Gallagher

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

October 5, 1979

Ms. Mary Sinclair
5711 Summerset Drive
Midland, Michigan 48640

Dear Ms. Sinclair:

This is in response to your letter dated September 11, 1979. We had previously seen the news article which you sent, and therefore we were aware of comments attributed to Congressman Albosta regarding letters received from workers at the Midland Site. He has not provided us with copies of these letters, but has informed our agency that he will provide information from these letters which is appropriate for NRC review.

Minor acts of vandalism at construction sites are not unusual, nor are they unique to nuclear power plants. It is possible that certain acts of vandalism have occurred at the Midland site of which we may not be aware. There are no NRC requirements for physical security programs at construction sites until such time as nuclear fuel is received onsite (a matter of concern to Congressman Albosta). Consequently, licensees are not required to inform the NRC of such matters. Safety systems are tested prior to operation, and any act of vandalism which could cause disruption of any safety system would likely be identified during the testing process.

Resolution of the diesel generator building settlement problem has not been finalized. Consumers Power has completed the preloading procedure on the diesel generator building. Measurements were made of the compression of soils beneath the diesel generator building under the preload conditions, and monitoring will be continued to determine the soil response. Based on this information, the settlement of the building will be projected for the life of the plant. This information will be factored into the final resolution of the problem.

We are not aware that any pipes beneath the diesel generator building have been sheared because of excessive differential settlement. We were aware that some of the pipes were stressed and somewhat deformed. As a part of the preloading process discussed in the previous paragraph, certain of the pipes were disconnected so that they would not prevent the building from settling under the pre-loading test conditions.

Please contact us if you have further questions.

Sincerely,

James G. Keppler

James G. Keppler
Director

cc: Congressman Donald Albosta

Midland PSAR info

2.11 Section 2.4 of Amendment 5 did not include the data and methods of analyses requested in Question 2.4 of Enclosure A to our letter dated September 26, 1969. This information is needed to complete our evaluation.

Answer:

Refer to Pages 2.4-1, 2.4-2 and 2.4-3 of Amendment No. 6 transmitted to Dr. Peter A. Morris on December 29, 1969.

2.14 In Section 5.1.11 of Amendment 5, you have stated that certain Class I components or piping will be founded or placed on the upper, loose sands. Justify the placement of Class I equipment on the loose sands considering densification from vibratory loading. Discuss the possibility and significance of relative differential settlement between structures or components.

Answer:

As noted in Section 5.1.11, certain Class 1 components and piping will be founded on the upper natural undisturbed sands or on controlled compacted fill placed above the undisturbed sands. Standard penetration tests and/or in-place density tests will be performed to identify for removal all loose sands. (For the purposes of this project, sands with a relative density of 50 percent or less are classified as loose sands to be removed.) The only sands remaining under Class 1 items will be undisturbed sands with a relative density greater than 50 percent. The controlled compacted fill is either a cohesive material or a granular material, the latter compacted to not less than 75 percent relative density as recommended in the Dames & Moore Report "Foundation Investigation and Preliminary Explorations for Borrow Materials," Page 16. Therefore, because loose sands will not be present in the undisturbed sand layer or the compacted fill, the question of densification of loose sands from vibratory loadings does not arise.

For those remaining sands with a relative density greater than 50 percent, some settlement under vibratory loads theoretically could occur. However, no significant settlements due to densification under vibratory loadings are expected because of the following reasons:

1. Vibratory loadings with limited duration (ie, earthquakes) are of insufficient length to significantly densify the relatively thin soil layer.

2. Vibratory loadings with a sustained duration (ie, machine vibrations) occur at only one Class 1 structure where the structure is founded over natural undisturbed sands. This structure (the Emergency Diesel Generator Building) is located at plant grade over approximately 25 feet of controlled compacted fill. The vibration effects will be largely dissipated through this 25-foot thick layer of fill and, thus, significant settlements in the underlying natural sands are not probable.

Settlement from densification of either the cohesive backfill or the granular backfill (placed with relative density greater than 75 percent) is not anticipated. *

Although settlement from densification under vibratory loadings is not foreseen, settlements from other load conditions, ie, overburden, structural loadings, etc, have been estimated and are summarized in answer to Question 8.0 of Amendment No. 6. However, as discussed in Paragraph 5.1.3, design provisions have been included to preclude overstressing of components due to differential settlement. For example, Class 1 piping will include sleeves and/or mechanical joints to insure flexibility where piping enters the structures.

4.0 The text of the Dames and Moore Report, titled "Report, Foundation Investigation and Preliminary Explorations for Borrow Materials, Proposed Nuclear Power Plant, Midland, Michigan, for Consumers Power Company," which was submitted as Amendment No. 1 to the application, indicates on Page 5 that you have been provided with the results of geologic studies made by others. Provide these results for our review. *

Answer:

The material referred to has been issued as a site report dated March 22, 1968. Six (6) copies were left with the DRL staff at a meeting in Bethesda, Maryland, in May 1968. Further, two (2) copies of this material were transmitted by Consumers Power letter by Mr. G. B. Matheney to Mr. J. Murphy dated August 15, 1969. This material is included as a formal part of our application. The material is comprised of:

1. "Seismic Measurements and Overburden Amplification Curves" by Western Geophysical Engineers, Inc.

2. Soils Exploration by Michigan Drilling Company dated October 19, 1956.

3. Soils Exploration by Michigan Drilling Company dated March 13, 1968.

7.0 Provide sub-surface profiles for all Class 1 structures and soil strata penetrated by the soil borings, as discussed at the July 24, 1969, meeting between representatives of Consumers Power Company and the DRL staff. (An example of such drawings has been presented in figures 8.2-1 and 8.3-1 through 8.3-6 of Amendment No. 5 to the Cook PSAR).

Answer:

Attached are Figures A7-1 and A7-2 showing subsurface profiles for all Class 1 structures except the service water intake structure, which is shown in Figure A9-1. These profiles are based on borings by Dames and Moore and Michigan drilling soils investigations.

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REASON

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8.0 As discussed at our July 24, 1969, meeting, provide information and calculations in support of your settlement tabulations on Pages 19 and 20 of the Dames and Moore Supplement to the Foundation Report, submitted as Amendment No. 3, to the application.

Answer:

The settlements tabulated on Pages 19 and 20 were evaluated from a consideration of the following conditions:

I. Settlements due to lowering of the water level to El 560 and pressure relief due to excavation of overburden soils above foundation level (short-term conditions).

II. Settlements due to placement of fill to grade and application of structural loads prior to flooding of the reservoir water level at El 600 (short-term condition).

III. Settlements due to fill and structural loads after reservoir filled water level at El 625 (long-term conditions).

For our settlement computations, a total of 72 settlement points were established on a grid and at selected structural locations as shown on Figure A8-1. Thirteen consolidation tests were performed for use in settlement analyses. The boring numbers and locations, and the elevations at which the consolidation tests were performed are also indicated on Figure A8-1.

Four loading areas were delineated for the Case I condition (Figure A8-2) and 17 loading areas were delineated for Cases II and III (Figure A8-1). Loading criteria, including net stresses at foundation elevation, are indicated on Figure A8-2 for the Case I condition and Table A8-1 for the Case II and III conditions based on the respective loading conditions, site soil conditions, and soil consolidation characteristics as evaluated from test data. Settlements at each of the 72 points were calculated utilizing an in-house computer program because of the variation in the thickness of the upper sands across the plant area.

Two computer runs were made for each case:

A. Soils consisting entirely of clays.

B. Upper 20 feet of soil consisting of sands which are underlain by clays.

The soil conditions in the plant construction area, as determined from test boring data, indicate that a sand layer of variable thickness overlies very stiff to hard silty clay. In portions of the Turbine Building and Auxiliary Buildings A and D, the sand is practically nonexistent. The maximum sand thickness in the area is 59 feet, in Boring 9, northeast of the reactor buildings. In the building area, the thickness of sand is generally less than 20 feet. Computer Run A (soils consisting entirely of clay) and Run B (upper 20 feet of soil consisting of sand, underlain by clay) were made to bracket the nonuniform soil conditions in the building area. Settlements at a specific point were then selected or interpolated from the Computer Run A and B values based on the estimated soil conditions, as determined from test borings, at that point. As indicated on Table A8-4, practically all of the evaluated settlements are the clay condition (Computer Run A) settlements. This occurs because excavation to building foundation levels will remove all of the sand with the exception of the Turbine Building area where some sand will remain. The above described method utilizing Computer Runs A and B to approximate actual site conditions is the most realistic approach to the settlement analyses.

Thus, the computer-run settlement analyses were made as follows:

1. IA - dewatering and excavation case, clay soil conditions.

2. IIA - fill and structural loading case prior to flooding reservoir, clay soil conditions.

3. IIIA - fill and structural loading case after reservoir filled, clay soil conditions.

4. IB - dewatering and excavation case, upper sandy soil conditions.

5. IIB - fill and structural loading case prior to flooding reservoir, upper sandy soil conditions.

6. IIIB - fill and structural loading case after reservoir filled, upper sandy soil conditions.

The Case I duration was 15 months; the Case II duration was 2 years. Case III was a permanent condition.

Computer printout sheets for the above six settlement calculations are attached as Figures A8-3 to A8-8. Results for a few of the settlement points are presented for purpose of illustration.

Our settlement computer program has been developed based on current soils engineering practice. Settlements at a point are computed by summing the individual compressions of soil slices of a predetermined thickness. The stress influence from all the loaded areas, as determined by the Boussinesq formula, is computed for each slice.

The compression of each soil slice is calculated by the following general forumla:

$$S = C \times T \times \log \left(\frac{P_o + \Delta P}{P_o} \right)$$

S = slice of compression

C = slope of consolidation curve (percent per log cycle)

P_o = overburden pressure

ΔP = total accumulated stress influence due to the loaded areas considered

T = thickness of soil slice

Values of C used in the computer settlement analyses were evaluated from the thirteen consolidation tests performed and correlations with other laboratory test data. Adjustments were made to the consolidation test curves to correct for sample disturbance. The evaluated values at various depths, for both virgin and recompression conditions, are presented on Figures A8-3 to A8-5 for the clay condition (Computer Run A) and on Figures A8-6 to A8-8 for the upper sand condition (Computer Run B). As noted on the figures, the values did not vary for the various cases analyzed.

Tabulations of the calculated settlements in the structure areas for Cases IA, IIA and IIIA are presented on Table A8-2. Similar tabulations for Cases IB, IIB and IIIB are presented on Table A8-3.

As noted in the tabulations, Cases IA and IB were adjusted for excavation relief and time effects, and those for Cases IIA and IIB for time effects (calculated settlements are ultimate settlements that must be modified for short-term loading conditions).

The adjusted settlements were then summed to obtain the total settlement at each point for the clay and upper sand condition. These settlements are also indicated on Tables A8-2 and A8-3. Based on estimated sand thickness as determined from borings in the plant area, settlements at specific points were then selected or interpolated from the appropriate values in Tables A8-2 and A8-3.

These resulting values are tabulated on Table A8-4. Based on the Westergaard stress distribution theory, these settlement values were modified by a factor of 2/3. The Boussinesq equations for calculating stresses are based on an elastic, isotropic, homogenous mass, whereas Westergaard's equations consider a stratified, nonisotropic condition. The test boring and laboratory data indicate that the soils at the site are nearer to the conditions upon which the Westergaard solutions are based. Therefore, it was concluded that the Westergaard stress distribution theory was more applicable than the Boussinesq theory for calculating stress distributions at the site.

Finally, the modified settlements were further evaluated in light of our past experience with similar soils to obtain the estimated settlements noted on Pages 19 and 20 of our report. These settlement tabulations are also presented in Table A8-4.

10.0 In reference to the compaction curve of brown fine sand, with some silt, from boring G at 5 feet as presented in plate A-9 of the Dames and Moore Report submitted in Amendment No. 1 to the application, provide the following information:

1. Is this curve reproducible using similar material from the same source?

2. Is this curve representative of the uppermost granular sand?

3. State the maximum and minimum relative densities as defined in ASTM designation D2049-64T of the material from which the compaction curve was obtained.

4. State the values of the dry density and moisture content from which this curve was constructed.

Answer:

1. The curve is essentially reproducible using similar material from the same source if similar test procedures are utilized. The results were obtained from compaction tests performed in accordance with ASTM Test Designation D1557-66T. Small variations in the values of maximum dry density and optimum moisture content should be expected, however, due to slight variations that exist even among similar soils, and to the scatter of results that is generally obtained when compacting fine sands.

2. The indicated curve is representative of the uppermost granular sand (fine sand with silt) encountered in various areas of the plant and cooling pond sites. Grain-size distribution tests performed on the upper fine sands in the area (see Plate A-10 presented in Dames and Moore Report submitted in Amendment No. 1) indicate the similarity of these sands and thus it is expected that the compaction characteristics would be similar.

3. A relative density test was not performed on the material from which the compaction curve was obtained. However, a relative density test was performed during a subsequent investigation at the site on a sandy soil which had similar grain-size characteristics. This test was performed in accordance with ASTM procedures on a fine sand with some silt, obtained from a boring located approximately 4000 feet west of Boring G, and the results were as follows:

Maximum Density	116 Lb/Cu Ft
Minimum Density	85 Lb/Cu Ft

The dry density and moisture content values from which the compaction curve was constructed are as follows:

Trial No.	Dry Density (Lb/Cu Ft)	Moisture Content (of Dry Density)
1	106.5	0
2	106.6	4.4
3	108.2	7.9
4	109.4	11.9
5	108.7	13.5

11.0 Indicate if the upper, natural, undisturbed sands will be used to support any critical appurtenances such as piping.

Answer:

The answer to this question is found in the answer to Question 5.1.11 in the Enclosure A to Peter Morris' letter to R. D. Allen dated September 26, 1969.

8. Criteria for Foundation Soils

The following comments by AEC soil consultants relate to the criteria to be used in foundation soils and were communicated by telephone to Consumers Power Company on March 13, 1970. Following each comment is a discussion which is intended to resolve the issue.

1. We would not concur with the conclusion stated by the applicant in Section 2.14-1 of Amendment No. 8 that vibratory loadings with limited durations (i.e., earthquake) are insufficient to significantly densify a thin soil layer. This conclusion does not take into account reported instances of densification of granular soils during earthquakes. The applicant has indicated that soils with relative densities less than 50% will be removed and replaced with compacted fill having a minimum relative density of 75%. In the absence of any analytical basis for the above procedure, it is our opinion that where Class I comparisons are to be founded upon upper sand layers, any sands with relative density less than 75% should be removed and replaced with compacted fill having a relative density of at least 75%.

Discussion:

The design criteria calling for the removal of all natural sands with a relative density of 50% or less was developed from published data as discussed in Section 5.1.11.

However, as a result of the concern of the DRL consultants for the adequacy of the 50% criteria, the design criteria for these Class I structures will be modified to remove all natural sands with a relative density of less than 75% and to replace these sands with a controlled backfill compacted in accordance with Page 16 of the report titled FOUNDATION INVESTIGATION AND PRELIMINARY EXPLORATIONS FOR BORROW MATERIALS PROPOSED NUCLEAR POWER PLANT dated March 15, 1969. In addition, beneath the non-Class I structures so sited that their failure could endanger the adjacent Class I structures, all in-situ sands with relative densitites less than 75% will also be removed. For example, in those areas of the Turbine Building adjacent to the Emergency Diesel Generator Building, existing sand will be removed if further tests show that the relative density of this sand is less than 75 percent. Please note that the sand depths beneath the Turbine Building are generally small, as shown in Fig. No. A7-2.

The updating of previously supplied PSAR information reflecting the 75% relative density foundation criteria will be supplied with a future amendment.

2. The discussion presented in Section 5.2.20-1 of Amendment No. 8 refers to the work of Seed and Idriss, but does not fully explain how the values of Young's Modulus were obtained. They appeared to be too high by about an order-of-magnitude: reference Barkan (1962) and the values computed from seismic velocities are valid at low stress levels and can therefore be considered to be upper bound. In summary, we cannot concur that the values of Young's Modulus presented by the applicant are conservative.

Discussion:

The Young's Modulus values listed in Appendix A to the FOUNDATION INVESTIGATION AND PRELIMINARY EXPLORATIONS FOR BORROW MATERIALS PROPOSED NUCLEAR POWER PLANT, dated March 15, 1969 are being reviewed and a relationship will be established between Young's Modulus and varying strains as determined both from field seismic surveys as well as laboratory tests performed on representative samples of cohesive foundation materials over a range of strains.

We note, however, that the Young's Modulus values based on the seismic surveys by Weston Geophysical, which can be considered as the upper bound, are 7×10^6 psf for the upper 50 feet and 63×10^6 psf for depths from 50 feet to 140 feet.

Pursuant to the AEC soil consultants' comments relating to the Young's Modulus (reference Item 2 of Page 8.00-2 included in Amendment No. 9 of this report), a further review was made of the moduli of elasticity values proposed for the plant design criteria. The following is a summary of the various studies conducted to establish the moduli of elasticity (E) values:

a. Dynamic E Based on Laboratory Testing - In mid-1968, two samples of undisturbed silty clay containing some sand and gravel were subject to cyclic triaxial tests to determine the dynamic modulus of elasticity of hard silty clay stratum which underlies the site at a depth of up to 30 feet below existing ground surface. Sample descriptions and test results are detailed below:

Boring Number	1	2
Sample Elevation (Ft)	533	562
Dry Density (PCF)	119	116
Moisture Content (%)	16	17
Confining Pressure (PSF)	5,000	3,000
Peak Shear Strength (PSF)	10,000	10,000
Poisson's Ratio (Assumed)	0.42	0.42
Dynamic E (PSF)	$E = 1.31 \times 10^6\ \varepsilon^{-0.14}$	$E = 1.22 \times 10^6\ \varepsilon^{-0.15}$

Based on these results, the dynamic modulus of elasticity of the soils supporting the containment vessels at their initially proposed locations was estimated to be 2.92×10^6 PSF, for an anticipated cyclic shear strain level of approximately 0.02 percent. An E value of 3×10^6 psf was included in the original soils report submitted with Amendment No. 1.

b. On-Site Seismic Work - Based on Western Geophysical Engineers, Inc, on-site seismic survey measured shear and compression wave velocities, soil properties and E values for the very low strain levels caused by seismic investigation work are as follows:

	From Ground Surface to Approximately 50 Feet Deep (Sand)	From Approximately 50 Ft to Approximately 140 Ft Deep (Silty Clay)
Dry Density (PCF)	110	135
Shear Wave Velocity (Ft/Sec)	850	2300
Compression Wave Velocity (Ft/Sec)	5200	6100
Poisson's Ratio	.49	.42
Modulus of Elasticity (PSF)	7.34×10^6	63×10^6

c. <u>Dynamic E Based on Seismic Survey and Published Data</u> - Although the above dynamic E value was based on laboratory tests, it appeared conservative in comparison with the results of the site seismic survey. Accordingly, in October, 1968, Dr. I. M. Idriss performed a reevaluation and Bechtel Corporation provided site seismic velocity data to assist the reevaluation. For a depth of approximately 50 to 140 feet below existing ground surface, the shear velocity was measured at approximately 2,300 feet per second. The shear modulus (G) calculated from this velocity was approximately 20×10^6 psf for the low strain levels of site seismic survey work. As this value corresponded reasonably well with the cyclic shear strain versus shear modulus divided by unconfined shear strength data published by Seed and Idriss in the December, 1968, issue of the Bulletin of the Seismology Society, the published data were used to correlate strain level with bulk modulus and thus dynamic E. Assuming that the shear stress resulting from an earthquake equaled the total weight of the column of soil above the depth in question by the maximum acceleration coefficient, then at a depth of 50 feet where the site soils have an average unconfined shear strength of approximately 8,00 psf, the dynamic E was determined equal to the following:

EQ Acceleration at Surface	Assumed Poisson's Ratio	E at 50 Feet Depth (PSF)
0.05 g	0.4	30×10^6
0.10 g	0.4	22×10^6
0.15 g	0.4	17×10^6

It was recommended that these E values be varied by plus or minus 50 percent during analysis to check the effect and allow for possible variation in E from the computed values. These results were incorporated in the soils supplement in PSAR Amendment No. 3.

d. <u>Short-Term Static E and Dynamic E Based on Additional Laboratory Testing</u> - Subsequently, static and dynamic laboratory testing was performed to develop more refined data. Two available undisturbed samples were subjected to a comprehensive testing by cyclic triaxial, resonant column, and static triaxial tests. Sample descriptions, soil properties and laboratory E values in terms of cyclic shear strain are tabulated below. No marked variation between laboratory static E and dynamic E was apparent from the test results.

Boring Number	14	15
Sample Elevation (Ft)	587.5	546.6
Soil Description	Gray Silty Clay	Gray Silty Clay With Some Sand & Gravel
Dry Density (PCF)	109	136
Moisture Content (%)	20.4	13.9
Confining Pressure (PSF)	6,000	6,000
Peak Shear Strength (PSF)	7,500	3,600
Poisson's Ratio (Assumed)	0.42	0.42
Dynamic or Static E (PSF)*	$E = 1.00 \times 10^6 \varepsilon^{-0.53}$	$E = 0.32 \times 10^6 \varepsilon^{-0.48}$

*where ε is shear strain.

Based on these test results, the laboratory dynamic or short-term static E values for various shear strain levels are as follows:

Shear Strain (Percent)	Dynamic or Short-Term E, (PSF)	
	Boring 14 at 587.5	Boring 15 at 546.6
1.0	1.0×10^6	0.32×10^6
0.1	3.4×10^6	1.0×10^6
0.01	11.5×10^6	3.0×10^6
0.001	39.2×10^6	9.0×10^6

An overall review of the previously outlined field and laboratory testing and analyses indicates the following:

1. The initial dynamic E value of 2.92×10^6 developed by the first set of dynamic triaxial tests is conservative and has not been substantiated by subsequent studies.

2. The dynamic E values calculated by Idriss are the E values that would be expected for the site soils on the basis of their strength characteristics. For an acceleration of 0.12 g, the dynamic E value at a depth of 50 feet should be assumed equal to 20×10^6 psf ± 50 percent. This value assumes the cyclic shear strain will be approximately 0.005 percent.

3. The final set of laboratory tests indicates that the laboratory static and dynamic E values are approximately equal for short-term loading conditions. At least one of these tests indicates similar E values at comparable strain levels as would be expected from (2) above.

4. The field seismic survey indicates that the upper limit of the dynamic E is on the order of 60×10^6 psf for very low strains.

As the average unconfined shear strength of the in situ soils supporting the subject foundations is approximately 8,000 psf, it is considered appropriate to increase laboratory E values in proportion to their unconfined strengths to give interpolated E values for soil with an 8,000 psf strength. Considering all other data available, averaging test results and adjusting from laboratory measured E values to probable field values, by a correction factor of 1.5, the field E value for various strain levels is estimated to be as follows:

Cyclic Shear Strain (Percent)	E (PSF)
.001	45×10^6
.01	14×10^6
.1	4.4×10^6
1.0	1.3×10^6

The E value is related to cyclic shear strain in the above values by the equation:

$$E = 1.3 \times 10^6 \, \varepsilon^{-0.5}$$

As the cyclic shear strain during seismic loading will be in the range of 0.001 percent to 0.01 percent, it is concluded that the E value used in seismic design (22×10^6 ±50 percent psf) is appropriate, and that the initial E values developed in 1968 (2.92×10^6 psf) should be disregarded.

E

FSAR

Consumers Power Company

Stephen H. Howell
Senior Vice President

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

November 2, 1978
Howe 227-78

Director of Nuclear Reactor Regulation
Attn: Mr Roger Boyd, Director
Division of Project Management
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
DOCKET NO 50-329, 50-330
RESPONSES TO SUPPLEMENTAL REQUESTS FOR
ADDITIONAL INFORMATION: PART 2
FILE: 0485.11 SERIAL: 6026

Enclosed with this letter are Consumers Power Company's responses to Supplemental Requests for Additional Information: Part 2, transmitted by Mr S A Varga's letter of October 13, 1978. In addition, responses from previous requests for additional information are provided where new updated information has become available.

Responses are provided via letter format to meet your schedule date for receipt of responses from Supplemental Q-1's by November 6, 1978 so that Regulatory Staff Positions (Q-2's) can be issued by the NRC Staff on December 1, 1978.

The enclosure contains printed pages containing responses and updated FSAR pages. New and updated information provided with the enclosure is marked as Revision 15. Information contained in the enclosure will be submitted as Revision 15 to the Midland Plant Units 1 and 2 FSAR as an amendment to the Company's application for construction permits and operating licenses on November 30, 1978.

Should changes occur that necessitate revising the technical content in the enclosure between now and the November 30, 1978 amendment, they will be clearly designated in the November 30, 1978 submittal to facilitate staff review.

We are available to discuss these and previous responses should the staff find it desirable to do so prior to the issuance of Regulatory Staff Positions on December 1, 1978.

Stephen H Howell

SHH/jbg

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

December 11, 1978

Docket Nos. 50-329
50-330

Mr. S. H. Howell, Vice President
Consumers Power Company
212 West Michigan Avenue
Jackson, Michigan 49201

Dear Mr. Howell:

SUBJECT: STAFF POSITIONS AND REQUESTS FOR ADDITIONAL INFORMATION (PART 1)

During the course of our review of Midland Plant Units 1 and 2, we have adopted several positions that differ from those in your FSAR. We also find that we need additional information in some areas to complete our evaluation. Positions not provided in previous correspondence and further information requests are contained in Enclosure 1.

Several of your responses to our previous positions and requests were not provided to our established schedules and some of our technical review branches have been unable to adjust other workload assignments to review the delayed information recently provided. Still other branches have found that issuance of positions must await receipt of information previously requested. Accordingly, additional staff positions will be issued as they become available. We presently anticipate issuance of additional positions in mid-December and late-December, 1978.

We will need response and resolution to Enclosure 1 by January 19, 1979. If you cannot meet this date, inform us within seven days after receipt of this letter so that we may revise our schedule accordingly.

Should you desire a meeting to clarify Enclosure 1 or to discuss preliminary responses, please contact me.

Sincerely,

Steven A. Varga

Steven A. Varga, Chief
Light Water Reactors Branch 4
Division of Project Management

Enclosure: As stated

cc: Listed on following page

7901020099

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ENCLOSURE 1

STAFF POSITIONS (Q-2s) AND REQUEST FOR ADDITIONAL INFORMATION

PART 1

MIDLAND PLANT UNITS 1 & 2

These positions and requests for additional information are numbered such that the three digits to the left of the decimal identify the technical review branch and the numbers to the right of the decimal are the sequential request numbers. The number in parenthesis indicates the relevant section in the Safety Analysis Report. The initials RSP indicate the request represents a regulatory staff position.

Branch Technical Positions referenced in these requests can be found in "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-75/087.

* 130.0 STRUCTURAL ENGINEERING BRANCH

130.21 (3.8) (2.5) Provide an evaluation of the ability of those seismic Category I structures which are located upon backfill and which are experiencing settlement in excess of that predicted, to withstand appropriate loading combinations, including SSE, throughout plant life. Describe how stresses associated with differential settlement of the structural foundations and any corrective preloading activities have been or will be factored into these evaluations. Also provide a comparison of the stresses predicted due to settlement to those allowable stresses permitted by the ACI Code.

362.0 GEOTECHNICAL ENGINEERING

362.11 (2.5) The March 15, 1969 report by Dames & Moore for foundation investigation and preliminary exploration for borrow materials which is included in your PSAR provided final foundation design criteria, including:

> "d) Recommended foundation type and estimated total settlement for the auxiliary building which is located between the two reactor buildings. Its structure and foundation will be separate from those of the adjacent three buildings to allow for possible differential settlement which must not exceed 3/4 inch." [Emphasis added]

The June 28, 1968 report by Dames & Moore on this same subject also states their understanding that the maximum allowable differential settlement between the radwaste building and the adjacent reactor containment building is 3/4 inch.

Provide documentation that this maximum differential settlement between buildings has not and will not be exceeded throughout plant life.

362.12 .5.4) Describe your preloading program which is planned to further consolidate backfill material underneath the Diesel Generator Building. Include your schedule for these activities.

362.13 (2.5.4) Provide your program for reassessing the properties of the backfill materials after completion of the preloading program of request 362-12. This program should differentiate between:

1. Areas affected by the vertical conduits in the Diesel Generator Building area, and

2. Areas not affected by the conduits.

Also, provide your program for confirming the dynamic characteristics of the fill materials used in seismic analyses of supported structures. Include your schedule for this program.

GENE GALLAGHER

ADDITIONAL INFORMATION Midland

Reg. III

DISTRIBUTION:

FSAR

NRC PDR
Local PDR
Docket file
Branch file
RSBoyd
DBVassallo
FJWilliams
SAVarga
Project Manager Darl Hood

QUESTIONS ON SETTLEMENTS

MService
RJMattson
JKnight
RTedesco
RDeYoung
VAMoore
RHVollmer
MLErnst
RPDenise
OELD
IE (3)

bcc:
JRBuchanan
TBAbernathy
ACRS (16)

JAN 2 1979

RESAR-414

Midland 50-329/330

BACKUP

REQUEST FOR ADDITIONAL INFORMATION MIDLAND

2/24/78

Questions on FSAR

Distribution:
NRC PDR
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R. S. Boyd
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Project Manager DARL HOOD
M. Service
R. J. Mattson
D. Ross
J. Knight
R. Tedesco
H. Denton
V. A. Moore
R. H. Vollmer
M. L. Ernst
W. P. Gammill
W. McDonald
~~ELD~~
IE (3)
W. Haass

bcc: ACRS (16)
T. Abernathy
J. R. Buchanan

PART #1

(Part #2 + #3 is attached with this part - please do not separate - Jaiden)

PART #1 - 2/24/78
PART #2 - 3/15/78
PART #3 - 4/21/78

1978

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

February 24, 1978

Docket Nos.: 50-329 & 50-330

Consumers Power Company
ATTN: Mr. S. H. Howell
Vice President
212 West Michigan Avenue
Jackson, Michigan 49201

Gentlemen:

SUBJECT: REQUEST FOR ADDITIONAL INFORMATION - PART ONE

In continuing our review of the FSAR for Midland Plant Units 1 & 2, we find we need additional information to complete our evaluation. This information request is contained in Enclosure 1.

The information requests provided in Enclosure 1 use a sequential numbering system continuing from those following our acceptance review and provided by our letter of November 11, 1977. As indicated in our letter of December 27, 1977, we have scheduled our round-one requests in three separate parts for which this is the first part. Enclosure 1 is based upon our review of FSAR revision numbers three or four.

We will need complete and adequate responses to Enclosure 1 by April 14, 1978. If you cannot meet this date, inform us within seven days after receipt of this letter so that we may revise our schedule accordingly.

Some of our requests also represent Regulatory Staff Positions and are identified by the initials RSP. If, during the course of our review, you believe there is a need to appeal a staff position because of disagreement, this need should be brought to our attention as early as possible so that the appropriate meeting can be arranged on a timely basis. A written request is not necessary and all such requests should be initiated through our staff project manager assigned to the review of your application. This procedure is an informal one, designed to allow oportunity for applicants to discuss, with management, areas of disagreement in the case review.

Please contact us if you desire clarification or other discussions of the information requested.

Sincerely,

for S. A. Varga, Chief
Light Water Reactors Branch No. 4
Division of Project Management

Enclosure: As Stated

~~8006110429~~

Consumers Power Company

ccs:

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Enclosure 1

REQUEST FOR ADDITIONAL INFORMATION (Q1s)

PART 1 of 3

MIDLAND PLANT UNITS 1 & 2

These requests for additional information are numbered such that the three digits to the left of the decimal identify the technical review branch and the numbers to the right of the decimal are the sequential request numbers. The number in parenthesis indicates the relevant section in the Safety Analysis Report. The initials RSP indicate the request represents a regulatory staff position.

Branch Technical Positions referenced in these requests can be found in "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants," NUREG-75/087 dated September 1975.

362.0 GEOTECHNICAL ENGINEERING

362.1 (2.5.4.5.3) Provide a summary of the results of field density tests for compaction and moisture control of structural fill beneath and adjacent to Category I structures.

362.2 (2.5.4.5.1) Question 1 and the resulting discussion on page 8.00-1 included in Amendment Number 9 to your PSAR stated that all natural sands with relative densities less than 75% would be removed beneath all Class I structures and beneath non-Class I structures so sited that their failure could endanger the adjacent Class I structures. Discuss the methods employed in mapping and removing the sands having less than 75% relative density. Provide plan and sectional figures showing the areas where these materials were removed. Figure A9-2 of the PSAR which displays sub-surface profiles of Class I piping should be updated to show removal of sands of less than 75% relative density and be presented in the FSAR. Figure 2.5-21 of the FSAR shows loose sands beneath the Class I tanks although they were to have been removed. Explain this inconsistency, and provide proper documentation of as-built conditions.

362.3 (2.5.4.10.2.3) Reference is made in section 2.5.4.10.2.3 to Table 2.5-14 for design values of passive pressure. The table number is incorrect and should read Table 2.5-15.

362.4 (2.5.4.13) Provide the results of all benchmark survey measurements taken during construction. Graphically, compare the measured results to predicted settlements. Provide a commitment and schedule to submit the results of future survey settlement measurements.

362.5 (2.5.6.4.2) Provide gradation curves for the 12 inch thick crushed rock bedding layer beneath the riprap. Discuss the adequacy of the bedding material with respect to the requirements of a filter.

362.6 (2.5.6.5.3) Provide figures showing the failure surfaces that resulted in the minimum computed factors of safety for all slope stability conditions studied.

362.7 (2.5.6.5.4) Paragraph four of section 2.5.6.5.4 states that the outer slope of cross-section I was used to simulate the plant area fill and a seismic coefficient of .12g was used. However, Table 2.5-20 indicates that cross-section G was used for this condition. Explain and correct this inconsistency.

362.8 (2.5.6.8) Provide a detail of a typical piezometer as installed in the cooling pond dike. Also provide cross sections showing the development of the phreatic surface from initial piezometric head to full pond steady-state condition and a comparison to the phreatic surface assumed for the stability analysis of the steady-state condition.

FSAR

CONSUMERS POWER COMPANY

APPLICATION FOR

REACTOR CONSTRUCTION PERMIT AND OPERATING LICENSE

DOCKET NO. 50-329
DOCKET NO. 50-330
AMENDMENT NO. 50

Enclosed herewith, revising and supplementing the above-entitled application, are revised pages for incorporation in the Final Safety Analysis Report. The Final Safety Analysis Report was submitted with Amendment 33 to the above dockets on November 18, 1977. The enclosed material consists of the following:

1) A revised description of the Reactor Building Spray System showing the replacement of sodium hydroxide and sodium thiosulfate additives with hydrazine and disodium phosphate additives has been partially incorporated into the FSAR text, tables, and figures. Completion of these revisions will be made in subsequent amendments.

2) Appendix 7A has been added to provide a failure modes and effects analysis for the safety related portion of the Control Rod Drive Control System.

3) Additional information the FSAR stated would be submitted at this time.

4) Changes in various FSAR sections resulting from routine design developments.

5) Reduction of the page size for selected "11 x 17" tables.

6) Correction of minor errors and omissions.

7) Changes relating to the above (Tables of Contents, Figures, Tables, etc.).

These new and revised pages bear the notation "Revision 13 8/78", and are marked in the margin to indicate where changes have been made. Additional pages and figures have been added as reflected on the revised Midland Plant FSAR "List of Effective Pages".

Consumers Power Company

Dated: August 29, 1978

by /s/ Stephen H. Howell
Stephen H. Howell, Vice President

Sworn and subscribed to before me on this 29th day of August, 1978.

(S E A L)

/s/ Beverly A. Avery
Notary Public, Jackson County, Michigan
My commission expires March 14, 1981.

<u>Question 362.2</u> (2.5.4.5.1)

Question 1 and the resulting discussion on Page 8.00-1 included
in Amendment Number 9 to your PSAR stated that all natural sands
with relative densities less than 75% would be removed beneath
all Class I structures and beneath non-Class 1 structures so
sited that their failure could endanger the adjacent Class 1
structures. Discuss the methods employed in mapping and removing
the sands having less than 75% relative density. Provide plan
and sectional figures showing the areas where these materials
were removed. Figure A9-2 of the PSAR which displays subsurface
profiles of Class 1 piping should be updated to show removal of
sands of less than 75% relative density and be presented in the
FSAR. Figure 2.5-21 of the FSAR shows loose sands beneath the 8
Class 1 tanks although they were to have been removed. Explain
this inconsistency, and provide proper documentation of as-built
conditions.

<u>Responses</u>

Subsection 2.5.4.5.1 has been revised in response to this
question. The request to provide plan and sectional figures of
areas where the loose sands were removed will be responded in
more detail by amendment in October 1978. 13

estion 362.1 (2.5.4.5.3)

rrovide a summary of the results of field density tests for compaction and moisture control of structural fill beneath and adjacent to Category I structures. 8

Response

Subsection 2.5.4.5.3 has been revised in response to this question.

2.5.4.3.2 Exploration Programs (Q&R 362.2)

The borings taken for this project are plotted in Figures 2.5-16, 2.5-17 2.5-35, and 2.5-36 and also arranged by area in Table 2.5-8. Several preconstruction field exploration programs were undertaken between 1956 and 1969. The major part of the subsurface investigation for plant foundation, cooling pond dike, railroad bridge, and other related facilities at the plant site and borrow source was done by Dames & Moore[58] during the period 1968-1969, and by the Michigan Drilling Company[58] during the year 1968. Additional borings were made in 1956 by Michigan Drilling, and in 1967 by Dames and Moore, for Dow Chemical Company.

Exploration programs were also performed during the plant construction period of 1969 to 1974 for various specific purposes.

a. The borings designated by the letter "C" were performed by Walter Flood Company under the supervision of Bechtel in 1969 and 1970. These were made in conjunction with the cooling pond dike construction operations. They were intended to identify sand pockets, determine the depth of the dike cutoff trench, and estimate the location and extent of the slurry trenches.

b. A series of borings designated by the letter "D" were performed by Canonie Construction Company under the supervision of Bechtel in 1970, along the originally planned route of Seismic Category I buried piping, mainly to locate loose sand pockets for liquefaction evaluation. A series of borings designated by the letters DG, T, Q, CT, CL, DF, and TR was performed by Raymond International under the supervision of Bechtel in 1978 over the entire plant area to locate any loose 15
sand pockets after the plant construction. These are listed in Table 2.5-25.

c. Two exploration programs supervised by Bechtel were performed by Soil and Materials Engineers and Raymond Drilling Company in August 1973. These borings are designated by hole numbers 800 and the 900 series. They were made to evaluate the possible changes due to frost action and flooding on the in-place foundation soils for plant excavation and the partially completed northeast dike that might have occurred during the construction shutdown period from May 1970 to August 1973. Also, several borings were drilled to inspect the completed foundation dike materials.

d. A series of borings designated by the letter "M" were performed by Raymond International Company under the supervision of Bechtel in 1974 in the Mergard property area for borrow source investigation.

e. Other letter designated borings presented in Table 2.5-8 were made by Raymond International Company and supervised by Bechtel in 1974 for the foundation investigation of the makeup water pump structure, the service water pump structure, the river intake structure, and the cooling tower.

Appendix 2A contains a tabulation of the bore hole information, which includes depth of boring, ground surface elevation, purpose of boring, type of drilling, and number and type of samples taken. The borings have been arranged numerically according to three major groupings: Bechtel supervised borings, Dames & Moore borings, and Michigan Drilling Company borings. The boring logs are given in Appendix 2A following the tabulation.

TABLE 2.5-25

PENETRATION RESISTANCE OF
NATURAL SAND IN PLANT AREA

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type
I. Borings made in 1970 during construction			
D-1	595	30	SM
	590	85	SM
	589	95	SM
	586	100	SM
D-2	594	100	SP
	590	100	SP
	586	100	SP
D-4	600	22	SP
	595	64	SP
	590	100	SP
	588	98	SP
D-9	595	78	SP
	590	61	SP
	585	80	SP
D-10	597	39	SP
	591	72	SP
	587	100	SP
	582	76	SP
D-11	597	92	SP
	592	91	SP
	587	100	SP
D-12	597	34	SP
	592	90	SP
	587	84	SP
	582	100	SP
D-13	600	39	SP
	595	41	SP
	590	84	SP
	585	86	SP
	580	91	SM
D-14	600	29	SP
	595	56	SP
	590	83	SP
	585	84	SP

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TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type
D-15	600	25	SP
	595	77	SP
	590	98	SP
	585	70	SP
D-16	597	49	SP
	592	53	SP
	590	85	SP
	586	60	SP
D-17	595	30	SP
	590	47	SP
	585	57	SP
	580	90	SP
D-22	599	15	SP
	594	64	SP
	590	89	SP
	588	87	SP
	585	100	SP
	583	100	SP
D-24	594	80	SP
D-31	600	40	SP
	595	48	SP
D-32	600	40	SP
	582	69	SP
	580	100	SP
D-33	595	42	SP
	590	72	SP
D-41	600	18	SP
	595	26	SP
D-42	600	78	SP
	595	71	SP
	590	21	SP
	585	100	SP
	580	100	SP
D-45	601	18	SP
	596	41	SP
	595	100	SP
	591	23	SP

15

TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type
D-47	600	41	SP
	595	74	SP
	593	68	SP
	590	77	SP
	587	90	SP
D-48	595	5	SP
	590	29	SM
	585	100	SM
	580	100	SM
	575	63	SM
D-48A	603	20	SP
	599	57	SP
	596	76	SP
	594	91	SP
	591	89	SP
D-49	600	39	SP
	595	78	SP
	593	57	SP
	590	78	SP
	587	100	SP
D-51	601	22	SP
	596	54	SP
	595	82	SM
	592	29	SP
D-52	600	18	SP
	596	33	SP
	589	83	SM
D-53	600	33	SP
	595	73	SP
	592	89	SP
	590	83	SP
	587	79	SP
D-56	602	26	SP
	600	78	SP
	597	93	SP
	594	86	SP
	591	81	SP

15

TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type
D-57	602	22	SP
	600	54	SP
	597	83	SP
	594	86	SP
	591	81	SP
D-58	602	16	SP
	600	27	SP
	597	85	SP
	595	81	SP
	591	86	SP
	590	80	SP
D-59	602	16	SP
	600	37	SP
	597	86	SP
	595	90	SP
	592	87	SP
	590	85	SP
D-60	601	37	SP
	599	91	SP
	596	92	SP
	593	89	SP
	591	84	SP
1-MICH	607	9	SP
	604	18	SP
	602	6	SP
II. Borings made in 1978 after plant area fill			
DG-1	606	93	SM
	604	91	SM
	602	150	SP
	598	100	SP
	593	100	SP
	588	100	SP
	583	100	SP
DG-2	605	40	SP
	604	100	SP
	599	100	SP
	594	100	SP
	589	100	SP
	584	100	SP

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TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type
DG-3	601	100	SP
	599	100	SP
	593	100	SP
	588	100	SP
	583	100	SP
DG-5	605	57	SP
	604	55	SP
	598	100	SP
	593	100	SP
	588	100	SP
DG-7	604	17	SP-SW
	603	25	SP-SW
	601	17	SP-SW
	600	10	SP-SW
	599	15	SP-SW
DG-8	606	57	SP
	604	50	SP
	599	26	SP
	596	34	SP
DG-9	603	21	SP
	599	24	SP
DG-11	606	68	SP
	604	57	SP
	599	72	SP
DG-12	599	69	SP
	594	100	SP
	589	100	SP
DG-13	604	39	SP
	601	44	SP
	599	31	SP
	594	51	SP
DG-14	606	100	SP
	603	100	SP
	598	74	SP
DG-15	606	66	SP
	602	100	SP
	597	66	SP

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TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type	
DG-16	606	37	SP	
	603	68	SP	
	598	100	SP	
DG-17	603	35	SP	
	602	77	SP	
	599	52	SP	
DG-19	602	59	SP	
	597	100	SP	
DG-20	600	34	SP	
	597	100	SP	
DG-21	603	78	SP	
	597	104	SP	
DG-23	604	85	SP	
	599	57	SP	
	594	33	SP	15
	589	100	SP	
DG-25	603	76	SP	
	600	154	SP	
DG-27	600	41	SM	
DG-28	601	9	SP	
	599	37	SP	
	596	89	SP	
DG-29	602	26	SP	
	592	70	SP	
T-1	602	61	SP	
	596	100	SP	
	591	100	SP	
	587	100	SP	
	581	100	SP	
T-2	601	80	SM	
T-4	593	65	SP	
	588	100	SP	
	583	76	SP	

TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type
T-8	595	100	SP
	590	100	SP
T-9	601	66	SP-SM
T-10	586	100	SP
	582	100	SP
	577	100	SP
	572	100	SP
T-12	599	138	SP
	597	131	SP
T-13	593	140	SP
	588	100	SP
T-14	599	197	SP
	595	103	SP
	589	100	SP
T-16	597	165	SM
	593	183	SM
	588	100	SM
T-18	598	171	SP, GP
CT-1	603	11	SP
	600	24	SP
	598	29	SP
	593	40	SP
	588	49	SP
CT-5	603	140	SP
DF-2	600	59	SP
	595	59	SP
TR-7	599	155	SP
C-1	613	68	SP
	608	33	SP
Q-1	595	156	SP
Q-2	601	32	SP
	596	102	SP
	591	54	SP

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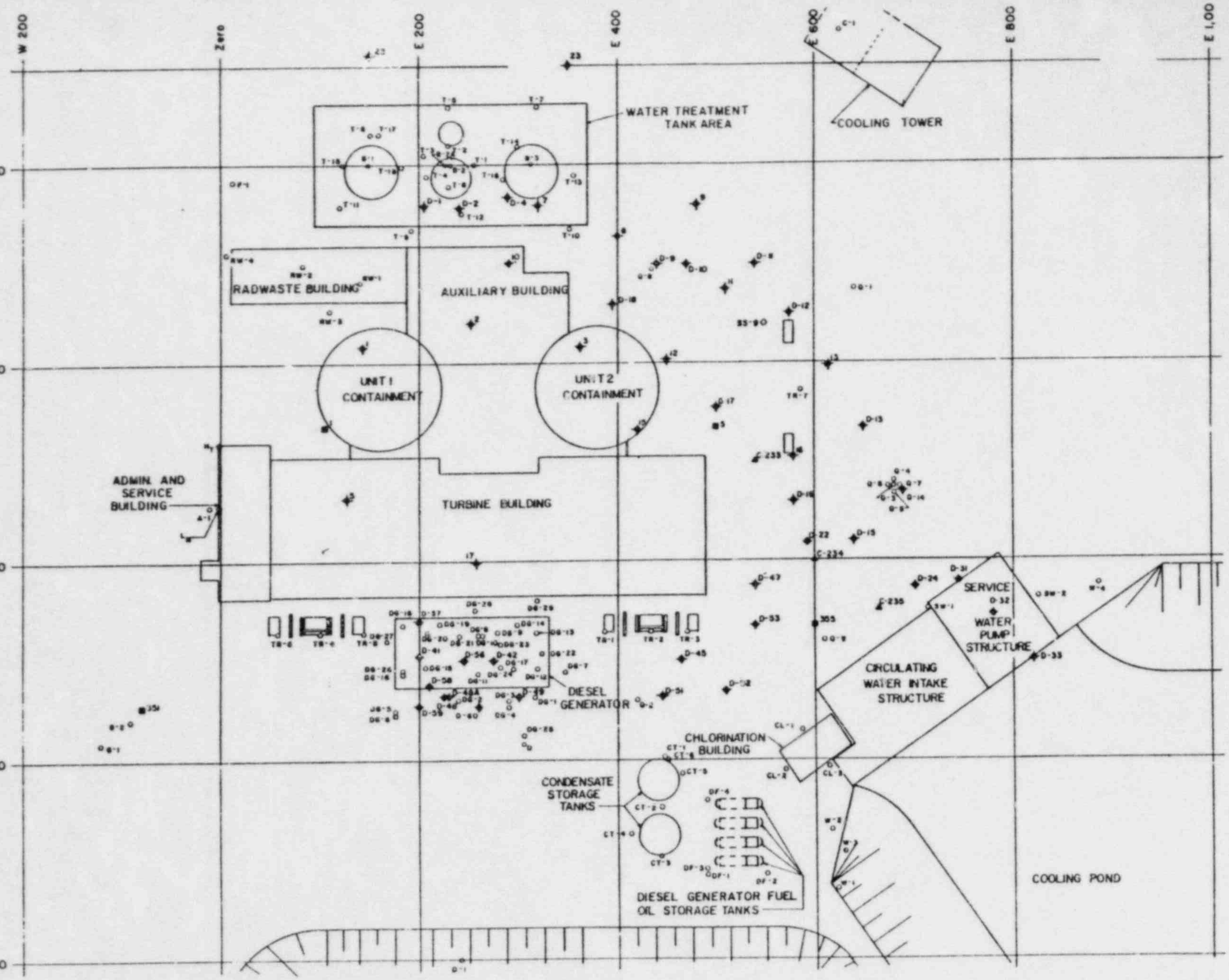
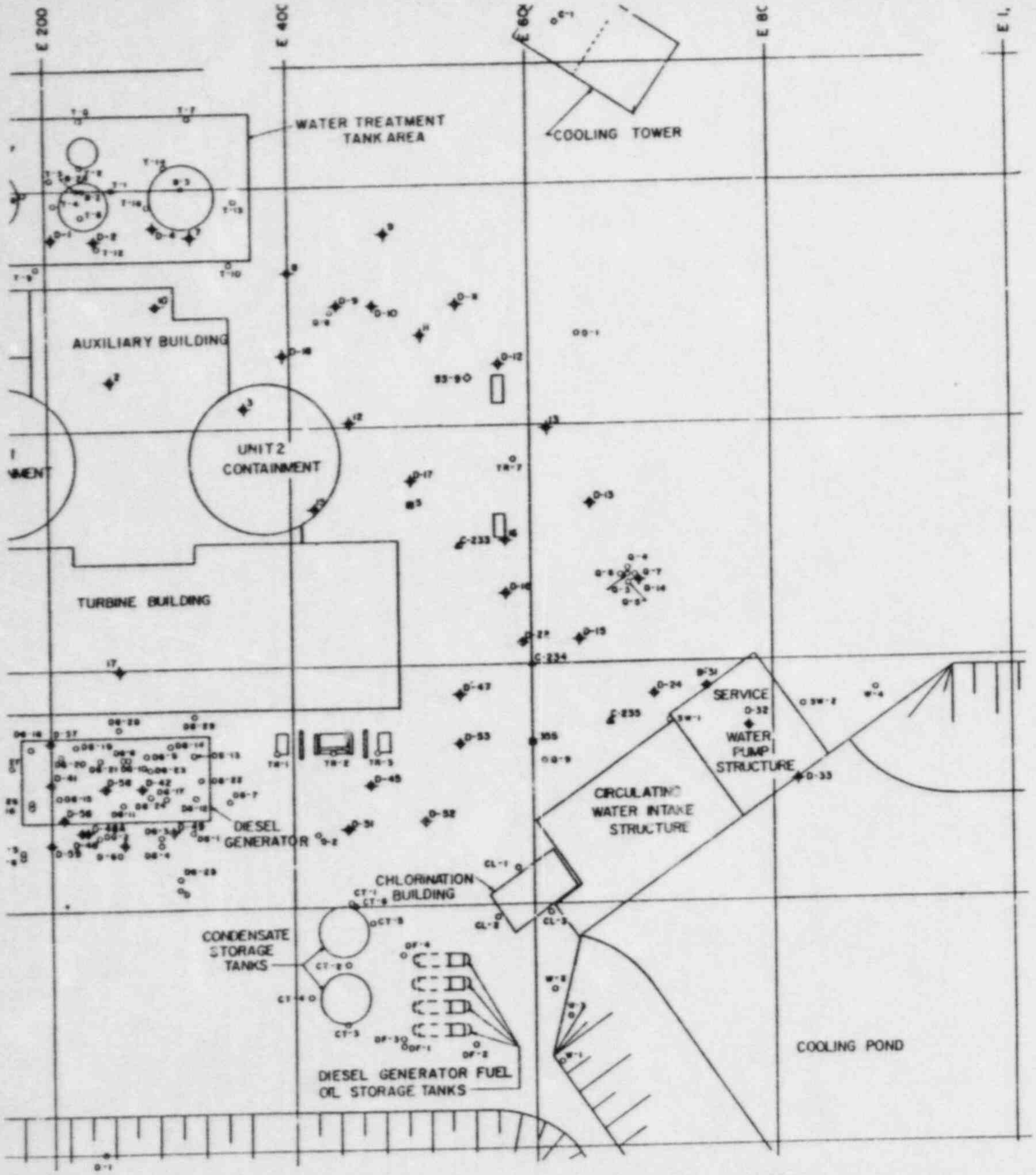
TABLE 2.5-25 (continued)

Boring Number	Elevation (ft)	Blowcount (blow/ft)	Material Type	
Q-3	595	100	SP	
Q-8	595	105	SP	
	590	108	SP	
	585	95	SP	
	576	100	SP	
RW-3	590	100	SP-SM	
W-1	599	100	SP	
	594	100	SP	
W-3	597	54	SP	
	594	56	SP	
	589	100	SP	15
CL-1	598	100	SP	
CL-2	602	81	SP	
CL-3	597	76	SP	
LN	583	100	SP	
	581	100	SP	
	579	100	SP	
HT	582	111	SM	
E	600	88	SP	
	597	81	SP	
D	604	107	SM	
	601	113	SM	
	598	102	SM	



W 200
Zero
E 200
E 400
E 600
E 800
E 1,00
S 4,600
S 4,800
S 5,000
S 5,200
S 5,400
WATER TREATMENT TANK AREA
COOLING TOWER
RADWASTE BUILDING
AUXILIARY BUILDING
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
ADMIN. AND SERVICE BUILDING
TURBINE BUILDING
DIESEL GENERATOR
SERVICE WATER PUMP STRUCTURE
CIRCULATING WATER INTAKE STRUCTURE
CHLORINATION BUILDING
CONDENSATE STORAGE TANKS
DIESEL GENERATOR FUEL OIL STORAGE TANKS
COOLING POND

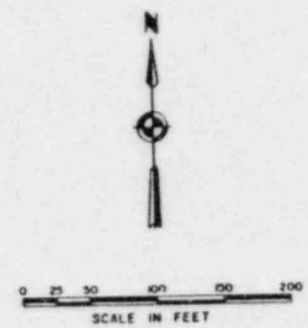


EXPLANATION

- ■ Michigan Drilling Company Borings: 1956 & 1968
- ◆ Dames & Moore Borings: 1968 & 1969
- ◆ Bechtel Borings: 1970
- ▲ Waller Flood Company Borings: 1969 & 1970
- ○ Bechtel Borings: 1977 & 1978
- ○ Bechtel Borings; August through October, 1978

NOTES:

1. The Bechtel Borings, August through October, 1978, were drilled to determine whether the natural sand pockets with relative density less than 75% have been removed or not
2. All Borings drilled prior to 1977 as shown indicates the locations with sand pockets (See FSAR Figures 2.5-40, Rev. 1)



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

All Plant Area Boring Locations to Determine the Removal of Natural Sands with Relative Density Less than 75%

FSAR Figure 2.5-40A

11/78 Revision 15

FSAR

Question 362.2 (2.5.4.5.1)

Question 1 and the resulting discussion on Page 8.00-1 included in Amendment Number 9 to your PSAR stated that all natural sands with relative densities less than 75% would be removed beneath all Class I structures and beneath non-Class 1 structures so sited that their failure could endanger the adjacent Class 1 structures. Discuss the methods employed in mapping and removing the sands having less than 75% relative density. Provide plan and sectional figures showing the areas where these materials were removed. Figure A9-2 of the PSAR which displays subsurface profiles of Class 1 piping should be updated to show removal of sands of less than 75% relative density and be presented in the FSAR. Figure 2.5-21 of the FSAR shows loose sands beneath the Class 1 tanks although they were to have been removed. Explain this inconsistency, and provide proper documentation of as-built conditions.

Responses

Numerous borings were made in August and September 1978 to determine that all natural sands with relative densities less than 75% have been removed beneath all Class I structures, piping, and non-Class I structures so that their failure could endanger the adjacent Class I structures. These boring locations are shown in Figure 2.5-40A. Up to a 35 foot thick compacted fill has been placed in the plant area to achieve a final plant grade elevation of 634 feet.

Split spoon samples were obtained for the natural sands encountered using a standard split spoon sampler. This procedure utilized a 140-pound hammer falling 30 inches to drive a 1-3/8 inch inside diameter split spoon sampler (ASTM D 1586). 15
Blows required to advance the sampler through each 6 inch increment were recorded. The standard penetration test blowcount is the number of blows corresponding to the last foot of sampler penetration. Standard penetration test blowcounts are presented on these boring logs. Standard penetration test blowcounts are presented on these boring logs. A tabulation of the blowcounts associated with the natural sands is shown in Table 2.5-25. These logs and updated soil cross-sections will be presented in a later amendment.

The blowcount obtained from the standard penetration test can be used as a measure of the relative density of sand in situ as described by Gibbs and Holts. Based on such a relationship, a standard penetration test with the range of 20 to 25 blows would be required to obtain the 75% relative density (see FSAR Figure 2.5-42). By examining all the borings, most blowcounts of the natural sands are greatly in excess of required blowcount range of 20 to 25 blows with the exception of the few sand lenses encountered in the following borings.

Boring Number	Elevation	Blowcount (blows/ft)
DG-7	604	17
DG-7	601	17
DG-7	600	10
DG-7	599	15
DG-28	601	9
CT-1	603	11

15

It is seen that the existing natural sands are dense with a relative density much greater than 75%. The sand lenses with the relatively low blowcounts encountered in the borings DG-7, DG-9, DG-28, and CT-1 are isolated and will not endanger the integrity of plant structures.

H.J. Gibbs and W.G. Holtz, "Research on Determining the Density of Sands by Spoon Penetration Testing," Proceedings-Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol I (1957), London, England, pp 35-39

Question 362.10 (2.5.4)

The SER on the PSAR stated that continued surveillance for subsidence should be maintained throughout the life of the plant. Provide in Substction 2.5.4.13 of the FSAR a discussion on the scope and details of the subsidence monitoring program. Include a commitment to monitor subsidence throughout the life of the plant, and indicate the proposed survey frequency. Submit all subsidence data measured since installation of the benchmarks.

Response

Subsection 2.5.4.13.3 discussed the Midland subsidence surveillance monitoring program.

A discussion on the effects of salt mining operations in the plant vicinity is presented in Subsection 2.5.1.2.5.4.1. Included is reference to the subsidence monitoring program initiated to determine any ground movement caused by the removal of salt. First order surveys of the 24 monitoring points that make up the system will be made at least annually for the operational life of the plant to detect any subsidence in the area. The results and analysis of these surveys will be presented in future amendments when they become available.

COPY

Question 362.2 (2.5.4.5.1)

Question 1 and the resulting discussion on Page 8.00-1 included in Amendment Number 9 to your PSAR stated that all natural sands with relative densities less than 75% would be removed beneath all Class I structures and beneath non-Class 1 structures so sited that their failure could endanger the adjacent Class 1 structures. Discuss the methods employed in mapping and removing the sands having less than 75% relative density. Provide plan and sectional figures showing the areas where these materials were removed. Figure A9-2 of the PSAR which displays subsurface profiles of Class 1 piping should be updated to show removal of sands of less than 75% relative density and be presented in the FSAR. Figure 2.5-21 of the FSAR shows loose sands beneath the Class 1 tanks although they were to have been removed. Explain this inconsistency, and provide proper documentation of as-built conditions.

Responses

Numerous borings were made in August and September 1978 to determine that all natural sands with relative densities less than 75% have been removed beneath all Class I structures, piping, and non-Class I structures so that their failure could endanger the adjacent Class I structures. These boring locations are shown in Figure 2.5-40A. Up to a 35 foot thick compacted fill has been placed in the plant area to achieve a final plant grade elevation of 634 feet.

Split spoon samples were obtained for the natural sands encountered using a standard split spoon sampler. This procedure utilized a 140-pound hammer falling 30 inches to drive a 1-3/8 inch inside diameter split spoon sampler (ASTM D 1586). Blows required to advance the sampler through each 6 inch increment were recorded. The standard penetration test blowcount is the number of blows corresponding to the last foot of sampler penetration. Standard penetration test blowcounts are presented on these boring logs. Standard penetration test blowcounts are presented on these boring logs. A tabulation of the blowcounts associated with the natural sands is shown in Table 2.5-25. These logs and updated soil cross-sections will be presented in a later amendment. 15

The blowcount obtained from the standard penetration test can be used as a measure of the relative density of sand in situ as described by Gibbs and Holts. Based on such a relationship, a standard penetration test with the range of 20 to 25 blows would be required to obtain the 75% relative density (see FSAR Figure 2.5-42). By examining all the borings, most blowcounts of the natural sands are greatly in excess of required blowcount range of 20 to 25 blows with the exception of the few sand lenses encountered in the following borings.

Boring Number	Elevation	Blowcount (blows/ft)
DG-7	604	17
DG-7	601	17
DG-7	600	10
DG-7	599	15
DG-28	601	9
CT-1	603	11

15

It is seen that the existing natural sands are dense with a relative density much greater than 75%. The sand lenses with the relatively low blowcounts encountered in the borings DG-7, DG-9, DG-28, and CT-1 are isolated and will not endanger the integrity of plant structures.

H.J. Gibbs and W.G. Holtz, "Research on Determining the Density of Sands by Spoon Penetration Testing," Proceedings-Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol I (1957), London, England, pp 35-39

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the sands having less than 75% relative density. Provide plan
and sectional figures showing the areas where these materials
were removed. Figure A9-2 of the PSAR which displays subsurface
profiles of Class 1 piping should be updated to show removal of
sands of less than 75% relative density and be presented in the
FSAR. Figure 2.5-21 of the FSAR shows loose sands beneath the 8
Class 1 tanks although they were to have been removed. Explain
this inconsistency, and provide proper documentation of as-built
conditions.

Responses

Subsection 2.5.4.5.1 has been revised in response to this
question. The request to provide plan and sectional figures of
areas where the loose sands were removed will be responded in
more detail by amendment in October 1978. 13

To JLCorley

From BHPeck/RMW

Date September 26, 1978

Subject MIDLAND PROJECT GWO 7020 - NRC QUESTION #362.2
REMOVAL OF LOOSE NATURAL SANDS
File: 0505.2 Serial: 3448

Consumers Power Company

Internal Correspondence

CC DBMiller
GSKeeley

The purpose of this memorandum is to inform you of the results of Bechtel and CPCO-PMO efforts to answer the NRC licensing question relating to whether a natural sand layer near elevation 600' was removed during construction or if the sand tested out to be greater than 75% relative density. A copy of this question is attached.

A search of the records to date has not yielded any verification the sands were ever removed. Also, a search of the test records indicates that no tests were performed to confirm the in place density of the natural sands. The current boring program for the Diesel Generator Building problem will also be used as a data base for confirming the in place condition of the natural sand.

We will keep you informed as the situation develops.

2.5.4.2.5 Specific Gravity

Specific gravity of solids was determined in accordance with ASTM D 854-58 in conjunction with consolidated-drained triaxial tests for cooling pond foundation and embankment soil samples. Results are presented in Table 2.5-3.

2.5.4.2.6 Compaction

Compaction tests were performed to develop criteria for placement of fill underneath and around structures, and for embankments. Two compaction methods were used. These are the ASTM D 1557-66T method and the ASTM D 1557-66T method modified to achieve a compaction energy of 20,000 foot-pounds per cubic foot of soil. Compaction tests were performed on bulk samples retrieved from the borrow source. Results are presented in Appendix 2B, Section 2B.3.

2.5.4.2.7 Relative Density

Relative density tests were performed on bulk samples of granular soils. These were made in accordance with ASTM D 2049-64T. Results are presented in Appendix 2B, Section 2B.4.

2.5.4.2.8 Permeability

Constant head permeability tests were conducted in the manner described in Appendix 2B, Section 2B.5 on undisturbed samples from the cooling pond foundation soils and on compacted samples from embankment borrow material. Most compacted samples were prepared at optimum moisture content and compacted to 95% of maximum dry density as determined by ASTM D 1557-66T modified to achieve 20,000 foot-pounds of compactive energy per cubic foot of soil or 70% relative density as determined by ASTM D 2049-64T. Three samples were compacted to 95%, 93%, and 100% of maximum dry density in accordance with ASTM D 1557-66T. The permeability data are presented in Appendix 2B, Section 2B.5, and summarized in Table 2.5-4.

2.5.4.2.9 Consolidation

Thirteen consolidation tests were performed by the dead load-pneumatic consolidometer developed by Dames & Moore. The test procedure is described in Appendix 2B, Section 2B.6. Samples were loaded (at the field moisture content) with a pressure equal to or greater than the existing overburden and were rebounded prior to performing standard consolidation tests. The standard test was then made under submerged conditions. An additional test was performed on a compacted specimen prepared from a bulk sample from the cooling pond area. The data are

2.5.4.5 Excavation and Backfill

2.5.4.5.1 Excavation Plan and Sections

The plant area excavation plan and sections are presented in Figures 2.5-37 and 2.5-38. The excavation extended through the sandy surface soils into relatively impervious clay soils. Slopes were no steeper than 1.5 horizontal to 1 vertical.

The maximum depth of excavation was approximately 40 feet (to elevation 561.5) at the auxiliary building location. The safety of the slope geometry was verified by stability analysis and is discussed in Subsection 2.5.5.

A lean concrete blending mat was used to prevent disturbance of the soil structure during construction. The working mat thickness was no less than 6 inches for the two containments and auxiliary buildings and other structures as needed for workable conditions.

2.5.4.5.2 Dewatering

Dewatering during construction is discussed in Subsection 2.5.4.6.2.

2.5.4.5.3 Fill

Up to 35 foot thick compacted fill was required to attain final plant grade elevation 634. Fill was also required to achieve the foundation elevation for portions of the auxiliary and turbine buildings. The compaction criteria of the plant area fill for various functions are presented in Table 2.5-9. Select sand backfill adjacent to structures was also required and placed according to Table 2.5-9 around all structures. Onsite excavated soils meeting gradations as shown in Table 2.5-10 were used for fill material.

All fill and backfill were placed according to Table 2.5-9. The uncompacted lift thickness was not more than 12 inches. Sheepsfoot rollers and vibratory compaction equipment were used to meet the minimum compaction criteria as shown in Table 2.5-9. In areas not accessible to heavy compaction equipment the material was placed in 4 inch layers and compacted to the required density by mechanical hand tampers.

One hundred and sixty-eight proctor tests have been performed on various fill source materials to establish moisture-density relationship curves and used to determine the percent of compaction for in-place fill. Figure 2.5-39 shows representative moisture-density relationship curves obtained by the ASTM D 1557-66T method, modified to achieve 20,000 foot-pounds of compactive energy per cubic foot of soil. Frequency of

original issue of FSAR

TABLE 2.5-9

MINIMUM COMPACTION CRITERIA

Function	Zone[1] Designation	Soil Type	Compaction Criteria Degree	ASTM Designation
Adjacent to structures	Structural backfill	Sand	80%	ASTM D 2049
Support of structures		Clay	95%	ASTM D 1557-66T (modified)[2]
Plant area fill	1 or 1A	Clay	95%	
	2	Clay or sand	95%	
	3	Sand	95%	
Cooling pond embankment	1 or 1A	Clay	95%	
	2	Clay or sand	95%	
	3	Sand	95%	

[1]For zone designation see Table 2.5-10

[2]The method was modified to get 20,000 foot-pounds of compactive energy per cubic foot of soil.

2.5.4.5 Excavation and Backfill

2.5.4.5.1 Excavation Plan and Sections

The plant area excavation plan and sections are presented in Figures 2.5-37 and 2.5-38. The excavation extended through the sandy surface soils into relatively impervious clay soils. Slopes were no steeper than 1.5 horizontal to 1 vertical.

Engineering design drawings required that loose sands be removed as part of the work scope to be performed in the earthwork subcontract. These loose sands were identified by shallow depth borings made before and during construction operations.

Figure 2.5-21 was prepared to show the original soil profile, 8
including the loose sands, and is based on these shallow depth
borings. This figure represents the condition before
construction.

The request to provide plan figures of areas where the loose sands were removed will be responded to in more detail by August 1978.

The maximum depth of excavation was approximately 40 feet (to elevation 561.5) at the auxiliary building location. The safety of the slope geometry was verified by stability analysis and is discussed in Subsection 2.5.5.

A lean concrete mud mat was used to prevent disturbance of the
soil structure during construction. The mud mat thickness was no 5
less than 6 inches for the two containments and auxiliary
buildings and other structures as needed for workable conditions.

2.5.4.5.2 Dewatering

Dewatering during construction is discussed in Subsection 2.5.4.6.2.

2.5.4.5.3 Fill

Up to 35 foot thick compacted fill was required to attain final
plant grade elevation 634. Fill was also required to achieve the
foundation elevation for portions of the auxiliary and turbine
buildings. The compaction criteria for fill in different areas
are presented in Table 2.5-9. Onsite excavated soils meeting
gradations as shown in Table 2.5-10 were used for fill material.
Select sand backfill adjacent to all safety-related structures
was also required and placed according to Table 2.5-9 around all 1
structures.

All fill and backfill were placed with an uncompacted lift thickness of not more than 12 inches. Sheepsfoot rollers and

vibratory compaction equipment were used to meet the minimum 1
compaction criteria as shown in Table 2.5-9. In areas not accessible to heavy compaction equipment the material was placed in 4 inch layers and compacted to the required density by mechanical hand tampers.

One hundred and sixty-eight proctor tests have been performed on various fill source materials to establish moisture-density relationship curves and used to determine the percent of compaction for in-place fill. Figure 2.5-39 shows representative moisture-density relationship curves obtained by the ASTM D 1557-66T method, modified to achieve 20,000 foot-pounds of compactive energy per cubic foot of soil. Frequency of laboratory and field testing for control of materials is shown in Table 2.5-11; additional testing was done when required by field
engineering. The numbers of field in-place density control tests 1
taken in structural and plant areas are summarized in Table 2.5-12.

Figures 2.5-66 through 2.5-69 show summaries of field density
tests for compaction and moisture contents of fill placed beneath 8
and around Seismic Category I structures.

The quality assurance program during construction was performed in accordance with Chapter 17. Quality control was a daily program that checked both field and laboratory work. The program was approved by a quality control engineer before any work was accepted; any deviation from specifications required a nonconformance report which had to be satisfied before that portion of the work could proceed or be accepted.

Test fills were constructed to evaluate compaction equipment to be used on the site. Given below are the results of this investigation.

Three types of compaction equipment were used to compact a 1 foot lift of similar Zone 1 material on the three pads. They consisted of the following units:

1. Bros roller, having four pneumatic rubber tires on one axle, which has been loaded to a gross weight of 50 tons, pulled by a Terex 8240 dozer

2. A smooth steel drum vibratory roller, Raygo Rumbler, pulled by a Michigan 280 tractor with the following specifications:

gross weight	20,000 pounds
drum diameter	60 inches
drum length	100 inches
dynamic vibration force	45,000 pounds
vibration frequency	1,100 to 1,500 vpm

3. A CF43 Vibroplus Sheepsfoot roller, pulled by a Michigan 280 tractor with the following specifications:

static weight	12,000 pounds
centrifugal force	11.5 tons
total applied load at 1,600 vpm	35,000 pounds
vibration frequency	1,400-1,600 vpm
diameter of drum	63 inches
length of drum	75 inches

Each roller made four passes over its respective test pad. All the material placed in the test pads corresponds to the same compaction curve with optimum moisture content being 10.3% and the maximum dry density being 124.7 lb/ft^3. The ASTM D 1557-66T compaction method modified to achieve a compaction energy of 20,000 foot-pounds per cubic foot of soil was used. The results of the tests are tabulated below:

Test Pad	Moisture %	Dry Density (lb/ft^3)	% Compaction	% Passing #200 Sieve	Type Roller
1	10.9	110.2	88.4		50 ton rubber tire
1	17.9	112.5	90.2	64	50 ton rubber tire
1	12.0	111.4	89.3		50 ton rubber tire
2	8.0	117.3	94.1		vibratory steel drum
2	12.8	121.2	97.2	60	vibratory steel drum
2	9.7	123.3	98.9		vibratory steel drum
3	8.0	117.5	94.2		vibratory sheepsfoot
3	12.4	123.8	99.3	58	vibratory sheepsfoot
3	9.8	128.5	103.0		vibratory sheepsfoot

According to these results, both substitute rollers achieved higher soil densities than the 50 ton roller. Both substitute rollers were accepted for compaction of Zone 1, 1A, and 2 material. The four passes were required for each substitute roller.

See table 2.5-21

2.5.4.6 Groundwater Conditions

2.5.4.6.1 Effects on Stability of Facilities

The groundwater effects on the structural design of the plant facilities are discussed in Subsection 2.4.13. All plant structures, systems, and components are designed to withstand hydrostatic loading resulting from the site probable maximum flood of elevation 631. This level is greater than any potential groundwater level in the site area. 1

2.5.4.6.2 Dewatering During Construction

No permanent dewatering system was required during the foundation excavation and subsequent construction of the plant facilities. Only minor quantities of groundwater entered the excavations. Occasional ponding of water occurred from precipitation and surface runoff. This situation was relieved either by direct removal of water by small pumps or by diverting the water, by means of surface ditches, to nearby sumps. 1

relationship between field values of cyclic stress ratio and standard penetration test data. It is seen that both analyses indicated that, for an SSE of 0.12g, there is no liquefiable soil at the Midland power plant. Furthermore, placing up to 35 feet of fill on the plant area should further decrease the potential for liquefaction because of the additional confinement provided by the fill.

2.5.4.9 Earthquake Design Basis

The maximum earthquake for the tectonic province in which the site is located is intensity VI. The SSE, as described in Subsection 2.5.2.6, is based on a local event of epicentral intensity VI on the Modified Mercalli (MM) Scale. A number of relationships between MM intensity and epicentral acceleration[55,56,57] show this intensity to be associated with a peak acceleration of approximately 0.06g. Consequently, 0.10g would be suitable for the safe shutdown earthquake (SSE) for the site. However, for additional conservatism, an acceleration of 0.12g was used for design purposes. As described in Subsection 2.5.2.7, the operating basis earthquake (OBE) is one-half that of the SSE, or 0.06g. Design response spectra for the OBE and SSE are presented in Section 3.7.

2.5.4.10 Static Stability

This section deals with the static stability of all safety-related facilities.

The containments and certain portions of the auxiliary building are founded on the layer of very stiff to hard cohesive soils. Other portions of the auxiliary building are founded at various elevations. The original ground surface elevation in this area was between elevations 605 and 612. The surface soils encountered in this area were sand pockets of varying thickness overlying very stiff to hard cohesive soils. The sandy soils were removed and foundation grade attained, if necessary, by the placement of compacted fill.

All in situ sands, soft or compressible clay soils, and organic soil were excavated in the turbine building area. The turbine building and turbine generators are supported on mat foundations on controlled compacted fill.

The remaining plant facilities, diesel generator building, yard tanks, solid radwaste building, and borated water storage tanks were constructed on compacted fill. Building foundations are placed at least 4-1/2 feet below the plant grade to mitigate potential frost penetration effects.

2.5.4.10.1 Bearing Capacity

To adequately design the foundations against shear failure of the supporting soil, it is necessary to determine the ultimate bearing capacity of the soil. Bearing capacities, as shown in Tables 2.5-13 and 2.5-14, were determined by Dames & Moore for both mat foundations and spread footings. The plant facilities were established either on mat or spread foundations. Table 2.5-14 shows the contact stress beneath footings subject to static and static plus dynamic loadings, the foundation elevation, and the type of supporting medium for various plant structures. Foundations placed at elevation 580 and below were founded on the in situ stiff clay layer. Structures with foundations at elevations 602 and above were supported on compacted fill.

It is noted that the ratios between the calculated ultimate net bearing capacity versus the maximum contact stress beneath footings shown in Table 2.5-14 for various plant facilities are greater than three for the combination of dead and live loads and greater than two for the combination of dead, live, and seismic loads. A factor of safety of three is used for maximum loads normally expected to act upon the foundation and a factor of safety of not less than two is used for the maximum loads ever to be expected.

2.5.4.10.2 Lateral Earth Pressures

The walls of structures below grade, elevation 634, are subjected to horizontal earth pressures imposed by backfill materials, hydrostatic pressures, and lateral pressures from adjacent structured loads. The earth pressure depends on the soil strength, groundwater conditions, the method used in placing the backfill, the degree of compaction of the backfill, and the amount of wall movement. The principal earth pressure conditions are categorized as the active earth pressure, the at-rest earth pressure, the passive earth pressure, and dynamic earth pressures. The earth pressures resulting from any of these conditions are calculated by appropriate earth pressure theory. The equivalent fluid weight concept is used to express earth pressures. Equivalent fluid weights for all conditions are discussed in detail in the following paragraphs.

2.5.4.10.2.1 Active Earth Pressure

A nonrigid retaining wall, which is free to move laterally at the top, causes the active earth pressure condition to develop. Most of the unrestrained nonrigid retaining walls and sheet pile walls can move sufficiently to permit the development of the active earth pressure condition.

Table 2.5-15 shows equivalent fluid weights used in design for the active case (nonrigid walls) as conservatively derived by Dames & Moore for sand and clay above and below the water table.

2.5.4.10.2.2 At-Rest Earth Pressure

Rigid walls and walls sufficiently restrained can cause at-rest soil pressures to develop. At-rest pressures are those pressures developing at a point in the ground not subject to any lateral movement. For in situ clean sands, the theoretical at-rest earth pressure coefficient k varies from about 0.35 for dense sands to about 0.5 for loose sands. However, backfilling and compaction processes may cause the lateral earth pressure to increase the above theoretical at-rest value. Table 2.5-15 shows equivalent fluid weights used in design for at-rest case (rigid walls) as derived by Dames & Moore for sand and clay above and below the groundwater table. For sandy soils, the results are based on k of 0.5.

2.5.4.10.2.3 Passive Earth Pressure

When a wall is pushed into the backfill, the horizontal stresses in the soil will increase until the shear strength of the soil is fully mobilized. The horizontal stress developed under this condition is known as the passive earth pressure. However, the movement necessary to develop full passive pressure is quite large. This movement is on the order of 5% of the height of the wall. Because movements of this magnitude cannot normally be tolerated, a factor of safety of two is usually applied to the total passive pressure. Design values for passive pressure are included in Table 2.5-14.

2.5.4.10.2.4 Dynamic Earth Pressure

During earthquakes, active and at-rest pressures will increase, while, under worst conditions, the passive pressure will reduce. The simplified design procedures for dynamic soil loads are based on the Mononobe-Okabe analysis of dynamic pressure in dry cohesionless materials. See Seed and Whitman.[71]

Based on the Mononobe-Okabe approach, dynamic lateral pressures were estimated for sand backfill. These pressures, along with the method used to combine them with active, at-rest, or passive pressures, are shown in Figure 2.5-45 for clean sand backfill under the water table.

2.5.4.10.2.5 Surcharge Load Due To Adjacent Structures

Surcharge loads caused by adjacent structures can generally be defined both in magnitude and area of application. The pressure developed by adjacent structures is additive to the lateral pressure directly applied by the backfill material. This additional earth pressure can best be determined by using methods derived from the theory of elasticity which are available for most loading shapes encountered in engineering applications. Suitable solutions are given by Bowles.[72] If the wall is considered to be rigid, the earth pressure will be twice that due to the elastic solutions as described and accounted for in this reference.

2.5.4.10.2.6 Live Load Surcharge

The lateral earth pressures due to live load depend on the load intensity, location, and shape; therefore, these lateral pressures can best be determined by elastic methods. Several possible load configurations that may be anticipated are given by Bowles.[72]

Surcharge pressures caused by dead or live loads were added to the pressures shown in Figure 2.5-45.

2.5.4.10.3 Settlements

This section deals with the evaluation of vertical ground movements (heave or settlement) under the plant facilities caused by construction. An excavation up to 40 feet below the original ground surface was made to enable the construction of the containment and portions of the auxiliary building. A large area fill up to 35 feet high, measuring approximately 1,000 feet by 1,100 feet, has been placed as shown in Figure 2.5-46. Heavy structural loads will be applied on this fill. The groundwater table at the plant area will be raised to elevation 627 when the cooling water reservoir is filled.

*

The effects of the above construction operations on ground movements at the Midland site are as follows:

a. First, when the site was excavated to depths of 40 feet, the resulting removal of material caused the underlying soils to rebound upward.

review record

b. Next, as the large area fill was placed and structures were constructed, the resulting loads recompressed the prior upward rebound and then caused additional settlement.

c. Finally, raising the groundwater table will reduce the net foundation pressures. However, some settlement will confirmation of predicted settlements. Permanent benchmarks and control monuments will be established at the site and used for survey reference points. Periodic elevation checks against the benchmarks and control monuments will be measured.

2.5.4.13.2 Survey Frequency

Review records

a. During Construction

For Seismic Category I and II structures, survey measurements will be made every 60 days.

b. During Plant Operation

1. For Seismic Category I and II structures, survey measurements will be made every 90 days during the first year of operation.

2. Frequency of survey measurements for subsequent years will be established after evaluating the measurements taken during the first year.

c. Seismic Category I and II Tanks

1. Survey measurements will be made after the tanks are erected and just prior to hydrostatic testing of the tanks.

2. During hydrostatic testing

3. Immediately after the hydrostatic testing is complete with the tanks empty

4. At the completion of filling of tanks for plant operation

5. At 90 day intervals for the first year of plant operation

6. Frequency survey measurements for subsequent years will be established after evaluating the measurements taken during the first year.

2.5.4.14 Construction Notes

The earthwork operation was started in July 1969 and was suspended between May 1970 and August 1973. During the first construction period, the work generally involved the clearing of selected portions of the cooling pond and plant site, and the clearing and grubbing of the foundation areas. The plant area was excavated to grade and concrete was poured for some footings, floor slabs, and walls. During construction stoppage, the excavation was left open protected with straw cover. Five soil borings (899, 900, 901, 904, 904A) were drilled in June 1973 by Soil and Materials Engineers, Inc., ranging from 20 to 71 feet deep, to ensure that no major changes had taken place in the subsoil as a result of flooding and frost penetration within the plant area. Three of these borings were drilled as near as possible to the containment structures and the other two borings were made in the plant dike. Both undisturbed (Shelby tube) and disturbed (Split Spoon) samples were obtained at various depths in all borings. Various laboratory tests were performed on these samples in order to evaluate the shear strength of the in situ soils. See Subsection 2.5.4.3 for details.

The boring locations are shown in Figures 2.5-16 and 2.5-17. Logs of borings, together with other pertinent information, are presented in Appendix 2A. A summary of the test results is given in Tables 2.5-17 and 2.5-18.

how about heave

Although the clayey soils in all borings were fully saturated or close to being so, it should be noted that in every case the in situ moisture content was very near the plastic limit (shown on the boring logs). This indicated that even though the area had been inundated, the subsurface soils had not absorbed much, if any, additional water. Thus, no softening or weakening of the soil was evident. This was ~~further substantiated by unconfined~~ compression tests performed on selected undisturbed samples which showed undrained shear strengths of 3.7 to 5.9 ksf. Somewhat lower shear strengths of 1.9 ksf in boring 899 and 2.5 ksf in boring 900 were noted, although significantly higher values were found at higher elevations ~~leading to the conclusion~~ that the weaker soil strengths noted were not due to inundation. Considering the type of soil, it was likely that these samples contained very thin sand or silt lenses as noted in the Split Spoon samples and in some of the triaxial test specimens, thus accounting for their lower strengths. Based on the information obtained from laboratory tests performed on undisturbed samples from the three test borings, the subsurface soils in the plant area did not appear to have been adversely affected by water standing and frost penetration within the open excavations.

Therefore, changes in design and/or construction procedures were unwarranted and earthwork operations were resumed normally.

2.5.5 STABILITY OF SLOPES

This section deals with the static and dynamic stability of all soil slopes, both natural and manmade, at the plant site. The stability evaluation of embankment slopes associated with the main power plant facilities is discussed in the following paragraphs. The stability of embankments related to the cooling pond is discussed in Subsection 2.5.6.

continue until equilibrium is reached under the net increase in load.

Ultimate heave or settlement values were estimated by calculating the stress changes from elastic half-space theory and then computing the settlement or heave using Terzaghi's theory of one-dimensional consolidation.

Parameters to establish the analytical model are discussed in the following subsections.

2.5.4.10.3.1 Plant Layout and Loads

As shown in Figure 2.5-47, the two units and the contiguous structures occupy a total area measuring approximately 600 feet by 600 feet. Preconstruction grade at the site is approximately elevation 603. Finished grade at the plant site is 31 feet higher, at elevation 634. Compacted fill was used to raise the original ground surface to grade elevation.

Each containment was founded on a circular mat having a diameter of 128 feet and located at a depth of 20 feet below original ground surface. Portions of the auxiliary building were established 40 feet below original ground surface on the layer of very stiff to hard cohesive soils. The mat foundation grades for the rest of the auxiliary building, the turbine building, and associated facilities were placed at various elevations on compacted fill. The building loads superimposed by the structures on undisturbed soil or compacted fill are given in the soil pressure plan, Figure 2.5-47.

2.5.4.10.3.2 Subsurface Conditions

The plant site was essentially flat, and the ground surface was at about elevation 603. A detailed description of soil conditions together with generalized soil profiles through the plant site is given in Subsection 2.5.4.3.5. For the purpose of analysis, the soil profile is divided into the layering system shown in Table 2.5-16.

2.5.4.10.3.3 Soil Parameters

The soil compressibility parameters used in the settlement calculation are presented together with soil profile in Table 2.5-16. The normalized compression and swelling indexes (C_c, r/1+e_o) were evaluated by two methods. The first method used, presented by Dames & Moore,[58] is based on laboratory consolidation tests with adjustments for the effects of sample disturbance as discussed in Subsection 2.5.4.2.9.

The other method is based on mathematical relationships among compression index, constrained modulus, and Young's Modulus as illustrated by Lambe and Whitman.[73] Young's Modulus ($E = 600\ Su$)[74] is based on a statistical relationship with the unconfined compressive strength or undrained shear strength. The undrained shear strength used is interpreted conservatively from the summation plot of shear strength vs elevation given in Figure 2.5-33.

The sampling of overconsolidated glacial clays is usually difficult due to the stiffness of the clays. Sample disturbance is inevitable. This evidence is clearly shown from all the laboratory consolidation test curves. Furthermore, experience indicated that the estimated soil compressibilities from consolidation tests are influenced and increase by the specimen preparation of trimming and ring fitting. On the other hand, the empirical compressibilities are derived from shear strength test results, which are not affected by sample disturbance to the same degree as laboratory consolidation test results. The normalized compression and swelling indexes ($C_{c,r}/1+e_o$) adopted in settlement calculations are the weighted average values derived from both methods.

2.5.4.10.3.4 Groundwater Conditions

Check

For settlement evaluation, the static groundwater level is conservatively estimated at or near the existing ground surface before construction. The post-construction long-term water level in the plant area is taken to be elevation 627. This elevation will be the maximum operational level of the filled cooling pond.

2.5.4.10.3.5 Analysis

The settlement evaluation for the plant structures was made from a consideration of the following cases:

review records

a. Settlements due to placement of fill to grade and application of building net loads prior to flooding of the cooling water reservoir, water level at elevation 603 (short-term condition)

b. Settlements due to fill and building net loads after reservoir is filled, water level at elevation 627 (long-term condition)

Heave from pressure relief due to excavation of overburden soils above the foundations is not analyzed because: 1) pressure relief due to excavation would decrease quickly to zero by the subsequent placement of fill and building loads, 2) the heave associated with stress reduction is relatively small compared to the settlement due to large area fill and building loads, and is essentially elastic due to the highly overconsolidated nature of the in situ glacial till, and 3) the ultimate settlement analyzed for the above Case a and b loading conditions was based on the application of appropriate building net loads.

For settlement computations, a total of 41 settlement points are established on a grid and at selected structure locations as shown in Figure 2.5-48.

Loading criteria for Case a and b conditions plus the other pertinent parameters are presented in Figure 2.5-47. Based on the respective loading conditions, site soil conditions, and the selected soil compressibility characteristics, ultimate settlements at each of the 41 points are calculated for load conditions -- Cases a and b. Settlement values resulting from each loading condition are calculated by evaluating the stresses from elastic half-space theory(75) and then computing the settlement using Terzaghi's theory of one-dimensional consolidation.

To account for possible time-dependent relationship, the estimated tota settlements at each of the 41 points were obtained respectively by adding 25% of the calculated settlement values of loading Case a to the calculated ultimate settlement values of loading Case b. These values are presented in Figure 2.5-48.

2.5.4.10.4 Discussion

Settlements at the 41 points calculated for Units 1 and 2 show the best estimates of settlement expected. Because of the possible variations in loads, soil conditions, and soil properties, deviations from the estimated values are possible.

It is known that if clays have previously been consolidated by pressures equal to or greater than those to be added by new construction, their settlement is relatively small and occurs so rapidly that it may be considered to be elastic. On the other hand, if the added pressures exceed the preconsolidation load, the settlements are larger and occur with appreciable time lag. With respect to the Midland site, the glacial till at the site is heavily preconsolidated and the pressure added by new construction does not exceed the estimated preconsolidation pressures. Therefore, it is concluded that the settlement of the most heavily loaded portions of the plant will be essentially elastic. Most of the settlement occurs as the fill is placed and as the dead weight of the structures is added. It is estimated that settlements on the order of 20% of the calculated ultimate settlements can be expected after the vital pipe connections are made. It is anticipated that maximum differential settlements on the order of one inch may occur between adjacent structures. The differential settlements will be appreciably smaller than the maximum settlements.

To ensure the integrity of the plant facilities and verify the settlement predicted by analysis, settlement measurements will be monitored at each instrument location to provide a history of time-movement. The measurements reflect what the structures will actually experience. The monitoring program is discussed in Subsection 2.5.4.13.

2.5.4.11 Design Criteria

The design criteria and methods of design related to the stability studies of all safety-related facilities have been discussed previously in Subsection 2.5.4.10.

Settlements at various locations of Units 1 and 2 are calculated by the Terzaghi one-dimensional consolidation theory. It is estimated that the essentially elastic settlements in the power block will be between 2 and 3 inches. Maximum differential settlements among buildings are not expected to exceed 1 inch. ~~Refer to Subsection 2.5.4.10.4.~~

Will Change

Gross bearing capacity of the soil of various mat foundations is determined by conventional Terzaghi theory. The computed factors of safety (ratio values between gross ultimate bearing capacity versus the maximum contact stress beneath footing) for various plant facilities are greater than three for dead and live loads combined and greater than two for the combination of dead, live, and seismic loadings. See Table 2.5-14.

The design values for the principal earth pressure conditions are conservatively derived by neglecting the wall friction force. Furthermore, the design values for passive earth pressures have reduced by a factor of two from the calculated theoretical values.

2.5.4.12 Techniques to Improve Subsurface Conditions

Because of the competent nature of the subsurface soil conditions, measures (such as grouting, vibroflatation, dental work, rock bolting, and anchors) to improve foundations were not required. Slurry cutoff trench treatments were placed to prevent seepage loss during construction of the cooling pond dikes and plant area fill. This is discussed in Subsection 2.5.6.3.

2.5.4.13 Subsurface Instrumentation

2.5.4.13.1 Benchmark Locations

Settlement measurements are to be taken at benchmark locations installed at the various plant structures to provide a history of settlement versus time. These measurements will provide a record of movements experienced and they will be used to provide

TABLE 2.5-11

FREQUENCY OF FILL TESTING

Test	Approximate Frequency
Equipment calibration	Frequency to be based on Bechtel Field Inspection Manual, or, if not otherwise stated, upon manufacturer's suggested frequency.
Field densities, moisture content	One per 500 yards of fill
Compaction, grain size, specific gravity	One per 10,000 cubic yards of fill

TABLE 2.5-9

MINIMUM COMPACTION CRITERIA

Function	Zone[1] Designation	Soil Type	Compaction Criteria Degree	ASTM Designation
Adjacent to structures	Structural backfill	Sand	80%	ASTM D 2049
Support of structures	2	Clay	95%	ASTM D 1557-66T (modified)[2]
Plant area fill	1 or 1A	Clay	95%	
	2	Clay or sand	95%	
	3	Sand	95%	
Cooling pond embankment	1 or 1A	Clay	95%	
	2	Clay or sand	95%	
	3	Sand	95%	

[1]For zone designation see Table 2.5-10

[2]The method was modified to get 20,000 foot-pounds of compactive energy per cubic foot of soil.

% Comp = $\frac{\gamma_{dry\ in\ place}}{\gamma_{proctor}}$

TABLE 2.5-14

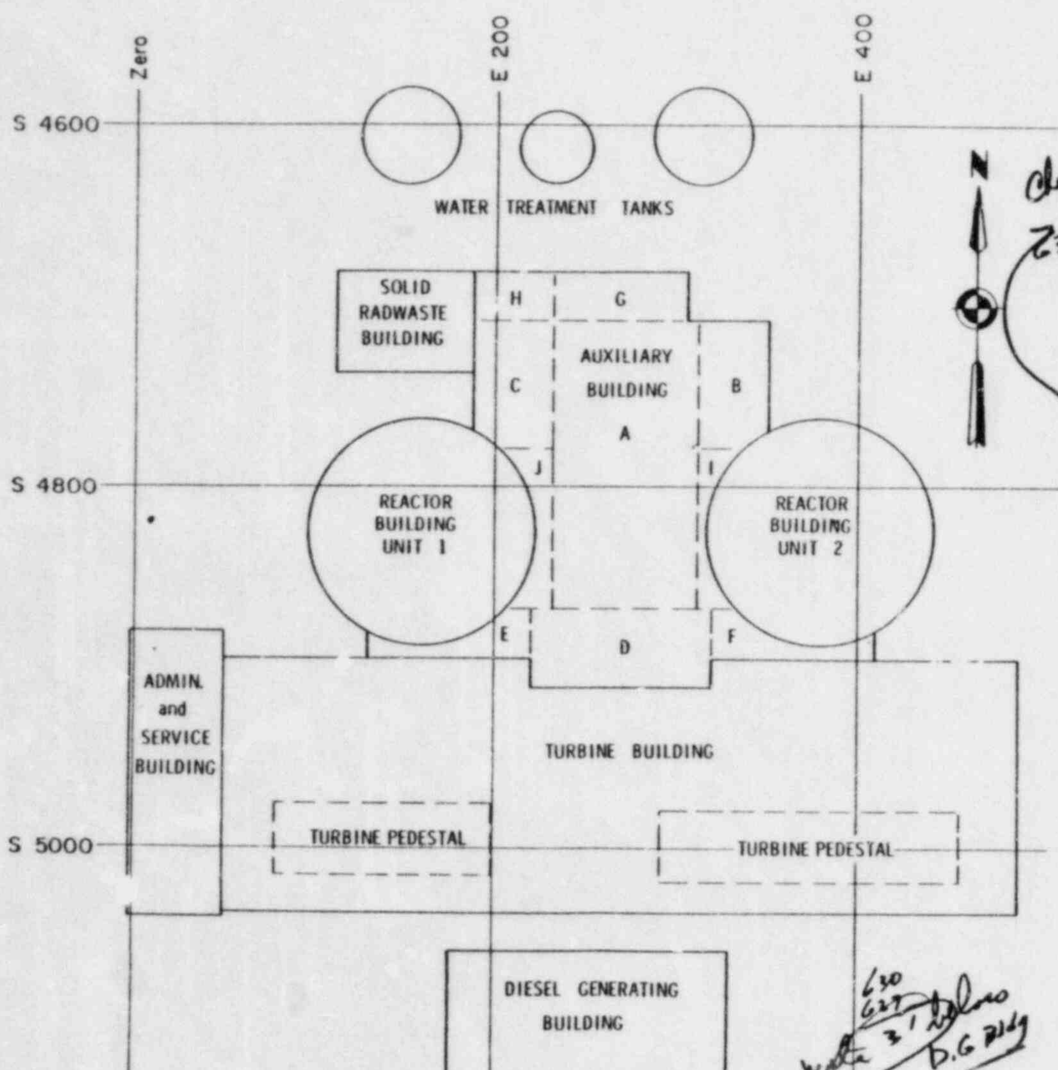
SUMMARY OF CONTACT STRESSES AND ULTIMATE
BEARING CAPACITY FOR MAT FOUNDATIONS
SUPPORTING SEISMIC CATEGORY I AND II STRUCTURES

Unit	Supporting Soils	Foundation Elevation	Contact Stress Beneath Footing (lb/ft^2) Dead Plus Live Load	Contact Stress Beneath Footing (lb/ft^2) Dead, Live, and Seismic Load	Ultimate Bearing Capacity (lb/ft^2)	Factor of Safety Dead Plus Live Load	Factor of Safety Dead, L and Seismi
Reactor containment buildings	Very stiff to hard natural cohesive soils	582.5	8,000	17,500	45,000	5.6	2.6
Auxiliary building A	Very stiff to hard natural cohesive soils	562	7,000	14,000	50,000	7.1	3.6
Auxiliary building B&C	Very stiff to hard natural cohesive soils	579	8,000	16,000	50,000	6.2	3.1
Auxiliary building D	Controlled compacted cohesive fill	609	6,000	12,000	30,000	5.0	2.5
Auxiliary building E&F	Controlled compacted cohesive fill	609	6,000	12,000	30,000	5.0	2.5
Auxiliary building G	Controlled compacted cohesive fill	630	4,000	8,000	15,000	3.8	1.9
Auxiliary building H	Controlled compacted cohesive fill	609	3,000	6,000	30,000	10.0	5.0
Auxiliary building I&J	Very stiff to hard natural cohesive soils	569	6,500	13,000	50,000	7.7	3.8
Turbine mat	Controlled compacted cohesive fill	602	5,000	10,000	30,000	6.0	3.0
Turbine building	Controlled compacted cohesive fill	609	3,000	6,000	30,000	10.0	5.0
Solid radwaste building	Controlled compacted cohesive fill	629.5	2,500	5,000	15,000	6.0	3.0
Diesel generator building	Controlled compacted cohesive fill	629.5	4,000	6,000	15,000	3.8	[illegible]

TABLE 2.5-14 (continued)

Unit	Supporting Soils	Foundation Elevation	Contact Stress Beneath Footing (lb/ft²) Dead Plus Live Load	Dead, Live, and Seismic Load	Ultimate Bearing Capacity (lb/ft²)	Factor of Dead Plus Live Load	De and S
Condensate and primary storage tank	Controlled compacted cohesive fill	629.5	2,500	5,000	15,000	6.0	
Borated water storage tank	Controlled compacted cohesive fill	629.5	2,500	5,000	15,000	6.0	

NOTE: Factor of safety is defined as the ratio of ultimate bearing capacity to contact stress beneath footing.



Check bottom elevation 634 or 630?

		El_m	P_s	P_{n1}	P_{n2}
WATER TREATMENT TANKS		634	2.5	2.5	2.5
SOLID WASTE BLDG.		634	6.0	6.0	6.0
AUXILIARY BLDG.	A	562	7.0	1.57	-0.12
	B & C	579	8.0	4.83	3.15
	D	609	6.0	6.0	4.88
	E & F	609	6.0	6.0	4.88
	G	630	4.0	4.0	4.0
	H	609	6.0	6.0	4.88
	I & J	569	6.5	1.82	0.13
REACTOR BLDGS. 1 & 2		582.5	8.0	5.29	3.60
TURBINE BLDG.		609	3.0	3.0	1.88
TURBINE PEDESTALS (2)		602	5.0	4.87	3.31
DIESEL GEN. BLDG.		634	4.0	4.0	4.0
CONDENSATE STORAGE TANKS		634	2.5	2.5	2.5
AREA FILL LOAD		603	4.09	4.09	2.60

NOTES :

1. El_m is the elevation of the bottom of the foundation.
2. P_s is the superimposed load intensity.
3. P_{n1} is the short term net load intensity (before the cooling water reservior filling) $P_{n1} = P_s$ - Excavation load
4. P_{n2} is the long term net load intensity (after the cooling water reservior filling) $P_{n2} = P_{n1}$ - Hydrostatic pressure
5. All units for load intensity in kips per square foot (ksf), elevations in feet from U. S. G. S. datum.
6. Reference Table 2.5-14.

4.87
3.31
1.56 kip/ft²

.560 #/ft² / 62 #/ft³ = 25 ft of water under Turbine Bldg

630
627
water 3' below D.G Bldg

Fig 2.5-47

TABLE 2.5-21

SUMMARY OF COMPACTION REQUIREMENTS

Zone	Compaction Equipment	Minimum Number of Passes	Maximum Lift Thickness Before Compaction (inches)	Compaction Criteria: Degree	Compaction Criteria: ASTM Designati
1 Impervious	50 ton rubber tired roller, vibratory steel drum and sheepsfoot	4	12	95%	
1A Impervious adjacent to concrete structures	Power tampers	4	4	95%	ASTM D 1557-66 modified to ge 20,000 foot-pounds of comp tive energy pe cubic foot
DG Bldg. 2 Random fill	50 ton rubber tired roller, vibratory steel drum and sheepsfoot	4	12	95%	
2A Random fill adjacent to concrete structures	Power tampers		4	95%	
3 Sand	50 ton rubber tired roller		12		No requirement
4 Gravel	Construction equipment				
5 Riprap	Compaction not required				
6 Topsoil	Compaction not required				

NO include IN spec

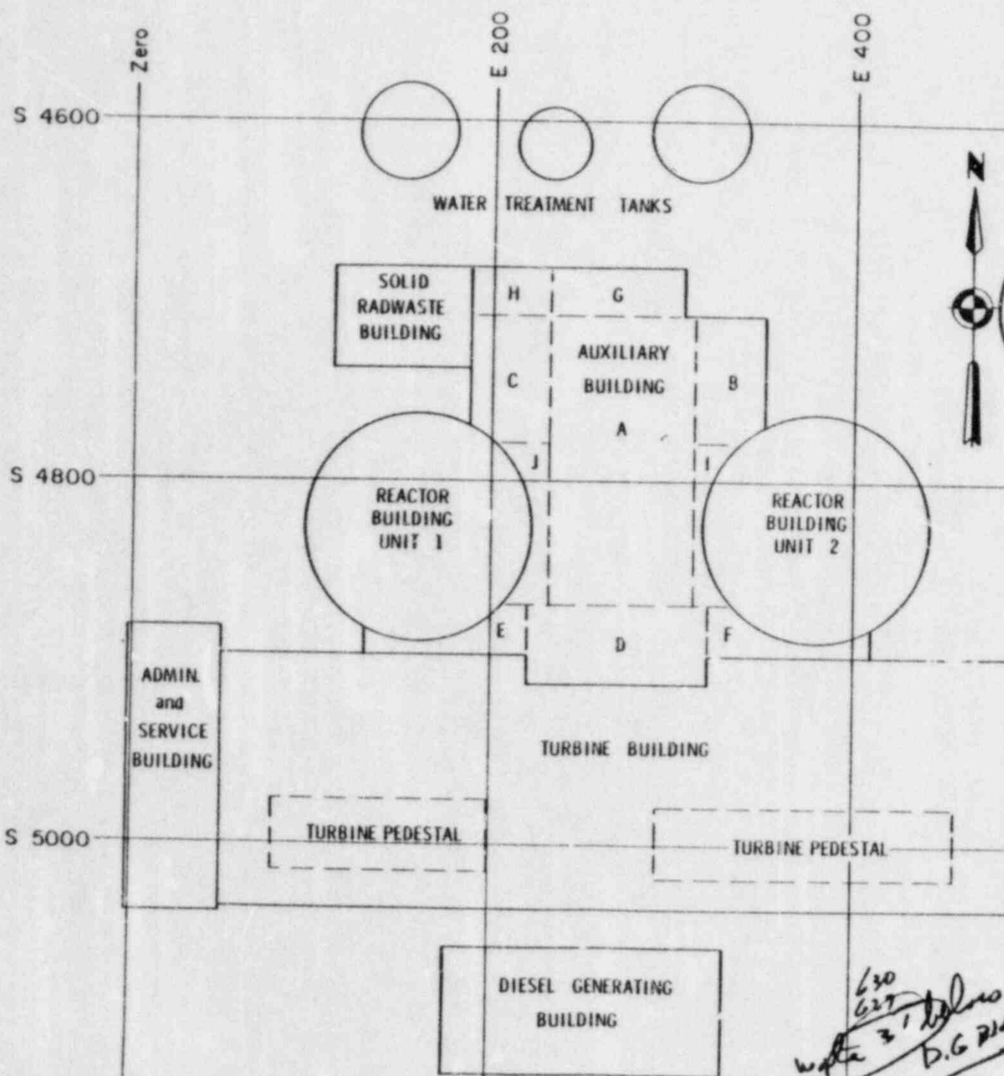
NOTE: In areas not accessible to rollers, particularly those adjacent to the outlet structure, it was necessary to control moisture and lift thicknesses carefully to achieve the required density with hand operated power tampers. The power tamping was such that the same standard of compaction was achieved as required for the contiguous material in the embankment, compacted by the specified rollers. Areas where hand tamping had to be carried out were apt to be most vulnerable to seepage, and thus great care was necessary to ensure that they were adequately compacted.

MIDLAND 1&2-F..

TABLE 2.5-10

GRADATION RANGES FOR FILL MATERIAL

Zone	Type	Description	Source	Gradation	
1	Impervious fill	Sandy silty clays or sandy silts with some clay	Designated borrow area and all required excavation	Not less than 20% passing No. 200 sieve	
				U.S. Std Series Sieve Sizes	Percent Passing
1A	Impervious fill	Native broadly graded sandy glacial till	Designated borrow area and all required excavation	No. 4	40-100
				No. 30	30-100
				No. 100	25-80
				No. 200	20-70
				0.01 (millimeters)	10-40
				0.002 (millimeters)	0-20
2	Random fill	Any material free of humus, organic or other deleterious material	Designated borrow area and all required excavation	No restrictions	
3	Sand drain	Clean sand graded as specified	Imported	3/8 inch	100
				No. 8	55-100
				No. 30	20-55
				No. 100	0-10
				No. 200	0-3
Structural backfill	Sand fill	Clean sand graded as specified	Imported	1 inch	100
				No. 4	75-100
				No. 10	50-100
				No. 40	5-60
				No. 200	0-5



Check bottom elevation 634 or 630?

		El_m	P_s	P_{n1}	P_{n2}
WATER TREATMENT TANKS		634	2.5	2.5	2.5
SOLID WASTE BLDG.		634	6.0	6.0	6.0
AUXILIARY BLDG.	A	562	7.0	1.57	-0.12
	B & C	579	8.0	4.83	3.15
	D	609	6.0	6.0	4.88
	E & F	609	6.0	6.0	4.88
	G	630	4.0	4.0	4.0
	H	609	6.0	6.0	4.88
	I & J	569	6.5	1.82	0.13
REACTOR BLDGS. 1 & 2		582.5	8.0	5.29	3.60
TURBINE BLDG.		609	3.0	3.0	1.88
TURBINE PEDESTALS (2)		602	5.0	4.87	3.31
DIESEL GEN. BLDG.		634	4.0	4.0	4.0
CONDENSATE STORAGE TANKS		634	2.5	2.5	2.5
AREA FILL LOAD		603	4.09	4.09	2.60

NOTES :

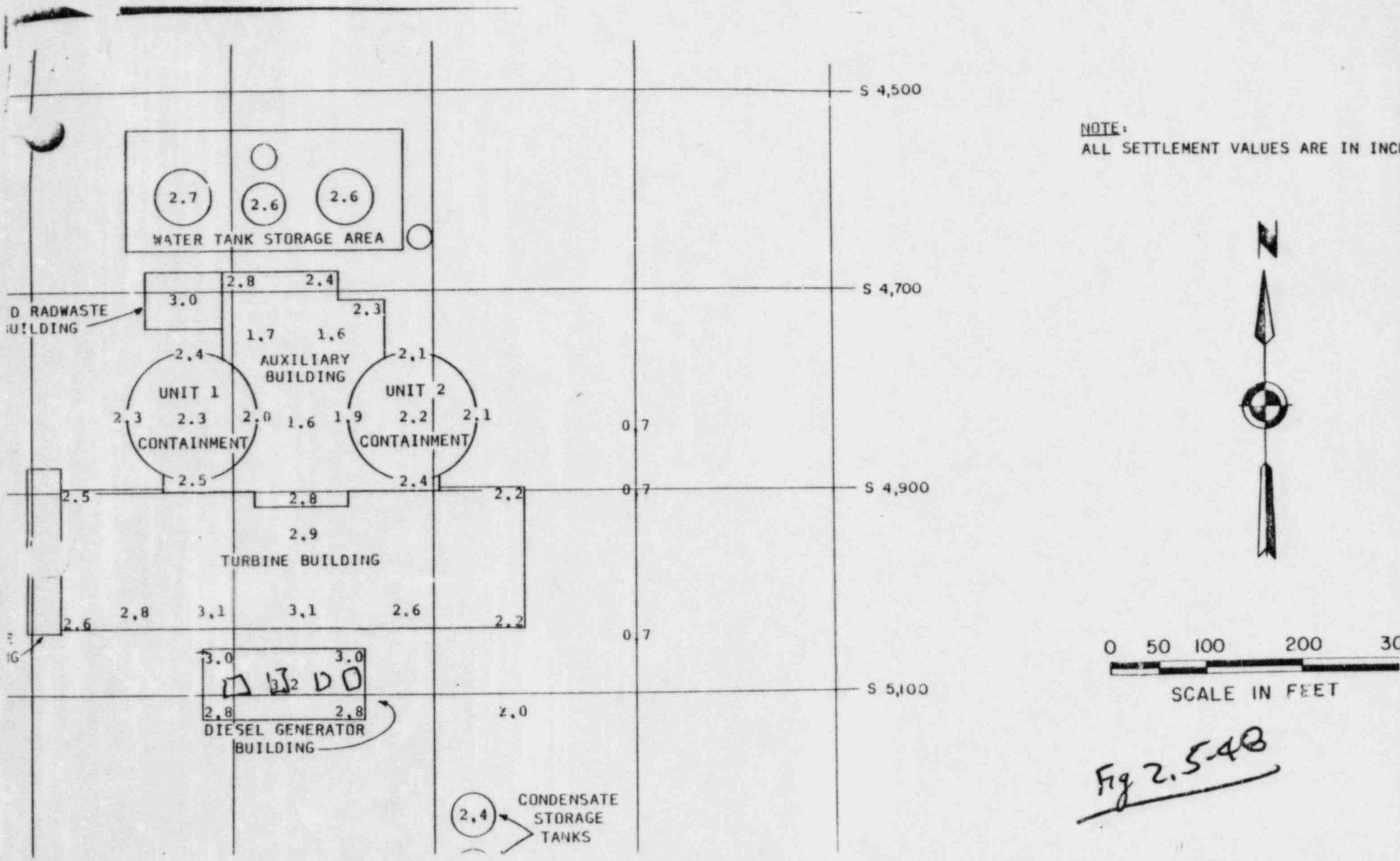
1. El_m is the elevation of the bottom of the foundation.
2. P_s is the superimposed load intensity.
3. P_{n1} is the short term net load intensity (before the cooling water reservior filling) $P_{n1} = P_s$ - Excavation load
4. P_{n2} is the long term net load intensity (after the cooling water reservior filling) $P_{n2} = P_{n1}$ - Hydrostatic pressure
5. All units for load intensity in kips per square foot (ksf), elevations in feet from U. S. G. S. datum.
6. Reference Table 2.5-14.

$\frac{4.87 - 3.31}{1.56}$ k/ft²

Fig 2.5-47

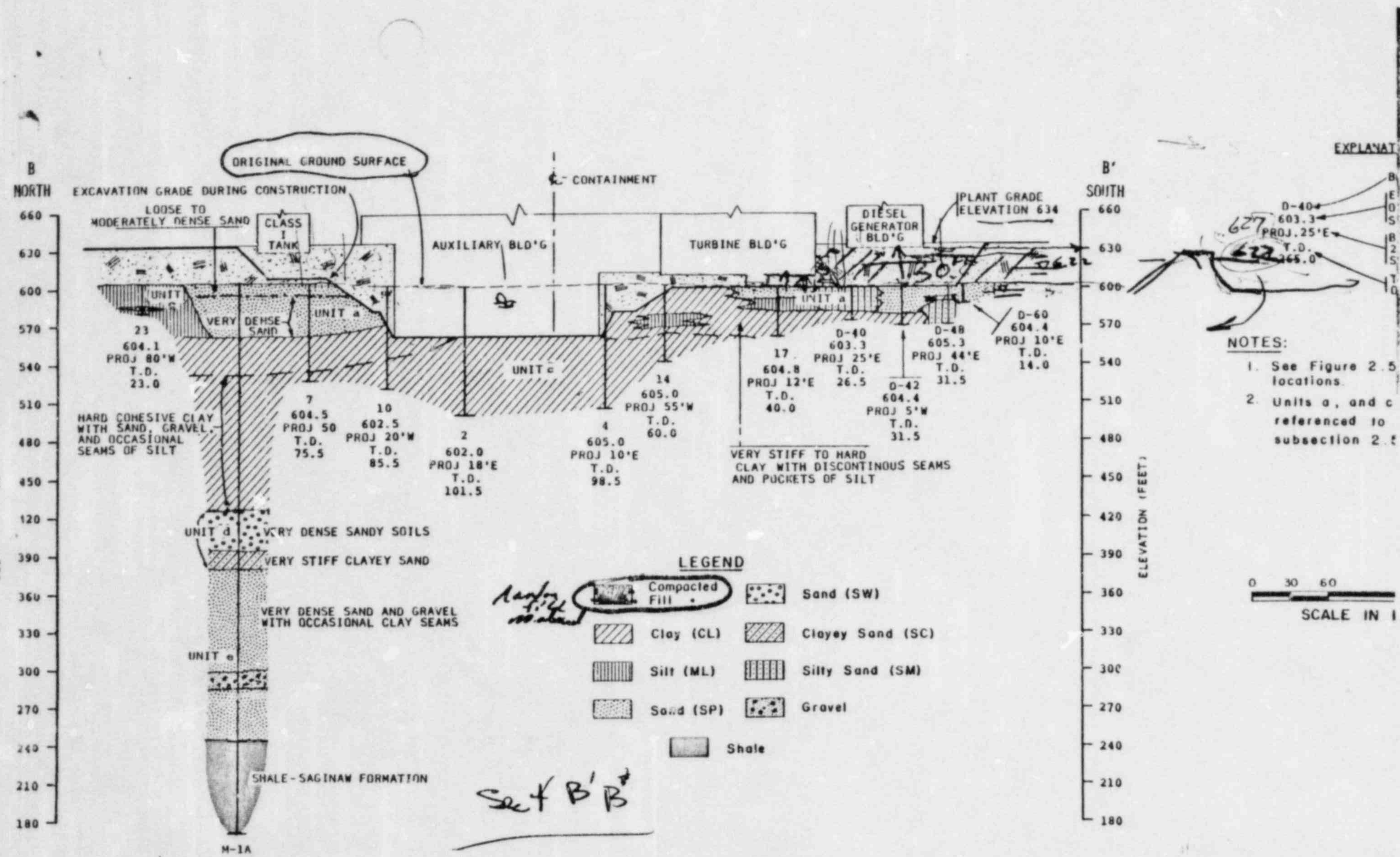
630
627
water 3' below D.G Bldg

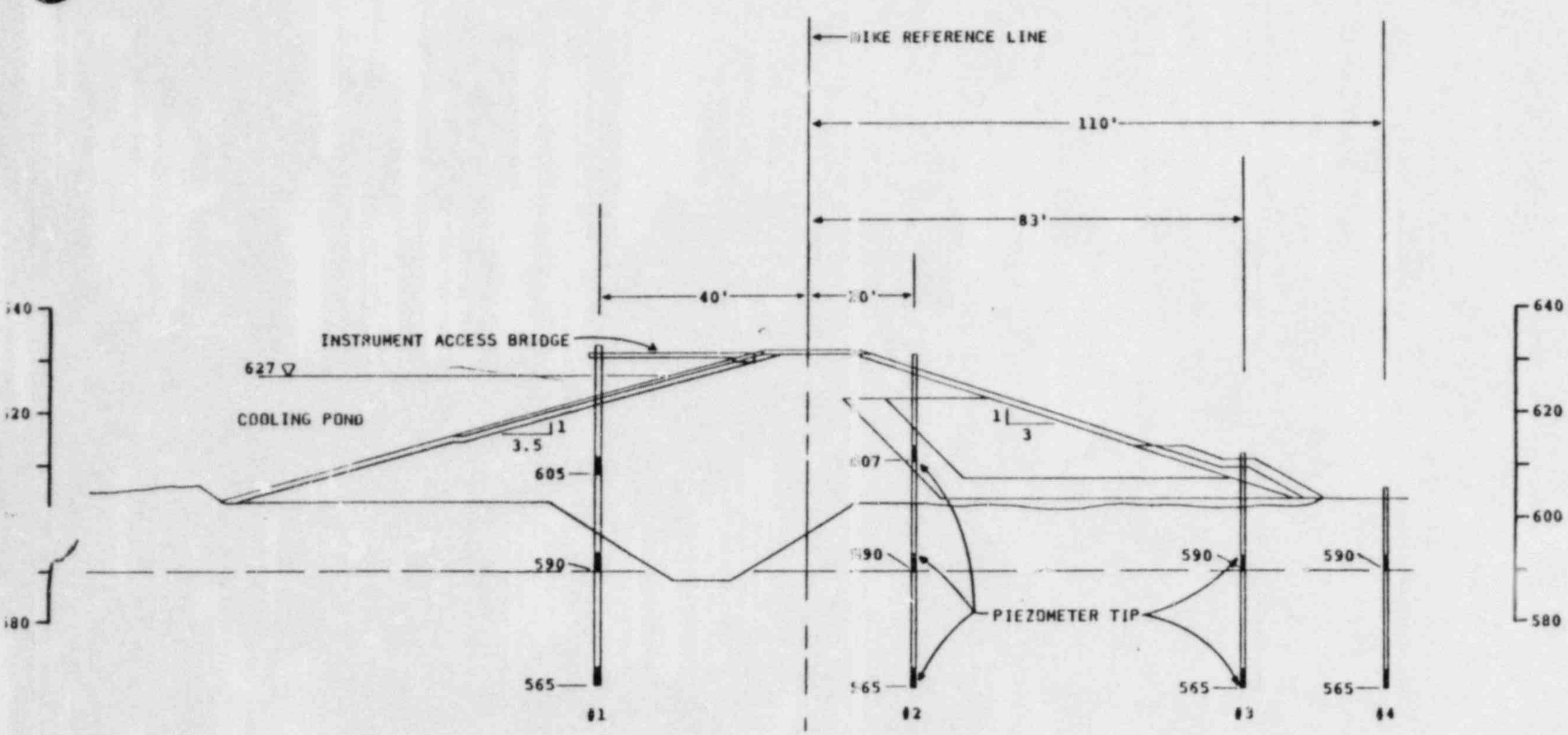
$\frac{1560\#/ft^2}{62\#/ft}$ = 25 FT of water under Turbine Bldg



NOTE:
ALL SETTLEMENT VALUES ARE IN INCH
S 4,500
S 4,700
S 4,900
S 5,100
WATER TANK STORAGE AREA
2.7
2.6
2.6
D RADWASTE
UILDING
3.0
2.8
2.4
2.3
1.7
1.6
AUXILIARY
BUILDING
UNIT 1
CONTAINMENT
2.4
2.3
2.3
2.0
2.5
1.6
UNIT 2
CONTAINMENT
2.1
1.9
2.2
2.1
2.4
0.7
0.7
0.7
2.5
2.8
2.2
2.9
TURBINE BUILDING
2.6
2.8
3.1
3.1
2.6
2.2
3.0
3.0
3.2
2.8
2.8
DIESEL GENERATOR
BUILDING
2.0
2.4
CONDENSATE
STORAGE
TANKS
N
0 50 100 200 30
SCALE IN FEET

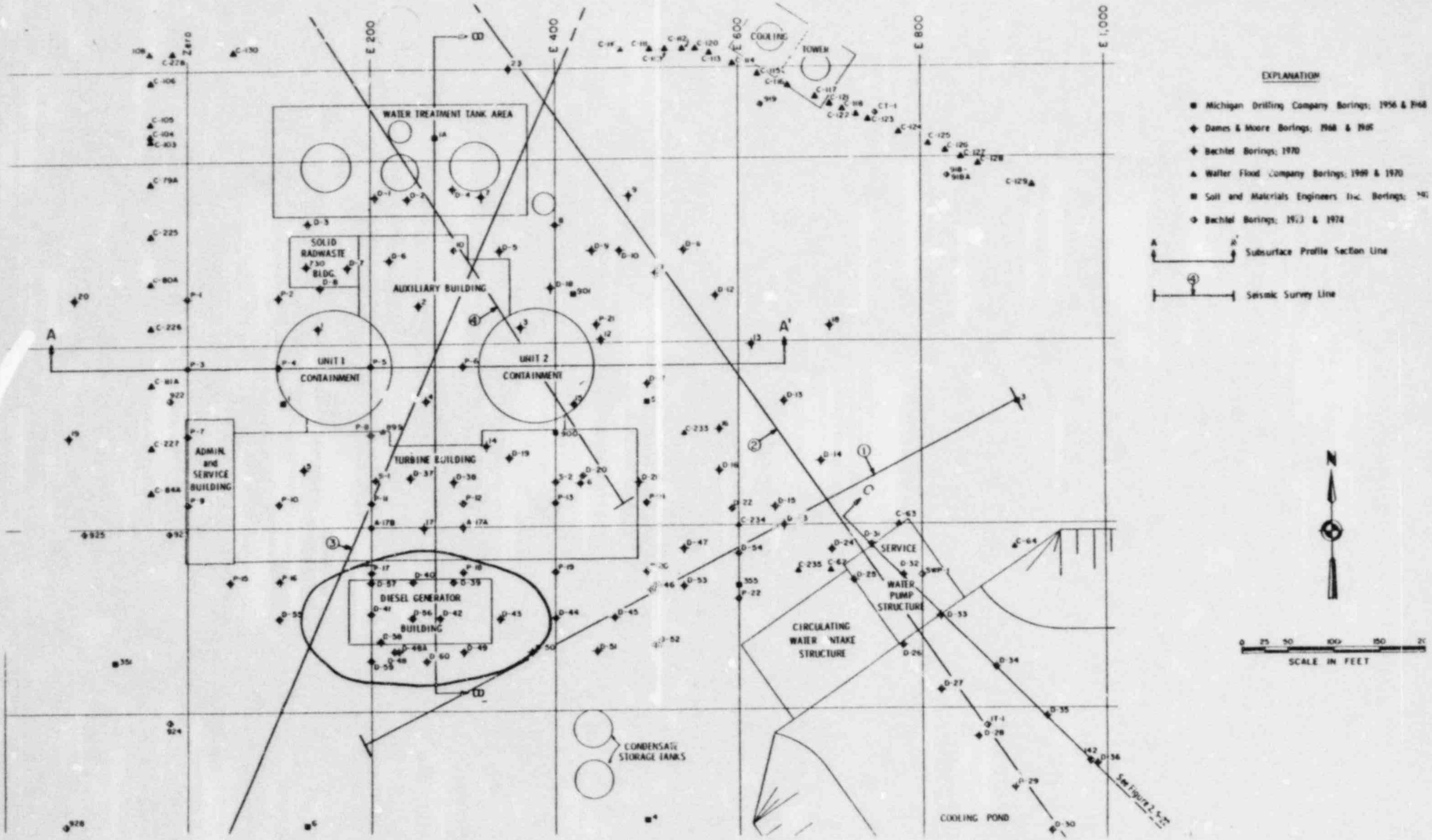
Fig 2.5-48

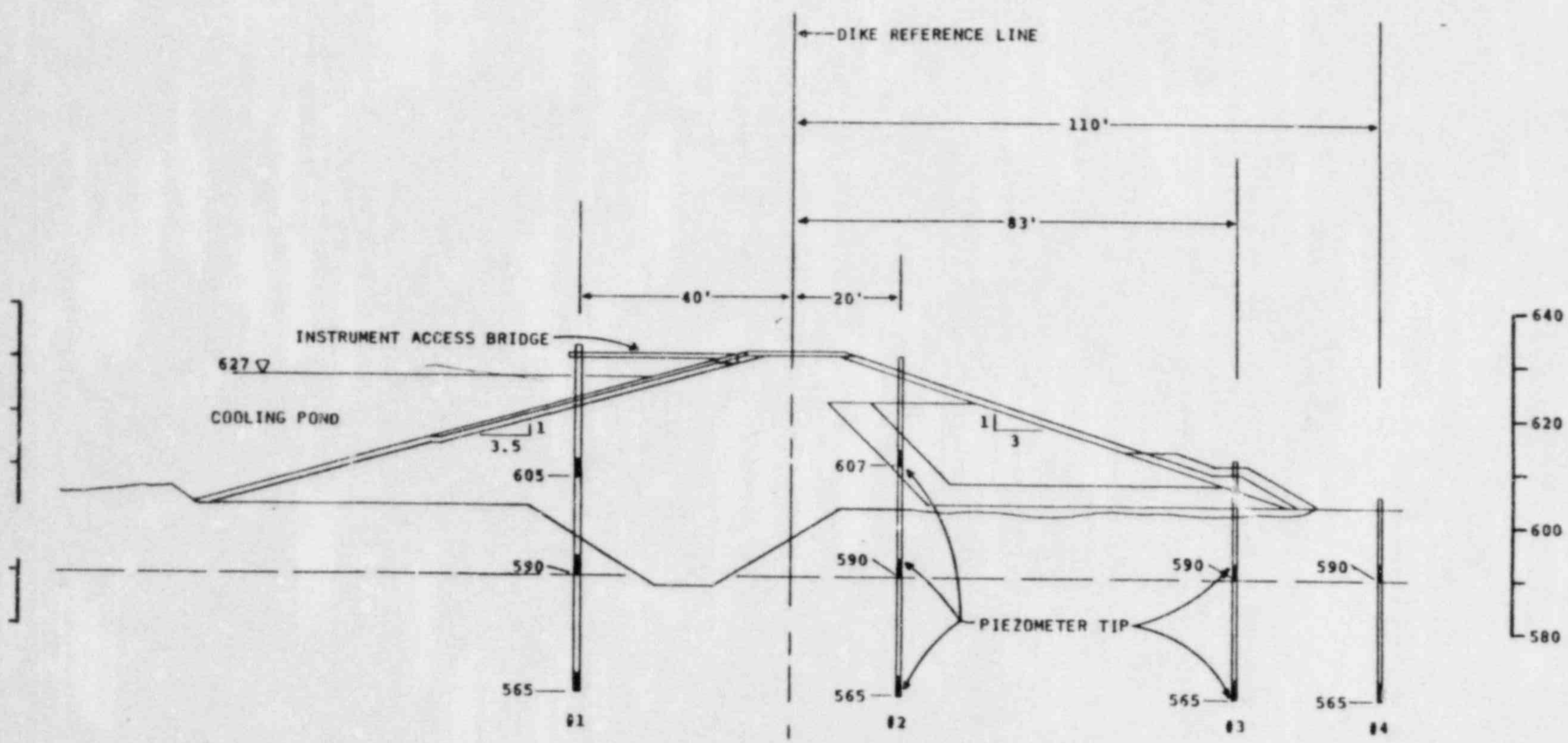

B
NORTH
B'
SOUTH
ORIGINAL GROUND SURFACE
EXCAVATION GRADE DURING CONSTRUCTION
LOOSE TO MODERATELY DENSE SAND
CLASS I TANK
CONTAINMENT
AUXILIARY BLD'G
TURBINE BLD'G
DIESEL GENERATOR BLD'G
PLANT GRADE ELEVATION 634
UNIT c
VERY DENSE SAND
UNIT a
UNIT c
UNIT a
23
604.1
PROJ 80'W
T.D.
23.0
7
604.5
PROJ 50
T.D.
75.5
10
602.5
PROJ 20'W
T.D.
85.5
2
602.0
PROJ 18'E
T.D.
101.5
4
605.0
PROJ 10'E
T.D.
98.5
14
605.0
PROJ 55'W
T.D.
60.0
17
604.8
PROJ 12'E
T.D.
40.0
D-40
603.3
PROJ 25'E
T.D.
26.5
D-42
604.4
PROJ 5'W
T.D.
31.5
D-48
605.3
PROJ 44'E
T.D.
31.5
D-60
604.4
PROJ 10'E
T.D.
14.0
HARD COHESIVE CLAY WITH SAND, GRAVEL, AND OCCASIONAL SEAMS OF SILT
VERY STIFF TO HARD CLAY WITH DISCONTINOUS SEAMS AND POCKETS OF SILT
UNIT d
VERY DENSE SANDY SOILS
VERY STIFF CLAYEY SAND
VERY DENSE SAND AND GRAVEL WITH OCCASIONAL CLAY SEAMS
UNIT e
SHALE-SAGINAW FORMATION
M-1A
ELEVATION (FEET)
660
630
600
570
540
510
480
450
420
390
360
330
300
270
240
210
180
LEGEND
Compacted Fill
Sand (SW)
Clay (CL)
Clayey Sand (SC)
Silt (ML)
Silty Sand (SM)
Sand (SP)
Gravel
Shale
Sec't B'B'
EXPLANAT
D-40
603.3
PROJ.25'E
T.D.
265.0
627
622
NOTES:
1. See Figure 2.5
locations.
2. Units a, and c
referenced to
subsection 2.5
0 30 60
SCALE IN



NOTES:
1. ALL PIEZOMETER DEP
LOCATIONS ARE APPR
2. EACH PIEZOMETER WI
INSTALLED IN A DIFF

REFERENCE:
SEE FIGURE 2.5-46 FO
CROSS SECTION


EXPLANATION
Michigan Drilling Company Borings; 1956 & 1968
Dames & Moore Borings; 1968 & 1969
Bechtel Borings; 1970
Walter Flood Company Borings; 1969 & 1970
Soil and Materials Engineers Inc. Borings
Bechtel Borings; 1973 & 1974
Subsurface Profile Section Line
Seismic Survey Line
SCALE IN FEET
N
WATER TREATMENT TANK AREA
SOLID RADWASTE BLDG.
AUXILIARY BUILDING
UNIT 1 CONTAINMENT
UNIT 2 CONTAINMENT
ADMIN. and SERVICE BUILDING
TURBINE BUILDING
DIESEL GENERATOR BUILDING
COOLING TOWER
SERVICE WATER PUMP STRUCTURE
CIRCULATING WATER INTAKE STRUCTURE
CONDENSATE STORAGE TANKS
COOLING POND
Zero
E 200
E 400
E 600
E 800
E 1,000
A
A'

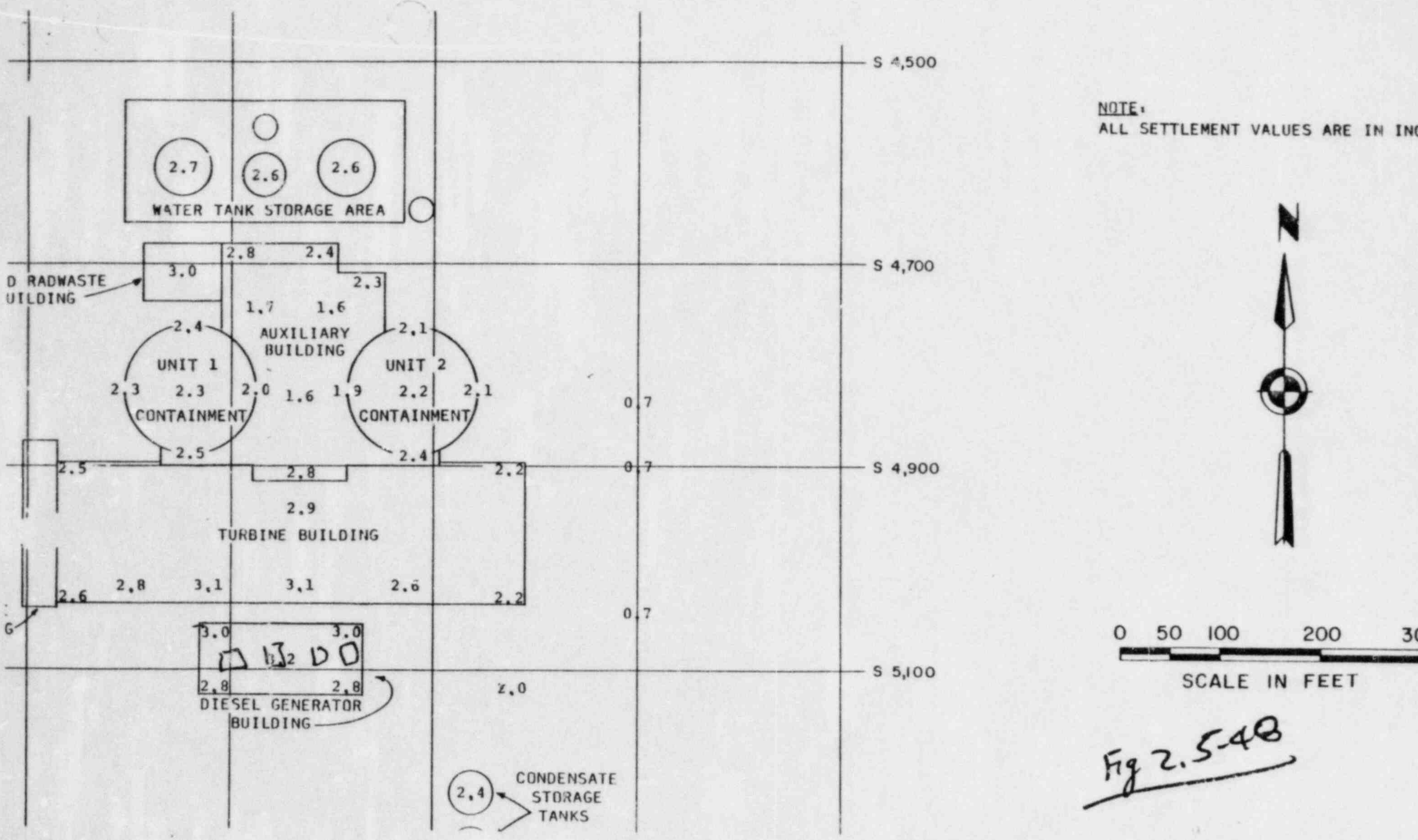


NOTES:

1. ALL PIEZOMETER DEP
   LOCATIONS ARE APPR
2. EACH PIEZOMETER WI
   INSTALLED IN A DIFF

REFERENCE:
SEE FIGURE 2.5-46 FO
CROSS SECTION

0 10 20
SCALE IN F



S 4,500
S 4,700
S 4,900
S 5,100
2.7
2.6
2.6
WATER TANK STORAGE AREA
D RADWASTE
UILDING
3.0
2.8
2.4
2.3
1.7
1.6
AUXILIARY
BUILDING
2.4
2.1
UNIT 1
UNIT 2
2.3
2.3
2.0
1.6
1.9
2.2
2.1
CONTAINMENT
CONTAINMENT
2.5
2.4
2.5
2.8
2.2
0.7
0.7
2.9
TURBINE BUILDING
2.6
2.8
3.1
3.1
2.6
2.2
0.7
G
3.0
3.0
2.8
2.8
2.0
DIESEL GENERATOR
BUILDING
2.4
CONDENSATE
STORAGE
TANKS
NOTE:
ALL SETTLEMENT VALUES ARE IN INCH
N
0
50
100
200
30
SCALE IN FEET

Fig 2.5-48

settlements are essentially elastic and occur as the loads are applied.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The materials, quality control, and special construction techniques used for the foundations are the same as for the structures themselves and are presented in Subsection 3.8.1.6.

3.8.5.7 Testing and Inservice Surveillance Requirements

Settlements of the foundations are monitored during and after construction. The details of the program are presented in Subsection 2.5.4.13.

3.8.6 GENERAL DESIGN OF SEISMIC CATEGORY I STRUCTURES

3.8.6.1 Design Criteria Used in All Seismic Category I Structures

The following subsections summarize the bases which are common to the design and construction of all Seismic Category I structures. Any bases which are pertinent to only one of the Seismic Category I structures will be discussed in the appropriate subsection related to that structure.

3.8.6.2 Applicable Codes, Standards, and Specifications

Codes, industry standards, specifications, design criteria, NRC Regulatory Guides, and Bechtel Topical Reports, which are referenced for the design and construction of all Seismic Category I structures, are described in the following subsections. Codes, standards, etc, which are applicable to a particular structure or modifications made to meet specific requirements of the structure, are indicated in the subsections related to that particular structure.

3.8.6.2.1 Codes

ACI	American Concrete Institute
ACI 318-63, ACI 318-71	Building Code Requirements for Reinforced Concrete
AISC	American Institute of Steel Construction

3.8.5.3 Loads and Load Combinations

Containment foundation loads and loading combinations are discussed in Subsection 3.8.1.3.

Foundation loads and loading combinations for other Siesmic Category I structures are discussed in Section 3.8.6.

3.8.5.4 Design and Analysis Procedures

Design and analysis procedures used in the design of the foundations are discussed in Subsections 3.8.1.4 for the containment and in 3.8.4.4 for other Siesmic Category I structures. Assumptions made on boundary conditions are discussed in detail in the computer program description presented in Appendix 3C. Lateral forces and overturning moments are transmitted to the foundation without exceeding the allowable bearing capacity limits. Values of the factors of safety against overturning, sliding, and flotation for the containment are presented in Table 3.8-23.

3.8.5.5 Structural Acceptance Criteria

The foundations of all Seismic Category I structures are designed to meet the same structural acceptance criteria as the structures. These criteria are discussed in Subsections 3.8.1.5 for the containment, 3.8.3.5 for the internal structures, and 3.8.4.5 for other Seismic Category I structures.

Minimum allowable factors of safety against sliding, overturning, and flotation are presented in Table 3.8-23.

Estimated maximum differential settlements which could occur between adjacent structures are presented in Table 3.8-24.

It is estimated that one-tenth to one-half of the maximum settlement occurs as elastic compression immediately after load application. The remainder of the settlements occurs in accordance with the rates estimated from consolidation test data as presented below:

Approximate Percent of Consolidation Settlement	Time in Years
20	2
50	10
90	50

Settlements of shallow spread footings founded on compacted fills are estimated to be on the order of 1/2 inch or less. These

3.8.5.1.2 Auxiliary Building

The auxiliary building is founded on reinforced concrete mat foundations at six different elevations as shown in Figure 3.8-61. The figure shows the bottom elevations and thicknesses of the mat foundations at different areas. The major portion of the auxiliary building (between column lines A and H in the north-south direction and between column lines 5.6 and 7.4 in the east-west direction) rests on a 6 foot thick reinforced concrete mat, 158'-3" long and 79'-0" wide, founded on glacial till, with the bottom elevation at 562 feet. The southern portion of the auxiliary building, south of column line H, rests on a 5 foot thick reinforced concrete mat with the bottom elevation at 609 feet. It is founded on compacted fill. The elevations and the thicknesses of the mat foundations in the other areas are shown in Figure 3.8-61. All the mat foundations with their bottom elevations above 570 feet are founded on compacted fill. Figure 3.8-62 and 3.8-63 show the cross-sections of the foundations and typical reinforcement details.

3.8.5.1.3 Diesel Generator Building

The foundation for the exterior and interior walls of the diesel generator building consists of continuous reinforced concrete footings, ~~10'-0" wide and~~ 2'-6" thick, with their base at elevation 628 feet. Adjacent to any pit location (e.g., sump pit), the exterior wall footing base is locally lowered to elevation 625 feet. The diesel generators rest on 6'-6" thick concrete pedestals. The overall arrangement of the foundation in relation to the superstructure is shown in Figure 3.8-55. The footings are placed on compacted fill.

3.8.5.1.4 Service Water Pump Structure

The foundation for the service water pump structure consists of two reinforced concrete mats at elevations 592 feet and 620 feet. The lower mat is 90 feet long, 74 feet wide, and 5 feet thick, and is founded on glacial till. The upper mat is 86 feet long, 38 feet wide, and 3 feet thick, and is founded on compacted fill. The details of the foundation and reinforcement are shown in Figure 3.8-56.

3.8.5.2 Applicable Codes, Standards, and Specifications

The applicable codes, standards, and specifications used in the structural design, fabrication, and construction of foundations are discussed in Subsection 3.8.1.2 for the containment, in Subsection 3.8.3.2 for the internal structures, and in Subsection 3.8.4.2 for other Seismic Category I structures.

3.8.4.7 Testing and Inservice Surveillance Requirements

A system of leak chase channels is connected to the outside surface of the fuel pool liner plate. These channels connect to piping that terminates in the sampling room below the spent fuel pool. Liner plate leakage may be checked by opening valves on the leak chase piping. Other testing and inservice surveillance is not required and a formal program of testing and inservice surveillance is not planned.

3.8.5 FOUNDATIONS FOR SEISMIC CATEGORY I STRUCTURES

3.8.5.1 Description of the Foundation

Subsequent subsections include a description of the foundation of each Seismic Category I structure.

Each foundation is designed to act independently by means of physical separation from adjacent structures. This independence permits differential settlement without adverse consequences and simplifies the seismic analysis.

The foundation design incorporates a waterproof membrane up to elevation 632 feet for the containment, the auxiliary building, and portions of the turbine building. Due to the multilevel configuration of the foundation, shear transfer will not be affected by the membrane.

3.8.5.1.1 Containment

Each containment foundation is a circular mat conventionally reinforced with bonded reinforcing steel. The diameter of the mat is 127'-10" and the thickness of the mat varies from 9 feet at the outer edge to 13 feet in the central portion. Figure 3.8-1 shows the containment foundation in relation to the rest of the structure. A continuous access gallery is provided beneath the mat foundation for installation and inspection of vertical tendons. A base liner is installed on the top of the mat and covered with concrete. Figure 3.8-4 shows a cross-section of the mat foundation with typical reinforcement details. The mat foundation is founded upon glacial till at the site. The engineering properties and bearing capacity of the glacial till are discussed in Subsection 2.5.4.

The internal structures that support large equipment, such as the reactor vessel, steam generators, and the primary and secondary shield walls, are anchored to the mat in order to transfer the loads. Figures 3.8-30 and 3.8-31 show the typical details of anchorage of the reactor vessel and steam generator to the base.

Figure 3.8-13 shows the typical reinforcement details at the junction of the base mat and the containment wall.

F

Inter-office Memorandum

BEBC- 2047

To

J. F. Newgen

Subject

Midland Plant Units 1 & 2
Job 7220
Administration Building
Foundation Settlement
Investigation

Copies to

File: 0274, C-1700, C-2600 both w/a

S. L. Blue w/a
F. E. Meyer w/o
P. A. Martinez w/o

Date January 13, 1978

From R. L. Castleberry

Of Engineering

At Ann Arbor

Gat.

Attached for your use is a copy of a report on the above subject which was prepared by the Geotechnical Services department. It is Project Engineerings understanding that this completes our participation in the subject investigation.

R. L. Castleberry

GAT/sg

Attachment

G. Richardson's (CGT)

Admin Bldg Report

rec'd by NRC on 1/4/79 cgs.