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May 6, 1983

Mr. Harold R. Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Subject: Braidwood Station Units 1 and 2
Additional FSAR Information
NRC Docket Nos. 50-456/457

References (a): B. J. Youngblood letter to L. O. DelGeorge dated January 14, 1983

(b): B. J. Youngblood letter to L. O. DelGeorge dated February 1, 1983

Dear Mr. Denton:

The above References requested that the Commonwealth Edison Company provide certain additional information concerning our FSAR for Braidwood Station Units 1 and 2.

The Attachment to this letter provides our response to Questions 241.1, 241.2, 241.7, and a revision to 241.6-3. Our FSAR will be amended to include the information contained in the Attachment to this letter as appropriate. Additionally, in supplement to Question 361.5 Part (a), photographs with identifiable sections of the excavations of the main power block have been sent directly to Ms. Janice A. Stevens as listed in the Attachment.

Please address any questions that you or your staff may have concerning this matter to this office.

One (1) signed original and fifteen (15) copies of this letter with Attachment are provided for your use.

Very truly yours,

E. Douglas Swartz

E. Douglas Swartz
Nuclear Licensing Administrator

Attachment

cc: J. G. Keppler - RIII
RIII Inspector - Braidwood

D014

6150N

QUESTION 241.1

"Discuss details of rock excavation by blasting. Discuss how the operation was monitored and what parameters were monitored to control the damage to the bedrock as a result of blasting."

RESPONSE

The criteria for blasting used for rock excavation at the Braidwood Station is covered in Sargent & Lundy Specification L-2714, entitled "Preliminary Site Work." A minimal amount of blasting was required for excavation of the plant foundations. Only eight blasts were used, all occurring between December 31, 1975 and January 22, 1976. No concrete was inplace for any structures at the time of the blasts. The blasts were monitored at the site boundaries using seismographic tests to insure that no damage was caused to residential structures. Blast data for the eight blasts are presented in Table Q241.1-1. The majority of the plant foundations were excavated using conventional construction techniques such as ripping and ram-hoe methods.

TABLE Q241.1

BLAST DATA

BLAST	DATE & TIME	BLAST LOCATION	TYPE OF BLAST	MAXIMUM BLAST LOADING lb/DELAY	MONITORING DISTANCE, Ft.	BLAST MONITORING DATA PEAK VELOCITY, In/Sec.	 PEAK AIR PRESSURE, Lb/In^2
A	12/31/75 4:45 p.m.	Unit 1	Presplit	50 to 128	≃1800	0.11	0.0006
B	12/31/75 4:50 p.m.	Unit 1	Presplit	106 to 110	≃1800	0.12	0.0011
C	01/06/76 4:41 p.m.	Unit 2	Presplit	40 to 96	≃1800	0.12	0.0028
D	01/06/76 4:48 p.m.	Unit 2	Presplit	40	≃1800	0.18	0.0019
E	01/07/76 4:26 p.m.	Unit 2	Production & Presplit	40 to 280	≃1800	0.50	0.0003
F	01/12/76 4:48 p.m.	Unit 1	Production & Presplit	120 to 260	≃2300	0.11	0.0026
G	01/15/76 4:36 p.m.	West of Unit 1&2	Production & Presplit	153	≃2800	0.04	Less than wind & background noise
H	01/22/76 4:30 p.m.	West of Unit 1&2	Production & Presplit	189	≃1900	0.15	-

NOTES:

1. Presplit blasts utilized presplit explosives in the holes; individual holes were detonated with premacord surface line to down hole primacord lines; blasts detonated electrically.

2. Production blasts were loaded with conventional explosives, detonated by electric millisecond (ms) delay firing techniques. All explosive products used were manufactured by Atlas, except for Ensign-Bickford "Primacord."

QUESTION 241.2

"Provide information on the gradation, method of compaction, placement density, and mositure content specified for the granular backfill used beneath and surrounding all Category I structures and buried pipes. Furnish plots presenting results of the quality control field tests performed to verify that the actual construction is in compliance with the specifications."

RESPONSE

Category I granular backfill for the main plant and essential service cooling water pipelines has been discussed in detail in Subsections 2.5.4.5.2.2 and 2.5.4.5.4.2, respectively. Project specifications specified that the backfill material be approved material from previous excavations or borrow areas onsite. The sand backfill used was approved. Figures 2.5-261 and 2.5-262 give an envelope of 58 grain size curves for the granular backfill used in the main plant area and three grain size curves for the granular backfill used in the essential service water pipeline trench also in the main plant area. Figure Q241.2-1 gives an envelope of 12 grain size curves for essential service water pipeline backfill within the essential service water cooling pond.

Specifications required the Category I granular backfill to be compacted by vibratory compactors to minimum 85% Relative Density. A discussion of the results of 273 inplace density tests (ASTM D 1556) for the main plant area is presented in Subsection 2.5.4.5.2.2. These test results indicate compliance with project specifications. Results of the inplace density tests for compacted granular fill placed outside the main plant area of the buried pipeline also indicated compliance with project specifications. The frequency of field density and laboratory testing exceeded the minimum specified. Specifications required the following material testing and frequency.

Material Testing and Frequency

Field and laboratory test measurements shall be performed to the following minimum test frequencies.

TEST	REQUENCY (SEE NOTE 1)
FIELD DENSITY	
Controlled Compacted Fill	A,B,C,E,F,K,L,M
Regular Compacted Fill	A,B,(L*)
COMPACTION	
Controlled Compacted Fill	D,F,G,(L*),M
Regular Compacted Fill	F,J,D
MOISTURE CONTENT	
Borrow	C,D,H
Controlled Compacted Fill	C,H,K,(L*),M
Regular Compacted Fill	C,D,H
GRAIN SIZE	
Controlled Compacted Fill	F,J
Regular Compacted Fill	L
LIFT THICKNESS	
Controlled Compacted Fill	C,D,I
Regular Compacted Fill	C,D,I
RELATIVE DENSITY	
Controlled Compacted Fill	L

NOTE 1 - Letter designations represent the following frequencies or areas for the tests:

A = In areas where degree of compaction is doubtful.

B = In areas where earth fill operations are concentrated.

C = At least one for each earth fill shift.

D = One for every 8,000 yd^3 of fill for control and record.

E = For record tests at location of any embedded items.

F = Where material identity is questionable.

G = One for each field density test as needed.

H = Where soil appears too wet or too dry.

I = Periodic surveillance and measurement checks.

J = One for every 4,000 yd^3 for record.

K = One for every 500 yd^3 for record and control (in confined areas only).

L = One for every 4,000 yd^3 for record and control.

M = One for every 500 linear feet of dike for slurry trench cap.

* Indicates a requirement for the Lake Work in addition to the listed requirements.

Results of inplace density tests made for the essential service water discharge structure are discussed in the response to Question 241.5.

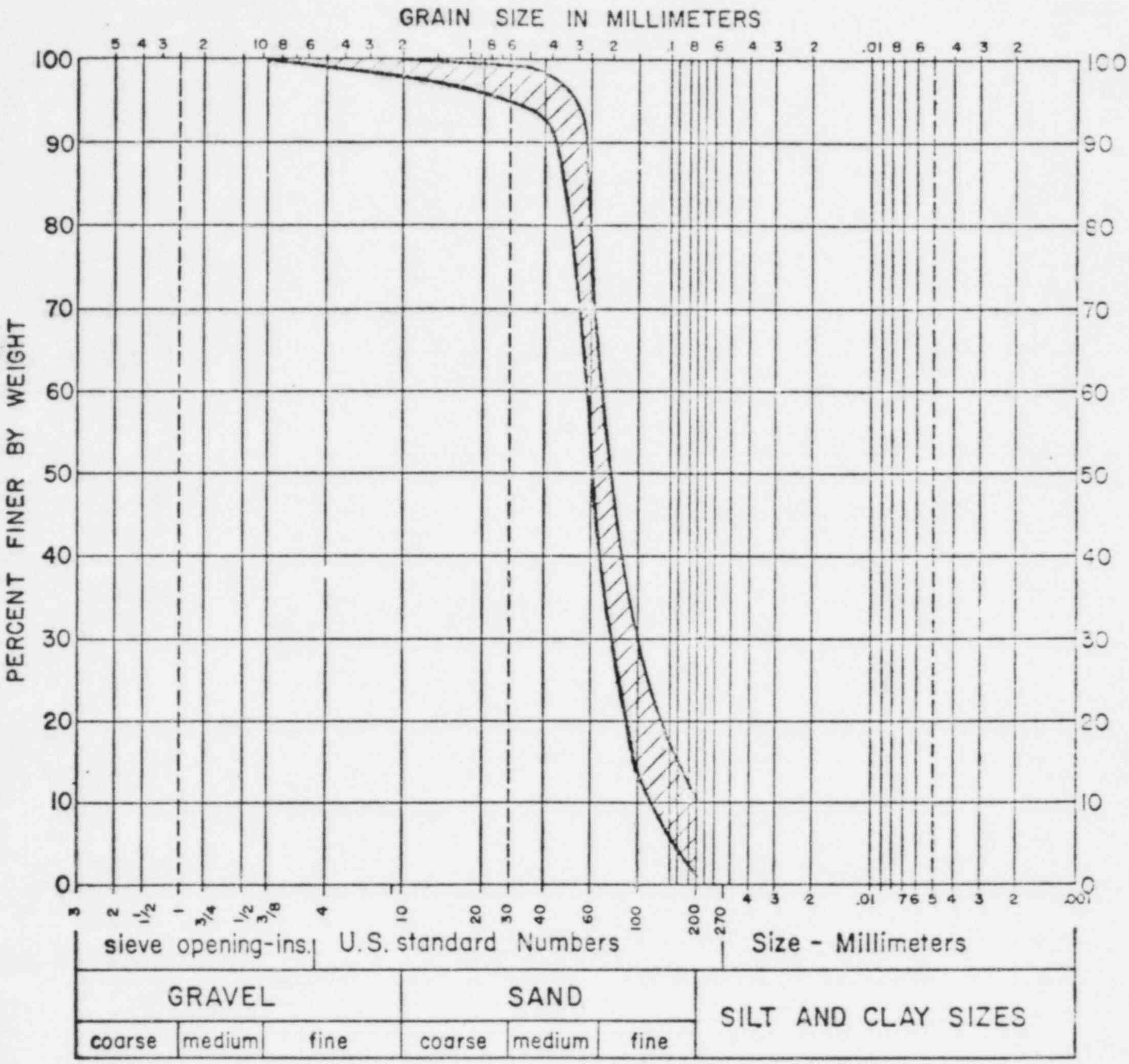

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FIGURE Q241.2-1
GRAIN SIZE ENVELOPE, SAND BACKFILL, ESWP WITHIN ESCP

QUESTION 241.6 - PAGE 3 REVISION

At the ESW discharge structure interface, the discharge pipes are encased in lean concrete and backfilled with granular fill to minimum 85% relative density. Cross-sections are given in Figure Q362.8-1. Backfill has been discussed in response to Question 241.4. Total settlement is calculated to be 1/8 inch or less since the structure and pipeline are supported on Wedron silty clay till. See the response to Question 241.4 for further discussion of the ESW discharge structure.

(4) The ESW pipelines are either founded on Wedron glacial till or compacted granular fill within the main plant excavation. The pipelines are backfilled with bash, concrete, or compacted fill. The compacted fill was placed to a minimum 85% relative density. The glacial till and compacted fill are not susceptible to liquefaction. For further discussion, see Subsection 2.5.6.5.2 on liquefaction potential and the response to Question 241.7.

(5) The ESWS pipelines are founded on Wedron silty clay till and are backfilled with bash to the top of the pipes. Figure 2.5-25 shows a profile along the pipeline alignment. The top of the till is above the top of the pipes in most areas and in all cases is above the pipe centerline. The till and bash will not erode if the circulating water supply pipes should break.

(6) <u>Quantitiative and Procedural Details of the Dynamic Analysis of the Seismic Category I Buried Piping</u>

The methodology used to perform the dynamic analysis of the seismic Category I buried piping is described in the response to Question 130.33.

The variability of the supporting soil strata has been accounted for in the dynamic analysis by conservatively choosing the design particle velocity and the apparent shear wave velocity. The static properties of the in situ soil and compacted fill have been accounted for by conservatively choosing the modulus of subgrade reaction.

QUESTION 241.7

"Provide the following information for the Essential Service Cooling Pond (ESCP) slopes:

"1. Present a figure showing the critical section of the ESCP slope analyzed for static stability. Show the critical failure surface and the corresponding factor of safety against failure.

"2. What was the seismic coefficient used in evaluating the dynamic stability of the ESCP slope by pseudostatic method of analysis? What are the minimum factors of safety for seismic coefficients of 0.20g and 0.26g?

"3. Evaluate the dynamic stability of the ESCP slope.

"4. The liquefaction study using the SHAKE program evaluated the case with the level ground at elevation 590.0 ft. Provide the results of a similar study for the ESCP bottom, at elevation 584.0 feet.

"5. Discuss the static and dynamic stability of the Category I sheet pile wall adjoining the Lake Screen house. Present a detailed cross section and plan of the critical section analyzed for stability.

"6. Investigate the potential for blockage of the entrance to the Lake Screen house as a result of a catastrophic flow type of failure of the ESCP slopes in the immediate vicinity of the screen house."

RESPONSE

1. The critical section of the ESCP slope analyzed for static stability is given in Figure Q241.7-1. The analysis is for end of construction condition with a minimum factor of safety of 5.9 as discussed in Subsection 2.5.6.5.1.2.

2. The seismic coefficient used in evaluating the dynamic stability of the ESCP slope by pseudostatic method of analysis was 0.2g. The minimum factor of safety is 1.3 as discussed in Subsection 2.5.6.5.1.2. The ESCP slope has also been analyzed with a seismic coefficient of 0.26g and the minimum factor of safety is 1.1.

3. The dynamic stability of the ESCP slope by finite element methods was not performed because the pseudostatic analysis used yields conservatiave results and a greater minimum

factor of safety would be obtained if a finite element method were used. This is the case because the method of analysis employed assumes application of the seismic force at the base of each slice rather than at the centroid. It should be noted that factor of safety is determined by a comparison of overturning moments and resisting moments and that no consideration is given to the effects of side forces on slices in making the computations. The seismic force is assumed to increase only the overturning moment and to have no influence on the resisting moment. The soil strength properties have also been based on triaxial compression tests rather than plane strain tests. This is also conservative. Discussion of the conservative nature of these assumptions can be found in the paper by H. B. Seed, K. L. Lee, and I. M. Idriss on the Analysis of Sheffield Dam Failure, Journal of the Soil Mechanics and Foundations Division, November 1969.

The minimum factor of safety for slope stability using pseudostatic analysis with a seismic coefficient of 0.2g was 1.3. With a seismic coefficient of 0.26g the minimum factor of safety is 1.1, which is considered acceptable.

4. The liquefaction potential of the ESCP bottom at the Braidwood Station was evaluated by calculating a factor of safety (τ_f/τ_d) defined as the ratio of the shear stress required to cause liquefaction (τ_f) and the shear stress induced by the SSE (τ_d). The shear stress required to cause liquefaction was calculated based on laboratory cyclic strength tests on reconstituted test specimens and corrected for the effects of specimen reconstitution and to adjust for differences in stress conditions between the field and laboratory (refer to FSAR Equation 2.5-14). The stresses induced by the SSE were computed using the program SHAKE. The resulting stress distribution induced in 10 cycles for level ground at elevation 590 feet is shown in Figures 2.5-116 and 2.5-117.

   Calculations indicating the various correction factors and the resulting factors of safety are presented in Tables Q241.7-1 through Q241.7-4. Factors of safety against liquefaction are calculated and presented for level ground at elevations 590 feet and 584 feet, and "average" and "low average" relative density conditions corresponding to the development of initial liquefaction (IL), ±5% axial strain, and ±10% axial strain.

The induced stresses, (τ_d), and the stresses required to cause $\pm 10\%$ strain (τ_f) for level ground at elevation 590 feet are compared in Figures 2.5-116 and 2.5-117, and for level ground at elevation 584 feet in Figures Q241.7-3 and Q241.7-4. Figures Q241.7-3 and Q241.7-4 are plots of data from Table Q241.7-3 which correspond to average relative density conditions.

Based on results of the liquefaction potential evaluation presented, it is concluded there is an ample margin of safety against liquefaction of the sand deposits within the ESCP for level ground surface at both elevations 590 feet and 584 feet. Details of the analysis are discussed below.

Selection of Parameters

The following parameters were used to calculate τ_f/τ_d:

Parameter	Description and Source
τ_d	Shear stress induced by SSE. τ_d values are plotted with depth of the soil profile in Figures 2.5-116 and 2.5-117. These were computed based on a SHAKE analysis for level ground at elevation 590 feet were calculated by using a simplified procedure for evaluating stresses using a simplified procedure for evaluating stresses described in Reference 1 (Seed & Idriss, 1982). The SHAKE program has been run for level ground at elevation 584 feet and verifies that the τ_d values shown in Tables Q241.7-3 and Q241.7-4 are conservative.
$(\sigma_d/2\sigma_{ec})$	Stress ratio representative of laboratory cyclic strength of reconstituted test specimens at N=10 stress cycles. $(\sigma_d/2\sigma_{3c})$ values vs. N are plotted in Figures 2.5-100 and 2.5-101 for soil type and relative density/fines content properties.
D_c	Correction factor to adjust for effect of specimen reconstitution. D_c values are plotted in Figure 2.5-110 and are dependent or relative density and strain condition.

C_r Correction factor dependent on relative density or K_o, as appropriate. The selection of C_r based on D_r was made on the basis of an equivalent Sacramento River Sand D_r = 90% and the curve presented in Figure 2.5-111. The C_r value for D_r = 90% is 0.75. The selection of C_r based on K_o was obtained from Figure 2.5-113. The value of K_o was obtained based on OCR from Figure 2.5-115. The OCR was calculated as shown in Figure 2.5-114 for level ground at elevation 590, and in Figure Q241.7-2 for level ground at elevation 584. For OCR greater than 4.5 a K_o value of 0.88 is selected. For K_o = 0.88, a value of C_r = 0.83 is selected from Figure 2.5-113.

Calculation Method

The calculated factors of safety (FS) are presented in Tables Q241.7-1 through Q241.7-4. Three FS values are calculated (columns (8), (11), and (14) for each elevation and strain condition considered. The method used to calculate FS is as follows:

(1) FS in column (8)

$$FS = \frac{C_r \cdot \sigma_v \cdot (\sigma_d/2\sigma_{3c})}{\tau_d}$$

where C_r = 0.75

(2) FS (C_r based on D_r) in column (11)

FS (C_r based on D_r) = FS $\cdot$ D_c

(3) FS (C_r based on K_o) in column (14)

$$FS\ (C_r \text{ based on } K_o) = FS\ (C_r \text{ based on } D_r) \cdot \frac{C_r}{0.75}$$

where C_r obtained from Figure 2.5-113.

<u>Reference</u>

1. "Ground Motions and Soil Liquefaction During Earthquakes," Seed & Idriss, Earthquake Engineering Research Institute, 1982.

5. There is no Category I sheetpile wall adjoining the lake screen house. The retaining walls adjoining the lake screen house are reinforced concrete wing walls founded on Wedron silty clay till between elevations 561 feet 9 inches and 569 feet 0 inch. The walls are designed as Category I and extend as much as 100 feet east and west of the screen house. Plans and sections of the walls are given in Figures Q241.7-2, Q241.7-3, and Q241.7-4.

6. The ESCP slopes in the immediate vicinity of the lake screen house are 10 horizontal to 1 vertical and are protected with a 2-foot thick layer of bedding and riprap. The ESCP slopes have been shown to be stable and have an ample margin of safety against liquefaction during the unlikely event of the postulated SSE as discussed in Subsection 2.5.6.5 and this question response. A plan of the ESCP slopes in the vicinity of the lake screen house is shown in Figure Q241.4-3.

TABLE Q241.7-1

FS Against Liquefaction for Average Relative Density Conditions

Level Ground at Elev. 590.0 ft

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $(\frac{\sigma_d}{2\sigma_{3c}})$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
588	Brown	48.2	IL	134	0.56	56.3	1.17	1.00	56.3	1.17	0.83	62.3	1.30
	Fine	48.2	±5%	134	0.70	70.5	1.46	1.19	83.7	1.74	0.83	92.7	1.94
	Silty	48.2	±10%	134	0.83	83.5	1.72	1.38	114.0	2.36	0.83	127.0	2.63
	Sand												
585	Brown	115.0	IL	335	0.56	141.0	1.22	1.00	140.3	1.22	0.83	154.1	1.34
	Fine	115.0	±5%	335	0.70	176.0	1.53	1.19	203.6	1.77	0.83	226.6	1.97
	Silty	115.0	±10%	335	0.83	208.0	1.80	1.38	280.0	2.43	0.83	310.0	2.70
	Sand												
585	Gray	115.0	IL	335	0.54	135.8	1.18	1.03	143.8	1.25	0.83	159.9	1.39
	Fine	115.0	±5%	335	0.63	158.0	1.37	1.25	196.7	1.72	0.83	219.6	1.91
	Sand	115.0	±10%	335	0.75	188.5	1.64	1.42	266.0	2.32	0.83	296.0	2.58
570	Gray	368.0	IL	1340	0.54	543.0	1.48	1.03	581.4	1.58	0.65	515.2	1.40
	Fine	368.0	±5%	1340	0.63	633.0	1.72	1.25	802.2	2.18	0.65	706.6	1.92
	Sand	368.0	±10%	1340	0.75	755.0	2.05	1.42	1070.0	2.92	0.65	945.0	2.57

TABLE Q241.7-2

FS Against Liquefaction for Low Average Relative Density Conditions

Level Ground at Elev. 590.0 ft

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condi- tion	(5) σ_v σ_v	(6) $(\frac{\sigma_d}{2\sigma_{3c}})$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o	(13) τ_f (psf)	(14) FS (C_r based on K_o)
588	Brown	48.2	IL	134	0.53	53.3	1.11	0.99	53.0	1.10	0.83	59.3	1.23
	Fine	48.2	+5%	134	0.62	62.3	1.29	1.17	72.8	1.51	0.83	81.0	1.68
	Silty	48.2	±10%	134	0.79	79.5	1.65	1.36	109.4	2.27	0.83	121.9	2.53
	Sand												
585	Brown	115.0	IL	335	0.53	133.0	1.13	0.99	130.0	1.13	0.83	144.9	1.26
	Fine	115.0	+5%	335	0.62	156.0	1.36	1.17	178.2	1.55	0.83	199.0	1.73
	Silty	115.0	±10%	335	0.79	198.5	1.72	1.36	265.6	2.31	0.83	295.6	2.57
	Sand												
585	Gray	115.0	IL	335	0.48	120.5	1.05	1.00	120.8	1.05	0.83	134.6	1.17
	Fine	115.0	+5%	335	0.55	138.0	1.20	1.19	164.4	1.43	0.83	182.8	1.59
	Silty	115.0	±10%	335	0.61	153.0	1.33	1.38	212.8	1.85	0.83	236.9	2.06
	Sand												
570	Gray	368.0	IL	1340	0.48	482.0	1.34	1.00	493.1	1.34	0.65	434.2	1.18
	Fine	368.0	+5%	1340	0.55	552.0	1.50	1.19	666.1	1.81	0.65	588.8	1.60
	Sand	368.0	±10%	1340	0.61	613.0	1.67	1.38	853.8	2.32	0.65	754.4	2.05

TABLE Q241.7-3

FS Against Liquefaction for Average Relative Density Conditions

Level Ground at Elev. 584.0 ft

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $(\frac{\sigma_d}{2\sigma_{3c}})$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
582	Gray	48.2	IL	134	0.54	54.3	1.12	1.03	55.9	1.16	0.83	61.9	1.28
	Fine	48.2	±5%	134	0.63	63.3	1.31	1.25	79.1	1.64	0.83	87.6	1.82
	Sand	48.2	±10%	134	0.75	75.4	1.56	1.42	112.4	2.33	0.83	124.4	2.58
579	Gray	115.0	IL	335	0.54	135.7	1.18	1.03	139.7	1.21	0.83	154.6	1.34
	Fine	115.0	±5%	335	0.63	158.3	1.38	1.25	197.9	1.72	0.83	219.0	1.90
	Sand	115.0	±10%	335	0.75	188.4	1.64	1.42	267.6	2.33	0.83	296.1	2.58
577.5	Gray	148.8	IL	435.5	0.54	176.4	1.18	1.03	181.7	1.22	0.83	201.5	1.35
	Fine	148.8	±5%	435.5	0.63	205.8	1.38	1.25	257.2	1.73	0.83	285.3	1.92
	Sand	148.8	±10%	435.5	0.75	245.0	1.65	1.42	347.8	2.34	0.83	385.9	2.59
570	Gray	309.0	IL	938	0.54	379.9	1.23	1.03	391.3	1.27	0.73	380.8	1.23
	Fine	309.0	±5%	938	0.63	443.2	1.43	1.25	554.0	1.79	0.73	539.2	1.74
	Sand	309.0	±10%	938	0.75	527.6	1.71	1.42	749.2	2.42	0.73	729.2	2.36

TABLE Q241.7-4

FS Against Liquefaction for Low Average Relative Density Conditions

Level Ground at Elev. 584.0 ft

(1) Elev	(2) Soil	(3) τ_d (psf)	(4) Strain Condition	(5) σ_v (psf)	(6) $(\frac{\sigma_d}{2\sigma_{3c}})$ N=10	(7) τ_f (psf)	(8) FS	(9) D_c (based on D_r)	(10) τ_f (psf)	(11) FS (C_r based on D_r)	(12) C_r (based on K_o)	(13) τ_f (psf)	(14) FS (C_r based on K_o)
582	Gray	48.2	IL	134	0.48	48.2	1.00	1.00	48.2	1.00	0.83	53.5	1.11
	Fine	48.2	+5%	134	0.55	55.3	1.15	1.19	65.8	1.36	0.83	73.0	1.51
	Sand	48.2	±10%	134	0.61	61.3	1.27	1.38	84.6	1.76	0.83	93.8	1.95
579	Gray	115.0	IL	335	0.48	120.6	1.05	1.00	120.6	1.05	0.83	133.8	1.16
	Fine	115.0	+5%	335	0.55	138.2	1.20	1.19	164.4	1.43	0.83	182.4	1.59
	Sand	115.0	±10%	335	0.61	153.3	1.33	1.38	211.5	1.84	0.83	234.6	2.04
577.5	Gray	148.8	IL	435.5	0.48	156.8	1.05	1.00	156.8	1.05	0.83	173.9	1.17
	Fine	148.8	+5%	435.5	0.55	179.6	1.21	1.19	213.8	1.44	0.83	237.1	1.59
	Sand	148.8	±10%	435.5	0.61	199.2	1.34	1.38	274.9	1.85	0.83	305.0	2.05
570	Gray	309.0	IL	938	0.48	337.7	1.09	1.00	337.7	1.09	0.73	328.7	1.06
	Fine	309.0	+5%	938	0.55	386.9	1.25	1.19	460.4	1.49	0.73	448.2	1.45
	Sand	309.0	±10%	938	0.61	429.1	1.39	1.38	592.2	1.92	0.73	576.4	1.86

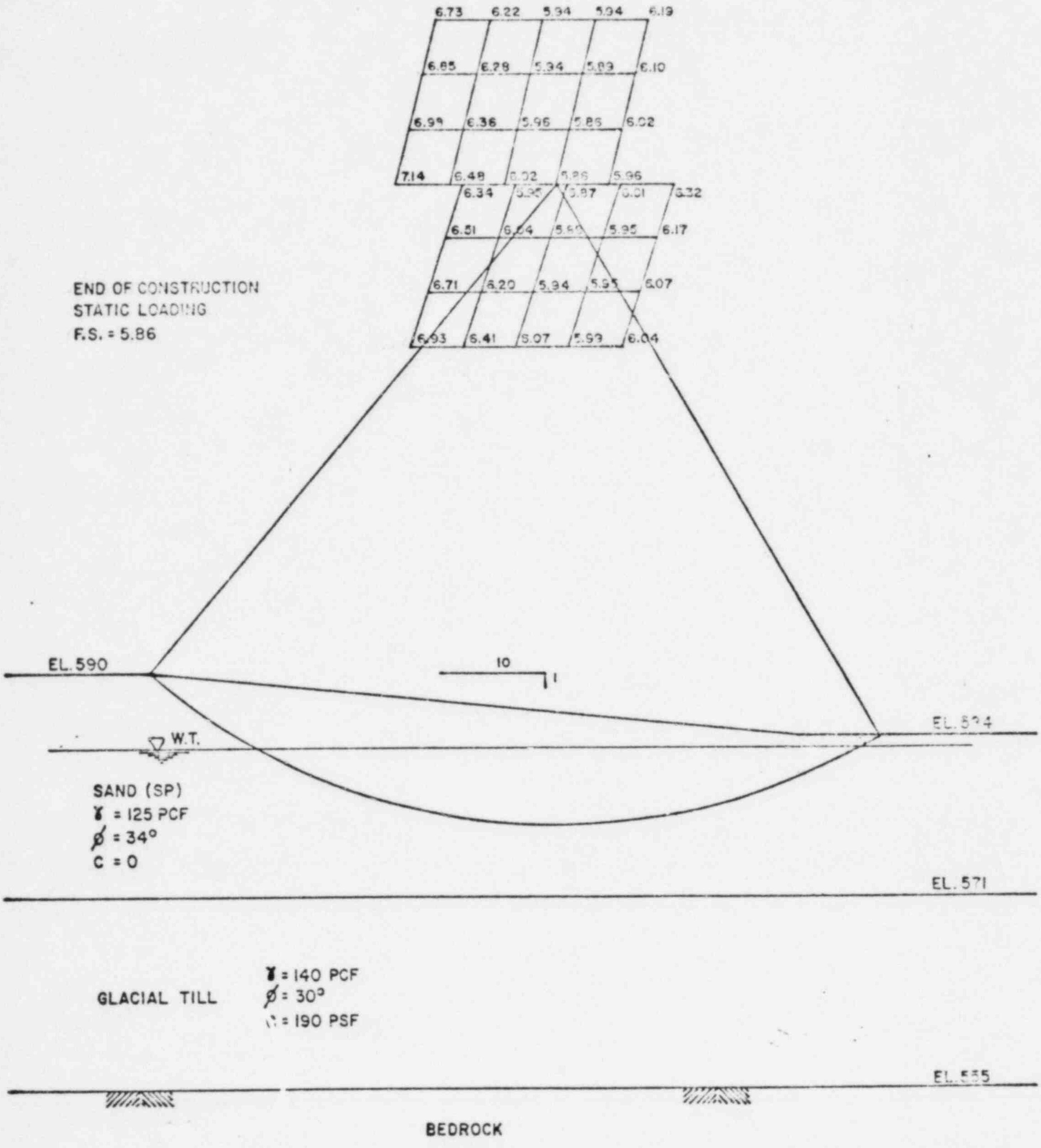

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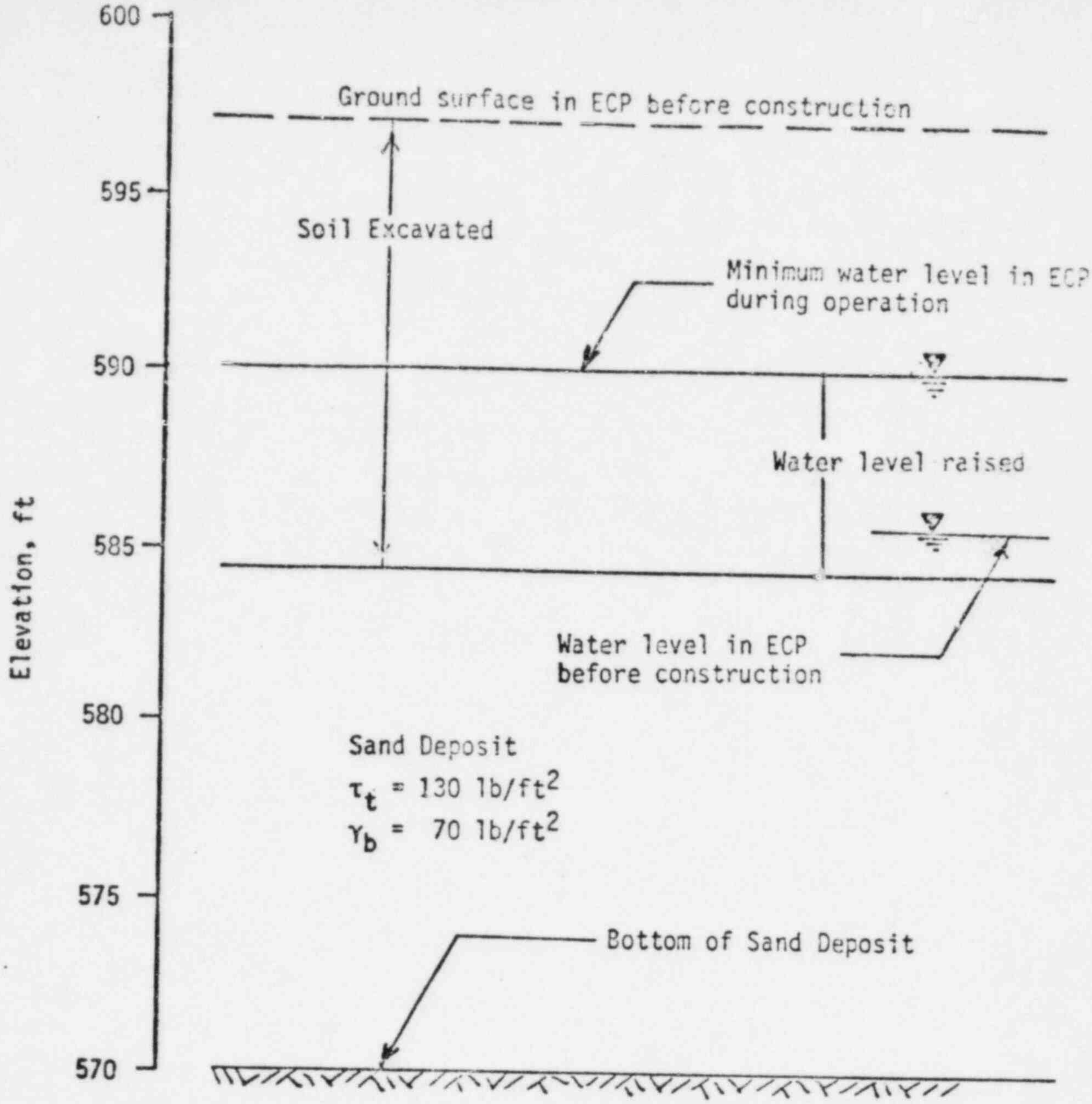
FIGURE Q241.7-1

CRITICAL SECTION FOR STATIC ESCP SLOPE STABILITY ANALYSIS



Before Construction

el.ft	$\bar{\sigma}_b$ lb/ft^2	K_o	$\bar{\sigma}_h$ lb/ft^2
577.5	2085	0.40	834
575.0	2260	0.40	900
570.0	2610	0.40	1040

During Construction

$\bar{\sigma}_a$ lb/ft^2	OCR*	K_o
455	4.5	0.88
630	3.6	0.78
980	2.7	0.70

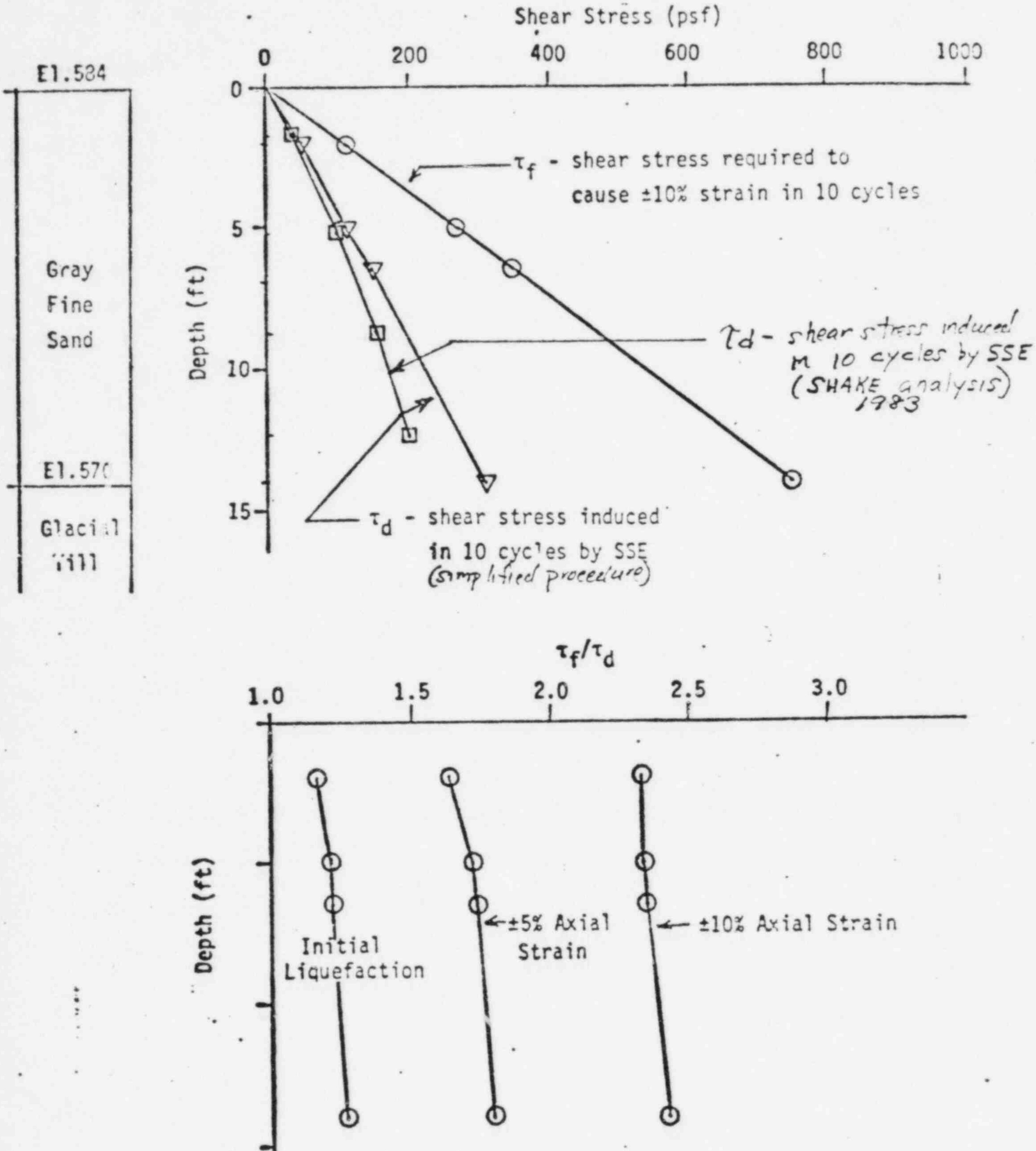
* OCR = Overconsolidation Ratio = $\frac{\bar{\sigma}_b}{\bar{\sigma}_a}$ = $\frac{\text{Effective Overburden Pressure Before Construction}}{\text{Effective Overburden Pressure After Construction}}$

BRAIDWOOD STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.7-2

MINIMUM PRINCIPAL STRESS RATIO WITHIN SAND DEPOSIT DURING OPERATION AT EL 584 FT

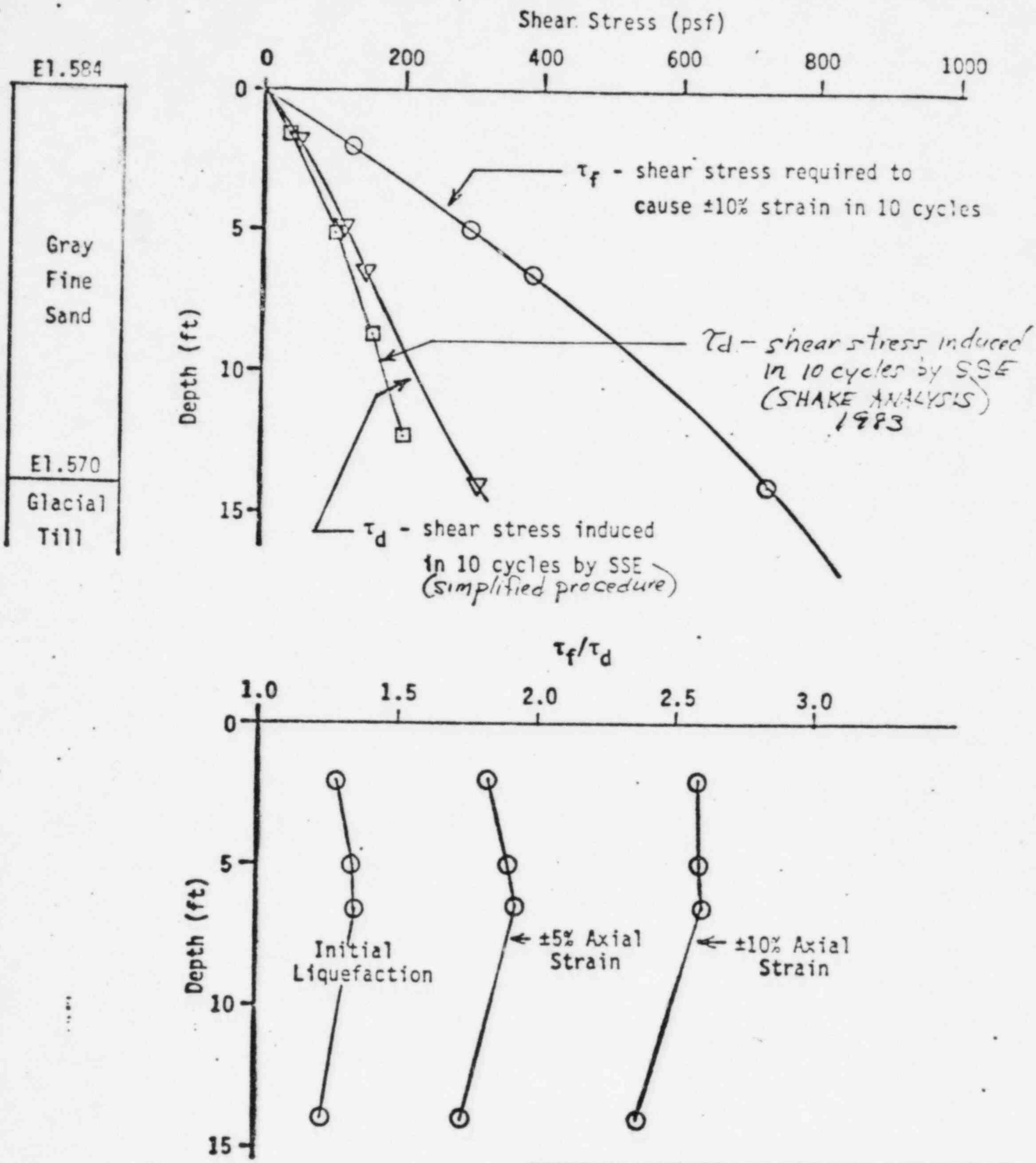


BRAIDWOOD STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q241.7-3

EVALUATION OF LIQUEFACTION POTENTIAL - LEVEL GROUND AT El 584 FT - C_r BASED ON D_r



BRAIDWOOD STATION

FINAL SAFETY ANALYSIS REPORT

FIGURE Q 241.7 - 4

EVALUATION OF LIQUEFACTION POTENTIAL - LEVEL GROUND AT EL 584 FT - C_r BASED ON K_o

Supplemental Information Requested by

NRC Following Review of Response to

Braidwood Question 361.5, part a

Question 361.5, part a, indicated that 190 photographs of 145 locations have been taken in excavations of the main power block. A map of these locations is given in Figure Q361.5-1. Prints of the above photographs with identifiable sections, control points, and photograph numbers are enclosed (sheets 2 through 34). Also enclosed is a table listing the photographs, control points, and section numbers (sheets 1 and 1a).

SUPPLEMENTAL INFORMATION FOR RESPONSE TO BRAIDWOOD QUESTION 361.5, PART A

TABLE OF SECTIONS, CONTROL POINTS, AND PHOTOGRAPHS

DATE OF WORK	PHOTO NUMBER	SECTION NUMBER	CONTROL POINT NUMBER	
2-25-76	1	1	1	
"	2-11			
"	12	2	no control point	No contact in section
"	13-26			
"	27	3	3	
"	28-38			Photos 37-38=2 Photos each
"	39	4	4	2 photos
"	40-50			Photos 40-42=2 photos, photo 44 is a polaroid duplicate, photos 45-50=2 photos
"	51	5	5	2 photos
"	52-61			Photo 52=3 photos, 53=2, 54=1, 55-59=2, 60-61=3
"	62	6	6	2 photos
"	63-70			Photo 69 = 2 photos, photos 63 & 64 = 2 photos
2-26-76	71-72	7	7	
"	73-74			
"	75	8	8	Photo 75 = 2 photos
"	76-86			Photo 76 = 2 photos, Photo 77 = 3 photos
"	87	9	9	
"	88-93			Photo 93 = 2 photos
"	94	10	10	
"	95-100			
"	101	11	11	In excavation mapping report
"	102-111			Photos 102, 103,& 111 in excavation mapping report, photo 108 = 2 photos
"	112	12	12	In excavation report
"	113-119			Photo 113 in excavation mapping report
2-27-76	120	13	13	Control point 5-13 was destroyed by construction.
"	121			
"	122	14	14	
"	123			
"	124	15	15	
"	125			
"	126	16	16	
"	127-128			3 photos
"	129	17	17	
"	130			
"	131	18	18	

SUPPLEMENTAL INFORMATION FOR RESPONSE TO
BRAIDWOOD QUESTION 361.5, PART A

TABLE OF SECTIONS, CONTROL POINTS, AND PHOTOGRAPHS (Cont'd)

DATE OF WORK	PHOTO NUMBER	SECTION NUMBER	CONTROL POINT NUMBER	
2-27-76	132	19	19	
"	133			
"	134	20	20	
2-28-76	135	21	21	
"	136			
"	137	22	22	
"	138	23	23	soil section, 3 photos, one photo in excavation mapping report
"	no photos	no sections	24-27	*T.O.R control points; control point S-24 was destroyed by construction.
3-1-76	139	28	28	T.O.R soil section, 2 photos
"	no photos	no sections	29-30	T.O.R control points, control point S-30 was destroyed by construction.
"	140	37	37	2 photos
"	141	38	38	4 photos
3-1-76 & 3-2-76	142	31	31	T.O.R. soil section, 2 photos, control point S-31 was destroyed by construction
"	no photos	no sections	32	T.O.R control point, control point S-32 was destroyed by construction.
"	143	39	39	T.O.R
"	144	33	33	T.O.R Soil section, control point S-33 was destroyed by construction.
3-1-76 & 3-3-76	no photos	no sections	34-35	T.O.R control points
"	145	36	36	T.O.R soil section, 3 photos

Note:
T.O.R = Top of Rock