SUPPLEMENT TO REPORT

FOUNDATION INVESTIGATION AND
PRELIMINARY EXPLORATIONS FOR BORROW MATERIALS
PROPOSED NUCLEAR POWER PLANT
MIDLAND, MICHIGAN

FOR

CONSUMERS POWER COMPANY

5697-004-07

DAMES & MOORE

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March 15, 1969

Bechtel Corporation
P.O. Box 3965
San Francisco, California 94119

Attention: Mr. J. H. Blasingame,
Project Engineer

Gentlemen:

This letter transmits fifteen copies of our "Supplement to Report, Foundation Investigation and Preliminary Explorations for Borrow Materials, Proposed Nuclear Power Plant, Midland, Michigan for Consumers Power Company," dated March 15, 1969.

The scope of this investigation was planned in collaboration with Messrs. Flach, Martinez, Kulesza and Cherrington of Bechtel Corporation.

The data and recommendations presented in this report are intended to supplement those presented in our "Report of Foundation Investigation, and Preliminary Explorations for Borrow Materials," dated June 28, 1968, and are considered appropriate for final plant design.

It has been a pleasure to be of service to Consumers Power Company and Bechtel Corporation on this project, and we trust that you will contact us if you should have any questions or comments.

Yours very truly,

DAMES & MOORE

George D Leal

George D. Leal

GDL:WWM:mf

SUPPLEMENT TO REPORT

FOUNDATION INVESTIGATION AND

PRLIMINARY EXPLORATIONS FOR BORROW MATERIALS

PROPOSED NUCLEAR POWER PLANT

MIDLAND, MICHIGAN

FOR

CONSUMERS POWER COMPANY

INTRODUCTION

This report presents the results of a supplementary foundation investigation performed at the site of the Proposed Nuclear Power Plant to be constructed in Midland, Michigan for Consumers Power Company.

An initial foundation investigation was performed by Dames & Moore and the results presented in our "Report, Foundation Investigation and Preliminary Explorations for Borrow Materials, Proposed Nuclear Power Plant, Midland, Michigan," dated June 28, 1968. Subsequent to the initial investigation, the plant structures were relocated 150 feet to the east and 60 feet to the north of the original location. Because of subsurface conditions encountered at the new location, the plant structures were relocated a second time to a position 40 feet south and 20 feet east of the original location. The data and recommendations presented in this supplementary report are appropriate for the final plant location.

SCOPE

The purpose of the supplementary foundation investigation was to develop data and recommendations appropriate for fina. plant design. The specific program discussed and agreed upon for investigating the site consisted of the drilling and sampling of exploration test borings, the performance of a limited number of supplementary laboratory tests, the performance of appropriate engineering analyses, and the preparation of final recommendations and substantiating data.

This report is intended to be supplementary in nature and does not repeat discussion of items covered in the initial report unless required. Emphasis is given to the following specific information:

1 - Modified site description as necessitated by the additional explorations.

2 - Soil boring logs which include information on ground water levels at the time of drilling.

3 - Results of supplementary laboratory tests.

4 - Final foundation design criteria, including:

a) Allowable bearing pressure for shallow spread foundations on the compacted plant fill as a function of width for an allowable total settlement of 3/4 inch.

b) Lateral earth pressure against structure walls as a function of depth. In developing these data, the maximum probable flood has been assumed at elevation 632 feet, and the top of plant fill has been assumed at elevation 634 feet. For normal conditions, the ground water level has been assumed at elevation 625 feet, the reservoir water surface elevation.

c) Recommended foundation type for the reactor buildings, the turbine building, and for the turbine generators. The estimated total settlement and maximum differential settlement are provided for recommended foundation types.

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.

e) Differential settlements between auxiliary building, reactor, and turbine buildings.

5 - Review of recommendations regarding site preparation and earthwork, as follows:

a) Recommended excavation slopes in natural soils and in plant fill.

b) Control of ground water in excavations for the reactor and turbine buildings.

c) Compaction requirements for the plant fill (I) beneath structures, and (II) adjacent to structures.

d) Minimum depths of footings in compacted soil for frost protection or other reasons.

The results of our supplementary field explorations and laboratory tests are presented in the Appendix to this report.

DESIGN CONSIDERATIONS

Subsequent to the completion of the initial investigation, planned foundation elevations of all the major structures have been modified and more detailed structural design data has become available. A summary of pertinent structural data is given below.

The reactor building foundations will be established at elevation 582.5. They will be structurally separated from the adjacent auxiliary building, and the maximum allowable differential settlement between auxiliary building and reactor buildings has been established at three-quarters of an inch.

The auxiliary building plan dimensions will be 166 feet by 161 feet. This building abuts both of the reactor buildings and the turbine building. The central portion of the auxiliary building, 76 feet by 131 feet in plan dimensions, will be founded at elevation 562.0 feet; both parts of the auxiliary building abutting the reactor buildings will be founded at elevation 580. The remainder of the auxiliary building, located adjacent to the turbine building, and between it and the reactor buildings, will be founded at elevation 610.

Plan dimensions of the turbine building will be approximately 132 feet by 436 feet with a base elevation of 610. This building will house two turbine-generators supported on mat foundations established at approximately elevation 602 feet. The turbine-generator mat foundations will have plan dimensions of 145 feet by 45 feet and 185 feet by 45 feet.

Foundation loads imposed by the various structures under normal operating conditions and under seismic loading conditions are tabulated below.

STRUCTURE	FOUNDATION ELEVATION, FEET	FOUNDATION LOADING, LBS./SQ.FT. DEAD AND LIVE LOAD	DEAD, LIVE AND SEISMIC LOAD MAXIMUM	MINIMUM
Reactor Building	582.5	8,000	16,000	0
Auxiliary Building	562.0	6,500	13,000	0
	580.0	5,000	10,000	0
	610.0	3,500	7,000	0
Turbine Building	610.0	3,000	5,000	1,000
Turbine Generator Mat Foundations	602.0	4,500	9,000	0

The locations and foundation loading data relative to the appurtenant structures have not been provided to us.

Final plant grade has been raised approximately six feet and will be established at approximately elevation 634. Normal ground water level as in the initial investigation, is assumed to be at the existing ground surface, approximately elevation 603. However, this may be a perched water level. The water level in the cooling pond reservoir will be at approximately elevation 625. The underdrainage system considered in the initial report has been eliminated; consequently, it is assumed that the ground water level in the plant area will rise concurrently to approximately elevation 625. The maximum probable flood level will remain at elevation 632.

SITE CONDITIONS

SUBSURFACE CONDITIONS

General geologic conditions, and surface conditions at the site have been discussed in our initial report.

The subsurface conditions at the site were further investigated by drilling 11 supplementary exploration test borings and 22 probings to depths ranging from 10 to 80 feet at the locations shown on Plate 2.

The supplementary borings and probings provided more detailed information regarding the sandy soils, which generally underlie the topsoil and/or organic silty soils. These sandy soils consist of brown and gray fine sands which grade from loose near the surface to very dense with increasing depth. Although there is little or no sand within the central part of the plant area, the sand stratum does extend to approximately elevation 585 feet at both the east and west ends of the turbine building. Similarly, the bottom of the sand stratum varies from approximately elevation 600 in the vicinity of the west reactor building area to approximately elevation 575 feet near the north-eastern edge of the east reactor building area and along a part of the northern edge of the auxiliary building area.

The presence of very stiff to hard cohesive soils, predominantly gray silty clay, underlying the surface sand deposits was confirmed by the supplementary boring program.

More detailed descriptions of the subsurface soil penetrated by the supplementary borings are presented on the Log of Borings in the Appendix to this report.

SURFACE WATER

The site is presently subjected to periodic flooding. We understand that maximum probable flood level has been estimated at elevation 632 feet, which is the same elevation assumed in our initial report.

GROUND WATER

Seepage water entered some of the borings through the sand stratum blanketing the site. Ground water observations in the supplementary borings were consistent with those discussed in the initial report. A perched water condition probably exists in the sandy surface soils, and it has been conservatively estimated that the perched ground water level is at or near the existing ground surface. The underlying silty clay soils are saturated, but the present ground water level in these impervious materials could not be determined during the short term of our field investigations.

LABORATORY TESTS

The results of the laboratory tests performed in connection with the supplemental investigation, together with a description of the test procedures, are presented in the Appendix to this report.

A summary of all laboratory strength tests, and moisture and density tests, performed on soil samples extracted from borings drilled in the power plant area are presented on Plate 5, Summary of Test Data.

DISCUSSION AND RECOMMENDATIONS

GENERAL:

The results of our supplementary investigation confirm that the site is suitable, from a foundation standpoint, for the support of the proposed plant structures. Initial recommendations regarding suitable foundation types for various structures are considered applicable. These recommendations are summarized below.

It is recommended that the reactor buildings and the lower portion of the auxiliary building be supported on mat foundations established at the planned elevations, in the very stiff to hard cohesive soils.

It is recommended that the turbine building, the higher south portion of the auxiliary building, and the turbine-generators be supported on mat foundations established in controlled compacted fill at the planned elevations. Prior to the placement of fill, it is recommended that all topsoil, loose sand and other unsuitable soils be excavated from the turbine building area and the south portion of the auxiliary building area. The exposed natural soils should be thoroughly proof-rolled prior to commencing filling operations.

It is recommended that appurtenant structures be supported on spread foundations established in the controlled compacted fill.

The more detailed structural design data and the additional subsurface data available at this time permit a final analysis of total and differential settlements. Foundation design data and the results of the settlement analysis are presented in subsequent sections of this report.

Recommendations regarding earthwork operations are presented in the following section.

EARTHWORK:

The supplementary investigation requires certain modifications in our initial recommendations regarding dewatering, excavating, filling and backfilling.

Dewatering - The supplementary investigation has indicated that more extensive dewatering operations will be required than originally anticipated due to the greater amount of sandy surface soils encountered in the immediate plant area.

Plant excavations will extend into sandy surface soil below the ground water level and into relatively impervious clay soils. The depth of the sandy surface soils in the vicinity of the plant structures ranges from 0 to approximately 35 feet, with the maximum depth of sand occurring near the south western corner of the turbine building. The maximum depth of excavation will be on the order of 40 feet, to elevation 562.0, for the auxiliary building.

Only minor water seepage is anticipated in the lower clay soils. However, dewatering operations will be required in connection with excavations into the upper sandy soils. The ground water level, presently assumed to be at approximately elevation 603, may vary during the construction period in response to rainfall, surface runoff conditions, and the water level in the adjacent Tittabawasse R ar.

We understand that a seepage cutoff wall will be installed which will minimize the flow of seepage water through the sandy soils into the plant excavations. The location of the seepage cut off wall is shown on Plate 2, Site Plan. In order to supplement ground water control in the excavations, it is recommended that the ground water level inside the seepage cutoff wall be lowered as required by a well-point or deep-well dewatering system.

The subsurface conditions at the site have been discussed with a representative of the Griffin Wellpoint Corporation, a qualified dewatering contractor. After having been familiarized with the soil conditions, the following schemes were proposed by the Griffin Wellpoint Corporation.

1 - A single stage well-point system would be installed to lower the water level in the sandy soils inside the seepage cutoff wall to approximately elevation 575. In areas where the depth of sandy soils exceeds approximately elevation 575, a second stage of well-points would be installed to lower the water level to approximately elevation 560. It is anticipated that well-points will have to be installed with vertical sand filter-wicks to maintain the required drainage and draw-down. A copy of correspondence from Griffin Wellpoint Corporation and their sketch of proposed locations of the upper and lower dewatering systems is attached to the Appendix of this report.

2 - As an alternative to the above, particularly in areas where the sandy soils extend to depths below the bottom of excavations, it may be more economical to install several peripheral wells to depths below the plant excavations. These deep wells should be designed and operated such that the ground water level in the vicinity of the plant excavations is maintained below the bottom of the excavations.

The dewatering schemes outlined above are considered suitable, but appropriate field pumping tests should be performed prior to selecting a dewatering contractor. The field pumping tests would provide data to allow the choice of the most suitable type of dewatering system (well-points or deep wells), and would provide additional data for contractor bidding purposes. We would be pleased to provide guide specifications and technical supervision during the performance of field pumping tests, if required.

The dewatering system should maintain the water level in the sandy soils at least three to five feet below exposed excavated surface. Piezometers should be installed and monitored to insure that the water level in the sandy soils is continuously maintained at the recommended level.

In peripheral areas where the sandy surface soils are shallow, and surface water is not intercepted by other means, it is recommended that a peripheral drainage trench system be installed around the outside of excavations. The perimeter drainage system should consist of trenches excavated through the sandy surface soils and graded to drain away from the plant area. The trenches should be backfilled with clean gravel or other pervious material. Inside the excavation it is recommended that ground water seepage be controlled by a system of shallow peripheral trenches and sumps. Pumps will be required to remove water which accumulates in the trench-sump system.

Excavating - This section presents recommendations pertaining to excavating operations required to attain the modified planned grades and to prepare soils for the support of foundations or fill materials.

The maximum depth of excavation will be on the order of 40 feet in the vicinity of the auxiliary building.

Providing stripping is carried out in the manner recommended in our previous report and stripped soils are wasted, all remaining soils to be excavated will be suitable for use as fill or backfill. Detailed recommendations for the use of these soils are given in a subsequent section.

In addition to the excavation required to attain foundation levels, it is recommended that all on-site sands be excavated from below foundation level in the reactor building and auxiliary building areas, and that these soils be replaced by either compacted sand or clay fill soils. Based on the results of our field explorations, we anticipate that only very minor amounts of in-situ sands may be encountered at the foundation level of these structures. Where over-excavation is required, subgrade preparation and the backfilling to attain foundation levels should be carried out in the manner outlined in subsequent sections.

All loose in-situ sands, soft or compressible clay soils, and organic soils should be excavated in the turbine building area. Based on the results of the supplementary field explorations, it is anticipated that the depth of excavation of unsuitable soils will vary from one to five feet with an average over the area of approximately three feet. The excavation of these unsuitable soils, and subsequent backfilling with controlled compacted fill where required, is necessary in order to provide uniform foundation support for the turbine building and turbine-generator foundations. The plan dimensions of the excavated area should include the "zone of influence" of the mat foundations established in the controlled compacted fill. For purposes of excavation and filling, the "zone of influence" of a foundation is defined as the zone within planes extending downward and outward from the bottom outside edge of a foundation at an angle of 45 degrees with the horizontal.

Engineering studies have been performed to evaluate the stability of slopes constructed through the upper dewatered sandy soils and the underlying very stiff to hard clay soils. Based on the results of these studies, it is recommended that the banks of excavations through the dewatered sandy soil be cut on a slope of one vertical to one and one-half horizontal or flatter. Banks of excavations cut through the clay soils may be cut on a slope of two vertical to one horizontal or flatter. Banks of temporary excavations within the clay soils which are not subject to surcharge loading may be cut vertically with an unsupported height of up to 15 feet. It is anticipated that localized sloughing and spalling of the banks of excavations will occur due to drying and shrinking of the banks and also due to the presence of discontinuous lenses and pockets of silt in the clay soils.

Subgrade Preparation - Following stripping and excavating it is recommended that the exposed surfaces be thoroughly proof-rolled under the supervision of a qualified soils engineer. Where practical both foundation and fill subgrades should be proof-rolled to compact the exposed surfaces and to detect any localized zones of soft soils. As a guide, the proof-rolling operation could be considered equivalent to making approximately two passes over the entire exposed subgrade with a 20-cubic yard capacity loaded motor scraper. In deep excavations or limited access areas, smaller equipment making more passes would be suitable for proof-rolling.

Zones of loose or soft soils delineated by proof-rolling should be compacted if possible or removed and replaced with controlled compacted fill.

Upon attainment of final foundation grade in each area, it is recommended that a working mat of lean concrete be poured. The installation of a lean concrete "mud mat" or similar protection should minimize disturbance of the subgrade soils due to water seepage and construction operations. The mud mat will not provide protection against freezing and thawing of the subgrade soils.

The clay soils are susceptible to loss of strength due to frost action, disturbance and/or the presence of water. If the construction schedule requires that foundation excavations be left open during the winter, it is recommended that excavating operations be performed such that at least three and one-half feet of natural soils or similar cover remain in place over the final subgrade or overlying the "mud mat." This layer of protective material is necessary to prevent the softening and disturbance of the subgrade soils due to frost action.

Mud mats or similar means of protection should also be installed on the banks of excavations which lie within the building areas. The mud mat will provide protection against drying and resaturation which could lead to weakening and spalling of slopes.

<u>Filling and Backfilling</u> - Fills up to approximately 35 feet in thickness will be required in the attainment of the final plant grade elevation 634. In addition, fills and backfills will be required below and adjacent to structures.

As previously mentioned, on-site excavated soils, both sands and clay soils are considered suitable fill materials. Provided either soil type is placed and compacted in accordance with the criteria recommended below, it is considered unnecessary, from performance considerations, to specify the selective use of one or other of these soil types for any of the fills or backfills which will be required; however, as sands are more readily compacted with small equipment such as hand operated vibratory equipment it is recommended that sand fill be used in areas of limited access.

All fill and backfill materials should be placed at or near the optimum moisture content in nearly horizontal lifts approximately six to eight inches in loose thickness. Each lift should be compacted in accordance with the following criteria for the construction of controlled compacted fill and backfill.

In addition, no compacted soils should be allowed to freeze. If filling or backfilling operations are discontinued during periods of cold weather, it is recommended that all frozen soils be removed or recompacted prior to the resumption of operations.

Engineering studies have been performed to evaluate the stability of slopes constructed through the plant fill. Based on the results of these studies, it is recommended that the banks of temporary excavations through dewatered sand fill soils be cut on a slope of one vertical to one and one-half horizontal or flatter. Banks of temporary excavations through compacted clay fill soils which are not subject to surcharge loading may be cut vertically with an unsupported height of up to ten feet.

It is recommended that permanent slopes through granular compacted fill soils be constructed on slopes of one vertical to four horizontal or flatter. Permanent slopes through cohesive compacted fill soils may be constructed on slopes of one vertical to two horizontal.

Filling operations should be performed under the continuous technical supervision of a qualified soils engineer who would perform in-place density tests in the compacted fill to verify that all materials are placed and compacted in accordance with the recommended criteria.

RECOMMENDED MINIMUM COMPACTION CRITERIA

PURPOSE OF FILL	ON-SITE SAND SOILS PERCENT RELATIVE DENSITY*	ON-SITE CLAY SOILS PERCENT OF MAXIMUM DENSITY**
Support of Structures	85	100
Adjacent to Structures	75	95
Areal Fill (Not supporting or adjacent to structures)	70	90

* Maximum and Minimum density of sand soils should be determined in accordance with A.S.T.M. Test Designation D-2049-64T.

** Maximum dry density and optimum moisture content should be determined in accordance with A.S.T.M. Test Designation D-698, modified to require 20,000 foot-pounds of compactive energy per cubic foot of soil.

FOUNDATION DESIGN DATA

General - Foundation design data presented in this section assumes that individual building areas will be prepared in the manner previously recommended. It is our opinion that the major plant structures may be satisfactorily supported on mat foundations established at the presently planned elevations. Similarly, shallow spread foundations founded on controlled compacted fill soils will provide satisfactory support for the appurtenant structures.

Mat Foundations - The ultimate bearing capacity of the supporting soils underlying each of the major structures has been re-evaluated to reflect modified foundation elevations. The results of these analyses are tabulated below:

UNIT	SUPPORTING SOILS	FOUNDATION ELEVATION (FEET)	ULTIMATE BEARING CAPACITY LBS./SQ.FT.
Reactor Building	Very stiff to hard natural clay soils	582.5	45,000
Auxiliary Building	Very stiff to hard natural clay soils	562.0 580.0	50,000 45,000
	Controlled compacted fill	610.0	30,000
Turbine Building	Controlled compacted fill	610.0	30,000
Turbine-Generators	Controlled compacted fill	602.0	30,000

The above tabulation assumes that fill will be composed of compacted clay soils; if compacted sand fill is used the ultimate bearing capacities listed above will be greater than the tabulated values. The tabulated ultimate bearing pressures are gross values; thus the weight of foundations should be included in computing the foundation loads. The effects of overburden to elevation 634, and the effects of ground water at elevation 625 have been considered in the bearing capacity analysis.

The following tabulation presents a summary of the factors of safety revised to reflect the modified loading conditions and ultimate bearing capacities for the various units:

UNIT	FACTOR OF SAFETY DEAD AND LIVE LOADS	FACTOR OF SAFETY DEAD, LIVE AND SEISMIC LOADS
Reactor Buildings	5.6	2.8
Auxiliary Building		
@ Elevation 562.0	7.7	3.8
@ Elevation 580	9.0	4.5
@ Elevation 610	8.6	4.3
Turbine Building	10.0	6.0
Turbine-Generators	6.7	3.3

Shallow Spread Foundations

The recommended bearing pressures for shallow spread foundations have been calculated assuming the ground water level to be at elevation 625 and assuming that the supporting compacted fill materials may be either clay or sand soils.

FOUNDATION WIDTH	ALLOWABLE NET BEARING PRESSURES (POUNDS PER SQUARE FOOT) CLAY SOILS	ALLOWABLE NET BEARING PRESSURES (POUNDS PER SQUARE FOOT) SAND SOILS
2	5,000	2,800
4	5,000	3,100
8	5,000	3,700
12	5,000	4,300

The factor of safety and allowable increase for seismic loads are the same as previously recommended.

SETTLEMENT

General - Settlement analyses are based on the results of consolidation tests performed on undisturbed and recompacted soil samples. Consolidation test data are presented in the Appendix of this report. The consolidation tests performed in connection with the supplemental investigation confirm that the very stiff to hard clay soils have been preconsolidated under overburden pressures of at least 15,000 to 20,000 pounds per square foot.

The settlement analyses consider the effects of lowering the ground water level, excavating, placement of areal fill, subsequent raising of ground water level and the associated time considerations.

Mat Foundations

The results of our settlement analyses for structures supported on mat foundations are tabulated below:

UNIT	ESTIMATED MAXIMUM SETTLEMENT INCHES	ESTIMATED MAXIMUM DIFFERENTIAL SETTLEMENT INCHES
Reactor Buildings	1 - 1½	¼ - ½
Auxiliary Building		
@ Elevation 562	½ - 1	¼ - ½
@ Elevation 580	½ - 1	¼ - ½
@ Elevation 610	1½ - 2	¼ - ½
Turbine Building	1½ - 2	¼ - ½
Turbine-Generator Mats	1½ - 2	¼ - ½

It has been further estimated that the maximum differential settlement which will occur between adjacent structures will be as follows:

ADJACENT UNITS	ESTIMATED MAXIMUM DIFFERENTIAL SETTLEMENTS BETWEEN STRUCTURES INCHES
Auxiliary @ Elevation 562 and @ Elevation 580	1/2
Auxiliary @ Elevation 562 and @ Elevation 610	1
Auxiliary @ Elevation 580 and Reactor	1/2
Auxiliary @ Elevation 610 and Reactor	3/4
Auxiliary @ Elevation 610 and Turbine Building	1/2
Turbine Building and Turbine Mat	1/2

The results of the dynamic settlement analysis presented in the initial report are considered applicable to the revised plant design and final location. Additional settlement under dynamic loading should not exceed one-quarter inch. The appropriate range of values for modulus of elasticity for dynamic settlement analysis is discussed in the Appendix to this report.

Appurtenant Structures - The total and differential settlements of buildings supported on shallow-spread foundations will depend on (1) the surface settlement of the areal fill and (2) the settlement caused by the individual foundations imposing bearing pressures on the order of the allowable bearing pressures previously recommended.

Neither building locations nor the individual column loads have been made available to us at this time. Analysis shows that the areal fill will undergo long term settlements on the order of $1\frac{1}{2}$ to 2 inches. It is estimated that shallow spread foundations supporting a total design load of up to 30,000 pounds and proportioned utilizing the bearing pressures presented above will undergo settlement on the order of one-half inch or less.

If necessary, the long term total and differential settlement of each appurtenant structure will be analyzed when the locations and structural loads of these structures are known.

Time-Rate of Settlement - It is estimated that one-~~third~~ tenth to one-half of the maximum settlements tabulated previously will occur, as elastic recompression, essentially simultaneously with the load application. The remaining one-half to two-thirds of the maximum settlements will occur in accordance with the time-rates estimated from consolidation test data and presented below.

APPROXIMATE PERCENT OF SETTLEMENT, AFTER RECOMPRESSION	TIME YEARS
20	2
50	10
90	50

Settlement of conventional spread foundations, established on an appreciable thickness of controlled compacted granular fill will occur essentially as the load is applied to the foundation.

LATERAL PRESSURES

The walls of structures below final plant grade, elevation 634, will be subjected to horizontal loads imposed by backfill materials, hydrostatic pressures, and the horizontal components of adjacent foundation loads. Excluding the horizontal components of adjacent foundation loads, it is recommended that long term lateral pressures against rigid and non-rigid walls be computed using the equivalent fluid unit weights tabulated below:

	BACKFILL MATERIAL ADJACENT TO STRUCTURE	EQUIVALENT FLUID UNIT WEIGHT (LBS./CU.FT.) ABOVE WATER LEVEL	BELOW WATER LEVEL
NON-RIGID WALLS:			
	Sand Soils	40	80
	Clay Soils	50	90
RIGID WALLS:			
	Sand Soils	60	100
	Clay Soils	80	110

Lateral pressures developed adjacent to rigid walls immediately following placement and compaction of backfill materials may exceed the long term pressures in the portion of the wall near the ground surface. Therefore, we recommend that rigid walls be designed for the equivalent fluid unit weights presented above or a uniformly distributed pressure of 600 pounds per square foot, whichever is greater at any particular depth.

The above recommended equivalent fluid pressures assume backfill soils will be placed in a carefully controlled manner. The stiff to hard on-site clay soils should not be placed as layers of chunky soil which require excessive compactive effort to obtain a homogeneous compacted fill. Such a procedure would increase the equivalent fluid pressure on the order of 50 percent. The use of clay backfill in any areas of limited access is not recommended.

Substructure walls which are established below adjacent foundations should also be designed to resist the horizontal components of adjacent foundation loads. For preliminary analysis of lateral foundation pressures we suggest the method of analysis presented in Spangler and Mickle's* paper "Lateral Pressures on Retaining Walls Due to Backfill Surface Loads." For final analysis, after the final arrangement of facilities, type of backfill, and final loading conditions are known, it is suggested that horizontal components of foundation loads acting on adjacent walls be evaluated by finite element analysis.

UPLIFT PRESSURES

Uplift loads will be resisted by the dead weight of the structures, the weight of the backfill materials, directly overlying the foundations, if any, and the frictional resistance between the structure and the adjacent backfill materials. The unit weight of the backfill materials may be taken as 120 pounds per cubic foot above the assumed ground water level, and 60 pounds per cubic foot below the assumed ground water level. The frictional resistance may be computed by assuming a coefficient of lateral earth pressure equal to 0.35 and a coefficient of friction between soil and concrete of 0.35.

These values apply to backfill soils composed of clean sand and pertain to ultimate frictional resistance to uplift. An appropriate factor of safety on the order of 1.5 for normal operating conditions and 1.2 for maximum probable flood conditions should be applied to the ultimate values.

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* Spangler, M.G. and Jack L. Mickle, "Lateral Pressures on Retaining Walls Due to Backfill Surface Loads," Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, P. 155, 1936.

If clay backfill soils are used, the ultimate frictional resistance to uplift may be computed in a similar manner, except that the coefficient of friction between soil and concrete should be reduced to 0.25.

Floor slabs established below the design floor level should be designed for full hydrostatic pressure or should be provided with adequate drainage facilities.

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The following Plates and Appendix are attached and complete this report:

Plate 2	-	Site Plan (Revised Reservoir and Power Plant Areas)
Plate 3	-	Plot Plan (Power Plant Area, Revised)
Plate 4B	-	Generalized Subsurface Section B-B (Revised)
Plate 5	-	Summary of Test Data (Revised)
Appendix	-	Field Explorations and Laboratory Tests

Respectfully submitted,

DAMES & MOORE

George D. Leal
Registered Professional Engineer
State of Michigan
Certificate No. 17383

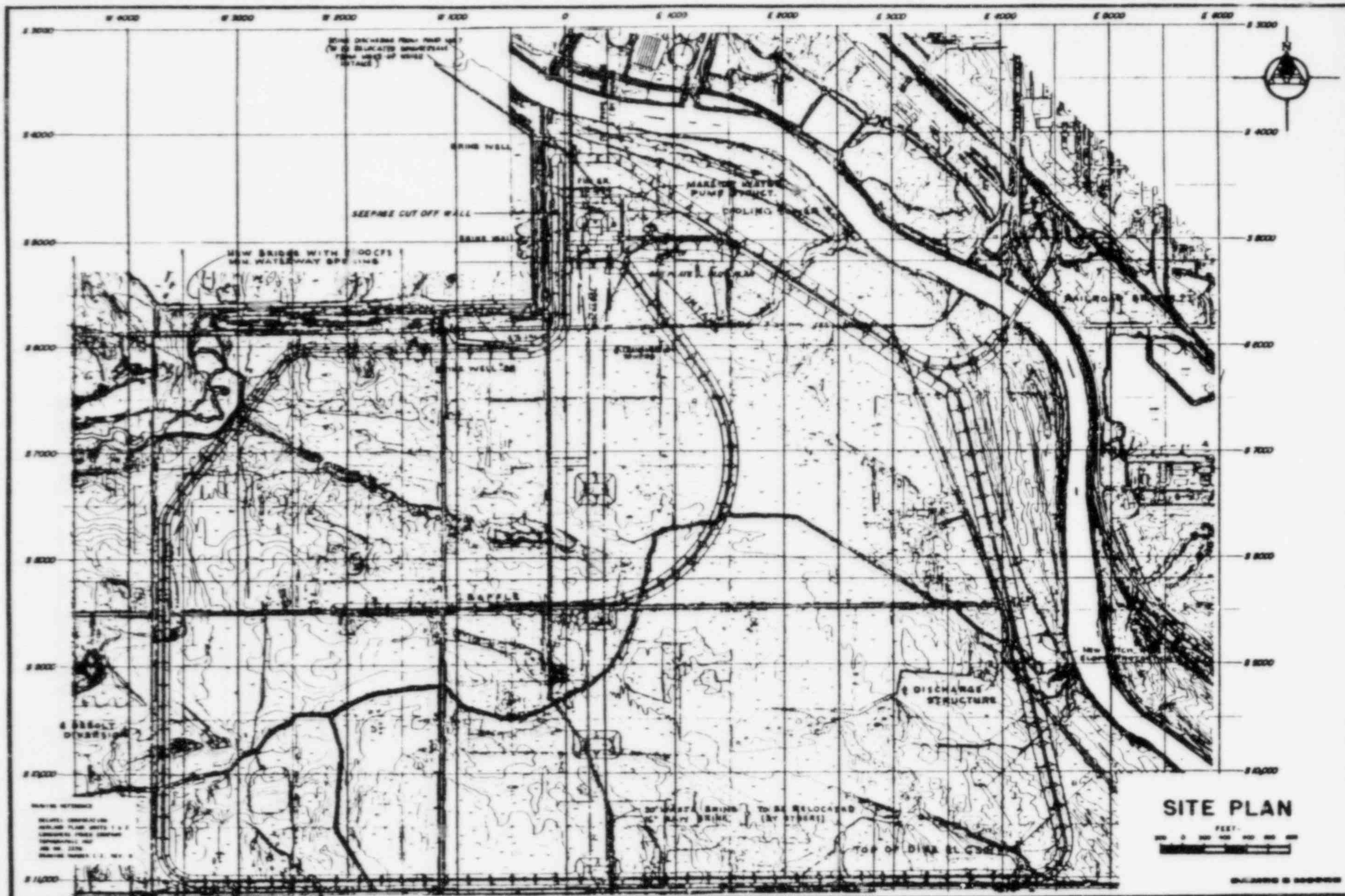
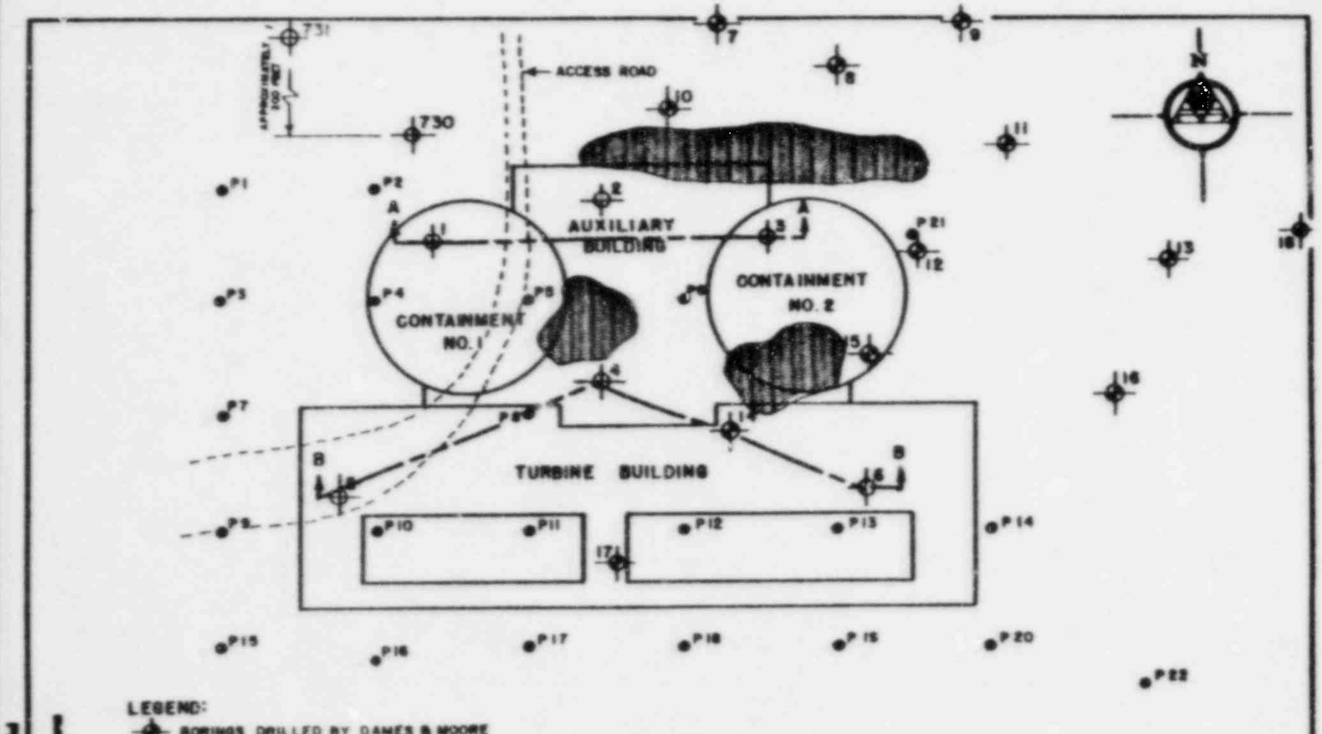

SITE PLAN
FEET
BRINE WELL
SEEPAGE CUT OFF WALL
NEW BRIDGE WITH 7,000 CFS MIN. WATERWAY OPENING
BAFFLE
℄ DISCHARGE STRUCTURE
TOP OF DIKE EL

PLATE 2

BY RH DATE 2-29-60 FILE 5697-004 REVISIONS BY ____ DATE ____
CHECKED BY ____



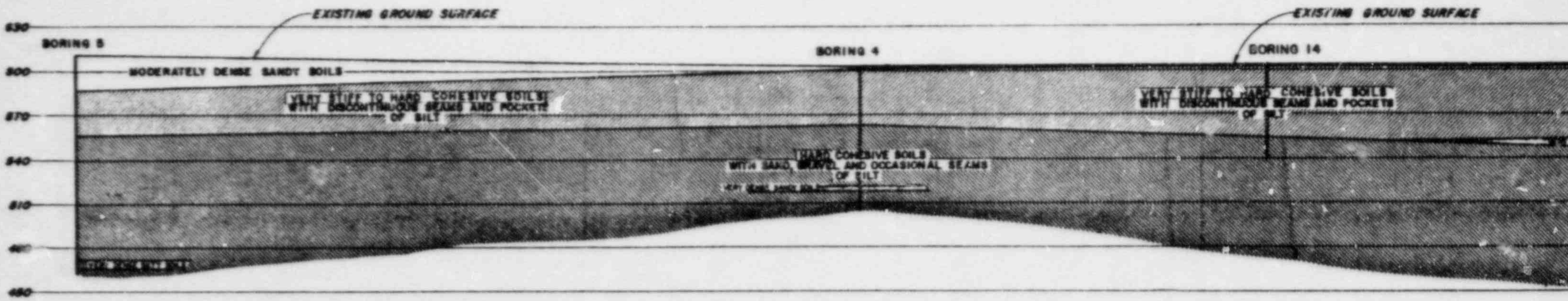
PLOT PLAN

SCALE: 1" = 80'

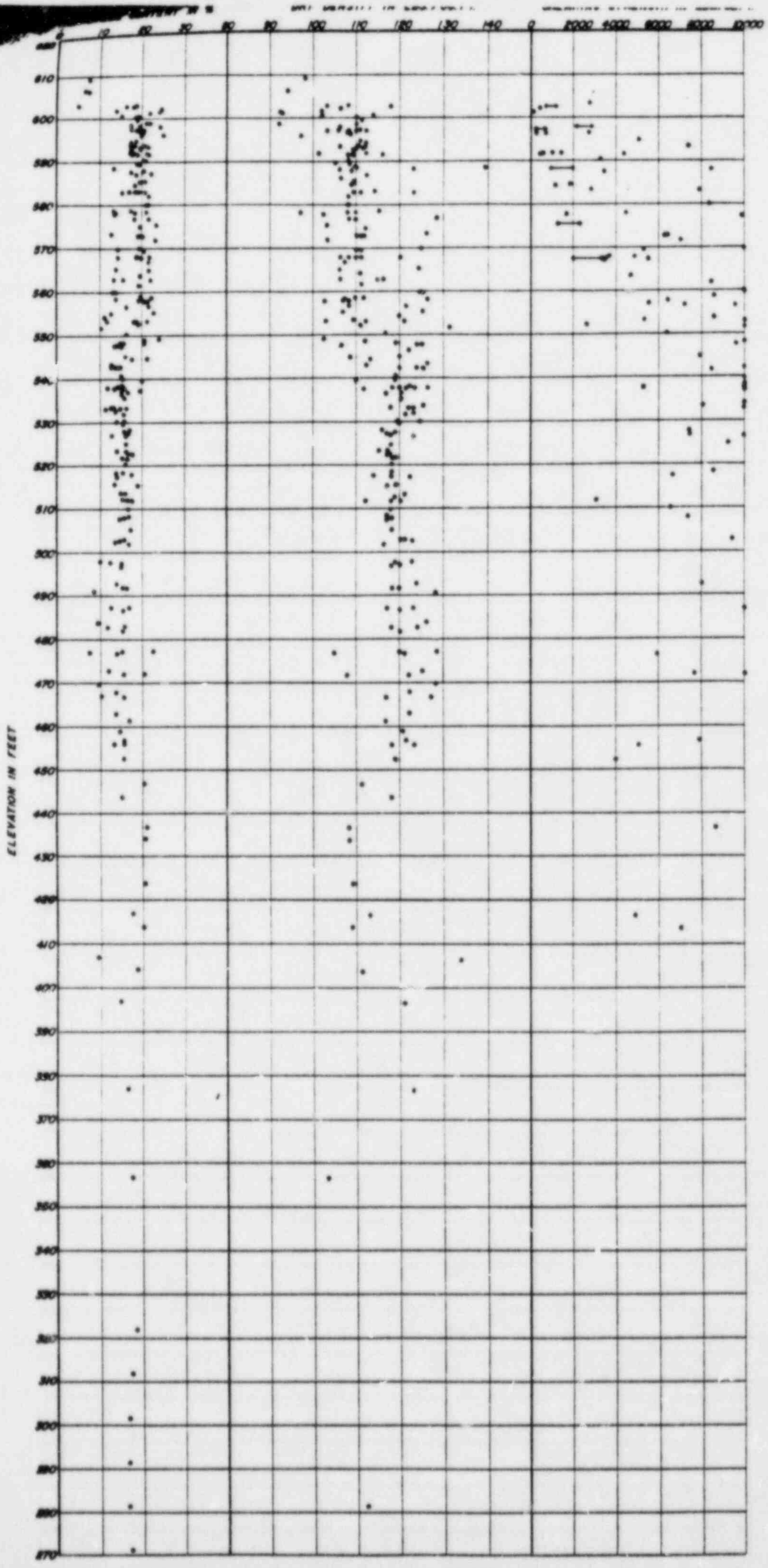
DRAWING REFERENCE:
BECHTEL CORPORATION
JOB NO. 7220
DRAWING NO. SK-C-99
REVISION A

DAMES & MOORE

PLATE 3



GENERALIZED SUBSURFACE SECTION B-B



ELEVATION IN FEET
0
10
20
30
40
50
100
110
120
130
140
0
2000
4000
6000
8000
10000
610
600
590
580
570
560
550
540
530
520
510
500
490
480
470
460
450
440
430
420
410
400
390
380
370
360
350
340
330
320
310
300
290
280
270

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTS

FIELD EXPLORATIONS

Power Plant Area - The subsurface conditions at the site of the Proposed Nuclear Power Plant were further investigated by drilling 11 additional four-inch diameter exploration test borings to depths ranging from approximately 40 feet to 80 feet below the existing ground surface utilizing truck-mounted rotary wash and rotary auger type drilling equipment. Exploration test borings 730 and 731 were drilled as part of a previous investigation. In addition to the exploration test borings, 24 probe holes were drilled to depths ranging from approximately 10 feet to 45 feet below the existing ground surface utilizing truck mounted rotary auger type drilling equipment.

The drilling operations were supervised by our field engineers who maintained logs of the borings, obtained undisturbed samples of the various soil strata penetrated utilizing Dames & Moore Soil Samplers and supervised the performance of Standard Penetration Tests. Graphical representations of the soils penetrated by the borings and probe holes are shown on Plates A-1D through A-1U, Log of Borings. The method utilized in classifying the soils is defined on Plate A-2, Unified Soil Classification System.

Undisturbed samples of the soils penetrated by the exploration test borings were obtained in Dames & Moore Soil Samplers of the type illustrated on Plate A-3, Soil Sampler Type U. The Dames & Moore soil samplers were driven approximately 18 inches into the soil with a hammer weighing approximately 340 pounds falling approximately 24 inches. The

Standard Penetration Tests were performed utilizing a split spoon sampler having an outside diameter of two inches and an inside diameter of one and three-eighths of an inch. The split spoon sampler was driven 18 inches into the ground with a hammer weighing 140 pounds falling 30 inches. The number of blows required to drive the Dames & Moore soil samplers and the split spoon sampler for the second and third six inches of penetration are recorded on the Log of Borings.

The boring locations and the elevations of the ground surface were provided to us by a survey crew from the firm of Hunter, Whittier and Solberg located in Midland, Michigan. The ground surface elevation is shown above the log of each boring. These elevations refer to the U.S.G.S. Datum.

LABORATORY TESTS:

Strength Tests - Direct shear, unconfined compression and triaxial compression tests were performed on selected undisturbed samples to evaluate the strength characteristics of the various soils penetrated by the borings.

The direct shear tests were performed in the manner described on Plate A-4, Method of Performing Direct Shear and Friction Tests. Unconfined compression and triaxial compression tests were performed in the manner described on Plate A-5, Methods of Performing Unconfined and Triaxial Compression Tests. Stress-strain curves were plotted for each static strength test. For the direct shear tests, the shear strength is yield point strength or the strength at a deflection of one-tenth of an inch whichever occurs first. For the unconfined compression and triaxial compression tests, shearing strengths were chosen assuming that the angle of internal friction of the cohesive soils was equal to zero. The shear strengths presented are either peak strengths or the strengths at an axial deflection of ten percent of the sample height, whichever occurred first. Determination of the moisture content and dry density were made in conjunction with each strength test. The results of the strength tests, together with the associated moisture-density determinations are presented to the left of the Log of Borings in the manner described by the Key to Test Data shown on Plate A-2.

Consolidation Tests - Consolidation tests were performed on representative undisturbed samples and a remolded sample of the soils penetrated by the borings to provide additional data for estimating settlements of fill and foundations. The results of the consolidation tests are presented on Plates A-7F through A-7H, Consolidation Test Data.

Moisture-Density Tests - Moisture-density tests were performed in conjunction with each strength and consolidation test. Additional moisture and/or density tests were performed on selected samples for correlation purposes. The results of the moisture and/or density tests are presented to the left of the Log of Borings in the manner described by the Key to Test Data shown on Plate A-2.

Grain Size Distribution - A determination of the grain size distribution of selected samples of sandy soils extracted from borings was made to facilitate classification of these soils. The results of the mechanical analyses performed to determine the grain size distribution are presented on Plate A-11 Grain Size Analyses.

DYNAMIC MODULUS OF ELASTICITY:

A revised derivation of appropriate values of dynamic modulus of elasticity (E) for the very stiff to hard clay soils underlying the site is as follows:

EARTHQUAKE ACCELERATION AT SURFACE	E @ 50 FEET DEPTH LBS./SQ.FT.
0.05g	30×10^6
0.10g	22×10^6
0.20g	17×10^6

Poisson's Ratio may be assumed equal to 0.4. The above modulus of elasticity values are approximate and it is recommended that they be varied by plus or minus 50 percent in analyses to evaluate their influence. It is anticipated that soil damping will be in the range of five to ten percent.

The above values are derived from the data of Idriss and Seed published in the December 1968 issue of the Bulletin of the Seismological Society of America.

--oOo--

The following Plates are attached and complete this Appendix:

Plate A-1D - Log of Borings (Boring 7)

Plate A-1E - Log of Borings (Boring 8)

Plate A-1F - Log of Borings (Boring 9)

Plate A-1G - Log of Borings (Boring 10)

Plate A-1H - Log of Borings (Boring 11)

Plate A-1I - Log of Borings (Boring 12)

Plate A-1J - Log of Borings (Boring 13)

Plate A-1K - Log of Borings (Boring 14)

Plate A-1L - Log of Borings (Boring 15)

Plate A-1M - Log of Borings (Boring 16)

Plate A-1N - Log of Borings (Boring 17)

Plate A-1O - Log of Borings (Boring 18)

Plate A-1P - Log of Borings (Boring 730)

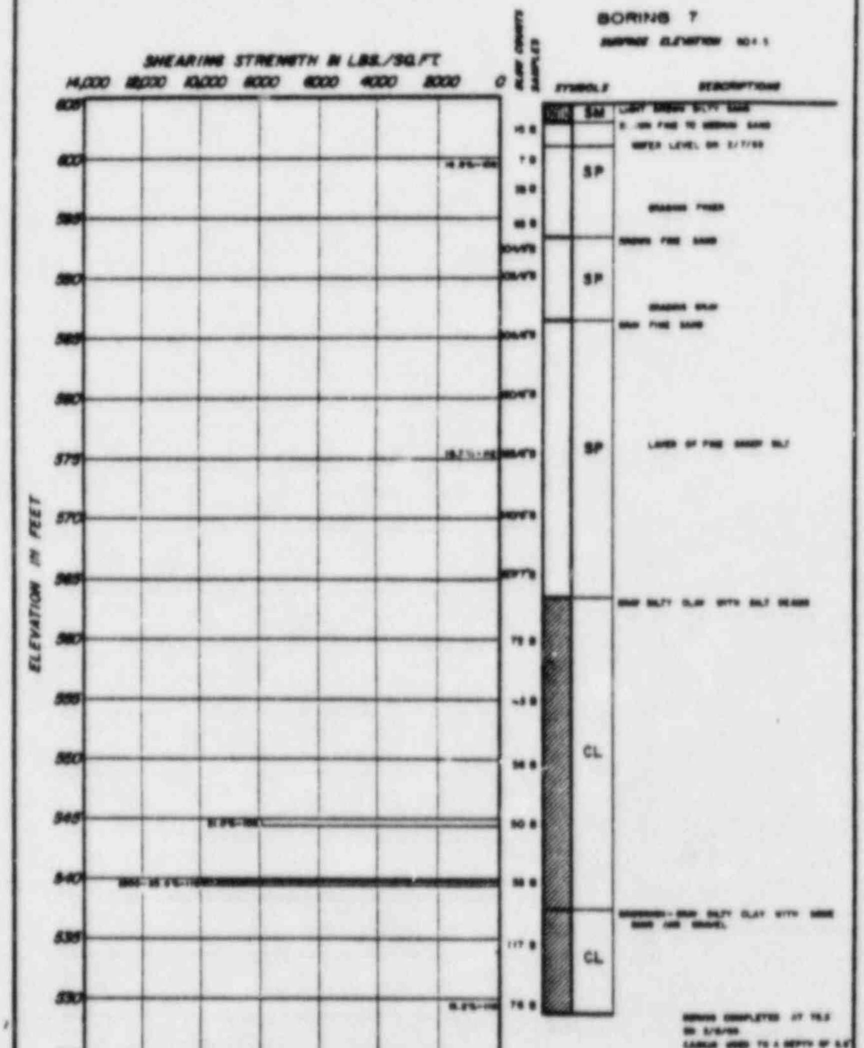
Plate A-1Q - Log of Borings (Boring 731)

Plate A-1R - Log of Probe Borings (Probe Borings P1, P2, P3, P4, P5, P6)

Plate A-1S - Log of Probe Borings (Probe Borings P7, P8, P9, P10, P11)

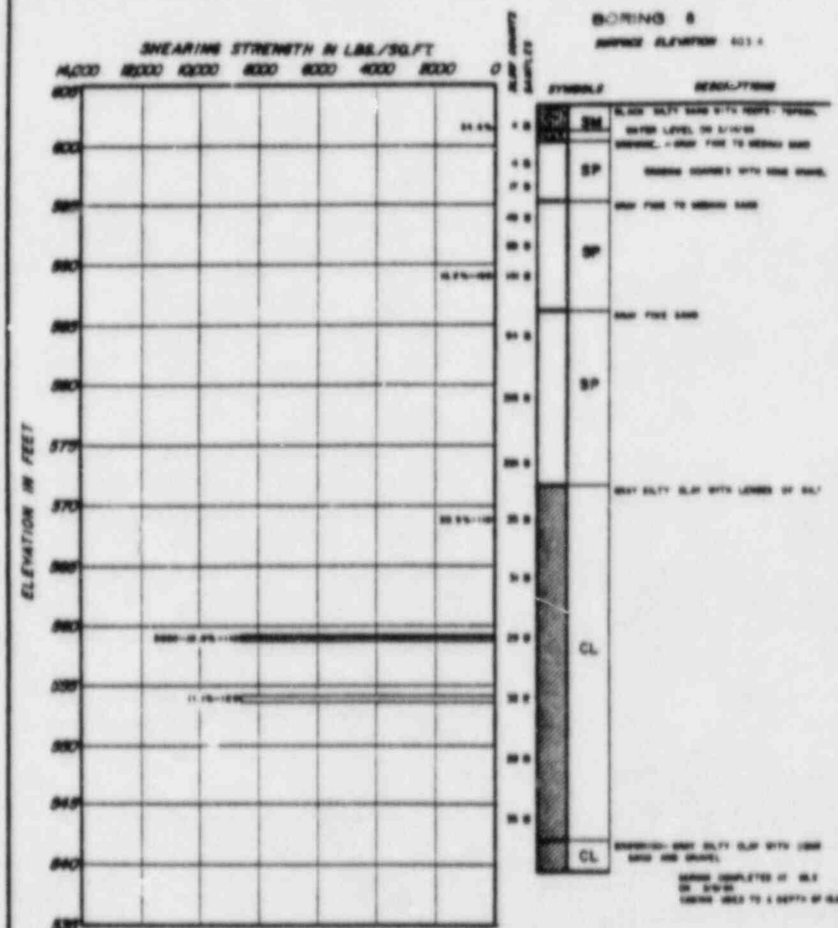
Plate A-1T - Log of Probe Borings (Probe Borings P12, P13, P14, P15, P16, P17)

Plate A-1U - Log of Probe Borings (Probe Borings P18, P19, P20, P21, P22)



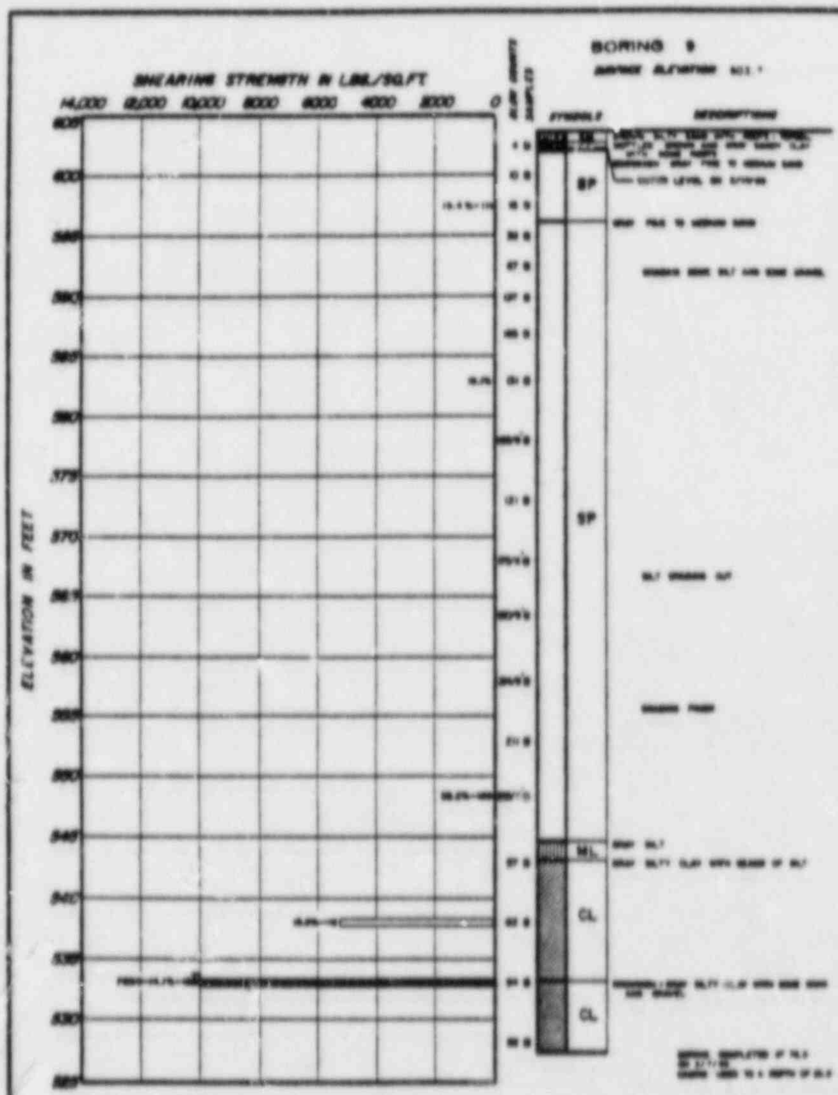
LOG OF BORINGS

PLATE A-10



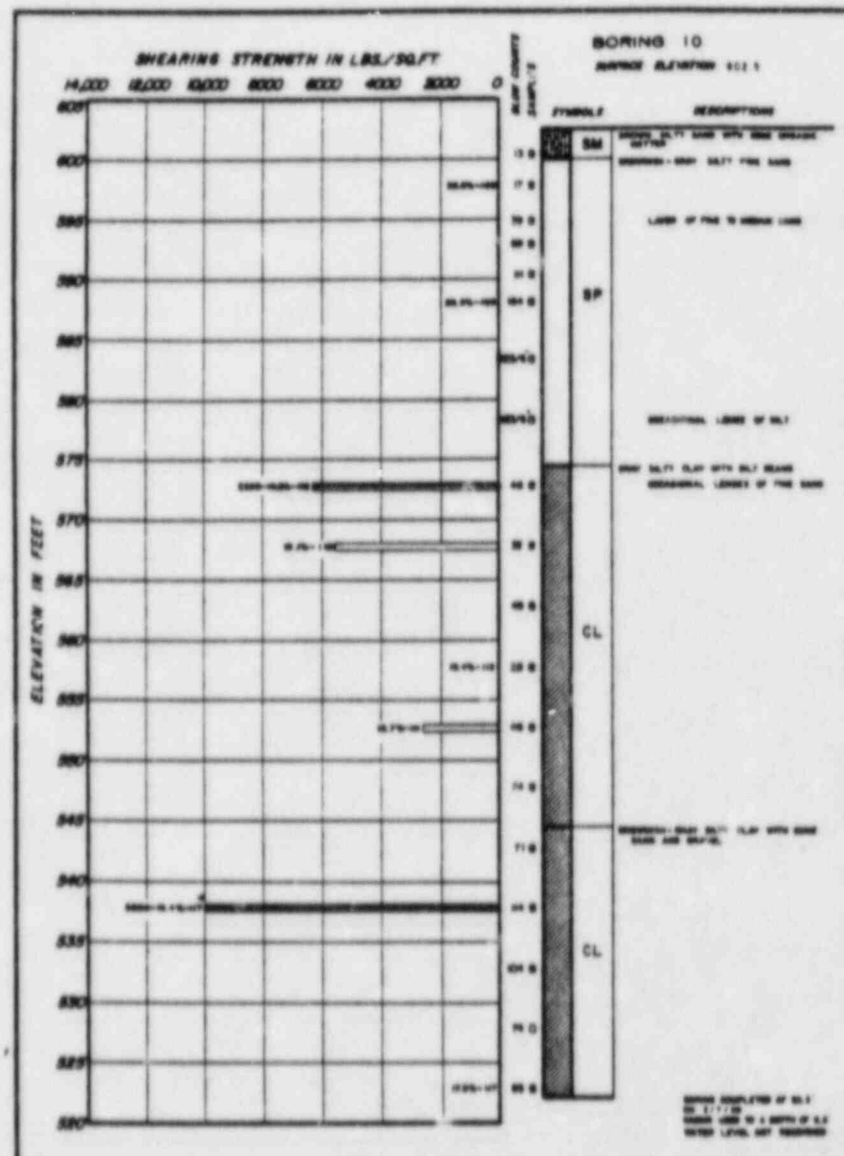
LOG OF BORINGS

PLATE A-12



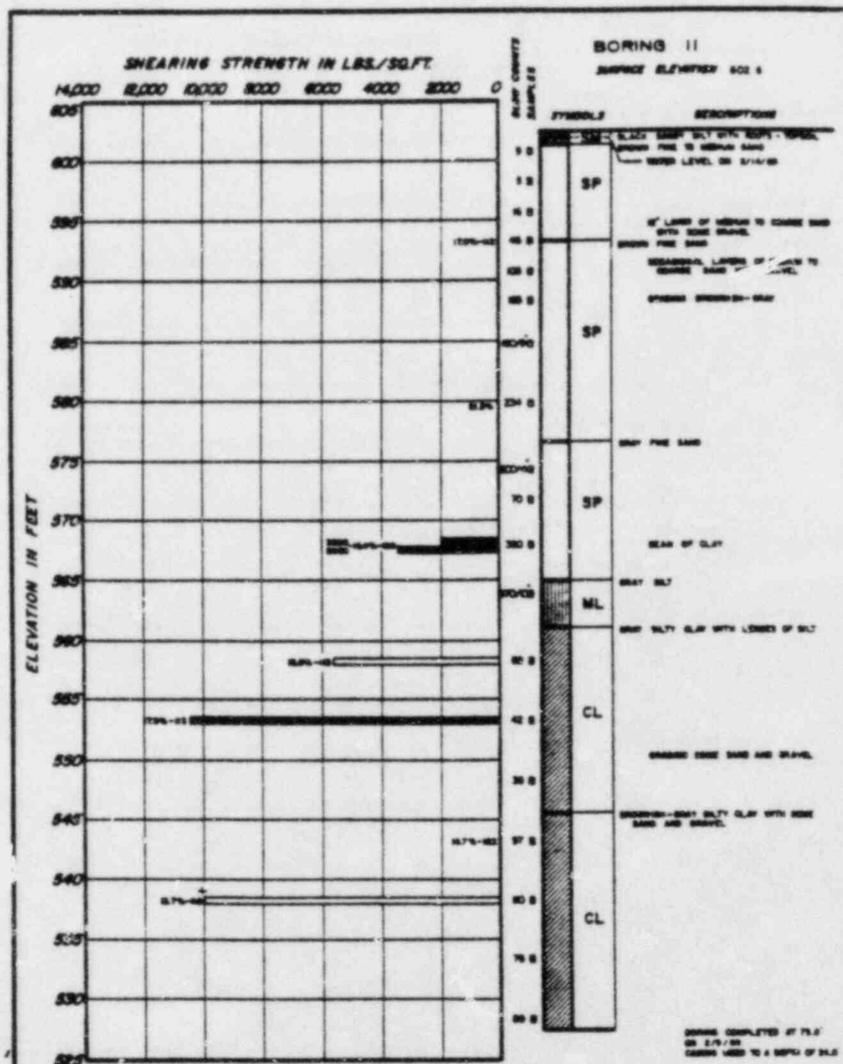
LOG OF BORINGS

PLATE A-17



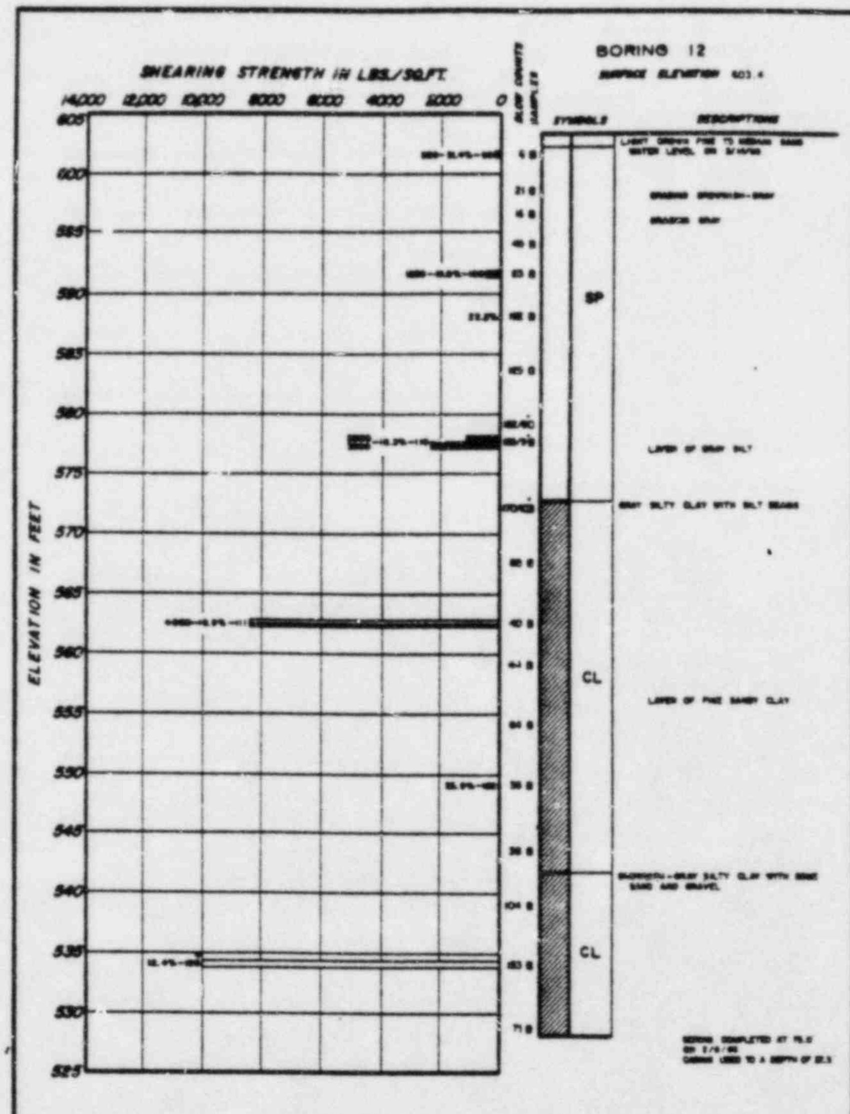
LOG OF BORINGS

PLATE A-16



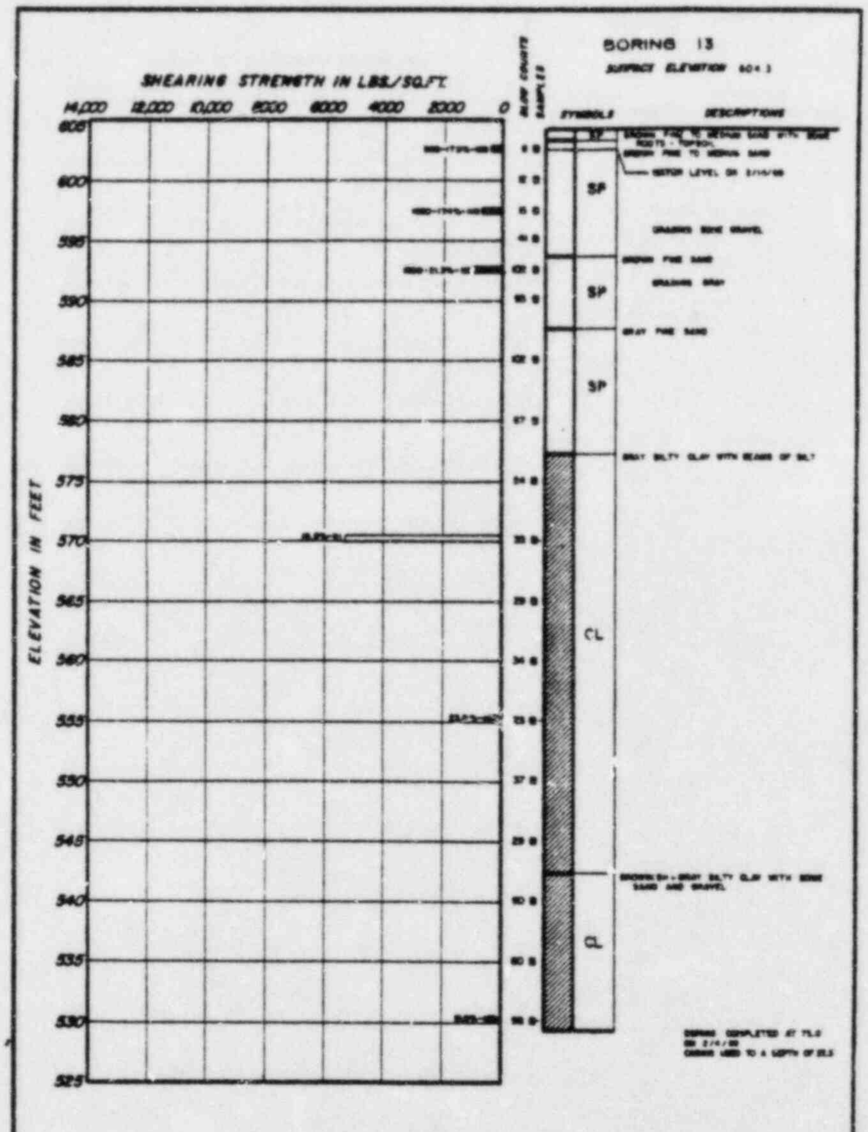
LOG OF BORINGS

PLATE A-11



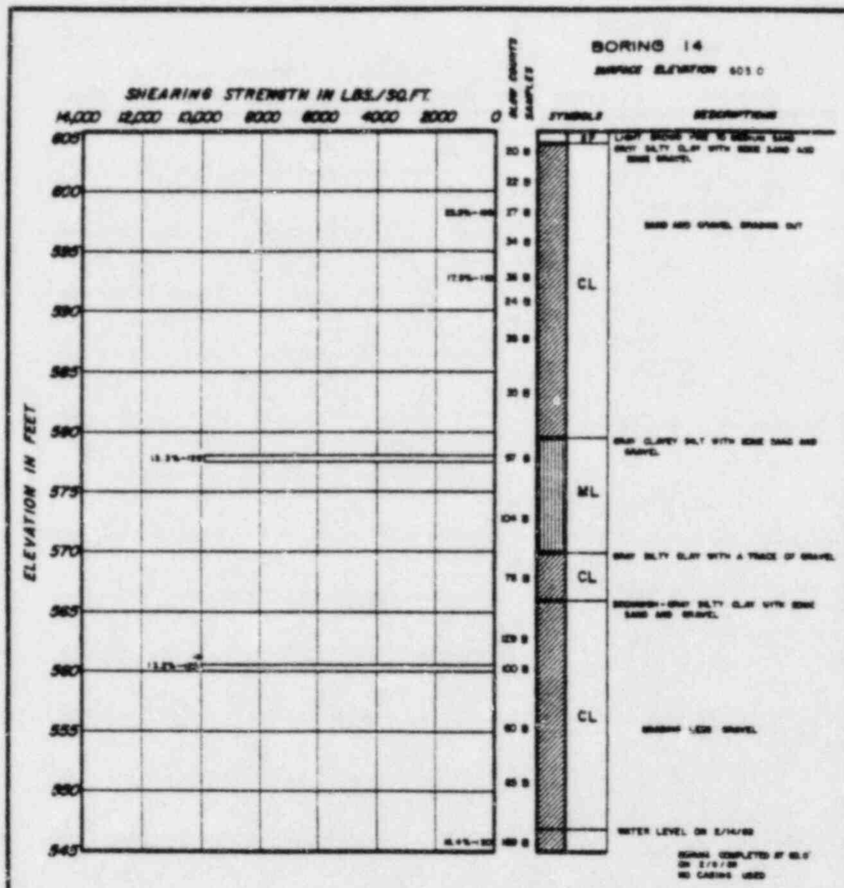
LOG OF BORINGS

PLATE A-12



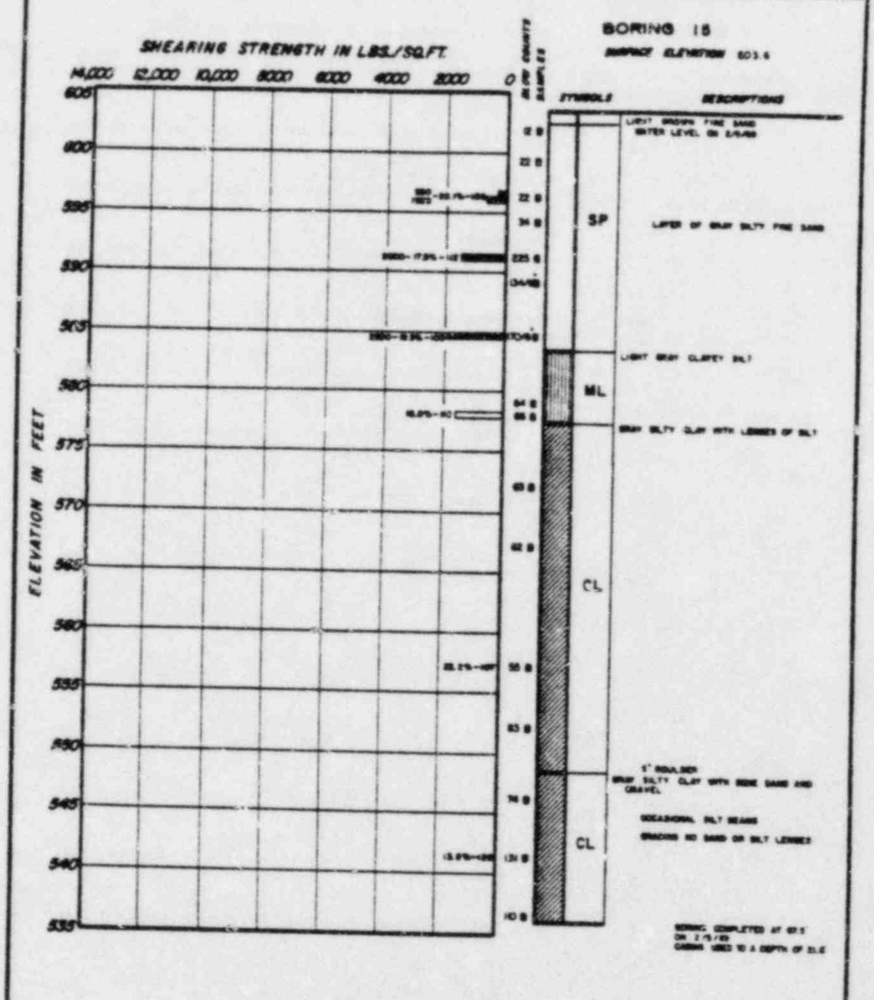
LOG OF BORINGS

PLATE A-13



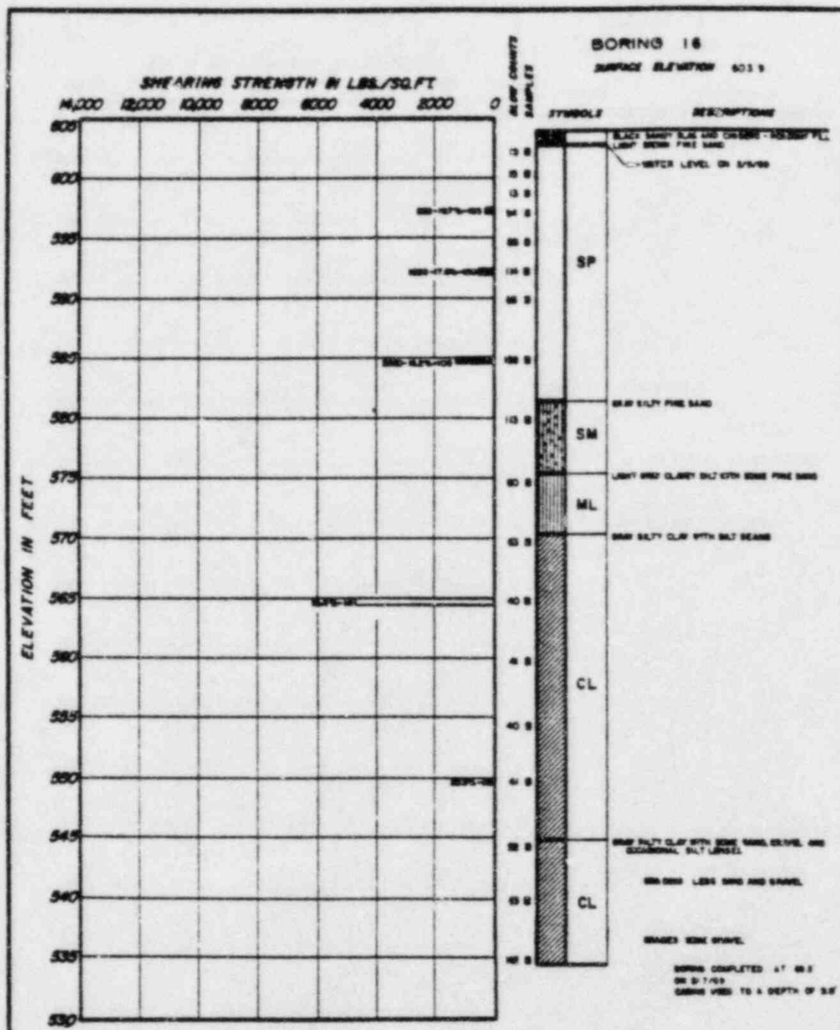
LOG OF BORINGS

PLATE A-1K



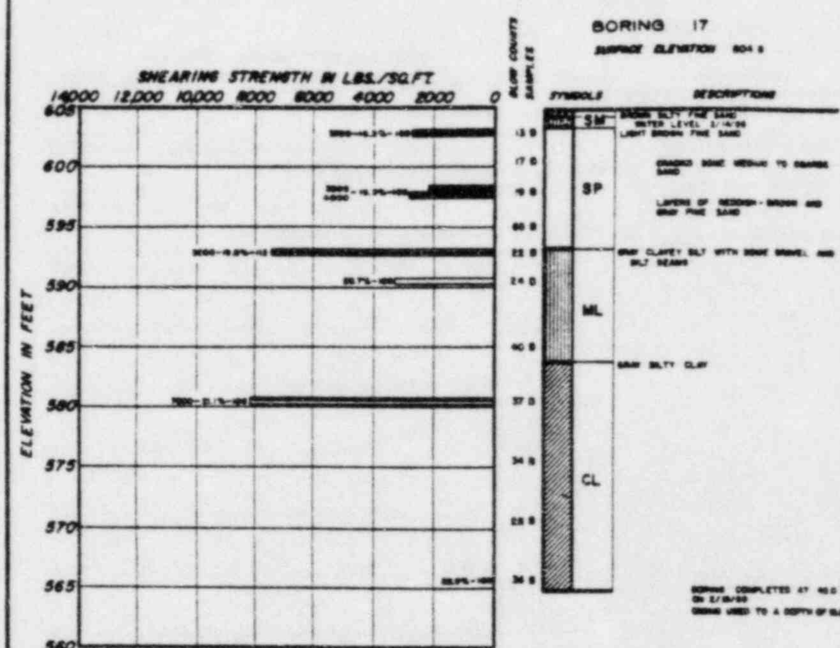
LOG OF BORINGS

PLATE A-1 L



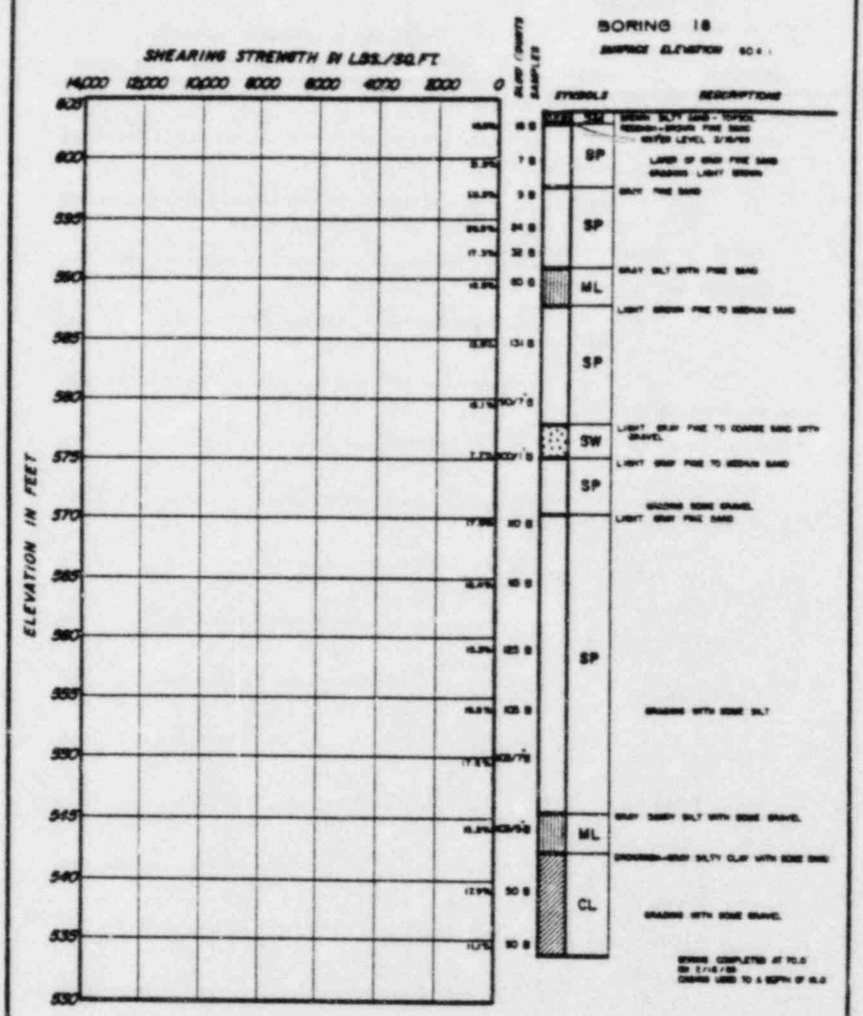
LOG OF BORINGS

PLATE A-1 M



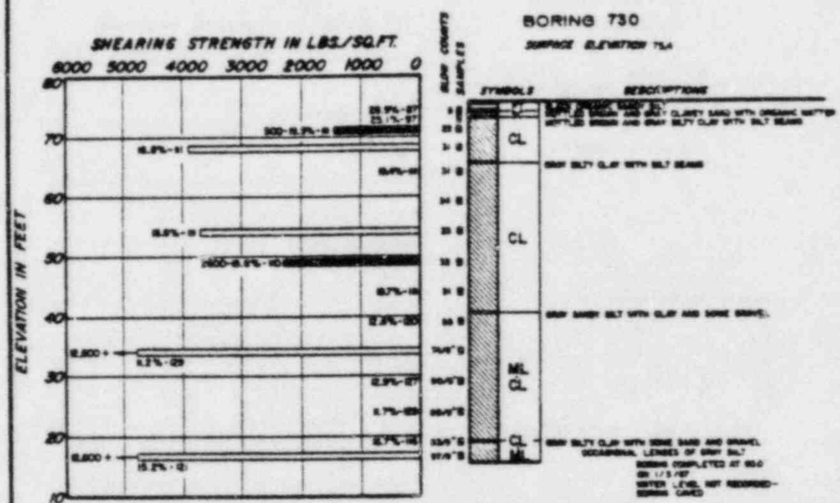
LOG OF BORINGS

PLATE A-1N



LOG OF BORINGS

PLATE A-10

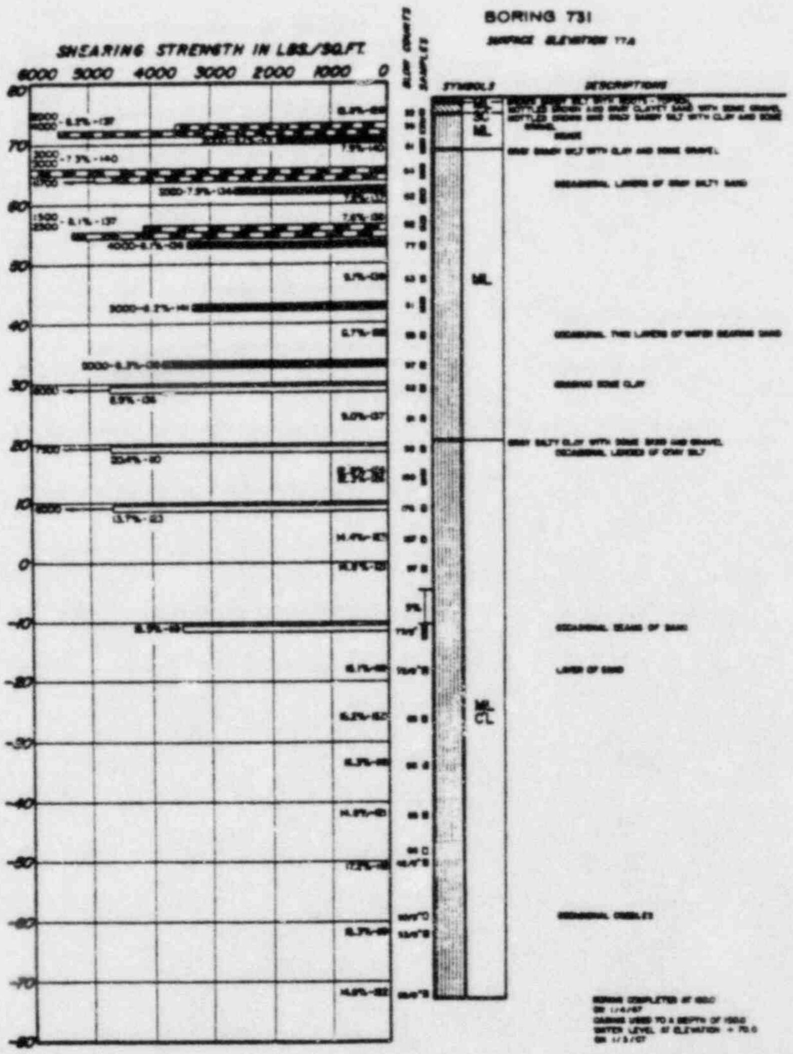


LOG OF BORINGS

NOTE
ELEVATIONS REFER TO PLANT DATUM

DAMES & MOORE

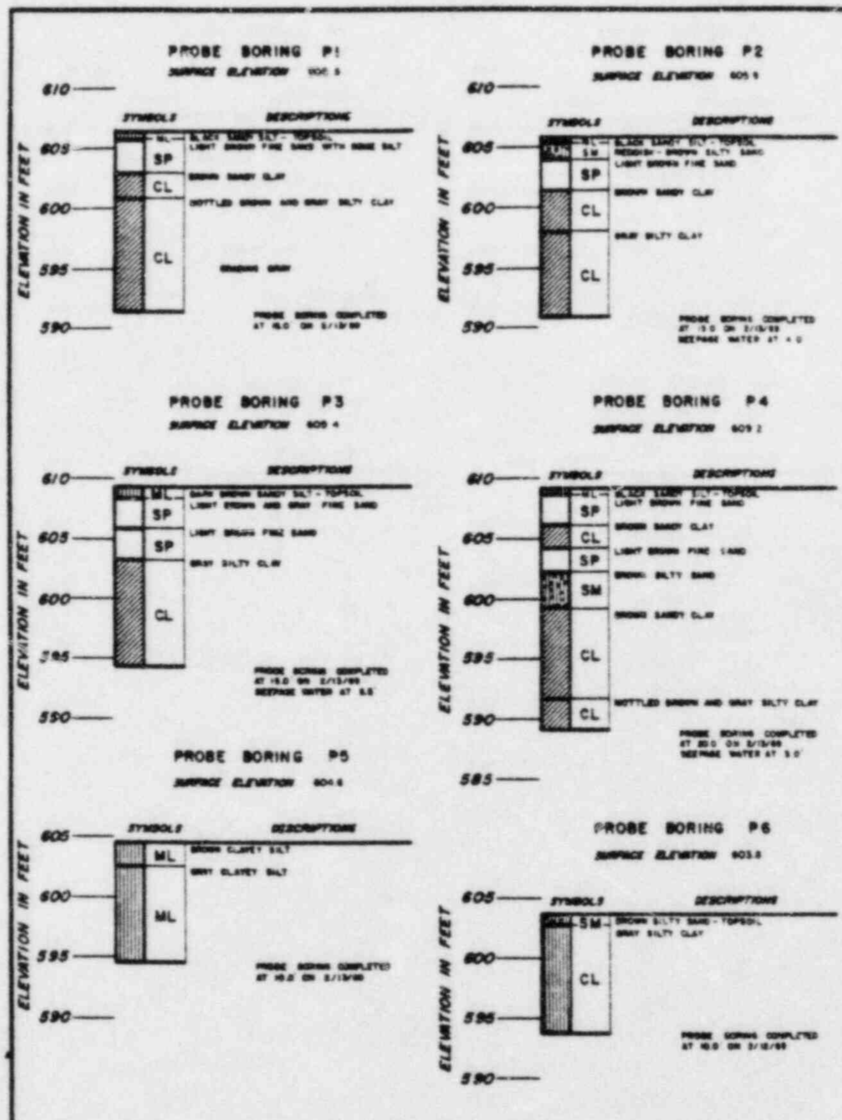
PLATE A-1P



LOG OF BORINGS

NOTE:
ELEVATIONS REFER TO PLANT DATUM

PLATE A-10



LOG OF PROBE BORINGS

DAMES & MOORE

PLATE A-1R

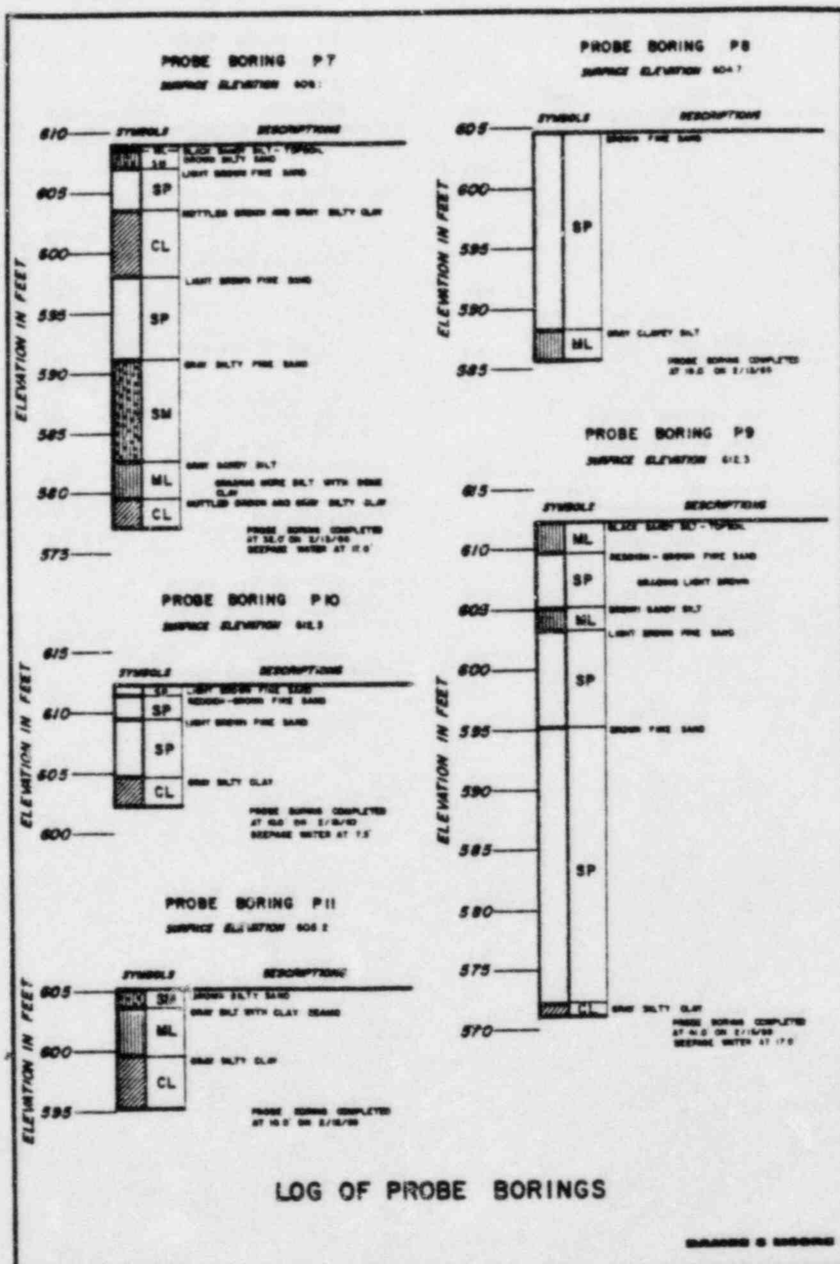

PROBE BORING P7
PROBE BORING P8
PROBE BORING P9
PROBE BORING P10
PROBE BORING P11
ELEVATION IN FEET
SP
CL
SM
ML
LOG OF PROBE BORINGS

PLATE A-18

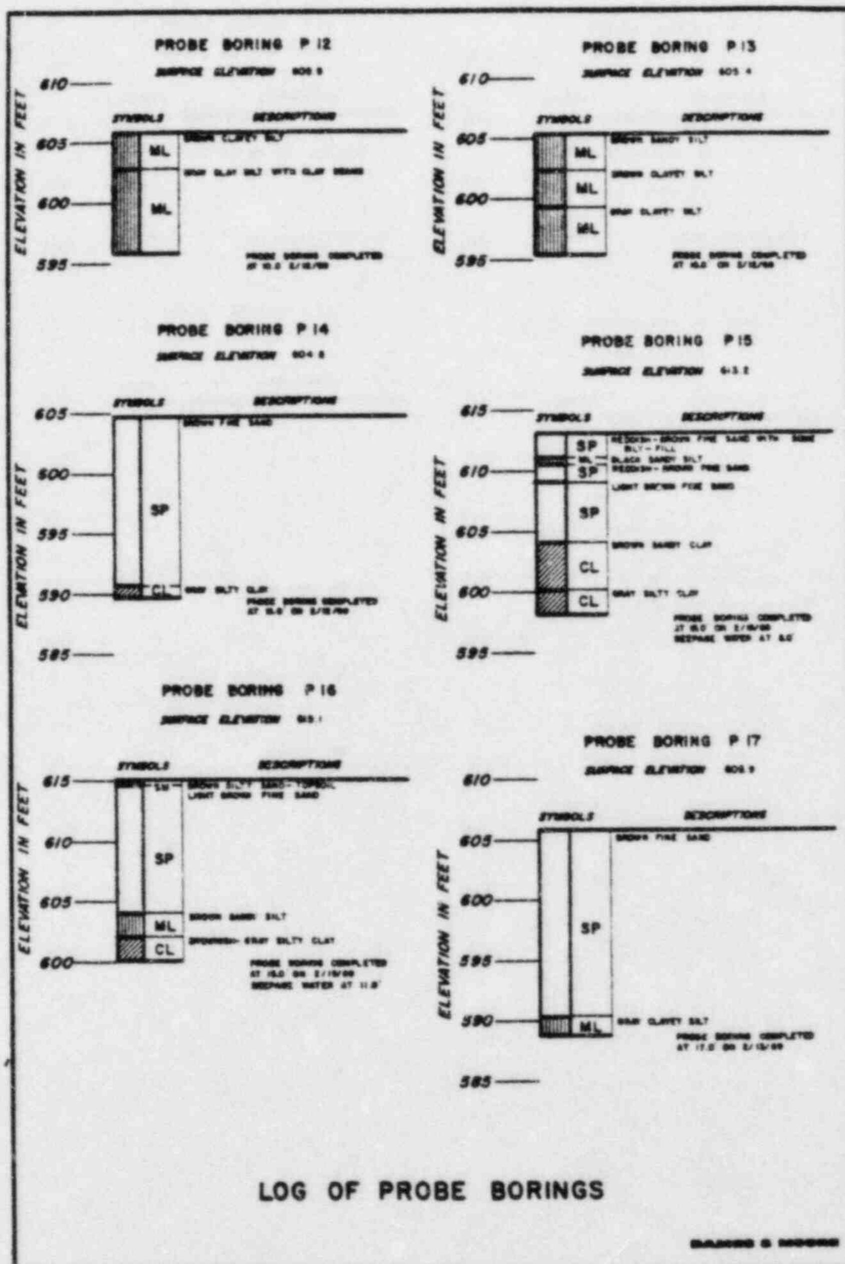

PROBE BORING P 12
PROBE BORING P 13
PROBE BORING P 14
PROBE BORING P 15
PROBE BORING P 16
PROBE BORING P 17
SYMBOLS
DESCRIPTIONS
ELEVATION IN FEET
ML
SP
CL
LOG OF PROBE BORINGS

PLATE A-17

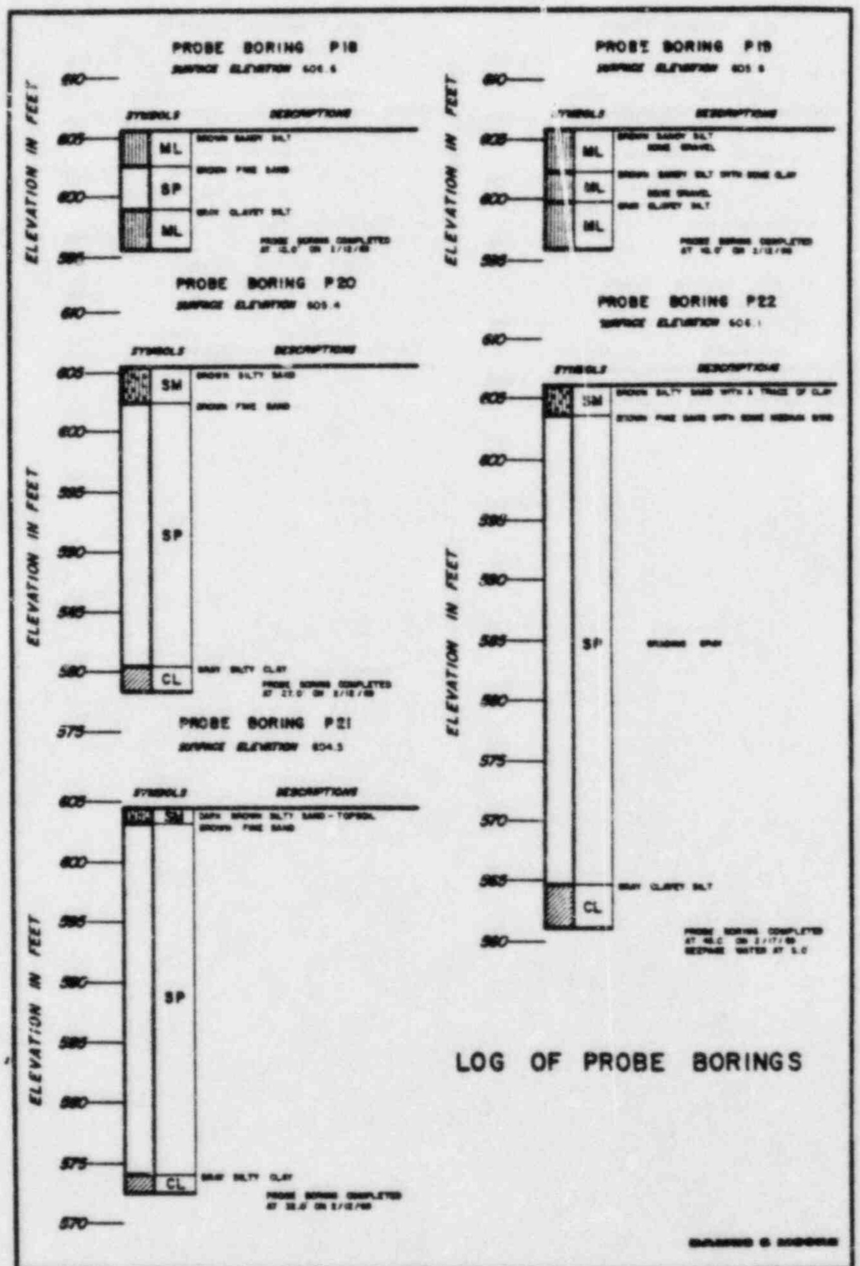

PROBE BORING P18
SURFACE ELEVATION 605.5
ELEVATION IN FEET
610
605
600
SYMBOLS
DESCRIPTIONS
ML
SP
ML
BROWN SANDY SILT
BROWN FINE SAND
GRAY CLAYEY SILT
PROBE BORING COMPLETED
PROBE BORING P19
SURFACE ELEVATION 605.5
ML
ML
ML
BROWN SANDY SILT
SOME GRAVEL
BROWN SANDY SILT WITH SOME CLAY
SOME GRAVEL
GRAY CLAYEY SILT
PROBE BORING COMPLETED
PROBE BORING P20
SURFACE ELEVATION 605.4
605
600
595
590
585
580
575
SM
SP
CL
BROWN SILTY SAND
BROWN FINE SAND
GRAY SILTY CLAY
PROBE BORING COMPLETED
PROBE BORING P22
SURFACE ELEVATION 606.1
610
605
600
595
590
585
580
575
570
565
560
SM
SP
CL
BROWN SILTY SAND WITH A TRACE OF CLAY
BROWN FINE SAND WITH SOME MEDIUM SAND
GRAY CLAYEY SILT
PROBE BORING COMPLETED
SEEPAGE WATER AT 5.0
PROBE BORING P21
SURFACE ELEVATION 604.3
605
600
595
590
585
580
575
570
SM
SP
CL
DARK BROWN SILTY SAND - TOPSOIL
BROWN FINE SAND
GRAY SILTY CLAY
PROBE BORING COMPLETED
LOG OF PROBE BORINGS
DAMES & MOORE

PLATE A-10

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

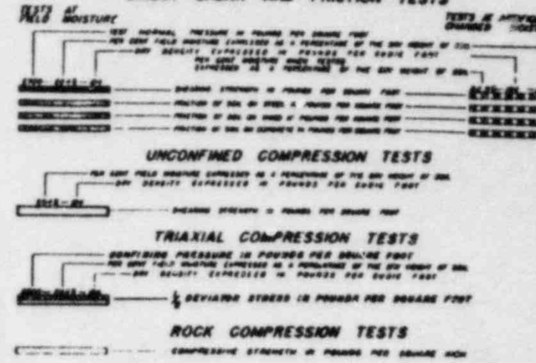
SOIL CLASSIFICATION CHART

DIRECT SHEAR AND FRICTION TESTS

UNCONFINED COMPRESSION TESTS

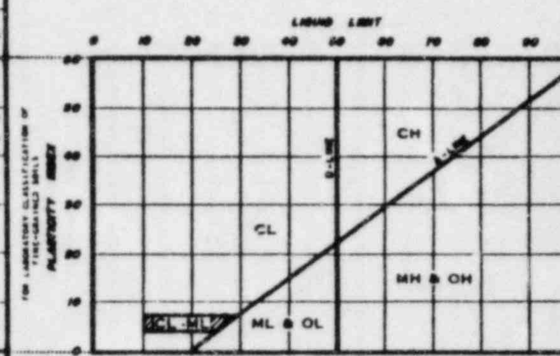
TRIAXIAL COMPRESSION TESTS

ROCK COMPRESSION TESTS



KEY TO TEST DATA

KEY TO SAMPLES



PLASTICITY CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

DAMES & MOORE

PLATE A-2

BY ______ DATE ______
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COPY TO GO ______ FILE ______

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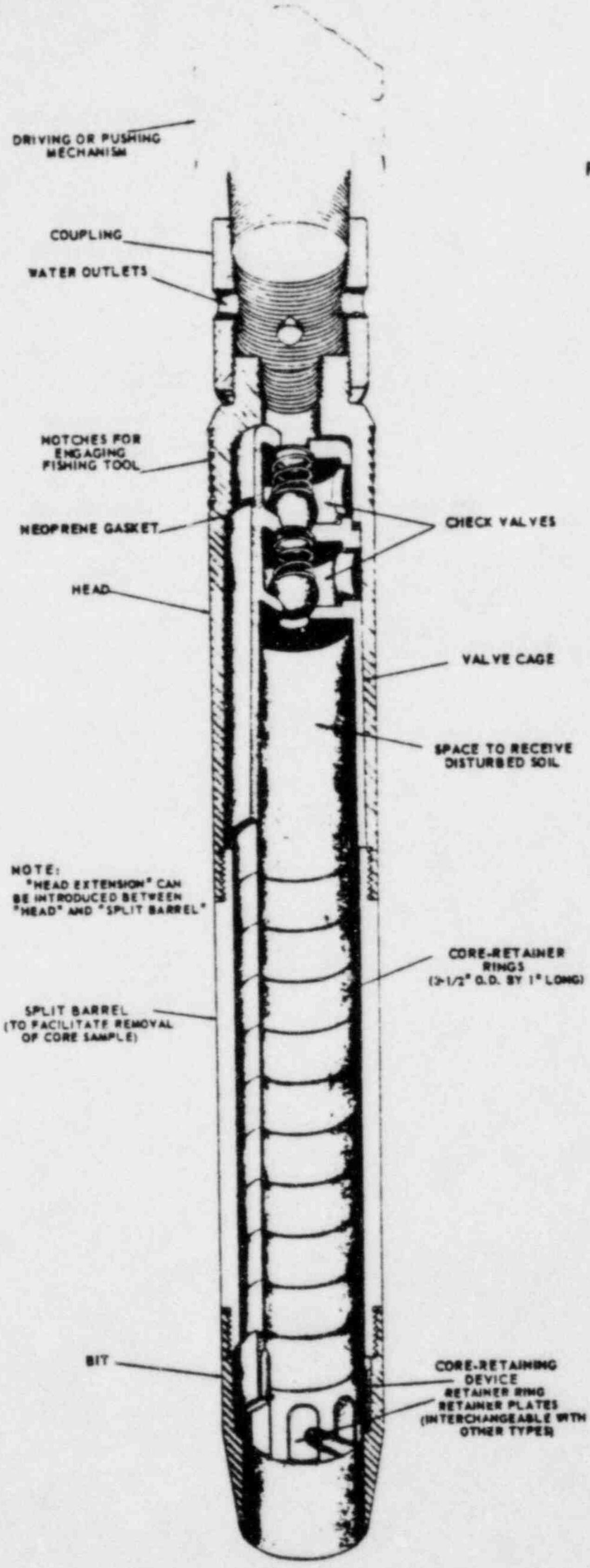
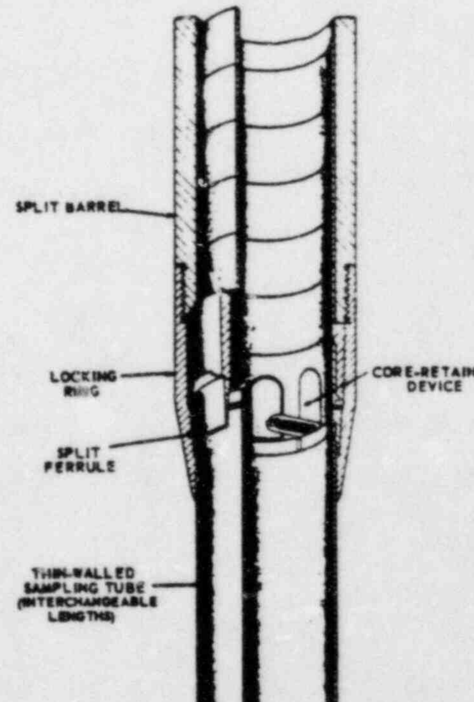
SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER
U. S. PATENT NO. 2,318,062



ALTERNATE ATTACHMENTS



DAMES & MOORE
APPLIED EARTH SCIENCES

METHOD OF PERFORMING DIRECT SHEAR AND FRICTION TESTS

DIRECT SHEAR TESTS ARE PERFORMED TO DETERMINE THE SHEARING STRENGTHS OF SOILS. FRICTION TESTS ARE PERFORMED TO DETERMINE THE FRICTIONAL RESISTANCES BETWEEN SOILS AND VARIOUS OTHER MATERIALS SUCH AS WOOD, STEEL, OR CONCRETE. THE TESTS ARE PERFORMED IN THE LABORATORY TO SIMULATE ANTICIPATED FIELD CONDITIONS.

DIRECT SHEAR TESTING & RECORDING APPARATUS

EACH SAMPLE IS TESTED WITHIN THREE BRASS RINGS, TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.

DIRECT SHEAR TESTS

A THREE-INCH LENGTH OF THE SAMPLE IS TESTED IN DIRECT DOUBLE SHEAR. A CONSTANT PRESSURE, APPROPRIATE TO THE CONDITIONS OF THE PROBLEM FOR WHICH THE TEST IS BEING PERFORMED, IS APPLIED NORMAL TO THE ENDS OF THE SAMPLE THROUGH POROUS STONES. A SHEARING FAILURE OF THE SAMPLE IS CAUSED BY MOVING THE CENTER RING IN A DIRECTION PERPENDICULAR TO THE AXIS OF THE SAMPLE. TRANSVERSE MOVEMENT OF THE OUTER RINGS IS PREVENTED.

THE SHEARING FAILURE MAY BE ACCOMPLISHED BY APPLYING TO THE CENTER RING EITHER A CONSTANT RATE OF LOAD, A CONSTANT RATE OF DEFLECTION, OR INCREMENTS OF LOAD OR DEFLECTION. IN EACH CASE, THE SHEARING LOAD AND THE DEFLECTIONS IN BOTH THE AXIAL AND TRANSVERSE DIRECTIONS ARE RECORDED AND PLOTTED. THE SHEARING STRENGTH OF THE SOIL IS DETERMINED FROM THE RESULTINC LOAD-DEFLECTION CURVES.

FRICTION TESTS

IN ORDER TO DETERMINE THE FRICTIONAL RESISTANCE BETWEEN SOIL AND THE SURFACES OF VARIOUS MATERIALS, THE CENTER RING OF SOIL IN THE DIRECT SHEAR TEST IS REPLACED BY A DISK OF THE MATERIAL TO BE TESTED. THE TEST IS THEN PERFORMED IN THE SAME MANNER AS THE DIRECT SHEAR TEST BY FORCING THE DISK OF MATERIAL FROM THE SOIL SURFACES.

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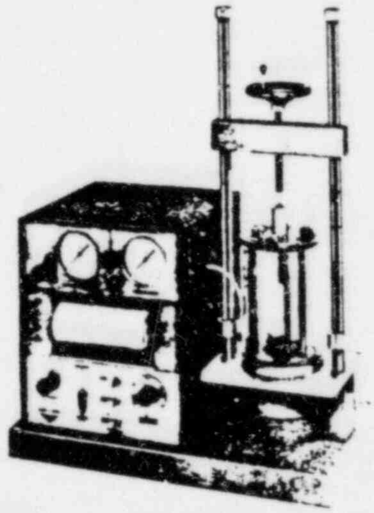
Methods of Performing Unconfined Compression and Triaxial Compression Tests



TRIAXIAL COMPRESSION TEST UNIT

The shearing strengths of soils are determined from the results of unconfined compression and triaxial compression tests. In triaxial compression tests the test method and the magnitude of the confining pressure are chosen to simulate anticipated field conditions.

Unconfined compression and triaxial compression tests are performed on undisturbed or remolded samples of soil approximately six inches in length and two and one-half inches in diameter. The tests are run either strain-controlled or stress-controlled. In a strain-controlled test the sample is subjected to a constant rate of deflection and the resulting stresses are recorded. In a stress-controlled test the sample is subjected to equal increments of load with each increment being maintained until an equilibrium condition with respect to strain is achieved.

Yield, peak, or ultimate stresses are determined from the stress-strain plot for each sample and the principal stresses are evaluated. The principal stresses are plotted on a Mohr's circle diagram to determine the shearing strength of the soil type being tested.

Unconfined compression tests can be performed only on samples with sufficient cohesion so that the soil will stand as an unsupported cylinder. These tests may be run at natural moisture content or on artificially saturated soils.

In a triaxial compression test the sample is encased in a rubber membrane, placed in a test chamber, and subjected to a confining pressure throughout the duration of the test. Normally, this confining pressure is maintained at a constant level, although for special tests it may be varied in relation to the measured stresses. Triaxial compression tests may be run on soils at field moisture content or on artificially saturated samples. The tests are performed in one of the following ways:

Unconsolidated-Undrained: The confining pressure is imposed on the sample at the start of the test. No drainage is permitted and the stresses which are measured represent the sum of the intergranular stresses and pore water pressures.

Consolidated-Undrained: The sample is allowed to consolidate fully under the applied confining pressure prior to the start of the test. The volume change is determined by measuring the water and/or air expelled during consolidation. No drainage is permitted during the test and the stresses which are measured are the same as for the unconsolidated-undrained test.

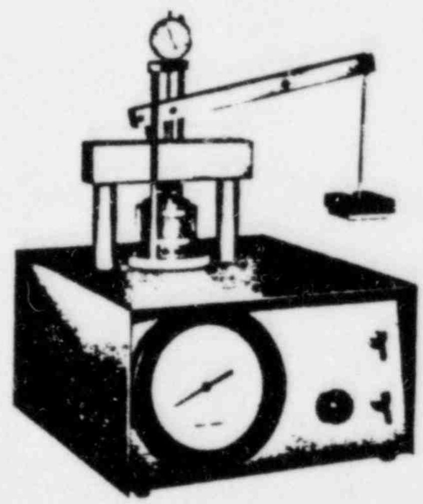
Drained: The intergranular stresses in a sample may be measured by performing a drained, or slow, test. In this test the sample is fully saturated and consolidated prior to the start of the test. During the test, drainage is permitted and the test is performed at a slow enough rate to prevent the buildup of pore water pressures. The resulting stresses which are measured represent only the intergranular stresses. These tests are usually performed on samples of generally non-cohesive soils, although the test procedure is applicable to cohesive soils if a sufficiently slow test rate is used.

An alternate means of obtaining the data resulting from the drained test is to perform an undrained test in which special equipment is used to measure the pore water pressures. The differences between the total stresses and the pore water pressures measured are the intergranular stresses.

Method Of Performing Consolidation Tests

Consolidation tests are performed to evaluate the volume changes of soils subjected to increased loads. Time-consolidation and pressure-consolidation curves may be plotted from the data obtained in the tests. Engineering analyses based on these curves permit estimates to be made of the probable magnitude and rate of settlement of the tested soils under applied loads.

Each sample is tested within brass rings two and one-half inches in diameter and one inch in length. Undisturbed samples of in-place soils are tested in rings taken from the sampling device in which the samples were obtained. Loose samples of soils to be used in constructing earth fills are compacted in rings to predetermined conditions and tested.



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

In testing, the sample is rigidly confined laterally by the brass ring. Axial loads are transmitted to the ends of the sample by porous disks. The disks allow drainage of the loaded sample. The axial compression or expansion of the sample is measured by a micrometer dial indicator at appropriate time intervals after each load increment is applied. Each load is ordinarily twice the preceding load. The increments are selected to obtain consolidation data representing the field loading conditions for which the test is being performed. Each load increment is allowed to act over an interval of time dependent on the type and extent of the soil in the field.

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BY RH DATE 8-29-69
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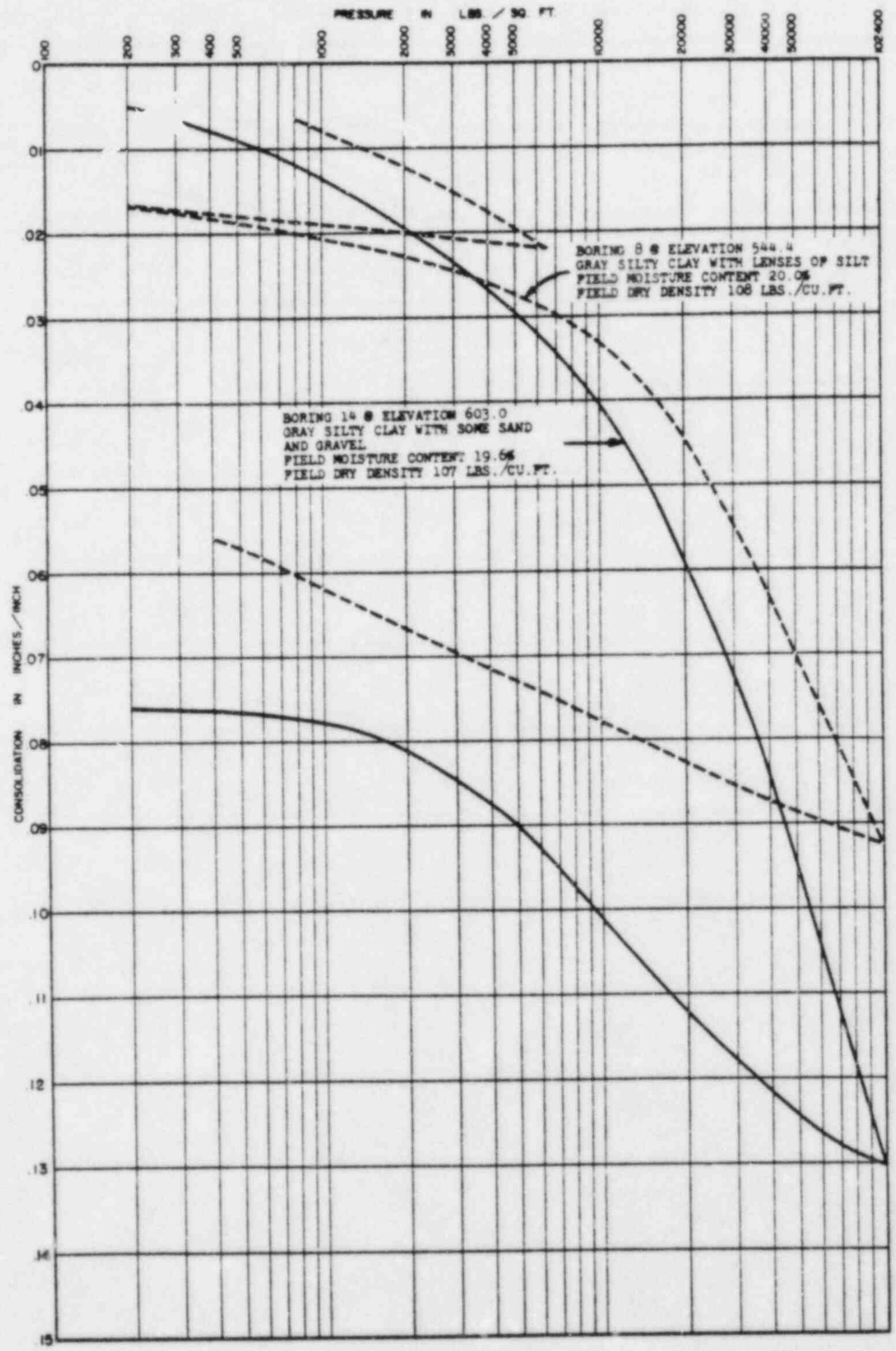
BY DATE
BY DATE
PLATE OF



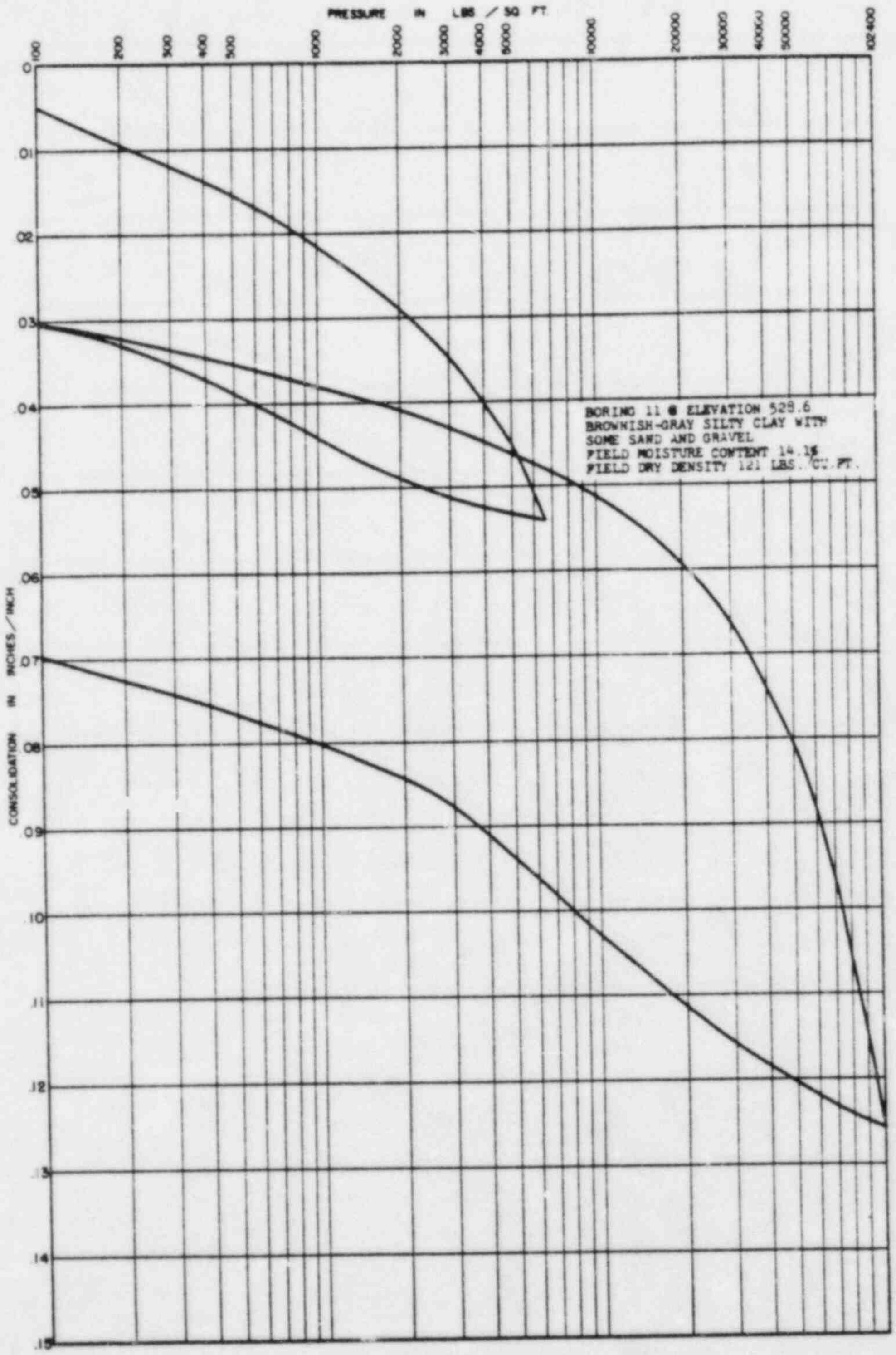
CONSOLIDATION TEST DATA

DAMES & MOORE

PLATE A-7F

REVISIONS
BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

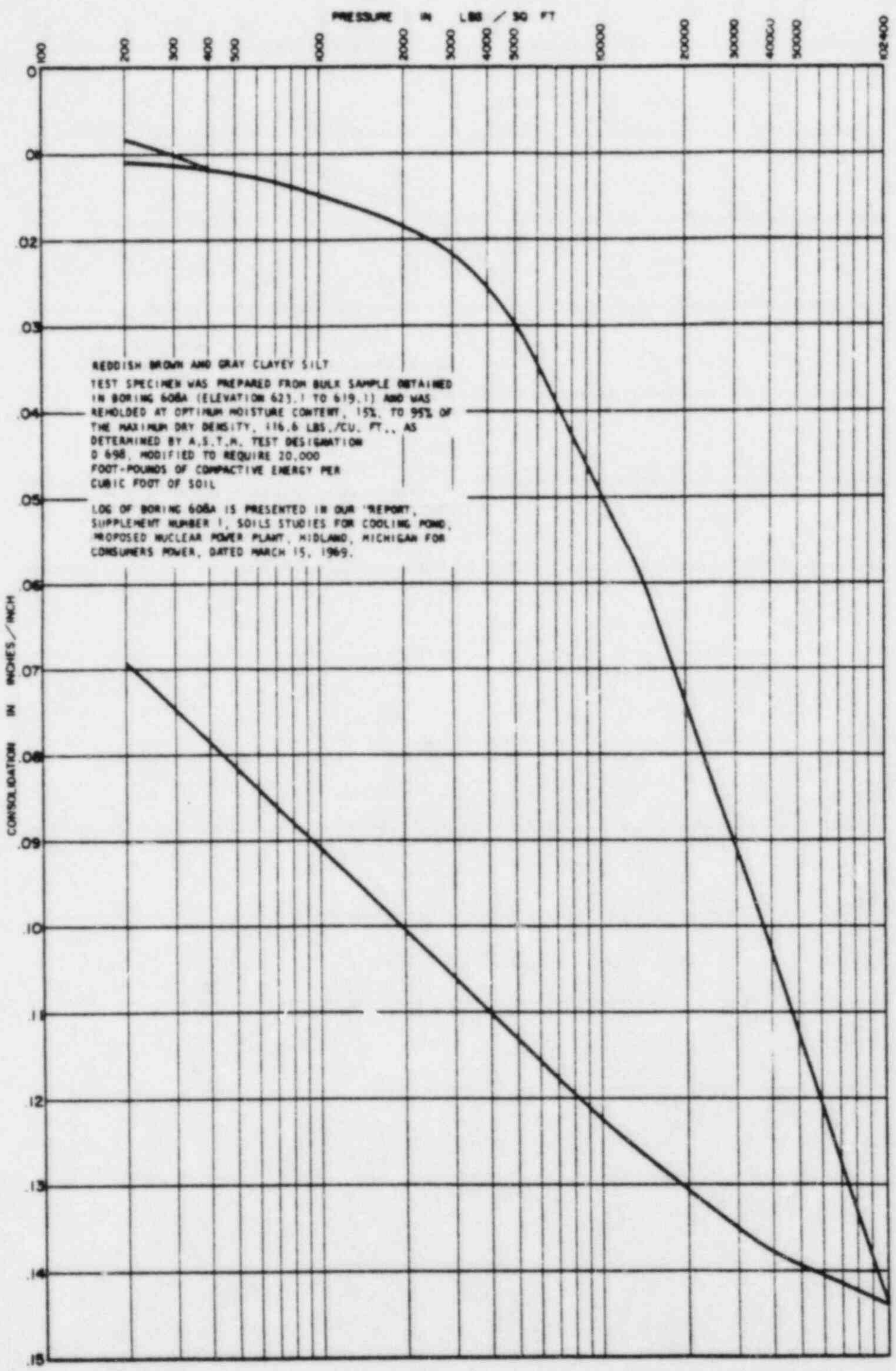
FILE 5697-004
BY R.H. DATE 5-24-62
CHECKED BY ____ DATE ____



CONSOLIDATION TEST DATA

DAMES & MOORE

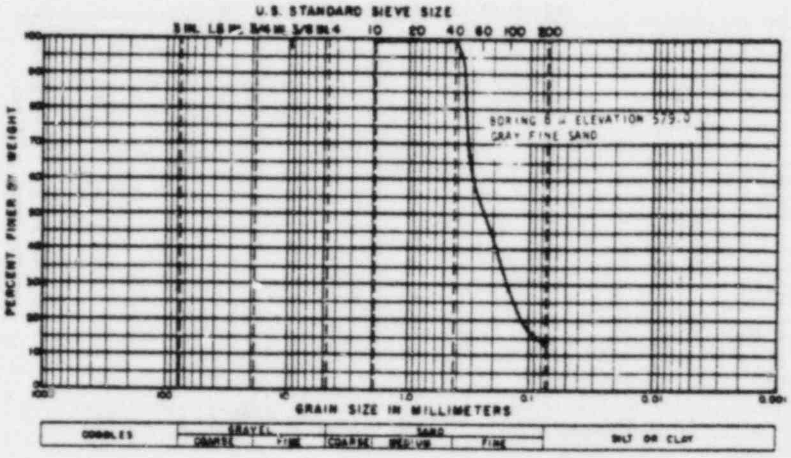
PLATE A-76

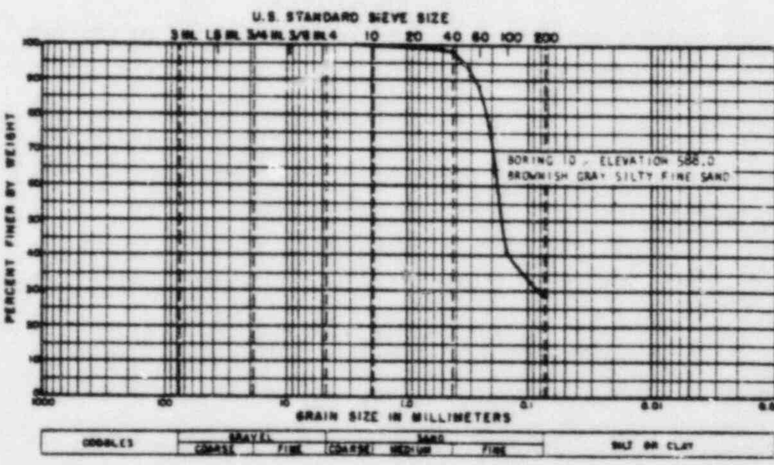


CONSOLIDATION TEST DATA

DAMES & MOORE

PLATE A-7H





GRAIN SIZE ANALYSES

DAMES & MOORE

PLATE A-11

Griffin Wellpoint Corporation

JACKSONVILLE, FLORIDA 904-388-7612
HOUSTON, TEXAS 713-923-2724
HAMMOND, INDIANA 219-931-1662
WEST PALM BEACH, FLORIDA 305-683-0702
NORFOLK, VIRGINIA 703-625-6524
NEW YORK, N. Y. 212-292-1800
CHICAGO, ILLINOIS 312-374-2255
QUEBEC, CANADA 663-3231

DAMES & MOORE CHICAGO, ILLINOIS RECEIVED FEB 24 '69

JBT		WWM
GD		
WG		
JVT		
EFC		ANW
DGC		JM
PSF		FILE

P.O. BOX 207 3450 CALUMET AVENUE HAMMOND, INDIANA 219 - 931-1662 CHICAGO

February 22, 1969

Dames & Moore Company
309 West Jackson Blvd.
Chicago, Illinois 60606

Re: Nuclear Power Plant
Midland, Michigan

Attention: Mr. Bill Moore

Gentlemen:

From a study of available preliminary plans, boring data, soil samples, grain-size curves and a soil profile of the proposed excavation area, we propose the excavation be open-cut on the Northeast side on approximately two (2) horizontal to one (1) vertical slopes, (to allow for berms at elevation 600.0 and 585.0 for unwatering and stabilizing this area of pervious material with a 2-stage interconnected wellpoint system. (See attached sketch).

Although the soil samples visually appear to be a fine sharp and clear sand, the grain size analysis, and previous wellpoint dewatering in this area, indicates that the wellpoints must be installed with vertical sand filter-wicks for required drainage and drawdown.

Since the clay strata varies in depth over this Northeast Side, there may be some dips in the clay that will require a small amount of sand-bagging. However, since the pervious soils get deeper away from the excavation and toward the Northeast, this sandbagging should be a minimum item.

Our estimate of the cost of this dewatering (with no mark-up) is approximately \$ 68,000.00 for six months pumping, plus (or minus) \$ 270.00 per calendar day thereafter.

If there are any questions on the above.....or changes in the plant location .....please call us.

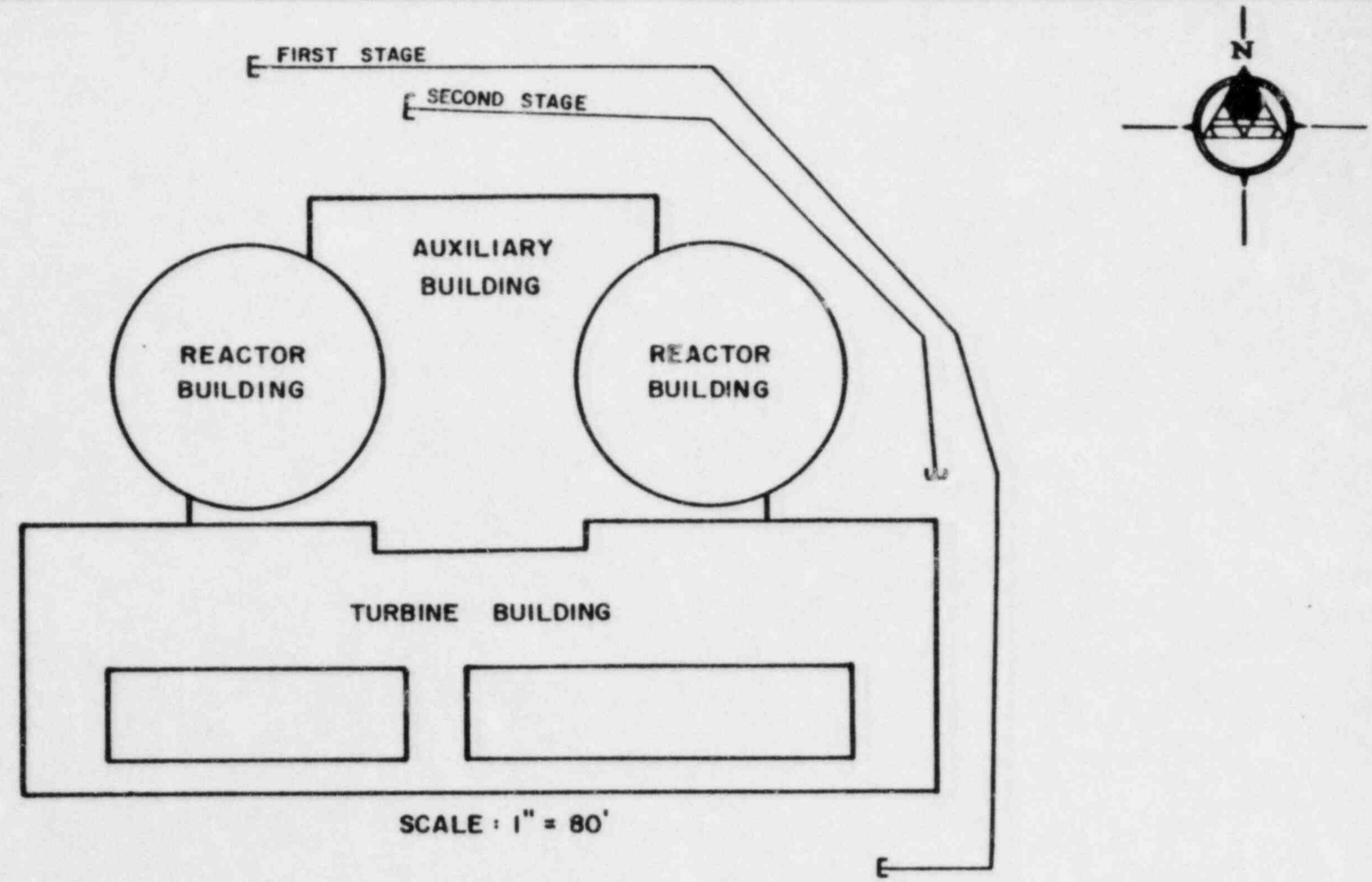
Very truly yours,
GRIFFIN WELLPOINT CORPORATION
(Indiana)

R H Hockberger

R. H. Hockberger
Vice President

RHH/ms
Enclosure: Sketch of 2-stage wellpoint location

BY ____________ DATE __________ REVISIONS
CHECKED BY __________________ FILE ______________________________ BY _________ DATE ______



SKETCH OF PROPOSED LOCATIONS OF UPPER AND LOWER DEWATERING SYSTEMS

DRAWING REFERENCE:

ORIGINAL SKETCH PREPARED BY
GRIFFIN WELLPOINT CORPORATION

DAMES & MOORE

PLATE A-13

3

Gentlemen:

I am Thiru Thiruvengadam. I am with Consumers Power Company as a Staff Engineer in the Project Engineering Services Department.

3.0 Remedial Work

I will be describing the remedial measures in progress or planned for the first five items on the agenda under Article 3 (Slide 3.1.1). Namely, DGB, SWB, tank farm, diesel oil storage tanks and underground facilities. These remedial measures are discussed in great detail in our responses to 50.54(f) questions and 50.55(e) submittals. Therefore, I will be presenting only a brief outline of the remedial work.

3.1 Diesel Generator Building (Slide 3.1.2)

The diesel generator building is a box-shaped structure. Its main purpose is to provide a housing for the four emergency diesel generators. The structural walls are very rigid. The building is supported on strip footings. The building and the generator pedestal are founded on approximately 30 feet of fill. In summer of last year, settlements more than anticipated values were observed. A detailed soil investigation was conducted. The backfill was found to consist of soft to very stiff clay with pockets and layers of very loose to dense sand backfill. The conclusion of the investigation was that the fill was not adequately compacted. Based upon the recommendation of our soil consultants, Professors Peck and Hendron, the remedial measure chosen was to preload the existing backfill by layers of sand surcharge.

(Slide 3.1.3) - This slide shows in plan the extent of sand surcharge. The surcharge was gradually applied in steps. To date, the backfill under the diesel building is subjected to 20 feet thick of sand surcharge. This slide (Slide 3.1.4) shows a cross section of the building and the surcharge. The surcharge will produce stresses in the fill greater than the amount the fill would experience when the structure is operational. This surcharge will remain until excess pore pressures are essentially dissipated and the rate of residual settlement becomes small and can be predicted conservatively by extrapolation.

The preload consolidates soft areas of clay fill; however, will not significantly improve the quality of loose sands. The potential of liquefaction of these sands and aerial dewatering of the plant site as a remedial measure for this problem will be presented later in detail.

(Slide 3.1.5) - This slide shows plan and cross-sectional elevation of a typical diesel generator pedestal. This is a reinforced concrete structure having a minimum compressive strength of 4000 psi. The fill beneath the pedestals have also consolidated resulting in differential settlement. Differential settlement of the pedestals will have no effect on alignment of the engine and the generator because they are both mounted on the same foundation. Furthermore, because of the enormous stiffness of the pedestal, no significant warping is expected and the top of the pedestal will generally lie within one plane. The diesel generator will be set in a level position irrespective of the amount of differential settlement between the corners of the pedestal. It will be achieved either by a suitable layer of grout on the pedestal or by chipping a few inches of top concrete and refinishing it to the required level.

The machine itself has considerable tolerance limits for tilt and roll. Delaval Turbines, the manufacturer of the diesel generator, stated that of the [illegible] forward tilt of 1.4° and roll of 5° combined

a 5° combined backward tilt and roll will not affect the performance of the engine and the generators. Furthermore, during the operation of the plant, if further differential settlement causes to exceed this tolerance, the manufacturer states that the generators can be shimmed back to level position. Therefore, in summarizing for the DGB, the remedial work of preload is in progress and dewatering of site is being planned for implementation soon. No further remedial work on the pedestal than that mentioned before is anticipated.

3.2 Service Water Pump Structure

(Slide 3.1.1) - The service water pump structure is located in the southeast end of the site adjacent to the cooling pond. This (Slide 3.2.1) slide shows a plan view of the structure. The cooling pond is on the southern side. Major portion of the structure is founded on natural soil material except for the northern portion which is founded on fill. (Slide 3.2.2) - This slide shows a cross-sectional view of the structure. As mentioned earlier, the northern section, which is cantilevered off the main building, is founded on backfill material. As a follow-up to the investigation of all Class I structures on fill, several borings were taken in this area. The borings indicated that the backfill consists of soft to very stiff clay and loose to very dense sand. The conclusion was that some areas of the fill material under the northern part of the structure were not sufficiently compacted.

However, no significant settlement of the structure has been noted. The reason for this is that the existing dead loads from this portion are being supported by the rest of the structure through cantilever action.

The remedial measure chosen was to support the north wall on piles driven [illegible] till. The choice of piles is an economical and expedient solution with minimal impact on the schedule.

(Slide 3.2.3) - This slide shows in plan the layout of piles. A total of 16 piles is planned at this time. The piles will have a capacity of 100 tons and are designed as bearing piles to carry only vertical load. The piles will be pipe piles filled with concrete. They will be predrilled through the fill and driven into the glacial till. The length of piles is expected to be 50 feet.

(Slide 3.2.4) - This slide shows the method of transferring vertical load from the wall to the piles by a system of reinforced concrete corbels.

(Slide 3.2.5) - The concrete corbels will be anchored to the wall by a system of anchor bolts. The pipe piles in turn would be jacked against the corbels to effect the transfer of load.

A test pile will be load tested to determine its capacity.

3.3 Tank Farm

(Slide 3.3.1) - This slide shows tank farm in plan. There are two BWSTs, a utility tank and a primary storage tank. Of these, only BWSTs are safety related. The BWST has a capacity of 500,000 gallons, 52 feet in diameter and 32 feet in height.

(Slide 3.3.2) - The tank is supported on a short concrete ring girder ending in a strip footing. The tank by itself is quite flexible.

Adjoining the ring girder for each tank there is a small box-shaped structure called [illegible] pit. This [illegible] valves and other controls. At present, construction of ring girder and valve pits are complete and installation of piping is in progress. As a follow-up to to the investigation of all Class I structures founded as fill, several borings and test pit examinations were done in the tank farm area. The results of the investigation indicate that the tanks are supported on medium to very stiff clay backfill with occasional medium to very desne sand layers. The condition of the fill is suitable for the support of the tanks. To confirm this, the tanks will be constructed and filled with water in order to make a full-scale test of the foundation soil.

The (Slide 3.3.3) slide shows the layout of borated water lines entering the tank through the valve pit. The piping connections are being made to allow start-up, flushing, filling and testing of the tank. Selected points on the piping between BWST and the auxility building will be monitored for settlement during construction phase. Any differential settlement ~~that was~~ measured will be analyzed in accordance with established procedures.

In summary, the backfill material on which the BWSTs are founded is satisfactory and will be confirmed by a load test. Borated water lines will be monitored and evaluated for any differential settlements. Therefore, no remedial action is anticiapted for these structures.

3.4 <u>Diesel Oil Storage Tanks</u>

(Slide 3.1.1) - The oil storage tanks are located southeast of the diesel generator building. There are 4 tanks, each 1[illegible] feet in diameter and 44 feet in length.

(Slide 3.4.1) - There is six feet of earthen cover over the top of the tank. The tank is supported at three points anchored to concrete pedestals. The tanks are founded on backfill and results of boring program indicated that the tanks are supported on medium to stiff sandy clay backfill. This soil condition is adequate to support the tanks. Moreover, the weight of the tanks is approximately equal to the fill that it replaced. In order to verify that the fill is satisfactory, these tanks have been filled with water and settlements are being monitored. It has been three months since the tanks have been filled with water and no appreciable settlements have been noted yet. Therefore, the backfill is adequate and no remedial measures are anticiapted.

3.5 Underground Facilities

The underground facilities that will be discussed are Seismic Category I piping and electrical duct banks. This (Slide 3.1.1) slide shows safety-related piping, namely Service Water Lines, from the auxiliary building to the service water structure and diesel generator building to the service water structure. Borated water lines from the auxiliary building to BWST and diesel oil lines from the diesel oil storage tanks to the diesel generator building. Electrical duct banks are also shown in this slide.

To evalute the present condition of piping, a representative group of piping was selected and profiled by a Nold Aquaducer Profile Settlement Gauge. This (Slide 3.5.1)slide shows for illustrative purposes a plot of one of the lines profiled. All the pipes profiled were reanalyzed taking into account the measured differential settlement in accordance with the provisions of current codes. The analyses showed that the effect of differential settlement on the stresses were minimal and much below the level of allowable stresses. A detailed discussion of piping stress analysis would be covered later, if required.

In summary, the pipes are very ductile and calculations show that effects of differential settlement undergone so far have minimal effect on stresses. Therefore, no remedial work is anticipated with regards to buried piping.

Electrical Duct Banks

The duct banks are reinforced concrete elements enclosing PVC and rigid steel conduits thus providing voids for the cables. Earlier, Mr Tom Cooke described the continuity checks that were performed by passing a rabbit through all the voids. This program establishes the fact that, to date, the duct banks are intact. Furthermore, the duct banks are reinforced with nominal amount of steel therefore possesses considerable amount of ductility in bending.

(Slide 3.5.2) - A preliminary calculation indicated that a typical duct bank of 100 feet in length can undergo a maximum of 12" of central deflection in pure bending at ultimate load.

In summary, the integrity of the duct bank is established by passing a rabbit through during construction three and the duct bank by itself is ductile and can absorb considerable amount of differential settlement without significant stresses. Therefore, no remedial measures are anticipated for duct banks.

1.

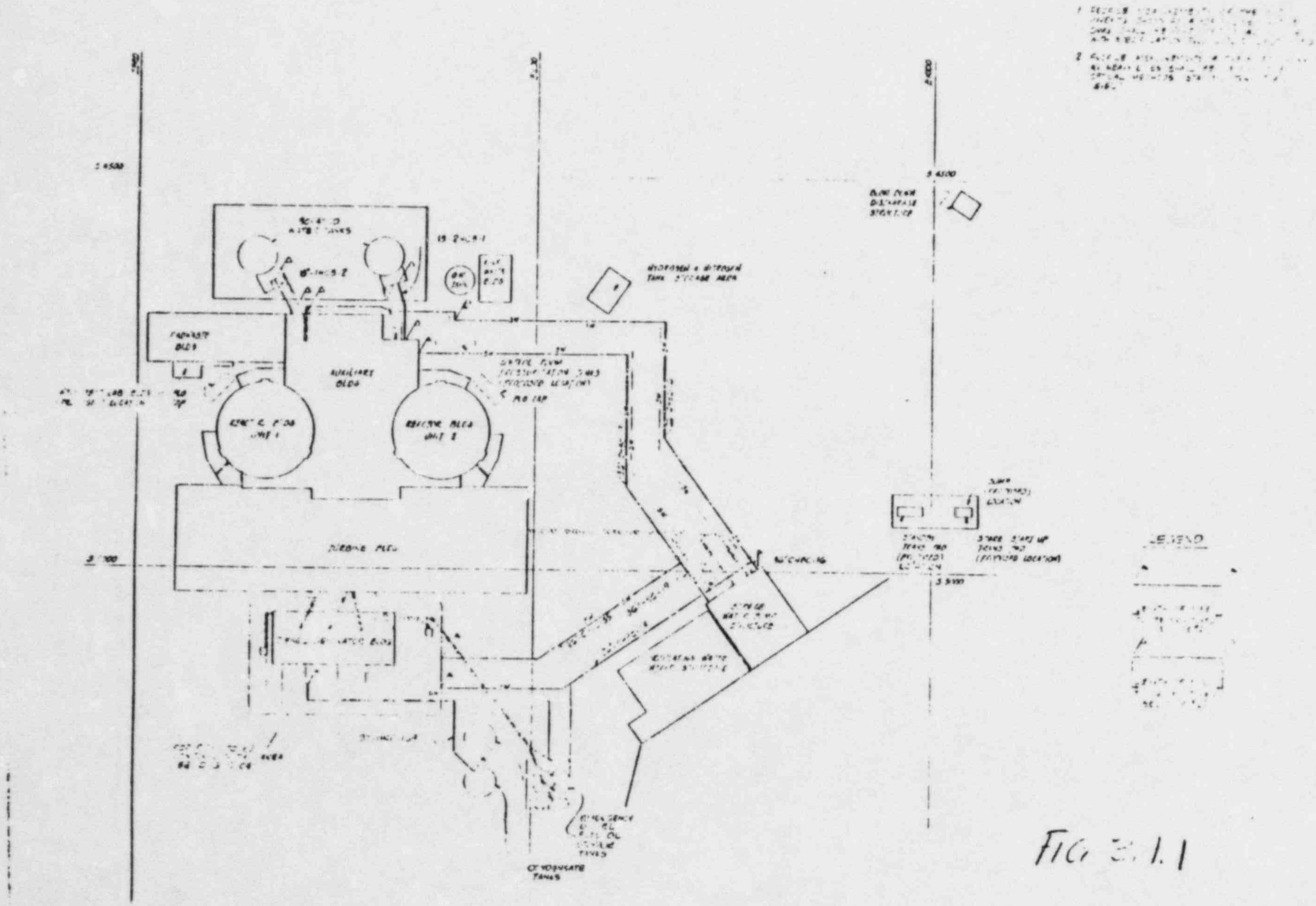
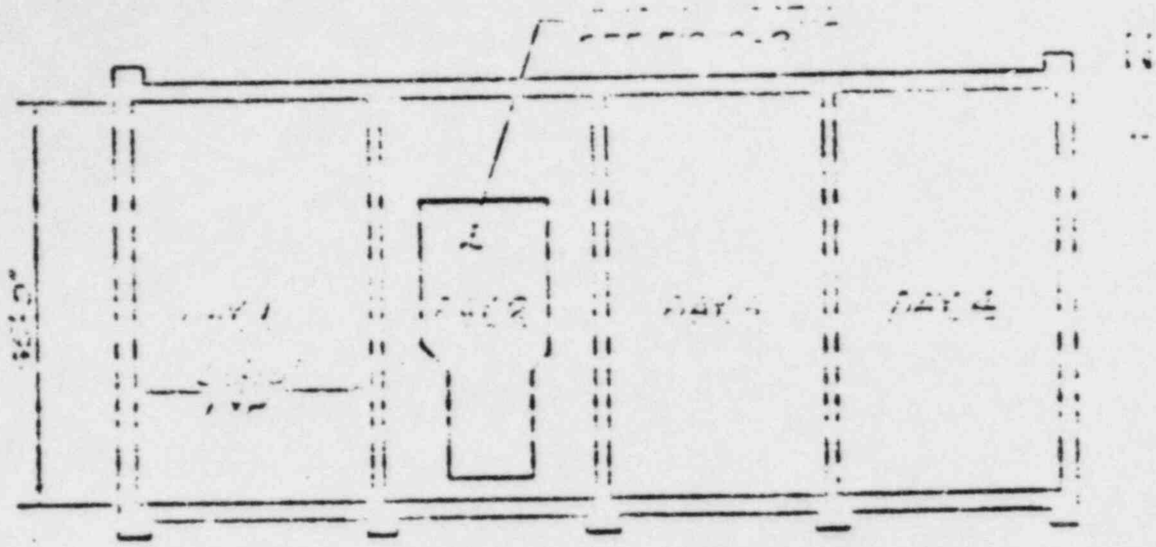
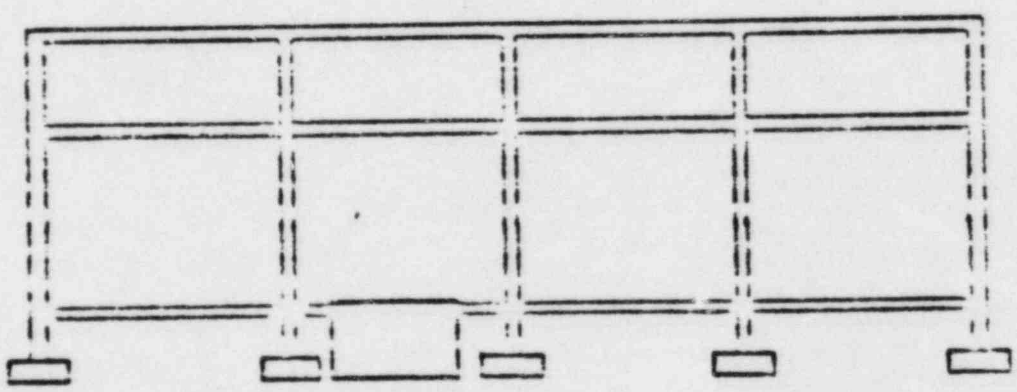

AUXILIARY BLDG
REACTOR BLDG UNIT 2
TURBINE BLDG
HYDROGEN & NITROGEN TANK STORAGE AREA
LEGEND
CONDENSATE TANKS

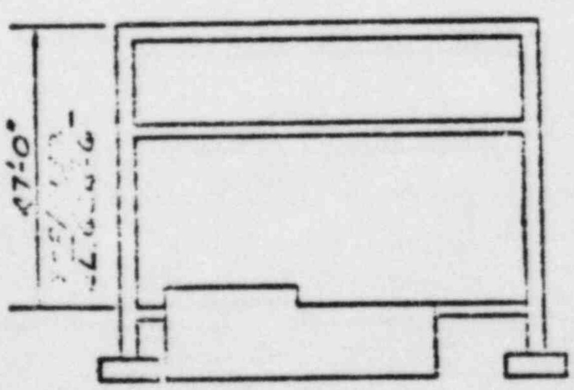
FIG 3.1.1



PLAN



SECTION
LOOKING SOUTH



SECTION
LOOKING WEST

FIG. 3.1.2

MIDLAND PLANT UNITS 1 & 2 CONSUMERS POWER COMPANY
DIESEL GENERATOR BLDG PLAN & SECTIONS
FIGURE ~~13-9~~ DATE: 4/24/79

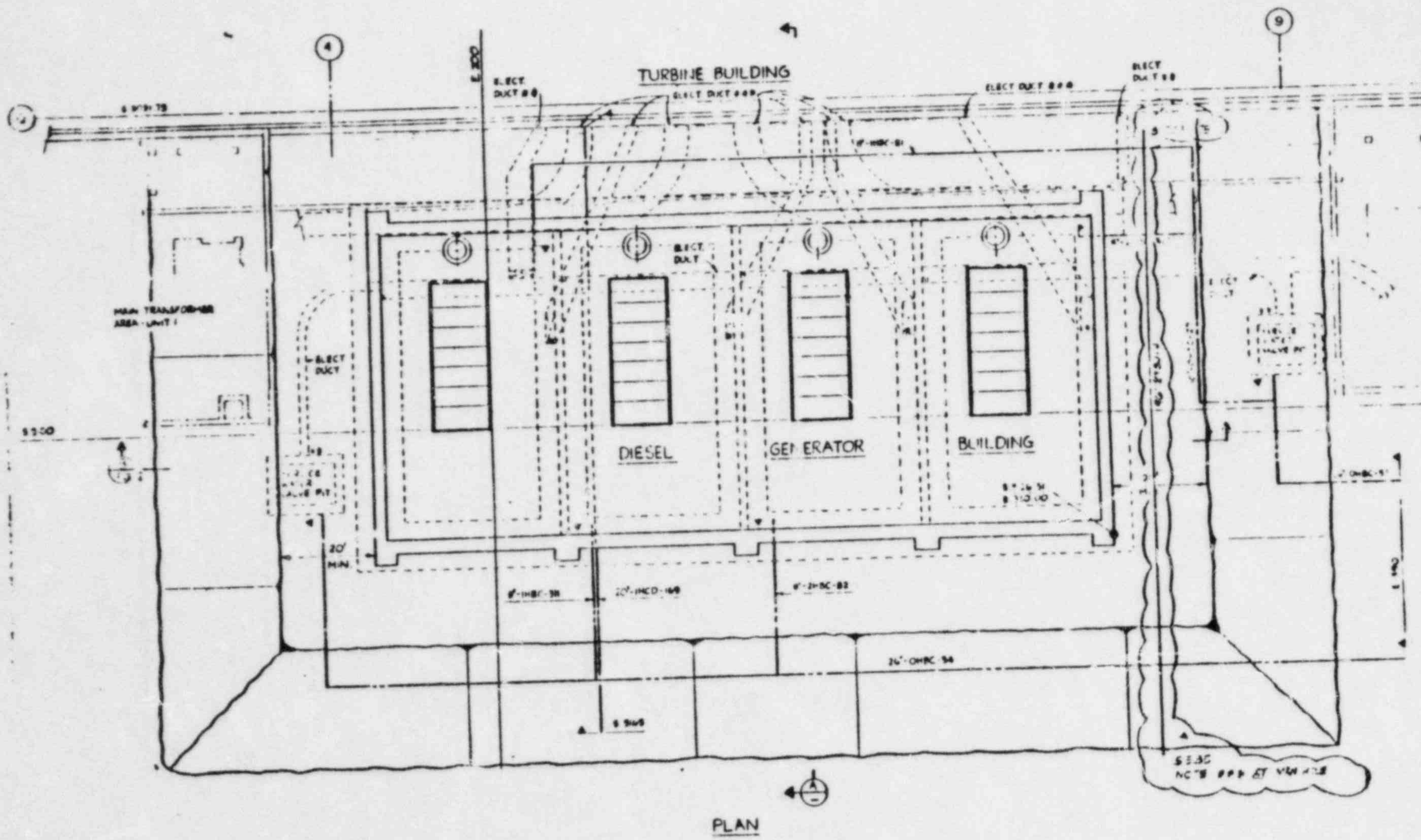

TURBINE BUILDING
ELECT DUCT
DIESEL
GENERATOR
BUILDING
MAIN TRANSFORMER AREA-UNIT 1
20' MIN.
PLAN

Fig. 3.1.3

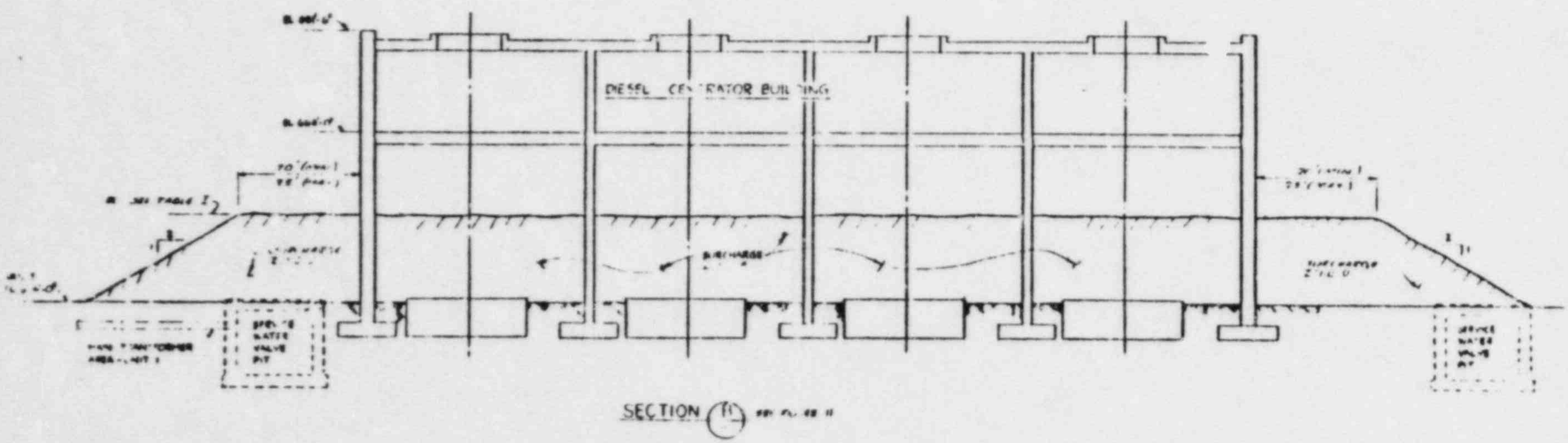


FIG. 3.1.4

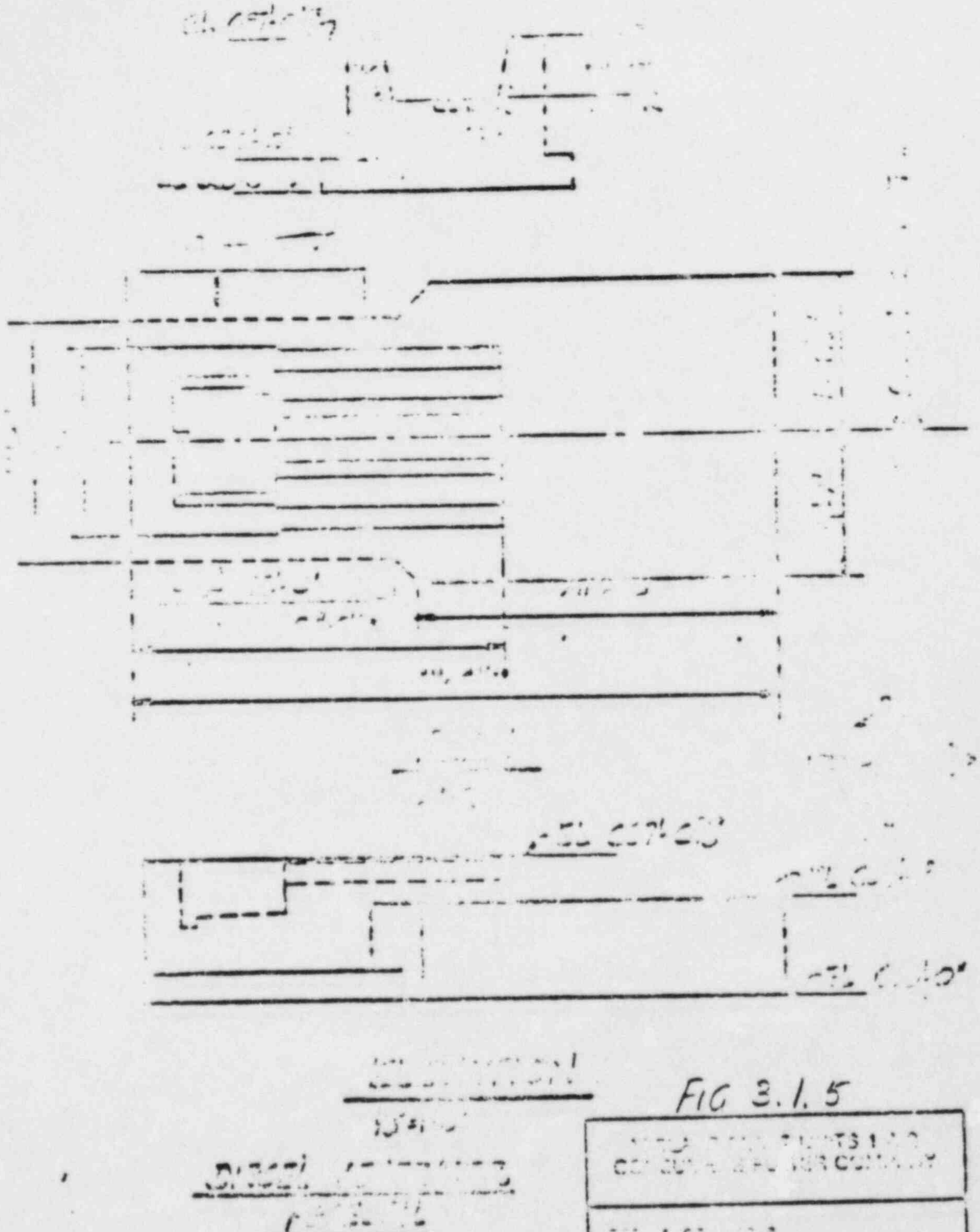


FIG 3.1.5

DATE

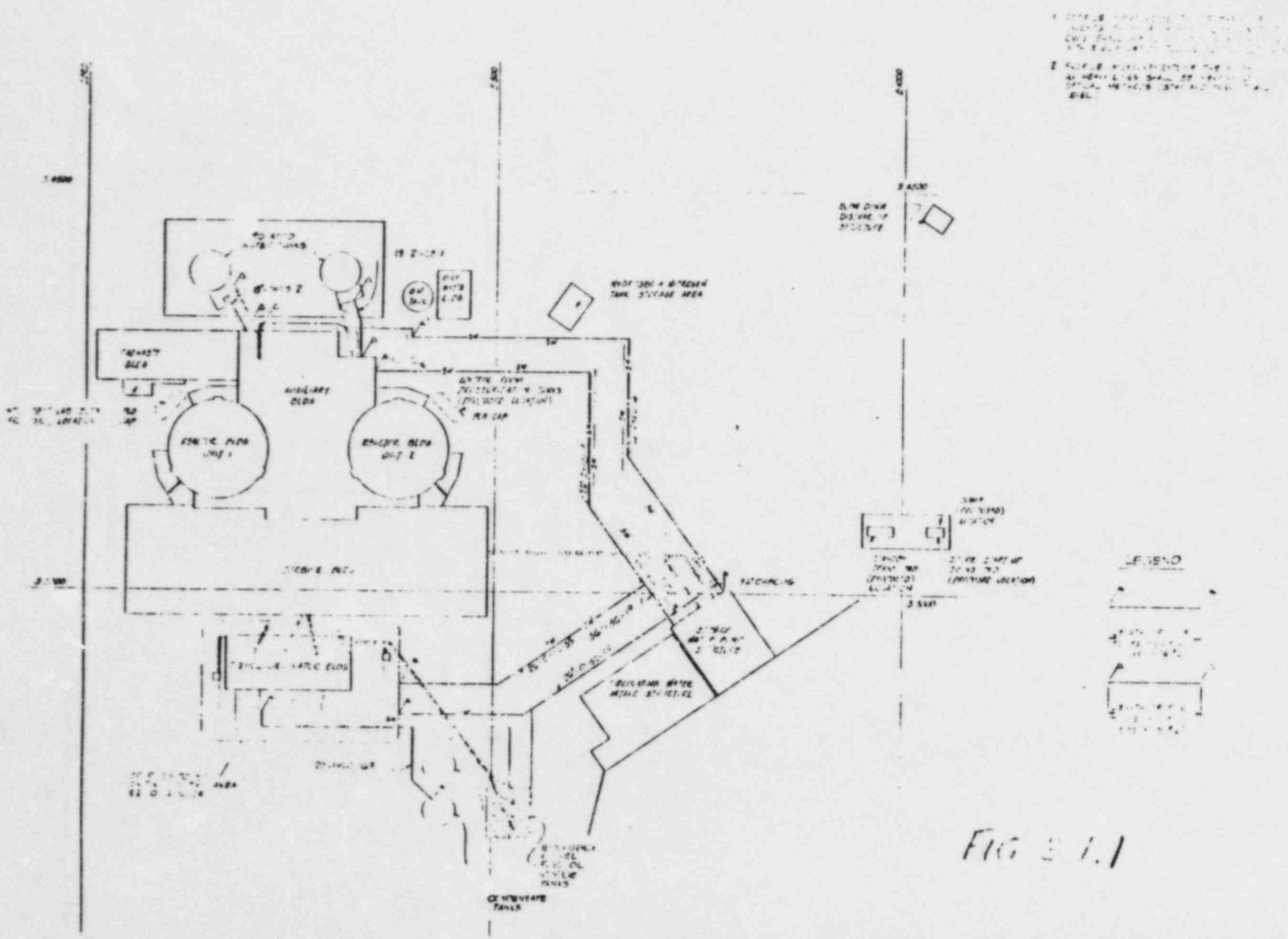
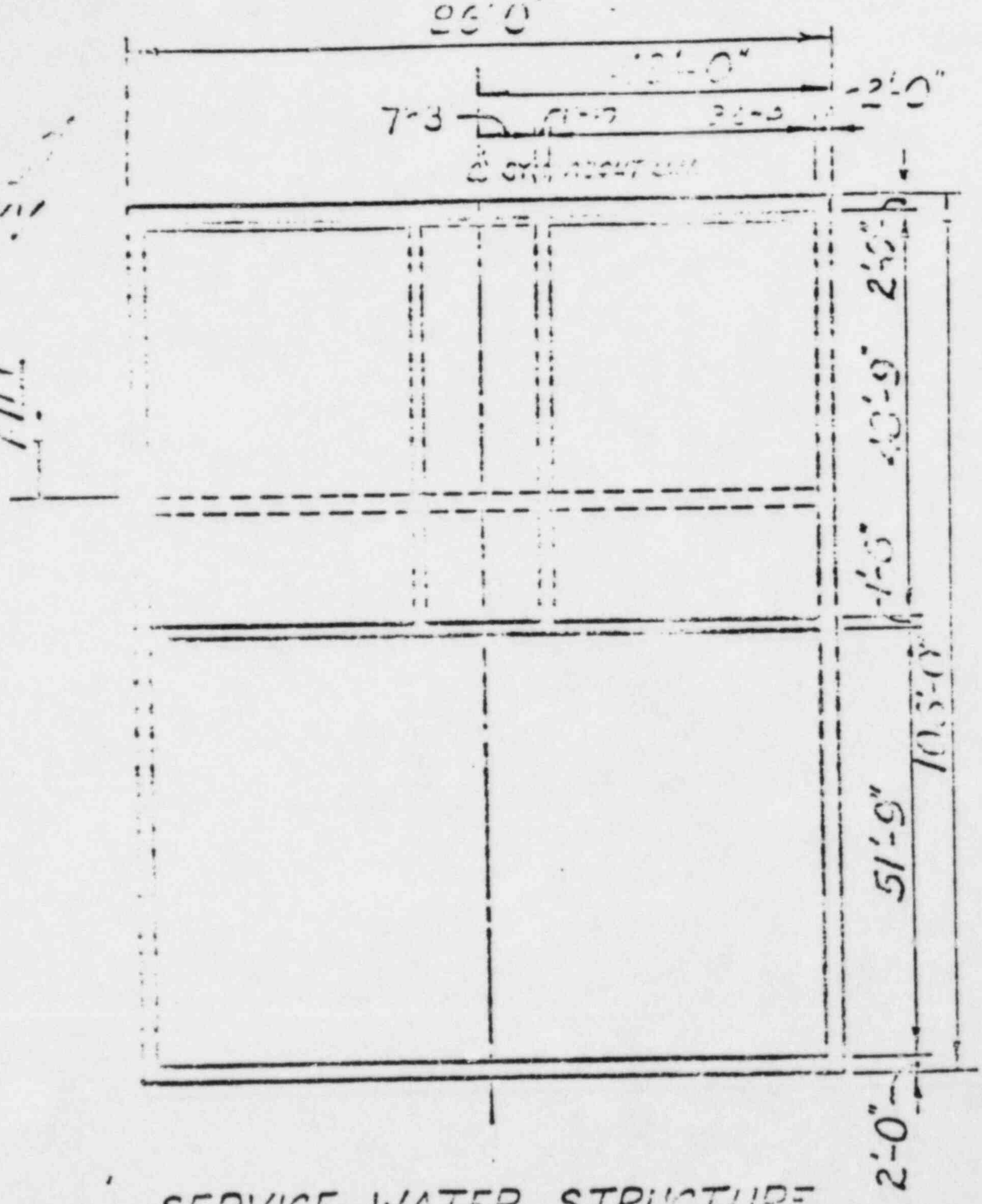
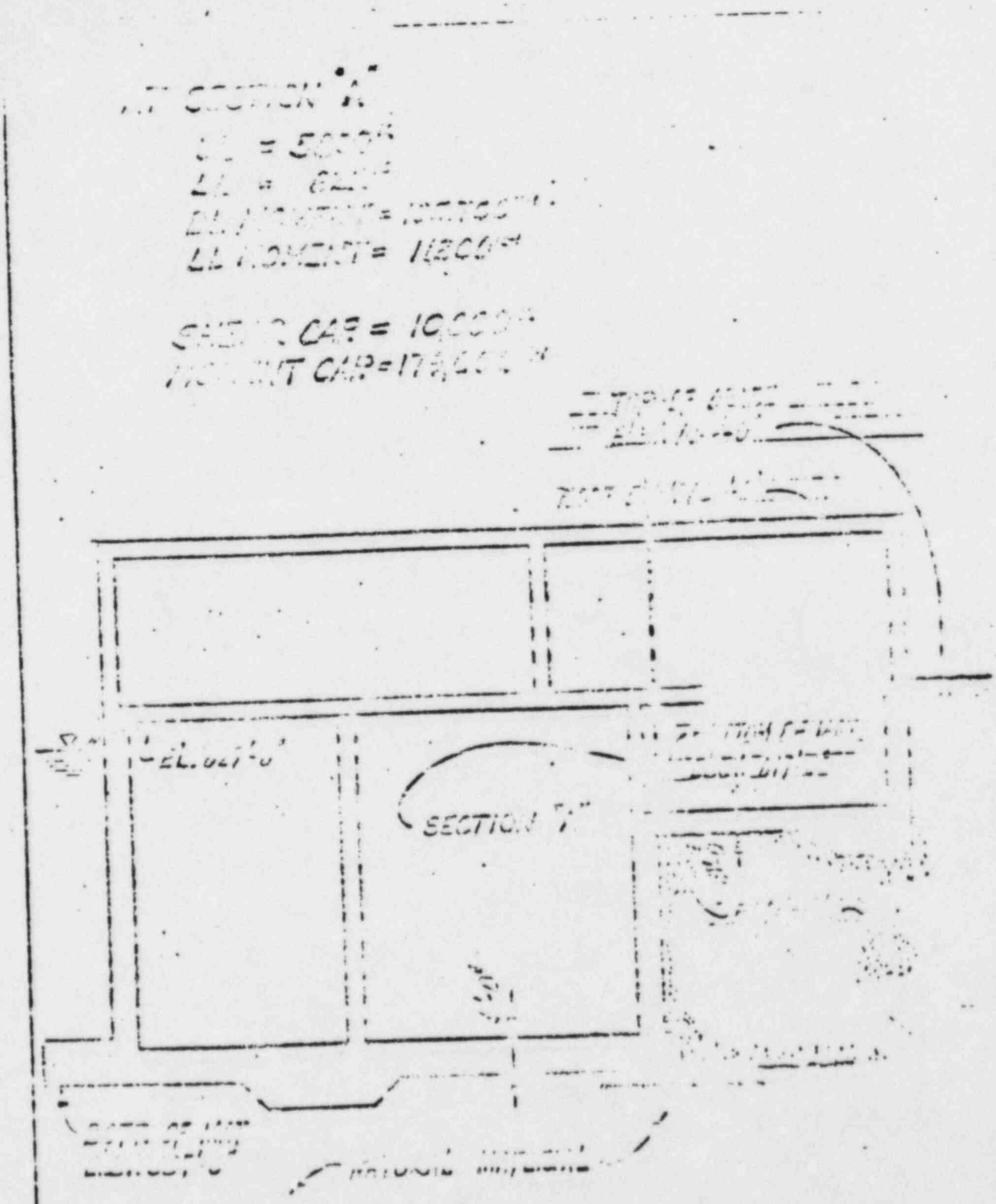

AUXILIARY BLDG
REACTOR BLDG UNIT 2
TURBINE BLDG
HYDROGEN & NITROGEN TANK STORAGE AREA
LEGEND
CONDENSATE TANKS

FIG 3.1.1



SERVICE WATER STRUCTURE
PLAN AT EL. 634'-6"

FIG. 3.2.1



TYPICAL SECTION

SERVICE WATER

STRUCTURE

FIG. 3.2.2

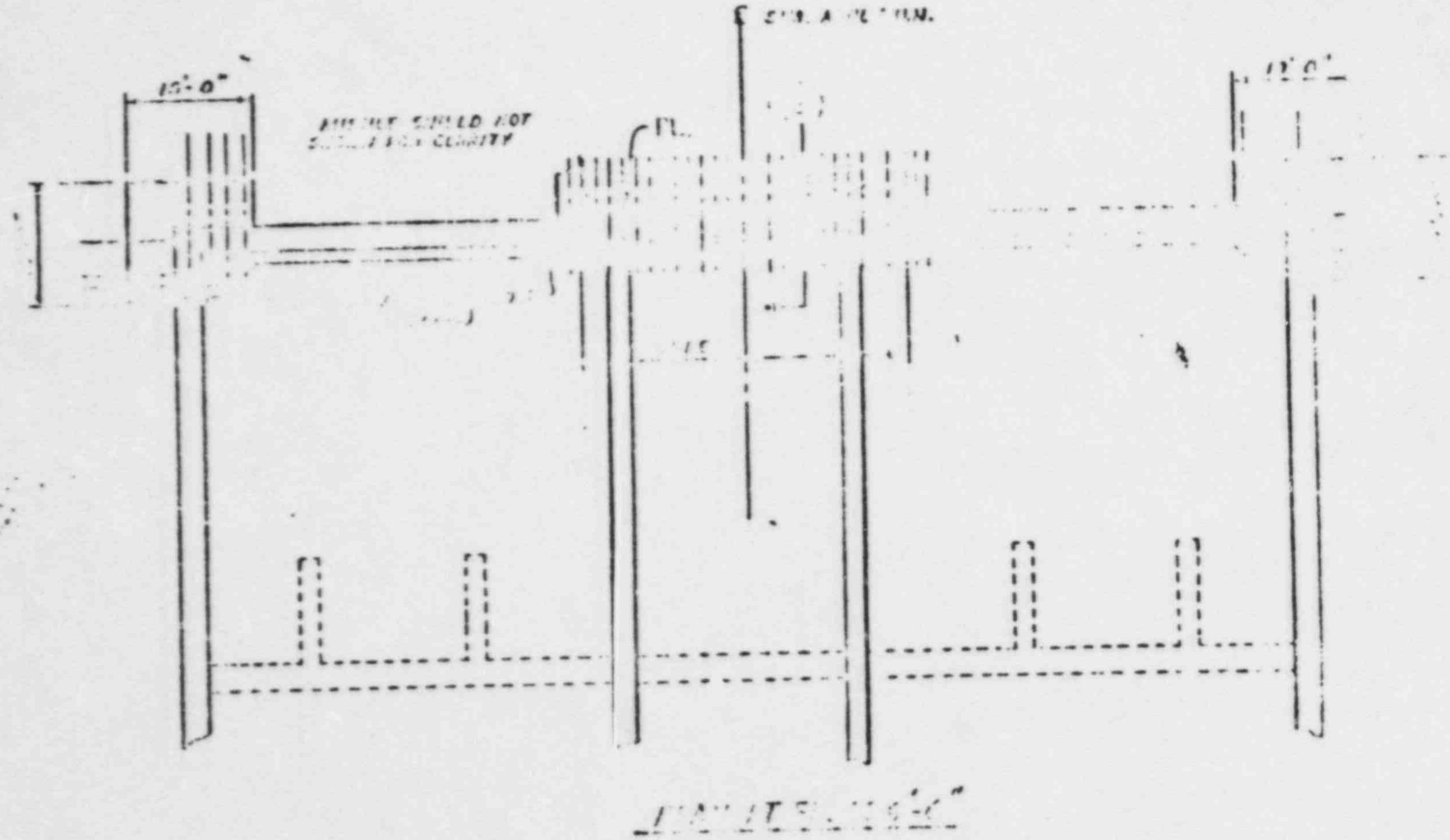
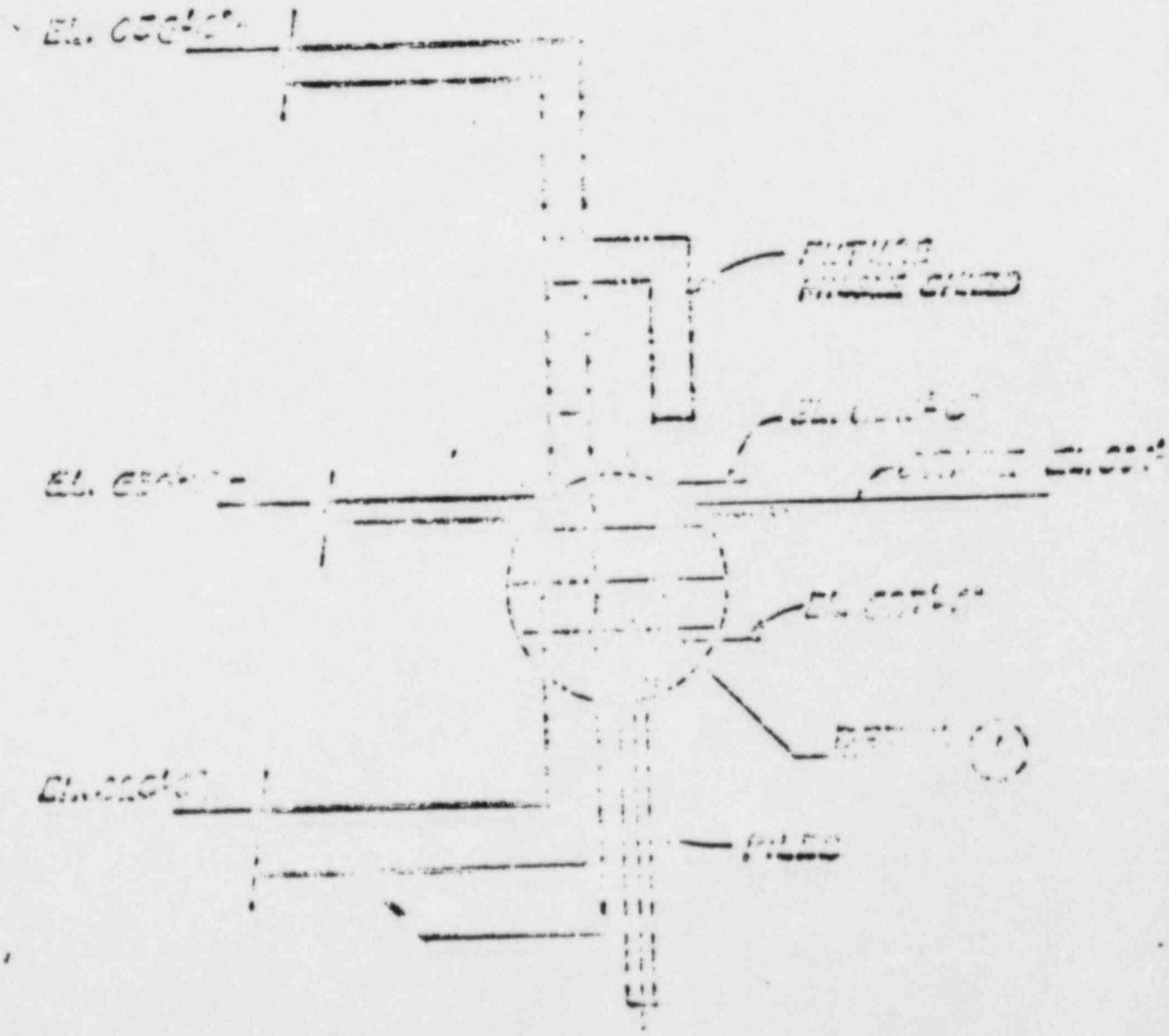


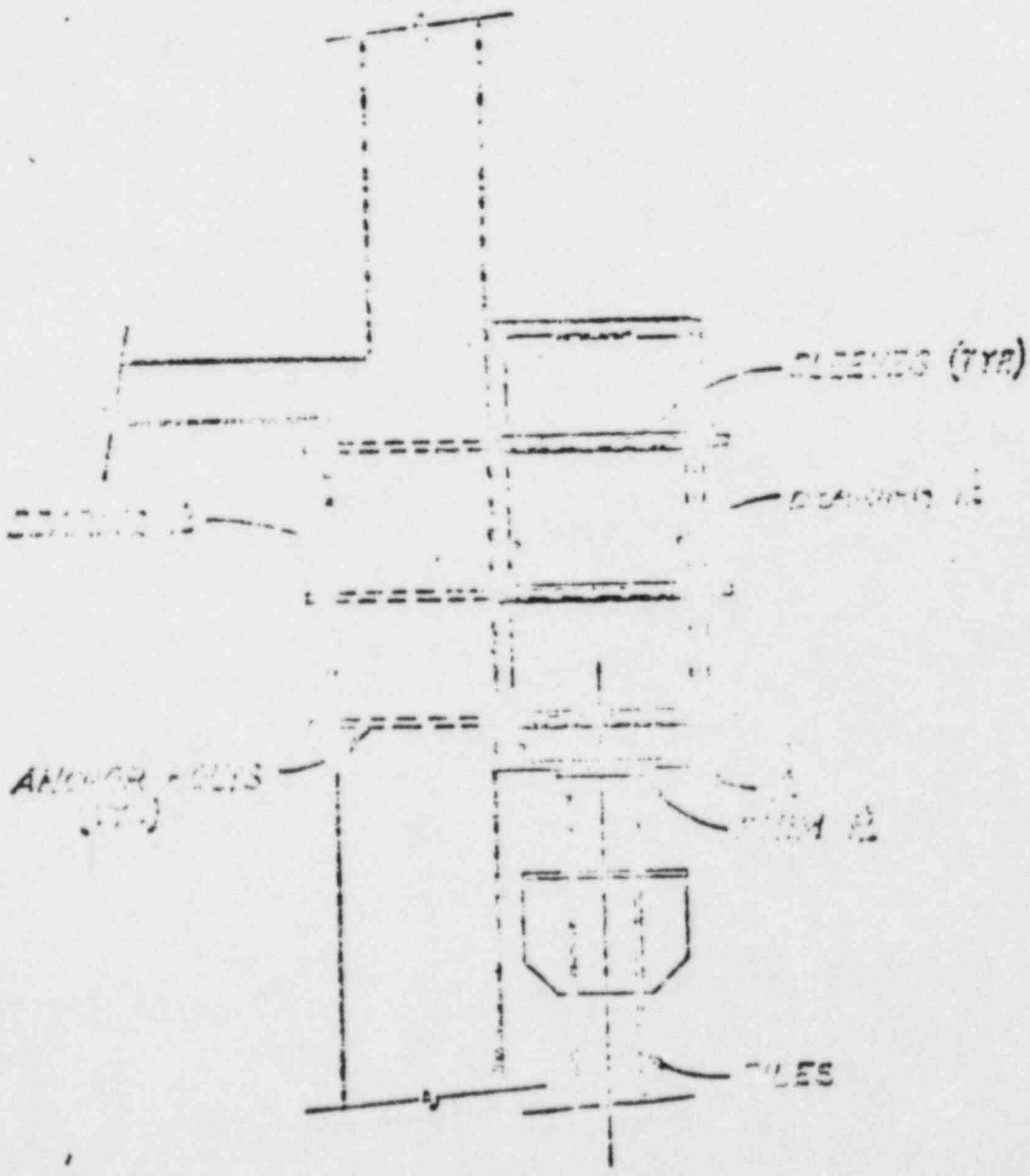
FIG. 2.2.3



STATION

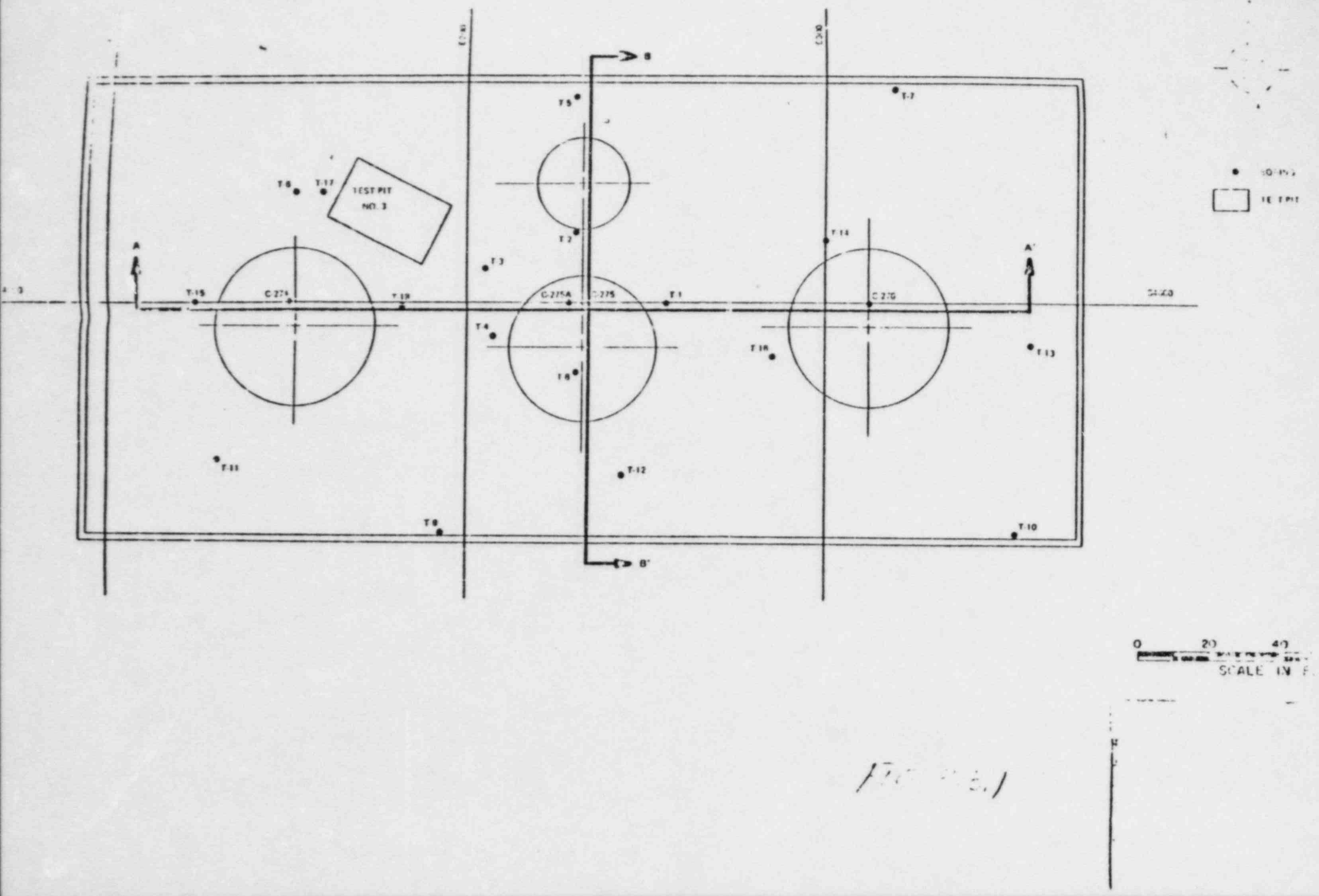
FIG 3.2 4

[illegible] CAPACITY OF A-540 [illegible]
[illegible] TO ACCOUNT FOR [illegible]
[illegible] CREEP AND BEA[illegible]

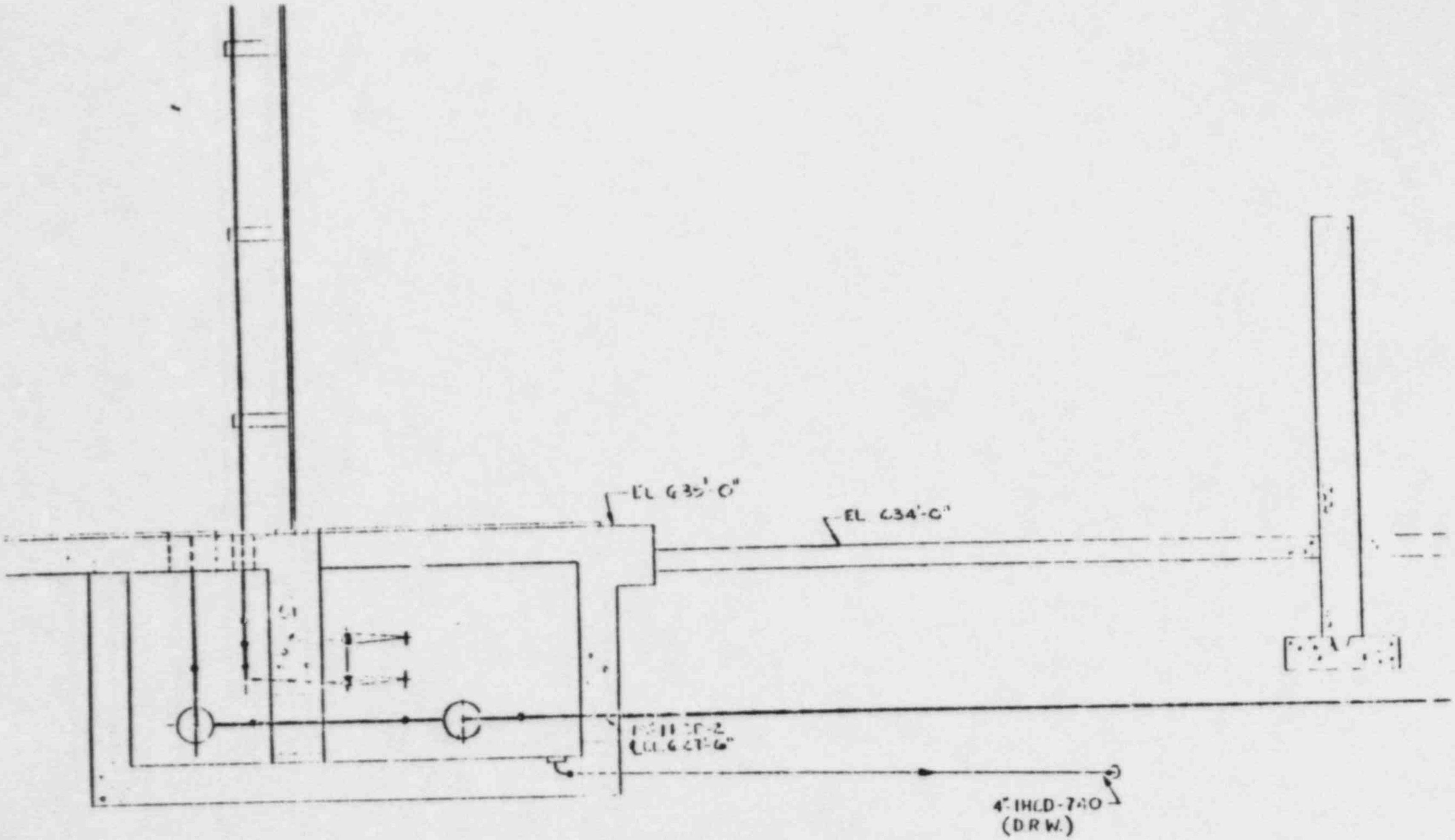


DETAIL

FIG 3.2.5



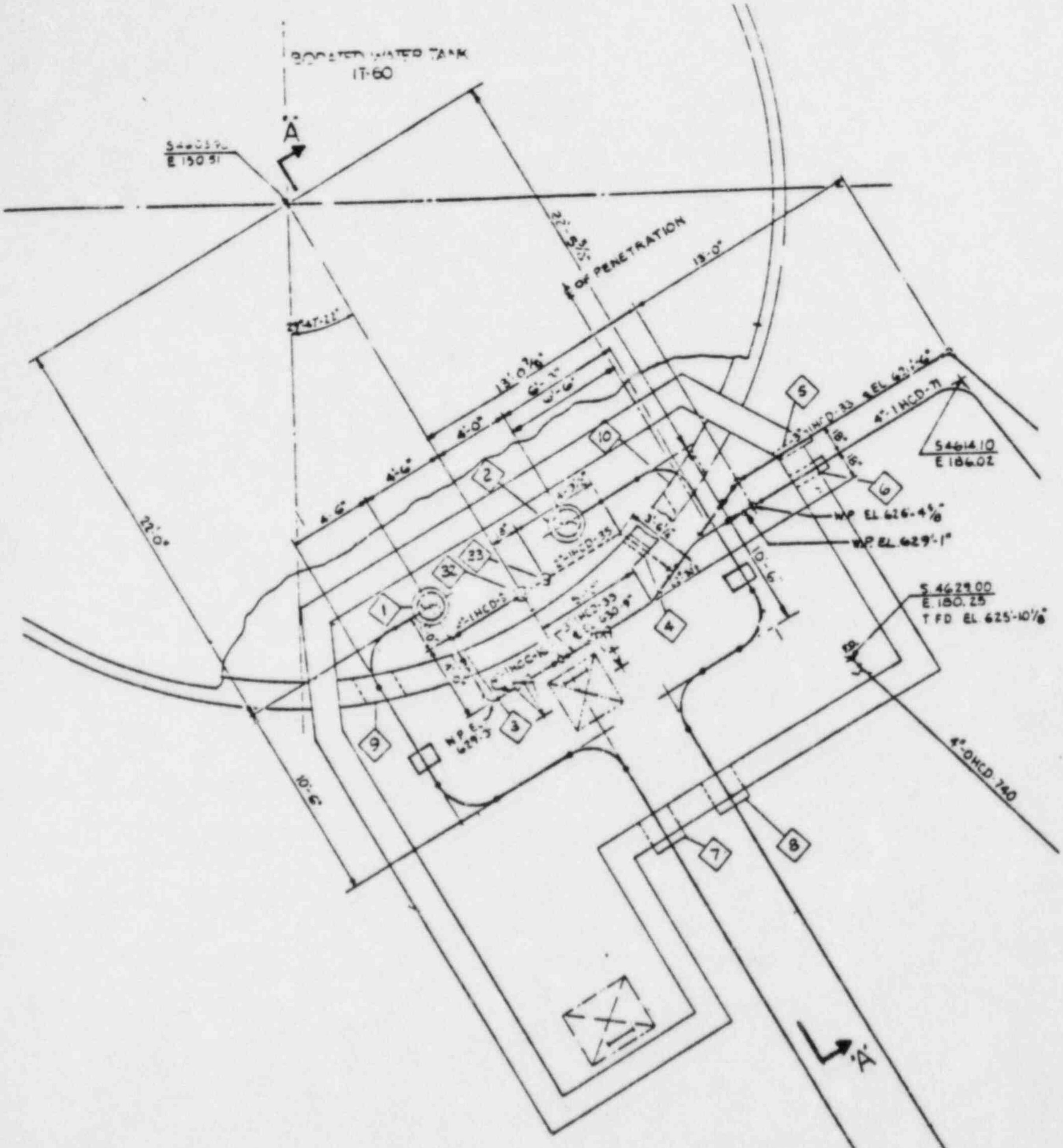
A
A'
B
B'
TEST PIT
NO. 3
T-15
C-274
T-19
T-3
T-4
C-275A
C-275
T-1
T-2
T-5
T-6
T-17
T-8
T-9
T-11
T-12
T-14
C-276
T-16
T-7
T-13
T-10
0
20
40
SCALE IN F



SECTION "A-A"

FIG 3.3.2

BORATED WATER STORAGE



BORATED WATER TANK
1T-60
A
E 150.51
℄ OF PENETRATION
13'-0"
22'-0"
4'-0"
4'-6"
S4614.10
E 186.02
W.P. EL. 629'-1"
S.4629.00
E. 180.25
T.F.D. EL. 625'-10⅛"
4"-OHCD-740
10'-6"
1
2
3
4
5
6
7
8
9
10
23
32
A

PLAN

FIG. 3.3.3

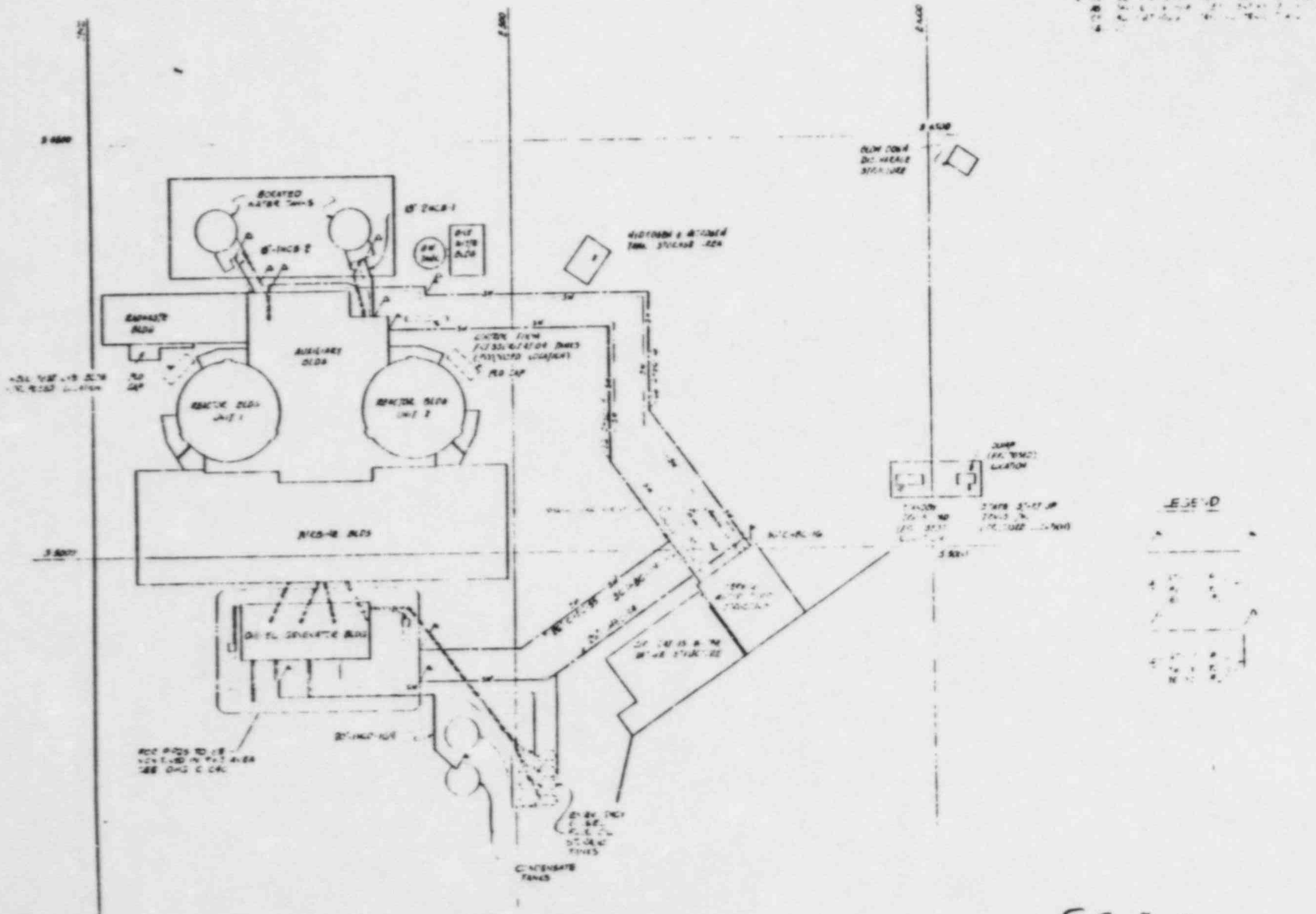
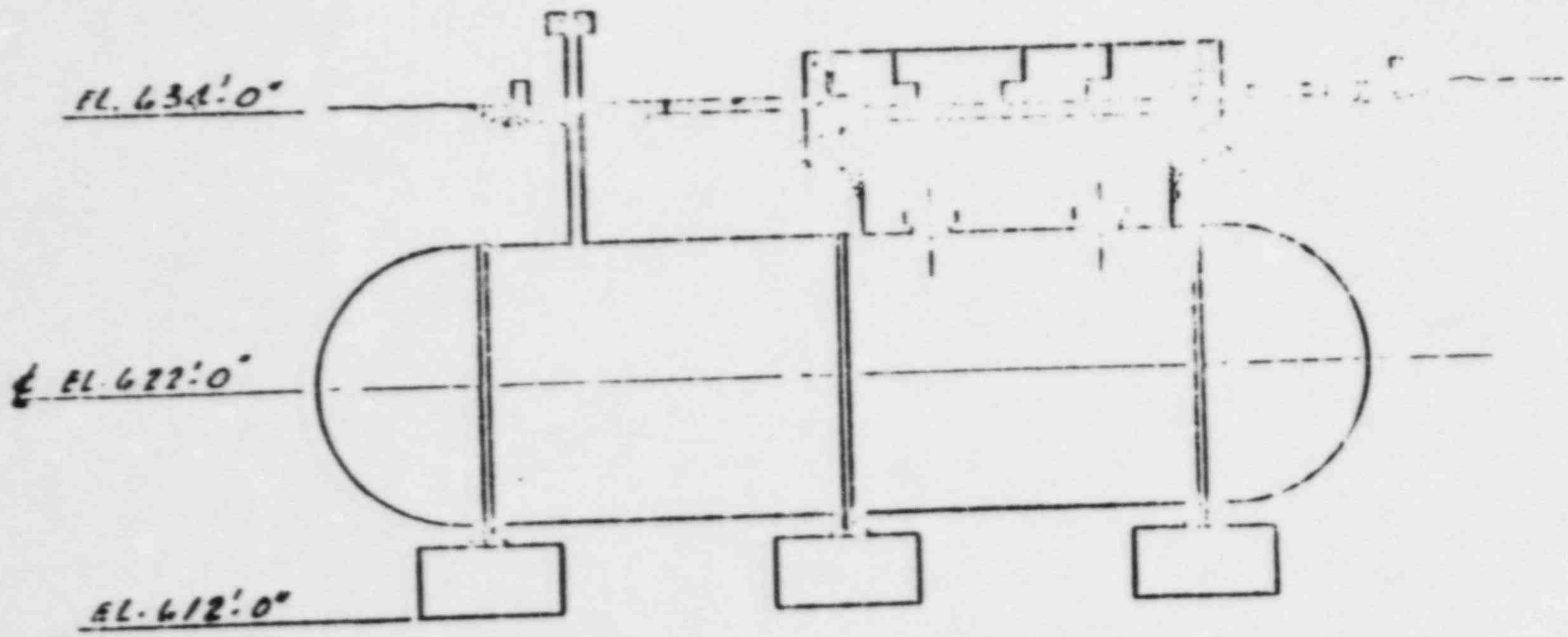

BORATED WATER TANKS
RADWASTE BLDG
AUXILIARY BLDG
REACTOR BLDG UNIT 1
REACTOR BLDG UNIT 2
TURBINE BLDG
DIESEL GENERATOR BLDG
CONDENSATE TANKS
LEGEND

FIG. 3.1.1



ELEVATION

EMERGENCY DIESEL FUEL OIL
STORAGE TANKS (Q)

FIG. 3.4.1

G-0817-04

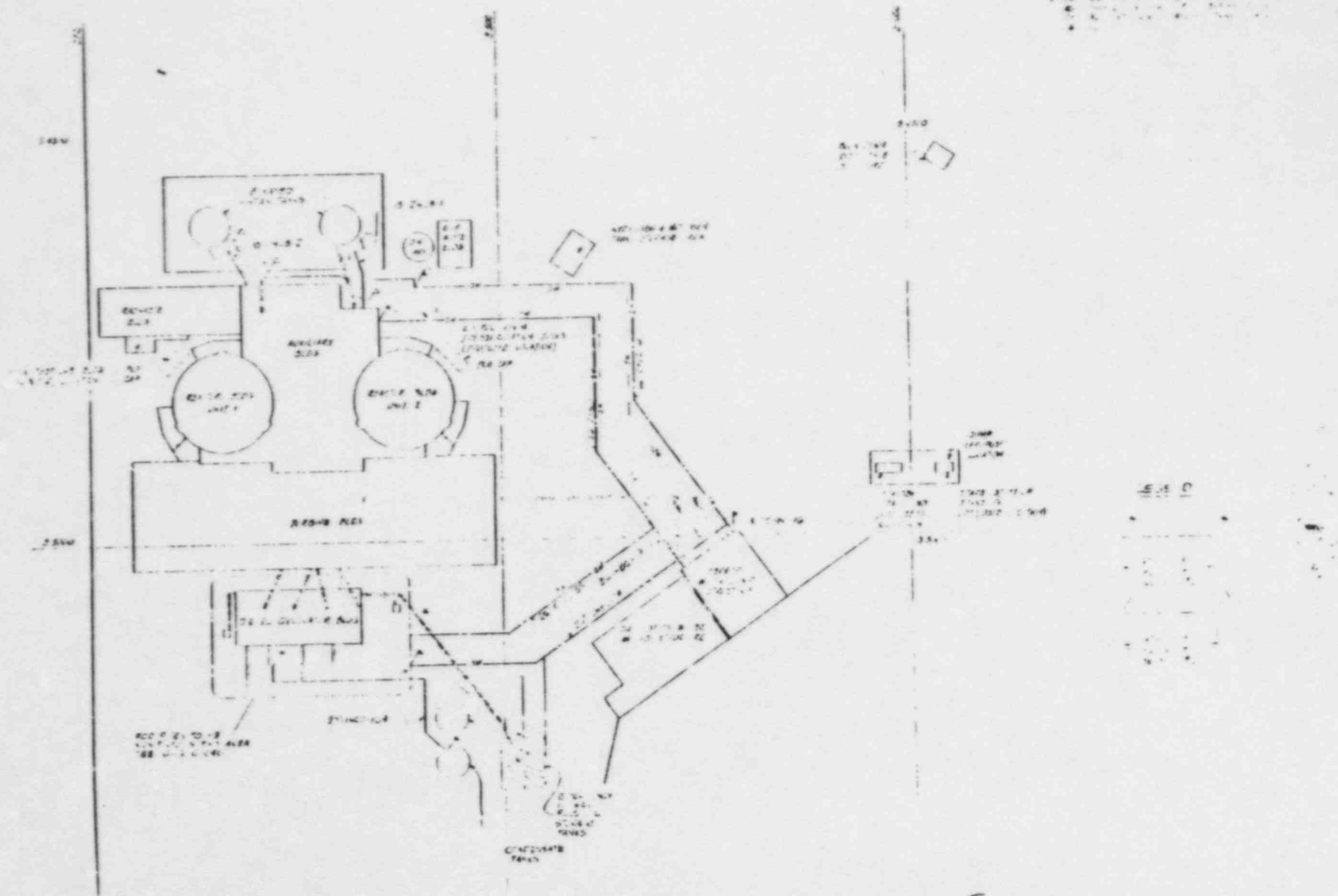
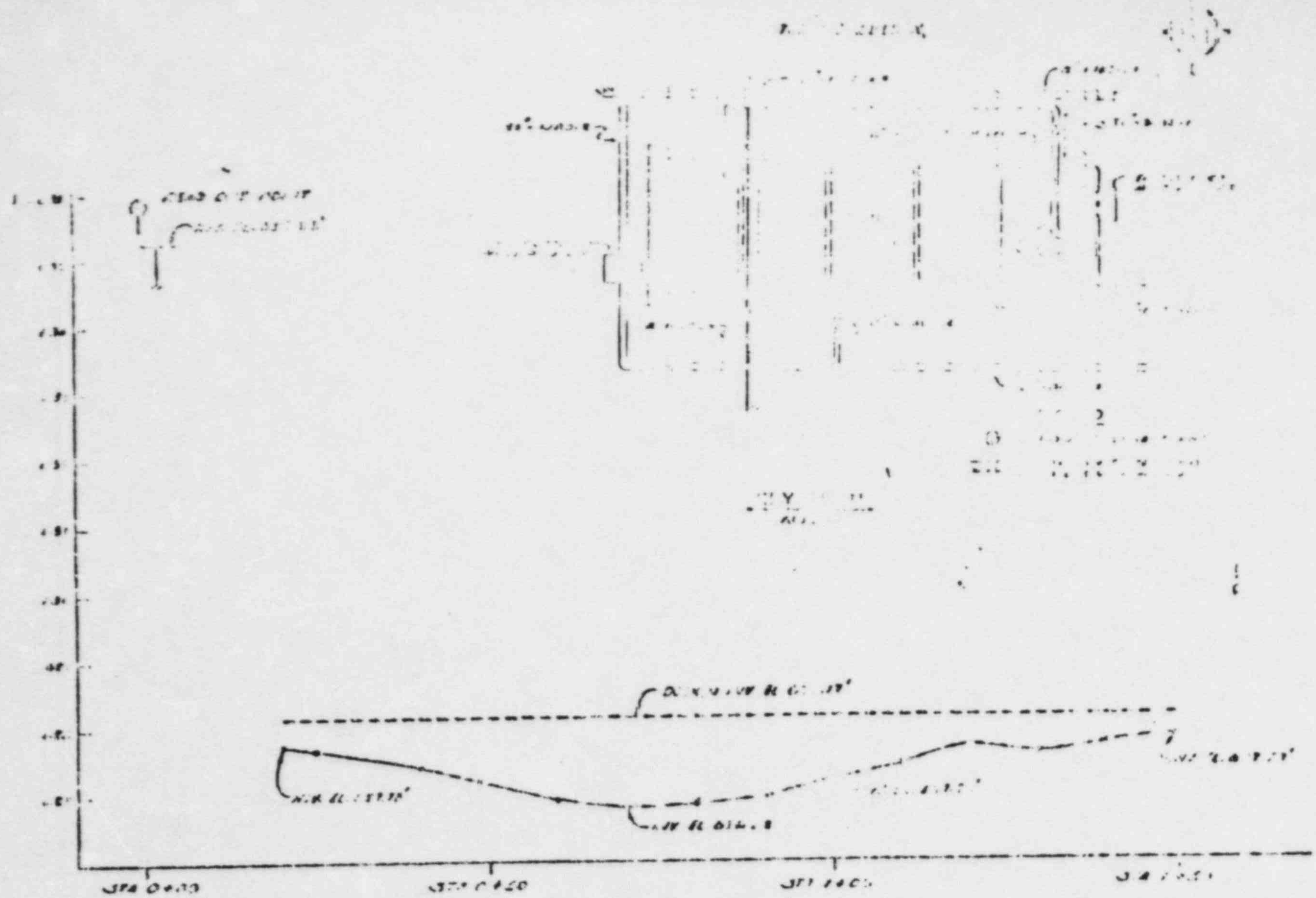


FIG 3.1.1



PROFILE

[illegible] Structural Backfill C-[illegible] 30

4

11:45 a.m.
4/19/74

CAHunt and/or ~~[illegible]~~:

Would you be availatle this afternoon to get involved in a real hot technical approval requirement for Q-List material which is on the Critical Path right now of construction at the site? If so, I would like you to review the facts with Don Sibbald (80016 or -25) at the site and then get together with John Hink so that we can get a proper approval of the material that engineering would approve the Field to use.

MPHanson

jmb

Rixford — J. Hink

[illegible]: structural backfill

[illegible] Work – not included [illegible] C-2[illegible]
Structural backfill to be done by Bechtel Proced.
C-301 [illegible] [illegible] [illegible]
[illegible] material + compaction.
Field can not find specified material.
C-301 written with spec C-[illegible]

1. [illegible] included
2. [illegible] not included
Soil testing not incl.
3. ASTM [illegible]
4. [illegible] Standard
Materials D 422-63 Gradation
D 2049-69 [illegible]
D 2216-71 Moisture [illegible]
5. Backfill
Wall [illegible],
Requires much inspection

Cohesionless free draining
[illegible] -
[illegible] backfill

5.2 Placement clean

subgrade ok

No large lumps clods

No frozen surface + no frozen soil

12"

5.3 Power tampers - size required & adequate compaction.

5.4 90% Relative Density

5.5 Testing

frequency 1 in 500 CY large areas

1 in 100 CY

CAH:

Found attached on my desk after lunch Friday

Talked to Don Sibbald who told me that Bechtel is currently using lean concrete as backfill under the extended portions of the Aux Bldg at a cost of about $20/yd³ + labor and equipment. About 150 to 200,000 yd³ are involved.

Sibbald told me that Bechtel had a procedure (C-301 Rev 0 Dated 10-10-73) specifying the gradation of backfill material thinking it was available on site. The material on site will not meet that spec and since no Tech Spec had ever been made up to purchase, Bechtel went to concrete. Sibbald has prices for offsite material to meet spec at delivered cost of about $3.25/yd³. Material on-site can be supplied at a cost of about $0.75/yd³ that comes close to meeting original spec and it was Sibbald's understanding that the material would be useable if the spec were changed slightly. This is Q-List material. Sibbald and MPH were asking for verbal approval of the gradation of C-- So a procurement spec could be ~~made~~ up to purchase off-site material. Sibbald would like Bechtel to buy enough to keep the job moving while the on-site material is checked to see if it could be used

Called John Hink but he was arranging a conference call with Sibbold, Hink, Bechtel Soils and myself to resolve.

Results of Conference call: Sibbold not included

C-301 had been prepared assuming on-site availability
Gradation specifies very low fines content (passing 200) to avoid Liquifaction probabilities. On-site material runs about 20% passing 200

C-211 is being prepared based on C-301 to enable purchase of off-site material. Spec will not be ready until Monday after J. Allen returns. Allen was orig. of ~~author~~ soils engineer on project.

Hink will call again monday to clean up
I told Hink I felt ~~the~~ cost differences appear to make it preferable to use sand even if it had to be imported

Hink also would like to arrange a meeting with you at A² to review the PMF report prior to a meeting with the AEC coming up shortly

April 25 or 26

Called Sibbold after talking to Hink. His question was why Bechtel had waited till now to reject on-site material. Feels Bechtel sits on a problem until it has to be resolved on a crash basis usually at considerable cost to CP or CP is accussed of holding up the project.
At 4:00 pm 1300 yd³ of concrete had been poured and still pouring. He (Sibbold) would like problem resolved as early monday as possible as cost are going up fast.

Gradation for Structural Backfill as received by telephone

size	% retained fine	coarse
1"		0
#4		25%
#10	0 %	50%
#40	40%	95%
#200	95%	