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Robert L. Mittl General Manager
Nuclear Assurance and Regulation

April 24, 1984

Director of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
7920 Norfolk Avenue
Bethesda, MD 20814

Attention: Mr. Albert Schwencer, Chief
Licensing Branch 2
Division of Licensing

Gentlemen:

NRC REVIEW OF STRUCTURAL/GEOTECHNICAL TOPICS
HOPE CREEK GENERATING STATION
DOCKET NO. 50-354

Pursuant to the agreements reached at the meetings held on January 10, 11, and 12, 1984, to review HCGS structural/geotechnical topics with the NRC, attached is one (1) set of responses to those items denoted as Category III target date. A listing of the attached Category III target date items, broken down by date of meeting, is as follows:

Meeting of January 10, 1984:
Items A.5, A.8, and A.13

Meeting of January 11, 1984:
Items A.4, A.5, A.14, and A.16

Meeting of January 12, 1984:
Item A.4

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In addition, please note that four (4) advanced sets of these responses were transmitted to D. Wagner via Federal Express on April 23, 1984.

Should you have any questions in this regard, do not hesitate to contact us.

Very truly yours,

R J McCutt

Attachment: Resolution of NRC Comments on Structural/Geotechnical Topics

C D. H. Wagner (w/attach.)
USNRC Licensing Project Manager

Mr. W. H. Bateman
USNRC Senior Resident Inspector

FA25 1/2

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: A.5

QUESTION: Verify that soil shear moduli variation of $\pm$ 50 percent is at least one standard deviation based on soil test data.

RESPONSE: Dynamic properties of the soils used in the analyses are based on both geophysical survey and laboratory testing (FSAR Section 2.5.4.2.4). The laboratory data from strain controlled cyclic triaxial tests and resonant column tests for Vincentown, Hornerstown and basal sand deposits are plotted (Shear strain versus K_2) as shown on Figure 1. The factor K_2 is the coefficient in the Seed and Idriss equation for shear modulus at low shear strains:

$$G(PSF) = 1000 \times K_2 x(\sigma_m)^{1/2}$$

where σ_m is the Mean Confining Stress

A mean curve to these data is shown by the solid line in Figure 1. Standard deviation was computed separately for strain controlled cyclic triaxial test data and resonant column test data. Limits for one standard deviation are shown by dotted lines. Figure 1 also shows the design curve used in dynamic analyses and the range of soil shear moduli variation of $\pm$ 50 percent. The seismic induced effective shear strains are typically in the range of 10^{-3} to 10^{-4} in/in.

Comparison of the curves indicates that the soil shear moduli variation of $\pm$ 50 percent envelopes the standard deviation curves at seismic induced strain levels. The soil shear moduli variation of $\pm$ 50 percent, therefore, includes the variation in laboratory data in the area of interest.

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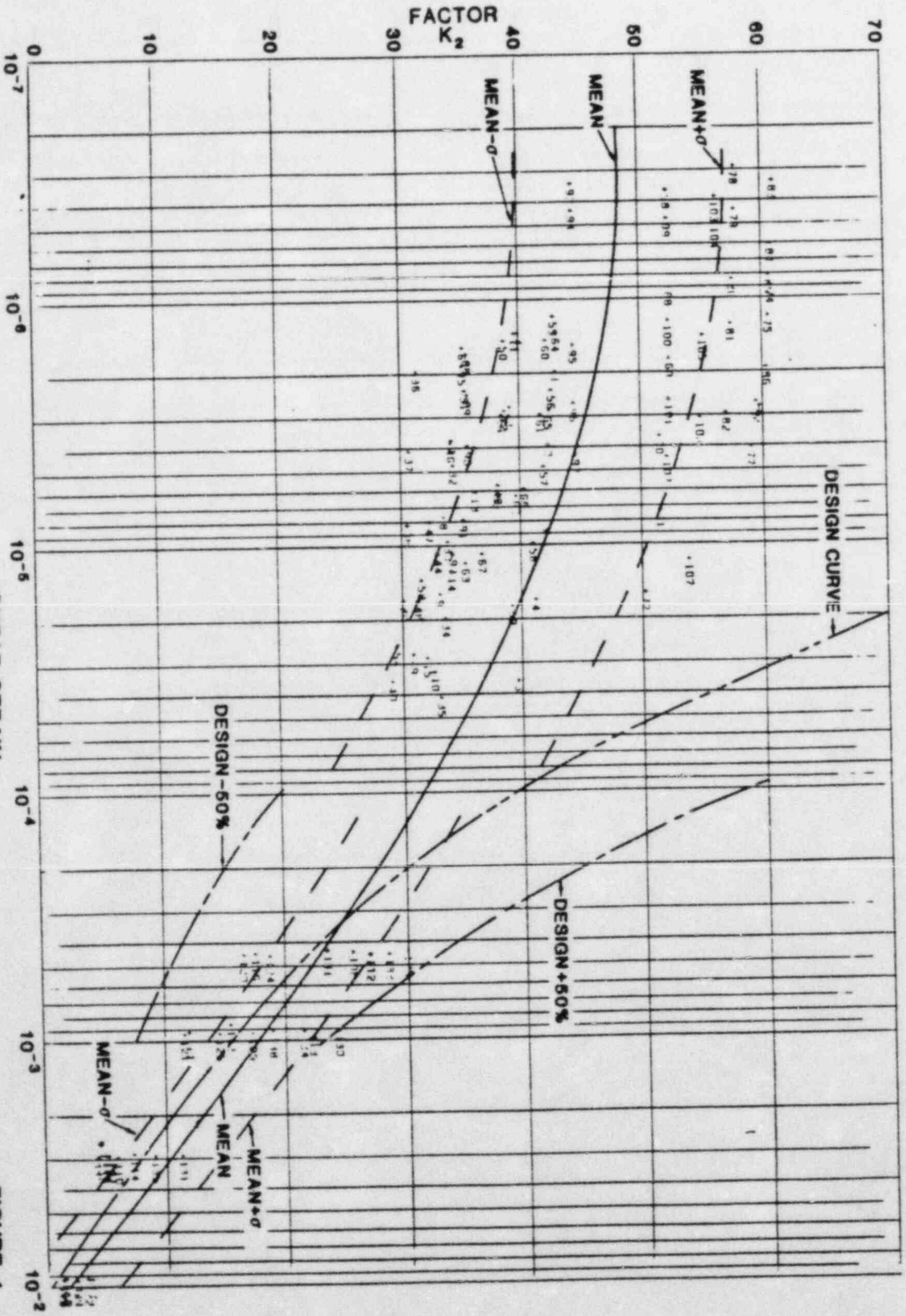

FIGURE 1

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: A.8

QUESTION: Lab test shear modulus values for Vincentown differ from values used in the analysis. Investigate the impact of the use of lab test values on soil structure interaction.

RESPONSE: The design curve (FSAR Fig. 2.5-41) recommended for the Vincentown sands for use in the soil structure interaction analysis was derived from laboratory tests which included resonant column tests and dynamic strain-controlled cyclic triaxial tests and field geophysical surveys. For many soils, especially the variably cemented Vincentown sands, the major shortcoming of all laboratory tests for determining the shear modulus lies in the fact that undisturbed samples are difficult to obtain and test. In general, the greater the degree of sample disturbance, the lower the shear modulus. On the other hand, field tests, such as geophysical tests, do not suffer from this sample disturbance limitation and, therefore, provide more reliable data at low strains: i.e., 10^{-6}.

For strain values observed in the soil-structure interaction analysis results, the effect of the laboratory test data variation was evaluated by varying the design shear modulus curve ±50%. This 50% shear modulus variation envelopes one standard deviation based on soil laboratory test data for strain levels observed in the soil-structure interaction analysis results (See response to Question No. A.5, NRC Structural Audit Meeting Date January 10, 1984). Therefore, the effect of the use of laboratory test values on soil-structure interaction has been taken into account in the design basis analysis.

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: A.13

QUESTION: Provide comparison of Bechtel Independent Verification results with The Design Basis Results.

RESPONSE: A response to the above question will be provided in July 1984.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.4

QUESTION:

Current settlement calculations for the service water pipe are based on average soil properties. Provide additional settlement estimates using actual soil properties at various sections along length of service water pipe line.

RESPONSE:

SETTLEMENT ESTIMATES

Settlements of the service water pipeline were estimated using average soil properties and a typical construction sequence. For the load-time history shown on Figure 1, the maximum total static settlement of the pipeline is estimated to be approximately 3 inches corresponding to a 30 foot thick Kirkwood clay layer. The settlements would be less for a smaller thickness of Kirkwood clay. Differential settlements would primarily depend on the variability of the Kirkwood clay layer thickness. This variability however will be negligible within each 20 foot segment of the pipeline, except near the power block area where the pipeline crosses the main excavation slope. The maximum differential settlement within a pipe segment is expected to be on the order of 1 inch in that area.

SETTLEMENT MEASUREMENTS

Settlements of the service water pipeline were measured for the former Unit 2 pipe by comparing invert elevations of the pipe segments from the as-built data with those from an optical survey performed in March 1984. Unit 1 pipes were not readily accessible. Because of the proximity of the Unit 1 and 2 pipes, actual settlements of the Unit 1 pipes should be similar to those measured for the Unit 2 pipes. The alignment of the pipeline and locations of field measurements of settlements are shown on Figure 2. The measured settlements are shown on Figure 3. The measured maximum static settlement and maximum differential settlement between the pipe joints are 2.28 inch and 0.6 inch, respectively. It should be noted that the location of the measured maximum total settlement is in the area of maximum Kirkwood clay thickness along the pipeline. Settlements in other parts of the pipeline, therefore, are expected to be smaller.

PREDICTED FUTURE SETTLEMENTS AND CONCLUSIONS

After decommissioning of the dewatering system, groundwater at the HCGS site had been slowly brought back to its original natural level as of December 1983. This has an unloading effect on the soils, resulting in reduction of the effective stresses. Future movement of the pipeline, if any, is expected to be small and in the upward direction. The measured settlements are, therefore, considered to be representative of the maximum settlements experienced by the pipeline in the respective areas. Comparison of the measured and estimated settlements indicates that the average soil properties used in settlement calculations are appropriate.

The structural integrity and flexibility of the joints of the service water pipes were verified by testing. The pipe joints have been qualified for 1 inch axial displacement and 1 degree rotation (corresponding to 4 inches of allowable differential settlement along each 20 foot section) based on the test results. These tolerance values exceed the predicted static plus dynamic differential settlement and provide an adequate margin of safety.

LOADING HISTORY KIRKWOOD CLAY AT SERVICE WATER TRENCH

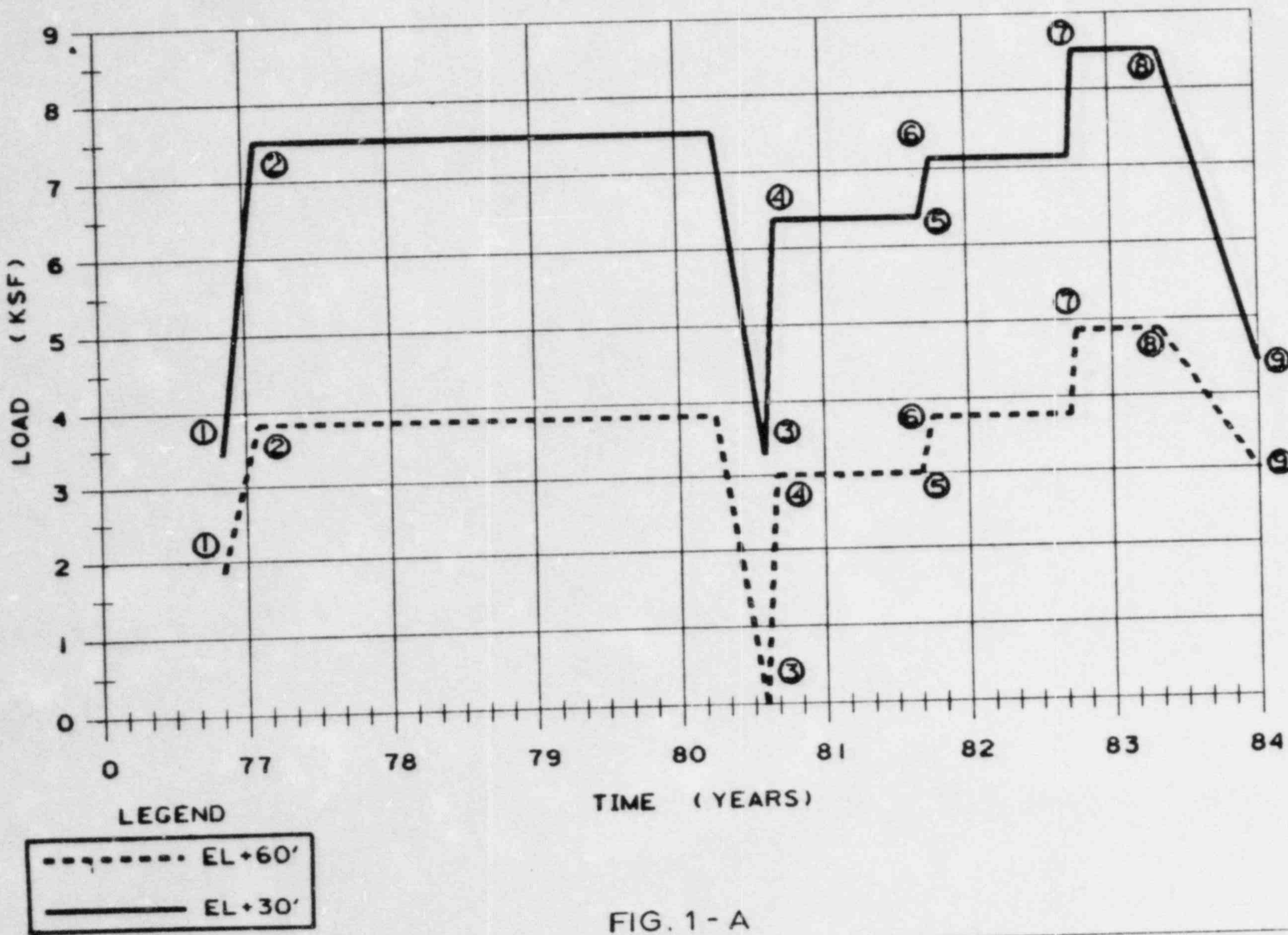

FIG. 1-A

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TYPICAL STRESS HISTORY (TOTAL STRESS AT W 800)

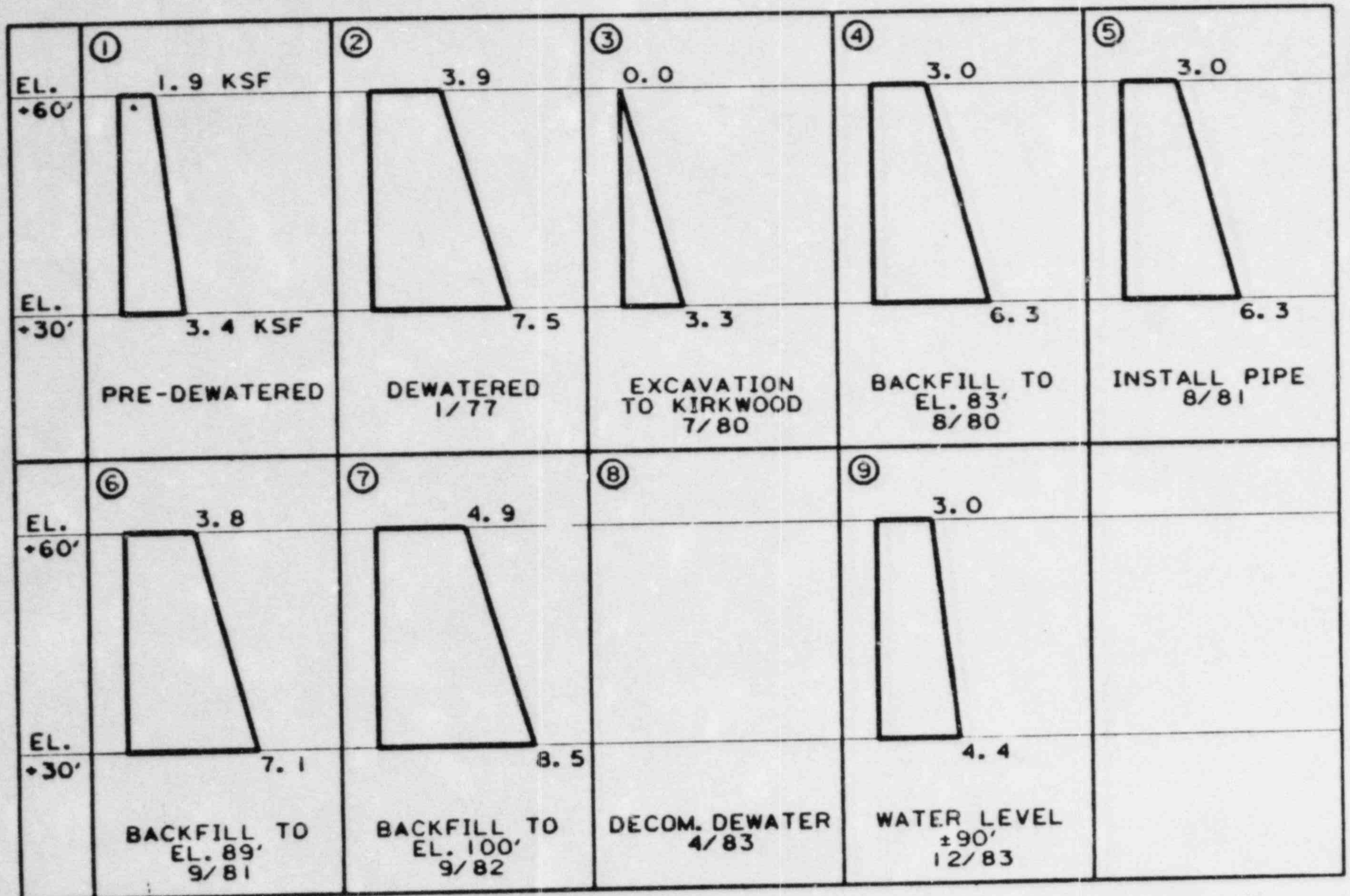

FIG. 1 - B

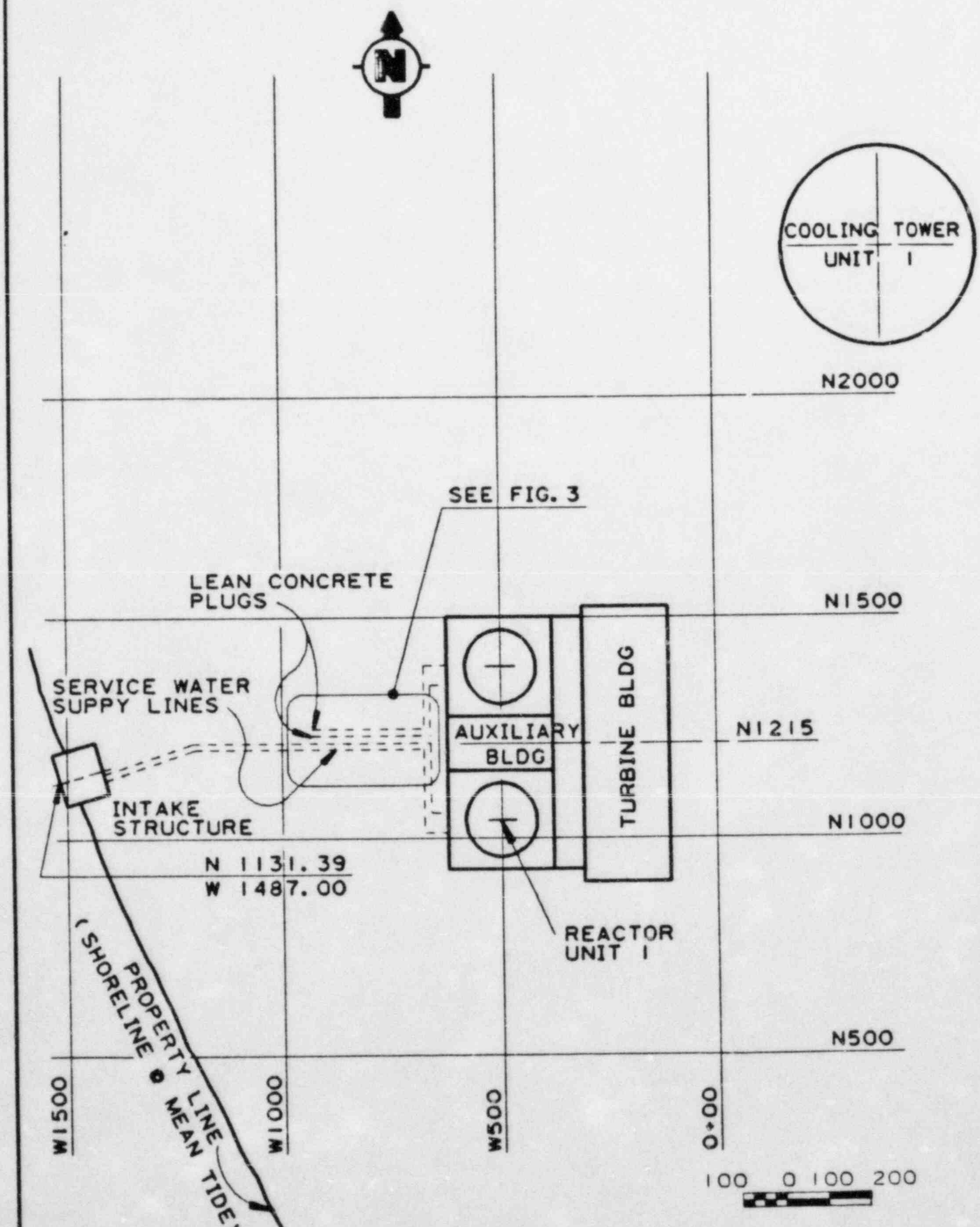

FIG. 2 GENERAL ROUTING OF PIPING

Station / Date	AS-BUILT ELEV. (ft)	MEASURED EL. (ft) (MARCH 15, 1984)	SETTLEMENT (In)	DIFFERENTIAL SETTLEMENT BETWEEN PIPELINE JOINTS (In)
W8+90, 7/20/81	83.50	83.42	0.96	
				0.04
7/20/81	83.50	83.42	0.92	
				0.16
7/20/81	83.51	83.42	1.08	
				0.0
7/20/81	83.50	83.41	1.08	
				0.48
7/20/81	83.56	83.43	1.56	
				0.60
6/10/81	83.48	83.40	2.16	
				0.12
6/10/81	83.51	83.32	2.28	
				0.12
8/1/80	83.50	83.32	2.16	
				0.12
8/1/80	83.55	83.38	2.04	
				0.60
8/1/80	83.52	83.40	1.44	
				0.60
8/1/80	83.46	83.39	0.84	

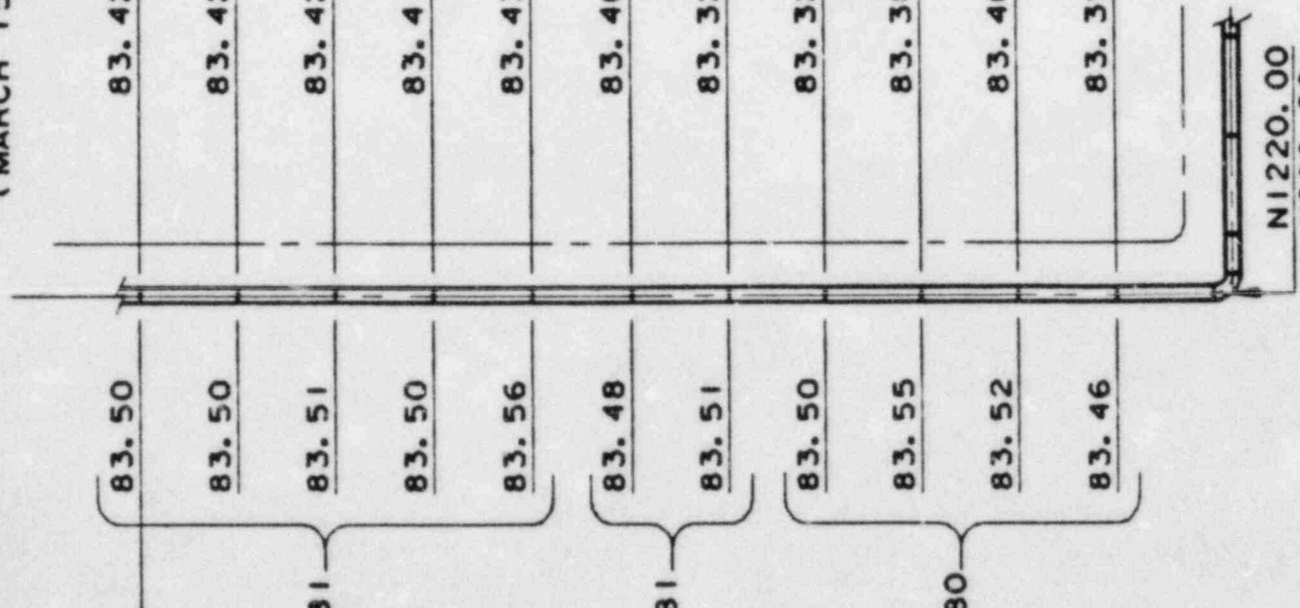

FIG. 3

SETTLEMENT READINGS (MARCH, 1984)

SERVICE WATER PIPELINE 2B

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.5

QUESTION: The response to FSAR question 241.29 should include information on gantry crane piles (including pile tip elevations) and cofferdam configuration adjacent to intake structure. Provide design summary for Cofferdam stability during seismic events without considering liquefaction of soils.

RESPONSE: The Service Water Intake Structure gantry crane support consists of six pile supported foundations. These foundations are independent of the cellular cofferdams. Each foundation contains ten-14 inch diameter by 3/8 inch wall concrete filled pipe piles. The piles are driven to refusal for a 100 ton pile with tip elevation generally varying between elevation 0 feet to 20 feet. All piles are protected by a cathodic protection system. A design summary for cofferdam stability during an SSE event without considering liquefaction of soil is attached. The cofferdam configuration adjacent to the intake structure is shown in the design summary.

CELLULAR COFFERDAM STABILITY ANALYSES

I. Overview

Load combinations, key assumptions, and summary of safety factors for the cofferdam stability analyses are presented. Figures 1 and 2 show the cofferdam and surrounding area.

II. Load Combinations

The following load combinations are evaluated for the cofferdam stability analyses to verify that the structural acceptance criteria, as represented by the corresponding minimum factors of safety for each load combination, are satisfied:

Minimum Factors of Safety

	Load Combination	
Failure Mode	$D + H + E_o$	$D + H + E_s$
Sliding	1.5	1.1
Overturning	1.5	1.1
Centerline Shear	1.5	1.1
Bursting Pressure	4.0	4.0

The load combination $D+H+E_s$ controls the factor of safety of the stability calculations. Figure 3 summarizes all of the loads.

III. Key Assumptions

Key assumptions for the stability check during an SSE event are as follows:

a. Soil layers and properties are based on information discussed in Section 2.5 of HCGS FSAR.

b. The horizontal component of the SSE case is assumed to be 0.2 g and the vertical component is assumed to be 40 percent of the peak acceleration in the upward direction concurrent with horizontal peak acceleration.

III. (Cont'd)

c. The saturation line inside and landward side of the cofferdam is assumed to be the same as the mean water level (el. 89'), therefore, the hydrostatic effects cancel each other.

d. The grouted (or concreted) portion (el. 62' to ± 77') of the fill inside the cofferdam is capable of withstanding a shear stress of $1.1 \sqrt{f_c'}$ or 70 psi.

IV. Summary of Safety Factors for Stability Checks

The factors of safety are shown on Table 1. All calculated safety factors for the stability checks exceed the minimum safety factors specified by Section II.

TABLE 1

SUMMARY OF RESULTS FOR STABILITY CHECKS

Stability Check Against	Controlling Load Combination	Calculated Factor of Safety
Sliding	$D + H + E_s$	1.3 > 1.1 min.
Overturning	$D + H + E_s$	1.14 > 1.1 min.
Centerline Shear	$D + H + E_s$	1.84 > 1.1 min.
Bursting Pressure	$D + H + E_s$	4.6 > 4.0 min.

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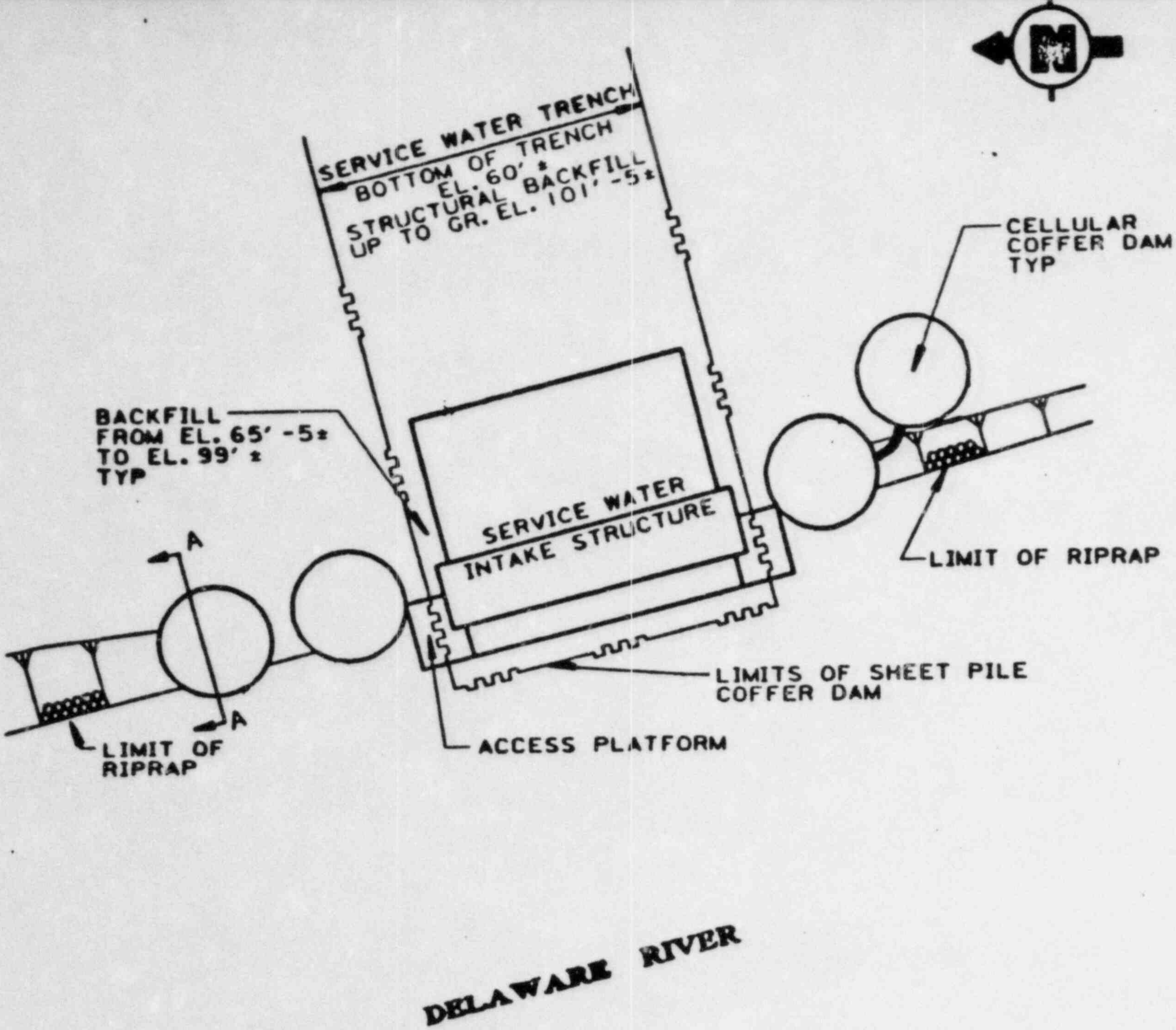
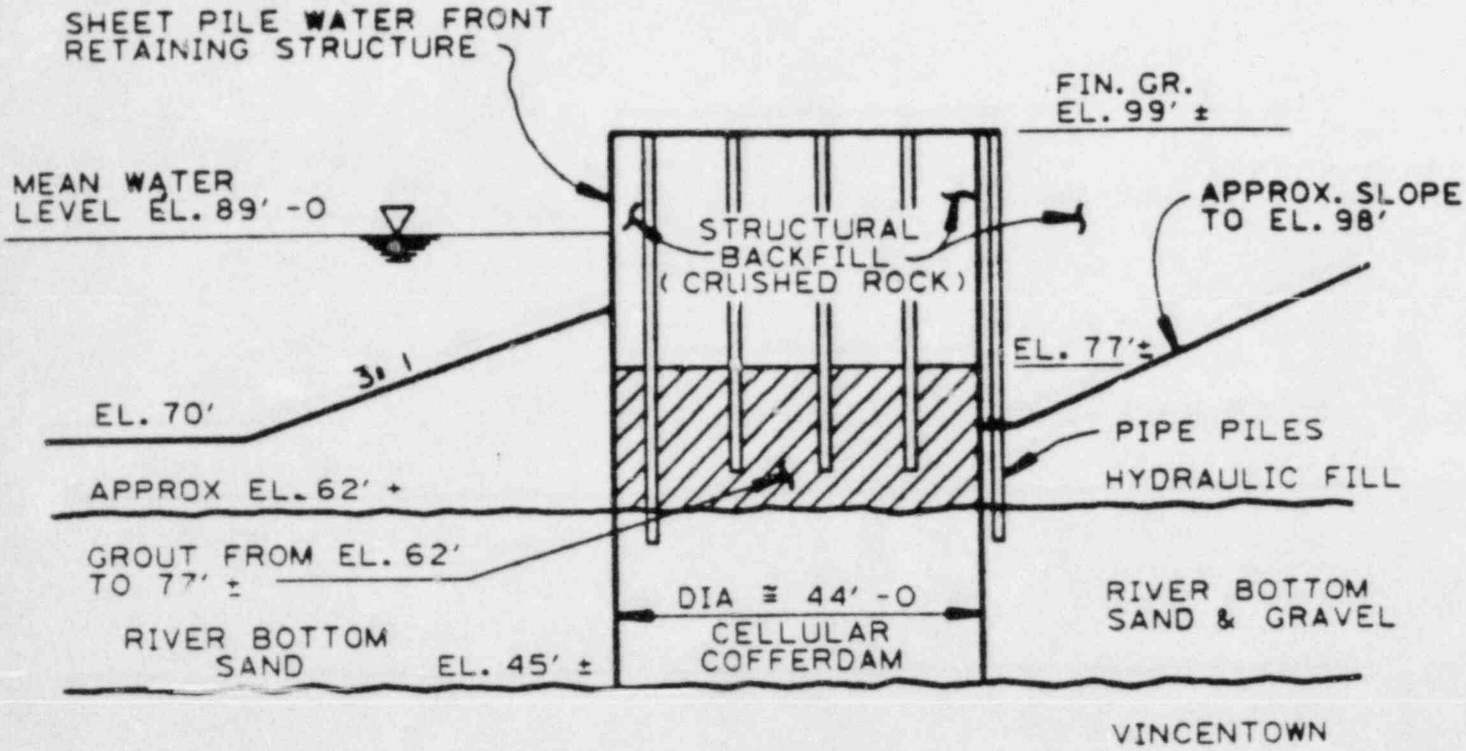
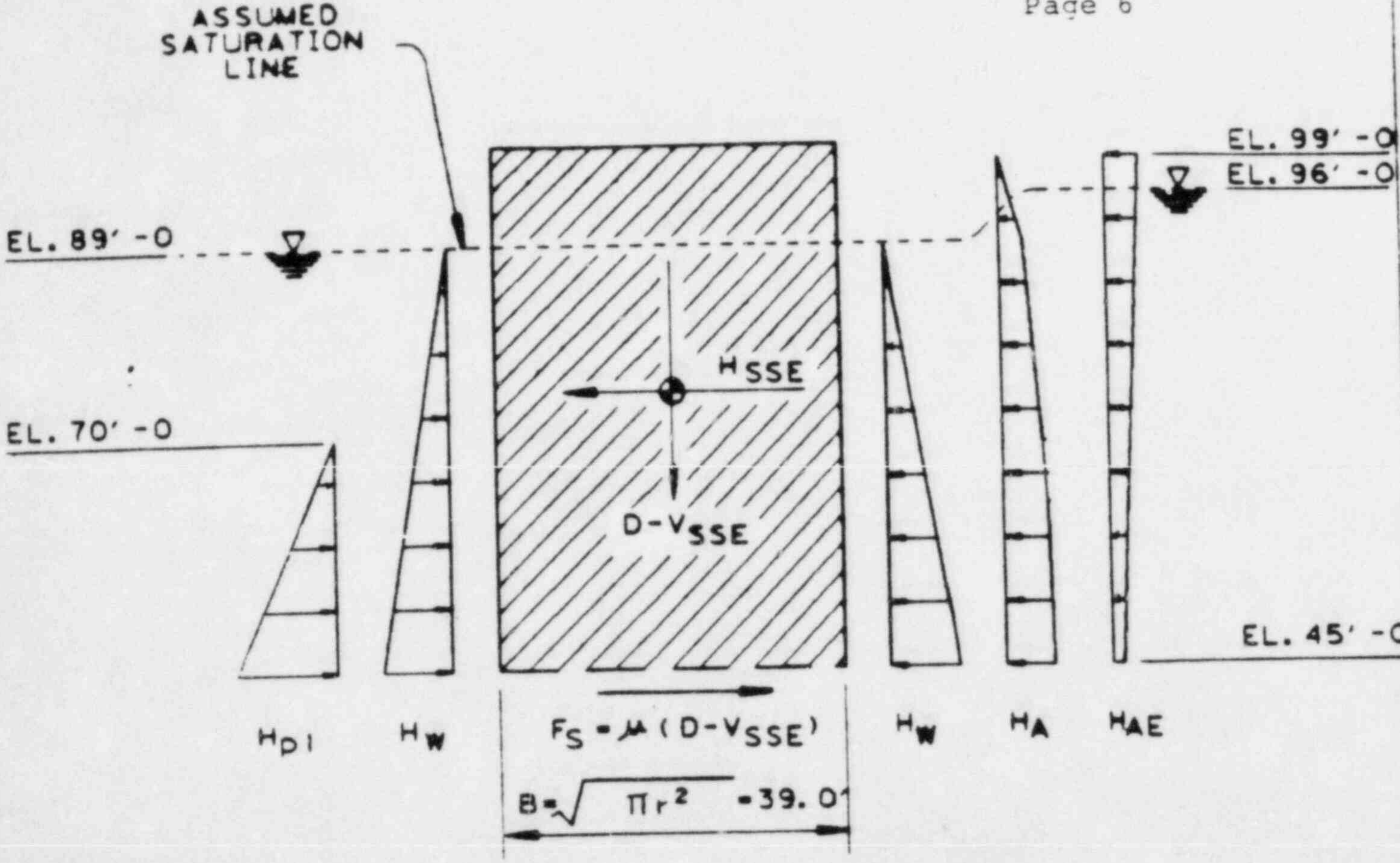


FIGURE - 1



SECTION A-A

FIGURE - 2



FORCE	RESISTING		DRIVING	
DYNAMIC SOIL	-	NEGLECTED	H_{AE}	26.5 K/Ft
STATIC SOIL	H_{DI}	78.4 K/Ft	H_A	45.4 K/Ft
HYDROSTATIC	H_W	60.4 K/Ft	H_W	60.4 K/Ft
DYNAMIC INERTIA	-	-	H_{SSE}	54.8 K/Ft
FRICTION	F_S	90.7 K/Ft	-	-

NOTE:

D = EFFECTIVE WEIGHT = 167 K/Ft

V_{SSE} = VERTICAL DYNAMIC INERTIA = 21.9 K/Ft

μ = COEFFICIENT OF FRICTION = 0.625

FIGURE - 3

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.14

<u>QUESTION</u>: Assess and justify that the current soil modeling for the intake structure adequately accounts for:

- Soil property variability along the depth
- Sheet piling
- Layering of soil including inclined layering

<u>RESPONSE</u>: A response to the above question will be provided in July 1984.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.16

QUESTION: Perform an independent seismic verification analysis (impedance analysis) for the intake structure and compare the results with design basis results. Consider the effects of side boundaries, embedment and the presence of water masses in the analysis.

RESPONSE: A response to the above question will be provided in July 1984.

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Response to NRC Audit

Meeting Date: January 12, 1984

Question No.: A.4

QUESTION: Review the seismic design of all Seismic Category I tanks to determine whether the flexibility of the tank wall and the water mass within the tank were considered. For those tanks where these effects have not been considered, assess the impact of including these effects.

RESPONSE: All Seismic Category I tanks were reviewed to determine whether the flexibility of the tank wall and the water mass within the tank were considered. Review has indicated that in all tanks, except the diesel fuel oil storage tank, fluid mass and tank wall flexibility are addressed adequately. In the case of the diesel fuel oil storage tank, an analysis to qualify the tank to include the effect of fluid mass and tank wall flexibility is in progress.