Consumers Power Company

J A Mooney
Executive Manager
Midland Project Office

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0774

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Mr J J Harrison
Midland Project Section
U S Nuclear Regulatory Commission
Region III
799 Roosevelt Road
Glen Ellyn, IL 60137

Subject: Midland Energy Center GWO7020
Auxiliary Building Underpinning
NRC Audit of September 14-15, 1983
and Subsequent Discussions
File: 0485.16 UFI: 42*05*22*04 Serial: CSC-6960
12*32

This letter summarizes the discussions during the subject audit. It also includes the applicants' responses to the open items resulting from the subject audit and the subsequent discussions.

Audit

During the NRC audit of September 14-15, 1983, the capacity of the Auxiliary Building for a soil modulus of 1500 ksf and differential settlement of one-half inch was reviewed and it was concluded that the building is structurally adequate.

During this audit, presentations were made and exhibits provided to the NRC. These exhibits are included as Attachment 1. Also, updated settlement plots of the Diesel Generator Building were provided and are included as Attachment 2.

The NRC also reviewed the design and details of the slab fix at Elevation 659 feet. Consumers will provide the final drawings of this fix as a work package to NRC Region III prior to implementation of this work.

Included in the audit were four additional points of discussion. These points and their responses are listed below.

1. Building stresses after lock-off of the permanent wall with regard to residual stresses and upward building movements during underpinning.

   Response: Attachment 3 provides response and concludes that the assumptions made, regarding existing stress, in the analytical models are justified and the calculated stresses resulting from these models are reasonable.

2. Request for an alteration to the soil consolidation acceptance criteria for the permanent underpinning wall included in our letter of June 9, 1983.

   Response: This request is withdrawn, the criteria will be as referenced in SSER Section 3.8.3.1, Pages 3-9.

3. Results of a local stress analysis of the EPA/Control Tower connection at Elevation 704.

   Response: The connection at Elevation 704 is being reviewed. The results of this review will be submitted to the NRC before removal of the temporary prestressing strands in the EPA.

4. Long term settlement values as defined in the previously submitted Technical Specifications.

   Response: These values are being reviewed and if necessary revised values will be submitted to the NRC by revision to the Technical Specifications.

Subsequent Discussion

1. Approximately how much upward movement of the existing structure (EPA and Control Tower) will be allowed during jacking operations?

2. How was the value (and conditions related to value) in Answer No. 1 determined?

   Response to Questions 1 and 2 is provided in Attachment 4 wherein it is concluded that the structure will be allowed to move upward as necessary to accommodate the design jacking loads during temporary underpinning for EPA and the initial support piers for the Control Tower.

3. In what sequence will the remaining underpinning and associated jacking work be performed?

   Response: The sequence for jacking (temporary and permanent) is consistent with the SSER (Appendix I) except that during the initial jacking of Control Tower piers, CT 3/10 will be completed prior to CT 2/11. This information was provided to the NRC in the March 7-8, 1983, telephone conversation regarding access from the UAT.

4. When initial jacking of an independent pier or pier/grillage system is performed, what evaluations are made if AUM occurs?

   Response: Attachment 5 provides this response and shows that an adequate evaluation of the structure is performed prior to proceeding with further jacking.

5. Provide an explanation for jacking 160% of the specified load into the grillage at 8, as the reserve capacity load.

   Response: Sometime after jacking grillage at Pier 8, excavation for the grillage at Pier 5 will be performed. The loss of building support due to this excavation can result in additional load being transferred passively to the grillage at 8. This additional load can cause additional building movement due to pier settlement, grillage deflection, etc. In order to minimize this building movement, a reserve capacity load (RCL) in increments of 5% will be jacked into the grillage at 8 prior to excavation for grillage at 5. The load which is based on estimated loss of building support at 5 has been calculated to result in an increase in the load of 50% of the specified load (S.L.) at grillage 8. The S.L. is the design force defined in Paragraph 6.3.4b of Specification 7220-C-195. The building has been checked for, and found to be adequate, for 160% S.L. i.e., the total load in grillage at 8 when the grillage 5 area is undermined.

   Similarly a RCL will be jacked into the grillage at 5 before excavation for the grillage at 2. At this time the load at the grillage 8 will be maintained at 160% S.L. While loading the grillage at 2, the loads at grillages 5 and 8 are reduced to the S.L.

6. For grillage jacking at Pier 8, why was the 24 hour acceptance criteria changed to 125% of specified load instead of 110% of specified load.

   Response: Since it is planned to go to RCL, which is higher than 110% S.L., it was considered more conservative and prudent to satisfy the 24 hour acceptance criteria at 125% S.L., instead of reducing the load to 110% S.L. The 24 hour criteria will be again met when the RCL is jacked.

JAMooney

J A Mooney
Executive Manager
Midland Project Office

JAM/nj

MEC/D

Rev 4 SWPS Incl

	ACTION	INFORMATION
J W Cook, P26-336B		
R A Wells, MPQAD		
A J Boos, Bechtel Ann Arbor		
J A Mooney, P14-115A		
J E Brunner, M-1079		
J R Schaub, P14-305		
R J Cook, Midland Resident Inspector		
D S Hood, USNRC Project Manager Department of Licensing USNRC Washington, DC 20555		
J K Meisenheimer, MPQAD Soils		
Hearings File, P-24-517		
B J Walraven, P-24-517		
NRC Correspondence File, P-24-517		
D L Quamme Midland		
A R Mollenhopf, P14-408A		
D H Lavelle, FSO		
T A Buczwinski, Admin 207		
J N Leech, P24-507A		
N J Saari, Midland		
D F Lewis, Bechtel Ann Arbor		
D J Vandewalle, P-24614B		
Mr. Mike Miller Isham, Lincoln & Beale 3 First National Plaza, Suite #5100 Chicago, IL 60602		
F C Williams Isham, Lincoln & Beale 1120 Connecticut Avenue N.W. Washington, D.C. 20036		
R M Wheeler, Midland		
A E Blocher, Midland		
T R Thiruvengadam, P14-400		
Neil Swanberg, Bechtel Ann Arbor		
Mr. Ron Callen Michigan Public Service Commission 6545 Mercantile Way Lansing, MI 48909		
NRC Correspondence Book		
File: 0485.16		
UFI: 42*05*22*04		
Serial File		
Chronilogical File, P14-403 (2)		

LS0883-0002A-CN01

ATTACHMENT #1

AUDIT EXHIBITS

SUMMARY OF SOILS DATA FOR AUXILIARY BUILDING UNDERPINNING ANALYSES

	EPA				CONTROL TOWER				MAIN AUX.		
Case	E (KSF)	Total Settl. (IN)	After Lockoff Settl. (IN)	Unit Soil Spring (KCF)	E (KSF)	Total Settl. (IN)	After Lockoff Settl. (IN)	Unit Soil Spring (KCF)	After Lockoff Settl. (IN)	Unit Soil Spring (KCF)	Comments
I	3000	0.6	0.2	410	3000	0.9	0.3	350	0.1	1160	Based on Bechtel Testimony
II	1333	1.35	0.45	180	2000	1.35	0.45	240	0.2	580	NRC
III	857	2.1	0.7	128	1286	2.1	0.7	175	0.2	580	0.5 inch differential

AUXILIARY BUILDING UNDERPINNING
DESIGN CRITERIA

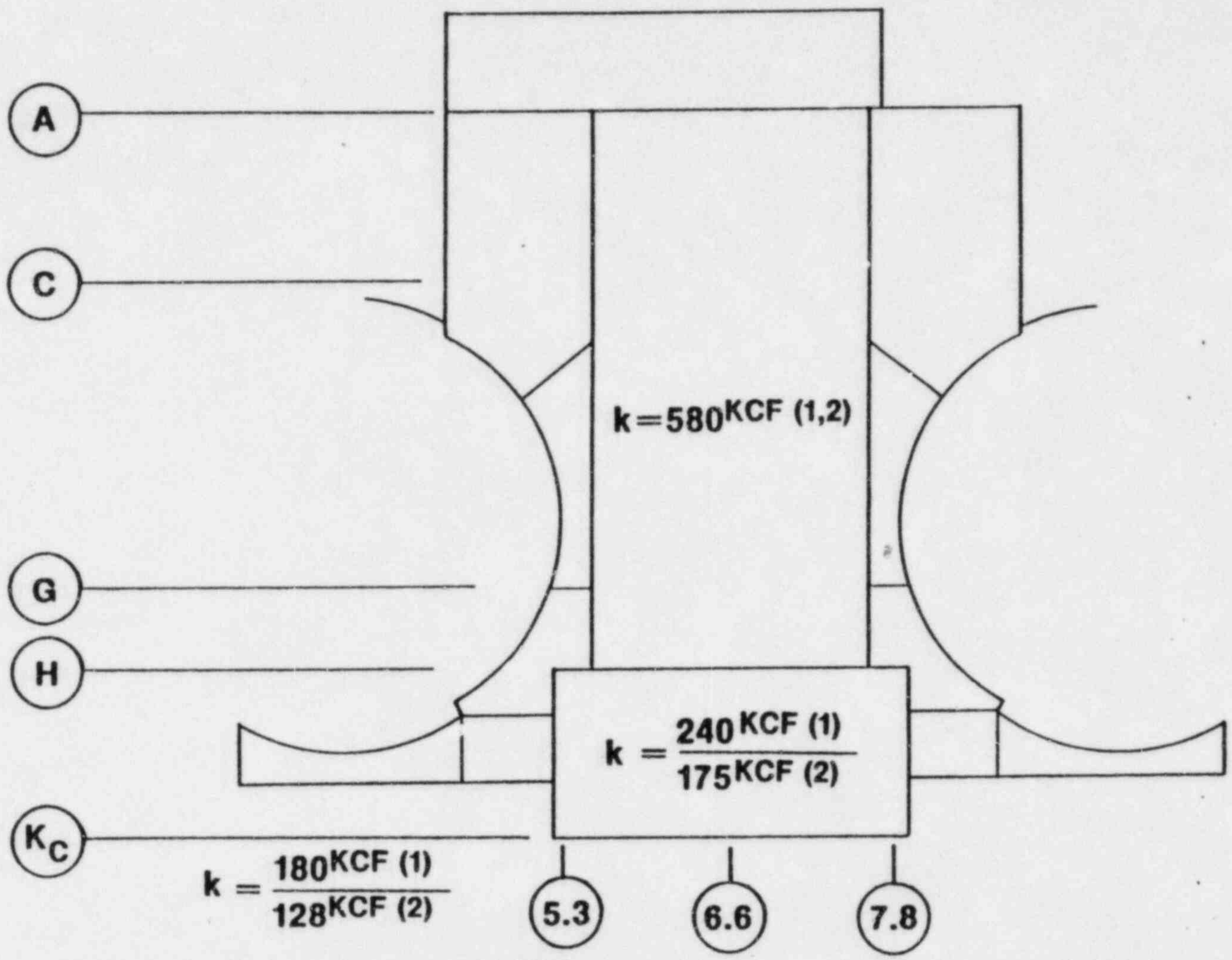
- EXISTING STRUCTURE EXCLUDING UNDERPINNING WALL + CONNECTIONS
  - Designed in accordance with Subsections 3.8.6.3.1 through 3.8.6.3.3 of FSAR (ACI 318-71, including settlement effects)
  - Some loading combinations include settlement effects; others do not
- UNDERPINNING WALL + CONNECTIONS
  - Designed in accordance with Subsection 3.8.6.3.5 (ACI 349.80)
  - All load combinations have settlement effects

34-G-3067-03

AUXILIARY BUILDING UNDERPINNING
SETTLEMENT ANALYSIS

- Used same methodology as before
- Revised soil springs and added settlement stresses to other stresses in accordance with FSAR

AUXILIARY BUILDING UNDERPINNING
SOIL SPRINGS UNDER AUXILIARY BUILDING



NOTE:

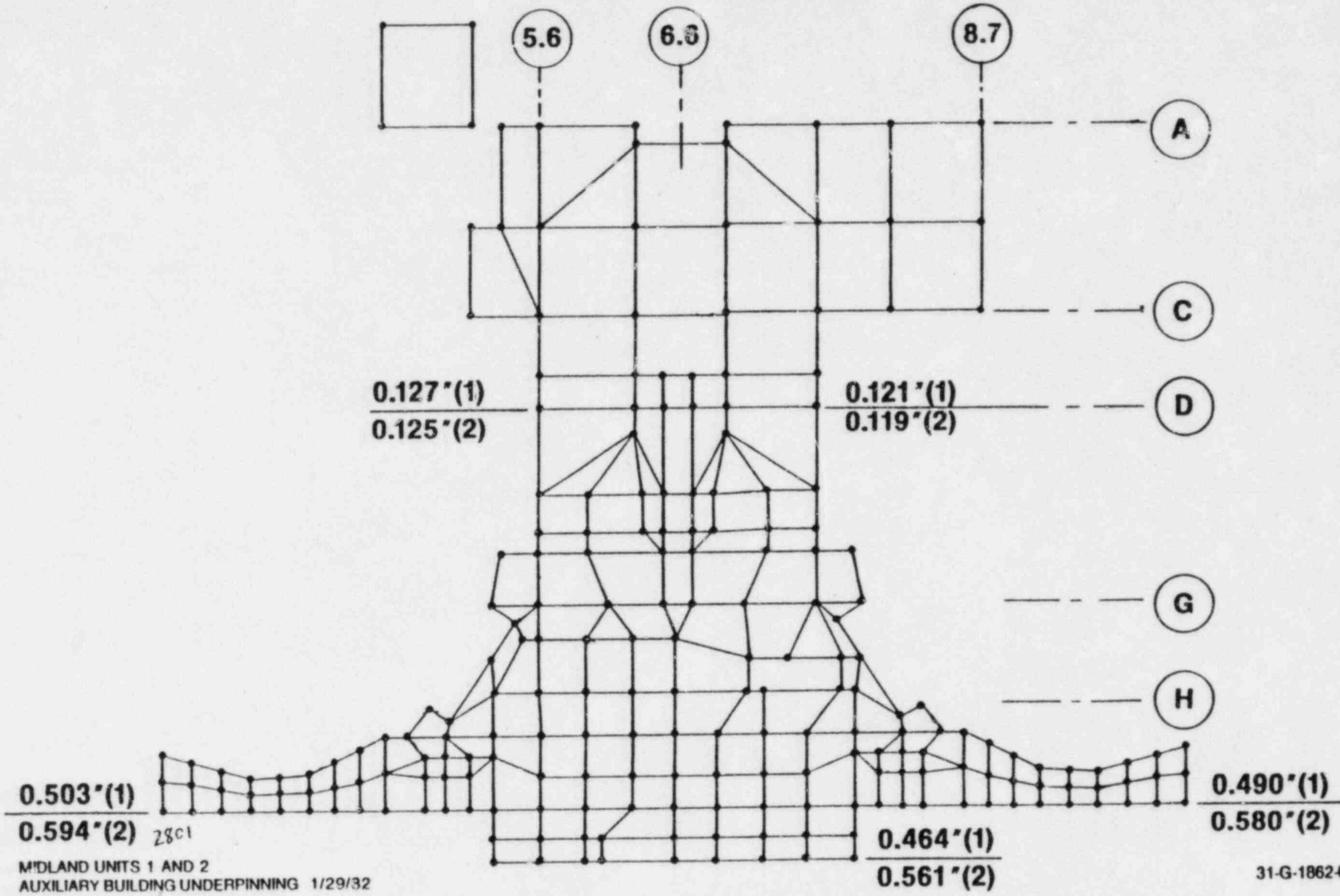
(1) for lower differential settlement
(2) for higher differential settlement

MIDLAND UNITS 1 AND 2
AUXILIARY BUILDING UNDERPINNING 1/26/82

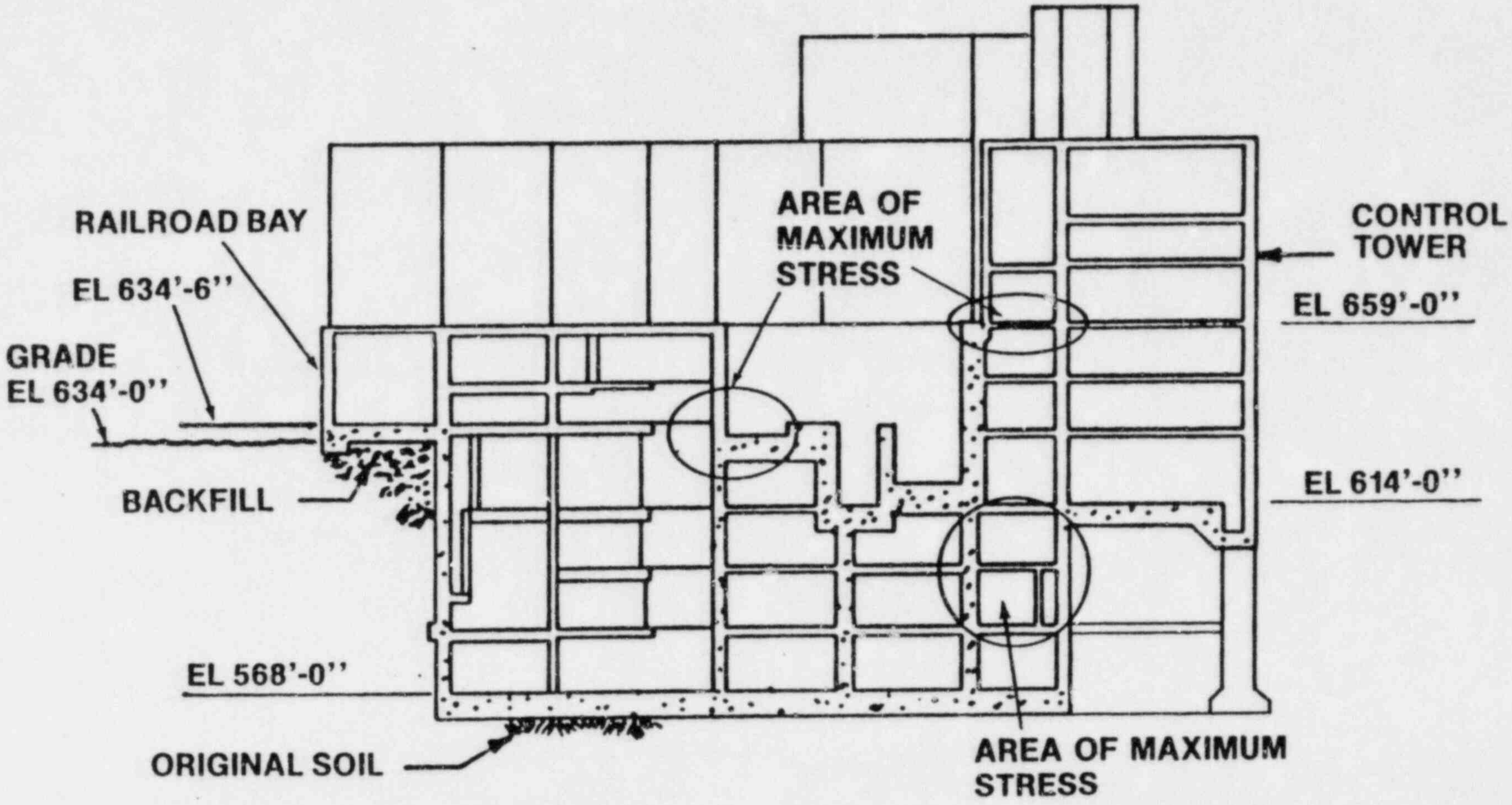
31-G-1862-24

AUXILIARY BUILDING UNDERPINNING

NODAL MESH AT ELEVATION 614'
PLAN VIEW



AUXILIARY BUILDING UNDERPINNING
TYPICAL SECTION
LOCATION OF MAXIMUM STRESS
(Looking East)



Page 6 of 22

G-1555-07

AUXILIARY BUILDING UNDERPINNING
REVIEW OF CRITICAL AREAS

DESCRIPTION	STRESS/LOAD			
	For Lower Diff Settlemt	For Higher Diff Settlemt	Other Load Combin	Capacity of Section
Slab at El 659′ between column lines G and H	3,480^{K}	3,850^{K}	5,900^{K} 7,150^{K} *	6,230^{K}
N-S walls on column lines 5.3 and 7.8 below El 614′	19.1^{KSI}	24.5^{KSI}	42.8^{KSI} 59.2 " *	54^{KSI}
Slab at El 634′-6 between column lines C and F and 5.6 and 6.2	41.4^{KSI}	48.1^{KSI}	42.2^{KSI}	54.0^{KSI}
Slab at El 659′ between column lines 4.7 and 5.6 and D and G	47.5^{KSI}	50.0^{KSI}	37.3^{KSI}	54.0^{KSI}

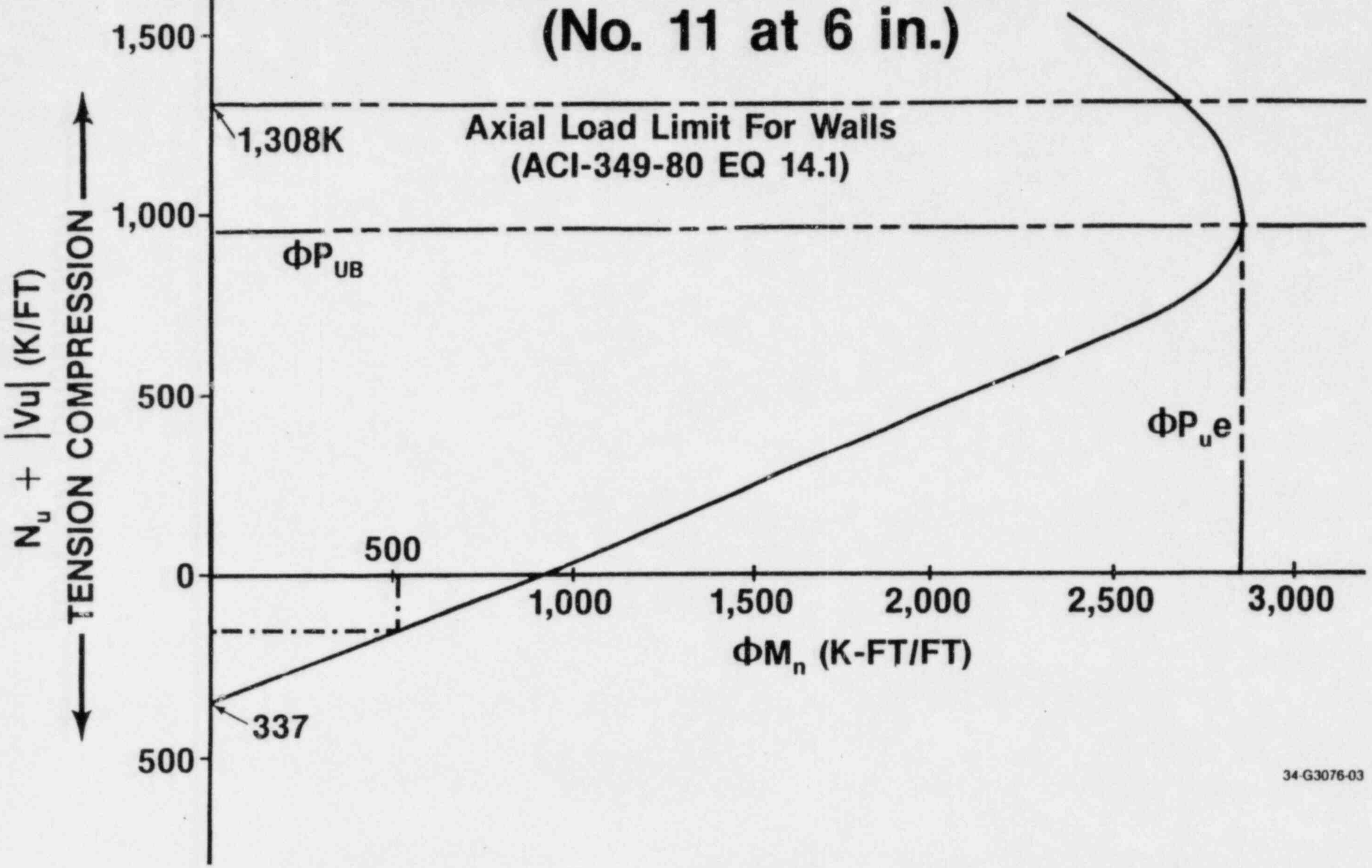
31-G-3067-01

* VALUES FOR ACI 349-80 LOAD COMBINATIONS WITH HIGHER ASSUMED DIFFERENTIAL SETTLEMENT (FOR INFORMATION ONLY). THESE CORRESPOND TO MIDLAND FSAR RESPONSE SPECTRA.

AUXILIARY BUILDING UNDERPINNING

INTERACTION DIAGRAM FOR HORIZONTAL REBAR ON COLUMN LINE 5.3 AND 7.8

(No. 11 at 6 in.)



AUXILIARY BUILDING UNDERPINNING

UNDERPINNING WALL DESIGN CRITICAL LOADS

NORTH-SOUTH WALLS IN EPA AND CONTROL TOWER									
LOCATION	ELEVATION	HORIZONTAL REBAR				VERTICAL REBAR			
		N_u (K/FT)	V_u (K/FT)	M_u (K-FT/FT)	ϕM_n (K-FT/FT)	N_u (K/FT)	V_u (K/FT)	M_u (K-FT/FT)	$\phi M_{u/n}$ (K-FT/FT)
Just North of Column Line K_c on Column Line 5.3	Between EL 565 and EL 574′	124	159	211	260	133	159	9.6	−120
Just North of Column Line K_C on Column Line 5.3	Between EL 603′ and EL 614′	112	−231	−51.3	120	−139	−231	−61.6	−650
Just South of Column Line H_K	Between EL 565′ and EL 574′	90.6	57.1	459	510	−54.1	57.1	−16.5	−900

34-G3076-02

AUXILIARY BUILDING UNDERPINNING

UNDERPINNING WALL DESIGN CRITICAL LOADS

WALLS ON COLUMN LINES K and K_c
(E-W EPA AND CONTROL TOWER WALLS)

LOCATION	ELEVATION	HORIZONTAL REBAR				VERTICAL REBAR			
		N_u (K/FT)	V_u (K/FT)	M_u (K-FT/FT)	ϕM_n (K-FT/FT)	N_u (K/FT)	V_u (K/FT)	M_u (K-FT/FT)	$\phi M_{\cancel{u}\,n}$ (K-FT/FT)
Between Column Lines 4.1 and 4.6	Between EL 603′ and EL 614′	75.5	74.6	118	168	47.4	74.6	23.2	180
Just West of Column Lines 5.3	Between EL 603′ and EL 614′	63.6	77.8	24.7	170	9.3	77.8	19.7	190

34-G-3076-01

3.8.6.3 Loads and Loading Combinations

The containment, internal structures, other Seismic Category I structures, and foundations are designed for all credible conditions of loadings, including normal loads, loads resulting from a loss-of-coolant accident, thermal loads, test loads, missile-generated loads, adverse environmental conditions, and loads resulting from a pipe rupture where applicable.

Wind and tornado loads, flood design bases, and seismic loads are given in Sections 3.3, 3.4, and 3.7. Missile effects and the postulated pipe rupture effects are discussed in Sections 3.5 and 3.6.

All the loads postulated in the plant are listed. All loads listed, however, are not necessarily applicable to all the structures and components in the plant. The loads and the applicable load combinations for which each structure is designed depend on the conditions to which that particular structure could be subjected.

Steel structures other than pipe whip restraints were designed by the working stress method. Soil bearing pressure was checked for the actual loads. All reinforced concrete structures were designed by the ultimate strength method except the containment.

The loads used in the design of containment are presented in Subsection 3.8.1.3. The loads used in the design of the remaining Seismic Category I structures are presented in the following subsections.

The design of structures is separated into two parts.

a. The portions of existing structures that were constructed before remedial work

b. The new remedial foundations including their connections to the existing structures

Design of the existing structure is based on the set of load and load combinations specified in Subsection 3.8.6.3.1 through 3.8.6.3.3. Design of remedial work including the connection to the existing structure is based on the load combinations given in Subsection 3.8.6.3.5. 44

The stability of all Category I structures including containments is investigated for the load combinations given in Subsection 3.8.6.3.4.

3.8.6.3.1 Loads and Definition of Terms

The following loads are considered: dead loads, live loads, earthquake loads, pipe rupture loads, thermal loads, wind and tornado loads, hydrostatic loads, differential settlement, and jacking preload effects. | 47 | 44

a. Dead Loads

The dead load includes the weight of the following:

1. Interior framing and slabs including base slabs

2. Walls, roofs, and floors

3. All internal structures including partitions, platforms, hangers, cable trays, and pipes with fluid

4. Electrical conductors and equipment as specified on the drawings supplied by the manufacturers of the equipment and installed within a structure

5. Hydrostatic and soil loads, where applicable

b. Live Loads

The live load includes the weight of the following:

1. The design floor and roof loads

2. Laydown loads

3. Pool and tank liquid loads

4. All vertical loads except dead load

5. Where applicable, lateral pressure of the soil

6. Main piping loads

7. Equipment live loads including fuel handling equipment and load materials

8. All live loads transmitted by internal structures

c. Seismic Loads

Seismic loads for safe shutdown earthquake load and the operating basis earthquake load were considered. A more detailed discussion is presented in Section 3.7.

d. Pipe Rupture Loads

Pipe rupture loads include the jet impingement forces from postulated pipe breaks, differential pressures that might build up across compartments, and loads due to pipe whipping or pipe restraint. Pipe rupture effects are further discussed in Section 3.6.

e. Thermal Loads

Thermal loads include the temperature gradients through the spent fuel pool walls and floor, the primary and secondary shield walls, forces on internal structures due to the thermal expansion and contraction of the liner plate, piping, and equipment, including increases in water temperature during operating and accident conditions.

f. Wind and Tornado Loads

Wind and tornado loads were considered and are discussed in detail in Section 3.3. Tornado missile effects are discussed in Subsection 3.5.3.

All structures whose failure could endanger Seismic Category I structures, systems, or equipment, are designed to withstand the effects of the wind and tornado loadings and to provide protection of Seismic Category I systems and components from tornado missiles.

The structures are analyzed for tornado loading not coincident with the safe shutdown earthquake.

g. Hydrostatic Loads

Lateral hydrostatic pressure loads and buoyant forces resulting from the displacement of groundwater or probable maximum flood (PMF) have been applied to the structures and are accounted for in the design and discussed further in Section 2.4.

h. Jacking Preload

The design considers the effects of jacking loads in the existing structure and the underpinning wall. 44

The following variables are used in the loading combination equations:

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

For the ultimate load capacity of a concrete section:

U is calculated in accordance with ACI 318-63 Part IV-8 for design calculations initiated prior to February 1, 1973

U is calculated in accordance with ACI 318-71 for design calculations initiated after February 1, 1973

F_y = Specified minimum yield strength for structural steel

f_s = Allowable stress for structural steel; f_s is calculated in accordance with the AISC Code, 1963 Edition for design calculations initiated prior to February 1, 1973. f is calculated in accordance with the AISC Code, 1969 Edition, with Supplements 1, 2, and 3 for design calculations initiated after February 1, 1973. | 33

D = Dead loads

P_L = Effects of jacking preload on structure | 44

L = Live loads

M = Loads due to hydrotest fluids

R = Local force or pressure on structure or penetration caused by rupture of any one pipe

T = Effects of differential settlement | 44

T_O = Thermal effects during normal operating conditions

H_O = Force on structure due to thermal expansion of pipes under operating conditions | 44

T_A = Total thermal effects which may occur during a design accident other than H_A | 44

H_A = Force on structure due to thermal expansion of pipes under accident condition | 44

E = Operating basis earthquake (OBE)

E' = Safe shutdown earthquake load (SSE)

B = Hydrostatic forces due to the PMF elevation of 635.5 feet

W = Design wind load

W' = Tornado wind loads, including missile effects and differential pressure

A cross reference of terminology used in SRP 3.8.4 and those listed above are presented in Table 3.8-26.

Ø = Capacity reduction factor

The capacity reduction factor (Ø) provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, while individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity.

In the load equations, the following factors are used:

Ø = 0.90 for reinforced concrete in flexure

Ø = 0.85 for tension, shear, bond, and anchorage in reinforced concrete, applicable only for calculations in accordance with ACI 318-63

Ø = 0.75 for spirally reinforced concrete compression members

Ø = 0.70 for tied compression members

Ø = 0.90 for fabricated structural steel

Ø = 0.90 for reinforced steel in direct tension

Ø = 0.85 for lap splices for reinforcing steel, applicable only for calculations in accordance with ACI 318-63

Ø = 0.90 for welded or mechanical splices of reinforcing steel

3.8.6.3.2 Loading Under Normal Conditions

For loads encountered during normal plant operation, the design is based on referenced codes and standards.

a. Concrete

Reinforced concrete structures are designed for ductile behavior, that is, with steel stresses controlling.

Design of concrete structures satisfies the most severe loading combinations, based on the load factors shown below:

1) $U = 1.5D + 1.8L$ - applicable to calculations started before February 1, 1973

$U = 1.4D + 1.7L + 1.0P_L$ - applicable to calculations started after February 1, 1973 |44

2) $U = 1.4 (D + L + M)$ |41

3) $U = 1.25 (D + L + H_O + E) + 1.0 T_O + 1.0P_L$

4) $U = 1.25 (D + L + H_O + W) + 1.0 T + 1.0P_L$

5) $U = 0.9 D + 1.25 (H_O + E) + 1.0 T + 1.0P_L$ 44

6) $U = 0.9 D + 1.25 (H_O + W) + 1.0 T + 1.0P_L$

In addition, for ductile moment resisting concrete frames and for shear walls: 41

7) $U = 1.4 (D + L + E) + 1.0 T + 1.25 H_O + 1.0P_L$

8) $U = 0.9 D + 1.25 E + 1.0 T + 1.25 H_O + 1.0P_L$

For structures which include settlement effects:

9) $U = 1.05D + 1.28L + 1.05T + 1.0P_L$

10) $U = 1.4D + 1.4T + 1.0P_L$ 44

11) $U = 1.0D + 1.0L + 1.0W + 1.0T + 1.0P_L$

12) $U = 1.0D + 1.0L + 1.0E + 1.0T + 1.0P_L$

For structural elements carrying mainly earthquake forces, such as equipment supports:

13) $U = 1.0 D + 1.0 L + 1.8 E + 1.0 T_O + 1.25 H_O + 1.0P_L$

b. Structural Steel

Design of steel structures satisfies the following loading combinations without exceeding the specified stresses:

1) $D + L + P_L$ 44
..................stress limit = f_s

2) $D + L + T_O + H_O + E + P_L$ 44
..................stress limit = $1.25f_s$

3) $D + L + T_O + H_O + W + P_L$ 44
..................stress limit = $1.33f_s$

4) $D + L + M$
..................stress limit = $1.33f_s$ 44

In addition, for structural elements carrying mainly earthquake forces, such as struts and bracing:

5) $D + L + T_O + H_O + E + P_L$ 44
..................stress limit = f_s

3.8.6.3.3 Loading Under Accident Conditions

The Seismic Category I structures, except as provided in BC-TOP-
9A and BN-TOP-2, are proportioned to maintain elastic behavior
when subjected to various combinations of dead, live, jacking 44
preload, differential settlement, seismic, hydrostatic, thermal,
tornado winds and differential pressure, and sustained accident
pressure loads. The upper limit of elastic behavior is 39
considered to be the yield strength of the effective
load-carrying structural materials. The yield strength F_y for 47
steel (including reinforcing steel) is considered to be the
guaranteed minimum given in appropriate ASTM specifications. The
yield strength for reinforced concrete structures is considered
to be the ultimate resisting capacity as calculated from the
"Ultimate Strength Design" portion of the ACI Code.

The deflections or deformations of structures and supports are evaluated to ensure required functional capabilities are maintained under all postulated loading conditions.

The engineered safeguards systems components are protected by
barriers from all credible missiles which might be generated. 39

a. Concrete

The concrete structures satsify the most severe of the following loading combinations:

1) $U = 1.05\ D + 1.05\ L + 1.25\ E + 1.0\ T_A + 1.0\ H_A + 1.0\ R + 1.0\ P_L$

2) $U = 0.95\ D + 1.25\ E + 1.0\ T_A + 1.0\ H_A + 1.0\ R + 1.0 P_L$

3) $U = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ T_O + 1.25\ H_O + 1.0\ R + 1.0\ P_L$

4) $U = 1.0\ D + 1.0\ L + 1.0\ E' + 1.0\ T_A + 1.0\ H_A + 1.0\ R + 1.0\ P_L$

5) $U = 1.0\ D + 1.0\ L + 1.0\ B + 1.0\ T_O + 1.25\ H_O + 1.0\ P_L$

6) $U = 1.0\ D + 1.0\ L + 1.0\ T_O + 1.25\ H_O + 1.0\ W' + 1.0\ P_L$

b. Structural Steel

Steel structures satisfy the most severe of the following loading combinations without exceeding the specified stresses:

1) $D + L + R + T_O + H_O + E' + P_L$
. ...stress limit[a] = $1.5 f_s$ 41

2) $D + L + R + T_A + H_A + E' + P_L$ 47
...................stress limit[a] $= 1.5f_s$

3) $D + L + B + T_O + H_O + P_L$
...................stress limit[a] $= 1.5f_s$ 41

4) $D + L + T_O + H_O + W' + P_L$
...................stress limit[a] $= 1.5f_s$

[a]For the cases above, the maximum allowable stress, except for local areas affected by missiles, whipping pipes, and jet impingement which do not affect overall stability, is limited to 0.9 F_y for bending, and axial tension or compression when buckling is precluded and 0.5 F_y for shear. Bearing allowables shall be as given in the AISC Specification. 47 39

In the above factored load combinations for steel, accident thermal loads are neglected when it can be shown that they are secondary and self limiting in nature, and that the material is ductile.

Design of energy absorbing steel elements to resist pipe break loads may consider the effects of strain hardening of the material.

The time phasing between loadings is used where applicable to satisfy the above equations.

Structural members subjected to postulated impact effects are designed in accordance with BC-TOP-9-A, Rev. 2. 39

Structural members subjected to missile and pipe break loads are designed in accordance with Bechtel's BC-TOP-9-A, Rev. 2, and Bechtel's BN-TOP-2, Rev. 2. Table 3.8-40 shall be used for ductility ratios. 39

3.8.6.3.4 Other Loadings

In addition to the previous load combinations listed, the structures were checked for overturning, sliding, and flotation utilizing the load combinations and minimum safety factors indicated below:

Load Combination	Minimum Factor of Safety Overturning	Sliding	Flotation
D + H + E	1.5	1.5	---
D + H + W	1.5	1.5	---
D + H + E'	1.1	1.1	---
D + H + W'	1.1	1.1	---
D + B	---	---	1.1

where H is the lateral earth pressure

3.8.6.3.5 Loads and Loading Combinations for the Underpinning Walls

The underpinning walls and piers and their connection with the existing structure are designed using the load combinations of this subsection only. The definitions of loads used especially for these combinations are shown as follows.

Normal loads which are encountered during normal plant operation and shutdown:

D = dead loads or their related internal moments and forces

L = applicable live loads or their related internal moments and forces. Only 25% of the floor design live load (except snow load) will be used in analysis of the building for global effects and under operating conditions. 44

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

P_L = effect of jacking preload

T_0 = thermal effects and loads during normal operating or shut-down conditions

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

T = effects of differential settlement

U = required strength to resist design loads or their related internal moments and forces. U is calculated in accordance with ACI 349-80.

Severe environmental loads which could infrequently be encountered during the plant life:

E_O = loads generated by the operating basis earthquakes

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

Extreme environmental loads are loads which are credible but highly improbable.

E_{ss} = loads generated by 1.5 times the safe shutdown earthquake (as defined in Section 3.7) for underpinning wall design

W_t = loads generated by the design tornado specified for the plant. They include combined loads due to the tornado wind pressure, tornado-created differential pressures, and tornado-generated missiles.

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

P_a = maximum differential pressure load generated by a postulated break 44

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

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

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

The underpinning walls satisfy the most severe of the following loading combinations:

1) $U = 1.4\ (D + T) + 1.4\ F + 1.7\ L + 1.7\ H + 1.7\ R_O + P_L$

2) $U = 1.4\ (D + T) + 1.4\ F + 1.7\ L + 1.7\ H + 1.9\ E + 1.7\ R_O + P_L$

3) $U = 1.4\ (D + T) + 1.4\ F + 1.7\ L + 1.7\ H + 1.7\ W + 1.7\ R_O + P_L$

4) $U = (D + T) + F + L + H + T_O + R_O + E_{ss} + P_L$

5) $U = (D + T) + F + L + H + T_O + R_O + W_t + P_L$

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

7) $U = (D + T) + F + L + H + T_a + R_a + 1.25\ P_a + (Y_r + Y_j + Y_m) + 1.25\ E_O + P_L$

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

9) $U = 1.05\ (D + T) + 1.05\ F + 1.3\ L + 1.3\ H + 1.3\ T_O + 1.3\ R_O + P_L$

10) $U = 1.05\ (D + T) + 1.05\ F + 1.3\ L + 1.3\ H + 1.4\ F_O + 1.3\ T_O + 1.3\ R_O + P_L$

11) $U = 1.05\ (D + T) + 1.05\ F + 1.3\ L + 1.3\ H + 1.3\ W + 1.3\ T_O + 1.3\ R_O + P_L$

3.8.6.4 Design and Analysis Procedures

Design and analysis procedures for the containment including the base slab are discussed in Subsection 3.8.1.4.

For all other Seismic Category I structures including foundations and containment internals, the basic techniques used for analysis and design are the conventional methods used in engineering practice such as the theory of concrete structures or beam theory, and those based on plate and shell theories of different degrees of approximation.

These are discussed in more detail in Subsections 3.8.3.4, 3.8.4.4, and 3.8.5.4.

The seismic analysis of these structures is covered in Section 3.7. The structures are proportioned to withstand the forces from all postulated loadings. 44

3.8.6.5 Structural Acceptance Criteria

The fundamental acceptance criterion for the containment is the successful completion of the structural integrity test, with measured responses within the limits predicted by analyses. The limits are predicted based on test load analyses, test load combinations, and code allowance values for stress, properties, and construction tolerances as described in Subsection 3.8.1. In this way, the margins of safety associated with the design and construction of the containment are, as a minimum, the accepted margins associated with nationally recognized codes of practice.

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	$\boxed{40}$ 19.4*	$\boxed{44}$	39	37	27	48	26	40	54 ksi Allowable
Slab At El 659' Between Column Lines G and H	$\boxed{15}$ 14.3*	$\boxed{17}$	13	12	0*	23	0*	20	54 ksi Allowable

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

K-70

TABLE 2-4

* THESE VALUES CORRESPOND TO A SOIL MODULUS OF 30 KCF UNDER THE MAIN AUXILIARY BUILDING & ARE AVERAGE STRESS VALUES.

G-2513-13

ATTACHMENT #2

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RESPONSE TO AUDIT POINT #1

1 INTRODUCTION

This attachment addresses two points:

a. How the "existing stress" in the Auxiliary Building structure has been considered in the design.

b. Plan for recovering building movements (elastic displacement).

Section 2 describes the causes and the nature of the existing stresses and the cause for the elastic displacement in the structure before the start of underpinning.

At the end of permanent jacking, theses existing stresses are mostly neutralized as explained in Section 3.

Section 4 explains how the existing stresses in the structure were used to check the structure during the different stages of underpinning construction.

Section 5 describes how the elastic displacement is recovered in the process of applying the temporary and subsequent permanent jacking.

Conclusions are provided in Section 6.

2 EXISTING CONDITION OF BUILDING BEFORE UNDERPINNING

A. EXISTING STRESSES

The Main Auxiliary (MA) building is supported on original soil. Before the start of underpinning, the electrical penetration areas (EPA's) and Control Tower (CT) are supported on backfill (shown by cross-hatching in Figure 1). The estimated weights are 9,300Kips for each of the EPA's and 29,000Kips for the CT. These weights include the dead weight of the structure and 25 percent live load on the structure (hereafter referred to as live load).

The soil supporting the MA is stiffer than the backfill supporting the EPA's and the CT. The Auxiliary Building structure has been analyzed for the existing condition considering relative stiffnesses of supporting soil under the MA, EPA's, and CT. It has been determined from this analysis that part of the dead and live loads of the EPA and CT are supported by the MA. This transfer of load to MA causes moments and shears at the interface of MA and CT. These moments and shears (M_e & V_e in figure 7(a)) result in "existing stresses" in the MA. The areas of maximum stresses are the floor slab at El. 659' between column lines G and H and walls between El. 614' and 584' between column lines G and H as shown in Figure 2. The existing stress is predominately tension in the slab at El. 659' and shear in the wall below El. 614'.

The total existing displacements under the CT and EPA are the sum of the following components:

(i) Building Translation: Uniform soil settlement results in rigid body translation of the structure and causes no stress in the building.

(ii) Building Rotation: Because the soil under the MA is stiffer than the soil under the EPA's and CT, the entire Auxiliary Building will rotate. This rigid body rotation induces no stress in the structure.

(iii) Elastic Movement: The soil reaction under the CT and EPA is less than the weights of these structures. The difference between the weights of the structures and the sum of the soil reactions is transmitted to MA by partial cantilever action inducing elastic movements in the CT and EPA. This elastic movement has caused the existing stress in the structure.

3 PERMANENT UNDERPINNING AND ITS EFFECTS ON EXISTING STRESS

To provide adequate support under the CT and EPA, permanent underpinning walls are constructed. The layout of these walls is shown in Figure 4. Building loads are transferred to the underpinning wall by jacking the wall against the building. The amount and location of jacking loads to be applied is shown in Figure 5. The basis of these jacking loads and their locations are: (1) they equal the tributary load from the structure above, and (2) their center of gravity coincide with the centroid of the tributary loads from the building.

The effect of applying permanent jacking load has been schematically shown in Figure 7. Due to the existing condition a Shear V_e and a moment M_e occur at the interface of the Main Auxiliary and the Control Tower as shown in Figure 7 (a). Existing stresses are caused due to these forces. Due to the soil removal and the application of permanent jacking load, shear V_p and moment M_p are caused at the interface as shown in Figure 7 (b). Stresses, opposite in nature to the existing stresses, are caused by these forces. As shown in Figure 7 (c), the existing shear and moment are eliminated, after the application of the permanent jacking load if the jacking load equals the tributary weight of the EPA and Control Tower; and the jacking load is applied at the center of gravity of the tributary load. Since the applied load is not exactly equal to the tributary load and there is a small eccentricity between the jacking and the tributary load (see Figure 6), small residual shear and moment remain in the structure causing small residual stresses.

For example, the existing stress in the slab at El. 659' before the start of underpinning was approximately 14.3 ksi tension with load factor. At the end of permanent jacking, this tensile stress is reduced to a small amount (2.1 ksi).

In the analytical model for the analysis of the permanent wall, neither of the loading conditions presented in Figures 7 (a) and (b) are considered, as they essentially neutralize each other as explained above. Therefore,

the model considers the loading cases starting from the lock off of the jacking loads. The forces and moments resulting from this model have already been presented in FSAR Table 3.8 - 19 and are shown to be acceptable.

4 BUILDING BEHAVIOR DURING TEMPORARY UNDERPINNING

The temporary jacking stage is a transient phenomenon in the underpinning construction. The aim of temporary jacking is to provide support for the building so that soil under the CT and EPA can be removed and the permanent walls constructed. The total amount of temporary jacking loads (J_L) to be applied are shown schematically in Figure 3.

An analysis of the structure was performed to ensure the safety of the structure during all construction stages. In this analysis the effect of existing stress was considered by using bounding values of soil springs under MA, EPA and CT. According to the analysis performed, the stress in the slab at El. 659' changes to small compressive stress from tensile stress, but the shear stress in the walls below El. 614' between column lines G and H does not reduce significantly (see Table 1). This analysis shows that during the temporary underpinning, the structural stresses are within allowables.

5 DISPLACEMENT

As temporary and permanent jacking are progressively applied, the elastic settlement will generally be recovered, (i.e., the building will move up). The phenomenon was confirmed during the jacking of the grillage beams at E/W8 (see Figures 8 and 9). The observed upward movement here was approximately 60 mils for the West EPA and 80 mils for the East EPA for the jacking load of 2,000Kips. Part of this upward movement can be attributed to atmospheric changes in temperature. Calculated values of upward movement, during the jacking, are only approximate and will vary from observed values because of the following factors in the analysis:

a. E and G Values of Concrete: The concrete has been poured at different times and under different conditions; therefore, there is much uncertainty regarding the values of E and G. The displacements are directly proportional to the values of E and G.

b. Degradation of shear modulus due to microcracking: Shear modules is proportional to concrete elasticity modulus. However, even with minute cracks in the concrete, the elastic modulus (and hence the shear modulus) reduces drastically.

c. Simplifying assumptions made in modeling a complex structure.

d. Nonlinear soil springs: The response of the soil depends on the strain level, the time the load is sustained, the previous loading history of the soil, and whether the soil is being loaded or unloaded. The soils have a nonlinear stress-strain relationship, and linear springs representing soil behavior cannot strictly be applied.

e. Atmospheric temperature variations: During a cold spell in October 1982, it was observed that the ends of the EPA moved up by approximately 25 mils.

It is not practicable to determine these factors accurately for the analytical model. Codes and standards provide necessary guidance to determine these factors. Structural analysis performed using code recommended factors results in a conservative design of the structure from strength considerations.

The displacement calculation is directly related to these factors and is sensitive to their variation. Hence, the calculated displacements due to the applied loads are approximate. To ensure serviceability, the calculated deflections based on code assumption are usually modified by conservative factors.

6 CONCLUSION

Existing building stresses before underpinning are reduced to small residual stresses at the end of soil removal under EPA and CT and the application of permanent jacking loads. Hence the assumptions made, regarding the existing stresses, in the analytical model are justified and the calculated stresses resulting from these models are reasonable. The resulting stresses are presented in FSAR, (Table 3.8 - 19) and meet the acceptance criteria.

Downward elastic displacement has occurred because of the difference in soil stiffness under different areas of the MA and backfill under the CT and EPA. It is expected that a major portion of the downward elastic displacement will be recovered by an upward movement of the structure. The amount of elastic displacement that will be recovered can only be predicted approximately by calculations. However, the plan is to apply the design jacking loads and allow the structure to move up.

Applying predetermined jacking loads and allowing the structure to move up will ensure that the stresses existing in the structure before the underpinning will be reduced to minimal values.

AUXILIARY BUILDING UNDERPINNING

SOIL SPRINGS UNDER AUXILIARY BUILDING

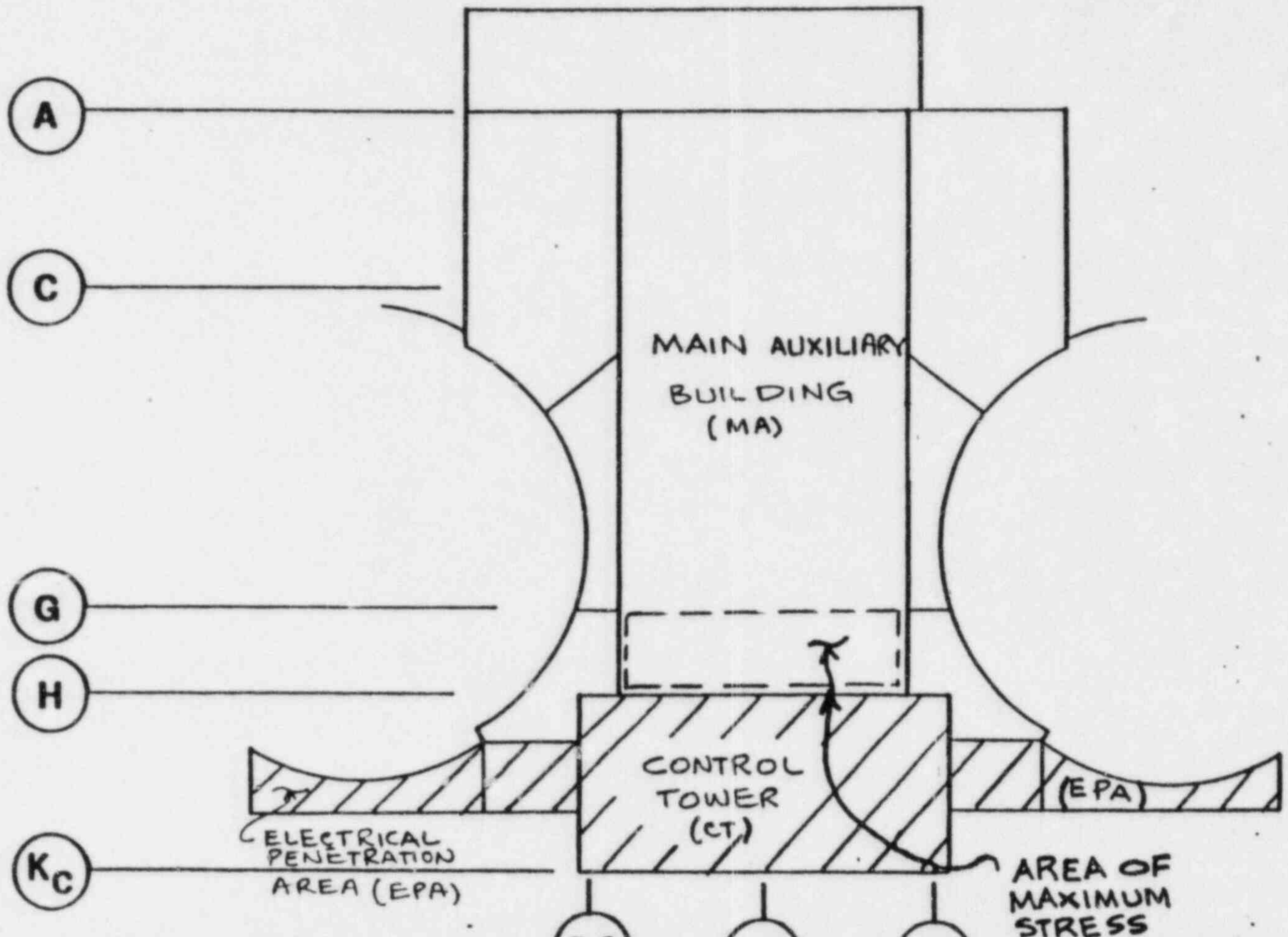


FIGURE 1

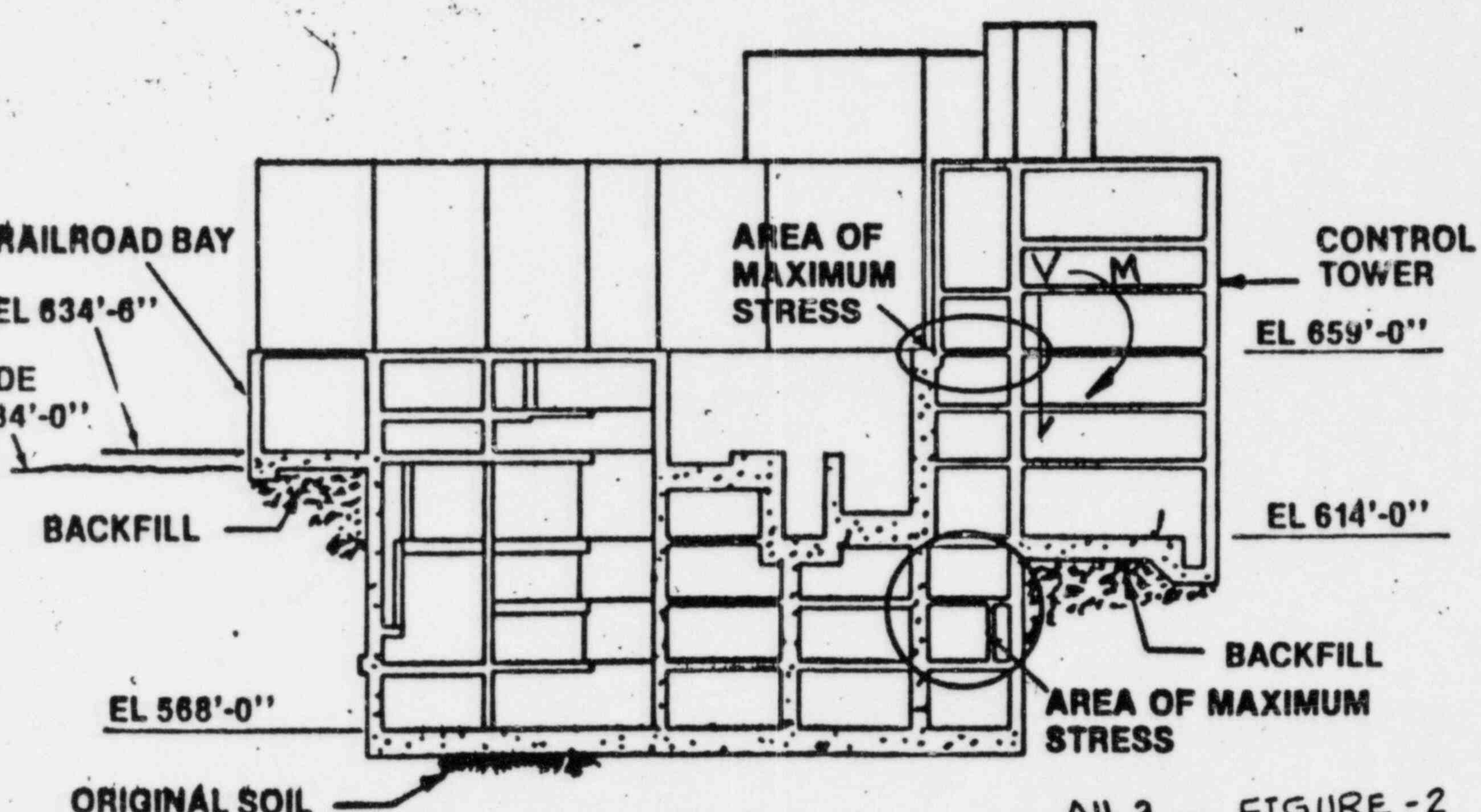
AH - 3

NOTE:

(1) for lower differential settlement

(2) for higher differential settlement

AUXILIARY BUILDING UNDERPINNING
TYPICAL SECTION
LOCATION OF MAXIMUM STRESS
(Looking East)

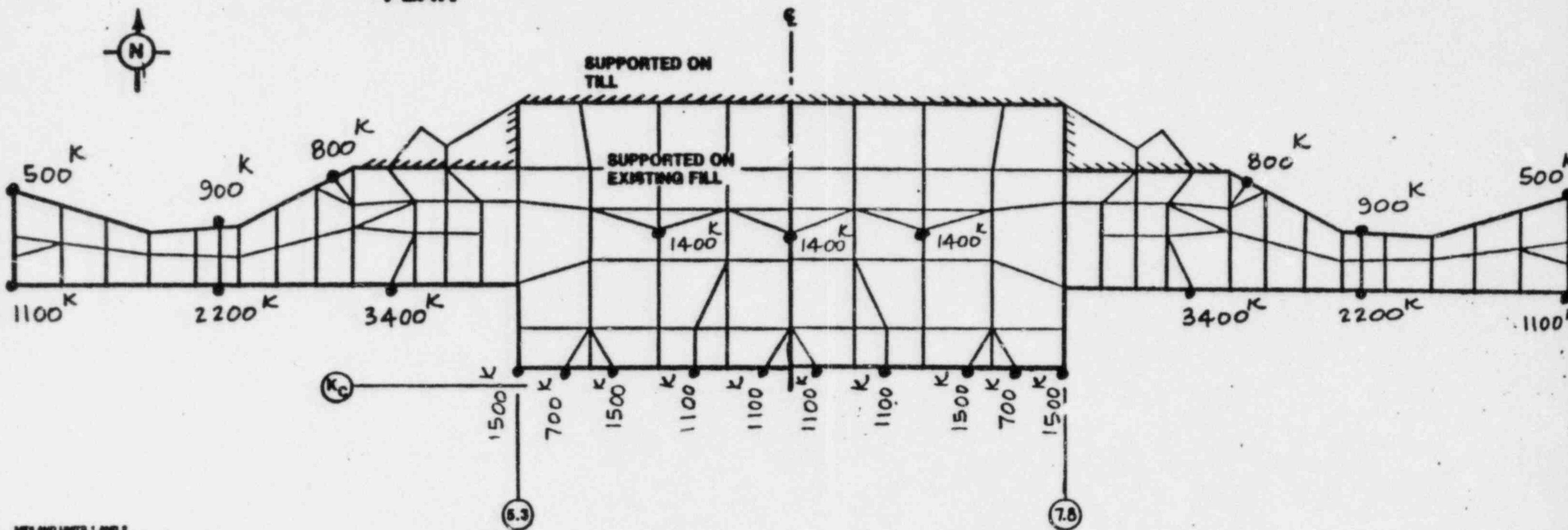


Att. 3 — FIGURE - 2

G-1555-07

MIDLAND UNITS 1 AND 2
AUXILIARY BUILDING UNDERPINNING 1/26/82

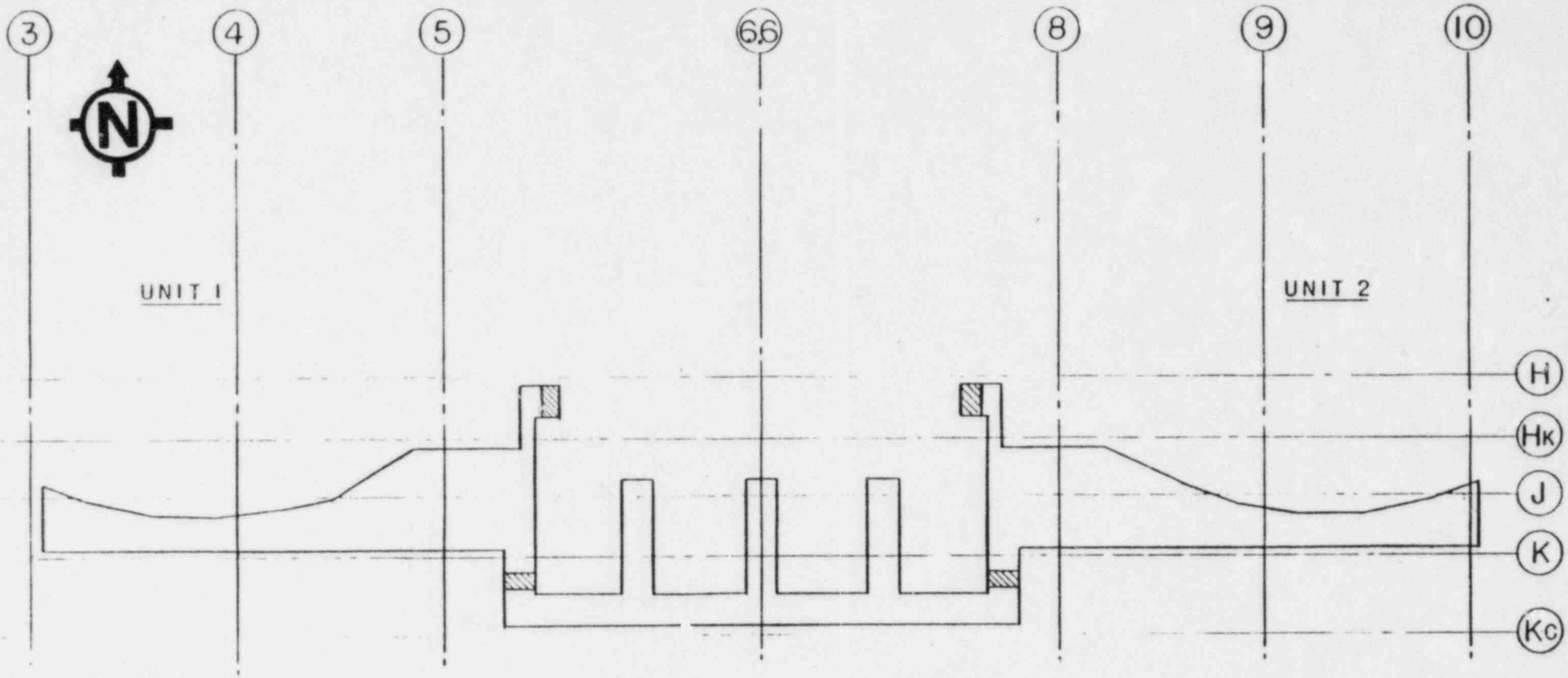
AUXILIARY BUILDING UNDERPINNING
CONSTRUCTION AREA
PLAN



TEMPORARY JACKING LOADS (J_L)

(TOTAL JACKING = 33,800^K)

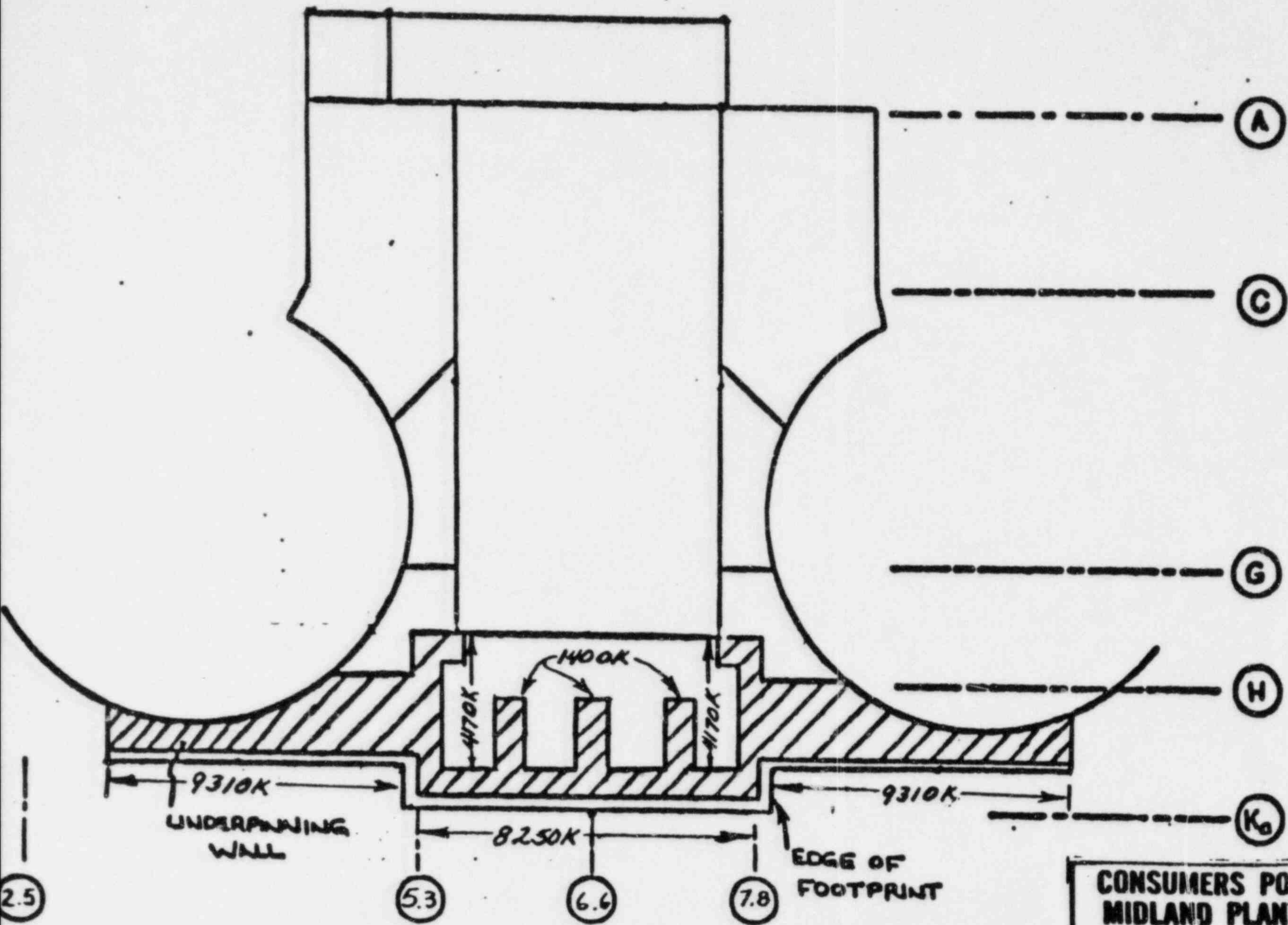
Attach - 3
FIGURE 3



PERMANENT UNDERPINNING WALL

ATTATCHMENT - 3
FIGURE - 4

▧ - AREAS TO BE POURED AFTER FINAL LOAD TRANSFER ACCEPTANCE CRITERION HAS BEEN SATISFIED

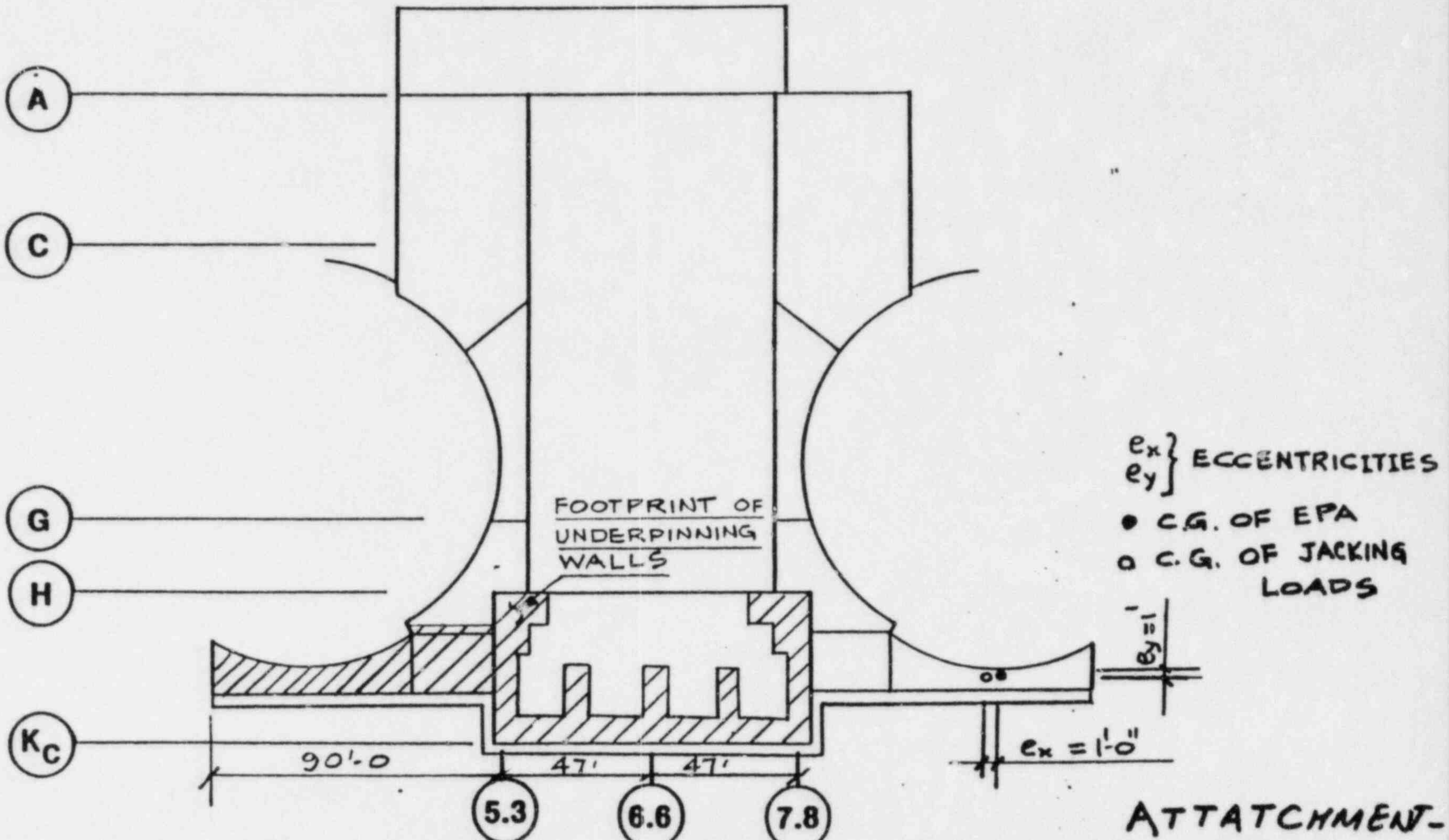


PERMANENT JACKING LOADS (P_L)
(TOTAL JACKING = 39410^k)

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

FIGURE 5
ATTATCHMENT-3

AUXILIARY BUILDING UNDERPINNING
SOIL SPRINGS UNDER AUXILIARY BUILDING



ATTATCHMENT-
FIGURE 6

NOTE:

(1) for lower differential settlement
(2) for higher differential settlement

MIDLAND UNITS 1 AND 2
AUXILIARY BUILDING UNDERPINNING 1/26/82

31-G-1862-24

NOMENCLATURE:

W = Weight of (CT & EPA)

S_R = Soil Reaction on Fill

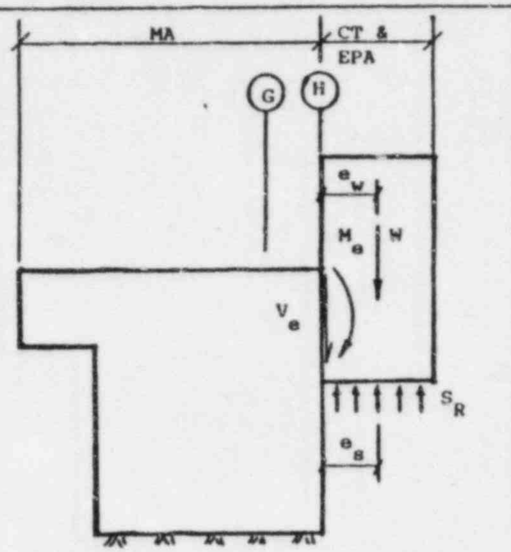
P_L = Total Permanent Jacking Load $\cong W$

e_w, e_s, e_p = Eccentricities of W, S_R, P_L W.R.T. 'H' Line; $e_p \cong e_w$

M_e, M_p = Moment Effects on 'H' Line for Existing and Permanent Jacking Respectively

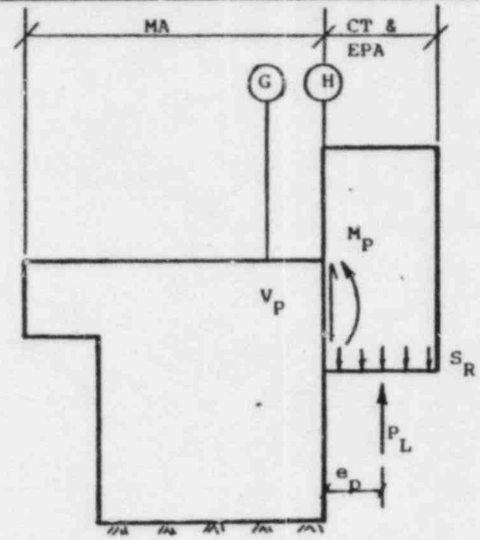
V_e, V_p = Shear Effects

(a) Existing Condition



1) W S_R
2) $W-S_R$ causes moments (M_e) and shear (V_e)
3) $V_e = W-S_R$
4) $M_e = (W)(e_w) - (S_R)(e_s)$

(b) Effect of Soil Removal and Permanent Jacking



1) $V_p = P_L - S_R$
2) $M_p = (P_L \times e_p) - (S_R \times e_s)$

(c) Residual Stresses after Permanent Jacking

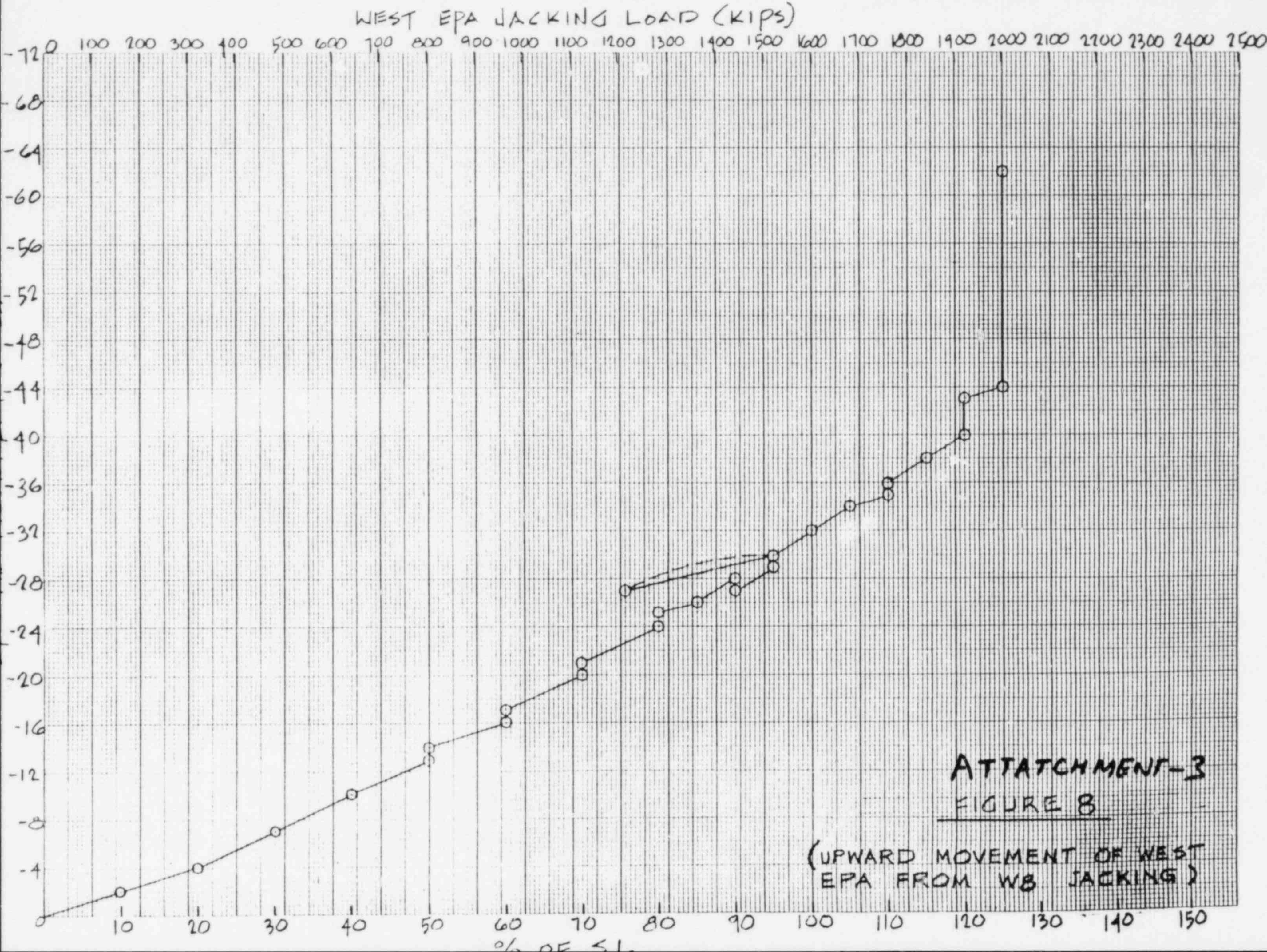
Shear $= V_e - V_p = W - P_L$

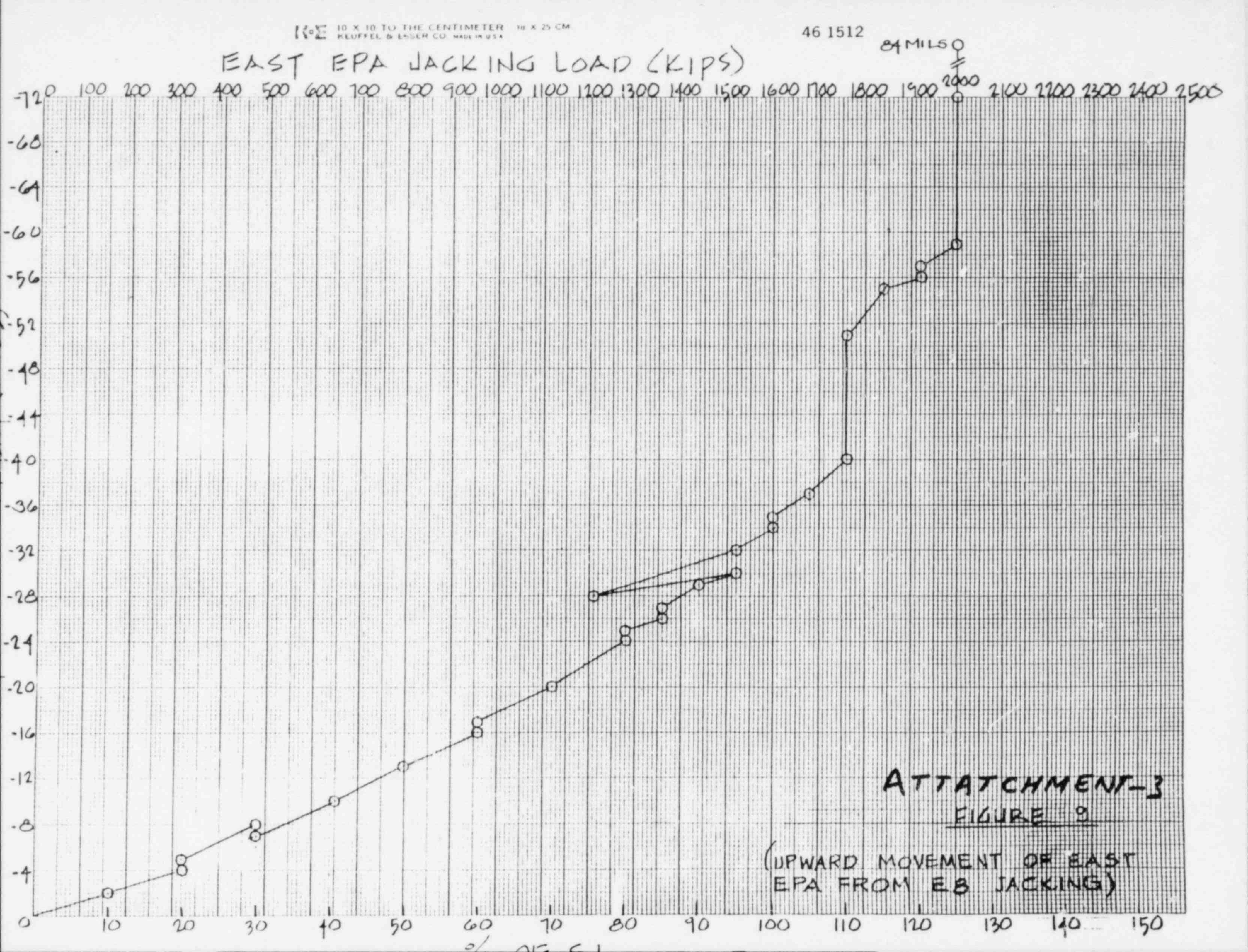
$= 0$ if $W = P_L$

Moment $= M_e - M_p = (W)(E_w) - (P_L)(e_p)$

$= 0$ if $W = P_L$ & $e_w = e_p$

FIGURE - 7 , ATTATCHMENT-3


WEST EPA JACKING LOAD (KIPS)
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500
-72 -68 -64 -60 -56 -52 -48 -44 -40 -36 -32 -28 -24 -20 -16 -12 -8 -4
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150
ATTATCHMENT-3
FIGURE 8
(UPWARD MOVEMENT OF WEST EPA FROM WB JACKING)



K+E 10 X 10 TO THE CENTIMETER 18 X 25 CM KEUFFEL & ESSER CO. MADE IN U.S.A.
46 1512
EAST EPA JACKING LOAD (KIPS)
0 100 200 300 400 500 600 700 800 900 1000 1100 1200 1300 1400 1500 1600 1700 1800 1900 2000 2100 2200 2300 2400 2500
84 MILS
-72 -68 -64 -60 -56 -52 -48 -44 -40 -36 -32 -28 -24 -20 -16 -12 -8 -4 0
10 20 30 40 50 60 70 80 90 100 110 120 130 140 150
ATTATCHMENT-3
FIGURE 9
(UPWARD MOVEMENT OF EAST EPA FROM EB JACKING)

TABLE 1

REBAR STRESSES IN CRITICAL AREAS

Description	Existing Stress	Stress (ksi) P_L Stress at End of Permanent Jacking	Stress (ksi) J_L Stress at End of Temporary Jacking	Remarks
Slab at el 659' between column lines G and H	14.3(1)	2.1	0(5)	P_L stresses added to stresses from other load combinations
N-S walls on column lines 5.3 and 7.8 between column lines G and H and between el 614' and 584'	16.37	6.7(2,4)	16.42(2,4)	J_L stresses are transient.

(1) For 1.4D + 1.7L, where D and L are stresses from dead and live loads, respectively
(2) For 1.0 P_L
(3) Includes membrane shear
(4) Includes membrane tension
(5) Compression in concrete; therefore, no tension in rebar

RESPONSE TO SUBSEQUENT DISCUSSION ITEMS 1 & 4

During the temporary jacking operation, uncontrolled upward displacement of the structure is not possible due to the following reasons:

1. The total applied temporary jacking load is less than the gravity load of the Control Tower and Electrical Penetration Areas (EPA's).

2. A substantial structural link exists between the Control Tower and the Main Auxiliary Building.

Some structural movement will result from the jacking operation. This movement has two components:

1. Rigid body rotation due to application of loads at the edge of a structure on an elastic medium. (This movement does not induce any internal structural stresses.)

2. Elastic deformation; individual jacking loads will cause the structure to move until resistance from the surrounding structure equals the jacking load. (This movement induces strains and stresses in the building).

The expected order of magnitude of upward movements at the EPA's during the temporary jacking operation is as follows: (See Figure 4-1 for Deep Seated Bench Mark (DSB) locations).

3/16" for extreme ends of EPA's at DSB-2E/W with respect to ends of Control Tower as measured at DSB-3E/W.

This value is based on a calculation which considers an EPA as a cantilever beam fixed at the Control Tower. A linear elastic analysis with stiffness based on uncracked sections is used. Applied loads are chosen from the current temporary jacking schedule as representing the condition that should produce the greatest displacement at the free end of an EPA. The deflection computed at the free end of an EPA is based on flexural and shear deformations. This calculated movement is approximate due to the reasons described in Section 5.0 of Attachment 3. Therefore, the observed movement may exceed the calculated values.

The allowable upward movement of the Control Tower during the temporary jacking operation is as follows:

1/2" for the south face of the Control Tower with respect to the north edge of the immediately adjacent Turbine Building foundation mat between Col. Lines 5.6 and 7.4 (see Figure 4-2).

This value is based on a conservative evaluation of the existing condition of the connection of eight duct banks to the south face of the Control Tower foundation (see Figure 4-2 for illustration). The purpose of this evaluation is to ensure that the relative movement between the Control Tower and Turbine Building foundations does not impair serviceability of the cables within the duct bank. For the evaluation, primary considerations are fill ratio of individual conduits (i.e., available space inside conduit) and a worst case shear displacement of the pipe across the joint. During jacking of the piers CT1/12, relative movement between the Control Tower and Turbine Building foundations will be monitored at nearby DSB's 3E/W. During jacking of the remaining CT piers, the relative movement will be monitored by additional instruments which will be installed shortly. The 1/2" allowable value may be increased if it can be shown that the deformation of the cables and the conduit can allow greater relative movement without impairing the serviceability of the cables.

During the temporary and permanent jacking operations, the intent is to allow the Control Tower and EPA's to move up as required for input of design jacking loads. Input of design jacking loads is important with regards to existing stresses in the structure as explained in Attachment 3. If the upward movement of an EPA relative to the Control Tower reaches 3/8" (twice the calculated value) or the upward movement of the Control Tower relative to the Turbine Building reaches 1/2" (the allowable value for the Control Tower), jacking shall be stopped. Jacking shall be resumed only after it is determined by subsequent evaluation that structural safety and serviceability will not be impaired. The jacking schedule and/or construction schedule may be revised if necessary to remain within these limits.

The estimated values of upward movement of the structure due to permanent jacking will be based on the observed movement values during temporary jacking.

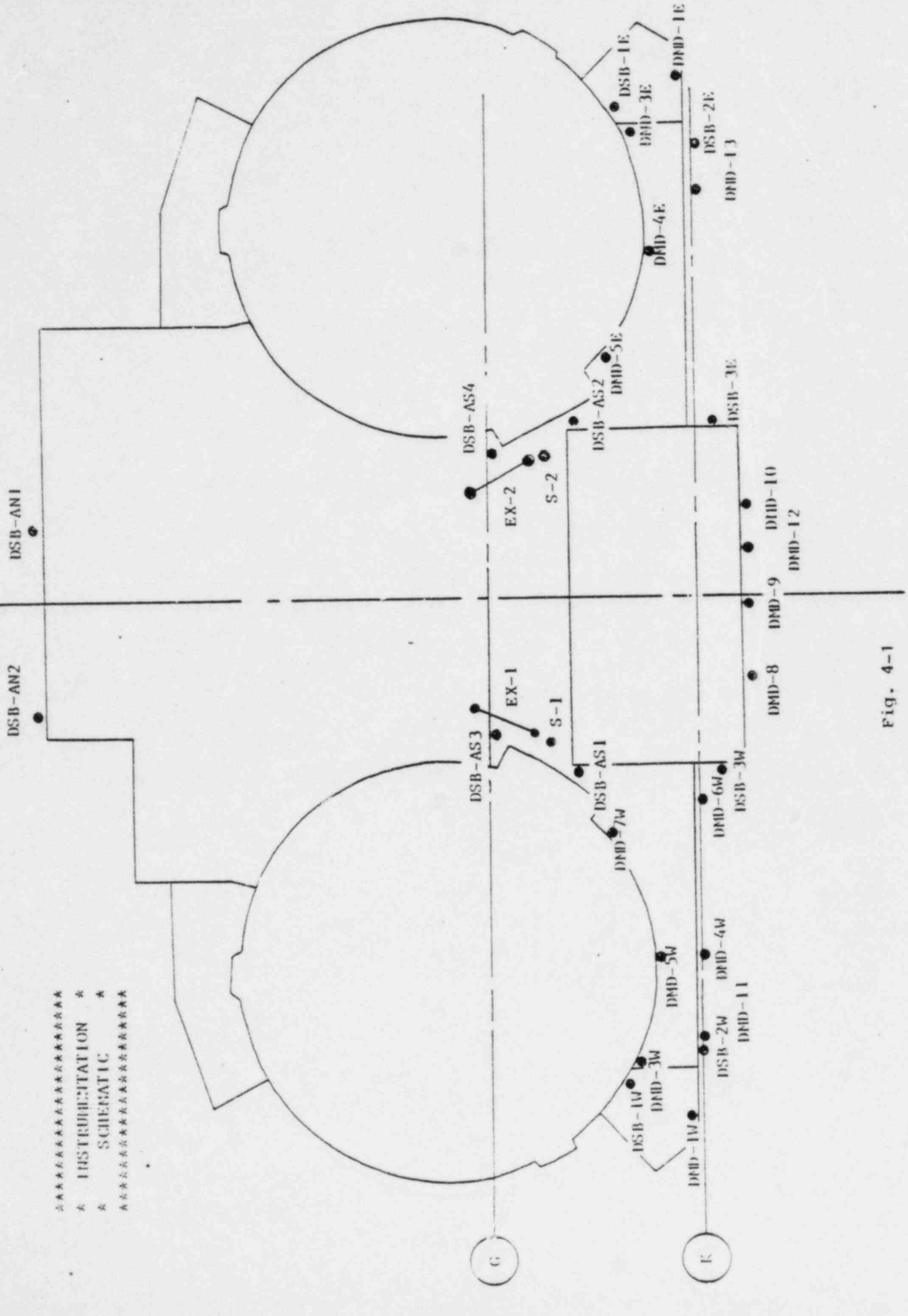
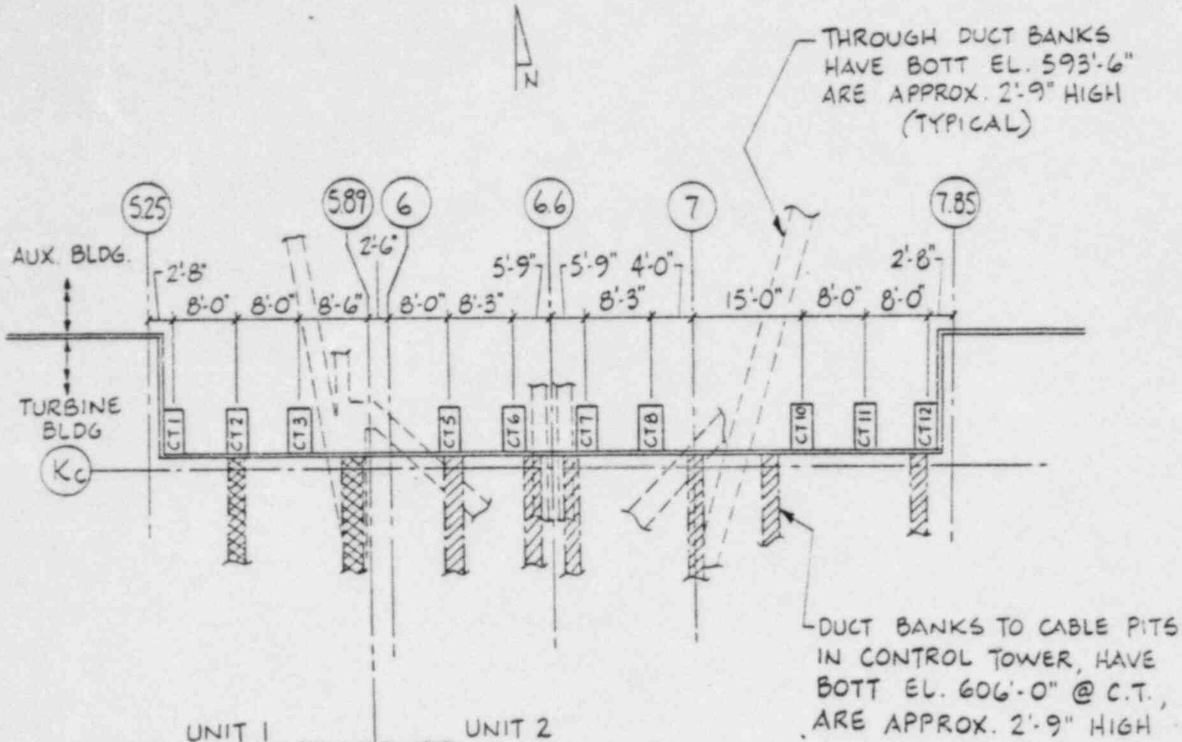
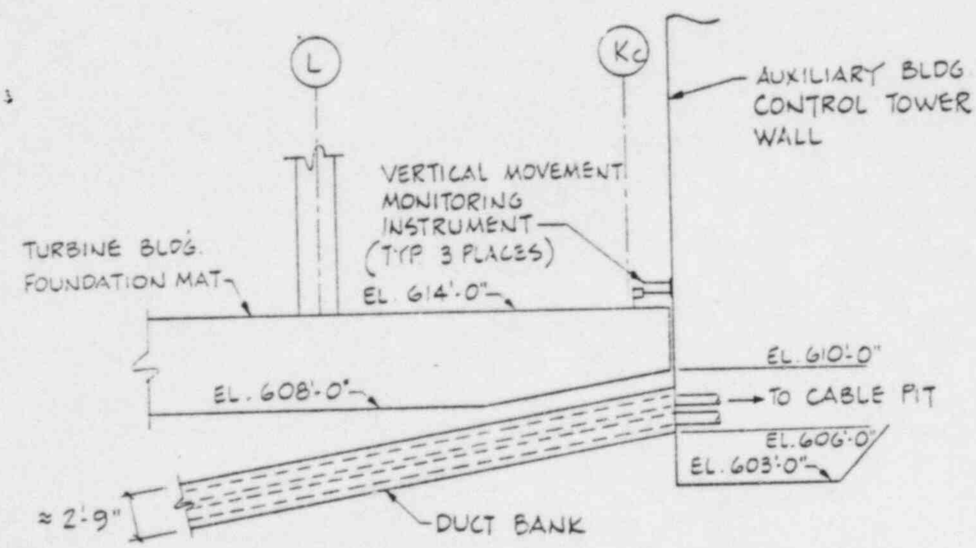

INSTRUMENTATION
SCHEMATIC
6.6
G
K
DSB-AN1
DSB-AN2
DSB-AS1
DSB-AS2
DSB-AS3
DSB-AS4
EX-1
EX-2
S-1
S-2
DSB-1E
DSB-2E
DSB-3E
DMD-1E
DMD-3E
DMD-4E
DMD-5E
DMD-8
DMD-9
DMD-10
DMD-11
DMD-12
DMD-13
DSB-1W
DSB-2W
DSB-3W
DMD-1W
DMD-3W
DMD-4W
DMD-5W
DMD-6W
DMD-7W

Fig. 4-1



PLAN



TYP SECTION
LOOKING WEST

Figure 4-2

RESPONSE TO SUBSEQUENT DISCUSSION ITEM 4

During each increment of initial jacking of a pier/grillage, building movements are determined by a computerized monitoring system operated by Wiss, Janey, Elstner and Associates (WJE). WJE transmits a copy of this data to the Bechtel Resident Structural Engineer (RSE). The RSE records the movement with other jacking data and verbally transmits the data to the Mergentime Jacking Engineer.

The RSE evaluates and plots key data such as jacking load versus building movements as the data is obtained. Extrapolations of plotted data are made to predict if the allowable upward movement (AUM) or other limits set by the RSE, may be exceeded and if so, at what jacking load. These evaluations are regularly transmitted to Mergentime engineering personnel so that actions can be planned if an AUM is forecast.

When an AUM occurs, the corresponding jacking load (MARL) shall be reduced to 80% MARL and maintained. The RSE in consultation with Mergentime makes an evaluation based on experience and the following data as appropriate:

* The current and past movement rates at the applicable deep seated benchmarks.

* The magnitude of the jacking load (MARL) and loads jacked at other support locations with respect to specified loads.

* The magnitudes of monitoring parameters for bending ($\Delta 1$, $\Delta 2$ and $\Delta 3$) with respect to alert and action limits.

* Magnitude of recovered settlements.

* Past experience from previous jacking locations.

* Strain data at building monitoring locations.

* Building behavior predicted by Project Engineering models.

* Pier load - settlement behavior.

* Results from crack monitoring, walk downs, and inspections at critical areas of the structure.

Direction to maintain 80% of MARL for acceptance criterion or resumption of jacking to a jacking load above 80% of MARL is given after RSE's and Mergentime's evaluations have been completed and discussed, and actions agreed upon. In addition to the on site evaluation of data, Project Engineering personnel are regularly updated concerning building behavior.

In summary, the above process ensures that no physical damage is imparted upon the structure. The step by step reviews provide a mechanism of checking that the jacking operation is proceeding as predicted, thereby giving confidence in the analysis. If conditions develop which are different than expected, jacking operations are stopped until the situation is evaluated and the problem resolved.