TENNESSEE VALLEY AUTHORITY
CHATTANOOGA, TENNESSEE 37401
400 Chestnut Street Tower II

August 28, 1981

RECEIVED
AUG 31 1981
U.S. NUCLEAR REGULATORY COMMISSION

Director of Nuclear Reactor Regulation
Attention: Ms. E. Adensam, Chief
Licensing Branch No. 4
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Ms. Adensam:

In the Matter of the Application of	)	Docket Nos. 50-390
Tennessee Valley Authority	)	50-391

Enclosed is TVA's response to NRC requests for information concerning the potential for liquefaction of soils at the Watts Bar Nuclear Plant site. Included are responses to NRC questions 362.29 through 362.35 provided by R. L. Tedesco's letter to H. G. Parris dated March 30, 1981 and question 362.36 provided by R. L. Tedesco's letter to H. G. Parris dated May 7, 1981.

If you have any questions concerning these responses, please get in touch with D. L. Lambert at FTS 857-2581.

Very truly yours,

TENNESSEE VALLEY AUTHORITY

L. M. Mills

L. M. Mills, Manager
Nuclear Regulation and Safety

Sworn to and subscribed before me
this 28th day of August 1981

Paulette H. White
Notary Public
My Commission Expires 9-5-84

Enclosure

B001
s
1/1

Question 362.29 (2.5.4)

The measured settlement data given in Figures Q362.19-1 through Q362.19-5 of the FSAR is provided only up to June 1978. Provide time vs settlement plots of up-to-date settlement data obtained for all Category I structures where settlements are being monitored. Tabulate values of the measured maximum differential settlements and show comparisons of the measured data with anticipated settlements assumed in the analysis of these structures and their appurtenances, and evaluate the impact of any differences between the measured and anticipated settlements on the design and construction of these structures and appurtenances. Staff requires that the settlement of safety related structures and appurtenances be monitored for a period of at least five years after the issuance of the operating license and the impact of observed settlement, if any, on the design limits of Category I structures be evaluated periodically.

Response

The time vs settlement plots of Unit 1 and 2 Reactor Building of Figure Q362.19-1 and Q362.19-2 reflect the latest data available. Readings were discontinued June 1978, because settlement stations became inaccessible. The updated time vs settlement plots are provided in Figures Q362.29-1 and -2 for the Auxiliary-Control Building, the Diesel Generator Building, and the Intake Pumping Station.

Tables Q362.29-1 through Q362.29-3 provide all the maximum and minimum movements for all the settlement stations in Category I structures. The differential settlement readings for the rock supported structures are provided in Table Q362.29-4. Settlement stations location are provided in Figure 3.8.4-66 and 3.8.4-67. The maximum settlement of .057 feet and the maximum differential settlement of .038 feet between the Reactor Building Unit 1 and the Auxiliary Building were recorded on August 3, 1977. This maximum value is virtually unchanged through April 1980. The measured differential settlement of .060 feet between settlement stations (SS) 18 and 23 was judged to be a measurement error for three reasons. First the differential settlements one month before and after were recorded to be .008 feet and .024 feet respectively, second the latest reading between SS18 and SS23 was recorded to be .018 feet of differential settlement, and third the maximum settlement recorded a year before and after the error was .033 feet between SS18 and SS23.

The measured settlements have not approached the design criteria of 1 inch of differential settlement between buildings or 1 to 2 inches of total settlements with respect to the surrounding area. In general the maximum settlements of rock-supported structures had occurred by 1977, and thereafter the settlements have been stable or decreasing.

362.29-1

For the Auxiliary-Control Building and the Intake Pumping Station readings were discontinued April 29, 1980. The Diesel Generator Building is a soil supported structure and is still being monitored. We have fulfilled our commitment of monitoring rock supported structures since the structure loading is essentially complete on all rock supported buildings, all the total and differential settlements are well within the design criteria allowables, and settlements have not increased in the rock supported structures during the past 2 years of monitoring.

Based on our evaluation, the total and differential settlements are not significant; there are no trends being exhibited; there has been no adverse structural performance; and there are not any anticipated problems from the settlement of Category I structures.

TABLE Q362.29-1

SETTLEMENT STATION READINGS

FOR THE REACTOR BUILDING UNITS 1 AND 2

AND THE AUXILIARY-CONTROL BUILDING

Settlement Station	Most Recent Reading *Settlement (Feet)	Most Recent Reading Date	Maximum Downward Movement (Feet)	Maximum Downward Date	Maximum Upward Movement (Feet)	Maximum Upward Date	Initial Reading Date
1	-0.002	02-02-74	-----	-------	-----	----------	12-17-73
1A	-0.008	03-03-78	0.025	08-03-77	0.012	06-06-77	03-17-75
1B	-0.014	04-29-80	0.016	12-09-76	0.015	08-10-76	10-31-75
2	-0.038	04-29-80	0.050	08-03-77	Note A	02-20-74	02-20-74
2A	-0.018	09-28-78	0.035	08-03-77	0.004	11-24-75	12-18-74
3	-0.028	04-29-80	0.030	08-03-77	0.006	07-15-74	04-15-74
4	-0.013	09-29-78	0.023	08-04-78	0.007	02-11-76	01-14-76
5	-0.021	07-10-78	0.035	08-03-77	0.012	07-14-76	02-19-75
6	-0.018	07-10-78	0.027	08-03-77	0.013	07-14-76	01-20-75
7	-0.024	06-05-78	0.042	08-03-77	0.00	07-14-76	02-19-75
8	-0.019	08-04-78	0.019	12-15-77	0.005	10-13-76	07-14-76
9	-0.044	04-28-80	0.044	04-28-80	Note A	02-11-76	02-11-76
10	-0.056	04-29-80	0.057	08-03-77	0.003	07-15-74	03-18-74
11	-0.040	04-29-80	0.041	08-03-77	Note A	02-11-76	02-11-76
12	-0.019	08-04-78	0.020	11-07-77	0.007	08-10-76	07-14-76
13	-0.035	03-03-78	0.050	08-03-77	Note A	09-16-76	09-16-76
14	-0.024	03-03-78	0.037	08-03-77	0.004	02-19-75	10-17-74
15	-0.016	10-11-77	0.042	08-03-77	0.004	07-21-75	09-16-74
16	-0.014	04-08-79	0.017	03-03-78	0.011	02-11-76	01-14-76
17	-0.015	04-09-79	0.038	08-03-77	Note A	04-15-74	04-15-74
18	-0.010	04-09-79	0.036	08-03-77	0.018	01-14-76	10-16-73
19	-0.018	04-25-80	0.041	08-03-77	0.007	12-17-73	11-19-73
20	+0.005	04-25-80	0.029	08-03-77	0.029	01-09-78	10-16-73
21	-0.028	04-25-80	0.053	08-03-77	Note A	10-17-74	10-17-74
22	0.000	04-28-80	0.023	08-03-77	0.022	02-02-78	08-18-75
23	+0.021	04-24-80	0.000	09-16-74	0.045	02-10-77	09-16-74

*Positive settlement is up.

TABLE Q362.29-2

SETTLEMENT STATION READINGS

FOR THE INTAKE PUMPING STATION

Settlement Station	Most Recent Reading *Settlement (Feet)	Most Recent Reading Date	Maximum Downward Movement (Feet)	Maximum Downward Date	Maximum Upward Movement (Feet)	Maximum Upward Date	Initial Reading Date
1	-0.004	06-06-77	0.009	05-10-77	0.036	05-11-76	10-17-74
1A	-0.010	04-23-80	0.026	02-02-79	0.007	07-10-78	03-15-77
2	-0.010	06-06-77	0.018	05-10-77	0.013	03-15-77	10-17-74
3	+0.001	06-06-77	0.018	03-21-75	0.011	05-09-75	12-19-74
3A	-0.011	04-23-80	0.032	02-02-79	0.003	07-10-78	08-03-77
4	-0.002	04-23-80	0.019	02-02-79	0.012	07-10-78	03-15-77

*Positive settlement is up.

TABLE Q362.29-3

SETTLEMENT STATION READINGS

FOR THE DIESEL GENERATOR BUILDING

Settlement Station	Most Recent Reading *Settlement (Feet)	Most Recent Reading Date	Maximum Downward Movement (Feet)	Maximum Downward Date	Maximum Upward Movement (Feet)	Maximum Upward Date	Initial Reading Date
1	-0.049	04-10-81	0.049	04-10-81	Note A	11-24-75	11-24-75
2	-0.049	04-10-81	0.049	04-10-81	0.003	12-16-75	10-31-75
3	-0.045	04-10-81	0.045	04-10-81	Note A	11-24-75	11-24-75
4	-0.040	04-10-81	0.040	04-10-81	0.005	12-16-75	10-31-75

*Positive settlement is up.
Note A: The Initial Reading was the maximum upward value.

TABLE Q362.29-4

DIFFERENTIAL SETTLEMENT BETWEEN ROCK SUPPORTED STRUCTURES

	Settlement Station	Initial Reading		Maximum Differential Settlement				Most Recent Differential Settlement			
		Date	Elevation (Feet)	Date	Elevation (Feet)	S (Feet)	ΔS (Feet)	Date	Elevation (Feet)	S (Feet)	ΔS (Feet)
Auxiliary Control Building and Turbine Building Settlement Stations	SS20	10-17-74	693.972	04-25-80	693.975	+0.003	0.031	04-25-80	693.975	+0.003	0.031
	SS21	10-17-74	710.006	04-25-80	709.978	-0.028		04-25-80	709.978	-0.028	
	SS19	08-18-75	694.027	08-04-78	694.042	+0.015	0.008	04-25-80	694.028	+0.001	0.001
	SS22	08-18-75	709.999	08-04-78	710.006	+0.007		04-25-80	709.999	0.000	
	SS18	09-16-74	694.032	06-11-76	694.022	-0.010	0.037*	10-19-79	694.029	-0.003	0.018
	SS23	09-16-74	709.840	06-11-76	709.867	+0.027		10-18-79	709.855	+0.015	
Reactor Building Unit 1 and Auxiliary Building Settlement Stations	SS15	01-14-76	704.764	08-03-77	704.726	-0.038	0.038	10-11-77	704.752	-0.012	0.000
	SS16	01-14-76	728.980	08-03-77	728.980	0.000		10-11-77	728.968	-0.012	
	SS12	07-14-76	728.995	01-12-77	728.991	-0.004	0.013	09-12-77	728.988	-0.007	0.010
	SS13	07-14-76	704.787	01-12-77	704.770	-0.017		09-12-77	704.770	-0.017	
Reactor Building Unit 2 and Auxiliary Building Settlement Stations	SS4	01-14-76	729.033	07-14-76	729.031	-0.002	0.029	07-10-78	729.015	-0.018	0.005
	SS5	01-14-76	705.284	07-14-76	705.311	+0.027		07-10-78	705.271	-0.013	
	SS7	07-14-76	705.336	12-09-76	705.300	-0.036	0.028	11-07-77	705.299	-0.037	0.018
	SS8	07-14-76	729.034	12-09-76	729.026	-0.008		11-07-77	729.015	-0.019	

*This is the second highest differential settlement for SS18 and SS23, the highest is peculiarly high in August of 1977.

S=Settlement ΔS=Differential Settlement

Question 362.30(2.5.4)

Indicate how much settlement of the structures has occurred since the connections between structures and safety-related utilities were made. Evaluate the effect of the past and anticipated future setttlement of structures on safety related utility connections.

362.30 Response

Connections between the structures and the safety related utilities were made at various times. The ERCW piping connections were made between November 1977 and June 1978. Safety-related IE electrical conduits were connected to structures from June 1976 to March 1978. The past settlement performance of the structures are provided in Question 362.29. The anticipated future settlement is expected to be less than 1 inch for the structures.

Direct settlement recordings of the safety-related utilities were not made. It is anticipated that very little differential settlement will occur at the connections for the following reasons:

1. When interfacing with the structures, the electrical conduit banks rest on reinforced concrete brackets, and the brackets prevent differential settlement at the interface.

2. The ERCW pipes enter the Diesel Generator Building (DGB) through an encasement that rests on reinforced concrete brackets at the interface with the DGB. Similiar to the electrical conduits, the brackets will prevent differential settlement at the interface.

3. The ERCW pipes enter the Auxiliary-Control Building through a pipe tunnel approximately 200 feet long, which rests on in situ gravel and eliminates any differential settlement problems.

4. The electrical conduit banks and ERCW pipes have a pile supported concrete slab to alleviate any differential settlement at the Intake Pumping Station.

5. When an ERCW pipe connects with a structure, a 2 to 6 inch clearance is maintained between the pipe and sleeve. The clearance if filled with a flexible watertight sealant.

6. The total settlements of the structures are less than 1 inch which means that the structures will not cause any significant differential settlements at the connections.

Differential settlements of the connections between structures and safety-related utilities is not anticipated to result in any significant problems.

362.30-1

Question:

362.31 (2.5.4) Your response to Question Number 371.23 indicates that you are relying on proper performance of weep holes to maintain water level at elevation 685 for retaining walls at the intake pumping station and that, based on the performance of weep holes, you have used this water elevation in the design of retaining walls. Provide the following information:

(i) The factors of safety for sliding and overturning of the walls based on water elevation of 685. Please provide analysis method and bases for assumptions made in the analysis.

(ii) The safety factors in the design of retaining walls, if weep holes were considered inoperative due to blockage or plugging?

(iii) Details of monitoring program, if any, to assure the proper performance of weep holes during the life of the plant.

Response:

Sheet Pile Wall.

(i) Although in "Response to Question 371.23 (2)" it is stated that the 685.0 elevation is maintained by weep holes, this fact was not used in determining the stability of the sheet pile retaining walls. The following were two of the assumptions considered in the design of the retaining walls: (1) Saturated soil up to elevation 700 with no water on opposite side. (2) Dry soil on one side, no water on other. Since these assumtions provide conservative results, no factors of safety were calculated for walls based on water elevation 685. The method used in the analysis of retaining walls was provided by C. W. Dunham's book, Foundations of Structures, Second Edition, pages 468-474.

(ii) In Dunham's book he acknowledges the need for conservation in design and therefore has provided a certain amount in his design procedures. The anchorage used in bracing the retaining walls has a factor of safety of 1.25 for the controlling design case of SSE.

(iii) No monitoring system is provided.

Concrete Wall:

(i) With water at elevation 685.0 the retaining walls are submerged ∴ no differential water pressure. Factor of safety against overturning 3.96. Wall keyed into rock ∴ no problem from sliding.

(ii) Same as above.

(iii) No monitoring program provided.

Question 362.32(2.5.4)

The information provided for the foundation soil conditions underneath several Category I structures, e.g., ERCW Discharge Overflow Structure, Refueling Water Storage Tanks and Waste Packaging Area is not sufficient to complete the review. Where applicable, provide the depth to bedrock, properties of in situ gravel, properties and thickness of granular fill under the structure, and excavation and backfill details for these Category I structures. Provide details of pile foundation design and installation for category I structures founded on piles (e.g., Condenser Demineralizing Building and ERCW Pipe Slabs).

Response

FSAR Figures 2.5-225, 2.5-226, and 2.5-226a show depth to bedrock and thickness of granular fill under the structure with backfill details for Category I structures.

The response to question 362.28 provides the properties of in situ gravel. Granular fill properties are provided in Table Q362.26-2. Table Q362.32-1 provides details of pile foundation design and installation for the Condensate Demineralizer Waste Evaporator Building and the ERCW Piping and IE Electrical Conduit Support Slab.

TABLE Q 362.32-1

	Condensate Demineralizer Waste Evaporator Building		ERCW Piping and Conduit Support Slab		
Design information					
Soil parameters					
Angle of internal friction (Ø)	32°		None		
Cohesion (C) psf	0		None		
Moist unit weight (γ_m) pcf	130		None		
Skin friction (f) psf	1800		Note E		
Foundation - Type	H-pile		H-pile		
- Section	HP12x74		HP12x74	HP12x53	
- Estimated length (Le) ft	30		50 to 60		
Reason for selection	Settlement		Settlement		
Design criteria, and capacity allowables	Criteria	Capacity	Criteria	Capacity	Capacity
Static - Compression	= 12 ksi[a]	260 k	*	425 k	340 k
Dynamic - Compression - OBE	= 12 ksi[a]	260 k			
- SSE	= 15 ksi[a]	325 k	*	425 k	340 k
Uplift -	Pnet x Le x f	216 k			
Lateral - OBE	Note A		Note A		
- SSE	Note A		Note A		
Construction information					
Installation requirements					
Driving criteria	Note B		Note F		
Tolerances - Location	3 Inches		6 Inches		
- Plumbness	2%		Not established		
- Rotation	Not established		Not established		
Corrosion evaluation	Note C		Note C		
Installation data					
Method	Pile driver		Pile driver		
Equipment used	15,000 ft-lb single-acting hammer		15,000 ft-lb single-acting hammer		
Pile/pier length					
Longest	46'		60.0'	55.2'	
Shortest	12'		55.0'	55.1'	
Average	30'		55.7'	55.15'	
Field inspection	Note D		Note D		
Problems encountered	None		Note G		

*Based on pile test data.
Pnet - Net perimeter of pile.
[a] - Allowable stress.

Note A: No criteria or specific load capacity was established. Piles were structurally designed to resist the applied lateral loads due to OBE and SSE conditions. There is no lateral load for static conditions.

Note B: 5 Blows of 15,000 ft-lb hammer at full listed speed producing penetration of 1/4 inch

Note C: Evaluation of corrosion indicates piles are not in a corrosive medium.

Note D: Before pile driving started the pile location was laid out by the survey party. A stake was driven at each pile location so craft personnel would locate pile in designed location. An inspector would be present when pile was set up for driving. He would check location and plumbness of pile before driving started. He would also check plumbness during driving to ensure pile remaining plumb and straight during driving operation. The inspector also ensured refusal was met according to drawing specifications by counting blows and taking measurements when refusal was expected. The inspector was there during entire driving operation to record length driven. The survey party would set cutoff grade and after pile cutoff check final pile location for compliance with tolerance given on drawing.

Note E: Designed for end-bearing only. Skin friction was not used in design.

Note F: 48 Blow/inch of 30,000 ft-lb hammer at full listed speed producing penetration less than 1 inch.

Note G: Field used 15,000 ft-lb rather than 30,000 ft-lb hammer, NCR CDB 79-3 was submitted with no corrective action necessary.

Question 362.33(2.5.4)

Provide quantitative and procedural details of the basis for the dynamic soil properties used for horizontal and vertical soil-structure interaction analysis of the diesel generator building. Indicate the design water table used in seismic analysis and describe how the effect of water table was considered in the vertical seismic analysis.

Response

The procedure in the analysis for soil-supported structures is to consider the soil deposit as an elastic medium, and to make a dynamic analysis of a slice of unit thickness considering only the horizontal shearing resistance of the soil.

The shear wave velocity in the analysis was influenced by the in situ soil measurements, ground water, slanted soil layers, soil density variations, and variations in bedrock elevation. The shear wave velocity (V_s) of the in situ firm gravel is approximately 1650 fps from the FSAR Table 2.5-16. Also see Q362.12 response for technique used for downhole seismic velocity. The shear wave velocity of the crushed stone backfill is assumed equal to the firm gravel. Due to uncertainties in the determination of the soil properties, the shear wave velocity of soil is varied ±30 percent to calculate the horizontal ground surface motions. A soil damping ratio of 10 percent is used for the soil deposit.

The maximum ground surface accelerations, based on 0.09 g horizontal and 0.06 g vertical accelerations at the top of rock, were 0.27 g horizontal and 0.18 g vertical for the 1/2 Safe Shutdown Earthquake. The vertical motion is considered to be two-thirds of the horizontal.

The shear wave velocity of the soil was also varied ±30 percent to calculate the soil springs used in the analysis of the structure. Analysis of Foundation Vibrations by R. V. Whitman was used to calculate the soil springs. Table Q362.33-1 lists the normal modes of vibration of the structure using the different soil springs. Using the ground surface motions, the analysis of the structure indicated the primary motion of the structure to be a translatory rigid body motion. This motion is predominant because approximately 70 percent of the structure's weight is concentrated at the base, and also because of the soil on which the structure is supported. Vibrations of Soils and Foundations by F. E. Richart explains that motion of this type results in a high damping ratio. Only 10 percent damping is used in the structural analysis, which results in conservative responses.

Due to the soil-structure interaction, the effects of the structure and soil springs amplify the horizontal ground surface acceleration at the base of the structure to 0.54 g for the 1/2 Safe Shutdown Earthquake.

TABLE Q362.33-1

NORMAL MODES OF VIBRATION

V_S = 1155 FPS

Mode No.	N-S Motion $K_T = 147 \times 10^4$ K/Ft $K_R = 300 \times 10^7$ ft-K/rad Period, Second	E-W Motion $K_T = 141 \times 10^4$ K/Ft $K_R = 425 \times 10^7$ ft-K/rad Period, Second
1	0.154	0.156
2	0.103	0.111
3	0.029	0.035

V_S = 1650 FPS

Mode No.	N-S Motion $K_T = 308 \times 10^4$ K/Ft $K_R = 614 \times 10^7$ ft-K/rad Period, Second	E-W Motion $K_T = 294 \times 10^4$ K/Ft $K_R = 887 \times 10^7$ ft-K/rad Period, Second
1	0.108	0.110
2	0.072	0.077
3	0.028	0.034

V_S = 2145 FPS

Mode No.	N-S Motion $K_T = 517 \times 10^4$ K/Ft $K_R = 1031 \times 10^7$ ft-K/rad Period, Second	E-W Motion $K_T = 493 \times 10^4$ K/Ft $K_R = 1490 \times 10^7$ ft-K/rad Period, Second
1	0.085	0.087
2	0.056	0.059
3	0.028	0.033

Question 362.34 (2.5.4)

In response to question 371.23 you indicate that use of a permanent dewatering system is required to permanently lower ground water levels at safety-related structures. Provide an evaluation of the effect of the lowered water table on the stability and settlement of Category I structural foundations.

Response

Category I structural foundations at Watts Bar Nuclear Plant are supported on 1032 crushed stone fill, basal gravel and rock. Information on site geology, material properties and foundation conditions is available in section 2.5. Briefly, however, rock at the site consists of consolidated, low porosity, interbedded limestone and shale of the Conasauga formation. Basal gravel extends to partially weathered rock and is essentially a firm to dense granular soil. The 1032 crushed stone fill extends to either rock or basal gravel and is also a dense granular soil.

Permanent lowering of the ground water table to the design level should not adversely affect Category I foundation performance. More specifically, the Conasauga formation is essentially unaffected by the lowered ground water level. However, the basal gravel and 1032 crushed stone should exhibit a positive response to the lowered water table typical of firm and dense granular soil. Basically, the bearing capacity of such granular soils increase with the increase of their effective unit weights. Also, the modulus of elasticity of firm granular soils increases with the increase of the effective confining pressure. This offsets the tendency for increased deformation due to the increased effective stress caused by the lowered ground water level. In conclusion, we anticipate that permanent lowering of the ground water level will result in equal or improved Category I foundation performance when compared to the design forecasts.

362.34-1

Question:

362.35 (2.5.4) In June of 1979, you reported that the piles supporting category I ERCW pipe slabs were not driven to drawing requirements. In March 1980, based on load tests on six piles driven to the same criteria you concluded that no corrective action is required. You also indicated at that time that field measurements show no settlement of the slabs. Provide the following information:

(a) quantitative and procedural details of the pile load tests conducted to verify the adequacy of installed piles. Provide the design loads, test loads, the location of test piles, comparison of soil conditions at the location of test piles and the piles installed under the ERCW pipe slabs and load test results.

(b) up-to-date time vs settlement plots at various locations of the slabs where settlements are being monitored. Tabulate the values of the measured maximum differential settlement of the slabs and evaluate its effect on the allowable stress levels in these slabs.

Response:

(a) Load tests were performed to establish (1) pile load capacity for the existing piles supporting the slab and (2) allowable design load for any additional piling that might be required. Two tests were performed for each of the following pile sizes:

For Load Capacity of Existing Piles	For Allowable Design Load For Additional Piles
HP 12 x 53 HP 12 x 74	HP 12 x 74

representative of the Civil Engineering and Design Branch (CDB) witnessed the driving and load tests.

Locations of the six test piles and other procedural details are shown on figures Q362.35-1 and 362.35-2. (Deviation of pile location did not exceed 3 inches. Vertical deviation for all piles was less than 1/4 inch per foot of longitudinal axis.)

The driving criteria varied according to the function of pile testing, i.e., (1) determining load capacity or (2) determining allowable design load. The four piles tested to determine load capacity were driven to a penetration count of 48 blows for the last inch with a Vulcan Iron Works piledriver, model 1, developing 15,000 foot-pounds of energy with a hammer weight of 5,000 pounds and a 3.3-foot stroke. Cushion material for the Vulcan 1 hammer was 1-1/2 inches of plywood. The two piles tested to determine allowable design load were driven to penetration counts as shown in the table below with a Kobe K-22 diesel hammer. Note that the actual hammer stroke attained while setting the pile controls the specified blow count.

Energy Developed	Approximate Stroke	Blows Per Final Inch	Average Blows Per Inch for Last 6 Inches
41,300	8.51'	30	30
37,500	7.73'	32	32
35,000	7.22'	40	40
32,500	6.70'	48	48

Cushion material for the Kobe K-22 hammer was 3 inches of micarta. Piles which have not reached the required penetration rate per blow when the top has been driven to within 18 inches of grade were spliced in accordance with detail shown on figure Q362.35-2.

After the piles were driven, there was a waiting period of 14 days before the piles were tested. The piles have a maximum test load as shown below:

Pile	Maximum Test Load
HP 12 x 53	232 tons ÷ 16 = load per increment
HP 12 x 74	327 tons ÷ 16 = load per increment
HP 14 x 73	322 tons ÷ 16 = load per increment

The load tests were performed and reported as outlined in section 4.0 of Civil Design Guide DG-C1.6, "Design Guide End-Bearing H-Piles."

Results of the pile load tests are presented in the form of settlement versus load plots. These graphs are shown on figures Q362.35-3 through -6. Design loads and test loads are found in table Q362.35-1.

To eliminate the effect of different soil conditions, the piles tested were located in the same general area as the piles under the ERCW pipe support slab.

(b) To obtain the field measurements of the elevations of the slabs, the field had to dig through 18± inches of earth, chip through 18 inches of missile protection slab, and then dig through the earth surrounding the pipe to the top of the slabs. At that time the slab had been in place approximately 2 years. These field measurements showed no settlement of the slab even though subjected to crane and equipment loadings during construction of the intake pumping station. The slabs are adequate structures as designed and constructed. Since the time of the measurements, repairs have been made without provisions being made to monitor any settlement. Therefore the information in question 362.35(b) is not available. However, the elevations measured by the field and the required elevation of the slabs are shown in table Q 362.35-2.

TABLE Q362.35-1

SLAB	CASE	LOAD (K)	AREA (IN)	f_s (KSI)	f_u (KSI)	f_u (KSI)	F.S.	F.S.	MAX. *S (IN)
					(1)	(2)			
A (EXT)	NORMAL	194.36	21.8	8.92	19.72	23.86	2.21	2.67	0.14
	OBE	227.04	"	10.41	"	"	1.89	2.29	0.18
	SSE	259.72	"	11.91	"	"	1.66	2.00	0.21
					(3)	(4)			
A (INT)	NORMAL	180.8	15.6	11.59	21.8	22.11	1.88	1.91	0.15
	OBE	211.2	"	13.54	"	"	1.61	1.63	0.18
	SSE	241.6	"	15.48	"	"	1.41	1.42	0.24
					(1)	(2)			
B (EXT)	NORMAL	224.74	21.5	10.45	19.72	23.86	1.89	2.28	0.18
	OBE	262.52	"	12.21	"	"	1.62	1.95	0.21
	SSE	300.31	"	13.97	"	"	1.41	1.71	0.28
					(1)	(2)			
B (INT)	NORMAL	209.5	21.5	9.72	19.72	23.86	2.03	2.45	0.15
	OBE	244.2	"	11.36	"	"	1.74	2.10	0.20
	SSE	279.35	"	12.99	"	"	1.52	1.84	0.24

(1) PILE TEST # 5 (12×74)
(2) " " # 6 (12×74)
(3) " " # 1 (12×53)
(4) " " # 4 (12×53)

* SETTLEMENT (INCLUDING ELASTIC COMPRESSION)

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

TABLE Q362 35-1

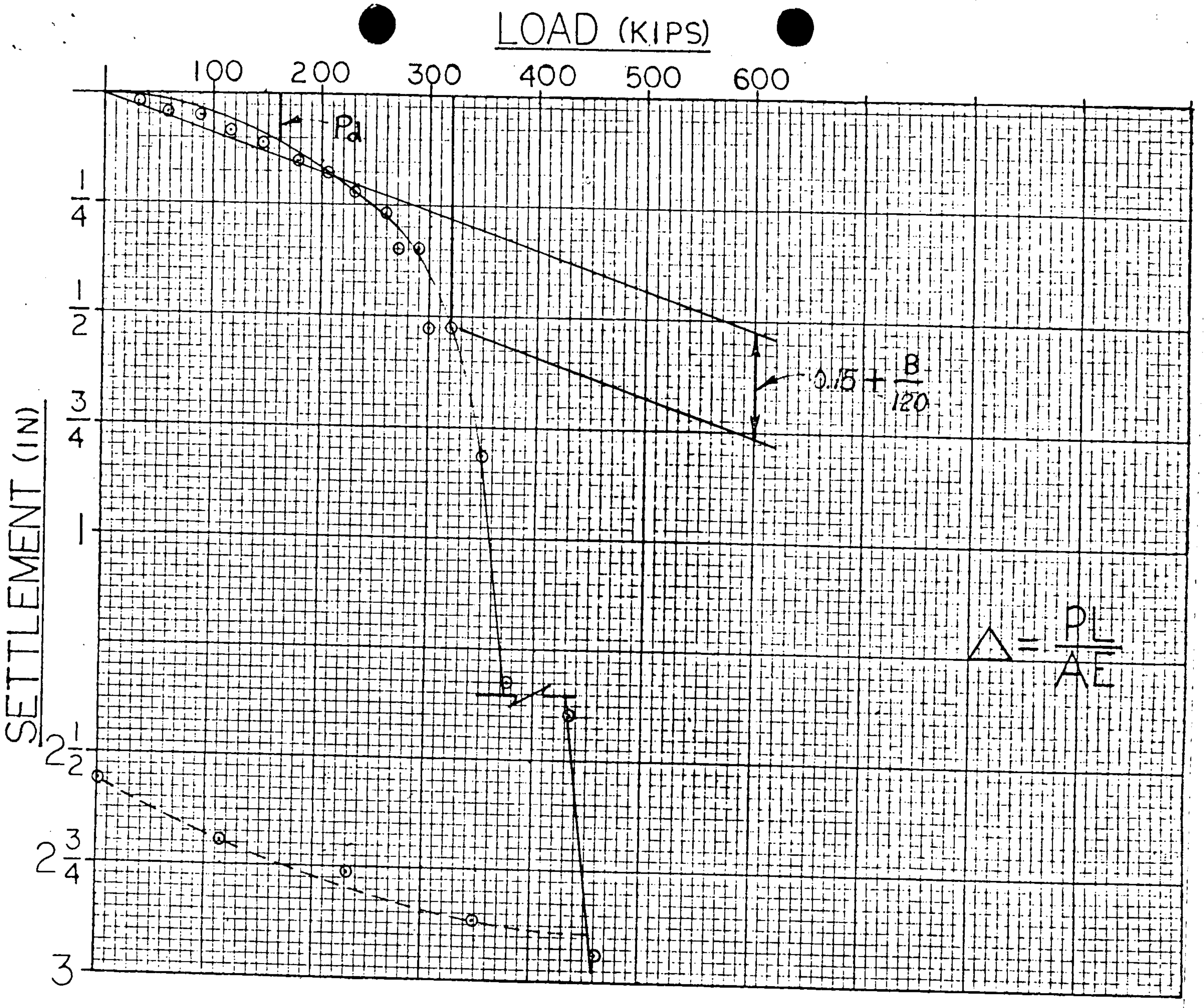


FIG. Q362.35-3

PILE NO. 1
HP 12×53
VULCAN HAMMER
Pd = 160 K
σ PILE = 10.2 KSI

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

FIGURE Q362.35-3

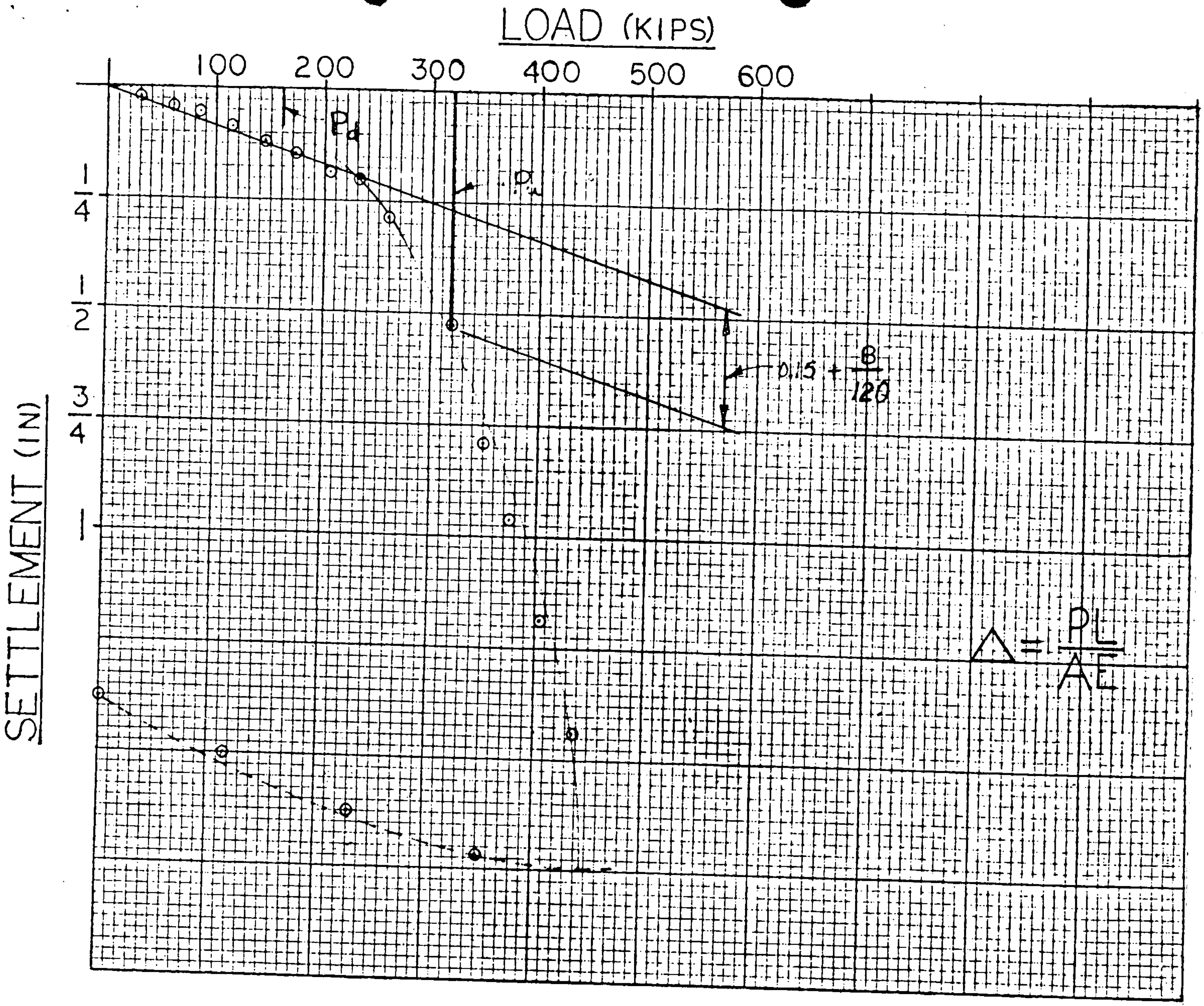


FIG. Q362.35-4

PILE NO. 4
HP 12×53
VULCAN HAMMER
P_d = 159 K
σ_{PILE} = 10.2 KSI

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

FIGURE Q362.35-4

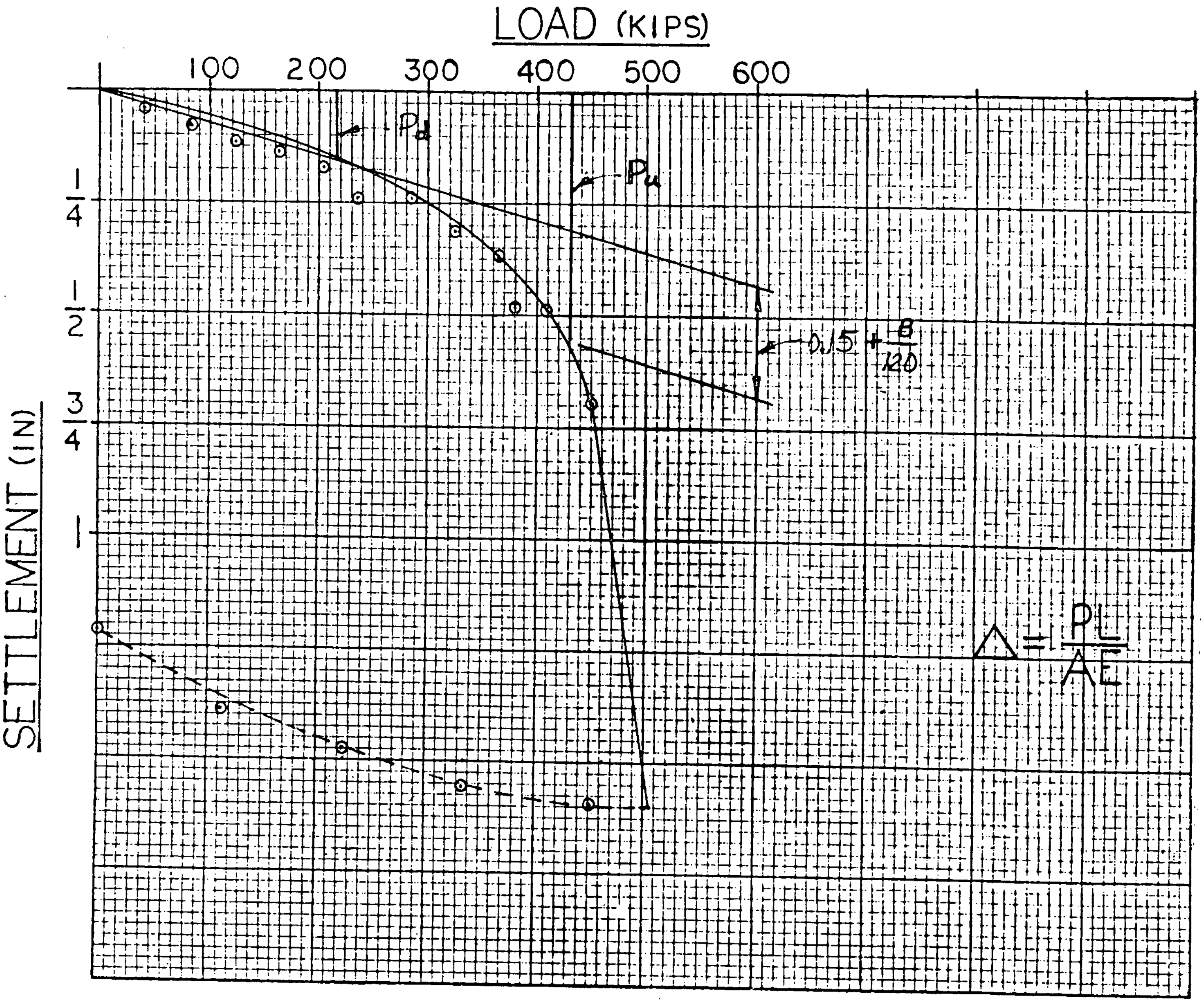


FIG. Q362.35-5

PILE NO. 5
HP 12×74
VULCAN HAMMER
P_d = 215 K
σ_{PILE} = 9.86 KSI

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

FIGURE Q362.35-5

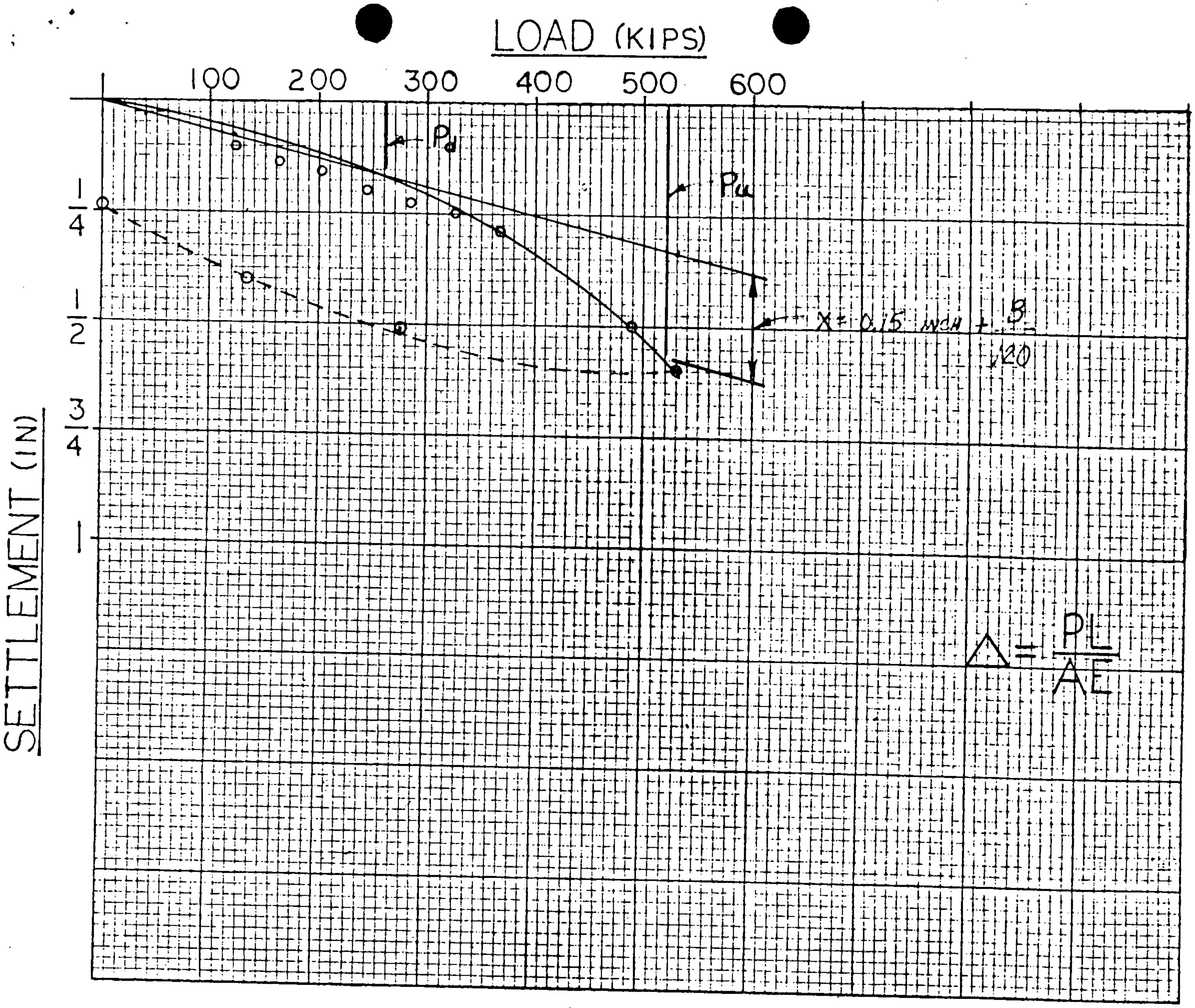


FIG. Q362.35-6

PILE NO. 6
HP 12×74
VULCAN HAMMER
P_d = 260 K
σ_{PILE} = 11.93 KSI

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

FIGURE Q362.35-6

TABLE Q 362.35-2		
*POINT	ACTUAL ELEV	ORIGINAL ELEV
A	703.53	703.5
B	703.54	
C	703.52	
D	703.55	
E	703.56	
F	703.57	
G	703.56	
H	703.56	703.5

* SEE FIG. Q362.35-7

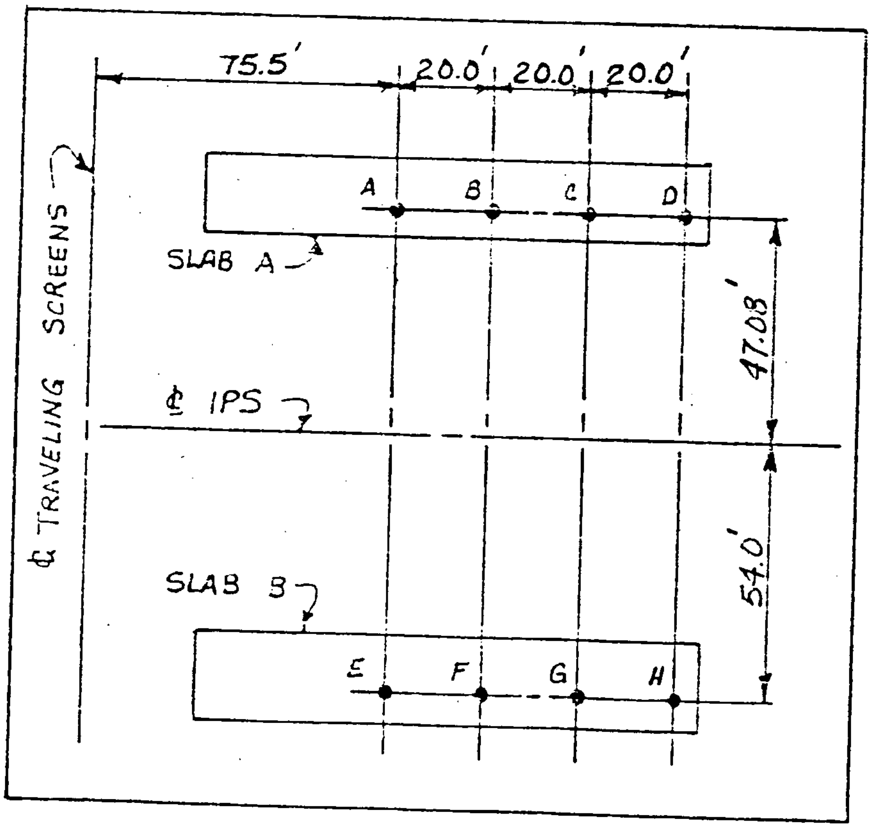


FIG Q362.35-7

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

FIGURE Q362.35-7

Question 362.36

Your response to Q362.27, in Ammendment 2 has not provided sufficient information to establish that the zone of alluvial silty sands and sandy silts within the foundation of the Class IE Electrical Conduit and the Essential Raw Cooling Water pipeline are not loose and potentially susceptible to liquefaction. The information required by the staff for an adequate review was requested earlier in Q362.14, Q362.24 and Q362.27, but has not been provided to the staff. We request again that you provide the following information in sufficient detail for an independent staff review.

1. Provide following plots drawn to scale on two large size drawings (approximately 22 inches by 34 inches) for category I Essential Raw Cooling Water pipeline and Class IE Electrical Conduit. Provide one drawing for essential Raw Cooling Water pipeline and the other for Class IE Electrical Conduit.

   (a) Locations and routing from one end of the utility to the other, clearly identifying the lines.

   (b) Locations of the borings along the route of the pipeline and the conduit. Indicate by legend the type of sampling in these borings (split spoon or undisturbed sampling) and show the spacing between individual borings. Show the locations of the pertinent borings that provide information about the liquefaction potential of soils under question.

   (c) Show the contours of the as-built ground surface along these utilities after placement of fill.

2. Explain your basis for using borings spaced as much as 200 to 400 feet apart along the routes of these lines to provide reasonable assurance that the soil profile underneath the utilities does not contain materials susceptible to liquefaction. Note that boring log SS-50 shows about 10 feet of loose alluvial material below water table and because of the wide spacing of the borings, the lateral extent of the loose zone cannot be established in this area. Explain how the extent of loose alluvial material was determined from the widely spaced borings in different sections along the routes.

3. On two large size drawings (approximately 22 inches by 34 inches), provide the following details to scale for Category I Essential Raw Cooling Water pipeline and Class IE Electrical Conduit. Provide one drawing for ERCW pipeline and the other for Class IE conduit.

   (a) The pertinent boring logs along the routes of the conduit and pipeline showing the fill above the pipeline and conduit. The spacing between the logs should be to scale as well. Provide the classification and blow count information on this plot. If some of the borings along the routes are not used in the liquefaction potential evaluation (e.g., borings 52, 66, 89, 91, 98, 100, and 102), identify and provide logs for these borings. Provide justification for not using these borings in your analysis.

(b) Show soil stratification and top of shale boundary on the profile.

(c) Draw 25-year high water level on profile and discuss how it corresponds to the water table information presented in Section 2.4.13.2.

(d) Draw the invert and top of Class IE Electrical Conduit and ERCW pipeline on these logs.

(e) Show the as-built fill above the pipeline and the conduit and indicate the ground surface elevation on the logs.

4. Based on the information provided in items 1, 2, and 3, discuss in detail the probable vertical and lateral extent of the alluvial solid with N 30 that is below the 25-year high water table. Discuss the gradation, relative density and cyclic strength characteristics of material in this strata.

5. Provide details of the dynamic response computations and the factors of safety for liquefaction potential of alluvial soils in the profile along the routes of the utilities. Include the following information:

(a) The cross-section of the one dimensional soil profile analyzed. Indicate the water table elevation used in the analysis. Discuss any conservatism in selecting the profile.

(b) The dynamic soil moduli and damping values of the various soils in the profile. Provide the value of the coefficient of earth pressure at rest used for the analysis.

(c) The characteristics of the seismic input used for liquefaction analysis, viz, response spectrum of the input motion, and its point of application in the soil profile.

(d) The method of dynamic response analysis, various assumptions used for converting the irregular shear stress time history to 5 cycles of equivalent uniform cyclic stress. Show typical results.

(e) Provide the results of analysis for the entire profile.

(f) Justify the use of the cyclic strength properties curve given in response to Q362.27. Explain the scatter in the laboratory test data and justify your interpretation of the data.

(g) Provide a table of factors of safety for the alluvial material at various depths against liquefaction potential.

Response

1. The Category I Essential Raw Cooling Water pipeline and Class IE Electrical Conduit locations are provided in Figure Q362.36-1. The plan view provides the following:

(a) Location and routing from one end of the utility to the other end.

(b) Location of the borings along the route of the pipeline and the conduit.

(c) And contours of the as-built ground surface.

2. The original borings along the ERCW route were spaced at 200 to 400 foot centers. A later (November 1979) investigation reduced the spacing between borings to approximately 100 feet. These boring locations and data are shown on Figure Q362.36-1.

   The original borings for the IE Electrical Conduit route (when different from ERCW route) were spaced on 200 foot grid centers. The IE Electrical Conduits pass diagonally through part of this area and traverse a course through the switchyard to the main plant area. The spacing of these holes is sufficient to define the soil profile for routing of the conduit banks. No low blow count sand or potentially liquefiable material was identified in these higher elevation residual soils.

3. A profile of the ERCW pipes are provided in Figures Q362.36-2 through Q362.36-6. The IE Electrical Conduit profiles are provided in Figures Q362.36-7 through Q362.36-9. The profiles provide the following information:

   (a) Pertinent boring logs along the routes showing blow counts and the material classification of the in situ soil.

   (b) The elevation of original grade, final grade, and the top of rock. (Note: Fill material was used to backfill around the pipes and to achieve final grade.)

   (c) The electrical conduit and ERCW pipelines and their elevation to scale.

   (d) The 24 hour water table.

   Borings 52, 66, 89, 91, 98, 100, and 102 were not drilled. Some of the borings were back-fitted with borings 137, 143, 146, 154, 157, and 160. A generalized soil stratification is provided in Figures Q362.36-10.

   A 25-year water table was not established. In section 2.4.13 of the FSAR the ground water is discussed in detail. The Knox Dolomite is the principle source of flow to streams of the region. Other formations within the site region, described in detail in section 2.5.1.1, include the Rome Formation, a poor water-bearing formation; the Conasauga Shale, a poor water-bearing formation; and the Chickamauga Limestone, a poor-to-moderate water-bearing formation that normally yields only 25 gpm to wells.

   The plant site is underlain by the Conasauga Shale, which is made up of about 84 percent shale and 16 percent limestone, which occurs as thin discontinuous beds. Surficial material are older terrace deposits and recent alluvial soils mostly finegrained, poorly sorted, and poorly water-bearing.

All recharge to the ground water system is from local precipitation. There is no regional subsurface transport of water. All ground water discharge from the site is to Chickamauga Lake, either directly or via Yellow Creek.

Six observation wells were set up in 1973 at the site to monitor the ground water. The wells in FSAR Figure 2.4-104 are not in the near vicinity of the ERCW piping or electrical conduits.

Monitoring of the wells show that they fluctuate ±5 feet due to seasonal change. The water level in the wells will rise in the winter and spring and drop in the summer and fall, typical of the local precipitation.

4. The blow counts of the alluvial soils are given in Figures Q362.36-2 through Q362.36-9. The extent of the alluvial sand are provided in Figure Q362.36-10. The characteristics of alluvial soil susceptible to liquefaction are given in Figures Q362.27-1 through Q362.27-15 and tables Q362.27-1 and Q362.27-2.

5. (a) The profile selected and analyzed is based on boring SS-50-1. This boring is shown in Figure Q362.27-1. This boring contained the most SM Material. Surface elevation is 716.9 feet. Around elevation 685 and 690 the blow count increases to +50 and is identified as "top of weathered shale." This is assumed as "top of rock" for the liquefaction evaluation. Thus the depth of the profile is 30 feet. The water table is about 15 to 20 feet below the ground surface in boring SS-50, SS-50-1, SS-65, and SS-65-1. Thus the water table is assumed to be 15 to 20 feet below the ground surface. The profile analyzed is fairly typical of those along the ERCW route. This generalized soil profile is shown graphically in Figure Q362.36-11.

   (b) The soil unit weight (moist) is taken as 120 pcf. The shear wave velocity of the soil is taken as 1000 ft/s. This value is in agreement with data obtained from the intake channel and elsewhere on the site. The strain dependent shear modulus and damping ratio properties of these soils are assumed to conform with the relationships developed by Seed for sand. The coefficient of earth pressure at rest (K_o) is conservatively taken as 0.5. All soil properties are assumed to be constant with depth.

   The rock has a unit weight of 165 pcf and a shear wave velocity of 5900 ft/s.

   (c) The seismic input at the site is defined as a 0.18g earthquake at top of rock. Four artificial accelerograms are used to define this event. This is inappropriate for use in a liquefaction evaluation and is not used. The liquefaction evaluation is performed using another artifical accelerogram which conforms to Reg. Guide 1.60 requirements. Peak accelerations of 0.18g, 0.225g, and 0.25g are considered. The accelerogram was also high band pass filtered to eliminate frequencies greater than 5 Hz for three cases and 25 Hz for two cases. In all, five different analyses are performed and are listed below.

Case	Maximum Acceleration	Applied at Top of	Upper Frequency Cutoff
1	0.25g	Ground	5 Hz
2	0.18	Ground	5
3	0.225	Ground	5
4	0.25	Ground	25
5	0.18	Rock	25

The most appropriate seismic loading is case 1 where the 0.25g accelerogram is applied at top of ground with a 5 Hz uppper frequency cutoff. Its results essentially envelope all cases except for case 5 where the input is at top of rock.

(d) The dynamic response analysis is performed using the computer program SHAKE. Irregular shear stress time histories are not calculated. The equivalent uniform cyclic stress is taken as 65% of the maximum cyclic shear stress within each layer of the profile as calculated by SHAKE.

(e) The results of the analyses are given in the attached table. The maximum and equivalent uniform stresses within each layer and the peak accelerations at the top of each layer are summarized in Table Q362.36-1 for all five earthquake input conditions.

For material located about 17.5 feet below the surface (approximately the elevation of the samples tested cyclically), the max shear stress is:

$$\tau_{max} = 500 \text{ psf}$$

The average shear stress is:

$$\tau_{avg} = 0.65\ \tau_{max} = 325 \text{ psf}$$

The vertical pressure at 17.5 feet is:

$$\bar{\sigma}_v = \gamma h = (120 \text{ pcf})(17.5 \text{ feet}) = 2100 \text{ psf}$$

Assuming a $K_o = 0.5$, the horizontal stress is:

$$\bar{\sigma}_h = 0.5\bar{\sigma}_v = 1050 \text{ psf} \qquad \text{use } 1000 \text{ psf}$$

The cyclic stress ratio is:

$$\frac{\sigma_d}{2\ \sigma_3} = \frac{\tau}{\sigma_3} = \frac{325 \text{ psf}}{1000 \text{ psf}} = 0.32$$

From Figure Q362.27-15 the most susceptible sample will survive six load cycles with this stress ratio. We expect only five uniform load cycles from our 0.18g to 0.25g event. This event is an intensity VIII earthquake and is characterized as a m_{blg} 5.8. Extrapolating Seed and Idriss's data,

Magnitude	Number of Cycles	Equivalent Uniform Cyclic Stress
7	10	0.65 max
7-1/2	20	0.65 max
8	30	0.65 max

we conservatively have 5 cycles of uniform load for a magnitude 5.5 to 6.0 event. Factors of safety against the development of 5 percent strain are given in Tables Q362.36-2 and Q362.36-3 and Figure Q362.36-12. These factors of safety are calculated only for seismic loading case 1. Results are presented for cases where the water table is not considered and where it is located 16.5 feet below the surface. The 16.5 is in the upper range as given in the borings and the exact number 16.5 is chosen for convenience only. Factors of safety are calculated for both the reconstituted sample (sample No. 3) and for the in situ sample (sample No. 2). The in situ sample is more representative of field behavior. It should be repeated that these factors of safety are against the development of 5 percent strain and not against actual liquefaction which, if it occurs, occurs at strains in excess of 10 percent for the samples tested.

(f) The scatter of the test data is to be expected. These tests were conducted on in situ soil samples. Variations in the soil and the results were anticipated. Only the soil judged most susceptible to liquefaction were selected for testing. Of the three samples selected for testing, all available specimens were tested. Sample No. 3 shows some scatter. All specimens from sample No. 3 are reconstituted due to the presence of a large gravel particle.

Three of the four test points form the classical cyclic curve. However, the fourth point (the lower point at three cycles) is out of place. The curve was constructed giving extra weight to the upper point at three cycles and then giving equal weight to both test points at stress ratios of 0.26 and 0.27. The other two curves are constructed essentially parallel to this first curve.

(g) See response to Part E.

TABLE Q362.36-1

ERCW ROUTE LIQUEFACTION EVALUATION

Maximum and Average Element Stresses and Peak Accleration at the Top of Each Layer

Layer	Depth (Feet)	Top of Ground 0.25g 5 Hz	Top of Ground 0.18g 5 hz	Top of Ground 0.225g 5 Hz	Top of Ground 0.25g 25 Hz	Top of Rock 0.18g 25 Hz
Max Element Stresses (psf)						
1	1.5	44	32	39	50	80
2	4.5	132	95	118	149	239
3	7.5	220	159	196	244	395
4	10.5	308	221	275	339	549
5	13.5	396	283	351	433	692
6	16.5	484**	344	429	520	824
7	19.5	566**	407	502	600	942
8	22.5	645	466	574	671	1044
9	25.5	720	522	643	734	1130
10	28.5	790	575	709	793	1198
Average Element Stresses* (psf)						
1	1.5	29	21	25	33	52
2	4.5	86	62	77	97	155
3	7.5	143	103	127	159	257
4	10.5	200	144	179	220	357
5	13.5	257	184	228	281	449
6	16.5	315	224	279	338	536
7	19.5	368	265	326	390	612
8	22.5	419	303	373	436	679
9	25.5	468	339	418	477	735
10	28.5	514	374	461	515	779
Top of Layer Accelerations (g)						
1	0	.24	.17	.22	.28	.44
2	3	.24	.17	.22	.28	.44
3	6	.24	.17	.22	.27	.44
4	9	.24	.17	.21	.26	.43
5	12	.24	.17	.21	.25	.41
6	15	.23	.17	.21	.25	.39
7	18	.23	.16	.20	.24	.36
8	21	.22	.16	.20	.22	.32
9	24	.21	.15	.19	.22	.27
10	27	.20	.15	.18	.22	.22
11	30	.20	.14	.18	.23	.2[illegible]

Average element stress = 0.65 max element stress.

**Assume 500 psf at 17.5 feet.

TABLE Q362.36-2

Factors of Safety with Depth When the Water Table is not Considered

Layer	Depth (Feet)	$\bar{\sigma}_v$ (psf)	$\bar{\sigma}_h$ (psf)	τ/σ_3	τ_f	τ_{avg}	$FS = \tau_f/\tau_{avg}$
For Sample 3 - Reconstituted							
1	1.5	180	90	0.34	31	29	1.07
2	4.5	540	270	0.34	92	86	1.07
3	7.5	900	450	0.34	153	143	1.07
4	10.5	1260	630	0.34	214	200	1.07
5	13.5	1620	810	0.34	275	257	1.07
6	16.5	1980	990	0.34	337	315	1.07
7	19.5	2340	1170	0.34	398	368	1.08
8	22.5	2700	1350	0.34	459	419	1.10
9	25.5	3060	1530	0.34	520	468	1.11
10	28.5	3420	1710	0.34	581	514	1.13
For Sample 2 - In situ							
1	1.5	180	90	0.60	54	29	1.86
2	4.5	540	270	0.60	162	86	1.88
3	7.5	900	450	0.60	270	143	1.89
4	10.5	1260	630	0.60	378	200	1.89
5	13.5	1620	810	0.60	486	257	1.89
6	16.5	1980	990	0.60	594	315	1.89
7	19.5	2340	1170	0.60	702	368	1.91
8	22.5	2700	1350	0.60	810	419	1.93
9	25.5	3060	1530	0.60	918	468	1.96
10	28.5	3420	1710	0.60	1026	514	2.00

Notation:

$\bar{\sigma}_v$ = effective vertical stress

$\bar{\sigma}_h$ = effective horizontal stress

τ/σ_3 = cyclic stress ratio

τ_f = cyclic shear stress corresponding to 5% strain

τ_{avg} = average on effective shear stress

FS = Factor of Safety against 5% cyclic strain potential

TABLE Q362.36-3

Factors of Safety with Depth Assuming the Water Table is 16.5 feet Below Ground Surface

Layer	Depth (Feet)	$\bar{\sigma}_v$ (psf)	$\bar{\sigma}_h$ (psf)	τ/σ_3	τ_f	τ_{avg}	FS = τ_f/τ_{avg}
For Sample 3 - Reconstituted							
1	1.5	180	90	0.34	31	29	1.07
2	4.5	540	270	0.34	92	86	1.07
3	7.5	900	450	0.34	153	143	1.07
4	10.5	1260	630	0.34	214	200	1.07
5	13.5	1620	810	0.34	275	257	1.07
6	▽ 16.5	1980	990	0.34	337	315	1.07
7	19.5	2160	1080	0.34	367	368	1.00
8	22.5	2340	1170	0.34	398	419	.95
9	25.5	2520	1260	0.34	428	468	.91
10	28.5	2700	1350	0.34	459	514	.89
For Sample 2 - In situ							
1	1.5	180	90	0.60	54	29	1.86
2	4.5	540	270	0.60	162	86	1.88
3	7.5	900	450	0.60	270	143	1.89
4	10.5	1260	630	0.60	378	200	1.89
5	13.5	1620	810	0.60	486	257	1.89
6	▽ 16.5	1980	990	0.60	594	315	1.89
7	19.5	2160	1080	0.60	648	368	1.76
8	22.5	2340	1170	0.60	702	419	1.68
9	25.5	2520	1260	0.60	756	468	1.62
10	28.5	2700	1350	0.60	810	514	1.58

Notation:

$\bar{\sigma}_v$ = effective vertical stress

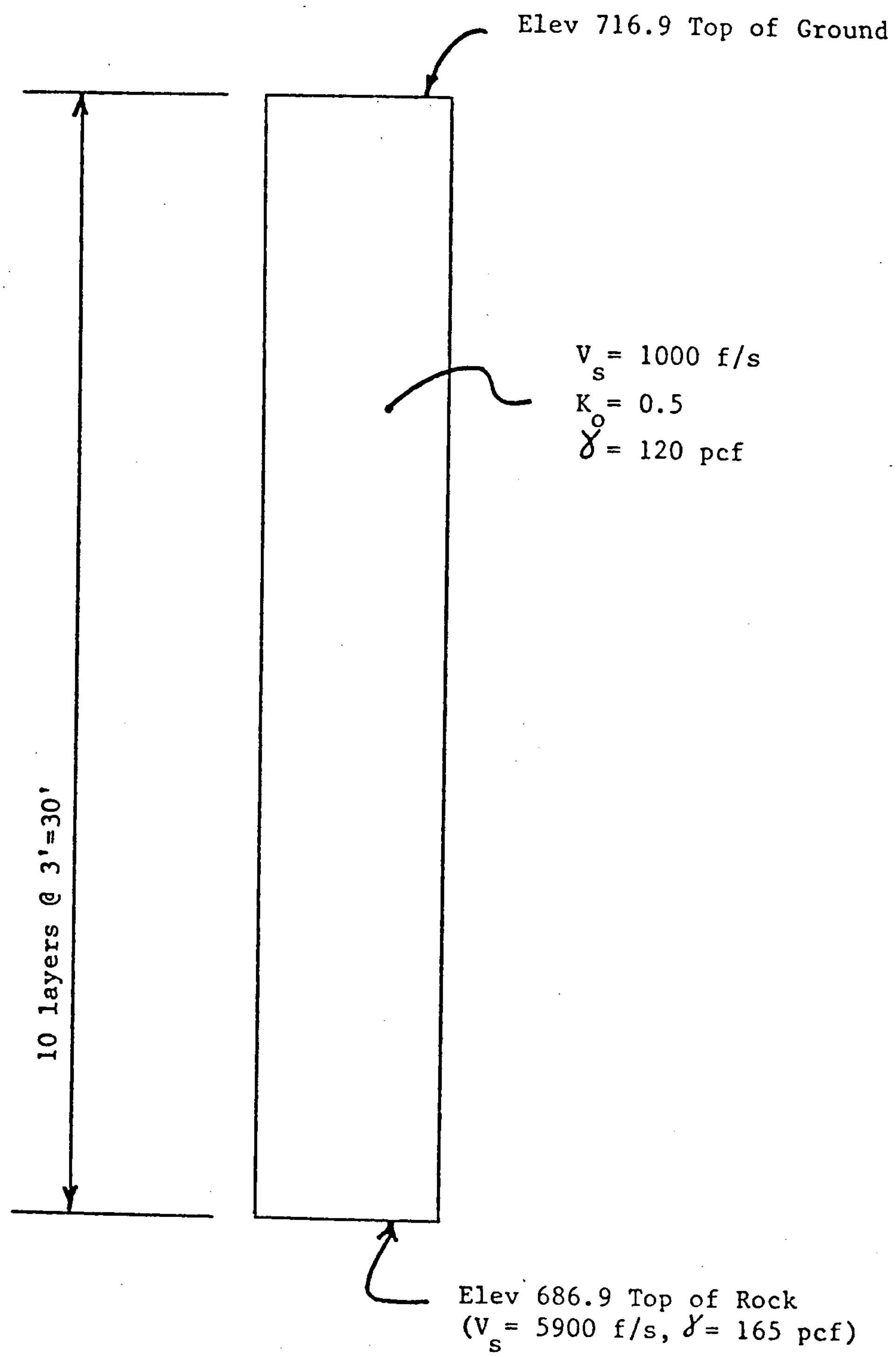
$\bar{\sigma}_h$ = effective horizontal stress

τ/σ_3 = cyclic stress ratio

τ_f = cyclic shear stress corresponding to 5% strain

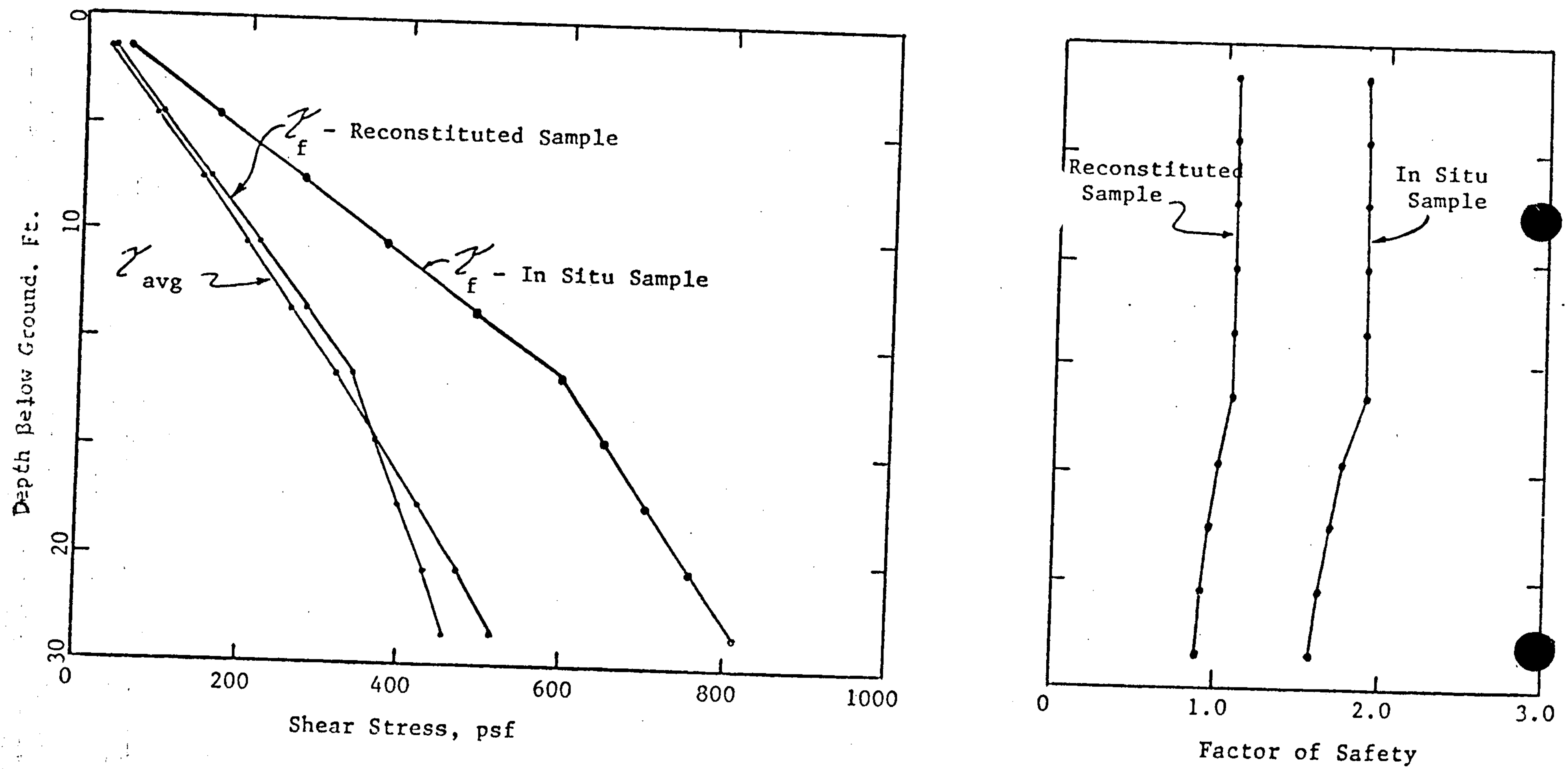
τ_{avg} = average on effective shear stress

FS = Factor of Safety against 5% cyclic strain potential



One-Dimensional Soil Profile Used for Liquefaction Evaluation

Figure Q362.36-11



Comparison of Induced Shear Stress (τ_{avg}) and Shear Stress Required to cause 5% strain (τ_f) and Resulting Factors of Safety with Depth Below Ground Surface.

Figure Q362.36-12