3/03

RECORD OF TELEPHONE CONVERSATION

DATE: March 8, 1982, 3:30 pm   PROJECT: Midland

RECORDED BY: Joseph D. Kane   CLIENT: ____________

TALKED WITH:

Bechtel	CPC	GEI	NRC
J. Anderson	T. Thruvengadam	S. Poulos	J. Kane
M. Das Gupta	K. Razdan		

ROUTE TO: INFORMATION

G. Lear
L. Heller
D. Hood
F. Rinaldi
S. Poulos
H. Singh
R. Landsman
J. Kane

MAIN SUBJECT OF CALL: ADOPTED SOIL SPRING STIFFNESSES USED IN DESIGN OF AUXILIARY BUILDING UNDERPINNING AND START OF PHASE 2 CONSTRUCTION

ITEMS DISCUSSED:

1. Attachments 1 and 2 to this telephone record provide the design cases and soil spring stiffnesses adopted by Bechtel as soils input in their structural analysis of the Auxiliary Building. The values of stiffness also on Attachment 2 under the column labeled NRC are the results of extensive discussions between NRC Consultants, S. Poulos, GEI, H. Singh, COE and J. Kane, NRC and represent the staff and its Consultants determination of the range of reasonable stiffness values which should be considered in design. The NRC values had been provided to Bechtel via telephone on March 5, 1982 as committed to by the Staff in the meeting of February 26, 1982 in Bethesda.

   The NRC recommended value of 70 KCF for the Main Auxiliary Building versus the Applicant's adopted 30 KCF for Case 2 is important because this difference has the potential to affect settlements which are to be tolerated during underpinning. Allowable settlements using the stiffness of 30 KCF had been provided on February 26, 1982 by M. DasGupta of Bechtel Corp.

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2. Following considerable discussion on NRC recommended stiffness values (in both March 5 and March 8 telephone calls), Consumers expressed a willingness to use these values in their structural analysis but indicated the time needed to complete the required computer runs would impact their Phase 2 construction plans. As an alternative, J. Kane suggested that Phase 2 work be subdivided into two parts, the initial one beginning with work which would not affect the EPA and Control Tower area and the second part beginning after the analysis using the NRC recommended stiffness values had been completed by CPC and the results evaluated by the NRC staff. An acceptable line of demarcation between these two portions of Phase 2 work was tentatively identified as column lines 2.5 and 10.5 on the Construction Sequence drawing provided for the underpinning work at the February 3-5 design audit. These lines, respectively, are sufficiently west and east of the EPA and Control Tower to conclude that these structures would be unaffected by underpinning operations permitted by this initial portion of Phase 2 work.

3. Consumers agreed to provide a letter to NRC giving details which would permit the Staff to fully understand what work would be performed under this initial portion of Phase 2 work.

4. The following comments were given to Consumers concerning the monitoring plans during underpinning of the Auxiliary Building.

   a. Drawing C-1493(Q), "Monitoring Matrix," should be updated and values provided in the tolerance criteria column for staff concurrence before any portion of Phase 2 work is started.

   b. Sheet 8 of M. DasGupta's presentation on February 26, 1982 does not agree with previous drawings provided (Drwgs. C-1490 (Q) and C-1491 (Q)). Corrections in proper labeling of the deep seated bench mark locations on Sheet 8 and on Sheet 10 are needed and should be provided to the NRC.

   c. NRC expressed a concern for measurement of horizontal movement between the EPA and the Turbine Building and between the Control Tower and the Turbine Building during underpinning operations and suggested three monitoring devices be installed. One device at the top of each wing of the EPA's and one at the top of the Control Tower was recommended. Consumers responded that they were now planning to place instruments at those locations in response to questions raised by ASLB but had not yet updated the monitoring locations on Drawings C-1490(Q), C-1491(Q) and C-1493(Q). The Staff indicated that criteria on tolerable relative horizontal movement for these instruments should be established and furnished on the Monitoring Matrix drawing along with the basis for these limits.

   d. As previously discussed at the February 26, 1982 meeting in Bethesda, the Staff anticipates a submittal by Consumers identifying the acceptance criteria for the strain gages to be placed at El.659 on the Auxiliary Building.

5. Consumers indicated that the six deep seated bench mark instruments located on Sheet 8 of M. DasGupta's presentation will be in operation before beginning Phase 2 work. Installation of the additional instruments at top of the EPA's and Control Tower and the strain gages at El 659 and the results of the structural analysis using NRC recommended stiffness values are to be completed before the second portion of Phase 2 work is started.

6. J. Kane indicated that subdivision of Phase 2 underpinning work into two portions is subject to the approval of NRC Project Management and Structural Engineering Branch. It was also indicated that other conditions which could affect the start of Phase 2 work may be identified by the Staff. The original intent of this telephone conference call was to discuss soil spring stiffnesses but was not intended to address the start of Phase 2 work.

SOIL SPRING STIFFNESSES

Cases Considered

1. Normal Soil Springs - Springs used to represent subgrade for analysis of structure for FSAR loading conditions. (A subcase of this is the seismic condition).

2. Existing Condition - Springs used to represent subgrade for analysis of existing state of stress in the structure.

3. Long Term Settlement Condition - Springs which represent the behavior of the structure due to secondary consolidation of the structure after lock-off.

The springs for Case 1 are based on settlement data obtained since 1977 and the load increment added during that time. For the seismic subcase the springs are based on the stiffness used in the seismic model.

For the second case (existing condition) the springs are computed at the center of each area using elastic half space theory and assuming a flexible footing.

For the long term settlement case the springs are computed from the estimated settlement after lock-off and the estimated loads. There are two subcases which were considered: 3a) Where the underpinned areas settle more than the main auxiliary building; and 3b) where the main auxiliary building settles more than the underpinned areas.

Design Conditions	UNIT SPRING STIFFNESSES (KCF) — BECHTEL			NRC		
	E.P.A	C.T.	M.A	E.P.A.	C.T.	M.A.
Case 1 Normal Soil Springs	180	180	80	Acceptable to NRC		
Case 2 Existing Condition	17	18	30	Acceptable to NRC		70
Case 3(a) Long Term Settlement	410	350	1,160	180	240	580
Case 3(b) Long Term Settlement	160	350	230	Acceptable to NRC		

E.P.A. – Electrical Penetration Area
C.T. – Control Tower
M.A. – Main Auxiliary Building

Attachment 2

Feb. 83

- The attached two boring logs (for MP-10 & ME-27B) were the borings where loose sands were encountered when drilling to install the FREEZE WALL. This matter was discussed during Dr. Wood's testifying at the November 1982 ASLB hearing session on liquefaction and is discussed in a Feb 7, 1983 letter to the ASLB from CPC.

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Midland Units 1 and 2	7220	1 of 2	MP-10

SITE	COORDINATES		ANGLE FROM HORIZ.	BEARING
West of Administration Building	S4942.6	W7.74	90°	NA

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH
4-27-82	4-27-82	Mergentime/Moretrench	Mobile B-62	3-7/8"	NA	NA	61.5'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING	GROUND EL.	DEPTH/EL. GROUND WATER	DEPTH/EL. TOP OF ROCK
NA	NA	11	635.6	634.0	33.9'/600.1	NA

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY
140 lbs/30 in	1"/61.5'	R.J. Kelleher

SAMPLE TYPE AND DIAMETER	SAMPLER ADVANCE / LENGTH CORE RUN	SAMPLE RECOVERY / CORE RECOVERY	SAMPLE BLOWS 6" / PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	ELEVATION	DEPTH	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
							634.0	0		0-9.0' No samples	0-8.0' Drilled with a 4" nom. O.D. continuous flight auger. 4" I.D. surface casing set. 8.0-61.5' Rotary drilled with a 3-7/8" tricone roller bit and recirculating Johnson revert drilling fluid.
SS 2"	18"	11"	4	2	2	2	625.0	9	1	9.0-12.5' Sand, light brown to brown, very loose, very fine- to fine-grained, trace fine gravel. (SP) (Fill)	
							621.5	12.5		12.5-22.0' Concrete. (Fill)	
							612.0	22		22.0-45.0' Sandy Clay, gray, mottled, hard, very fine- to fine-grained sand, some fine gravel. (CL) (Fill)	
SS 2"	18"	18"	35	10	17	18		25	2		
SS 2"	18"	0	44	24	20	24		30	3		30.0-31.5' No sample recovery.
SS 2"	18"	18"	35	8	18	17			4		
							599.0	35			5-5-82

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER

SITE		HOLE NO.
West of Administration Building	Revision 14 12/82	MP-10

4/83

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Midland Units 1 and 2	7220	2 OF 2	MP-10

Sampler Type and Diameter	Sampler Advance Length Core Run	Sample Recovery Core Recovery	Sample Blows "N" / Percent Core Recovery	Penetration Blows 1st 6"	2nd 6"	3rd 6"	Elevation	Depth	Sample	Description and Classification	Notes on: Water Levels, Water Return, Character of Drilling, etc.
							599.0	35			
SS 2"	18"	2"	29	8	11	[illegible]			5	35.0' Boulder	
SS 2"	18"	6"	19	6	8	11			6		
SS 2"	18"	10"	11	5	5	6		40	7		
SS 2"	18"	7"	23	6	11	12	589.0	45	8	45.0-57.0' Sand, brown to gray, medium to very dense, very fine- to fine-grained, trace clay and silt. (SP) (Lacustrine)	Fill / Lacustrine
SS 2"	8"	8"	100+	92	100/2"	-		50	9		
SS 2"	12"	12"	100+	26	100	-		55	10	55.0-57.0' Increased amounts of clay and silt.	
							577.0	57		57.0-61.5' Silty Clay, gray, hard. (CL) (Lacustrine)	
SS 2"	18"	18"	97	24	37	60	572.5	61.5	11		
										Bottom of boring at 61.5' Ground water observation well installed. See observation well construction summary.	

SS = SPLIT SPOON; ST = SHELBY TUBE
D = DENNISON; P = PITCHER; O = OTHER

SITE		HOLE NO.
West of Administration Building	Revision 14 12/82	MP-10

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Midland Units 1 and 2	7220	1 of 2	ME-27B

SITE	COORDINATES		ANGLE FROM HORIZ.	BEARING
SE of Turbine Bldg East of Unit 2 Transformer	S5062.35	E524.39	90°	NA

BEGUN	COMPLETED	DRILLER	DRILL MAKE AND MODEL	HOLE SIZE	OVERBURDEN (FT.)	ROCK (FT.)	TOTAL DEPTH
4/15/82	4/15/82	Mergentime/Moretrench	Mobile B-61	3-7/8"	NA	NA	59.5'

CORE RECOVERY (FT./%)	CORE BOXES	SAMPLES	EL. TOP OF CASING	GROUND EL.	DEPTH/EL. GROUND WATER	DEPTH/EL. TOP OF ROCK
NA	NA	11	635.5	634.0	Not Determined	NA

SAMPLE HAMMER WEIGHT/FALL	CASING LEFT IN HOLE: DIA./LENGTH	LOGGED BY:
140 lbs/30 in	3"/49.5'	D.J. Kelleher

SAMPLE TYPE AND DIAMETER	SAMPLER ADVANCE / LENGTH CORE RUN	SAMPLE RECOVERY / CORE RECOVERY	SAMPLE BLOWS / PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	ELEVATION	DEPTH	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
							634.0	0		0-8.0' No samples	0-8.0' Drilled with a 4" nom. O.D. continuous flight auger. 4" I.D. surface casing set to 7.5 ft.
SS 2"	18"	13"	18	4	10	8	626.0	8	1	8.0-13.0' Sandy Clay, light brown to brownish gray, mottled, very stiff, trace fine gravel.(CL) (Fill)	8.0-59.5' Rotary drilled with a 3-7/8" tricone roller bit and recirculating Johnson Revert drilling fluid.
SS 2"	18"	8"	4	2	2	2	621.0	13	2	13.0-22.5' Sand, brown to gray, very loose, very fine- to coarse-grained, poorly sorted, subrounded to subangular, some gravel.(SP) (Fill)	
SS 2"	18"	11"	3	2	2	1			3		
SS 2"	18"	8"	12	4	5	7	611.5	22.5	4	22.5-32.5' Sandy Clay, gray, mottled, stiff, fine- to coarse-grained sand, some fine gravel.(CL) (Fill)	
SS 2"	18"	5"	26	14	15	11			5		
							601.5	32.5		32.5-57.0' Sand, light brown to gray, very dense, very fine- to fine-grained, well sorted. (SP) (Lacustrine)	Fill Lacustrine
SS 2"	18"	8"	68	16	28	40	599.0	35	6		

SS = SPLIT SPOON; ST = SHELBY TUBE; D = DENNISON; P = PITCHER; O = OTHER	SITE		HOLE NO.
	East of Unit 2 Transformer	Revision 14 12/82	ME-27B

BORING LOG

PROJECT	JOB NO.	SHEET NO.	HOLE NO.
Midland Units 1 and 2	7220	2 OF 2	ME-27B

SAMPLER TYPE AND DIAMETER	SAMPLER ADVANCE / LENGTH CORE RUN	SAMPLE RECOVERY / CORE RECOVERY	SAMPLE BLOWS "N" / PERCENT CORE RECOVERY	PENETRATION BLOWS 1ST 6"	2ND 6"	3RD 6"	ELEVATION	DEPTH	SAMPLE	DESCRIPTION AND CLASSIFICATION	NOTES ON: WATER LEVELS, WATER RETURN, CHARACTER OF DRILLING, ETC.
							599.0	35			
SS 2"	8"	8"	100+	50	100/ 2"	-			7		
								40			
SS 2"	6"	6"	100+	100/ 6"	-	-			8		
								45			
SS 2"	5"	5"	100+	100/ 5"	-	-			9		
								50			
SS 2"	18"	13"	150	58	60	90			10		
								55			
							577.0	57		57.0-59.5' Silty Clay, gray, hard. (CL) (Lacustrine)	
SS 2"	18"	18"	50	16	22	28	574.5	59.5	11		
								60		Bottom of boring at 59.5' Construction dewatering well installed. See pumping well construction summary.	

Revision 14
12/82

SS = SPLIT SPOON; ST = SHELBY TUBE
D = DENNISON; P = PITCHER; O = OTHER

SITE: East of Unit 2 Transformer

HOLE NO.: ME-27B

5/B3

J. Kane
Rec'd 4/19/82
from R. Huston

Consumers Power Company

James W Cook
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April 19, 1982

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

MIDLAND PROJECT
MIDLAND DOCKET NO 50-329, 50-330
SUMMARY OF SOILS-RELATED ISSUES
AT THE MIDLAND NUCLEAR PLANT
FILE: 0485.16, 0485.18 SERIAL: 16629
ENCLOSURES: SUMMARY OF SOILS-RELATED ISSUES
AT THE MIDLAND NUCLEAR PLANT

As a result of recent discussions between the NRC Staff management and Consumers Power Company management, it was concluded that a summary report addressing all of the soils-related issues at the Midland Nuclear Plant would be beneficial in completing the Staff's extensive review of the remedial actions proposed with regard to these issues. The enclosed report is a technical summary which provides a history of the soils problem at the Midland plant and a discussion of the design and construction details concerning the remedial measures for the diesel generator building (DGB), auxiliary building, service water pump structure foundation, permanent dewatering system, and underground utilities. The quality assurance program for the underpinning activities is also discussed. Finally, the enclosed report presents the status of design, licensing, and construction of the remedial activities for the various affected structures and utilities on the Midland site.

It is our expectation that this report will serve several purposes. Our objective in providing this technical report is to summarize the soils-related remedial measures for use in the NRC's staff management review and as an introduction to this topic for the Adivsory Committee on Reactor Safeguards (ACRS) Subcommittee.

We believe that this report, together with all the other exhaustive soils-related information provided to the NRC Staff, should assist the Staff in completing its review, issuing a Safety Evaluation Report (SER) on the soils remedial actions and in providing its concurrence on remaining items of soils-related construction. In further support of this continuing effort, we are providing by separate correspondence reference document tabulations of the detailed information available to the Staff. These tabulations of the

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reference information available to the Staff are arranged to correspond to the areas of review identified in those Standard Review Plans pertinent to the Midland soils issues.

James W. Cook

JWC/RLT/mkh

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
DCFischer, ACRS, w/a (6)
SGadler, w/o
JHarbour, ASLB, w/o
GHarstead, Harstead Engineering, w/a
RWHernan, NRC, w/a
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
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RBLandsman, NRC Region III, w/a
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JPMatra, Naval Surface Weapons Center, w/a
WOtto, Army Corps of Engineers, w/o
WDPaton, Esq, w/o
SJPoulos, Geotechnical Engineers, w/a
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o

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SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

April 19, 1982

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SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

EXECUTIVE SUMMARY

Consumers Power Company, the applicant for an operating license for the Midland Nuclear Plant, has been engaged in a comprehensive program to resolve soils-related issues identified during plant construction.

Excessive settlement of the diesel generator building (DGB), resulting from inadequately compacted plant fill, was identified in July 1978. Since then, extensive exploratory tests and studies have been conducted to determine the exact cause and extent of this problem. Subsequently, other soils-related problems have been identified.

In addition to the soils-related issues, remedial actions are necessary to correct a problem affecting the two borated water storage tank (BWST) foundations. Failure of the design to consider nonuniform loading led to overstressing during a load test. This condition was aggravated by the soils conditions.

Together with the architect-engineer, Bechtel Associates Professional Corporation, and numerous other renowned consultants, the Applicant has performed comprehensive and detailed analyses in order to develop satisfactory remedial actions for identified problems.

Throughout this process, the Applicant has maintained an extensive dialogue with the NRC staff through technical reports, responses to questions, meetings, and direct presentations. Concurrence has been received on many of the analyses and remedial design concepts while others are still under review.

The status of soils-related issues as of April 1982 at the Midland Nuclear Plant can be summarized under the following programs:

- The settlement problem of the DGB has been essentially resolved by preloading the area in and around the building to achieve accelerated consolidation of plant fill which supports the building.

Not yet resolved stresses due to settlement

- In Adequately compacted fill under portions of the auxiliary building and feedwater isolation valve pit (FIVP) will be resolved by constructing underpinning under the auxilary building and replacing the existing backfill under the FIVP. When completed, the new foundations will carry the loads to the undisturbed natural soils underlying the site. These new foundations will meet newly established seismic design criteria promulgated by the NRC.

- Inadequately compacted fill under the overhang portion of the service water pump structure will be resolved by constructing underpinning similar to that under the auxiliary building.

- Design problems associated with the BWST foundation will be resolved by the preload of the valve pit, which has been completed, and reinforcing the old ring beam with a new concentric ring beam.

- Potential liquefiable pockets of backfill supporting some Seismic Category I structures and utilities will be resolved by providing a permanent plant dewatering system.

- The adequacy of all underground Seismic Category I utilities will be ensured by a variety of actions ranging from acceptance of existing facilities to complete replacement.

- Concerns relating the the quality assurance program for the unique underpinning have been resolved by developing a special quality assurance plan for this work.

This report provides a brief history of the soils-related problems at the Midland plant and presents design and construction details of the remedial measures developed to address these problems. It is intended for use in NRC management reviews and as an introduction to this topic to the Advisory Committee on Reactor Safeguards.

SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

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SUMMARY OF SOILS-RELATED ISSUES

AT THE

MIDLAND NUCLEAR PLANT

BACKGROUND

A construction permit for Midland Plant Units 1 and 2 was issued by the Atomic Energy Commission on December 15, 1972. Soils-related problems were first identified in July 1978 when the settlement monitoring program detected excessive settlement of the diesel generator building (DGB). The building had settled 3.5 inches at the point of greatest settlement, compared to design predictions of 3 inches for the 40 years of expected plant operation. Shortly thereafter, the Applicant verbally reported the matter to the NRC site inspector, and formally reported it under 10 CFR 50.55(e) in September 1978.

The plant design called for the placement of foundations for certain structures and portions of others on approximately 30 feet of compacted fill material overlying the natural material of the site. Specifications governing the placement and compaction of fill material required typical controls over moisture content, lift thickness, compactive energy, and in situ testing by the traditional soils engineering methods. As was later determined, controls in the areas of both placement and testing were deficient.

Soil placement activities were conducted largely from 1975 to 1977. In August 1977, some settlement was detected for one of seven foundation grade beams of the administration building. This is a nonsafety-related structure that houses plant offices. The settlement was investigated by conducting test borings in the near vicinity and by load testing the remaining grade beams. In addition, two borings outside the immediate area of the failure were taken. The results of the investigation, which was completed in September 1977, demonstrated adequately compacted soils, apart from those directly beneath the beam that had settled.

The foundation construction of the DGB, for which construction was started in October 1977, rests entirely on plant fill material. The Applicant's initial response after discovering the settlement problem in 1978 was to halt DGB construction, pending investigation. Drs. R.B. Peck and A.J. Hendron, Jr., renowned soils consultants, were retained.

The Applicant also initiated a soils boring program, which was later extended to the entire site and resulted in over 350 soil borings. The NRC, for its part, initiated an investigation that continued into the early part of 1979.

Based on results of soil boring samples taken from under the DGB, the Applicant concluded that the soil beneath the DGB was inadequately compacted. The consultants recommended in November 1978 that the Applicant "preload" or "surcharge" the structure. This involved placing a 20-foot layer of sand around the perimeter of and within the structure to accelerate settlement, or more accurately, to "consolidate" the fill material. In the consultant's opinion, a significant advantage of the preload process is its self-verifying nature. That is, when the preload is complete and effective, settlements under the structure approach a straight line on a settlement-versus-log-time graph. In addition, excess pore pressures are dissipated, a fact which can be observed directly by piezometer measurements.

VU-GRAPH
4 piez.
rebound
needed

After a through review of the options available, the Applicant elected to institute a surcharge loading program, which subsequently was started in January 1979. In early November 1978, the NRC staff was advised that preloading was the recommended remedial action for the DGB. The staff visited the site in December of that year. Although the staff expressed no opinion at the time, it later objected to the Applicant's actions on grounds that the staff had not been provided adequate acceptance criteria before application of the preload. In the December meeting, the staff indicated that if the Applicant implemented the preload, the Applicant would be proceeding at its own risk.

VU-GRAPH
that demonstrate
Staff concern

In August 1979, results from the preload indicated to the satisfaction of the Applicant and its consultants that the criteria for reaching secondary consolidation had been achieved. Accordingly, the Applicant began removing the surcharge in August 1979. The removal operation was completed within a month.

Meanwhile in 1979, while the preload was in place, the results of an extensive boring program elsewhere on the site showed inadequately compacted soil under the electrical penetration areas of the auxiliary building and under a portion of the cantilevered section of the service water pump structure (SWPS), i.e., the portion of the structure that rests on plant fill. Neither building had undergone unusual or excessive settlement. Nevertheless, the Applicant decided to underpin portions of both structures to obtain adequate predictability of structural behavior under design conditions.

The possibility of liquefaction of inadequately compacted sandy soils during seismic conditions also was studied. Grouting of localized sand pockets was considered. However, the Applicant decided upon a permanent dewatering system, because demonstrating that all sand pockets had been successfully grouted was considered difficult and because a dewatering system was both practical and conclusive.

The NRC staff review of the Applicant's soils proposals was delayed by the Three Mile Island accident. Late in 1979, the NRC staff retained the U.S. Army Corps of Engineers as its consultant. On December 6, 1979, the staff issued an order halting all remedial construction until such time as the Applicant could prove to the staff that its proposed and completed remedial actions were technically sound.

During 1979, the Applicant had responded at length to two sets of 10 CFR 50.54(f) requests. However, the staff did not find the responses adequate. The Applicant requested a hearing and voluntarily agreed not to undertake further remedial construction without concurrence of the NRC staff, although a request for a hearing suspended the effect of the staff order. As a result of the hearing, staff concurrence has been secured on the dewatering system, portions of the auxiliary building underpinning, and certain other work.

In June 1980, the staff, still not assured that the preload had brought about secondary consolidation of the fill under the DGB, requested a series of borings to demonstrate, among other things, that the preload had accomplished its purpose. The staff also asked for borings at other locations, including the cooling pond dike. The Applicant's consultants advised against the borings because they believed errors inherent in this approach would lead to unpredictable results of little or no value. Because the staff believed that the information relied upon by the consultants was ambiguous, the NRC staff maintained its view and the Applicant took the requested borings against the advice of its consultants. Subsequently, the staff has come to believe that the borings confirm the Applicant's predictions of future settlement of the DGB.

VU-C-RHM Plot depth vs. press Also vol test results of se...

The next event of major consequence occurred on October 14, 1980, when the staff changed its position concerning seismic criteria for the Midland site safe shutdown earthquake (SSE). The new staff position, which was announced by a letter, was a departure from criteria approved by the NRC when a construction permit for the plant was issued.

At the staff's request, the Applicant has agreed to revise its underpinning proposals for the SWPS and auxiliary building in order to incorporate this new criteria as a design basis.

The previous underpinning scheme for the SWPS used drilled piles attached to the overhang portion of the structure by corbels. This was found lacking under the heightened seismic loads. A new scheme making use of walls that extended from the structure's original walls to the undisturbed natural material under the cantilevered portion was adopted.

Regarding the auxiliary building, a scheme involving caissons under the electrical penetration area was also abandoned because of increased seismic loads in favor of a wall extending under the electrical penetration area and control tower. The modified schemes were developed in mid-1981 and were presented to the NRC staff in September 1981. The NRC staff has concurred with the concept of the new underpinning schemes.

To resolve the seismic issue raised in the staff's October 1980 letter, the Applicant proposed a site-specific response spectrum (SSRS) for the design of structural remedial work and for a seismic margin analysis of existing structures. The staff has concurred with this proposal. With regard to the auxiliary building underpinning proposal, the staff agreed to conduct its review in four phases to avoid construction delays associated with obtaining staff concurrence. In late 1981, after the staff approved Phase 1, the Applicant started excavations for the access shaft for the underpinning.

not only structural design error

During 1981, the Applicant discovered a problem with the borated water storage tank (BWST) foundations. These foundations, which consist of a concrete ring beam and valve pit, are placed on fill. A structural design error resulted in overstressing the ring beam, creating cracks and the potential for yielding of reinforcing steel. To resolve this problem, the Applicant decided to reinforce the old ring beam with a new concentric ring beam to be constructed after preloading the valve pit. The NRC staff has concurred with this remedial concept.

Because of the widespread nature of the fill problems, the Applicant conducted additional plant fill analyses and proposed remedial measures for underground piping located in plant fill around the site. In some cases, existing pipes were proven adequate by analysis. In other instances, the Applicant opted to excavate and rebed pipes. The NRC staff has concurred with the decision regarding which pipes are to be rebedded. The Applicant has also committed to replace a portion of the piping due to an inability to reach agreement with the NRC staff on the acceptance criteria for that portion of the existing piping.

Hearings have been conducted on some aspects of the soils problem and the resulting remedial work. This includes the auxiliary building, the BWST and its foundation, the cooling pond dike, underground piping, and the proposed SSRS. The NRC staff has conducted extensive reviews into the preload plan and its effect on the DGB. In addition, the staff conducted extensive audits on the SWFS and auxiliary building during early 1982.

Since the inception of the soils issues, the Applicant has provided the staff with substantial information through 10 CFR 50.55(e) reports, responses to 10 CFR 50.54(f) questions, technical reports, and direct presentation in meetings. The Applicant has participated in over 50 meetings with the staff on soils-related issues. The 10 CFR 50.54(f) responses alone occupy over 11 volumes of material.

Because of the complexity of these soils-related issues, a summary of the technical details of the remedial work and the quality assurance program applied to the work are presented in seven parts, as follows:

Part I	Diesel Generator Building
Part II	Auxiliary Building and Feedwater Isolation Valve Pit
Part III	Service Water Pump Structure Structure
Part IV	Borated Water Storage Tanks
Part V	Permanent Dewatering
Part VI	Underground Utilities
Part VII	Quality Assurance

PART I: DIESEL GENERATOR BUILDING

1.0 INTRODUCTION

The diesel generator building (DBG) is a reinforced concrete structure with three crosswalls that divide the structure into four cells; each cell contains a pedestal to support a diesel generator unit. The building is supported on continuous footings that are founded at el 628' and rest on backfill that extends down to approximately el 603' (see Figure I-1).

In July 1978, approximately 60% of the building was completed and the pedestals were already in place. The recorded settlements of the building at that time exceeded those which should be anticipated under normal conditions. It appeared that the building was settling due to the consolidation of the backfill and was supported along the north portion by four electrical duct banks acting as vertical piers and resting on the natural soil below the fill.

The Applicant decided to halt construction while an exploration program was initiated to determine the quality of the backfill. Drs. R.B. Peck and A.J. Hendron, Jr. were retained as consultants to advise on the selection and the execution of any remedial action.

The exploration program confirmed that the backfill did not meet the specified compaction requirements at all points and that the fill consisted of cohesive soil, granular soil, and lean concrete. The backfill ranged from very soft to very stiff for cohesive soil and from very loose to dense for granular soil. At the time of the exploration, the groundwater level ranged from el 616' to el 622', and the cooling pond, located 275 feet south of the building, had water level at approximately el 622'.

2.0 REMEDIAL ACTION

After review of settlement observations and results of an exploration program, it was decided that remedial action was necessary and several options were evaluated. Based on consultants' recommendations, it was decided to surcharge the area within and around the building.

The purpose of the surcharge was to accelerate the settlement so that under the operating loads of the structure future settlement would be within tolerable limits. Furthermore, the procedure would permit a conservative and reliable estimate of the future settlement. Before the surcharge was placed, the duct banks were separated from the building and soil instrumentation was installed (see Table I-1).

Surcharging consisted of placing 20 feet of sand above grade (el 634') with the geometry shown in Figure I-1. The surcharge was added in two principal increments as shown by the idealized load history in Figure I-2. Surcharge was effectively begun on January 26, 1979. At the same time, construction of the remainder of the building was resumed and approximately 94% of the structural dead load was completed by the time the surcharge reached maximum level. The cooling pond level was also raised to el 627'. Removal of the surcharge started on August 15, 1979, when it had been determined that primary consolidation of the soil had been achieved.

The Applicant and its consultants have concluded that the surcharge has consolidated the fill beneath the DGB such that the future settlement can be predicted. The Applicant has included this prediction in a structural reanalysis of the building and concludes the DGB is capable of meeting its design requirements over the operating life of the Midland plant. *

The NRC staff has concurred with the prediction of future settlement. Discussions with the staff on the structural reanalysis of the building are continuing.

3.0 DATA INTERPRETATION - SETTLEMENT PREDICTIONS

Figure I-3 is a typical plot of settlement versus time for a point on the DGB, along with piezometer readings, cooling pond elevation changes, and the idealized surcharge load history. The same settlement data points have been replotted as settlement versus the logarithm of time as shown in Figure I-2. This semi-log plot shows the typical consolidation behavior with primary consolidation completed and the secondary consolidation beginning at approximately 100 days from the start of surcharge placement. This typical behavior permitted extrapolations to be made to forecast the building settlement during its service life under the conservative assumption that the surcharge remains in place for 40 years. Results of this extrapolation are shown in Figure I-4.

Upon surcharge removal, the building showed the expected rebound of about 0.2 inch. Following rebound and until the start of dewatering in September 1980, the building showed a maximum settlement of 0.1 inch. This is less than the range of 0.2 to 0.5 inch which was predicted on the basis of the previously mentioned straight line extrapolation. Following dewatering activities, the building settled 0.4 to 0.5 inch (see Figure I-5) due to lowering the groundwater table from approximately el 620' to el 595' and the resulting settlement of the fill and natural soil. This range is about half of that predicted on the basis of theoretical calculations.

Check basis for this statement

4.0 SOIL EXPLORATION AFTER SURCHARGE

At the request of the NRC, 11 soil borings were drilled in the DGB area during April and May 1981 as a part of additional soil investigation. Details of this investigation program were coordinated with the NRC staff and its consultants, the Army Corps of Engineers. The results of the field investigation and laboratory testing programs were provided to the NRC staff and its consultants.

4.1 SETTLEMENT CALCULATIONS

At the request of the NRC, one-dimensional consolidation tests were performed on the samples to provide an estimate of maximum past consolidation pressure. The maximum past consolidation pressures interpreted from the laboratory tests showed a scatter predictable for consolidation laboratory tests on heterogeneous fill. The data showed some of the interpreted maximum past consolidation pressures were lower than would have been expected after surcharging; a greater number were higher. Based on the assumption that the lower maximum past consolidation pressures interpreted from the laboratory tests demonstrated that parts of the fill had not achieved full primary consolidation under surcharge loading, a settlement analysis was made to estimate future primary consolidation under the DGB loading. This analysis predicted future primary consolidation settlement values ranging from 0 to 0.4 inch. Because this range is on the same order as that measured as a result of dewatering, the settlements predicted by this analysis were replaced with actual measured settlement values shown in Figure I-5. During the meeting with the NRC staff on February 23, 1982, the settlements calculated on the basis of consolidation tests and measured settlements were discussed and the staff concurred with using measured dewatered settlements plus predicted 40-year secondary consolidation settlements to represent future settlements for the structure.

4.2 BEARING CAPACITY

The results of the strength tests on cohesive soils obtained after surcharging provided shear strength parameters required for evaluation of the factors of safety against bearing capacity failure under static and seismic conditions. The factor of safety against a static bearing capacity failure is greater than 5, compared to the minimum acceptable value of 3. The factor of safety against a bearing capacity failure for combined static and earthquake loads consistent with an SSE of 0.12g is greater than 2.7, compared to the minimum acceptable value of 2.

5.0 EARTHQUAKE SETTLEMENT OF SAND

Involved area? SPT results? Method used to estimate settlement

On the basis of standard penetration tests conducted before surcharge, it is estimated that the settlement of sand due to earthquake ground shaking would be about 0.25 inch.

6.0 DYNAMIC PROPERTIES OF BACKFILL

Seismic cross-hole testing was performed at two locations within the DGB during November and December 1979 to determine the shear wave velocity of the fill for seismic analysis. The data showed the shear wave velocity can be represented by a value of 500 ft/sec from ground surface to el 615' and by a value of 850 ft/sec from el 615' to el 600'.

7.0 SURCHARGE EFFECTIVENESS

Figure I-6 presents a comparison between the pressures that existed during surcharge and those expected during the operating life of the structure. This comparison shows that at all depths the pressures that existed during surcharge exceeded those that are expected while the structure is operational. This comparison confirms that the settlements predicted on the assumption that the surcharge remains in place 40 years (see Figure I-4) are conservative in that all loads added after surcharge removal, including those due to permanent dewatering, were less than the surcharge loading at all depths.

8.0 STRUCTURAL REANALYSIS

At the conclusion of the surcharge when program, a structural reanalysis of the DGB was performed. This reanalysis accounted for the actual settlement which had ocurred since the removal of the surcharge, and for the additional settlement predicted to occur over the 40-year life of the plant.

This reanalysis proceeded by defining the acceptance criteria for the structure. These acceptance criteria differ from the acceptance criteria used in the original analysis and design of the structure and set forth in the FSAR only in the addition of four load combinations that include the effect of settlement. These additional load combinations are described in Section 8.1.

8.1 STRUCTURAL ACCEPTANCE CRITERIA

Because of the settlement problem, a structural reanalysis of the DGB was performed in accordance with the structural acceptance criteria which are consistent with FSAR Subsection 3.8.6.3, with settlement effects included as outlined in the response to NRC

Requests Regarding Plant Fill, Question 15 (Revision 3, September 1979). In accordance with an NRC staff request, an additional comparative analysis was performed on the DGB in accordance with the load combinations of ACI 349-1976 as supplemented by Regulatory Guide 1.142.

8.1.1 Diesel Generator Building Analytical Model

The structural reanalysis of the DGB uses a finite-element model. The required load combinations were applied to this model and the resulting forces were investigated for compliance with the structural acceptance criteria. The DGB was modeled as an assemblage of plate, beam, and boundary elements to represent soil.

8.1.2 Structural Adequacy Computations

The final structural reanalysis of the DGB indicated that in no case was the maximum allowable rebar stress exceeded. In nearly 70% of the structure, the tornado load combination produced the largest rebar stress levels. (The largest rebar stress value calculated was 39.15 ksi.)

8.2 LICENSING STATUS

During the meeting of February 24, 1982, the NRC staff, in its review of the testimony being prepared for the public hearings, requested additional analysis of the DGB. In particular, the staff was concerned that settlement stresses induced in the structure prior to and during the surcharge program may be significant. Consequently, an additional analysis is presently being performed to establish rebar stress values which existed prior to surcharge removal.

8.3 CONCLUSIONS

The DGB is a massive, reinforced concrete structure with extensive reserve strength. The structural reanalysis performed on the DGB verifies that the integrity of the structure will be maintained under the most critical load combinations. Based on the analysis performed, it can be stated that the settlement has had minimal effect on the structure, and it can be concluded that the DGB will safely perform its intended function over the operating life of the Midland plant.

9.0 CONCRETE CRACKS

A set of electrical duct banks located beneath the building foundation initially acted to restrain the even movement of the structure during fill settlement. A systematic crack pattern was observed in walls resting on the duct banks. Cracks in walls that do not rest on duct banks are attributable to restrained volume changes during curing and drying of the concrete. Cracks were first mapped after the duct banks were separated from the DGB and prior to surcharge placement. Another crack mapping of the DGB was performed after surcharge removal to acertain the effect of surcharge.

The concrete cracks within the DGB were formally addressed in the response to Question 29 of the NRC Requests Regarding Plant Fill. In this response, the cause and significance of the concrete cracks in all structures were presented. Subsequently, during the NRC structural technical audit of April 1981, further discussion was held concerning the effects of the cracks and the additional rebar stress resulting from the concrete cracks. To evaluate the additional rebar stresses associated with the concrete cracking, a number of analytical approaches have been used and the results forwarded to the NRC in the response to Question 40 of the NRC Requests Regarding Plant Fill. These results indicated that because these stresses are strain-induced secondary stresses, they do not affect the ultimate strength capacity of the cracked member.

In response to an NRC request for a nonlinear, finite-element analysis to evaluate the effects of cracks on the integrity of the DGB, an additional computer analysis of the DGB was performed. This analysis was perfomed using a finite-element program, Automated Dynamic Incremental Nonlinear Analysis (ADINA), which is a three-dimensional, nonlinear program capable of considering concrete crushing, cracking, crack widening, and reinforcement yielding. The east wall of the DGB was selected for the ADINA analysis. A crack was modeled into the east wall, and the ADINA analysis was performed for two governing load combinations. The analysis indicated that the effect of concrete cracks was localized and minor in nature. The results of this ADINA analysis were submitted to the NRC followed by meetings with the NRC staff to discuss these results.

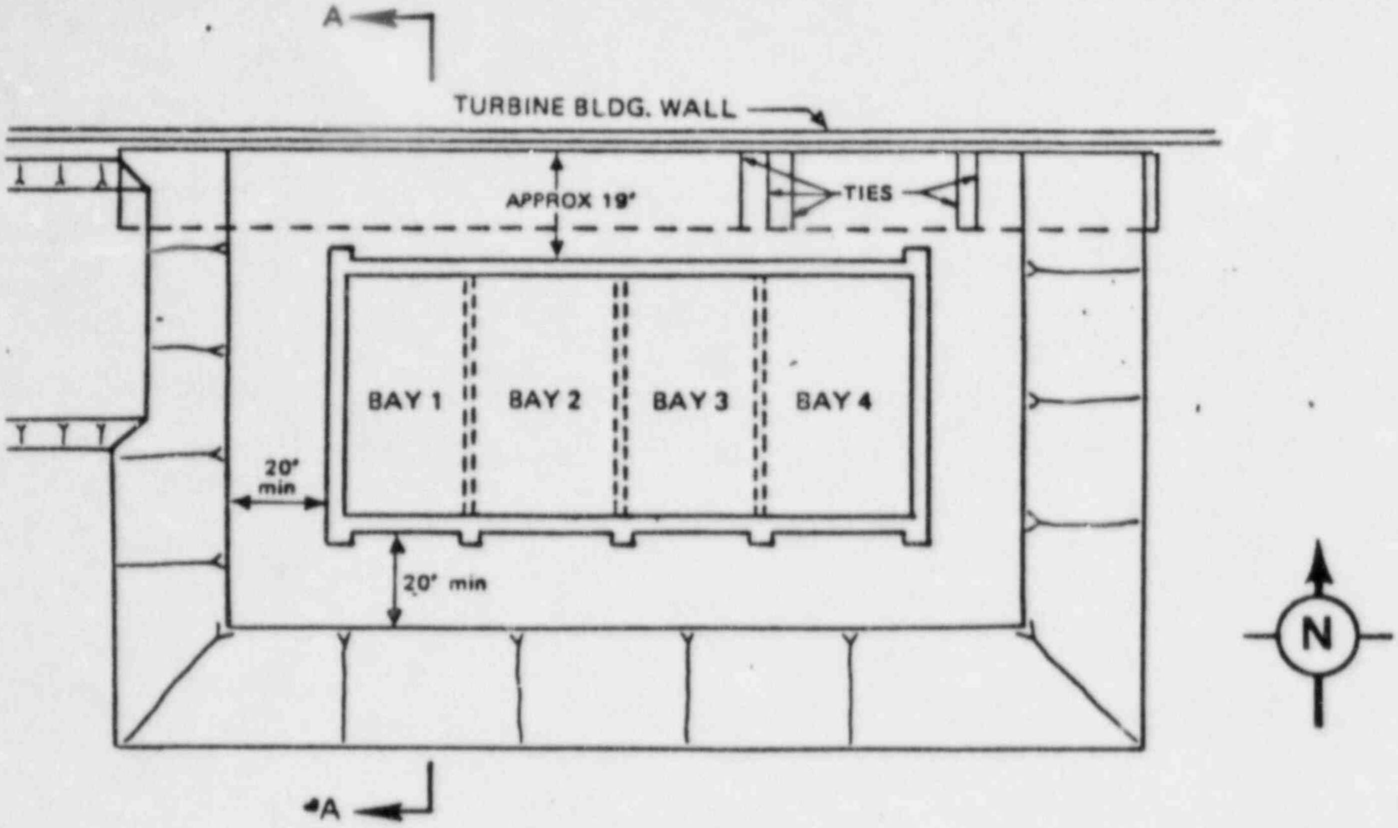
To address additional staff concerns, further evaluation of the existing concrete cracks was performed by Dr. Mete Sozen of the University of Illinois and Dr. W. Gene Corley of Portland Cement Association. The consultants agree that the DGB is capable of withstanding the loads it was initially designed for, despite the existence of concrete cracks. A report addressing the evaluation of cracks by the consultants has been presented to the NRC staff; three meetings have subsequently been held to discuss the crack report. A report on a crack repair program by Portland Cement Association for all cracks in all structures will be submitted to the staff in the near future. Furthermore, crack mapping for the DGB continues at approximately yearly intervals.

A final resolution of the crack issue is still pending with the NRC staff.

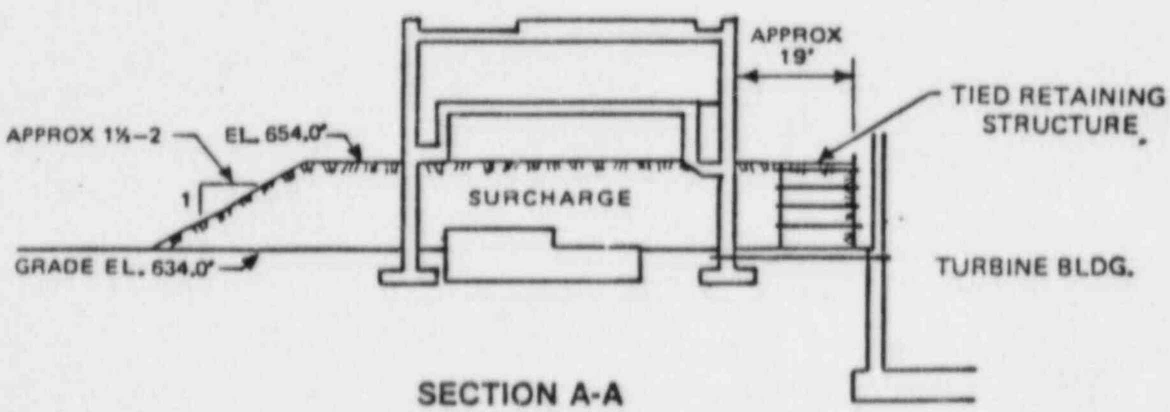
TABLE I-1

DIESEL GENERATOR BUILDING INSTRUMENTATION

Type	Number
Building Settlement Markers	28
Settlement Plates	52
Borros Anchors	60
Deep Borros Anchors	4
Sandex Gages	5
Piezometers	48



PLAN



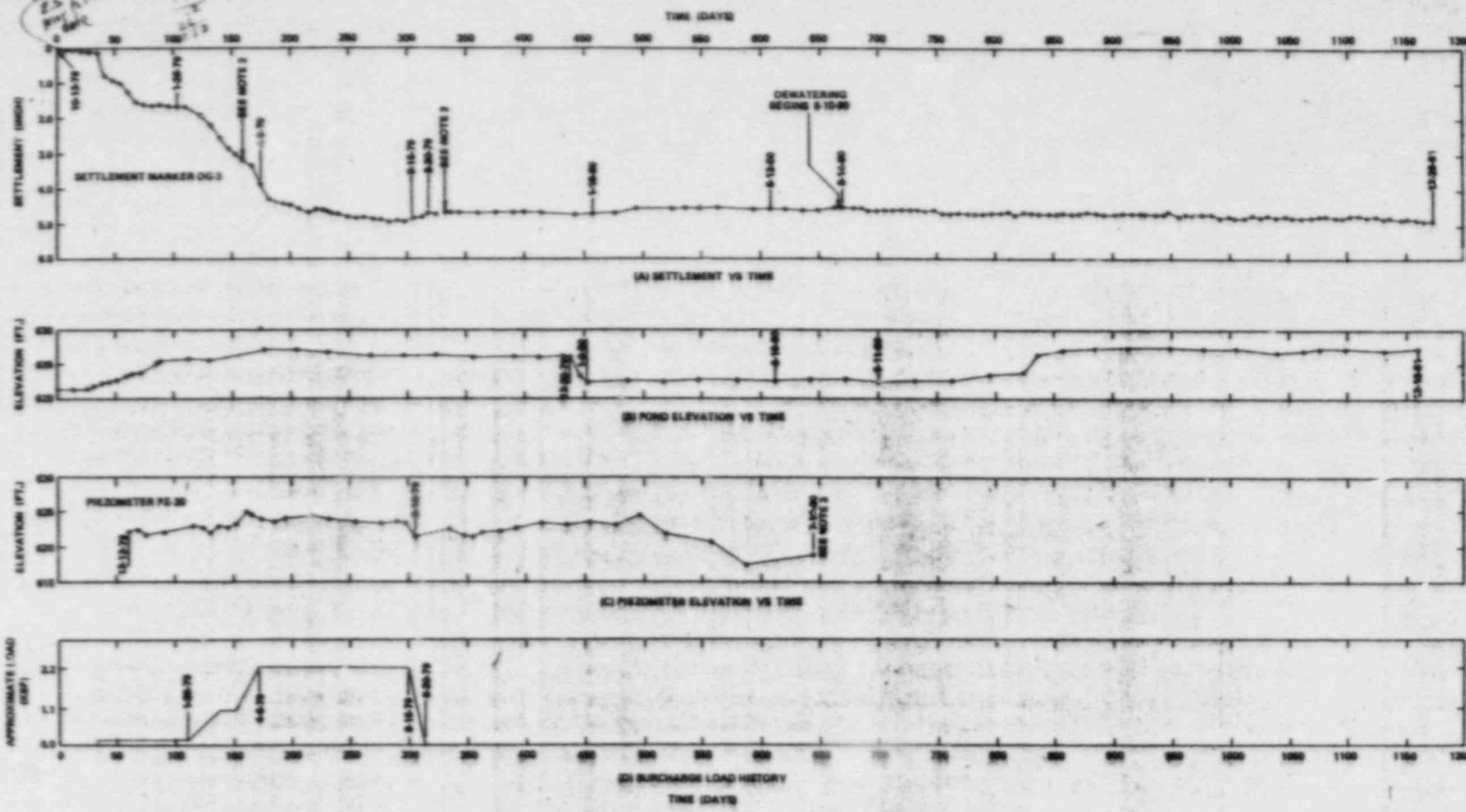
SECTION A-A

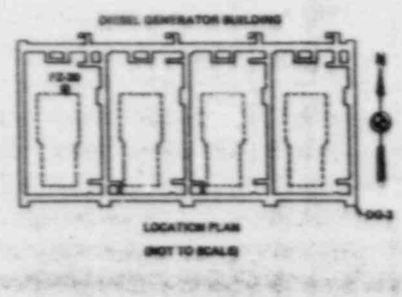
0 50

SCALE IN FEET

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
GENERAL LAYOUT OF SURCHARGE LOAD DIESEL GENERATOR BUILDING
FIGURE I-1

G-1986-41





NOTE:

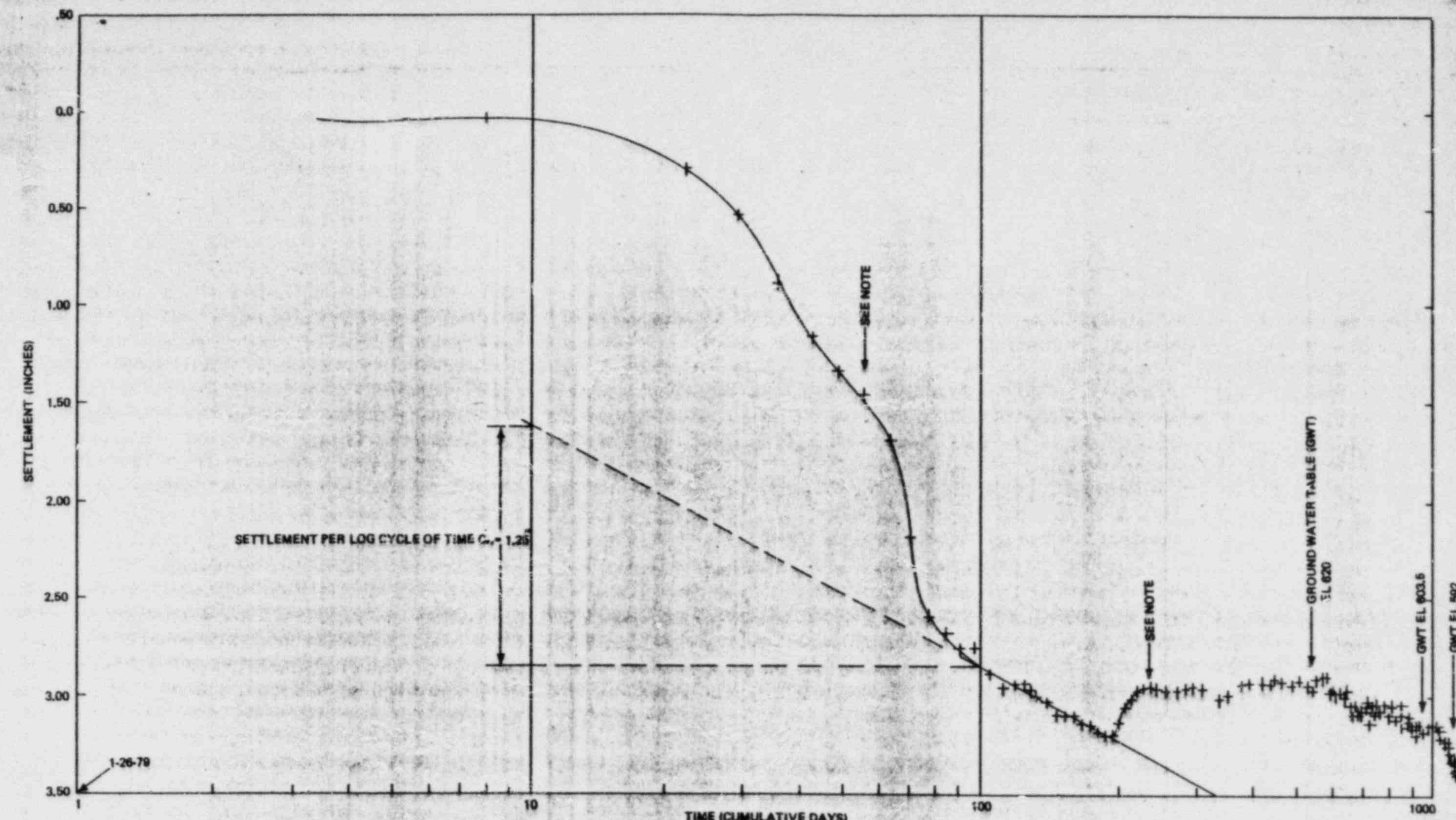
1. [illegible]
2. [illegible]
3. Piezometer destroyed.

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

DIESEL GENERATOR BUILDING TYPICAL SETTLEMENT, COOLING POND LEVEL, PIEZOMETER LEVEL, AND SURCHARGE LOAD HISTORY

FIGURE I-2

Q-1000-30



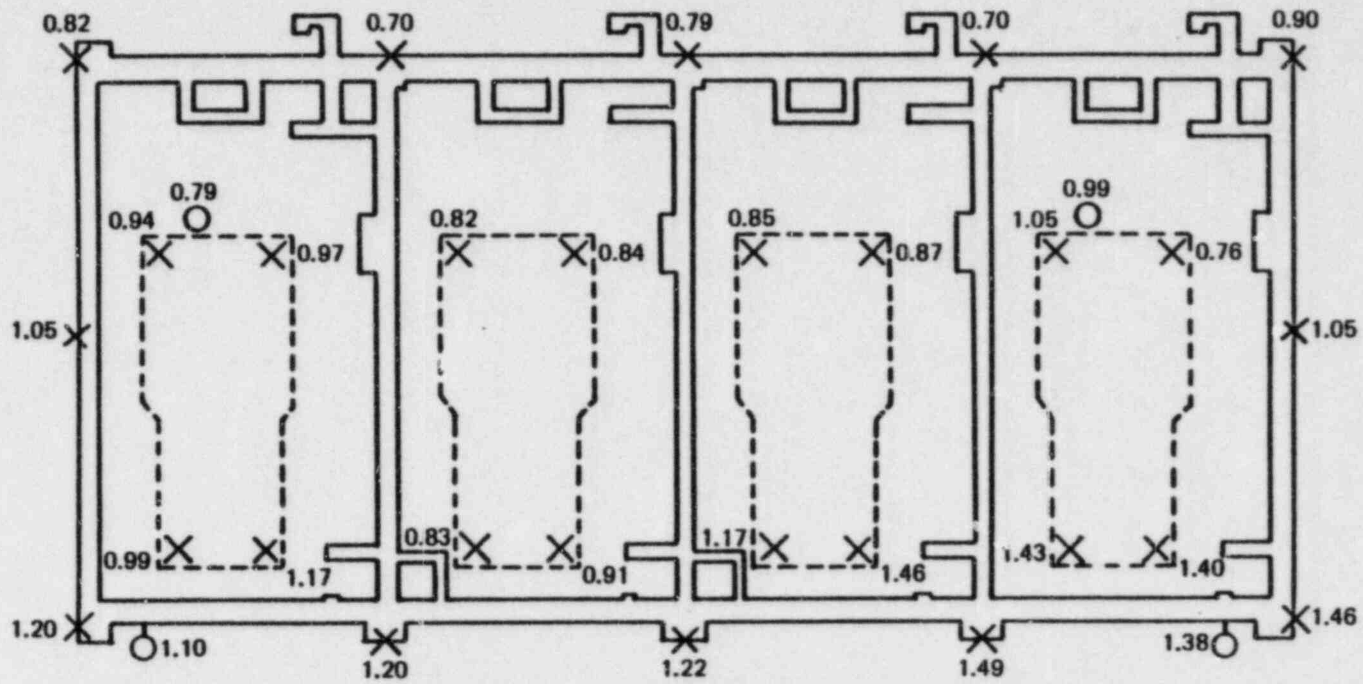
NOTE
The permanent marker could not be monitored from 3-22-79 to 9-14-79 due to surcharge. Temporary markers at elevation 664'-0" ± were used during this period to estimate the settlement of the permanent markers. On 9-14-79 the settlement was again based directly upon the permanent markers.

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

TYPICAL SETTLEMENT VS LOGARITHM OF TIME DURING AND AFTER SURCHARGE (MARKER DG-3)

FIGURE I-3

DIESEL GENERATOR BUILDING



LEGEND

○ —— DEEP BORROS ANCHOR

X —— BUILDING / PEDESTAL SETTLEMENT MARKER

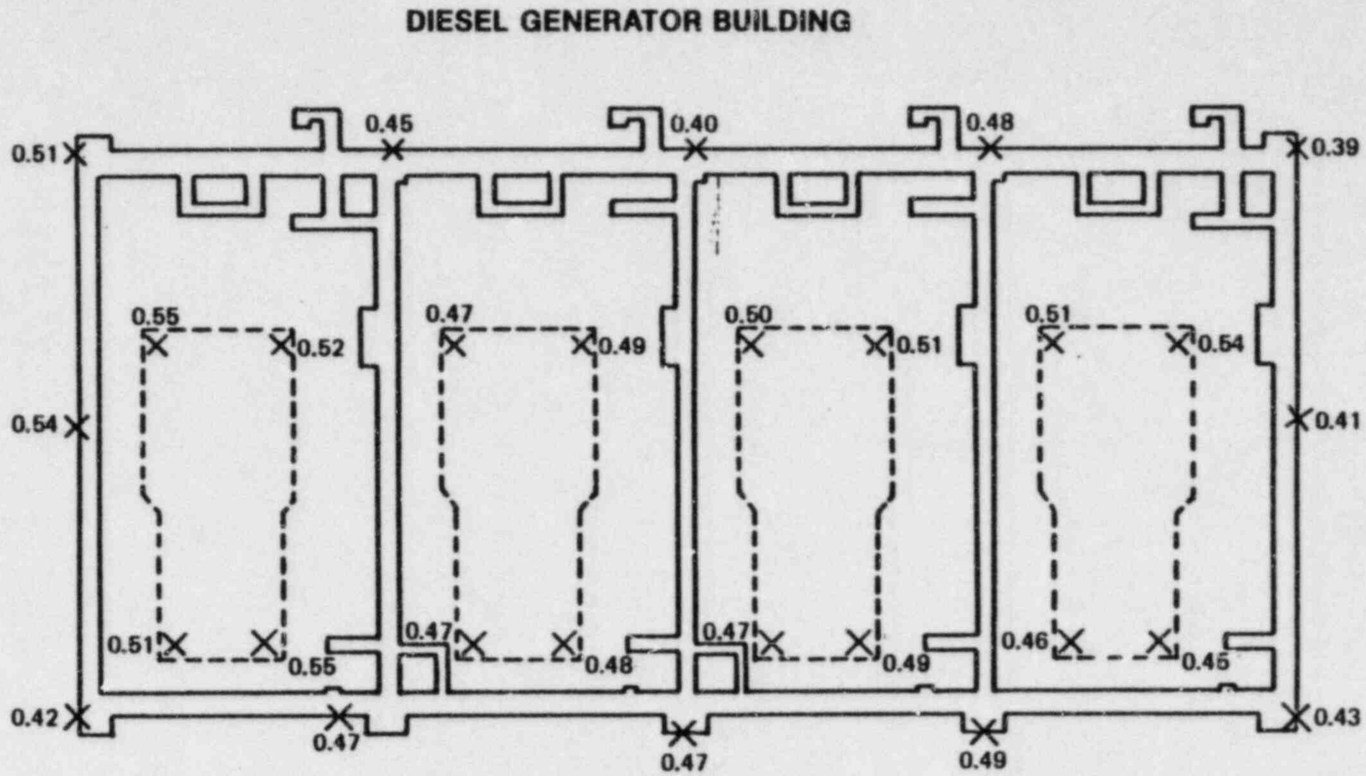
1.20 —— SETTLEMENT IN INCHES

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

ESTIMATED SECONDARY COMPRESSION SETTLEMENTS FROM 12/31/81 TO 12/31/2025 ASSUMING SURCHARGE REMAINS

FIGURE I-4

G-1986-39



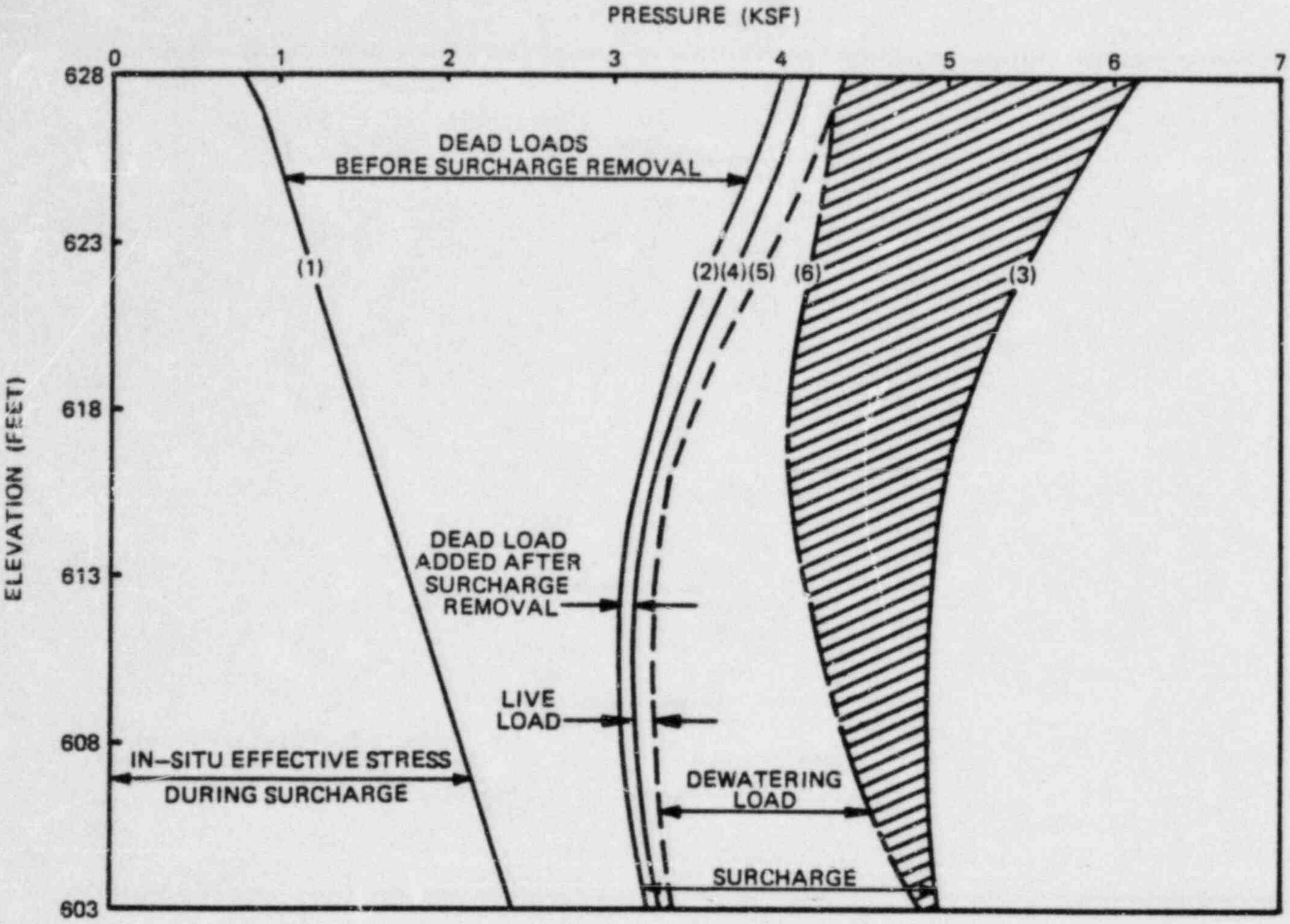
LEGEND

X ——— BUILDING / PEDESTAL SETTLEMENT MARKER

0.42 ——— MEASURED SETTLEMENT BETWEEN 9/14/79 AND 12/31/82.

CONSUMERS POWER COMPANY **MIDLAND UNITS 1 AND 2**
MEASURED SETTLEMENT S_2, FROM 9/14/79 TO 12/31/81
FIGURE I-5

G-1986-38



EXPLANATIONS

(1) In–situ effective overburden pressure (GWT at 627).

(2) Total effective pressure before surcharge removal due to In–situ effective overburden pressure and structural dead loads present during surcharge.

(3) Total effective pressure at the end of surcharge due to In–situ effective overburden pressure, structural dead loads, and surcharge loads.

(4) Total effective pressure due to In–situ effective overburden pressure and total structural dead loads (loads present during surcharge plus dead loads added after surcharge removal).

(5) Total effective pressure due to In–situ effective overburden pressure, total structural dead loads, and expected live loads.

(6) Total effective pressure during the life of plant operation due to In–situ effective overburden pressure, structural dead loads, dewatering loads, and expected live loads.

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
COMPARISON OF EFFECTIVE STRESS BEFORE AND AFTER SURCHARGE SOUTHWEST CORNER DIESEL GENERATOR BUILDING
FIGURE I-6

G-1986-37

PART II: AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PIT

1.0 INTRODUCTION

The 1978 investigation of the plant fill revealed inadequately compacted fill under some areas of the auxiliary building and feedwater isolation valve pits (FIVPs).

The auxiliary building houses a number of safety-related systems, including control and fuel handling. The general arrangement and layout of this building is shown in Figures II-1 and II-2. The auxiliary building is constructed of reinforced concrete.

Parts of the auxiliary building foundations rest on plant area fill; namely, the railroad bay on the north side, the electrical penetration areas for Units 1 and 2, and the control tower on the south side. The rest of the auxiliary building is founded on natural material.

The FIVPs are symmetrically located at the sides of each containment building and are adjacent to the auxiliary building, electrical penetration areas, turbine building, and the buttress access shaft. Each pit is C-shaped with the open end in contact with, but structurally separate from, the containment building. Primarily, the pits enclose the Seismic Category I feedwater pipe isolation valves. The FIVPs for both Units 1 and 2 are founded on plant fill. Exhibit II-1 is a photograph of a scale model of the auxiliary building and shows subsurface conditions under the electrical penetration areas and control tower.

The inadequately compacted fill under the electrical penetration area of the auxiliary building and the FIVPs led to the need for remedial actions for these structures.

2.0 DESIGN CONCEPTS

As agreed upon with the NRC staff, remedial actions consist of the following (see Figures II-3 and II-4):

a. Installing a system of concrete walls below the existing foundations of the electrical penetrations areas and the control tower

b. Installing new concrete foundations for the FIVPs which rest on new compacted granular fill.

The new foundation system provides permanent underpinning that will transfer the load of the affected structure from the existing fill to undisturbed material.

2.1 IMPLEMENTATION OF PLAN

The structure, including the underpinning, has been analyzed for the loads from the building, the effects of the 40-year settlement of soil, and environmental effects such as earthquakes and tornados. The dimensions and major details of the underpinning have been finalized based on a design which used the results of the analyses. The existing structure has been found to be adequate, based on these structural analyses and design. The supporting, undisturbed material also has been found to be adequate.

Before construction of the permanent underpinning can be started, temporary support for the electrical penetration areas and the control tower and lateral earth support are needed. This is the temporary underpinning system. It allows equipment to be used for mass excavation under the areas to be permanently supported.

The temporary underpinning consists of constructing concrete piers under the turbine buildings and installing temporary beams under the electrical penetration areas. The piers provide vertical support for turbine building column loads and support the south end of the temporary beams. They also retain earth during construction. Support for the north end of the temporary beams is provided by steel columns resting on the ledge of the reactor building foundation. The control tower is suppored by piers under the south wall and building columns. The piers are constructed by hand digging pits and filling these with concrete. After the pits are completed, the load is transferred by jacking.

To construct the temporary underpinning, which is below the existing foundations, access is needed from the present grade. Vertical access will be provided by two access shafts. Horizontal access, which is required for pit construction, will be provided by drifts (horizontal tunnels).

The construction of temporary piers and permanent underpinning must be done in a dry condition. Because the present dewatering system is not adequate to lower the groundwater to the bottom of the underpinning, an additional construction dewatering system is needed. This will be accomplished by constructing a freeze curtain dam around the area supplemented by additional dewatering inside the dam.

The freeze curtain dam is constructed by installing a network of vertical pipes in the ground connected to a common supply and return system. Chilled coolant is circulated throughout the system to freeze the ground in the area of the pipes.

After completing the temporary underpinning, mass excavation under the electrical penetration area, the control tower, and the FIVPs is accomplished. During this excavation, the temporary piers are tied by bracing to existing structures.

Completion of mass excavation provides the necessary access to construct the permanent underpinning. After the permanent, reinforced concrete underpinning is complete, the load is transferred from the temporary to the permanent underpinning. The underpinning is connected to the structure with dowels (see Figures II-5 and II-6). The excavations are backfilled with fill material and concrete. At this stage, the permanent foundation rests on undisturbed natural material and the underpinning operation is complete.

During the underpinning operations, extreme care must be taken to protect the existing structure. This is accomplished by removing only small portions of supporting soil during temporary underpinning installation, and replacing it with a temporary system with greater load bearing capacity. In addition, the structure is monitored frequently for movements to ensure that these movements are below predetermined limits.

2.2 LICENSING STATUS

The design concept for the auxiliary building underpinning has been presented and discussed with the NRC staff using several methods: technical reports, testimony for the Atomic Safety and Licensing Board (ASLB) soils hearings, design audits by the NRC staff, and technical meetings.

A technical report describing the underpinning was submitted on September 30, 1981. This was supplemented by responses to NRC staff requests for additional information on November 16, 1981, and by addendum on December 3, 1981. This provided preliminary analytical results. Specialized reports regarding the effects of cracking of concrete on the FIVP and the auxiliary building were submitted on January 25, 1982, and January 29, 1982, respectively.

Testimony presented at the ASLB soils hearings in December 1981 also provided the staff with information about the underpinning system.

Design audits were conducted in the Bechtel offices at Ann Arbor, Michigan, on three occasions: January 16 through 19, 1981; February 2 through 5, 1982; and March 16 through 19, 1982. During these audits, the staff reviewed in detail the design concepts and calculations for the temporary underpinning.

Meetings between the staff and the Applicant were held on October 1, 1981; November 4, 1981; and February 26, 1982; to discuss both the concept and details of the design. In addition, meetings were held December 10, 1981; and January 11, 1982; to specifically discuss effects of concrete cracking.

The design concept has received NRC staff concurrence.

3.0 STRUCTURAL ANALYSIS AND DESIGN

Structural analysis of the auxiliary building and its underpinning is performed in two parts:

a. A seismic analysis using a mathematical model to analyze the structure for the dynamic conditions during a seismic event

b. A static analysis, where the static loads imposed on the structure, such as dead load, live load, wind load, etc, are analyzed.

The loads from these two analyses are combined in accordance with applicable load combinations. Load combinations presented in Final Safety Analysis Report (FSAR) Subsection 3.8.6 and supplemented by the Responses to NRC Requests Regarding Plant Fill, Question 15, (Revision 3, September 1979) are used for the structure and the underpinning and its connections to the structure. Additional loading combinations based on American Concrete Institute (ACI) Code 349-76 and supplemented by NRC Regulatory Guide 1.142 are used for the underpinning and its connections to the structure.

3.1 SEISMIC ANALYSIS

A seismic model is developed to evaluate overall building response to seismic loadings as well as to generate in-structure response spectra for equipment design. The responses from this model provide input to other static analyses. The building is represented by a three-dimensional, lumped-mass stick model with plate elements used to represent the stiffness of the shear walls and underpinning in the electrical penetration area and control tower.

By NRC staff direction, the underpinning is designed to withstand the effects of the site-specific response spectra (SSRS) ground motion. The existing structure is evaluated for the effects of the plant's original design basis as stated in the FSAR ground motion description. In order to proceed with the underpinning design while NRC concurrence with the proposed SSRS was being obtained, the structural forces resulting from the FSAR safe shutdown earthquake (SSE) ground motion were multiplied by a factor of 1.5 for design of the underpinning. The response from a 1.5 times FSAR SSE envelops the final SSRS response.

The seismic analysis of the underpinned structure has been completed and the results are being used for the static analysis of the underpinning and reevaluation of auxiliary building equipment for seismic loadings.

3.2 STATIC ANALYSIS

3.2.1 Finite-Element Models

The superstructure and underpinning of the auxiliary building are analyzed by a finite-element method. The structure is analyzed for four conditions with four different finite-element models. Each model is briefly described below. The modeled conditions are:

a. Construction sequence of the proposed underpinning

b. Long-term loading without connecting the underpinning to the building

c. Long-term loading with full connection between the underpinning and building

d. Short-term loading with full connection between the underpinning and building

The models consist primarily of plate elements. Beam elements are used to represent columns, minor concrete elements, and major steel components of the structure. The nodal mesh is intensified in the areas significantly affected by underpinning. The soil subbase is represented by boundary springs placed under the foundation areas. The spring constants are based on appropriate soil response predictions as dictated by the load duration.

The underpinning is modeled as a continuation of the main shear walls in the control tower and the auxiliary building electrical penetration areas and extends the full length under these areas.

3.2.2 Construction Model

A construction sequence model reflects loadings on the structure during various stages of temporary underpinning. This model is used to investigate the construction sequence as the existing soil support of the structure is sequentially replaced by jacking loads.

Several variations of this model are utilized, modeling differences in the total number of boundary springs which are replaced by jacking loads. The temporary underpinning is reflected as a jacking load in this model. The spring constants for the boundary springs reflect the soil properties prior to underpinning. The load cases applied to the model include dead load, live load, jacking loads, external hydropressures, soil pressures, and wind loads.

3.2.3 Models for Long-Term Loads

3.2.3.1 Underpinning and Structures Disconnected

This model is used to investigate the effects of long-term loads with the underpinning disconnected from the superstructure. This model represents the construction stage when the superstructure and underpinning are separated by a series of hydraulic jacks and shims with the jacks and shims totally supporting the underpinned areas. Structural interaction is produced by placing upward jacking loads on the superstructure and placing equal and opposite loads on the underpinning.

The boundary springs have spring constants based on the predicted soil response to long-term loads. The load cases applied to the model are dead load, live load, external hydropressures, soil pressures, jacking loads, and wind loads.

3.2.3.2 Underpinning and Structures Connected

This model is used to investigate the effects of long-term loads with the underpinning fully connected to the superstructure. The load cases applied to the model include dead load, live load, soil and water pressures, and differential settlement loads. The differential settlement is considered in the model by calculating appropriate spring constants based on settlements.

Based on the properties of the natural materials, over the 40-year life of the underpinning, the settlement after construction is predicted to be 0.3 inch at the control tower and 0.2 inch in the electrical penetration area. The main portion of the auxiliary building is predicted to settle in the range of 0.1 inch to 0.5 inch. These predicted settlements are based on an investigation conducted by Woodward-Clyde Consultants (WCC), who performed soil borings and laboratory testing of the undisturbed natural materials. These tests show the preconsolidation pressure of the natural materials to be between 30 to 40 tons/sq ft.

3.2.4 Model for Short-Term Loads

This model is used to investigate the effects of short-term loads with the underpinning fully attached to the superstructure. The spring constants for the boundary springs are based on the predicted soil response to short-term loads. The load cases applied to the model are east-west earthquake, north-south earthquake, vertical earthquake, tornado, wind, and pipe rupture loads.

3.3 DESIGN

The results of the structural analyses are factored and added in specific combinations to evaluate the structural adequacy of the structure and underpinning. This verification ensures that computed stresses and loads will be lower than or equal to the allowable stresses and capacities.

3.3.1 Temporary Underpinning

Salient design features of the temporary support system include (see Figure II-8):

a. Steel frames, as shown in Figure II-9, supporting the FIVPs.

b. Thirty-six concrete piers at the north end of the turbine building - These piers support the turbine building column load on Column Lines K and K_C and also retain soil under the turbine building basemat. These piers are permanently left in place. The piers are braced with struts and tie rods to transmit lateral loads to the containment wall.

c. Three frame supports under each electrical penetration area - Each frame support consists of a concrete pier, needle beams, and steel columns supported on the reactor building foundation slab or on another concrete pier (see Figure II-10). These frames also support part of the turbine building load.

d. Ten concrete piers under the south side wall of the control tower - These piers are a part of the underpinning wall for the control tower. Struts are provided to transmit lateral loads from the soil under the turbine building to the auxiliary building.

e. Additional concrete piers under each of the three existing steel columns inside the control tower - These piers are part of the permanent underpinning.

f. Two concrete piers below each buttress access shaft - These support the reaction load from the temporary steel frames which support the FIVPs and retain soil under the buttress access shaft. These piers are permanently left in place.

g. Tunnels under the turbine building and access drift tunnels - These tunnels and drifts are constructed by the usual construction methods utilizing lagging and steel frames.

h. Temporary post-tensioning - The temporary dewatering system removes the buoyancy force normally provided by groundwater under the electrical penetration areas. To compensate for this effect during construction, a temporary system of post-tensioning ties is installed to apply a compressive force to the upper part of the east-west walls of the electrical penetration areas. The post-tensioning ties are removed when the temporary supports are installed and jacking loads are applied under the electrical penetration areas.

The temporary support system is designed to resist the calculated imposed loads using ACI and American Institute of Steel Construction (AISC) codes.

3.3.2 Permanent Underpinning

Design features of the electrical penetration and control tower areas are (see Figures II-3, 4, 5, 6, and 7):

a. The proposed underpinning for the Unit 1 and 2 penetration areas are a 6-foot thick, reinforced concrete wall 38 feet high belled out to 10 feet thick at the bottom. The belling limits bearing pressures to the allowable values. The underpinning walls under the control tower are 6 feet thick, 41 to 47 feet high, and are belled out to 14 feet thick. The walls are constructed to act as a continuous member under the perimeter of the structures. Individual piers are provided to underpin interior columns of the building. The entire wall and pier system is founded on undisturbed natural material.

b. Allowable bearing pressures for the undisturbed natural material is based on a safety factor of 2 for dynamic loading and 3 for static loading. The ultimate bearing capacity for the natural material is based on the undrained triaxial tests performed on the WCC boring samples. These yielded a median shear strength of 7.6 ksf.

c. A design jacking force is applied to the existing structure to provide adequate load transfer from the structure to the permanent underpinning. These jacking forces transmit the structural loads through the permanent underpinning wall to the bearing stratum.

d. Dowels connect the underpinning walls and the existing structure at the vertical and horizontal interfaces. The dowels are designed to transfer shear and tension forces between the structure and the underpinning wall.

These dowels are connected after the permanent load transfer is accomplished.

3.4 LICENSING STATUS

The structural analysis for the underpinning was presented in technical reports, ASLB hearing testimony, design audits, and meetings as previously indicated in Section 2.2.

The seismic analysis was covered in detail during testimony by Dr. R.P. Kennedy of Structural Mechanics Associates (SMA) and Dr. P. Hadala of the U.S. Army Corps of Engineers (representating the NRC staff) during the ASLB soils hearings of December 14, 1981.

As indicated in Section 2.2, three design audits have been performed by the NRC staff. During these audits, structural design calculations for the temporary underpinning and the resulting structural stresses have been reviewed in detail.

Preliminary analysis of the permanent underpinning has been completed and the results presented to the staff. Analysis of the temporary underpinning also has been completed and audited by the staff. Analysis of the final underpinning is being completed and when finished will be presented to the NRC staff.

Design of the temporary underpinning is complete and has been presented in technical reports, meetings, and design audits. Drawings are being issued for construction. Start of construction is currently awaiting NRC concurrence and is scheduled for May 1982.

As directed by the NRC, the Applicant is performing a parametric analysis by varying the subgrade reaction modulus for the till under the auxiliary building to a value of 70 kcf. The Applicant also will perform, at the NRC's direction, an analysis of the electrical penetration area for the effects on existing soil support caused by the adjacent access tunnel under the turbine building. A confirmatory load test on the bearing stratum will be performed.

4.0 CONSTRUCTION SUPPORT PROGRAMS

4.1 GROUNDWATER CONTROL

At the start of underpinning work it is anticipated that the groundwater level will be at about el 600'. Because this work will extend at least 29 feet below that level, the control of groundwater level will be an important prerequisite for successful completion.

The underpinning work is in a location with limited access, bounded by the two containment buildings, the main auxiliary building, and the turbine building. In the immediate construction area, groundwater will be removed by pumping from dewatering wells.

To reduce recharge of groundwater into this narrow area, an underground freeze curtain dam will be constructed. The proposed layout of the dam is shown in Figure II-11. The dam will be formed by drilling a line of boreholes at approximately 4-1/2-foot spacing and circulating glycol coolant at low temperatures through pipes in the boreholes. The coolant will freeze the soil in a narrow strip along the line from el 610' down to the undisturbed glacial till. The frozen soil will act as a dam and reduce subsequent seepage of groundwater from the pond side toward the underpinning construction area. The freeze curtain dam will be formed in permeable sandy soil that exists above the glacial till and below el 610'. The actual extent of these sandy soils will be determined by the initial borehole drilling.

The existing clay cutoff dike along the western edge of the power block will form a part of the underground dam. The effectiveness of the dewatering system will be monitored by measurements of the groundwater levels using piezometers located in the work area.

Design of the groundwater control system is complete and has been presented to the NRC staff in a technical report, meetings, and audit discussions. NRC concurrence has been received for installation and activation of the groundwater control system. Installation is approximately 75% complete. The safety-related utilities crossing the freeze curtain dam will be isolated by excavating so that they are unaffected by any potential heave of the ground due to freezing operations.

4.2 ACCESS SHAFT

Immediately east and west of the two FIVPs and adjacent to the turbine building, shafts are being constructed to provide access for workers and equipment for the underpinning work. The location of the west access shaft is shown in Figure II-12. The east access shaft will be symmetrically located. Each shaft will be about 16 feet by 26 feet in clear plan dimensions.

The shafts will be excavated in three phases. Initially, they will be excavated to el 609' to permit installation of the initial underpinning piers beneath the adjacent turbine building basemat. These piers will constitute permanent underpinning for the turbine building. When the initial turbine building underpinning is completed, the access shafts will be lowered to el 600' to provide access for excavation beneath the FIVPs.

After all temporary underpinning is completed for the FIVPs and electrical penetration areas, the two access shafts will be gradually lowered from el 600' to el 571'. At that time, a level working surface extending into the shafts will be constructed for the general excavation and removal of soil down to el 571' beneath the FIVPs, electrical penetration areas, and control tower.

The shafts will be constructed using standard methods and utilizing soldier piles, wales, and lagging.

The access shaft design is complete and has been presented in a technical report, meetings, and the audit of January 18 through 20, 1982. NRC concurrence has been received for installation to el 609' and this installation is complete. El 609' is the foundation level of the FIVP, auxiliary building, and turbine building.

5.0 CONSTRUCTION PROGRAMS

5.1 TEMPORARY UNDERPINNING

In order to construct the permanent underpinning, it is necessary first to install a temporary underpinning system to support the FIVPs and portions of the turbine building, electrical penetration areas and the control tower. The temporary underpinning system is shown in Exhibit II-2, which is a photograph of a scale model.

The following is a summary of the construction sequence of the temporary underpinning on the east side. The sequence for the west side is similar. The layout and the identification numbers of the underpinning system are shown in Figures II-8 and II-9.

The initial effort for the temporary underpinning was to construct access shafts to el 609'. This is the bottom of the turbine building and electrical penetration area foundations. It is also necessary to support the FIVP with steel framing. The purpose of these activities is to obtain access to the initial turbine building supports. Construction of both of these activities has been completed.

The next step will be to provide support to the turbine building near the electrical penetration area by constructing Piers E-9 and E-12. Before constructing these piers, the freeze curtain dam, which is near completion, will be activated to control groundwater. The completion of these turbine building piers is necessary to construct the tunnel/drift under the turbine building and to access the first support, Pier E-8, for the electrical penetration area.

Pier E-8 will be completed next and the first excavation under the electrical penetration area will be begun to install the needle beams needed to provide the first support for the electrical penetration area (see Figure II-10). The completion of Pier E-8 and the needle beams is very important to the temporary underpinning operation because after their completion, the entire weight of the electrical penetration area can be supported and any loss of soil support under the electrical penetration area is no longer critical. With Pier E-8 and the needle beams in place, the tunnel under the turbine building can be extended to access the first corner Pier E-1 of the control tower. While extending the tunnel, additional piers on Column Line K_C, to support the turbine building columns, are constructed.

The corner Pier E-1 of the control tower will be completed and jacked next. The completion of the control tower corner piers is crucial because after this the remaining control tower and electrical penetration area temporary underpinning piers can be simultaneously constructed.

With completion of the temporary underpinning piers, the weight of the electrical penetration area and control tower can be completely supported and the mass excavation under the electrical penetration area and control tower can begin. For performing the mass excavation, the access shaft will be extended to el 571'.

With completion of the mass excavation, the permanent underpinning can be started.

5.2 PERMANENT UNDERPINNING

A continuous underpinning wall resting on undisturbed natural material will be provided under the control tower and the electrical penetration area exterior walls. Also, a new concrete foundation resting on new concrete, which, in turn, is set on new compact granular fill, will be provided for the FIVPs. This underpinning provides the necessary vertical and horizontal support to the affected part of the structure. The details of the permanent underpinning are shown in Figures II-3, 4, 5, 6, and 7.

A summary of the construction sequence for the permanent underpinning follows.

After the completion of mass excavation, the permanent wall under the electrical penetration areas and the permanent section of the wall in the control tower area can be constructed. At this stage, compacted backfill will be placed below the FIVP area and a new slab will be poured at el 600'.

After completion, jacks will be placed on the wall. Jacking forces will be transferred from the temporary to permanent walls in stages. Adjustments will be made until all the load is transferred from the temporary to the permanent underpinning and the wall has reached the final design jacking load. The slab under the FIVP foundation also will be jacked against the FIVP to transfer the load from the temporary steel support to the new slab.

Jacking loads will be held on the permanent underpinning and the settlements monitored. When the settlement rate has reached a predetermined value, the jacking load will be locked off. The permanent underpinning walls will be connected to the existing structure by grouting and the gaps filled with grout. For the FIVP, the area between the new slab and the FIVP existing foundation slab will be filled with lean concrete. At this stage, the excavation will be backfilled with fill or lean concrete and the permanent underpinning will be complete.

The design of the underpinning is complete to the preliminary safety analysis report (PSAR) level and has been presented in the technical report and in meetings. NRC concurrence to proceed with construction has not been received.

There are no unresolved issues regarding the permanent underpinning and an operating license level design audit will be conducted by the NRC staff.

5.3 BUILDING MODIFICATIONS

Preliminary analysis indicates that strengthening may be required for one area of an existing slab at el 659' for certain loading combinations, including seismic loads. This area is between the control tower and spent fuel pool at the operating floor level. Detailed analysis is being performed to resolve this concern.

Because this strengthening, if required, is needed only to resist loads during a seismic event, it is not required prior to or during underpinning but will need to be installed prior to fuel load. The present plan is to finalize the design for this strengthening, if required, after the final analysis of the building and underpinning is completed.

6.0 MONITORING PROGRAM

To ensure that installation of the underpinning system is proceeding within acceptable limits, a monitoring program will be implemented during construction. This program has three parts: building movement and strain, cracking, and underpinning.

6.1 BUILDING MOVEMENT AND STRAIN MEASUREMENT

The underpinning methods to be used require that the soil be removed in small, discrete units and that these units be replaced with load bearing units of greater capacity than the unit that was removed. Discrete units are removed and replaced progressively, according to a predetermined plan, in a manner that will maintain the stresses in the structure below allowable limits.

Two systems will be used for detecting vertical and horizontal movements of the auxiliary building. The first system is for detecting movement of the reactor containment, auxiliary building, and turbine building with respect to a fixed datum. The second system is for detecting relative movement of the auxiliary building to the other structures.

The first system consists of seven deep-seated benchmarks to serve as reference points for measuring movement of the free ends of the electrical penetration areas, the east and west ends of the control tower, and the main auxiliary building. Movement will be measured with dial gages and electronic linear variable differential transducers (LVDTs). The precision of this instrumentation is ±0.001 inch and the accuracy is ±0.005 inch.

The second system will measure relative vertical movement between the structures described above by means of dial gages and LVDTs. Those relative readings will have an accuracy of ±0.005 inch. In addition, movements of the FIVPs will be monitored using LVDTs and one deep-seated benchmark in each pit.

Because of direct reading and high precision, the benefit of the movement measurement system is that data is readily produced for sensing differential movements and developing trends.

Relative horizontal movement will be measured at vertical measurement locations with relative movement dial gages and LVDTs. In addition, relative horizontal movement between the turbine building and auxiliary building will be measured at the roof level of these two structures.

Strains will be monitored in critical areas, which include the slab at el 654', the walls at el 614', and the connection of the electrical penetration area and control tower roof. Additionally, selected steel beams at el 659' will be provided with strain gages.

6.2 CRACKS

6.2.1 Existing Crack Evaluation

The existing cracks in the control tower, electrical penetration area, and FIVPs have been monitored. The size and location of existing cracks have been recorded on crack map drawings. The Applicant's consultant, Portland Cement Association (PCA), evaluated the structural significance of these cracks based on its site visit and review of the crack maps. The consultant concluded that all cracks are attributable to restrained volume changes that occur during curing and drying of concrete. PCA also did not observe any structural distress during the visit.

The consultant's evaluations and conclusions are contained in reports submitted to the NRC staff on January 25 and 29, 1982.

6.2.2 Crack Monitoring During Underpinning

Existing cracks will be monitored for changes in length and width during various phases of construction. The areas containing cracks will be inspected for new cracks that, if present, will be similarly mapped and monitored. need temp inside and outside

Because of the sequence of construction procedures, it is not anticipated that existing cracks will significantly widen or that significant new cracks will appear. However, any new structural cracks exceeding 0.01 inch in width or any crack exceeding 0.03 inch in width will be evaluated by PCA to determine whether underpinning operations should stop or continue. If development of yield strain is inferred from any observed crack, underpinning will be stopped and an evaluation made by PCA before continuing underpinning operations.

6.2.3 Repair of Cracks

A report on a crack repair program by PCA for all cracks in all structures will be submitted to the NRC staff in the near future.

6.3 UNDERPINNING

During underpinning installation, each temporary pier will be instrumented to monitor deflection of the pier tops and bottoms. Pier top movement will be monitored with readings taken between the underside of the foundation slab and the pier top. Monitoring will begin after pier concrete is placed and will include measurements during and after initial jacking. In addition, the underpinning wall movements will be similarly monitored.

Pier and wall bottom movement will be monitored by a rod attached to a plate at the base of the underpinning. The rod will be greased and enclosed in a small diameter pipe sleeve. The rod and sleeve will extend to the top of the pier before the pier concrete is placed. Rod movements will be recorded by dial gage extensometers simultaneously monitoring the movement of the pier or wall top. These instruments produce measurements relative to the position of the base slab. Absolute top and bottom movement values can be obtained by adding the measurements of movement, if any, of the base slab obtained from the deep benchmark monitoring.

The instrument readings for the movement of the pier base and top will be compared to anticipated values for creep and shrinkage of concrete and for the soil settlement. Actual values will be compared to expected values to determine when the final jacking loads can be locked off.

Carlson gages will be used to measure loads in selected temporary piers.

6.4 LICENSING STATUS

The design of the monitoring program is complete and was presented in a technical report, the meeting of February 26, 1982, and design audits. NRC concurrence has been received for installation and operation. Installation is currently in progress.

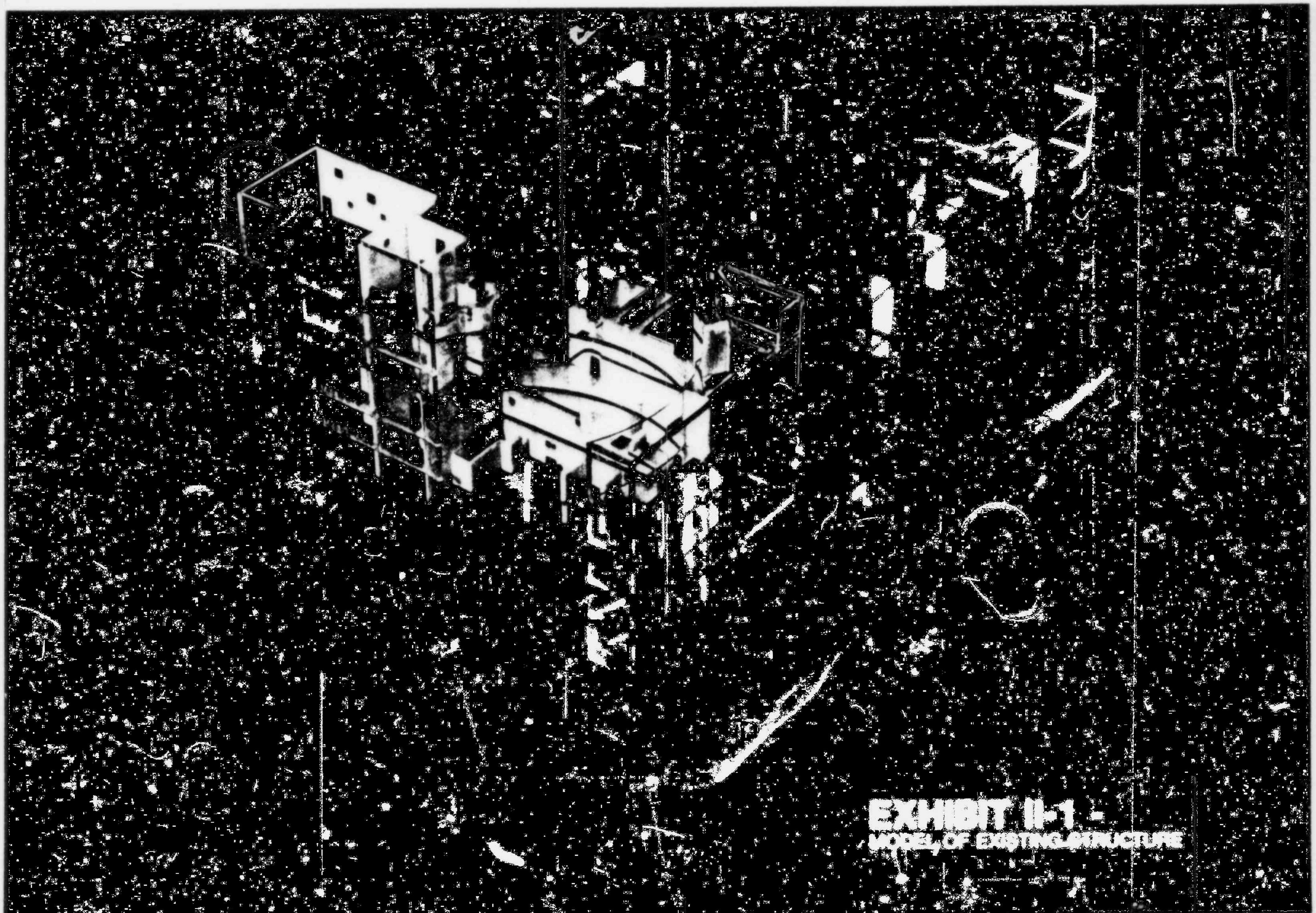


EXHIBIT II-1 -
MODEL OF EXISTING STRUCTURE

EXHIBIT II-2 -
MODEL OF TEMPORARY UNDERPINNING

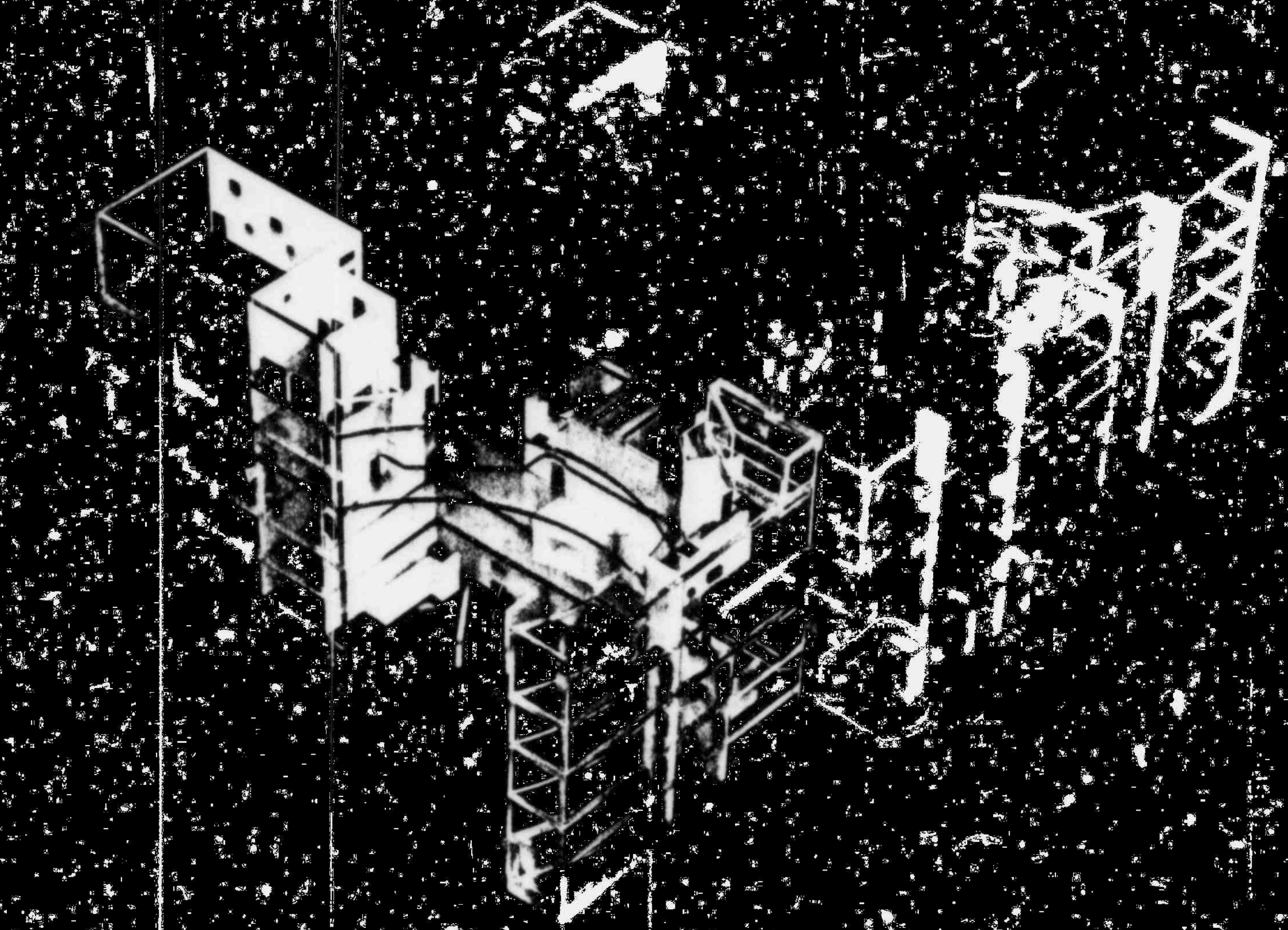
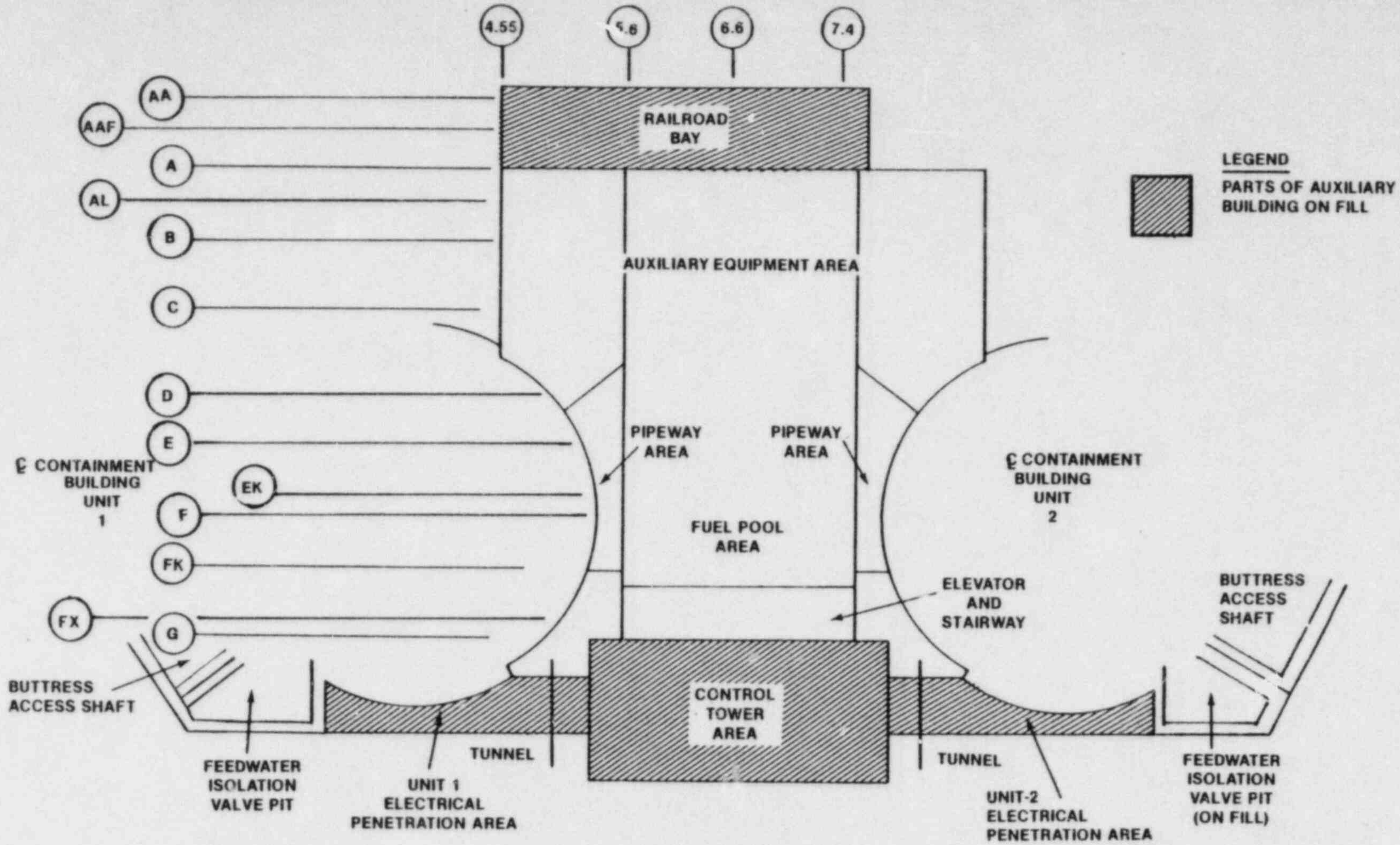
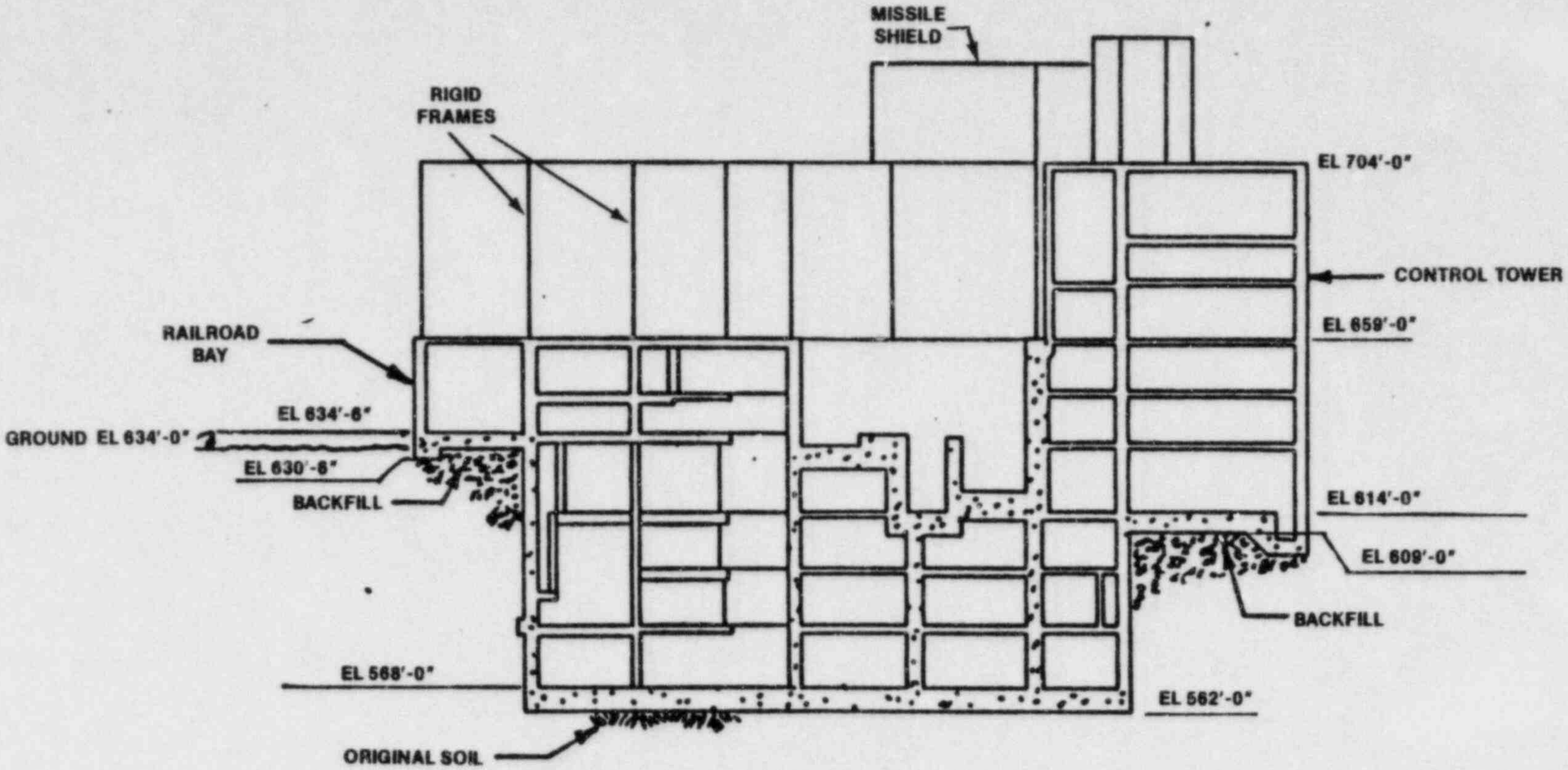


EXHIBIT II-3 -
MODEL OF PERMANENT UNDERPINNING



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
AUXILIARY BUILDING PLAN
FIGURE II-1

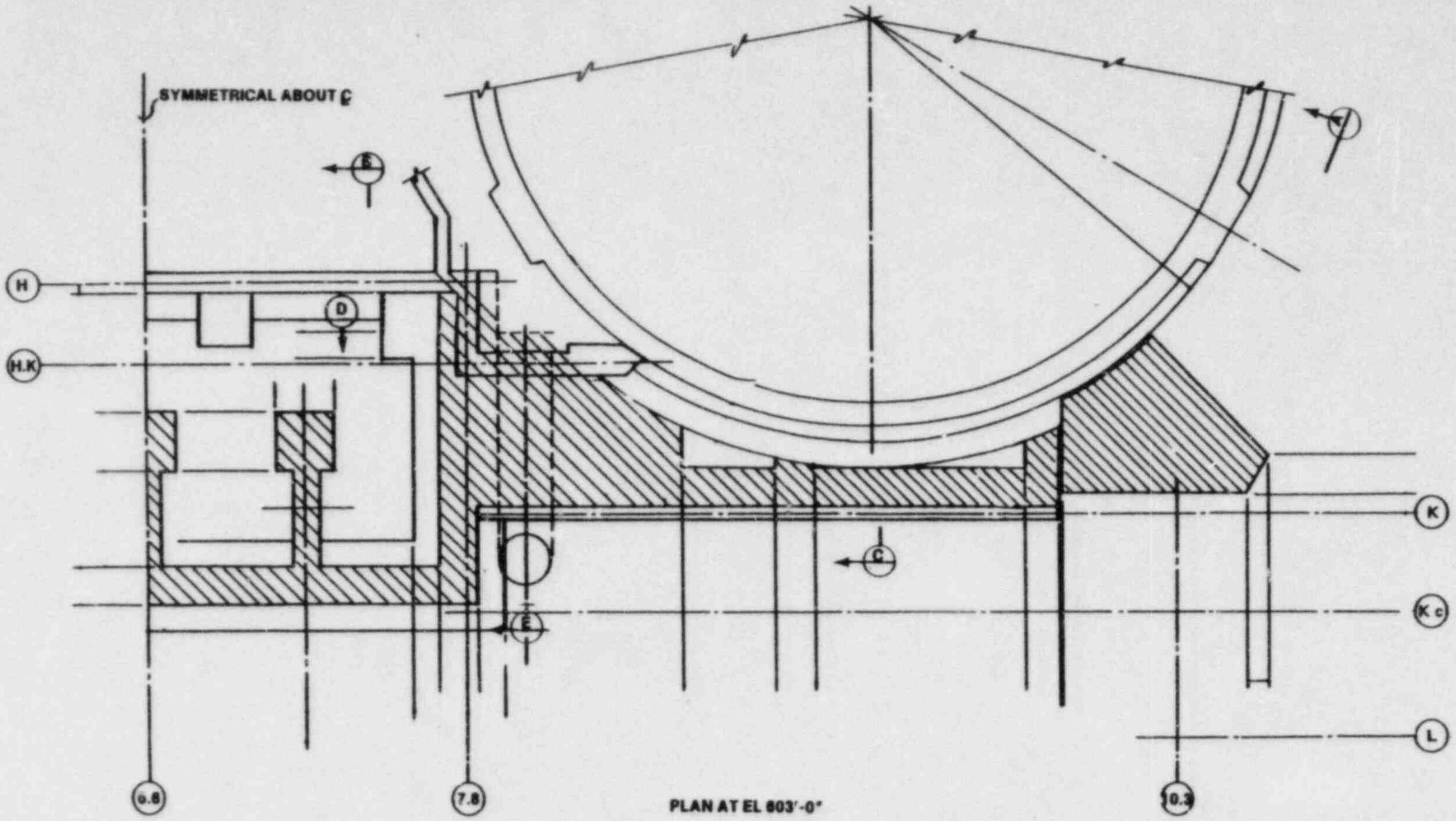
G-1986-12



TYPICAL SECTION (LOOKING EAST)

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
AUXILIARY BUILDING SECTION
FIGURE II-2

G-1986-10



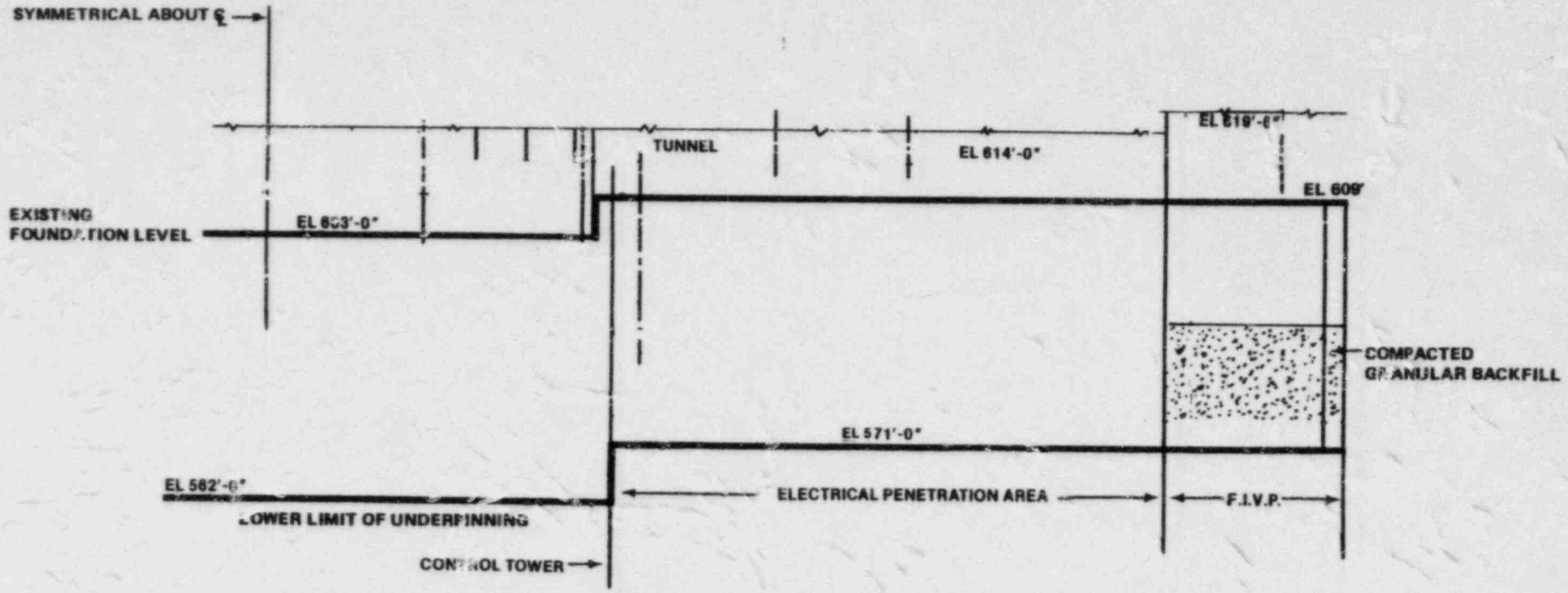
G-1986-08

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING PLAN AT
ELEVATION 603'

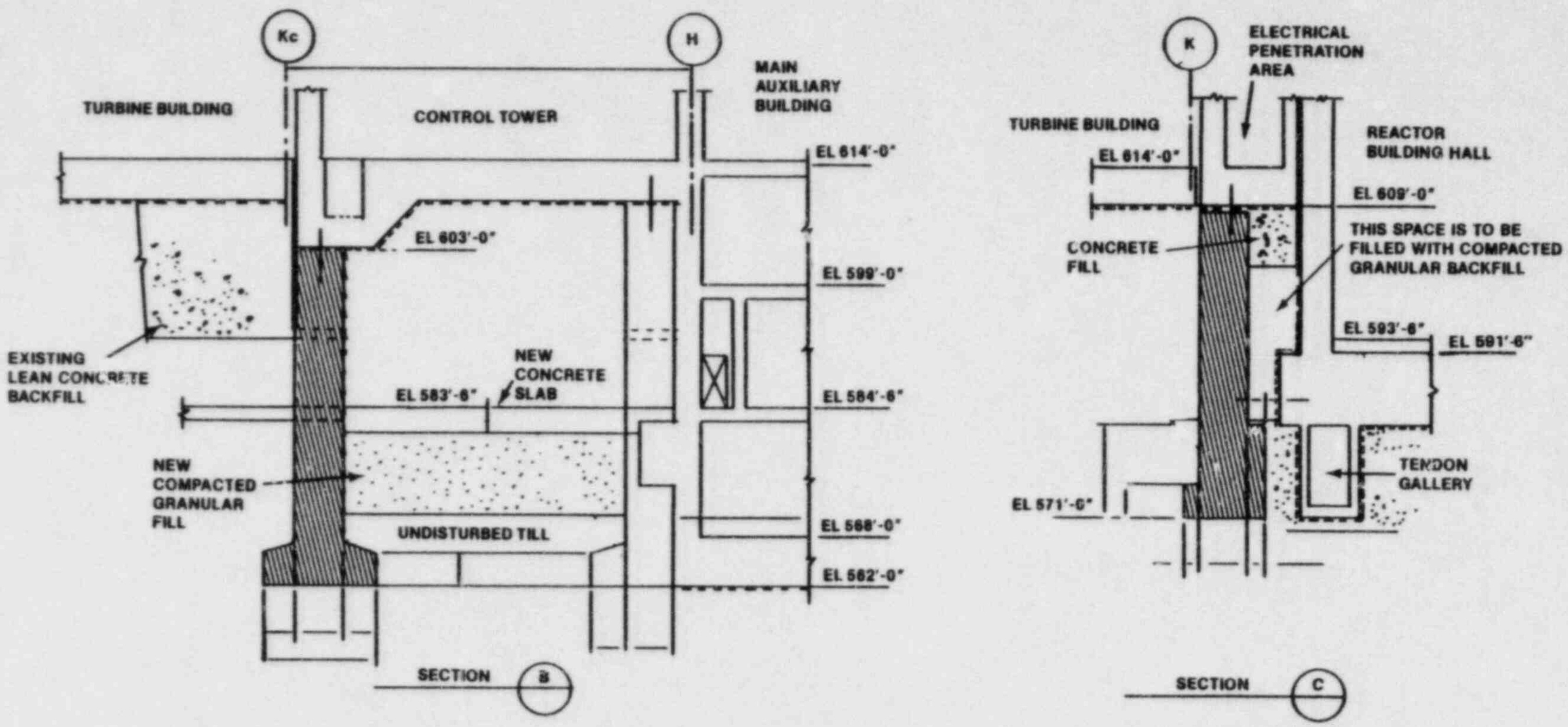
FIGURE II-3

G-1986-08



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
ELEVATION OF UNDERPINNING WALL
FIGURE II-4

G-1986-07

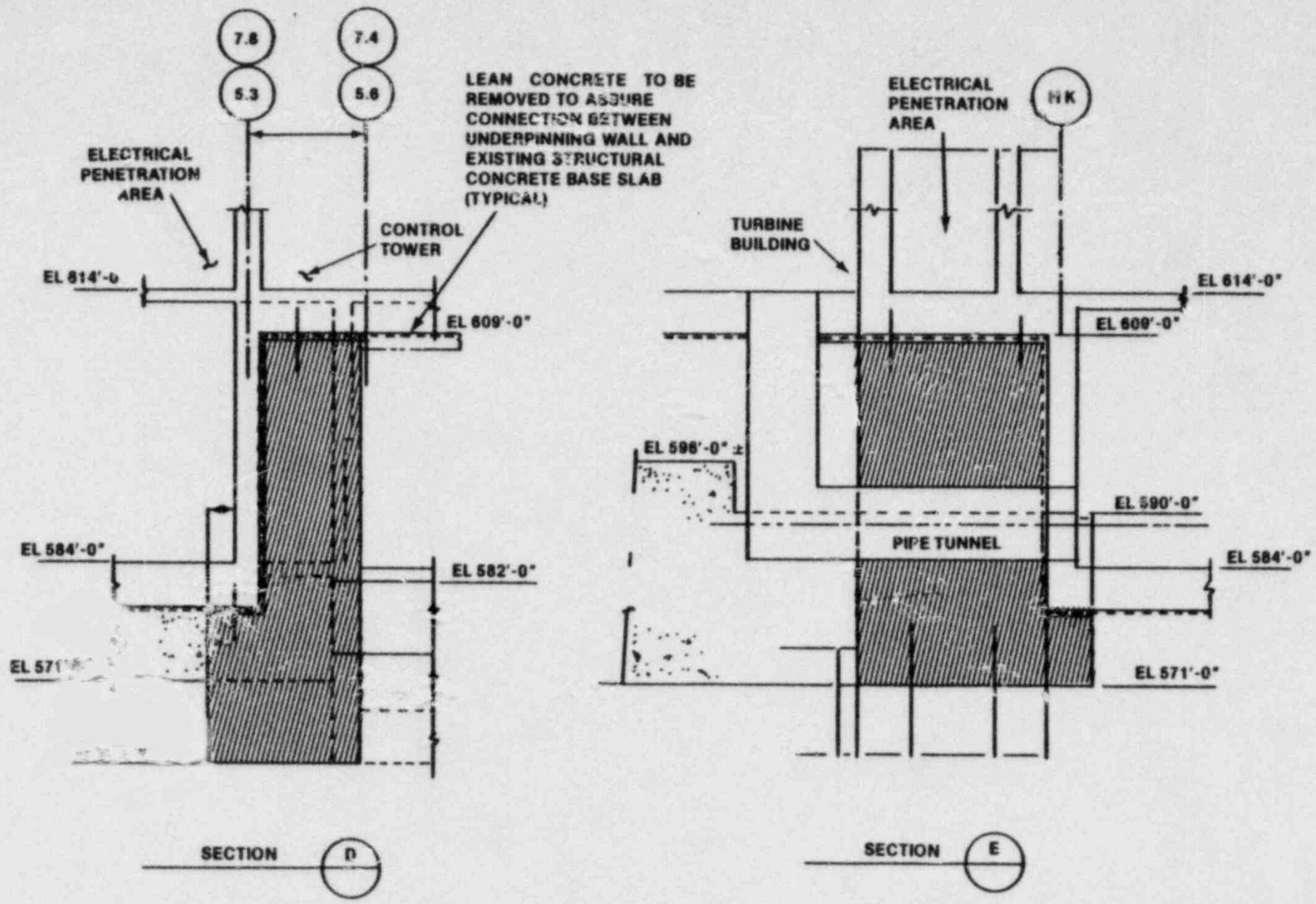


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING WALL SECTIONS

FIGURE II-5

G-1986-05

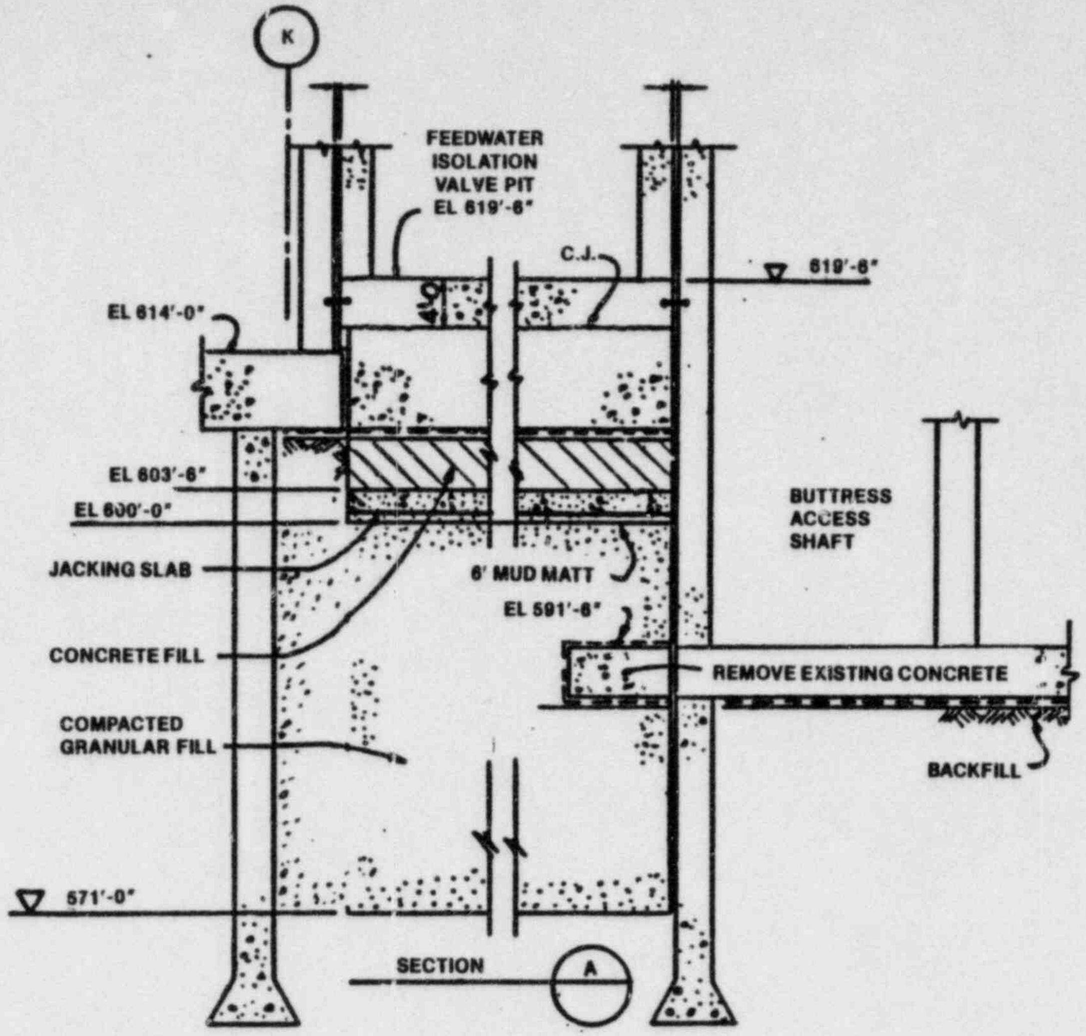


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING WALL SECTIONS

FIGURE II-6

G-1986-06

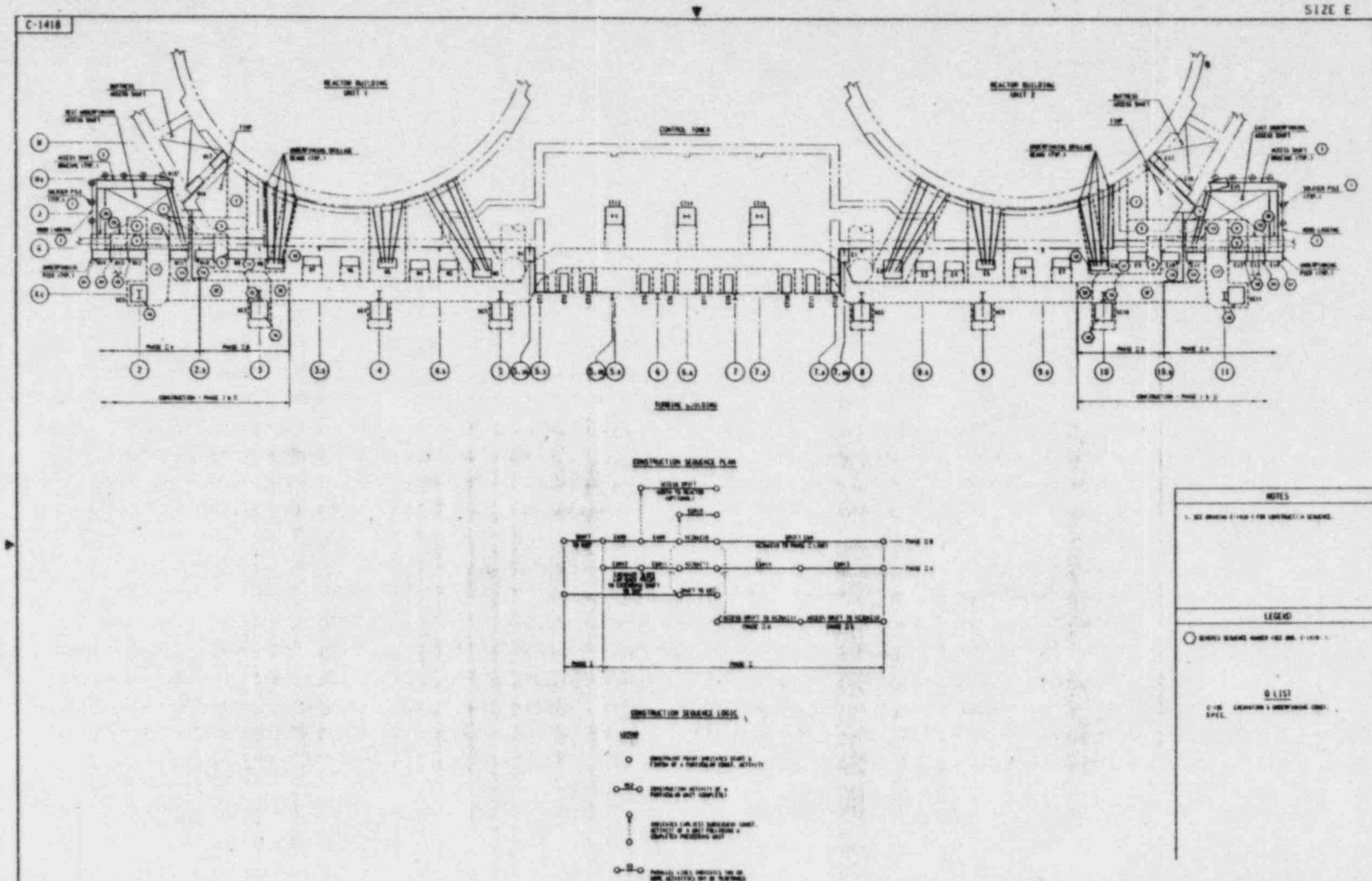


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SECTION AT FIVP

FIGURE II-7

G-1986-11

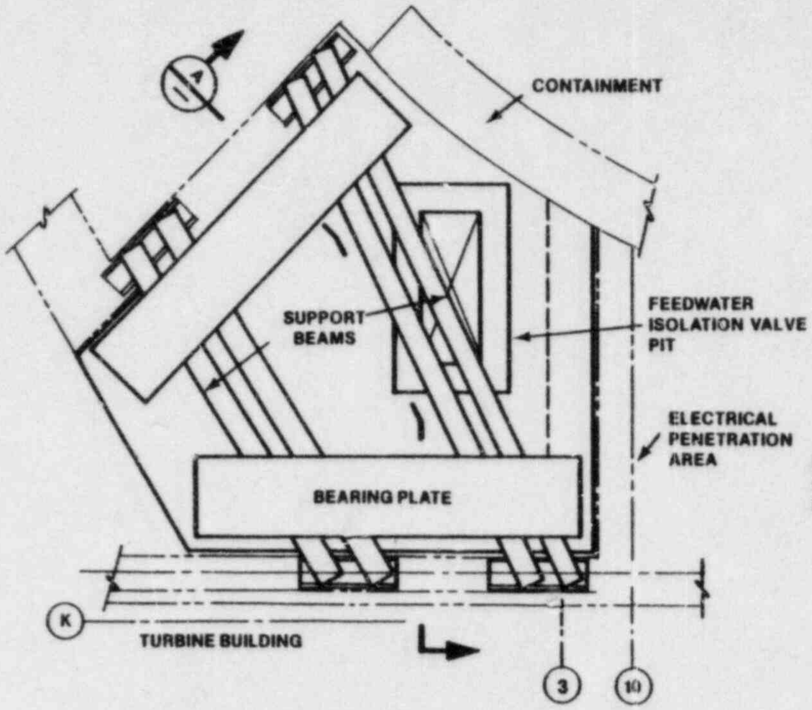


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

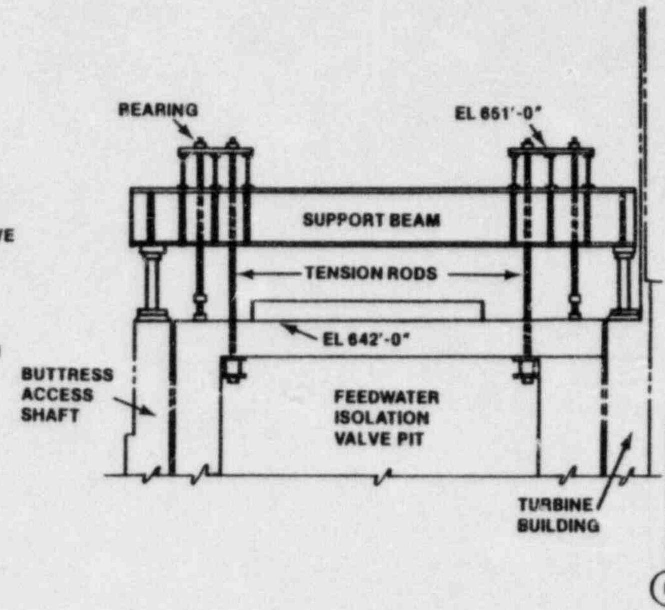
AUXILIARY BUILDING UNDERPINNING CONSTRUCTION

FIGURE II-8

G-1986-13



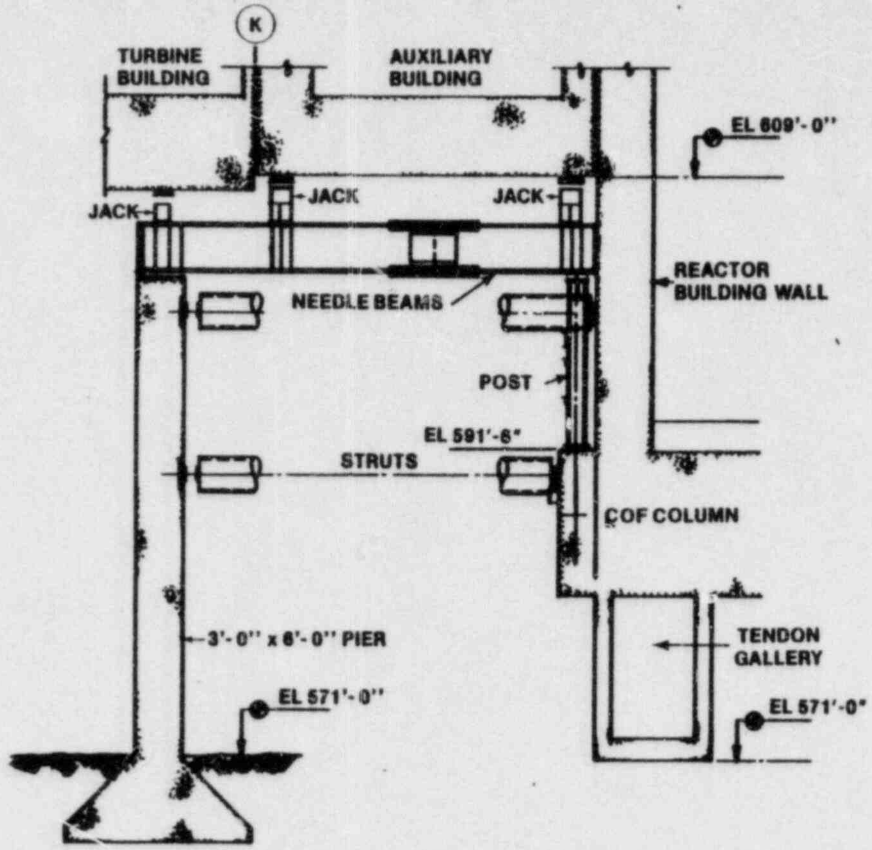
PLAN AT EL 651'-0"



SECTION A

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
TEMPORARY SUPPORT FOR FEEDWATER ISOLATION VALVE PIT
FIGURE II-9

G-1986-02

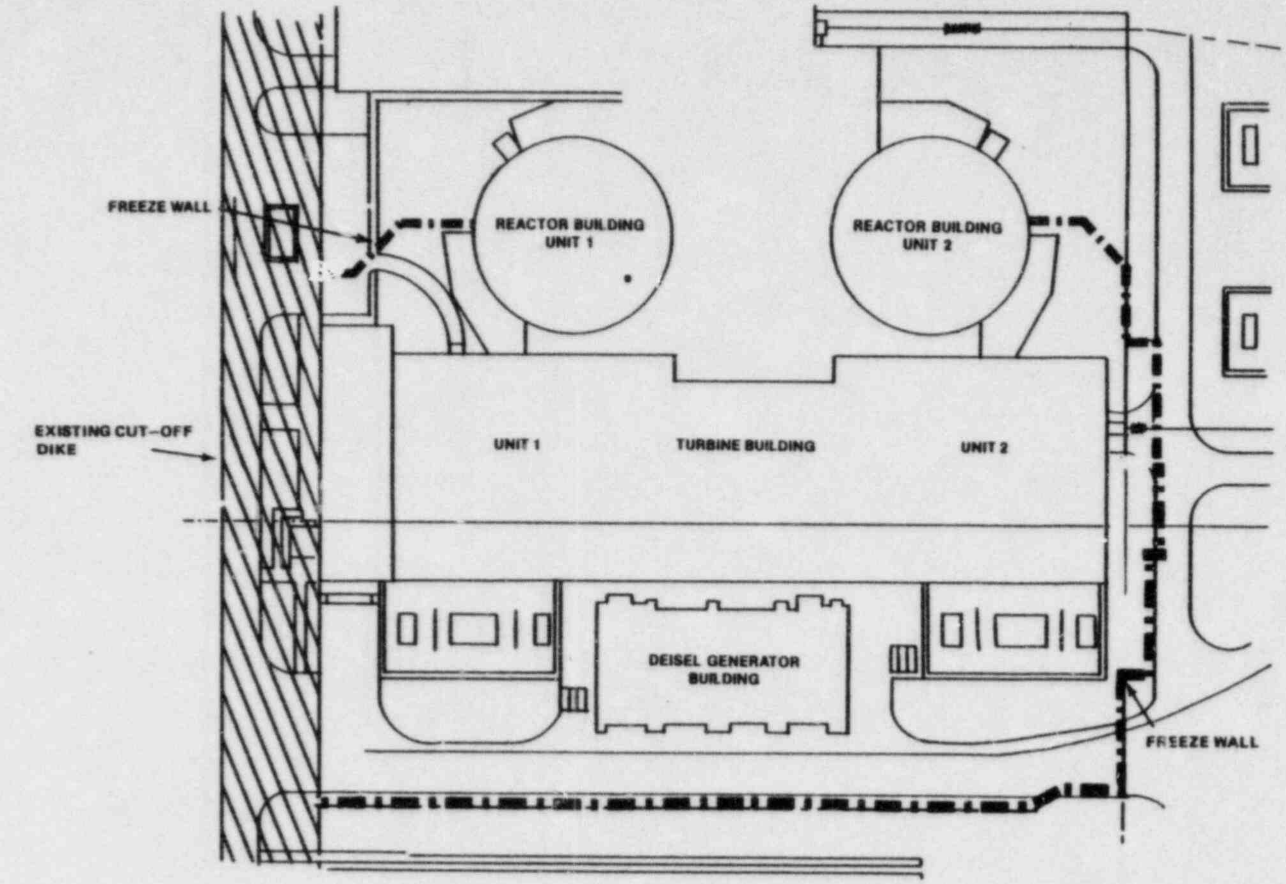


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

UNDERPINNING SECTION AT ELECTRICAL PENETRATION AREA

FIGURE II-10

G-1986-04

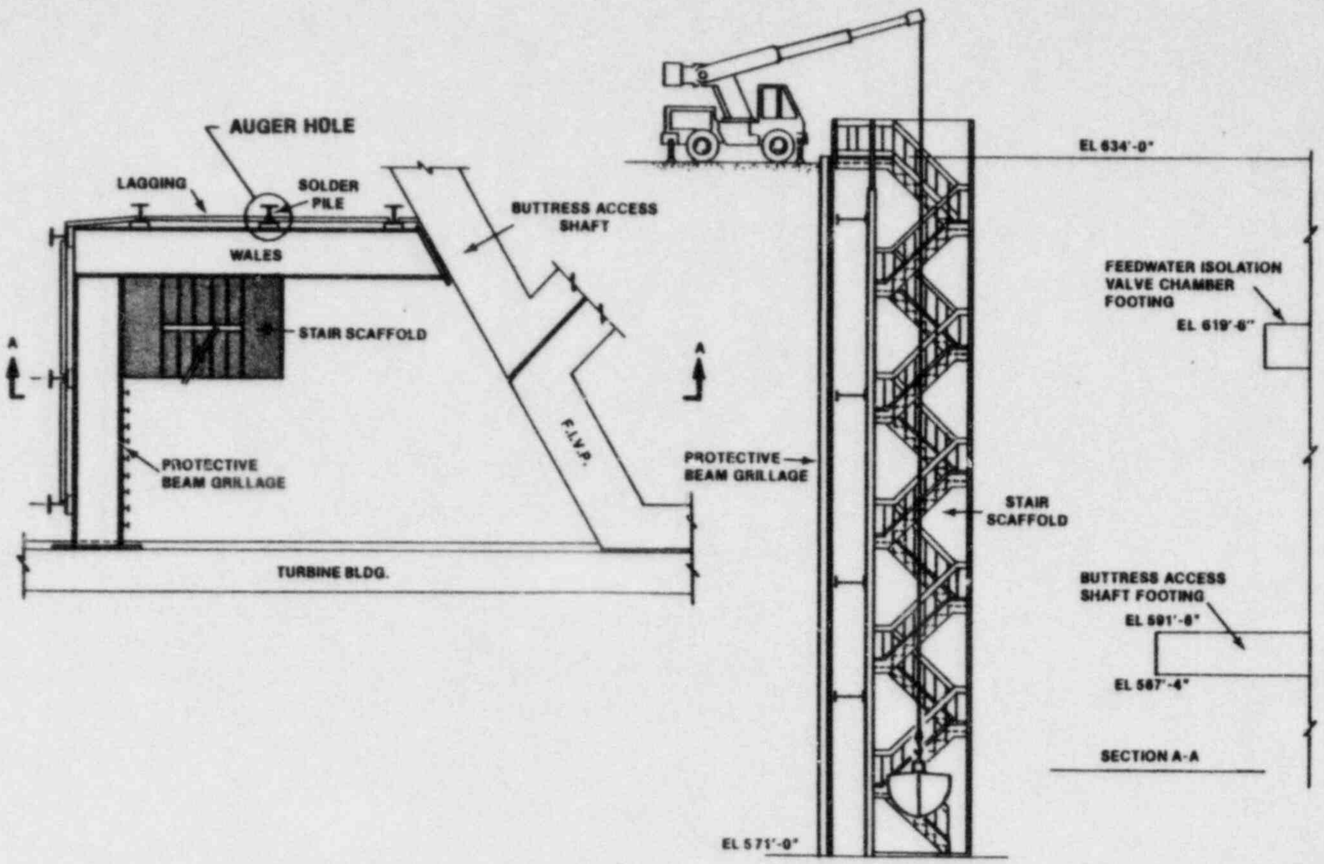


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

FREEZE CURTAIN DAM

FIGURE II-11

G-1986-03



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

ACCESS SHAFT

FIGURE II-12

G-1986-01

PART III: SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

The 1978 settlement of the diesel generator building (DGB) and subsequent plant soil investigation revealed inadequately compacted fill under a portion of the service water pump structure (SWPS).

The SWPS is a two-level, rectangular, reinforced concrete structure. Figure III-1 shows a general arrangement of this building. The foundation slab for the lower part of the building rests on undisturbed natural material. The foundation slab for the upper part of the building rests on plant fill.

The inadequately compacted fill resulted in the need for remedial action for the overhang portion of the structure (the portion founded on fill material). The remedial action is described below.

2.0 DESIGN CONCEPT

The remedial action agreed upon with the NRC staff consists of installing a permanent, continuous underpinning wall under the foundation of the overhang portion of the structure (see Figure III-1). The wall transfers the loads of this part of the structure from the fill to undisturbed natural material. The wall is connected to the existing structure.

2.1 IMPLEMENTATION OF PLAN

The structure, including the underpinning, has been analyzed for the loads from the building, the effects of the 40-year settlement of soil, and environmental effects such as earthquakes and tornados. The dimensions and major details of the underpinning have been finalized, based on a design which used the results of the analyses. The existing structure has been found to be adequate based on these structural analyses and design. The supporting undisturbed material has also been found to be adequate.

The underpinning wall is constructed in small sections (piers) which are tied together to form a continuous wall. The piers are constructed by hand digging pits and filling them with concrete. After a pier is completed, the load from the structure is transferred by jacking to a predetermined value known as initial jacking load.

To construct the underpinning piers, which are below the existing foundations, access is needed from the grade elevation. This access is provided from the outside of the building by open excavation for the piers on the north and the east walls. The access for the piers on the west wall is provided by an access shaft from the grade and a tunnel under the base slab for the overhang portion.

The underpinning is to be constructed in a dry condition. Because the present site dewatering is not adequate to lower the groundwater to the bottom of the underpinning, additional dewatering is accomplished by installing dewatering wells around the areas to be excavated.

The first piers to be constructed are three corner piers at the two corners of the underpinning walls. The completion of these piers is very important to the underpinning operation, because at this stage the entire weight of the overhang can be supported without depending on the fill. Therefore, the loss of fill support is not critical after this stage.

After the corner piers are completed, the remaining piers, except four sections on the east and west walls, are completed based on a predetermined sequence. At this stage the building is supported by initial jacking loads.

The jacking is now adjusted to the final design jacking loads. The settlements are monitored and after the rate of settlements has reached a predetermined value, the jacking load is locked off.

The underpinning is now connected to the structure by anchor bolts and dowels and by constructing the remaining sections on the east and west walls. Also, the gaps between the underpinning and the existing structure are filled with grout. All the excavations are backfilled with fill or concrete. At this stage, the underpinning wall rests on undisturbed material and the underpinning operation is complete.

During the underpinning operation, extreme care must be taken to protect the existing structure. This will be accomplished by removing only small portions of supporting soil and replacing these with piers of greater load-bearing capacity. In addition, the structure will be monitored frequently for strains to ensure that these remain below predetermined limits.

2.2 LICENSING STATUS

The design concept for the SWPS underpinning has been presented and discussed with the NRC staff using several methods: technical reports, meetings, and design audits by the staff.

A technical report describing the underpinning was submitted on August 26, 1981. This was supplemented by responses to NRC staff requests for additional information on November 16, 1981, and by an appendix, dated February 23, 1982, to the August report.

A meeting between the staff and the Applicant was held on September 17, 1981, to discuss both the concept and details of the design. Additional meetings were held on February 23, through 26, 1982, to discuss the finite-element model, construction aspects, and geotechnical issues.

A design audit was conducted in the Bechtel offices at Ann Arbor, Michigan, on March 16 through 19, 1982. During the audit, the staff reviewed the design calculations for the SWPS underpinning.

The design concept of the SWPS underpinning has concurrence from the NRC staff.

3.0 STRUCTURAL ANALYSIS AND DESIGN

The structural analysis of the SWPS and its underpinning is performed in two parts:

a. Seismic analysis using a mathematical model to analyze the structure for the dynamic conditions during a seismic event

b. A static analysis using different models to analyze the structure for the static loads, such as dead, live, and wind loads, etc imposed on the structure.

The results of these two analyses are combined in accordance with applicable load combinations. Load combinations presented in Final Safety Analysis Report (FSAR) Subsection 3.8.6 and supplemented by the Responses to NRC Requests Regarding Plant Fill, Question 15, (Revision 3, September 1979) are used for the structure and the underpinning and its connections to the structure. Additional loading combinations based on American Concrete Institute (ACI) Code 349-76 and supplemented by NRC Regulatory Guide 1.142 are used for the underpinning and its connections to the structure.

3.1 SEISMIC ANALYSIS

A seismic model is developed to evaluate overall building response to seismic forces as well as to generate in-structure response spectra for equipment design. The seismic forces are determined using a lumped-mass model with the response spectrum modal superposition technique. The computed seismic response accelerations are multiplied by the structural element masses to provide the seismic forces for the seismic structural analysis.

The underpinning is designed by staff direction to withstand the effects of the site-specific response spectra (SRSS) ground motion, while the existing structure is evaluated and found acceptable for the effects of the FSAR ground motion description. In order to proceed with the underpinning design while NRC concurrence with the proposed SRSS was being obtained, the structural forces resulting from the FSAR SSE ground motion were multiplied by a factor of 1.5 for design of the underpinning. The response from 1.5 times the FSAR SSE envelops the final SSRS response.

The seismic analysis of the underpinned structure has been completed and the results have been used for the static analysis of the underpinning.

3.2 STATIC ANALYSIS

The static structural analysis uses a finite-element analytical model capable of representing the structure behavior. The interface between the existing structure and the underpinning wall is modeled to transfer loads. The soil media are represented by springs of appropriate stiffness at the base of the structure.

The analysis uses different analytical systems requiring two different models and appropriate springs. The two analytical models that have been developed are used in the following manner.

3.2.1 Disconnected Model

A disconnected model, in which the underpinning wall is not connected to the structure, is used to investigate various construction stages. This model is also utilized in combination with the connected model to determine preload effects on the existing structure due to jacking.

3.2.2 Connected Model

A model in which the underpinning wall is connected to the structure is used to investigate the effects of long-term loading such as differential settlement and short-term loading such as seismic forces. The differential settlement is considered in the model by calculating appropriate spring constants based on settlements of the underpinning and the existing structure.

Based on the properties of the natural materials, it is estimated that the settlement of the underpinned structure after construction is completed will range from 0.1 inch to 0.2 inch for the 40-year life of the structure. The settlement of the main SWPS will range from 0.2 inch to 0.3 inch for the 40-year life of the structure. These predicted settlements are based on an investigation conducted by Woodward-Clyde Consultants (WCC), who performed soil borings and laboratory testing of the undisturbed natural materials. These tests show the preconsolidation pressure of the natural materials to be 48 tons/sq ft.

3.3 DESIGN OF UNDERPINNING

The results of these structural analyses are then factored and added in specific combinations. The results are used to evaluate the structural adequacy of the structure and the underpinning. The computed stresses or loads are ensured to be lower than the allowable stresses or capacities.

The underpinning walls and their connections are designed to meet the requirements set forth in FSAR Subsection 3.8.6 as supplemented by the Responses to NRC Requests Regarding Plant Fill, Question 15, and ACI 349-76 as supplemented by NRC Regulatory Guide 1.142. The capacity of the existing structure is reviewed in accordance with FSAR Subsection 3.8.6 requirements and Question 15 of the Responses to NRC Requests Regarding Plant Fill.

3.3.1 <u>Underpinning</u>

The design features of the underpinning are described below.

The proposed underpinning, as shown in Figure III-1, is a 4-foot thick, reinforced concrete wall that is 30 feet high and is constructed to act as a continuous member under the perimeter of the structure overhang. The entire wall is founded on undisturbed natural material. The base of the north underpinning wall is belled out to a 6-foot thickness to limit bearing pressures to the allowaable values, whereas the bases of the east and west side walls are 4 feet wide.

The allowable bearing pressures for the undisturbed natural material are based on a safety factor of 2 for dynamic loading and 3 for static loading. The ultimate bearing capacity for the natural material is based on the undrained triaxial tests performed on the WCC boring samples. These yielded a median shear strength of 18 ksf.

A jacking force is applied to the overhang perimeter to provide adequate load transfer from the structure to the underpinning. These jacking forces transmit the structural loads through the permanent underpinning wall to the bearing stratum.

Dowels and anchor bolts connect the underpinning walls and the existing structure at the vertical and horizontal interfaces.

The dowels and anchor bolts are designed to transfer shear and tension forces between the structure and the underpinning wall.

3.3.2 Temporary Post-Tensioning

A temporary post-tensioning system is designed to apply a compressive force to the upper part of the building along the north-south exterior walls. This post-tensioning is required to compensate for the loss of buoyancy, which results in additional forces on the overhang, when the construction site dewatering is installed.

The post-tensioning and the access shaft design are based on the ACI 318 and American Institute of Steel Construction (AISC) codes.

3.4 LICENSING STATUS

Structural analysis and design for the underpinning was presented in the technical reports, meetings, and a design audit by the staff, which have been previously identified in Section 2.2.

The seismic analysis was covered in detail during testimony by Dr. R.P. Kennedy of Structural Mechanics Associates and Dr. P. Halada of the U.S. Army Corp of Engineers (representing the NRC staff) during the Atomic Safety and Licensing Board soils hearings of December 14, 1981.

As indicated in Section 2.2, a design audit has been performed by the NRC staff. During this audit, structural analysis and design calculations for the underpinning, access shaft, and post-tensioning were reviewed. The NRC audit resulted in a list of confirmatory issues which the Applicant is addressing and will be prepared to discuss with the NRC staff in the near future.

4.0 CONSTRUCTION SUPPORT PROGRAM

4.1 GROUNDWATER CONTROL

At the start of the underpinning work, it is anticipated that the groundwater level will be about el 600'. Because this underpinning will extend at least 15 feet below this level, the control of groundwater is an important prerequisite for successful completion.

The groundwater level will be lowered below el 585' by using temporary dewatering wells. As part of the temporary dewatering procedure, piezometers will be installed to monitor the groundwater level. These wells will be sealed after the underpinning wall is completed.

The design of the temporary dewatering well system is complete and is shown in Figure III-2.

4.2 CONSTRUCTION ACCESS

The Applicant has recently decided to employ an improved method of access for installation of the hand-dug pits. The method utilizes external access from the outside in lieu of the tunnels shown in Figure III-3. The advantage of this proposal is that it can be better coordinated with the proposed replacing/rebedding of the service water piping north of the SWPS. An access shaft and tunnel will still be installed along the west wall of the overhang because the circulating water intake structure (CWIS) is adjacent to the SWPS on the west side. The underpinning installation sequence will not be altered by adoption of the improved access method.

4.3 LICENSING STATUS

The groundwater control and the improved access method have been discussed with the NRC staff during its recent audit mentioned in Section 2.2. Permission has been received for the installation and activation of the dewatering system.

5.0 CONSTRUCTION PROGRAM

5.1 BUILDING POST-TENSIONING

A temporary post-tensioning system has been installed at the upper part of the building along the north-south exterior walls. The post-tensioning system will be removed after the initial jacking loads are applied.

5.2 UNDERPINNING

This section describes the construction sequence of the underpinning wall. The layout and the sequence are shown in Figure III-3. The underpinning wall is constructed in small sections (piers) to preserve the structural integrity of the building. The first piers to be constructed are approximately 30-foot deep, 5-foot by 4-foot hand-dug sheeted pits located at each corner of the overhang. After the subgrade for these pits is inspected and approved by a geotechnical engineer, reinforcement, subgrade settlement and stress monitoring instrumentation, and anchor bolt assemblies to tie the pier to the underside of the slab are installed. The piers are then encased with concrete. An initial jacking load is applied to the overhang from jacks placed on the pier tops. After jacking, the remaining piers are constructed in the sequence outlined in Figure III-3.

Stress monitoring instrumentation will be installed in designated piers. The piers are tied together with threaded reinforcing bar couplers and shear keys to form a continuous underpinning wall.

The final jacking loads are applied after No. 10 piers (see Figure III-3) are constructed and the underpinning wall has progressed to within 6 feet of the vertical interface with the existing structure. Settlements caused by this load are monitored. When the geotechnical engineer determines that the settlement has decreased to a predetermined rate, the load is transferred from the jacks to wedges positioned between the top of the piers and the underside of the overhang, and the jacks are removed. No. 11 piers are poured, encasing dowel bars that were previously drilled and grouted into the vertical face of the existing structure and thereby connecting the underpinning wall to the existing structure. The space between the top of the underpinning wall and the underside of the base slab is filled with nonshrink grout and previously placed anchor bolt assemblies are tightened. The underpinning wall is connected to the structure at both the vertical and horizontal interfaces. Piers 12 are then constructed, completing the underpinning wall.

5.3 LICENSING STATUS

The construction details and sequence have been discussed with the NRC staff in meetings and during an NRC audit mentioned in Section 2.2. The audit resulted in a list of confirmatory issues which the Applicant is addressing and will be prepared to discuss with the NRC staff in the near future.

6.0 MONITORING PROGRAM

To ensure that installation of the underpinning is proceeding within acceptable limits, a monitoring program will be implemented during construction. This program has four parts:

a. Building settlement

b. Building strain

c. Cracking

d. Underpinning

6.1 BUILDING SETTLEMENT

In addition to the pier settlement monitoring program, a program to closely monitor the overall structure settlement has been planned. Besides the four existing settlement markers at each corner of the building, five additional markers have been installed on the building. A settlement dial indicator has been installed at each of the two north building corners where the underpinning will be constructed. The dial indicators measure displacement between the building and permanent benchmarks founded in undisturbed soil approximately 50 feet below the bottom of the underpinning wall. The depth at which the tip of the benchmark is located ensures that the benchmark movement will be negligible. The settlement markers will be monitored before and after major construction events.

Based upon a request from the NRC during the audit on March 16, through 19, 1982, one additional deep-seated benchmark is being placed on the south side of the structure to monitor settlements.

6.2 STRAIN MONITORING

Before the actual construction of the underpinning wall begins, strain indicating devices with gage lengths of approximately 20 feet will be installed near the top and bottom of the exterior north-south walls at the location of their connection to the existing structure. The strain will be monitored to ensure that it is lower than predetermined levels.

6.3 CRACKS

6.3.1 Existing Crack Evaluation

The existing cracks in the SWPS have been monitored. The size and location of existing cracks have been recorded on crack map drawings. The Applicant's consultant, Portland Cement Association (PCA), evaluated the structural significance of these cracks based on its site visit and review of the crack maps. The consultant concluded that cracks observed in this structure are attributable to restrained volume changes that occur during curing and drying of concrete. PCA also did not observe any structural distress during its visit. Furthermore, PCA concluded that while occurence of stress-related cracking because of differential building settlement cannot be completely dismissed, it did not appear that such hypothesized settlements were a primary cause of cracks observed in this structure. PCA's evaluations and conclusions are contained in a report submitted to the NRC staff on March 3, 1982.

6.3.2 Crack Monitoring During Underpinning

Existing cracks will be monitored for changes in length and width during various phases of construction. The areas containing cracks will be inspected for new cracks that, if present, will be similarly mapped and monitored.

Because of the sequence of construction procedures, it is not anticipated that existing cracks will significantly widen or that significant new cracks will appear. However, any new structural cracks exceeding 0.01 inch in width or any crack exceeding 0.03 inch in width will be evaluated by PCA to determine whether underpinning operations should stop or continue. If development of yield strain is inferred from any observed crack, underpinning will be stopped and an evaluation made by PCA before continuing underpinning operations.

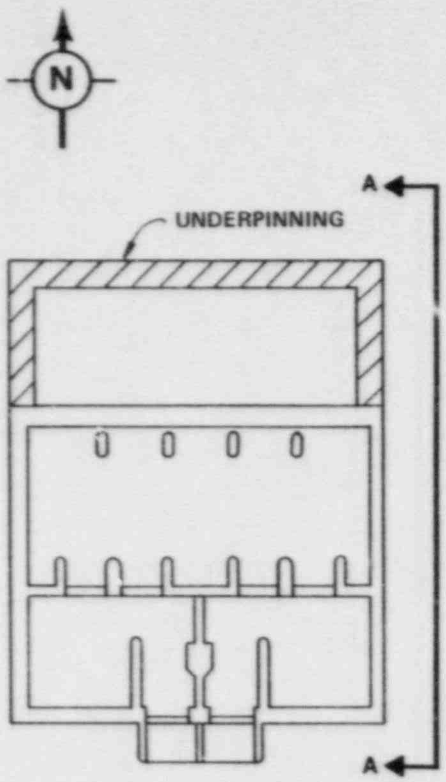
6.3.3 Repair of Cracks

A report on a crack repair program by PCA for all cracks in all structures will be submitted to the NRC staff in the near future.

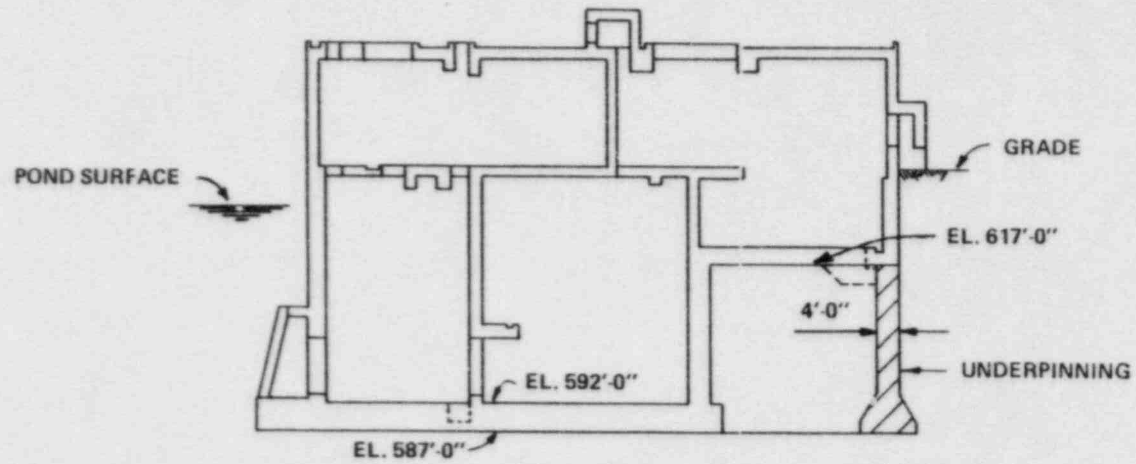
6.4 UNDERPINNING

A settlement monitoring program for the top and base of each pier begins immediately after pier construction. Instruments accurate to 0.001 inch are installed before the initial jacking is applied. The information from this monitoring program is used to evaluate the time required to dissipate shrinkage and creep of the concrete and the time when settlement of the undisturbed natural material below the underpinning wall has reached a predetermined rate.

Stress meters will be cast in concrete near the top and bottom of designated piers. These instruments will monitor variations in applied loads.



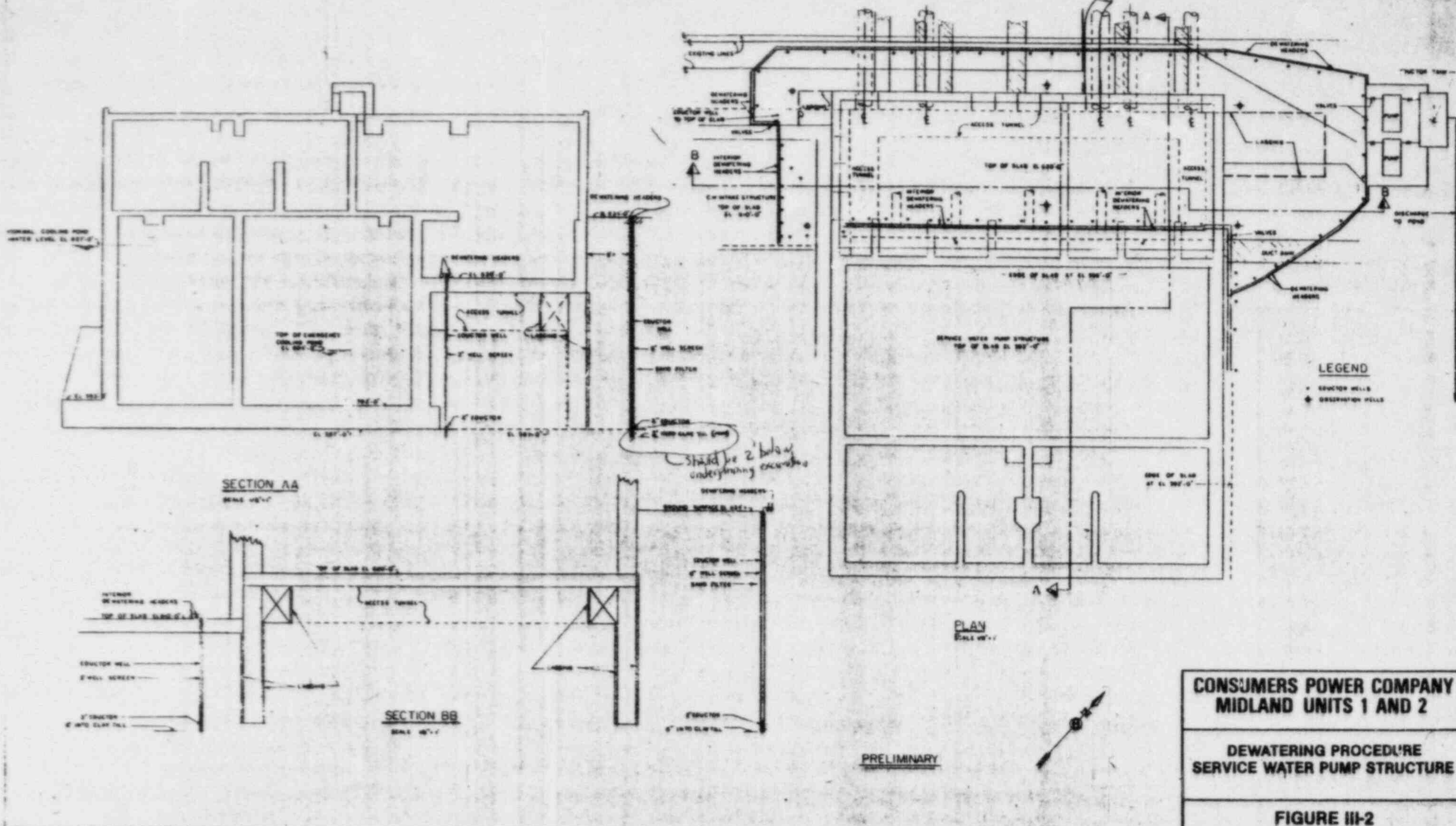
PLAN AT EL 592'-0"



ELEVATION A-A

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
UNDERPINNING GENERAL LAYOUT
FIGURE III-1

G-1986-30

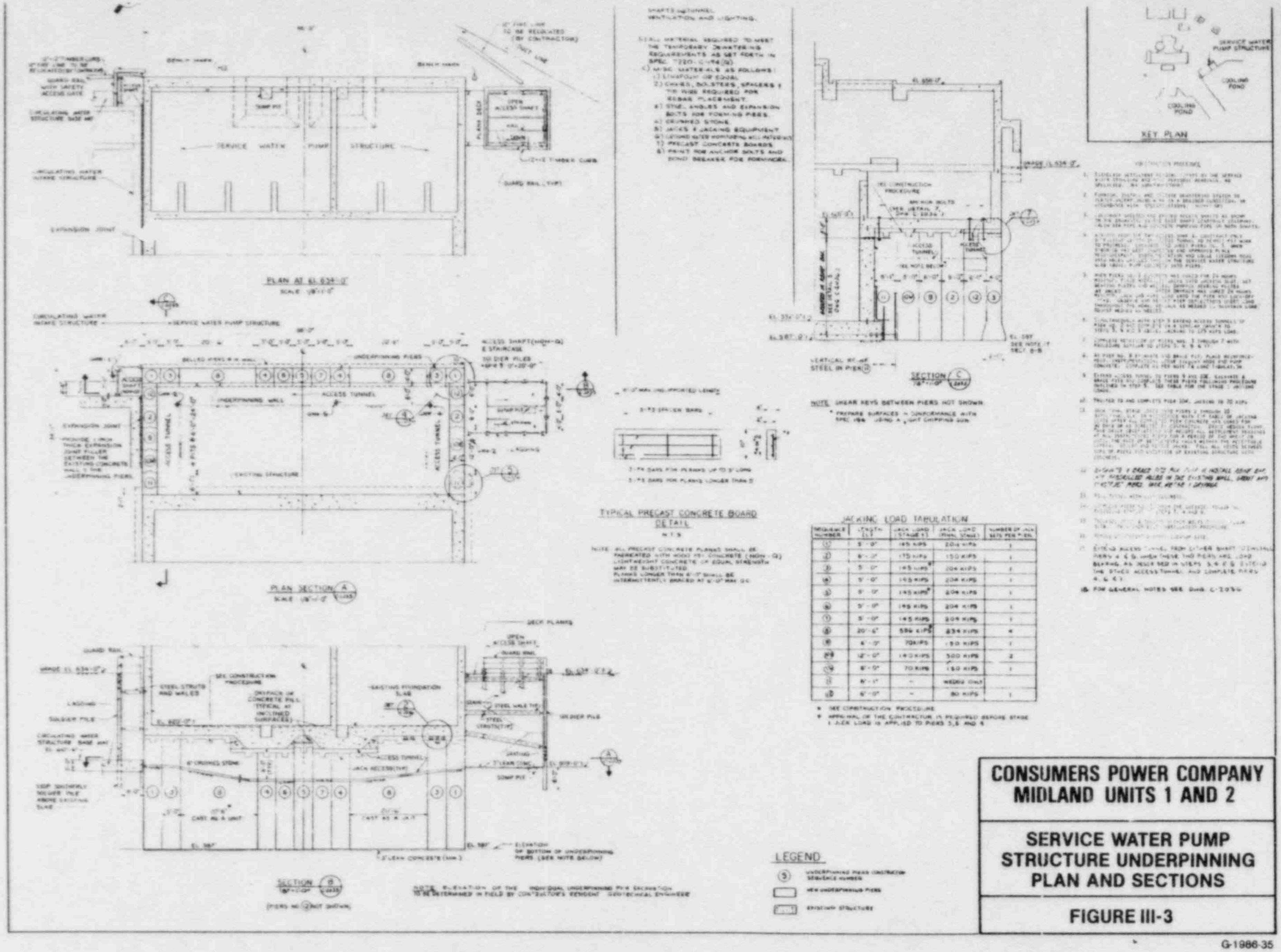


CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

DEWATERING PROCEDURE
SERVICE WATER PUMP STRUCTURE

FIGURE III-2

G-1960-33



JACKING LOAD TABULATION

SEQUENCE NUMBER	LENGTH (L)	JACK LOAD (STAGE I)	JACK LOAD (FINAL STAGE)	NUMBER OF JACK SETS PER PIER
1	5'-0"	145 KIPS	204 KIPS	1
2	6'-0"	175 KIPS	150 KIPS	1
3	5'-0"	145 KIPS•	204 KIPS	1
4	5'-0"	145 KIPS	204 KIPS	1
5	5'-0"	145 KIPS•	204 KIPS	1
6	5'-0"	145 KIPS	204 KIPS	1
7	5'-0"	145 KIPS	204 KIPS	1
8	20'-6"	586 KIPS•	834 KIPS	4
9	6'-0"	70 KIPS	150 KIPS	1
[illegible]	12'-0"	140 KIPS	300 KIPS	2
[illegible]	8'-0"	70 KIPS	150 KIPS	1
[illegible]	8'-1"	–	WEDGE ONLY	
[illegible]	6'-0"	–	80 KIPS	1

* SEE CONSTRUCTION PROCEDURE

• APPROVAL OF THE CONTRACTOR IS REQUIRED BEFORE STAGE I JACK LOAD IS APPLIED TO PIERS 3, 5 AND 8

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SERVICE WATER PUMP STRUCTURE UNDERPINNING PLAN AND SECTIONS

FIGURE III-3

G-1986-35

PART IV: BORATED WATER STORAGE TANKS

1.0 INTRODUCTION

Each unit of the Midland plant has a 500,000 gallon stainless steel borated water storage tank (BWST) located in the tank farm north of the auxiliary building. The tanks are 32 feet high and 52 feet in diameter and sit on a concrete foundation (see Figure IV-1).

A soils investigation program of the fill in the tank farm area, consisting of 40 borings, two test pits, and two plate load tests, was conducted. This program revealed that the fill in the area of the Unit 1 and 2 BWSTs varied from medium to very stiff clay backfill with occasional medium to very dense sand layers over dense to very dense natural sand. This fill was determined adequate to provide support for the postulated loadings from the tanks.

To develop a conservative, long-term settlement prediction, a load test was performed. This test consisted of filling the completed tanks with water. Several weeks after initiation of the test for the Unit 1 tank, a discrepancy was noted between measurements of settlement and the computed displacements derived from the structural analysis used at that time. As a result, the analysis was modified to include a finite-element model of the soil subgrade. A number of analyses were performed using various values for the modulus of elasticity (E) of the soil until the calculated foundation curvature became more severe than observed. The results of the analyses predicted that greater than allowable moments existed at several locations in the foundation (see Figure IV-2).

The foundation at these locations was examined to verify whether visible signs of high reinforcement strain existed. Cracks were found in the structure at those locations indicated by the analysis as having greater than allowable moments. The largest crack measured 0.063 inch. Subsequently, the Unit 2 tank foundation was also examined; similar cracks were found, and the largest crack measured 0.035 inch.

Additional engineering analysis determined that the valve pit, which was lightly loaded, acted as a partial end support and resulted in nonuniform loading of the foundation. This loading condition created differential settlement and localized areas of overstress.

2.0 DESIGN CONCEPT

2.1 CONCRETE FOUNDATION

A two-stage corrective action plan has been adopted for the concrete foundation repair for each tank.

2.1.1 Surcharge Program

Figure IV-3 shows the outline of surcharges that were applied to each valve pit for 4 months. The surcharge consolidated the fill beneath the valve pits, thereby reducing the amount of residual differential settlement of the foundation structure over the 40-year life of the plant. It also provided the added benefit of reducing the ring wall distortion.

2.1.2 Additional Ring Beam

Figures IV-4 and 5 show details of a reinforced concrete ring beam which will be constructed around each existing ring beam. The new ring beam is sized to resist all imposed loading from the tank, including additional future bending induced by the 40-year predicted residual differential settlement between the ring wall and the valve pit. (The predicted value, which was determined from the more severe extrapolated Unit 1 data before applying the surcharge, has not been reduced to account for the beneficial effects of the surcharge and, therefore, is conservative.) All cracks in the existing ring wall that exceed 10 mils will be repaired by pressure grouting. Shear connectors will be installed to transfer the force from the existing ring wall to the new ring beam. One end of the shear connectors will be installed in the existing ring wall by drilling and grouting. The other end will be cast in the new ring beam.

2.2 TANK

The Unit 1 tank (BWST 1T-60) will be releveled after new ring beam construction is complete. The Unit 2 tank (BWST 2T-60) need not be releveled because stresses associated with present plus future predicted differential settlement effects remain within Code-allowable values. Details of the analysis for BWST 1T-60 are provided in Section 3.3.

A detailed procedure has been developed to define a plan of action to relevel BWST 1T-60. This procedure is supported by an analysis that demonstrates that the tank will not be overstressed during this operation. Strain gaging of the tank will be used as a backup to this analysis. This procedure is to be submitted to the NRC staff for review and concurrence prior to performing the work. A brief summary of the procedure is provided below.

a. Vent and drain the tank

b. Mount strain gages

c. Attach electromechanical jacks to the anchor bolt chairs

d. Lift the vessel approximately 3 feet. (All jacks will be controlled from a central control panel and will lift at the same rate and time.)

e. Support tank with cribbing

f. Install Celotex cofferdam around the inner diameter of the ring wall to contain grout placed in Steps l and m below

g. Add and contour oil-impregnated sand

h. Clean the top surface of the ring wall

i. Place stainless steel shims on the original concrete ring wall. Level to a common datum plane above the ring wall. Set shims to the following standard:

   1) 1/8 inch within any 30 feet of circumference

   2) 1/4 inch over total circumference

j. Place Celotex in the areas between the shims

k. Remove cribbing and lower the tank

l. Add nonshrink grout under the tank bottom and allow grout to set

m. Remove the shims, install Celotex, and grout the remaining gaps

3.0 STRUCTURAL ANALYSIS AND DESIGN

3.1 SEISMIC

The preliminary seismic analyses for the BWST foundations are described in Appendix A of the design report submitted by the Applicant to the NRC on November 13, 1981. The final seismic analyses were explained in a November 24, 1981, addendum to the design report. The final seismic analyses are also discussed in testimony by Dr. R.P. Kennedy during the ASLB hearing on December 14, 1981.

The preliminary analyses conservatively determined the seismic shear and overturning moment on the BWST ring foundation from a horizontal final safety analysis report (FSAR) safe shutdown earthquake (SSE). The model included the sloshing and impulsive behaviors of fluid in the tank along with the soil-structure interaction effects. The tank shell was assumed to be rigid. A similar model was used to conservatively determine the seismic forces on the foundation from a vertical FSAR SSE, except there is no sloshing of fluid involved in this case.

Final seismic analyses were performed by Dr. R.P. Kennedy of Structural Mechanics Associates (SMA). The models were the same as those used for the preliminary analyses, except the tank shell was modeled in greater detail for the horizontal seismic load case.

The preliminary and final analyses were also performed to determine the seismic forces on the BWST foundation from earthquakes corresponding to site-specific response spectra (SSRS). The results showed that the forces from the SSRS are smaller than those from 1.5 times FSAR SSE. Also, the preliminary analyses gave consistently higher forces than those from the final analyses. The forces from preliminary analyses for 1.5 times FSAR SSE were used for the BWST ring foundation modification; hence, the design is conservative.

3.2 CONCRETE FOUNDATION DESIGN

3.2.1 Loads, Loading Combinations, and Acceptance Criteria

The modified BWST foundations are designed in accordance with the loading requirements and acceptance criteria for Seismic Category I structures using the load combinations presented in the FSAR.

Because of the presence of differential settlement, four additional load combinations as outlined in the response to NRC Requests Regarding Plant Fill, Question 15 (Revision 3, September 1979) have also been included in the design.

The new ring beam and shear connectors have been designed to withstand the load combinations of American Concrete Institute (ACI) 349-76 as supplemented by Regulatory Guide 1.142.

3.2.2 Static Finite-Element Model

The modified BWST foundation was analyzed by the finite-element method using the Bechtel Structural Analysis Program (BSAP). Because the tank has a flexible bottom, the water and tank bottom loads above the soil are transferred directly to the soil. To account for the settlement effect of the soil from this load, the soil subgrade was modeled in the analysis. The model is divided into two parts: the foundation structure and the soil subgrade. These two parts are connected at the common nodal points at the bottom of the foundation and the outside periphery. At the locations where significant cracks were observed in the ring wall and footing, the thickness of the existing ring wall was reduced by 50% in calculating the thickness of the elements in the model. This increase in flexibility of the foundation structure simulated the effect of cracks.

3.2.3 Soils

3.2.3.1 Elastic Modulus of Soil

Short-term and long-term modulii were developed and utilized in the static finite-element analysis. The long-term modulus is used when considering the effects of settlement combined with dead and live load. The short-term modulus is used for all other loading conditions.

The predicted foundation differential settlement from the finite-element analysis using the long-term modulus is more severe than the 40-year differential settlement prediction based on the Unit 1 load test; hence, the design of the new ring beam is conservative.

3.2.3.2 Foundation Bearing Pressures

The results of the finite-element analysis indicate all the soil elements immediately beneath the foundation structure are in compression for dead load and live load conditions. This behavior indicates that the structure is not lifting off the soil or the soil is not settling down away from the structure at any point. In short, the soil and foundation are displacing in a compatible manner without separation. The maximum calculated soil pressures are within the allowable values for the static and dynamic conditions.

3.3 TANK

3.3.1 Condition Prior to Foundation Repair

A finite-element analysis was conducted on BWST 1T-60 to determine the condition of the tank. Information used in the analysis included survey measurements of the elevations, field measurements of the anchor bolt loads (determined by strain gaging the bolts), a history of the tank filling and draining, and the compressibility of the asphalt-impregnated fibreboard (located between the tank bottom plate and the ring foundation) determined by laboratory testing.

All loads were known from the experimentally determined anchor bolt loads and the weight of tank components. The nonuniform support reactions and resulting tank wall stresses were computed utilizing the finite-element model.

The normal operating stress limits of the governing design code [American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, Subsection NC, 1974, supplemented by ASME Code Case 1607-1 to establish allowable stresses for conditions other than normal operation (infrequent events)] were met with two exceptions.

One exception was that the most highly loaded bolt chair top plate did not meet normal operating stress limits, but the emergency event loading criteria for an ASME Code Class 1 plate and shell-type component support were met. A subsequent dye penetrant examination of the top plate welds verified that no cracking was present.

The other exception was local tank wall compressive stresses which did not meet normal operating stress limits. The emergency event buckling criterion was used to verify freedom from buckling. A buckling factor of safety of 2.46 was also calculated to demonstrate that a large margin existed for tank buckling. A visual examination of the tanks was performed while they were under their most highly stressed conditions to verify that no buckling was present.

It is concluded that the uneven tank support which resulted from soil settlement has not resulted in any damage to the BWSTs, that their design basis has not been violated, and that their safe operating life has not been reduced.

3.3.2 Condition After Releveling

A finite-element analysis has been conducted on BWST 1T-60 to determine the tank condition over the operating life of the Midland plant after releveling. An analysis for BWST 2T-60 was not required because BWST 1T-60 had the more severe predicted future settlement pattern. Two loading cases were evaluated: 1) normal operating loads plus settlement, and 2) normal operating loads plus settlement, combined with the effects of the SSRS earthquake. The modeling technique used was that described in Section 3.3.1. The computed stresses are within Code allowables for each case.

4.0 MONITORING PROGRAM

After the new ring beam is constructed, two observation pits will be provided for each BWST foundation at the high stress locations. The new ring beams will be monitored monthly for possible cracks under service conditions for 6 months after filling the tanks. At the end of the monitoring period, a report evaluating cracks will be submitted to the NRC. If during the monitoring period any cracks are noted which are 30 mils or larger, an engineering evaluation will be conducted to determine corrective action.

BWST foundation settlement will also be monitored as part of the foundation survey. Foundations are surveyed at 60-day intervals during construction and at 90-day intervals for the first year of plant operation. Subsequent survey frequency will be established after evaluating the data taken during the first year of plant operation. As a minimum, the tank foundation would be monitored annually for the next 5 years of operation and at 5-year intervals thereafter.

The critical areas of each foundation at the transition zone between the ring wall and the valve pit will be monitored using a strain gage system. This system will be monitored at the same frequency as the foundation survey using established acceptance criteria.

5.0 CONSTRUCTION

The NRC has given its concurrence to the repair of cracks in the existing foundation. Preparation for this work is under way.

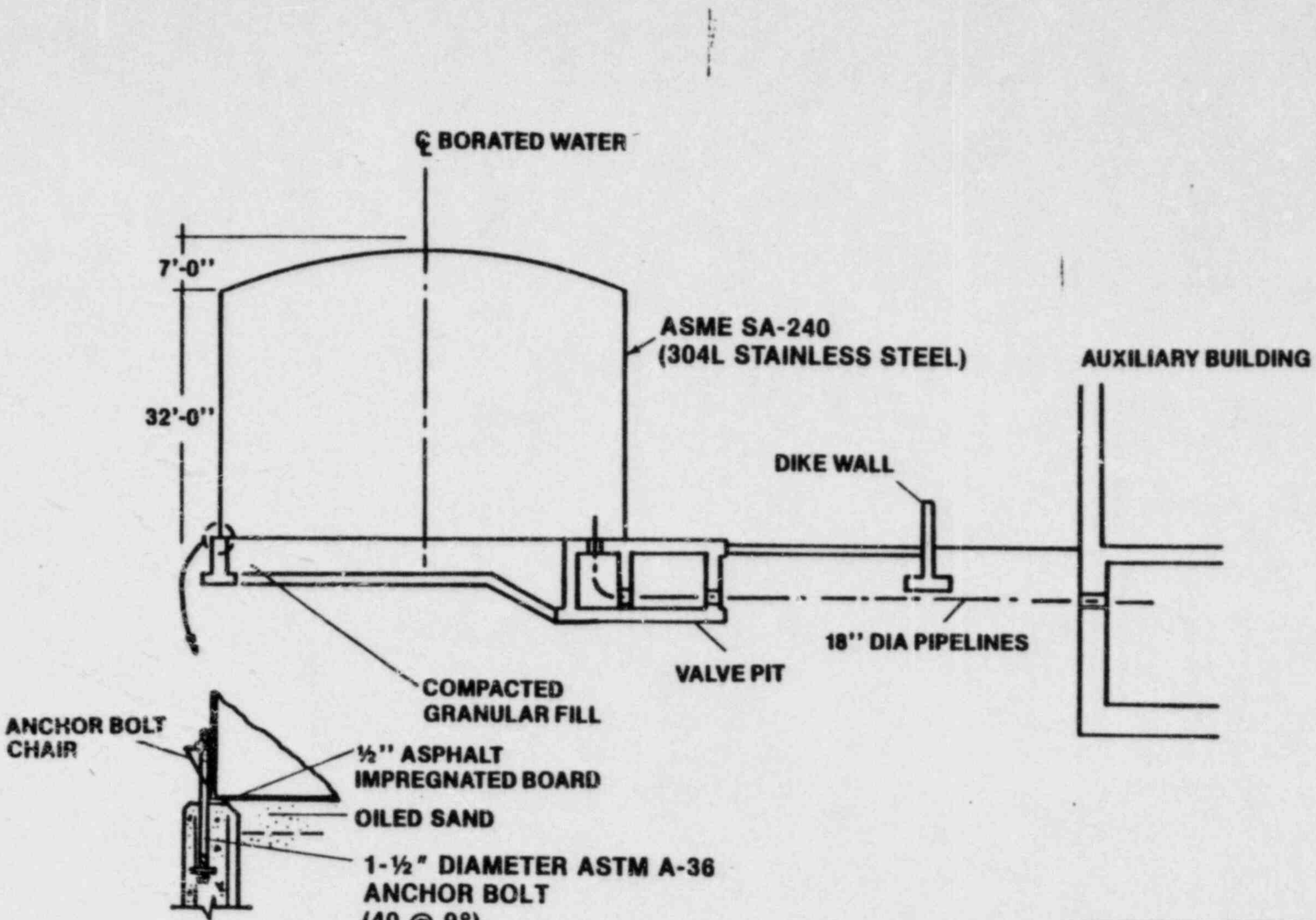
6.0 LICENSING STATUS

The remedial plan for the BWSTs has been presented in meetings and reports to the NRC and in the Atomic Safety and Licensing Board (ASLB) hearing. Meetings were held on May 7, 1981, to discuss the concept of building an additional ring; on August 3, 1981, to discuss application of the surcharge to the valve pit; on January 13, 1982, to discuss the analysis of the existing condition of the tanks; and during the January 18 through 20, 1982, audit to discuss crack repair, tank releveling, and analysis techniques.

A technical report was submitted on November 13, 1981, which described the design concept, provided details of the seismic and static analytical methods, and presented construction details. An addendum to the report was submitted on November 24, 1981, which provided the results from the final seismic analysis and verification that design acceptance criteria had been met.

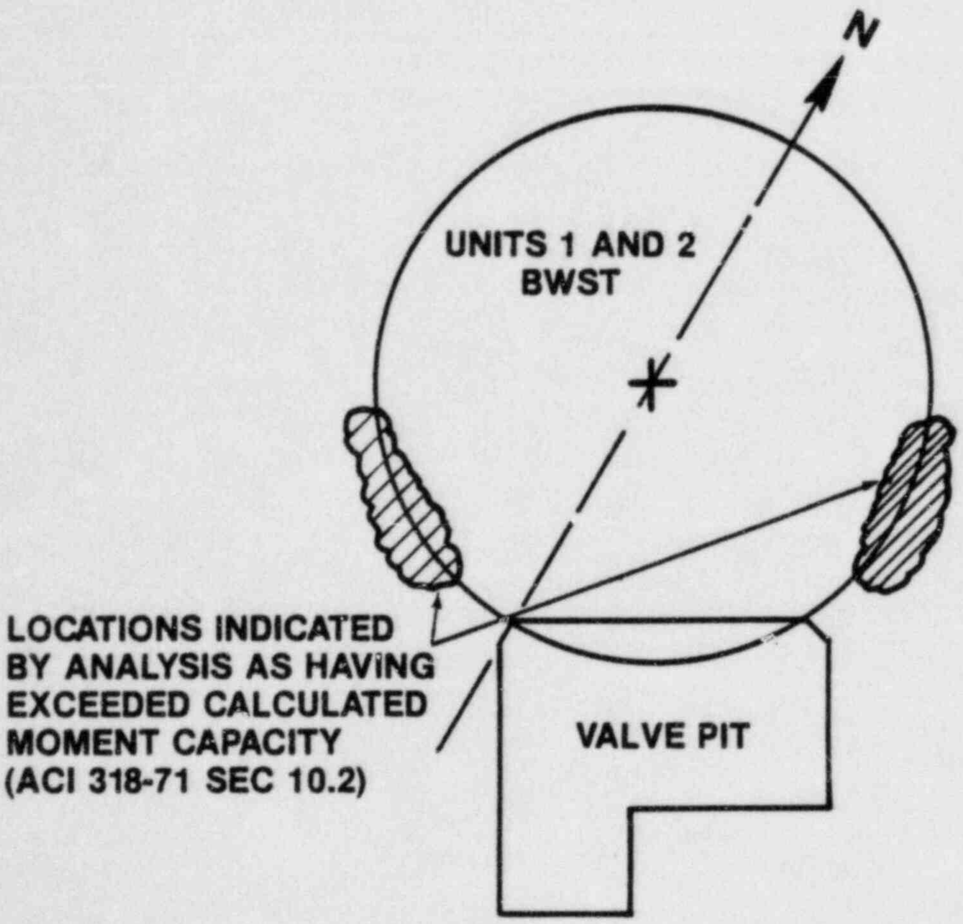
In the December 14, 1981, ASLB hearing, Drs. R.P. Kennedy of SMA and P. Hadala of the U.S. Army Corps of Engineers (consultant to the NRC) testified to the adequacy of the seismic model and associated analyses. During the February 16 through 19, 1982, ASLB hearing, the Applicant and NRC staff testified to the adequacy of the proposed remedial plan and the acceptance of the tanks.

The NRC staff intends to audit the final design calculations. The NRC staff has documented its concurrence on the application and removal of the surcharge and the repair of cracks in the existing foundation.



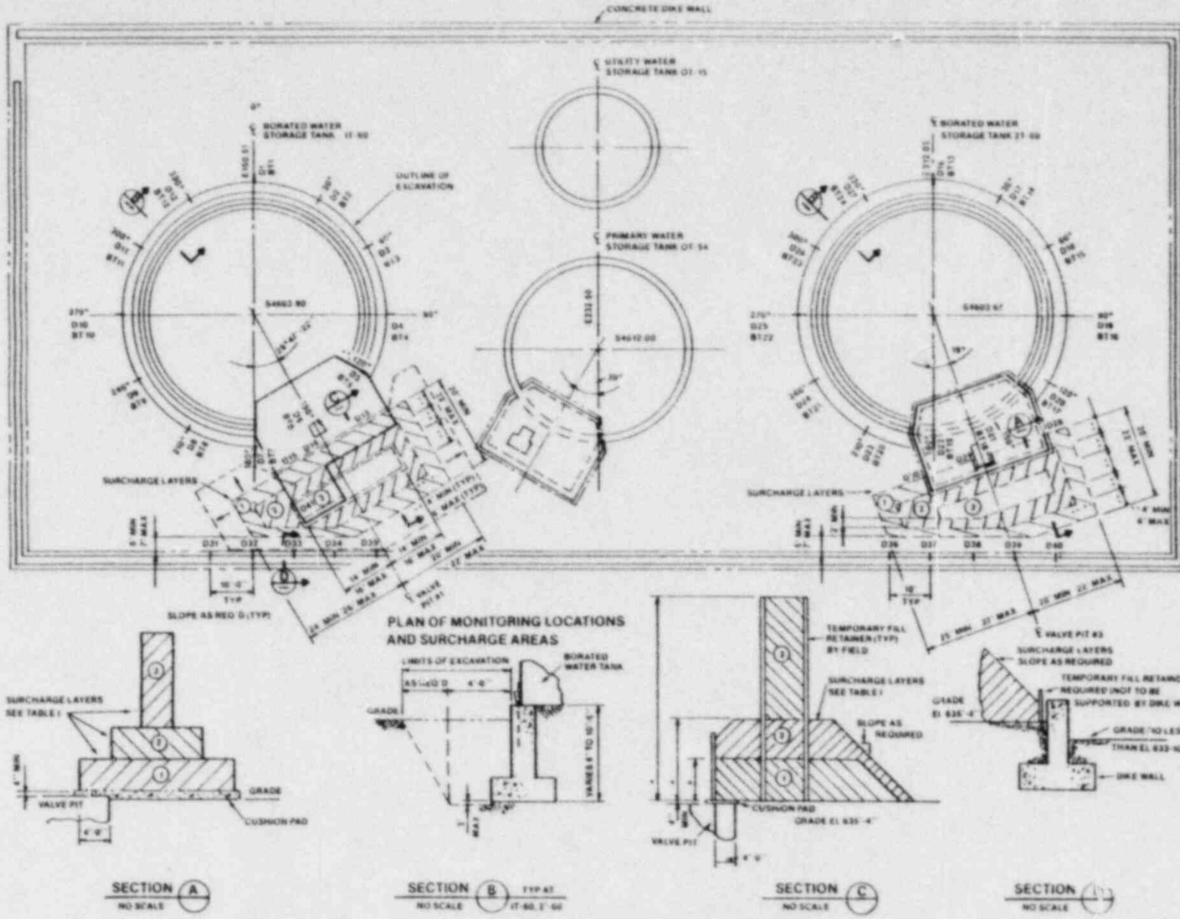
CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
BORATED WATER STORAGE TANK
FIGURE IV-1

G-1986-27



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
LOCATIONS OF EXCESSIVE MOMENT
FIGURE IV-2

G-1986-18



* Granular fill shall be placed to a height such that its weight is equivalent to the weight of the concrete blocks in the corresponding area.

TABLE II
SETTLEMENT MONITORING

DATA BASE NO.	LOCATION 1T-60	ELEVATION (approx)	DATA BASE NO.	LOCATION 2T-60	ELEVATION (approx)
D1	0°	635'-0"	D16	0°	635'-0"
D2	30°	635'-0"	D17	30°	635'-0"
D3	60°	635'-0"	D18	60°	635'-0"
D4	90°	635'-0"	D19	90°	635'-0"
D5	120°	635'-0"	D20	120°	635'-0"
D6	150°	635'-0"	D21	150°	635'-0"
D7	180°	635'-0"	D22	180°	635'-0"
D8	210°	635'-0"	D23	210°	635'-0"
D9	240°	635'-0"	D24	240°	635'-0"
D10	270°	635'-0"	D25	270°	635'-0"
D11	300°	635'-0"	D26	300°	635'-0"
D12	330°	635'-0"	D27	330°	635'-0"
D13	VALVE PIT	635'-0"	D28	VALVE PIT	635'-0"
D14	VALVE PIT	635'-0"	D29	VALVE PIT	635'-0"
D15	VALVE PIT	635'-0"	D30	VALVE PIT	635'-0"
D41	VALVE PIT	628'-4"			
D31	DIKE WALL	637'-6"	D36	DIKE WALL	637'-6"
D32	DIKE WALL	637'-6"	D37	DIKE WALL	637'-6"
D33	DIKE WALL	637'-6"	D38	DIKE WALL	637'-6"
D34	DIKE WALL	637'-6"	D39	DIKE WALL	637'-6"
D35	DIKE WALL	637'-6"	D40	DIKE WALL	637'-6"
BT1	0°	635'-6"	BT13	0°	635'-6"
BT2	30°	635'-6"	BT14	30°	635'-6"
BT3	60°	635'-6"	BT15	60°	635'-6"
BT4	90°	635'-6"	BT16	90°	635'-6"
BT5	120°	635'-6"	BT17	120°	635'-6"
BT6	150°	635'-6"	BT18	150°	635'-6"
BT7	180°	635'-6"	BT19	180°	635'-6"
BT8	210°	635'-6"	BT20	210°	635'-6"
BT9	240°	635'-6"	BT21	240°	635'-6"
BT10	270°	635'-6"	BT22	270°	635'-6"
BT11	300°	635'-6"	BT23	300°	635'-6"
BT12	330°	635'-6"	BT24	330°	635'-6"

TABLE I
SURCHARGE SEQUENCE

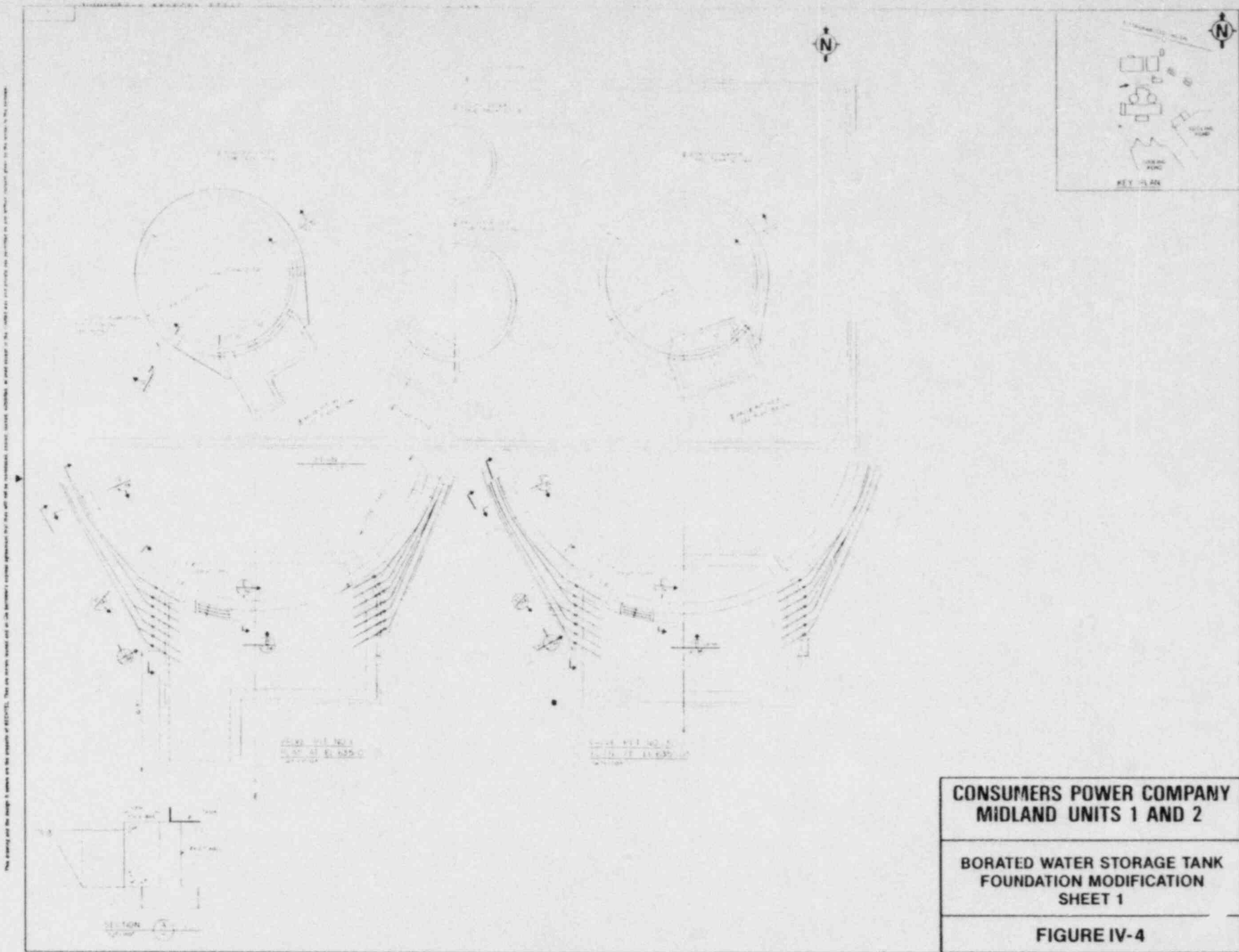
STEP	ACTIVITIES	HT OF SURCHARGE SEE NOTE 3	
		1T-60	2T-60
I	1. UTILITY MONITORING SHALL BE COMPLETED 2. CUSHION PAD SHALL BE COMPLETED 3. WATER HEIGHT IN TANK AND ANCHOR BOLT STATUS SHALL BE AS DIRECTED BY PROJECT ENGINEERING	0'-0"	0'-0"
II	1. PLACE SURCHARGE LAYER 1 AND HOLD FOR TWO WEEKS	SEE SECTION C	4'-0" MIN 6'-0" MAX
III	1. PLACE SURCHARGE LAYER 2 AND HOLD FOR TWO WEEKS	SEE SECTION C	8'-0" MIN 10'-0" MAX
IV	1. PLACE SURCHARGE LAYER 3 AND HOLD FOR SETTLEMENT DATA AS DIRECTED BY PROJECT ENGINEERING	SEE SECTION C	20'-0" MIN 22'-0" MAX
V	1. REMOVE SURCHARGE LAYER 3 AND HOLD FOR TWO WEEKS	SEE SECTION C	10'-0" MIN 12'-0" MAX
VI	1. REMOVE REMAINDER OF SURCHARGE LAYERS 1, 2 AND TAKE REBOUND DATA	0'-0"	0'-0"

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

BORATED WATER STORAGE TANKS VALVE PIT SURCHARGE PROGRAM

FIGURE IV-3

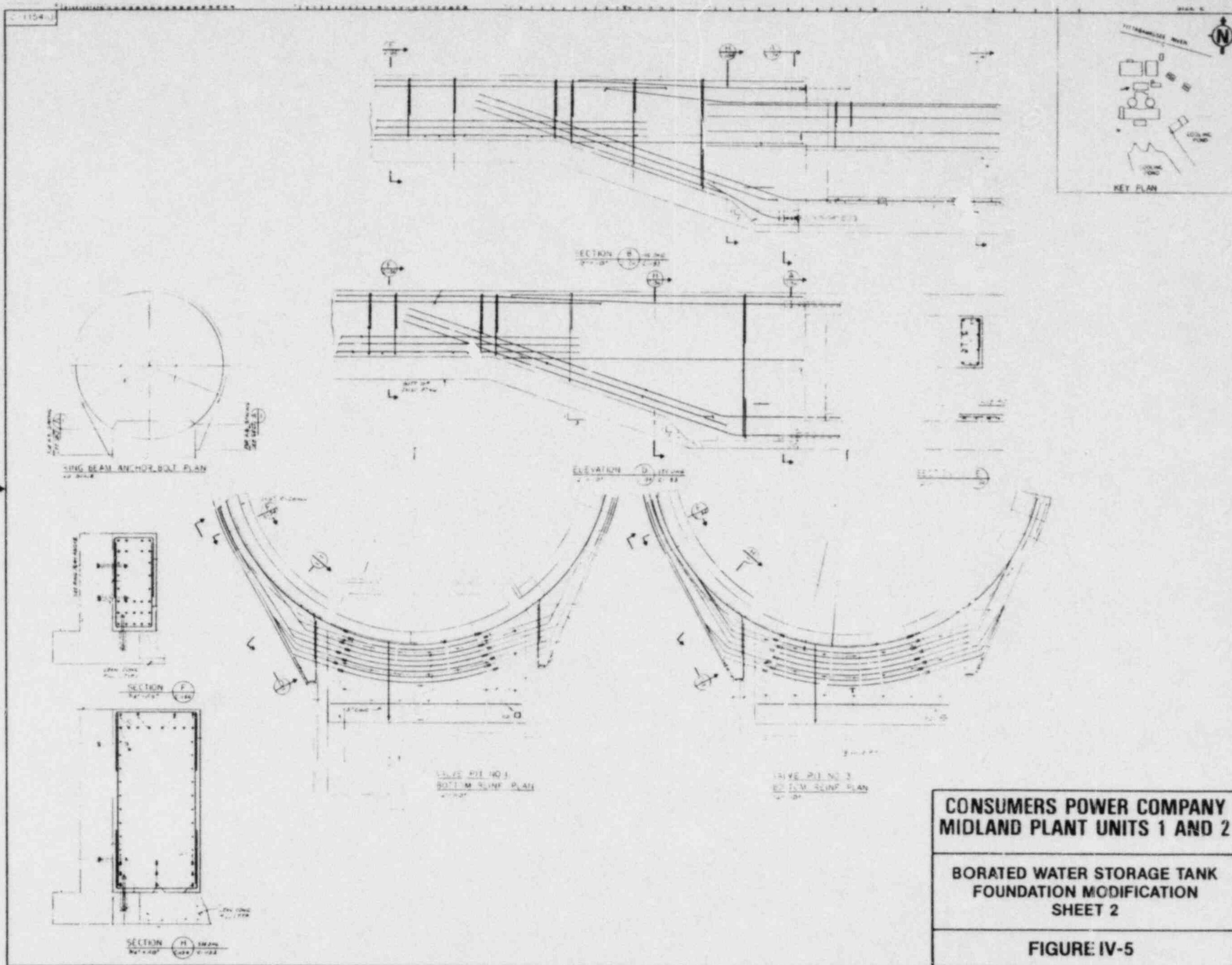
G-1986-28



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

BORATED WATER STORAGE TANK
FOUNDATION MODIFICATION
SHEET 1

FIGURE IV-4



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

BORATED WATER STORAGE TANK
FOUNDATION MODIFICATION
SHEET 2

FIGURE IV-5

PART V: PERMANENT DEWATERING

1.0 INTRODUCTION

As a result of the site soils exploration program conducted after the discovery of the diesel generator building (DGB) settlement problem, pockets of potentially liquefiable granular backfill materials have been discovered supporting some Seismic Category I structures and buried utilities. Facilities affected include: the DGB, auxiliary building electrical penetration areas, auxiliary building railroad bay, the cantilevered section of the service water pump structure (SWPS), and a portion of the service water piping adjacent to the DGB, auxiliary building, circulating water intake structure (CWIS), and the SWPS.

Evaluation of site exploration data performed by the Applicant, the NRC staff, and its consultant (the U.S. Army Corps of Engineers) concluded that loose granular backfill supporting Seismic Category I facilities is safe against liquefaction for earthquakes that produce a peak ground surface acceleration of 0.19g or less provided the groundwater elevation in the backfill is maintained at or below el 610'.

The auxiliary building electrical penetration areas and the cantilevered portion of the SWPS will be underpinned. The service water piping adjacent to the CWIS and SWPS will be excavated to at least el 610' and rebedded to meet design requirements. These remedial steps will eliminate liquefaction as a potential problem in these areas.

In the area of the DGB and auxiliary building railroad bay, there is still a potential during the safe shutdown earthquake (SSE), which is less than 0.19g, for liquefaction in saturated backfill sands that exist above el 610'. Critical areas where the groundwater levels have to be maintained below el 610' in granular backfill supporting Seismic Category I structures and buried utilities are shown in Figure V-1.

2.0 DESIGN CONCEPT

To eliminate the potential for liquefaction during the design SSE in loose, saturated granular backfill materials for the areas designated in Figure V-1, a permanent plant dewatering system has been designed to remove the water from the backfill sands and maintain it below el 610'.

The permanent dewatering system operating level has been selected to be el 595'. This level was selected, based upon site tests, to provide time for repair or replacement of the system before groundwater levels would rise above el 610' at the critical areas.

The permanent dewatering system consists of two subsystems: interceptor wells and area dewatering wells. The design of these systems accounts for the two basic findings of the exploration and testing program: 1) The granular backfill materials are hydraulically connected to the underlying natural sands, and 2) The cooling pond, at el 627', is the main source of recharge, and seepage from the pond is occurring primarily at the CWIS and SWPS.

The dewatering system will be monitored during plant operation. This will ensure that the water level stays below el 610', that soil particles removed are below predetermined levels, and that water quality is acceptable for disposal. X

The system has also been designed to ensure its operation during various accident conditions, including power outages, loss of wells, and pipe breakage.

The NRC staff has been provided information about the dewatering system design in response to 10 CFR 50.54(f), Questions 24, 47, and 49 through 53, and letters from the Applicant to H.R. Denton dated April 24, 1981; May 28, 1981; and September 16, 1981. The NRC staff have concurred with the proposed system.

3.0 ANALYSIS AND DESIGN

The permanent dewatering system design is based on an evaluation of design drawings and construction records, test boring information, field and laboratory test results, observation well and piezometer data, and pumping test results. The data obtained from these activities include type, distribution, and permeability of materials; zones of recharge and drawdown; and recharge and pumping rates. This information has been used to determine the location, spacing, size, and depth of the dewatering wells.

As stated earlier, the system consists of two subsystems: interceptor wells and area dewatering wells.

3.1 INTERCEPTOR WELLS

The first subsystem is a line of 20 interceptor wells around the CWIS and SWPS area (see Figure V-2). This line of wells was designed to prevent cooling pond water from moving through the backfill and natural sands toward the DGB and auxiliary building railroad bay areas. It will also help lower groundwater levels in the backfill and natural sands near the cooling pond so that if the dewatering wells become inoperable, the rate of groundwater level rise in the plant area will be slow enough to allow either activation of the backup dewatering system (20 backup wells corresponding to 20 interceptor wells) or effect repair or replacement of defective wells before the groundwater level reaches el 610' at either the DGB or auxiliary building railroad bay areas.

3.2 AREA DEWATERING WELLS

The second subsystem, consisting of 24 area wells distributed over the plant site area, was designed to remove the groundwater stored within the backfill and natural sands and then to maintain the groundwater level (see Figure V-2). This subsystem design utilizes the extensive natural sands underlying the backfill as a drain.

4.0 RECHARGE TIME

Analysis of data from pumping tests and from groundwater level responses to changes in cooling pond level indicates there is time available to repair or even replace the entire system before the design groundwater level would be exceeded at the critical areas. To further verify this conclusion, a full-scale test was performed between February 4 and April 5, 1982, after the groundwater levels had been lowered to el 595' or as low as practical and with the cooling pond at el 627'. The groundwater levels were lowered using only 20 permanent backup dewatering wells, existing construction dewatering wells, selected individual observation wells equipped with self-contained eductors, and temporary dewatering wells. During this test, groundwater level-versus-time curves were plotted to determine the actual recharge time at the DGB and auxiliary building railroad bay areas. The results of this test indicate that groundwater levels rise faster at the DGB than at the auxiliary building railroad bay and that there is at least 60 days' recharge time available to repair or perform maintenance on the dewatering system before groundwater levels would reach el 610' at the DGB (see Figures V-3 and V-4).

Results and progress of the recharge testing program were presented to the NRC staff in Bethesda, Maryland, on February 23 and March 3, 1982, and by telephone communication on April 5, 1982.

5.0 WELL INSTALLATION

On March 23, 1981, the Applicant sent a letter to the NRC staff requesting staff concurrence with the installation of 20 backup interceptor wells. After discussions in April, May, and part of June, the staff agreed to a slightly modified version of the proposal. Staff concurrence at that time included only 12 of the 20 wells, because the staff required additional information regarding soil conditions at the locations of the remaining eight wells. Concurrence regarding the final eight permanent wells was secured on September 2, 1981. After these wells were moved northward based on explorations

The 20 permanent backup dewatering wells were installed between August 17, 1981, and October 29, 1981, by a dewatering subcontractor. The architect-engineer's geologist/hydrogeologist prepared as-built drawings of each well installation, including well number, location, diameter of hole, total depth, and description of each type of casing; a log of subsurface materials encountered; and a complete compilation of field data obtained during drilling, installation, and developing of the wells including data requested by the NRC.

NRC concurrence to install the remaining permanent dewatering wells (20 interceptor, 24 area, and 6 monitoring) was given on October 22, 1981. The remaining wells are currently being installed in accordance with the same procedures, criteria, materials, methods, supervision, and inspection used for the installation of the 20 permanent backup wells. Construction of the permanent wells is about 65% complete.

6.0 MONITORING SAFEGUARDS

6.1 INITIAL OPERATING PERIOD

Groundwater quality, pumping rates, drawdown levels, and hours of operation will be monitored during the initial operating period so that an operating history of each well is established prior to plant operation. By comparing collected data, any decrease in production efficiency will be detected.

Near the end of the initial operating period, after the groundwater in storage has been removed and the groundwater levels have stabilized at or below el 595', the frequency of monitoring groundwater levels, soil particle content, and water quality will be determined for implementation during plant operation.

6.2 PLANT OPERATION

During plant operation, monitoring procedures will be performed under a quality assurance program. When it is determined by analyzing available data that a well or group of wells is no longer functioning, corrective measures will be taken. These corrective measures may include cleaning the well screens, repairing or replacing screens or any mechanical parts, or installing a new dewatering well, if necessary.

A complete set of replacement parts will be stored onsite for any repair, replacement, or installation that may be required. As a result of the proposed monitoring of the well system, any significant rise in the groundwater level will be detected in time to take remedial actions before the critical groundwater elevation (el 610') is reached at the DGB or auxiliary building railroad bay areas.

6.3 GROUNDWATER MONITORING

The dewatering system is self-verifying. This means that many design parameters and most design analyses used in the permanent dewatering system may be verified by direct observation of water levels at the Midland site. In addition, monitoring is an integral part of the system operation.

Six permanent monitoring wells are planned. Each permanent monitoring well is of the same design as a permanent well, except each permanent monitoring well will contain an ultrasonic level transmitter to continuously record the groundwater level. The locations of the permanent monitoring wells are shown in Figure V-2. These locations were selected based on their proximity to the critical areas and their position in the backfill and natural sand (two at the DGB, two at the auxiliary building railroad bay, and two north of the interceptor well system).

Currently, over 50 observation wells exist at the site to monitor various depths within the backfill and natural sands. A select number of these wells will be maintained for measurement over the life of the plant.

7.0 SYSTEM DESIGN FOR ACCIDENT CONDITIONS

The dewatering system is not a Seismic Category I system; it is not required to operate during or after an SSE. Instead, the system design is based on the conclusion that, following natural circumstances that may cause total or partial failure of the system, time exists to make necessary repairs before the potential for liquefaction develops. A worst-case assumption (the total failure of all pumping capacity in the system) would still permit time to repair or reinstall the system before the water level in liquefiable soils in the DGB and auxiliary building train bay areas reaches el 610'. This conclusion was verified by the full-scale recharge test described in Section 4.0. A summary of well failure mechanisms and repair times is presented in Table V-1. Additional discussions with the NRC staff concerning accident conditions and system response occurred at meetings with the staff on February 23 and March 3, 1982.

7.1 POWER OUTAGES

Less severe accident conditions (e.g., a partial break in the dewatering header system, line breaks outside the dewatering system, or power outages) have also been accounted for in the system design. Electrical wiring of the system will be designed such that the temporary outage of one or more wells will have no effect on the remaining wells. In addition, should any disruption in the overall power supply occur, backup diesel generator power will be available for temporary operation of the primary interceptor wells and/or backup well pumps until normal power is restored.

7.2 UNINTERRUPTED SERVCE

Assurance of uninterrupted service in the event of a partial loss of system wells is also provided by a number of redundancies built into the dewatering system. Twenty backup wells located at the CWIS and SWPS will provide standby pumping capacity for the 20 interceptor wells in this area. Another 24 area wells are available to remove any water not collected by the interceptor wells. Thus, 64 wells have been incorporated into the dewatering system design, each with a submersible pump having the capacity of at least 10 gpm. Normal operations to maintain the groundwater level at or below el 595' during the life of the plant is estimated to require only 22 of these wells.

7.3 PIPE BREAKS

The dewatering system design also accounts for pipe breaks, both at the interceptor wells and at the critical areas. Pipe breaks that would immediately impact the interceptor well system include breaks of a dewatering system header line, concrete pipe cooling pond blowdown line, concrete pipe cooling tower line, or service water discharge line. At the request of the NRC staff, the Applicant also analyzed a nonmechanistic failure of both the Unit 2 circulating water discharge pipe and the 20-inch diameter condensate water pipe near the DGB.

7.3.1 <u>Damage to the Dewatering System Header Line</u>

Damage to the dewatering system header line could result in return flow to the dewatering wells in the vicinity of the broken line. In that event, the combination of groundwater recharge and surface water inflow could exceed the capacity of the affected pump, producing a rise in groundwater level. To account for this possibility, the dewatering system will be designed to permit a flexible hose to be attached to the individual wells. If a header line breaks, a hose would be attached to each well to temporarily divert flow to the system's catch basins until the header line is repaired. In the case of an interceptor well header failure, the backup wells can be activated because they are on a separate header system. This arrangement will prevent an overload of the pumping capacity of an individual well or of a group of wells.

7.3.2 Break of Either Concrete Pipe Blowdown or Cooling Tower Lines

A break of either the concrete pipe blowdown line or the cooling tower line at the CWIS and SWPS could result in the loss of three dewatering wells. The impact of such a pipe break on the entire dewatering system, however, would be minimal. The total amount of water released by a break in either of these low-pressure lines would not produce a significant rise in the overall plant groundwater levels, even if all the released water entered the groundwater system.

Following a pipe break, the flow of the water would be shut off and the backup interceptor wells would automatically activate. The backup interceptor wells and remaining primary wells will have sufficient capacity to remove recharge from the cooling pond until the damaged wells can be replaced. Excess water introduced into the area by the pipe break would be removed by the area dewatering system.

7.3.3 Nonmechanistic Failure of the Unit 2 Circulating Water Pipe

Potential hazards from the nonmechanistic failure of the circulating water discharge pipe near the DGB were assessed by determining the time necessary for the rise in water level to activate a permanent area dewatering well. It was determined that groundwater levels would be significantly below the critical elevation when the permanent area dewatering wells would be activated.

7.3.4 Nonmechanistic Failure of the 20-Inch Condensate Pipe

A nonmechanistic failure of the 20-inch diameter condensate water pipe, which is located directly beneath the DGB, was analyzed. Using a simplified analysis, it was assumed that the entire contents of the condensate water tank (300,000 gallons) were spilled directly beneath the DGB. Further, it was conservatively assumed that all the water would be contained beneath the building. From this analysis, it was determined that the groundwater elevation would not rise above el 610'.

Because the volume of water in the condensate storage tanks is less than the volume required to fill the area beneath the DGB to el 610', a failure of the condensate water pipe would be accommodated even if no permanent area dewatering wells were operating in this area.

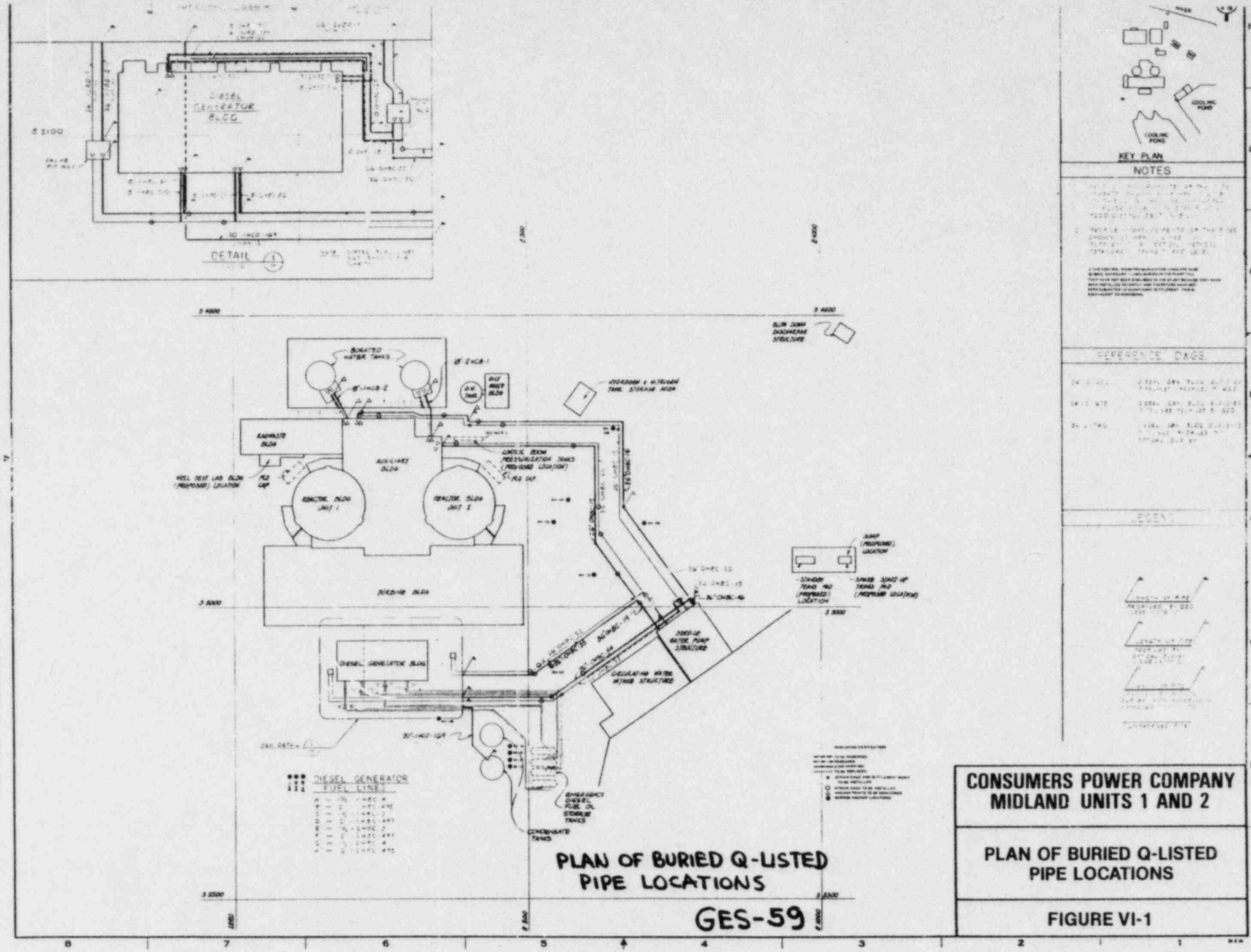
8.0 TECHNICAL SPECIFICATIONS

After the plant operator has verified that a water level measurement higher than el 595' is a correct reading and the repair measures given in Table V-1 do not affect the rise in groundwater level at the DGB or auxiliary building railroad bay, the plant will be shut down when any observation well at either critical structure exceeds el 607' (see Figure V-5). A technical specification will be prepared detailing the coordination of the shutdown.

TABLE V-1

WELL FAILURE MECHANISMS AND RESPONSES

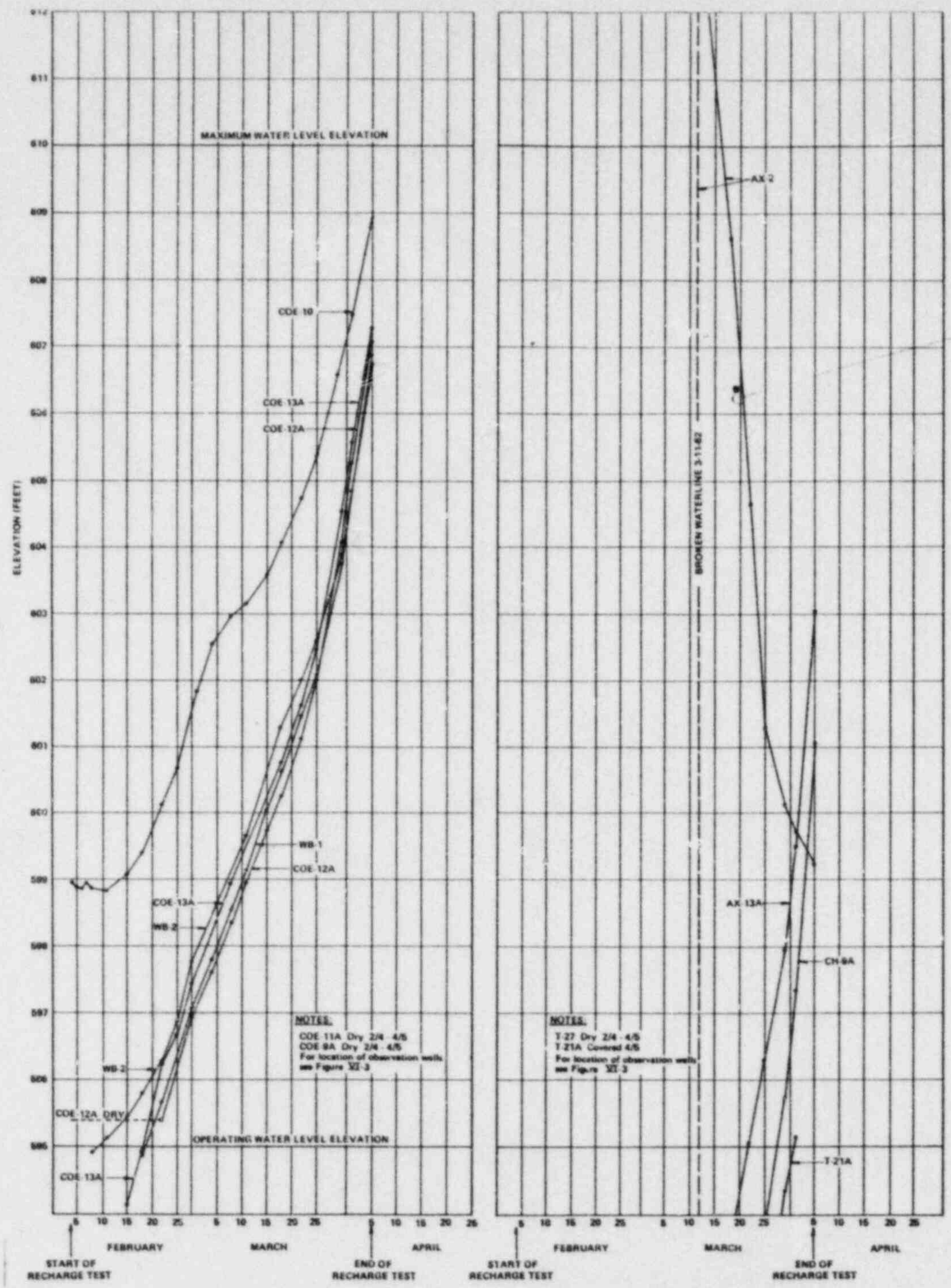
Event	50.54(f) Reference	Repair Time
1. Electrical Failure		
a. Single well (wired in parallel)	24.a, 24.c, 47.1.b	Less than 1 day.
b. Multiple wells due to power outage	24.a, 24.c, 47.1.b	1 day to initiate operation of backup diesel power to interceptor wells. Operate until normal power can be restored. Backup interceptor wells automatically begin pumping if water levels exceed el 595'.
2. Failure of timers/ pumps/check valves	24.c, 47.1.b, 47.6	Less than 1 day; replacement parts onsite.
3. Header pipe break	24.c	1 day to attach flexible hose to each well affected and pump water to storm drains. In case of interceptor well header failure, initiate backup wells (on separate header system).
4. Well screen encrustation	24.h, 47.6, 47.8	2 days to acidize well.
5. Complete loss of well	24.c, 47.1.b	4 days to replace one well using cable tool rig. 1 day if other drilling method used. If well or wells need to be replaced, there is enough redundancy and pumping capacity to prevent water levels from rising in plant fill, while the replacement wells are being installed.



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

PLAN OF BURIED Q-LISTED PIPE LOCATIONS

FIGURE VI-1



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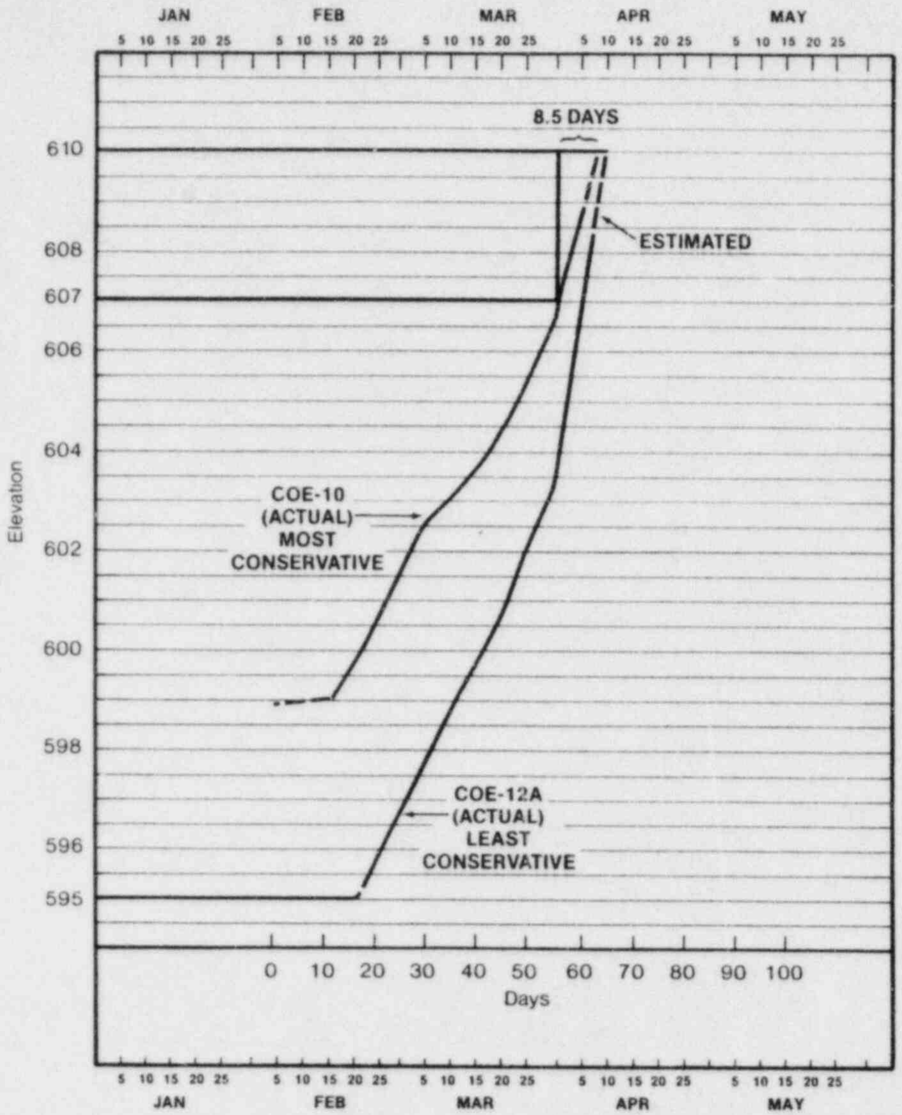
CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
GROUNDWATER LEVELS MEASURED AT CRITICAL AREAS DURING RECHARGE TEST
FIGURE V-4

G-1986-21



MIDLAND UNITS 1 AND 2
ACRS 4/7/82

ASSUMPTIONS:

1. 1½ DAYS TO COLD SHUTDOWN
2. 7 DAYS TO OPERATE DIESELS AFTER COLD SHUTDOWN
3. WELL OR WELLS CANNOT BE REPAIRED OR REPLACED IN SUFFICIENT TIME

CRITERIA:

IF GROUND WATER LEVEL EXCEEDS ELEVATION 607.0 AT ANY OBSERVATION WELL AT THE DIESEL BUILDING OR AUXILIARY BUILDING TRAIN BAY THE PLANT WILL BE SHUT DOWN.

NOTE: FOR LOCATION OF OBSERVATION WELLS SEE FIGURE VI-3.

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
DEWATERING CRITERIA FOR PLANT SHUTDOWN
FIGURE V-5

G-1986-15

PART VI: UNDERGROUND UTILITIES

1.0 INTRODUCTION

The Applicant has conducted an investigation to evaluate the adequacy of underground Seismic Category I utilities. The underground utilities included are:

a. Diesel fuel oil piping and tanks - This system provides fuel supply and return between the emergency diesel generators and diesel fuel oil storage tanks buried in the vicinity of the diesel generator building (DGB). There are four 1-1/2-inch supply lines, four 2-inch return lines, and four tanks 12 feet in diameter and 44 feet long.

b. Borated water piping - This piping provides borated water for volume and reactivity control from the borated water storage tanks (BWSTs) for normal functions and for such postulated accidents as a pipe break in the reactor coolant system. There are four 18-inch lines.

c. Control room pressurization piping and tanks - This system supplies overpressurization air to the main control room during postulated accidents such as releases of hazardous gases. There is one 4-inch line, one 1-inch line, and two tanks, each 5 feet in diameter and 25 feet long, buried in the vicinity of the auxiliary building.

d. Electrical duct banks - These concrete duct banks encase electrical power and control cables for various systems needed under normal and accident conditions.

e. Service water piping - This piping supplies water to various systems needed under normal and accident conditions. There are 22 lines ranging from 8 to 48 inches in diameter.

Table VI-1 contains a detailed listing of the Seismic Category I piping. Figure VI-1 shows the locations of the buried piping and tanks.

Because of the location of these utilities and the depth at which they are buried, all pipes, associated tanks, and duct banks listed above rest on compacted backfill material.

The investigation included test borings, measurements, and analysis. The remedial plan resulting from these investigations ranges from acceptance of the existing utilities to selected replacement. A selective monitoring program has also been adopted to ensure that intended functions are maintained over the life of the Midland plant.

2.0 REMEDIAL PLAN

The remedial plan for the Seismic Category I underground utilities is summarized below:

a. Diesel fuel oil lines and tanks - As a result of piping flexibility and small expected settlements for the piping and tanks, no remedial measures are indicated.

b. Borated water lines - This piping will be partially rebedded. This action, in conjunction with the settlement monitoring of the BWSTs, will provide assurance of the piping's continued serviceability.

c. Service water piping - Extensive measurements have been taken to define the present condition of the service water piping. A monitoring program for strain measurement and settlement will provide assurance of continued serviceability for a majority of the piping. The 36-inch diameter piping will be replaced. The two 26-inch diameter pipelines adjacent to the circulating water intake structure (CWIS) will be rebedded and the material beneath them replaced to preclude the potential for soil liquefaction.

d. Control room pressurization piping and tanks - The predicted differential settlement effects have been included in the design. No further action is required.

e. Electrical duct banks - The predicted settlement will not adversely impact the ability of the electrical duct banks to perform their function.

Details of the investigation, analysis, and agreements that support this remedial plan are presented in the remaining sections.

3.0 GEOTECHNICAL INVESTIGATIONS AND RESULTS

3.1 RESULTS OF TEST BORINGS

The records of exploration borings throughout the site indicate that the consistency of the fill at the location of buried utilities varies from soft to hard for silty clays and loose to dense for sands. Generally, the fill soils can be classified as medium stiff or medium dense below invert elevations of buried piping and other utilities. Fill foundation conditions have been greatly improved in the vicinity of the DGB as a result of the surcharge loading program that was conducted in 1979. Exploration borings in the area of the BWST indicate that the fill soils generally range from stiff to very stiff.

3.2 SETTLEMENT

Settlements that have been observed at buried utilities are primarily a result of the fill settling under its own weight. Areas that have been subjected to surcharge loading, such as the DGB and BWST areas, exhibit additional settlement from surcharging. The buried utilities add little, if any, weight to the fill; therefore, they have very little impact on present and future settlement below their invert elevations.

Records of monitored settlement within the fill have been utilized to predict future settlement for buried utilities. Borros anchors have been installed at nine locations in the vicinity of buried utilities not influenced by surcharge loadings. Settlement readings for anchors that have been established at depths of 7 feet to 12 feet below the surface were used in the analysis, because this depth represents the depth of most buried utilities. Soil conditions at these locations represent the variable soil conditions encountered throughout the fill.

Based on these records, future maximum settlement of buried utilities is conservatively estimated to be 3 inches or less. This maximum settlement estimate also includes future predicted settlement resulting from site dewatering and possible seismic shakedown. Future settlement of buried utilities in the vicinity of the DGB and BWST will be considerably less than the maximum value predicted because better fill conditions exist in these areas. Future settlement of the service water lines to be reinstalled in the vicinity of the service water pump structure (SWPS) and CWIS will be approximately 1-1/2 inches or less.

4.0 ANALYSES OF EXISTING UTILITIES

The analyses for buried utilities because of the remedial soils activities were initially presented in a technical report submitted December 15, 1981. They were discussed in meetings held with the staff in Bethesda, Maryland, on October 6, 1981; January 21 and 22, 1982; February 11, 1982; and were addressed in testimony at the Atomic Safety and Licensing Board (ASLB) soils hearings February 18 and 19, 1982. The following paragraphs summarize those reports, discussions, and testimony.

4.1 DIESEL FUEL PIPING AND STORAGE TANKS

The diesel fuel oil lines were installed in June 1980 after completion of the DGB surcharge program. The small diameter lines are flexible enough to accept the predicted future plant fill settlement without exceeding allowable limits. The maximum settlement stress was calculated for the maximum predicted settlement and was found to be within the allowable value.

The diesel fuel oil storage tanks were installed approximately 2 years after the fill was placed. This isolated the tanks from the effects of the initial settlement of the fill. The tanks were filled with water and the settlement monitored for approximately 8 months. Tank settlement during this period was minimal (less than 0.2 inch). It has been estimated that during plant life the tanks will experience about 1-1/4-inch long-term settlement, which includes settlement from site dewatering and seismic shakedown. The buried tanks will settle with the surrounding soil. The connecting pipes will also settle with the tanks in the surrounding soil. Thus, the differential settlement between the pipes and tanks will be small. Nozzle loads due to settlement have been calculated and are insignificant.

4.2 BORATED WATER PIPING

The borated water lines will be rebedded from the BWST valve pits to the dike around the tanks (see Figure VI-1). These lines have been cut loose from the valve pits to isolate them from the settlement caused by the valve pit surcharge. This partial rebedding in conjunction with the existing program to monitor future settlement of the BWST, settlement of the auxiliary building, and strain at the pipe anchors will provide sufficient ensurance of the piping's continued serviceability.

4.3 CONTROL ROOM PRESSURIZATION LINES AND TANKS

The control room pressurization lines and tanks were installed in early 1981. Installation after the occurrence of major fill settlement provides sufficient ensurance of continued serviceability of the pipes and tanks in this system.

4.4 ELECTRICAL DUCT BANKS

The seismic analysis of buried electrical duct banks complies with the requirements in FSAR Subsection 3.7.3.12 and was discussed in detail in the response to Question 30 of NRC Requests Regarding Plant Fill.

4.5 SERVICE WATER PIPING

4.5.1 Locations and Alignment

Extensive measurement data have been taken to define the present condition of the service water piping. The original position immediately after installation is not clearly defined. It is difficult to ascertain precisely how much of the current profile resulted from construction tolerances. To ensure serviceability, it has been conservatively assumed that all deviations from design location are due to settlement.

In 1979, elevation or profile data were taken for one pipeline in each pipe trench. In June 1981, the Applicant retained Southwest Research Institute to develop a more accurate measurement technique and to reprofile all the service water piping that is 26 inches and larger in diameter using the new technique. The measurement technique uses pressure and ultrasonic transducers and is accurate to 1/16 inch. The current location of the piping is very well defined from these accurately measured profile data taken at 5-foot intervals along the pipe length. The circumferential weld joints have also been identified between pipe spool lengths.

The results of these measurements show that the service water pipe is 8 to 12 inches from the design elevation in some extreme locations and the majority of the piping is, on the average, approximately 5 inches from its design location.

4.5.2 Ovalization

For the service water piping, the relationship between out-of-roundness/ovalization and strain was used to establish its serviceability. Ovalization is an indirect measurement of the bending stress of the pipe, which may have occurred due to fill settlement. These ovalization measurements were taken internally at the same locations as the profile points.

The results indicate general ovalizations of 1 to 1.5% with some locations of 2% and greater. The maximum ovalization recorded was 3% in one 36-inch diameter pipe where the pipe enters the SWPS.

4.5.3 Terminal End Analysis

A terminal end analysis considering weight, operating, and seismic forces was performed. This analysis started inside the structure at a fixed point (equipment nozzle or anchor) and continued to an assumed anchor point outside the structure. Soil springs were added along the pipe to model soil interaction. An analysis has also been performed to verify that displacements from settlement and seismic motion will not cause pipe contact with the building wall.

4.5.4 Acceptance Criteria

4.5.4.1 American Society of Mechanical Engineers (ASME) Code

The acceptance criteria for those portions of the analyses addressed by the ASME code were easily determined. These acceptance criteria are listed below:

a. Allowable stress in the pipe - Subsection NC

b. Combination of seismic stresses with stresses from other loading conditions - Subsection ND

c. Allowable stresses for the materials and operating temperature relevant to the piping being analyzed - Subsection ND

d. Allowable stress in pipe supports - Subsection NF

4.5.4.2 Ovalization

An acceptance criterion of 4% ovality for 26-inch pipe has been agreed upon with the NRC staff.

No agreement was reached between the Applicant and the NRC staff on appropriate acceptance criteria for the existing 36-inch diameter buried service water piping. Therefore, during the ASLB soils hearings, the Applicant agreed to replace the 36-inch pipe.

On March 16, 1982, the Applicant submitted a technical report describing the monitoring program, which resulted from a series of discussions with the staff. The report presented the relationship between ovalization and longitudinal strain in the pipe. Figure VI-2 shows the relationship used to convert the historical measured ovality to strain for comparison to the acceptance criteria.

4.5.5 Vertical Settlement

The acceptance criteria for settlement markers are based on the conservative upper limit of 3 inches for maximum future settlement. The NRC staff will be notified if 75% of the 3-inch upper limit is reached, and the staff and the Applicant will evaluate the appropriate action to be taken.

4.5.6 Reinstallation Program

The Applicant's March 16, 1982, report includes a reinstallation program that describes the engineering and construction activities necessary to replace the 36-inch diameter pipes and rebed a portion of two 26-inch diameter lines (26"-0HBC-53 and 26"-0HBC-54) immediately adjacent to the CWIS.

Rebedding the 26-inch diameter piping is an additional commitment since the soils hearings, based on the recently evaluated results of the dewatering recharge test. The results indicate that the area immediately north of the SWPS and the CWIS has only 3 days following a dewatering system failure before the groundwater would reach the level for potential soil liquefaction during a seismic event. As a consequence, the fill in the affected area will be replaced down to el 610'. The area covers a zone where the 36-inch diameter piping is being replaced and also a zone where pipelines 26"-0HBC-53 and 26"-0HBC-54 are buried. The fill replacement with acceptable fill will eliminate the potential for liquefaction.

The reinstallation program identifies the structures, facilities, and utilities that may be affected by the reinstallation activities. The underground utilities that will be exposed during the excavation work will be supported and protected as necessary to preclude damage. The quality program requirements applying to the reinstallation work were also discussed.

4.5.7 Monitoring Program

The future monitoring program submitted March 16, 1982, covers two types of monitoring: vertical settlement monitoring and pipe strain monitoring. The monitoring program describes the monitoring station locations and the details of selection criteria, monitoring frequency, acceptance criteria, and instrumentation for both types of monitoring. The reinstalled pipe will have no special monitoring program because the underlying fill will be replaced with suitable fill material.

The effect of future soil settlement on the service water piping will be monitored using externally mounted strain gages. The location of these instruments has been presented in the monitoring program submitted March 16, 1982. The location of these monitoring points are shown in Figure VI-1.

The initial monitoring frequency will be every 90 days, with reevaluation after 5 years. All locations are to be monitored immediately following an unusual event. If the technical specification limit is reached at a monitoring station, the frequency will be increased to monthly until remedial measures have been established.

The submittal of this monitoring program and the reinstallation program on March 16, 1982, provided the remedial action necessary to resolve the NRC concerns expressed in the ASLB soils hearing February 18 and 19, 1982.

TABLE VI-1

SEISMIC CATEGORY I LINES

A. Service Water Lines:

8"-1HBC-310	26"-0HBC-53
8"-2HBC-81	26"-0HBC-54
8"-1HBC-81	26"-0HBC-55
8"-2HBC-310	26"-0HBC-56
8"-1HBC-311	26"-0HBC-15
8"-2HBC-82	26"-0HBC-16
8"-1HBC-82	26"-0HBC-19
8"-2HBC-311	26"-0HBC-20
10"-0HBC-27	36"-0HBC-15
10"-0HBC-28	36"-0HBC-16
	36"-0HBC-19
	36"-0HBC-20

B. Diesel Fuel Oil Lines:

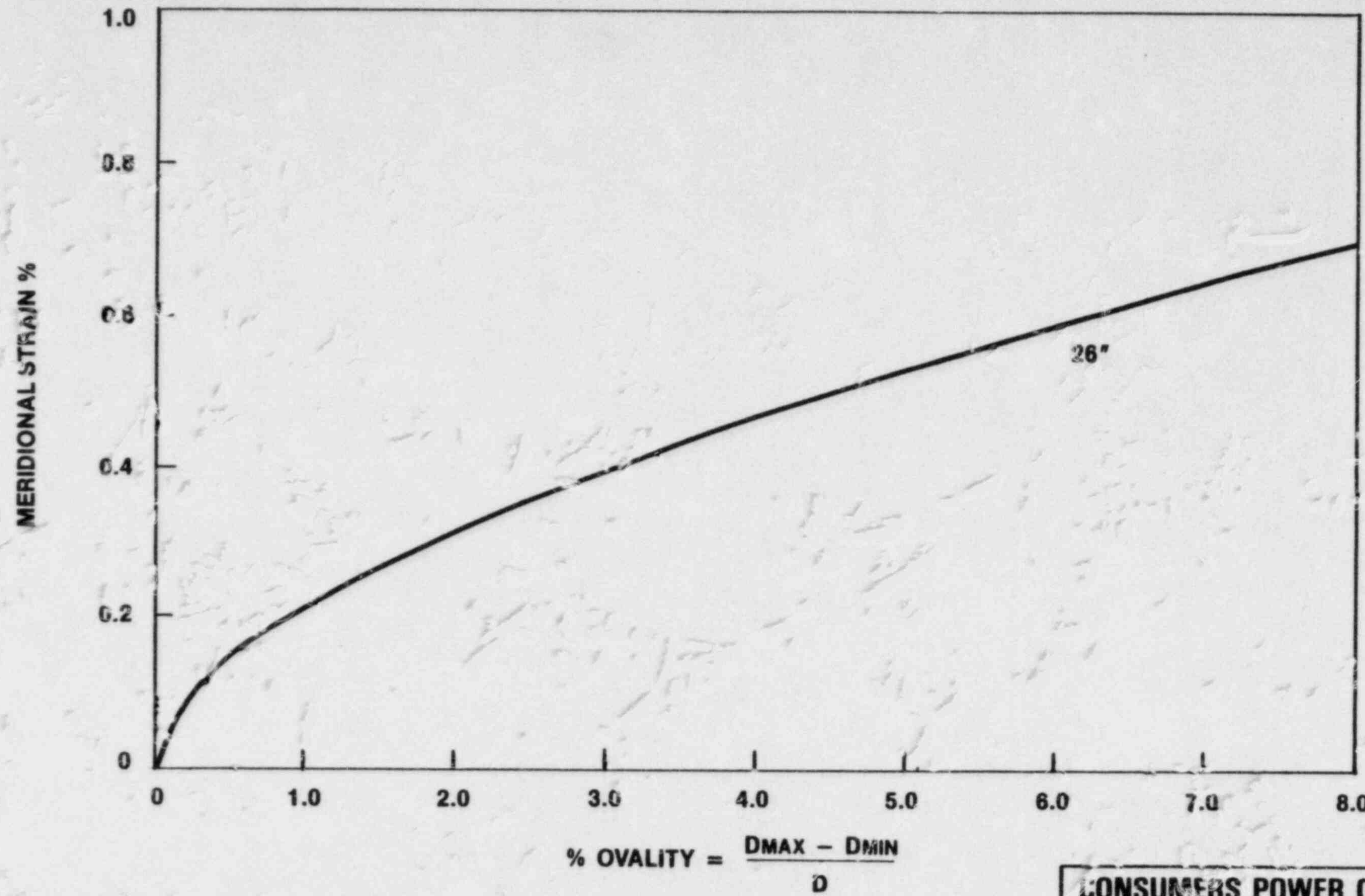
1-1/2"-1HBC-3	2"-1HBC-497
1-1/2"-1HBC-4	2"-1HBC-498
1-1/2"-2HBC-3	2"-2HBC-497
1-1/2"-2HBC-4	2"-2HBC-498

C. Borated Water Lines:

18"-1HCB-1
18"-1HCB-2
18"-2HCB-1
18"-2HCB-2

D. Control Room Pressurization Lines:

4"-0DBC-1
1"-0CCC-1



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

MIDLAND STRAIN vs
OVALITY CURVE

FIGURE VI-2

MIDLAND UNITS 1 AND 2
ACRS 4/7/82

G-1986-14

Part VII: QUALITY ASSURANCE

1.0 INTRODUCTION

All remedial soils work, except for underpinning, will be done in accordance with the existing Midland Project Quality Assurance Program. The underpinning activities are unique in that the few technically competent contractors who do this type of specialized work have no formal quality assurance (QA) programs and have little, if any, experience in the nuclear field. To accommodate the acquisition of only the most experienced contractors, a special Quality Assurance Plan for Underpinning has been devised to extend the Midland Project Quality Assurance Program to those contractors.

2.0 QUALITY ASSURANCE PLAN FOR UNDERPINNING

The Quality Assurance Plan for Underpinning, MPQP-1, was transmitted to the NRC on January 7, 1982. In addition to the information provided in the plan, in January and March there were presentations to and discussions with the NRC staff and Region III personnel relative to the plan. The plan has been found acceptable.

Under this plan, a special QA organization has been established for the underpinning work. The organization consists of two groups: a QA engineering group with an authorized staff of six engineers (degreed civil engineers), and an inspection, examination, and test verification group with an authorized staff of five civil inspectors (some of whom have experience directly related to the Midland underpinning work). These two groups report to a soils and remedial QA supervisor (a civil engineer) who, in turn, reports to the civil QA section head (also a civil engineer). Thus, there will be a total of 13 QA persons directly engaged in the underpinning work within the Midland Project Quality Assurance Department (MPQAD), which is independent of the architect-engineer/constructor and which is headed by a director reporting to the the Applicant's vice president for projects, engineering, and construction.

A special quality control (QC) organization also exists for which 23 inspectors are authorized for remedial soils inspection. The inspectors, through the lead inspectors, report to an underpinning QC coordinator who, in turn, reports to the lead civil QC engineer. This QC organization is part of the architect-engineer/constructor organization, but it is independent of the architect-engineer/constructor field construction management. Furthermore, this QC organization is overseen by the totally independent MPQAD described above.

The MPQAD performs the primary QA activities for the underpinning work, whereas the QC organization performs the primary inspection activities to the standards and requirements established by MPQAD. The following is a brief description of the major MPQAD activities and the objectives of each.

Design documents are originated and issued through the architect-engineer's design process with all controls of the existing Project Quality Assurance Program being applied to the design process. However, before their issuance, MPQAD reviews and approves the documents to ensure that they are sufficiently specific with regard to the quality characteristics and to ensure that these characteristics are inspectable or testable.

For construction contracts, MPQAD establishes the requirements by which the contractors attain quality, although the QC and MPQAD organizations will ensure that quality is attained. Requirements applied to contractors may deal with document controls, preparation of detailed construction procedures, personnel training, handling and storage of materials, and performing process corrective action, when necessary. These types of requirements are intended to promote the prevention of nonconformances or, at worst, their early detection and the correction of their root causes.

MPQAD reviews and approves construction procedures to ensure that the procedures impose the necessary quality prerequisites, that they provide sufficient specificity with which to ensure the consistent attainment of the design requirements, and that the QC inspection hold points are integrated into the construction procedures at the appropriate points in the process. MPQAD also integrates the MPQAD overinspection hold points into the construction procedures.

MPQAD reviews and approves the detailed QC inspection procedures to ensure that they are complete with regard to the necessary inspections and to ensure that they are sufficiently specific with regard to the methods of inspection, the points of inspection, and the inspection data to be recorded

MPQAD plans and performs its own overinspections. These overinspections are on a large sampling basis and are applied to the most significant quality characteristics for the purpose of ensuring that the construction work is being done properly and ensuring that the QC inspection decisions are being made properly. On a periodic basis, quality system audits of the constructor and contractors are also performed by MPQAD to ensure compliance with the QA standards and requirements. In addition to MPQAD, an entirely separate Applicant audit section performs periodic system audits. MPQAD ensures the correction of nonconformances as well as the identification and elimination of their root causes.

3.0 QUALITY ASSURANCE COVERAGE

As a result of the March discussion with the NRC, it has been agreed that the Quality Assurance Plan for Underpinning will be implemented for essentially all elements of the underpinning work and not just for the specific activities or structures deemed to be safety related. The plan is being modified to reflect this additional coverage. A mechanism will be provided by which to take any exception which may be desired, but this mechanism will include assurance that Region III personnel have concurred with the exception prior to doing the work. The MPQAD and QC staffing levels described above were arrived at in recognition of this extended coverage. The NRC has concurred that the staffing levels to date have been appropriate to the level of work.