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RECIP.NAME RECIPIENT AFFILIATION
TEDESCO,R.L. Assistant Director for Licensing

SUBJECT: Forwards "Foundation Instrument Rept," response to SER Section 2.5.4.3 & summary of 811020-21 meeting.
See Rpts

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P.O. BOX 21666 - PHOENIX, ARIZONA 85036

December 4, 1981
ANPP-19610 - JMA/WFQ

Mr. R. L. Tedesco
Assistant Director for Licensing
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Palo Verde Nuclear Generating Station (PVNGS) Units 1, 2 and 3
Docket Nos. STN-50-528/529/530
File: 81-056-026; G.1.10

References: (1) NUREG-0587, Safety Evaluation Report related to the operation of Palo Verde Nuclear Generating Station Units 1, 2 and 3, November 1981

(2) Letter from E. E. Van Brunt, Jr., APS, to R. L. Tedesco, (NRC) (ANPP-19239 - JMA/WFQ) dated October 22, 1981, subject: APS/HGEB Meeting Summary

Dear Mr. Tedesco:

The SER (Reference (1)) discussion in Section 2.5.4.3 resulted from our meeting with NRC's HGEB representative on October 20-21, 1981. A meeting summary was provided by Reference (2).

The APS responses to the SER Section 2.5.4.3 items (settlement monitoring), as well as those items of Reference (2) (settlement monitoring, lateral pressures and liquifaction analysis) are enclosed.

Based on the information presented, we now consider these items resolved. If you have any further questions, please contact me.

Very truly yours,

JM Allen for

E. E. Van Brunt, Jr.
APS Vice President,
Nuclear Projects
ANPP Project Director

B001
s
1/1

EEVBJr/WFQ/av
Enclosure
cc: J. Kerrigan (w/a)
P. Hourihan (w/a)
A. C. Gehr

STATE OF ARIZONA)
) ss.
COUNTY OF MARICOPA)

I, John M. Allen, represent that I am Nuclear Engineering Manager of Arizona Public Service Company, that the foregoing document has been signed by me for Edwin E. Van Brunt, Jr., Vice President Nuclear Projects, on behalf of Arizona Public Service Company with full authority so to do, that I have read such document and know its contents, and that to the best of my knowledge and belief, the statements made therein are true.

John M Allen

John M. Allen

Sworn to before me this 4th day of December, 1981.

Robert K Waltz

Notary Public

My Commission expires:

October 2, 1985

SETTLEMENT MONITORING

Reference 1, SER Section 2.5.4.3 (Settlements)

NRC Position

The staff concurs with the applicant's position (on post construction settlements) provided that the applicant formally submits the material presented at the (October 20-21, 1981) meeting.

Response

The material presented at the meeting is provided as Attachment 1.

Reference 1, SER 2.5.4.3 and Reference 2, Item IA (Settlement Monitoring)

NRC Position

a) Increase the frequency of post construction settlement monitoring.

b) Add three monitoring points to the essential spray pond structural fill area.

c) Include Items a) and b) into the monitoring program for Units 1, 2 and 3.

Response

a) Refer to the revised FSAR Table 2.5-18 (Attachment 2).

b) Locations for four additional settlement marker monitoring points per unit have been selected to monitor settlements of backfill supported segments of the spray ponds. These locations are shown on revised Figure 2 (Attachment 3) to our Foundation Instrumentation Report.

c) Items a) and b) have been incorporated as part of our heave, settlement and subsidence monitoring program for Units 1-3. Refer to the revised Foundation Instrumentation Report (Attachment 4).

LATERAL PRESSURES

Reference 2, Item I.B.1

NRC Position

APS will provide the design criteria for lateral earth pressures and incorporate into the FSAR, Section 3.8. (Currently there is no mention of surcharge effects.)

Response

The response is provided in Attachment 5 which will be incorporated into the FSAR in a future amendment.

STABILITY ANALYSIS

Reference 2, Item I.B.2.

NRC Position

Provide an updated stability analysis with reasonably conservative interpretation of shear strength along the backfill to native soil interface.

Response

The response to this resolution is presented in Attachment 6.

SLOPE PREPARATION

Reference 2, Item I.B.3

NRC Position

Add statements in the FSAR text regarding slope preparation before backfilling.

Response

The first paragraph on Page 2.5-141, Section 2.5.4.5.3, FSAR will be revised in a future amendment as follows:

"2.5.4.5.3 Backfill

Soil backfill placed adjacent to Category I structures and pipelines meets the requirements of the PVNGS 1, 2, and 3 PSAR. The shape of construction excavations and the extent of backfill at each powerblock is shown in Figures 2.5-74, 2.5-75, and 2.5-76. During backfilling, fills were benched into firm, undisturbed native soils along construction slopes. This was done in order to remove any loose, eroded soil at the construction slope surface, and to facilitate uniform compaction to the edges of the backfill.

Structural backfill under...."

LIQUIFACTION ANALYSIS

Reference 2, Item I.C

NRC Position

Provide results of an additional SHAKE analysis to verify that a surface motion derived from a deconvolved deep motion would be the same as the original surface motion.

Response

The response to this resolution is presented in Attachment 7.

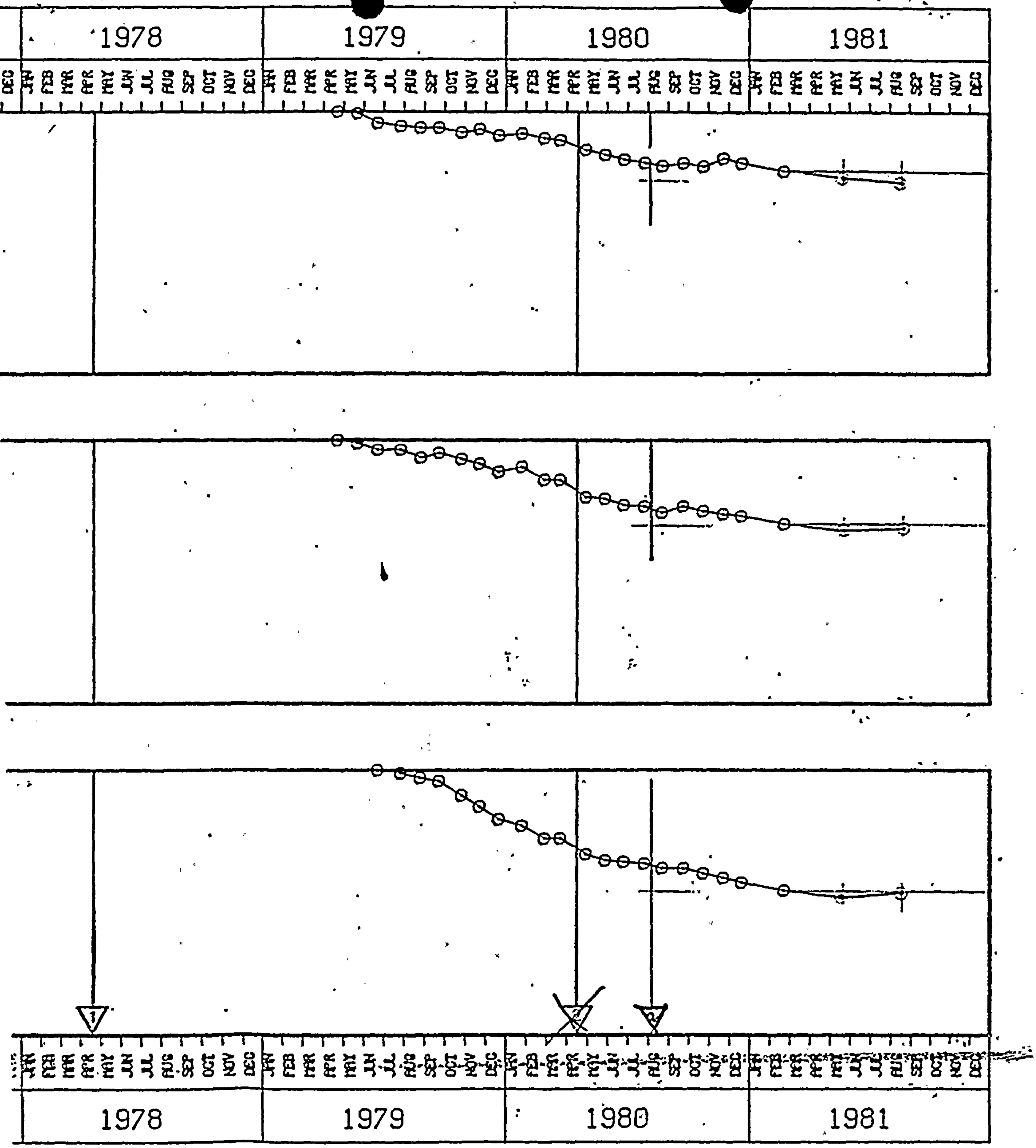
Attachment 1

TOTAL WEIGHTS

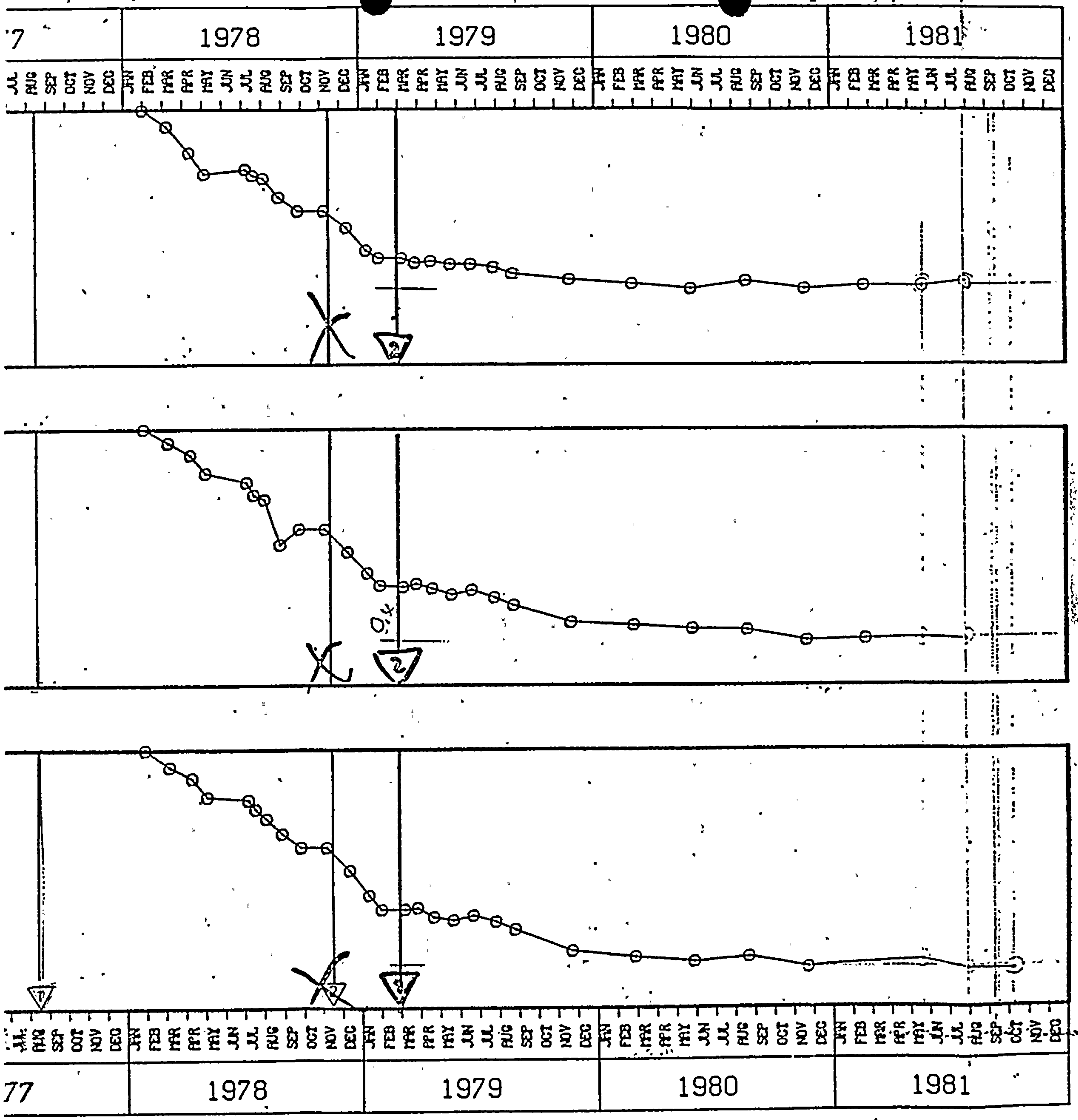
EXCAVATION	−	1,250,000	KIPS
BACKFILL	+	930,000	KIPS
STRUCTURES	+	610,000	KIPS
NET CHANGE	+	290,000	KIPS

STRUCTURAL LOADS

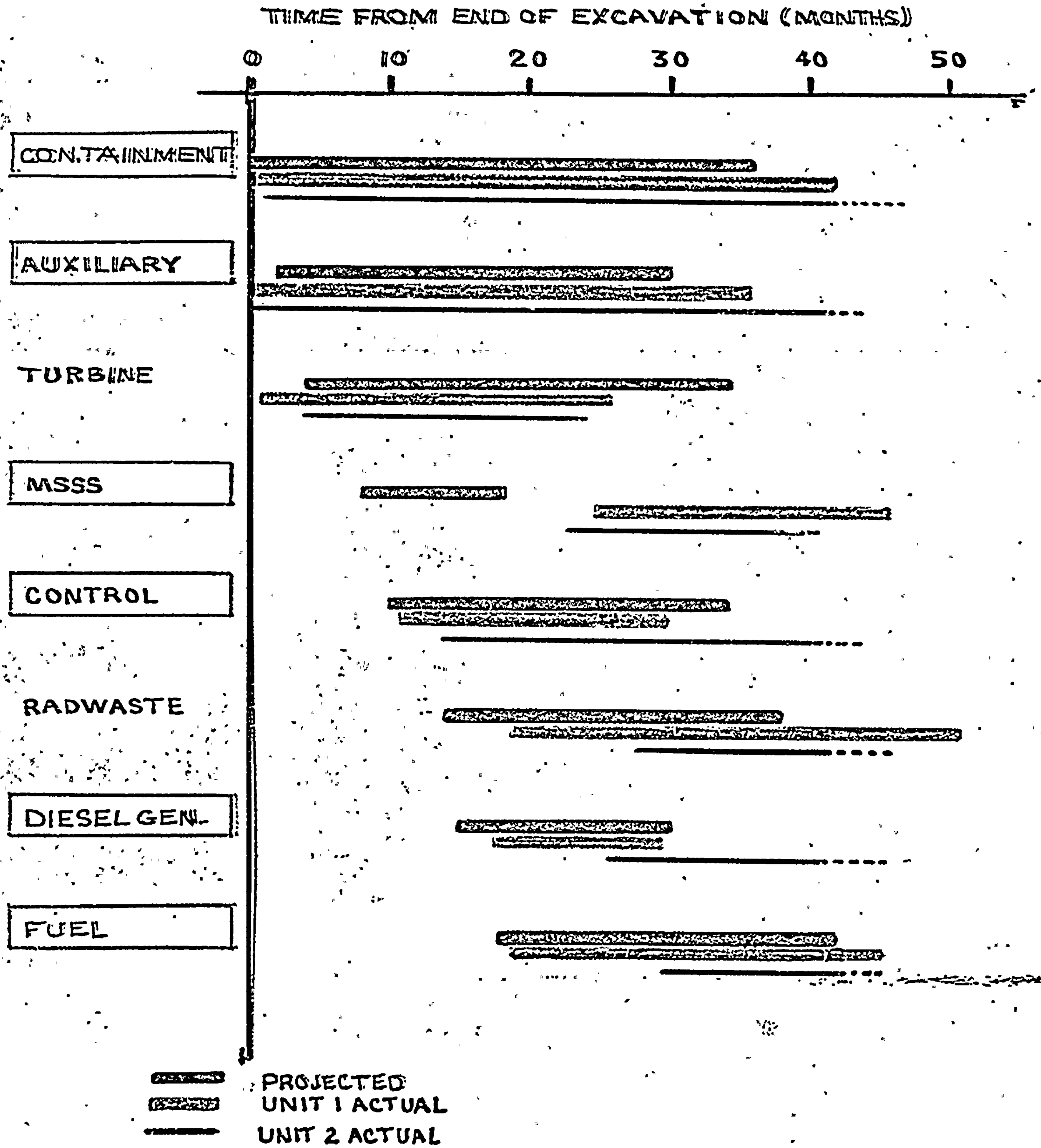
STRUCTURE	FOUNDATION PRESSURE (KSF)	TOTAL WEIGHT (KIPS)
CONTAINMENT	7.9	160,000
AUXILIARY (BASE)	6.2	120,000
" (WINGS)	4.8	
TURBINE	3.0	160,000
MSSS	7.1	12,000
CONTROL	3.3	31,000
RADWASTE	3.0	41,000
DIESEL GEN.	3.1	15,000
FUEL (POOL)	5.3	35,000
(REST)	2.5	

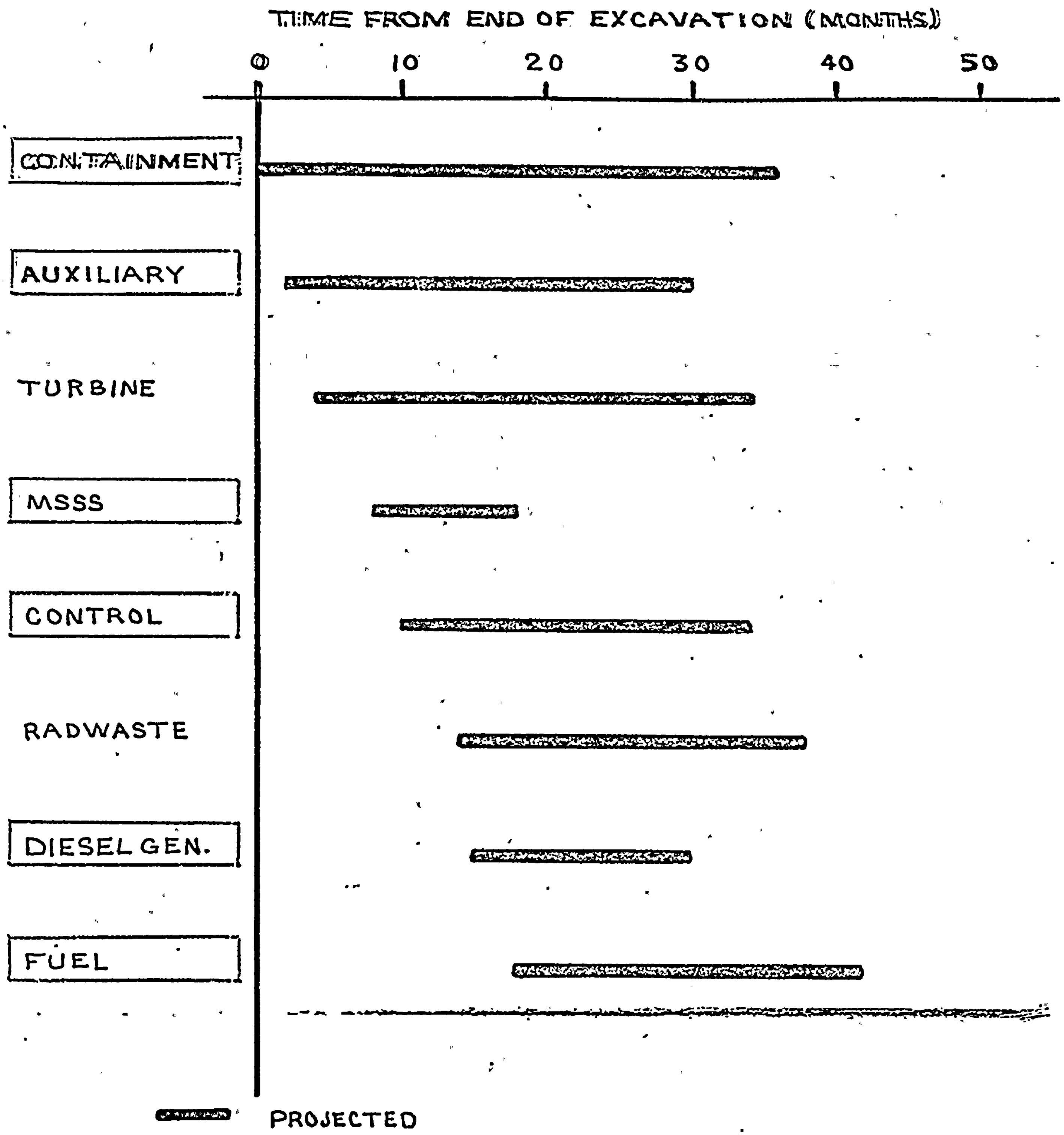


FUEL BUILDING



CONTROL BUILDING


TIME FROM END OF EXCAVATION (MONTHS)
0
10
20
30
40
50
CONTAINMENT
AUXILIARY
TURBINE
MSSS
CONTROL
RADWASTE
DIESEL GEN.
FUEL
PROJECTED
UNIT 1 ACTUAL
UNIT 2 ACTUAL


TIME FROM END OF EXCAVATION (MONTHS)
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RADWASTE
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FUEL
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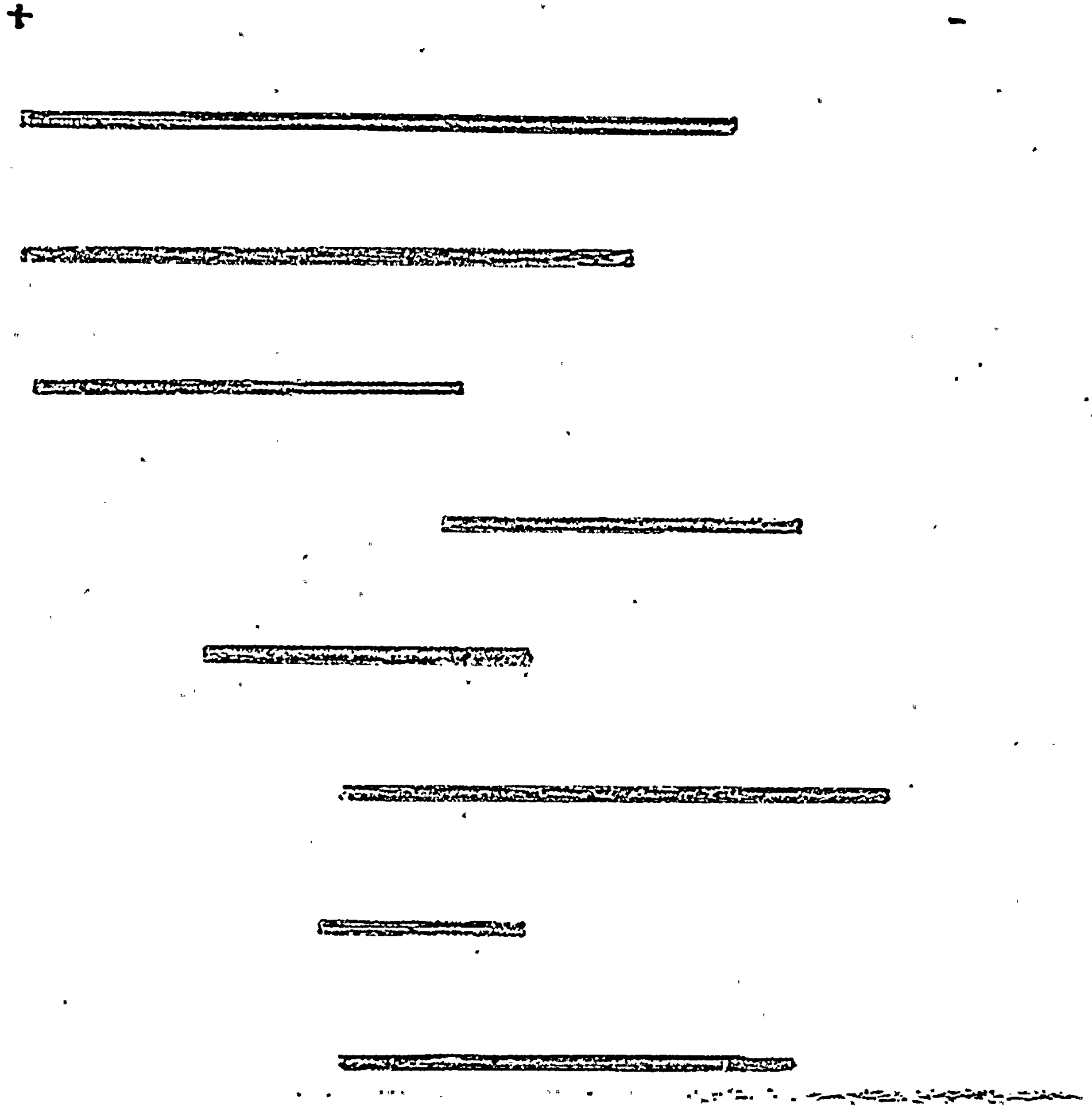
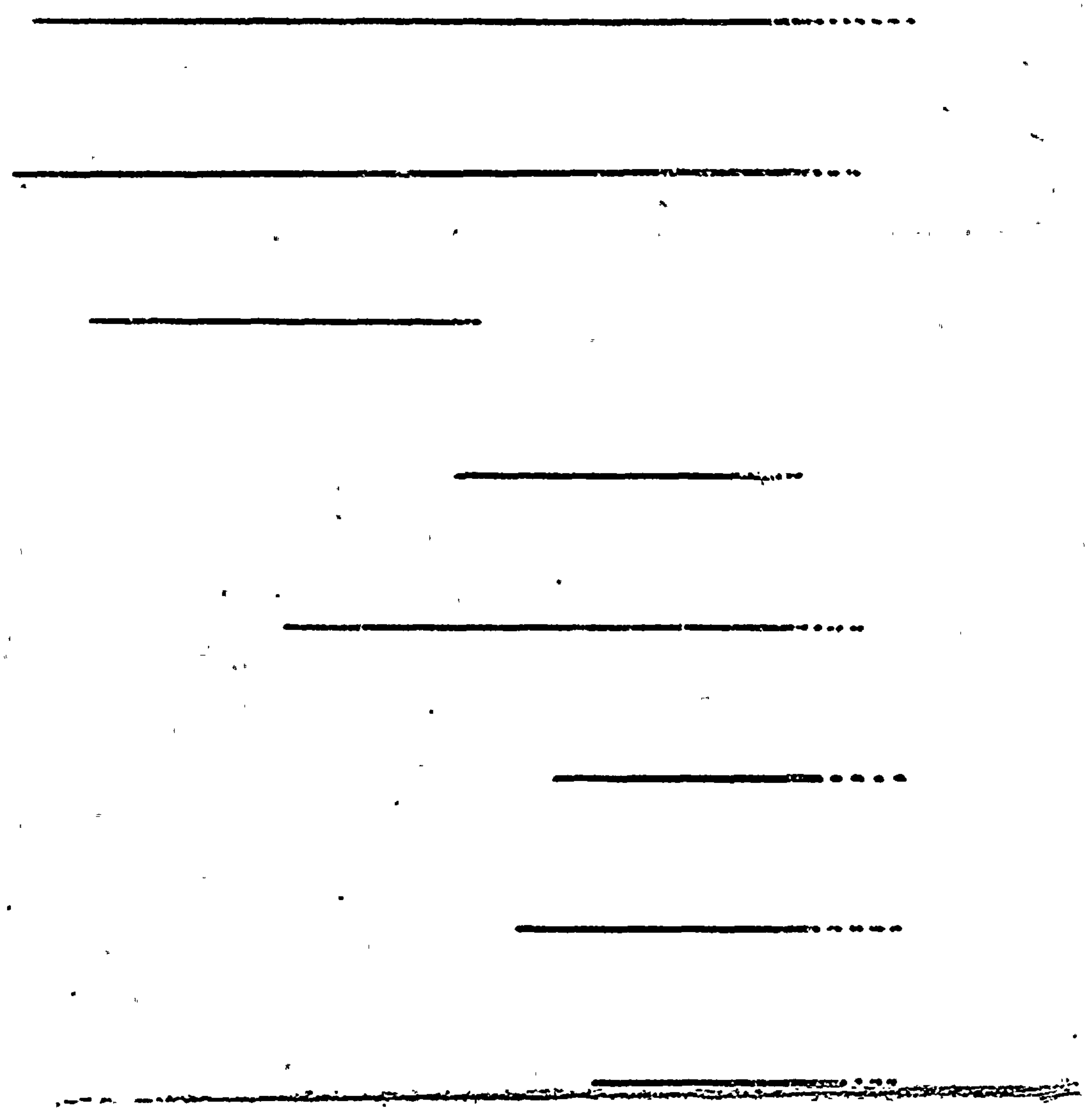
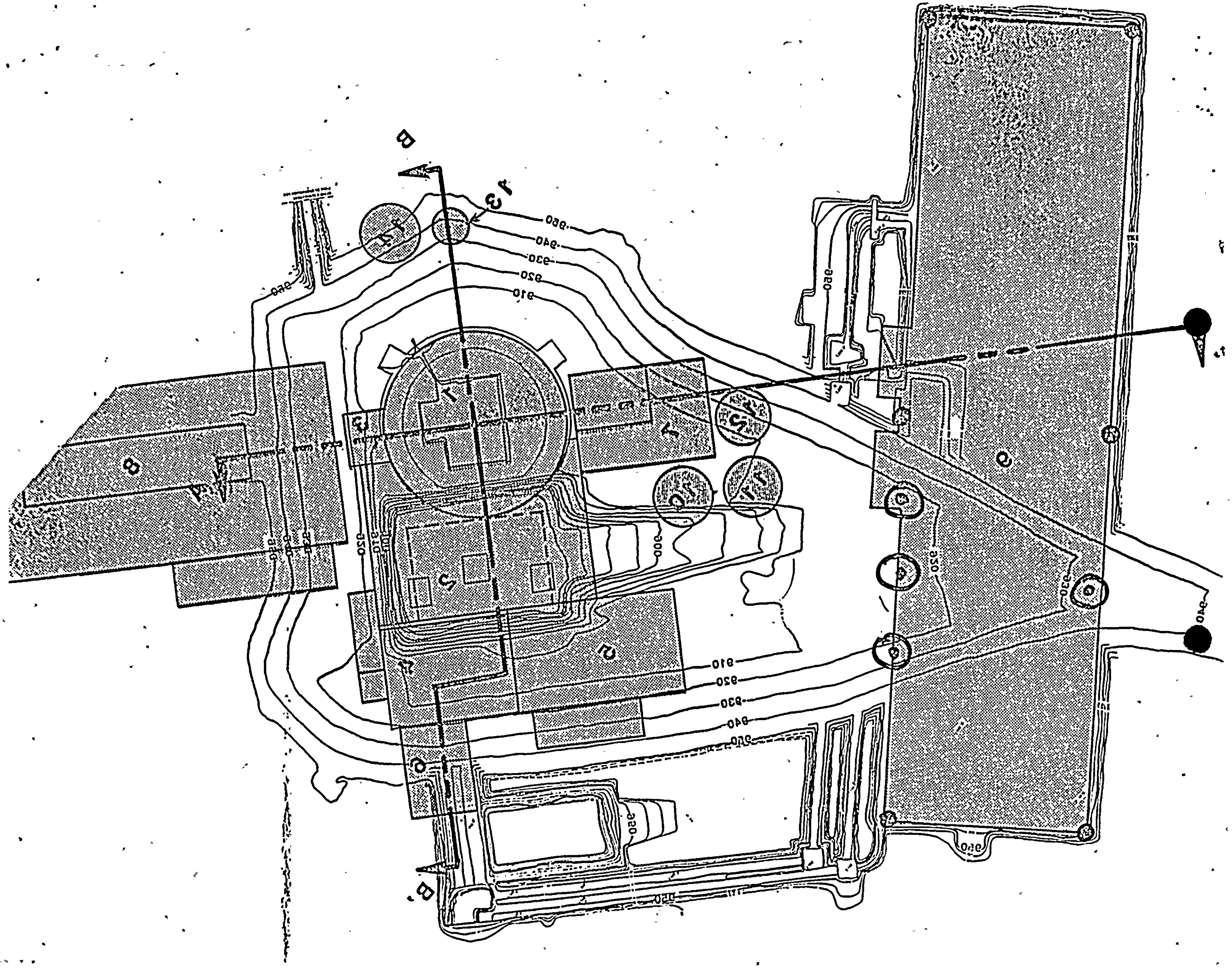


UNIT 1 ACTUAL



UNIT 2 ACTUAL



Attachment 2

TABLE 2.5-18

SUBSURFACE INSTRUMENTATION DETAILS

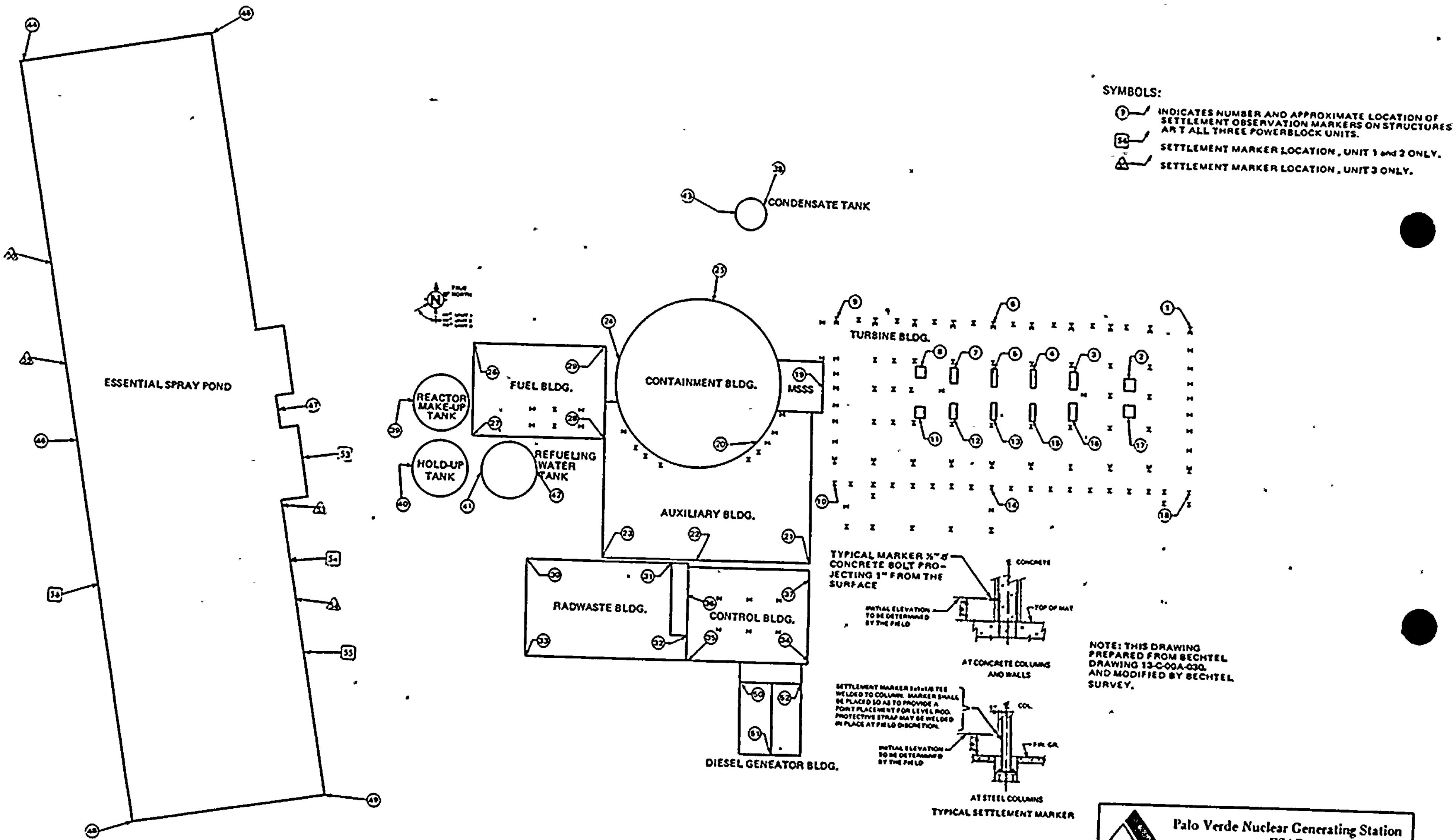
INSTRUMENTATION SYSTEM NAME	RESPONSE BEING MONITORED			MINIMUM SPECIFIED MONITORING FREQUENCY-TIME BETWEEN READINGS	
	HEAVE RESULTING FROM EXCAVATION	RECOMPRESSION DUE TO STRUCTURAL AND BACKFILL LOADING	REGIONAL SUBSIDENCE		
Extensometers (MPE, E)	X	X	X	1) Pre-excavation, excavation, pouring of overlying foundations.	-1 week
				2) The following 18 months	-1 month
				3) Until end of construction	-3 months
				4) For a 3 year period following end of construction[1]	-6 months
				5) After last 6 month reading	-5 years[3]
Mechanical Rebound Anchors (MRA)	X			1) Pre-excavation, excavation	-1 week
Settlement Markers		X	X	1) 18 months following first concrete placement for given structure	-1 month
				2) Until end of construction of the last major powerblock structure.	-3 months
				3) For a 3 year period following end of construction[2]	-6 months
				4) After last 6 month reading	-5 years[3]
Subsidence Monitoring Network			X	1) During construction	-1 year
				2) After end of construction	-5 years[3]

[1]End of construction is defined as the last major concrete placement in a given powerblock unit.

[2]The settlement markers for the essential spray ponds shall be monitored at 6-month intervals for a period of at least 3 years after the ponds are initially filled.

[3]After the first 5 year reading (8 years after end of construction), the owner may terminate the monitoring program either totally or at selected points, based on the need for continuing data.

Attachment 3



Palo Verde Nuclear Generating Station
FSAR

SETTLEMENT MARKER LOCATION PLAN

FIGURE 2

[illegible]

physical features. A steel liner plate with a leak chase system covers the entire interior base mat. A concrete slab 2 feet 9 inches thick is cast in place on top of the liner plate to protect the liner against damage during erection and maintenance and to reduce the thermal effects in the base mat. The added concrete slab also provides the foundation for some small equipment and steel columns so that their anchorage does not have to penetrate the liner plate underneath.

3.8.5.1.2 Other Seismic Category I Structures

A continuous reinforced concrete slab is used as foundation for each of the other Seismic Category I structures. Foundations of Seismic Category I structures are separated from one another and from other structures, either by using an expansion joint or by providing an adequate structural gap in between. The size and thickness of foundations vary with each individual Seismic Category I structure. 2

3.8.5.2 Applicable Codes, Standards, and Specifications

Refer to section 3.8.1.2 for the containment and to section 3.8.3.2 for other Seismic Category I structures.

3.8.5.3 Loads and Load Combinations

Containment foundation loads and loading combinations are discussed in sections 3.8.1.3.

Foundation loads and loading combinations for other Seismic Category I structures are discussed in section 3.8.4.3.

No special measures are required in the design and construction of foundations for Category I structures to alleviate the effects of ground subsidence. As discussed in section 2.5.4.1.1, subsidence, if any, will be negligible.

PROCEDURES for DETERMINING LATERAL EARTH PRESSURE LOADINGS FOR ALL CATEGORY I FOUNDATIONS ARE ~~LISTED~~ presented IN APPENDIX 3H.

[illegible]

APPENDIX 3H

LATERAL EARTH PRESSURE ON FOUNDATION AND RETAINING WALLS

LATERAL EARTH PRESSURE ON FOUNDATIONS AND RETAINING WALLS

The total lateral earth pressure on foundation and retaining walls shall be based on the sum of the appropriate static and dynamic lateral forces. Static forces shall be based either on the active case (P_A) for the case of a retaining wall free to rotate and translate, or on the compacted backfill case (P_B) for the case of a rigid foundation wall. Dynamic forces shall be based on the dynamic increment for a level backfill condition (P_{AE}) plus the surcharge effect (P_{AES}), if applicable.

A. Static conditions:

	Equivalent Fluid Unit Weight (lb/ft^3) (Horizontal Backfill)	
Case	Above Water Table	Below Water Table
Active (P_A)	36	19
Passive (P_P)	228	118
Backfill (P_B)	90	47

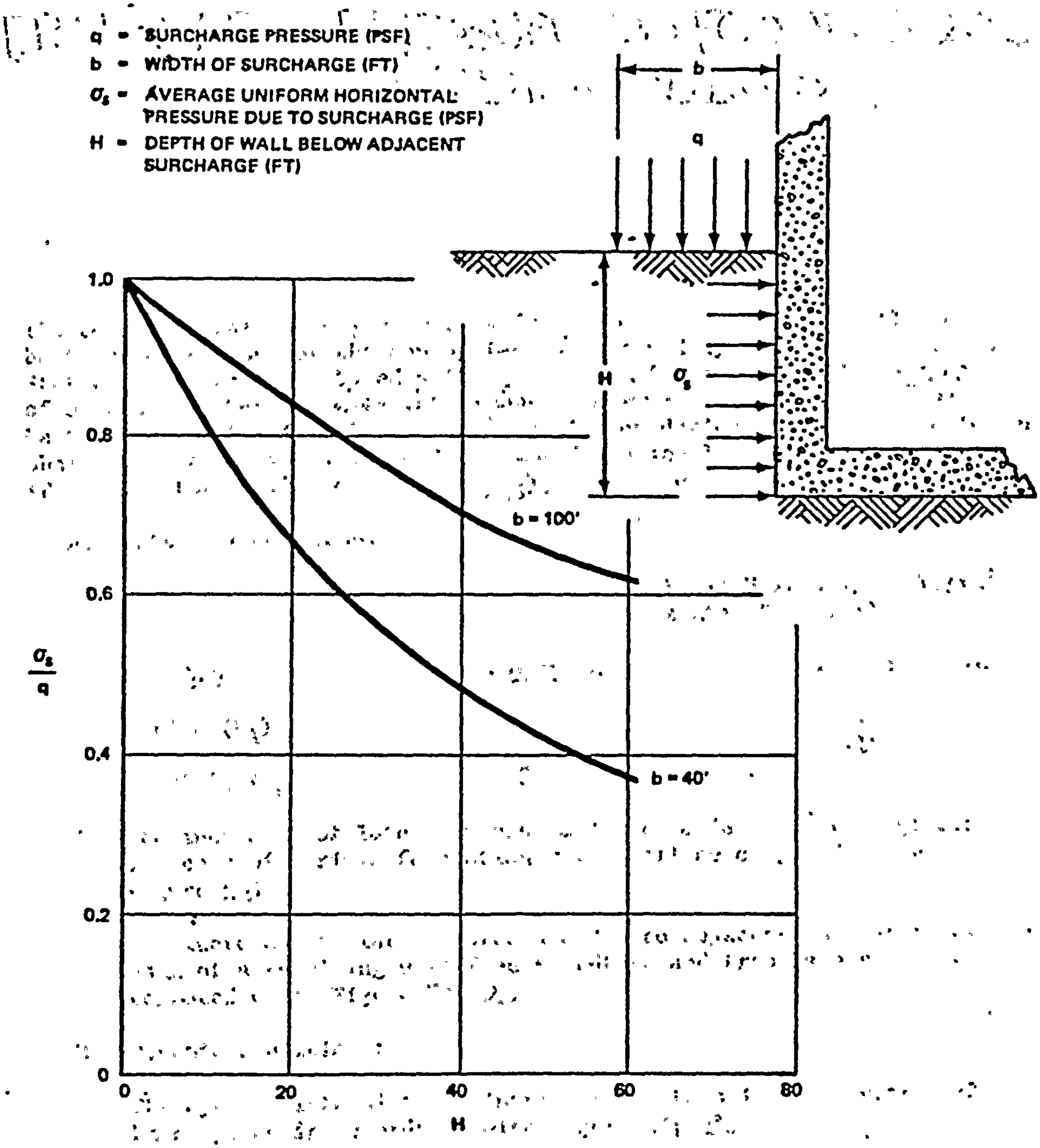
The increment of lateral pressure due to adjacent surcharge for the case of a rigid foundation wall shall be computed using Figure 3H-1.

The increment of lateral pressure due to adjacent surcharge for the case of a retaining wall free to rotate and translate shall be computed using Figure 3H-2.

B. Dynamic conditions:

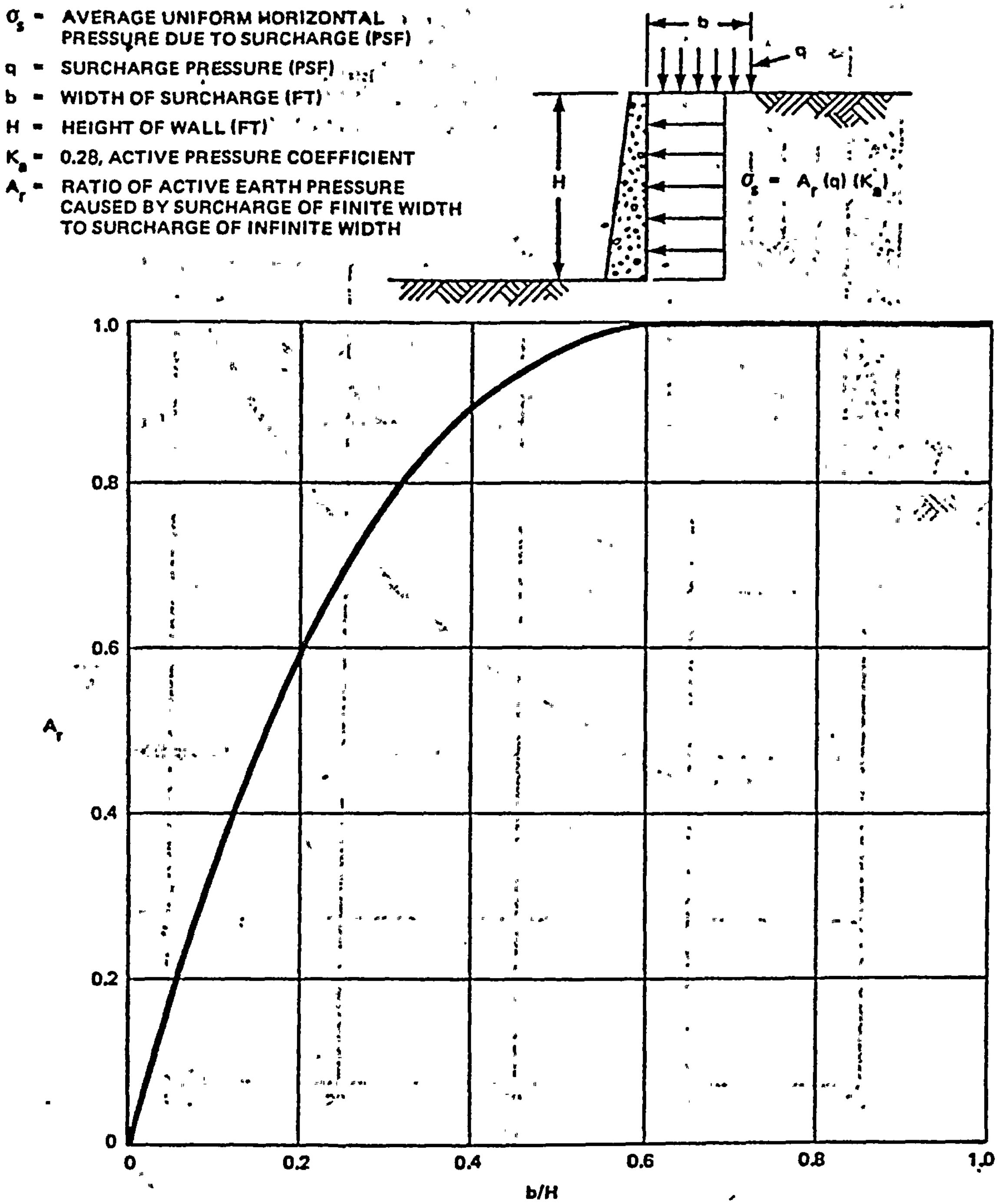
The dynamic lateral force increment due to seismic effects shall be computed in accordance with Figure 3H-3.

The total dynamic lateral force increment, P_{AE} or $P_{AE} + P_{AES}$, shall be added to the lateral force calculated for either the active case or the compacted backfill case. The lateral force is calculated in the usual manner and includes hydrostatic pressure if a water table is present. The dynamic lateral force increments are independent of the water table.



AVERAGE LATERAL PRESSURE DUE TO ADJACENT SURCHARGE FOR RIGID RETAINING WALL

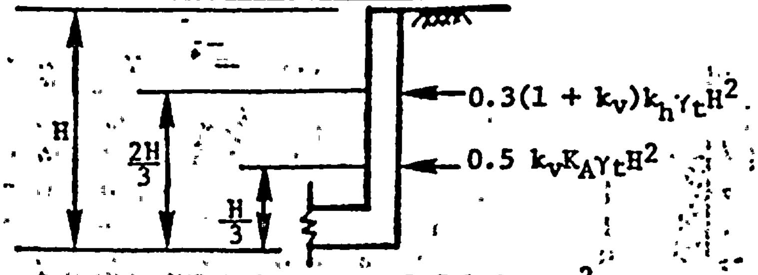
FIGURE 3H-1



AVERAGE LATERAL PRESSURE DUE TO ADJACENT SURCHARGE FOR RETAINING WALL FREE TO ROTATE

FIGURE 3H-2

Level Backfill Condition



$$P_{AE} = 0.5\ k_v K_A \gamma_t H^2 + 0.3(1 + k_v)k_h \gamma_t H^2$$

where: P_{AE} = dynamic lateral force increment due to seismic effects (lb)

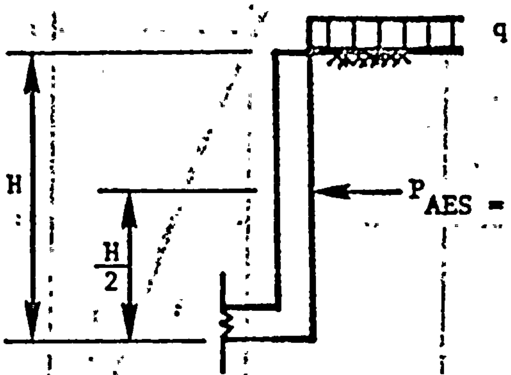
K_A = 0.28, coefficient of active lateral earth pressure.

k_h = k_v = 0.25 under SSE conditions, and
0.13 under OBE conditions

H = wall height (ft)

γ_t = total unit weight (129 lb/ft^3)

Surcharge Effect



$$P_{AES} = qH[k_v K_A + 0.6\ k_h(1 + k_v)]$$

where: P_{AES} = dynamic lateral force increment due to adjacent surcharge loading (lb)

q = Surcharge pressure (lb/ft^3) (lb/SF)

H = Depth of wall below adjacent surcharge (ft)

K_A = 0.28

k_h = k_v = 0.25 under SSE conditions, and
0.13 under OBE conditions

DYNAMIC LATERAL FORCE INCREMENT DUE TO SEISMIC EFFECTS

FIGURE 3H-3

APPENDIX 3H REFERENCES

1. Nazarian, H.N. and Hadjian, A.H., (1979) "Earthquake Induced Lateral Soil Pressures on Structures," Journal of the Geotechnical Engineering Division, ASCE Vol. 105, No. GT9, September 1979, pp. 1049-1066.

2. Wood, J.H., (1973) "Earthquake Induced Soil Pressures on Structures," PhD Thesis, California Institute of Technology, EERL 73-05 Pasedena, California.

ATTACHMENT 6

UPDATED STABILITY ANALYSIS FOR DYNAMIC LATERAL EARTH PRESSURES

The potential effects of hypothetical weak zones along the backfill to native soil interfaces on dynamic lateral earth pressures were evaluated by pseudo-static analyses. Three cases, depicted in Figure A-1, were analyzed. These cases represent the most adverse lateral loading conditions for Category 1 Structures.

The analyses were based on a comparison of peak inertial forces induced by simulated SSE conditions and the combined lateral resistance offered by the wall being analyzed and the assumed zone of weakness. The horizontal force equilibrium can be represented by:

$$F_I = P + R$$

or

$$R = F_I - P$$

where F_I = Combined inertial forces due to horizontal acceleration of backfill and structures supported on backfill adjacent to wall that is being analyzed.

P = Maximum allowable lateral load on wall (values provided by Bechtel)

R = Horizontal component of resistive shearing force that needs to be mobilized along the assumed zone of weakness to provide equilibrium.

The combined inertial force, F_I, was computed as follows:

$$F_I = (\Sigma H_i^2)^{1/2}$$

where H_i represents the individual peak inertial forces. These individual peak horizontal loads were derived from dynamic analy-

ses of simulated SSE conditions. This procedure is commonly used in structural dynamics modal superposition.

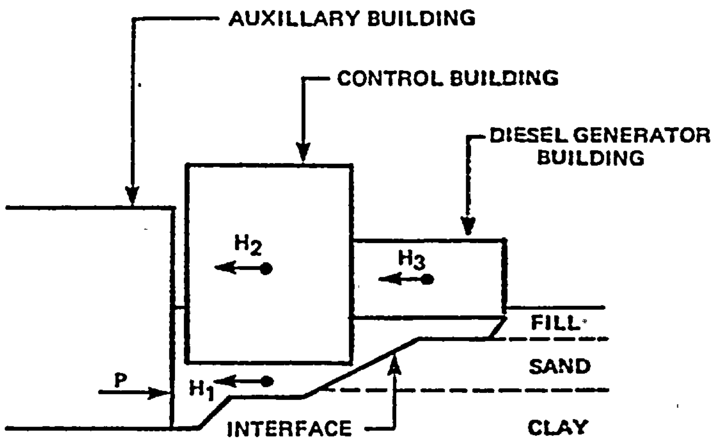
The results of the analysis as well as the loading conditions that were analyzed are presented in Table A-1. It can be seen that in all three cases the shear stress that needs to be mobilized along the backfill to native soil interface is a small fraction of the full shear strength of the soils involved. This confirms that the dynamic lateral earth pressures would not be significantly affected by the assumed presence of weak zones along the backfill to native soil interfaces. Furthermore, as noted in our original response to Question No. 241.3, there is no reason to suspect the presence of such weak zones along the backfill to native soil interfaces at the Palo Verde Nuclear Generating Station.

TABLE A-1

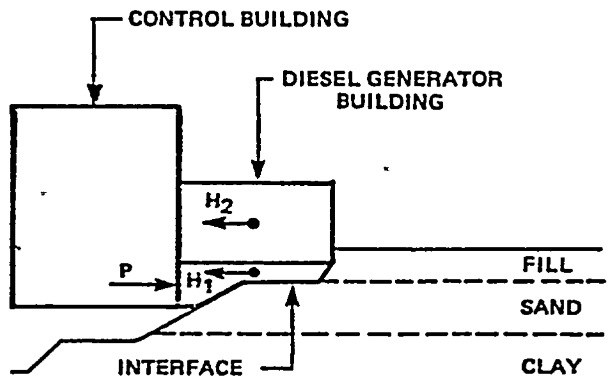
SUMMARY OF LOADING CONDITIONS AND CALCULATED RESULTS

Case No. (Fig. A-1)	H_1	H_2	H_3	$\sqrt{\Sigma H_i^2}$	P	Required Interface Strength c(ksf)	ϕ
Case I	71	92	74	138	46	0.20	*30°
Case II	26	74	--	78	27	--	19°
Case III	117	65	15	135	118	0.24	*30°

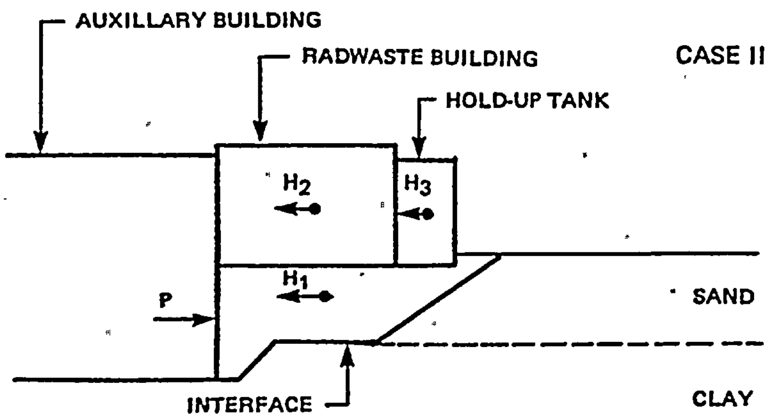
*ϕ - values conservatively assumed to compute c required.



CASE I AUXILLARY BUILDING WALL ADJACENT TO CONTROL BUILDING



CASE II CONTROL BUILDING WALL ADJACENT TO THE DIESEL GENERATOR BUILDING



CASE III AUXILLARY BUILDING WALL ADJACENT TO RADWASTE BUILDING

FIGURE A-1 CASES ANALYZED

ATTACHMENT 7
ADDITIONAL SHAKE ANALYSES

Two computer analyses were carried out using the one-dimensional, equivalent linear response program SHAKE (Schnabel, Lysmer & Seed, 1972) to verify that a surface motion derived from a deconvolved deep motion would be the same as the original surface motion in the SHAKE analysis. These analyses were performed to confirm that the depth of the soil model used in the SHAKE analyses had no affect on response calculation, performed as part of the liquefaction studies. A brief discussion of the theory together with input and results of these two runs are presented below.

SHAKE. The computer program SHAKE utilized an equivalent linear analysis procedure with response computed in the frequency domain. For a given control motion at a specific depth, motions at any other depth are defined by means of transfer functions. After a set of strain-dependent, equivalent-linear dynamic shear modulus and damping values are is defined, a transfer function between the control motion at any other point within the soil column can be uniquely determined by the mass density, soil column thickness and the equivalent linear dynamic shear modulus and damping values of the soil between the two points. In other words, for a given set of equivalent linear soil properties, the SHAKE analyses are essentially a linear elastic system.

Input Data. Two additional computer solutions were carried out to demonstrate that the motion is uniquely defined. The procedure in which this demonstration was performed is shown schema-

tically in Figure B-1. The properties of the 100-foot soil column, as summarized in Table B-1, are identical to properties utilized in previous studies (see original response to NRC Question No. 241.2).

During the analyses the Bechtel artificial earthquake record scaled to 0.2g was assigned at the surface in the first run. The base motion obtained at the 100-foot depth in the first run was then fed back into the base of the soil column in the second run to obtain another surface motion. The original input motion and the output (surface) motion from the second run were subsequently compared in terms of time histories of acceleration, response spectra for 2 and 5 percent damping, and induced shearing stress.

Results. Table B-2 and Figure B-2 show the comparisons for response spectra and time histories respectively. A comparison of the earthquake-induced shearing stress profile for both runs is given in Table B-3. These results clearly show that the output surface acceleration from the second run is essentially identical to the input Bechtel motion in every respect. Likewise, the induced shearing stress profiles are essentially identical.

It is concluded from this study that the surface motion derived from a deconvolved deep motion is the same as the original surface motion in the SHAKE analysis. Hence, the depth of the soil model had no effects on the ground response calculations.

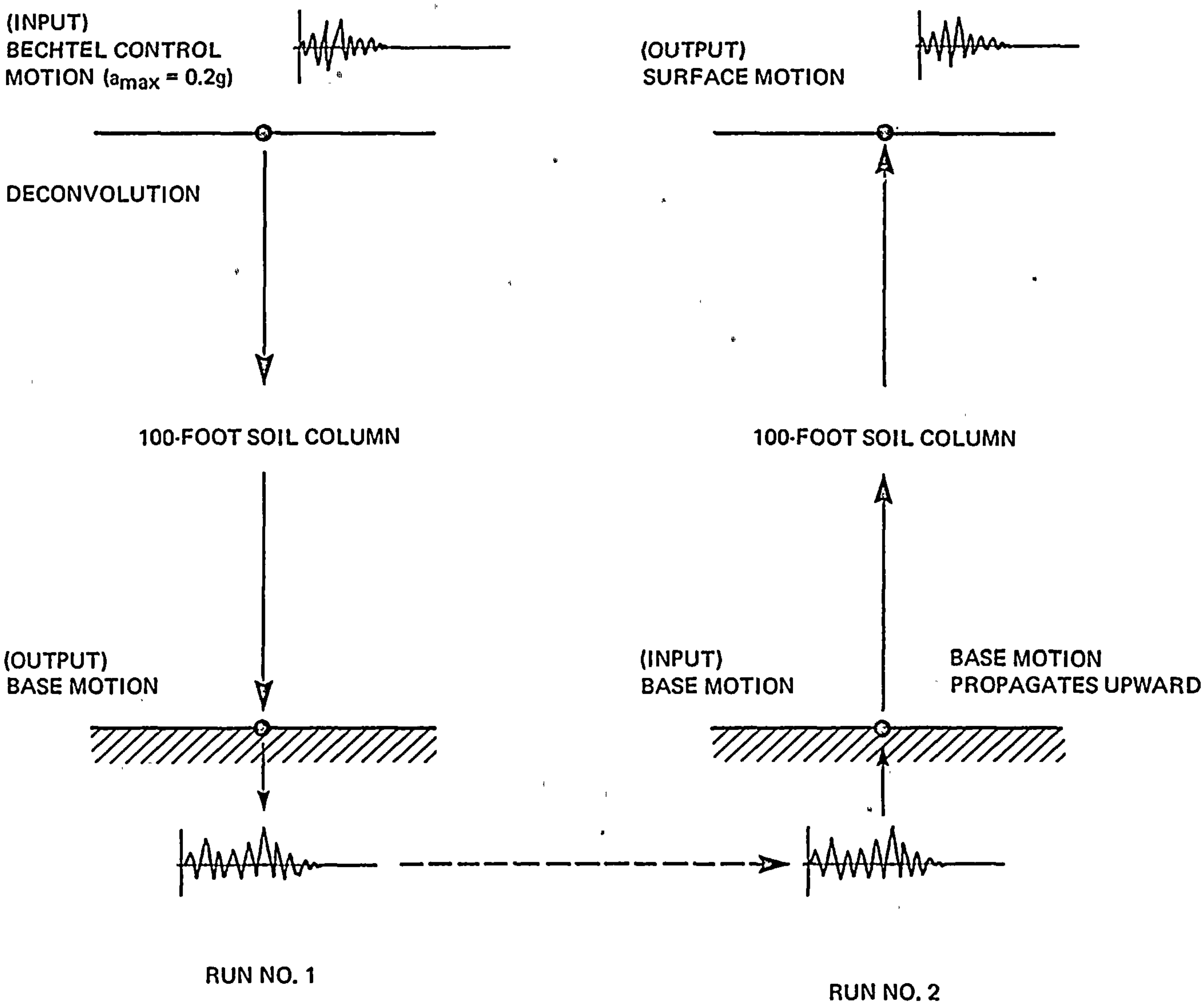
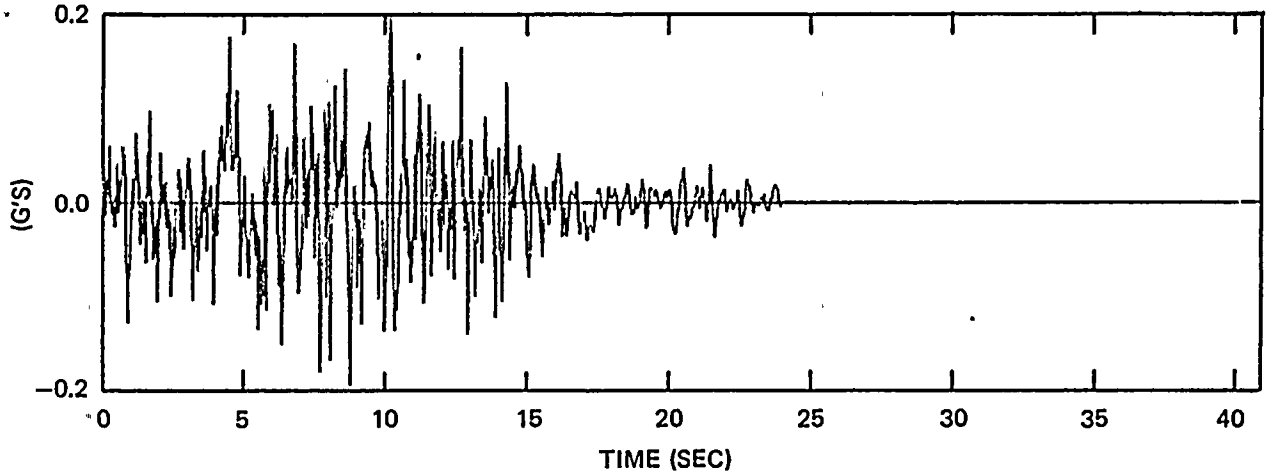
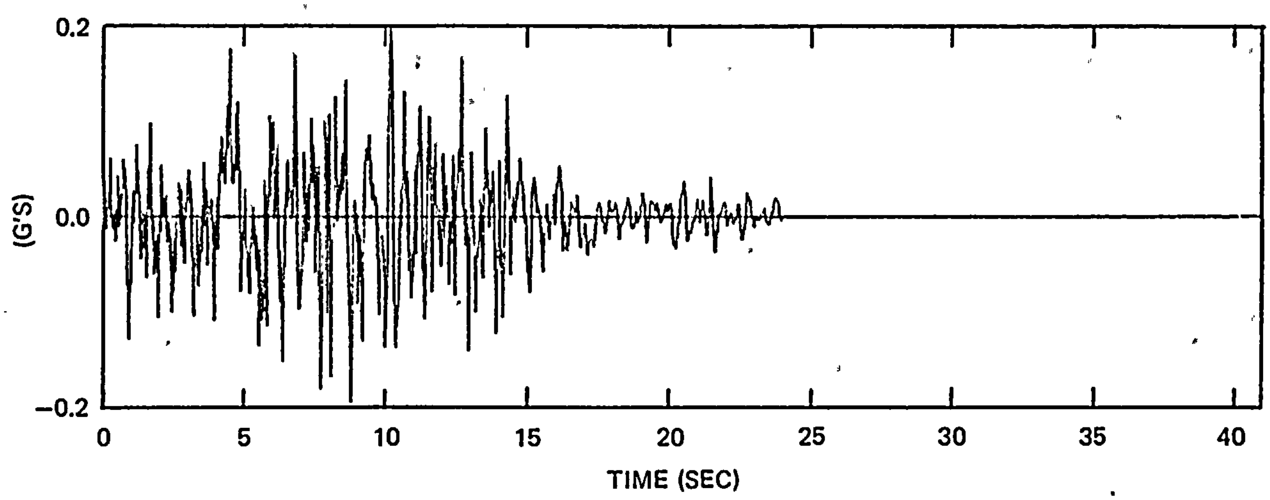


FIGURE B-1 ADDITIONAL SHAKE ANALYSES



(A) BECHTEL INPUT MOTION AT GROUND SURFACE (RUN NO. 1)



(B) OUTPUT SURFACE MOTION IN RUN NO. 2 OF SHAKE

FIGURE B-2 COMPARISON OF TIME HISTORIES

TABLE B–1

SOIL MODEL LIQUEFACTION POTENTIAL ANALYSES–UNIT 3

LAYER NO.	LAYER THICKNESS (FT.)	SOIL TYPE[1]	TOTAL UNIT WEIGHT (PCF)	LOW--STRAIN SHEAR MODULUS x 10^6 (PSF)
1	5	2	115	3.76
2	5	2	115	3.76
3	5	2	121	3.76
4	5	2	121	3.76
5	5	2	128	6.68
6	5	2	131	6.68
7	5	2	129	6.68
8	5	2	128	6.68
9	5	2	128	6.68
10	5	1	123	6.68
11	5	1	123	6.68
12	5	1	123	6.68
13	5	1	123	5.98
14	5	1	123	5.98
15	5	1	123	5.98
16	5	1	124	5.98
17	5	1	124	6.03
18	5	1	124	6.03
19	5	1	124	6.03
20	5	1	124	6.03
21	5	1	125	6.57

[1] SOIL TYPE: 1–CLAYS AND SILTY CLAYS; 2–SAND AND SILTY SAND

TABLE B-2

RESPONSE SPECTRUM COMPARISON (5% DAMPING)

Run # 1 Input Bechtel Motion (Amax = 0.2g)		Run # 2 Output Surface Motion (Amax = 0.2g)	
Period (Sec)	Absolute Acceleration(g)	Period (Sec)	Absolute Acceleration(g)
0.0010	0.2000	0.0010	0.2000
0.0692	0.3215	0.0692	0.3214
0.0832	0.3338	0.0832	0.3337
0.1000	0.4095	0.1000	0.4095
0.1202	0.4720	0.1202	0.4720
0.1445	0.4445	0.1445	0.4445
0.1738	0.5178	0.1738	0.5178
0.2089	0.4980	0.2089	0.4980
0.2512	0.5887	0.2512	0.5887
0.3020	0.5899	0.3020	0.5899
0.3631	0.5819	0.3631	0.5819
0.4365	0.5379	0.4365	0.5380
0.5248	0.5182	0.5248	0.5182
0.6310	0.4380	0.6310	0.4380
0.7586	0.4012	0.7586	0.4013
0.9120	0.3186	0.9120	0.3186
1.0965	0.2806	1.0965	0.2805
1.3183	0.2408	1.3183	0.2408
1.5849	0.2200	1.5849	0.2199
1.9055	0.1913	1.9055	0.1913
2.2909	0.1620	2.2909	0.1619
2.7542	0.1283	2.7542	0.1283
3.3113	0.1071	3.3113	0.1071
3.9811	0.0858	3.9811	0.0858
4.7863	0.0653	4.7863	0.0653
5.7544	0.0400	5.7544	0.0400
6.9183	0.0306	6.9183	0.0306
8.3176	0.0229	8.3176	0.0229
10.0000	0.0119	10.0000	0.0118

TABLE B-3 STRESS AND STRAIN PROFILE COMPARISON

LAYER	SOIL TYPE[1]	THICKNESS (FT)	DEPTH (FT)	RUN 1 MAX SHEAR STRAIN (%)	RUN 1 MAX SHEAR STRESS (PSF)	RUN 2 MAX SHEAR STRAIN (%)	RUN 2 MAX SHEAR STRESS (PCF)
1	2	5.0	2.5	0.00169	57.38	0.00169	57.34
2	2	5.0	7.5	0.00522	171.72	0.00521	171.63
3	2	5.0	12.5	0.00859	288.07	0.00859	287.97
4	2	5.0	17.5	0.01232	409.22	0.01232	409.16
5	2	3.0	22.5	0.01355	532.59	0.01355	532.59
6	2	6.0	28.0	0.01347	667.06	0.01347	667.09
7	2	4.0	33.0	0.01450	785.47	0.01450	785.48
8	2	5.0	37.5	0.01611	888.54	0.01611	888.51
9	2	5.0	42.5	0.01820	998.03	0.01820	997.94
10	1	5.0	47.5	0.01808	1102.08	0.01808	1101.91
11	1	5.0	52.5	0.02012	1200.67	0.02012	1200.46
12	1	5.0	57.5	0.02238	1293.22	0.02238	1293.00
13	1	5.0	62.5	0.02462	1379.03	0.02462	1378.85
14	1	5.0	67.5	0.02685	1456.10	0.02684	1455.96
15	2	5.0	72.5	0.03857	1514.01	0.03856	1513.93
16	1	5.0	77.5	0.03030	1577.41	0.03030	1577.31
17	1	5.0	82.5	0.03173	1621.55	0.03173	1621.44
18	1	5.0	87.5	0.03291	1655.41	0.03291	1655.25
19	1	5.0	92.5	0.03374	1679.87	0.03373	1679.67
20	1	5.0	97.5	0.03429	1696.01	0.03428	1695.78

[1]SOIL TYPE: 1– CLAYS AND SILTY CLAYS; 2– SAND AND SILTY SAND