Enclosure 2

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO NRC STAFF REVIEW CONCERNS

FOR UNDERPINNING OF THE AUXILIARY BUILDING

JUNE 3, 1982

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO NRC STAFF REVIEW CONCERNS

FOR UNDERPINNING OF THE AUXILIARY BUILDING

CONTENTS

TABLES

FIGURES

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ATTACHMENTS

MIDLAND PLANT UNITS 1 AND 2

RESPONSE TO NRC STAFF REVIEW CONCERNS

FOR UNDERPINNING OF THE AUXILIARY BUILDING

REVIEW CONCERN 2: CONSTRUCTION PHASE 4

Furnish results of analysis of the auxiliary building permanent underpinning walls, and the feedwater isolation valve pits

RESPONSE

1.0 INTRODUCTION

This response summarizes the results of the analysis of the auxiliary building superstructure, underpinning walls, and the feedwater isolation valve pits (FIVPs) for the completed conditions, that is, following soils remedial actions. Results of the preliminary analysis were submitted in December 1981. The stress analysis of the structure during the construction of the underpinning walls has been submitted in a separate report.

2.0 ANALYTICAL MODEL

The three-dimensional analytical model as shown on Sketches 7220-SK-C-767-1 through 24 (Reference 1) has been used for the analysis. Sketches 7220-C-767-1 through 21 have been provided to the NRC staff during the audits of February 1 and 26, 1982, and Sketches 7220-C-767-22 through 24 are included as Attachments 2-1, 2-2, and 2-3. The basic description of the model has been submitted to the NRC in Appendix A of Reference 2. The model has a total of 3,292 nodes and 4,811 elements consisting of plate, beam, truss, boundary, and dummy elements to simulate the structure. The FIVP structure has been analyzed by hand calculations.

3.0 LOAD COMBINATIONS

For the analysis of the auxiliary building superstructure, applicable FSAR load combinations with jacking loads (P_L) and long-term settlement effects incorporated as shown in Table Aux-1 of Reference 3 have been used. Dead, live, seismic, main steam line pipe break, settlement, and tornado loads with a global effect on the building have been analyzed by the finite-element model. Local effects of other loads such as normal and accidental thermal gradients, jet impingement loads, missile loads, and pipe support loads other than those for main steam line rupture have been added to local areas as appropriate. In addition, the superstructure has been analyzed, for information only, with the load combinations of Article 9.2 of American Concrete Institute (ACI) 349-80 (Reference 4) as modified by NRC Regulatory Guide 1.142.

The underpinning walls and the connections to the existing superstructure have been designed to satisfy the following requirements:

a. Midland FSAR as amended for the effects of jacking load (P_L) and long-term soil settlement loads

b. ACI 349-80 code requirements as modified by NRC Regulatory Guide 1.142.

The underpinning walls and connections to the existing structure have been designed to withstand seismic loads from a safe shutdown earthquake (SSE) with a multiplier of 1.5. This has been done to provide assurance that the underpinning walls could resist the forces resulting from the site-specific response spectra (SSRS) earthquake. The superstructure has been analyzed and designed for FSAR earthquake loads.

The FIVP foundation has been designed for the effects of dead, live, jacking, settlement, and seismic loads. The seismic acceleration values have been determined by hand calculation assuming a ground acceleration for an SSE of 1.5 times the FSAR value.

4.0 SUMMARY OF ANALYSIS AND DESIGN

4.1 AUXILIARY BUILDING SUPERSTRUCTURE

Analysis with FSAR and ACI 349-80 load combinations has been completed. The analysis indicates that for the superstructure south of Column Line G, the walls above el 659' and the slab at el 659' between Column Lines G and H do not meet the criteria for allowable stresses. The membrane shear stress in the wall exceeded $3\sqrt{f_c'}$, which causes it to crack. It was decided to strengthen the slab and to reduce the stiffness of the walls in the reanalysis. The reanalysis was performed assuming reduced stiffness for the walls and increased stiffness of the slab (from strengthening which shall be added). The analysis indicated that the acceptance criteria can be satisfied with modified slab and walls. Strengthening the slab either by adding plates on top of the slab or by adding rebar to the existing slab is being evaluated. The forces and capacities at critical sections for the superstructure north of Column Line G is also being reviewed and the results will be provided in future FSAR amendments.

4.2 UNDERPINNING WALLS

The design concepts of the underpinning walls below the electrical penetration areas (EPAs) and the control tower have been described in Reference 5. Figures 2-1 and 2-2 show the underpinning wall and reinforcing detail and Figure 2-3 shows typical connection details with the EPA and control tower superstructures. Figure 2-4 shows the soil pressure data points and Tables 2-1 and 2-2 include the design load capacities and soil pressures, respectively.

4.3 FEEDWATER ISOLATION VALVE PITS FOUNDATION

The analysis concept for the FIVP foundation has been described in Reference 5. The support detail is shown in Figure 2-5; Table 2-3 shows the soil pressures under the foundation and rebar details for the 3'-0" thick jacking slab.

REVIEW CONCERN 2: CONSTRUCTION PHASE 3

Provide the following:

a) Results of analysis of the auxiliary building during construction of the underpinning walls with a soil modulus of 70 kcf under the main auxiliary building.

b) Results of analysis for loss of support under the EPAs because of tunneling under the turbine building.

RESPONSE

5.0 INTRODUCTION

The auxiliary building temporary support system was analyzed at appropriate sequential stages of excavation and jacking planned during construction of the underpinning wall. The analysis was based on the estimated 30 kcf subgrade modulus of the existing soil under the building (shown in parentheses in Figure 2-6). The results of the analysis indicated that these were acceptable safety margins at the various construction stages. The results of this analysis were presented to the NRC staff during the structural audit conducted by them during the week of February 1, 1982.

At the conclusion of the audit, the NRC staff requested that two parametric studies mentioned above be performed. The studies are described below. Additionally, the staff had expressed a concern about 20 feet for Stage 1 soil removal. The staff felt that 30 feet should be used for Stage 1. This concern was also incorporated in the parametric analysis.

6.0 ANALYTICAL MODEL

The three-dimensional, finite-element model, as shown in Drawing 7220-SK-C-767-1 through -21 and previously provided to the NRC during the February 1 and February 26, 1982 audits (Reference 6), has been used. The following assumptions were made in the analysis.

6.1 LOADS

Loads include dead weight, weights of blockwall and equipment, and 25% live load on the structure, along with jacking loads applied as construction progresses.

6.2 ALLOWABLE STRESSES AND LOAD FACTORS

These values are based on ACI 318-71 and the American Institute of Steel Construction manual, Seventh Edition. The computer results were multiplied by a factor of 1.43 to correspond to 1.4D + 1.7L. This is the same as in the previous analysis.

6.3 SOIL SPRINGS

The soil springs are based on the values of soil modulii as shown in Figure 2-6.

6.4 MODULUS OF CONCRETE

The Young's modulus of concrete is based on E_c = 57,000 fc' in accordance with Article 8.3.1 of ACI 318-71. No reduction due to creep has been assumed.

6.5 REDUCTION OF STIFFNESS

The stresses in different elements of the finite-element model were evaluated using the previous analysis (K_{soil} = 30 kcf under the main auxiliary building) for the existing condition and Stage 1 of soil removal. Elements whose membrane shear stress exceeded $3\sqrt{fc'}$ or whose membrane tensile stress exceeded $4\sqrt{fc'}$ were identified (open items list, Reference 6). These include some elements on the floor at el 659'-0" (shown in Figure 2-7 and Drawing 7220-SK-C-767-7) and on one wall below el 659'-0" between H and Fx on Column Line 5.3 (Figure 2-8 and Drawing 7220-SK-C-767-17). In accordance with Reference 6, the stiffnesses of these elements were reduced to:

$$p \times n$$

where

p = percentage of rebar

n = modular ratio between rebar and concrete (assumed to be 8)

This reduced stiffness decreased the stresses on these elements; however, the average stress on a total length of the slab as shown in Figure 2-7 increased by a small amount compared to the uncracked analysis (with soil modulus K = 30 kcf under the main auxiliary building).

7.0 DESCRIPTION OF VARIOUS ANALYSES

The analyses performed in response to Review Concern 2a and 2b are described in Sections ~~6~~7.1 and ~~6~~7.2 below. For all stages of construction, the effect of soil removal has been simulated by applying a downward load at the ends of a weightless structure as shown in Figure 2-12. The magnitude of this downward load is equal to the sum of the reactions of the springs removed.

In all analyses described hereafter, the change in stress due to any subsequent construction has been analyzed separately and added to the existing stress. The total stresses at any stage thus obtained are shown in Table 2-4.

7.1 REVIEW CONCERN 2a

7.1.1 Existing Stress

In determining the existing stress in the structure, two models have been used to represent the progress of the original construction. The structure above el 659' was assumed to cause stress for structural elements at el 659' and above as shown in Figure 2-9; for all other areas, the structure was assumed to be loaded as shown in Figure 2-10.

7.1.2 Stage 1 Construction

In Stage 1 of construction, the soil at the two extremes of the EPAs is removed (Figure 2-11). To satisfy staff concerns, the width of soil removal is assumed to be 30 feet, compared to 20 feet assumed for Stage 1 in the previous analysis with K = 30 kcf under the main auxiliary building.

Upon completion of soil removal, grillage beams will be placed under the ends of the EPAs and predetermined jacking loads will be applied to the structure.

7.1.3 Stage 2 Construction

The analysis for this stage combines the analyses for Stage 2 and part of Stage 3 of construction as presented in the February 1, 1982, structural audit and, therefore, is an upperbound analysis. This was done in accordance with the agreement with the NRC staff (Reference 6). Actual excavation limits and the extent of deletion of springs are shown in Figure 2-14. At the end of Stage 1 of construction, additional jacking capacity will be available at the ends of the EPA (capacity shown in parentheses). Piers CT1 and CT12 on the south corners of the control tower will be installed before further excavation under the EPA. However, the structure will be monitored to detect unanticipated movements. If necessary, either of the following actions can be taken before a large amount of soil is removed.

a. The jacking loads (shown in parentheses in Figure 2-16) at the ends of the EPA and piers CT1 and

CT12 at the south corners of the control tower can be increased.

b. Four additional piers on the south side of the control towers (CT2, CT3, CT10, and CT11) can be constructed.

The structure has been analyzed for a large amount of soil removal as shown in Figure 2-14 and for each of the above conditions. The more critical results from the two cases are incorporated in Table 2-4.

At the end of excavation in Stage 2, the design jacking loads will be applied to the structure as shown in Figure 2-15.

7.1.4 Stage 3 Construction

The design conditions for total soil removal and with jacking loads applied are shown in Figures 2-16 and 2-17, respectively.

7.2 REVIEW CONCERN 2b

This study has been performed to analyze the effect of tunneling under the turbine building after the ends of the EPA have been supported by jacks, as shown in Figure 2-18. It has been assumed that, because of tunneling under the turbine building, a strip approximately 6 feet wide on the south side of the EPA will lose soil support.

8.0 SUMMARY OF ANALYSIS AND CONCLUSION

The areas of maximum stress have been identified in Figure 2-19, and Table 2-4 shows the average stress in the rebar during the various construction stages. As Table 2-4 indicates, despite the conservative assumptions, there is no overstressing of the structure.

REFERENCES

1. Sketches 7220-SK-C-767-1 through 24, Three-Dimensional Finite-Element Model

2. Consumers Power Company, Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits for Midland Plant Units 1 and 2, Consumers Power Company Docket Numbers 50-329 and 50-330, Enclosure 3 to J.W. Cook's letter to H.R. Denton (NRC), September 30, 1981

3. Consumers Power Company, Testimony to the Atomic Safety and Licensing Board Regarding Remedial Measures for the Midland Plant Auxiliary Building and Feedwater Isolation Valve Pits by T.E. Johnson, Docket No. 50-329 and 50-330

4. American Concrete Institute, Standard Code Requirements for Nuclear Safety Related Concrete Structures, ACI 349-80

5. Consumers Power Company, Addendum to Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits, December 2, 1981

6. Attachment 2 to CPCo letter to the NRC, Serial 16597, March 31, 1982, Midland Docket No. 50-329, 50-330

REVIEW CONCERN 5

A) Provide an updated version of Drawing 7220-C-1495.

B) Should the strain gage on the wall on Column Row 5.3-5.6 at elevation 646' be oriented diagonally similar to the strain gages below elevation 614'?

C) Should the wall on Column Row 7.4-7.8 shown on Drawing 7220-C-1495 have Columns G and H reversed? Should the orientation of the strain gages also be reversed at this location?

D) What are the temperature requirements for the strain gages?

E) Provide details of strain gages and gage reading frequencies.

RESPONSE

A) Attachment 5-1 is an updated version of Drawing 7220-C-1495.

B) The strain gage (called extensometer on Drawing C-1495) was originally oriented vertically based on a preliminary survey. Further investigation showed that a diagonal orientation of the strain gage is feasible. Thus, the strain gage is now oriented as shown in Drawing 7220-C-1495.

C) Columns G and H and the strain gages should be reversed as shown in Drawing 7220-C-1495.

D) The strain gages use temperature-insensitive invar wire. Also, all strain gages are located within the temperature-controlled environment of the auxiliary building. The effect of the temperature range will be minimal; therefore, temperature requirements are not needed.

E) Strain gage details are shown in Drawing 7220-C-1495 (Attachment 5-1) and in Attachment 5-2. The reading frequency is shown in Drawing 7220-C-1493 (Attachment 5-3).

REVIEW CONCERN 6

a) Commitment to perform test load above design load (e.g., 1.30 times) on installed pier to develop load-deflection curve for verification of hard clay soil modulus. Identify pier.

b) Consider loading pier to the allowable bearing capacity for the seismic condition (22 ksf) or consider performing a plate load test to that load level.

RESPONSE

a) A load test will be performed on Pier W11 which is beneath the turbine building. The load test performed for this pier will generally have the same procedure as the test planned for an initial pier in the service water pump structure (SWPS). The procedure for the SWPS has conceptually been discussed in the response to Confirmatory Issue 14 in the April 23, 1982, submittal of Additional Information for Review of the Borated Water Storage Tank and Service Water Pump Structure Underpinning. This response will provide a more detailed discussion regarding the procedure which will be used for the auxiliary building (and SWPS) test pier.

An appendix to the underpinning specification is being developed for the test procedure. The procedure is based on ASTM D 1143-81, Standard Method of Testing Piles Under Static Axial Compressible Load. However, several modifications have been made because of the special nature of the proposed test. The load test will be supervised by the resident geotechnical engineer.

The load test for Pier W11 will be made to a jacking load which induces a maximum bearing pressure of 19 ksf. This is approximately 30% greater than the design static maximum bearing pressure of 14.7 ksf. At present, it is anticipated that a load producing 19 ksf bearing pressure load can be jacked into the system without damaging the turbine building.

The apparatus used for applying the load to the pier will be the jacking system specified to transfer load to the pier. Measuring devices to detect pier movement will, as a minimum, be the dial gages specified to measure the deflection at the top and bottom of the pier. In addition, Carlson pressure cells will be installed near the top and bottom of the shaft.

The load will be applied in increments of 25% (or less) of the jacking load (hereafter referred to as the design jacking load) required to induce a 14.7 ksf bearing pressure. Each load will be maintained until the rate of settlement is not greater than 0.01 in./hr, but not longer than 2 hours. When 100% of the design jacking load is reached and the criteria have been met, the pier will be unloaded incrementally to zero load. Each decrement of load will be held for 20 minutes. The pier will be reloaded at the same increments as initial loading allowing 20 minutes between increments until reapplication of 100% of the design jacking load is complete. At 100% of the design jacking load, the load will be maintained until the rate of settlement is not greater than 0.005 in./hr.

After the settlement criterion (0.005 in./hr) at 100% of the design jacking load is met, the load will be increased in increments of 10% (or less) of the design jacking load until the load is approximately 130% of the jacking load. Each load increment will be held until the rate of settlement is not greater than 0.01 in./hr, but not longer than 2 hours. The load at approximately 130% of the design jacking load will be held until the rate of settlement is not greater than 0.005 in./hr.

On completion of the final test loading, the pier will be unloaded in accordance with specified production jacking procedures and the wedges will be driven tightly to lock off the force as specified by the design documents.

Measures will be taken to eliminate the potential for load to be transferred via skin friction between the pier and the surrounding soil. Two options are being specified. The first consists of lining the inside of the pit with 1/2-inch thick plywood placed over the lagging and 1/2-inch thick fiberboard (Celotex). The second option consists of lining the inside of the pit with 1/2-inch thick plywood, greasing the surface of the plywood, and placing another sheet of 1/2-inch thick plywood over the first layer of plywood. In either option, no nails or fasteners will be placed between the two sheets.

Before placing the mud slab for Pier W11, a number of tests will be performed using the miniature cone penetrometers. In addition, two hand-cut, 10-inch undisturbed cube samples will be obtained in the soil directly above the bearing stratum.

b) As stated earlier, the design static maximum bearing pressure for the pier foundation is 14.7 ksf. This represents a factor of safety (FS) of 3 with respect to the ultimate bearing capacity of 44 ksf. The pier design has also been analyzed using allowable bearing pressures of 17.6 ksf (FS = 2.5) for construction conditions and 22 ksf (FS = 2) for seismic loading. The bearing pressure for the construction condition is temporary. The seismic condition represents pier loadings which are transient. In particular, the bearing pressure associated with the seismic loading is extremely short-lived and is applied dynamically rather than statically as is the pier test load.

Discussions with the NRC staff have indicated that the staff would like the pier load test to be taken to a loading with a bearing pressure in excess of 22 ksf (the allowable bearing pressure including seismic loading). It is not possible to do this practically for the pier load test because at this stage of construction, available reactions will not be sufficient. In addition, the soil modulus which would be applicable to deformations caused by earthquake-induced forces cannot be determined by loading the pier to 22 ksf. This modulus has been established by previous dynamic soil property evaluations which are presented in the FSAR.

To attain a bearing pressure exceeding 22 ksf, the NRC staff has recommended that a plate load test be performed at the bottom of the pier excavation prior to placement of the mud mat, reinforcement, instrumentation, and pier concrete. Such a test would increase the risk associated with construction and would yield results which require considerable extrapolation to the design conditions.

It is important to note that performing a plate load test at the bottom of a pier excavation will require leaving the excavation open and the subgrade exposed to environmental effects. In underpinning construction, it is prudent to minimize the time during which the excavation is left open. The longer the pier pit remains open and exposed, the greater the amount of risk to the excavation, subgrade, and adjacent structures.

The results of a plate load test would not be directly applicable to predicting the performance of a pier. Such a test would be run using an 18- to 36-inch diameter plate on the pier subgrade surface. A plate system has a considerably smaller zone of influence than a 6' x 6' rectangular pier. Also, the pier will have 35 to 40 feet of soil confinement, which would not be present in the test of a small plate. If the results of a plate load test are extrapolated to an actual pier, the results would be extremely conservative. In addition, the previously discussed comments related to the soil modulus for the seismic condition would also apply to a plate load test.

Based on the above discussion, the performance of a load test in Pier W11 to 1.3 times the design jacking load will provide sufficient verification of the hard clay soil modulus in the static load range anticipated for the underpinning foundations.

UNDERPINNING WALLS

LOCATION (SEE FIG. 1)			AXIAL K/FT	MOM'T K-FT/FT	MOM'T CAP. K-FT/FT	IN PLANE SHEAR K/FT	IN PLANE SHEAR CAP. K/FT
A	VERT. SECT.		84.5	115	378	66.4	132
B	HORIZ. SECT.		73.2	-4.2	389	42.0	143
C	VERT. SECT.		117	431	590	12	201
D	HORIZ. SECT.		-162	587	1054	13	215

INTERFACES

LOCATION	INTERFACE	AXIAL K/FT	SHEAR K/FT	SHEAR CAP. K/FT
A (FIG. 2-1)	HORIZ	21.6	65.0	87.1
E (FIG. 2-3)	VERT	49.4	113.	193

NOTE: 1) THE CAPACITIES CORRESPOND TO THE EXISTING AXIAL LOADS.
2) +VE AXIAL LOAD IS TENSION
3) THE CRITICAL OUT OF PLANE SHEAR IN THE UNDERPINNING WALL IS 23 k/ft WHILE THE CAPACITY IS 109 k/ft

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Review Concern 2
Auxiliary Building Underpinning
Design Loads

Table 2-1

NET SOIL PRESSURE (KSF)					ULT. NET BEARING CAPACITY (KSF)
POINT (SEE FIG.)	EL.	Σ(DL+LL) ± E' SETTLEMENT CASE 1	Σ(DL+LL) ± E' SETTLEMENT CASE 2	D+L CASE-1	
A	571	-19.8	-19.6	-5.2	44.0
B	571	-15.4	-15.2	-5.0	"
C	562	-13.8	-18.9	-5.9	"
D	"	-8.5	-14.7	-3.0	"
E	"	-9.8	-14.6	-4.9	"
F	"	-12.1	-16.9	-6.0	"
G	"	-13.4	-18.6	-5.9	"
H	"	-8.2	-14.5	-2.9	"
J	571	-15.0	-14.9	-5.0	"
K	571	-19.2	-19.1	-5.1	"

1. Case 1 corresponds to maximum compression @ PT. A for settlement case 1.
2. Case 2 corresponds to maximum compression @ Pt. A for settlement case 2.
3. Compression is negative

Note: Net pressure is total pressure minus the pressure due to the removed soil up to original ground elevation (el 600').

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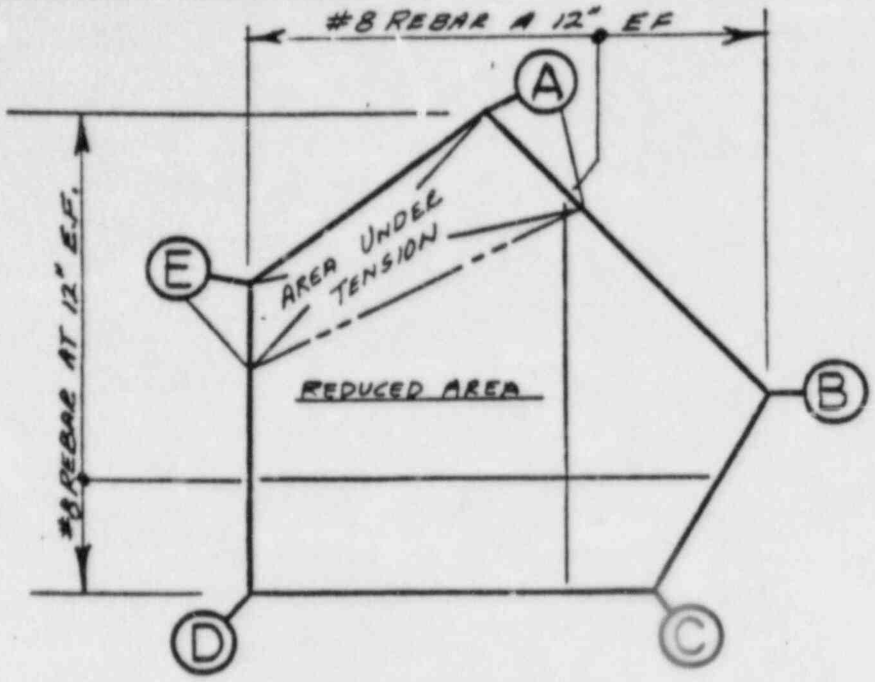
Review Concern 2
Auxiliary Building Underpinning
Soil Pressure

TABLE-2-2



SOIL PRESSURE (KSF)

	D+L+E'		D+L
POINT	CASE 1	CASE 2	CASE 3
A	3.0	.1	-3.1
B	-6.5	-6.6	-4.1
C	-10.1	-10.9	-5.3
D	-6.8	-8.0	-5.7
E	.8	-.5	-3.4

1) CASE 1 CORRESPONDS TO MIN. COMPRESSION ON TOTAL AREA

2) CASE 2 CORRESPONDS TO MIN. COMPRESSION ON REDUCED AREA

3) COMPRESSION IS NEGATIVE

4) ULTIMATE BEARING CAPACITY = 25 KSF (ESTIMATED MINIMUM VALUE)

5) THE MAXIMUM MOMENT IS 31 K-FT/FT AND THE MOMENT CAPACITY IS 112 K-FT/FT

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Review Concern 2

FIUP Soil Pressure and Rebar Detail

TABLE-2-3

REBAR STRESSES FOR PARAMETRIC STUDIES

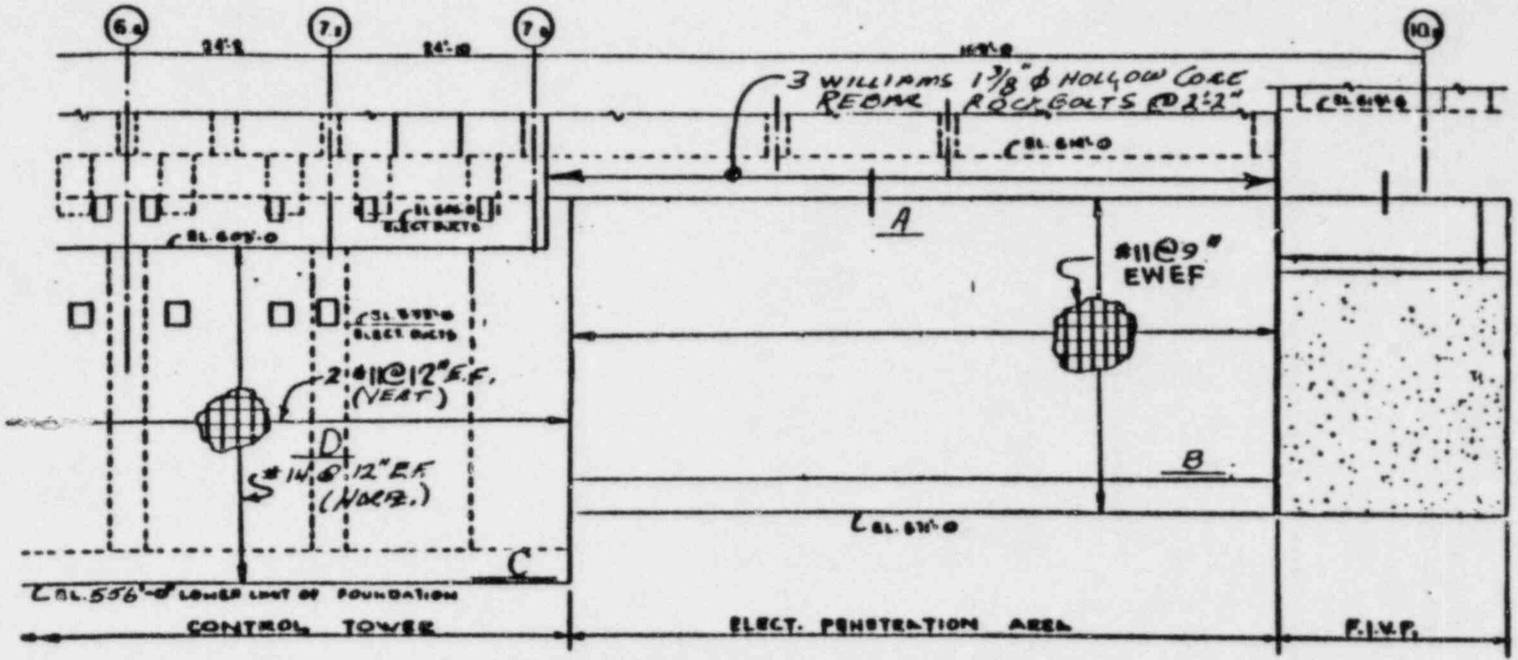
Description	Existing Stress ksi	Parametric Study I						Parametric Study 2	
		Construction Stage 1		Construction Stage 2		Construction Stage 3			
		After Soil Removal	With Jacking Load	After Soil Removal	With Jacking Load	After Soil Removal	With Jacking Load		
Wall Below El 614'-0" On Line 5.3 Between Column Lines G and H	40	44	39	37	27	48	26	40	54 ksi Allowable
Slab At El 659' Between Column Lines G and H	15	17	13	12	0*	23	0*	20	54 ksi Allowable

*Compressive stress in slab; Hence, no tensile stress in rebar.

TABLE 2-4

G-2513-13

AUXILIARY BUILDING UNDERPINNING



SOUTH ELEVATION OF UNDERPINNING WALL

CRITICAL SECTION FOR WALL DESIGN

(A) (B) ELECTRICAL PENETRATION WING

(C) (D) CONTROL TOWER APPROX. 25' N OF K_C

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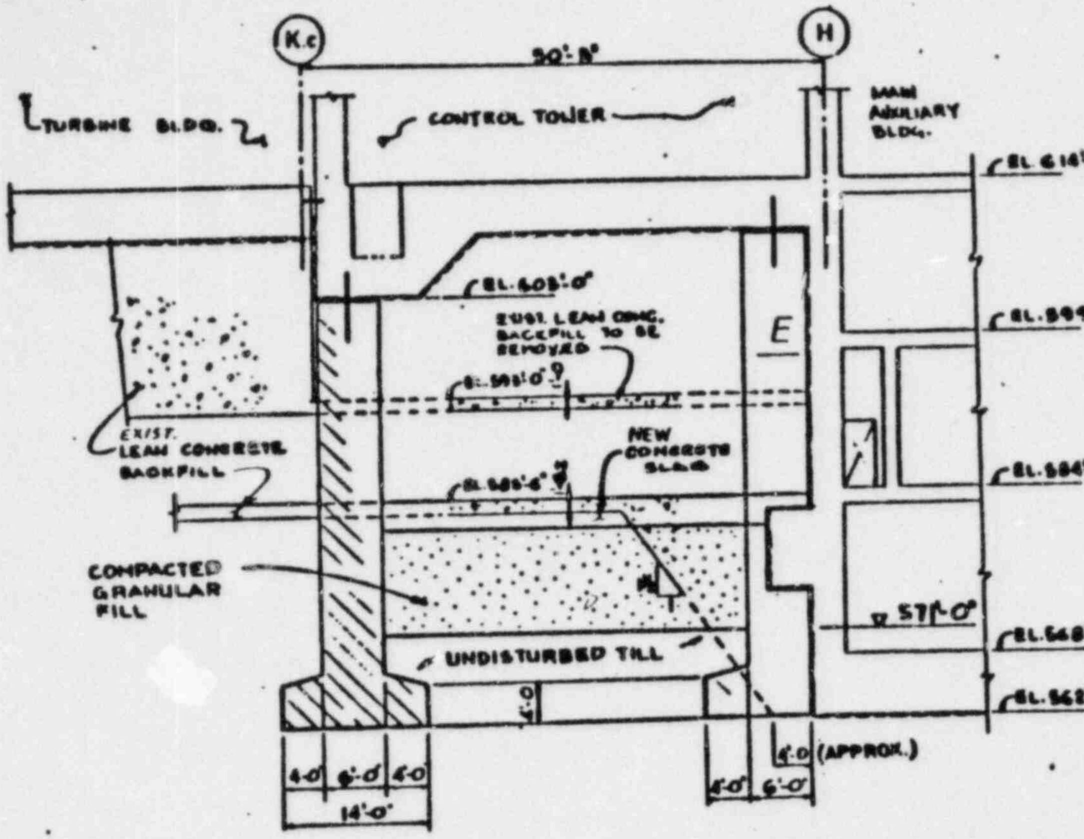
MIDLAND PLANT UNITS 1 & 2

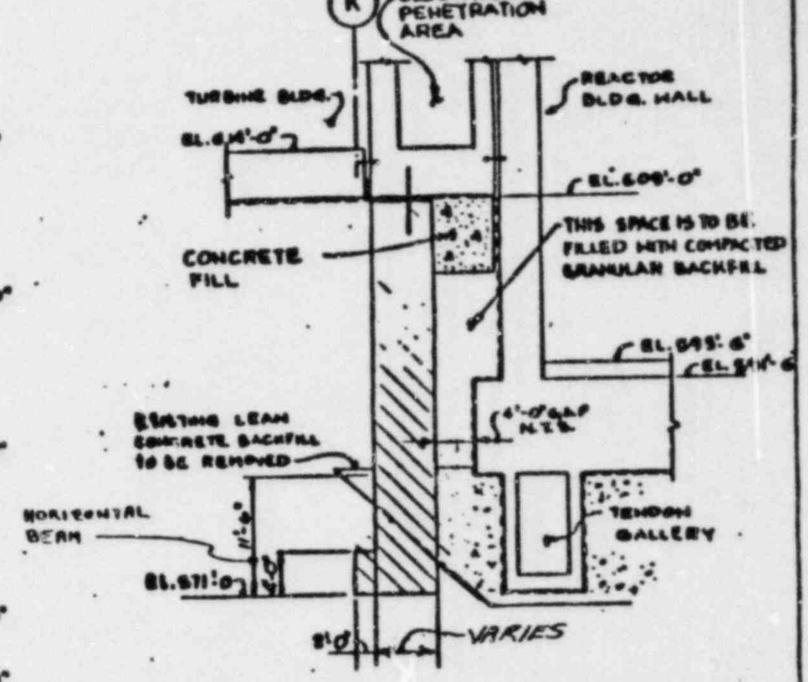
Review Concern 2

Auxiliary Building Underpinning Wall Reinforcing Detail

Figure 2-1

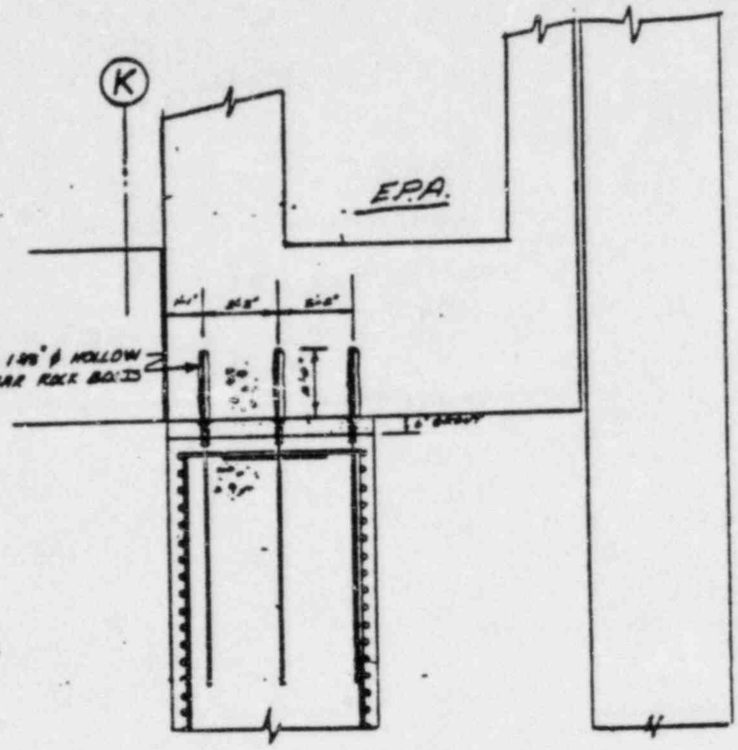


WALL UNDER CONTROL TOWER

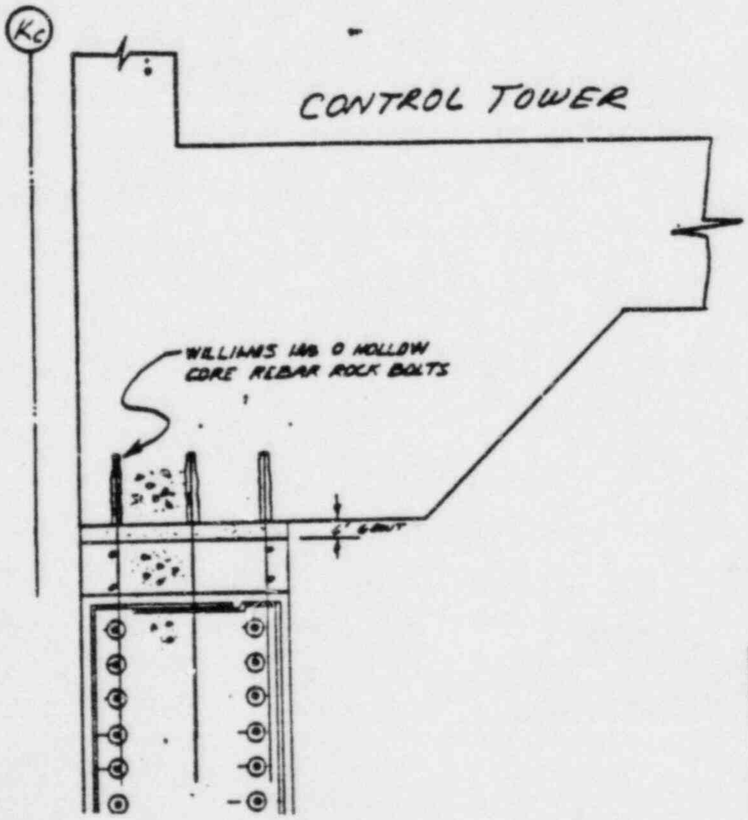


WALL UNDER ELECTRICAL PENETRATION AREA

CONSUMERS POWER COMPANY **MIDLAND PLANT UNITS 1 & 2**
Review Concern 2 UNDERPINNING WALL DETAILS
FIGURE 2-2



DETAIL OF U/P TO EPA CONNECTION



DETAIL OF U/P TO C.T. CONNECTION

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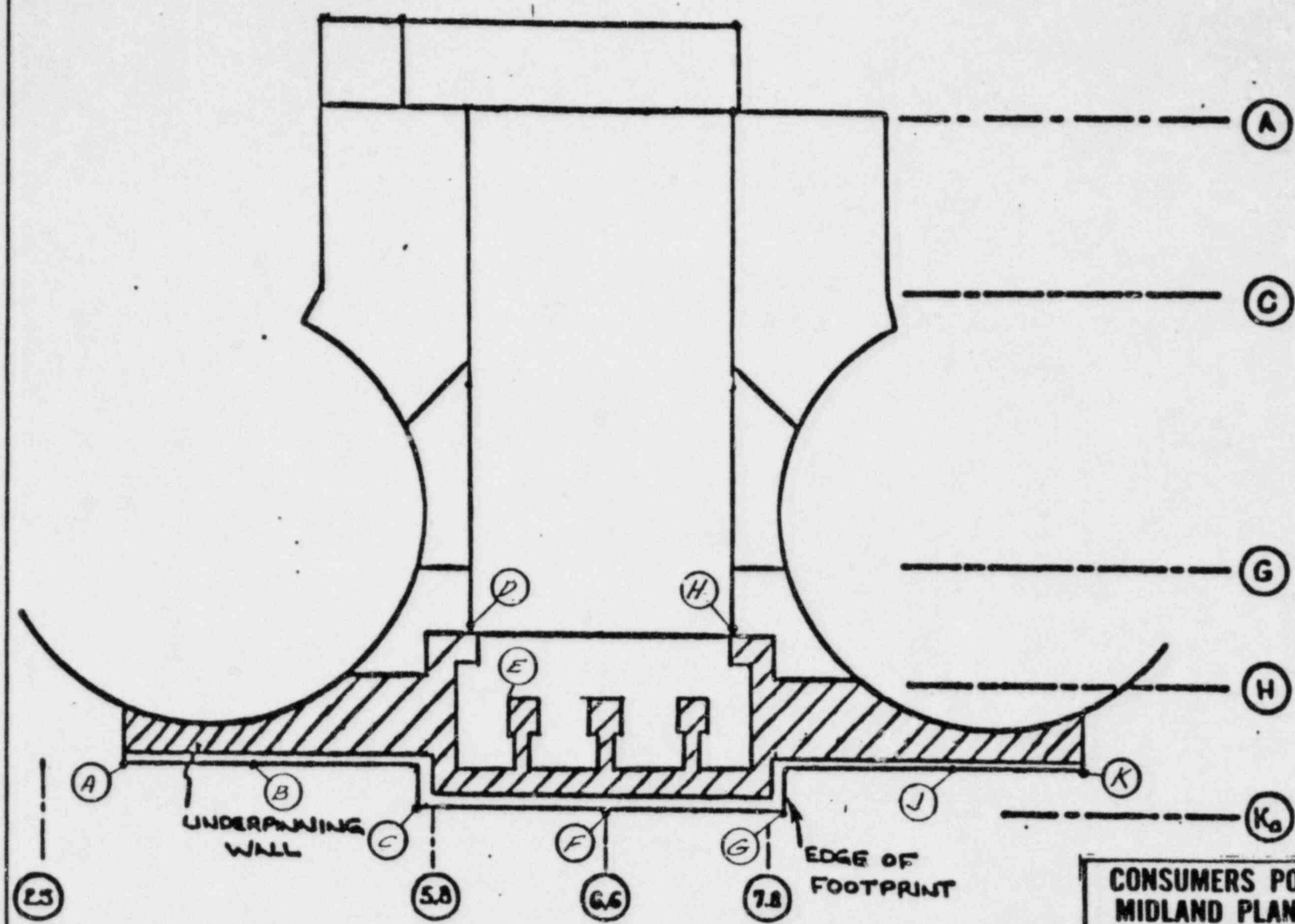
Review Concern 2
Auxilliary Building Underpinning
Wall Connection Details

Figure 2-3



PLAN-AUXILIARY BUILDING UNDERPINNING

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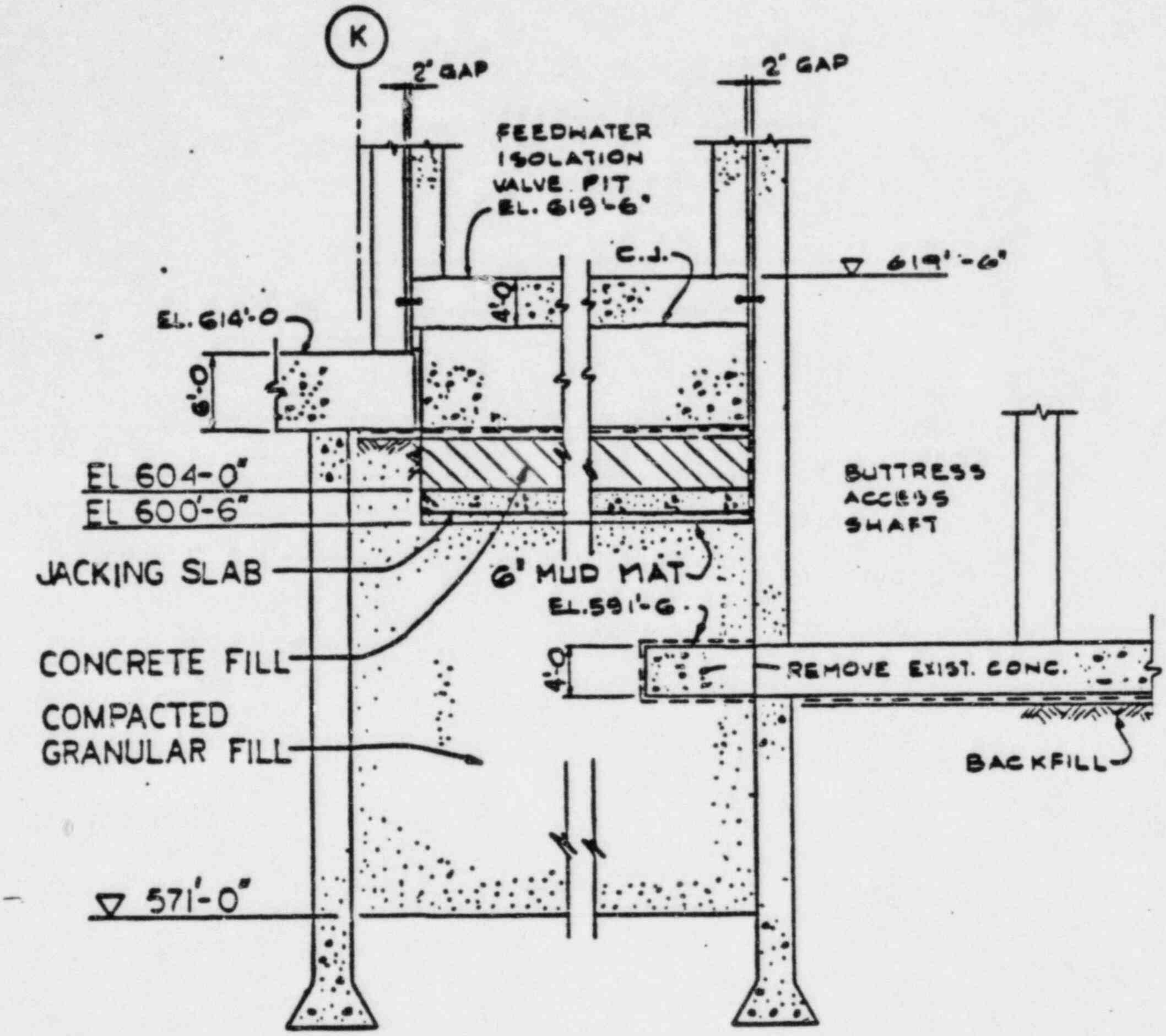
MIDLAND PLANT UNITS 1 & 2

Review Concern 2

Soil Pressure Data Points

FIGURE 2-4



F.I.V.P. SUPPORT DETAIL

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Review Concern 2
Feedwater Isolation Valve Pit
Support Detail

FIGURE 2-5

AUXILIARY BUILDING UNDERPINNING

EXISTING SOIL SPRINGS UNDER AUXILIARY BUILDING

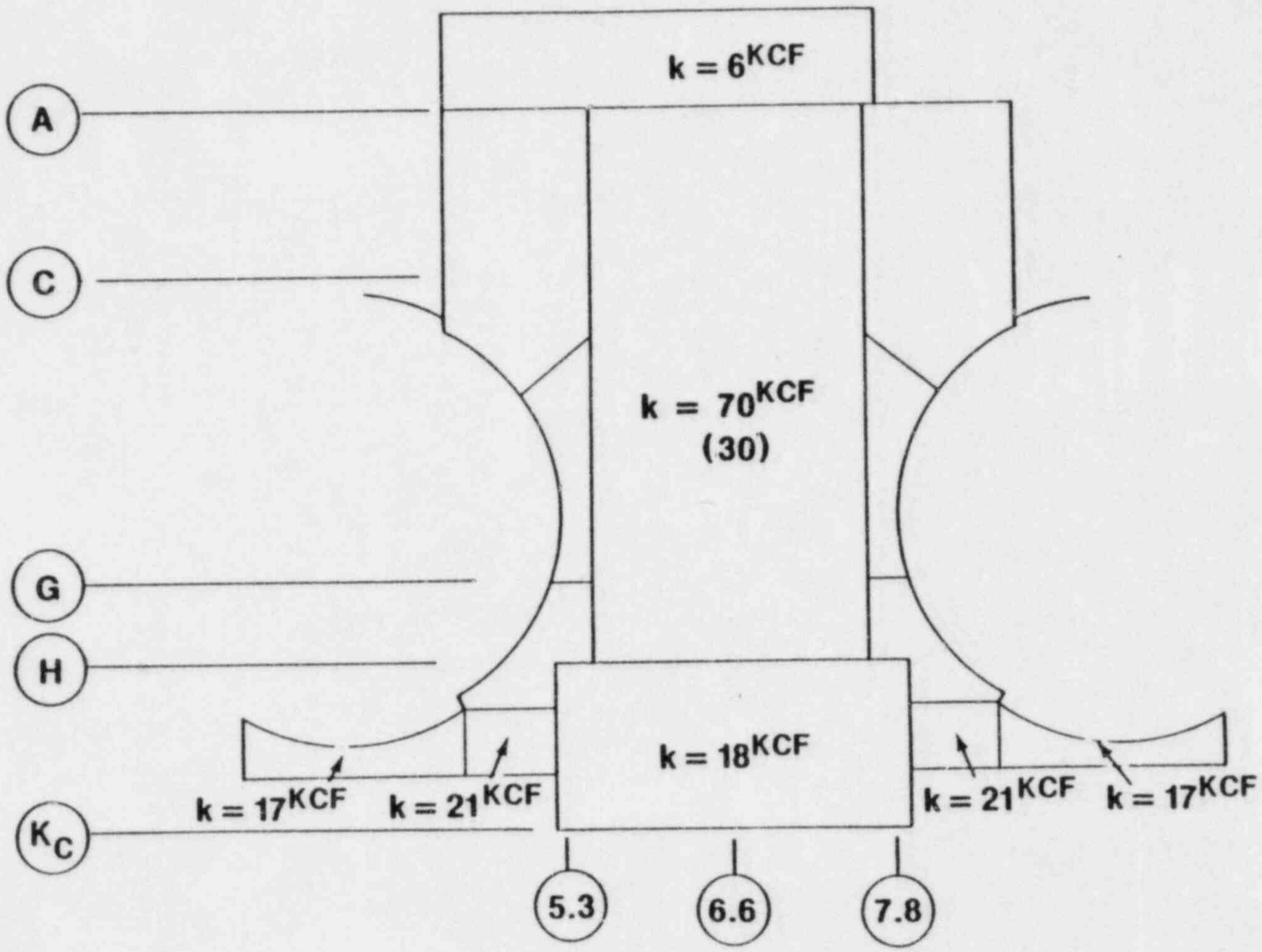


Figure 2-6

G-2513-03

AUXILIARY BUILDING UNDERPINNING
REDUCTION OF STIFFNESS

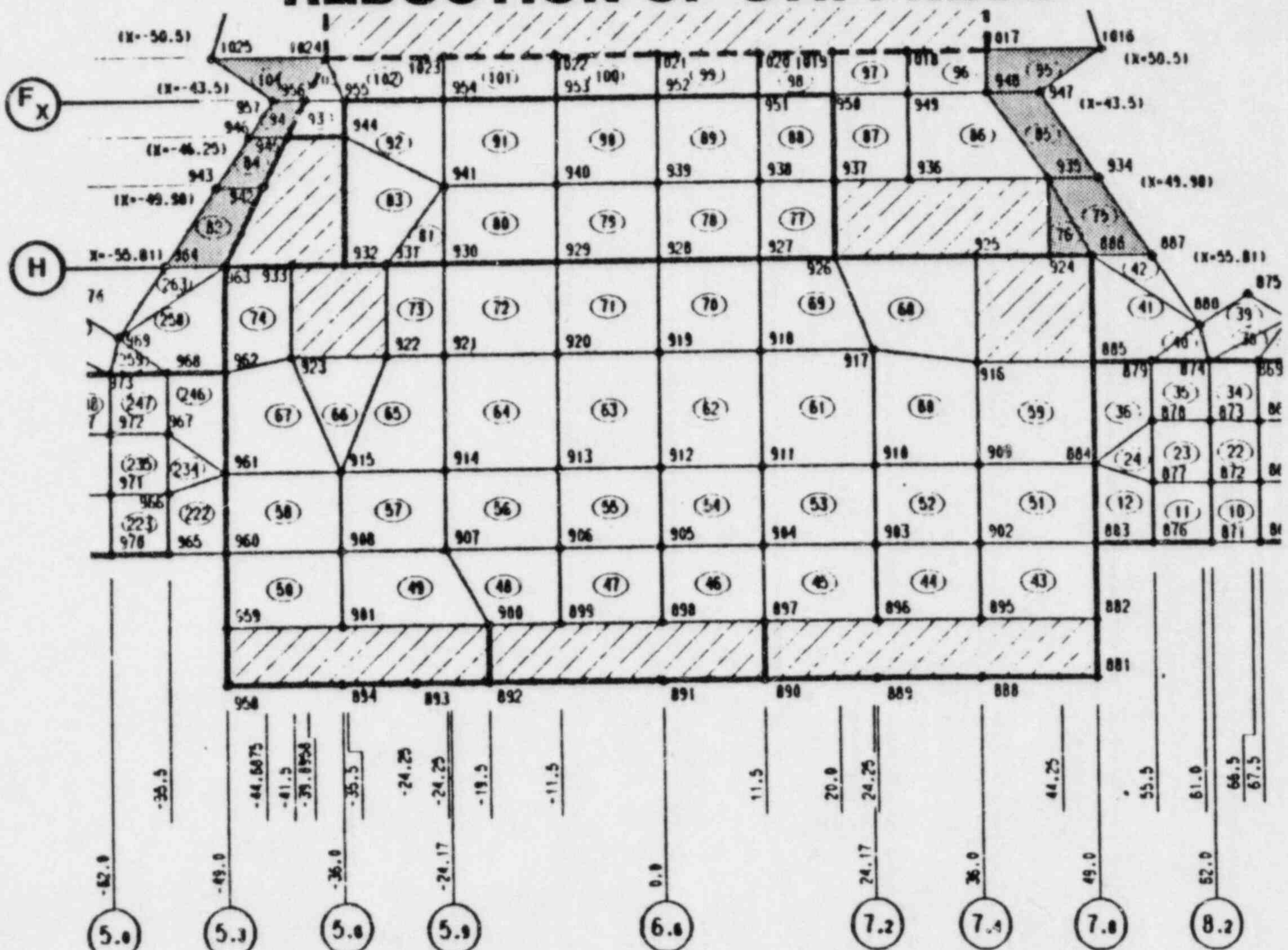


Figure 2-7

G-2513-01

AUXILIARY BUILDING UNDERPINNING REDUCTION OF STIFFNESS

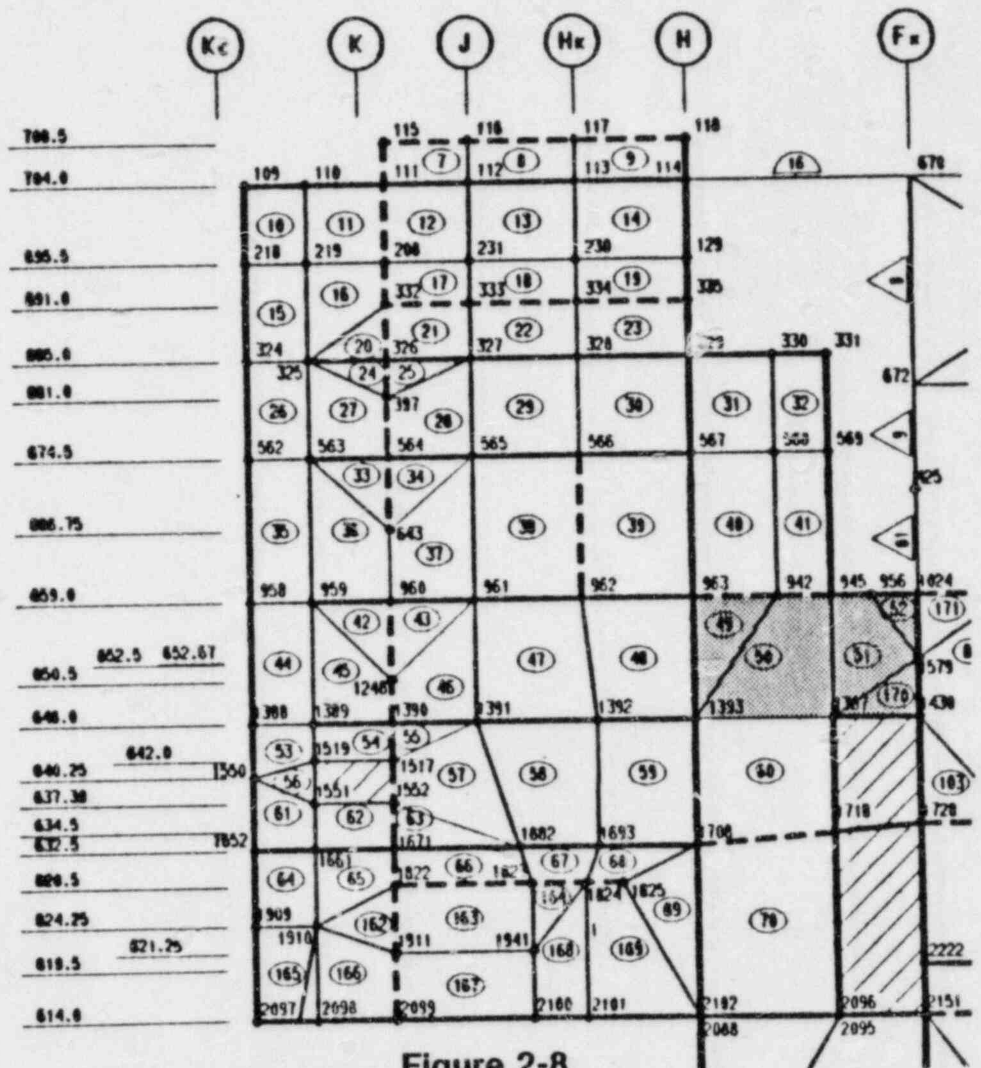
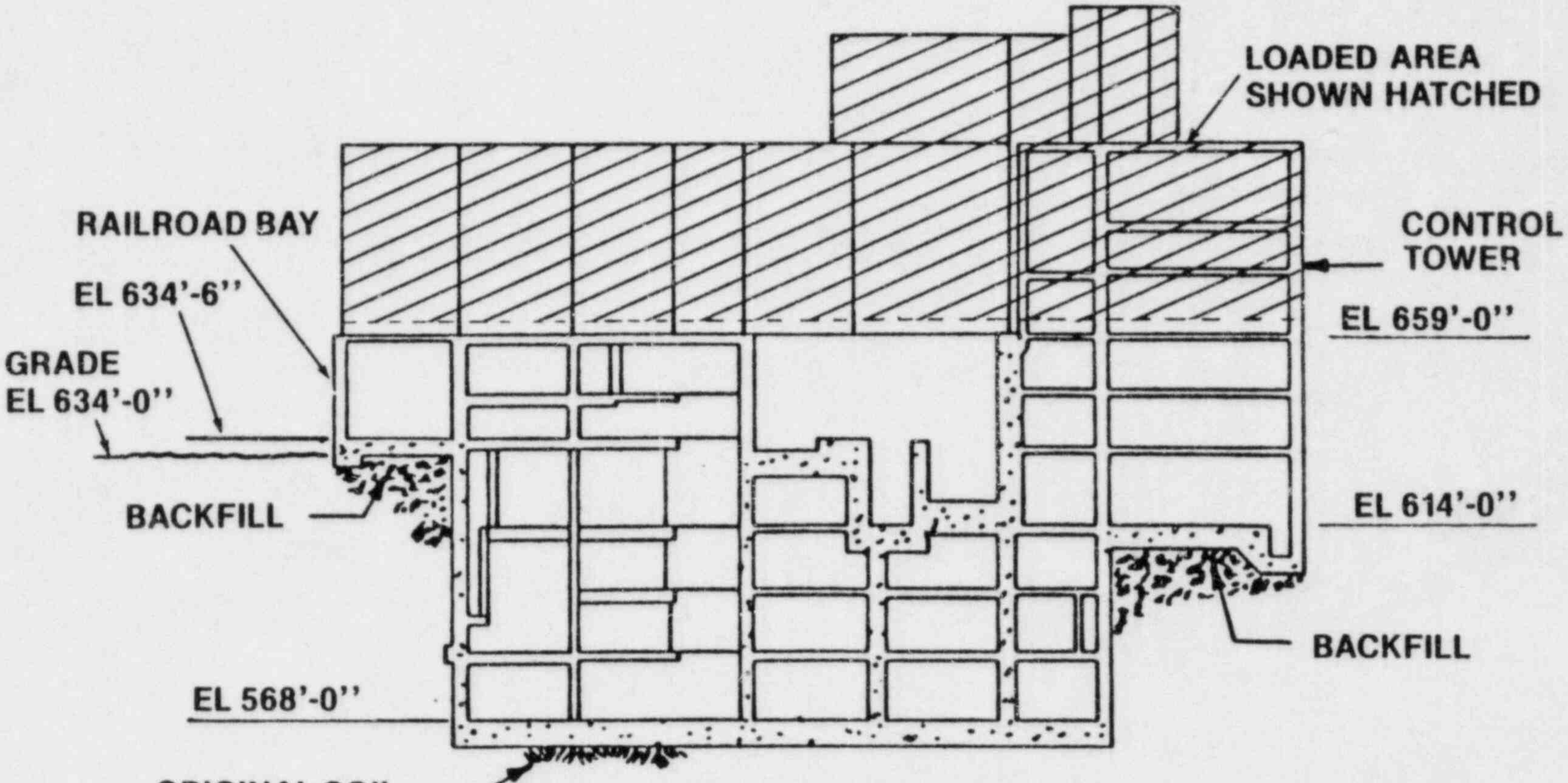


Figure 2-8
WALL NEAR COLUMN LINE 5.3

G-2513-02

AUXILIARY BUILDING UNDERPINNING
TYPICAL SECTION
(Looking East)



EXISTING STRESS ANALYSIS

LOADING CONDITION FOR EL 659'-0" AND ABOVE

Figure 2-9

G-2513-02

AUXILIARY BUILDING UNDERPINNING
TYPICAL SECTION
(Looking East)

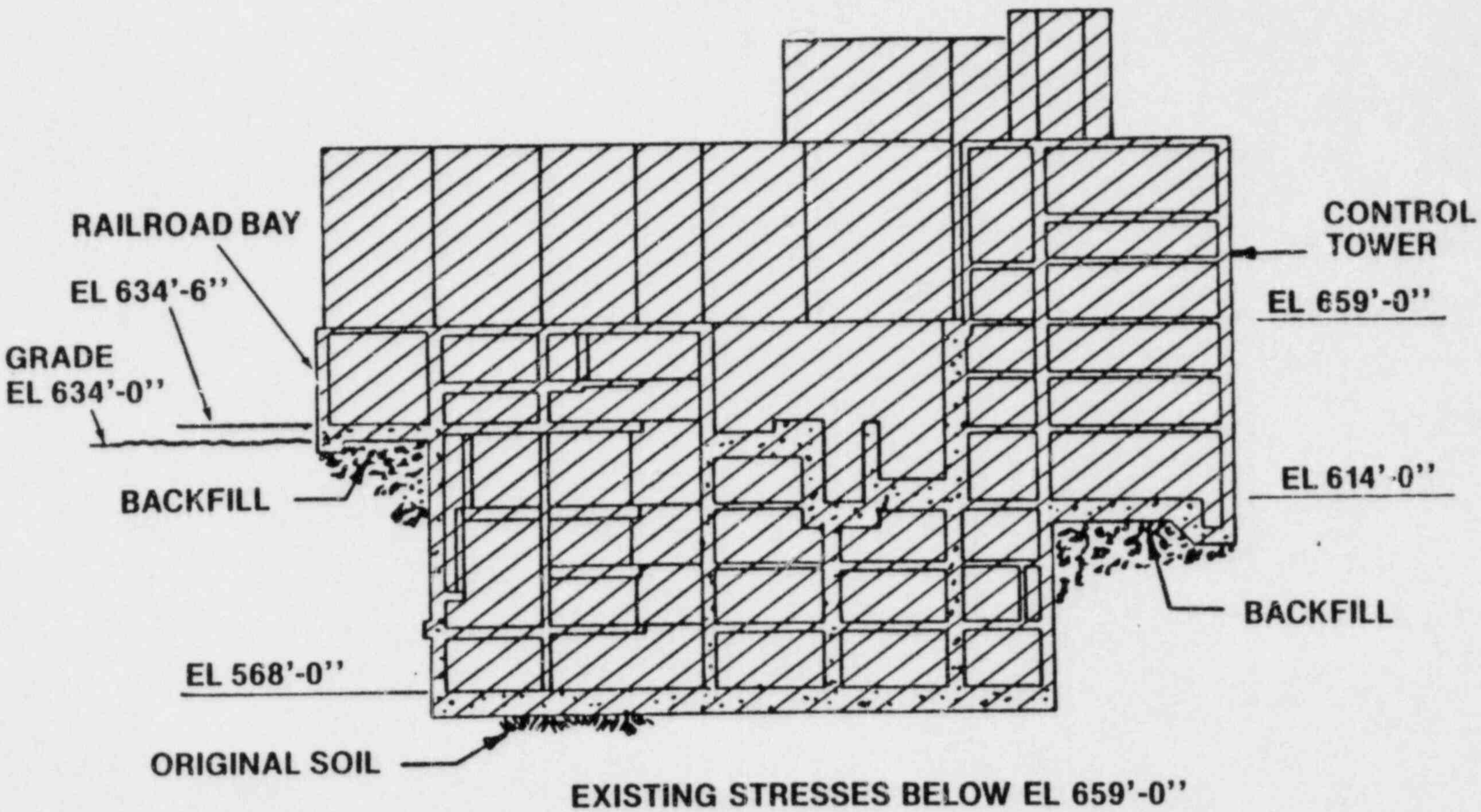


Figure 2-10

G-2513-01

AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN
CONSTRUCTION STAGE 1

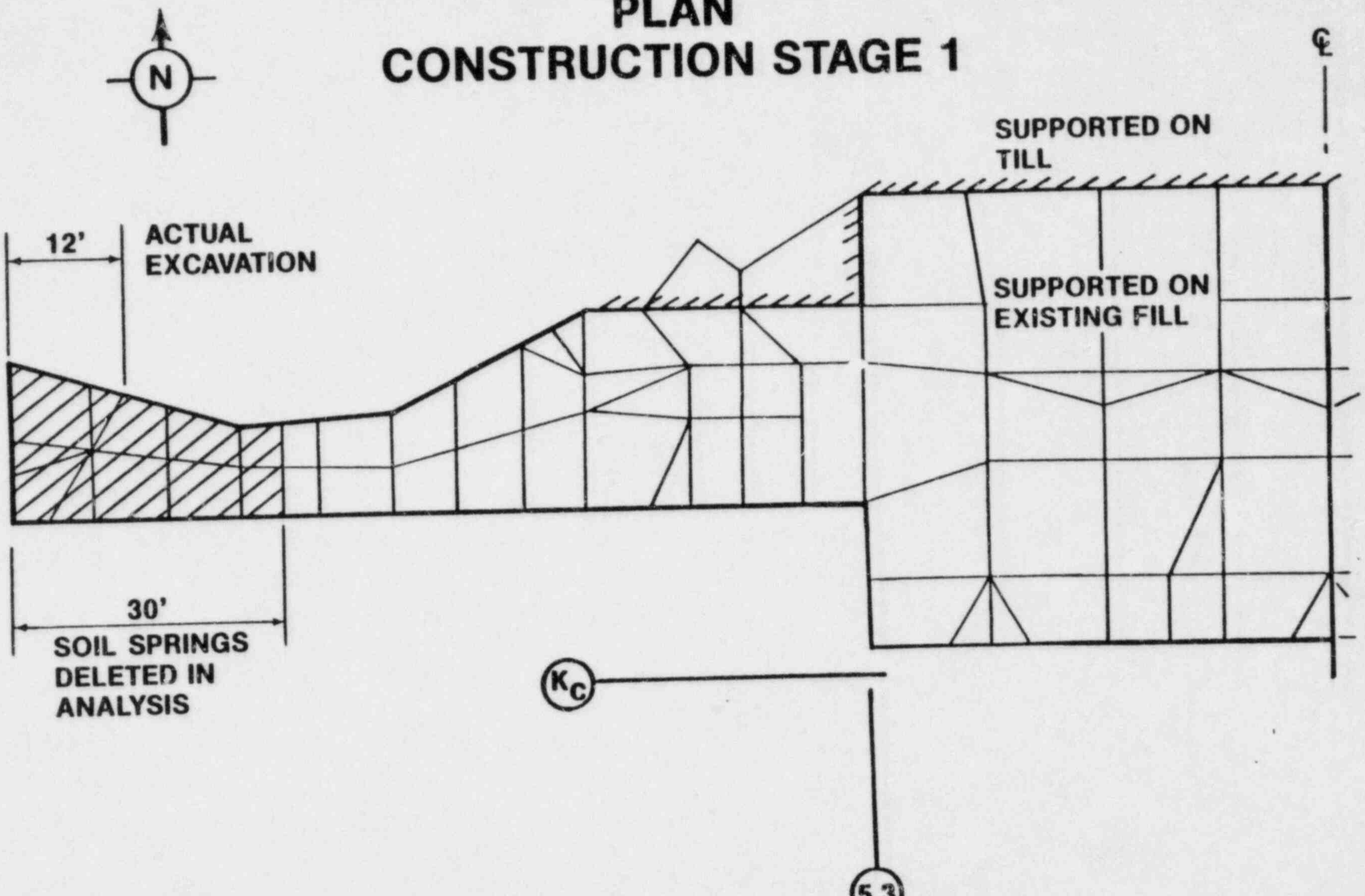


Figure 2-11
PARAMETRIC STUDY I

G-2513-05

AUXILIARY BUILDING UNDERPINNING
ELEVATION VIEW AT K_C LINE
CONSTRUCTION STAGE 1
(Soil Removal)

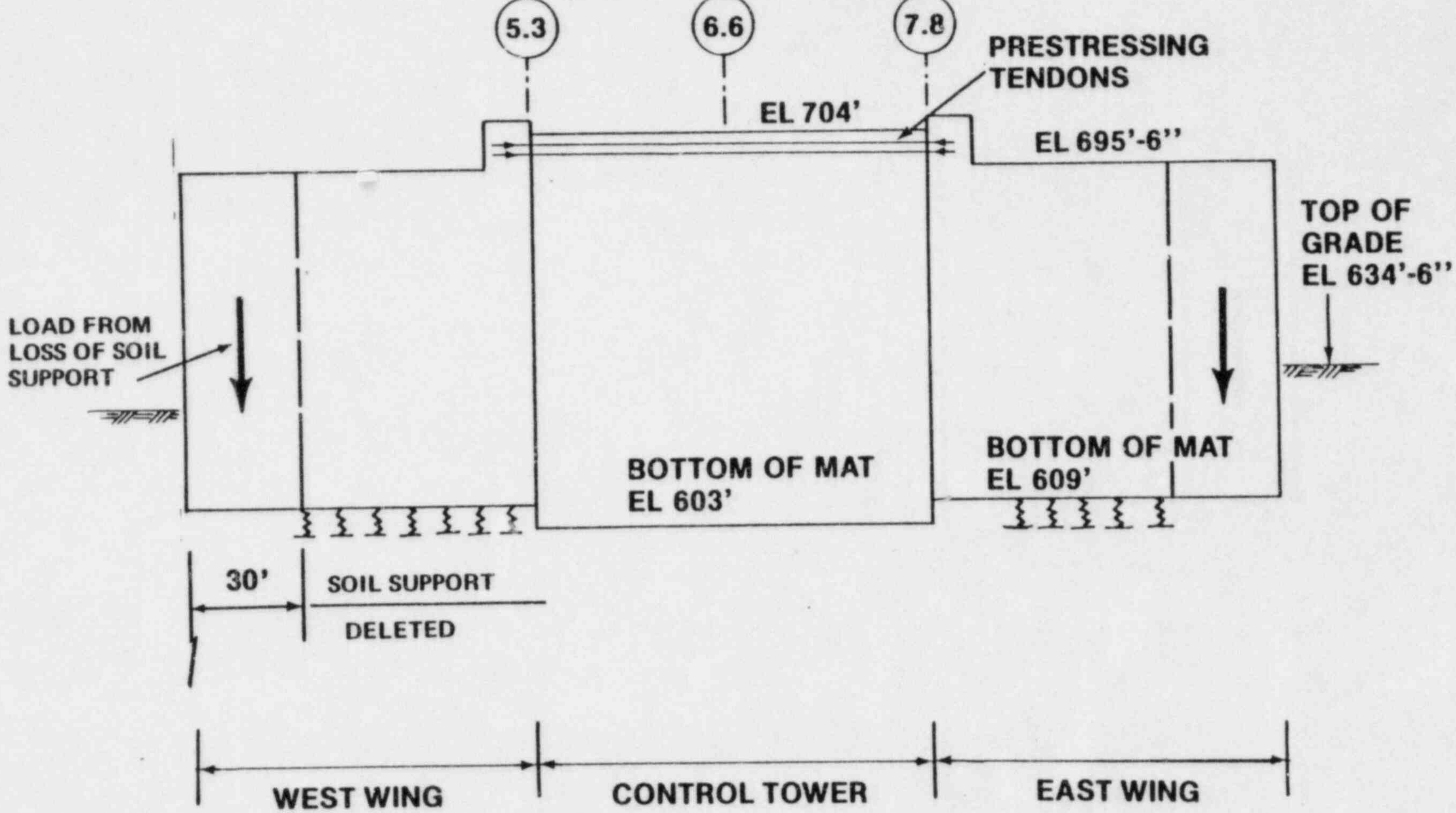


Figure 2-12

G-2513-12

AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN
CONSTRUCTION STAGE 1
(Jacking Loads Applied)

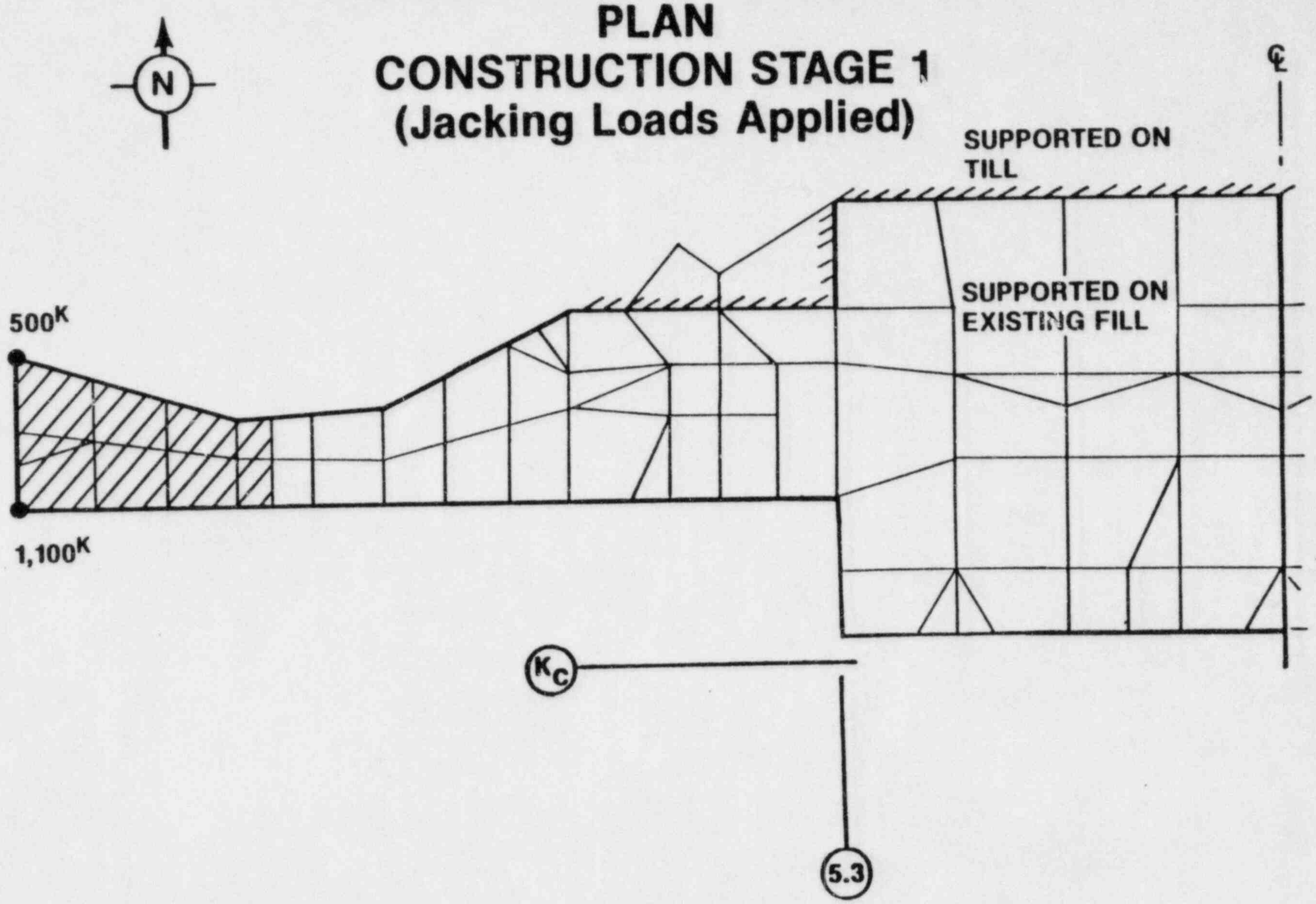
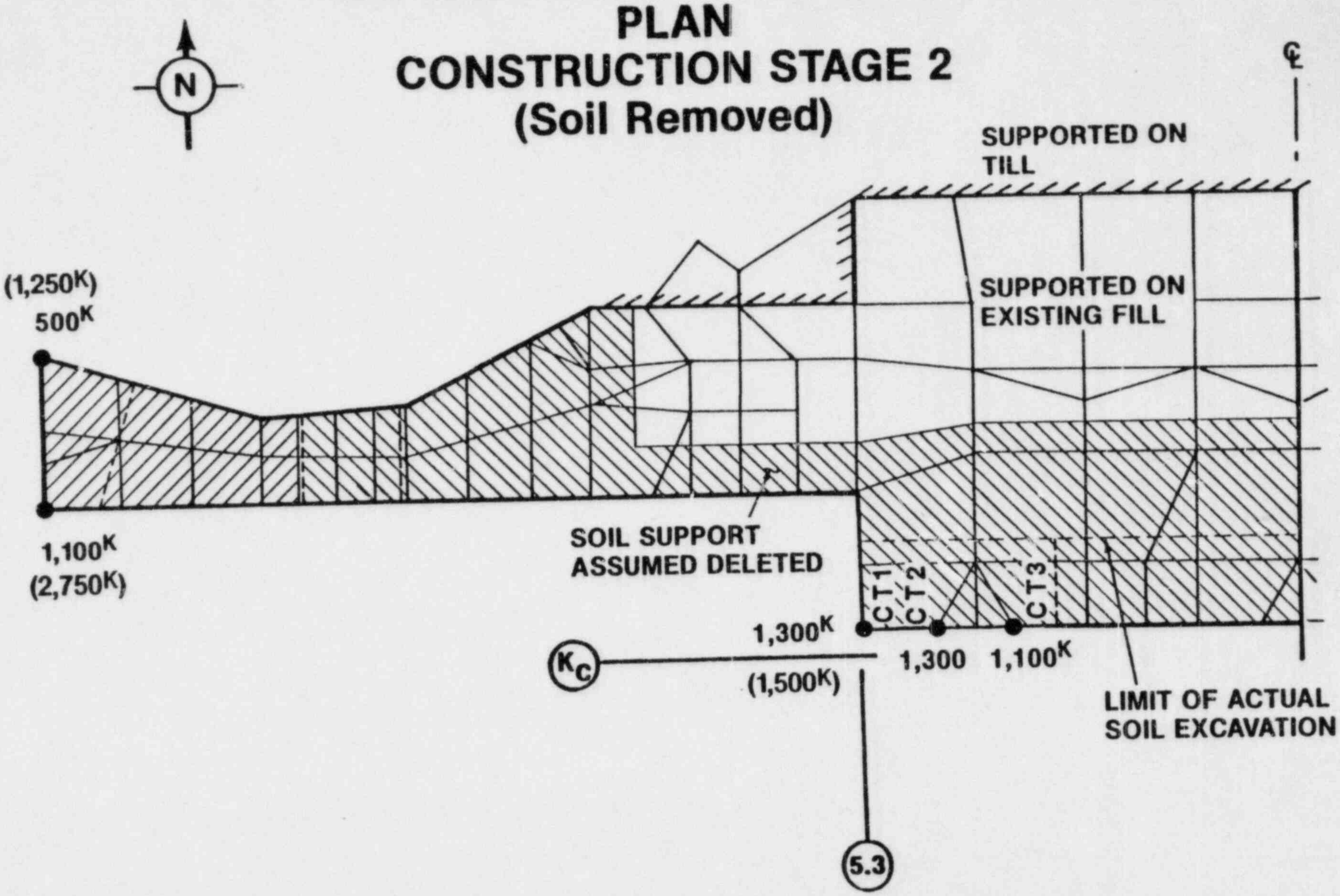


Figure 2-13
PARAMETRIC STUDY I

G-2513-08

AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN
CONSTRUCTION STAGE 2
(Soil Removed)



PARAMETRIC STUDY I
Figure 2-14

G-2513-07

AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN
CONSTRUCTION STAGE 2
(Jacking Loads Applied)

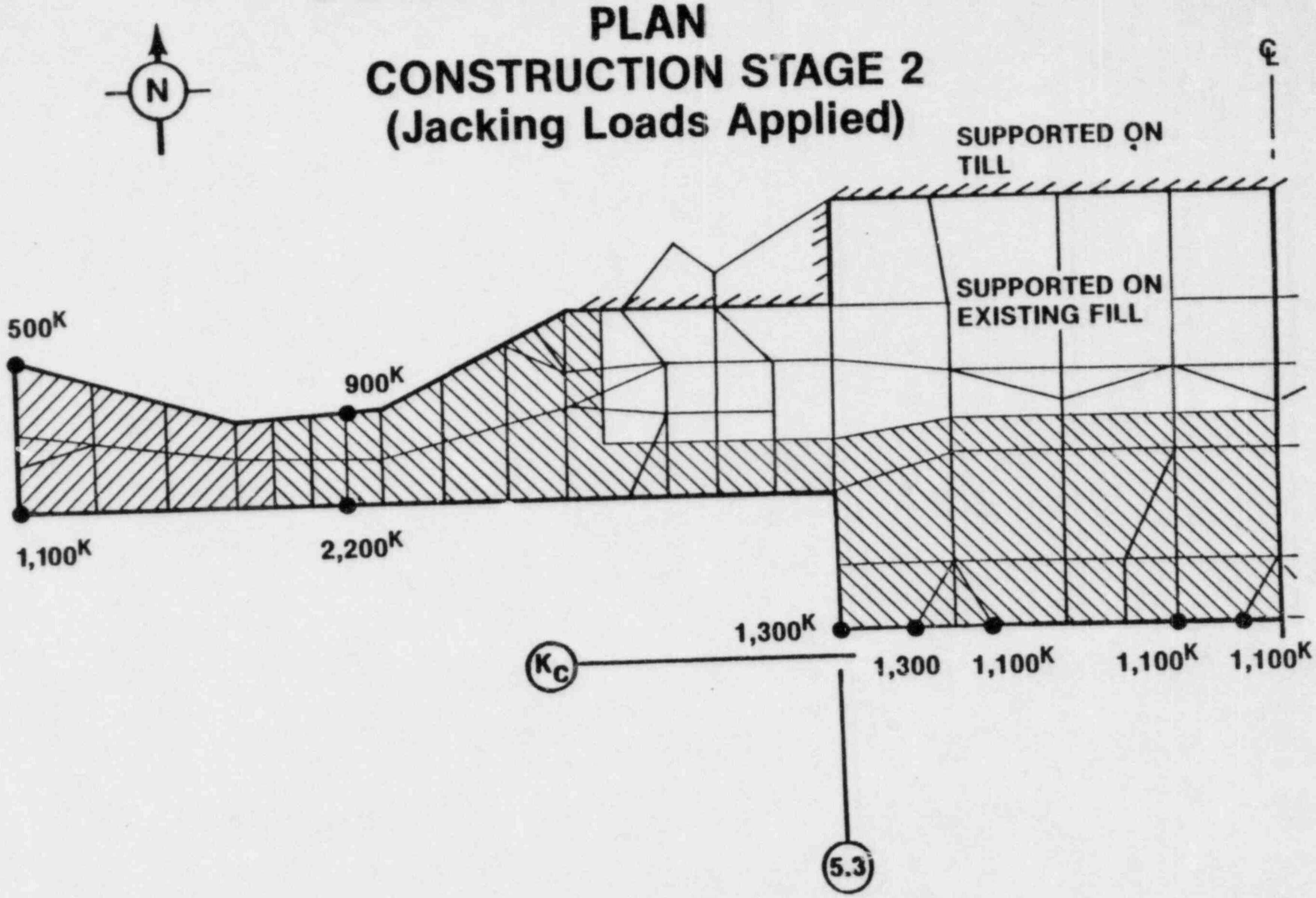


Figure 2-15
PARAMETRIC STUDY I

G-2513-08

AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN
CONSTRUCTION STAGE 3
(Soil Removed)

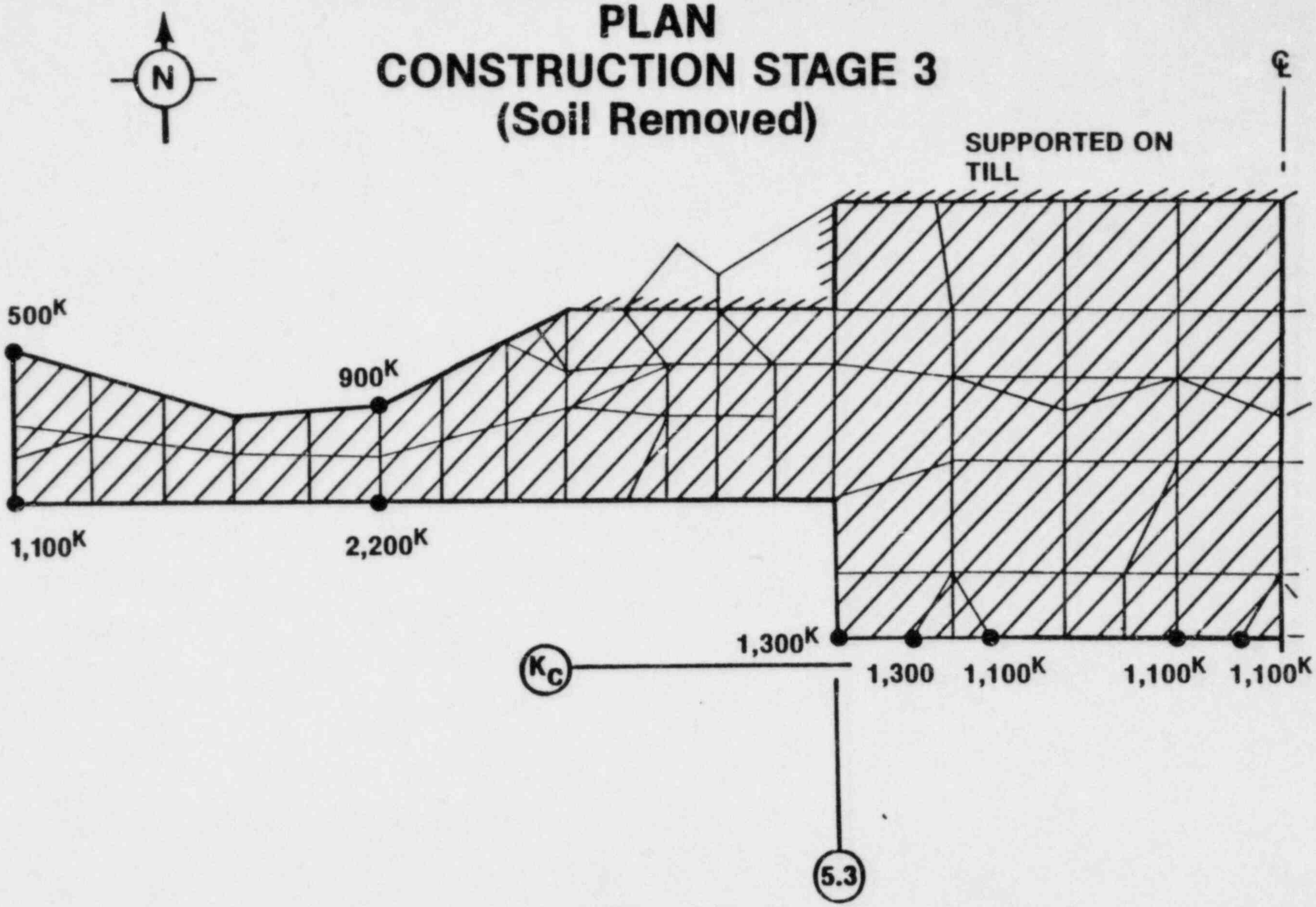


Figure 2-16
PARAMETRIC STUDY I

G-2513-11

AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN
CONSTRUCTION STAGE 3
(Jacking Loads Applied)

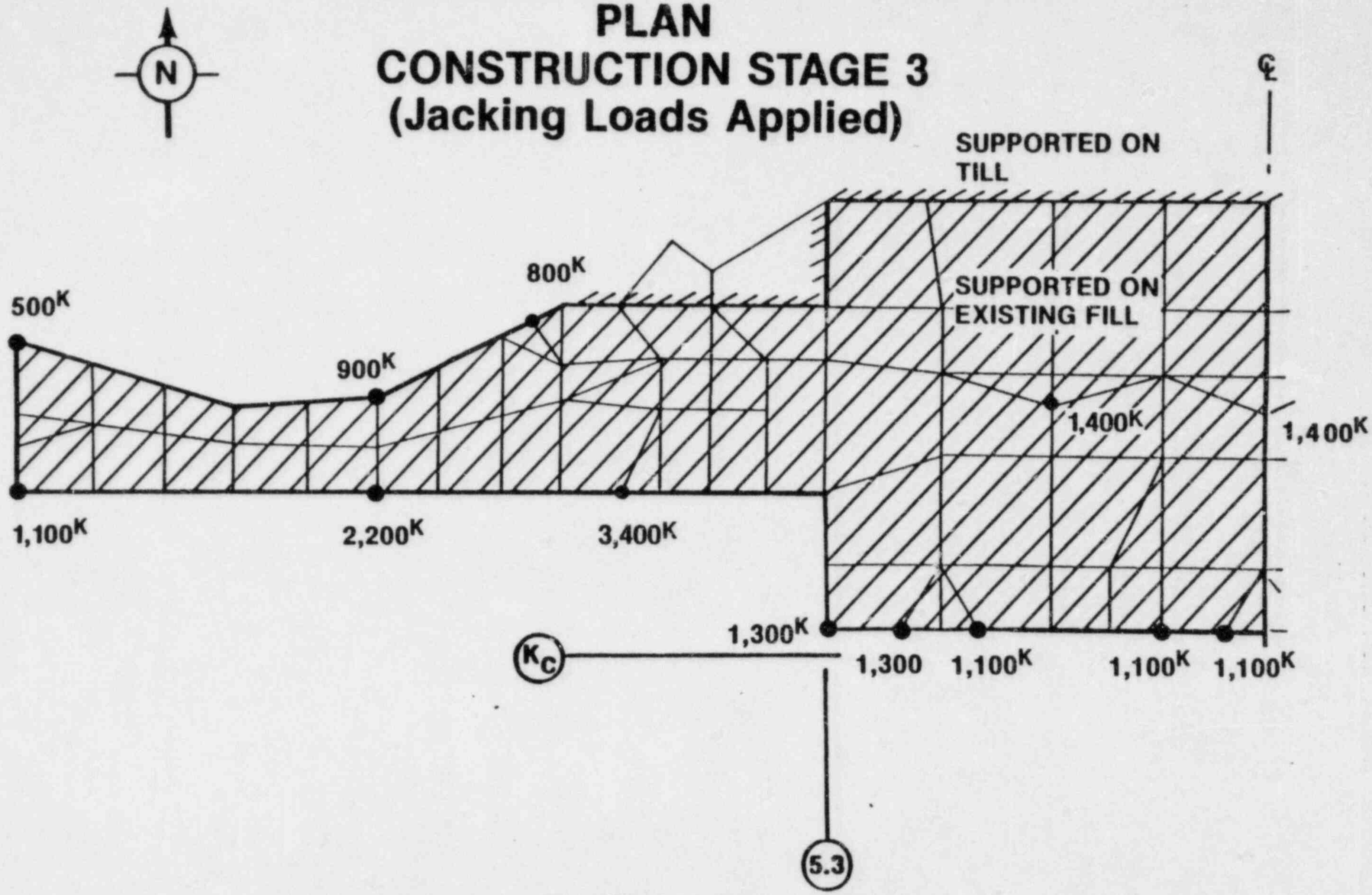


Figure 2-17

G-2513-11

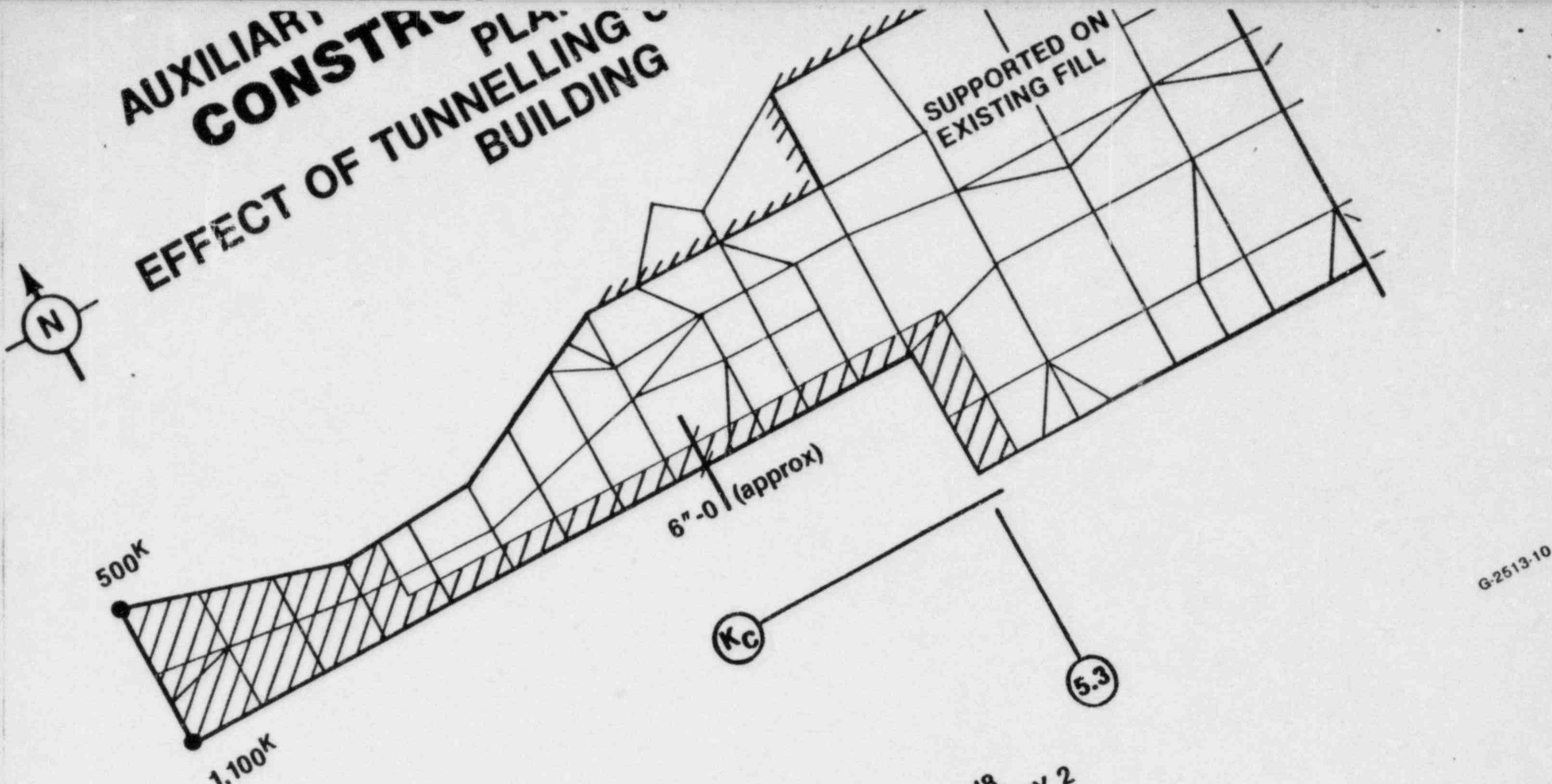


Figure 2-18
PARAMETRIC STUDY 2

AUXILIARY BUILDING UNDERPI

TYPICAL SECTION

LOCATION OF MAXIMUM STR

(Looking East)

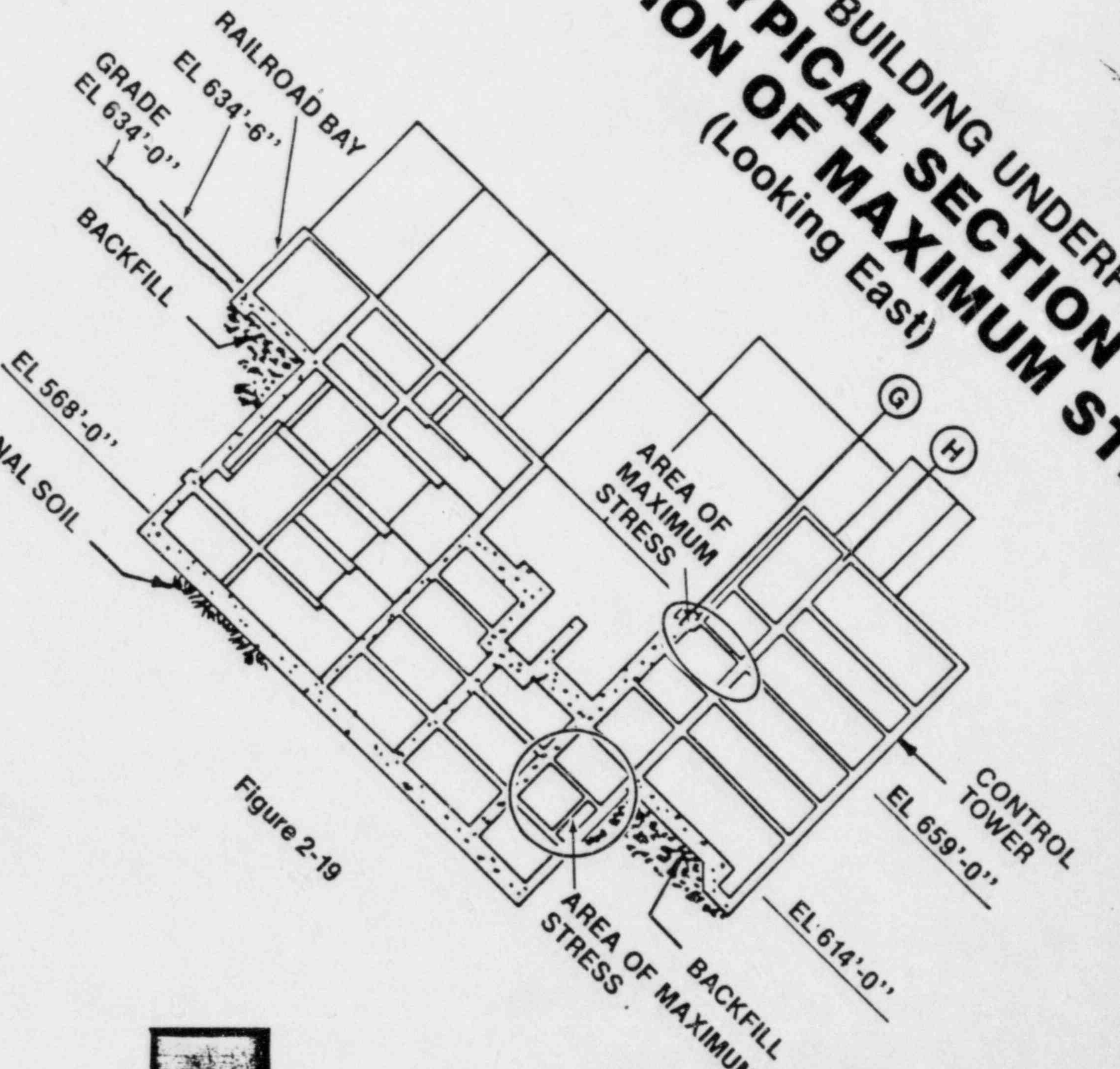


Figure 2-19

G-2513-04

Conference Call June 8, 1982

Hod, Gilray, Bird, Mooney, Schaub

permanent dewatering under QA program

only exception on attachment 1 to 4-5-82 letter

~~[illegible]~~

everything called "Q"

Consumers
Power
Company

James W Cook
Vice President - Projects, Engineering and Construction

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

June 1, 1982

PRINCIPAL STAFF			
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DEP&OS		File	

Landsman

Please return to file

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
RESPONSE TO THE NRC STAFF REQUEST FOR
SETTLEMENT-RELATED ANALYSES FOR THE
DIESEL GENERATOR BUILDING
FILE: 0485.16, B3.0.3 SERIAL: 17228
ENCLOSURE: (1) STRUCTURAL STRESSES INDUCED BY DIFFERENTIAL SETTLEMENT OF THE DIESEL GENERATOR BUILDING

As a result of meetings with the NRC during the week of February 23-26, 1982, a number of analyses were completed to resolve concerns identified by the Staff for the diesel generator building. These analyses included:
(1) analysis of the diesel generator building, including the effect of settlements which occurred before the removal of the surcharge;
(2) statistical evaluation of the diesel generator building settlement data to support the conclusion that the structure is settling as a rigid body; and
(3) analysis of the diesel generator building using zero springs and/or reduced spring values.

The diesel generator building was analyzed as documented in the technical report for the governing loading contributions including the effects of the surveyed settlements recorded from the start of construction (6-6-78) to the removal of surcharge (8-3-79), and also for the effect of the predicted forty-year settlement. The maximum rebar stresses are within the allowable of 54 ksi and are, therefore, within the strength capacity of the building to withstand the design loads specified in the FSAR and Question 15 of the NRC Requests Regarding Plant Fill.

In Attachment I-1 of the technical report the statistical evaluation of the surveyed settlement data verifies that the data contains both systematic and erratic errors due to optical surveys at different elevations due to the inaccessibility of permanent markers during the surcharge. This data lends further support to our conclusion that the diesel generator building is undergoing rigid body motion.

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In Attachment I-2 of the technical report the potential bridging of the building over soft soils was also analyzed and by comparison with the original design analysis it is concluded that the structure will withstand the stresses of this hypothesis.

We believe these analyses represent a complete response to the concerns identified by the Staff and the enclosed technical report completes the analytical activities associated with the diesel generator building.

Russell B Dewitt/For JWC

JWC/WJC/mkh

CC Atomic Safety and Licensing Appeal Board, w/o
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RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
GHarstead, Harstead Engineering, w/a
DSHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, w/o
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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TECHNICAL REPORT

STRUCTURAL STRESSES INDUCED BY DIFFERENTIAL SETTLEMENT OF THE DIESEL GENERATOR BUILDING

CONSUMERS POWER COMPANY

MIDLAND PLANT UNITS 1 AND 2

~~8206080344~~

MIDLAND PLANT UNITS 1 AND 2
TECHNICAL REPORT
STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT OF THE
DIESEL GENERATOR BUILDING

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MIDLAND PLANT UNITS 1 AND 2
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STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT OF THE
DIESEL GENERATOR BUILDING

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1.0 STRUCTURAL REANALYSIS

To account for the effect of the observed and predicted settlement on the diesel generator building, a structural reanalysis was performed. This reanalysis proceeded by defining the acceptance criteria for the structure (see Subsection 1.1). These acceptance criteria differ from the acceptance criteria used in the original design and analysis of the structure and set forth in the Final Safety Analysis Report (FSAR) only in the addition of four load combinations that include the effect of settlement. These additional load combinations are described in Subsection 1.1.2, Equations 1 through 4.

To investigate the effects of the load combinations on the structure, the structural reanalysis uses two different mathematical models of the diesel generator building: a dynamic, lumped mass model and a static, finite-element model. The dynamic, lumped mass model (described in Subsection 2.1.6 and illustrated in Figure I-1) is used to generate seismic forces in the building, given the input ground motion from the operating basis earthquake (OBE) and safe shutdown earthquake (SSE) specified in the FSAR.

The finite-element model (described in Subsection 2.0 and illustrated in Figure I-2) is a more complex mathematical model that reduces the diesel generator building to an interrelated system of plate, beam, and boundary elements representing the walls, slabs, foundation, and soil. The finite-element model is used to assess the effect on individual elements of various load combinations applied to the structure as a whole. (These load combinations include seismic forces generated with the dynamic, lumped mass model.) The finite-element model thereby allows the identification of those sections of the diesel generator building that will experience the greatest forces due to the postulated load combinations. The allowable stress is then calculated and compared to the actual stress level in these sections based on the forces derived from the finite-element model. This comparison shows that even those sections of the building experiencing the highest forces meet the acceptance criteria.

1.1 STRUCTURAL ACCEPTANCE CRITERIA

Because of the settlement problem, a structural reanalysis of the diesel generator building was performed to determine if the structure met 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 (Reference 1).

1.1.1 Load Cases

The following loads are considered in the reanalysis:

a. Dead loads (D)

b. Effects of settlement combined with creep, shrinkage, and temperature (T)

c. Live loads (L)

d. Wind loads (W)

e. Tornado loads (W')

f. OBE loads (E)

g. SSE loads (E')

h. Thermal effects (T_o)

Thermal effects appear twice in this list (Items b and h). For load combinations committed to in the response to Question 15 of the NRC Requests Regarding Plant Fill, thermal effects are contained within the settlement effects term, T. For load combinations committed to in FSAR Subsection 3.8.6.3, thermal effects are contained in the thermal term, T_o (Refer to Table I-1).

All other load cases appearing in the load combinations for Seismic Category I structures listed in FSAR Subsection 3.8.6.3 (e.g., rupture of pipe lines) do not occur in the diesel generator building and are not addressed.

1.1.2 Load Combinations

The load combinations employed for the original analysis and design of the diesel generator building are provided in FSAR Subsection 3.8.6.3. The original FSAR load combinations did not contain a settlement effects term (T). For the structural reanalysis performed in response to Question 15 of the NRC Requests Regarding Plant Fill (September 1979), four additional load combinations were established and committed to be considered. These additional combinations consider the effects of differential settlement in combination with long-term operating conditions and with either wind load or OBE. Table I-1 provides the load combinations listed in FSAR Subsection 3.8.6.3 and the four additional load combinations. These load combinations comprise the acceptance criteria for the diesel generator building and are hereinafter referred to as the Midland acceptance criteria.

By requiring combination of differential settlement with wind loads and OBE, the Midland acceptance criteria are more stringent than the requirements of American Concrete Institute (ACI) 318. ACI 318 only requires combining the effects of differential settlement with the dead loads and live loads. The Midland acceptance criteria are less stringent than ACI 349, because ACI 349 (as supplemented by Regulatory Guide 1.142) includes load combinations that combine the effects of differential settlement with extreme loads such as tornados and SSEs. In the response to Question 26 of NRC Requests Regarding Plant Fill, a commitment was made to do a separate structural reanalysis of the diesel generator building in accordance with ACI 349, as supplemented by Regulatory Guide 1.142, for comparative purposes only. Table I-2 provides the load combinations of ACI 349 as supplemented by Regulatory Guide 1.142.

It is unnecessary to use all Table I-1 load combinations in the structural reanalysis. A number of combinations can be eliminated from the analysis after comparison with more severe loads or load equations. For example, Equations 6 and 10 from Table I-1 are:

a. $U = 1.25\ (D + L + H_o + E) + 1.0T_o$ (6)

b. $U = 1.4\ (D + L + E) + 1.0T_o + 1.25H_o$ (10)

Because there are no significant forces on the structure due to thermal expansion of pipes (H_o), these two expressions can be rewritten in simpler forms:

a. $U = 1.25\ (D + L + E) + 1.0T_o$ (6)

b. $U = 1.4\ (D + L + E) + 1.0T_o$ (10)

The second expression is more critical than the first. Therefore, Equation 10 is used in the analysis and is considered to envelop the lower force components resulting from an analysis using Equation 6. Utilizing this approach with the entire set of load combinations eliminates the less critical equations and condenses the list to 10 load combinations.

	Load Combinations	Table I-1 Equation No.
a.	1.05D + 1.28L + 1.05T	(1)
b.	1.4D + 1.4T	(2)
c.	1.0D + 1.0L + 1.0W + 1.0T	(3)
d.	1.0D + 1.0L + 1.0E + 1.0T	(4)

e. $1.4D + 1.7L$ (5)

f. $1.25 (D + L + W) + 1.0T_o$ (7)

g. $1.4 (D + L + E) + 1.0T_o$ (10)

h. $0.9D + 1.25E + 1.0T_o$ (11)

i. $1.0 (D + L + E') + 1.0T_o$ (15)

j. $1.0 (D + L + W') + 1.0T_o$ (18)

1.1.3 Allowable Material Limits

In accordance with regulatory requirements and the recommendations of the American Concrete Institute (ACI 318 and ACI 349), the maximum rebar tensile stress allowed in the diesel generator building rebar equals 0.90 f_y (where f_y equals yield stress) for computation of section capacities. Because the diesel generator building rebar has an f_y value of 60 ksi, the maximum allowable tensile rebar stress due to flexural and axial loads is 54.0 ksi. Rebar stress values subsequently calculated for critical, reinforced concrete sections of the diesel generator building were based on this maximum allowable rebar stress value (54 ksi) and a maximum allowable concrete strain level of 0.003 in./in.

2.0 DIESEL GENERATOR BUILDING ANALYTICAL MODEL

The structural reanalysis of the diesel generator building 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 diesel generator building was modeled as an assemblage of plate, beam, and boundary elements. The structure is defined by a set of 853 nodal points and 1,294 elements. Of these elements, 901 are plate elements representing walls and slabs, 141 are beam elements, and 252 are boundary elements (translational springs, in both the vertical and horizontal directions) representing varying soil pressures. Vertical springs were used for dead load, live load, and settlement analysis. Sets of vertical and horizontal springs were used for other load cases. Certain items, such as steel platforms and lightly reinforced interior secondary structural walls, have not been included in the model for the reasons listed in subsequent sections. Figure I-2 illustrates an isometric view of the finite-element model.

2.1 APPLICATION OF LOADS TO THE BUILDING MODEL

The following loads have been applied to the model in the manner noted.

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2.1.1 Dead Loads

The dead load of the structure was simulated by specifying a mass acceleration value equaling that of gravity (32.2 ft/s). Secondary structural walls and platforms were not included in the model because their contribution to the gross weight of the structure is minimal (less than approximately 3 percent) relative to the sum of the other loads considered. Their exclusion does not significantly affect the magnitude or distribution of stresses. The louvers on both the north wall and south wall, along with the doors on the north and south walls of the building, were modeled simply as penetrations, with dimensions equivalent to those of the doors and louvers. This is acceptable because the doors and louvers contribute insignificantly to the building stiffness and total building weight. The diesel generator pedestals and the ground floor slabs were omitted from the finite-element model because they were not constructed monolithically with the remainder of the structure. Consequently, they do not add stiffness to the structure.

2.1.2 Settlement Loads

The settlement effects were modeled into the structure with vertical springs as boundary elements representing varying soil conditions. At 84 locations along the building footing, springs with varying properties were applied to represent the nonhomogenous nature of soil conditions existing beneath the diesel generator building.

Values for vertical springs were developed for two general cases: those springs calculated for long-term loading (dead load, live load, surcharge load, and differential settlements) and those springs calculated for short-term loading (wind, tornado, and seismic).

For long-term loading, the settlement analysis addresses four distinct time periods. A unique set of measured or estimated settlement values then corresponded to each of the following periods.

a. July 10, 1978, to August 15, 1978: Although construction of the diesel generator building began in spring 1978, survey data on the diesel generator building were available only as of July 10, 1978, August 15, 1978, represents the closest survey date prior to the halt of diesel generator building construction.

b. August 15, 1978, to January 5, 1979: Diesel generator building construction resumed and the ductbanks were separated from the structure during this period.

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January 5, 1979, is the last survey date prior to the start of surcharge activities.

c. January 5, 1979, to August 3, 1979: Surcharge activities occurred within the structure during this period. August 3, 1979, is the last survey date available prior to the start of surcharge removal from the diesel generator building.

d. Forty-year Settlement Estimates: This estimte is comprised of the following:

   1) Actual measured settlements from September 1979 to December 1981. These settlements are small when compared with the predicted settlements and are mainly due to dewatering.

   2) Predicted secondary consolidation from December 1981 to December 2025. These values are based on the conservative assumption that the surcharge remains in place over the 40-year life of the plant, thus exceeding the settlement which will actually occur.

To determine forces resulting from settlement, an analysis was performed separately for each of the above four cases. Analysis results were first combined with each other to form one settlement term, then combined with other load cases (e.g., tornado, seismic, etc) to form the required load combinations of the Midland position, and of ACI 349, as supplemented by Regulatory Guide 1.142.

For settlement case a, a longhand analysis was performed to account for stresses in the partially completed structure. With the actual settlement values from survey data imposed on the partially completed structure (represented as a grade beam up to el 635) this calculation indicated that the measured displacements would result in a maximum rebar stress of 2 ksi. For the other three settlement cases, individual finite-element models were used. For settlement case b, the finite-element model represents the structure as-built to el 662'-0". For settlement cases c and d, the finite-element model represents a fully constructed structure. In each of the three finite-element analyses, the diesel generator building was analyzed for "best fit" settlements resulting from a statistical analysis of the recorded or estimated settlements. For cases b, c, and d, springs were typically calculated at each nodal point along the foundation by dividing the structural load represented at the selected point by the measured or predicted settlement at that point. The finite-element analysis of each case then involved several iterations in which the soil springs were varied until

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the deflected shape of the diesel generator building, as calculated by the model, approximated the "best fit" settlements.

Figure I-3 summarizes the actual and estimated settlements employed in the finite-element settlement analyses (cases b, c, and d). Figures I-3A, I-3B, and I-3C give individual isometric presentations of measured and predicted settlements and also show settlement values resulting from the finite-element analysis of the diesel generator building model for cases b, c, and d. The comparison shows good correlation between values resulting from the finite-element model and the measured values and also for the predicted settlement values. Because of the great overall stiffness of the structure (shear walls are over 50 feet high and 2-1/2 feet thick) in particular when compared with the stiffness of the underlying soil, the building will undergo mainly rigid body motion. (For a complete discussion showing that the structure has been experiencing primarily rigid body motion, refer to Attachment I-1, Settlement Data Analysis.) Differences between calculated and measured settlements are small and are within the accuracy of the survey.

The maximum total rebar stress resulting from all settlement analyses (cases a, b, c, and d) is on the order of 21 ksi, which occurs in the south wall in the vertical direction. The maximum horizontal rebar stress resulting from all settlement analyses is on the order of 18 ksi, which occurs in the south shield wall. The location of maximum settlement stresses typically does not coincide with the location of maximum seismic or tornado stresses. Actual calculated moment and forces for settlements have been combined with other load cases and are included in Table I-4 in accordance with the governing load equations. (A second method of addressing settlement, involving the use of zero and near zero values for soil spring constants, is discussed in Attachment I-2.)

Other springs were developed for short-term loading, in which it was assumed that the structural movement was small enough to assume the soil was linearly elastic. The modulus of elasticity was estimated using soil density and measured shear wave velocity values. Springs were developed for the vertical and horizontal modes. These springs were calculated by determining the amount of force required to produce a unit displacement in the direction indicated by the particular mode. The footings of the diesel generator building were assumed to be resting on a large mass of elastic soil for the vertical mode and embedded within the mass of soil for the horizontal mode.

The settlement due to seismic shakedown was also identified as a possible occurrence during a seismic event. The maximum differential settlement due to seismic shakedown, as stated in Question 27 of the NRC Requests Regarding Plant Fill, is

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approximately 1/2 inch. The effects of seismic shakedown settlement will act to reduce the effects of differential settlement and for this reason the effect of seismic shakedown was not the governing case in the structural reanalysis of the diesel generator building.

2.1.3 Live Loads

Live loads were applied to the modeled structure by applying pressure loads on the plate elements which represent the floor slab at el 664'-0" and the roof at el 680'-0". During the plant life, a maximum live load of 100 psf is predicted to occur on the roof slab, whereas for the floor at el 664'-0", a maximum live load of 250 psf is postulated. One hundred percent of the live load was used in the design of individual structural members, such as floor slab at el 664'-0" and roof slab at el 680'-0". For overall building response, however, the live loads considered were limited to 25 percent of the above maximum loads. This 25-percent value represents the live load expected to be present when the plant is in operation, i.e., 100 percent of the live load will not act simultaneously on every square foot of the floor space.

2.1.4 Wind Loads

Loads resulting from the design wind (100-year recurrence with a velocity of 85 mph) were applied to the modeled structure as a pressure load on the plate elements that represent the exposed walls. Wind loads on the roof and south wall hatch covers were determined assuming the hatch covers were in place. These loads were then distributed to the nodal points which define the perimeter of the respective hatches.

2.1.5 Tornado Loads

As specified in BC-TOP-3-A (Reference 2), various combinations of velocity wind pressure, differential pressure, and local pressures were applied to the modeled structure. The maximum wind velocity of the tornado was 360 mph.

The original structural analysis performed in accordance with the FSAR considered various tornado-generated missiles. The analysis considered missiles equivalent to a 4" by 12" by 12' wooden plank (108 pounds) traveling end-on at 300 mph at any height; a 4,000 pound automobile with a velocity of 72 mph no higher than 30 feet above the ground with a contact area of 20 square feet; a 1-inch diameter, 3-foot long, 8-pound steel bar traveling at 216 mph at any height in any direction, and a 35-foot long utility pole, 13-1/2 inches in diameter, weighing 1,490 pounds, traveling at 144 mph, and striking the structure not more than

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30 feet above the ground. For tornado-generated missile loads, the structure was allowed to locally exceed the yield strain.

The results of the original tornado-generated missile load analysis showed the diesel generator building was acceptable. Results of missile impact tests conducted over the last 6 years indicate that reinforced concrete walls, thinner than the exterior walls of the diesel generator building, have a considerable margin against local damage. The tests indicate that a wall thickness of 12 inches would sufficiently preclude unacceptable local damage (spalling) from these missiles. (The thinnest exterior wall of the diesel generator building is 30 inches thick.)

2.1.6 Seismic Loads

The seismic response of a structure depends on the stiffness properties and mass of the structure, the input seismic motion at the structure location, and the soil properties of the foundation medium. Of these parameters, only soil properties are affected by insufficient compaction of backfill. The following paragraphs describe how the effects of insufficient compaction and eventual surcharging were accounted for in the revised diesel generator building seismic analysis. The design spectra and design time history as defined in FSAR Section 3.7 have been used in this reanalysis.

The analytical models used for the original seismic analysis and for the seismic reanalyses described in this report are one-dimensional, stick-type, lumped mass models using beam elements to represent the structural stiffness and impedence functions of the foundation medium (see Figure I-1).

The effect of soil-structural interaction is accounted for by coupling the structural model with the foundation media. The foundation media are represented by impedance functions which represent the equivalent spring stiffness and radiation damping coefficients as specified in BC-TOP-4-A (Reference 3).

The structural stiffness of the lumped mass model was not revised in the new dynamic analysis. The difference in the new model was confined to the treatment of the soil-structural interface. The revised analysis developed the impedance functions based on the building's foundation dimensions and the modification in the soil properties described below. In addition, for the horizontal accelerations, the weight of the soil and the concrete pedestals and diesel generator pedestals within the building were included in this revised model.

The original (presettlement) diesel generator building seismic analysis was based on the underlying till material, which has a

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shear wave velocity value of 1,359 ft/s (see Table I-3). This value was not adjusted for the 30 feet of plant fill between the till and building foundation elevation. The first seismic reanalysis accounted for the soil properties of the fill by averaging the measured shear wave velocity of the fill and underlying till (Figure I-4) over a depth of 75 feet, which is the smallest dimension of the building. This resulted in the value of 796 ft/s, which was used in the seismic reanalysis. However, the effect of decreasing shear wave velocity to a lower bound estimate of 500 ft/s was also analyzed. Both the measured shear wave velocity value of 796 ft/s and the lower bound shear wave velocity value of 500 ft/s were supplied by soil consultants.

The floor spectra at all elevations of the diesel generator building were generated using a shear wave velocity value of 796 ft/s. The resulting floor response spectra were combined in an enveloping fashion with the spectra developed in the original analysis which used a shear wave velocity value of 1,359 ft/s. The floor response spectra were further broadened to account for a lower bound shear wave velocity of 500 ft/s. Thus, conservative floor response spectra were generated.

The results of the seismic reanalysis indicated that the seismic forces at all elevations of the diesel generator building were somewhat higher than the forces determined in the original analysis. The highest seismic acceleration was derived from an analysis using a shear wave velocity value of 796 ft/s. This increased seismic load was conservatively simulated by applying the maximum structural acceleration occurring in the dynamic model to the finite-element model in north-south, east-west, and vertical directions. The combined effect of the three directional responses was assessed using the square-root-of-the-sum-of-the-squares method recommended in NRC Regulatory Guide 1.92.

The ability of the structure to withstand these increased seismic forces in combination with the other loads is described in Section 3.0.

2.1.7 Thermal Loads

Thermal effects were included in the analysis as a linear variation of temperature across the thickness of an element. The thermal effect due to linear variation of temperature across the thickness of an element (also called gradient) results in bending moments being applied to the element.

In general, the temperature gradient which is of most concern for the diesel generator building is that anticipated to occur in the winter. In accordance with the Handbook of Concrete Engineering

(Reference 4) and FSAR meteorological data, the equivalent steady-state exterior winter temperature of 14.6F was calculated. The corresponding maximum interior ambient air temperature was 75F. For information on how thermal effects were applied to the model, see Section 3.0.

3.0 ANALYSIS PROCEDURE

To determine force components in accordance with accepted analysis techniques, the force components resulting from each load condition of Section 1.1 are calculated separately. Applicable loads are applied to any of three models. (The three models are identical in every aspect except for the spring elements used to represent the soil pressures.) Various load factors are applied to the separate load conditions which are then assembled to create the required load combinations. Using this combined response, the structure is examined to ensure that the allowable stress limit is not exceeded.

3.1 SETTLEMENT/LONG-TERM MODEL

The soil moduli used to calculate the soil springs for this condition are based on the actual measured settlement data (for settlement prior to fall 1981) and estimated 40-year settlement values (for settlement subsequent to fall 1981). Dead load is applied to the model causing differential settlement to occur. As detailed in Section 2.1.2, three different models (for three different time periods) are used for this purpose. For each settlement model, an analysis iteration occurs to produce a deflected shape which best approximates the appropriate "best-fit" settlements for the particular time period being investigated. The settlement forces corresponding to each unique time period are then obtained by imposing the calculated deflection values on a finite-element model and removing the dead load.

3.2 SHORT-TERM MODEL

The soil moduli used to calculate soil springs for this model corresponds to short-term loads (i.e., wind, tornado, seismic).

3.3 ZERO-SETTLEMENT MODEL

The dead load and live load case are constructed on the zero-settlement model. To approximate zero settlement, large values are entered for the soil springs into this model.

3.4 STRUCTURAL ADEQUACY COMPUTATIONS

The computations necessary to verify structural adequacy were performed using a computer analysis program (OPTCON) capable of analyzing reinforced concrete sections. This reinforced concrete analysis program models a portion of the diesel generator building and analyzes it for forces that resulted from the BSAP finite-element model analysis. Refer to Appendix A for additional information concerning OPTCON.

To determine the structural adequacy of the diesel generator building, the modeled structure was partitioned into structural categories (i.e., north wall, center wall, roof, etc). Critical elements from each category were then selected for further investigation based on their axial force, moment, and in-plane shear force. Using OPTCON, rebar stress values were then calculated in these critical elements to verify that the allowable rebar tensile stress value was not exceeded. To facilitate the calculation process, a computer program was specifically written for selecting critical elements that would undergo OPTCON investigation. This program was written so that its selection of critical elements was based on a comparison of the axial force, bending moment, and in-plane shear force of each separate element within a structural category with all other elements of the same structural category.

Once these critical elements were selected, a thermal gradient was assigned to each element based on the location of that element within the building.

Based upon the procedure discussed above, all structural categories of the diesel generator building were investigated and found to meet the structural acceptance criteria. Table I-4 shows the results of the analysis. The left-hand column of Table I-4 describes the various structural categories of the diesel generator building. The second column shows the load combination which produces the highest stress, i.e., the load combination which is critical for a particular structural category. The third column presents the rebar stress value computed by OPTCON for the critical element within each structural category. The highest rebar stress value (reflecting the combined effects of flexural, axial, and in-plane shear loads) exist in the south wall where the rebar stress value is 44.0 ksi. The fourth column indicates the concrete compressive stress associated with the maximum rebar tensile stress in each structural category.

The final structural reanalysis of the diesel generator building showed that the critical load combinations (Table I-1) are those which include either the tornado load case (W'), the SSE load case (E'), or the settlement load case (T), specifically:

a. $1.0D + 1.0L + 1.0W' + 1.0T_o$ (18)

b. $1.0(D) + 1.0(L) + 1.0(E') + 1.0(T_o)$ (15)

c. 1.4(D) + 1.4(T) (2)

In approximately 70 percent of the diesel generator building, the tornado load combinations produce the these stress levels.

4.0 CONCLUSIONS

The diesel generator building is a massive, reinforced concrete structure with extensive reserve strength. The structural reanalysis performed on the diesel generator building verifies that the integrity of the structure will not be violated even 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 there is reasonable assurance that the diesel generator building will safely perform its intended function over the operating life of the Midland plant.

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REFERENCES

1. Consumers Power Company, Response to NRC Requests Regarding Plant Fill, Docket 50-329, 50-330

2. Bechtel Power Corporation, Tornado and Extreme Wind Design Criteria for Nuclear Power Plants, Revision 3, August 1974 (BC-TOP-3-A)

3. Bechtel Power Corporation, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Revision 3, November 1974 (BC-TOP-4-A)

4. M. Fintel, Handbook of Concrete Engineering, Van Nostrand Reinhold Company, September 1974

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APPENDIX A

OPTCON

The OPTCON computer code is a versatile and complete design and analysis program for reinforced concrete structures. The program may be used for the investigation of an existing reinforced concrete section where the reinforcing steel area is predetermined. Alternatively, it can be used for obtaining an optimum design by allowing the program to determine the minimum reinforcement required.

The computer program operates on the axial force/moment interaction diagram (IAD) of a section, where an IAD is a plot of the maximum allowable resistance of a section for given stress and strain limitations. Combinations of moment (M) and axial load (P) falling within this area are acceptable. Figure IA-1 depicts the appropriate IAD for a symmetrically reinforced, symmetrically shaped section subjected to a combination of flexural and axial loads.

The OPTCON program handles loads consisting of axial forces and corresponding bending moments due to different types of loads. Special subroutines are provided to incorporate the thermal effects into the design and/or investigation. The cracking effect of the concrete and the yielding effects of the reinforcement (as allowed by the appropriate stress/strain yielding criteria) are considered in the calculation of the thermal loads and moments computed by the program.

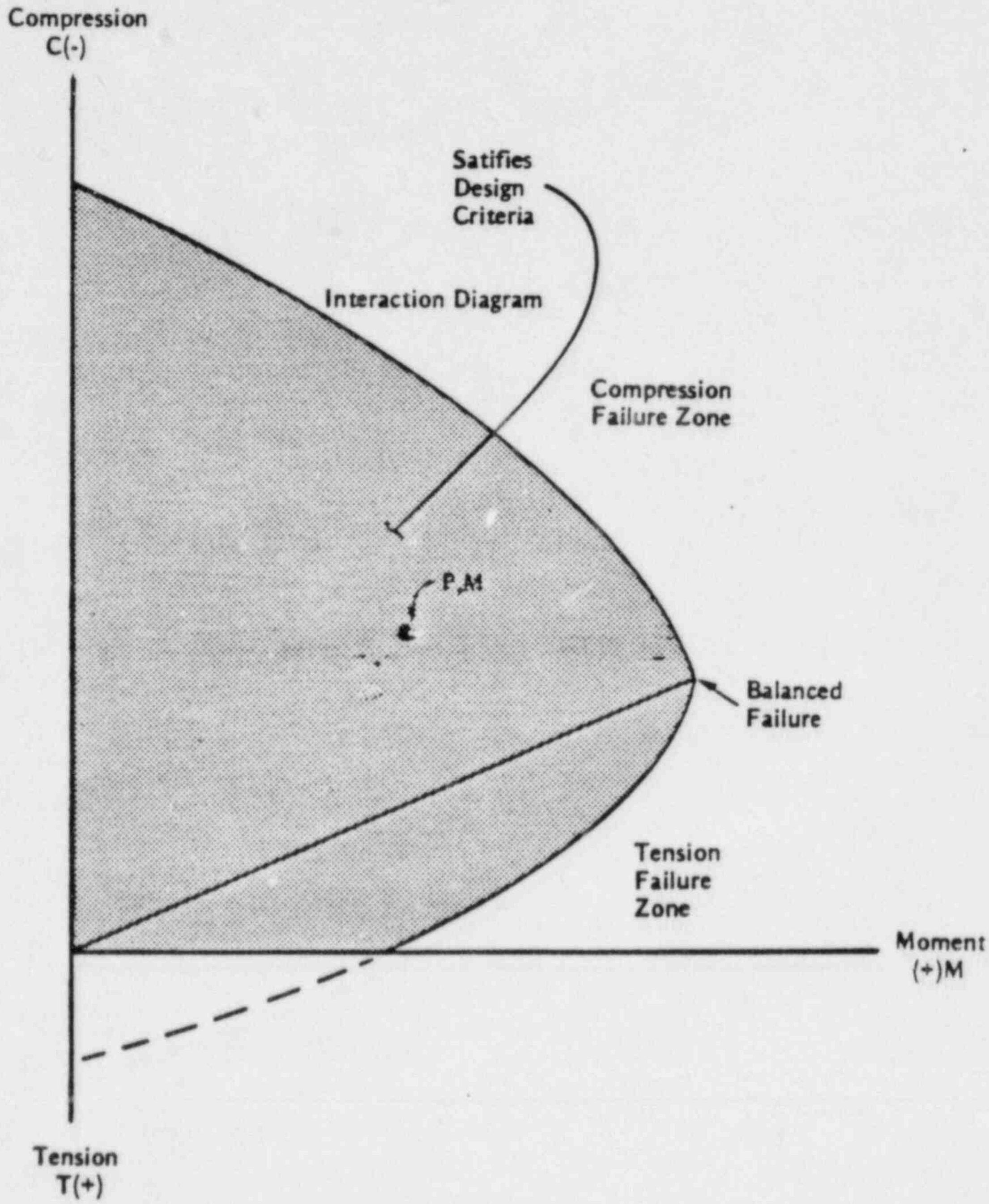

Figure IA-1

TYPICAL INTERACTION DIAGRAM
(for single axis bending on a section
with symmetrical re.. forcement)

000720-0

TABLE I-1

LOADS AND LOAD COMBINATIONS FOR CONCRETE STRUCTURES OTHER THAN THE CONTAINMENT BUILDING FROM THE FSAR AND QUESTION 15 OF RESPONSES TO NRC REQUESTS REGARDING PLANT FILL

Responses to NRC Requests Regarding Plant Fill, Question 15

a. Service Load Condition

$U = 1.05D + 1.28L + 1.05T$ (1)

$U = 1.4D + 1.4T$ (2)

b. Severe Environmental Condition

$U = 1.0D + 1.0L + 1.0W + 1.0T$ (3)

$U = 1.0D + 1.0L + 1.0E + 1.0T$ (4)

FSAR Subsection 3.8.6.3

a. Normal Load Condition

$U = 1.4D + 1.7L$ (5)

b. Severe Environmental Condition

$U = 1.25\ (D + L + H_0 + E) + 1.0T_0$ (6)

$U = 1.25\ (D + L + H_0 + W) + 1.0T_0$ (7)

$U = 0.9D + 1.25\ (H_0 + E) + 1.0T_0$ (8)

$U = 0.9D + 1.25\ (H_0 + W) + 1.0T_0$ (9)

c. Shear Walls and Moment Resisting Frames

$U = 1.4\ (D + L + E) + 1.0T_0 + 1.25H_0$ (10)

$U = 0.9D + 1.25E + 1.0T_0 + 1.25H_0$ (11)

d. Structural elements carrying mainly earthquake forces, such as equipment supports

$U = 1.0D + 1.0L + 1.8E + 1.0T_0 + 1.25H_0$ (12)

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Table I-1 (continued)

e. Extreme Environmental and Accident Conditions

$$U = 1.05D + 1.05L + 1.25E + 1.0T_A + 1.0H_A + 1.0R \quad (13)$$

$$U = 0.95D + 1.25E + 1.0T_A + 1.0H_A + 1.0R \quad (14)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_0 + 1.25H_0 + 1.0R \quad (15)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_A + 1.0H_A + 1.0R \quad (16)$$

$$U = 1.0D + 1.0L + 1.0B + 1.0T_0 + 1.25H_0 \quad (17)$$

$$U = 1.0D + 1.0L + 1.0T_0 + 1.25H_0 + 1.0W' \quad (18)$$

where

B = hydrostatic forces due to the postulated maximum flood

D = dead loads of structures and equipment and other permanent load contributing stress

E = operating basis earthquake (OBE)

E' = safe shutdown earthquake load (SSE)

H_0 = force on structure caused by thermal expansion of pipes under operating conditions

H_A = force on structure caused by thermal expansion of pipes under accident conditions

L = conventional floor and roof live loads (includes moveable equipment loads or other loads which very in intensity)

R = local force, pressure on structure, or penetration caused by rupture of pipe

T = effects of differential settlement, creep, shrinkage, and temperature

T_0 = thermal effects during normal operating conditions, including linear expansion of equipment and temperature gradients

T_A = total thermal effects which may occur during a design accident

U = required strength to resist design loads or their related internal moments and forces

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Table I-1 (continued)

W = design wind load

W' = tornado wind loads, excluding missile effects, if applicable (refer to Subsection 2.2.3.5)

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TABLE I-2

LOADS AND LOAD COMBINATIONS FOR COMPARISON ANALYSIS REQUESTED IN QUESTION 26 OF NRC REQUESTS REGARDING PLANT FILL

ACI 349 as Supplemented by Regulatory Guide 1.142

a. Normal Load Condition:

$U = 1.4\ (D + T) + 1.7L + 1.7R_o$

$U = 0.75\ [1.4\ (D + T) + 1.7L + 1.7T_o + 1.7R_o]$

b. Severe Environmental Condition:

$U = 1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.9E_o + 1.7R_o$

$U = 1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.7W + 1.7R_o$

$U = 0.75\ [1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.9E_o + 1.7T_o + 1.7R_o]$

$U = 0.75\ [1.4\ (D + T) + 1.4F + 1.7L + 1.7H + 1.7W + 1.7T_o + 1.7R_o]$

c. Extreme Environmental Conditions:

$U = (D + T) + F + L + H + T_o + R_o + W_t$

$U = (D + T) + F + L + H + T_o + R_o + E_{ss}$

d. Abnormal Load Conditions:

$U = (D + T) + F + L + H + T_a + R_a + 1.5P_a$

$U = (D + T) + F + L + H + T_a + R_a + 1.25P_a + 1.0(Y_r + Y_j + Y_m) + 1.25E_o$

$U = (D + T) + F + L + H + T_a + R_a + 1.0P_a + 1.0(Y_r + Y_j + Y_m) + 1.0E_{ss}$

Table I-2 (Continued)

where 0007200

Normal loads are those loads encountered during normal plant operation and shutdown, and include:

T = settlement loads

D = dead loads or their related internal moments and forces

L = applicable live loads or their related internal moments and forces

F = lateral and vertical pressure of liquids or their related internal moments and forces

H = lateral earth pressure or its related internal moments and forces

T_0 = thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition

R_0 = maximum pipe and equipment reactions if not included in the above loads

Severe environmental loads are those loads that could infrequently be encountered during the plant life and include:

E_0 = loads generated by the operating basis earthquake (BOE)

W = loads generated by the operating basis wind (OBW) specified for the plant

Extreme environmental loads are those loads which are credible but highly improbable, and include:

E_{ss} = loads generated by the safe shutdown earthquake (SSE)

W_t = loads generated by the design tornado specified for the plant

Abnormal loads are those loads generated by a postulated high-energy pipe break accident and include:

P_a = maximum differential pressure load generated by a postulated break

T_a = thermal loads under accident conditions generated by a postulated break and including T_0

Table I-2 (Continued)

R_a = pipe and equipment reactions under accident conditions generated by a postulated break and including R_0

U = required strength to resist design loads or their related internal moments and forces

Y_r = loads on the structure generated by the reaction on the broken high-energy pipe during a postulated break

Y_j = jet impingement load on a structure generated by a postulated break

Y_m = missile impact load on a structure generated by or during a postulated break, such as pipe whipping

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TABLE I-3

SOIL PROPERTIES USED IN

THE SEISMIC ANALYSIS

	Original Analysis	First Revised[1] Analysis	Second Revised[1] Analysis
Modulus of Elasticity (E)	22,000 ksf	6,598 ksf	2,609 ksf
Poisson's Ratio	0.42	0.45	0.40
Unit Weight (w)	135 pcf	116 pcf	120 pc/s
Shear Wave Velocity (V_s)	1,359 ft/s	796 ft/s	500 ft/s
Shear Modulus	7,746 ksf	2,275 ksf	971 ksf

[1] Note different shear wave velocity values.

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TABLE I-4

REBAR STRESS VALUES FOR THE DIESEL GENERATOR BUILDING STRUCTURAL MEMBERS (ACCORDING TO THE FSAR AND RESPONSES TO NRC REQUESTS REGARDING PLANT FILL, QUESTION 15)

Description of Members/Location	Load[1] Combination	Tensile Rebar Stress Value (ksi) Allowable = 54 ksi	Compressive Concrete[2] Stress Value (ksi) Allowable = 3.4 ksi
Exterior - West			
2'-6" thick wall horizontal reinforcement	Tornado	25.03	0.425
Exterior - South			
2'-6" thick wall horizontal reinforcement	Seismic	44.04	0.000[3]
Elevation - 664'-0"			
2'-0" floor slab N-S reinforcement	Tornado	39.15	0.068
Elevation - 680'-0"			
1'-9" floor slab E-W reinforcement	Tornado	36.06	0.834
South			
2'-0" missile shield wall south, horizontal reinforcement	Settlement	42.79	0.185
Interior			
2'-0" interior missile shield wall, vertical reinforcement	Tornado	28.06	0.000[3]
North			
2'-0" missile shield wall north, horizontal reinforcement	Tornado	13.85	0.000[3]

TABLE I-4 (continued)

Description of Members/Location	Load[1] Combination	Tensile Rebar Stress Value (ksi) Allowable = 54 ksi	Compressive Concrete Stress[2] Value (ksi) Allowable = 3.4 ksi
Exterior - North			
2'-6" thick wall horizontal reinforcement	Tornado	21.90	0.313
Exterior - East			
2'-6" thick wall horizontal reinforcement	Tornado	23.64	0.403
Interior			
1'-6" thick wall horizontal reinforcement	Tornado	16.66	0.000[3]
South			
2'-0" thick box missile shield/south, horizontal reinforcement	Tornado	8.02	0.000[3]
Footing			
2'-6" thick footing	Tornado	35.22	-

NOTES:

[1]The tornado load combination is $1.0\ (D + L) + 1.0W' + 1.0T_o$.
The settlement combination is $1.4D + 1.4T$
The seismic load combination is $1.0\ (D + L) + 1.0E' + 1.0T_o$.

[2]Concrete stresses shown are associated with maximum rebar tensile stresses shown in this table.

[3]Section is in tension.

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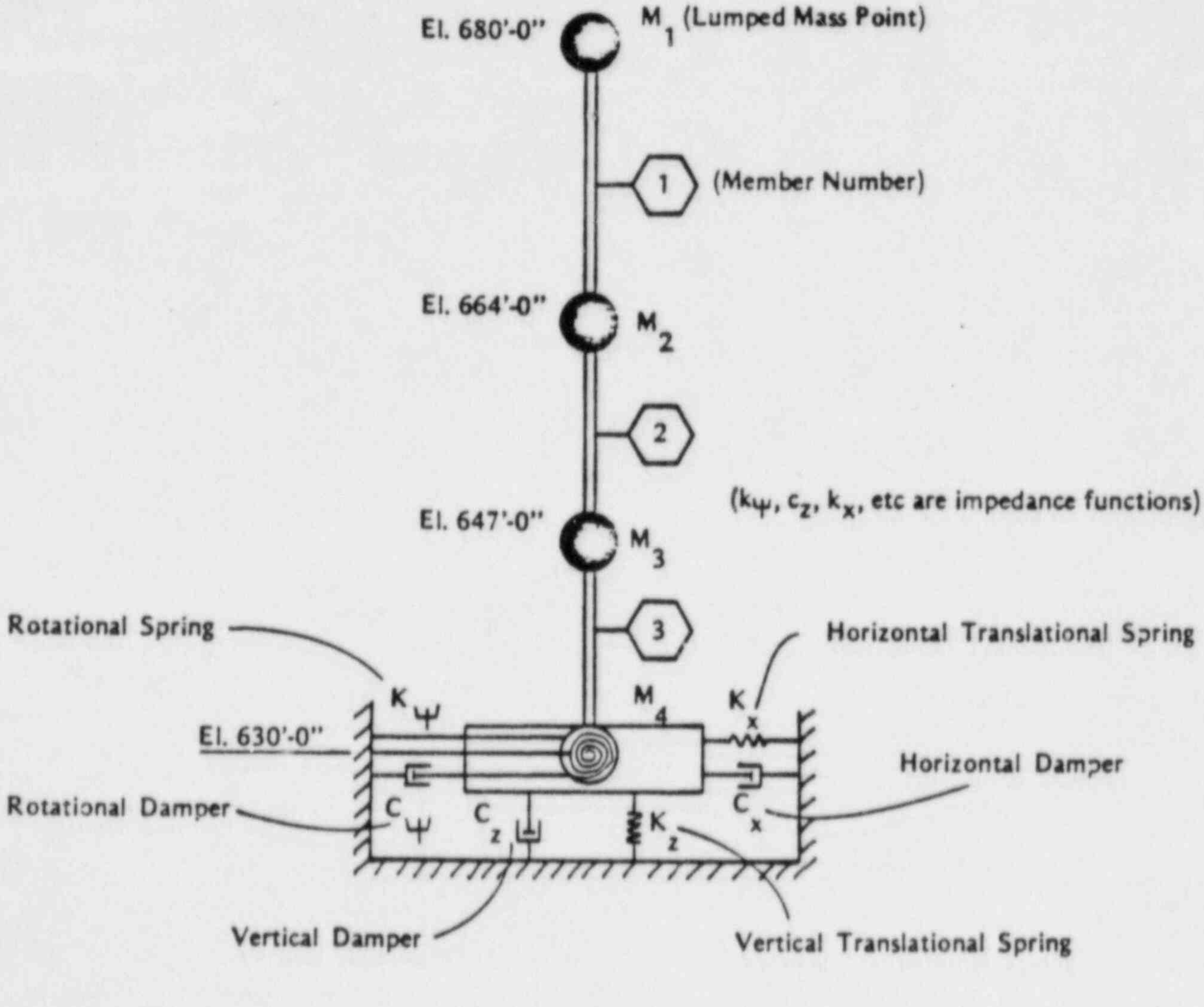

FIGURE I-1
DIESEL GENERATOR BUILDING DYNAMIC LUMPED MASS MODEL
FOR SEISMIC ANALYSIS

Midland Plant Units 1 and 2
Structural Stresses Induced by
Differential Settlement in the
Diesel Generator Building

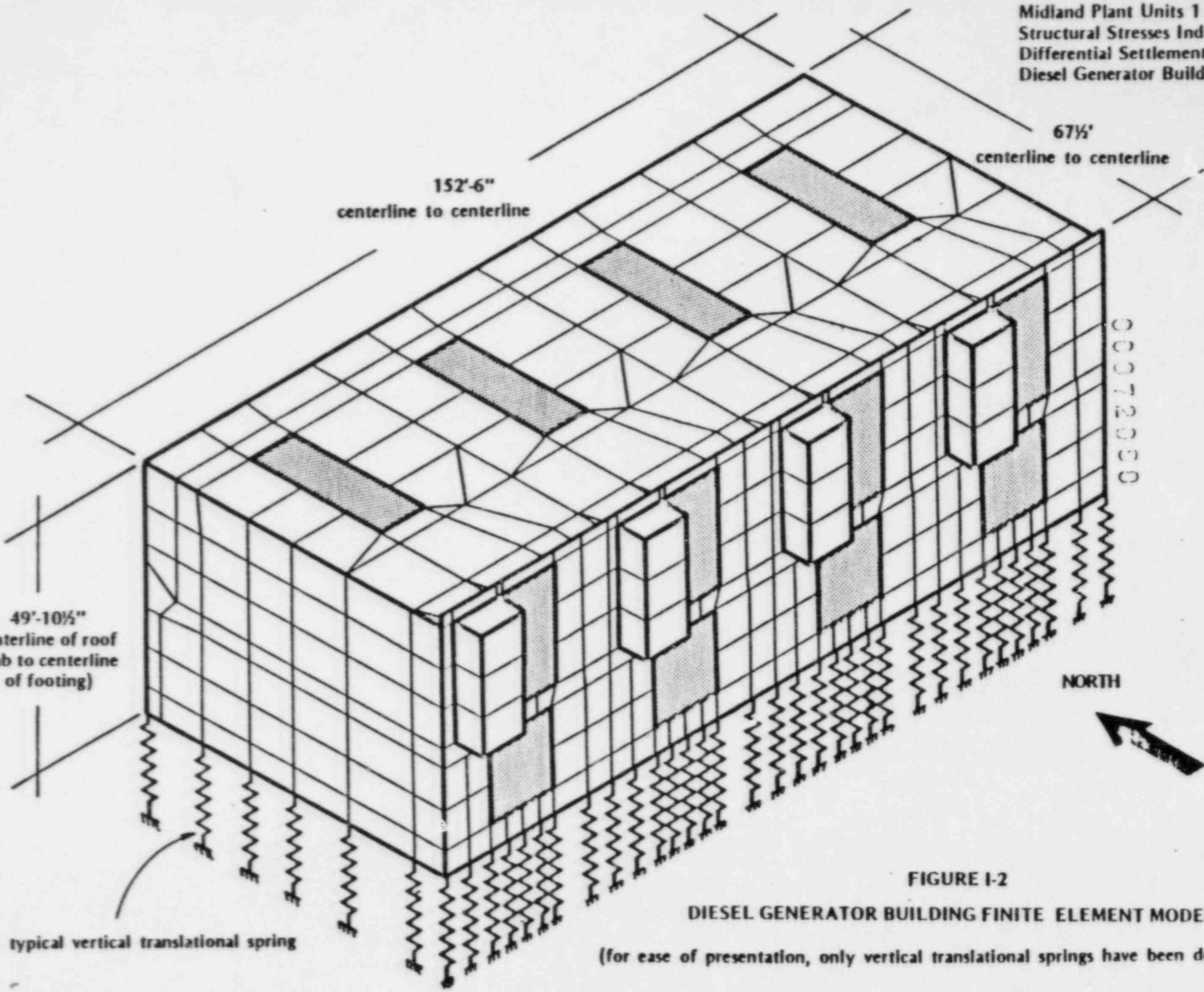

FIGURE I-2

DIESEL GENERATOR BUILDING FINITE ELEMENT MODEL

(for ease of presentation, only vertical translational springs have been depicted)

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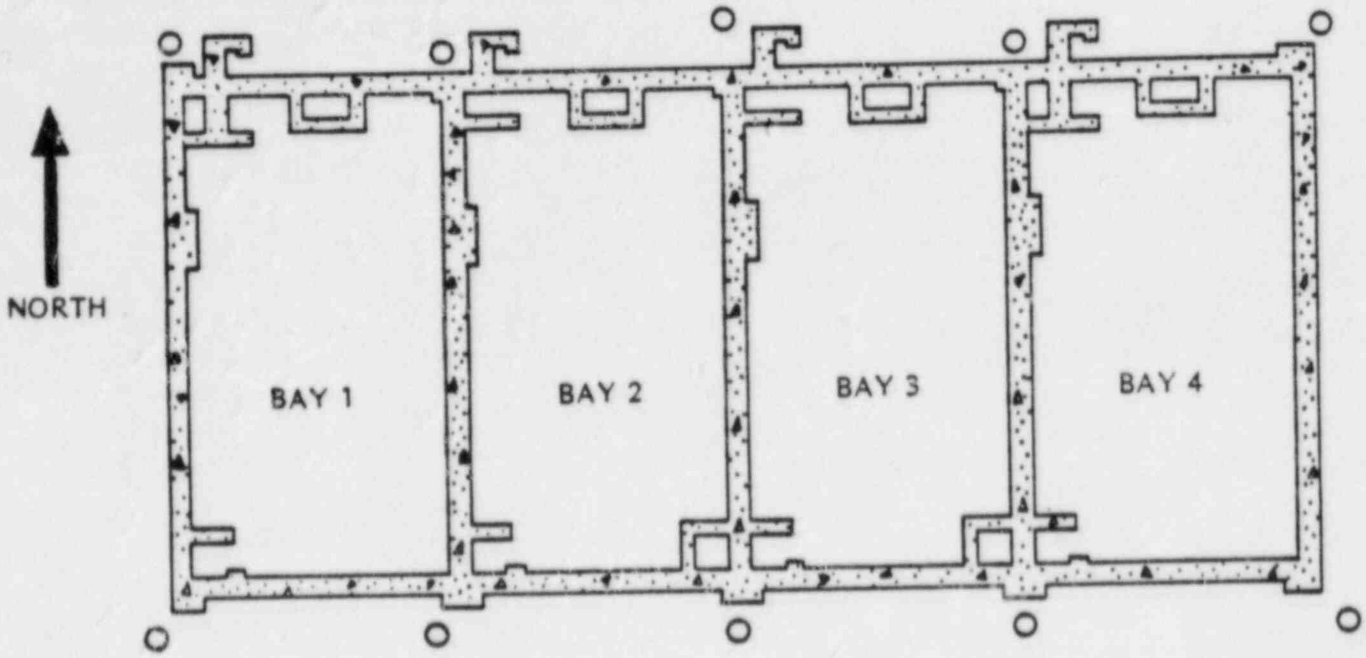
LINE A	0.77	1.09	1.54	1.98	2.41
LINE B	1.50	1.51	1.78	1.86	1.91
LINE C	1.33	1.15	1.19	1.18	1.29
TOTAL	3.60	3.75	4.51	5.02	5.61



LINE A	1.14	1.12	1.46	1.92	2.21
LINE B	3.00	2.92	3.16	3.37	3.24
LINE C	1.62	1.67	1.69	1.98	1.89
TOTAL	5.76	5.71	6.31	7.27	7.34

LEGEND

O — DIESEL GENERATOR
BUILDING SETTLEMENT MARKER
SETTLEMENT IN INCHES
FOR
PRE-SURCHARGE PERIOD (8/78-1/79 . . . . LINE A
SURCHARGE PERIOD 1/79 (1/79-8/79) . . . . LINE B
POST SURCHARGE PERIOD (9/79-12/2025) . . . . . LINE C
ASSUMING SURCHARGE REMAINS IN PLACE

FIGURE I - 3
SUMMARY OF ACTUAL AND ESTIMATED SETTLEMENTS
(for finite element analysis)

MIDLAND PLANT UNITS 1 AND 2
STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT IN THE
DIESEL GENERATOR BUILDING

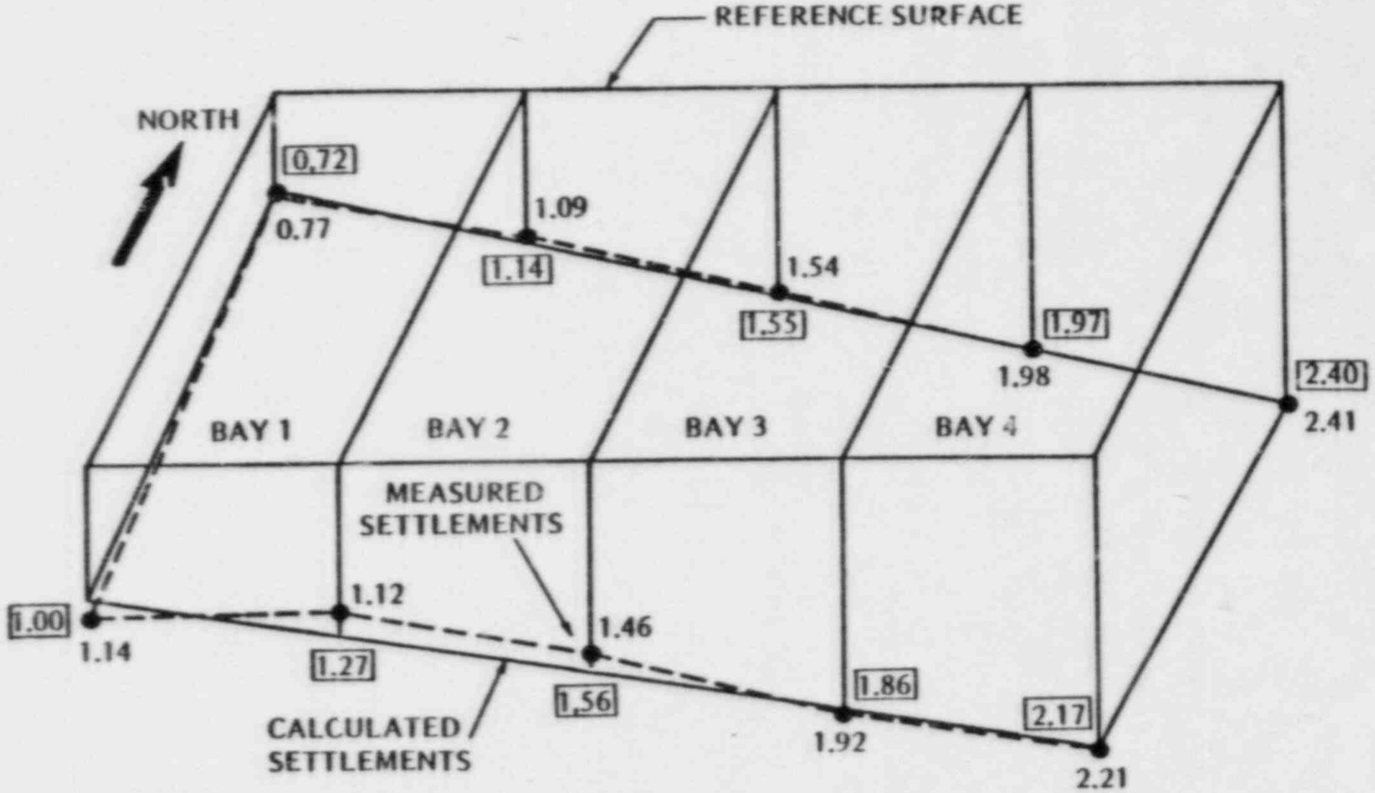


FIGURE I - 3A
COMPARISON OF MEASURED SETTLEMENT VALUES WITH SETTLEMENT VALUES
RESULTING FROM A FINITE ELEMENT ANALYSIS OF THE DIESEL GENERATOR BUILDING
PRE-SURCHARGE PERIOD
AUGUST 1978 - JANUARY 1979

MIDLAND PLANT UNITS 1 AND 2
STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT IN THE
DIESEL GENERATOR BUILDING

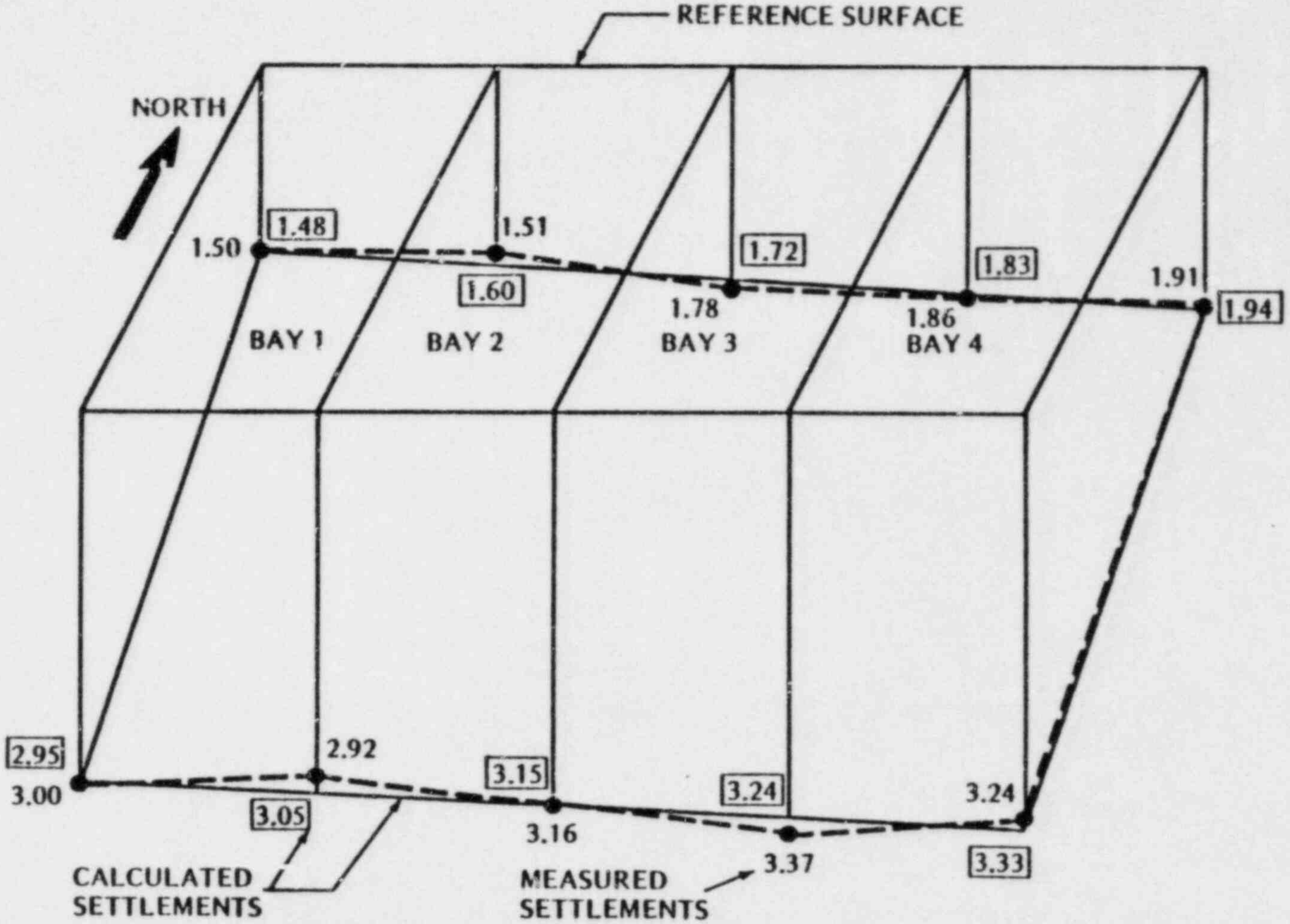
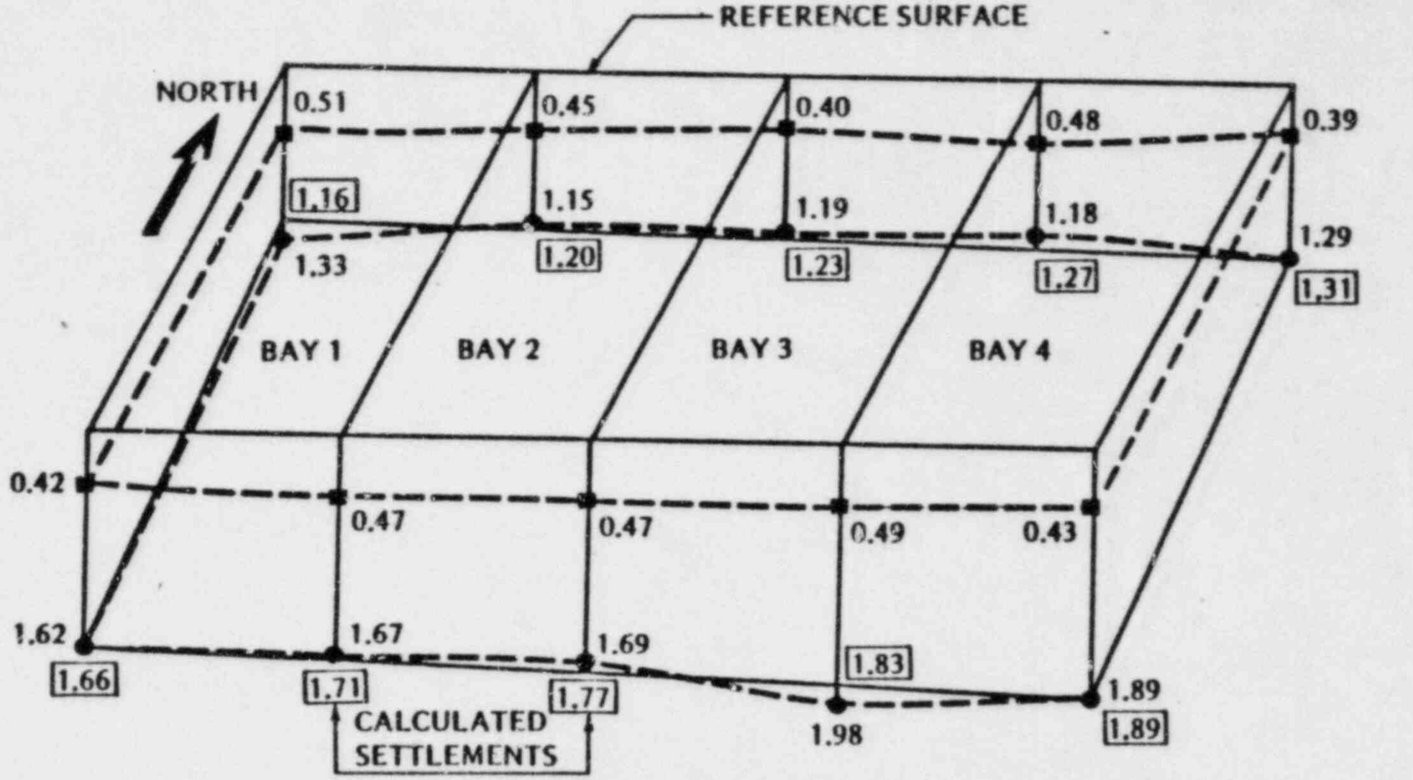


FIGURE I - 3B
COMPARISON OF MEASURED SETTLEMENT VALUES WITH SETTLEMENT VALUES
RESULTING FROM A FINITE ELEMENT ANALYSIS OF THE DIESEL GENERATOR BUILDING
<u>SURCHARGE PERIOD</u>
JANUARY 1979 - AUGUST 1979

MIDLAND PLANT UNITS 1 AND 2
STRUCTURAL STRESSES INDUCED BY
DIFFERENTIAL SETTLEMENT IN THE
DIESEL GENERATOR BUILDING



ACTUAL MEASURED SETTLEMENT FROM SEPT. 14, 1979 TO DEC. 31, 1981. THESE INCLUDE EFFECT OF DEWATERING TO APPROXIMATELY EL. 595', AND REPRESENT MOVEMENT OF THE STRUCTURE DUE TO SETTLEMENT OF THE FILL AND NATURAL SOIL BELOW.

ACTUAL MEASURED SETTLEMENTS FROM SEPT. 14, 1979 TO DEC. 31, 1981 PLUS ESTIMATED SECONDARY COMPRESSION SETTLEMENT FROM DEC. 31, 1981 TO DEC. 31, 2025 ASSUMING SURCHARGE REMAINS IN PLACE.

FIGURE I - 3C
COMPARISON OF ACTUAL MEASURED SETTLEMENTS PLUS ESTIMATED SECONDARY COMPRESSION SETTLEMENT WITH SETTLEMENT VALUES RESULTING FROM A FINITE ELEMENT ANALYSIS OF THE DIESEL GENERATOR BUILDING

<u>POST-SURCHARGE PERIOD</u>
SEPTEMBER 1979 - DECEMBER 2025

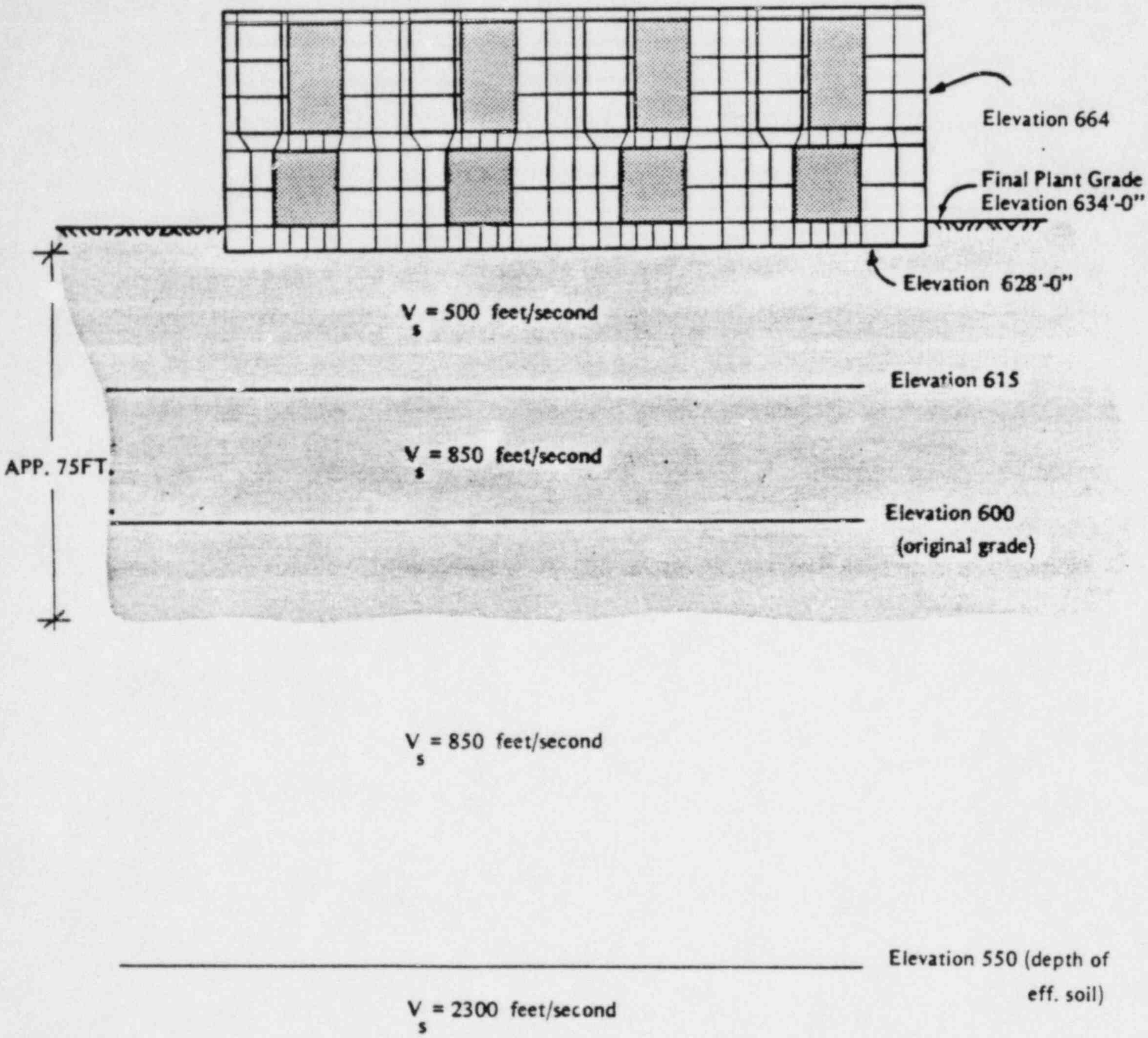


FIGURE I-4
BASIS FOR CALCULATION OF
EQUIVALENT SHEAR WAVE VELOCITY VALUES (V_s)
(Shaded region represents the area over which measured shear wave velocity values (V_s) were averaged, resulting in a V_s value of 796 ft/sec.)

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ATTACHMENT I-1

TO

TECHNICAL REPORT

STRUCTURAL STRESSES INDUCED BY

DIFFERENTIAL SETTLEMENT OF THE

DIESEL GENERATOR BUILDING

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MIDLAND PLANT UNITS 1 AND 2

DIESEL GENERATOR BUILDING

SETTLEMENT DATA ANALYSIS

CONTENTS

TABLES

FIGURES

MIDLAND PLANT UNITS 1 AND 2

DIESEL GENERATOR BUILDING

SETTLEMENT DATA ANALYSIS

1.0 INTRODUCTION

This report presents the analysis of the surveyed settlement data of the diesel generator building (DGB). The reported settlement data obtained between November 24, 1978, and November 19, 1979, were studied.

Section 2.0 presents a general discussion of the structural response due to differential settlement. (Differential settlement is defined as structural deformation which induces stresses, i.e., rigid body motion is not considered to be differential settlement.) As indicated in this section, an accurate settlement data set is required for structural analysis.

A description of the settlement data, measurement location, and methodology used to derive the original settlement data is presented in Section 3.0. The settlement data in a time-history form is presented in this section. The effectiveness of settlement in the time-history form is discussed.

Section 4.0 presents the four different analyses made on the original settlement data. The original data analyzed in this section do not indicate a consistent structural deformation. A further discussion of the accuracy of the settlement data is provided in Section 5.0.

Conclusions of this study are presented in Section 6.0.

2.0 GENERAL CONSIDERATION OF BUILDING SETTLEMENT AND STRUCTURAL RESPONSE

Figure 1 illustrates the building settlement data and differential settlement derived from the settlement data. The stresses induced on the structure from date i to date j are functions of the relative differential displacements and are defined as D_2, D_3, and D_4 in Figures 1b and 1c.

Figure 1a indicates that the elevation measurement is subjected to an assumed measurement error (E). The accuracy of the measured absolute total settlement is higher than the accuracy of the calculated relative differential settlement. Letting S_n be the absolute settlement of a particular measurement point, the error of total settlement is E/S_n. The error of differential settlement is E/D_n. It is obvious that E/D_n is much larger than E/S_n.

If E/D_n is large, the differential settlement value (D_n) should not be imposed on the structure for the structural analysis. The absolute settlement value (S_n), however, has a higher accuracy and, therefore, may be utilized. The soil stiffness derived from S_n may be used to determine the structural responses.

3.0 SETTLEMENT DATA, MEASUREMENT LOCATIONS, AND METHODOLOGY TO DERIVE THE ORIGINAL SETTLEMENT DATA

The settlement data of the DGB were obtained at different locations during different time periods. Figure 2 illustrates the locations of "scribe" and permanent "markers."

Before installation of the permanent building markers (DG markers 1, 3, and 20 through 29), settlements had been monitored by surveys on construction scribes which were elevation marks placed on the inside of the building exterior walls 3 or 4 feet above final grade. A total of 26 such construction scribes were placed between March 28, 1978, and May 12, 1978. Elevation surveys of these scribes began on July 10, 1978, and continued at weekly intervals until November 24, 1978.

The first permanent building settlement marker, DG-3, was installed May 9, 1978, marker DG-1 was installed September 9, 1978, and markers DG-20 through 29 were installed November 15, 1978. The permanent markers were installed on the outside of the building walls 1 to 4 feet above final grade and consisted of short steel rods grouted into the walls. When the surcharge was placed, these permanent markers were no longer accessible and temporary markers were set in the mezzanine floor at elevations 663.5 to 664. Temporary markers consisted of nails set in the concrete in locations generally above the corresponding permanent markers.

The settlement record included settlements monitored by the construction scribes which had occurred up to November 24, 1978. The settlement data had been calculated by assuming the settlement of a given DG marker on November 24, 1978, equal to the settlement recorded at the scribe for that particular area of the building. Beginning December 1, 1978, and up to and including March 22, 1979, only the permanent DG markers were optically surveyed. Placement of the surcharge prevented the use of the permanent markers after March 22, 1979, and temporary markers were installed to continue monitoring the settlements. The first survey of the temporary DG markers was made on March 24, 1979 (2 days after the final survey of the permanent markers), except for temporary markers DG-23 and 29 which could not be surveyed until April 9, 1979 (18 days after the final survey of the permanent markers). Temporary DG markers were surveyed during surcharge and surcharge removal until

September 14, 1979, according to the settlement record table. By this time, the permanent DG markers were accessible.

The procedure used to obtain and calculate the original settlement data was to:

a. Convert the settlements of the construction scribes to the corresponding permanent markers for the period between July 10, 1978, and November 24, 1978.

b. Set the settlements of the permanent DG markers on November 24, 1978, equal to the settlements measured by construction scribes up to that date, for the particular area of the building where a given DG marker was located.

c. Obtain the elevations of the DG markers by optical surveys and calculate the settlement of a marker on a given day by adding the settlement of the marker on November 24, 1978, to the change in elevation of the marker between November 24, 1978, and the day of the survey. This procedure continued until March 22, 1979, when the permanent DG markers were no longer accessible.

d. Install temporary DG markers above the level of the surcharge and obtain their elevations on March 24, 1979 (except for temporary markers DG-23 and 29 which were not surveyed until April 9, 1979). The settlements of the permanent markers on March 22, 1979, were added to the elevations of the corresponding temporary markers on March 24, 1979, to establish base elevation for the temporary markers. Because temporary markers DG-23 and 29 were not surveyed until several days after the final survey of the permanent markers, settlements of these markers between March 22 and April 9, 1979, were estimated from the behavior of nearby markers and these estimated settlements were added to the April 9, 1979, elevations to establish base elevations for these two markers.

e. Calculate the settlements of the temporary DG markers on a given day by subtracting the marker elevation determined by surveys from the base elevation established on March 24, 1979 (April 9, 1979, in the case of markers DG-23 and 29). Settlements of the temporary markers were calculated in this manner until September 14, 1979.

f. Obtain elevations of the permanent markers on September 14, 1979, and calculate settlements of the permanent markers on that date by subtracting the marker elevations from base elevations for the permanent markers. The base elevations for the permanent markers were established for December 2, 1978, by adding the settlements which had occurred up to that date (these settlements were estimated from scribes up to November 24, 1978) to the elevations of the markers obtained from surveys on December 2, 1978.

The settlement data were plotted in Bechtel Drawings SK-C-628 and SK-C-629 (Reference 1). Figure 3 illustrates the settlement values of the south wall for several dates. The settlement data plotted in Reference 1 for permanent markers DG-20, 23, 24, 25, 26, 27, and 29 for the period from July 10, 1978, to November 24, 1978, were derived from the settlement data of the nearby scribes by taking the numerical average values. Because the structure was only partially constructed before November 24, 1978, and the structural analysis shows that the stress level is low because of high structural flexibility, data earlier than November 24, 1978, are less important and, therefore, are not considered in this study.

The reported settlements after November 24, 1978, are listed in Table 1 and are plotted in a time-history form in Figure 4. These data were originally used in the settlement and structural evaluations.

The settlement-time relation shown in Figure 4 is a better form for studying the accuracy of the survey. The presentation method used in Reference 1 and Figure 3 (i.e., the settlement-marker location relationship) is misleading. For example, the structural shapes plotted in Figure 3 are based on the premise that the structure deformed according to the reported data without considering survey accuracy.

Figure 4 reflects survey errors. A discussion of these errors is presented in Section 5.0. Section 4.0 presents numerical analyses based on the original data.

4.0 DATA ANALYSIS

The settlement history data for the exterior wall settlement markers shown in Figure 2 are listed in Table 1. The data were analyzed and are presented in this section. The analyses include:

a. Difference of settlements between two consecutive measurement dates

b. Relative displacement along north and south walls

c. Angle variation analysis

d. Warpage analysis

These analyses are discussed as follows.

4.1 DIFFERENCE OF SETTLEMENTS BETWEEN TWO CONSECUTIVE MEASUREMENT DATES

The values of S_j - S_i for all marker points on the exterior wall of the DGB as shown in Figure 2 are listed in Table 2. The negative values indicate that either the structure moved up or a potential measurement error existed. Because the structure cannot easily move up on its own weight, it is likely that negative values indicate a measurement error.

4.2 RELATIVE DISPLACEMENTS ALONG NORTH AND SOUTH WALLS

To establish a datum point, the displacements of the exterior corners are normalized to zero. The relative displacements of the interior points D_2, D_3, and D_4 as defined in Figure 1 are calculated and are listed in Table 3.

If the measurement was 100% accurate, these relative displacements should be positive, negative, or zero for differential settlement.

a. If the relative displacement is positive or negative, the structure is undergoing differential settlement and the curvature increases or decreases.

b. If the relative displacement is zero, the structure remains at the previous curvature.

Table 3 shows that data varies irregularly. It cannot be concluded from these data that the structure developed differential settlement in the period considered.

4.3 ANGLE VARIATION ANALYSIS

Figure 5 illustrates the method used to calculate the term called "angle." The variations, with respect to time, of "angles" between markers 1-22-21 and 21-20-3 are listed in Tables 4a and 4b.

If the measurement is 100% accurate, the angle will continue increasing or decreasing through the survey period for differential settlement or will remain constant for rigid body motion.

Observations of the angle are listed below:

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Angle 1-22-21	from 11/24/78 to 03/22/79	relatively constant in the range of 179.941 degrees
	from 03/30/79 to 09/06/79	relatively constant in the range of 179.864 degrees
	from 09/14/79 to 08/28/80	relatively constant in the range of 179.934 degrees

Angle 21-20-3 has a pattern identical to that of Angle 1-22-21.

Based on the difference between successive reading dates, the change in angle between marker points on the exterior south wall is small with a random change in algebraic sign.

Therefore, these results show that the structure developed rigid body motion in the periods during which settlements were measured and the random change in algebraic sign of the change in angle is due to the accuracy of the measurements being taken.

4.4 WARPAGE ANALYSIS

A review of the settlement data for the settlement markers on the four corners of the DGB indicates the amount of warpage the structure has attained. The method of analysis for warpage is illustrated in Figure 6. Results of this analysis are listed in Table 5.

As shown in Table 5, the warpage across the structure (ΣDIFD) is very small and varies with time between positive and negative values. It can be concluded from this analysis that the survey data is not accurate enough to prove that the structure has developed differential settlement (or warpage) across the corners.

5.0 DISCUSSIONS OF THE SURVEY DATA

The numerical data analyses presented in Section 4.0 reveal that the reported settlement data do not identify a consistent pattern of differential settlement in the overall period considered. This warrants a further consideration of the accuracy of survey data.

There are two types of errors in the original data (see Figure 4). The first type is the erratic error that occurred in a particular marker elevation reading on a particular date. This type of error occurred most often in the period between December 15, 1978, and March 30, 1979. Considering the consistency of relative elevation of the north wall in the periods of December 2, 1978, to December 8, 1978, and January 26, 1979, to February 16, 1979, the inaccuracy of readings on markers DG-27 and 28 in the period from December 15, 1978, to January 19, 1979, is quite obvious. Readings from marker DG-24 on January 19, 1979, is 0.012 ft lower than the average value of January 12 and January 26, 1979. Erroneous readings are also observed on May 3, 1979, for markers DG-1, 3, 22, 24, 25, and 28. These erratic errors are clearly reflected on the settlement-time curves shown in Figure 4.

The second type of error is the systematic error that is carried over in the period from March 30, 1979, to September 6, 1979. Inspecting the relative elevation in the periods after March 30, 1979, shows that a systematic inconsistency existed between September 6, 1979, and September 14, 1979.

The systematic error during the period from March 30, 1979, to September 6, 1979, had been studied by Mr. Peter A. Lenzini of the University of Illinois (Reference 2).

Both survey data records and Mr. Lenzini's report show that on September 14, 1979, the discrepancy between temporary and permanent markers is as high as 0.017 ft at marker DG-27, 0.016 ft at marker DG-3, 0.015 ft at marker DG-28, etc. Mr. Lenzini corrected the original data and calculated the settlement relative to January 26, 1979.

As discussed in Section 3.0, the procedure to obtain and calculate the original settlement data in the period between March 24, 1979, and September 14, 1979, is to determine the base elevation for the temporary markers by adding the settlement of permanent markers to the corresponding temporary marker elevation. The base elevation is then used to calculate the settlements for the subsequent dates. This procedures indicates that the erratic error during the time to establish a base elevation can be carried through the period of temporary marker survey. Therefore, the erratic error becomes a systematic error.

Because the error may be about 0.02 ft, settlement-time curves in Figure 4 are smoothed and illustrated in Figure 7.

Based on Figure 7, the differential settlements developed in the south wall are plotted in Figure 8. It is found that as long as the comparisons are made within the period of the same measurement location, deflection is a rigid body motion (Figures 8a and 8b). When settlements of different measurement locations are compared, a higher curvature was observed (Figure 8c). This indicates the structure was developing rigid body motion and differential settlement was due to a survey error. This observation agrees with the angle variation analysis, as indicated in Section 4.3.

As indicated in Section 2.0, the absolute settlement (S_n) has a higher accuracy than the relative settlement (D_n). To utilize the available data, the soil stiffness derived from S_n may be used for structural analysis. This approach can minimize the effect due to survey error.

6.0 CONCLUSIONS

Based on this study, the following conclusions concerning the Midland DGB settlement data are made.

6.1 The survey data varies up to 0.02 (erratic error) ft.

6.2 The existing data does not indicate a consistent pattern of differential settlement. This is proven in the differential displacement analysis, angle variation analysis, and warpage analysis.

6.3 Systematic errors are contained in the survey data.

6.4 By smoothing the settlement-time curves to correct the erratic error, the data reflect that the structure was developing rigid body motion in the period during which settlement was measured at the same locations.

6.5 Differential settlement is derived only when data obtained at different elevations were compared. This is due to systematic errors. Therefore, it is concluded that the structure is under rigid body motion during the period considered in this study.

6.6 The total settlement data has a higher degree of accuracy than the relative differential settlement values. Therefore, the soil stiffness derived from the total settlement data may be used for the structural analysis.

Because of the errors in the differential settlement values, these values should not be imposed on the structure for structural analysis.

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REFERENCES

1. Bechtel Power Corporation, Midland Project Drawings SK-C-628 and SK-C-619, Diesel Generator Building Settlement Data

2. Peter A. Lenzini, Review of Data

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TABLE 1

EXTERIOR WALL SETTLEMENT DATA* (Ft)

Date	1	3	20	21	22	23	24	25	26	27	28	29
781124	0.215	0.282	0.217	0.183	0.184	0.166	0.146	0.146	0.163	0.188	0.211	0.240
781202	0.217	0.295	0.238	0.195	0.188	0.167	0.146	0.155	0.178	0.202	0.226	0.249
781208	0.216	0.299	0.231	0.194	0.188	0.170	0.152	0.158	0.181	0.206	0.232	0.255
781215	0.218	0.318	0.243	0.196	0.188	0.168	0.153	0.166	0.190	0.206	0.259	0.283
781222	0.228	0.342	0.264	0.213	0.200	0.177	0.164	0.168	0.190	0.206	0.263	0.292
781229	0.229	0.350	0.272	0.219	0.204	0.177	0.159	0.168	0.190	0.[illegible]06	0.264	0.299
790105	0.234	0.350	0.280	0.229	0.211	0.188	0.163	0.176	0.203	0.236	0.264	0.299
790112	0.231	0.349	0.280	0.231	0.214	0.181	0.160	0.180	0.209	0.236	0.267	0.301
790119	0.238	0.354	0.287	0.234	0.218	0.192	0.174	0.180	0.209	0.236	0.271	0.305
790126	0.234	0.356	0.280	0.227	0.210	0.188	0.164	0.180	0.209	0.236	0.261	0.303
790201	0.237	0.357	0.284	0.236	0.214	0.192	0.168	0.189	0.220	0.250	0.272	0.306
790216	0.259	0.378	0.314	0.265	0.245	0.210	0.179	0.205	0.239	0.266	0.288	0.329
790223	0.277	0.398	0.335	0.282	0.261	0.216	0.181	0.201	0.232	0.267	0.289	0.340
790302	0.280	0.428	0.366	0.305	0.274	0.225	0.182	0.201	0.232	0.267	0.312	0.364
790309	0.322	0.451	0.401	0.338	0.315	0.251	0.207	0.224	0.256	0.297	0.324	0.383
790315	0.344	0.466	0.407	0.346	0.324	0.260	0.213	0.231	0.263	0.303	0.328	0.397
790322	0.354	0.476	0.411	0.352	0.327	0.266	0.215	0.235	0.271	0.312	0.340	0.401
790330	0.349	0.495	0.425	0.369	0.337	0.270	0.227	0.255	0.305	0.342	0.371	0.426
790406	0.400	0.536	0.475	0.421	0.380	0.303	0.242	0.274	0.321	0.359	0.384	0.453
790413	0.439	0.570	0.514	0.452	0.413	0.332	0.260	0.281	0.331	0.369	0.397	0.477
790420	0.442	0.577	0.522	0.458	0.420	0.336	0.260	0.284	0.330	0.372	0.398	0.479
790426	0.454	0.583	0.526	0.467	0.424	0.345	0.268	0.289	0.335	0.375	0.404	0.486
790503	0.449	0.583	0.528	0.465	0.423	0.341	0.266	0.283	0.334	0.374	0.402	0.485
790511	0.464	0.594	0.536	0.470	0.435	0.352	0.277	0.294	0.337	0.379	0.409	0.492
790518	0.464	0.600	0.543	0.479	0.439	0.354	0.274	0.296	0.344	0.385	0.412	0.496
790525	0.464	0.598	0.541	0.477	0.439	0.352	0.274	0.293	0.340	0.380	0.409	0.494
790531	0.464	0.598	0.543	0.478	0.439	0.350	0.273	0.294	0.340	0.381	0.410	0.496
790605	0.467	0.601	0.542	0.480	0.443	0.353	0.275	0.295	0.344	0.380	0.412	0.496
790607	0.471	0.603	0.546	0.481	0.443	0.357	0.279	0.297	0.341	0.383	0.413	0.499
790615	0.473	0.606	0.549	0.485	0.446	0.359	0.281	0.297	0.345	0.386	0.416	0.503
790622	0.477	0.612	0.555	0.487	0.447	0.361	0.283	0.300	0.347	0.389	0.420	0.507
790629	0.477	0.612	0.556	0.489	0.447	0.360	0.280	0.299	0.350	0.389	0.418	0.504
790706	0.478	0.612	0.557	0.491	0.451	0.361	0.281	0.300	0.349	0.389	0.419	0.506
790713	0.482	0.615	0.557	0.490	0.453	0.364	0.287	0.302	0.346	0.388	0.420	0.507
790720	0.482	0.616	0.560	0.492	0.454	0.365	0.285	0.302	0.348	0.389	0.419	0.508
790727	0.485	0.618	0.561	0.493	0.454	0.366	0.286	0.302	0.351	0.392	0.422	0.510
790803	0.484	0.620	0.561	0.495	0.454	0.366	0.288	0.302	0.351	0.391	0.423	0.510
790810	0.486	0.620	0.564	0.494	0.457	0.369	0.288	0.304	0.352	0.392	0.424	0.512
790817	0.479	0.615	0.559	0.491	0.453	0.364	0.285	0.306	0.352	0.394	0.423	0.511
790824	0.471	0.608	0.552	0.487	0.444	0.357	0.277	0.295	0.347	0.387	0.416	0.504
790831	0.466	0.605	0.546	0.480	0.439	0.351	0.273	0.291	0.341	0.382	0.410	0.499
790906	0.462	0.602	0.546	0.478	0.439	0.349	0.269	0.289	0.341	0.380	0.410	0.497
790914	0.464	0.616	0.544	0.477	0.448	0.358	0.271	0.298	0.330	0.363	0.393	0.493
790921	0.464	0.615	0.544	0.477	0.450	0.360	0.271	0.297	0.333	0.363	0.392	0.492
790928	0.464	0.616	0.544	0.477	0.450	0.359	0.271	0.295	0.334	0.362	0.392	0.492
800206	0.458	0.616	0.536	0.467	0.441	0.348	0.265	0.291	0.326	0.365	0.395	0.491
800627	0.459	0.615	0.538	0.469	0.441	0.349	0.264	0.289	0.323	0.361	0.422	0.487
800822	0.456	0.612	0.536	0.468	0.440	0.348	0.265	0.288	0.323	0.362	0.423	0.490
800828	0.456	0.612	0.537	0.468	0.440	0.350	0.269	0.288	0.326	0.364	0.424	0.491

*See Figure 2 for location of settlement markers.

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TABLE 2

DIFFERENCE OF SETTLEMENT BETWEEN TWO CONSECUTIVE MEASUREMENT DATES OF MARKERS FOR EXTERIOR WALL* (Ft)

Date 1	Date 2	1	3	20	21	22	23	24	25	26	27	28	29
781124	781202	.002	.013	.021	.012	.004	.001	.000	.009	.015	.014	.015	.009
781202	781208	-.001	.004	-.007	-.001	.000	.003	.006	.003	.003	.004	.006	.006
781208	781215	.002	.019	.012	.002	.000	-.002	.001	.008	.009	.000	.027	.028
781215	781222	.010	.024	.021	.017	.012	.009	.011	.002	.000	.000	.004	.009
781222	781229	.001	.008	.008	.006	.004	.000	-.005	.000	.000	.000	.001	.007
781229	790105	.005	.000	.008	.010	.007	.011	.004	.008	.013	.030	.000	.000
790105	790112	-.003	-.001	.000	.002	.003	-.007	-.003	.004	.006	.000	.003	.002
790112	790119	.007	.005	.007	.003	.004	.011	.014	.000	.000	.000	.004	.004
790119	790126	-.004	.002	-.007	-.007	-.008	-.004	-.010	.000	.000	.000	-.010	-.002
790126	790201	.003	.001	.004	.009	.004	.004	.004	.009	.011	.014	.011	.003
790201	790216	.022	.021	.030	.029	.031	.018	.011	.016	.019	.016	.016	.023
790216	790223	.018	.020	.021	.017	.016	.006	.002	-.004	-.007	.001	.001	.011
790223	790302	.003	.030	.031	.023	.013	.009	.001	.000	.000	.000	.023	.024
790302	790309	.042	.023	.035	.033	.041	.026	.025	.023	.024	.030	.012	.019
790309	790315	.022	.015	.006	.008	.009	.009	.006	.007	.007	.006	.004	.014
790315	790322	.010	.010	.004	.006	.003	.006	.002	.004	.008	.009	.012	.004
790322	790330	-.005	.019	.014	.017	.010	.004	.012	.020	.034	.030	.031	.025
790330	790406	.051	.041	.050	.052	.043	.033	.015	.019	.016	.017	.013	.027
790406	790413	.039	.034	.039	.031	.033	.029	.018	.007	.010	.010	.013	.024
790413	790420	.003	.007	.008	.006	.007	.004	.000	.003	-.001	.003	.001	.002
790420	790426	.012	.006	.004	.009	.004	.009	.008	.005	.005	.003	.006	.007
790426	790503	-.005	.000	.002	-.002	-.001	-.004	-.002	-.006	-.001	-.001	-.002	-.001
790503	790511	.015	.011	.008	.005	.012	.011	.011	.011	.003	.005	.007	.007
790511	790518	.000	.006	.007	.009	.004	.002	-.003	.002	.007	.006	.003	.004
790518	790525	.000	-.002	-.002	-.002	.000	-.002	.000	-.003	-.004	-.005	-.003	-.002
790525	790531	.000	.000	.002	.001	.000	-.002	-.001	.001	.000	.001	.001	.002
790531	790605	.003	.003	-.001	.002	.004	.003	.002	.001	.004	-.001	.002	.000
790605	790607	.004	.002	.004	.001	.000	.004	.004	.002	-.003	.003	.001	.003
790607	790615	.002	.003	.003	.004	.003	.002	.002	.000	.004	.003	.003	.004
790615	790622	.004	.006	.006	.002	.001	.002	.002	.003	.002	.003	.004	.004
790622	790629	.000	.000	.001	.002	.000	-.001	-.003	-.001	.003	.000	-.002	-.003
790629	790706	.001	.000	.001	.002	.004	.001	.001	.001	-.001	.000	.001	.002
790706	790713	.004	.003	.000	-.001	.002	.003	.006	.002	-.003	-.001	.001	.001
790713	790720	.000	.001	.003	.002	.001	.001	-.002	.000	.002	.001	-.001	.001
790720	790727	.003	.002	.001	.001	.000	.001	.001	.000	.003	.003	.003	.002
790727	790803	-.001	.002	.000	.002	.000	.000	.002	.000	.000	-.001	.001	.000
790803	790810	.002	.000	.003	-.001	.003	.003	.000	.002	.001	.001	.001	.002
790810	790817	-.007	-.005	-.005	-.003	-.004	-.005	-.003	.002	.000	.002	-.001	-.001
790817	790824	-.008	-.007	-.007	-.004	-.009	-.007	-.008	-.011	-.005	-.007	-.007	-.007
790824	790831	-.005	-.003	-.006	-.007	-.005	-.006	-.004	-.004	-.006	-.005	-.006	-.005
790831	790906	-.004	-.003	.000	-.002	.000	-.002	-.004	-.002	.000	-.002	.000	-.002
790906	790914	.002	.014	-.002	-.001	.009	.009	.002	.009	-.011	-.017	-.017	-.004
790914	790921	.000	-.001	.000	.000	.002	.002	.000	-.001	.003	.000	-.001	-.001
790921	790928	.000	.001	.000	.000	.000	-.001	.000	-.002	.001	-.001	.000	.000
790928	800206	-.006	.000	-.008	-.010	-.009	-.011	-.006	-.004	-.008	.003	.003	-.001
800206	800627	.001	-.001	.002	.002	.000	.001	-.001	-.002	-.003	-.004	.027	-.004
800627	800822	-.003	-.003	-.002	-.001	-.001	-.001	.001	-.001	.000	.001	.001	.003
800822	800828	.000	.000	.001	.000	.000	.002	.004	.000	.003	.002	.001	.001

*See Figure 2 for location of settlement markers.

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TABLE 3a

RELATIVE DISPLACEMENT ALONG NORTH WALL
FOR SETTLEMENT MARKERS* (Ft)

From Date	To Date	24	25	26	27	28
781124	781202	.000	-.005	-.007	-.003	.000
781202	781208	.000	.003	.003	.002	.000
781208	781215	.000	-.000	.005	.020	.000
781215	781222	.000	.007	.008	.006	.000
781222	781229	.000	-.004	-.002	-.001	.000
781229	790105	.000	-.005	-.011	-.029	.000
790105	790112	.000	-.005	-.006	.002	.000
790112	790119	.000	.012	.009	.007	.000
790119	790126	.000	-.010	-.010	-.010	.000
790126	790201	.000	-.003	-.004	-.005	.000
790201	790216	.000	-.004	-.005	-.001	.000
790216	790223	.000	.006	.009	.000	.000
790223	790302	.000	.006	.012	.017	.000
790302	790309	.000	-.001	-.005	-.015	.000
790309	790315	.000	-.002	-.002	-.002	.000
790315	790322	.000	.001	-.001	.000	.000
790322	790330	.000	-.003	-.012	-.004	.000
790330	790406	.000	-.005	-.002	-.004	.000
790406	790413	.000	.010	.005	.004	.000
790413	790420	.000	-.003	.002	-.002	.000
790420	790426	.000	.002	.002	.004	.000
790426	790503	.000	.004	-.001	-.001	.000
790503	790511	.000	-.001	.006	.003	.000
790511	790518	.000	-.003	-.007	-.005	.000
790518	790525	.000	.002	.003	.003	.000
790525	790531	.000	-.002	.000	-.001	.000
790531	790605	.000	.001	-.002	.003	.000
790605	790607	.000	.001	.006	-.001	.000
790607	790615	.000	.002	-.001	-.000	.000
790615	790622	.000	-.001	.001	.001	.000
790622	790629	.000	-.002	-.006	-.002	.000
790629	790706	.000	.000	.002	.001	.000
790706	790713	.000	.003	.007	.003	.000
790713	790720	.000	-.002	-.004	-.002	.000
790720	790727	.000	.001	-.001	-.001	.000
790727	790803	.000	.002	.002	.002	.000
790803	790810	.000	-.002	-.001	-.000	.000
790810	790817	.000	-.004	-.002	-.003	.000
790817	790824	.000	.003	-.002	-.000	.000
790824	790831	.000	-.001	.001	-.001	.000
790831	790906	.000	-.001	-.002	.001	.000
790906	790914	.000	-.012	.003	.005	.000
790914	790921	.000	.001	-.004	-.001	.000
790921	790928	.000	.002	-.001	.001	.000
790928	800206	.000	.000	.006	-.002	.000
800206	800627	.000	.008	.016	.024	.000
800627	800822	.000	.002	.001	.000	.000
800822	800828	.000	.003	-.001	-.000	.000

*Settlement marker locations are shown in Figure 2.

TABLE 3b

RELATIVE DISPLACEMENT ALONG SOUTH WALL
FOR SETTLEMENT MARKERS* (Ft)

From Date	To Date	1	22	21	20	3
781124	781202	.000	.001	-.004	-.011	.000
781202	781208	.000	.000	.002	.010	.000
781208	781215	.000	.006	.009	.003	.000
781215	781222	.000	.001	.000	-.000	.000
781222	781229	.000	-.001	-.002	-.002	.000
781229	790105	.000	-.003	-.008	-.007	.000
790105	790112	.000	-.005	-.004	-.002	.000
790112	790119	.000	.002	.003	-.002	.000
790119	790126	.000	.005	.006	.007	.000
790126	790201	.000	-.001	-.007	-.002	.000
790201	790216	.000	-.009	-.008	-.009	.000
790216	790223	.000	.002	.002	-.002	.000
790223	790302	.000	-.003	-.006	-.008	.000
790302	790309	.000	-.004	-.000	-.007	.000
790309	790315	.000	.011	.010	.011	.000
790315	790322	.000	.007	.004	.006	.000
790322	790330	.000	-.009	-.010	-.001	.000
790330	790406	.000	.006	-.006	-.007	.000
790406	790413	.000	.005	.006	-.004	.000
790413	790420	.000	-.003	-.001	-.002	.000
790420	790426	.000	.007	.000	.003	.000
790426	790503	.000	-.003	-.000	-.003	.000
790503	790511	.000	.002	.008	.004	.000
790511	790518	.000	-.002	-.006	-.002	.000
790518	790525	.000	-.001	.001	.000	.000
790525	790531	.000	.000	-.001	-.002	.000
790531	790605	.000	-.001	.001	.004	.000
790605	790607	.000	.003	.002	-.001	.000
790607	790615	.000	-.001	-.002	-.000	.000
790615	790622	.000	.003	.003	-.000	.000
790622	790629	.000	.000	-.002	-.001	.000
790629	790706	.000	-.003	-.002	-.001	.000
790706	790713	.000	.002	.005	.003	.000
790713	790720	.000	-.001	-.002	-.002	.000
790720	790727	.000	.003	.001	.001	.000
790727	790803	.000	-.000	-.002	.001	.000
790803	790810	.000	-.001	.002	-.003	.000
790810	790817	.000	-.003	-.003	-.000	.000
790817	790824	.000	.001	-.003	-.000	.000
790824	790831	.000	.000	.003	.002	.000
790831	790906	.000	-.004	-.001	-.003	.000
790906	790914	.000	-.004	.009	.013	.000
790914	790921	.000	-.002	-.000	-.001	.000
790921	790928	.000	.000	.000	.001	.000
790928	800206	.000	.004	.007	.007	.000
800206	800627	.000	.001	-.002	-.003	.000
800627	800822	.000	-.002	-.002	-.001	.000
800822	800828	.000	.000	.000	-.001	.000

*Settlement marker locations are shown in Figure 2.

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TABLE 4a

ANGLE VARIATION FOR MARKERS 1-22-21 ALONG EXTERIOR SOUTH WALL

Settlement Data From Tbl 1							
Date	1	22	21	Angle*(Deg)	Date i	Date j	ΔAngle** (Deg)
781124	.215	.184	.183	179.95467377	781124	781202	-.00884819
781202	.217	.188	.195	179.94582558	781202	781208	.00325775
781208	.216	.188	.194	179.94908333	781208	781215	-.00675774
781215	.218	.188	.196	179.94232559	781215	781222	-.00448990
781222	.228	.200	.213	179.93783569	781222	781229	.00159454
781229	.229	.204	.219	179.93943024	781229	790105	-.00159454
790105	.234	.211	.229	179.93783569	790105	790112	.01124763
790112	.231	.214	.231	179.94908333	790112	790119	-.00325775
790119	.238	.218	.234	179.94582558	790119	790126	-.00838280
790126	.234	.210	.227	179.93744278	790126	790201	-.00561142
790201	.237	.214	.236	179.93183136	790201	790216	.01725197
790216	.259	.245	.265	179.94908333	790216	790223	-.00459671
790223	.277	.261	.282	179.94448662	790223	790302	.00000000
790302	.280	.274	.305	179.94448662	790302	790309	.01018715
790309	.322	.315	.338	179.95467377	790309	790315	-.01800728
790315	.344	.324	.346	179.93666649	790315	790322	-.01517487
790322	.354	.327	.352	179.92149162	790322	790330	.01215744
790330	.349	.337	.369	179.93364906	790330	790406	-.02564049
790406	.400	.380	.421	179.90800858	790406	790413	-.00667191
790413	.439	.413	.452	179.90133667	790413	790420	.00801086
790420	.442	.420	.458	179.90934753	790420	790426	-.01971245
790426	.454	.424	.467	179.88963509	790426	790503	.00709915
790503	.449	.423	.465	179.89673424	790503	790511	.00635338
790511	.464	.435	.470	179.90308762	790511	790518	-.00150299
790518	.464	.439	.479	179.[illegible]0158463	790518	790525	.00302887
790525	.464	.439	.477	179.90461349	790525	790531	-.00152588
790531	.464	.439	.478	179.90308762	790531	790605	.00492096
790605	.467	.443	.480	179.90800858	790605	790607	-.00765800
790607	.471	.443	.481	179.90035057	790607	790615	.00000000
790615	.473	.446	.485	179.90035057	790615	790622	-.00618935
790622	.477	.447	.487	179.89416122	790622	790629	-.00318718
790629	.477	.447	.489	179.89097404	790629	790706	.00767326
790706	.478	.451	.491	179.89864731	790706	790713	.00121307
790713	.482	.453	.490	179.89986038	790713	790720	.00049019
790720	.482	.454	.492	179.90035057	790720	790727	-.00618935
790727	.485	.454	.493	179.89416122	790727	790803	-.00160599
790803	.484	.454	.495	179.89255524	790803	790810	.00730515
790810	.486	.457	.494	179.89986038	790810	790817	.00322723
790817	.479	.453	.491	179.90308762	790817	790824	-.00915718
790824	.471	.444	.487	179.89393044	790824	790831	.00280380
790831	.466	.439	.480	179.89673424	790831	790906	.00969124
790906	.462	.439	.478	179.90642548	790906	790914	.02540588
790914	.464	.448	.477	179.93183136	790914	790921	.00600433
790921	.464	.450	.477	179.93783569	790921	790928	.00000000
790928	.464	.450	.477	179.93783569	790928	800206	-.00269508
800206	.458	.441	.467	179.93514061	800206	800627	-.00473022
800627	.459	.441	.469	179.93041039	800627	800822	.00323868
800822	.456	.440	.468	179.93364906	800822	800828	.00000000
800828	.456	.440	.468	179.93364906			

*See Figure 5
**ΔAngle is the angle increment between Date i and Date j.

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TABLE 4b

ANGLE VARIATION FOR MARKERS 21-20-3 ALONG EXTERIOR SOUTH WALL

Settlement Data From Tbl 1							
Date	21	20	3	Angle*(Deg)	Date i	Date j	ΔAngle** (Deg)
781124	.183	.217	.282	179.95256424	781124	781202	.02645493
781202	.195	.238	.295	179.97901917	781202	781208	-.02593613
781208	.194	.231	.299	179.95308304	781208	781215	.00495338
781215	.196	.243	.318	179.95803642	781215	781222	.00058746
781222	.213	.264	.342	179.95862389	781222	781229	.00306892
781229	.219	.272	.350	179.96169281	781229	790105	.00863457
790105	.229	.280	.350	179.97032738	790105	790112	-.00081253
790112	.231	.280	.349	179.96951485	790112	790119	.00836945
790119	.234	.287	.354	179.97788429	790119	790126	-.01285362
790126	.227	.280	.356	179.96503067	790126	790201	-.00269508
790201	.236	.284	.357	179.96233559	790201	790216	.01554871
790216	.265	.314	.378	179.97788429	790216	790223	.00647736
790223	.282	.335	.398	179.98436165	790223	790302	.00574875
790302	.305	.366	.428	179.99011040	790302	790309	.02967072
790309	.338	.401	.451	180.01978111	790309	790315	-.01978111
790315	.346	.407	.466	180.00000000	790315	790322	-.01211357
790322	.352	.411	.476	179.98788643	790322	790330	-.00886726
790330	.369	.425	.495	179.97901917	790330	790406	.01109123
790406	.421	.475	.536	179.99011040	790406	790413	.02200317
790413	.452	.514	.570	180.01211357	790413	790420	.00352478
790420	.458	.522	.577	180.01563835	790420	790426	-.01563835
790426	.467	.526	.583	180.00000000	790426	790503	.01211357
790503	.465	.528	.583	180.01211357	790503	790511	.00000000
790511	.470	.536	.594	180.01211357	790511	790518	-.00222397
790518	.479	.543	.600	180.00988960	790518	790525	.00000000
790525	.477	.541	.598	180.00988960	790525	790531	.00574875
790531	.478	.543	.598	180.01563835	790531	790605	-.00574875
790605	.480	.542	.601	180.00988960	790605	790607	.00574875
790607	.481	.546	.603	180.01563835	790607	790615	-.00574875
790615	.485	.549	.606	180.00988960	790615	790622	.00574875
790622	.487	.555	.612	180.01563835	790622	790629	.00000000
790629	.489	.556	.612	180.01563835	790629	790706	.00000000
790706	.491	.557	.612	180.01563835	790706	790713	-.00352478
790713	.490	.557	.615	180.01211357	790713	790720	.00501823
790720	.492	.560	.616	180.01713181	790720	790727	-.00149345
790727	.493	.561	.618	180.01563835	790727	790803	-.00574875
790803	.495	.561	.620	180.00988960	790803	790810	.01109123
790810	.494	.564	.620	180.02098083	790810	790817	-.00384903
790817	.491	.559	.615	180.01713181	790817	790824	-.00149345
790824	.487	.552	.608	180.01563835	790824	790831	-.00574875
790831	.480	.546	.605	180.00988960	790831	790906	.00724220
790906	.478	.546	.602	180.01713181	790906	790914	-.02702141
790914	.477	.544	.616	179.99011040	790914	790921	.00988960
790921	.477	.544	.615	180.00000000	790921	790928	-.00988960
790928	.477	.544	.616	179.99011040	790928	800206	-.00574875
800206	.467	.536	.616	179.98436165	800206	800627	.00352478
800627	.469	.538	.615	179.98788643	800627	800822	.00000000
800822	.468	.536	.612	179.98788643	800822	800828	.00222397
800828	.468	.537	.612	179.99011040			

*See Figure 5
**ΔAngle is the angle increment between Date i and Date j.

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TABLE 5

RESULT OF WARPAGE ANALYSIS (Ft)

Date i	Date j	A	B	C	D	DP	DIFD	ΣDIFD*
781124	781202	.000	.002	.013	.015	.011	.004	.004
781202	781208	.004	-.001	.004	.006	.011	-.005	-.001
781208	781215	.001	.002	.019	.027	.018	.009	.008
781215	781222	.011	.010	.024	.004	.025	-.021	-.013
781222	781229	-.005	.001	.008	.001	.002	-.001	-.014
781229	790105	.004	.005	.000	.000	-.001	.001	-.013
790105	790112	-.003	-.003	-.001	.003	-.001	.004	-.009
790112	790119	.014	.007	.005	.004	.012	-.008	-.017
790119	790126	-.010	-.004	.002	-.010	-.004	-.006	-.023
790126	790201	.004	.003	.001	.011	.002	.009	-.014
790201	790216	.011	.022	.021	.016	.010	.006	-.008
790216	790223	.002	.018	.020	.001	.004	-.003	-.011
790223	790302	.001	.003	.030	.023	.028	-.005	-.016
790302	790309	.025	.042	.023	.012	.006	.006	-.010
790309	790315	.006	.022	.015	.004	-.001	.005	-.005
790315	790322	.002	.010	.010	.012	.002	.010	.005
790322	790330	.012	-.005	.019	.031	.036	-.005	.000
790330	790406	.015	.051	.041	.013	.005	.008	.008
790406	790413	.018	.039	.034	.013	.013	.000	.008
790413	790420	.000	.003	.007	.001	.004	-.003	.005
790420	790426	.008	.012	.006	.006	.002	.004	.009
790426	790503	-.002	-.005	.000	-.002	.003	-.005	.004
790503	790511	.011	.015	.011	.007	.007	.000	.004
790511	790518	-.003	.000	.006	.003	.003	.000	.004
790518	790525	.000	.000	-.002	-.003	-.002	-.001	.003
790525	790531	-.001	.000	.000	.001	-.001	.002	.005
790531	790605	.002	.003	.003	.002	.002	.000	.005
790605	790607	.004	.004	.002	.001	.002	-.001	.004
790607	790615	.002	.002	.003	.003	.003	.000	.004
790615	790622	.002	.004	.006	.004	.004	.000	.004
790622	790629	-.003	.000	.000	-.002	-.003	.001	.005
790629	790706	.001	.001	.000	.001	.000	.001	.006
790706	790713	.006	.004	.003	.001	.005	-.004	.002
790713	790720	-.002	.000	.001	-.001	-.001	.000	.002
790720	790727	.001	.003	.002	.003	.000	.003	.005
790727	790803	.002	-.001	.002	.001	.005	-.004	.001
790803	790810	.000	.002	.000	.001	-.002	.003	.004
790810	790817	-.003	-.007	-.005	-.001	-.001	.000	.004
790817	790824	-.008	-.008	-.007	-.007	-.007	.000	.004
790824	790831	-.004	-.005	-.003	-.006	-.002	-.004	.000
790831	790906	-.004	-.004	-.003	.000	-.003	.003	.003
790906	790914	.002	.002	.014	-.017	.014	-.031	-.028
790914	790921	.000	.000	-.001	-.001	-.001	.000	-.028
790921	790928	.000	.000	.001	.000	.001	-.001	-.029
790928	800206	-.006	-.006	.000	.003	.000	.003	-.026
800206	800627	-.001	.001	-.001	.027	-.003	.030	.004
800627	800822	.001	-.003	-.003	.001	.001	.000	.004
800822	800828	.004	.000	.000	.001	.004	-.003	.001

*ΣDIFD is the accumulated value of DIFD

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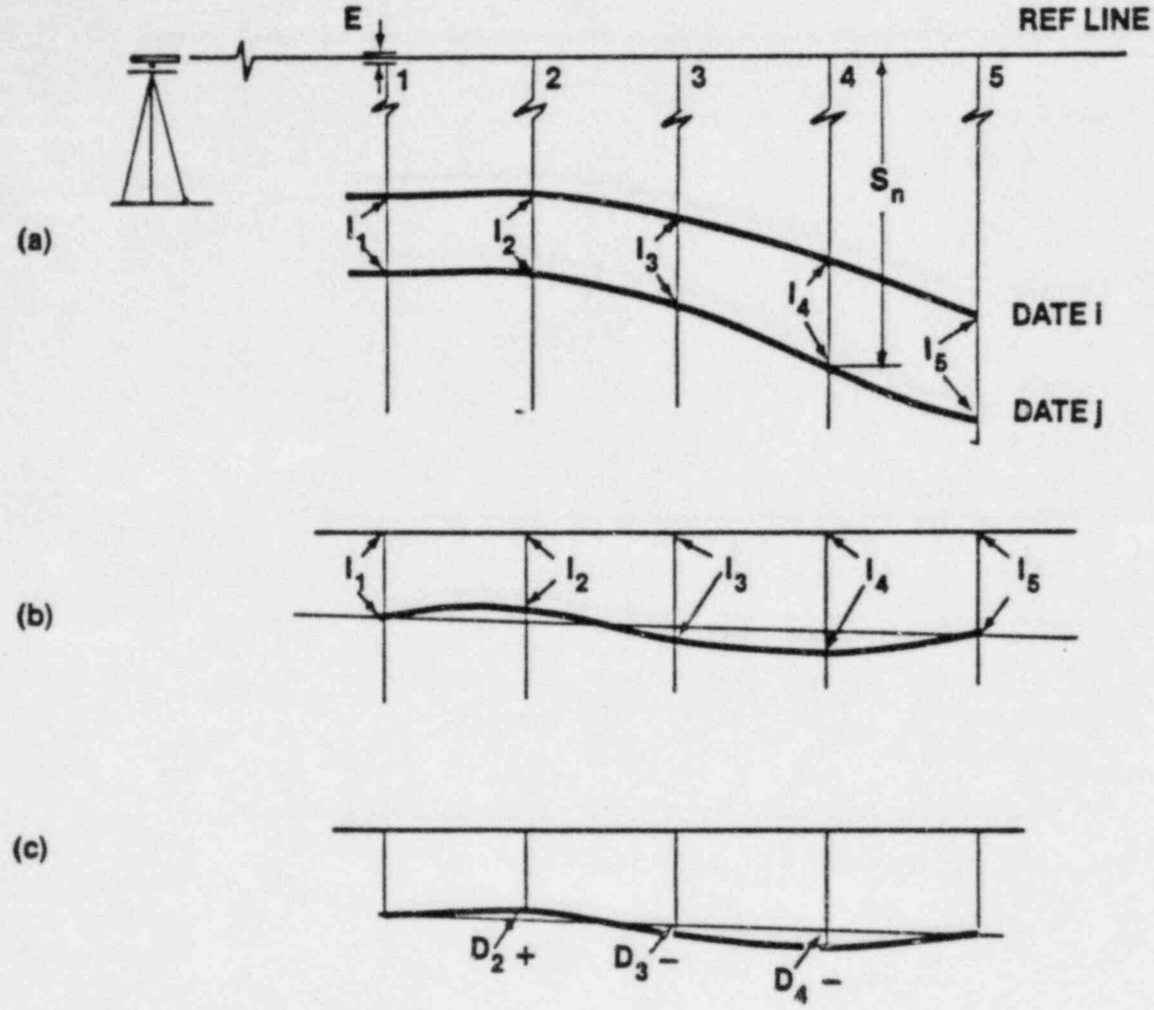


Where D_2, D_3 and D_4 are determined from the following equations:

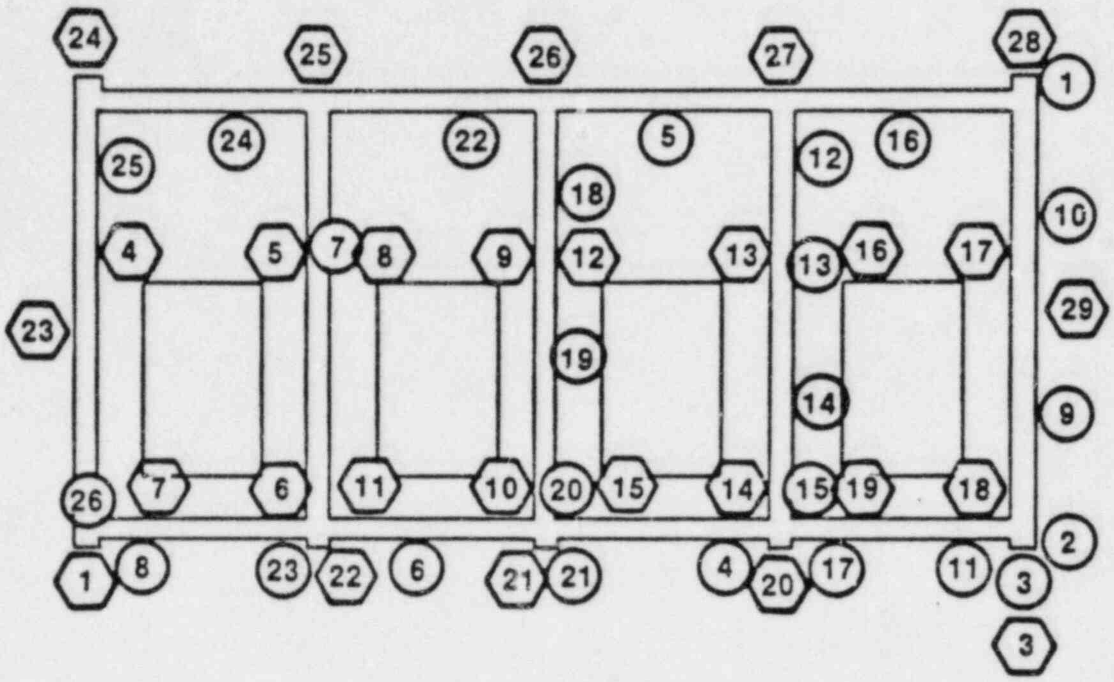
$$D_2 = [0.25(I_5 - I_1) + I_1] - I_2$$
$$D_3 = [0.50(I_5 - I_1) + I_1] - I_3$$
$$D_4 = [0.75(I_5 - I_1) + I_1] - I_4$$

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
DERIVATION OF DIFFERENTIAL SETTLEMENT FROM SETTLEMENT DATA
FIGURE 1

00072090

MARKER
EL 633'~ 638'

SCRIBE



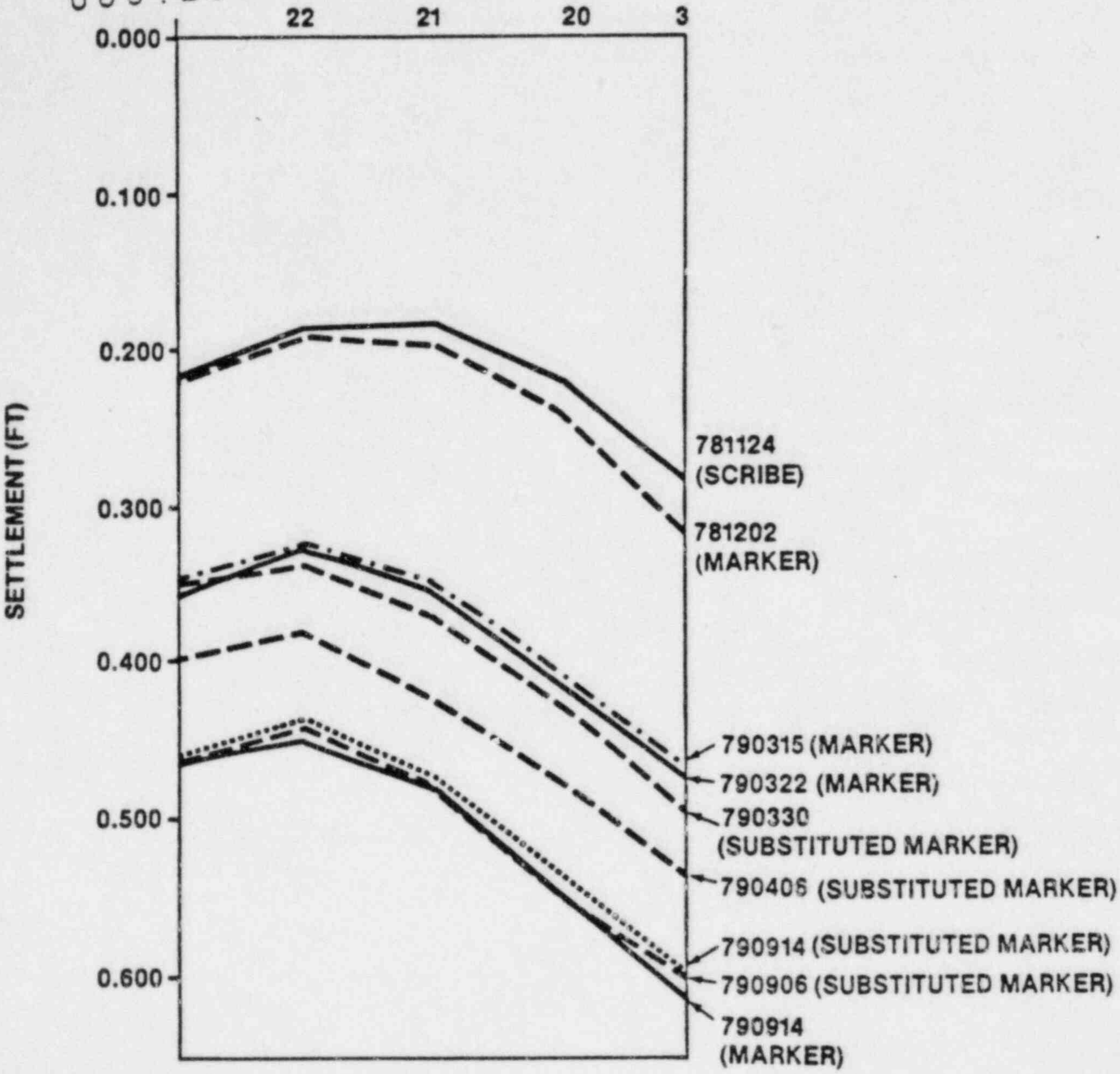
DATA DATE	DATA DERIVATION
7/10/78 - 11/24/78	Measured settlements on scribe, then converted to the equivalent settlement on marker location
12/2/78 - 3/22/79	Measured settlements directly from marker
3/30/79 - 9/14/79	Measured settlements from substituted marker inside the building on mezzanine floor el 663'
9/14/79 - Now	Measured settlements directly from marker

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

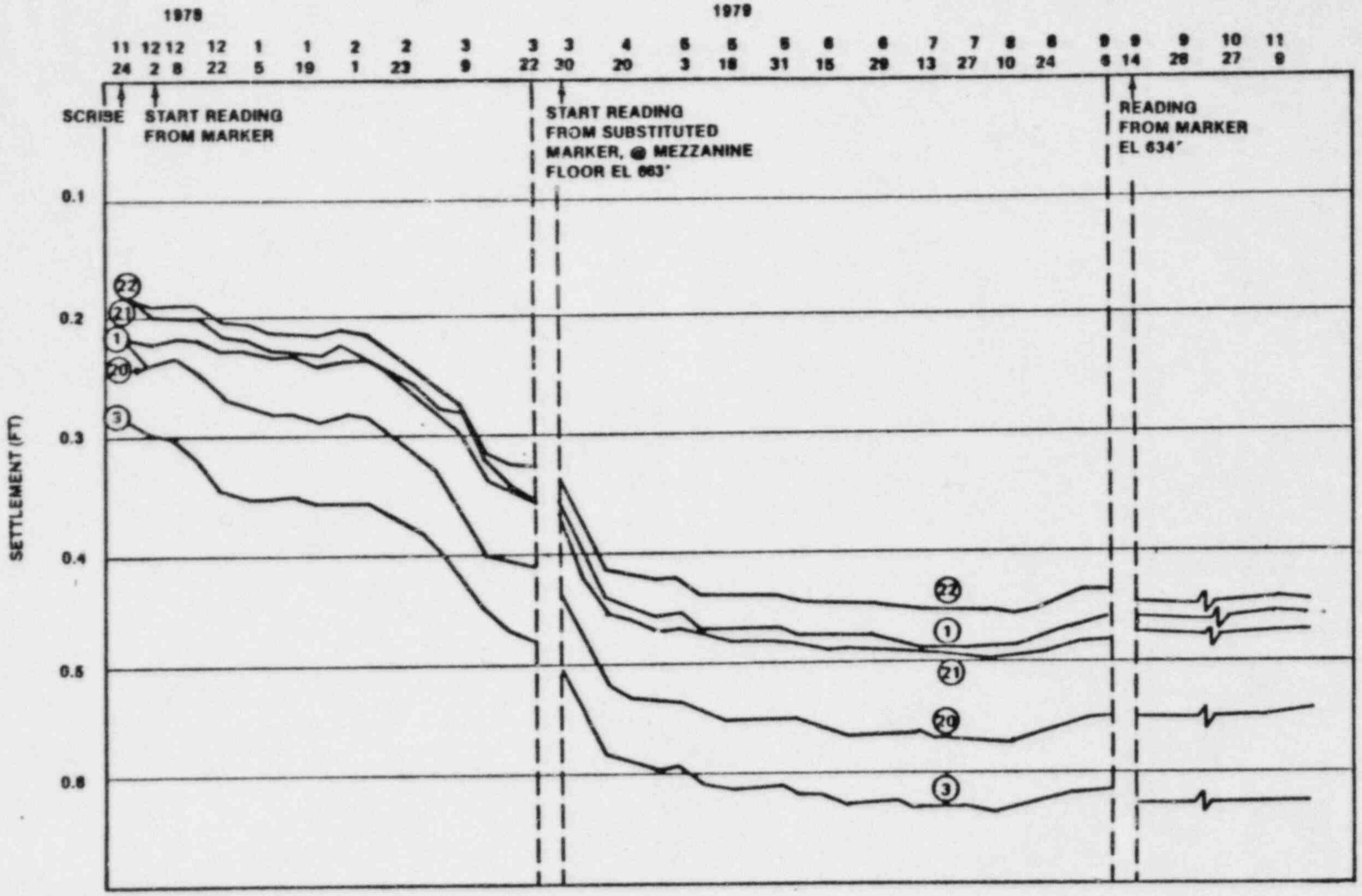
MEASUREMENT LOCATIONS

FIGURE 2

00072090



CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
SETTLEMENT ALONG SOUTH WALL
FIGURE 3



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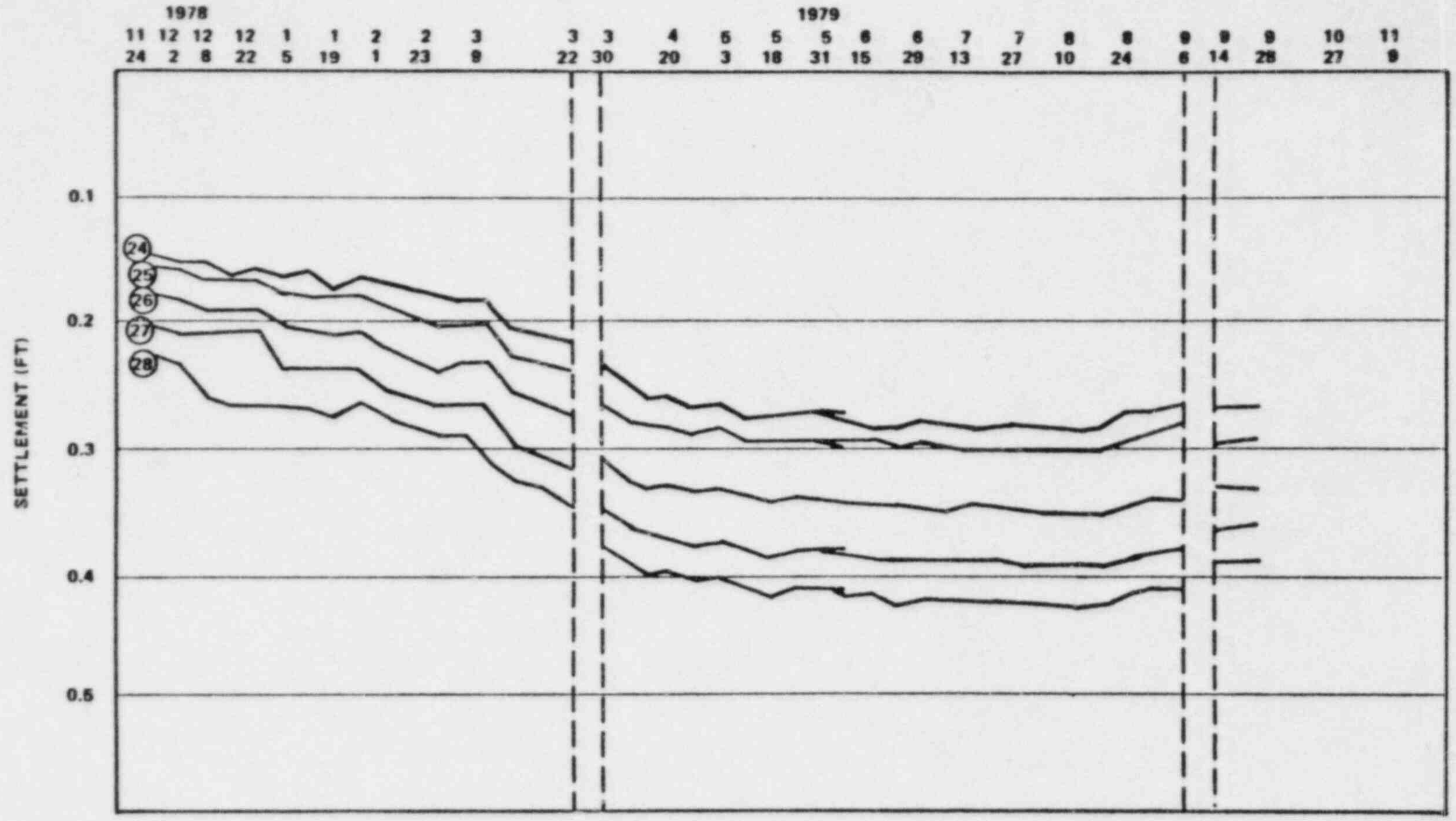
CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
SETTLEMENT-TIME CURVES FOR SOUTH WALL
FIGURE 4a



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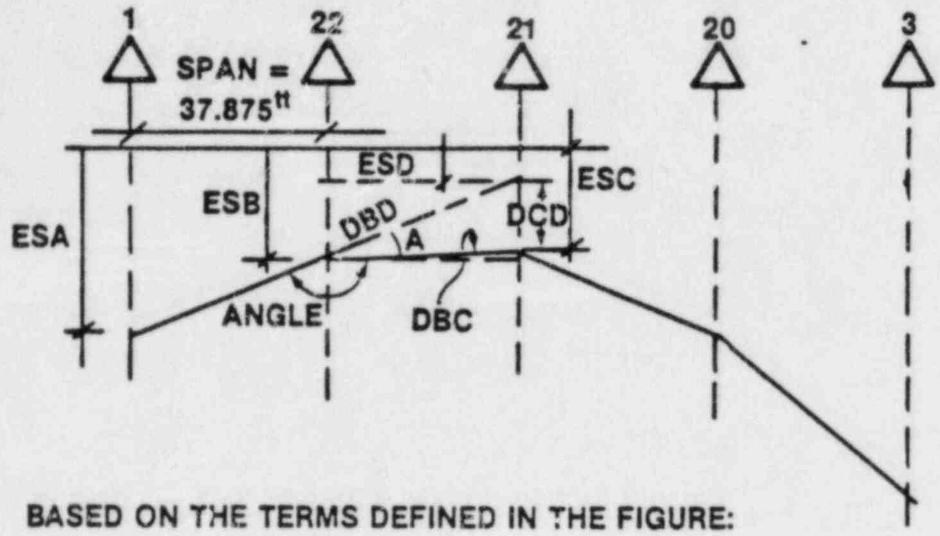
CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SETTLEMENT-TIME CURVES
FOR NORTH WALL

FIGURE 4b

00072090

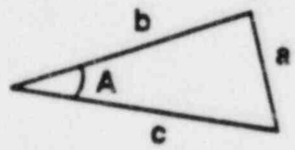


BASED ON THE TERMS DEFINED IN THE FIGURE:

$$ESD = ESB + (ESB - ESA)$$
$$DBD = [(ESB - ESD)^2 + SPAN^2]^{1/2}$$
$$DBC = [(ESB - ESC)^2 + SPAN^2]^{1/2}$$
$$DCD = |ESC - ESD|$$

FROM THE TRIANGLE RELATIONSHIP

$$a^2 = b^2 + c^2 - 2bc \cos A$$



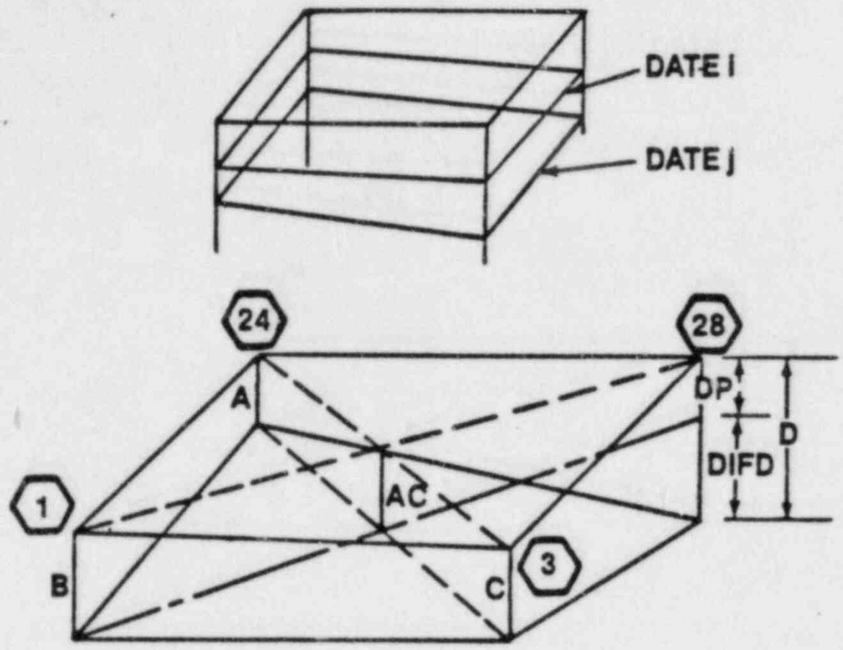
$$\therefore \cos A = (DBD^2 + DBC^2 - DCD^2)/(2DBC \times DBD)$$
$$A = \cos^{-1} (\cos A)$$

$\therefore$ IF ESC $\geq$ ESD, ANGLE = 180° − A

IF ESC $\leq$ ESD, ANGLE = 180° + A

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
ANALYSIS OF ANGLE VARIATION
FIGURE 5

00072090



$AC = (A + C)/2$

$DP = AC + (AC - B)$

$DIFD^{*} = D - DP$

IF SURVEY IS 100% ACCURATE,

Σ DIFD** SHOULD:

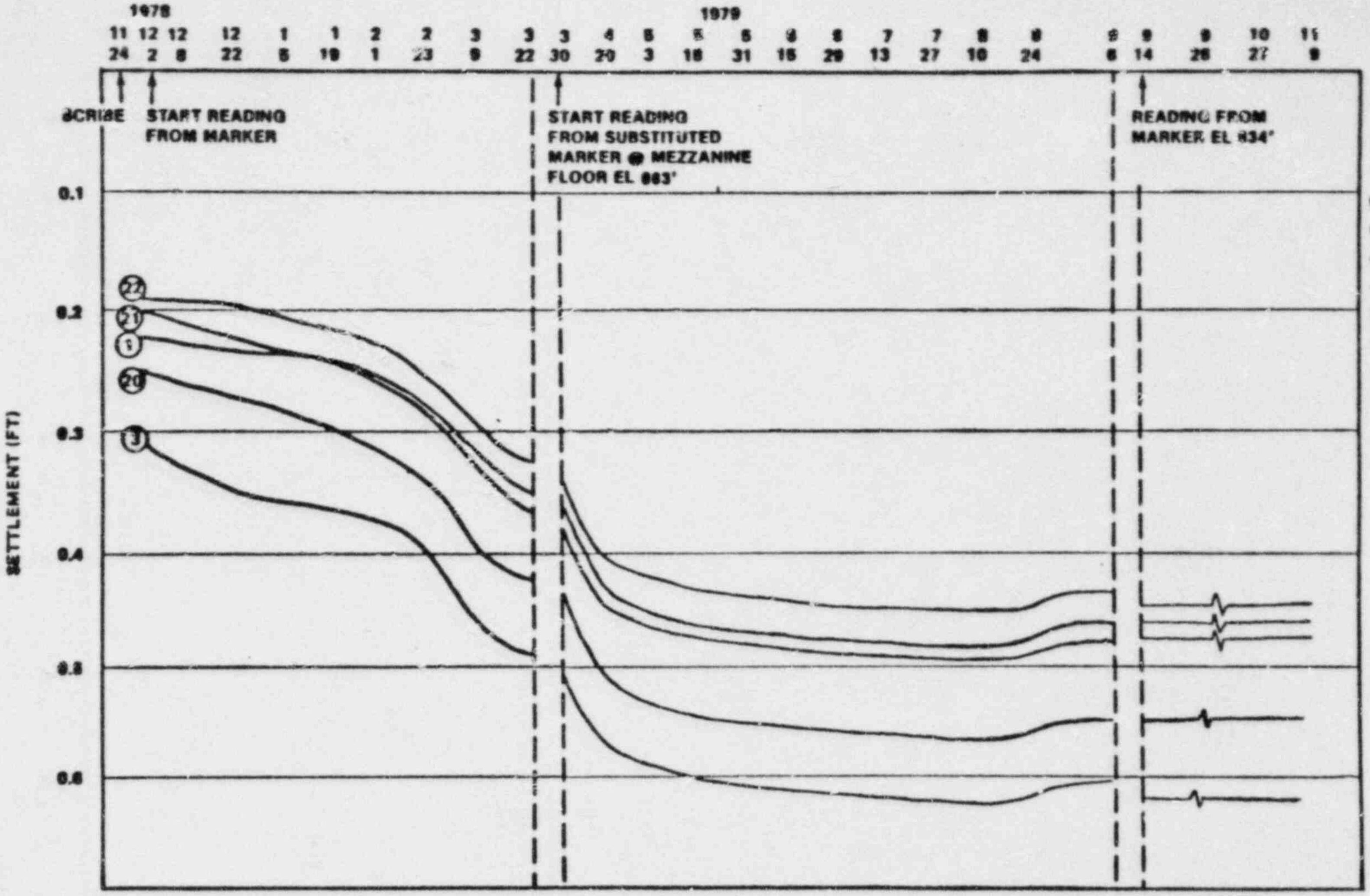
(1) KEEP INCREASING } STRUCTURE UNDERGOING TWISTING
(2) KEEP DECREASING }

(3) KEEP CONSTANT - RIGID BODY MOTION

*DIFD is the deviation of the corner from a plane which induces warping.

**Σ DIFD is the accumulated valve of DIFD.

CONSUMERS POWER COMPANY MIDLAND UNITS 1 AND 2
WARPAGE ANALYSIS
FIGURE 6



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

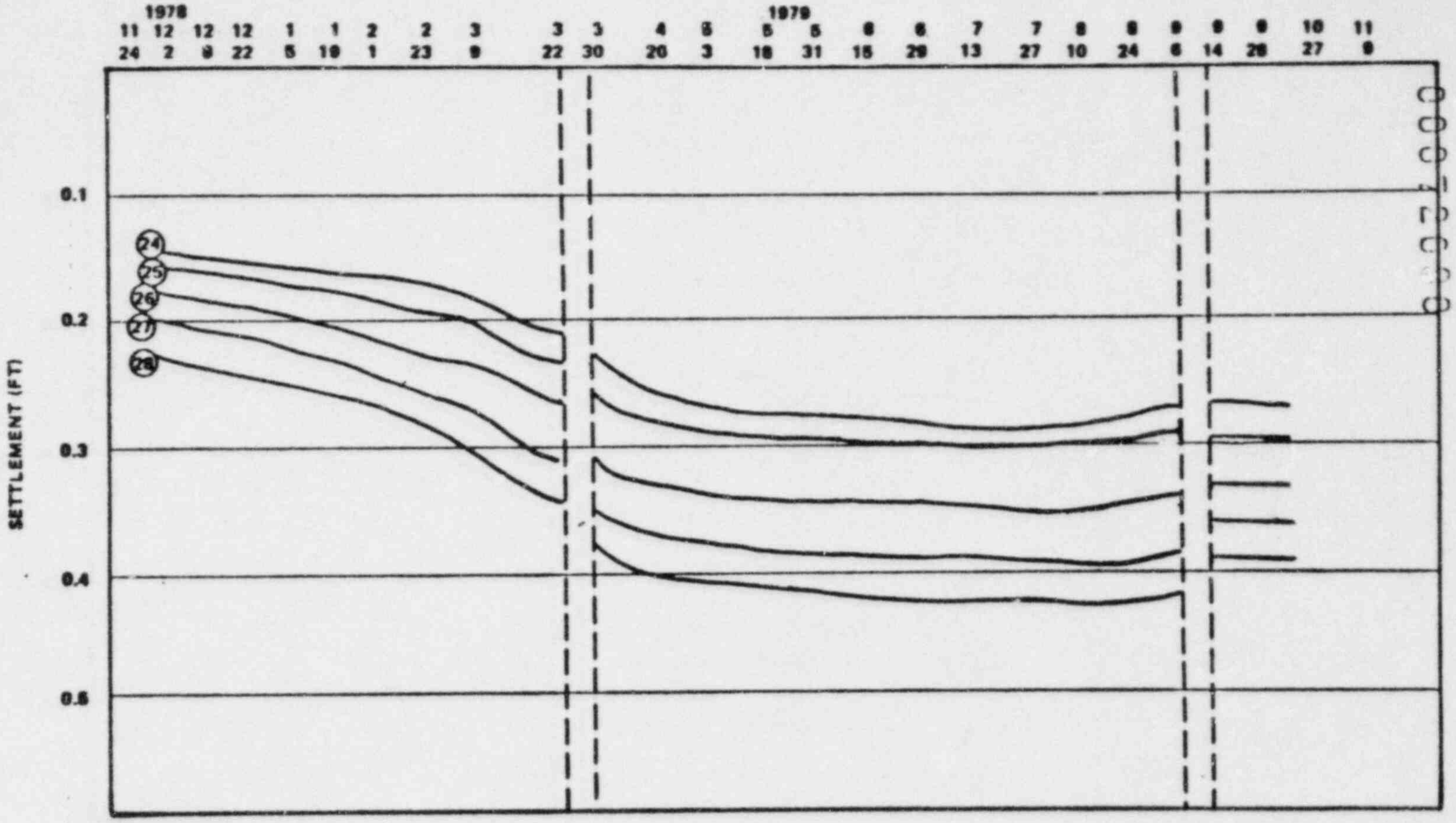
MODIFIED SETTLEMENT -TIME CURVES FOR SOUTH WALL

FIGURE 7a

00072090

MIDLAND UNITS 1 AND 2
DIESEL GENERATOR BUILDING SETTLEMENT DATA ANALYSIS 5/6/82

G-2508-07

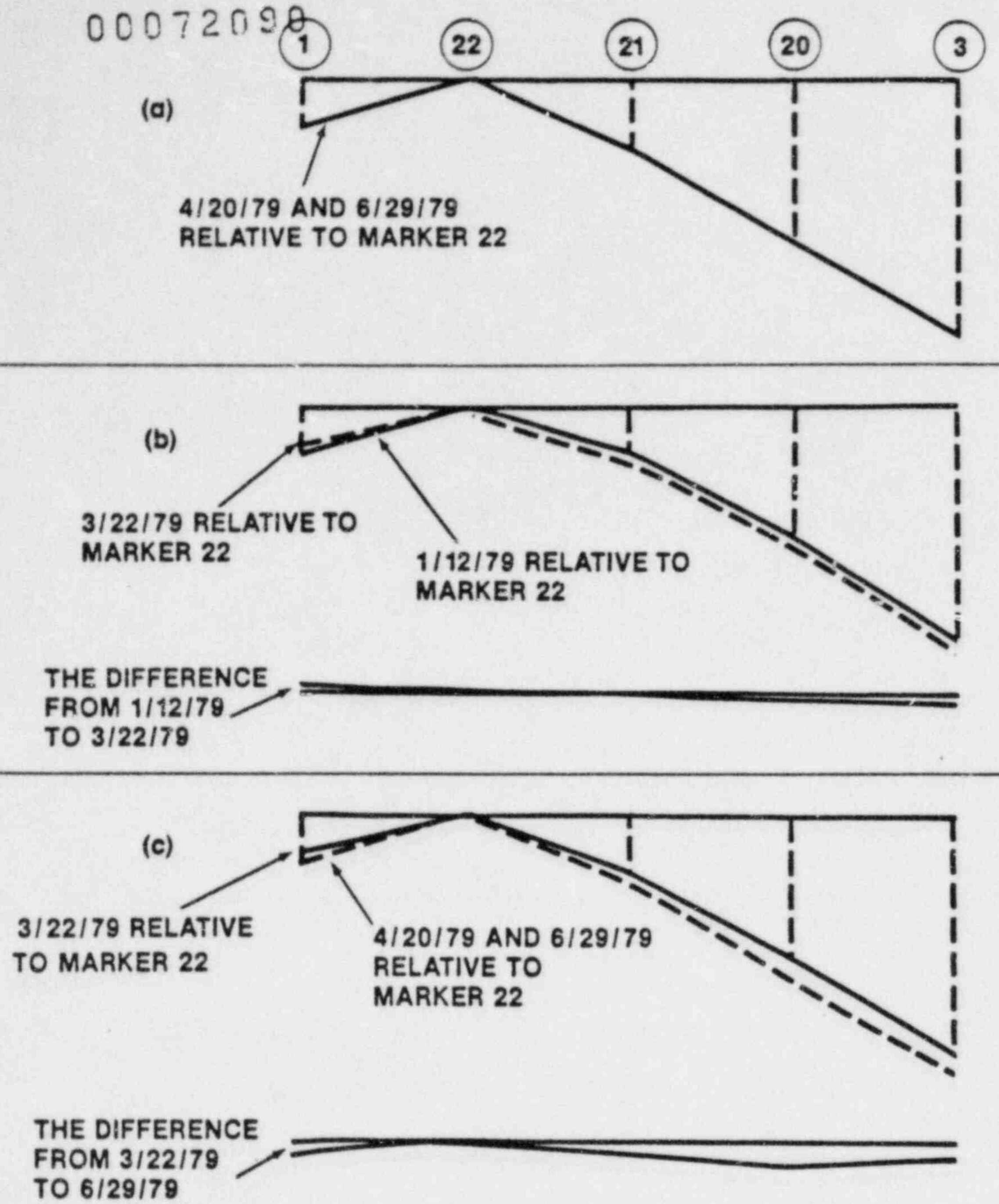


CONSUMERS POWER COMPANY
UNITS 1 AND 2

MODIFIED SETTLEMENT - TIME CURVES FOR NORTH WALL

FIGURE 7b

00072098



CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

DIFFERENTIAL SETTLEMENT
DETERMINATION

FIGURE 8

00072090

ATTACHMENT I-2

TO

TECHNICAL REPORT

STRUCTURAL STRESSES INDUCED BY

DIFFERENTIAL SETTLEMENT OF THE

DIESEL GENERATOR BUILDING

00072090 MIDLAND PLANT UNITS 1 AND 2
ANALYSIS OF DIESEL GENERATOR BUILDING
FOR
ZERO SPRING CONDITION ANALYSIS

CONTENTS

TABLES

FIGURES

MIDLAND PLANT UNITS 1 AND 2
ANALYSIS OF DIESEL GENERATOR BUILDING
FOR
ZERO SPRING CONDITION ANALYSIS

00072090

1.0 BACKGROUND

During the February 23 through 26, 1982, meeting with the NRC, it was requested that a finite-element analysis of the diesel generator building (DGB) be performed for the 40-year, dead load case, modified with zero and near-zero soil spring constants in areas to represent potential bridging. The primary purpose of this analysis would be to investigate the structure's ability to span any soft soil condition. It was subsequently decided that, in an attempt to approximate the predicted 40-year settlement profile of the south wall (as proposed by Dr. Affifi on February 23, 1982), a soil spring value of zero would be used at the junction of the south wall and east center wall. Soil spring values would then be linearly varied so that springs returned to their original 40-year values within a distance of approximately 15 feet from the zero spring (see Figure 1).

2.0 ANALYSIS PROCEDURE

A finite-element analysis of the DGB was therefore performed using 40-year soil spring values, modified along the south wall and east center interior partition wall as described above. Several analysis iterations were necessary to arrive at a settlement profile that approximated the desired "best fit" settlement profile (as obtained from a statistical analysis of Dr. Affifi's estimated 40-year settlement values). Figure 2 gives an isometric presentation of Dr. Affifi's 40-year settlement values and also the settlement values resulting from the finite-element analysis of the DGB for the zero spring condition.

Subsequent to the final analysis iteration, maximum rebar stress values were calculated for the dead load plus settlement case (i.e., "modified case"). These values were compared with the dead load plus settlement case previously calculated for the "unmodified" 40-year settlement case (see Table 1). Such a comparison shows that, except for an increase in the south wall, the footings, the box missile shield, and the south shield wall, the maximum rebar stress values remained essentially unchanged. Typically, stress level increases were limited to approximately 5 ksi except in the south shield wall, where the modeling technique causes the rebar stress value to increase 18 ksi, and in the footings where the nature of the analysis causes the rebar stress value to increase approximately 20 ksi.

As a result of this favorable comparison, it is apparent that it would be unnecessary to combine the "modified" 40-year settlement case with other load cases to form the load combinations of the FSAR and the response to Question 15 of the NRC Requests Regarding Plant Fill.

For comparative purposes, the last column of Table 1 also presents maximum rebar stress values for the governing load combinations of the FSAR and Question 15. A review of this table indicates that settlement stress is typically only a small portion of the overall maximum rebar stress values associated with the required load combinations (FSAR and Question 15).

Furthermore, because the maximum settlement stresses and maximum service load stresses generally do not occur at the same location, the component of settlement stress that actually exists in a maximum rebar stress value would typically be less than the values of Table 1.

3.0 CONCLUSIONS

As a result of the analysis performed, it can therefore be concluded that the DGB can successfully span the assumed soft soil spot introduced into the analysis without significantly increasing the rebar stress levels.

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TABLE 1

REBAR STRESS VALUES FOR THE DIESEL GENERATOR BUILDING
FOR ZERO SPRING CONDITION

Category	Tensile Rebar Stress Values (allowable = 54 ksi)		
	(D + T) for Unmodified 40-Year Case	(D + T) for Modified 40-Year Case	Max Rebar Stresses for FSAR and Q 15*
West wall	2.15	2.78	25.03
South wall	6.82	10.98	44.04
Slab at el 664'	16.94**	16.97**	39.15
Roof at el 680'-0"	5.61	6.19	36.06
South missile shield	10.79	28.82	42.79
Interior missile shield	5.51	5.30	28.06
North missile shield	2.71	2.72	13.85
East wall	2.24	2.80	23.64
North wall	3.85	4.26	21.90
Interior partition wall	3.71	4.01	16.66
Box missile shield	4.50	9.33	8.02
Footings (longitudinal bending)	14.35	37.14	20.95

* Consists of FSAR load combinations and load combinations contained in response to Question 15 of the NRC Requests Regarding Plant Fill

** A large portion of this value is attributable to the dead load component.

MIDLAND UNITS 1 AND 2
DIESEL GENERATOR BUILDING
ANALYSIS FOR ZERO SPRING
CONDITION

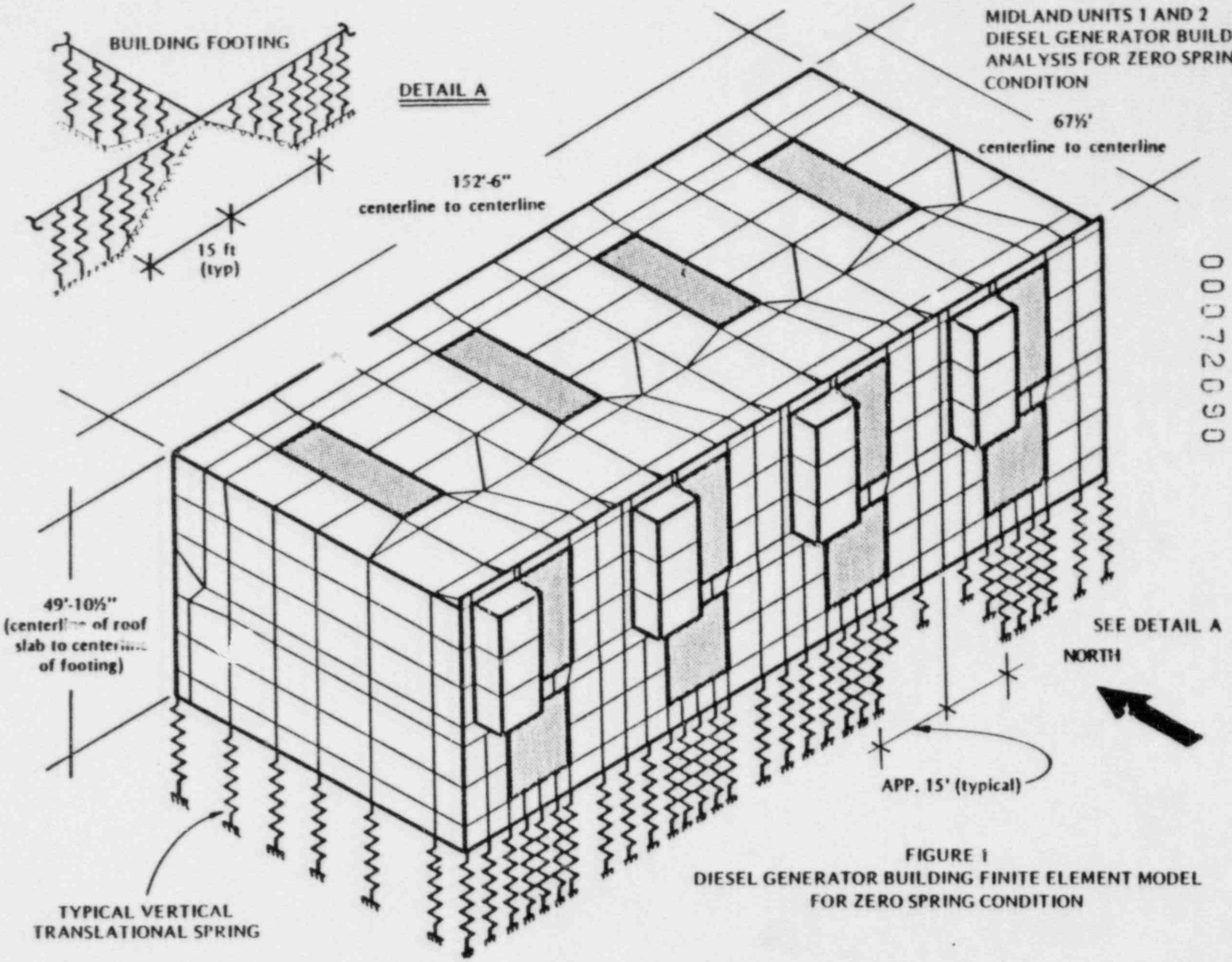

FIGURE 1
DIESEL GENERATOR BUILDING FINITE ELEMENT MODEL
FOR ZERO SPRING CONDITION

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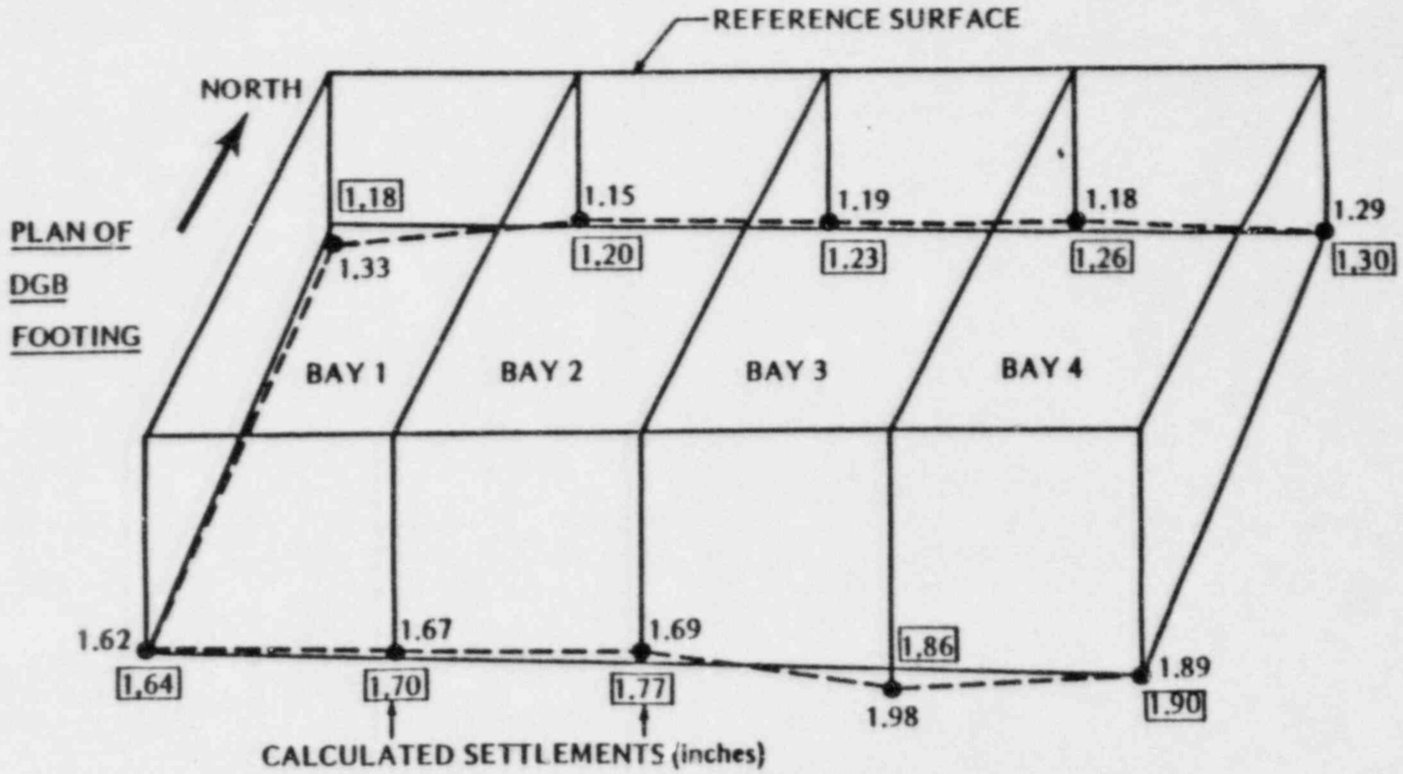
MIDLAND PLANT UNITS 1 AND 2
DIESEL GENERATORS BUILDING
ANALYSIS FOR ZERO SPRING CONDITION



ACTUAL MEASURED SETTLEMENTS FROM SEPT. 14, 1979 TO DEC. 31, 1981 PLUS ESTIMATED SECONDARY COMPRESSION SETTLEMENT FROM DEC. 31, 1981 TO DEC. 31, 2025 ASSUMING SURCHARGE REMAINS IN PLACE.

COMPARISON OF 40-YEAR ESTIMATED SETTLEMENT VALUES WITH SETTLEMENT VALUES RESULTING FROM A FINITE ELEMENT ANALYSIS OF THE ZERO SPRING CONDITION

FIGURE 2

R Landsman
R-III

MAY 2 5 1982

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Attorney, OELD
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BPCotter, ASLBP
ACRS (16)
CMiles, OPA

Docket Nos: 50-329 OM, OL
and 50-330 OM, OL

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

Dear Mr. Cook:

Subject: Completion of Soils Remedial Activities Review

In several meetings and discussions held during the months of April and May 1982, you were informed by the staff of the approach to be used for the review of the soils remedial activities at Midland Plant, Units 1 and 2. This approach is intended to make the review process more consistent with that followed by the staff for license applications and improve the efficiency of the staff review. Specifically, the previous staff practice of approving each individual construction step for each remedial measure as the review progresses will generally be discontinued by the staff. The staff intends to complete the entire review of the soils remedial activities and related matters as an integrated package and then proceed with ACRS meetings and hearing sessions in the normal fashion.

Although no activities directed to remedial actions for the soils deficiencies are expected to be approved prior to completion of the staff's integrated review, those for which staff review was substantially completed as of April 1, 1982, are, however, approved. These are discussed below.

On the basis of the staff technical review of documents listed in Enclosure 1, the staff concurs with your plan to proceed with Phase 2 underpinning activities (which involve excavation under the feedwater isolation valve pit and the turbine building) subject to the successful completion of conditions listed in Enclosure 2. Accomplishment of these conditions should be documented and Region III notified. Enclosure 3 provides a definition of Phase 2 on which the staff's approval is based, and further discusses the staff's understanding of approved quality assurance plans for this and other soils work.

We are further responding to your letter of May 10, 1982, which addresses certain soils construction work you believe had staff approval prior to the Licensing Board's Memorandum and Order of April 30, 1982. Staff comments and conclusions on Paragraphs I and II are provided in Enclosure 4.

~~8205280526~~

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With respect to your Paragraph III, you note you are continuing with certain soils remedial work with full awareness and concurrence of the staff for which explicit written approval had not been obtained. You also noted that this work has been stopped in accordance with the Order and requested that the staff verify its concurrence so that the work can be reactivated. The three work items you identified in this category are:

(1) installation of deep-seated benchmarks,
(2) installation and operation of construction dewatering wells that were not previously operating, and
(3) installation of monitoring system instruments and mounting.

Items (1) and (2) are conditionally approved as addressed by Enclosure 5 and 6, respectively. With respect to item (3), your letter notes that work on the monitoring system instruments and mounting for the auxiliary building is presently stopped because Region III concurrence has not been obtained. We are advised that Region III will provide explicit written confirmation of NRC approval following resolution of existing QA deficiencies.

Your letter of May 10, 1982, also forwarded Drawing 7220-C-45 for purposes of defining which soils at the Midland site are safety related (i.e., are considered to be under and around safety-related structures and systems). During a May 5, 1982, conference telephone call with the Licensing Board and hearing parties, Consumers proposed to use this drawing to define the bounds for the term "around" in Sections VI(1)(a), (b) and (c) of the Board's April 30, 1982, Memorandum and Order. The Board's subsequent Memorandum and Order of May 7, 1982, requested the staff to advise the Board of the results of its review of Drawing 7220-C-45. The results of our review are presented in Enclosure 7; and, on the basis of your commitments to modify the drawing, we find this drawing to be acceptable for the purpose of defining areas around safety-related structures and systems.

In addition, Enclosure 8 lists the information required by the staff to conclude its review of the soils remedial work. This list is based upon staff review of information provided by your letter of March 31, 1982, and earlier submittals. Certain of the information needs may already have been transmitted by you. You are requested to provide your response schedule within seven (7) days of receipt of this letter. Once your schedule is received, the staff will develop the review completion schedule for this effort.

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DATE							

Mr. J. W. Cook

The reporting and/or recordkeeping requirements contained in this letter affect fewer than ten respondents; therefore, OMB clearance is not required under P.L. 96-511.

Sincerely,

Original signed by
Darrell G. Eisenhut

Darrell G. Eisenhut, Director
Division of Licensing

Enclosures:
As stated

cc: See next page

OFFICE	DL:LB #4	LA:DL:LB #4	DL:LB #4	AD:L:DL	DIR:DL		
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DATE	5/24/82	5/ /82	5/24/82	5/24/82	5/ /82	5/24/82	5/24/

MIDLAND

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LISTING OF ENCLOSURES

Enclosure 1 - "Basis for Staff Concurrence for Start of Phase 2"

Enclosure 2 - "Conditions for Staff Acceptance of Phase 2"

Enclosure 3 - "Definition of Phase 2 Underpinning Activities and Quality Assurance Plans for Soils Activities"

Enclosure 4 - "Staff Comments on Continuing or Planned Soils Activities Previously Approved by the Staff"

Enclosure 5 - "Installation of Deep Seated Benchmarks"

Enclosure 6 - "Construction Dewatering Wells"

Enclosure 7 - "Staff Evaluation of Drawing 7220-C-45"

Enclosure 8 - "Additional Information Required to Complete Staff Review of Soils Remedial Work"

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ENCLOSURE 1

BASIS FOR STAFF CONCURRENCE FOR START OF PHASE 2

1. Letter to R. Vollmer from R. T. Hamilton, dated July 8, 1975, transmitting Bechtel quality assurance topical BQ-TOP-1, Revision 1A

2. Letter to H. R. Denton from J. W. Cook, dated September 30, 1981, Submitting the Auxiliary Building Dynamic Model, Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

3. Letter to H. R. Denton from J. W. Cook, dated November 16, 1981, on Response t the NRC Staff Request for Additional Information Pertaining to the Proposed Un pinning of the Auxiliary Building and Feedwater Isolation Valve Pits

4. Hearing testimony by CPC witnesses (Johnson, Burke, Gould, Corley and Sozen) o remedial underpinning work for the Midland Auxiliary Building, November 19, 19

5. Hearing testimony of D. Hood, J. Kane and H. Singh concerning the Remedial Und pinning of the Auxiliary Building Area, dated 11/20/81

6. Hearing testimony of F. Rinaldi, dated 11/20/81

7. Letter to H. R. Denton from J. W. Cook, dated 11/24/81 on Test Results, Auxili Building, Part 2, Soil Boring and Testing Program

8. Letter to H. R. Denton from J. W. Cook, dated December 3, 1981, with Addendum Technical Report On Underpinning the Auxiliary Building and Feedwater Isoloati Valve Pits

9. Letter to H. R. Denton from J. W. Cook, dated January 6, 1982, on Auxiliary Building Underpinning - Freezewall; Effects of Freezewall on Utilities and Str tures

10. Letter to H. Denton and J. Keppler from J. W. Cook, dated January 7, 1982, tra mitting general Quality Plan for underpinning activities and Quality Plans and Q-Listed activities for SWPS and Auxiliary Building Underpinning

11. Design audits of January 18-20, 1982 (Summary dated March 10, 1982); Feburary 1982; March 16-19, 1982; and meeting of February 23-26, 1982, (Summary dated March 12, 1982)

12. Letter to H. R. Denton from J. W. Cook, dated February 4, 1982, on Auxiliary Building Access Shaft - Augering Method for Soldier Pile Holes

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13. Letter to J. W. Cook from R. L. Tedesco, dated February 12, 1982, on Staff Concurrence for Activation of Freezewall

14. Letter to H. R. Denton from J. W. Cook, dated March 10, 1982, on Protection of Excavation Face - Auxiliary Building Underpinning Shaft

15. Summary of March 8, 1982 Telephone Conversation Regarding Soil Spring Stiffnesses for Auxiliary Building Underpinning and Phase II Construction, dated March 11, 1982

16. Letter to H. R. Denton from J. W. Cook, dated March 31, 1982, on Response to the NRC Staff Request for Additional Information Required for Completion of Staff review of Phases 2 and 3 of the Underpinning of the Auxiliary Building and Feedwater Isolation Valve Pits

17. Letter to J. Keppler from J. W. Cook, dated April 5, 1982, describing Quality Assurance for Remedial Foundation Work

18. Letter to H. Denton from J. W. Cook, dated April 26, 1982, transmitting quality assurance topical CPC-1-A, Revision 12

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Enclosure 2

CONDITIONS FOR STAFF ACCEPTANCE OF PHASE 2

1. Deep-seated bench marks DSB-AS1 and DSB-AS2. DSB-AS1 and DSB-AS2 shall be installed at a distance not to exceed 5-feet from the wall of the main auxiliar building which is founded at Elevation 562. Actual locations of these installe bench marks and any modifications in tolerance criteria required on Drawing C-1493(Q) due to changes from the original DSB-AS locations shall be documented

2. Monitoring instrumentation required to be installed. The following deep seated benchmarks and relative-absolute measurement devices identified on audited drawings shall be properly installed and operating for at least 7 days prior to drifting under the turbine building or Feedwater Isolation Valve Pit (FIVP):

Deep-Seated Benchmarks		Relative-Absolute Measurement Devices
DSB-1W	DSB-AS1	DMD-1W
DSB-1E	DSB-AS2	DMD-1E
DSB-2W	DSB-AN	DMD-11
DSB-2E		DMD-12
DSB-3W		DMD-13
DSB-3E		

3. Strain gauge installation. Revisions shall be made to the proposed instrumenta tion shown in drawing C-1495, "Instrumentation - Elevation 695 - 0 5/16" for Building Settlement Monitoring". On the sectional view at the wall at Column Lines 7.4 and 7.8, change the orientation of proposed lower strain gauges betwe Elevations 584 to 614 to be perpendicular to the orientation shown on Drawing C-1495, Figure 3 in the March 31, 1982 submittal. On this same sectional view, add an additional strain gauge between Elevations 646 to 659 at an inclination similar to the above recommended orientation. Also, correct the labeling of column lines H and G which is reversed on the copy of the sectional view submitted to the staff.

check

4. Pier load test procedures. The following modifications and additions shall be made to the pier load test procedures provided by the April 22, 1982 submittal from J. Cook to H. Denton, "Response to the NRC Staff Request for Additional Information Required for Completion of Staff Review of the Borated Water Storage Tank and Underpinning of the Service Water Pump Structure." (Consumers Power Company (CPCo) stated that, although the procedures were submitted for underpinning work for the service water pump structure, the procedures are applicable to the pier load test to be conducted during Phase 2 underpinning work for the auxiliary building.)

June 14

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a. The maximum required test load should be equal to 1.3 times the maximum anticipated design load. As an alternative, should there be structural difficulties in developing the required reaction load for the prior test, the staff would accept a procedure where the maximum test load for the pier load test was equal to 90 percent the maximum anticipated design load and a plate load test (ASTM D1194) was performed to a maximum test load equal to 130 percent of the maximum anticipated design load. (See Page 12 of submittal).

b. Significant modifications to the specified ASTM D1143-81 test procedures, as may be appropriate, require advanced notification and approval of the Region III Office. (See Page 12 of submittal.)

c. The rate of settlement shall not exceed 0.005 inch per hour when controlling the length of time that the 90% test load increment is to be maintained. (See Page 12 of submittal).

d. In order to provide a more positive reduction of skin friction, plywood sheeting coated with 1/8-inch thick bitumen (or equivalent) shall be installed on all test pier sides prior to performing the pier load test as a replacement for the plastic sheeting proposed by CPCo. (See Page 12 of submittal).

e. To permit correlation with the previously approved measures proposed by CPCo to demonstrate the adequate foundation capacity of the other installed piers, a minimum of two in situ density tests and five cone penetrometer tests shall be performed on the soil at the bottom of the pier selected for test loading.

5. Construction dewatering. During underpinning of the auxiliary building area, the upper phreatic surface shall be maintained a minimum of 2 feet in depth below the bottom of any underpinning excavation at any given time. The final plan for the dewatering system shall be established and implemented in advance of drifting under the turbine building or FIVP. The dewatering plan should include the locations and depths of the dewatering wells and piezometers (observation wells). Criteria for monitoring loss of soil particles due to pumping shall be the same as those previously approved by the staff for the construction dewatering of the service water pump structure (R. Tedesco letter of April 2, 1982) or for the permanent dewatering wells (R. Tedesco letters of June 18, September 2, and October 22, 1981).

June 17

6. Monitoring movement of FIVPs. Jacking of the FIVP back to its original position shall be required if the relative settlement between the reactor containment and the FIVP reaches a total settlement of 3/8-inches since the time piping connections were made.

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ENCLOSURE 3

DEFINITION OF PHASE 2 UNDERPINNING ACTIVITIES AND QUALITY ASSURANCE PLAN FOR SOILS ACTIVITIES

Phase 2 construction activities for the Midland auxiliary building underpinning are defined by Bechtel drawing C-1418-1(Q) Revision A, "Auxiliary Building - Underpinning Construction Sequence", and associated plan and logic drawing C-1418(Q), Revision A, both issued for information 3/19/82 and provided to the staff during an audit meeting on that date.

With respect to quality assurance requirements for Phase 2 work, CPCo's letter to H. Denton/J. Keppler dated January 7, 1982, transmitted a general Quality Plan for underpinning activities along with quality plans for the service water pump structure underpinning system and for the auxiliary building underpinning system and FIVPs. These plans describe the basic QA program controls to be applied to items and activities associated with the soils remedial work. We find these plans, including the QA programs described in Revision 12 of Consumer's QA Topical Report CPC-1A and Bechtel's QA Topical Report BQ-TOP-1, Rev. 1A, acceptable for the soils remedial work. However, a condition for this finding is that these quality assurance plans and programs are to apply to 1) all items and activities identified in the ASLB Memorandum and Order of April 30, 1982, and 2) all of the to-go underpinning Q-listed and non Q-listed work described in your April 5, 1982 letter to J. Keppler, except that work stated in attachment 1 of that letter. We interpret these plans and program to mean that the Midland Project Quality Assurance Department will be actively involved in reviewing contractor's, sub-contractor's, and consultant's quality assurance capabilities and assuring thorough review of procedures and verifications that hardware is built and work is performed in accordance with design, specification, and procedural requirements. Accordingly, we conclude that the above referenced Quality Plan is acceptable for implementation as described above. Since the foregoing conforms to the April 30, 1982, Board Order, any deviations must be reported to the staff.

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ENCLOSURE 4

STAFF COMMENTS ON CONTINUING OR PLANNED SOILS ACTIVITIES PREVIOUSLY APPROVED BY THE STAFF

The following comments are provided to clarify the staff's prior approvals of remedial soils activities at the Midland Plant. Each listed item in paragraphs I and II of CPCo's May 10, 1982, letter is presented and addressed.

"I.a. Phase I Work (Auxiliary Building Underpinning)"

The specific activities for Phase I work referred to in our letter of concurrence (Reference 5) for installation of the vertical access shafts were those defined by Consumer's Drawing "Underpinning Auxiliary Building Construction Sequency Logic" dated January 20, 1982.

"I.b. Access Shaft (Auxiliary Building Underpinning)"

This item is included in the staff's definition of "Phase I work" and is discussed under paragraph I.a. above.

"I.c. Freezewall Installation, Underground Utility Protection, Soil Removal Cribbing and Related Work in Support of the Freezewall Installation, Freezewall monitoring and Freezewall activation"

References 5 and 7 provided staff concurrences for freezewall installation and activation, respectively. These approvals were based upon CPCo's plan to eliminate the inducement of stresses to the conduits and piping because of heaving by excavating the soil directly beneath affected utilities within the projected area of influence of the freezewall before ground freezing begins. The approvals also recognized your commitments (1) to demonstrate to the staff's satisfaction that recompression of the foundation soils beneath the piping or ducts has been completed before backfilling the excavation, and (2) to notify Region III personnel prior to drilling near seismic Category I underground utilities and structures. The approval was further contingent upon the successful audit by the NRC Regional Office III of the implementation procedures for excavation and monitoring. 1 2

The information which provided the basis for staff review and approval was provided by CPCo's letters of November 16 and 24, 1981, and January 6, 1982, and by hearing testimony of your consultant, J. P. Gould.

Consequently, the staff agrees that prior explicit concurrence for the activities listed by paragraph I.c. of CPCo's letter, May 10, 1982 had been obtained from the staff prior to the April 30, 1982 Order, except for the ambiguous phase you included "and related work in support of...". Therefore, the staff did not approve "related work" in its letters of concurrence or other records.

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"I.d. Installation and Operation of the Permanent Site Dewatering System"

The identity and location of the 65 permanent dewatering wells approved by the staff are given in References (1), (2) and (4). Installation and monitoring aspects of the permanent site dewatering system, exculding seismic aspects, was to be performed as Q-listed activities following staff review and approval of associated quality assurance and quality control documents.

"I.e. Operation of Existing Construction Dewatering Wells"

The only construction dewatering wells approved by the staff are those identified by References (6) and (10). This item is further discussed in Enclosure 6. As noted therein, however, construction wells installed and monitored to procedures equivalent to those for permanent wells may be considered acceptable.

"I.f. FIVP Proof Load Test"

The staff has no record or recollection of concurrence for a FIVP proof load test. Therefore, this test is not approved.

"II.a. Installation and Activation of Dewatering System for the Service Water Pump Structure"

Staff approval was indicated by Reference (10), subject to certain committed changes specified therein.

"II.b. The Repair of Cracks in the Borated Water Storage Tank Ring Wall"

Staff approval was indicated by Reference (9), which noted your commitment to pressure grout at least all cracks with widths in excess of 10 mils. This activity follows the completion of the valve pit surcharge programs which were also the subjects of prior staff approvals (References (3) and (8)).

In summary, ambiguity associated with CPCo's use of the terms "Phase I work" and "related [freeze wall] work" preclude confirmation of specific prior approval of these activities. Similarly, failure by CPCo to identify the particular existing construction dewatering wells precludes us from determining whether previous staff concurrence had been indicated. No description or discussion is provided for a "FIVP proof load test" and no record of prior staff approval can be located. Consequently, continuation of these activities in conformance with the foregoing staff comments will be in accordance with the Board Memorandum and Order of April 30, 1982. Any deviations must be reported and approved by the staff.

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ENCLOSURE 4

References:

(1) R. Tedesco letter of June 18, 1981, "Staff Concurrence on Installation of Twelve Backup Dewatering Wells"

(2) R. Tedesco letter of September 2, 1981, "Staff Concurrence on Installation of Eight Backup Dewatering Wells"

(3) R. Tedesco letter of September 25, 1981, "Staff Concurrence on Surcharging of Valve Pits for Borated Water Storage Tank Foundations"

(4) R. Tedesco letter on October 22, 1981, "Staff Concurrence on Installation of Permanent Dewatering Wells and Request for Additional Information"

(5) R. Tedesco letter of November 24, 1981, "Staff Concurrence for Construction of Access Shafts and Freezewall in Preparation for Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits"

(6) R. Tedesco letter of December 28, 1981, "Staff Concurrence for Five Temporary Dewatering Wells"

(7) R. Tedesco letter of February 12, 1982, "Staff Concurrence for Activation of Freezewall"

(8) R. Tedesco letter of February 26, 1982, "Staff Concurrence on Removal of Surcharge from Borated Water Storage Tank Valve Pits"

(9) R. Tedesco letter of March 26, 1982, "Staff Concurrence for Grouting of Cracks in Concrete Foundations of Borated Water Storage Tanks"

(10) R. Tedesco letter of April 2, 1982, "Staff Concurrence for Installation and Operation of Construction Dewatering and Observation Wells for the Service Water Pump Structure"

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ENCLOSURE 5

STAFF CONCURRENCE ON INSTALLATION OF DEEP SEATED BENCHMARKS

CPCo's letter of May 10, 1982 states that installation of deep-seated benchmarks is being carried out by Woodward Clyde Consultants, which is subject to its own quality assurance program and procedures approved by Consumers and previously subject to staff inspections. We are advised that these NRC inspections have resulted in a finding that these activities are being conducted to an acceptable quality assurance program.

CPCo has also provided the staff with information on the installation of deep-seated benchmarks and relative-absolute instrumentation beginning with the design audit of January 18-19, 1982 and continuing through the submittal of March 31, 1982 (Letter from J. Cook to H. Denton, Response to the NRC Staff Request for Additional Information Required for Completion of Staff Review of Phases 2 and 3 of the Underpinning of the Auxiliary Building and Feedwater Isolation Valve Pits). The information for the auxiliary building underpinning work which has been provided includes locations, depths, elevations, instrumentation accuracy and typical installation details of the proposed instruments. This information is contained in the following documentation:

a. Technical Specification for Monitoring Instrumentation for Underpinning Construction, Specification 7220-C-198(Q), January 18, 1982 Rev. 0 (Provided at the February 3, 1982 Design Audit)

b. Drawings C-1490(Q) and C-1491(Q), Auxiliary Building, Instrumentation Location for Underpinning, January 20, 1982; Revision 1 (Provided at the February 3, 1982 Design Audit)

c. Drawing C-1493(Q), Auxiliary Building and F.I.V.P., Instrumentation System and Monitoring Matrix, May 29, 1982, Rev. A (Provided by applicant's letter of March 31, 1982)

d. Sketches of Carlson Stress Meter and Telltale Installations, Midland Plant Instruments for Pier Measurements, January 15, 1982

On the basis of the technical review by the Staff and its consultants of the information in the above documents, including the quality assurance program, the staff concurs with Consumer's proceeding with the installation of the deep-seated benchmarks and relative-absolute instrumentation for monitoring the auxiliary building underpinning work.

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ENCLOSURE 6

CONSTRUCTION DEWATERING WELLS

In the past Consumer's position with respect to temporary or construction dewatering has been that this work was not permanent, it was being conducted to enable performance of construction activities and, therefore, the work did not require staff approval. Consumers did not provide the details of the construction dewatering design and installation and did not seek staff approval for these activities.

More recently the staff has concluded that certain aspects of construction dewatering activities related to underpinning the service water pump structure (SWPS) and auxiliary building could potentially affect the foundation stability of these nearly completed structures. The staff has actively reviewed the temporary construction dewatering plan for the SWPS and has reached agreement with CPCo on an acceptable plan (April 2, 1982 letter with enclosures from R. Tedesco to J. Cook, Staff Concurrence for Installation and Operation of Construction Dewatering and Observation Wells for the Service Water Pump Structure). The staff has not presently obtained or evaluated the final plan for construction dewatering during auxiliary building underpinning but has specified conditions for Phase 2 concurrence (Enclosure 3).

It is the staff's position, with respect to the remaining construction dewatering wells that are already installed and operating, that these wells be monitored for the loss of soil particles due to pumping similar to the requirements agreed upon and recorded in Enclosure 3 to the April 2, 1982 letter.

The specifications for a construction dewatering well are dependent upon the specific application. Consequently, approval for typical field practices, on other than a case-by-case basis is not meaningful. Therefore, for the future, the design and installation details of construction dewatering wells that have not yet been operated or installed should be addressed on a case-by-case basis following appropriate notification of the staff by the CPCo. This procedure will permit an assessment of the safety significance of the proposed well. However, any construction well for which the procedures for installing and monitoring the loss of soil particles are equivalent to those previously approved for permanent dewatering wells (which was in accord with a staff approved quality assurance plan) may be considered acceptable, provided also that the upper phreatic surface is maintained two feet below the bottom of any excavation or as otherwise approved in advance by Region III.

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ENCLOSURE 7

STAFF EVALUATION OF DRAWING 7220-C-45

Staff requirements for this drawing were provided by the staff on May 7, 1982, to Messrs J. Mooney, J. Schaub and others of CPCo. These were:

(1) The seismic Category I retaining wall to the east of the service water pump structure is shown to be located in the non-Q zone. CPCo should revise the drawing to provide for Q-listed control in the vicinity of this wall.

(2) The drawing should be revised to provide for Q control of soils activities for the emergency cooling water reservoir (ECWR), the concrete service water discharge lines, and the perimeter and baffle dikes adjacent to the ECWR.

(3) CPCo should implement Q controls for certain aspects of work outside the Q zone of Drawing 7220-C-45 which could impact safety related structures and systems. Examples include potential removal of fines by dewatering wells, improper location of borings near the Q boundary, and soil excavations at the boundary involving both Q and non-Q areas.

(4) CPCo should re-confirm that no seismic Category I underground utilities extend beyond the Q area bounds of the drawing.

CPCo's letter of May 10, 1982 notes the intent to revise the drawing to address the ECWR components and other appropriate areas. CPCo has also identified during the May 7 telephone discussion additional measures being implemented to assure proper location for drillings.

On the basis of CPCo's commitment to extend the controls of soils activities to incorporate these staff requirements, the staff approves the use of Drawing 7220-C-45 for defining the areas around safety-related structures and systems within which the restrictions and requirements of the April 30, 1982, Memorandum and Order shall apply.

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ENCLOSURE 8

ADDITION INFORMATION REQUIRED TO COMPLETE STAFF REVIEW OF
SOILS REMEDIAL WORK

1. Provide the following information regarding the Auxiliary Building and Feedwater Isolation Valve Pits:

   1.1 redesign of stiffened bulkhead against earth pressures during drift excavation to install needle beam assembly
   1.2 revise report on crack evaluation to include consideration of the effects of multiple cracks
   1.3 analysis of the construction condition using a subgrade modulus of 70 KCF and provide results
   1.4 allowable differential settlements for Phase 3 (based on 1.3 above)
   1.5 horizontal movement acceptance criteria for Phase 3 for instruments at top of EPAs and control tower
   1.6 as-built report with confirmatory detail on underpinning in FSAR upon completion of construction
   1.7 acceptance criteria for strain monitors for Phase 3
   1.8 acceptability of 1.5 FSAR SSE versus SSRS as bounding design
   1.9 method to be followed for transfer of jacking load into permanent wall
   1.10 complete design analyses of permanent underpinning wall
   1.11 updated construction sequence for Phases 3 and 4
   1.12 settlement monitoring program to be required during plant operation with action levels and remedial measures identified (Tech. Spec.). Include RBA, EPA and Control Tower
   1.13 plans and details for permanently backfilling underpinning excavations including compaction specifications for granular fill under FIVP
   1.14 procedure to be required for detecting extent of planar openings uncovered in drift excavations and controls to minimize their effects.

2. Provide the following information regarding the Service Water Pump Structure:

   2.1 acceptability of 1.5 FSAR SSE versus SSRS as bounding design
   2.2 sliding calculation using site-specific response spectra (SSRS) seismic loads and provide results with basis for assumed soil input parameters
   2.3 stress condition for existing parts of structure:
      (a) Maximum stresses .
      (b) Critical combinations
      (c) Identify true critical elements based on actual rebar

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2.4 calculation for determining lateral earth pressures under dynamic loading
2.5 settlement monitoring program to be required during plant operation with action levels and remedial measures identified (Tech. Spec.)
2.6 as-built report with confirmatory data on underpinning in FSAR upon completion of construction
2.7 report on crack evaluation to include consideration of the effects of multiple cracks.

3. Provide the following information regarding the Borated Water Storage Tanks:

3.1 adequacy of governing load combination used in design
3.2 acceptability of 1.5 FSAR SSE versus SSRS as bounding design
3.3 settlement monitoring program to be required during plant operation with action levels and remedial measures identified (Tech. Spec.)
3.4 as-built report with confirmatory data in FSAR on completed construction

4. Provide the following information regarding underground pipes:

4.1 basis for modeling of the piping inside the building in the terminal end analyses
4.2 controls to be required during plant operation to pervent placement of heavy loads over buried piping and conduits
4.3 as-built report with confirmatory data in FSAR on completed construction
4.4 justification why the BWST lines are not to be rebedded from the tank farm dike to the auxiliary building
4.5 a list of all penetrations for underground seismic Category I piping. Revise and submit your pipe monitoring program to include periodic measurements of rattelspace for plant operating life. Provide justification for all exceptions.
4.7 justification for the high (beyond limits) reported settlement stesses

5. Provide the following information regarding the Diesel Generator Building:

5.1 a structural reanalysis considering:
    (a) Presurcharge conditions
    (b) Conditions during the surcharge
    (c) 40-year settlement effects
    (d) The combined effects of (a) through (c) above
5.2 a structual reanalysis assuming reduction in soil spring stiffnesses between bays 3 and 4 on the south side and beneath adjacent cross wall
5.3 a statistical evaluation of settlements to evaluate impact of survey inaccuracies versus actual differential settlements which have been experienced

5.4 acceptability of 1.5 X SSE (FSAR) versus SSRS for bounding design
5.5 criteria relating crack width and spacing to reinforcing steel stress
5.6 settlement monitoring program to be required during plant operation with action levels and remedial measures identified (Tech. Spec.)
5.7 evaluation of effect of past and future differential settlements to diesel lines from the day tank to the diesels.

6. Provide a settlement monitoring program to be required during plant operation with action levels and remedial measures identified (Tech. Spec.) for the underground Diesel Fuel Oil Storage Tanks.

7. Provide the following information regarding the permanent dewatering system:

7.1 results of the dewatering recharge tests
7.2 technical specification requirements on the permanent dewatering system.
7.3 a summary dicussion of your contingency plans which would be implemented in the event groundwater levels at critical locations exceed limits in the technical specifications.

8. Provide a settlement monitoring program to be required for structures founded on natural soils and plant fill which have not been identified above with action levels and remedial measures identified. (Tech. Spec.)

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Landsman

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

MAY 18 1982

MEMORANDUM FOR: E. Adensam, Chief, Licensing Branch 4 NRR

FROM: C. E. Norelius, Director, Division of Engineering and Technical Programs

SUBJECT: REVIEW OF THE ASLB ORDERS AND THE APPLICANT'S RESPONSE (MIDLAND)

In keeping with our discussions concluding on May 13, 1982, our comments on the subject documents are attached for your use in responding to the applicant. Attachment 1 sets forth our comments on the ASLB orders. Attachment 2 is our understanding regarding the NRR approval status of pertinent construction activities. Attachment 3 sets forth our comments on the Applicant's May 10, 1982 letter responding to the ASLB orders.

Please call Ross Landsman or me if you have questions.

C E Norelius

C. E. Norelius, Director
Division of Engineering
and Technical Programs

Attachments: As Stated

cc w/attachments:
D. Boyd

8307260021

ATTACHMENT 1

Comments on the ASLB orders:

1. We understand that any geotechnical work defined on drawing C-45 requires prior NRC approval with the exception of those already approved, as discussed in Attachment 2.

2. We further understand that any geotechnical work defined on drawing C-45 must be controlled by a staff-approved QA plan. The QA plan approved by Mr. Gilray (January 7, 1982, CPCo submittal) only addresses the "underpinning" activities. To comply with the Order, the licensee now needs to develop a fully comprehensive geotechnical QA plan which covers the broader range of remedial work.

3. We recommend that it be made clear in our reply to the applicant that the use of drawing C-45 to show the boundary of "Q" work does not necessarily limit the general applicability of the applicant's QA/QC programs to other areas that are determined to have safety significance.

4. CPCo's submittal, dated April 5, 1982 to Mr. Keppler, states that, "... the non-Q classification of the permanent dewatering system, except for the installation of wells and the monitoring of fines, had been specifically resolved previously with the NRR staff". We consider their conclusion to be not fully responsive in view of the Order. We contend that the total permanent dewatering system should be under the QA program.

ATTACHMENT II

The following represents Region III's understanding of the approval status of the various activities and issues at the site.

1. Activities previously approved by NRC and in progress:

   a. Freeze-wall installation (activation is subject to Region III concurrence that four monitoring pits over safety-related utilities and monitoring instrumentation have been installed adequately). March 24, 1981, February 24, 1982.

   b. Auxiliary building access shafts to El. 609. November 24, 1981 and March 12, 1982

   c. Permanent dewatering wells (See comment under Attachment 1). June 18, 1981, September 2, 1981, October 22, 1981, and December 28, 1981.

   d. Surcharge of BWST valve pits and subsequent removal. September 25, 1981, and February 26, 1982.

2. Activities previously approved by NRR, but not in progress:

   a. SWPS construction dewatering. April 2, 1982.

   b. Grouting of cracks in BWST foundation. March 26, 1982.

3. Activities not explicitly approved in writing, but in progress:

   a. Instrumentation monitoring system for auxiliary building underpinning (Region III has a confirmatory action letter from the licensee on this item and will restart activities only upon Region III approval).

   b. Deep-seated benchmarks in auxiliary building (10 already installed, 2 more to go).

   c. Auxiliary building construction dewatering wells (these were not covered by the QA/QC program and Region III cannot verify their adequacy).

4. Activities not explicitly approved in writing nor in progress:

   a. Crack mapping of FIVP and auxiliary building.

ATTACHMENT III

Comments on CPCo's May 10, 1982 response to the ASLB order of April 30, 1982 are set forth below. Items which have been covered in the proceeding two attachments will not be addressed again.

1. In Item I.f. (on page 2), we do not understand what a FIVP proof load test is or where it has been approved.

2. We do not concur with their statement in paragraph one on page 3, "The construction dewatering wells were installed to an acceptance criteria agreed upon by the staff." We are not aware of any acceptance criteria for the construction dewatering wells. Region III has not inspected any of the temporary construction dewatering wells because they were not on the Q-list.

MAY 1 7 1982

Docket Nos: 50-329 OM, OL
and 50-330 OM, OL

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D/D		PAO	
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DRSPI			
DE&TI			
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Landsman

APPLICANT: Consumers Power Company

FACILITY: Midland Plant, Units 1 and 2

SUBJECT: SUMMARY OF MAY 7, 1982, CONFERENCE TELEPHONE CALL ON PHASE 2 ISSUES FOR AUXILIARY BUILDING UNDERPINNING

On May 7, 1982, the NRC Staff participated in a conference telephone call with Consumers Power Company (the applicant), and Bechtel to discuss issues associated with Phase 2 of the construction activities for the Auxiliary Building underpinning.

Enclosure 1 is a summary of this telephone conversation.

/S/
Darl S. Hood, Project Manager
Licensing Branch No. 4
Division of Licensing

Enclosure:
As stated

cc: See next page

MAY 1 9 1982

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OFFICE	DL:LB #4	DL:LB #4					
SURNAME	DHood/mc	EAdensam					
DATE	5/14/82	5/14/82					

NRC FORM 318 (10-80) NRCM 0240 OFFICIAL RECORD COPY USGPO: 1981—335-

MIDLAND

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Geotechnical Engineers, Inc.
ATTN: Dr. Steve J. Poulos
1017 Main Street
Winchester, Massachusetts 01890

RECORD OF TELEPHONE CONVERSATION

DATE: May 11, 1982, 1:00 pm PROJECT: Midland

RECORDED BY: Joseph D. Kane CLIENT:

TALKED WITH:

CPC	Bechtel	NRC
J. Schaub	N. Swanberg	F. Rinaldi
J. Mooney	J. Anderson	D. Hood
	C. Russell	J. Kane
	B. Dhar	
	W. Paris	

ROUTE TO:

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

MAIN SUBJECT OF CALL: To discuss Phase 2 Issues - Auxiliary Building Underpinning

ITEMS DISCUSSED:

Consumers arranged this conference call to discuss review items related to Auxiliary Building underpinning. These items had been identified in a brief call on May 7, 1982 by J. Kane to J. Schaub where the NRC Staff had expressed their recommendations on the following items:

1. Location of deep seated benchmarks DSB-AS1 and DSB-AS2. The current hold on construction and field installation of monuments prevents the actual locations from being established. Consumers will provide actual locations when these benchmarks are installed and recognize these monuments are to be installed at a distance not to exceed 5 feet from the wall of the Main Auxiliary Building which is founded at Elevation 562.

2. Strain gage installation. The NRC Staff's comments for correction of drawing C-1495 were accepted and the drawing will be revised. (Lower strain gages at Elev. 584 to 614 on Sectional View-Wall at Col. Lines 7.4 and 7.8 are to be reorientated 90 degrees and column lines H and G will be corrected). Bechtel will check why strain gage at Elev. 646 to 659 range was not proposed for Wall at Col. lines 7.4 and 7.8 and will get back to Staff. The vertical alignment of strain gage on Col. Lines 5.3 and 5.6 at Elevation range 646 to 659 is being controlled by the need to avoid equipment obstructions on the wall. Consumers will make an analytical correction for the vertical alignment when evaluating strain gage readings.

3. Pier test procedures. Consumers indicated the dead load available in the existing structure for the reaction load in the pier load test is approximately 90 percent of the maximum design load. Consumers wished to further consider the Staff's recommendation to perform a plate load test where the maximum test load would be equal to 130 percent of the maximum design load and a pier load test at 90 percent of the maximum design load.

   Consumers accepted the Staff's recommendation for performing two in situ density tests and a minimum of five cone penetrometer tests on the soil at the bottom of the pier selected for load testing. Consumers also agreed to use bituminous coated plywood sheeting for reducing the effects of skin friction during the pier load test.

   Consumers wished to further consider the Staff's recommendation for requiring a rate of settlement that would not exceed 0.005 inch per hour when controlling the length of time that the 90 percent test load increment would be maintained.

   To better explain what the Applicant intended when it indicated that it would make modifications to ASTM D1143 as deemed appropriate, Consumers will provide the Staff with the pier load test procedures that identify the proposed modifications.

4. Construction dewatering. The Applicant indicated its plan for constructior dewatering during underpinning is nearly complete and will be provided to the Staff within a week. Most of the dewatering wells are already installed but additional wells are planned. The additional wells are to be installed with Q/A procedures that are similar to the permanent dewatering wells which were previously approved by the NRC Staff. Monitoring for loss of soil particles due to pumping will be conducted according to the agreements reached for construction dewatering of the SWPS. (April 2, 1982 letter with enclosures, R. Tedesco to J. Cook).

   Consultants to Consumers indicated the already installed construction dewatering wells extend to the natural clay layer at approximately El 585. The Staff indicated that the anticipated plan for construction dewatering to be provided by Consumers should address the problem of handling seepage on the sides and bottom of pier excavations which extend below the bottom of the already installed wells.

5. Movement of Feedwater Isolation Valve Pit (FIVP). Consumers indicated its intent to assure transfer of the FIVP loading to the Turbine Building and Buttress Access Shafts by jacking the installed support system. It is not the intent of this jacking to restore the FIVP to its original position but

rather assure transfer of the load. The procedure for future jacking which Consumers indicated they would follow at the February 1-5, 1982 design audit and which was found acceptable by the NRC Staff requires jacking of the FIVP back to its original position if the relative settlement between the Reactor Containment and the FIVP reaches a total settlement of 3/8-inches since the date that the piping connections were made.

MEETING SUMMARY DISTRIBUTION

MAY 17 1982

Docket Nos: 50-329/330 OM, OL
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NRC Participants:

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DHood
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RGonzales
RLandsman RIII

bcc: Applicant & Service List

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

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

May 3, 1982

Landsman

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

MIDLAND PROJECT
MIDLAND DOCKET NO 50-329, 50-330
UNDERGROUND PIPING INFORMATION REQUESTED DURING APRIL 16, 1982 MEETING
FILE: 0485.16 SERIAL: 16881
REFERENCES: (1) J W COOK LETTER TO H R DENTON, SERIAL 16269, DATED MARCH 16, 1982
(2) J W COOK LETTER TO H R DENTON, SERIAL 16638, DATED APRIL 15, 1982
ENCLOSURES: (1) TABLE 1.0 MONITORING STATION OVALITY AND CORRESPONDING STATION
(2) BURIED CATEGORY 1 LINES AND TANKS
(3) ADDITIONAL GEOTECHNICAL INFORMATION

The purpose of this letter is to provide confirmatory information regarding several issues discussed during a meeting between the NRC Staff and Consumers Power Company. The meeting was held in Bethesda on April 16, 1982.

Enclosure 1 is an expansion of the table previously submitted by our letter, Serial 16638, dated April 15, 1982. Additional information is provided specifying the future allowable strain based on an acceptance criteria and technical specification limit of 0.48% strain. The number of strain gages has also been specified in the table. The number of gages were determined by reviewing the pipe elevation profiles for abrupt inflection points and critical buckling zones. The strain gages are to be mounted one pipe diameter apart at a given monitoring station.

At the April 16 meeting a concern arose about the accuracy of the vibrating wire strain gages. In a telephone conference with the Irad Gage Company, they indicated the instrument is accurate to 10 (μinch/inch) as a worst case condition for any type of vibrating wire gage. This includes accounting for inaccuracies in installation and calibrations. This accuracy is an order of magnitude greater than the accuracy required for the strain measurements to be taken (.0001 in/in vs .00001 in/in).

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MAY 10 1982

A clarification on the technical specification limits and requirements proposed in the pipe monitoring program submitted March 16, 1982 is necessary. Our intention is to use the 4% ovality (equivalent .0048 inch/inch strain) which includes appropriate safety factors as the technical specification unless we can justify a higher value at a later date. If the specified limit is reached we would immediately notify the NRC Staff and increase the monitoring frequency to one month intervals. In parallel with the Staff notification an engineering evaluation of the situation would be performed. This evaluation would consider the remedial action necessary to restore the safety function and reliability of the service water system to overall plant operations. The actions necessary may very well include excavation of the piping in the affected zone for visual examination and possible replacement or sleeving.

The NRC Staff asked Consumers Power Company to verify that no other buried Category 1 pipes remain unidentified. Enclosure 2 is a current table of all the buried seismic Category 1 lines and tanks. The pressurization lines and tanks have been added to the list of buried Category 1 piping. The control room pressurization lines and tanks were installed during the summer 1981, and therefore not subjected to the soils settlement problems. The penetration pressurization lines and tanks have not been installed; however appropriate procedures for soil settlement will be followed. The list does not include the 48-inch diameter (48-OHBC-2) discussed in Enclosure 3 of our letter, Serial 16638, dated April 15, 1982.

The NRC Staff expressed a concern regarding the margins for future settlement at the wall penetration of pipeline 26-OHBC-15. Our investigations indicate that there is a 90° elbow fitting in this line immediately upon exiting the building. Any bending moment developed due to soils settlement will be transformed to an equal torque value. This load transformation causes the vertical deflection due to settlement to change to an angle of twist on the pipe at the penetration. This angle of twist has no effect on the annulus clearance of the wall penetration and therefore the only real clearance we need to assure is the seismic rattlespace (0.3693 inch). The margin we presently have is 0.6307 inches which is a factor of 1.7 times the conservative estimate of seismic rattlespace.

The NRC Geotechinical Branch requested information concerning soils and its relation to buried utilities. Enclosure 3 addresses the concerns expressed about the prediction of maximum future settlement for plant life (3.0 inches) and the isolated sand pocket near the diesel fuel tanks. A concern was also expressed about the soil properties used in estimating the soil forces required to deform condensate line (20-1HCD-169) into its present configuration. We have responded by seperately providing the Structural Mechanics Assoiciates calculations estimating the soil capacity at Midland.

We believe the information supplied satisfies the concerns the NRC Staff expressed during the recent April meeting.

J A Mooney
Executive Manager
Midland Project Office

For J W Cook

JWC/WJC/mkh

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
PChen, ETEC, w/a
FCherney, NRC, w/a
MMCherry, Esq, w/o
FPCowan, ASLB, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
SGadler, w/o
JHarbour, ASLB, w/o
DSHood, NRC, w/a (2)
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, w/o
WDPaton, Esq, w/o
BStamiris, w/o

TABLE 1.0
Monitoring Station Ovality and Corresponding Strain

Station*	Measured Ovality (%)	Meridional Strain (%)	Future Allowable Strain (%)	No of Strain Gages
Line: 26-OHBC 15 Reference: Figure 1			Allowable Strain = .48%	
1	2.34	0.35	0.13	2
2	1.88	0.32	0.16	3
3	2.34	0.35	0.13	2
4	2.34	0.35	0.13	2
5	1.24	0.25	0.23	2
Line: 26-OHBC 16 Reference: Figure 2				
1	2.18	0.34	0.14	3
2	2.18	0.34	0.14	2
3	2.34	0.35	0.13	3
4	2.18	0.34	0.14	2
5	1.12	0.23	0.25	2
Line: 26-OHBC 53 Reference: Figure 3				
1	1.40	0.27	0.21	2
2	2.96	0.40	0.08	2
3	2.18	0.34	0.14	3
4	2.18	0.34	0.14	2
Line: 26-OHBC 54 Reference: Figure 4				
1	2.50	0.36	0.12	2
2	2.50	0.36	0.12	3
3	2.18	0.34	0.14	2
4	2.03	0.32	0.16	2
5	2.50	0.36	0.12	3
6	2.03	0.32	0.16	2
Line: 26-OHBC 55 Reference: Figure 5				
1	2.03	0.32	0.16	2
2	1.47	0.27	0.21	2
3	1.56	0.28	0.20	2
4	1.56	0.28	0.20	2

Station*	Measured Ovality (%)	Meridional Strain (%)	Future Allowable Strain (%)	No of Strain Gages
Line: 26-OHBC 56 Reference: Figure 5				
1	1.09	0.22	0.26	2
2	1.87	0.31	0.17	2
3	0.90	0.21	0.27	2
4	2.49	0.36	0.12	2
Line: 26-OHBC 19 Reference: Figure 6				
1	1.87	0.31	0.17	2
2	1.87	0.31	0.17	3
3	1.87	0.31	0.17	2
4	0.89	0.21	0.27	2
Line: 26-OHBC 20 Reference: Figure 6				
1	1.87	0.31	0.17	2
2	1.87	0.31	0.17	2
3	1.87	0.31	0.17	3
4	1.79	0.30	0.18	2

*The station numbers are numbered from left to right from the given reference figures transmitted March 16, 1982.

BURIED SEISMIC CATEGORY I LINES AND TANKS

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 and Tanks

1-1/2"-1HBC-3	2"-1HBC-497	1T-77A
1-1/2"-1HBC-4	2"-1HBC-498	1T-77B
1-1/2"-2HBC-3	2"-2HBC-497	2T-77A
1-1/2"-2HBC-4	2"-2HBC-498	2T-77B

C. Borated Water Lines

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

D. Control Room Pressurization Lines and Tanks

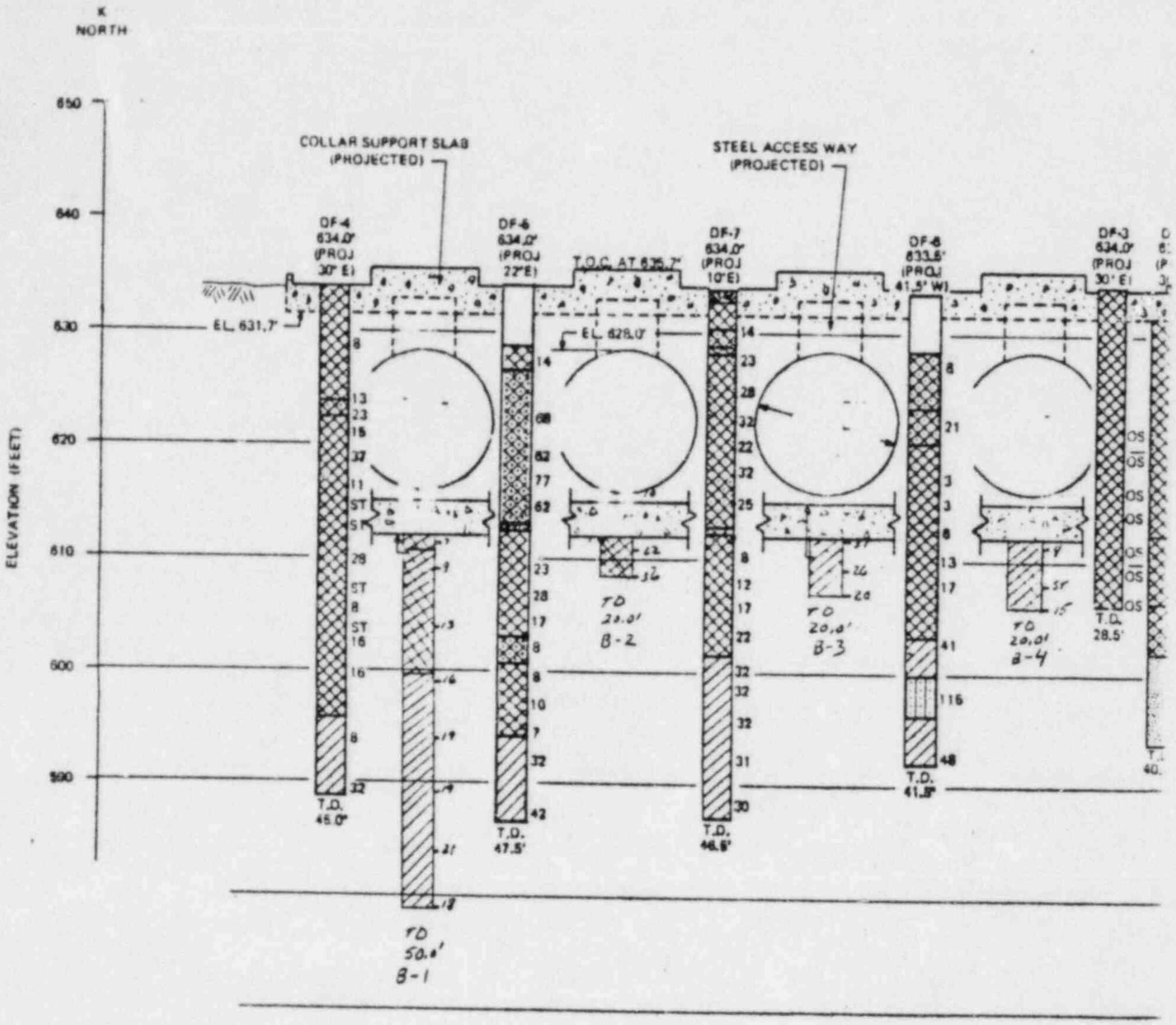
4"-0DBC-1	0VT 68A
1"-0CCC-1	0VT 68B

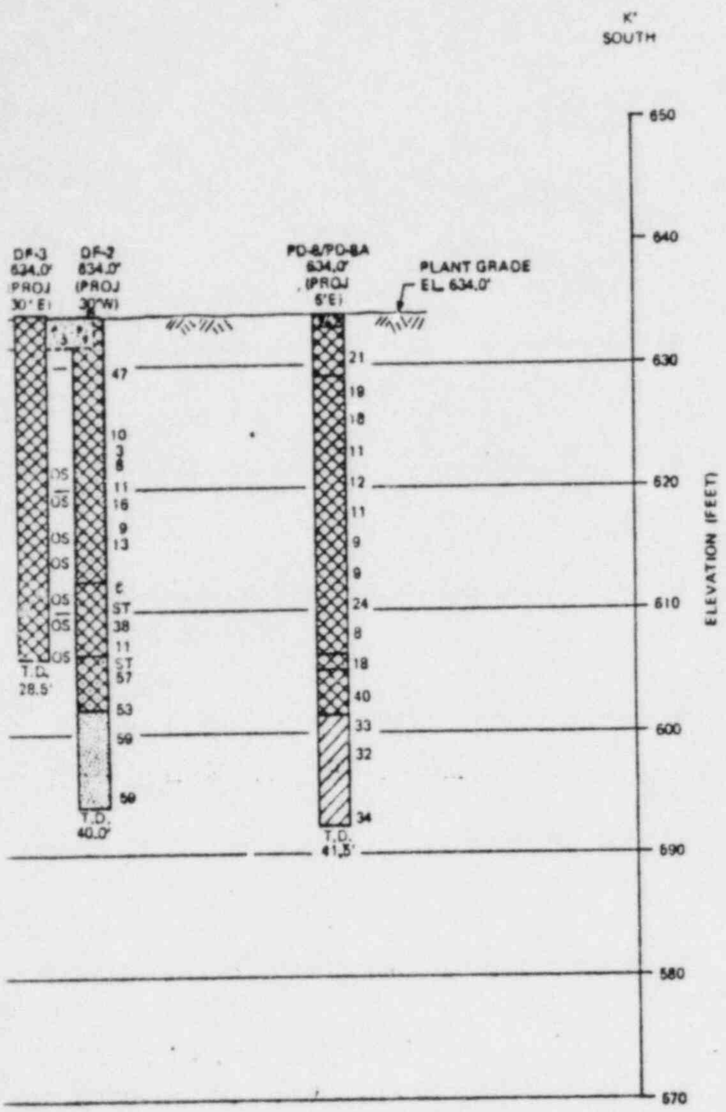
E. Penetration Pressurization Lines and Tanks

1"-1CCB-45	1T-114
1"-2CCB-45	2T-114

4/26/82

unable to reproduce drawing on next page.


K
NORTH
ELEVATION (FEET)
650
640
630
620
610
600
590
COLLAR SUPPORT SLAB (PROJECTED)
STEEL ACCESS WAY (PROJECTED)
DF-4 634.0' (PROJ 30' E)
DF-6 634.0' (PROJ 22' E)
T.O.C. AT 625.7'
DF-7 634.0' (PROJ 10' E)
DF-8 633.5' (PROJ 41.5' W)
DF-3 634.0' (PROJ 30' E)
EL. 631.7'
EL. 628.0'
T.D. 45.0'
T.D. 47.5'
T.D. 46.6'
T.D. 41.8'
T.D. 28.5'
TD 50.0' B-1
TD 20.0' B-2
TD 20.0' B-3
TD 20.0' B-4



EXPLANATION

- T.O.C. — TOP OF CONCRETE
- F.F.G. — FINAL FOOTING GRADE
- 2T-78A — TANK IDENTIFICATION NUMBER
- DF-8 — BORING NUMBER
- 633.5' — GROUND SURFACE ELEVATION
- (PROJ 41.5'W) — DISTANCE AND DIRECTION OF BORING TO SECTION LINE
- ST — SHELBY TUBE SAMPLE
- OS — OSTERBERG SAMPLE
- 21 — STANDARD PENETRATION BLOWCOUNT (BLOWS/FOOT)
- T.D. 41.5' — TOTAL DEPTH IN FEET

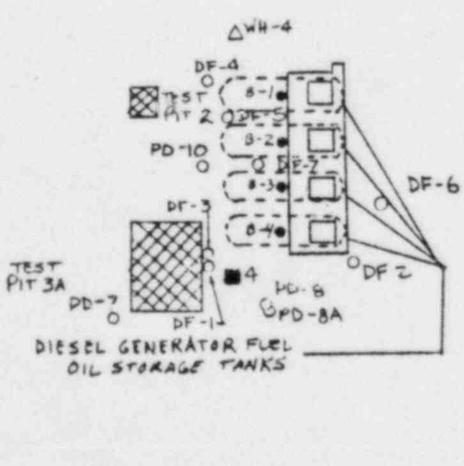
LEGEND

- SANDY GRAVEL FILL
- CLAY FILL
- SAND FILL
- CONCRETE
- CLAY (CL)
- SILTY SAND (SM)
- SAND (SP)
- NO SAMPLES

NOTES:

1. For the location of this cross-section see Figure 2.5-17.
2. Boring DF-3 has no SPT blowcounts.

0 5 10 20 30
SCALE IN FEET



1		ISSUED FOR PROJ USE			
0		FOR PROJ. USE			

BECHTEL
ANN ARBOR

MIDLAND POWER PLANT

SUBSURFACE CROSS-SECTION K-K'
DIESEL FUEL OIL STORAGE TANKS

JOB NO.	DRAWING NO.	REV.
7220	FIGURE 2.5-22H	1

SK G-339

ENCLOSURE 3.0

ADDITIONAL GEOTECHNICAL INFORMATION

Prediction of Maximum Future Settlement For Buried Utilities

To predict the maximum future settlement for buried utilities, settlement monitoring within the fill has been utilized in our analysis. There are nine (9) locations in the vicinity of buried utilities where Borros anchors have been installed and have not been influenced by surcharge loadings. Settlement readings for anchors that have been established at a depth of 7 feet to 12 feet below the surface were used in the analysis, since these depths are representative of the depth of most buried utilities. Soils conditions at the locations of the Borros anchors is also representative of the variable soil conditions encountered throughout the fill.

Borros anchors BA-13, BA-14, and BA-34 were installed in December 1978 and have over three years of data. Settlement plots for these anchors are shown on Figure 1.0. Borros anchors BA-100 through BA-106 were installed in September 1979 and have over two years of data. Settlement data from anchors BA-100 through BA-106 project less future settlement then shown for BA-34. The log of time versus settlement plots projected for most of these anchors predict on the order a maximum total 2.0 to 2.5 inches of additional settlement to occur over the next 40 years of buried utility life. Settlement projections for BA-34 are considered to provide a conservative estimate of the future maximum settlement expected beneath any buried utilities in the site fill. A total maximum future settlement during plant life has been estimated not to exceed 3 inches and includes settlement due to dewatering and seismic shakedown.

NO. 340-L510 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
5 CYCLES X 10 DIVISIONS PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.

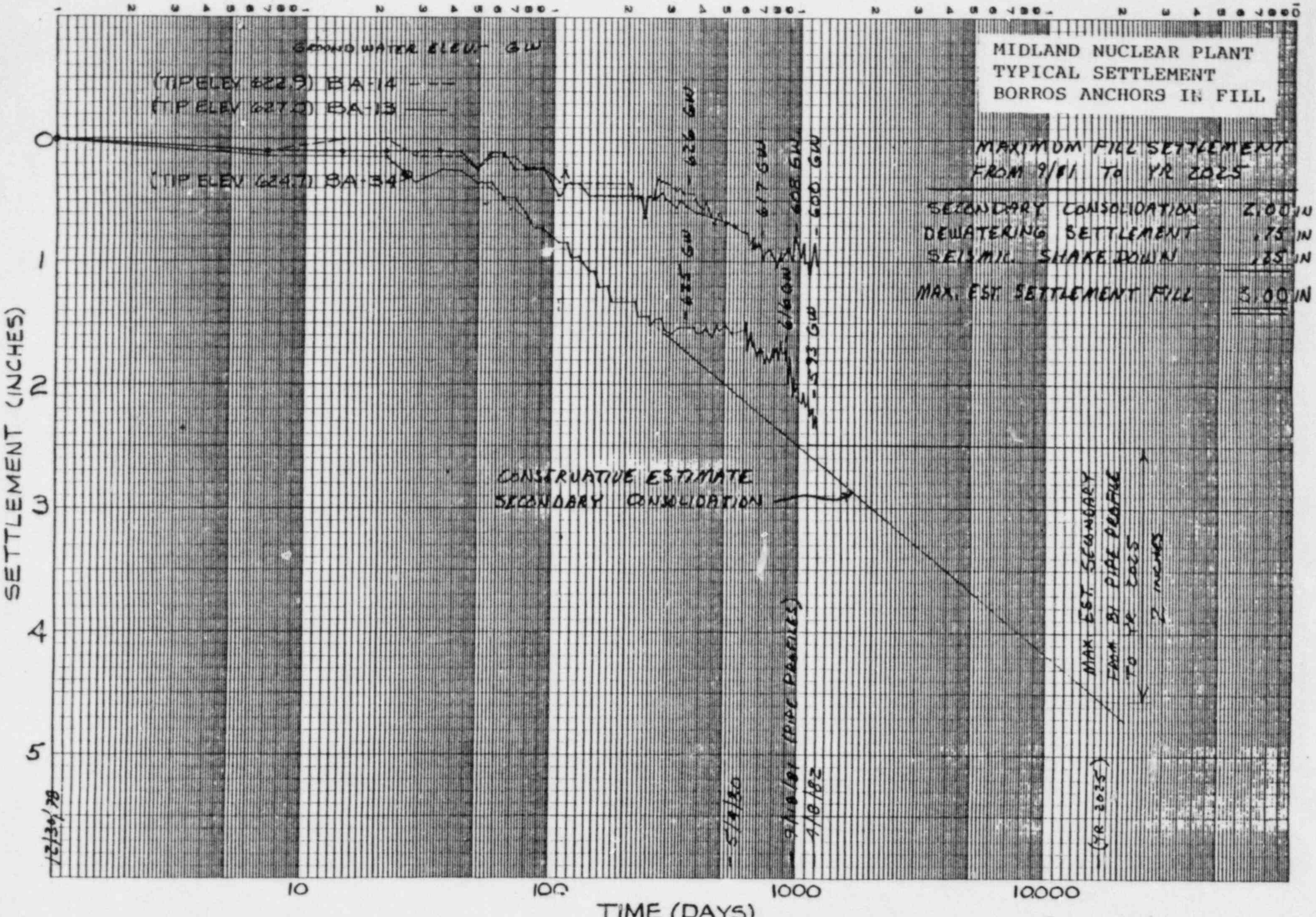
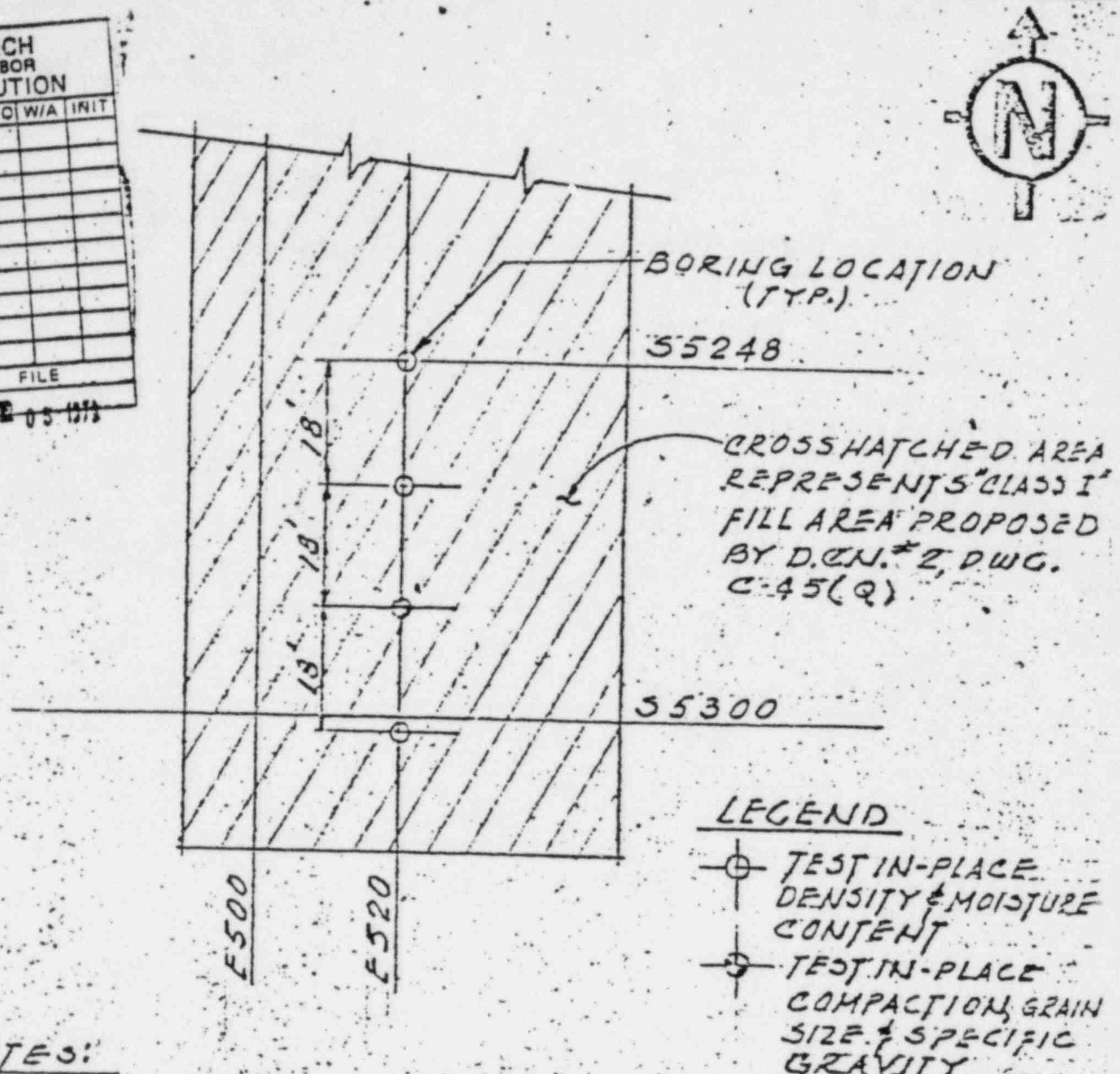


Fig. 1

DIESEL FUEL TANK SEISMIC STABILITY
RELATED TO LIQUEFACTION OF ISOLATED SAND POCKET

Figure 2.5-22H is a cross-section through the DOFT showing fill and natural soil conditions. The section includes 4 borings (B-1 through B-4) drilled in July 1977 before the excavation was made in the original plant fill to construct the tanks. The location plot and logs of these borings are also attached. It is seen from available information that the loose sand pocket in boring DF5 near elevation 600 is limited in extent and therefore considered confined by clay fill.

An analysis was made of the diesel fuel oil tanks assuming liquefaction does occur in a postulated thin layer of sand below the entire area of the tanks. Since the tanks are anchored down and have adequate resistance to flotation, any movement of the tanks under these postulated conditions would be resisted by the passive resistance of the fill surrounding the tanks. The safety factor against sliding of these tanks under these conditions was calculated to be at least 1.7. This analysis indicates that the tanks will be stable even if liquefaction of the loose sand pocket does occur. Lateral movement estimated under these conditions is less than 1/2 inch. The 1-1/2 to 2 inch diameter diesel fuel piping lines and tank connections have sufficient flexibility to accomodate this differential movement.



NOTES:

1- BORINGS SHALL BE MADE IN LOCATIONS SHOWN.
2- MATERIALS FROM BORINGS SHALL BE TESTED IN ACCORDANCE WITH SPEC. 7220-C-208
3- TESTS SHALL BE ADMINISTERED IN ACCORDANCE WITH THE QUALITY ASSURANCE PROGRAM SPEC. 7220-G-22

SB 19092

NO.	DATE	REVISIONS	BY	CHK'D	GROUP LEAD	GROUP SUPV.	PROJ ENGR.	CHIEF ENGR.
△								
△								
A	5-20-77	ISSUED FOR INFORMATION	LZC.	TJD	Rao	TLA	RLC/RLX	

SCALE	DESIGNED	DRAWN
ORIGIN	MIDLAND PLANT - UNITS 1 & 2 CONSUMERS POWER CO. EMERGENCY DIESEL FUEL TANK AREA-TEST BORING ARRANGEMENT	JOB No. 7220 DRAWING No. SK-C-541 REV. A

SERVICE CONTRACTS DIVISION
TEST BORING DEPARTMENT

Date JULY - 21 - 1977 Job No. ______

Job Address A.T.&T. MIDLAND, MICH.

Fixed Datum used is ______

Ground Surface this boring is ______

DEPTH From	DEPTH To	CLASSIFICATION Be Careful and Accurate	Sample Type	Sample No.	Depth	No. of 30" blows on Spoon 1st 6"	2nd 6"	3rd 6"	Recovery in.	Lost W... or Rema...
Grd. Surface	0'9"	GRAVEL FILL								
0'9"	4'6"	DENSE BROWN FINE TO	S.S.	1	2'6"	13	13	18		
		MEDIUM SILTY SAND	S.S.	2	5'0"	8	13	10		
4'6"	8'6"	LOOSE BROWN FINE SAND	S.S.	3	7'6"	5	4	3		
8'6"	11'0"	VERY LOOSE GRAY FINE	S.S.	4	10'0"	1	1	1		
		SILTY SAND TRACE OF ORGANIC								
11'0"	13'0"	MEDIUM GRAY CLAYEY SAND	S.S.	5	12'6"	5	7	3		
13'0"	18'0"	MEDIUM GRAY SANDY CLAY	S.S.	6	15'0"	2	2	2		
			S.S.	7	17'6"	3	3	4		
18'0"	29'0"	STIFF GRAY SANDY CLAY	S.S.	8	20'0"	4	4	5		
		SOME SMALL GRAVEL	S.S.	9	25'0"	4	6	7		
29'0"		VERY STIFF GRAY SILTY CLAY	S.S.	10	30'0"	5	7	9		
		SOME SMALL GRAVEL	S.S.	11	35'0"	5	8	11		
			S.S.	12	40'0"	5	8	11		
			S.S.	13	45'0"	7	9	12		
			S.S.	14	50'0"	5	8	10		
		WATER ENC. AT. 9'0"								

GEOTECH
ANN ARBOR
DISTRIBUTION

DISC	ACT	INFO	W/A	INIT
MGR				
ADMIN				
DRFT				
SOILS				
GEOL				
H&H				
EWP				
Proj Mgr				
Proj Eng				
JOB		FILE		
REC'D	[illegible] 05 1979			

NOT BECHTEL

BECHTEL'S DONE 1st JUNE

Ground Surface to 8'6" ft. used 3 " casing.

Water level is 8'8" ft. below Ground surface 12 hrs. after completion.

Water level is ______ ft. below Ground surface ______ hrs. after completion.

Boring stopped by ______

Foreman HERSCHEL BOYD

Boring No. B-1

SB 13094

RAYMOND INTERNATIONAL INC.
SERVICE CONTRACTS DIVISION
TEST BORING DEPARTMENT

Date JULY - 22 - 1977.
Job No.
Job Address / MIDLAND, MICH.
Fixed Datum used is
Ground Surface this boring is

DEPTH From	DEPTH To	CLASSIFICATION Be Careful and Accurate	Sample Type	Sample No.	Depth	No. of 30" blows on Spoon 1st 6"	2nd 6"	3rd 6"	Recovery in.	Lost Water or Remarks
Grd. Surface	0'9"	GRAVEL FILL								
0'9"	6'0"	MEDIUM BROWN FINE SILTY	S.S.	1	2'6"	13	11	13		
		SAND	S.S	2	5'0"	19	15	13		
6'0"	8'0"	LOOSE BROWN FINE TO	S.S.	3	7'6"	4	4	4		
		MEDIUM SILTY SAND								
8'0"	11'0"	LOOSE BROWN & GRAY FINE	S.S.	4	10'0"	1	1	5		
		SILTY SAND								
11'0"	16'0"	STIFF GRAY SANDY CLAY	S.S.	5	12'6"	5	5	5		
			S.S.	6	15'0"	4	6	12		
16'0"		VERY STIFF GRAY SILTY	S.S.	7	17'6"	5	10	12		
		CLAY SOME SMALL GRAVEL	S.S.	8	20'0"	9	16	20		
		WATER ENC. AT 10'0"								

GEOTECH
ANN ARBOR
DISTRIBUTION

DISC	ACT	INFO	W/A	INIT
MGR				
ADMIN				
DRFT				
SOILS				
GEOL				
H&H				
ENP				
Proj Mgr				
Proj Eng				
JOB		FILE		
REC'D		05 1979		

Ground Surface to ______ ft. used 4" AUGER casing.
Water level is 8'6" ft. below Ground surface 6 hrs. after completion.
Water level is ______ ft. below Ground surface ______ hrs. after completion.
Boring stopped by

Foreman HOSCHA BOYD
Boring No. B-2

SB 10095

SERVICE CONTRACTS DIVISION
TEST BORING DEPARTMENT

Date JULY - 22 - 1977 Job No. ____

Job Address ____ MIDLAND, MICH.

Fixed Datum used is ____

Ground Surface this boring is ____

DEPTH From	DEPTH To	CLASSIFICATION Be Careful and Accurate	Sample Type	Sample No.	Depth	No. of 30" blows on Spoon 1st 6"	2nd 6"	3rd 6"	Recovery in.	Lost Water or Remarks
Grd. Surface	2'0"	LOOSE BLACK SILTY SAND								
2'0"	6'0"	MEDIUM BROWN FINE TO	S.S.	1	2'6"	2	4	6		
		MEDIUM SILTY SAND	S.S.	2	5'0"	6	7	7		
6'0"	9'0"	LOOSE BROWN FINE SILTY SAND	S.S.	3	7'6"	2	2	1		
9'0"	12'0"	STIFF GRAY VERY SILTY CLAY	S.S.	4	10'0"	4	4	5		
		SOME SMALL GRAVEL	S.S.	5	12'6"	6	7	14		
12'0"		VERY STIFF GRAY VERY SILTY	S.S.	6	15'0"	8	15	22		
		SANDY CLAY SOME SMALL GRAVEL	S.S.	7	17'6"	8	12	14		
			S.S.	8	20'0"	7	9	11		

GEOTECH
ANN ARBOR
DISTRIBUTION

DISC	ACT	INFO	W/A	INIT
MGR				
ADMIN				
DRFT				
SOILS				
GEOL				
H&H				
EWP				
Proj Mgr				
Proj Eng				

JOB FILE
REC'D MAR 2 8 1978

WATER ENC. AT 6'4"

Ground Surface to 4" AUGER ft. used ____ " casing.

Water level is 6'6" ft. below Ground surface 4 hrs. after completion.

Water level is ____ ft. below Ground surface ____ hrs. after completion.

Boring stopped by ____

Foreman HERSCHEL BOYD

Boring No. B - 3

SB 19096

RAYMOND INTERNATIONAL INC.
SERVICE CONTRACTS DIVISION
TEST BORING DEPARTMENT

Date JULY 27-1977 Job No.

Job Address MIDLAND, MICH.

Fixed Datum used is

Ground Surface this boring is

DEPTH From	DEPTH To	CLASSIFICATION Be Careful and Accurate	Sample Type	Sample No.	Depth	No. of 30" blows on Spoon 1st 6"	2nd 6"	3rd 6"	Recovery in.	Lost Water or Remarks
Grd. Surface	1'6"	MEDIUM BLACK SILTY SAND								
1'6"	6'0"	MEDIUM BROWN FINE SILTY	S.S.	1	2'6"	5	7	9		
		SAND	S.S.	2	5'0"	5	7	8		
6'0"	10'6"	LOOSE BROWN + GRAY FINE	SS	3	7'6"	2	1	1		
		SILTY SAND TRACE OF ORGANIC	SS	4	10'0"	2	2	5		
10'6"		STIFF GRAY SANDY CLAY	3-T	5	11'0" To 13'0"				24"	
		SOME SMALL GRAVEL	S.S.	6	14'6"	3	4	4		
			3-T	7	16'6" To 18'6"				11"	
			S.S.	8	25'0"	8	8	7		
		WATER ENC. AT 6'6"								

Ground Surface to ______ ft. used ______" casing.

Water level is 7'0" ft. below Ground surface 1/2 hrs. after completion. Foreman HOSTETLER BOYL

Water level is ______ ft. below Ground surface ______ hrs. after completion.

Boring stopped by ______ Boring No. B-4

SB 19097

MEMORANDUM OF CALL

TO:

Ross

☐ YOU WERE CALLED BY— ☐ YOU WERE VISITED BY—

DARRELL Hood

OF (Organization)

☐ PLEASE CALL → PHONE NO. CODE/EXT. ☐ FTS

☐ WILL CALL AGAIN ☐ IS WAITING TO SEE YOU

☐ RETURNED YOUR CALL ☐ WISHES AN APPOINTMENT

MESSAGE

Conference CALL
w/ Consumers
@ 1:00 P.M.
If not OK, call him

RECEIVED BY	DATE	TIME
D		10:00

63-109
GPO : 1981 0 - 341-529 (106)

STANDARD FORM 63 (Rev. 8-76)
Prescribed by GSA
FPMR (41 CFR) 101-11.6

phone call May 3, 1982 CPCo, Bechtel, NRR

~~[illegible]~~

Dewatering Wells (Permanent)

even alarm system not Q

THEY had not received order yet

System not safety-related because of

BWST

NCR Feb [illegible], 1982 → Bechtel # 4000

18" of ice

3 construction dewatering wells

Duct Bank hit

mud into Turbine building

BWST

BWST - NOT UNDER remedial group

↑ if not remedial

[other group (structural group)

(Block Walls)