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GEOTECHNICAL ENGINEERS INC.

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Project 81907
File 2.0
Ref: 81907-27

Mr. Joseph Kane
NRR Project Officer
U. S. Nuclear Regulatory Commission
Division of Engineering, M/S P-214
Washington, D.C. 20555

Subject: Summary Comments Regarding Audit
September 14 and 15, 1983
Bechtel/Ann Arbor
Midland Underpinning

Dear Mr. Kane:

On September 14 and 15, 1983 an audit was held in the offices of Bechtel at Ann Arbor. The purpose was to gather information from the applicant relative to the effect on design and construction of reducing the modulus of the hard clay foundation soil from 3000 ksf to 1500 ksf. The latter value was obtained from the pier load test. Below I summarize and comment on the information received at the audit and related data received previously with respect to the geotechnical aspects of this question.

1. The allowable bearing pressure for design need not be altered as a result of reducing the modulus of the hard clay from 3000 to 1500 ksf. The long-term differential settlement may be expected to increase, however. Therefore, the jacks may have to be held active for a longer period prior to lockoff than previously expected. The measurements made while the jacks are active will provide the data needed to make this decision.

2. The data provided for the settlement of the EPA's and the Control Tower relative to the Main Auxiliary Building and the extensometer data at El 659 show that the stresses in the reinforcing at El 659 increase about 100 psi for each mil of differential settlement. (That is, when 1 increases one mil, the rebar stress at El 659 increases about 100 psi.) The results of Bechtel's computations provided at the audit, using the 3D finite element model and assuming an uncracked structure, show that the computed stresses at El 659 increase by about 110 psi/mil of differential settlement. Thus it appears that there is reasonable compatibility between computed and indirectly measured rebar stresses. Normally one does not expect such good agreement. However, the good agreement lends some credibility to the computed results and decisions that are based on them. The relationship between 1 values and stresses measured by means of the extensometers should be followed closely as the work progresses. The above data indicate that a differential settlement during underpinning of 0.1 in. will cause a rebar stress of about 10,000 psi at the critical locations in the Auxiliary Building.

3. Bechtel's computations showed that there are four locations in the main auxiliary building where the highest stresses were found due to the design loads, including stresses due to differential settlement after lockoff. These were:

	Elev. (ft)	Column Lines	Location
A	659	G-H	Slab
B	Below 614	5.3-7.8	N-S walls
C	634-635	C-F and 5.6-6.2	Slab
D	659	D-G and 4.7-5.6	Slab

The applicant provided a design for fixing Location A because the computed stresses are greater than code allowable. The NRR structural group is reviewing the design. The applicant noted that the computed stresses at Locations B, C, and D were within code allowable limits. Subsequently, the loading combinations that led to the computed results were reviewed at the meeting.

4. The loading combinations considered under accident conditions for stresses in the concrete of the Auxiliary Building were provided in the FSAR. They

do not include any stresses due to differential settlements that have occurred to date nor those that will occur during operation of the plant. The effects of differential settlement were included only for "normal" conditions. Since the differential settlements are always present, a reconsideration of these design loading conditions should be undertaken.

5. Stresses due to future differential settlements were taken into account for design of the underpinning.

6. The applicant should provide information on the stresses that exist at critical locations due to differential settlements that will remain in the structure after lockoff. The applicant previously has indicated that zero (or compressive) stresses will exist after lockoff due to differential settlements. During the audit NRC personnel questioned this assumption. To relieve the stresses that currently exist it would be necessary to lift the EPA's and the Control Tower (probably also the Turbine Building). But the applicant indicated that these structures will not be lifted during underpinning more than about 30 mils. Thus there is incompatibility between the intention of the applicant not to lift the structures and the applicant's assumption that existing differential settlement stresses will be relieved. Since the existing stresses may be in the range of 10,000 psi to 25,000 psi in the rebar at critical locations, they cannot be considered insignificant. Unless the extensometers at El 659 show a reduction in tensile stress when the jacking load is applied, it should be assumed that these stresses remain in the structure. The effects of these stresses on design should be considered by the structural group.

Sincerely yours,

GEOTECHNICAL ENGINEERS INC.

Steve J. Poulos

Steve J. Poulos
Principal

SJP:ms

AUXILIARY BUILDING 19/B3

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December 3, 1981

RECEIVED DEC 9 1981 U.S. NUCLEAR REGULATORY COMMISSION

Harold R Denton, Director
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MIDLAND PROJECT
MIDLAND DOCKET NOS 50-329, 50-330
UNDERPINNING OF THE AUXILIARY BUILDING - CALCULATIONAL RESULTS
FILE 0485.16, B3.0.1 SERIAL 14899
REFERENCE: JWCOOK TO HRDENTON, SERIAL 14110, DATED SEPTEMBER 30, 1981
ENCLOSURE: ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

Attached to the above-referenced correspondence of September 30, 1981, we submitted a design report entitled, "Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits." We are providing as an enclosure to this correspondence twenty-five (25) copies of an addendum to the above-referenced technical report.

The purpose of the enclosed addendum is to supplement Section 7.5 of the above-referenced technical report and Appendix A of the same document. The enclosed addendum contains the following information:

1. Soil pressure data under the auxiliary building and the feedwater isolation valve pits underpinning area.

2. Load combinations used for preliminary design of the underpinning reinforcement walls and the connection joints of the underpinning walls to the auxiliary building.

3. Design forces and moments at the critical sections.

4. Reinforcement details provided in the underpinning walls.

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5. A summary of results from recent preliminary auxiliary building structural analyses which reflect the modified dynamic model of the structure, actual natural soils properties and the proposed underpinnings. These results identify certain areas within the structure which may require some modification in order to meet design requirements. As further analyses are completed, we will forward our proposed plans for any additional remedial actions to the Staff for their review and concurrence.

The material presented in this addendum is based on preliminary analyses of the permanent underpinning configuration. Detailed calculational checks will be performed as a part of the final analysis to verify the design adequacy. We are also currently performing analyses and design checks for the auxiliary building construction condition for various construction stages. The results of these detailed design checks for both the permanent underpinning configuration and the construction condition will be available to the NRC Staff for their audit in accordance with agreements reached at our November 17, 1981 meeting in Bethesda.

This addendum along with our previous submittals and discussions with the NRC Staff should adequately respond to the concerns identified by the Staff. We believe this information continues to support our conclusion that the design of the auxiliary building and feedwater isolation valve pit structures combined with the proposed underpinning remedial actions are adequate and appropriate for these structures.

James W. Cook

JWC/WJC/RLT/dsb

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ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE

AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 AND 2

DECEMBER 2, 1981

MIDLAND PLANT UNITS 1 AND 2
ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE
AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

CONTENTS

MIDLAND PLANT UNITS 1 AND 2
ADDENDUM TO TECHNICAL REPORT ON UNDERPINNING THE
AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

1.0 INTRODUCTION

The purpose of this addendum is to supplement Section 7.5 of the Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits (Reference 1) with the following information:

a. Soil pressure data under the auxiliary building, feedwater isolation valve pits (FIVPs), and auxiliary building underpinning

b. Load combinations used for preliminary design of the underpinning reinforcement and the connection of the underpinning to the auxiliary building

c. Design forces and moments at the design sections

d. Reinforcement provided in the underpinning walls

e. Identification of the areas of potential overstress in the auxiliary building as indicated by the preliminary analysis

The material presented herein is based on preliminary analyses and design for the permanent underpinned configuration of the auxiliary building and the FIVPs. Detailed checking will be performed after final analysis to verify the design adequacy. The results of this detailed check will be provided later in an audit scheduled for May 17, 1982.

The results of the analysis for the construction condition with temporary support piers are not included. This analysis is in progress and the results will be provided later for the audit scheduled January 15, 1982.

2.0 SOIL PRESSURES

2.1 AUXILIARY BUILDING UNDERPINNING

Table 1 and Figure 1 show the magnitude and location of the net soil pressure under the main auxiliary building and underpinning under the control tower and the electrical penetration area. The soil pressures were computed for the following load combination considered to be critical for preliminary analysis.

$$D + L + R + E' + P_L$$

where

D = dead load

L = live load

R = pipe break load

E' = safe shutdown earthquake (SSE) loads corresponding to the ground acceleration given in the Midland FSAR Section 3.7

This load combination corresponds to the 19th load combination in Table 1 of Reference 1 without the thermal loads which are neglected in the preliminary design.

The allowable net bearing pressure is based on the allowable values submitted to the NRC in Subsection 7.2.1 of Reference 1 and Midland FSAR Section 2.5.

2.2 FEEDWATER ISOLATION VALVE PITS

The FIVPs will be supported on engineered sand backfill. A 3-foot thick concrete slab will be provided between the bottom of the pit and the top of the sand, as shown in Figure 2. The sand will be confined between the reactor building, electrical penetration area underpinning wall, turbine building underpinning, and buttress access shaft. The slab at the top of the engineered backfill will be jacked against the existing FIVP base slab. This jacking will minimize any future settlement due to compaction of the engineered backfill from the weight of the FIVP. After jacking, the space between the 3-foot slab and the bottom of the pit will be filled with concrete grout. The maximum bearing pressures on the engineered backfill are shown in Table 2.

The soil pressures (shown in Table 2) were computed for the following critical load combination considered in the preliminary analysis:

$$D + L + E' + P_L$$

This load combination corresponds to the 19th load combination in Table 1 of Reference 1 without the thermal loads which are neglected in the preliminary design.

3.0 UNDERPINNING WALL DESIGN

3.1 LOADS

The preliminary wall design is based on the following loads and load combinations:

a. $U = 1.4D + 1.7L + P_L$ (corresponds to the fifth case in Table 1 of Reference 1)

b. $U = D + L + R + 1.5E' + P_L$

For the above load combinations, the following loads have been considered:

a. Dead load - Includes soil pressure loads.

b. Jacking load applied as uniform load along the length of the underpinning

c. Live load

d. Seismic loads

e. Pipe break loads

3.2 UNDERPINNING BELOW THE ELECTRICAL PENETRATION AREA

The underpinning wall under the electrical penetration areas will carry the vertical loads which will be transferred to clay till at el 571'. The walls will also carry lateral loads due to seismic forces, soil pressure, and surcharge from the turbine building. These lateral loads will be resisted by the engineered sand backfill placed between the underpinning wall and the reactor building, as shown in Figure 4, and the friction between the concrete wall and the soil underneath (clay till). The net lateral loads in the second load combination exceed the available friction between the wall and soil. For this reason, an 11-foot wide, horizontal beam has been provided to resist the bending due to the net lateral loads (Figure 4).

The critical section for the wall is near column lines 5.3 and 7.8 (see Figure 3). The design forces are shown in Table 3 and reinforcement is presented in Figure 3.

3.3 UNDERPINNING BELOW THE CONTROL TOWER

The underpinning wall will be embedded in natural clay till between elevations 571 and 562, and will be restrained by a new slab at el 583'-6" to be constructed as shown in Figure 4. The space between el 571' and the slab at el 583' ;" will be backfilled with engineered granular material. Part of the lateral loads will be resisted by the clay till between elevations 571 and 562, and the balance will be transferred to the main building by the slab at el 583'-6".

The critical section for the wall is at column line 7.8. The location of the critical sections and reinforcement are presented in Figure 3. Design loads at the critical section are presented in Table 3.

4.0 STABILITY

The factors of safety against sliding and overturning are shown in Subsection 3.8.6.3.4 of the Midland FSAR (Reference 2). In the underpinned condition, the overall safety factors against sliding and overturning are expected to reduce or remain unchanged from the values shown in the Midland FSAR.

5.0 CONNECTION DETAIL

The connection of the underpinning to the auxiliary building will be designed to transfer shear and tension resulting from the seismic lateral loads and other concurrent loads. The design loads are presented in Table 3. The type and arrangement of dowels required for the connection are being finalized and will be provided during the structural audits.

At first, the dowels will be grouted only on one side, either at the building or the underpinning. The other side will be grouted only after jacking loads are applied and held. To achieve this for the horizontal dowels, the end portion of the underpinning wall will be poured after jacking loads are applied and held long enough for the till to be within secondary compression.

6.0 EXISTING STRUCTURE

Based on a preliminary analysis, the following areas between column lines G and H appear to be overstressed:

a. Slab at el 659'

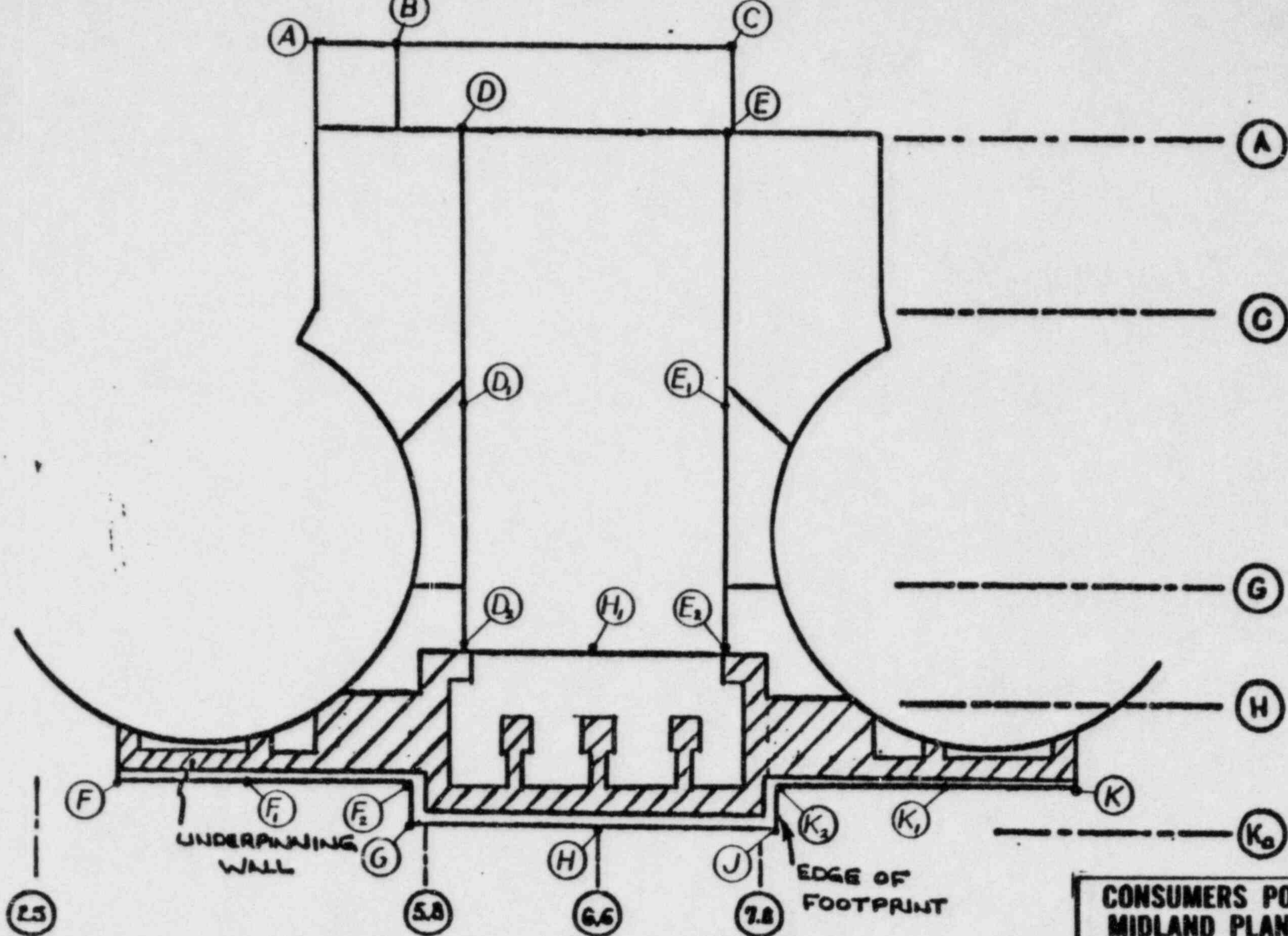
b. Shear walls on column lines 5.6 and 7.8 between elevations 584' and 614'

c. West staircase wall on column line 5.3 between elevations 646' and 685'

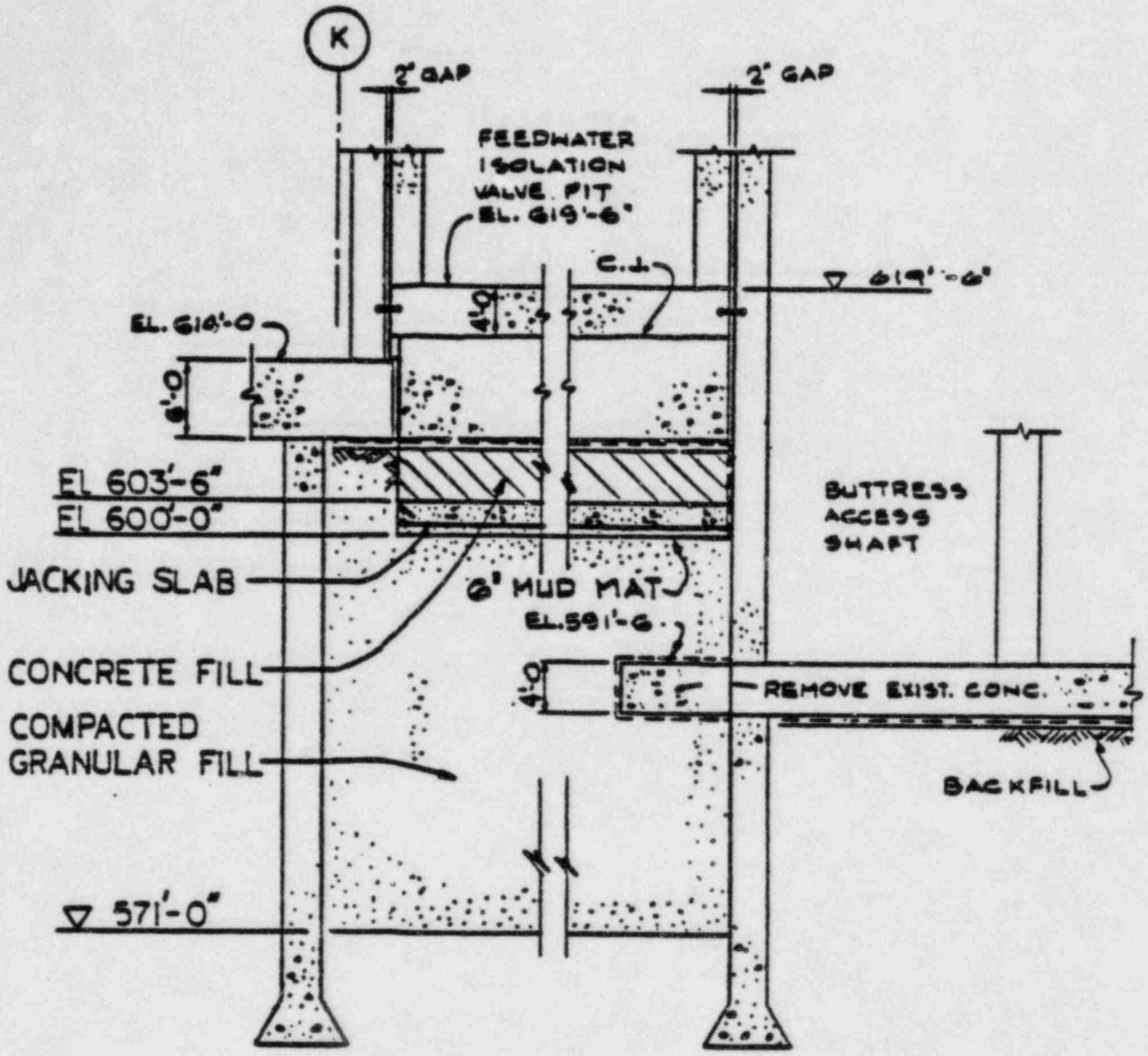
d. Walls on column lines 5.8 and 7.2 from elevations 659' to 699'

The above mentioned areas will be structurally upgraded to withstand all loads including 1.5 x E' if the more rigorous final analysis still indicates that these areas are overstressed.

AUXILIARY BUILDING UNDERPINNING



CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 & 2
SOIL PRESSURE DATA POINTS
FIGURE 1



F.I.V.P. SUPPORT DETAIL

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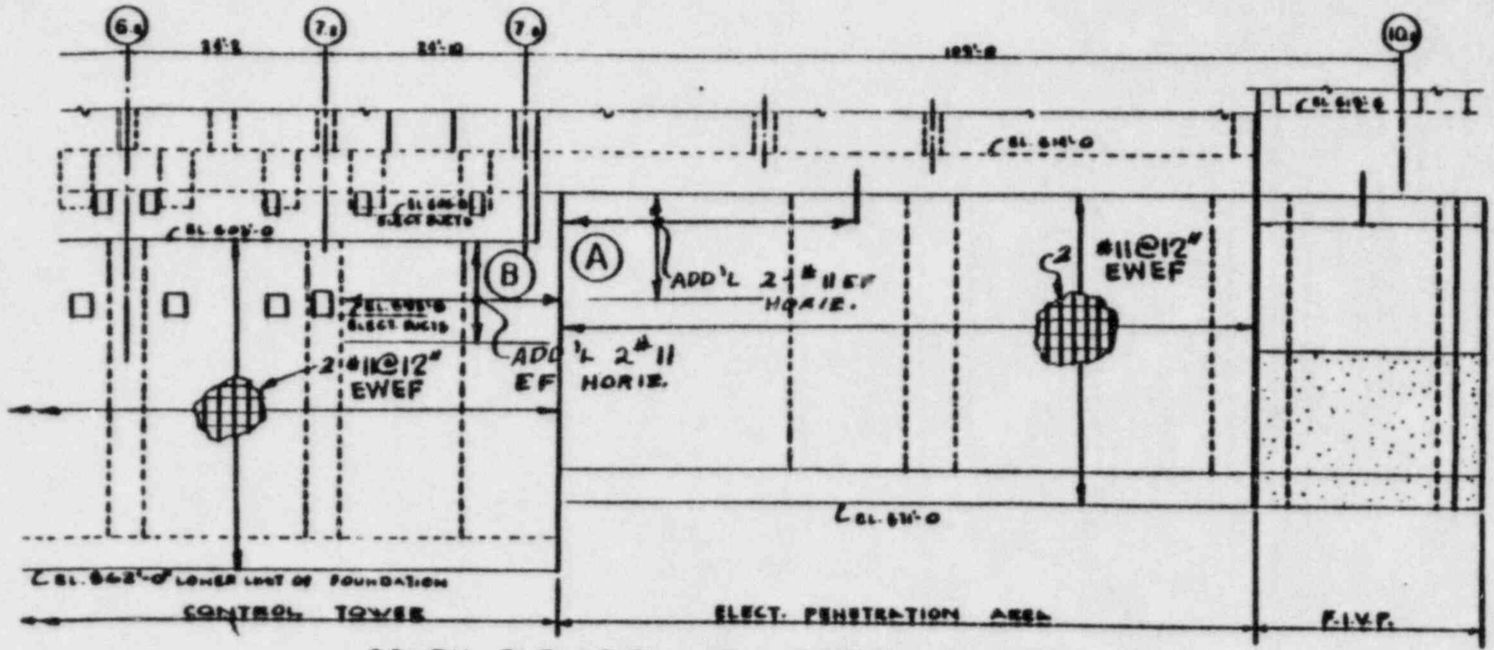
FEEDWATER ISOLATION
VALVE PIT
SUPPORT DETAIL

FIGURE 2



SOUTH ELEVATION OF UNDERPINNING WALL

CRITICAL SECTION FOR WALL DESIGN

(A) ELECTRICAL PENETRATION WING

(B) CONTROL TOWER

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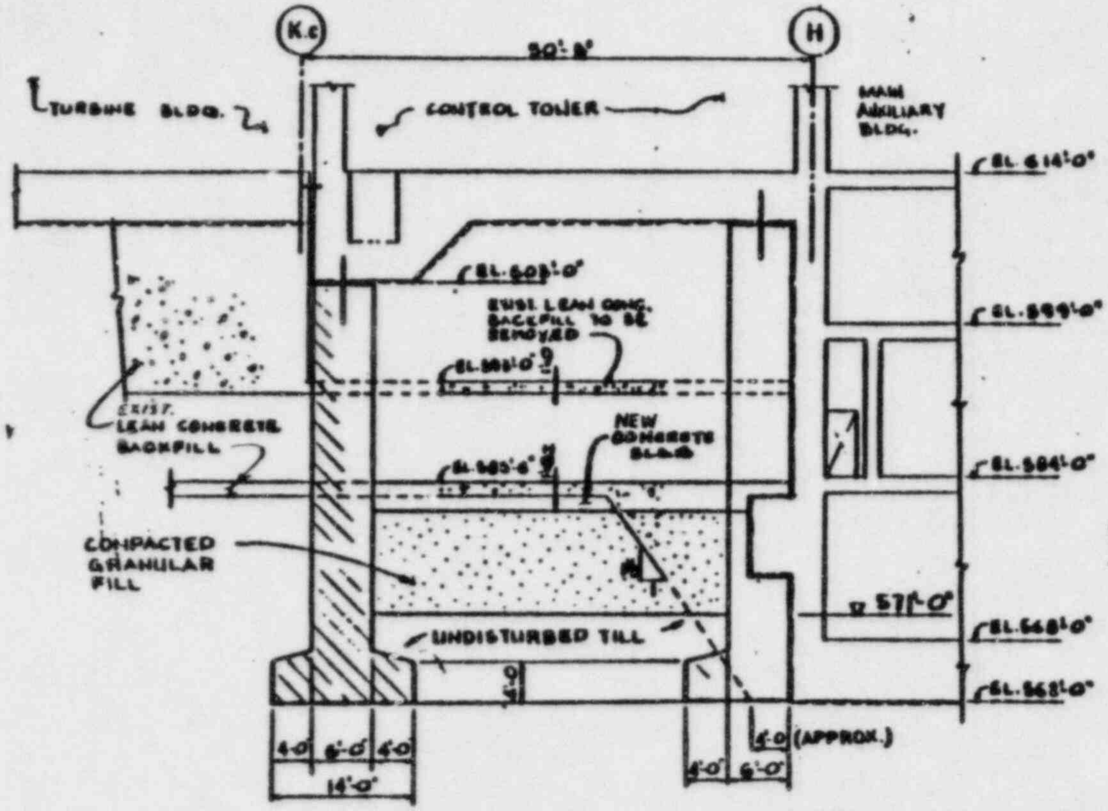
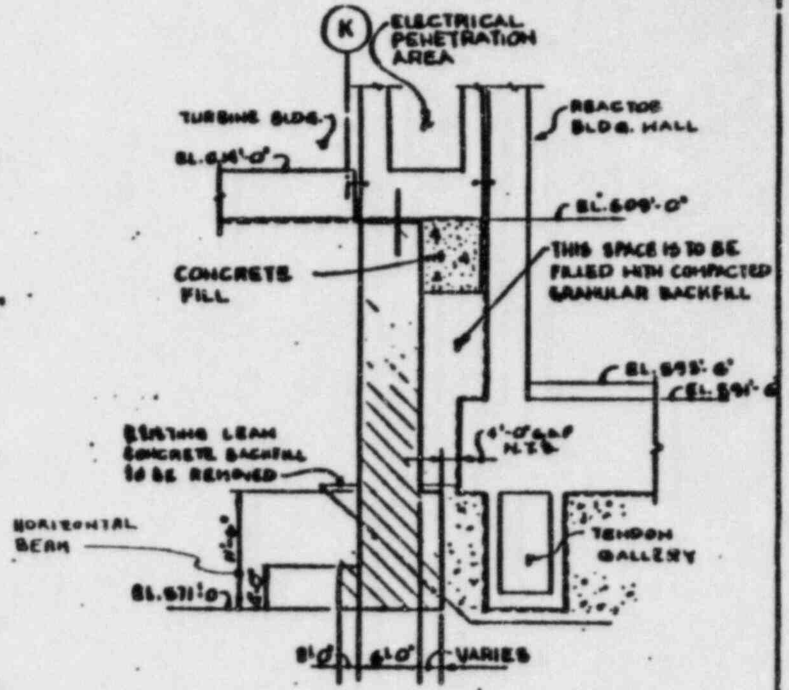
Auxilliary Building Underpinning Wall Reinforcing Detail

Figure 3



WALL UNDER CONTROL TOWER



WALL UNDER ELECTRICAL PENETRATION AREA

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UNDERPINNING WALL DETAILS

FIGURE 4

NET SOIL PRESSURE (KSF)					ULT. NET BEARING CAPACITY (KSF)
POINT (SEE FIG.)	EL.	D+L+R+E'+P_L		D+L+R+P_L	
		CASE 1	CASE 2		
A	609'-0	-3.4	0.8	-1.3	30
B	609'-0	-2.4	-0.3	-1.3	30
C	630'-6"	1.2	-3.7	-1.2	15
D	562'-0	-7.1	-5.3	-6.2	44
E	562'-0	-7.9	-3.9	-5.9	44
D1	562'-0	-6.5	-2.3	-4.4	44
E1	562'-0	-2.9	-5.3	-4.1	44
D2	562'-0	-10.2	-3.0	-6.6	44
E2	562'-0	-5.8	-6.8	-6.3	44
F	571'-0	-18.2	1.6(-3.0)	-8.3	44
F1	571'-0	-15.3	-0.7	-8.0	44
F2	571'-0	-12.8	-2.8	-7.8	44
G	562'-0	-15.3	-4.7	-10.0	44
H	562'-0	-12.7	-7.3	-10.0	44
H1	562'-0	-7.6	-5.0	-6.3	44
J	562'-0	-9.9	-9.9	-9.9	44
K	571'-0	-2.5	-13.5	-8.0	44
K1	571'-0	-5.2	-10.4	-7.8	44
K2	571'-0	-7.5	-7.9	-7.7	44

1. Case 1 corresponds to maximum compression @ PT. F
2. Case 2 corresponds to minimum compression @ PT. F
3. Gross soil pressure is given in parenthesis
4. Compression is negative

Note: Net pressure is total pressure minus the pressure due to the removed soil

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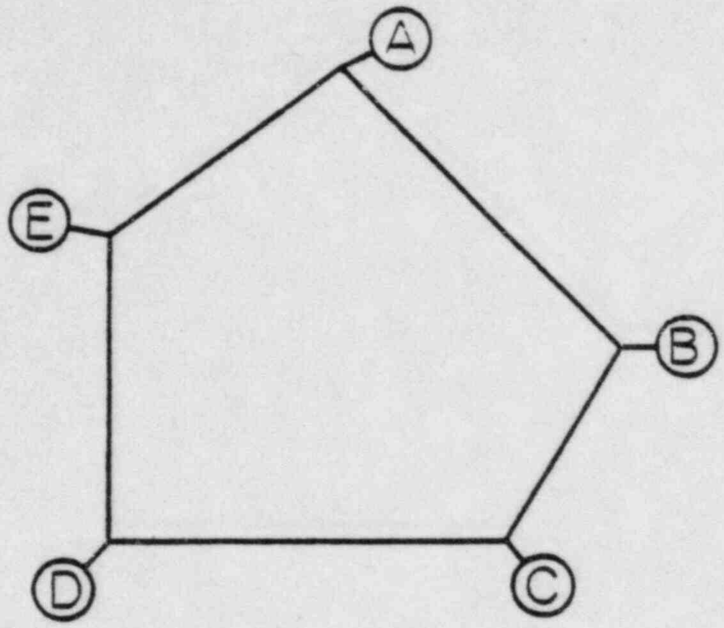
AUX BLDG UNDERPINNING
SOIL PRESSURE

TABLE-1



SOIL PRESSURE (KSF)

	D+L+E′		D+L
POINT	CASE 1	CASE 2	CASE 3
A	2.54	2.96	-3.07
B	-7.16	-6.52	-4.68
C	-10.83	-10.12	-5.27
D	-7.41	-6.78	-4.68
E	0.39	0.85	-3.40

1) CASE 1 CORRESPONDS TO MAX. COMPRESSION

2) CASE 2 CORRESPONDS TO MIN. COMPRESSION

3) COMPRESSION IS NEGATIVE

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

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FIVP
SOIL PRESSURES

TABLE-2

UNDERPINNING WALLS

LOCATION (SEE FIG. 3)	LOAD COMB.	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.	1	358	-387	± 816	22.6	±278
A HORIZ. SECT.	1	-48.5	-27.4	± 968	22.6	±318
B VERT. SECT.	1	278	370	± 969	-29.8	±358
B HORIZ. SECT.	1	-122.	30.1	±1100	-29.8	±318

INTERFACES (Load Comb. 2)

LOCATION	INTERFACE	AXIAL K/FT	SHEAR K/FT	SHEAR CAP. K/FT
A (FIG. 3)	HORIZ	15.7	117	*
E_2 (FIG. 1)	VERT	12.7	79.7	*

LOAD COMBINATIONS:

1. $U = 1.4D + 1.7L + P_L$
2. $U = D + L + R + 1.5E' + P_L$

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 21.3k/ft WHILE THE CAPACITY IS 94k/ft

*THE TYPE AND SPACING OF DOWELS WILL BE FINALIZED

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Aux. Bldg. Underpinning

Design Loads

Table 3