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Woodward-Clyde Consultants

May 15, 1979

Stearns-Roger, Inc.
P.O. Box 5888
Denver, Colorado 80217

Attention: Mr. John R. Stryker

Re: Additional Geotechnical Services, Bokum Resources, Canyon de Santa Rosa Tailing Dam near Marquez, New Mexico.
S-R No. 7007 C19620
WCC Job No. 19568-18971

Gentlemen:

As requested, we have prepared the following geotechnical information for the Diversion Ditch, Diversion Dam and the Seepage Evaporation Dam.

CREEP ANALYSIS AND STABILITY OF MANCOS SHALE, DIVERSION DITCH

Discussion of Creep

The occurrence of "Creep" or "Progressive Failure" is a well known time-dependent phenomenon related to the failure of slopes constructed or eroded in natural, overconsolidated, plastic clays and plastic-clay shales due to strain softening. Overconsolidation of these materials is caused by consolidation and formation of structural bonds under high overburden pressures at some past geologic period and the later reduction of loads by geologic processes to the present lighter load conditions. Experience from slides in overconsolidated plastic clays and clay shales has shown that the average shear stresses along the failure surfaces at large strains are much smaller than the peak shear stresses. The remaining shear stresses after sliding failures have occurred have been termed "residual stresses" ($\emptyset_r$) in geotechnical literature. The so-called "residual stresses" seem to be dependent mostly on the size, shape and mineralogical composition of the constituent particles.

Mancos Shale Materials

The Mancos Shale in the vicinity of the Bokum Resources Diversion Ditch and Tailing Dam is a silty, lean clay having sand and silt contents which average about 50. Liquid limits range from about 32 to 40, averaging 37 percent and the colloid contents (-0.002 mm size) range from about 25 percent to 39 percent, averaging about 30 percent. The shale is considered to be overconsolidated, based on high dry densities varying from about 125 to 142 pounds per cubic foot.

Because of the high sand-silt content, only moderate plasticity and lack of significant amounts of montmorillonite minerals, we do not consider the Mancos Shale at this site to be particularly susceptible to "creep" or "progressive failure" when slopes are constructed or eroded into the formation. Shale related landslides were not noted in the area and eroded channels have steep sides. For these reasons, we did not examine this type of failure previously.

However, at recent meetings with regulatory agencies the question of "creep" failures was posed and we were asked to evaluate such possibilities. We have now performed analyses required for such an evaluation. We evaluated the stability of the shale cuts for the Diversion Ditch by stability analyses utilizing so-called "residual stress" data obtained from our laboratory triaxial shear tests. We also compared our analyses and strength parameters with those obtained during a comprehensive study by U.S. Army Engineering Nuclear Cratering Group of plastic shale slope failures in the Missouri River Basin and Colorado.As shown on Figure 3, this shale is somewhat similar, from a plasticity standpoint, to the more silty and less plastic portion of the Dawson Shale formation at Chatfield Dam near Denver, being borderline between Dawson Lean-Clay Shale and Dawson Siltstone. It is much less plastic than the Ft. Union Shale of the Missouri River Basin.

Several shale samples from Test Holes A,C,F,H,I and J were tested in our laboratory (See Figure 1, Location of Test Holes and Tailing Dam, and Figure 2, Summary Logs of Test Holes). These test holes were sampled with core samples in connection with shale permeability determinations reported to you in our letter dated April 18, 1979. All of our additional laboratory test data is summarized in Table No. 1. The results of mineralogical examinations are given in Table II. Complete gradation data is given in Appendix A.

Residual Strength

In an effort to evaluate the after sliding-failure strength of the Mancos Shale, we performed triaxial shear tests on four shale specimens for Test No. 1 and 3 specimens for Test No. 2. Inasmuch as this strength represents the strength of a soil material after failure and remolding along the failure plane, we selected cores from Test Hole F at depth 54.9 to 57.0 ft. for Test No. 1 and at depth 51.0 to 52.4 ft. for Test No. 2, crushed the material and remolded the material to form specimens at near natural density and moisture conditions. The triaxial shear tests were performed with lateral chamber pressures of 25, 50 and 100 psi, and were conducted past the peak failure stresses to strains of about 15 to 20 percent. This provided stress-strain curves up to the peak stresses and then down to minimum stresses with continuing strains. These latter values were interpreted as after sliding-failure stresses. The stress-strain data are recorded on Figure B-1 and B-3, Appendix B. Good shear planes were obtained with definite slickensides being developed along the planes.

Mohr envelopes for limiting shear strengths were plotted for peak and final stress conditions as shown on Figures B-2 and B-4, Appendix B. The results were as follows:

Stress Conditions	Test No.	Friction Angle	Cohesion
Peak (Ø' and C')	1	31°	108 psi
	2	29°	126 psi
After sliding-failure ($Ø_r$ and C'_r)	1	21°	33 psi
	2	27°	9 psi

In an effort to compare the after sliding-failure friction angles, which we determined, with angles which have been determined for other shales, we made a comparison with reported results of the U.S. Army Engineers, previously referenced. The Dawson Shale characteristics appeared to be closest to the Marquez Mancos Shale, with the Ft. Union Shale being somewhat more plastic. The relationship they established for $\emptyset_r$ vs liquid limit is shown on Figure 4(A) and $\emptyset_r$ vs colloid content is shown on Figure 4(B). On these plots, we plotted the results of our tests, which fell within very reasonable positions. Based on both of these plots, their $\emptyset_r$ angles which they termed "residual friction" appear to be from about 20 to 21 degrees, which checks our determined after sliding-failure values from shear test No. 1 almost exactly.

Stability of Diversion Ditch Section

For our studies of the stability of the Diversion Ditch, we selected the section at Station 16+50, which we considered to be the most critical. We performed wedge type stability analyses for both the uphill and downhill slopes with the assumed erosion channel shown on your drawing 08-2-41. These analyses were performed for residual friction parameters ($\emptyset'$) of 5°, 10° and 20°, with no credit being given to any cohesion ($C' = 0$). The theoretical safety factors for static conditions using $\emptyset = 20°$ for the uphill and downhill slopes were 1.7 and 1.5, respectively, and 1.2 and 1.1 for the uphill and downhill slopes, respectively, for the dynamic state assuming an earthquake with 0.1g acceleration. The $C' = 0$ approach is often used for "residual" strength slope analyses. This is considered as a very conservative approach because some cohesion, adhesion and/or particle interlocking would normally be present even after movements and particularly so when safety factors were such that movements do not take place. The study section and results of our stability analyses are shown on Figure 5.

We made a comparison of the most critical downstream slope of the Diversion Ditch in Mancos Shale to data derived by the U.S. Army Engineers from slide observations for the more plastic Ft. Union Shale at Garrison Dam. Their design curves are shown on Figure 6(A). If we utilize these slope and slope height design curves, the indicated safety factors for the sliding surfaces selected would compute to be about as shown on Figure 6(B). This figure shows that theoretically the safety factors of the slopes from 1.18:1 to vertical would vary from 1.5 to some value greater than 1.0, respectively. It should be realized that this type of comparison is purely empirical and is only provided as a second "ballpark" evaluation of somewhat similar clay shale slopes. We included this evaluation to provide some additional relative information on the longtime stability of shale slopes.

Based on our test data and observations, we still do not believe that the silty, lean clay, Mancos shale cuts for the Diversion Ditch will be subject to the "creep" phenomenon. However, our analyses indicated safety factors within good engineering practice when we assumed after sliding-failure frictional strength and zero cohesion conditions.

STABILITY ANALYSES DIVERSION DAM

Analyses were made of the stability of the Diversion Dam in the vicinity of Canyon de Marquez. The section analyzed is shown on Figure 7 and represents Section A-A' shown on your Drawing No. 08-2-38. The soil property parameters used in these analyses were based on determinations made during previous studies reported in our report Geotechnical Services Tailing Dam, Bokum Resources near Marquez, New Mexico dated April 1978 (Job No. 18971). We used the values of 125 pcf wet weight, cohesion = 500 pcf and friction angle Ø' = 18°, which are very conservative. Our static and pseudo static circular arc stability analyses were performed on a computer using the Applied Geodata Systems, Inc. LEASE program after the Modified Bishop Method.

The soil parameters used and the analyses are shown on Figure 7. Based on the conservative parameters and conditions selected the minimum theoretical safety factors computed were 1.8 and 1.4 for the upstream and downstream slopes, respectively, for the static condition and 1.3 and 1.1 for the upstream and downstream slopes, respectively, for the dynamic state assuming an earthquake with 0.1g acceleration.

STABILITY ANALYSES SEEPAGE EVAPORATION DAM

An analysis was made of the stability of the most critical downstream slope of the Seepage Evaporation Dam, at maximum section. The section analyzed is shown on Figure 8. This section was developed from your Drawings Nos. 08-2-23 and 08-2-26 for the Seepage Evaporation Pond. The soil parameters and circular arc stability analyses used were the same as described above for the Tailing Dam and Diversion Dam.

The soil parameters and analysis are shown on Figure 8.

Based on the conservative parameters and the conditions selected the theoretical safety factors computed were 2.0 for the downstream slope under static conditions and 1.4 for the dynamic state, assuming an earthquake with 0.1g acceleration. The theoretical safety factors for the upstream slope were 5.6 for the static state and 3.1 for the dynamic state.

If you have questions on the above information, please call.

Yours truly,

Frank J. Holliday

Frank J. Holliday
Vice President

REGISTERED PROFESSIONAL ENGINEER
5376
NEW MEXICO
FRANK J. HOLLIDAY

WGH:et

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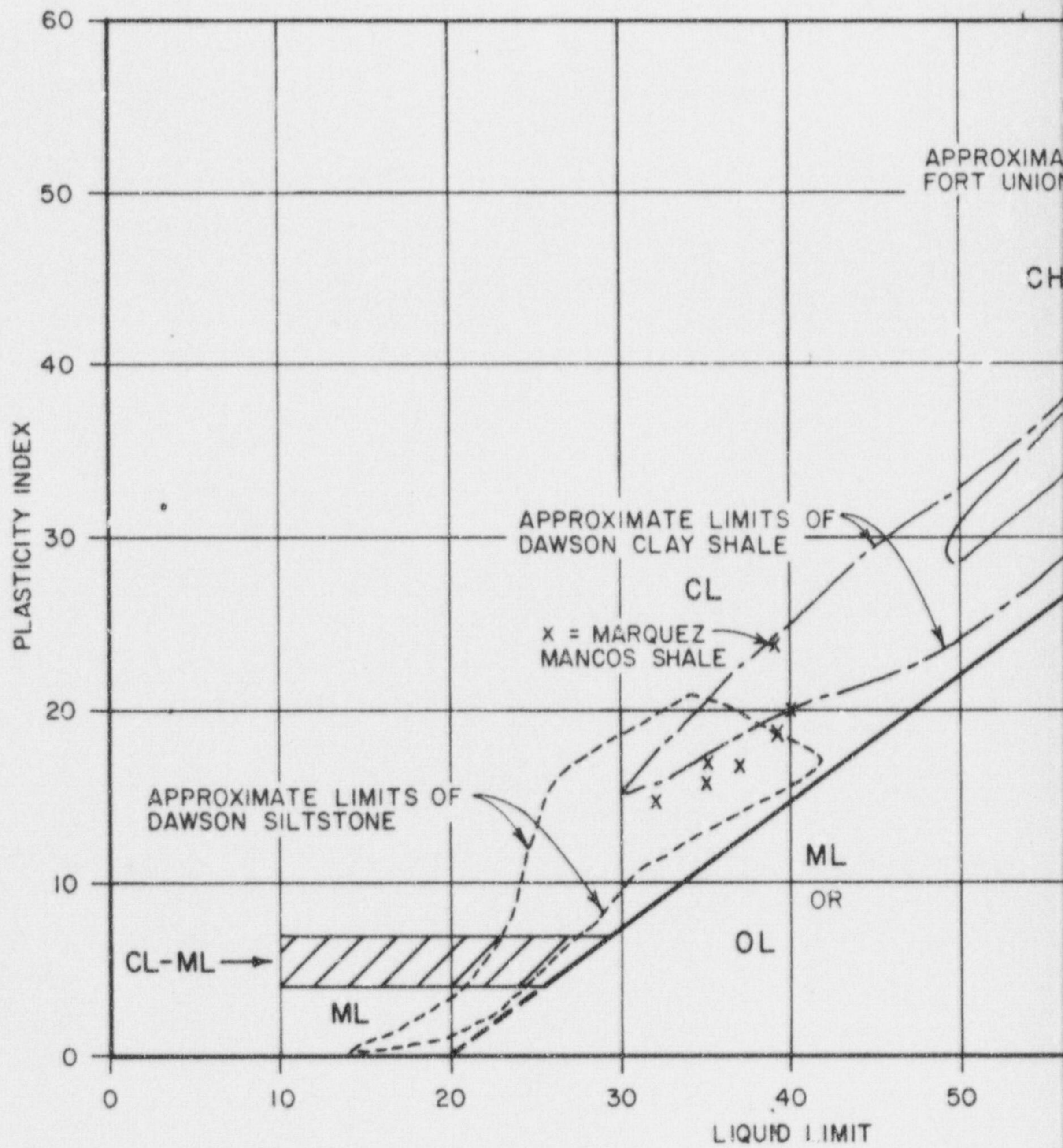

APPROXIMA
FORT UNION
CH
APPROXIMATE LIMITS OF
DAWSON CLAY SHALE
CL
x = MARQUEZ
MANCOS SHALE
APPROXIMATE LIMITS OF
DAWSON SILTSTONE
ML
OR
OL
CL-ML
ML
PLASTICITY INDEX
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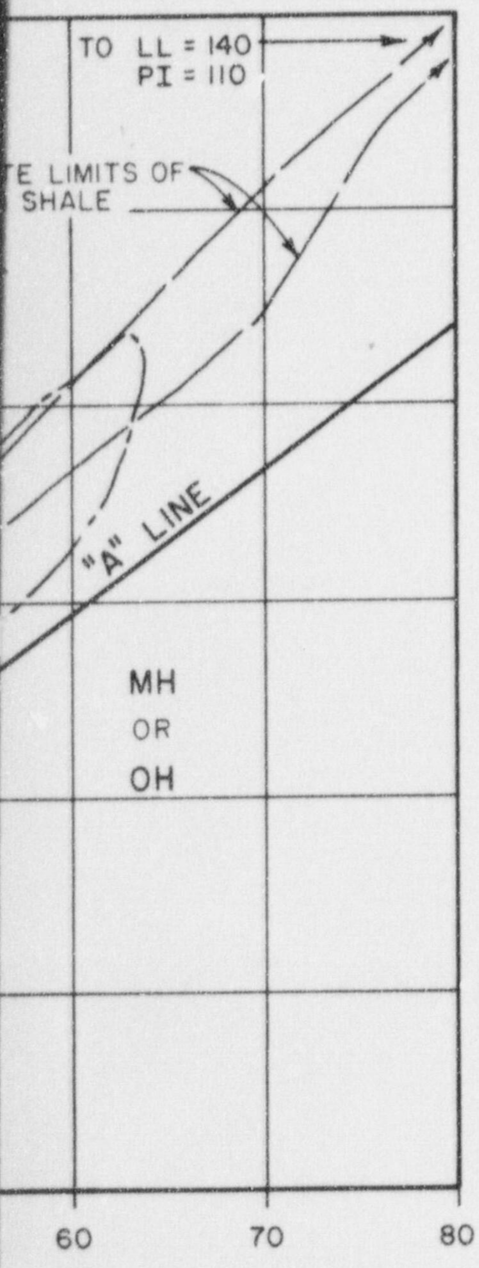

NOTE: DATA FOR FORT UNION AND DAWSON SHALES OBTAINED FROM USED NCG TECH. REPORT NO. 15, 1970.

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PLASTICITY CHART

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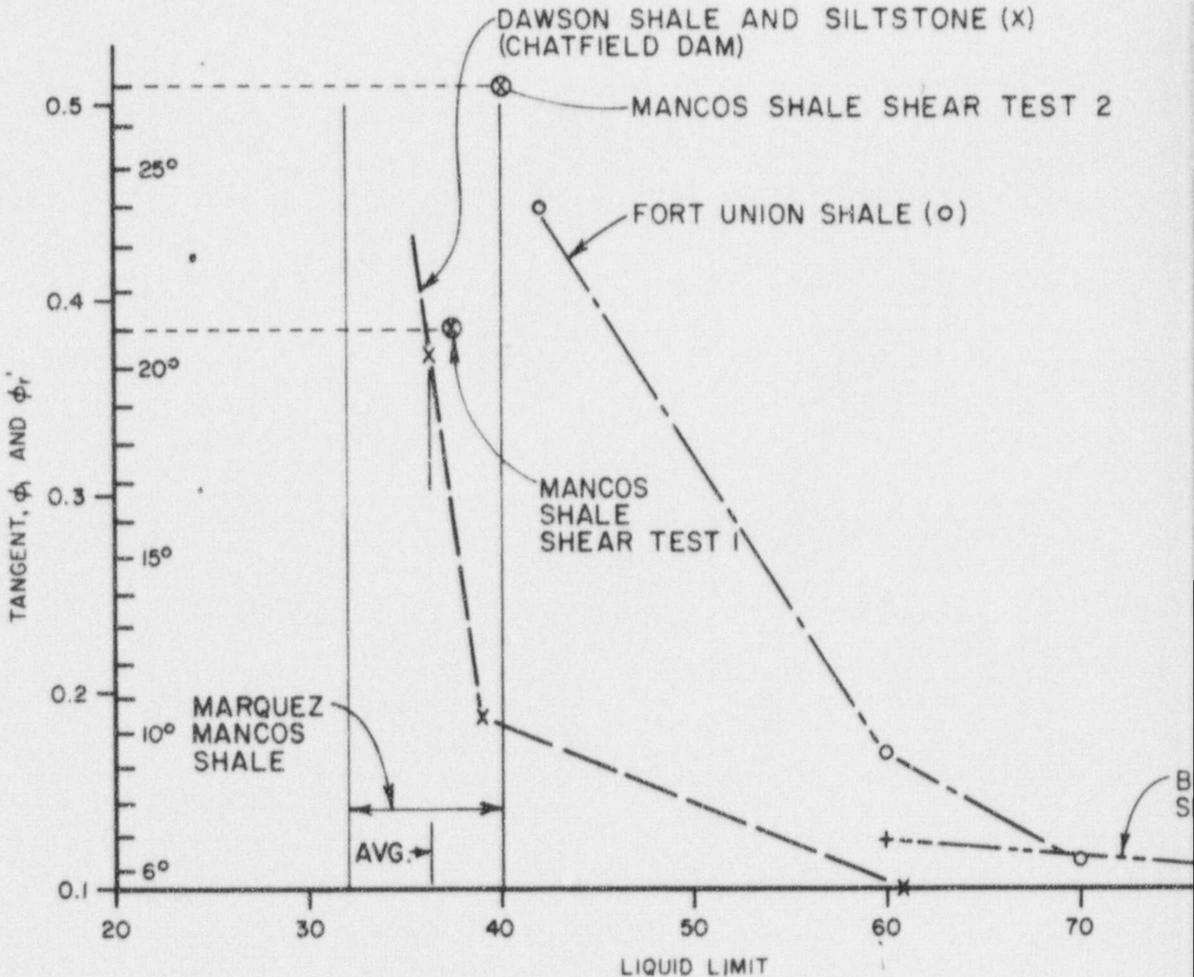
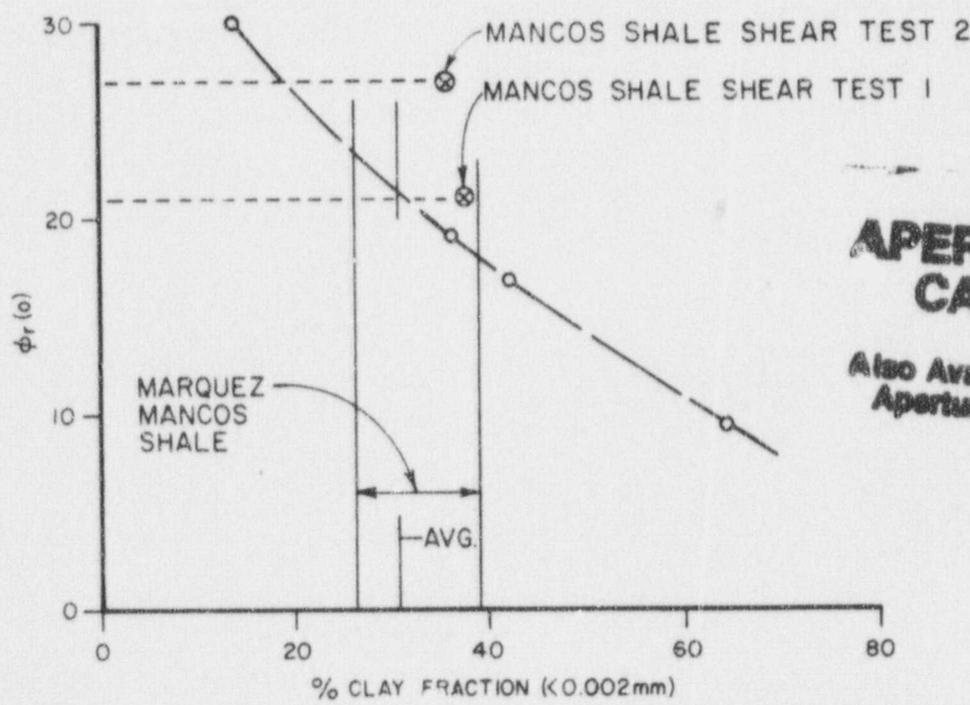
FIG. 3



ϕ_r vs. LIQUID LIMIT
(USAE NCG Technical Report No. 15, 1970)



ϕ_r vs. COLLOID CLAY FRACTION
(Skempton, 1964, USAE No. 15, 1970)

ARPAW
ALE (+)

80

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BOKUM RESOURCES
MARQUEZ TAILING DEPOSIT
SHEAR STRENGTH COMPARISONS

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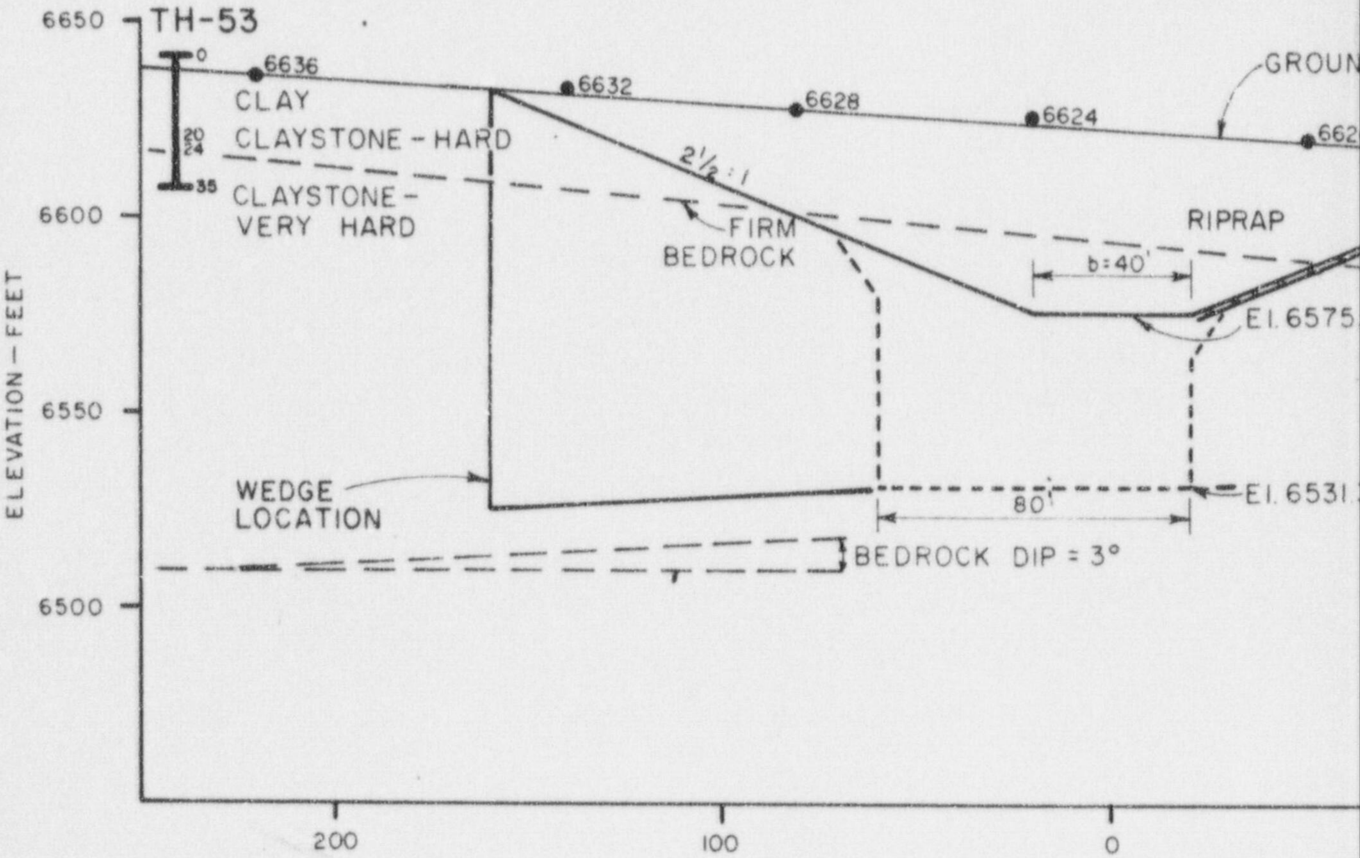
FIG. 4

LEFT BANK
WEDGE LOCATION FOR LOWEST
FACTOR OF SAFETY CONSIDERING
5 WEDGES

F.S. STATIC	F.S. SEISMIC	ϕ	C
0.4	0.3	5°	0
0.9	0.6	10°	0
1.7	1.2	20°	0

RIGHT BANK
WEDGE LOCATI
FACTOR OF SA
4 WEDGES

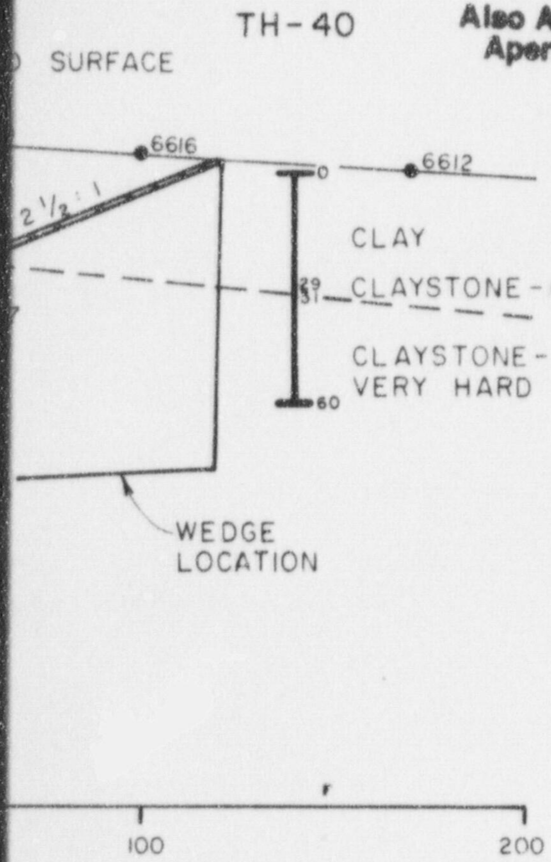
F.S. STATIC	F.S. SEISMIC
0.4	0.3
0.7	0.5
1.5	1.1



STA. 26 + 50 DIVERSION DITCH

N FOR LOWEST
ETY CONSIDERING

Ø	C
°	0
0°	0
0°	0



LEGEND

AS BUILT SECTION

80 YEAR ERODED SECTION

●6636 INDICATES GROUND SURFACE ELEVATION

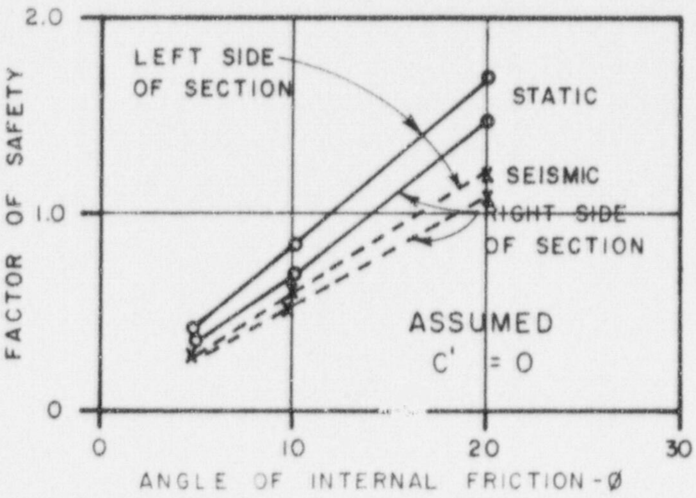
NOTES:

1. EMBANKMENT SECTION BASED ON DRAWINGS 08-2-40 AND 08-2-41 OF THE TAILING DISPOSAL SYSTEM PLANS BY STEARNS-ROGER INC., DATED JUNE 13, 1978.

2. STABILITY ANALYSIS PERFORMED USING THE WEDGE METHOD OF ANALYSIS.

3. LOCATION AND SOIL DEPTH INFORMATION FOR TEST HOLES 40 AND 53 WERE TAKEN FROM A REPORT TO STEARNS-ROGER INC., REPORT NO. 19253-18971.

4. ASSUMED SEISMIC FORCE = 0.1 g.



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BOKUM RESOURCES
STUDY SECTION
DIVERSION DITCH

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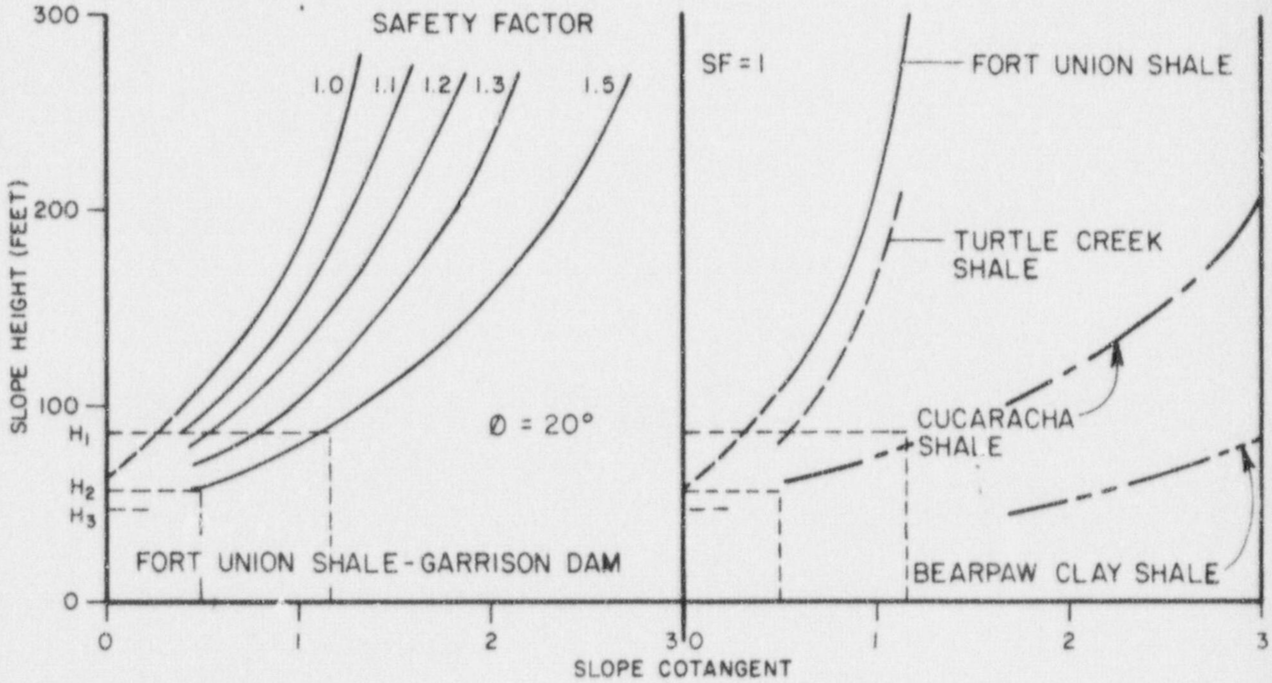
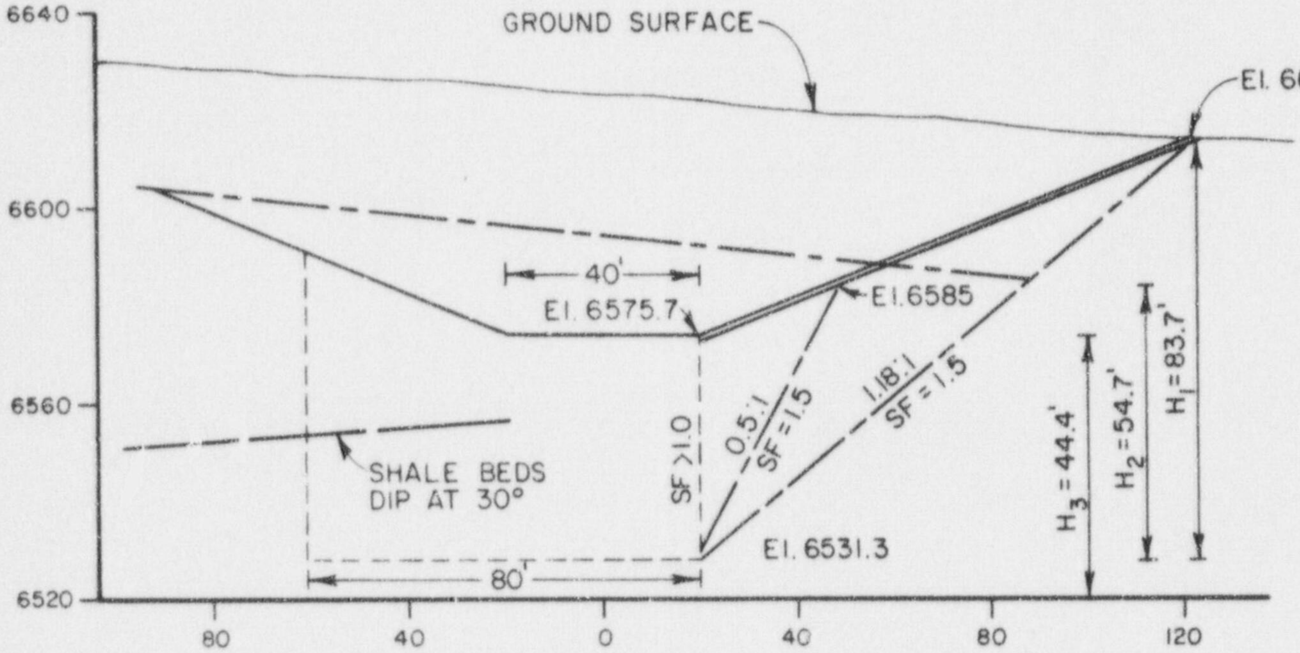
FIG. 5



SLOPE DESIGN CURVES
USAE , NCG Technical Report No. 15, 1970)



NOTE: SAFETY FACTORS SHOWN BASED ON FORT UNION SHALE DATA AT LEFT.

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STABILITY DIVERSION DITCH
STATION 26+50
USAE SLOPE DESIGN DATA

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FIG. 6

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ROCKY MOUNTAIN REGION

MARQUEZ MANCOS SHALE

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS			Deviator Stress Peak(psi)	TRIAXIAL SHEAR TESTS		Friction Angle		Cohesion (psi)		Colloids -0.002mm	Specific Gravity	Saturation %	Soil Type
				LIQUID LIMIT (%)	PI %	SL %		DEVIATOR STRESS Res(psi)	CONFINING PRESSURE (PSI)	Peak	Res	Peak	Res				
A	48.8-49.8			35	16	15			(Applied)					25			CL
C	33.6-34.1			37	17	15								28			CL
F	54.0-54.9			40	20	12								32			CL
H	46.6-47.8			39	24	16								24			CL
I	62.5-63.8			32	15	14								28			CL
J	56.0-56.5			35	17	12								37			CL
Average				36	22	14								29			
A	48.2	11.6	127.1												(2.78)	87	CL
C	34.6	10.4	129.9													86	CL
F	56.8	9.3	142.3													(100)	CL
I	66.2	8.0	139.9													91	CL
J	55.5	8.8	140.0													(100)	CL
Average		9.6	135.8													95	CL
*F (1)	54.9-57.0	8.5	130.9				575	230	100	(Shear Test No. 1						74	
*F(3-4)	54.9-57.0	8.5	130.0	37	19	12	507	146	50	31	21	108	33	39	2.77	78	CL
*F (2)	54.9-57.0	9.2	129.6				410	140	25							77	
*F (1)	51.0-52.4	8.7	130.2				586	232	100	(Shear Test No. 2)						72	
*F (2)	51.0-52.4	8.8	131.1	40	20	13	566	105	50	29	27	126	9	34	2.80	73	CL
*F (3)	51.0-52.4	9.3	130.1				444	94	25							78	
H	47.0-47.8	9.3	124.7	(Natural Core)			345		0	(Unconfined Test No.2)					(2.78)	66	CL
F	49.9-51.0	10.7	129.1	(Natural Core)			428	--	0	(Unconfined Test No.1)					(2.78)	86	CL

*Remolded

TABLE II

MINERAL ANALYSES OF MARQUEZ MANCOS SHALE

Minerals	Sample No.: Test Hole Depth:	Sample 1 A 45.0-45.5	Sample 2 J 55.2-55.7	Sample 3 H 54.95-65.65
Quartz (%±5)		45%	45%	45%
Kaolinite (%±3)		15%	15%	20%
Calcite (%±3)		20%	25%	15%
Illite (%±2)		12%	8%	10%
Feldspar (%±2)		5%	5%	5%
Rutile (TiO_2)(%±1)		1%	ND	3%
Iron Compounds (%±1)		2%	2%	2%
Montmorillonite		ND	ND	ND

Tests Performed by Vladimir E. Wolkodoff, P.E.

Note: ND = Not Dedectable

APPENDIX A

GRADATION DATA

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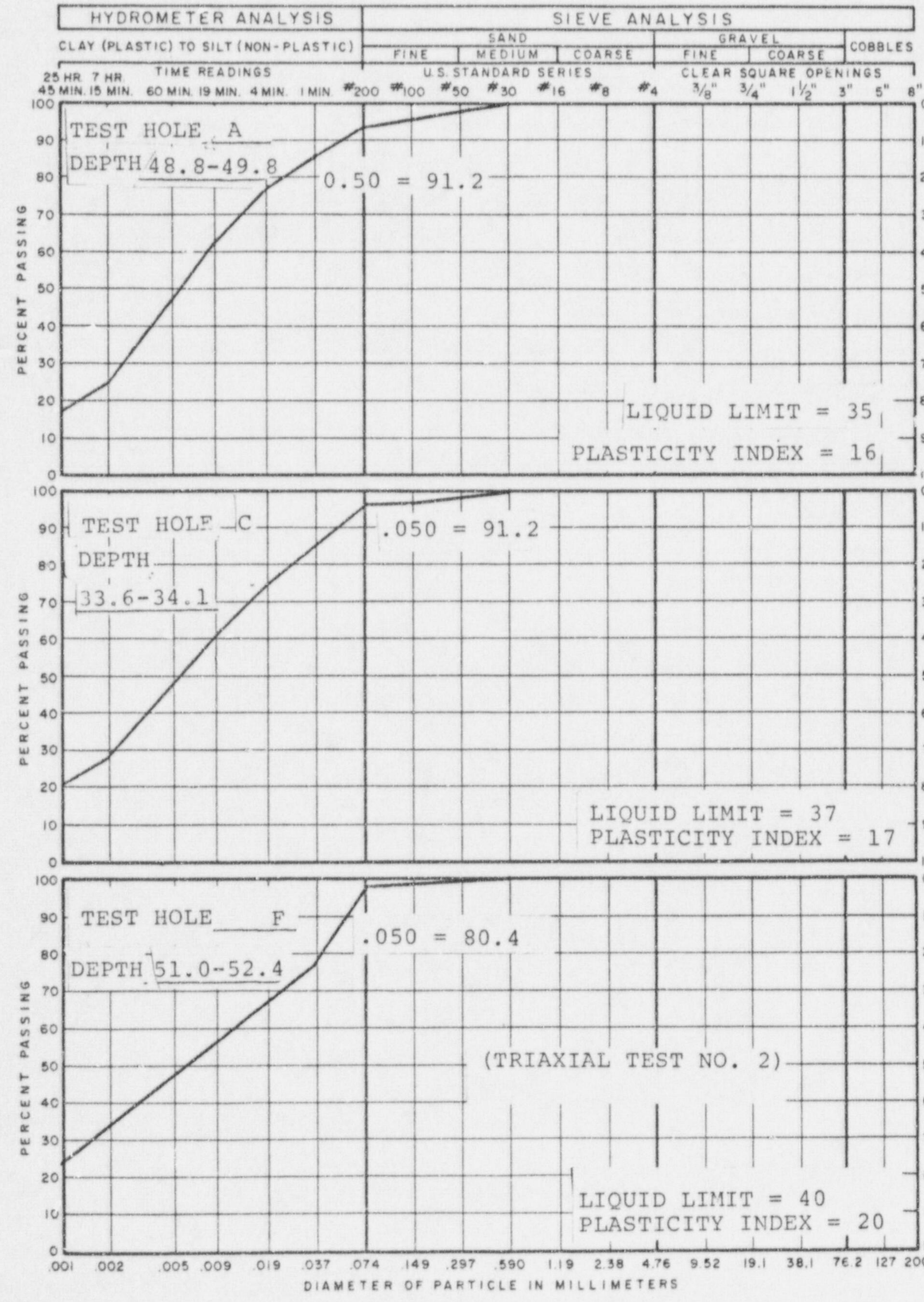
ROCKY MOUNTAIN REGION



GRADATION ANALYSIS

JOB NO. 19568-18971

FIG. A-1

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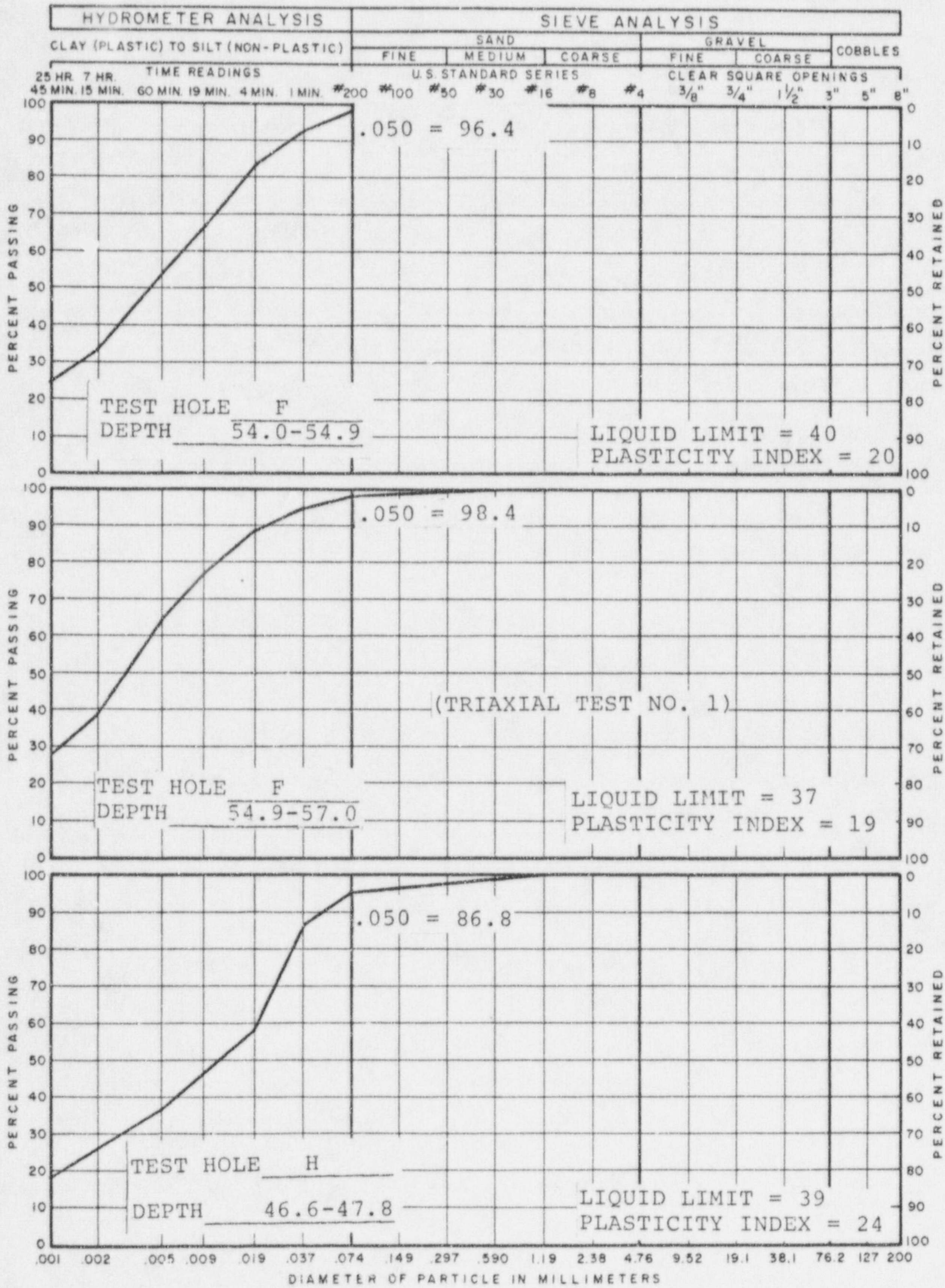

GRADATION ANALYSIS

JOB NO. 19568-18971

FIG. A-2

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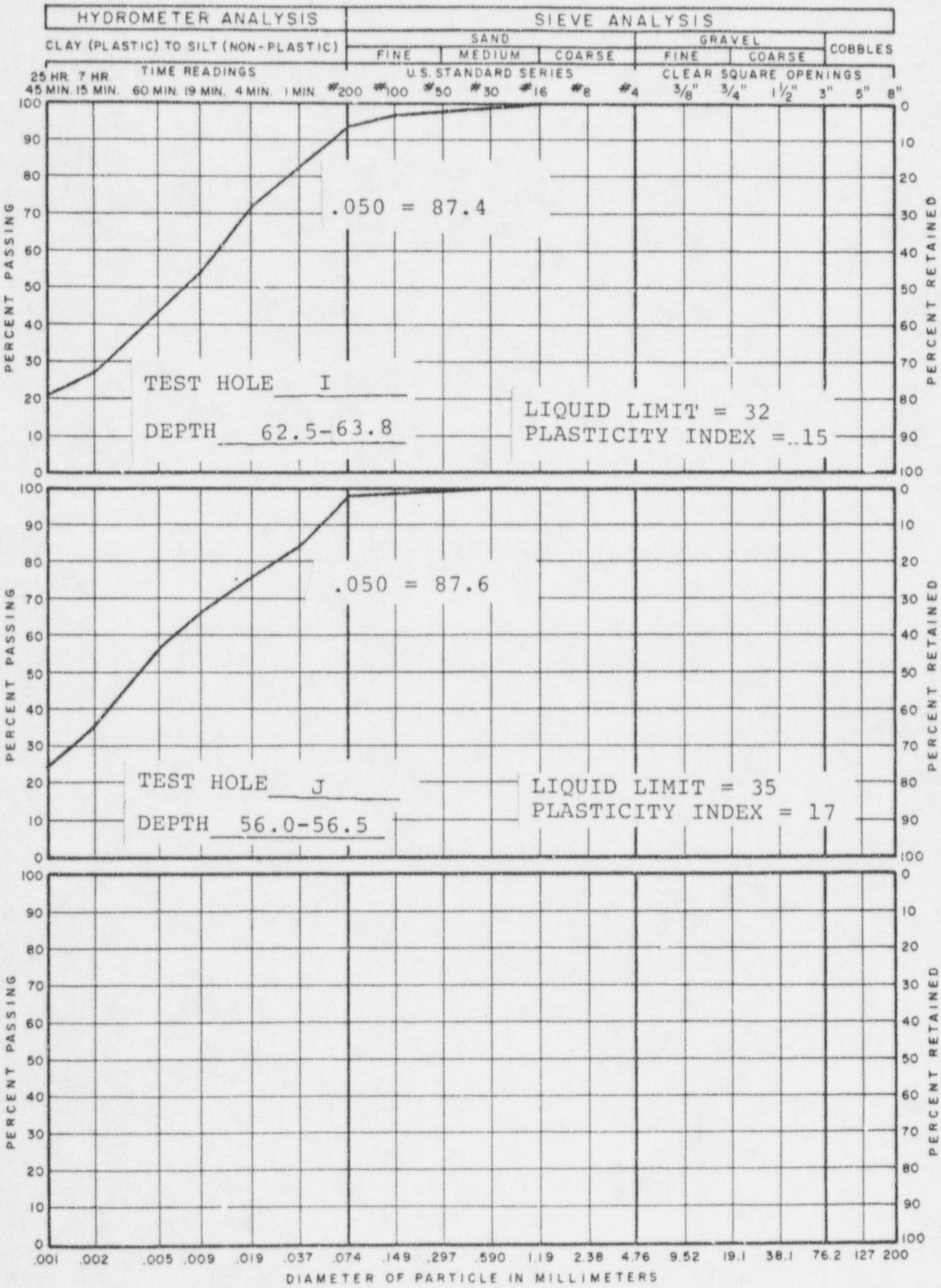


GRADATION ANALYSIS

JOB NO. 19568-18971

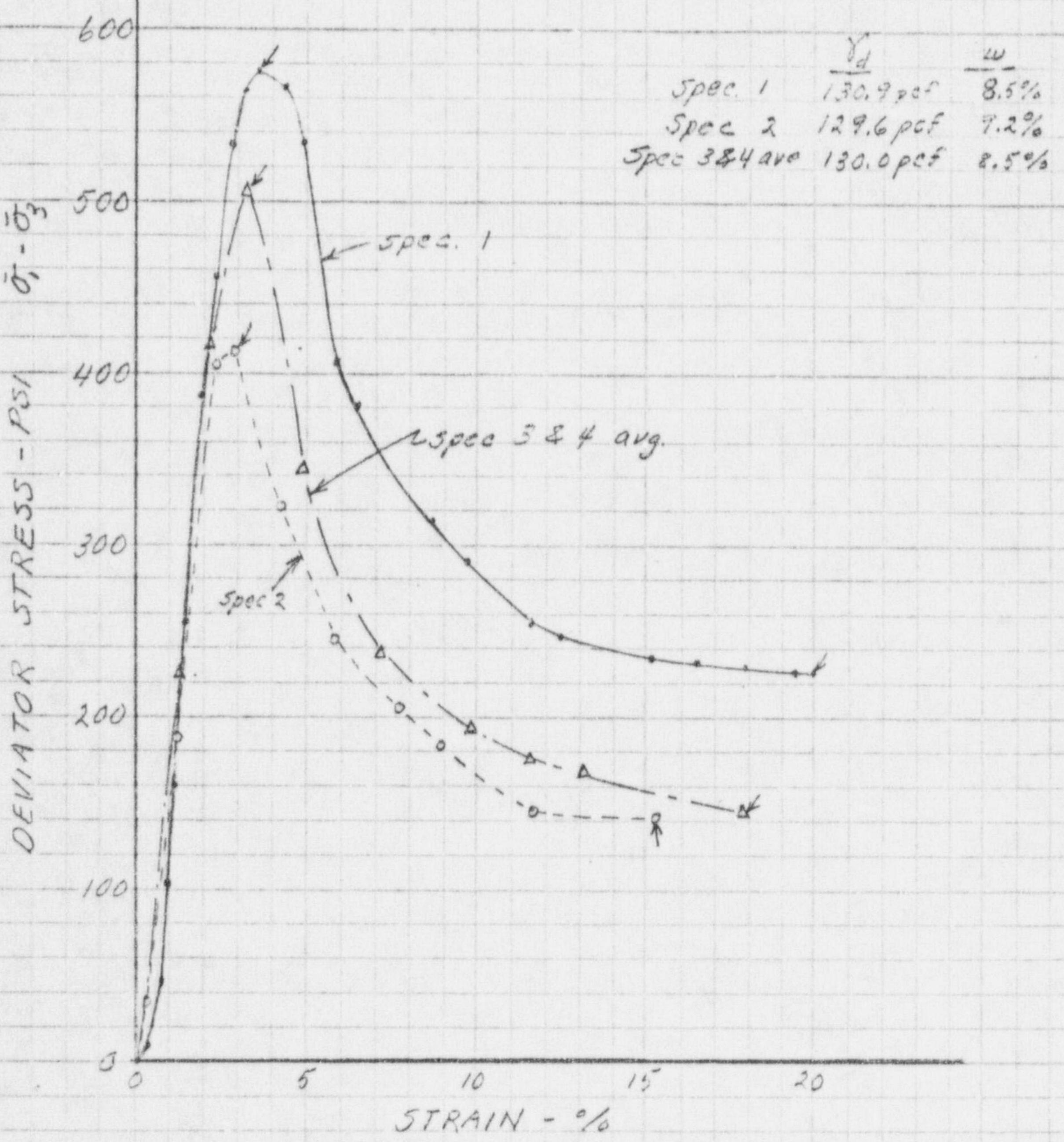
FIG. A-3

APPENDIX B

TRIAXIAL SHEAR TEST DATA

Woodward-Clyde Consultants

Problem
History
Given
Assumptions
Reasoning
Solution
Conclusions



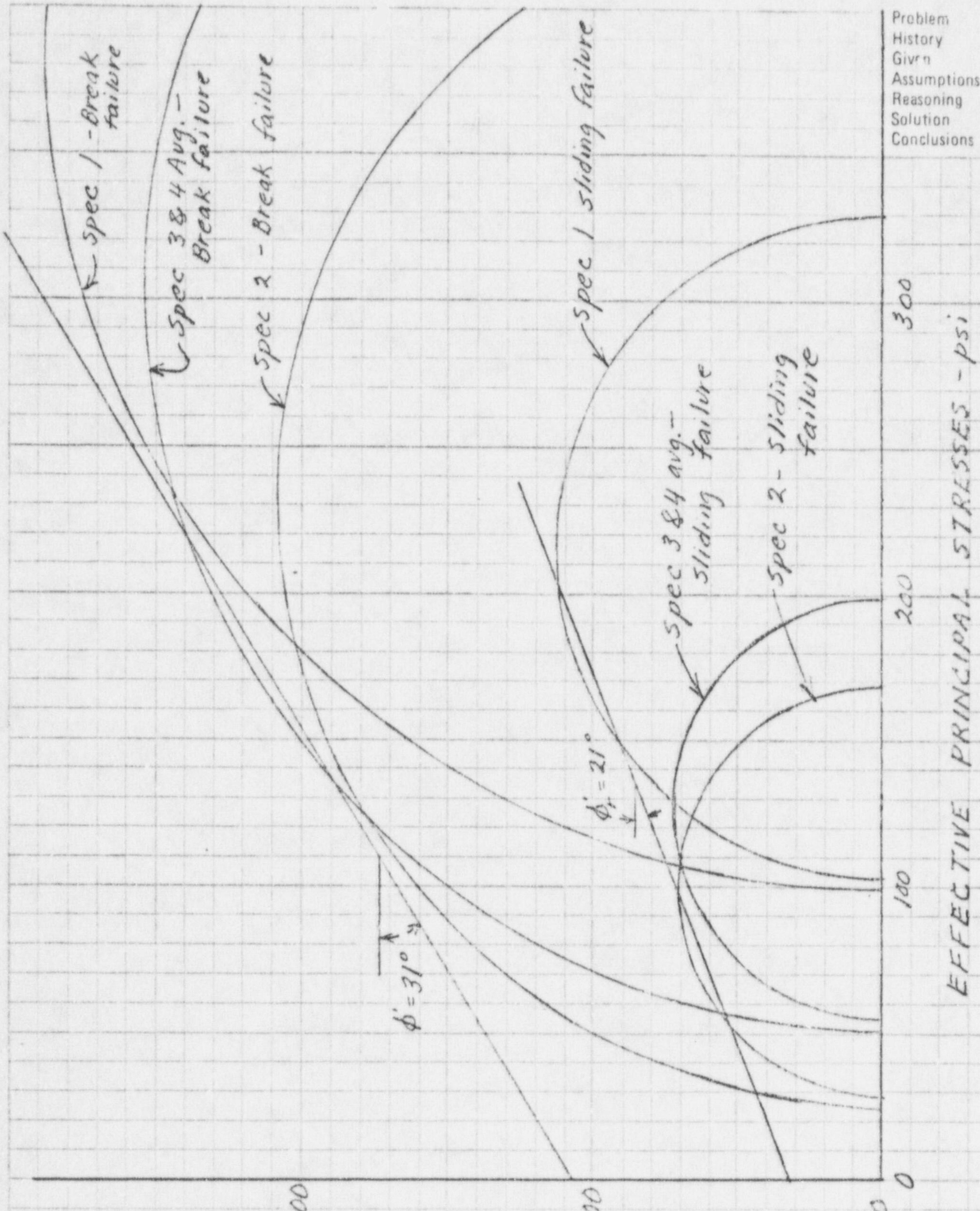
JOB *BOKUM RESOURCES* CALC: *Triaxial Shear Test 1 - Hole F - 54.9'-57.0'* SHT. ____ OF ____

BY *H.J.G* DATE *5/14/79* CHKD BY *W.G.H.* DATE *5/14/79* JOB NO. *19568 S*

Fig B-1

Woodward-Clyde Consultants

Problem
History
Given
Assumptions
Reasoning
Solution
Conclusions



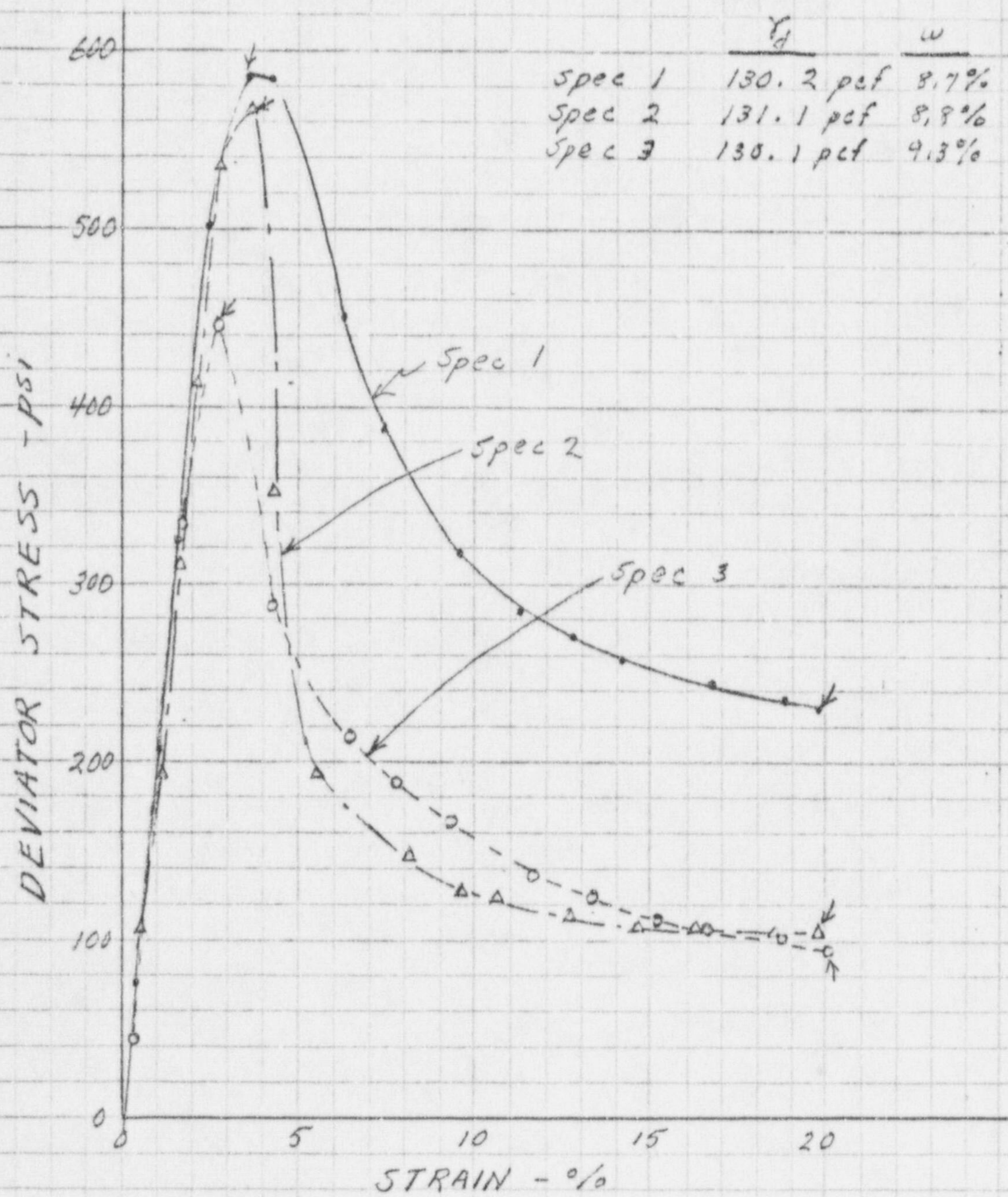
JOB BORUM RESOURCES CALC: Traixial Shear Test 1 · Hole F · 549'-570' SHT. ____ OF ____

BY H.J.G. DATE 5/14 CHKD BY W.G.M. DATE 5/14/79 JOB NO. 19568 S

Fig B-2

Woodward-Clyde Consultants

Problem
History
Given
Assumptions
Reasoning
Solution
Conclusions



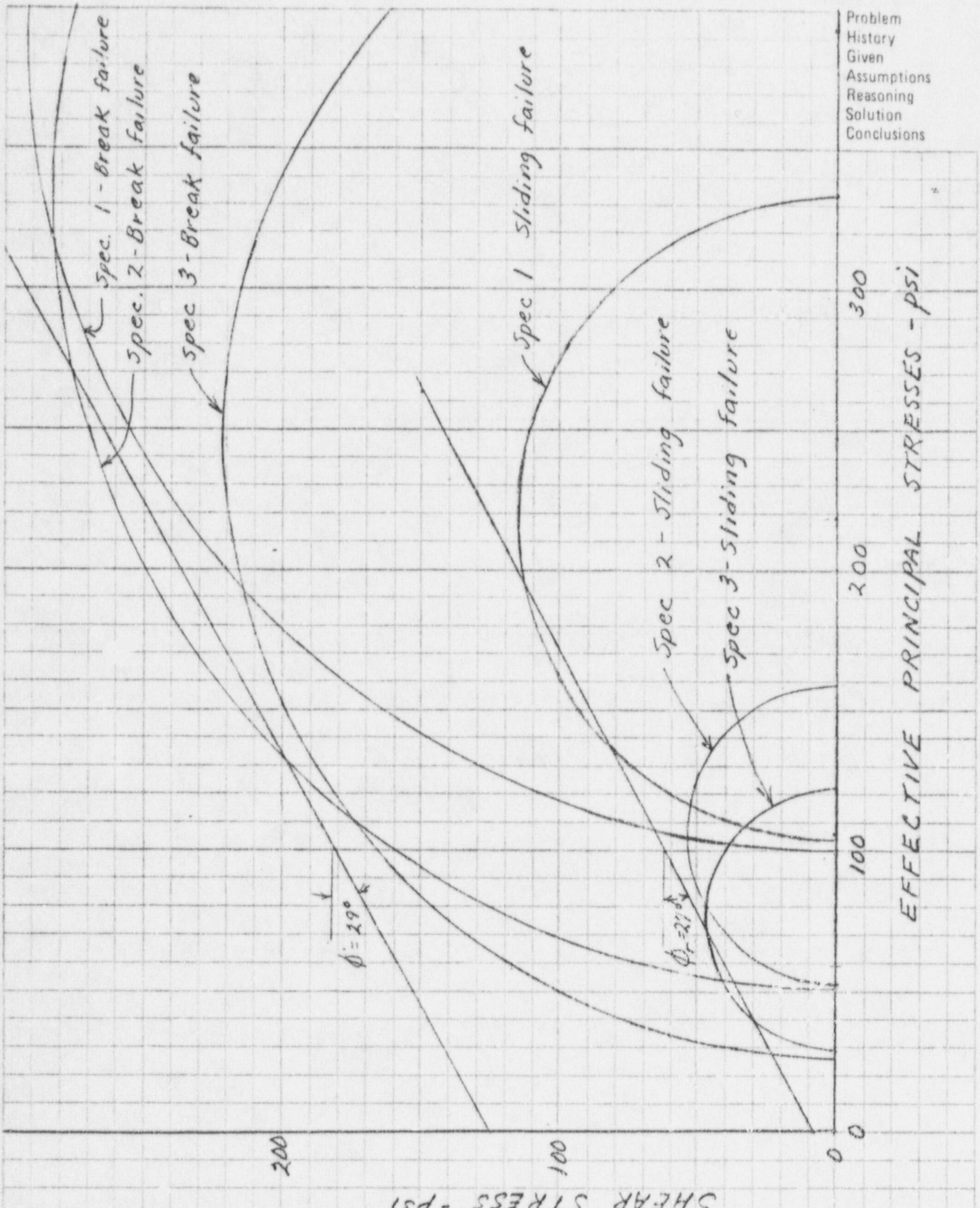
JOB BOKUM CALC: Triaxial Shear Test 2 - Hole F - 51.0'-52.4' SHT. ____ OF ____

BY HJG DATE 5/14/79 CHKD BY WGH DATE 5/15/79 JOB NO. 19568

Fig B-3

Woodward-Clyde Consultants

Problem
History
Given
Assumptions
Reasoning
Solution
Conclusions



JOB BOKUM CALC: Triaxial Shear Test 2 - Hole F - 51.0' - 52.4' SHT. ____ OF ____

BY HJG DATE 5/14/79 CHKD BY WGH DATE 5/15/79 JOB NO. 19568

Fig B-4