KANSAS GAS AND ELECTRIC COMPANY

GLENN L. KOESTER
VICE PRESIDENT - NUCLEAR

September 10, 1981

RECEIVED
SEP 11 1981
U.S. NUCLEAR REGULATORY COMMISSION

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

KMLNRC 81-110
Re: Docket Number STN 50-482
Ref: NRC Letter dated 8/7/81 from RLTedesco, NRC,
to GLKoester, KG&E

Dear Mr. Denton:

The referenced letter requested additional information in the area of geotechnical engineering. Transmitted herewith are responses to questions in the referenced letter. The outstanding response to question 241.5 WC, Items 2 thru 5, will be forwarded to you on October 1, 1981. This information will be formally incorporated into the Wolf Creek Generating Station, Unit No. 1 Final Safety Analysis Report in Revision 6. This information is hereby incorporated into the Wolf Creek Generating Station, Unit No. 1 Operating License Application.

Yours very truly,

Glenn L Koester

GLK:bb
Attach

cc: Dr. Gordon Edison (2)
Division of Project Management
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Thomas Vandel
Resident NRC Inspector
Box 311
Burlington, Kansas 66839

B001
s
1/1

OATH OF AFFIRMATION

STATE OF KANSAS)
) SS:
COUNTY OF SEDGWICK)

I, Glenn L. Koester, of lawful age, being duly sworn upon oath, do depose, state and affirm that I am Vice President - Nuclear of Kansas Gas and Electric Company, Wichita, Kansas, that I have signed the foregoing letter of transmittal, know the contents thereof, and that all statements contained therein are true.

KANSAS GAS AND ELECTRIC COMPANY

ATTEST:

By Glenn L Koester
Glenn L. Koester
Vice President - Nuclear

W B Walker
W.B. Walker, Secretary

STATE OF KANSAS)
) SS:
COUNTY OF SEDGWICK)

BE IT REMEMBERED that on this 10th day of September, 1981, before me, Evelyn L. Fry, a Notary, personally appeared Glenn L. Koester, Vice President - Nuclear of Kansas Gas and Electric Company, Wichita, Kansas, who is personally known to me and who executed the foregoing instrument, and he duly acknowledged the execution of the same for and on behalf of and as the act and deed of said corporation.

IN WITNESS WHEREOF, I have hereunto set my hand and affixed my seal the date and year above written.

EVELYN L. FRY
NOTARY PUBLIC
STATE OF KANSAS

Evelyn L. Fry
Evelyn L. Fry, Notary

My Commission expires on August 15, 1985.

Q241.1 In Figure 2.5-97a through 2.5-97e show the data points used in developing these curves. Also plot the mean and the standard deviation curves.

R241.1 Data points have been added to Figures 2.5-97d and e, Figure 241.1-1 and 241.1-2. No dynamic tests were performed on the pipe bedding material, since several alternative materials were to be used as pipe bedding. The pipe bedding materials ranged in gradation from gravelly sand to medium sand with little or no fines (see response to Question 241.3). The shear modulus range and strain degradation curves (Figure 2.5-97c) were, therefore, chosen as those for dense sands and gravelly sands as presented in "Soil Behavior Under Earthquake Loading Conditions"; State of the Art Evaluation of Soil Characteristics for Seismic Response Analysis; Shannon and Wilson, 1972.

Dynamic triaxial tests on the Heumader shale were not performed due to problems with slaking during coring and high fissility of the core. Resonant column tests were attempted on some samples (Table 2.5-38), however, due to uncertainty regarding the applicability of the resonant column tests on rock samples (insufficient apparatus stiffness) and the problems with slaking and fissility of the core samples, these test results were not regarded as reliable and were only used for evaluation of a possible lower bound shear modulus. The strain degradation curves on Figure 2.5-97a and b, were, therefore, based on the geophysical test results for anchor points at 10^{-4} percent shear strain, the strain degradation curves for the residual soils (Figure 2.5-97f), and judgment.

The shear wave velocities at the plant site, measured along an open end line using a sledgehammer for impact energy, indicated an average shear wave velocity for the Upper and Lower Heumader shales in the range of 1400 to 1500 feet per second. Since the strength of the Lower Heumader shale (being calcareous in nature) is higher than the Upper Heumader shale, the shear wave velocity in the Upper Heumader shale should be lower than that in the Lower Heumader shale. The resonant column tests on samples from the Upper Heumader shale showed shear wave velocities in the range of 500 to 800 feet per second. Considering that these tests results would be too low (insufficient testing apparatus stiffness and shale fissility), the shear wave velocity for the Upper Heumader shale

R241.1 (continued)

at the plant site was estimated to be 1000 feet per second. Thus, since the average velocity for the Heumader shale was measured in the range of 1400 to 1500 feet per second, the shear wave velocity of the Lower Heumader shale was estimated to be 1800 feet per second. These velocities correspond to shear moduli of approximately 5×10^6 and 15×10^6 pounds per square foot for the Upper and Lower Heumader shales, respectively.

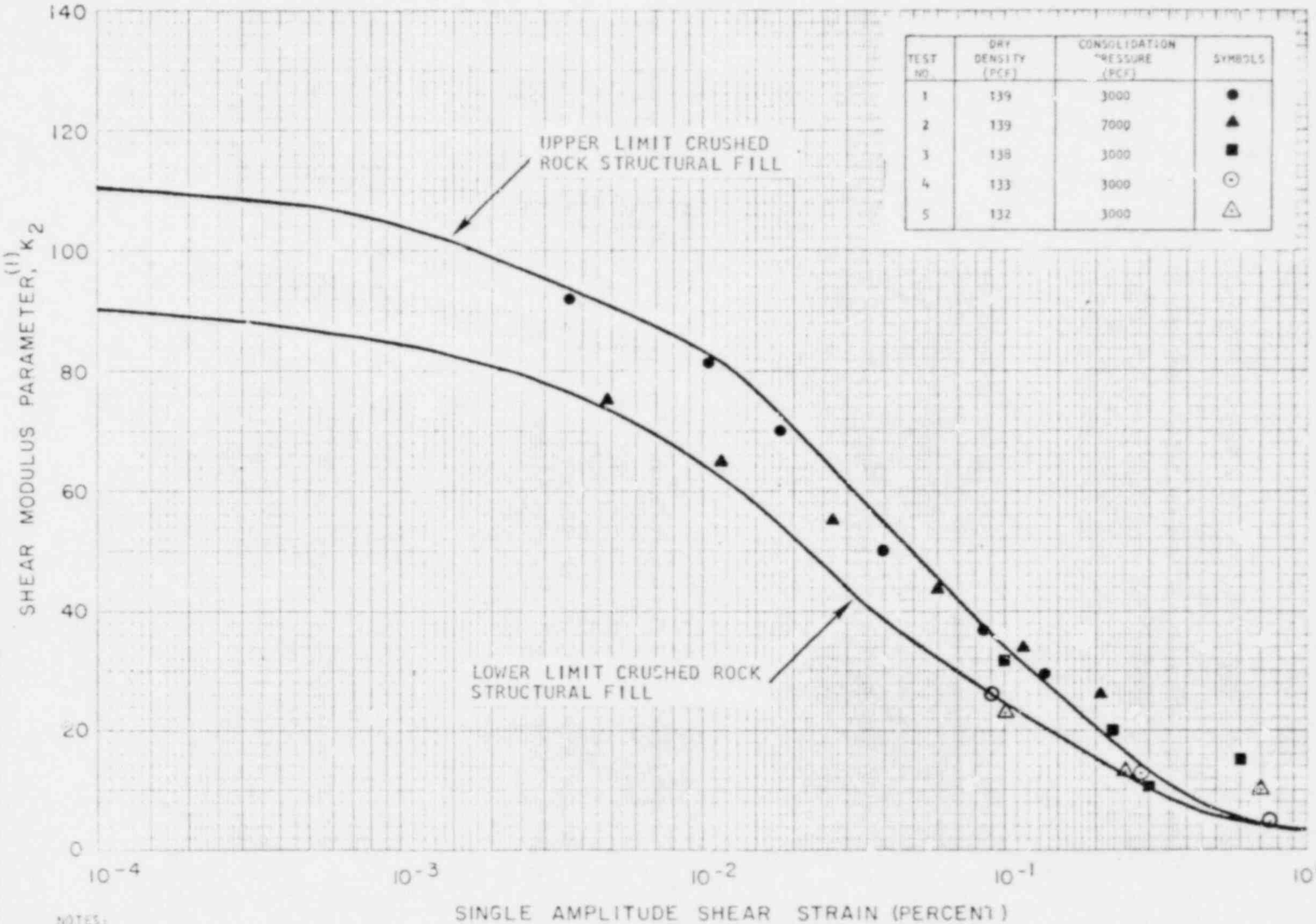
The strain degradation curves for the Upper Heumader shale was selected as that of the upper bound for the residual soils at the site (Figure 2.5-97f). However, the shear modulus for the Lower Heumader shale due to its calcareous nature and higher strength, was considered less strain dependent, and a flatter strain degradation curve was estimated for this material.

The compressional wave velocity in the Heumader shale near the ESWS Pumphouse (Boring HS-14, Figure 2.5-102c) was measured by an uphole compressional wave velocity survey. The average compressional wave velocity obtained was approximately 2625 feet per second for both the Upper and Lower Heumader shales. The Upper Heumader shale at Boring HS-14 is highly weathered and soil-like, and would, with a Poisson's ratio of 0.4 to 0.45, have a shear wave velocity in the range of 1050 to 800 feet per second. However, the Upper Heumader shale at the ESWS Pumphouse (Borings ESWS-28 and ESWS-29) is slightly less weathered than at Boring HS-14. The shear wave velocity for the Upper Heumader shale at the ESWS Pumphouse was, therefore. estimated to be the same as that at the plant site, namely 1000 feet per second. The Lower Heumader shale at the ESWS Pumphouse is also weathered to a lesser degree than at HS-14. The shear wave velocity for the Lower Heumader shale was, therefore, estimated to be 1300 feet per second, giving a shear modulus of 8×10^6 pounds per square foot at a strain of 10^{-4} percent and lower. This shear modulus corresponds to a Poisson's ratio of approximately 0.40 using a compressional wave velocity of 3200 feet per second. The strain degradation curves were taken as those for the Heumader shale at the plant site.

Since similar materials tend to have comparable strain degradation characteristics, the strain degradation curves obtain from dynamic triaxial tests on shale samples taken in Illinois (Maquoketa

R241.1 (continued)

Shale) are shown on Figure 241.1-3 (Carroll County Station Site Suitability, Site Safety Report Docket Nos. S50-599 and S50-600). Also shown are the strain degradation curves for the Heumader Shale from Figure 2.5-97b. The Maquoketa Shale contains the same type clay mineral constituents as the Heumader Shale, and, in addition, the fractional clay contents are within 10 to 20 percent of those of the Heumader Shale. The Atterburg Limits for both shales are between 30 and 40 percent for the Liquid Limit and between 15 and 20 percent for the Plastic Limit. No measurable swelling clay minerals were detected in either shale, and both shales exhibit similar strength properties. Figure 241.1-3 presents the mean strain degradation curve for 13 tests for the Maquoketa Shale and the test results for a sample from the Maquoketa Shale with the highest unconfined compressive strength (460 pounds per square inch as compared to 300 pounds per square inch obtained for the Lower Heumader Shale). As shown, the strain degradation characteristics for the Maquoketa Shale are quite similar to those estimated for the Heumader Shale. Therefore, the estimated strain degradation curves for the Heumader Shale are considered representative.



TEST NO.	DRY DENSITY (PCF)	CONSOLIDATION PRESSURE (PCF)	SYMBOLS
1	139	3000	●
2	139	7000	▲
3	138	3000	■
4	133	3000	⊙
5	132	3000	△

NOTES:

1. $G = 1000 K_2 (\sigma_m')^{1/2}$,
WHERE G IN PSF IS THE SHEAR MODULUS, K_2 IS A CONSTANT, AND σ_m IS THE MEAN EFFECTIVE STRESS, ALSO IN PSF.

2. GRADATIONS PER DMLK-419, JULY 6, 1977.

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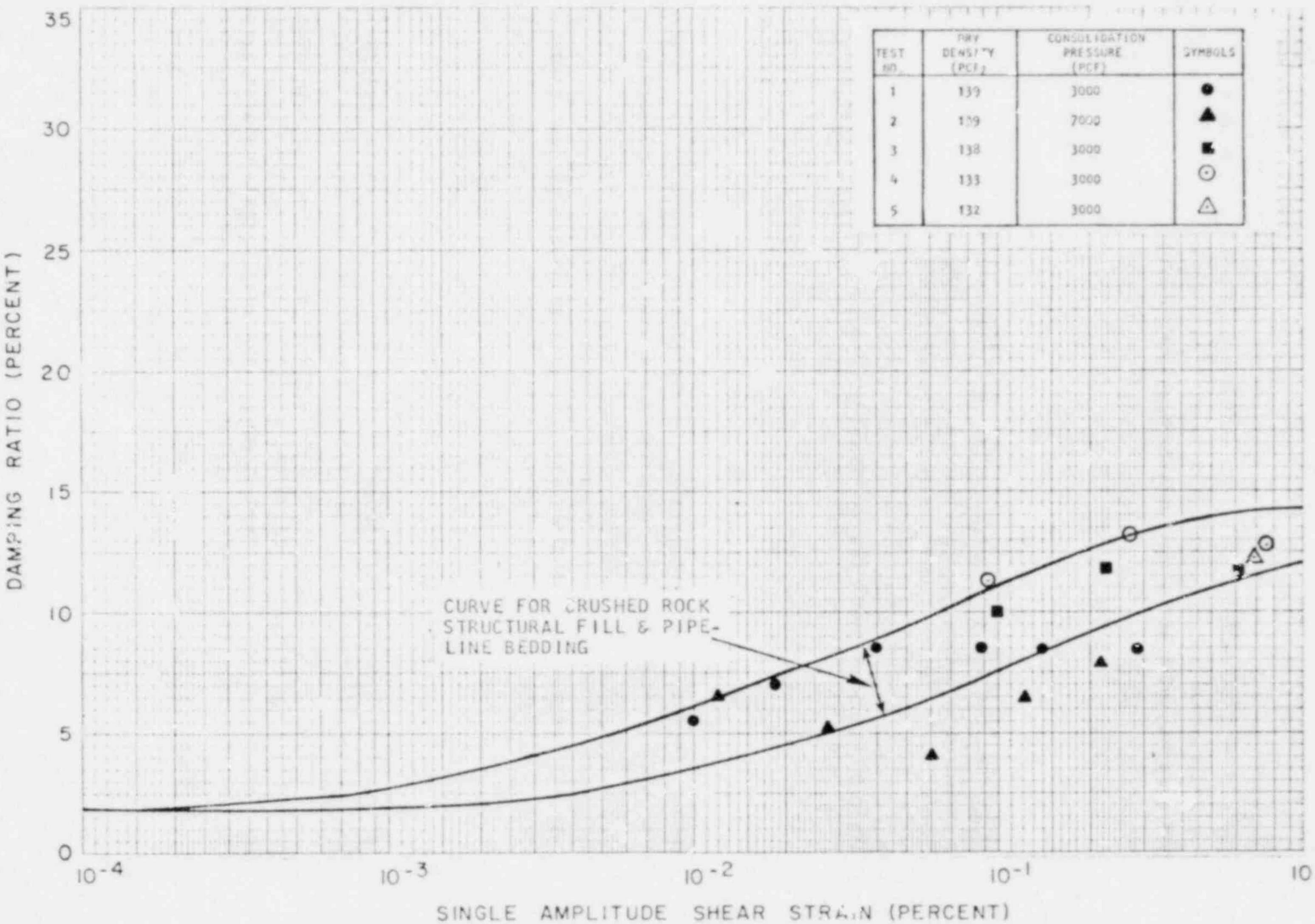
FIGURE 241.1-1
RECOMMENDED SHEAR MODULUS VERSUS SHEAR STRAIN CURVE FOR CRUSHED ROCK STRUCTURAL FILL



NOTES:

1. GRADATION PER DMLK-419, JULY 6, 1977 AND SARGENT & LUNDY ENGINEERS SPEC. NO. A-3852 SECTION 301.SC.

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FIGURE 241.1-2
RECOMMENDED DAMPING RATIO VERSUS SHEAR STRAIN CURVE FOR CRUSHED ROCK BACKFILL

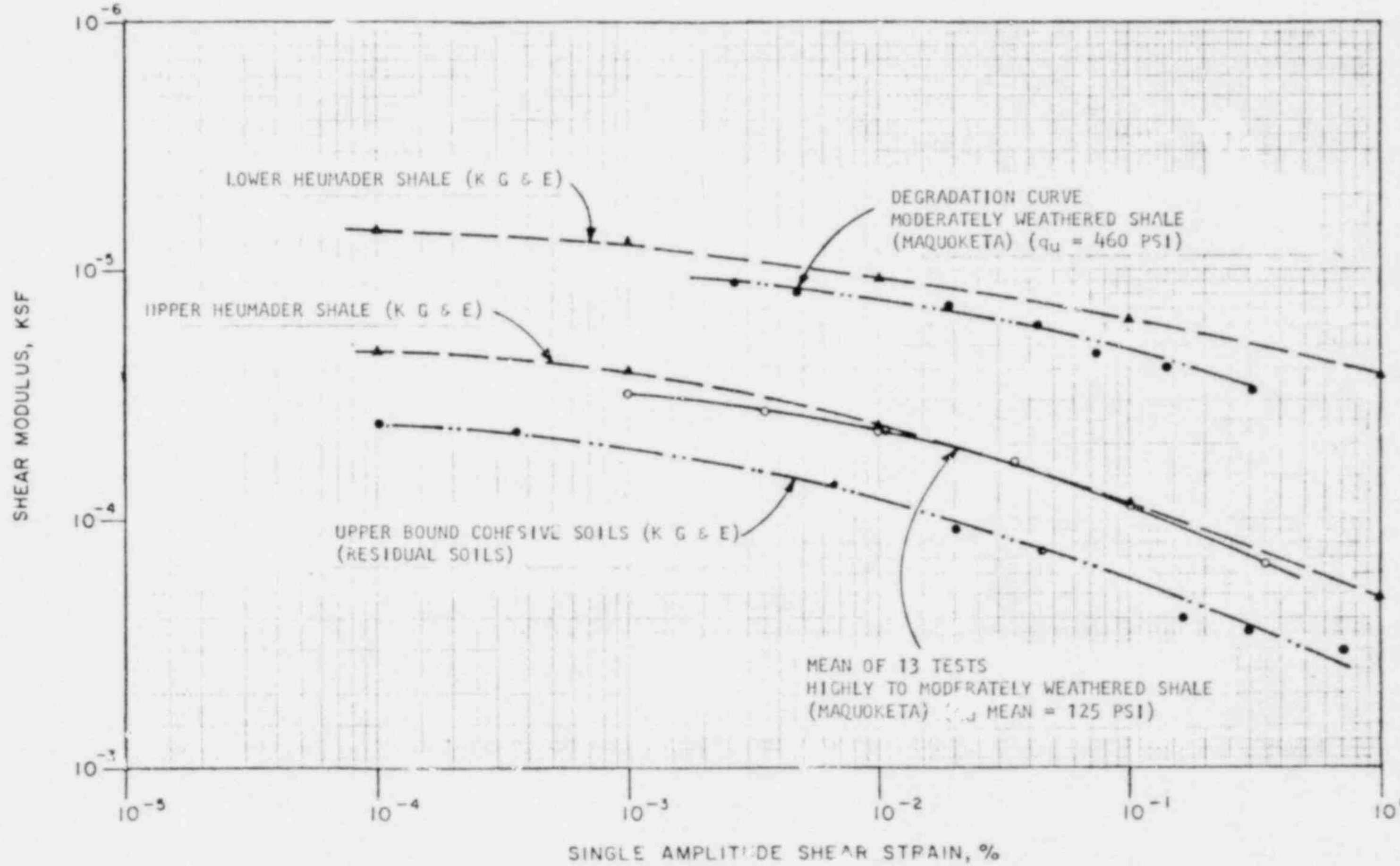

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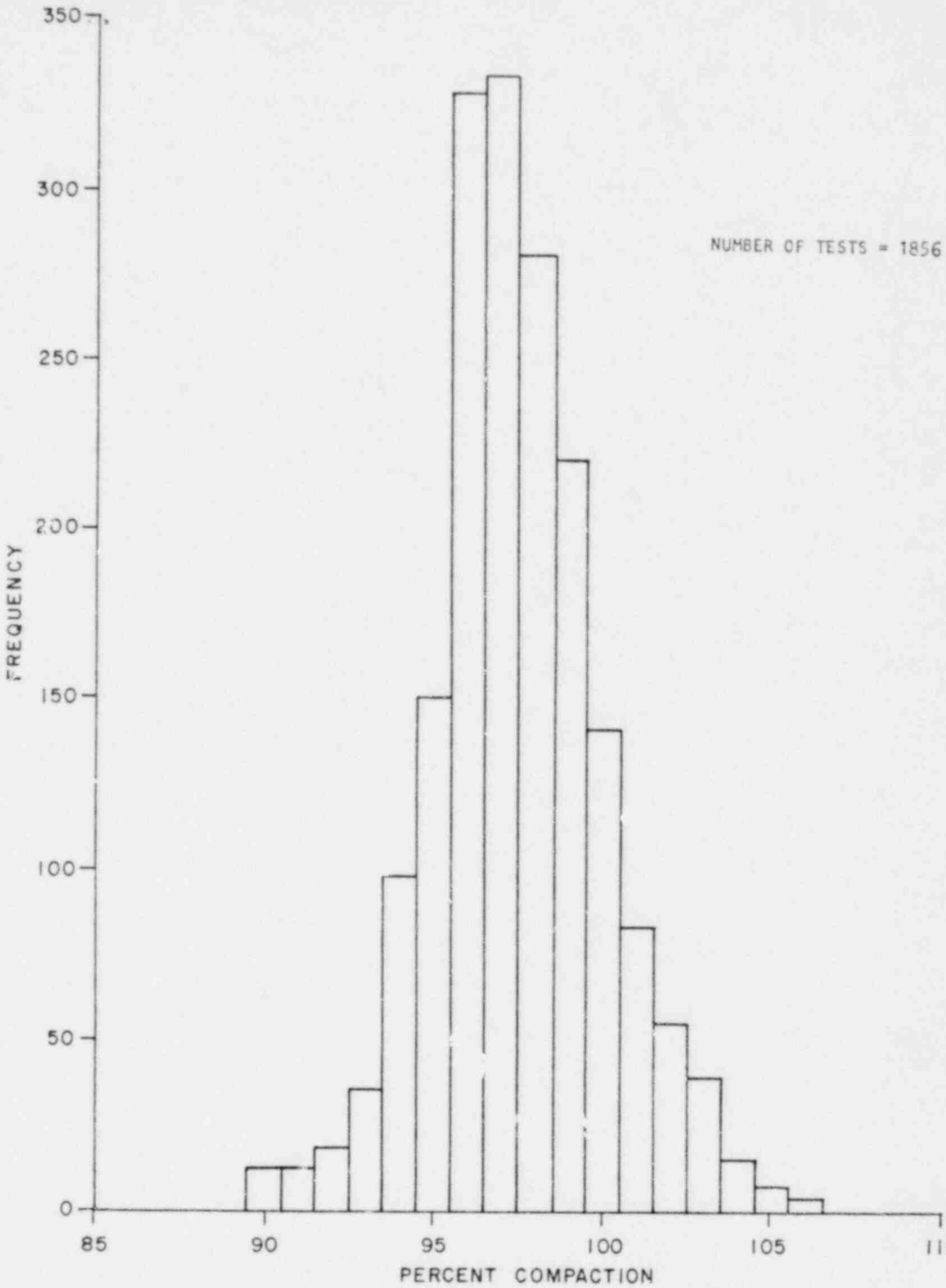
FIGURE 241.1-3
STRAIN DEGRADATION CURVES FOR
HEUMADER SHALE AND MAQUOKETA SHALE

Q241.2 Provide a summary of the results of field density and moisture content tests used for quality control during construction of structural fill under and backfill around the Category I structures. Present the results as a statistical distribution plot or by other convenient method(s) to be able to verify that the specified compaction has been attained. Provide the above data for each type of fill separately for the Power Block Unit, the ESWS pumphouse, the ESWS discharge structure and the seismic Category I pipelines and electrical duct banks.

R241.2 The information is provided on Figures 241.2-1 through 241.2-19.



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

2. MIMIMUM COMPACTION IS 95% ASTM D-1556.

7699-064-07

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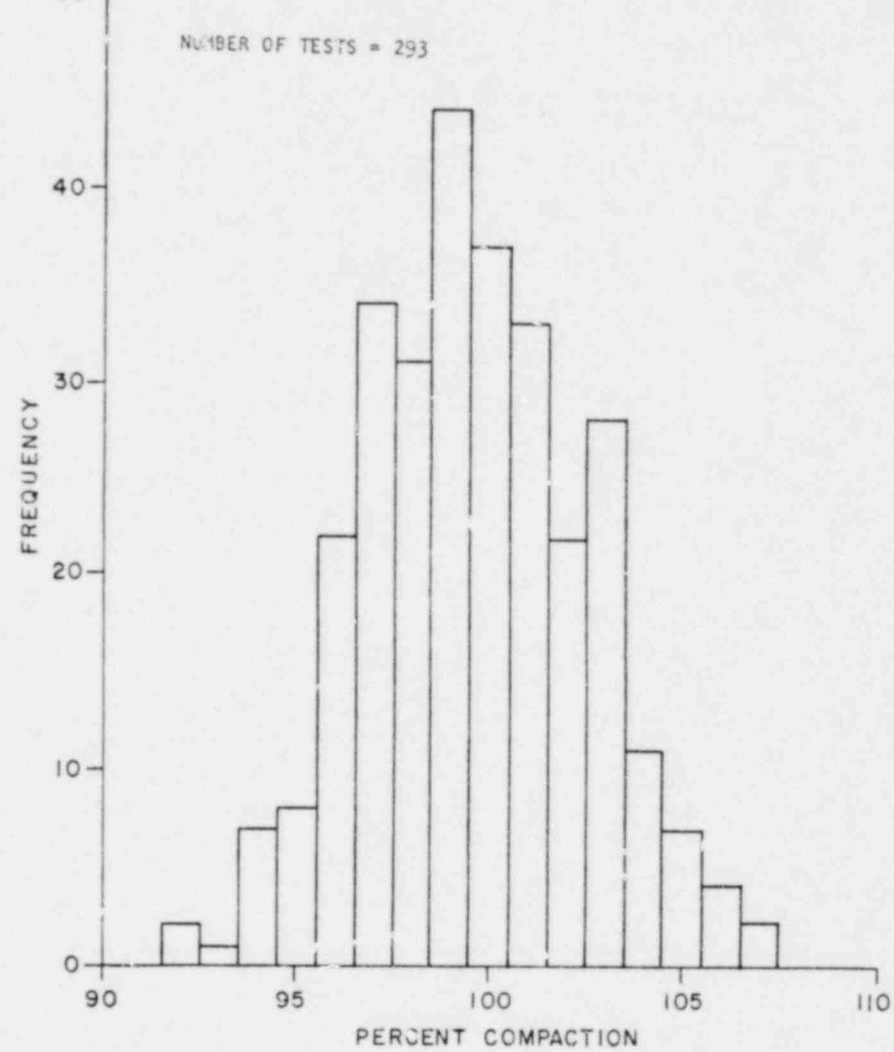
FIGURE 241.2-1

POWER BLOCK - STRUCTURAL FILL STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

2. MINIMUM COMPACTION IS 95% ASTM D-698.

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FIGURE 241.2-2
POWER BLOCK - COHESIVE FILL
STATISTICAL DISTRIBUTION PLOT

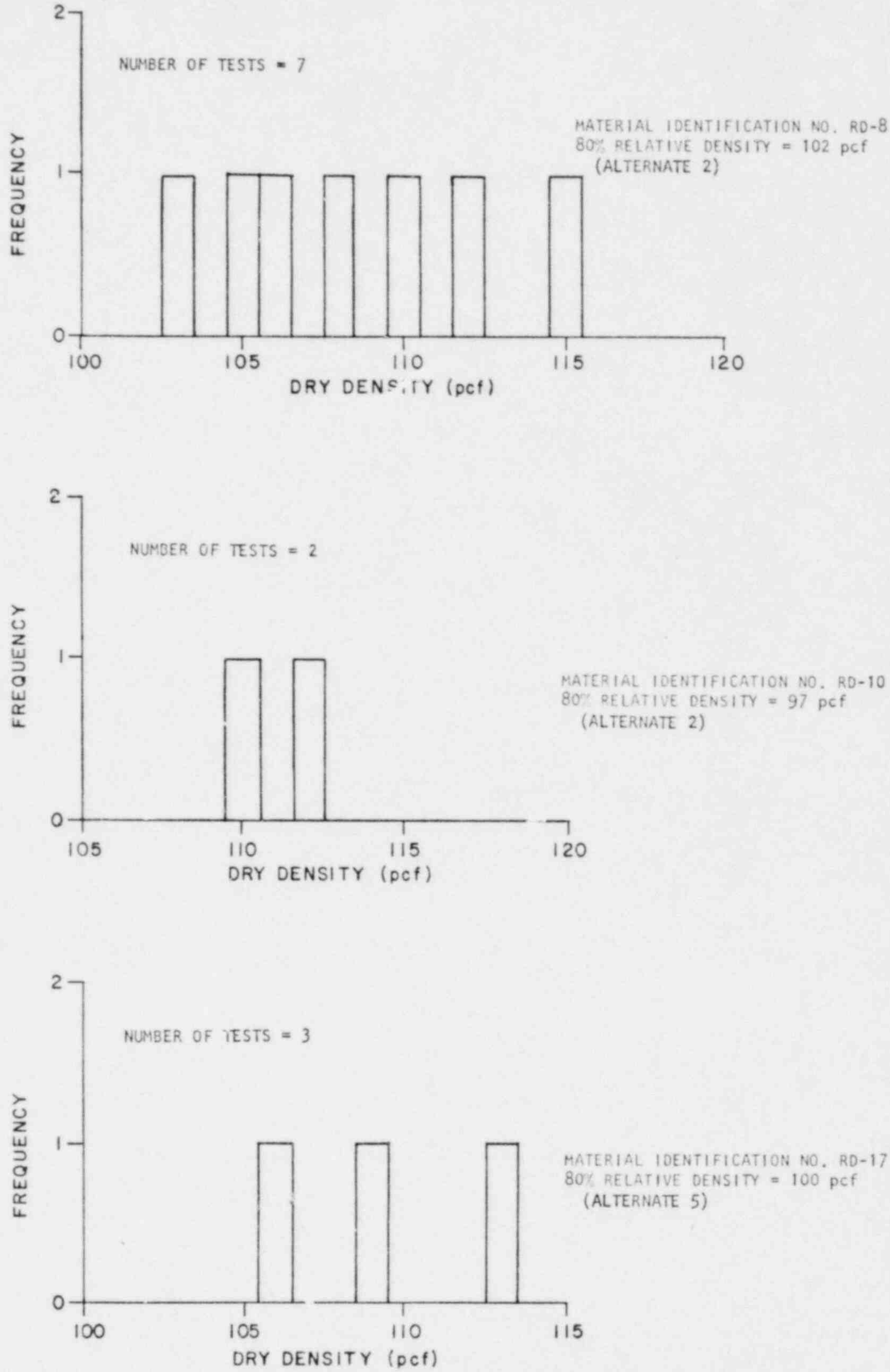

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FIGURE 241.2-3
POWER BLOCK - PIPE BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT

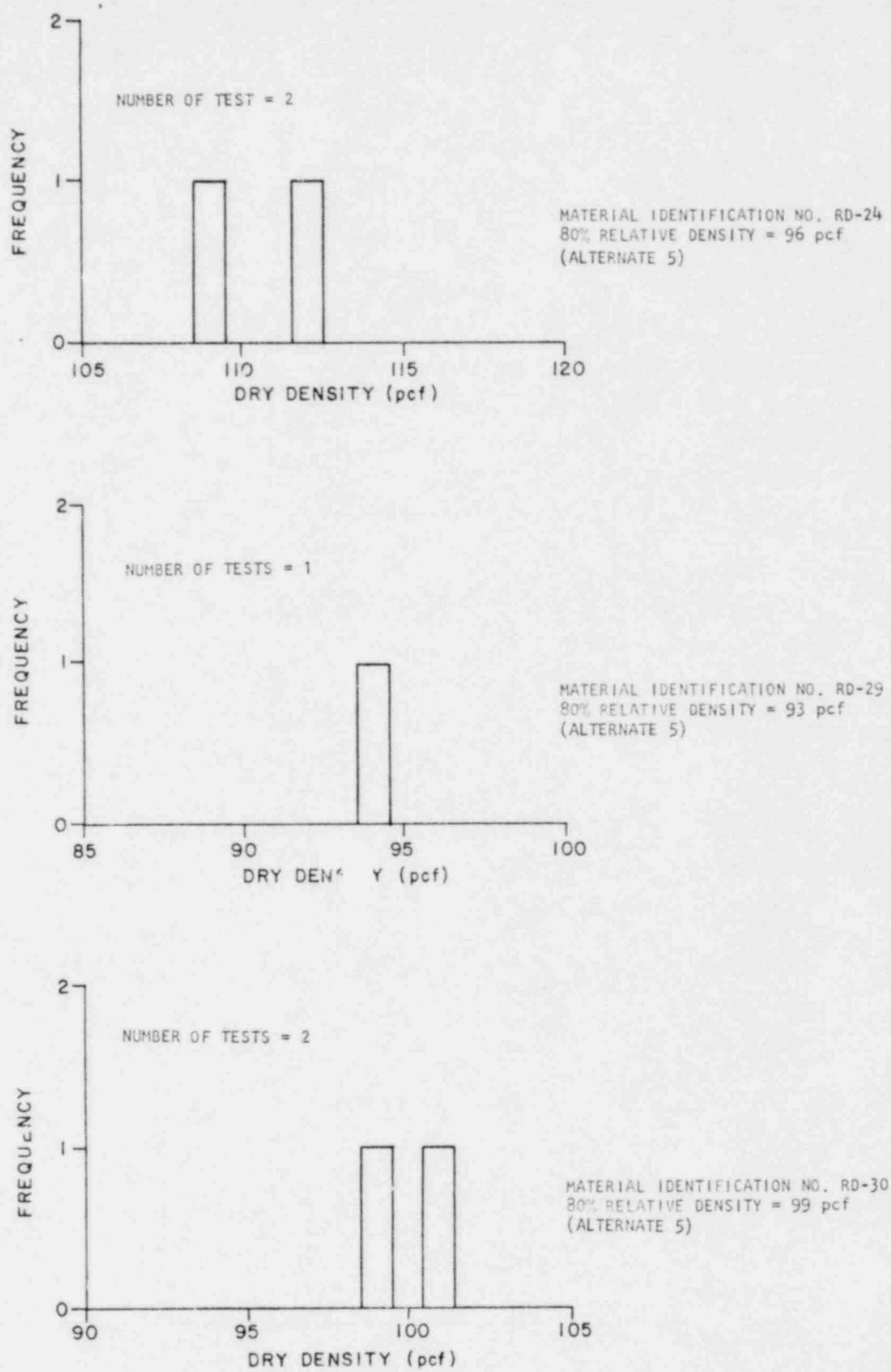

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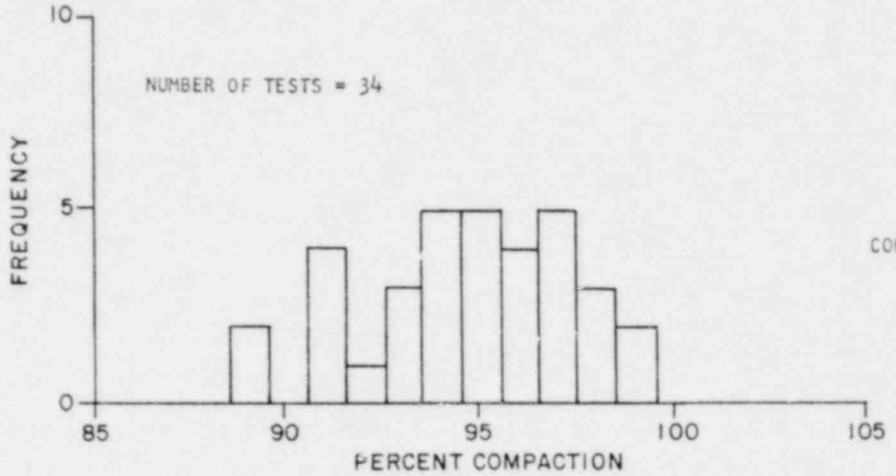
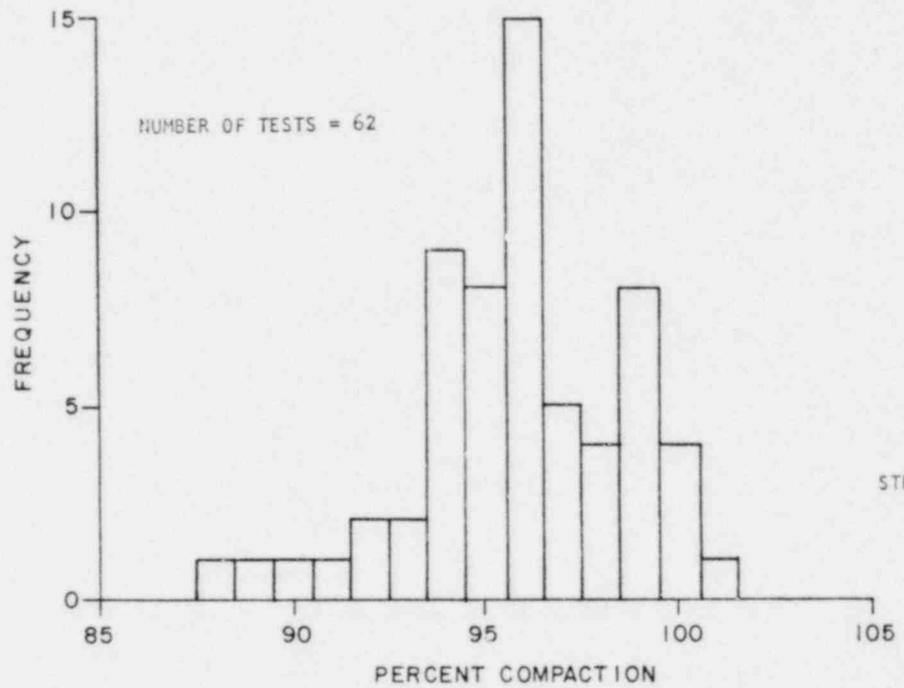
FIGURE 241.2-4
POWER BLOCK - PIPE BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT



COHESIVE FILL



STRUCTURAL FILL

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THESE PLOTS.

2. MINIMUM COMPACTION IS 95% ASTM D-698 FOR COHESIVE FILL, 95% ASTM D-1557 FOR STRUCTURAL FILL.

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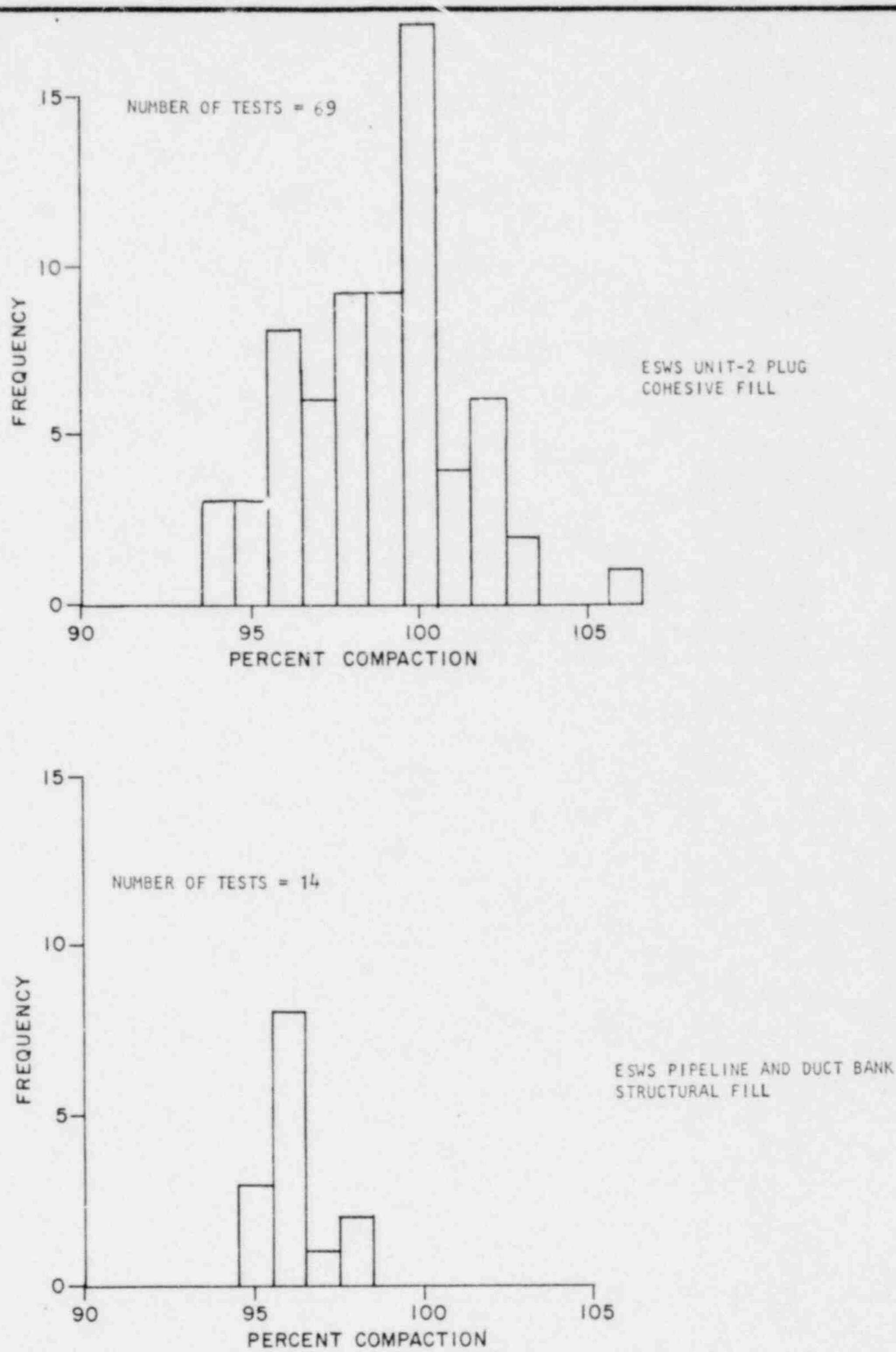
FIGURE 241.2-5
ESWS STRUCTURES
STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THESE PLOTS.
2. MINIMUM COMPACTION IS 95% ASTM D-698 FOR COHESIVE FILL, 95% ASTM D-1557 FOR STRUCTURAL FILL.

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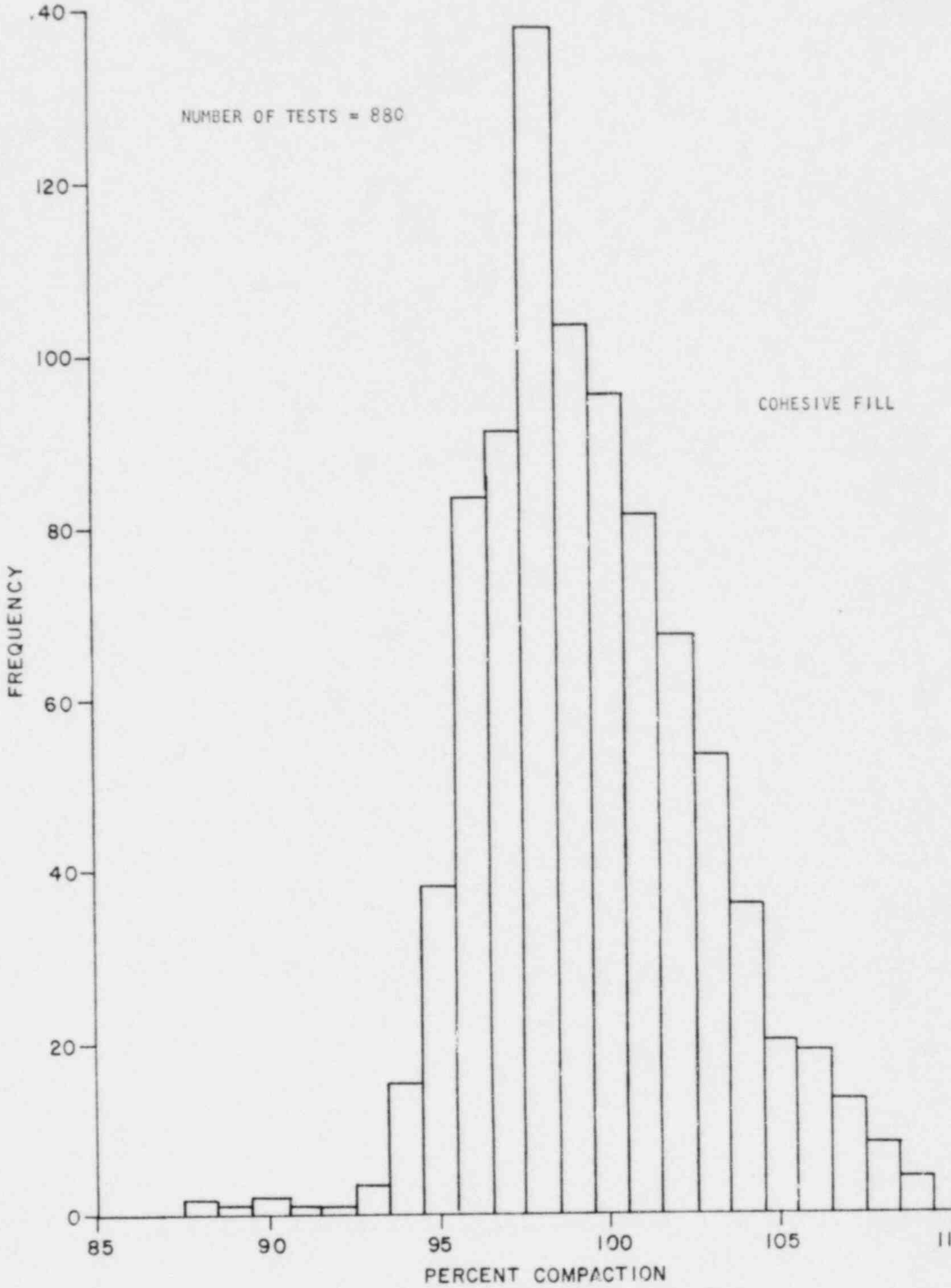
FIGURE 241.2-6
ESWS
STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.
2. MINIMUM COMPACTION IS ASTM D-1557.

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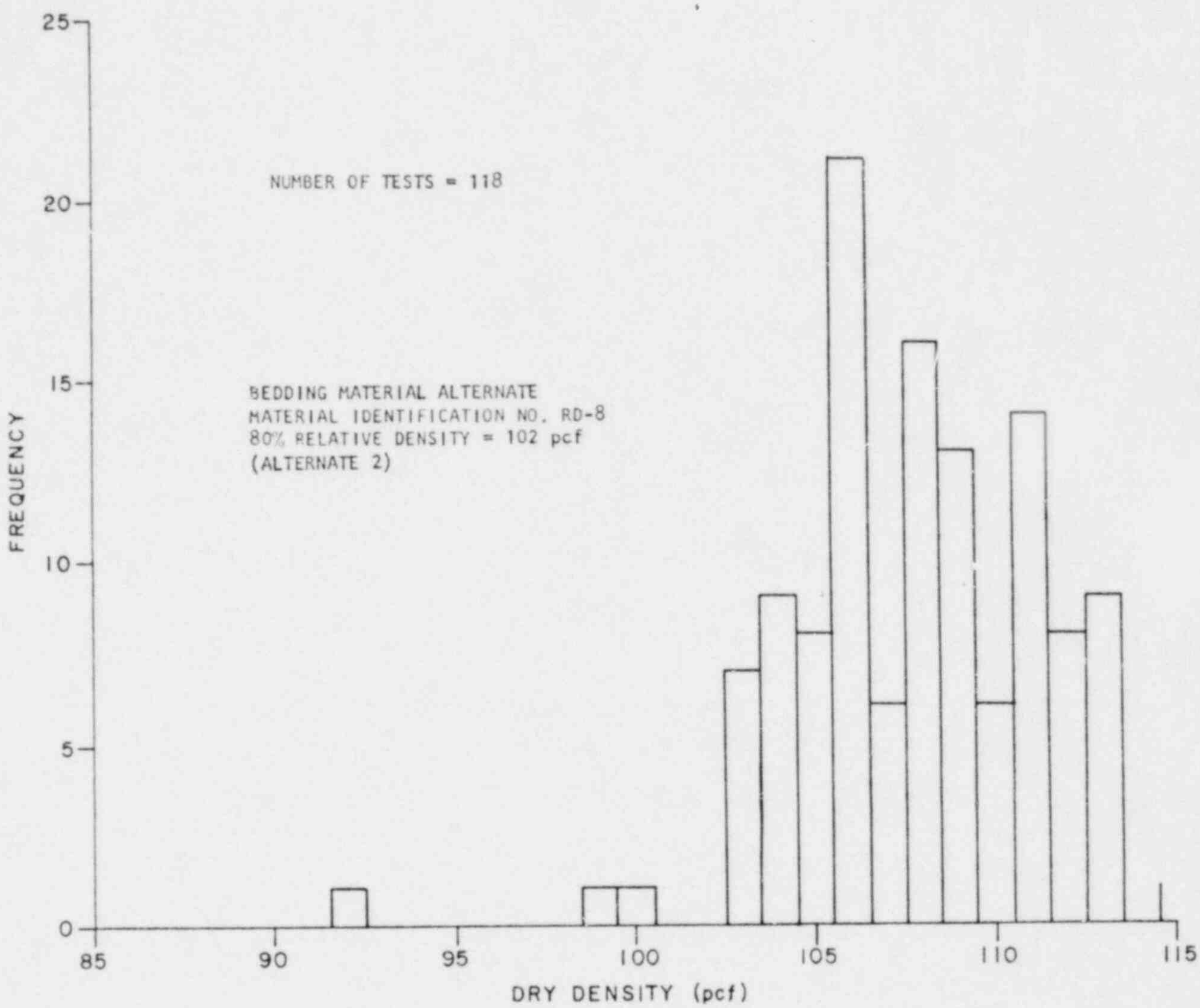
FIGURE 241.2-7

ESWS PIPELINE AND DUCT BANK STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

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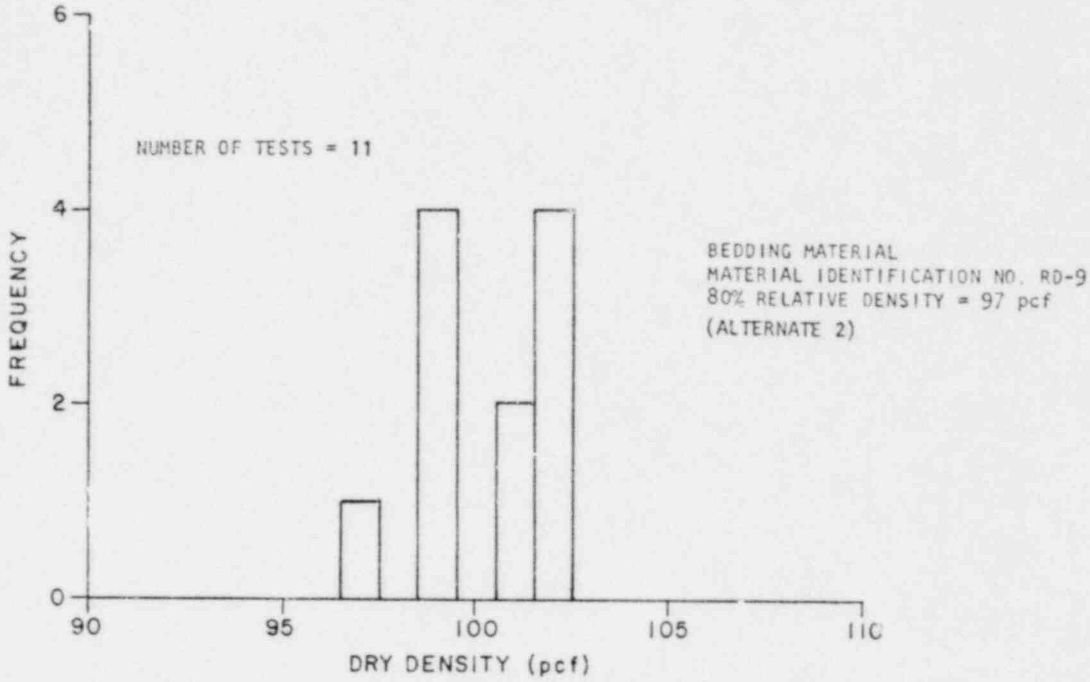
FIGURE 241.2-8
ESWS PIPELINE AND DUCT BANK STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

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FIGURE 241.2-9

ESWS PIPELINE AND DUCT BANK STATISTICAL DISTRIBUTION PLOT

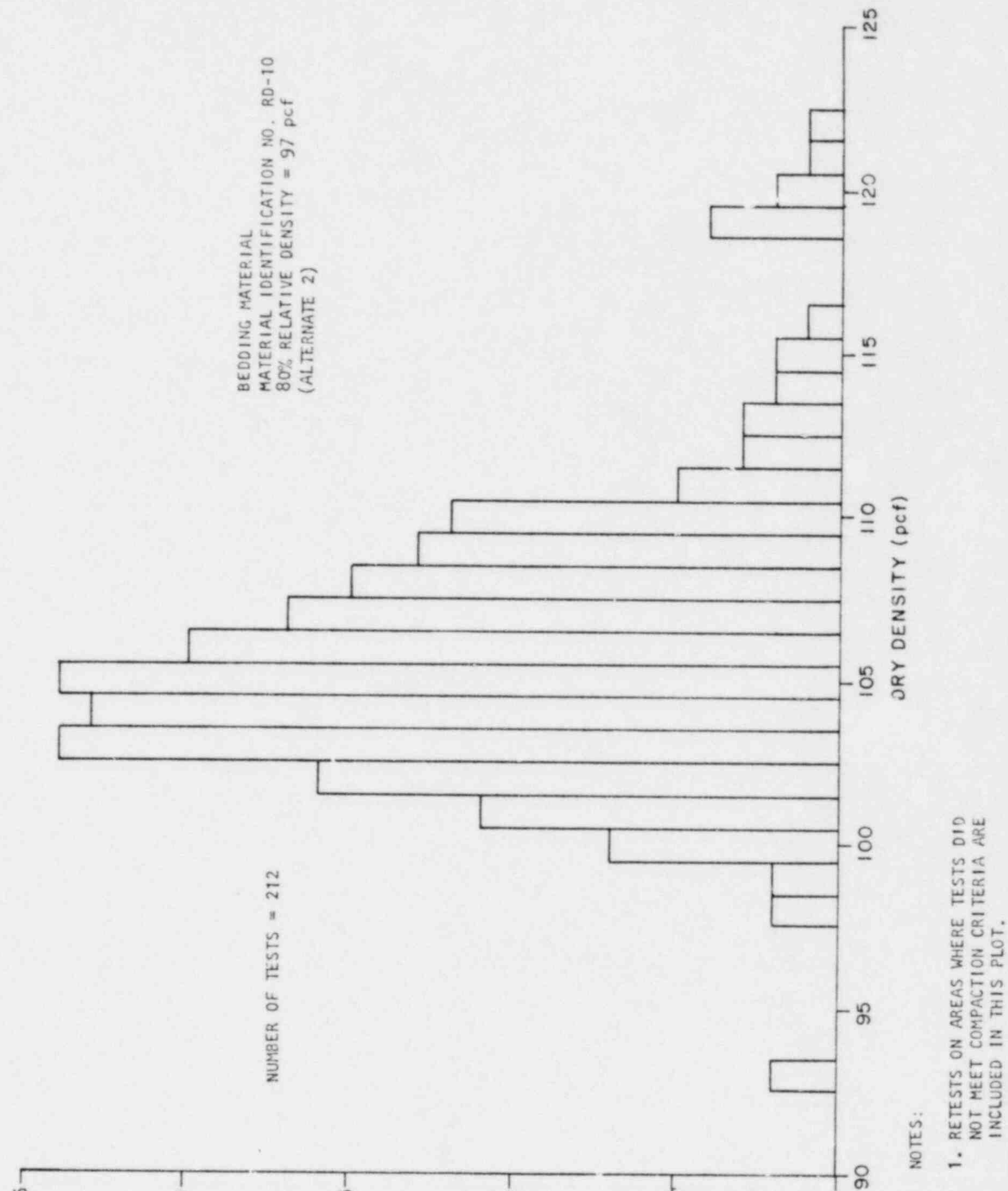

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FIGURE 241.2-10

ESWS PIPELINE AND DUCT BANK STATISTICAL DISTRIBUTION PLOT

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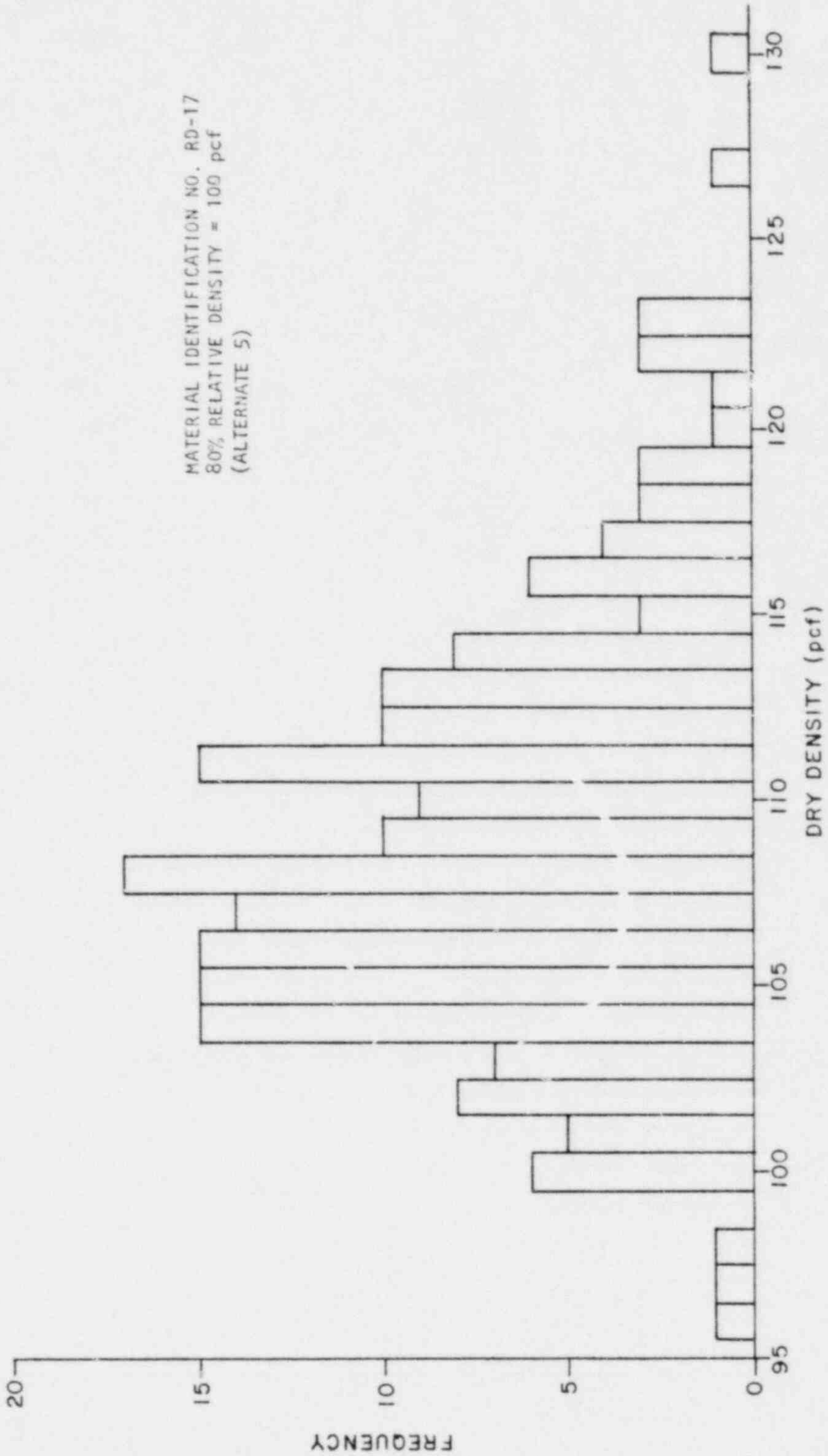

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

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FIGURE 241.2-11
ESWS PIPELINE BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT

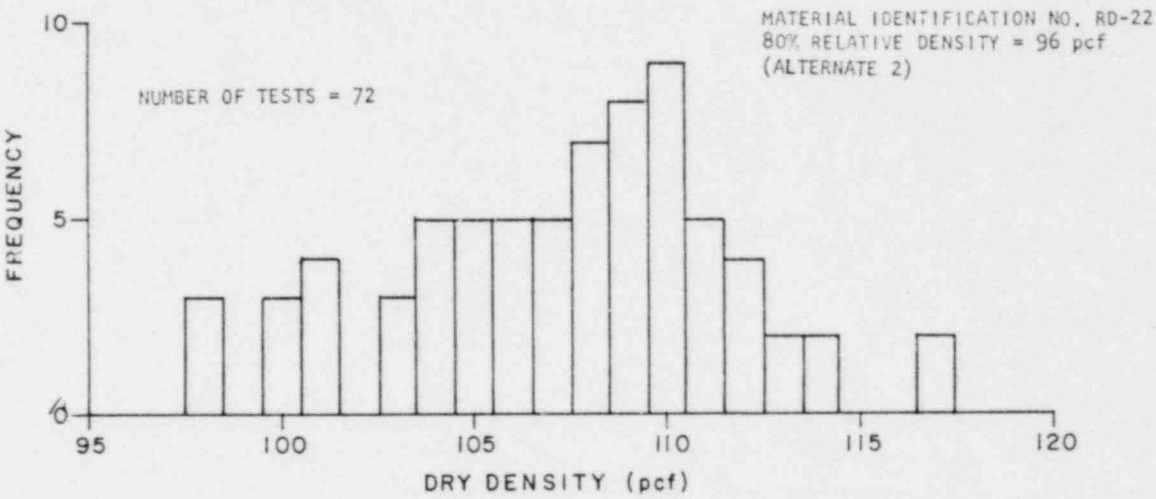

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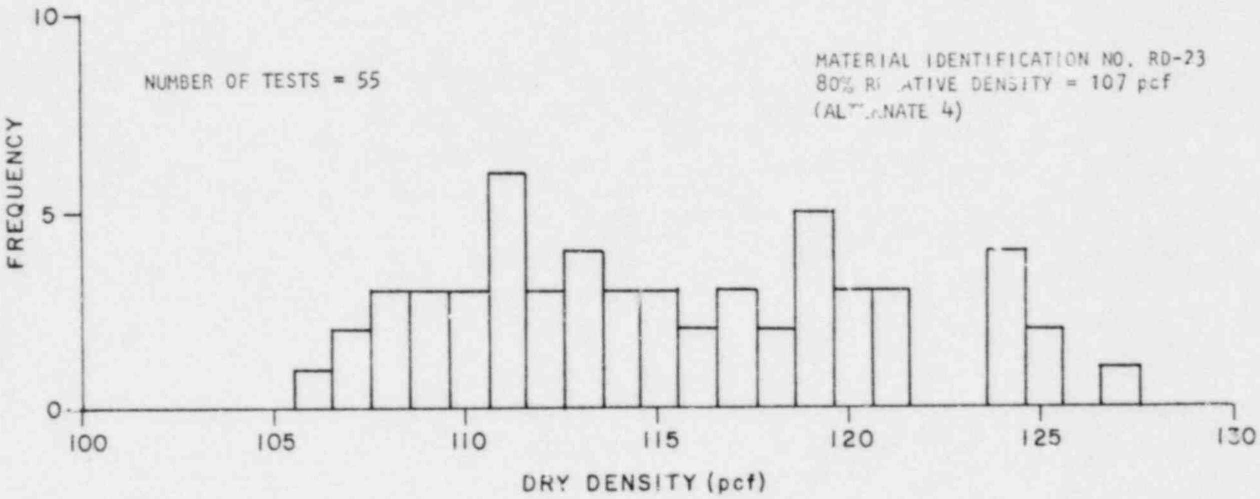
FIGURE 241.2-12

ESWS PIPELINE AND DUCT BANK
BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

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FIGURE 241.2-13

ESWS PIPELINE AND DUCT BANK
BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT

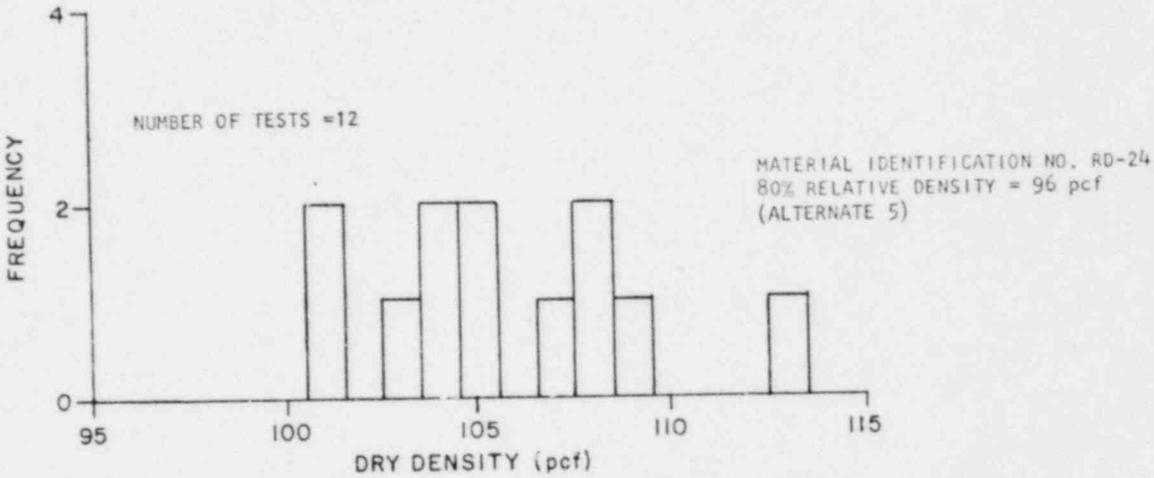

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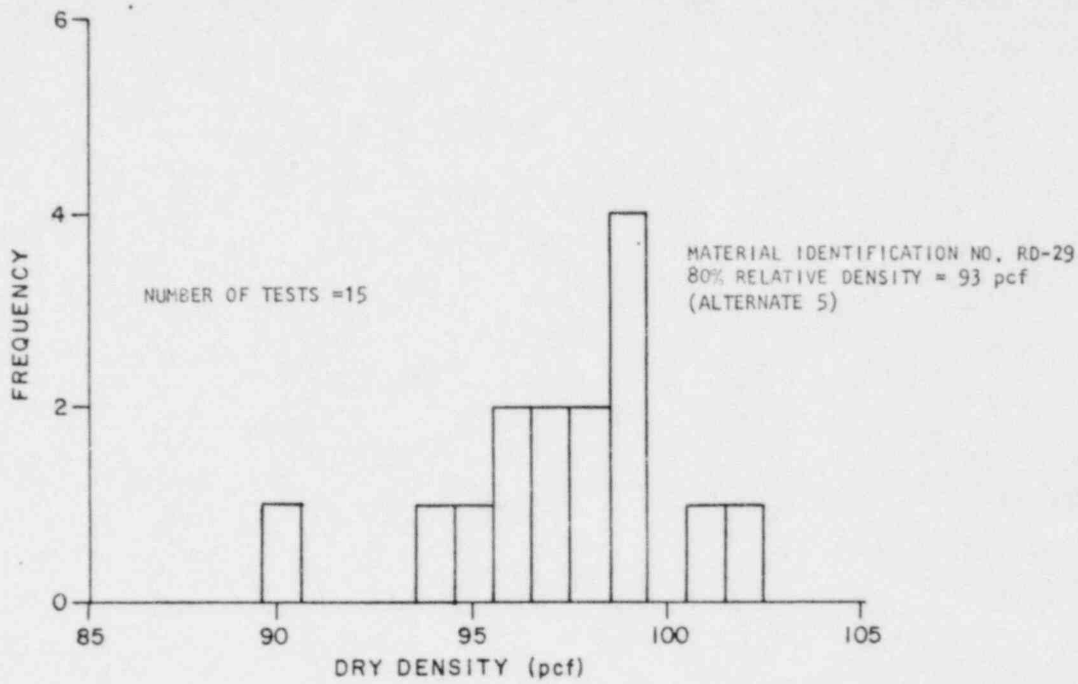
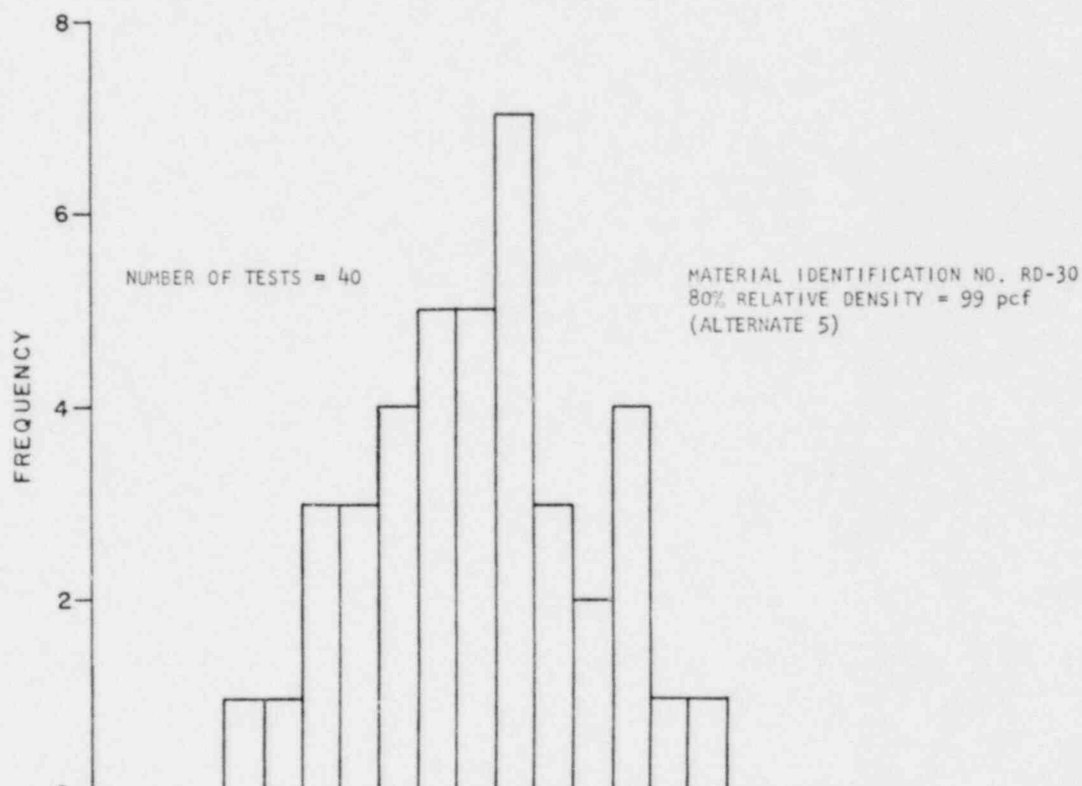
FIGURE 241.2-14

ESWS PIPELINE AND DUCT BANK
BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT





7699-064-07

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THESE PLOTS.

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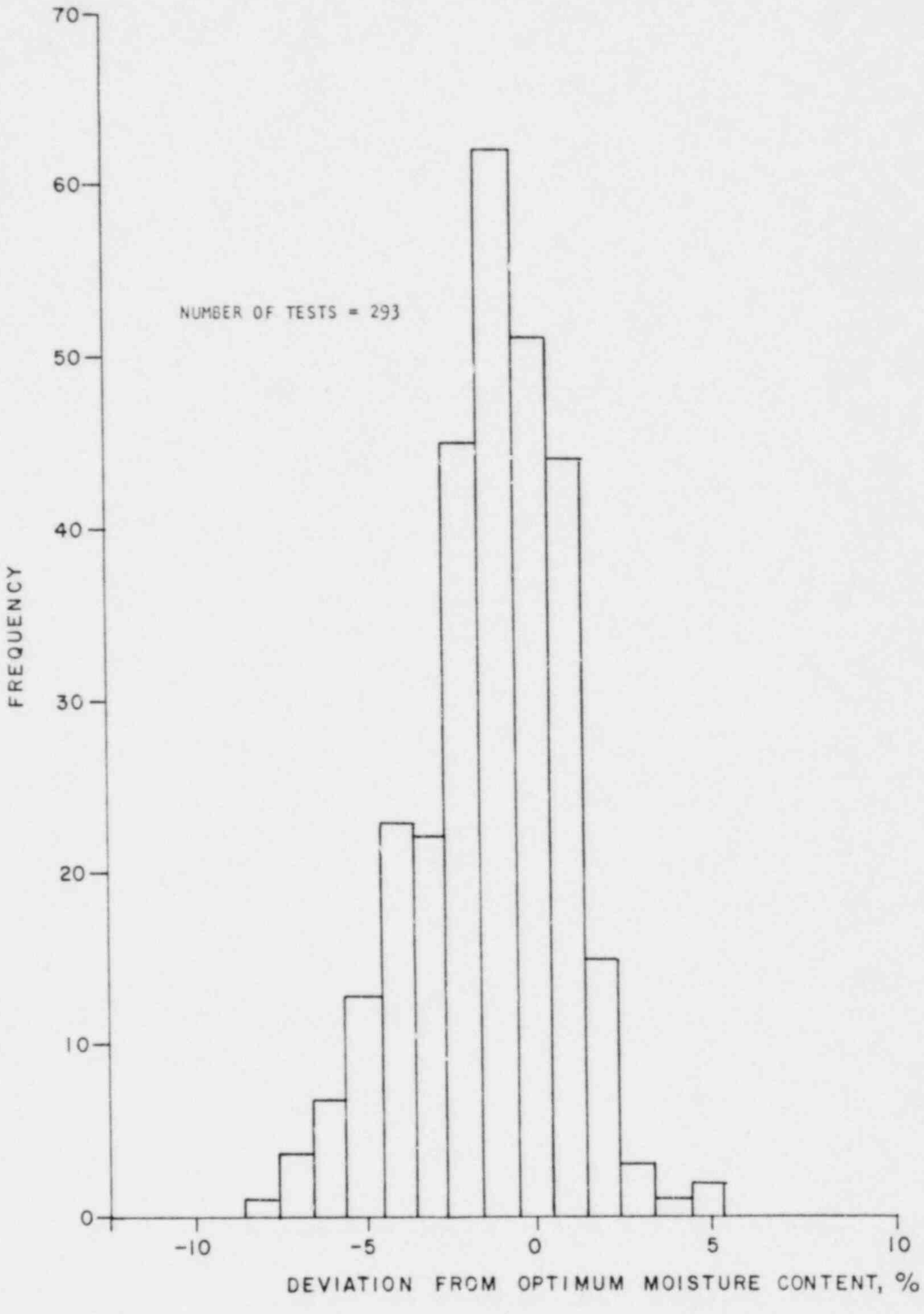
FIGURE 241.2-15

ESWS PIPELINE AND DUCT BANK
BEDDING MATERIAL
STATISTICAL DISTRIBUTION PLOT



7699-064-07

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

2. ACCEPTANCE CRITERIA IS ± 2% OF OPTIMUM MOISTURE CONTENT, HOWEVER FAILING TESTS ON THE DRY SIDE WERE GENERALLY ACCEPTED IF THE DENSITY REQUIREMENTS WERE MET.

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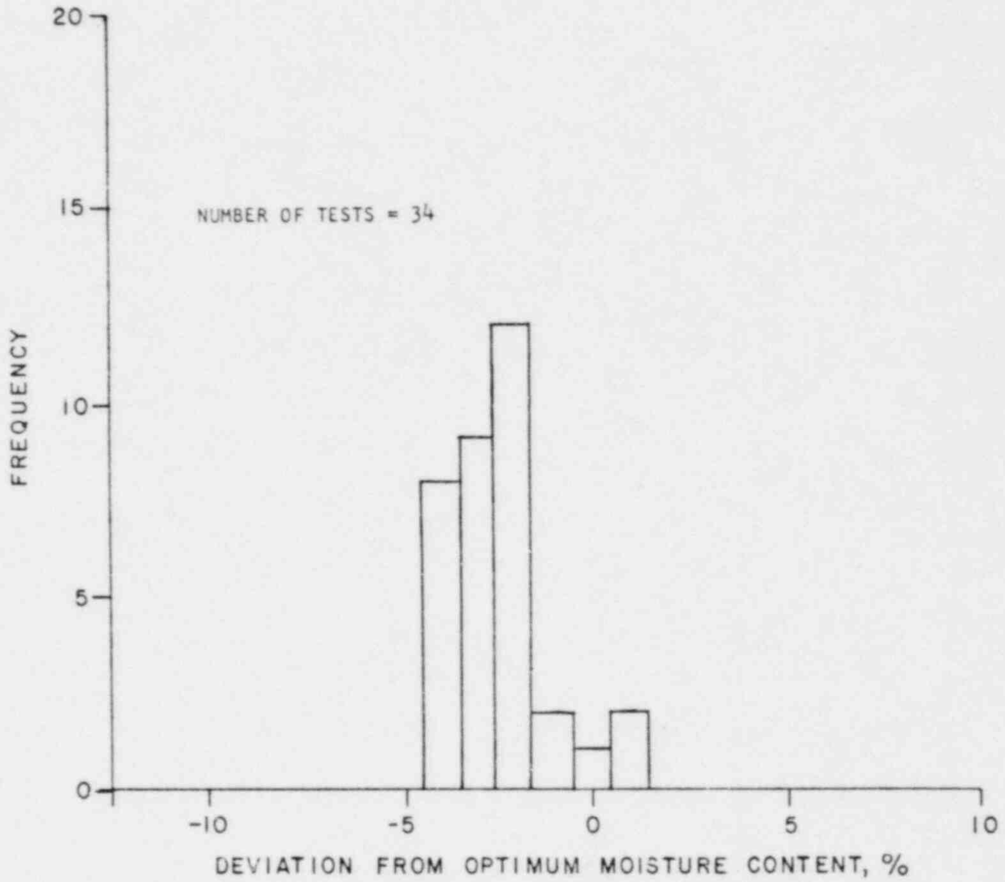
FIGURE 241.2-16
POWERBLOCK COHESIVE FILL
STATISTICAL DISTRIBUTION PLOT



7699-064-07

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

2. ACCEPTANCE CRITERIA IS ± 2% OF OPTIMUM MOISTURE CONTENT, HOWEVER FAILING TESTS ON THE DRY SIDE WERE GENERALLY ACCEPTED IF THE DENSITY REQUIREMENTS WERE MET.

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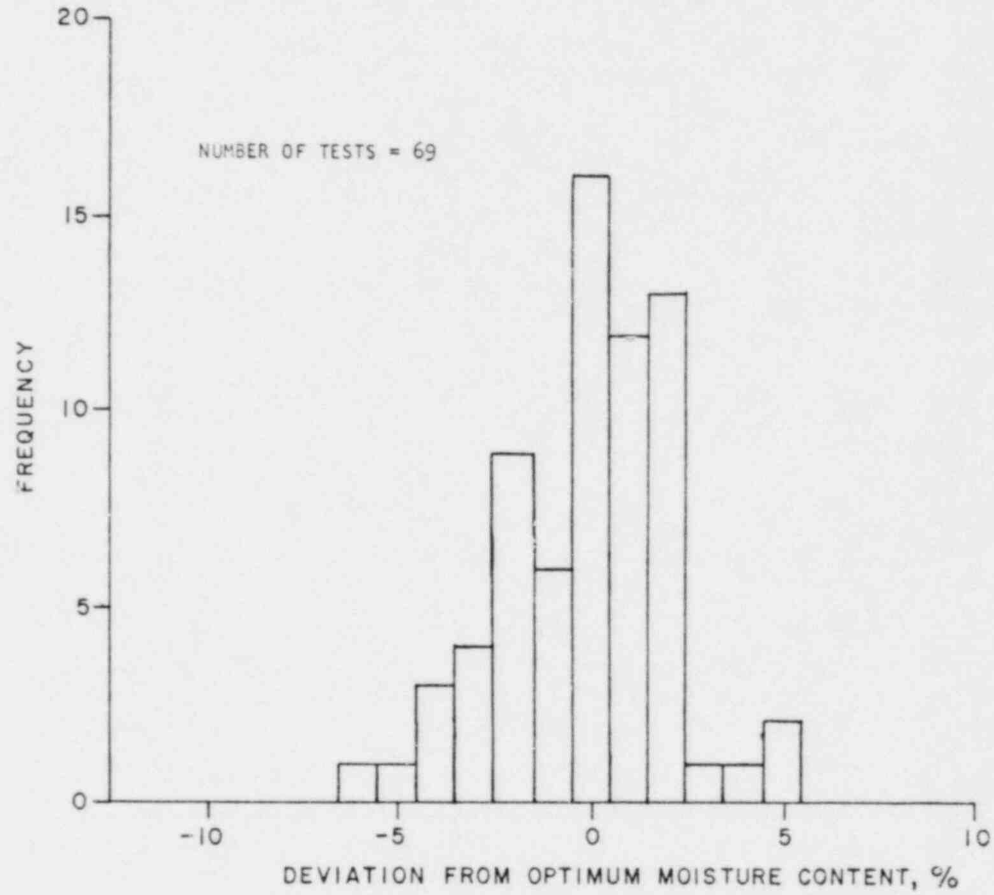
FIGURE 241.2-17
ESWS STRUCTURES COHESIVE FILL
STATISTICAL DISTRIBUTION PLOT



7699-064-07

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.
2. ACCEPTANCE CRITERIA IS ± 2% OF OPTIMUM MOISTURE CONTENT, HOWEVER FAILING TESTS ON THE DRY SIDE WERE GENERALLY ACCEPTED IF THE DENSITY REQUIREMENTS WERE MET.

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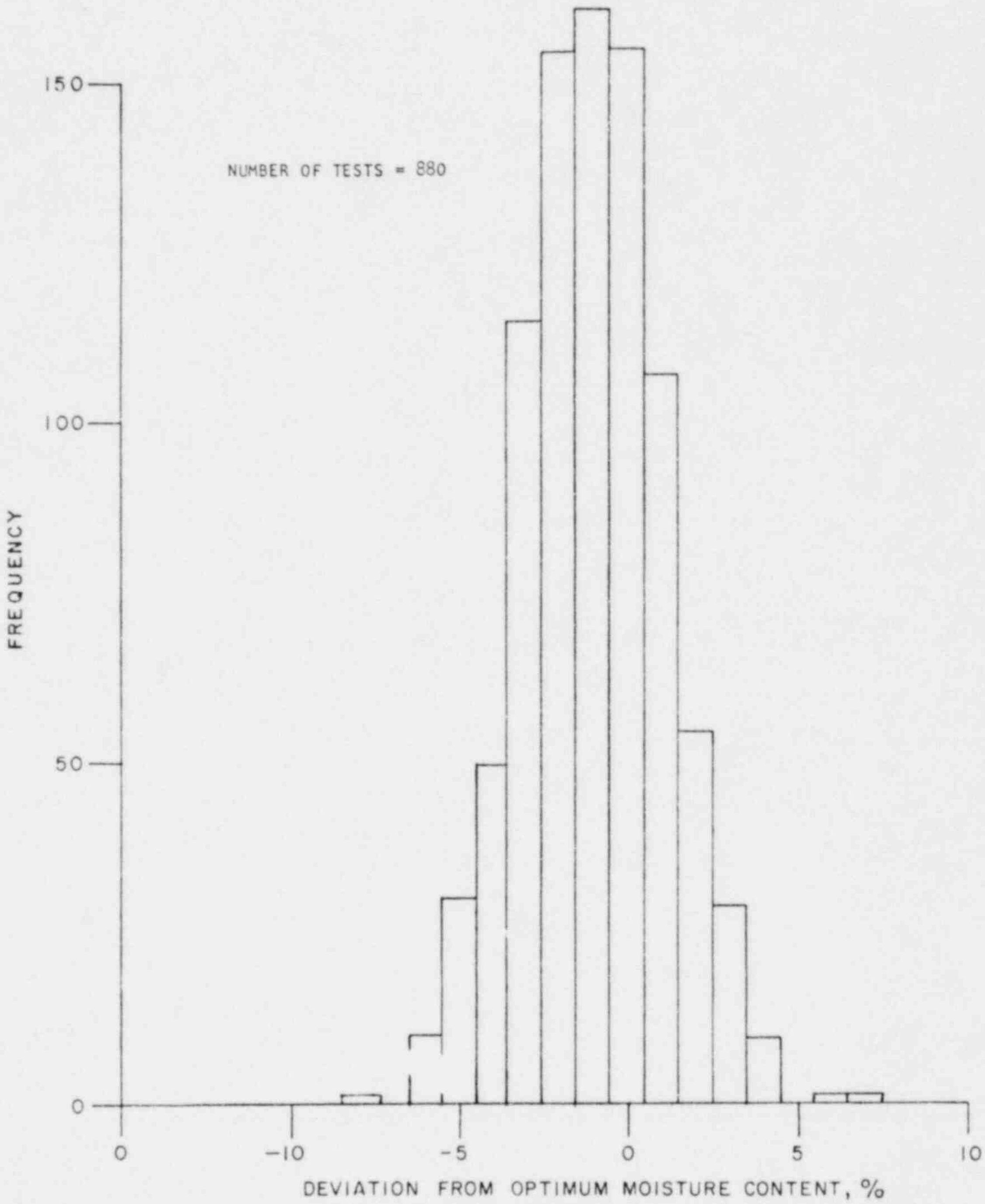
FIGURE 241.2-18
ESWS UNIT 2 PLUG COHESIVE FILL
STATISTICAL DISTRIBUTION PLOT



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.
2. ACCEPTANCE CRITERIA IS ± 2% OF OPTIMUM MOISTURE CONTENT, HOWEVER FAILING TESTS ON THE DRY SIDE WERE GENERALLY ACCEPTED IF THE DENSITY REQUIREMENTS WERE MET.

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FIGURE 241.2-19
ESWS PIPELINE COHESIVE BACKFILL
STATISTICAL DISTRIBUTION PLOT

Q241.3 Provide details of the six different types of backfill and the bedding materials used in the construction of ECCS seismic Category I piping and electrical duct banks including gradation and plasticity index requirements, and principal construction criteria.

R241.3 The information is provided in Sargent & Lundy's Specifications A-3852, (Section 301.5). These specifications are reproduced here as Attachment 241.3-1 (sheets 1 through 4).

301.5 Bedding for Circulating Water Pipeline, Warming Water Pipeline, Service Water Pipeline, ESWS Pipelines and ESWS Electrical Duct Banks: Amd.

a. The bedding shall be shaped to fit the underside of the pipe to provide a continuous firm bearing.

a1. There shall be a minimum of 6 inches of bedding below the pipe inverts where bottom of the trench is soil and a minimum of 12 inches of bedding where the bottom of the trench is rock. Amd.4

a2. The bedding shall extend to at least the mid height of the pipe for pipelines and to a minimum of 12 inches above the crown elevation of ESWS pipelines. A minimum of 12 inches of bedding material shall be placed along the sides of the pipes and the ESWS ductbanks that are not poured against in-situ materials. Where the ESWS ductbanks can be placed against in-situ material, bedding material is not required. Amd.4

b. When placing backfill the differential level from one side to the other side of the pipe or ductbank shall not exceed one foot. Amd.3

c. Bedding Material: Amd.4

c1. ESWS Pipeline, ESWS Electrical Duct Banks, Circulating Water Pipelines, Warming Water Pipeline and Service Water Pipeline: Amd.4

c1.1 Bedding material shall be a pea gravel or crushed stone with not less than 95% passing 1/2 inch and not less than 95% to be retained on the No. 4 sieve. The bedding material shall have less than 5 percent friable materials as determined by ASTM C-142 and less than 45 percent loss as determined by ASTM C-131.

c2. As an alternate to Paragraph c1 the following gradation may be used. Amd.5

c2.1 Bedding material shall conform to the applicable requirements of Paragraph 301.5. Bedding material shall have less than 5 percent friable materials as determined by ASTM C-142 and less than 45 percent loss as determined by ASTM C-131, except gradation shall be one of the following: Amd.4

(1) ALTERNATE NO. 1 Amd.4

Sieve Size	Passing %
3/4"	95-100
3/8"	40-60
#8	0-05

(2) ALTERNATE NO. 2

Sieve Size	Passing %
1/2"	95-100
#4	0-20
#8	0-08

(3) ALTERNATE NO. 3 Amd.4

Sieve Size	Passing %
3/4"	100
3/8"	85-100
#8	40-60
#30	5-30
#100	0-02

(4) ALTERNATE NO. 4 Amd.4

Sieve Size	Passing %
3/8"	100
#4	95-100
#8	50-85
#16	25-50
#30	10-35
#50	5-30
#100	0-15

(5) ALTERNATE NO. 5 Amd.5

Sieve Size	Passing %
1"	100
3/4"	90-100
3/8"	20-55
#4	0-10
#8	0-5

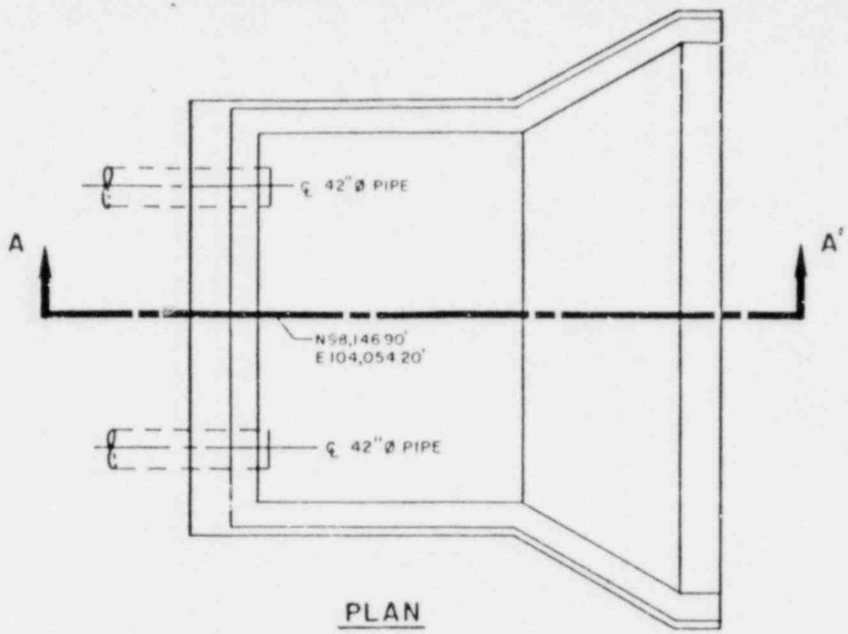
(6) ALTERNATE NO. 6 (Sand) Amd.5

Sieve Size	Passing %
3/8"	100
#4	95-100
#8	80-100
#16	50-85
#30	25-60
#50	10-30
#100	2-10

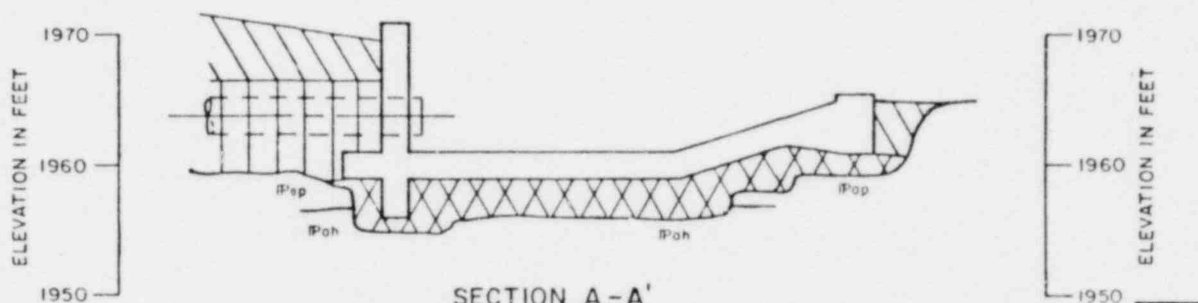
d. The bedding material shall be placed in not more than 6 inch layers and vibratory tamped to a relative density of not less than 80% as determined by ASTM D-2049.

Q241.4 For the ESWS discharge structure, submit drawings showing plans and typical cross-sections of the limits of excavation and types of fill and backfill materials.

R241.4 The information is provided on Figure 241.4-1.



PLAN



SECTION A-A'

EXPLANATION

SYMBOL	STRATIGRAPHIC MEMBER
Pop	PLATTSMOUTH LIMESTONE MEMBER
Poh	HEEBNER SHALE MEMBER

KEY:

- COHESIVE FILL
- PIPE BEDDING
- LEAN CONCRETE FILL

NOTE:

ELEVATIONS AND COORDINATES REFER TO SNUPPS SYSTEM.

N100,000 SNUPPS = N584,670 STATE PLANE COORDINATE SYSTEM

E100,000 SNUPPS = E2,807,250 STATE PLANE COORDINATE SYSTEM

ELEVATION 2,000' SNUPPS = 1,100' U.S.G.S. DATUM.

DRAWING REFERENCE:

1. DRAWING NO: C-KC 401, REV. A.
   BY: BECHTEL
   GAITHERSBURG, MARYLAND
2. SITE SURVEY DATA.

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FIGURE 241.4-1

ESWS DISCHARGE STRUCTURE
PLAN AND SECTION

Q241.5 1. In Figure 2.5-47 show locations and limits of soft material, if any, that was replaced by competent material during construction.

2. For the ECCS pipeline, provide typical transverse cross section showing the excavation limits, pipe, bedding, and different kinds of backfill materials.

3. Provide typical longitudinal section and cross section details of excavation and backfill near the interface between the ECCS pipes and the structures.

4. What are the estimated total and differential settlements of the ECCS pipe and the structures at their interface due to both static and dynamic loads?

5. What is the estimated settlement of the ECCS piping due to both static and dynamic loads?

R241.5 1. No soft material was encountered.

2. Response to be provided by October 1, 1981.

3. Response to be provided by October 1, 1981.

4. Response to be provided by October 1, 1981.

5. Response to be provided by October 1, 1981.

Q241.6 (2.5.4.7) Provide a copy of the Bechtel Topical Report, BC-TOP-4A, referenced on page 2.5-199 of the FSAR.

R241.6 Bechtel Topical Report, BC-TOP-4A, was approved by the NRC on October 31, 1974 (Reference) and has been used on many previous plant designs where Bechtel is the architect/engineer.

Reference: Letter of October 31, 1974 from R. W. Klecker, NRC, to John V. Morowski, Bechtel Power Corporation.

Q241.7 (2.5.4.10.1.3) Provide a plot of the magnitude and distribution of lateral earth and water pressures used in the design of subsurface walls and, on the same figure, plot the dynamic lateral pressures computed from the soil-structure interaction analyses due to the building and soil response under dynamic loading conditions. Provide such plots for the main powerblock structures, the ESWS pumphouse, and the ESWS discharge structure.

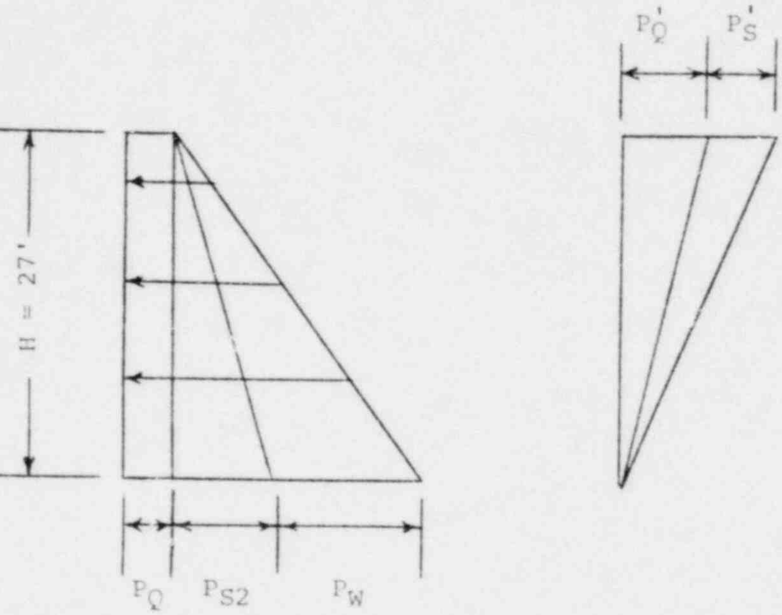
R241.7 The plots of the magnitude and distribution of lateral earth and water pressures as well as dynamic lateral pressures are provided in Figures 241.7-1 through 241.7-6.

Fig. 241.7-1

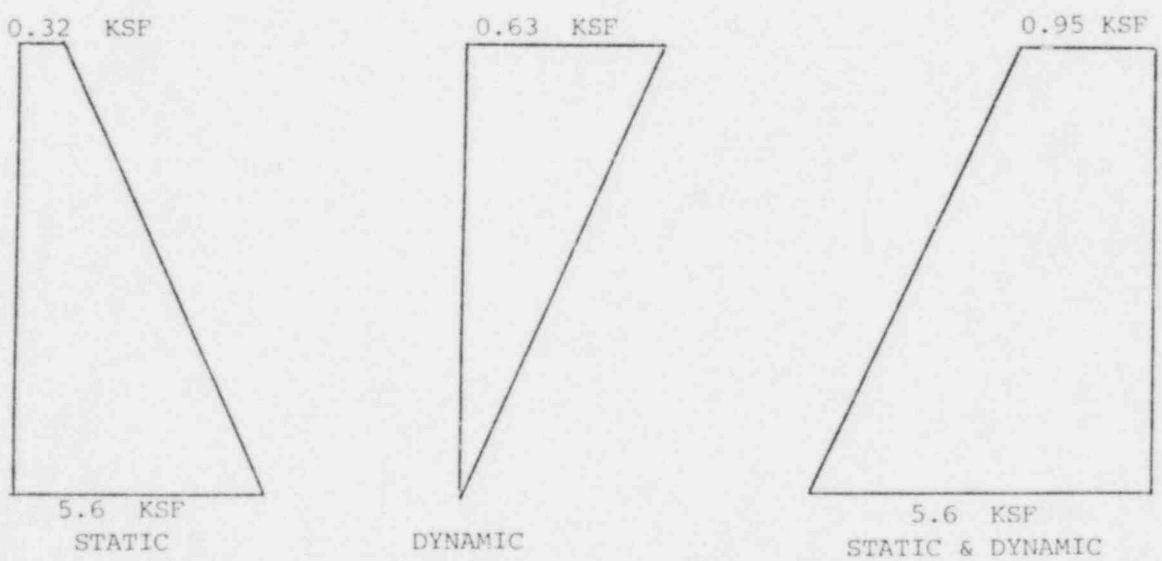
AUXILIARY BUILDING

EXTERIOR WALL DESIGN

NOTE: THE WATERTABLE WAS ASSUMED TO BE AT GRADE LEVEL.



STATIC X 1.7 (KSF)			DYN X 1.9 (KSF)		TOTAL WITH LOAD FACTORS	
					EARTH PRESS	EARTH PRESS
P_Q	P_{S2}	P_W	P'_Q	P'_S	@ EL. 2000'	@ EL. 1973'
0.32	2.43	2.85	0.10	0.53	0.95 KSF	5.6 KSF

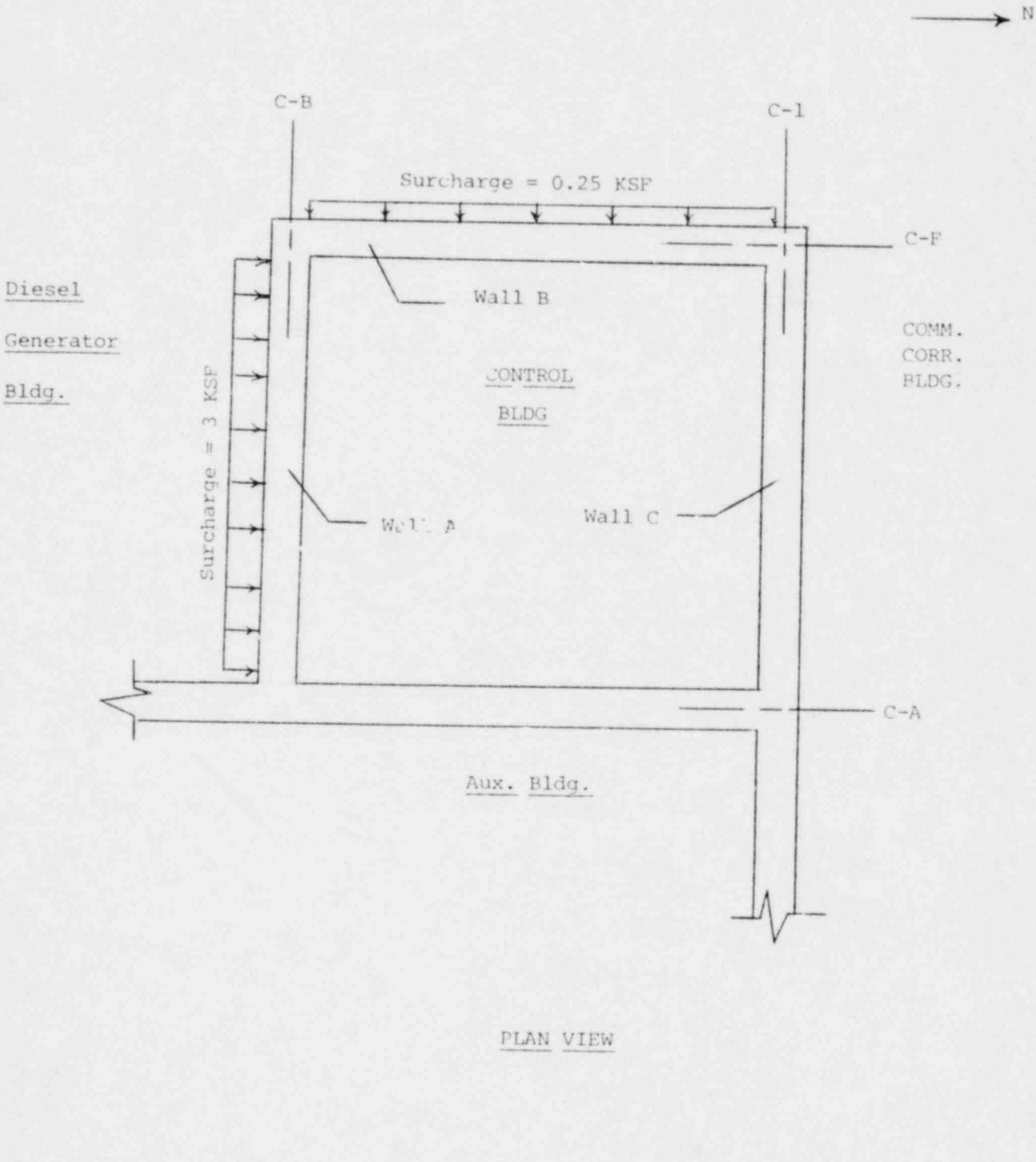


P_Q, which is the static pressure due to surcharge loading, varied according to the imposed load due to the building adjacent to the exterior wall of the auxiliary building.

Fig. 241.7-2

CONTROL BUILDING

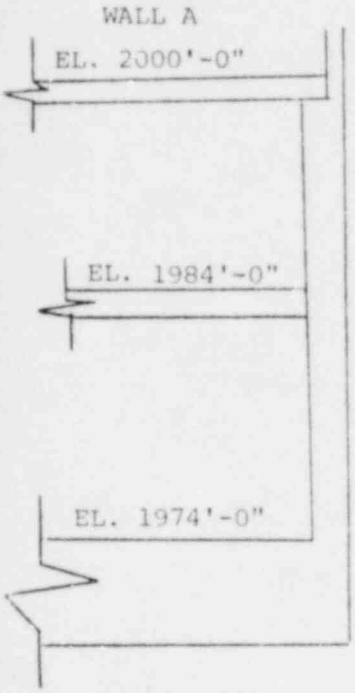
EXTERIOR WALL DESIGN

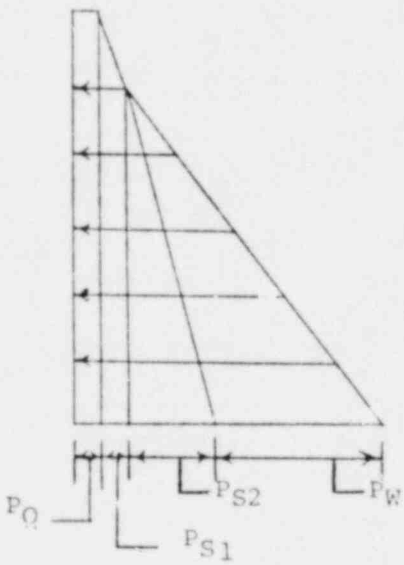


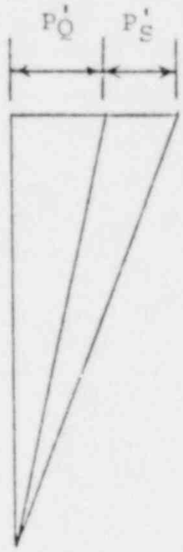
PLAN VIEW

CONTROL BUILDING

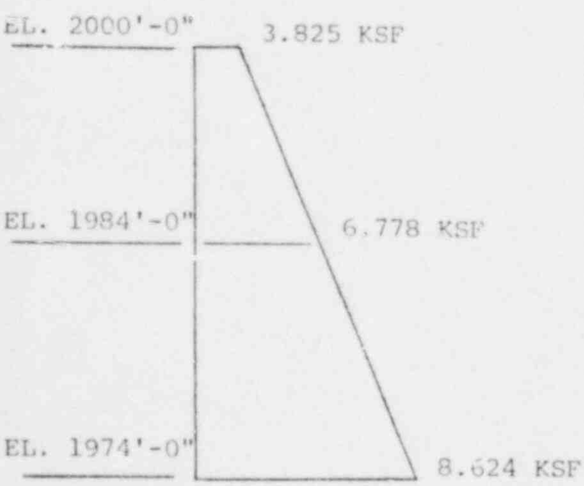
EXTERIOR WALL DESIGN



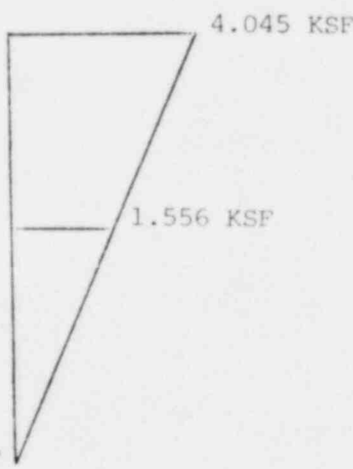




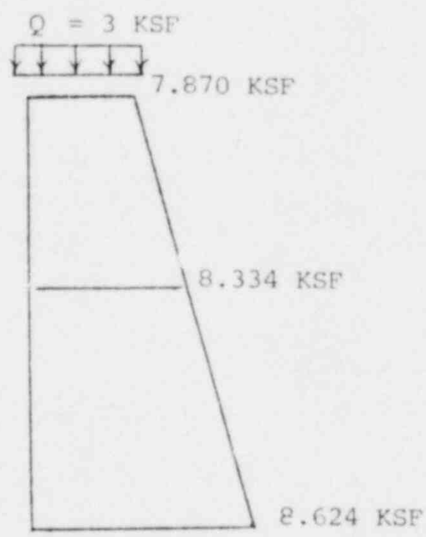
STATIC X 1.7 (KSF)				DYN X 1.9 (KSF)		TOTAL WITH LOAD FACTORS	
						EARTH PRESS @ EL. 2000	EARTH PRESS @ EL. 1974
P_Q	P_{S1}	P_{S2}	P_W	P'_Q	P'_S		
3.825	1.198	1.651	1.95	2.622	1.423	7.870	8.624



STATIC



DYNAMIC



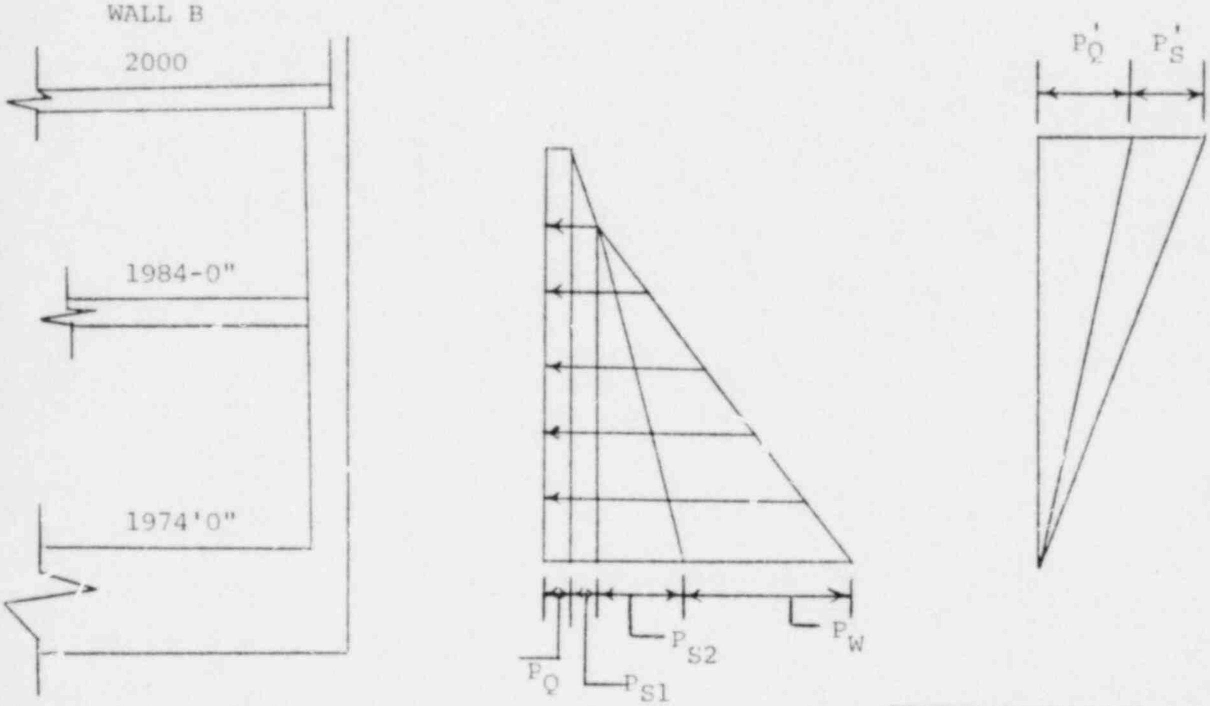
STATIC & DYNAMIC

(DESIGN)

Fig. 241.7-4

CONTROL BUILDING

EXTERIOR WALL DESIGN



STATIC X 1.7 (KSF)				DYN X 1.9 (KSF)		TOTAL WITH LOAD FACTORS	
P_Q	P_{S1}	P_{S2}	P_W	P_Q'	P_S'	EARTH PRESS @ EL. 2000'	EARTH PRESS @ EL 1974'
0.319	1.198	1.651	1.75	0.219	1.423	1.961	5.118

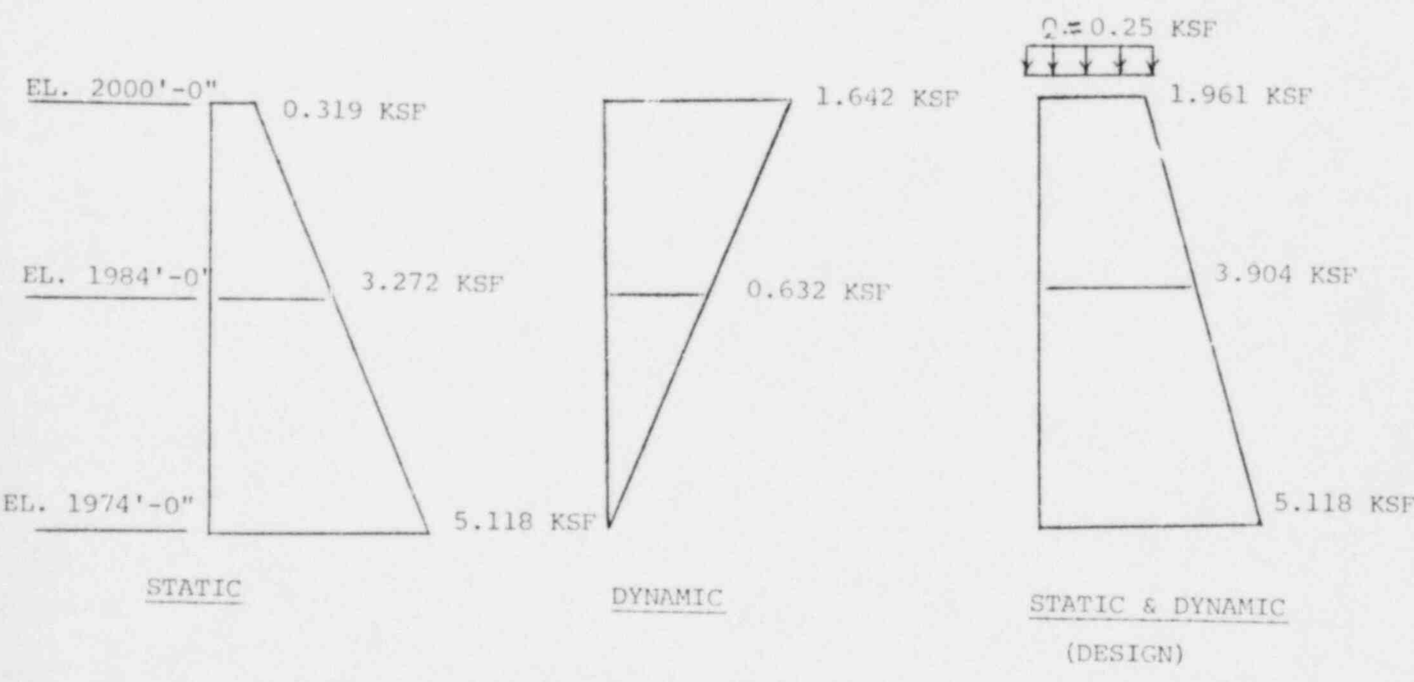


Fig. 241.7-5

ESWS PUMPHOUSE

EXTERIOR WALL DISIGN

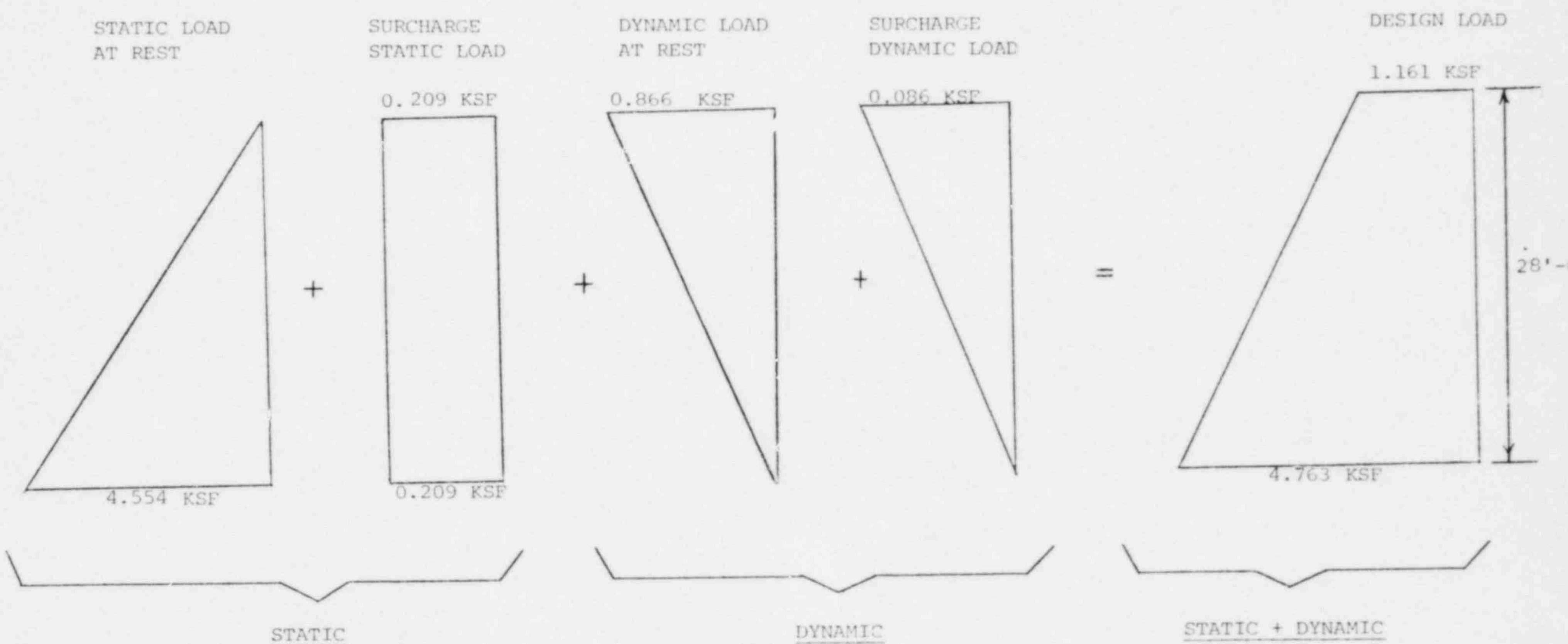
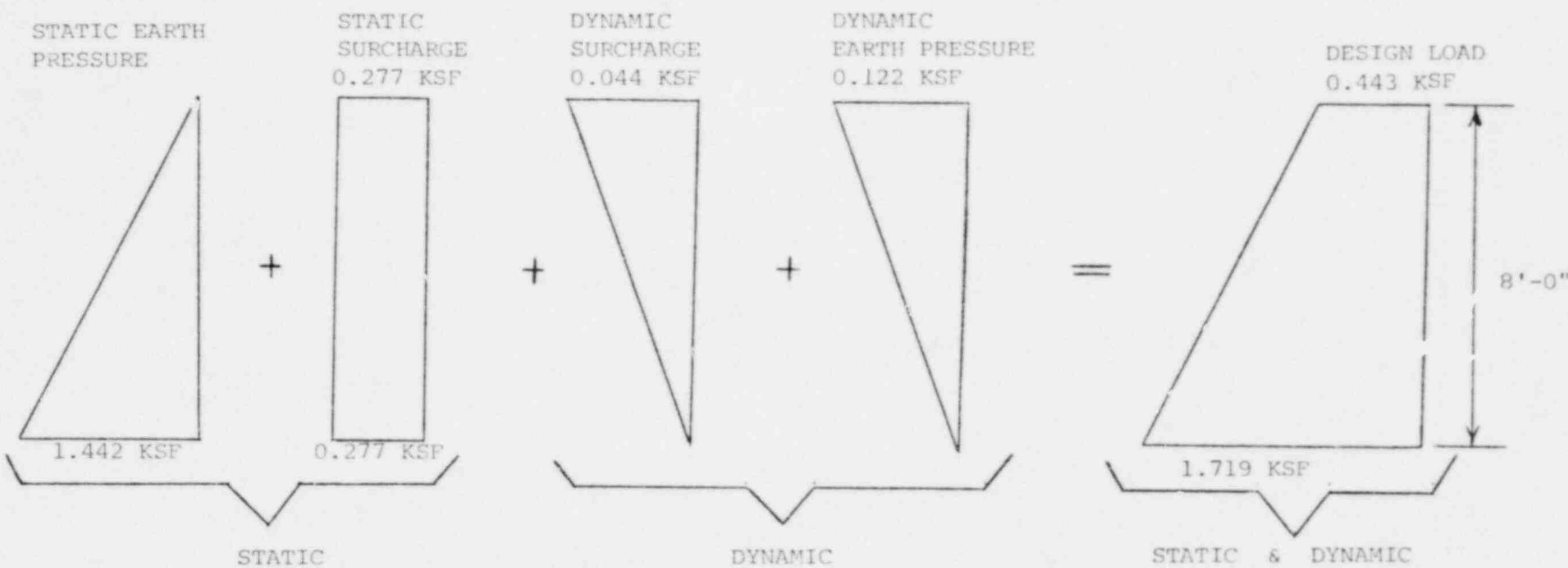


Fig. 241.7-6

ESWS DISCHARGE STRUCTURE

EXTERIOR WALL DESIGN



Q241.8 (Figure 2.5-111) Revise FSAR Figure 2.5-111 to show the location of sections GG and HH.

R241.8 Sections GG and HH are shown on Figure 2.5-108. Figures 2.5-111 and 2.5-108 of FSAR have been revised accordingly.

Q241.9 (Figure 2.5-112) In Figure 2.5-112 show the following missing information.

a) The water levels and the piezometric surfaces used in the stability analyses for all conditions analyzed.

b) Show the minimum factor of safety and the corresponding critical sliding wedge.

R241.9 Figure 2.5-112 of FSAR has been revised to show the water level and identify the critical failure wedge, soil parameters and minimum factor of safety for the cases investigated. The critical failure wedge is the same for each of the cases.

Q241.10 (Figure 2.5-113)

1. In Figure 2.5-113 show the following missing information:

   a. Subsurface soil profile and the soil parameters for each soil layer that were used in the slope stability analyses.

   b. Show the water levels and the piezometric surfaces used in the stability analyses for all conditions analyzed.

   c. Show the minimum factors of safety and the corresponding critical slip circles for each of the cases investigated.

2. Discuss the validity of using slip circle method of analysis, particularly for the side slopes of the pumphouse intake channel (3H:1V), considering that a) the hard rock layer is in the immediate vicinity of the toe of the slope, b) for the UHS slope you choose to use the sliding wedge method of analysis. Justify the validity of the slip circle method of analysis or investigate the stability of the slopes of the ESWS pumphouse intake channel using the sliding wedge method.

Q241.10 (continued)

3. For the cross section presented in Figure 2.5-113 explain why the minimum factor of safety for the stability of (3H:1V) slope is higher than the minimum factor of safety for the stability of (5H:1V) slope.

R241.10 1. The information requested is shown on Figures 2.5-113a through 2.5-113h. Section 2.5.5.2.2.2 had been revised to include a reference to these figures.

2. The 3H:1V side slopes of the ESWS Intake Channel are cut into the Heumader Shale material. During early stages of design and as presented in the PSAR these slopes were specified to be 1H:1V. During the excavation of the power block, it was discovered that this material weathers rapidly if it is not protected from exposure. Subsequently, the slopes of the ESWS Intake Channel were flattened to 3H:1V and the material was conservatively assumed to have the properties of residual clays which had been derived from weathering of similar shale material found where the Heumader Shale formation was exposed along the Wolf Creek Valley.

   The residual strength is much less than the strength of unweathered shale. When the material is assumed to be soil in and below the slope, the slip circle method of analysis is applicable.

3. For the cross-section presented in Figures 2.5-113a through 2.5-113h the minimum factor of safety for the stability of the 3H:1V slope is higher than the minimum factor of safety for the 5H:1V slope because the height of slope above the toe of the 3H:1V slope is 5 feet and the height of slope above the toe of the 5H:1V extends from elevation 1070 to existing grade and is much greater than 5 ft. The height of the 3H:1V slope is limited to 5 ft. because of the 55 ft. wide bench provided at elevation 1070.

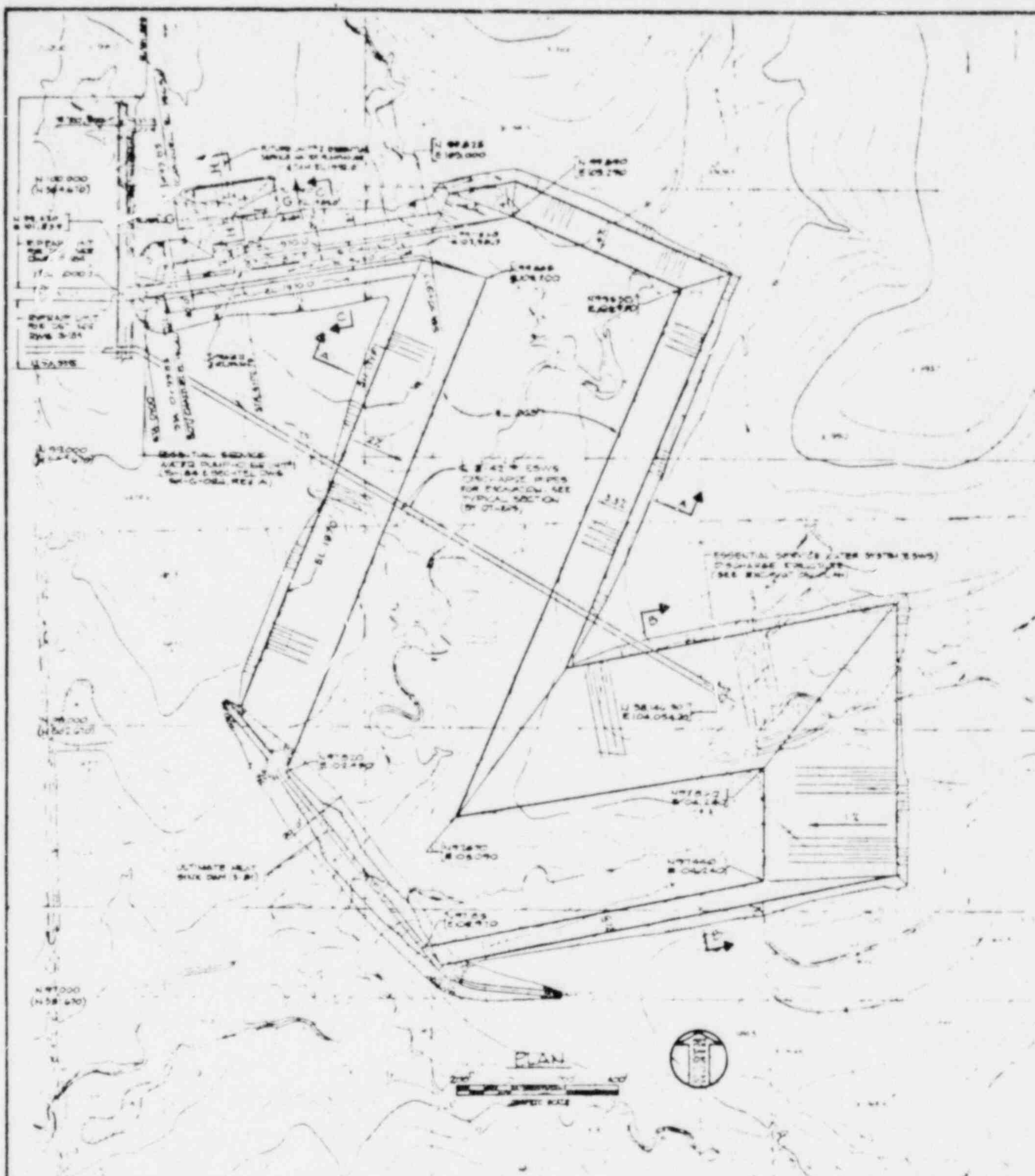


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FIGURE 2.5-108

ULTIMATE HEAT SINK

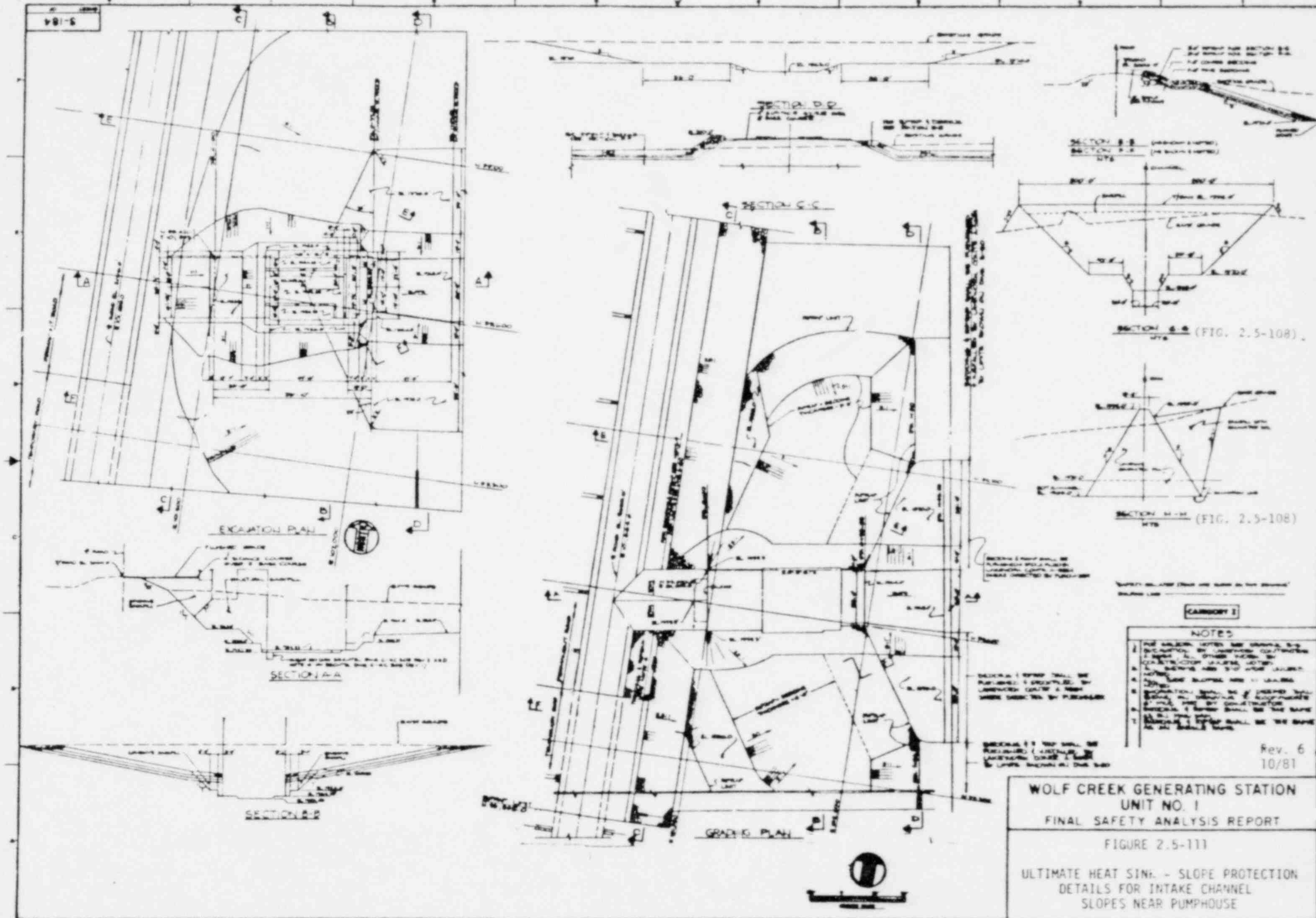

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FIGURE 2.5-111

ULTIMATE HEAT SINK - SLOPE PROTECTION DETAILS FOR INTAKE CHANNEL SLOPES NEAR PUMPHOUSE

CONDITION	SOIL PARAMETERS	MINIMUM FACTOR OF SAFETY
End of Construction	γ=124pcf ϕ_{cu}=10° c_{cu}=585 psf	7.8
Steady State	γ=124pcf ϕ' =20° c' =400psf	5.3
Steady State Plus SSE (p.12g)	γ=124pcf ϕ' =20° c' =400psf	3.5

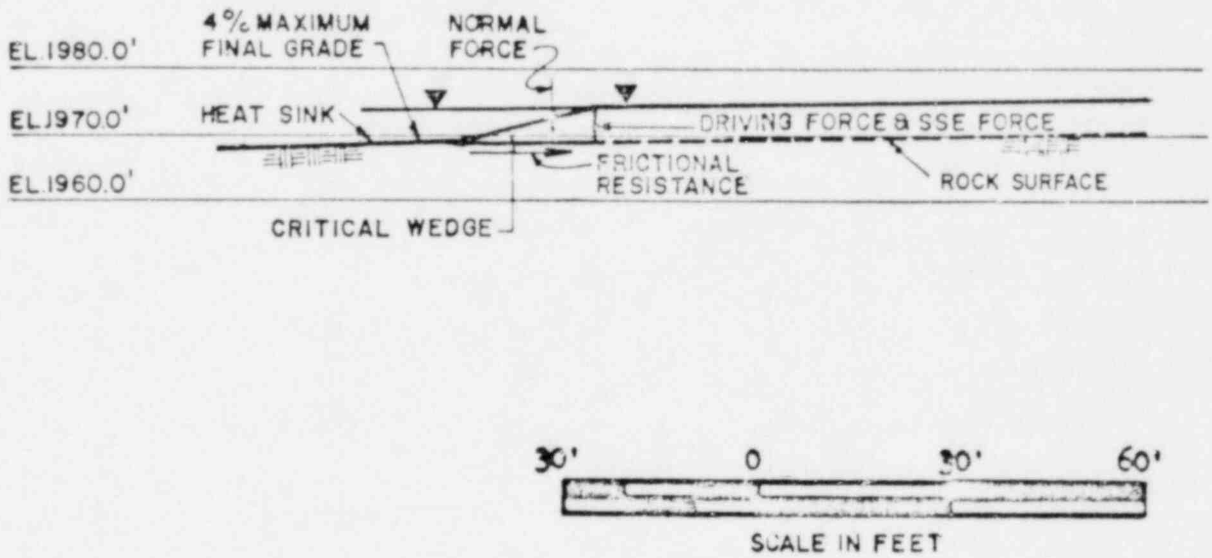

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FIGURE 2.5-112

ULTIMATE HEAT SINK -
WEDGE ANALYSIS OF EXCAVATED SLOPES

The two slopes (upper slope five horizontal to one vertical and lower slope three horizontal to one vertial) have been analyzed for the following conditions:

a. Steady state, water in channel at Elevation 1,070 feet;

b. Steady state with SSE of 0.12g;

c. End of construction;

d. End of construction with SSE of 0.12g; and

e. Rapid drawdown.

The BISHOP computer program was used to investigate the stability of slopes for all the above design conditions (Figures 2.5-113a through h). The details of this program are given in Section 3.12. The soil properties used in the analyses are given in Table 2.5-55. The shale was conservatively assumed to have the properties of residual clay for the stability analyses.

The effective stress method of analysis was used for evaluating the steady state condition with and without an SSE of 0.12g. The minimum factors of safety obtained for the static case are 7.13 for the three horizontal to one vertical slope and 3.37 for the five horizontal to one vertical slope; the minimum factors of safety with SSE effects are found to be 3.37 and 1.86, respectively. These factors are higher than required, as indicated in Table 2.5-57.

The total stress method was used to analyze the end of construction conditions. The minimum factor of safety obtained without SSE effect for the five horizontal to one vertical slope is 3.14. With SSE effect, the safety factors obtained are 3.88 and 1.74 for slopes three horizontal to one vertical and five horizontal to one vertical, and corresponding soil properties, respectively. For a three horizontal to one vertical slope, no analysis was performed for the static case, as the available factor of safety with the SSE effect is considerably higher than 1.5.

Consolidated undrained total stress parameters were used to evaluate the rapid drawdown condition. In this analysis, the drawdown is assumed to be instantaneous, and no drainage occurs during the time the water level is lowered. The minimum factors of safety obtained for rapid drawdown are 5.69 for the three horizontal and to one vertical slopes. The five horizontal to one vertical slope is not affected by rapid drawdown in the ESWS Intake Channel from Elev. 1070 to 1065. Table 2.5-57 summarizes the computed and required factors of safety for the various cases analyzed. Figures 2.5-113a through 2.5-113h show the critical slip circles and minimum factors of safety for the cases investigated. The intake channel slopes are safe under all conditions considered.

TABLE 2.5-57

RESULTS OF SLOPE STABILITY ANALYSES FOR
ESWS INTAKE CHANNEL EXCAVATED SLOPES

Condition	Slope 3:1	Slope 5:1	Required Minimum Factor of Safety
Steady State	7.13	3.37	1.5
Steady State plus SSE (0.12 g)	3.37	1.86	1.2
Total Stress Analysis	FS with SSE of 0.12 g is 3.88 (higher than 1.5); therefore, no analysis is performed	3.14	1.5
End of Construction plus SSE (0.12 g)	3.88	1.74	1.2
Rapid Drawdown	5.69	***	1.2

***The 5:1 slope is not affected by rapid drawdown in the ESWS Intake Channel from elevation 1070 to 1065.

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		UNIT WT γ_T PCF	COHESION C PSF	FRICTION ANGLE Ø DEGREE
1	DUMMY LAYER	——	——	——
2	WATER	62.4	0	0
3	RESIDUAL	124	400	20
4	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 7.13

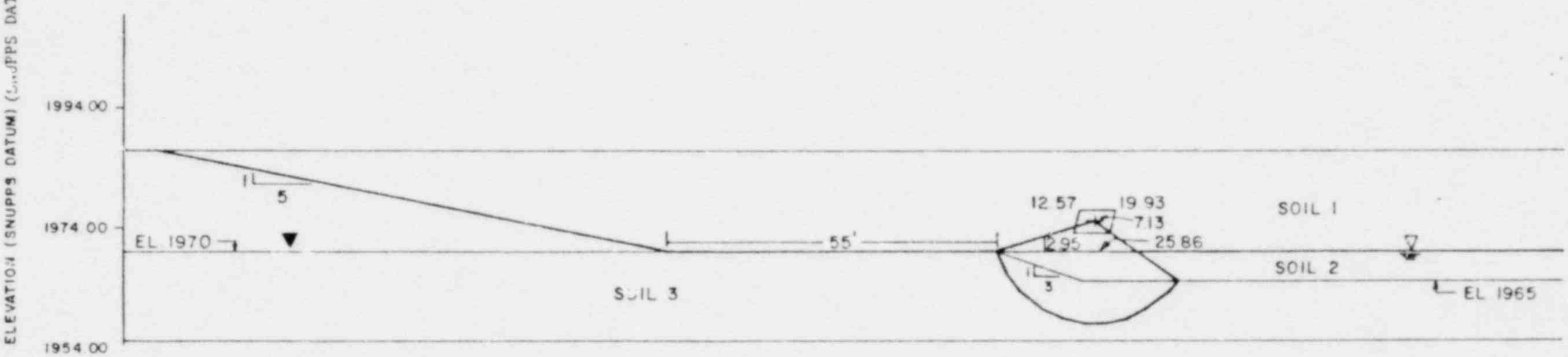

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FIGURE 2.5-113a

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
3:1 SLOPE STEADY STATE CONDITION

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		UNIT WT γ_T PCF	COHESION C PSF	FRICTION ANGLE ϕ DEGREE
1	DUMMY LAYER	——	——	——
2	WATER	62.4	0	0
3	RESIDUAL	124	400	20
4	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 3.37

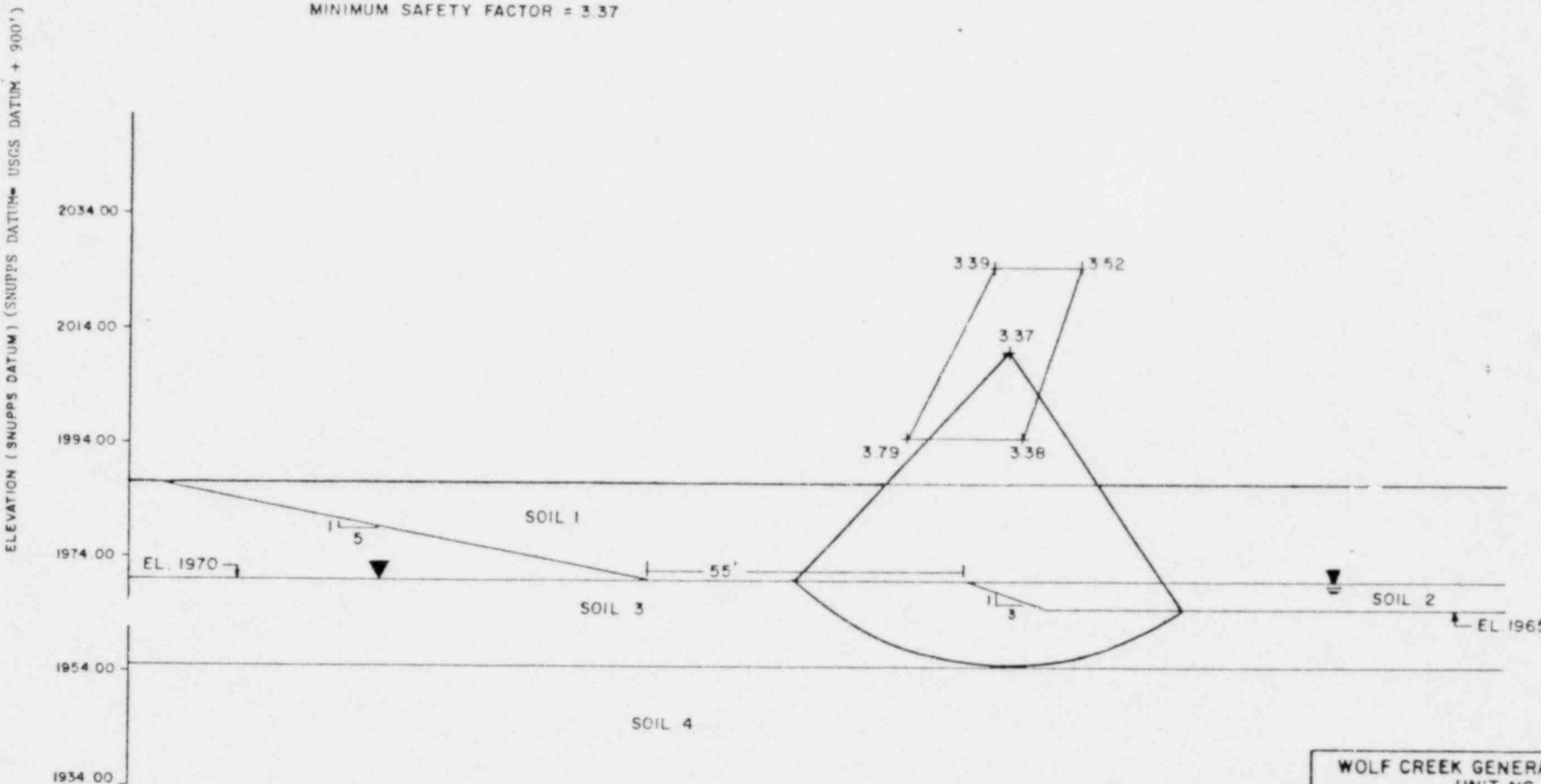

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FIGURE 2.5-113b

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
3:1 SLOPE STEADY STATE WITH SSE

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		UNIT WT. γ_T PCF	COHESION C PSF	FRICTION ANGLE ϕ DEGREE
1	DUMMY LAYER	——	——	——
2	WATER	62.4	0	0
3	RESIDUAL	124	585	10
4	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 3.88

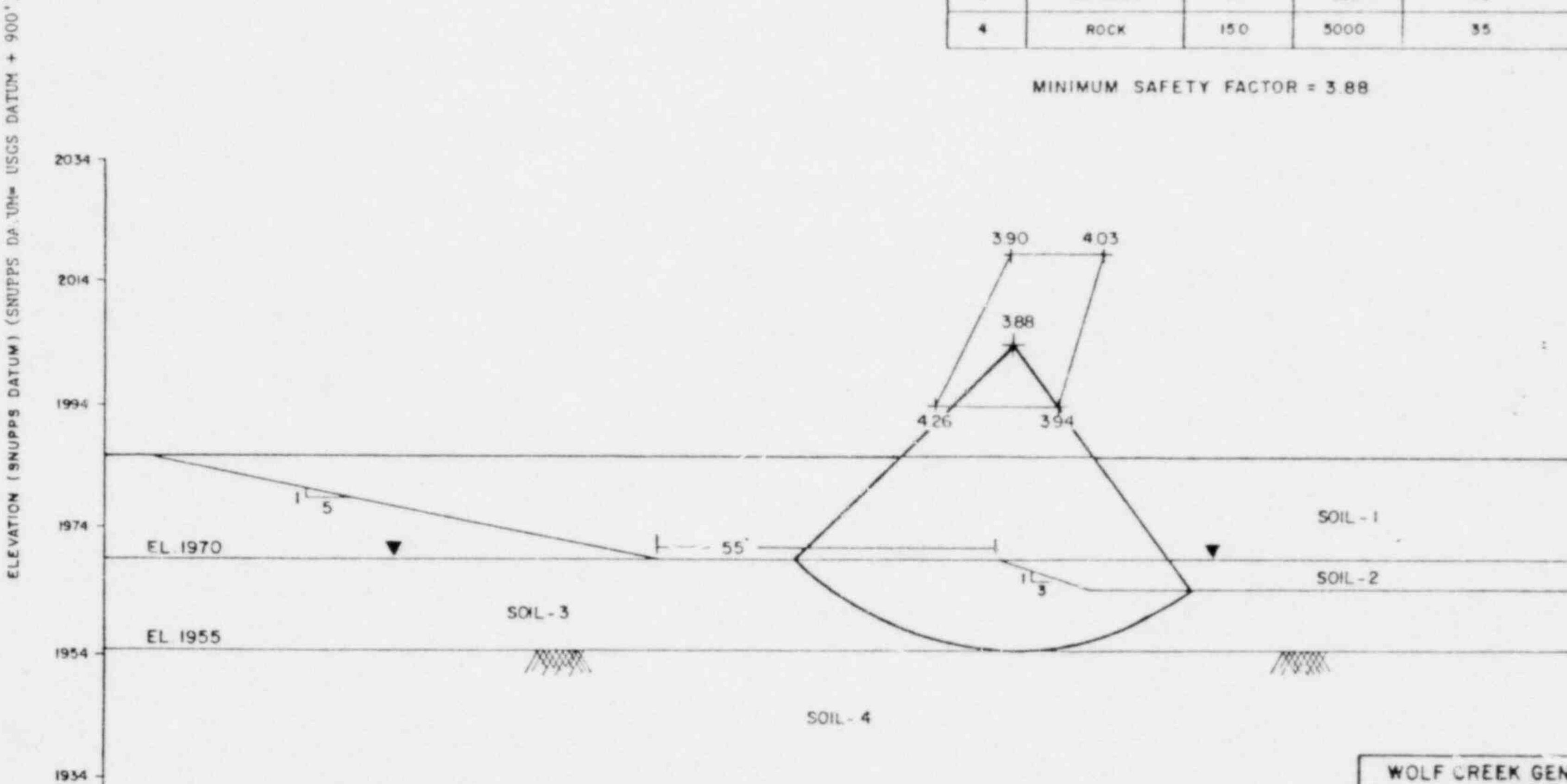

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FIGURE 2.5-113c

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
3:1 SLOPE END OF CONSTRUCTION WITH SSE

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C PSF	FRICTION ANGLE Ø DEGREE
1	DUMMY LAYER	0	0	0
2	RESIDUAL	124	585	10
3	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 5.69

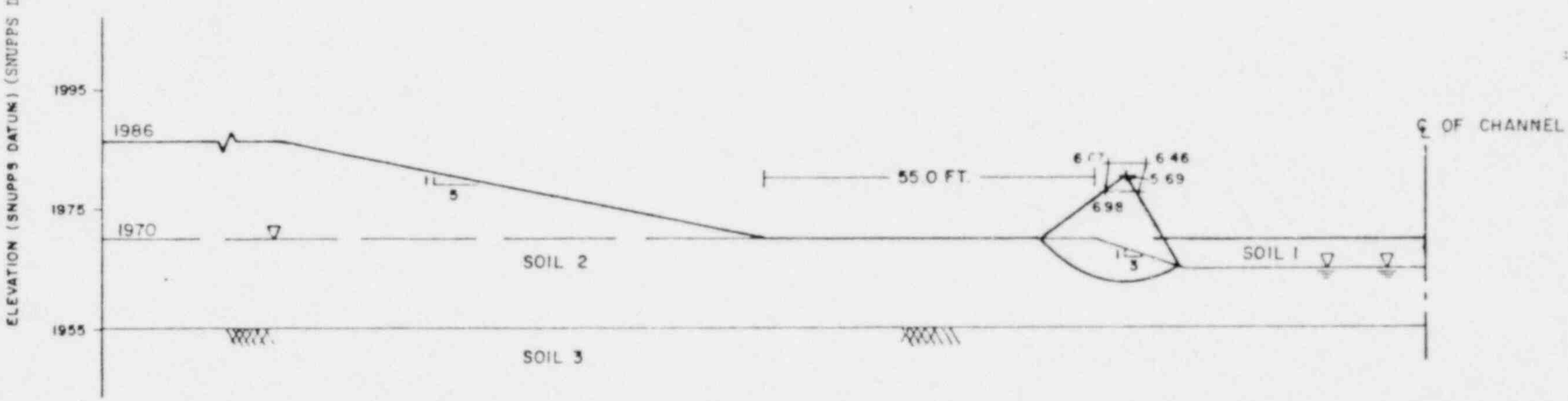

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FIGURE 2.5-113d

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
3:1 SLOPE RAPID DRAWDOWN CONDITIONS

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C PSF	FRICTION ANGLE ϕ DEGREE
1	WATER	62.4	0	0
2	RESIDUAL	124	400	20
3	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 3.37

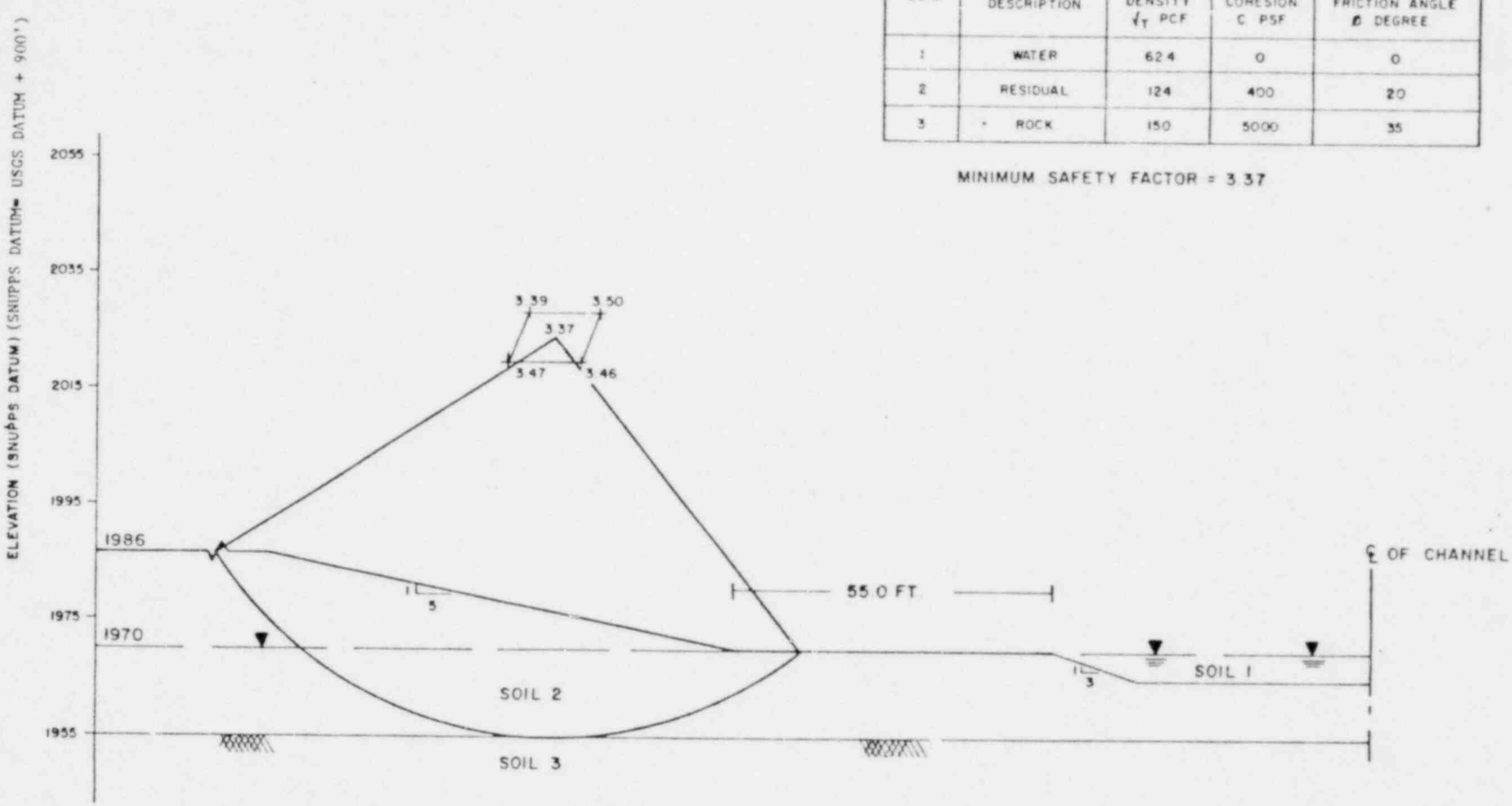

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FIGURE 2.5-113e

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
5:1 SLOPE STEADY STATE CONDITIONS

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C PSF	FRICTION ANGLE ϕ DEGREE
1	WATER	62.4	0	0
2	RESIDUAL	124	400	20
3	ROCK	150	5000	35

MINIMUM SAFETY FACTOR 1.86

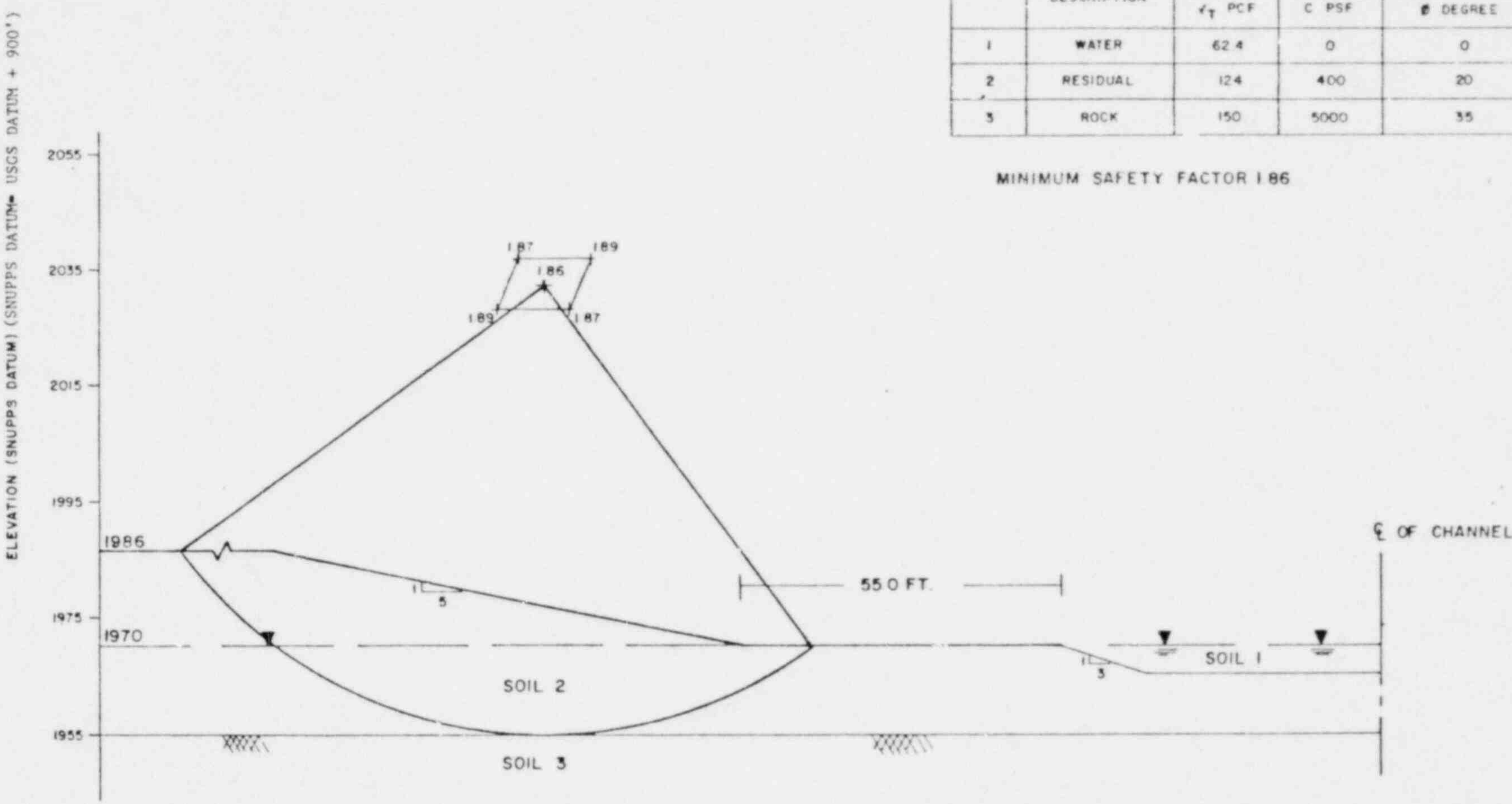

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FIGURE 2.5-113f

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
5:1 SLOPE STEADY STATE WITH SSE

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C PSF	FRICTION ANGLE Ø DEGREE
1	WATER	62.4	0	0
2	RESIDUAL	124	585	10
3	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 3.14

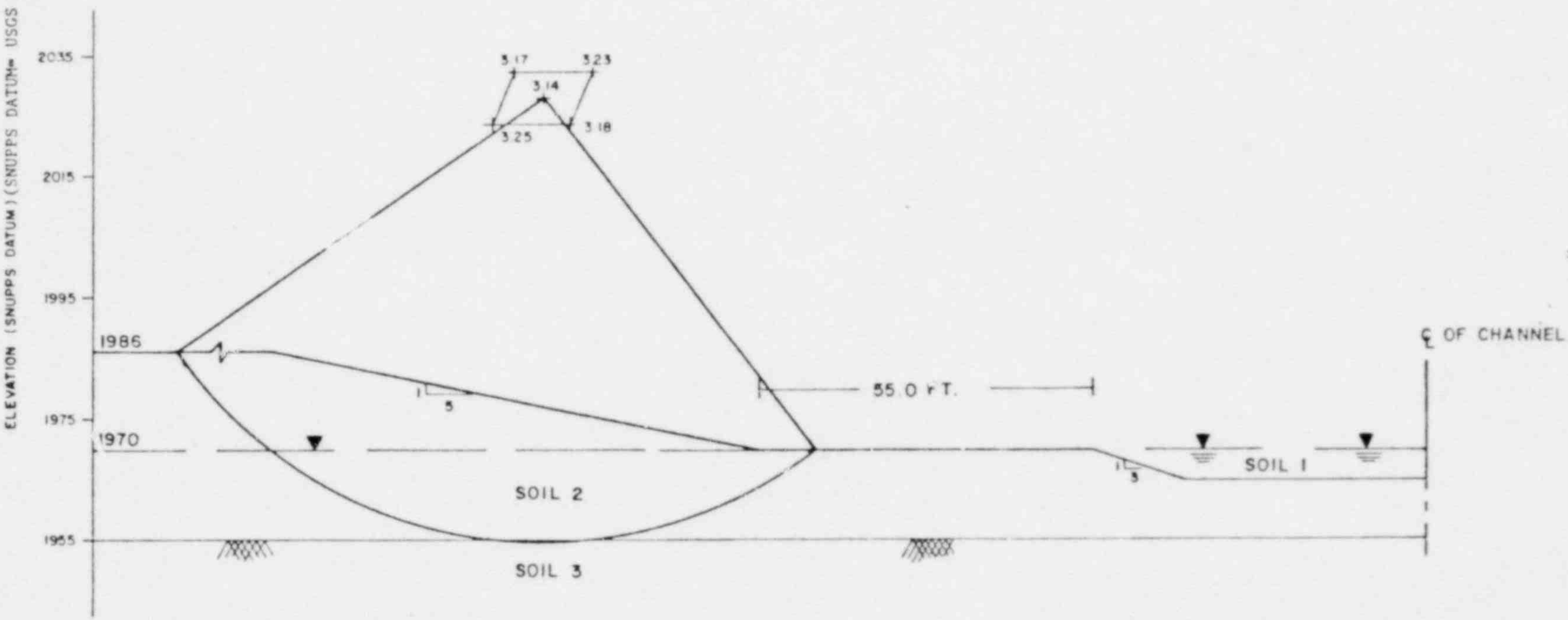

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FIGURE 2.5-113g

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS,
5:1 SLOPE END OF CONSTRUCTION WITH SSE

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY PCF	COHESION C PSF	FRICTION ANGLE Ø DEGREE
1	WATER	62.4	0	0
2	RESIDUAL	124	585	10
3	ROCK	150	5000	35

MINIMUM SAFETY FACTOR = 1.74

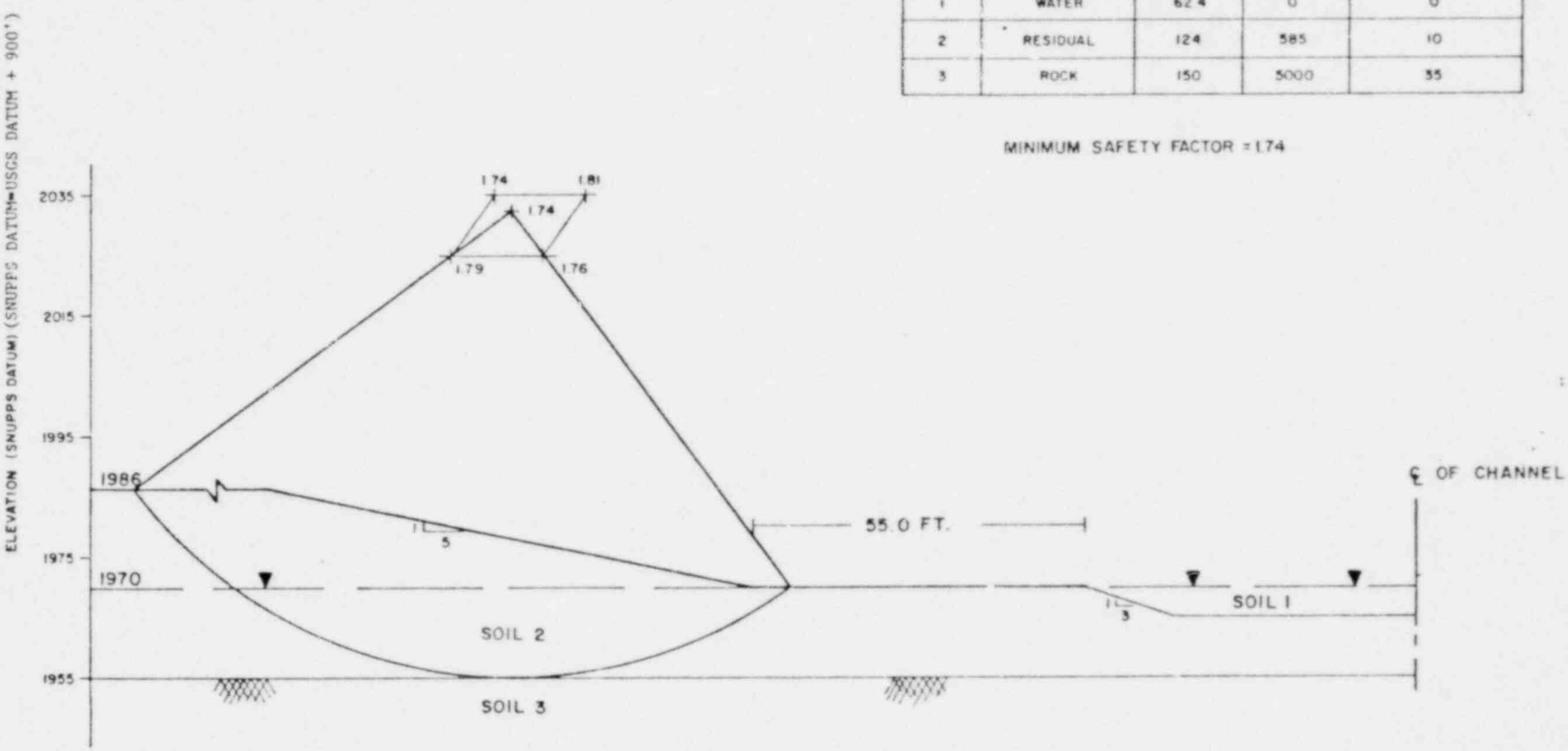

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FIGURE 2.5-113h

ESWS INTAKE CHANNEL
SLOPE STABILITY ANALYSIS.
5:1 SLOPE RAPID DRAWDOWN CONDITIONS

Q241.11 (Figure 2.5-115) Show the critical slip circle and the corresponding minimum factor of safety for the cases investigated in the stability analyses presented on Figure 2.5-115. Also, correct Detail A that shows the fine filter layer between the coarse filter layer and the riprap layer.

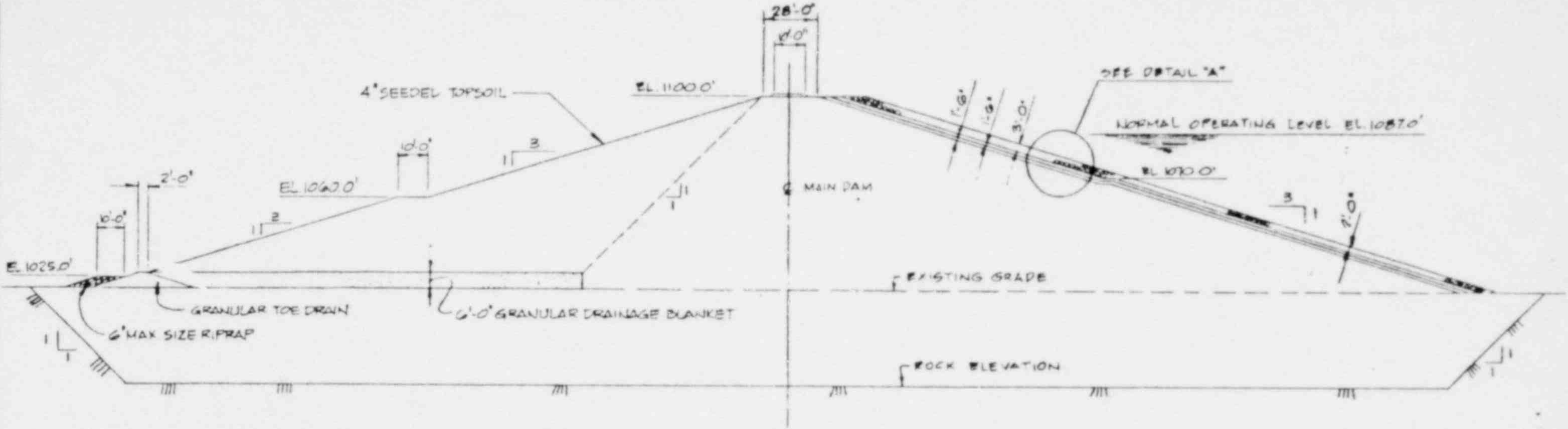
R241.11 Figures 2.5-115B through 2.5-115d show the critical slip circles and Factors of Safety for the cases investigated. Section 2.5.6.5.1.2 has been revised to include a reference to these figures. Detail A on Figure 2.5-115 (this is now Figure 2.5-115a) has been corrected.

Q241.12 (2.5.6.8.4) Provide a description of the monitoring system that is being used to measure the movements of the UHS dam. Summarize the data collected to date and compare the results with the estimated movements of the UHS dam. Comment on the results of this comparison and its safety implication.

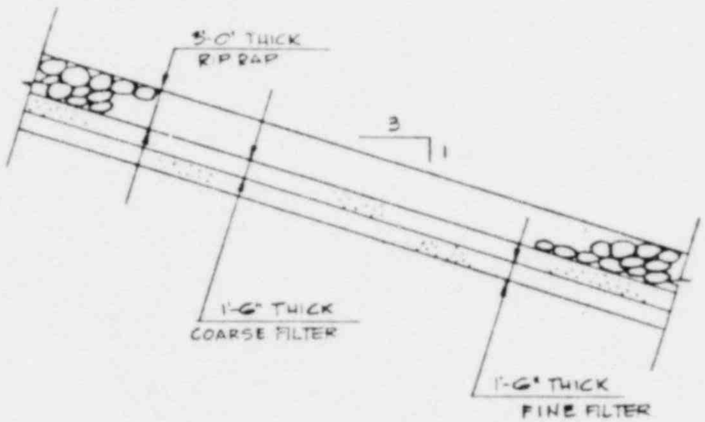
R241.12 The monitoring system that is being used to measure horizontal and vertical movements of the UHS dam consists of concrete piers located at Stations -2+00, 2+00, 4+00, 5+50, 7+00, 8+50, 10+00 and 12+00 along the centerline at the dam crest. These piers are 3 feet in diameter and are embedded in the embankment 5 feet and extend 1 foot above the riprap. A survey maker is embedded in the center of each pier. A comparison marker consisting of a 3 inch diameter pipe with a survey marker attached that extends up to elevation 1091 feet is also provided for making measurements when the piers become submerged.

These survey monuments were installed at completion of riprap placement on the UHS dam and initial elevations were measured on May 20, 1980 and initial coordinate locations were established by trilateration techniques from reference monuments on May 23, 1980.

Vertical settlement readings have been taken monthly and as of July 10, 1981, the maximum vertical settlement is 0.72 inches at Station 4+00. The height of dam embankment is the greatest at this location (Figure 2.5-116) and the observed settlement is well within the estimated settlment of 1.35 inches for an embankment height of 17 feet and has no influence on the safety of the UHS dam.



TYPICAL SECTION
MAIN DAM



DETAIL "A"

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FIGURE 2.5-115a

TYPICAL SECTION - MAIN DAM

Condition	Minimum Required Safety Factor
4. End of construction plus horizontal earthquake force (0.06g)	1.0
5. Steady state seepage with cooling lake at Elevation 1087 with horizontal earthquake force (0.06g)	1.0

As noted above, the steady-state cooling lake elevation was taken as 1,087 feet, and the rapid drawdown condition water level was taken down to Elevation 1,030 feet. For the steady-state seepage condition, an estimate for the phreatic line was based on a flow net construction.

The computer program SLOPE was used for evaluating the safety factors for the main dam slopes. The details of SLOPE program are described in Section 3.12.

For static stability analysis, the program SLOPE uses the simplified Bishop method. In this method, the failure surface is assumed to be an arc of a circle. The pore pressures developed in the embankment during construction are also considered in the analyses. The safety factor is defined as the ratio of the moment of the available resisting forces to the moment tending to cause sliding.

To evaluate the effect of an earthquake loading on the stability of slopes and embankments, a pseudo-static force is used in the computer program SLOPE to represent the deformation effects of earthquake motions. The static force is applied to a slope mass bounded by the slope profile and the assumed failure surface. The earthquake force for a slice is equivalent to the slope mass of that slice times a percent of the acceleration of gravity. Slope mass is calculated using the total unit weights, and does not take into account any pore pressure effects. The earthquake force for each slice is applied horizontally through the center of gravity of that slice.

In the analysis, an earthquake force equivalent to 0.06g corresponding to the OBE was used to determine the stability of the main dam.

The safety factors obtained from the stability analyses are greater than the minimums described above and are given in Table 2.5-83. Figures 2.5-115b through 2.5-115d show the critical slip circles for the cases investigated.

Riprap and filter layers are placed on the upstream slopes and a rock toe is placed on the downstream end of the dam to provide protection against tailwater erosion.

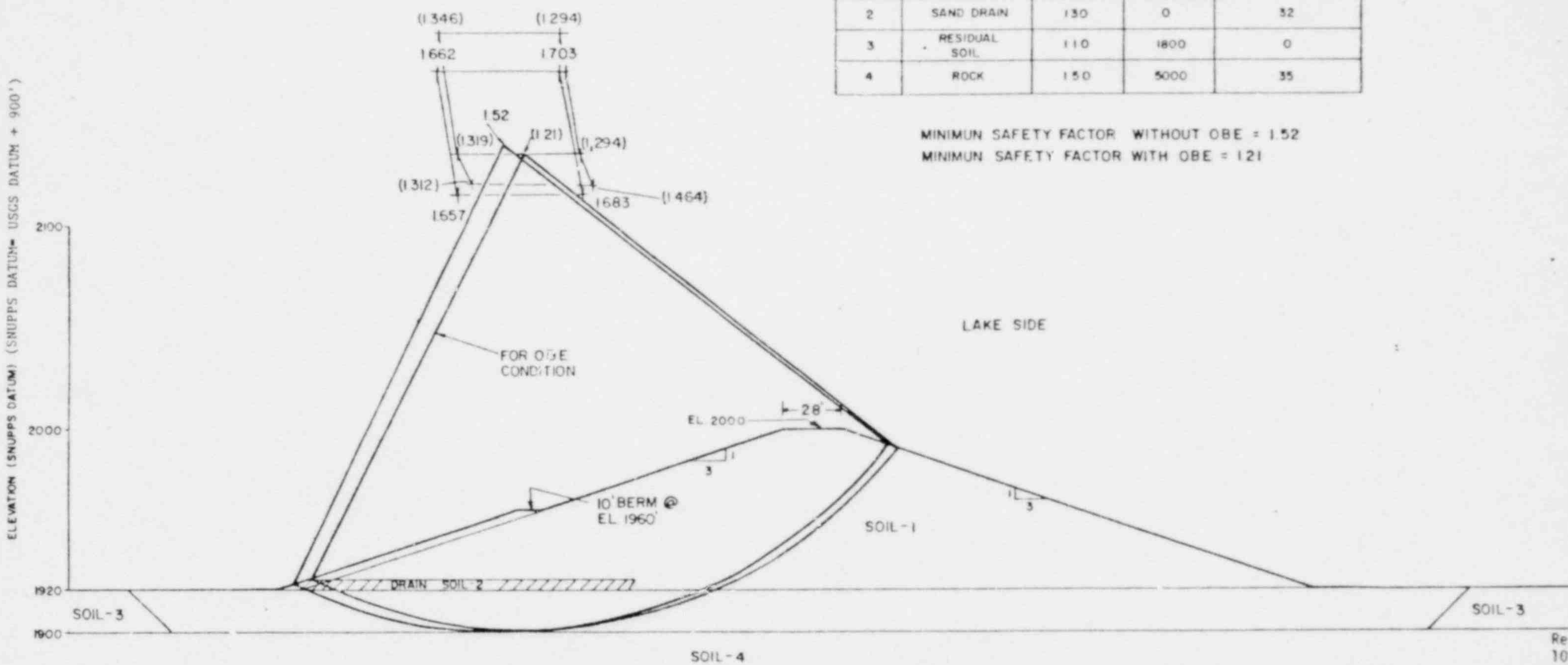
TABLE 2.5-83

RESULTS OF SLOPE STABILITY ANALYSIS FOR MAIN DAM

Condition	Computed Factor of Safety	Minimum Required Factor of Safety
End of construction	1.52	1.4
Steady state flow, cooling lake at El. 1,087 ft	1.70	1.5
Sudden drawdown, El. 1,087 ft to El. 1,030 ft	1.20	1.2
End of construction plus horizontal earthquake force (0.06 g)	1.21	1.0
Steady seepage with cooling lake at El. 1,087 with horizontal earthquake force (0.06 g)	1.38	1.0

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C PSF	FRICTION ANGLE Ø DEGREE
1	COMPACTED CLAY	121	1800	0
2	SAND DRAIN	130	0	32
3	RESIDUAL SOIL	110	1800	0
4	ROCK	150	5000	35

MINIMUN SAFETY FACTOR WITHOUT OBE = 1.52
MINIMUN SAFETY FACTOR WITH OBE = 1.21



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NOTE

1. Values in parenthesis are for OBE condition OBE of 0.06g

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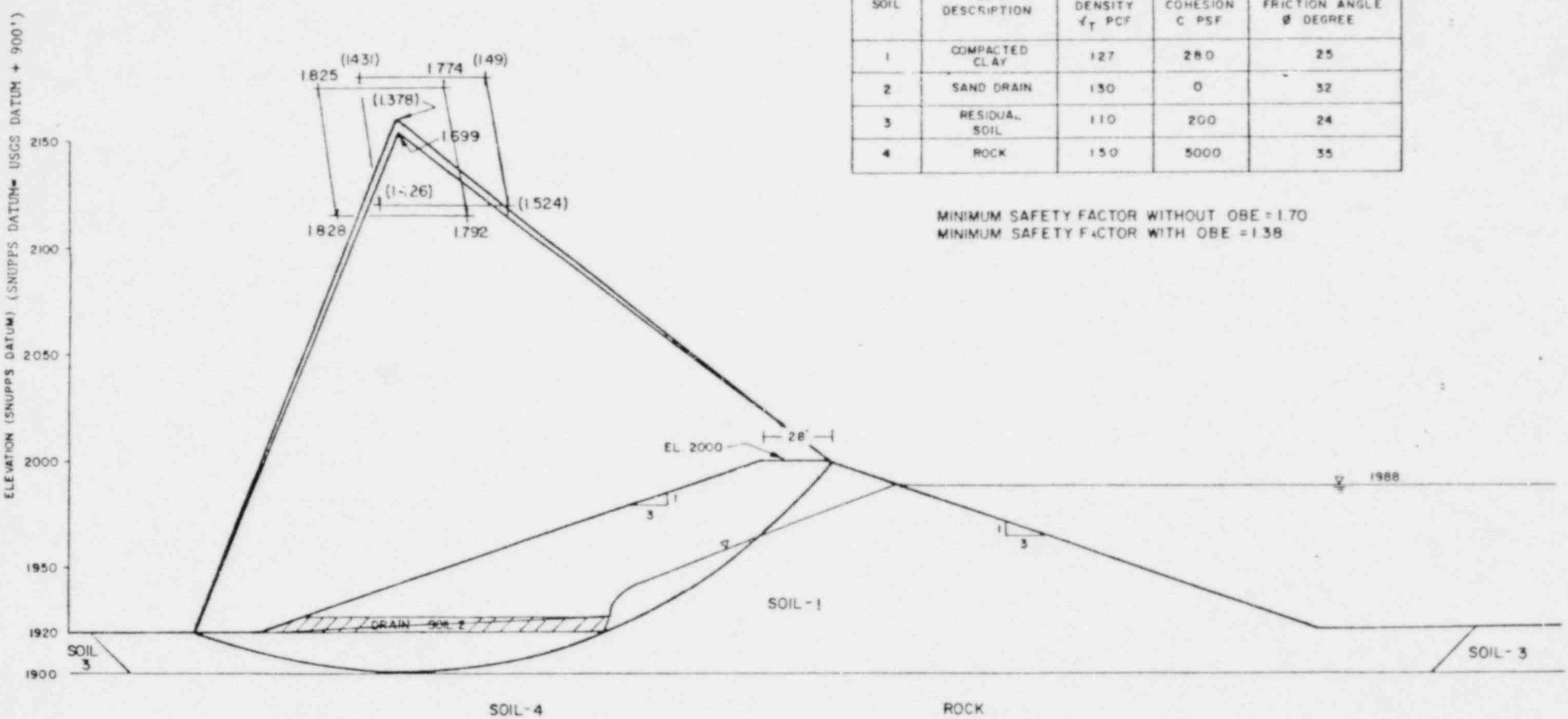
FIGURE 2.5-115b

MAIN DAM -
SLOPE STABILITY ANALYSIS -
END OF CONSTRUCTION



SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C PSF	FRICTION ANGLE Ø DEGREE
1	COMPACTED CLAY	127	280	25
2	SAND DRAIN	130	0	32
3	RESIDUAL SOIL	110	200	24
4	ROCK	150	5000	35

MINIMUM SAFETY FACTOR WITHOUT OBE = 1.70
MINIMUM SAFETY FACTOR WITH OBE = 1.38

NOTE

1. Values in parenthesis are for OBE condition. OBE of 0.06g

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FIGURE 2.5-115c

MAIN DAM -
SLOPE STABILITY ANALYSIS -
STEADY STATE CONDITIONS

SOIL	SOIL DESCRIPTION	SOIL PARAMETERS		
		DENSITY γ_T PCF	COHESION C' PSF	FRICTION ANGLE Ø DEGREE
1	COMPACTED CLAY	127	280	25
2	SAND DRAIN	130	0	32
3	RESIDUAL SOIL	110	200	24
4	ROCK	130	5000	35
5	RIPRAP	115	0	32

MINIMUM SAFETY FACTOR = 1.20

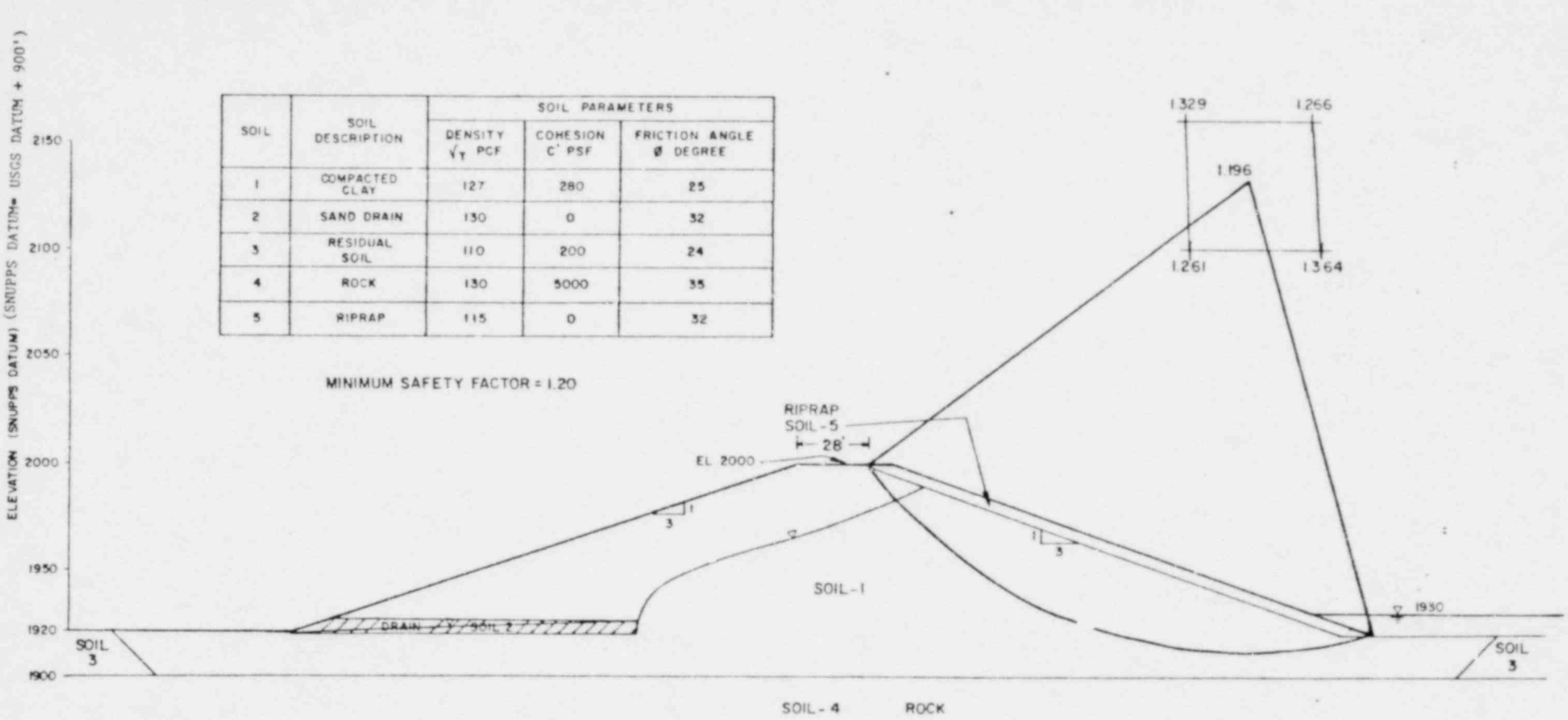

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FIGURE 2.5-115d

MAIN DAM -
SLOPE STABILITY ANALYSIS -
RAPID DRAWDOWN

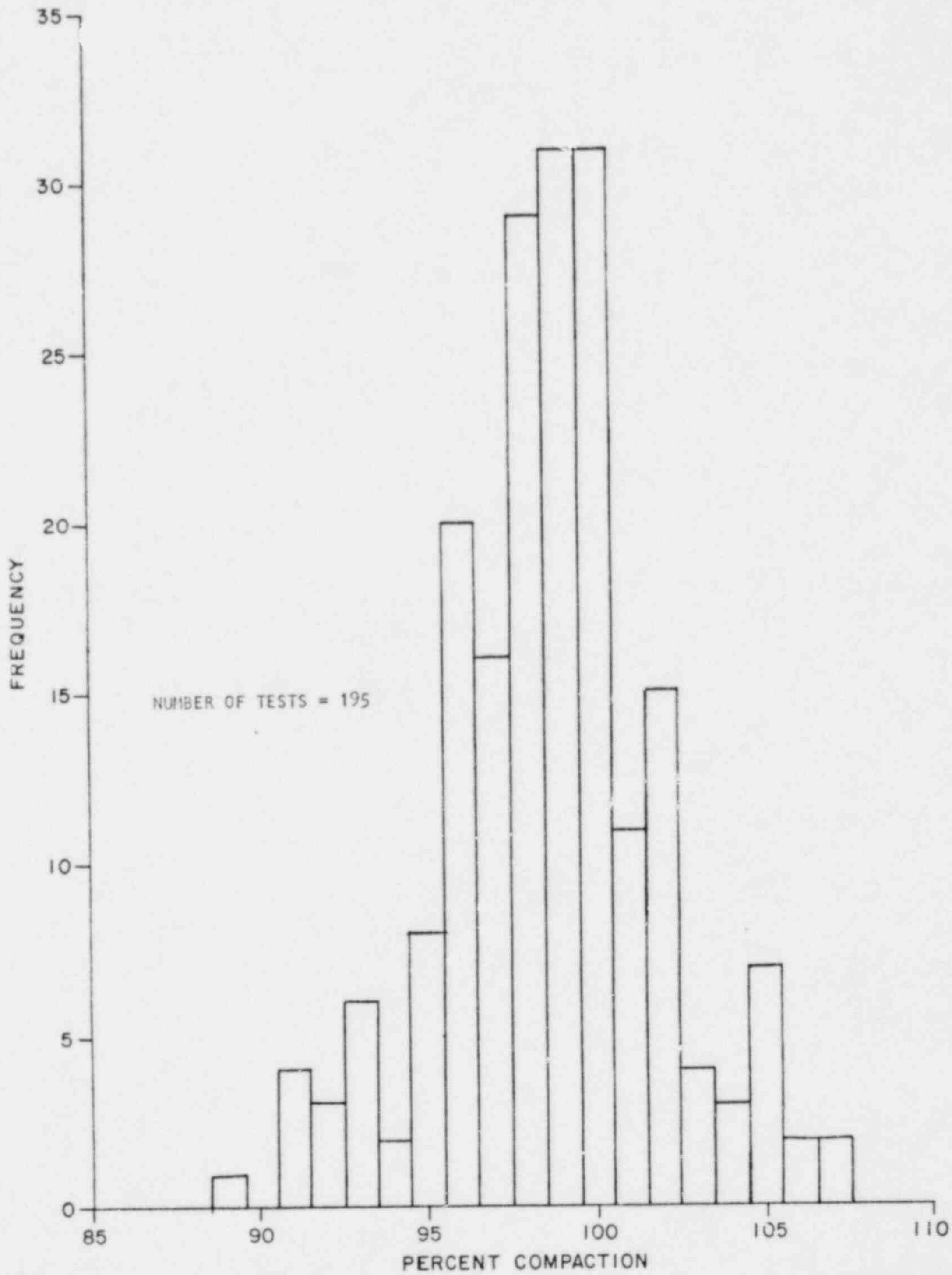
R241.12 (continued)

As of the February 16, 1981 (last readings available) measurement of horizontal movement, all of the movements were within 1.57 inches of their initial location as established on May 23, 1980. The magnitudes of the movements are within the expected survey accuracy and all or a portion of the movement could be attributed to this. Even if the measured movements have actually occurred, their magnitudes are not large and have no influence on the safety implication of the UHS dam.

Q241.13 Provide a summary of the results of field density and moisture content tests performed in connection with quality control during construction of the UHS dam. Present the results as a statistical distribution plot or by other convenient method(s) to verify that the specified compaction has been attained. Compare the compacted in-situ density and moisture content of the embankment fill with those of the test specimens from which the design strength parameters have been determined by laboratory testing. Based on the above comparison, comment on the validity of the physical and strength parameters used in the design.

R241.13 The summary of the field density tests are shown on Figures 241.13-1 and 241.13-2. Based on the field tests, 16 of the 195 field tests failed to meet the compaction criteria by 1 to 4 percent compaction. However, all failed areas were recompacted, or the failing material removed and replaced. Moisture content data are summarized on Figure 241.13-4.

Triaxial tests on six samples obtained from three different boreholes drilled in the UHS-embankment were also performed. The test results are shown on Figure 241.13-3 and Table 241.13-1. As can be seen, all tests yielded strengths higher than the design strength. Based on this information, the strength parameters used in the design are valid.



7699-064-07

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THESE PLOTS.

2. MINIMUM COMPACTION IS ASTM D-698.

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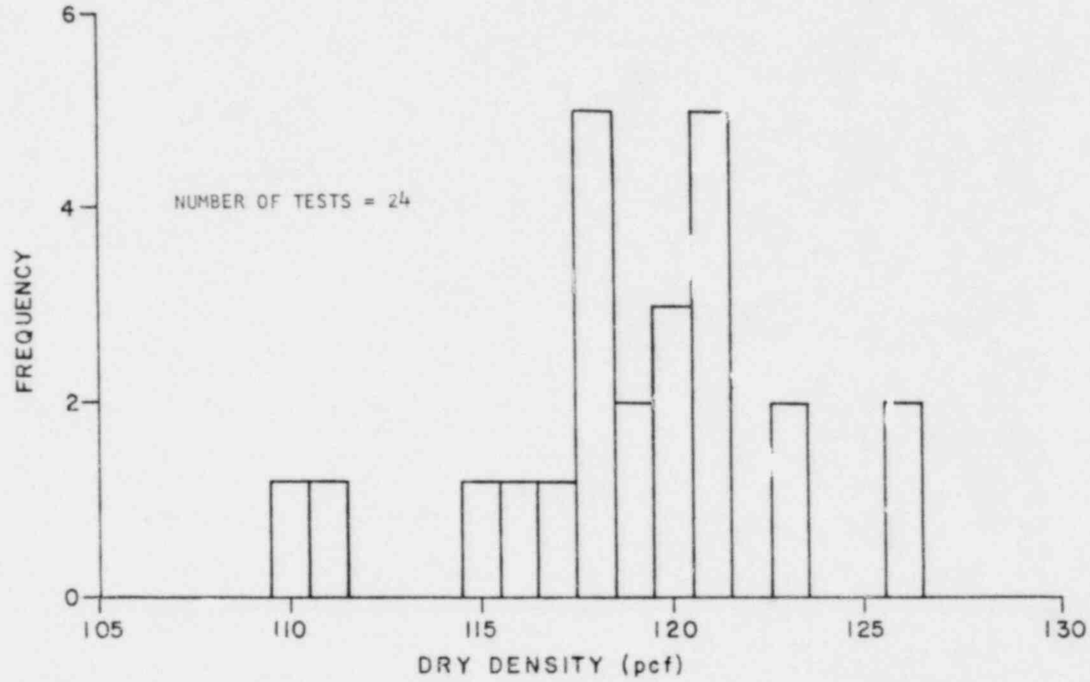
FIGURE 241.13-1

UHS DAM - EARTH FILL EMBANKMENT STATISTICAL DISTRIBUTION PLOT

MATERIAL IDENTIFICATION NO. LWRD-22
80% RELATIVE DENSITY = 117 pcf



NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

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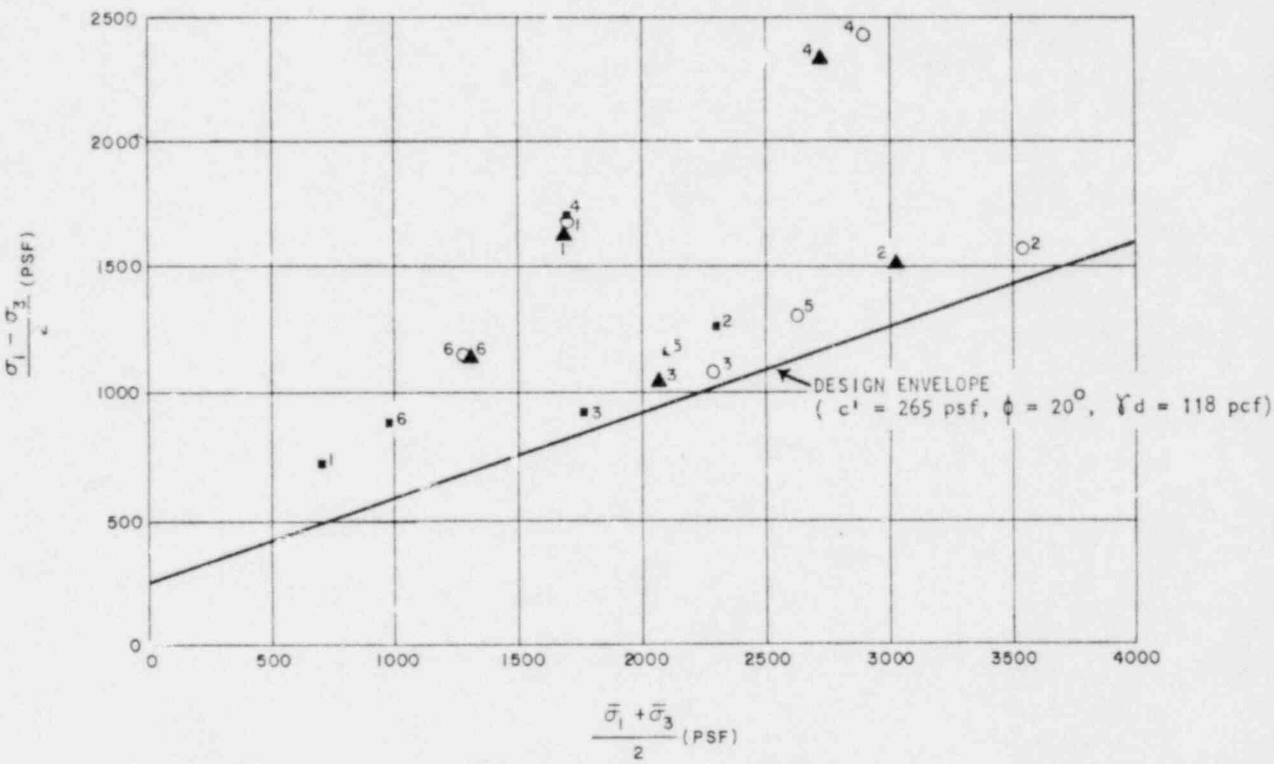
FIGURE 2[illegible].13-2

UHS DAM - FINE RIPRAP BEDDING
STATISTICAL DISTRIBUTION PLOT



KEY:

3 — INDICATES TEST NUMBER

STRESS CONDITIONS AT:

▲ 10% STRAIN

○ MAX $\sigma_1 - \sigma_3$

▪ MAX $\bar{\sigma}_1 / \bar{\sigma}_3$

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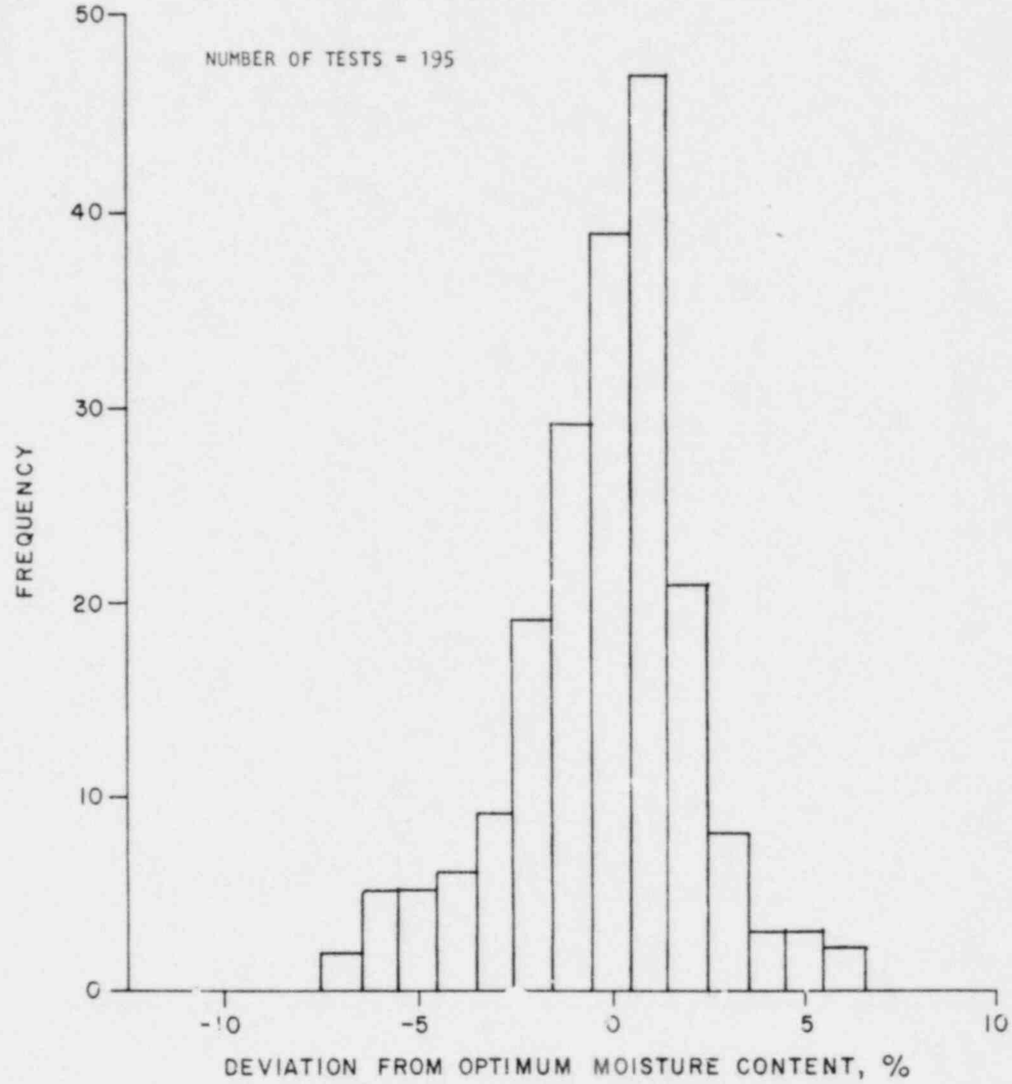
FIGURE 241.13-3

CONSOLIDATED-UNDRAINED
TRIAXIAL TEST RESULTS
ULTIMATE HEAT SINK DAM



7699-064-07

NOTES:

1. RETESTS ON AREAS WHERE TESTS DID NOT MEET COMPACTION CRITERIA ARE INCLUDED IN THIS PLOT.

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FIGURE 241.13-4
UHS DAM
STATISTICAL DISTRIBUTION PLOT

TABLE 241.13-1

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL TEST DATA ON UHS EMBANKMENT MATERIAL

TEST NUMBER		1		2		3		4		5		6	
BORING		CUHS-1		CUHS-1		CUHS-2		CUHS-2		CUHS-3		CUHS-3	
SAMPLE		2		7		3		6		3		4	
ELEV (FEET)		1966		1954		1963		1956		1963		1960	
INITIAL	w, %	13.9		16.3		18.3		16.4		17.9		23.4	
	γ_d, PCF	121.5		112.7		107.8		113.7		106.1		101.7	
	e_o	0.39		0.49		0.56		0.48		0.59		0.66	
	S	0.96		0.93		0.96		0.94		0.96		1.0	
FINAL	w, %	17.0		21.1		23.2		19.4		23.7		27.9	
	γ_d, PCF	120.1		111.7		106.2		114.8		105.6		100.2	
	e_f	0.40		0.50		0.58		0.46		0.60		0.68	
	S												
BACK PRESSURE (PSI)		48		46		56		64		55		66	
STRAIN RATE (% PER MINUTE)		0.002		0.002		0.002		0.002		0.002		0.002	
STRESS CONDITION		PEAK $\sigma_1-\sigma_3$	MAX. $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX. $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX. $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX. $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX. $\bar{\sigma}_1/\bar{\sigma}_3$	PEAK $\sigma_1-\sigma_3$	MAX. $\bar{\sigma}_1/\bar{\sigma}_3$
TOTAL STRESS	ε, %	11.6	1.59	15.4	5.3	14.3	5.2	15.7	3.6	6.6	2.6	10.7	2.2
	TIME TO FAILURE (MIN)	328	45	462	160	433	158	465	108	198	78	323	65
	σ_3, PSF	576	576	1872	1872	1152	1152	1152	1152	1872	1872	576	576
	$\sigma_1-\sigma_3$	3332	1410	3152	2507	2162	1857	4862	3381	2625	2333	2261	1769
	σ_1, PSF	3908	1986	5024	[illegible]	3314	3009	6014	4533	4496	4205	2837	2345
	$\frac{1}{2}(\sigma_1-\sigma_3)$	1666	705	1576	1254	1081	929	2431	1691	1313	1167	1131	885
	$\frac{1}{2}(\sigma_1+\sigma_3)$	2242	1281	3448	3126	2233	2081	2583	2843	3184	3039	1707	1461
	u, PSF	533	562	-101	821	-58	317	677	1138	547	936	418	475
	A, $u/(\sigma_1-\sigma_3)$	0.16	0.40	-0.03	0.33	-0.03	0.17	0.14	0.34	0.21	0.40	0.18	0.27
EFFECTIVE STRESS	ε, %	11.6	1.54	15.4	5.3	14.3	5.2	15.7	3.6	6.6	2.6	10.7	2.2
	TIME TO FAILURE (MIN)	328	45	462	160	433	158	465	108	198	78	323	65
	$\bar{\sigma}_3$, PSF	43	14	1473	1051	1210	835	475	14	1325	936	158	101
	$\bar{\sigma}_1-\bar{\sigma}_3$	3332	1410	3152	2507	2162	1857	4862	3391	2625	2333	2261	1769
	$\bar{\sigma}_1$, PSF	3375	1424	5125	3558	3372	2692	5337	3395	3949	3269	2419	1870
	$\frac{1}{2}(\bar{\sigma}_1-\bar{\sigma}_3)$	1666	705	1576	1254	1081	929	2431	1691	1313	1167	1131	885
	$\frac{1}{2}(\bar{\sigma}_1+\bar{\sigma}_3)$	1709	719	3549	2305	2291	1764	2906	1705	2637	2103	1289	986
	u, PSF	533	562	-101	821	-58	317	677	1138	547	936	418	475
	A, $u/(\bar{\sigma}_1-\bar{\sigma}_3)$	0.16	0.40	-0.03	0.33	-0.03	0.17	0.14	0.34	0.21	0.40	0.18	0.27
	$\bar{\sigma}_1/\bar{\sigma}_3$	78.5	102	2.6	3.4	2.78	3.22	11.2	24.3	3.0	3.5	15.3	18.5
LL													
PL													
PI													
SOIL CLASS													

Q241.14 Identify the local and federal agencies that have regulatory authority over the main dam, and the license or permit number(s); provide a brief description of the safety inspection program required and confirm your commitment to meet these requirements.

R241.14 The following description from Section 12.1.2.3 of the Wolf Creek Environmental Report (OLS) describes regulatory authority over the main dam:

> In compliance with the provisions of Kansas statutes KSA 82a-301 to 305 "regulating the placing of dams and other obstructions in streams and the making of changes in the course, current or cross-section of streams within the state .." KG&E has submitted appropriate applications and has received permits for all applicable structures for the construction of WCGS.
>
> The Division of Water Resources (Kansas Department of Agriculture) is responsible for the inspection of various structures in accordance with the provisions of the National Dam Safety Act. Representatives of the Division of Water Resources have been contacted to inspect the foundations of the Wolf Creek cooling lake dam and related structures after excavation was completed. Approval for each area excavated was granted before the construction and backfilling of the structure was undertaken.

The initial safety inspection of the cooling lake main dam will be performed by the Kansas Division of Water Resources and/or The U.S. Army Corps of Engineers following filling of the cooling lake. However, KG&E has initiated a periodic inspection program including an initial completion inspection, and a periodic inspection program to be performed during and following filling of the cooling lake. This inspection program will include those facilities required by Regulatory Guide 1.127 and also the main dam embankment and the saddle dams.

The inspection program will include: a complete visual inspection of the dam and the erosion protection (riprap and vegetative cover); inspection of the downstream area for seepage, wet areas and boils; periodic monitoring of vertical and horizontal movements; and observation of the water

R241.14 (continued)

levels in the installed piezometers. The frequency of the inspection is monthly during the filling of the cooling lake. After filling and during the inservice period, all performance instrumentation will be monitored monthly except for the horizontal movement surveys which will be performed on an annual basis. The visual inspection will be performed annually for the first four years, every two years for the next four years, and every five years thereafter. In addition, complete inspections will be performed following drawdown in excess of five feet and refilling to normal pool elevation.