UNITED STATES TESTING COMPANY, INC.
F. 7220 - C. 208 - QCP - 7 - 3
POWER GENERATION SERVICES

Controlled Copy No. 8 M

TITLE	PROCEDURE NO.	
SOIL FINES TESTING OF THE AREA DEWATERING SYSTEM	QCP-7	
	REVISION NO.	REVISION STATUS 1
	3	FA - I

SCOPE OF REVISIONS:

Revision 1: Extensive changes made in response to comments received from Bechtel Power Corporation on 11/15/79.

Revision 2: Incorporate Change Notice 1
Incorporate Change Notice 2
Incorporate Change Notice 3
Incorporate Bechtel Comments Throughout

Revision 3: Incorporate Bechtel Rev. 2 comments.
Incorporate Change Notice 1.
Revise Form MEI 225 dated 2/22/80 Formula clarification only.

RECEIVED
MAR 25 1980
BECHTEL POWER CORP.
JOB 7220
Per

I HAVE READ THIS PROCEDURE AND FULLY UNDERSTAND ITS REQUIREMENTS AS IT PERTAINS TO MY RESPONSIBILITIES. (Only sign when copy is issued to you) ______ Signature & Date

REVISION NO.	Original	1	2	3
NO. OF SHEETS	15	18	19	19
PREPARED BY	J. Speltz	J. Speltz	M. Rashid	M. Rashid
REVIEWED BY	A. J. Speltz	Arthur J Speltz	Arthur J Speltz	Arthur J Speltz
APPROVED BY	M. Rashid	M. Rashid	M. Rashid	D. Ellis
DATE OF ISSUE	11/2/79	12/20/79	2/4/80	3/17/80

1. STATUS CODES: FA - FOR APPROVAL; A-APPROVED H-HOLD. DO NOT IMPLEMENT; I-IMPLEMENT

USTF-PWC - 1.1 REV.3

7220-C208-16-24

SB 19172

TABLE OF CONTENTS

I. PURPOSE

Quality Control Procedure 7 identifies pertinent factors relative to determining the quantity of soil fines that may be contained in a water sample.

This procedure will facilitate the documentation of the requirements of Section 6A of Technical Specification 7220-C-88Q.

II. SCOPE

The requirements of this procedure shall apply as a minimum to all tests conducted for Section 6A of Technical Specification 7220-C-88Q, and whenever monitoring of the fines is directed by contractor to determine the adequacy of the dewatering system.

III. RESPONSIBILITIES

1.0 It is the responsibility of the U. S. Testing Project Supervisor to implement the requirements of this procedure.

2.0 It is the responsibility of the U. S. Testing dewatering technicians to perform sampling in accordance with Section IV, and testing in accordance with Section V of this procedure.

IV. SAMPLING

1.0 Responsibilities

Sampling points and sampling frequencies shall be identified by Bechtel. U. S. Testing personnel shall do the sampling. The sampling points shall be the only portion of the dewatering system operated by U. S. Testing personnel. Bechtel Quality Control shall be notified prior to sampling.

2.0 Procedure

2.1 Initial sampling of the individual wells, production sampling of temporary dewatering system during pumping and monthly sampling of the individual wells during pumping will be conducted in accordance with Attachment I to this Procedure and supplemented as follows: 2

2.1.1 Prepare a sample identification tag and record well number on it.

2.1.2 Attach a sampling tube to the petcock and flush for a minimum of one minute to remove accumulated dust, fines, and debris. Individual wells may be sampled directly from a well discharge hose when directed by Bechtel. 2

2.1.3 Rinse the container with the water being sampled.

2.1.4 Direct the flow into the sample container to obtain a minimum sample of 850 milliliters.

2.1.5 Stop the sampling flow and remove any excess if the sample is more than one liter as determined by reading the bottom of the sample meniscus.

2.1.6 Cover the sample.

2.2 Sample containers will be protected as necessary to assure that foreign materials are not included in the sample.

2.3 Whenever a test sample is transferred from one container to another, the container that held the test sample will be washed with distilled water. These wash waters will be included in the test sample but will not be included in the sample volume when calculating test results.

V. TESTING

1.0 Procedure

1.1 The quantification of soil fines in a sample shall be per Attachment I to this Procedure and supplemented as follows. 2

1.2 All weight determinations shall be to a minimum accuracy of 0.001 grams.

1.3 Filtration shall be by vacuum extraction thru a filter.

1.4 The filtering funnel shall be a Buchner type of serviceable size. An initial dry weight of filter shall be incorporated into the calculations. 2

1.5 Filter drying shall be for a minimum of two hours in a gravity convection oven at 221 to 239° F followed by cooling in a dessication chamber until the air temperature in the chamber is within 5° F of the ambient laboratory temperature in the test area. The filter shall remain in the dessicator until weighing.

1.6 The dessicating chamber and the balance weighing chamber shall contain a dessicant when in active use for this procedure.

1.7 Filters shall be receipt inspected to determing their serviceability under this procedure. Receipt inspection shall include the following tests as a minimum:

a) weight loss

b) verification of pore size. Note: Verification not necessary for filter medias less than 25 mircons. 2

c) oven drying time to constant weight.

Weight loss shall be determined by pre-testing a sample of the filters selected for use using distilled water to determine if any excess fibers are removed by the filtering process. If filter weight loss is noted, a maximum loss factor (f) 2 will be determined and applied to all filters. This factor will be 1.0 if no loss is recorded.

2.0 <u>Calculations</u>

2.1 One ppm of soil fin[illegible]fined as 0.001 gram of dry weight retained by the filter paper per 1000 milliliter sample.

2.2 The ppm of fines in a test sample (P_s) shall be calculated in accordance with the following formula.

$$P_s = \left\{\frac{W_2 - [(W_1)(f)]}{v}\right\}\left\{10^3\right\}$$ 2

W_1 = initial dry weight of filter in grams.

W_2 = dry weight of filter in grams after the test sample has been filtered thru it.

f = filter correction factor.

v = volume of test sample in liters.

3.0 <u>Documentation</u>

3.1 Data shall be documented on Attachments 2, 3 & 4 and transmitted to Bechtel Q.C. for review within 24 hours after the completion of the test (s). 2

3.1.1 Test data generated under this procedure shall be submitted to Bechtel for evaluation and application.

3.2 Report distribution shall be as directed by Bechtel Quality Control.

VI. <u>QUALITY ASSURANCE CONTROLS</u>

United States Testing Company Personnel certification and equipment for this procedure shall be controlled per the provisions of the Project Quality Assurance Manual 7220-C-208 with its supplementing procedures.

VII. ATTACHMENTS

Attachment Number	Description
1.	Monitoring Fines 2
2.	Report of Dewatering Effluent Test for Fines (form MEI-223)
3.	Log of Area Dewatering Flow Meter Readings (form MEI-224)
4.	Calculations for testing temporary dewatering system (form MEI-225)
5.	Summary Sheet 2

MONITORING FINES

1.0 The Subcontractor shall first test the effluent from each individual temporary dewatering well prior to production pumping to determine the amount of fines being removed by the dewatering well. The subcontractor shall collect a minimum of 850 milliliter sampled from the petcock of each individual temporary dewatering well after a minimum of 15 minutes and a maximum of 90 minutes of well development (test pumping) for each test. This time limit shall start over when a well is being retested. 3
In addition, when the dewatering system is in operation, the Subcontractor shall test each individual dewatering well on a monthly basis. These monthly test results shall be reported to the contractor's onsite geotechnical engineer. 2

The samples shall be tested in accordance with Item 2.2 below.

2.0 After the completion of testing each individual dewatering well, the effluent shall be tested every Monday and Thursday that the temporary dewatering system is in operation. The procedures for this testing are as follows:

2.1 Collect minimum of 850 milliliter samples of water from:

a. Overflow line at Location A (Pa), Figure 1
b. Return line at Location C (Pc), Figure 1
c. Hydrant at Location 7 (P_H), Figure 1 (only test water at hydrant on the first day of testing and whenever hydrant water is added to the system by the Contractor).

2.2 Procedure

a. Pretest a representative number of the glass fiber filter discs (no coarser than 0.05 millimeters) to determine the filter characteristics and weight loss potential. 2
b. Assemble the filtering apparatus; place the wrinkled surface of the filter disc facing upward and begin suction. Shake the sample vigorously and rapidly transfer the sample to a Buchner Funnel by means of a 1000 ml volumetric cylinder. Measure and record the exact amount of sample collected in the volumetric cylinder for use in the calculation.
c. Carefully remove the filter from the Buchner Funnel membrane filter apparatus. Dry at least two hours at 221-239 F. Cool in a dessicator and weigh on an analytical balance. Repeat the drying cycle until a constant weight is obtained or until weight loss is less than 0.5 mg.

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3.0 Calculations for testing individual wells

2.1 Calculate as follows:

$$mg/l = \frac{(A-B) \times 1000}{C}$$

Where:

A = weight of filter + residue
B = weight of filter (use filter correction factor if necessary)
C = litre of sample filtered

Report results to Contractor.

4.0 Calculations for testing temporary dewatering system

4.1 Record the flow in gallons from the flow meter on the hydrant, location 7, Figure 1, (ΣG_7 = total gallons at location 7).
4.2 Record the difference in the flow at the hydrant, location, 7, from the previous measurement (G_7).
4.3 Record the flow in gallons from the flow meter on the overflow line, location 8, Figure 1. (ΣG_8 = total gallons at location 8).
4.4 Record the difference in the flow at the overflow line, location 8, Figure 1, from the previous measurement (G_8).
4.5 Record the flow in gallons from the flow meter at the recirculation line, location 9, Figure 1. (ΣG_9 = total gallons at location 9).
4.6 Record the difference in the flow at the recirculation line, location 9, Figure 1, from the previous measurement (G_9).

P_{GW} is the average quantity of fines (ppm) measured for the interval since the last test was completed (Monday or Thursday).

P_{GWT} is the average quantity of fines (ppm) measured for the total gallons of ground water pumped.

For all other symbols see Items 2.1 and 4.1 through 4.6 above.

$$P_{GW} = \frac{P_c \times (G_8 + G_9) - P_a\,(G_9) - P_H\,(G_7)}{G_8 - G_7}$$

$$P_{GWT} = \frac{\Sigma(P_{GW} \times G_8 - G_7)}{\Sigma G_8 - \Sigma G_7}$$

Report results to Contractor.

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5.0 Purging Eductor Tank

5.1 Contractor will notify subcontractor whenever he indends to purge any collected fines from the eductor tank.

5.2 As the contractor purges his eductor tank, all material from the eductor tank shall be sieved through a No. 325 U. S. Standard screen over a 55 gallon drum by the Subcontractor.

5.3 The material collected on the sieve will be retained and stored for inspection by the on-site field geotechnical engineer. A representative 1-liter sample of the water that passes through the No. 325 U. S. Standard screen shall also be collected by the Subcontractor.

5.4 The materials collected on the screen and the corresponding water sample will be placed into separate suitabile sized containers, sealed and properly identified as follows:

Tank - dewatering sample No. -
Job No. -
Date -
Initials -

Total amount of ground water pumped at Location 8, Figure 1 on the date of tank purging. 2

Total amount of ground water purged shall be estimated by the Contractor and recorded by the Subcontractor.

6.0 Samples shall be stored with the other soils samples on-site. All used filter discs shall be stored in "zip lock bags" or equal with adequate documentation for traceability.

SAMPLE DATA SHEET FOR MONITORING FINES

(Temporary Dewatering System)

DATE	MEASUREMENT POINT	QUANTITY OF FINES (ppm)	TOTAL FLOW (Gallons)	DIFFERENCE SINCE LAST MEASUREMENT (Gallons) *
	Overflow line location A, Figure 1	(P_a)	-	-
	Return line location C, Figure 1	(P_c)	-	-
	Hydrant, location 7, Figure 1	(P_H)	(ΣG_7)	(G_7)
	Recirculation line, location 9, Figure 1	-	(ΣG_9)	(G_9)
	Overflow line location 8, Figure 1	-	(ΣG_8)	(G_8)

* On first day the previous readings are "0".

FOR MONITORING FINES

(Temporary Dewatering System)

DATE	MEASUREMENT POINT	QUANTITY OF FINES (ppm)	TOTAL FLOW (Gallons)	DIFFERENCE IN FLOW SINCE LAST MEASUREMENT (Gallons)**
Day 3*	Overflow line Location A, Figure 1	2 (P_a)	-	-
	Return line Location C, Figure 1	3 (P_c)	-	-
	Hydrant Location 7, Figure 1	1 (P_H)	-	-
	Hydrant, Location 7 Figure 1	-	3,000 (ΣG_7)	3,000 (G_7)
	Recirculation line, Location 9, Figure 1	-	3,456,000 (ΣG_9)	3,456,000 (G_9)
	Overflow line Location 8, Figure 1	-	1,728,000 (ΣG_8)	1,728,000 (G_8)
Day 6	Overflow line Location A, Figure 1	3 (P_a)	-	-
	Return line Location C, Figure 1	4 (P_c)	-	-
	Hydrant Location 7, Figure 1	-	3,000 (ΣG_7)	0
	Recirculation Location 9, Figure 1	-	6,912,000	3,456,000 (G_9)
	Overflow line Location 8, Figure 1	-	3,456,000	1,728,000 (G_8)

** On first day previous readings are "0"
* After pumping started

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SAMPLE (cont'd)

DAY 3

$$P_{GW} = \frac{[P_c \times (G_8 + G_9)] - [P_a (G_9)] - [P_H (G_7)]}{G_8 - G_7}$$

$$P_{GW} = \frac{3\ (3{,}456{,}000 + 1{,}728{,}000) - 2\ (3{,}456{,}000) - 1\ (3{,}000)}{1{,}728{,}000 - 3{,}000}$$

$$P_{GW} = \frac{15{,}552{,}000 - 6{,}912{,}000 - 3{,}000}{1{,}725{,}000}$$

$$P_{GW} = \frac{8{,}637{,}000}{1{,}725{,}000}$$

P_{GW} = 5.0 ppm (for first test cycle $P_{GW} = P_{GWT}$)

Day 6

$$P_{GW} = \frac{P_c \times (G_8 + G_9) - P_a (G_9) + P_H (G_7)}{G_8 - G_7}$$

$$P_{GW} = \frac{4\ (3{,}456{,}000 + 1{,}728{,}000) - 3\ (3{,}456{,}000) - 0}{1{,}728{,}000 - 0}$$

$$P_{GW} = \frac{4\ (5{,}184{,}000) - 10{,}308{,}000}{1{,}728{,}000}$$

$$P_{GW} = \frac{20{,}736{,}000 - 10{,}368{,}000}{1{,}728{,}000}$$

$$P_{GW} = \frac{10{,}368{,}000}{1{,}728{,}000}$$

P_{GW} = 6 ppm (Report results to Contractor)

$$P_{GWT} = \frac{\Sigma (P_{GW} \times G_8 - G_7)}{\Sigma G_8 - \Sigma G_7}$$

$$P_{GWT} = \frac{5.0\ (1{,}728{,}000 - 3{,}000) + 6.0\ (1{,}728{,}000 - 0)}{3{,}456{,}000 - 3{,}000}$$

$$P_{GWT} = \frac{8{,}625{,}000 + 10{,}368{,}000}{3{,}453{,}000}$$

$$P_{GWT} = \frac{18{,}993{,}000}{3{,}453{,}000}$$

P_{GWT} = 5.5 ppm (Report results to Contractor)

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ORIGINATOR ______ DATE ______ CHECK ______ DATE ______

PROJECT ______ JOB NO. ______

SUBJECT ______ SHEET NO. ______

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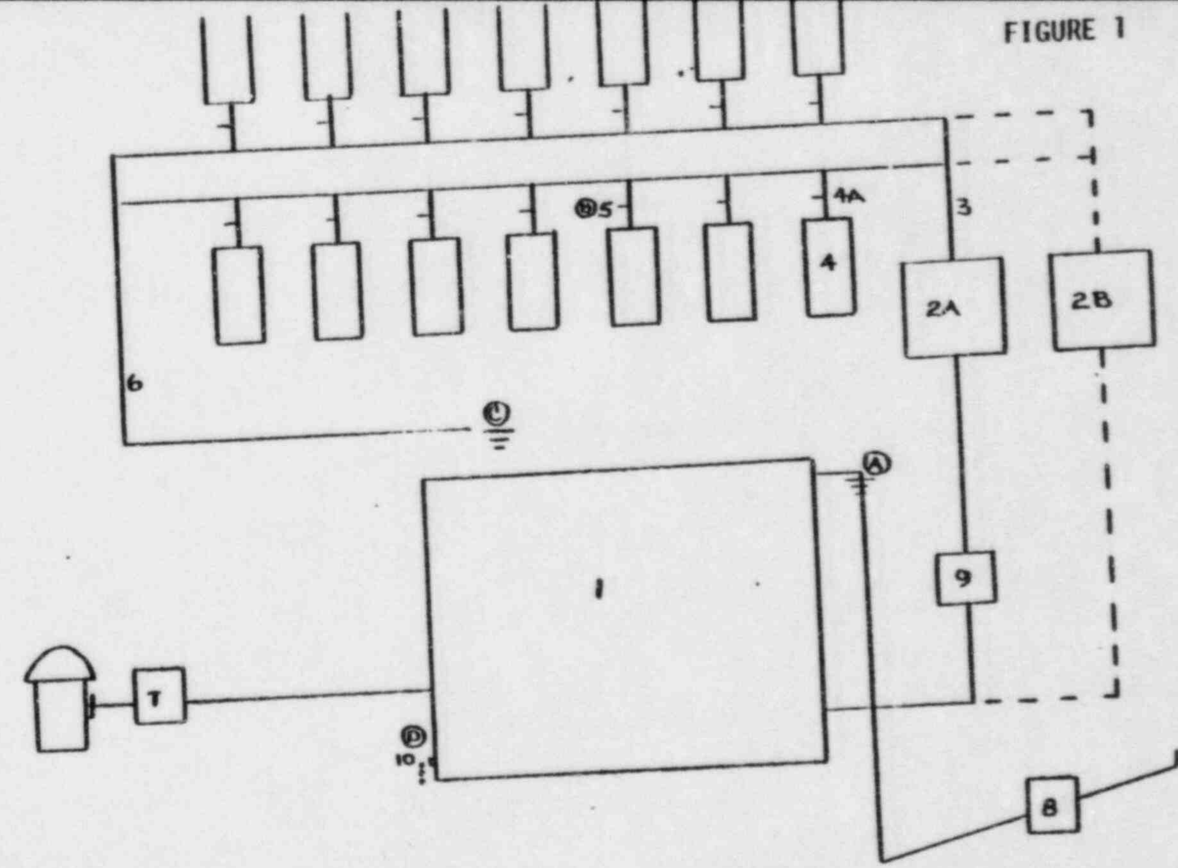
FIGURE 1



1. Loughney Water Tank: A 2000 to 3000 gallon tank supplied by Subcontractor.

2a System Pump: Centrifugal pump able to pump over 800 GPM (Electric)

2b Backup Pump: Centrifugal pump able to pump over 800 GPM (Diesel)

3. Outgoing Line: 8" pipe line going to eductors under approx 125 P.S.I. at max flow rate.

4. Well Point: Down hole eductor well needs 8.2 GPM to operate.

4a Eductor Well Line: Return line carries both eductor water and well water.

5. Sampling Petcock: 1/4" petcock and all necessary fittings needed for sampling.

6. Return Line: 10" return line under Atms. pressure.

7. Hydrant Meter: 3" Sparling inline meter series 1L2 to monitor water input.

8. Overflow Meter: 6" Sparling Inline, saddle mount series FM 103. Meter to monitor amount of system overflow.

9. Recirculation Line Meter: 8" Sparling In-line, saddle mount, series FM 103 meter.

10. Purge Valve: Tank purge location. A,B,C,D - Possible sample locations.

UNITED STATES TESTING COMPANY, INC.

MIDLAND NUCLEAR PROJECT

REPORT OF DEWATERING EFFLUENT

TEST FOR FINES

SAMPLING DATA

__________ (name) Q. C. Representative Notified on ______ (date) at ______ (time) by ________

Date ________ Sampled by ________ Valve No. ________ UST Log No. ________

Sample Container No. __________ Timing Device No. __________

TEST DATA

$$P_s = \left\{ \frac{W_2 - [(W_1)\ (f)]}{V} \right\} \left\{ 10^3 \right\}$$

W_1 = initial dry weight of filter in grams

W_2 = dry weight of filter in grams after the test sample has been filtered thru it.

f = filter correction factor

V = volume of test sample in litres

W_1 = ______ grms, W_2 = ______ grms, f = ______, V = ______ litre

$$P_s = \frac{\quad - [(\quad)\ (\quad)]}{\quad}$$

P_s = __________ ppm

Temperature in dessication chamber ______ °F Ambient Test Area Temperature ______ °F

Test Equipment	I.D. No.
Cylinder	
Oven	
Balance	
Thermometer	

Tested by ________ Date ________

Checked by ________ Date ________

Reviewed by ________ Date ________

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Sheet ______ of ______

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UNITED STATES TESTING COMPANY, INC.

MIDLAND NUCLEAR PROJECT

LOG OF AREA DEWATERING

FLOWMETER READINGS

Meter Number Location

Date	Time	Technician	Flow Quantities (gallons) FlowMeter Reading ( )	Flow ( )	Sample Log Ref.	Checked By

SB 19188

Checked by ____________ Date ________

Reviewed by ____________ Date ________

Sheet ________ of ________

7220·C208-16·4

CALCULATIONS FOR TESTING TEMPORARY DEWATERING SYSTEM

REPORT OF AVERAGE GROUND WATER FINES

Average Quantity of Fines Since Previous Measurement

Date of this measurement ____________________

$$P_{GW} = \frac{[(P_c)(G_8 + G_9)] - [(P_a)(G_9)] - [(P_H)(G_7)]}{G_8 - G_7}$$

P_{GW} =

P_{GW} =

Average Quantity of Fines for Total Pumping

Date of this measurement ______________

$$P_{GWT} = \frac{\Sigma [(P_{GW})(G_8 - G_7)]}{(\Sigma G_8 - \Sigma G_7)}$$

P_{GWT} =

P_{GWT} =

Note: The symbols used in these formulas are as defined in Sections 2 and 4 of Attachment 1 to UST Procedure QCP-7.

UNITED STATES TESTING COMPANY, INC.

DEWATERING VOLUME AND FINES

SUMMARY SHEET

DATE	FLOW SINCE PREVIOUS MEASUREMENTS			P_{GW}	TOTAL FLOW SINCE PUMPING			P_{GWT}
	Hydrant (G_7)	Overflow (G_8)	Recirc. (G_9)		Hydrant (ΣG_7)	Overflow (ΣG_8)	Recirc. (ΣG_9)	

Sheet ________ of ________

ALFRED J. HENDRON, JR.
Geotechnical Engineer

28 Golf Dr
Mahomet, Illinois 618
Phone: (217) 586-27

J256
17 November 1978

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JOB 7220		FILE 2140		
REC'D NOV 21 1978				

Dr. Sherif Affifi
Bechtel Associates
P. O. Box 1000
3621 South State Road
Ann Arbor, Michigan 48106

RE: Meeting on Midland Nuclear Station
Diesel Generator Bldg., November 7, 1978

Dear Dr. Afifi:

In the following letter I have briefly summarized what I believe to be the essential items of discussion and the conclusions which were reached at our meeting in Champaign, Illinois on November 7, 1978.

In this meeting the Bechtel engineers presented the settlement data, soil boring data and the lab data obtained on samples from the borings in the area of a diesel generator building at Midland. After these presentations, the various options which were considered for fixing the observed settlement problem at the diesel generator building were discussed. These options were:

- Option A - no correction plus grouting
- Option B - modify the continuous footings to a mat foundation under the structure
- Option C - preload the area around the structure and within the struct
- Option D - a combination of changing the footings to a mat and preload
- Option E - underpinning the structure with piles or piers
- Option F - remove and replace the building

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In the discussions of Option A, it was pointed out by Professors Peck and Hendron that no correction would mean that settlement of the fill wou continue under its own weight and that gaps may open up under the foundations of the building at a later time. In addition, the relative movements between the building and the piping and some of the piping systems with respect to eac other may continue over a period of 5 or 10 years. This would mean, if no instrumentation were added, that these possible harmful movements could go undetected until it was too late. Even if they were detected, Bechtel and the contractors would be off the job and corrections would be much more difficult to make and they may cause a regulatory shutdown of the plant if the plant were operating at that time.

Although modifying the strip foundations to a mat foundation would make the structure settle more uniformly it would not prevent the fill from settling due to its own weight over time and resulting in the same problems with the piping as Option A would yield.

Both consultants favored the preloading solution (Option C) because it would accelerate the settlements such that very little settlement would occur after the preload is removed. We can observe both the structure and the piping during this process and then determine which of these systems may need the remedial measures before the license is obtained for the plant. It has the advantages that remedial measures probably will not have to be made to the structure but there is some possibility that remedial measures will have to be employed to correct the differential movement between some of the piping systems and the building.

Some combination of the mat and preload (Option D) possibly could work, however, if the preloading is employed and works then the changing of the strip footings to a mat foundation would be an unneccessary expense.

Dr. Sherif Afifi
17 November 1978
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Underpinning [redacted]s ruled out because underpinning of the structure itself would still not relieve our problems with the relative movements between the piping and the structure. The piping would still be settling with the fill because it would be settling under its own weight. [redacted] piles or piers would also develop negative skin friction because the fill would still be settling under its own weight. We feel that this may be as expensive as a preloading solution [redacted]

One option would be possibly to remove and replace the building. However we do not feel this is necessary because even the removing and replacement of the building without preloading would still not solve any of the basic problems of the pipe settlements below the building. Preloading would have to be employed in between the removal of one structure and the rebuilding of another one. Since we feel the structure will undergo the preloading without any undue distress, removing and replacing the present building does not seem to offer a solution unless preloading is employed between removal and replacement.

In a discussion of the preferred option of preloading, some of the following points were discussed at the meeting.

The main problem at the location of the diesel generator building is settlement of the fill due to its own weight and the imposed loading of the diesel generator building. The preloading solution is simply a method employed to accelerate the settlement by applying a loading in and around the building which exceeds the design loading. The preload is then removed after

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sufficient consolidation has occurred such that very little additional settlement will occur under the design loadings. The key judgement which must be made in employing this method is the selection of the time at which the preload can be removed. This decision must be made on the basis of both settlement and piezometric observations and is one of the main reasons why extensive field observations of piezometric levels and settlement measurements are required.

A lot of discussion was given to the instrumentation which must be employed during the preloading phase. These included many settlement monuments to monitor settlement versus time, piezometers to monitor pore pressures within and below the fill, and special settlement points to measure the settlements at various depths in the fill. In this respect it was also suggested to have a series of settlement points installed at various depths at some point in the fill outside and away from the building to monitor as soon as possible how the fill is settling under its own weight. This location should preferably be between the diesel generator building and the reservoir in an area which is already being affected by saturation from the reservoir in the event that one of the major sources of compressibility of this material may be the initial wetting of the material because of compaction on the dry side.

The plan of action is in general for instrumentation and settlement points as well as piezometers to be installed immediately such that preloading can begin as soon as possible. Initial planning is to have on the order of 15 to 20 feet of preloading for a distance of about 20 to 30 feet around the structure with the slopes being sloped on probably about a 2 to 1 slope to the present grade elevation of 634. Because this depth of backfill will cause

SJ 19398

some problems for the retaining wall on the south side of the turbine building, consideration should be given to bracing that wall either by bracing the top of it with the floor system in the turbine generator building or possibly by anchoring the top of the wall back to the wall of the diesel generator building.

It was also decided that it would be advisable to grout under the footings where gaps have developed before preloading. It was also decided that, immediately after this grouting is done, to cut away the structure from the electrical duct banks that are concreted in vertical members. These duct banks are now essentially acting as a series of piers along the north side of the building. It was also indicated that grouting will also have to be done under the footings after the preloading operation is completed.

Within the structures the present grade elevation is proposed to be soil up to elevation approximately 635 and the interior floor slab will be supported on soil. It is possible to reduce the total weight of the structure causing settlement if the lower most slab in the diesel generator building were a structural slab and the soil surrounding the pedestals were removed to an elevation close to the foundation elevation for the building. This should be considered as an alternate for reducing the long term loading of the building.

Since the compressibility of the backfill could be affected by the rising reservoir level it was also recommended that the reservoir should be brought up to the maximum design elevation while the preload is on such that the embankment under the diesel generating building becomes saturated.

Respectfully submitted,

Alfred J. Hendron, Jr.

Alfred J. Hendron, Jr.

SD 19399

jrm
cc: Dr. R. Peck

Bechtel Incorporated

Interoffice Memorandum

To M. Mirsky

Date October 3, 1978

Subject Midland Project
Job 7220-101
Diesel Generator Building
Settlements

From Walter R. Ferris

Of H&CF - Soils

Copies to R. Schnaible
H. H. Burke
S. Afifi

At 45/31/C36
Extension 7834

On September 28, 1978, Dr. Ralph Peck visited the Midland site to inspect the emergency diesel generator building and to review the initial results of the exploration and testing program presently underway at the site.

Dr. Peck was given a brief summary of the geology and foundation conditions at the site. He expressed interest in the program developed to monitor areal subsidence that might result from pumping from Dow Chemical's brine production wells. To date there is no evidence of such subsidence. The emergency diesel generator building was inspected and also several other buildings on compacted backfill. Exposures of the fill were inspected at locations where trenches had been excavated into it.

Following the inspection Dr. Peck made some recommendations modifying the exploration program. He also concluded that, based on the data presented to him and his inspection of the site, the glacial till used for backfill is similar to that encountered in many places in this part of the United States and structures placed on compacted material of this kind have performed satisfactory.

Dr. Peck said that it was too early to discuss a fix as all of the exploration, testing and observational data were not yet available. He also said that he will read and digest the preliminary data given to him and may have some additional comments to make on investigation.

Dr. A. J. Hendron of the University of Illinois will visit the site on October 5. He was asked to participate as Dr. Peck's time is limited and Dr's. Peck and Hendron have cooperated previously for ~~us very~~ satisfactorily with such an arrangement.

Notes of the meeting on September 28 will be published t the Ann Arbor office.

W. R. Ferris

WRF/mz

W. R. Ferris

SB 19400

6060 (6-73)

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JOB 7220 FILE
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TECHNICAL SPECIFICATION

FOR

SUBCONTRACT FOR

UNDERPINNING, EXCAVATION, AND PLACING OF CONCRETE

FOR THE

CONSUMERS POWER COMPANY

MIDLAND PLANT UNITS 1 AND 2

MIDLAND, MICHIGAN

COORDINATION PRINT
JOB 7220
DATE 9·17·79

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ORIGIN			
BAPC	BECHTEL	CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 AND 2 MIDLAND, MICHIGAN	JOB No. 7220 SPEC/DES GUIDE No. C-95(Q) REV 0

SHEET	LATEST REV.	SHEET	LATEST REV.	SHEET	LATEST REV.	SHEET	LATEST REV.	SHEET	LATEST REV.	SHEET	LATEST REV.	SHEET	LATEST REV.
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BECHTEL

FACING SHEET

UNDERPINNING, EXCAVATION, AND PLACING OF CONCRETE
CONSUMERS POWER COMPANY
MIDLAND POWER PLANT UNITS 1 AND 2
MIDLAND, MICHIGAN

JOB No. 7220

C-95(Q)

REV. 0

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TECHNICAL SPECIFICATION

FOR

UNDERPINNING, EXCAVATION, AND PLACING OF CONCRETE

CONTENTS

APPENDICES

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1.0 SCOPE

1.1 ITEMS INCLUDED

The following work to be performed in accordance with this subcontract is located at Midland Plant Units 1 and 2. This specification includes Q-listed work where specifically noted and is to be performed in accordance with Subcontractor's QA program:

1.1.1 Underpinning the auxiliary building penetration rooms with caissons in the area indicated in the drawings

1.1.2 Removal of all unsuitable material as determined by Contractor and replacement of all removed material with concrete from under the feedwater valve pits

1.1.3 In addition, the following items are included under the scope of this subcontract:

a. Submit all drawings, calculations, and detailed procedures for a proposed type and method of underpinning which is best suited for the intended purpose and, as a minimum, meets all of the applicable criteria specified herein.

b. Furnish all labor, material, tools, equipment, supervision, design, and procedures to perform all operations and incidentals necessary to complete the work to the satisfaction of Contractor.

c. Provide (design, furnish, and install) local dewatering to remove and control all water which could cause soil movement.

d. Provide (design, furnish, and install) permanent support under the auxiliary building penetration rooms capable of withstanding approximately 4,000 kips vertical load at locations shown in the drawings without exceeding a 1/2-inch settlement (movement) of the structure at points indicated in the drawings.

e. Provide (design, furnish, and install) permanent support under the turbine building along the K line to support the column and slab loads at columns 2.0, 2.5, and 3.0 for Unit 1 and 10.0, 10.5, and 11.0 for Unit 2, resulting from and including the temporary support of the valve pits as indicated in the drawings.

f. Provide hydraulic jacking capacity sufficient to simultaneously lift approximately 4,000 kips of structural load utilizing the permanent support as a reaction point.

g. Provide lateral support for the soil in the zone of influence resulting from excavation made during underpinning to prevent soil movement.

h. Place and cure lean concrete backfill under the designated buildings including encasement of the permanent caissons.

i. Provide positive contact between the underside of the building foundation and the lean concrete backfill by methods acceptable to Contractor.

j. Install styrofoam or similar material as indicated on the drawings.

k. Monitor the buildings for settlement.

1.2 ITEMS NOT INCLUDED BUT PERFORMED BY OTHERS

1.2.1 Area dewatering prior to underpinning and excavation

1.2.2 Temporary support (above the ground) of the feedwater valve pit for Units 1 and 2 as indicated in the drawings.

1.2.3 Disposing of the waste material from a stock pile to be located at the ground surface near Subcontractor's access shaft.

1.2.4 Furnishing and testing of lean concrete backfill

1.2.5 Furnishing and testing structural concrete

1.2.6 Furnishing reinforcing steel

1.2.7 Furnishing styrofoam

1.2.8 Soil testing

1.2.9 Monitoring the auxiliary building for cracks

2.0 ABBREVIATIONS

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASTM	American Society of Testing Materials
AWS	American Welding Society

3.0 CODES, STANDARDS, AND REFERENCES

ACI 318-77	Building Code Requirements for Reinforced Concrete
ACI 543-74	Recommendation for Design, Manufacture, and Installation of Concrete Piles
AISC	Manual of Steel Construction, 7th Edition
ASTM A 36-77a	Specification for Structural Steel
ASTM A 53-77a	Specification for Steel, Black and Hot-Dipped, Zinc-Coated, Welded, and Seamless Pipe
ASTM A 252-77a	Specification for Welded and Seamless Steel Pipe Piles

AWS D1.1-74 Structural Welding Code

Reference 1 Soft Ground Tunneling with Steel Supports, Commercial Shearing Inc., Proctor and White, Youngstown, Ohio, 1977 (used for definition of state and behavioral characteristics of soil)

4.0 DOCUMENTATION REQUIREMENTS

4.1 Engineering and quality verification documents shall be submitted to Contractor by the underpinning Subcontractor. Permission to proceed, based upon Contractor's review of the procedures, does not constitute acceptance or approval of design details, calculations, analyses, test methods, or materials developed or selected by Subcontractor and does not relieve Subcontractor from full compliance with contractual obligations. The submittal requirements are summarized in Form G-321-D attached. These requirements are augmented by detailed requirements in this specification.

4.2 As a minimum, Subcontractor shall submit the following procedures (in detail, including hold points and inspection points) to Contractor's satisfaction:

4.2.1 General Underpinning Procedure - This procedure shall include the overall concept of the work involved, including the interface of all the operations listed below.

4.2.2 Detection and Monitoring of Structural Movement Procedure

4.2.3 Local Dewatering Procedure

4.2.4 Support of Excavation, Bracing, and Lagging Procedure

4.2.5 Installation of Access Shafts Procedure

4.2.6 Permanent Support Procedure for approximately 4,000-Kip Caisson Capacity Installed Under Each Electrical Penetration Room

4.2.7 Pile and/or Caisson Installation, Cleaning, Concreting, and Testing Procedure

4.2.8 Lifting Procedure for Raising Ends of the Electrical Penetration Rooms Utilizing Approximately 4,000 Kips of Hydraulic Jacking Force

4.2.9 Mass Excavation and Removal of the Material Under the Structures Procedure

4.2.10 Installation of Styrofoam Procedure

4.2.11 Mass Concreting Procedure

4.2.12 Dry Packing/Pressure Grouting Procedure

4.2.13 Soil Grouting Procedure (Chemical and Cement)

4.2.14 Pump Tremie Procedure

4.2.15 Final Cleanup Procedure

4.2.16 Welding Procedures and Qualifications

5.0 MATERIAL REQUIREMENTS

Materials shall conform to the above standard specifications, and Subcontractor shall submit to Contractor certified copies of mill test reports of all material used as specified by the applicable ASTM specifications.

5.1 ASTM A 36: For structural steel and bearing plates

5.2 ASTM A 53: Types E or S, Grade B for piles and caissons

5.3 ASTM A 252: Grade 2 seamless for piles and caissons

6.0 DESIGN PARAMETERS

6.1 PILE AND CAISSON DESIGN

Piles or caissons may be used to underpin the turbine building along the K line adjacent to the valve pit structure. Only caissons shall be used to underpin the auxiliary building penetration rooms.

6.1.1 Piles shall be located at a minimum center-to-center spacing of 3.0 feet for 80-ton piles and 3.5 feet for 100-ton piles. The pile center to the edge of concrete shall be 1'3" for 80-ton piles and 1'6" for 100-ton piles.

6.1.2 Combination bearing/soldier piles shall penetrate a minimum of 5 feet into the till or natural dense sand strata as determined by the soil removed from the interior of the pile and the jacking resistance.

6.1.3 Maximum calculated soldier pile lateral deflection shall be limited to 0.25 inch due to bending under the combined axial and lateral loading.

6.1.4 Maximum calculated lagging member deflection due to bending shall be limited to 0.25 inch.

6.1.5 The lateral pressure diagram for soil pressure shall be trapezoidal with break points at 0.2 of the full excavated depth.

6.1.6 Bending stress reduction of lateral loads due to arching shall not be allowed.

6.1.7 Pile and caisson locations shall be taken into account in the support of the mudmat to prevent separation from the foundation.

6.1.8 A full water head shall be considered for lateral support from el 627' to the bottom of mass excavation under the structures.

6.1.9 Caissons shall extend at least 4.0 feet into the till or natural dense sand strata as evidenced by inspection of in situ material. The design bearing capacity shall be 20 ksf times the diameter squared (ft^2) of the caisson. If the caisson is belled, the capacity shall be calculated at 17.7 ksf times the plan area of the bell. No increase in capacity is allowed for additional embedment of the caisson in the bearing strata.. Caissons acting as combination soldier and bearing elements are governed by the stress and deflection criteria from Sections 6.1.3 through 6.1.7.

6.1.10 The access shaft from el 634' to el 607' need not conform to Sections 6.1.3 to 6.1.8 but must be in accordance with acceptable industry practice.

6.1.11 For bearing plate design, the allowable bearing stress for concrete shall be 750 psi. Bearing stress for steel shall be 27,000 psi.

6.1.12 The design stresses shall not be greater than the allowable stress presented in AISC, ACI, or as specified herein. In the event of conflict, notify Contractor, who will determine the governing criteria.

6.2 DESIGN LOADS

6.2.1 Structure Loads

The load of each auxiliary building penetration room is equal to 8,300 kips (vertical). The center of gravity of the load is shown in the drawing. The allowable eccentricity of the underpinning support system during construction with respect to the east-west centroidal axis of the penetration room load is 3 feet.

Foundation pressure as well as local column loads for structures in the vicinity are shown in the drawings.

6.2.2 Soil Conditions

Soil conditions and interpretation of soil properties encountered are discussed below. In general, the backfill consists of heterogeneous mix of very loose to dense sand and very soft to hard clay fill. Details of soil information and boring logs are given in the foundation engineering sections of the PSAR and FSAR and are available from Contractor. The interpreted engineering properties of backfill presented below are based upon the laboratory and field investigations performed in the vicinity of the diesel generator building.

a. Soft to stiff, silty, sandy clay fill (CL)

Total unit weight γt = 110 - 130 pcf

Dry unit weight γd = 98 - 116 pcf

Undrained shear strength = 150 - 3,000 psf

b. Loose to dense, silty, fine to medium sand (sm-sp)

Total unit weight γt = 110 - 130 pcf

Dry unit weight γd = 94 - 116 pcf

Angle of internal friction 28 - 32 degrees

c. In-Situ Fill ? MATERIAL *

Total unit weight γt = 130 pcf

Average undrained shear strength El 580-560' 6,000 psf

Below 560' 8,000 psf

6.2.3 The permanent support for each auxiliary building penetration room provided by underpinning shall have a vertical resistance capacity sufficient to produce a moment equal to or greater than 325,000 foot kips at column rows 5.3 and 7.8, respectively. This moment capacity is the major design criterion and shall be met by Subcontractor. The approximate value of 4,000 kips of installed capacity as referenced throughout this specification is an estimate of the actual caisson capacity. The exact capacity required is to be determined by Subcontractor. All pumps, jacks, gages, equipment, and other hardware shall be of sufficient capacity and number to install and test, in accordance with this specification, the caissons that will produce a moment equal to or greater than 325,000 foot kips.

6.2.4 The permanent underpinning support for the turbine building shall be capable of safely resisting the column loads indicated in the drawings plus half the base slab pressure within the zone of influence. The zone of influence is defined by a slope of one horizontal to one vertical from the bottom of the excavation required for removal of the unsuitable material from under the valve pit structure. The permanent underpinning support for the turbine building shall take into account the 1,300 kips of force anticipated due to support of the valve pit structure.

7.0 PILE AND CAISSON LOAD TESTING

7.1 The vertical factor of safety established by in-place testing shall be not less than 1.5 for piles and caissons.

7.2 Test Method

7.2.1 The first caisson installed under each auxiliary building penetration room shall be load tested for 24 hours at 1.5 times the design load and 12 hours at 2.0 times the design load. The test load shall be applied in increments of 50%, 20%, 10%, 10%, and 10% of 1.5 times the design load at 1-hour intervals which are not included in the 24-hour period. At the completion of the 24-hour test at 1.5 times the design load, the load shall be increased to 2.0 times the design load in increments of 10% of the design load per hour. The load shall be maintained at 2.0 times the design load for 30 hours and then removed in decrements of 20% per hour to 80% of design load, then removed in decrements of 30%, and finally 50% at 1-hour intervals. This caisson selected for testing shall be representative of the majority of caissons to be installed.

7.2.2 All remaining caissons shall be loaded to 1.0 times the design load prior to concreting. This loading shall be referred to as the Empty Shell Test (EST). A satisfactory EST is defined as causing a caisson movement of less than 0.10 inch in a 5-minute period under a load equal to 1.0 times the design load.

7.2.3 After the EST and concreting of the caisson, the caisson shall be subjected to a Full Test Load (FTL). A satisfactory FTL is defined as causing a caisson movement of less than 0.05 inch in a continuous 1-hour period under a load equal to 1.5 times the design load. Extensometers (AMES) dial gages shall be calibrated to 0.001 inch per revolution and used to measure the movement of the caisson relative to the structure.

7.2.4 After installation of a group of caissons which constitute 500 design tons of underpinning resistance and prior to the completion of the caissons of the next group, one caisson in the previous group shall be chosen by Contractor for retesting. The caisson shall be tested for a period of 5 minutes at 1.5 times the design load. A satisfactory test shall be one where less than 0.010 inch of settlement occurs from the start of the test to the end of the test. If the test is satisfactory, the work on the next group may continue. If the test is not satisfactory, the caisson being tested shall be unloaded and retested in accordance with the requirements of FLT (Section 7.2.3). Then one more caisson in the group shall be subjected to the 5-minute and 0.01-inch settlement test. If the second caisson retested is satisfactory, the work on the next group may continue. If it is not satisfactory, repeat the procedure on caissons selected by Contractor until a satisfactory 5-minute and 0.01-inch settlement test is achieved before proceeding with the work.

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After completion of the FLT (Section 7.3.2) or 5-minute retest, the caissons shall be locked off (wedged tight) at 1.5 times the design load.

7.2.5 During the period of the test performed in Section 7.2.1, neither jacking against the structure nor any work which causes vibrations or may otherwise affect the test results shall be permitted.

7.3 The test method for the piles and/or caissons under the turbine building adjacent to the valve pit structure shall be in accordance with Section 7.2.

8.0 UNDERPINNING AND SUPPORT OF STRUCTURES

8.1 GENERAL

Subcontractor shall provide the necessary underpinning and permanent support for the auxiliary building penetration rooms for Units 1 and 2 in accordance with the requirement of this specification and as shown in the drawings.

In addition to the underpinning work shown in the subcontract drawings, Subcontractor shall take all other action necessary to maintain the integrity of these buildings. Subcontractor shall be responsible for supporting structures and taking all necessary precautions to prevent the settlement (movement) greater than specified in this specification or cracking of the buildings, including slabs on grade, supported slabs, appendages, interior partitions, and other columns and walls.

8.2 RESPONSIBILITY

Subcontractor shall conform to the following.

8.2.1 Assume full responsibility for all underpinning and related operations. Take necessary precautions for protection of persons, and preclude damage to property, including structures which are underpinned or affected by underpinning work.

8.2.2 All underpinning operations shall conform to applicable codes and also meet requirements of other authorities having jurisdiction over the work involved.

8.2.3 The maximum settlement (movement) of the auxiliary building penetration rooms and the turbine building along the K line for Units 1 and 2 shall not exceed 1/2 inch.

8.3 TEMPORARY SUPPORT

Prior to underpinning for Unit 1 and 2 operations, the feedwater valve pits will be temporarily supported by Contractor as shown in the drawings. The feedwater valve pits shall not be subjected to any additional vertical loads or external forces by Subcontractor during the underpinning operation.

8.4 PILE AND CAISSON INSTALLATION

8.4.1 General

The piling or caisson shall consist of a concrete-filled steel pipe. The steel pipe shall be installed by jacking against the existing structure. The pipe shall be precut to a convenient length for jacking. The pipe segments shall be joined together by full penetration welds around the circumference as the pipes are advanced into the subsurface.

The pipe shall be installed open-ended to the depths specified. The pipe shall then be subjected to the EST (Section 7.2.2), cleaned, inspected, concreted, subjected to the FLT (Section 7.2.3), and wedged tight to the structure under a load equal to 1.5 times the design load.

8.4.2 Materials

Steel pipe shall conform to ASTM A 252, Grade 2 seamless or ASTM A 53, Types E or S, Grade B. It shall have a minimum wall thickness sufficient to sustain the design load of the pile or caisson calculated at f_a = 18,000 psi. All cutting of the pipe shall be done in such a manner that the cut edge shall be perpendicular to the longitudinal axis of the pipe. Flame cutting is permitted. Beveling of edges for welding shall be done in such a manner to ensure a true cut. Internal or external pipe couplings, whether mechanical or welded, are not permitted. The pipe tip shall be reinforced by a steel shoe or by increasing the wall thickness of the lowermost section of pipe. The reinforcement shall be sufficiently strong to sustain a static point load equal to the maximum jacking force without permanent deformation of the lowermost pipe section. The shoe diameter or outside diameter of the lowermost pipe section shall not exceed the diameter of the remaining pipe section by more than 1/4 inch. Steel friction breakers may be installed on the outside of the pipe by welding. Friction breakers will not extend more than 1/8 inch beyond the outside of the pipe.

8.4.3 Pile Installation

a. The plan location shall be established at the bottom of the concrete slab at el 607'±. The tolerance for the plan location shall be ±5 inches, but the algebraic aggregate of the deviations shall not exceed ±5 inches.

b. The allowable top-to-bottom plumbness of the pile shall be not more than 2% out over the entire length. If the plumbness is more than 2% over the entire length, Subcontractor shall reduce the design load on the pile accordingly.

c. The maximum deviation for pile straightness shall be 1/2 inch in 10 feet and 1/2 inch in 20 feet.

d. If any criteria specified in Items a, b, or c of Section 8.4.3 are not satisfied, Subcontractor shall calculate the allowable reduced design capacity of the pile in accordance with generally acceptable methods. Calculations shall be submitted to Contractor for review. Subcontractor shall not be permitted to increase the load on other piles to compensate for the reduction caused by failure to achieve the requirements stated in Items a, b, or c of Section 8.4.3. All additional piles which shall be installed to compensate the loss of capacity shall be at the expense of Subcontractor.

e. The maximum jacking force shall be 1.50 times the design pile load or 0.85 f_y prior to the reduction for L/r ratio or 0.70 f_y as indicated in the certified material test report prior to reduction for L/r ratio. A commercial grade bentonite slurry may be injected between the pipe and the soil to minimize side wall friction.

f. Maintain a minimum of a 1-foot (vertical measurements) soil plug in the pile at all times unless an obstruction is encountered. If an obstruction is encountered, then maintain a liquid level inside the pile to within 5 feet of the bottom of the jacking pit, and limit the removal of material beyond the existing tip of the pile to less than 1 foot except by specific written instructions by Contractor.

g. No water jetting, slurry jetting, or air jetting for loosening or removal of material inside or in advance of the pile is permitted except by written permission from Contractor.

h. No airlifting or venturi principle lifting of the material in the pile or in advance of the pile is permitted within 5 feet of the pile tip except after final seating of the pile in the till. If air or venturi lifting is employed, Subcontractor shall maintain a liquid level in the pile at the bottom of the jacking pit during the entire operation. Contractor shall be notified in advance and in writing each time Subcontractor employs this procedure.

i. No impact or vibratory forces may be used for advancing the pile.

j. Inspection of piles prior to concreting shall be accomplished by visual inspection and measurements inside the pile. Subcontractor shall make a final cleaning of the pile and remove all water and deleterious matter from inside the pile. The pile will then be inspected by Contractor, who will give written permission to Subcontractor to place concrete in the pile. The maximum length of earth plug at the pile tip prior to concreting is 1'-6" of compact material.

k. The concrete for the pile shall be placed in such a manner that the concrete drops vertically from the top of the pile. Subcontractor shall take necessary measures to prevent segregation and voids in the concrete during placing.

1. Contractor reserves the right to direct Subcontractor to use the pumped tremie method for concreting the piles without employing the grout plug.

8.4.4 Caisson Installation

a. The plan location shall be established at the bottom of the concrete slab at elevation 607'±. The allowable deviation from the plan location shall be ±5 inches, but the algebraic aggregate of the deviation shall not exceed ±5 inches.

b. The allowable top-to-bottom plumbness of the caisson shall be not more than 2% out over the entire length. If the plumbness is more than 2% over the entire length, Subcontractor shall reduce the design load on the caisson accordingly.

c. The maximum deviation for caisson straightness shall be 1 inch in 10 feet and 1 inch in 20 feet.

d. If any criteria specified in Items a, b, or c of Section 8.4.4 are not satisfied, Subcontractor shall calculate the reduced allowable design capacity of the caisson in accordance with generally acceptable methods. Calculations will be submitted to Contractor. Subcontractor shall not be permitted to increase the load on other caissons to compensate for the reduction caused by failure to achieve the requirements stated in Items a, b, and c of Section 8.4.4. All additional caissons shall be at the expense of Subcontractor.

e. The maximum jacking force shall be 0.85 f_y prior to the reduction for the L/r ratio or 0.70 f_y as indicated in the certified material test report for the steel shell (prior to reduction for L/r ration) or 800 kips, whichever is less. A commercial grade bentonite slurry may be injected between the pipe and the soil to minimize side wall friction.

f. No water jetting, slurry jetting, or air jetting for loosening or removal of material inside or in advance of the caisson is permitted except by written permission from Contractor.

g. No airlifting or venturi principle lifting of the material in the caisson or in advance of the caisson is permitted within 5 feet of the caisson tip except after final seating of the caisson in the till. If air or venturi lifting is employed, Subcontractor shall obtain written permission from Contractor, and then maintain a liquid level in the caisson at the bottom of the jacking pit during the entire operation.

h. Caisson linings shall be of adequate thickness to ensure safe working conditions within the caissons.

i. The straight-shafted portion of the metal-lined caissons shall be limited in plan area to 16 square feet.

j. Metal-lined caissons with belled or wedge-shaped bottoms shall not have the top bell started until the shaft has penetrated only into the clay till a minimum of 2 feet.

k. The bell angle shall be a minimum of 60 degrees from the horizontal. The bell shall be lined for depths in excess of 4 feet below the straight shaft. The annulus (void) between the straight shafts liner and the ground shall be grouted and cured for 24 hours prior to starting excavation of the bell.

l. If squeezing ground (as defined in Soft Ground Tunneling with Steel Supports by Proctor and White) is encountered, Subcontractor shall proceed in such a manner that no ground is lost. Methods acceptable for this operation are as follows:

1. Jacking the lining past the squeezing zone without removal of material

2. Drilling under bentonite slurry and jacking (Drilling cannot proceed in excess of 1 foot beyond end of lining.)

m. If running ground (as defined in Soft Ground Tunneling with Steel Supports by Proctor and White) is encountered, the following methods for advancing are acceptable.

1. Jacking the lining past the running zone without removal of material

2. Drilling under bentonite slurry and jacking (Drilling cannot proceed in excess of 1 foot beyond end of lining.)

3. Grouting

n. If flowing ground (as defined in Soft Ground Tunneling with Steel Supports by Proctor and White) is encountered, stop excavation, fill caisson with water or slurry, give written notice of the condition to Contractor, and await written direction from Contractor prior to proceeding.

o. Concreting shall not be performed until Contractor has inspected the bottom and bell of the caisson and made measurements for plumbness, straightness, and location.

p. The concrete for the caisson shall be placed in such a manner that the concrete drops vertically from the top of the caisson. Subcontractor shall take necessary measures to prevent segregation and voids in the concrete during placing.

q. Contractor reserves the right to direct Subcontractor to use the pumped tremie method for concreting.

8.5 JACKING AND TESTING EQUIPMENT

8.5.1 The jacking and testing apparatus is to be actuated by hydraulic pressure generated by electrically or mechanically driven pumps. All high-pressure hydraulic apparatus shall be kept in a state of good maintenance.

8.5.2 All jacking and testing apparatus shall be equipped with two hydraulic pressure gages. The pressure gages for the production jacking equipment shall have a minimum face diameter of 2 inches. Testing equipment for the 24-hour load test (Section 7.2.1) shall have gages with a minimum 6-inch diameter face calibrated so that 270 degrees of dial movement covers the operating range of the equipment. Test equipment for the FLT (Section 7.2.3) shall have 4-inch diameter gage faces and be calibrated so the 270 degrees of dial movement cover the operating range of the equipment.

8.5.3 Subcontractor shall have the following hydraulic pressure gages at the job: three 8-inch face, three 6-inch face, and three 4-inch face gages calibrated with the specific rams intended to be used with each set of gages. The calibration shall be performed and certified by an independent laboratory which specializes in that type of work no earlier than 1 month prior to load testing. The calibration certificates shall be submitted to Contractor.

8.5.4 The gages and rams referred to in Section 8.5.3 shall hereafter be referred to as the master set. The master set shall not be used in the field operations except for the 24-hour load test. The master set shall be kept in a place suitable to Contractor and shall be used for calibrating all gages used in the field.

8.5.5 Subcontractor shall furnish a test manifold and test stand which will enable Subcontractor to field-calibrate four field gages simultaneously in the same hydraulic circuit as the respective three master gages and ram. Subcontractor shall notify Contractor in writing when field calibrating of gages will be done. Contractor will have a representative present during field calibration. The field gages shall be calibrated against a master set having a larger gage face. The results of the field calibration shall be forwarded to Contractor following the calibration.

8.5.6 Each field and master gage shall be assigned a unique identification number, and that number shall be engraved on the gage cover and also be painted on the gage back. In addition, a suitable calibration curve or tabulation for the particular gage shall be afixed to the back of the gage by means of a clear, sticky, celluloid cover. If gage face overlays are used, this is equally satisfactory.

8.5.7 Recalibration of the master sets is required at a minimum of every 3 months. This should be scheduled so there is at least one set of 6-inch or larger master gages on the project at all times. In addition, the master gages shall be recalibrated any time that one master gage varies by more than 5% of its calibrated rating as established by the other two master gages in the same master set. Field gages shall be recalibrated monthly or at any time when one gage is broken or damaged or when one gage varies by more

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than 10% from its calibrated rating as established by its companion gage.

8.5.8 The field testing apparatus shall contain a snubber valve in the hydraulic circuit. The snubber valve shall be located on the high-pressure side between the tandem gages and the pump, and immediately behind the gages. For the FLT (Section 7.2.3), the snubber valves shall be closed when the ram attains the test load and shall stay closed until either the movement of the pile exceeds 0.05 inch, or when the FLT (Section 7.2.3) with less than 0.05 inch has occurred. Movement (not gage pressure) is the testing criterion for the FLT (Section 7.2.3). However, if the gage pressure drops by more than 10% during the FLT (Section 7.2.3), the test shall be restarted. For the 24-hour load test, the gage pressure required shall be maintained throughout the test period.

8.5.9 During the underpinning operation, Subcontractor shall have available at the site all rams, gages, pumps, hose, and fittings required to simultaneouly lift the auxiliary building penetration area at the location of the permanent support. The total lifting capacity required is approximately 4,000 kips, which is to be distributed equally as specified in Section 6.2.3.

8.6 DETECTION OF MOVEMENT

8.6.1 General

Subcontractor shall submit and implement to the satisfaction of Contractor a procedure to monitor each structure which might be affected by the underpinning operations.

8.6.2 Horizontal movement detection points shall be located at or near the top and at the ground level of the structures. Vertical movement detection points shall be located near the ground level of the structures.

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8.6.3 Measurements shall be to the nearest 1/16 inch based upon survey observation.

8.6.4 Measurements shall be made daily during the underpinning operation, and Contractor shall be informed in writing if any movement is detected in excess of 1/16 inch of the previous reading or accumulated value of 1/8 inch.

9.0 WELDING

9.1 All welding shall be performed in accordance with AWS D1.1. Contractor shall approve all welding procedures.

9.2 All welders shall be qualified to the applicable welding procedures in accordance with AWS D1.1.

10.0 EXCAVATION

10.1 GENERAL

Subcontractor shall submit prior to excavation a detailed procedure describing the method of excavation. This procedure shall be to the satisfaction of Contractor and shall indicate coordination of the installation of lagging and bracing. The procedure shall also indicate the maximum amount of excavation below any previously installed lagging or structure. As a minimum, the excavation procedure shall include the following.

10.1.1 The location and dimension of each jacking pit

10.1.2 A flow diagram showing the excavation of all pits and the sequence of installation, testing, and locking off vertical support elements (caisson or jacked pile).

10.1.3 A table relating the information provided in Item 10.1.2 showing the following data with respect to each vertical support element:

a. Pile or caisson number

b. Pit number

c. Sequence number

d. Vertical capacity installed (kips)

e. Area undermined times 5.5 kips/ft (Note: The area undermined includes the disturbed but not excavated soil (influence zone) determined by a 1 horizontal to 3 vertical slope subtended from the bottom of the excavation prior to the installation of the lagging. Thus, a pit which is opened up to a depth of 6 feet prior to the installation of the lagging would have an influence zone which extended 2 feet beyond the edge of the excavation. If this same pit was subsequently (after installation of lagging) excavated to a greater depth, and lagging was installed as frequently as required by this specification, the influence zone would remain at 2 feet.)

f. Net overload (Item e) minus (Item d) (kips)

g. Percent net overload (Item f) times (100) divided by 8,300 kips

10.1.4 Sequence Control and Limits

In general, the objective is to install approximately 4,000 kips of underpinning resistance as close to the extreme end of the wings as possible. Subcontractor's plan shall be governed accordingly.

a. Piles which are adjacent to one another may be worked simultaneously provided the area of undermine is within allowable limits

b. Caissons which are adjacent to one another cannot be worked simultaneously. Where there are adjacent caissons, one must be excavated, concreted, tested, and stressed before work on any adjacent caisson can start. However, the first 7 feet below el 607' may be excavated in advance (if the space is required) for access to a nc adjacent caisson or pit.

10.2 Once an excavation is started, it must be worked continuously and without cessation until the lagging is in place and the lateral ground support restored.

10.3 Explosives shall not be used to dislocate and/or remove hardened material during excavation. Conventional tools, such as rock splitters and demolition tools, are acceptable.

10.4 Subcontractor shall submit the following procedure that is satisfactory to Contractor. This procedure shall describe in detail the measures, including hold points and inspection points, taken by Subcontractor to ensure slope stability and to prevent any movement of foundation material outside the excavation area.

10.5 Prior to excavation of material from under or within 7 feet from the foundations of the auxiliary or turbine buildings, the sand material shall be grouted. The pregrouted zone shall extend vertically from the underside of the existing foundation or mudmat to a depth of approximately 1 foot below the access excavation, or approximately 7 feet beyond the farthest lateral extent of the excavation.

10.6 The initial excavation below the bottom of the slab at el 607'+ shall not exceed 7 feet in depth. In addition, in no case shall the jacking pit exceed 7 feet in depth.

10.7 The percent net overload as established by Section 10.1.3 shall not exceed 10% during installation of approximately 4,000 kips of permanent support capacity under each electrical penetration room.

10.8 Subcontractor shall advance the mass excavation under the valve pit structures to the depth determined by Contractor. The bottom surface shall then be prepared for concreting as follows.

10.8.1 The surface shall be flat but sloped to a minimum of 1/4 inch per foot draining toward a sump located in one corner.

10.8.2 Subcontractor shall dispose of all water in the sump to the satisfaction of Contractor.

10.9 LAGGING INSTALLATION

10.9.1 All lagging material that will remain in the ground shall be metal or concrete. Any wooden lagging used for jacking pit lining or any other purpose must be removed. If removal of the lagging exposes earth, which will not be excavated, then the unremoved soil shall be grouted prior to removal of the lagging. The object of the grouting is to ensure intimate contact between the soil and the structure, and where granular soil is involved, to ensure the material in the zone of influence is self-supporting.

10.9.2 The initial excavation, where the earth is not supported, shall not exceed a depth of 4 feet if it is within 6 feet of the K line. Otherwise, a maximum depth of 7 feet shall be used.

10.9.3 After initial excavation, the lagging shall be installed and back packed or grouted. The lagging along the K line located below el 600.0' shall be grouted.

10.9.4 The excavation shall not proceed to a depth greater than 3 feet below the previously grouted lagging.

10.9.5 The procedure for installation of the lagging below el 600' along the K line shall indicate the maximum vertical distance between unpacked lagging and the bottom of the excavation. This distance shall not exceed 16 inches.

10.9.6 If squeezing ground (as defined in Soft Ground Tunneling with Steel Supports by Proctor and White) is encountered below el 600', Subcontractor shall employ a Chicago lagging system (near vertical-driven lagging) for advancing. The advance shall be limited to a maximum of one-half the length of the lag. After advancing one-half a lag, the lower breasting member shall be installed and the lagging wedged.

10.9.7 If running ground (as defined in Soft Ground Tunneling with Steel Supports by Proctor and White) is encountered below el 600', the zone shall be grouted in advance of excavation. Spieling is not a satisfactory procedure for running ground.

10.9.8 If flowing ground (as defined in Soft Ground Tunneling with Steel Supports by Proctor and White) is encountered, Subcontractor shall immediately close the opening from which ground is flowing, dewater the area to the extent that the ground behavior is changed to a running condition, and then proceed in accordance with Section 10.9.7.

11.0 CONCRETING

11.1 GENERAL

Subcontractor shall submit to the satisfaction of Contractor a detailed procedure, including hold points and inspection points, describing the placing of concrete under the structure. As a minimum, the placing and consolidating of the concrete shall be in accordance with Articles 11.0 and 12.0 of Specification 7220-C-231, unless otherwise specified herein.

11.2 Pile, pit, and caisson procedures are specified elsewhere.

11.3 Mass concrete shall first be placed in the corner opposite the sump such that any water shall be driven to the sump as the concrete is placed.

11.4 The first lift shall not exceed 2 feet in thickness; all subsequent lifts shall not exceed 5 feet in thickness.

11.5 Successive lifts shall be doweled into the preceding lift by using #8 bars on 18 -inch centers on a grit pattern. Dowels shall be located such that a minimum of 24 inches shall be embedded in both lifts.

Reinforcing steel will be furnished by Contractor.

11.6 The surface of the concrete lifts shall be horizontal within ± 3 inches for the entire area.

11.7 The top lift shall be placed to within 6 inches of the bottom of the existing slab. The remaining void between the lean concrete backfill and the foundation slab shall be either dry pack grouted or pressure grouted using a nonshrink grout. Subcontractor, shall submit a procedure to the satisfaction of Contractor, including hold points and witness points, describing in detail the method of dry pack grouting or pressure grouting.

11.8 Each lift of concrete shall be water cured for a minimum of 48 hours in accordance with ACI 318-1977 before continuing the placement.

11.9 Contractor will perform slump, percent air content, temperature, unit weight, and compressive strength cylinders on the lean concrete placed, and perform a compressive strength test on the grout used.

Subcontractor shall assist contractor when performing all grout and concrete testing.

11.10 Preceding the placing of concrete, Subcontractor shall install 2-inch styrofoam along the containment structure and 3-inch styrofoam between the containment and auxiliary buildings. Subcontractor shall submit a procedure for installing the styrofoam to the satisfaction of Contractor.

12.0 CLEANING AND RESTORATION

Subcontractor shall restore the work area to the same condition that existed prior to the start of operation and to the satisfaction of Contractor.

Subcontractor shall also submit a procedure on final cleaning.

13.0 QUALITY ASSURANCE REQUIREMENTS

13.1 GENERAL

The following operations are to be controlled in accordance with Subcontractor's approved QA program (Subcontractor's QA program shall be in accordance with Specification 7220-G-23, Appendix A, Attachment 2):

13.1.1 The design, materials, installation, testing, concreting, grouting, and all other incidentals for the permanent underpinning (caissons) of the auxiliary building penetration rooms

13.1.2 The excavation, mass concreting, and grouting under the valve pit structures

13.1.3 As a minimum, the following sections of this specification are under the scope of Subcontractor's QA program: 5.0, 6.1.4, 6.1.5, 6.1.6, 6.1.7, 6.1.8, 6.1.9, 6.1.11, 6.1.12, 6.2.1, 6.2.2, 6.2.3, 7.1, 7.2, 8.1, 8.2, 8.4.1, 8.4.2, 8.4.4, 8.5, 8.6, 9.0, 10.1, 10.4, 10.5, 10.6, 10.7, 10.9, and 11.0. These sections are applicable only with the work defined in Sections 13.1.1 and 13.1.2.

13.2 Because of the nature of the work, an independent overlay inspection will be performed by Contractor in accordance with this specification and Subcontractor's procedures.

14.0 MEASUREMENT OF PAYMENT

The measure of payment for this subcontract shall be in accordance with the pricing structure of the contract.

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Bechtel Power Corporation

Interoffice Memorandum

To: J. Wanzeck

Date: December 4, 1979

Subject: Job 7220 [redacted]t Comments on "U. S. Testing Comments of Bechtel Geo-Technical" "Review of U. S. Testing Field and Laboratory Tests on Soils"

From: J. Allen

Of: H&CF-Soils

Copies to: Stan Blue
Files: [redacted], 3410

At: Houston

1. The specification for soil testing was written for the use of the subcontractor (UST) and regardless of Bechtel responsibilities it is UST contractural duty to follow the specs. " Monitoring" usually means "looking over the shoulder of" not "directing."

2. At the bottom of Page 1 UST overlooks the fact that when fill soil is excavated and replaced it may not be represented by the original proctor since excavation would cover several soil layers, laid down under differnet conditions and possibly from different sources. A new proctor on the composite soil would have to be run in this case.

3. The first sentence at the top of Page 2 is unclear to me. I do not understand what it says.

4. Item 2, Page 2 is partially correct. The failed tests should have been reported at the time of failure, not days later as happened in some cases. It is difficult and very expensive to correct failed tests that were not reported immediately.

5. While it is true that specific gravity values varied widely on this project (from rock to sand to clay), any one material with similar description and proctor curves show very similar specific gravities. Therefore, I disagree with the UST statement at the bottom of Page 3 and the top of Page 4.

6. In reference to item 4, Page 4, I do not believe it is Bechtel's responsibility to check UST's arithmetic as implied in the last sentence.

7. In reference to item 5, Page 4, I disagree. However, I am not quite sure what the last sentence at the bottom of Page 4 says.

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8. In reference to item 6 Page 5, the nuclear density device was approved subject to verification by physical type tests.

9. The first sentence at the top of Page 6 indicates that it was a recent Bechtel idea to run wet and dry relative densities. ASTM D-2049 is clearly written and calls for wet and dry methods. The ASTM has not changed recently.

10. The method of determining relative density is well defined. I do not believe that Bechtel directed UST to use a different method, and I would like to review such instructions if they exist.

    Bechtel did not ask UST to evaluate field test data, but a competent soils testing technician or company representative is expected to recognize bad data, such as moisture-density points that plot above the zero-air-voids curve to which it is referenced.

J. H. Allen