GETR LANDSLIDE STABILITY ANALYSIS

Prepared for:

General Electric Company
Vallecitos Nuclear Center
Pleasanton, California 94566

By:

Earth Sciences Associates
701 Welch Road
Palo Alto, California 94304

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GETR LANDSLIDE STABILITY ANALYSIS

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ELEONORE H. JOHNSON
NOTARY PUBLIC-CALIFORNIA
PRINCIPAL OFFICE IN
SANTA CLARA COUNTY
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I. SUMMARY OF CONCLUSIONS

This report presents the results of stability analyses of the landslide complex present in the hills north of the General Electric Test Reactor facility near Pleasanton, California. The conclusions derived from these analyses are summarized as follows:

1. The landslide complex present today on the southern slope of the Vallecitos Hills is an erosional remnant of an ancient slide complex whose principal movement occurred more than 100,000 years ago.
2. Geologic evidence and field observation indicate that the landslide complex is presently stable and has been stable for at least the past 8,000 years.
3. Geologic evidence suggests that the landslide complex has remained stable even under strong seismic loadings.
4. Static stability analyses corroborate the field observation and geologic evidence indicating that the landslide complex is stable under present loading conditions. Factors of safety of 2.9 and 4.4 were calculated for the two modelled failure surfaces analyzed.
5. Seismic stability analyses indicate that the 0.75 g design earthquake postulated for the site could cause downslope movement of the landslide mass, conservatively estimated at 18 cm.
6. The landslide complex poses no hazard to the General Electric Test Reactor. However, a value of 18 cm of movement on the B-1/B-3 shear should be accommodated in the design of the proposed Fuel Flooding System.

II. INTRODUCTION

A. Purpose

An investigation has been conducted of the stability of a landslide complex present in the hills north of the General Electric Test Reactor (GETR) facility near Pleasanton in Alameda County, California. The objective of this investigation was to evaluate and characterize the potential hazard, if any, to the GETR or appurtenant safety-related facilities posed by the presence of the landslide complex. In particular, possible loadings on the proposed Fuel Flooding System (FFS), a portion of which crosses the surface projection of the basal shear of the landslide complex, were evaluated. This study is responsive to the Nuclear Regulatory Commission staff position that an investigation program should be conducted to evaluate the potential landslide hazard at the GETR site (NRC, 1980). The scope, objectives, and proposed methodologies of this investigation were reviewed by the NRC staff prior to the start of the program.

B. Related Studies

The results of several earlier studies related to the issue of landsliding at the GETR site have previously been submitted to the NRC. Geologic evaluations based on field mapping, aerial photo interpretation and some subsurface exploration of both the large-scale, deep-seated landslides and surficial slides were presented in the Phase I and Phase II Geologic Investigation reports for the GETR site (ESA, 1978a, 1979). In another investigation the relative stability of the landslide complex was assessed in terms of geologic, geomorphic, and climatic conditions and stability analyses were performed based on data available at the time (ESA, 1978b). An imdependent evaluation of potential landslide hazard at the site was included as part of the geologic studies conducted by the California Division of Mines and Geology (1979).

C. Investigation Procedure

The investigation of the stability of the landslide complex in the hills north of the GETR included review of geologic evidence, field and laboratory investigations and analysis and interpretation of the data collected.

Extensive geologic studies have been conducted at the GETR site as part of earlier investigations (ESA, 1978a; 1979). The geologic data relative to the structure and stratigraphy of the hills north of the GETR were utilized in formulating a model of the landslide complex for computer stability analyses. The geologic data also provide a useful check on the reasonableness of the results of the stability analyses.

The field exploration performed for this investigation had three main objectives: 1) to establish the stratigraphy at depth within the slope, and, to the extent possible, the location of the landslide failure surfaces; 2) to obtain samples of representative subsurface materials, including shear zone material if possible, for laboratory testing; and 3) to obtain information on the ground water conditions within the slope. During the field investigation four rotary wash borings were drilled along the section to be analyzed. The borings were logged, samples were taken, in-situ penetration tests were performed and piezometers were installed. Geophysical logs were run in the two deepest borings. The results of the field investigation are presented in Appendix A.

Samples acquired during the field investigation were tested in the laboratory to establish index properties and strengths of the subsurface materials. Both static triaxial and direct shear strength tests were performed to provide appropriate data for use in the slope stability analyses. Several triaxial tests were run at high pressures to simulate in-situ conditions at the base of the very deep landslide complex. The test procedures and results are discussed in Appendix B.

Data from the preceding tasks was analyzed and interpreted, and a model for the landslide complex amenable to slope stability analysis was formulated. The stability analyses were performed using a computer program specifically developed for this purpose. The procedures and results of the stability analyses are presented in Appendix C.

A slope monitoring program has been initiated at the site to detect and monitor ground movements of several inches or more which might accompany reactivation of the landslide complex, particularly as a result of seismic loadings from strong earthquakes in the vicinity. Although this monitoring program is not specifically part of the investigations reported herein, it should provide a useful means of checking the results of the stability analyses in the event of future seismic activity in the site vicinity. The survey monitoring program, including the results of the initial survey, was described in a memorandum submitted to the NRC (ESA, 1980).

III. SITE CONDITIONS

A. Geologic Units And Structure

The regional and site geology of the Vallecitos Nuclear Center vicinity have been described in detail in previous reports (ESA, 1978a, b; 1979). Of most interest for this investigation are the character and distribution of geologic units and structures within the Vallecitos hills just north of the GETR. The geologic section in this area consists predominantly of dense gravels with thinner interbedded very stiff to hard silty clays, both units being of the Livermore Gravels, and dipping gently to moderately to the northeast. The generalized distribution of these material types within the hillslope is shown on Figure 2, Geologic Section X-X'. The contacts shown on Figure 2 were interpreted from extensive trench data, geologic mapping and lithologic and geophysical logs of borings. Where boring control was not available contacts were projected to depth based on the dip of the section as exposed in the trenches, in outcrop, and in the Fuel Flooding System (FFS) cut.

Predominantly fine-grained materials were encountered throughout boring RD-1 and the middle portions of borings RD-2 and RD-4, although at least the upper 15-20 feet of RD-1 is known from previous studies to include late Quaternary colluvial deposits and superimposed paleosols derived from the Livermore Gravels (ESA, 1979). As a result, the samples from RD-1 cannot be considered representative of the materials existing down dip within the hills to the north. The fine grained units encountered in RD-2 and RD-4 include units of clayey silt to silty clay in the upper part of the section. The section grades to more plastic fines downward and includes variable amounts of sand and gravel. The interpretation shown on Figure 2 assumes that this generally fine-grained section consists of either material type 1 or material type 3 as defined in Appendix B.

The lower parts of borings RD-2 and RD-4 and of Shannon and Wilson's (1973) borings B-1/B-1A and B-2 all encountered clayey sand and gravel which has been designated as material type 4 as described in Appendix B. This material is interpreted to underlie the fine-grained unit discussed previously to the depths of interest in this study.

An interbedded sequence of fine and coarse grained units was encountered in the upper half of boring RD-3 as shown by the combined lithologic-geophysical logs on Figure A-2. Samples from the fine-grained units have been designated as material types 1 and 2 and are described in detail in Appendix B. For the purposes of this study the entire sequence in the upper part of RD-3 containing fine-grained units was directly correlated with the massive fine-grained unit exposed near the center of Trench G-1 (see Figure 2). Because the lateral extent of the two gravelly units interbedded with fine-grained units in RD-3 was uncertain, these coarse materials were conservatively replaced by fine-grained material in the interpretation shown on Figure 2.

As described in Appendix A, sampling of the coarse-grained unit encountered in the bottom half of boring RD-3 was not possible. However, very detailed log descriptions of the materials present in the downslope portion of Trench G-1 (see ESA, 1979) and direct observation of exposures in the FFS cut pad are available. These data indicate that this section is composed almost entirely of low plasticity silty gravelly sands/sandy gravels. No clayey fine-grained units were observed in the extensive trench and cut exposures, nor are any indicated from interpretation of the gamma or resistivity logs of boring RD-3 (see Figure A-3). For the purposes of this investigation, this section of the slope is conservatively assumed to have the index and strength properties of the clayey sand-gravel designated as material type 4 in Appendix B.

Similarly, samples are not available from the section upslope of RD-3 (see Figure 2). Based on trench log descriptions these units are, for the most part, interpreted to have properties intermediate to those of material types 3 and 4, as described in Appendix B. One fine-grained unit of material types 1 and/or 2 has been interpreted from trench log data within this section as shown on Figure 2.

As described in the Phase I and Phase II geologic reports (ESA, 1978a, and 1979, respectively) several shears have been encountered in exploratory trenches at the GETR site. These include the low-angle, northeast dipping B-1/B-3 and B-2 shears to the southwest and the steep southeast dipping shears (G-6 and G-9) to the northeast (see Figure 2). The B-1/B-3 and high-angle shears are slip surfaces of a large landslide complex on the southwest slope of the Vallecitos hills. The head-scarp, bench, toe geomorphic expression of this large landslide complex is shown by the distinctive amphitheater-like structure near the crest of the hills, the broad, gently sloping spur ridges at mid-slope and the lobate steeper slope at the foot of the hills (see Figures 1 and 2).

The stratigraphic sequence encountered in borings RD-2 and RD-4 was very similar to that penetrated by BH-3 in which several shears had been observed in situ during an earlier investigation, (ESA, 1978a). However, despite concentrating sampling attempts in the locations suggested by BH-3, no shear zone samples were recovered from borings RD-2 or RD-4. Due to the coarse-grained nature of most of the section penetrated by boring RD-3, evidence of the location or character of shears which may have been intercepted could not be interpreted from the samples, drilling progress or geophysical logs of the boring.

B. Ground Water Conditions

Ground water conditions within the hills north of the GETR have been interpreted from geologic conditions, the presence of springs or seeps within the drainages incised in the slope, and piezometer data from the field investigation. The piezometer data from borings RD-2, 3, and 4 provide an upper bound for the location of the main piezometric surface within the hills north of GETR (see Appendix A and Figure 2). If the portion of this surface defined by the boring piezometers is extended down gradient it coincides with the water level contours plotted by Farrar (in press) from well data in Vallecitos Valley. This piezometric surface is also consistent with the presence of springs and seeps in drainage courses adjacent to the spur ridge investigated. The elevations of the springs and seeps are all approximately the same as the elevations of the adjacent portion of the piezometric surface beneath the spur ridge (see Figures 1 and 2). Farrar (in press) points out that runoff on the hillsides in this area is very rapid and minimal infiltration occurs where Livermore Gravels are exposed on the hills. These factors, together with the great depth of the main piezometric surface, suggest that seasonal depth fluctuations of this surface will be small to non-existent. Piezometer readings in borings RD-2 and 4 may indicate the presence of perched ground water above the main piezometric surface (see Piezometer Installation and Monitoring in Appendix A). Perched water conditions were observed in-situ in large diameter bucket auger borings located about two-thirds of a mile east of the GETR (see ESA, 1978a). These borings penetrated approximately the same elevations as RD-2 and RD-4.

IV. SLOPE STABILITY

A. Geologic Evidence

The southern slope of the Vallecitos hills has been interpreted as the erosional remnant of an ancient, massive landslide complex (ESA, 1978a,b,; CDMG, 1979). Geologic evidence exposed in exploratory trenches at the toe of the slide mass indicates that principal movement of greater than 80 feet occurred more than 100,000 years ago (ESA, 1979). Minor additional movements totaling 6 to 12 feet occurred between about 17,000 to 100,000 years ago. There is evidence that no movement has occurred during at least the last 8,000 years. This evidence is compatible with the deep erosional dissection, subdued and modified geomorphic expression and overall gentle slope of the remaining landslide mass.

Geologic interpretation suggests that the landslide complex in the Vallecitos Hills was initially activated during a pluvial epoch when the climate was much wetter and erosional base levels were significantly lower than at present. There is also evidence that the Vallecitos hill front may have been over-steepened by a previous west-northwest flowing drainage. The role of earthquake loadings in the initiation of movement is not known, but may well have been a contributing factor.

The absence of movement during the past 8,000 years and the difference between the climatic and geomorphic conditions existing today and those interpreted to have existed at the time of major movement of the slide complex both attest to the present stability of the remaining landslide mass. Based on estimates of recurrence intervals for large earthquakes on the nearby Calaveras fault, it seems likely that the landslide complex has been subjected to at least several episodes of very strong ground shaking during the past 8,000 years. The fact that there is no evidence of offset during that time indicates that the remaining landslide complex is stable even under strong seismic loadings.

B. Stability Analysis

Even though geologic evidence indicates that the landslide complex in the hills north of the GETR is stable, a seismic stability analysis was performed to estimate the magnitude of permanent deformations which might occur under earthquake loading conditions. The methodologies and results of these stability analyses are described in detail in Appendix C and summarized briefly here.

The basic technique of analysis derives originally from the method of Newmark (1965) in which the landslide mass is considered capable of limited slip whenever the seismic coefficient acting on the mass causes shear stresses to exceed the yield stresses on the failure plane. Knowledge of (a) the seismic coefficient acting on the mass during the design earthquake and (b) the seismic coefficient necessary to cause incipient yielding are required as input. The output of the analysis is net final slip at the end of the earthquake.

To evaluate (a), we have determined that the design M 7.5 earthquake will cause seismic coefficients up to 0.34 to act on the landslide mass. This estimate is conservatively high because it neglects the "tau" effect arising from the great size of the landslide.

To evaluate (b), the coefficient causing yield of the landslide mass, we have determined the yield shear strength of the soils which we infer to lie along the failure plane. The residual undrained or remolded strength values are used, on the basis that the materials are disturbed by past landsliding, and that the disturbance has not "healed". Using these strength values, the seismic coefficient causing incipient movement is determined by trial-and-error stability analysis. The yield seismic coefficient is shown to be 0.18. This estimate is probably low because of the conservative interpretation of the laboratory test results (see Appendix B).

The results of the seismic stability analyses indicate that small movements, up to a maximum of about 18 cm, could occur along the existing landslide failure surfaces as a result of strong ground shaking associated with the most severe earthquake postulated for the site. On the basis of the geologic evidence of no movement in at least 8,000 years it appears that either these estimates of permanent deformation are quite conservative or the GETR site has not been subjected to seismic loadings equivalent to the design seismic event during the past 8,000 or more years.

Based on the results of the stability analyses it is our judgement that the landslide complex poses no hazard to the GETR. The analysis indicates that a value of 18 cm of movement of the landslide complex along the B-1/B-3 shear should be accomodated in the design of the proposed Fuel Flooding System. This value is well below the one meter of surface offset which has already been accepted as part of the seismic design criteria.

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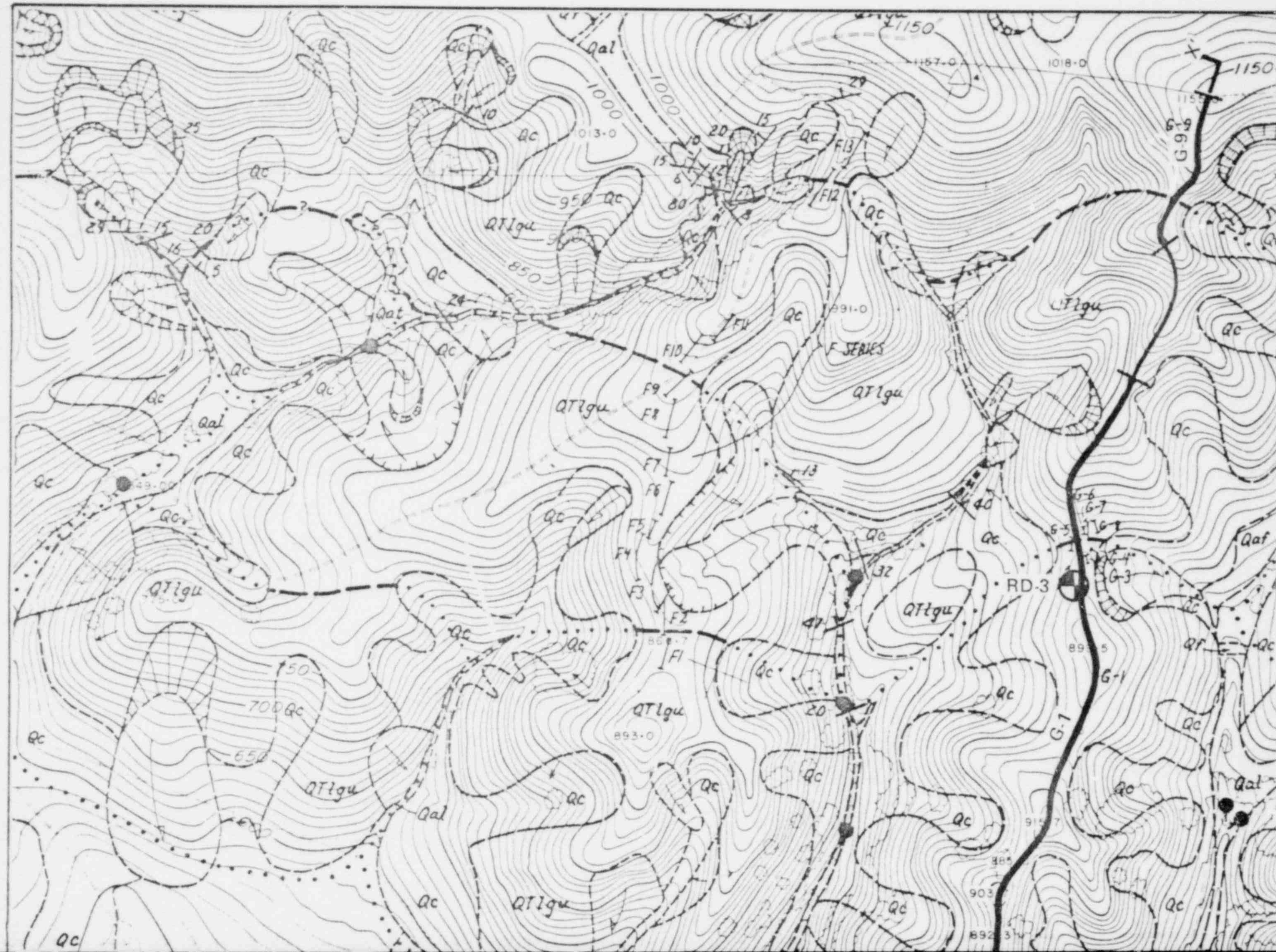

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G-9
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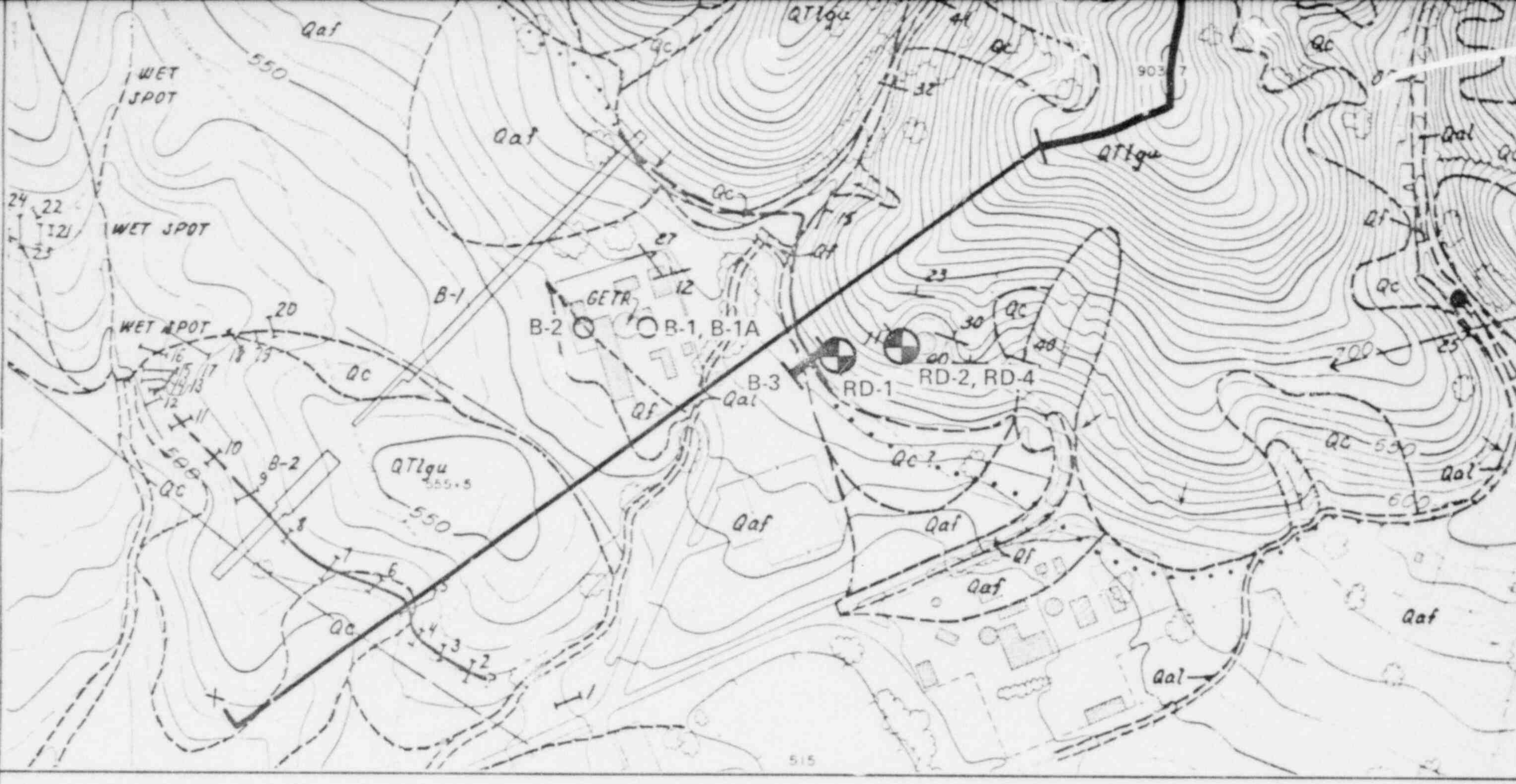

EXPLANATION

RD-2 Exploratory boring - ESA, 1980.

B-2 Exploratory boring - Shannon and Wilson, 1973.

G-1 Exploratory trench - ESA, 1979.

X X' Line of geologic/stability analysis section, see Figure 2.

NOTE

Base map from Figure No. 2, ESA, 1979. See Figure No. 2, ESA, 1979, for complete explanation of other map symbols.

SCALE

0 400 800 feet

Contour interval = 10 feet
(5 feet for dotted contour)

Earth Sciences Associates
Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS
EXPLORATION AND GEOLOGY MAP

Checked by Date 9/28/80		Project No.	Figure No.
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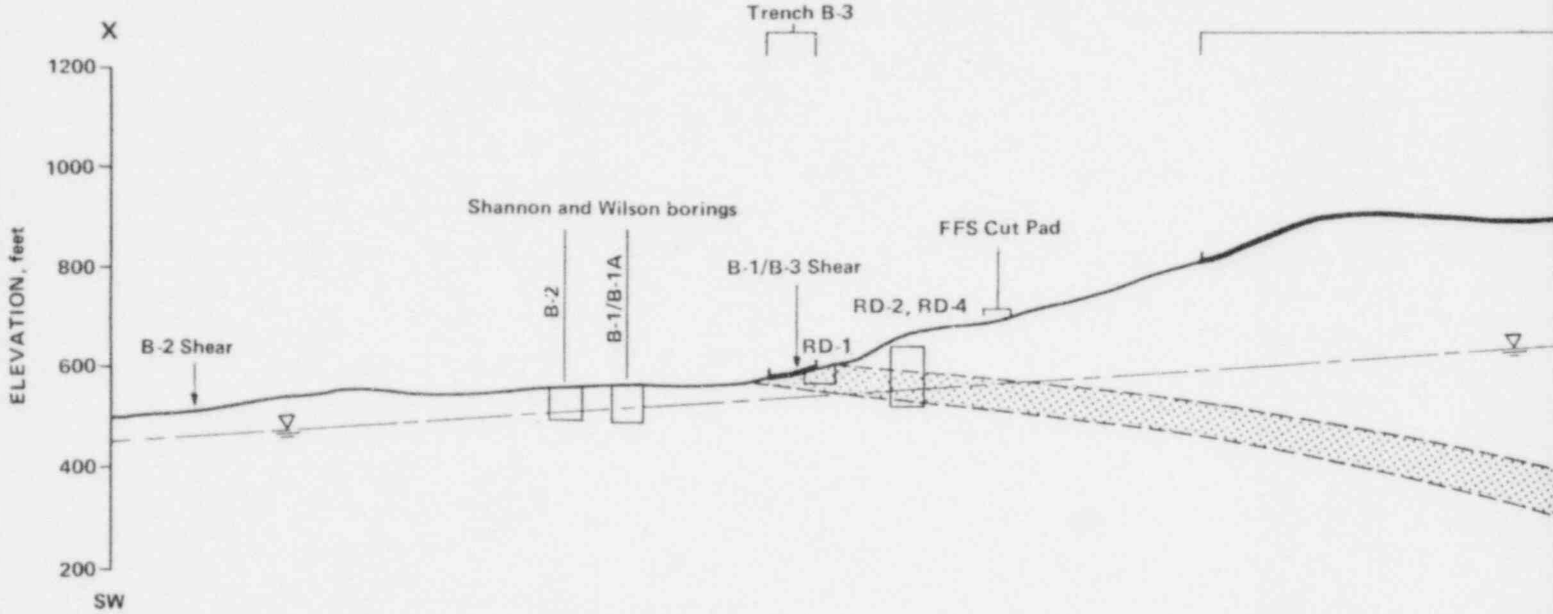


EXPLAN

Geologic contact.

Ground water table.

Material types 1 (clayey s

Material types 1 and 2 (cl

Material types 3 (sandy cl

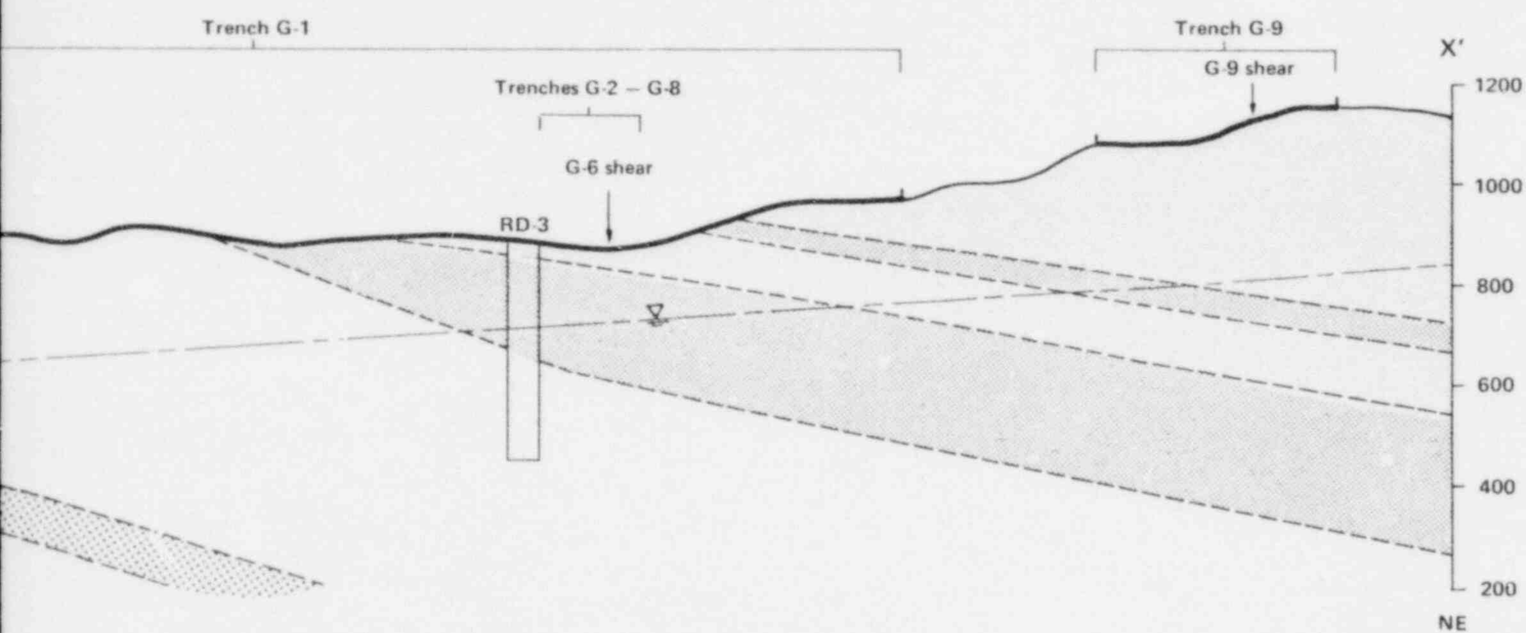
Material type 4 (clayey gr
(Note: see Appendix A ar

RD-3 Exploratory boring: ESA,
(Note: RD-1, 2, 4, and B-

Trench G-9 Exploratory trench; ESA,

NOTE: See Figure 1

Horizontal =



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ay) and 4 (clayey gravel and sand).

avel and sand).
d B for detailed descriptions of material types)

1980; Shannon and Wilson, 1973.
2, B-1/B-1A all projected to section)

1979.

for location of section.

Vertical Scale

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Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS

GEOLOGIC SECTION X–X'

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APPENDIX A
FIELD INVESTIGATION

Introduction

Subsurface exploration for this investigation was conducted during late June and early July of 1980. The field investigation involved drilling, logging, sampling and standard penetration testing of four rotary-wash borings in the GETR vicinity. Piezometers were installed in all borings and water levels are being monitored. All borings were drilled using a Failing 1500 truck-mounted drill rig. The borings were located as shown on Figure 1, Exploration and Geology Map. The borings, identified as RD-1, RD-2, RD-3, and RD-4, were sampled using Pitcher Barrel and Modified California Drive Samplers to obtain materials for laboratory index property determination and testing. The borings ranged in depth from 30 to 433 feet, with a total footage of 673 feet. Standard Penetration Tests were performed in 3 of the 4 borings.

Drilling operations were conducted by J.N. Pitcher Drilling Company of East Palo Alto, California, under the direction and field supervision of Earth Sciences Associates (ESA). All borings were logged by ESA personnel and copies of the field boring logs are included at the end of this appendix. ESA personnel handled, packed and transported all samples to the ESA geotechnical laboratory in Palo Alto, California.

Sampling

Samples for index property determination and laboratory strength testing were recovered from all borings using either a Pitcher Barrel Sampler (3-inch OD) or a modified California Drive Sampler (3-inch OD). The Pitcher Barrel Sampler yields relatively undistrubed samples except in gravelly materials and was used whenever possible to obtain good quality samples. ESA's field engineer encountered numerous difficulties in using the Pitcher Sampler in the formations being drilled. The local substrata consisted predominantly of clayey, gravelly sands to sandy gravels. Scattered cobbles up to several inches in diameter were common in much of the section penetrated. As a result two particular problems often occurred. First, cobbles protruding from the side wall of the bore hole often blocked lowering of the sampler to the bottom of the hole, or the end of the sample tube was bent as it was introduced. The borehole then had to be reamed with a rock bit before a new sample run was attempted. This procedure was timeconsuming and often occurred repeatedly so that further attempts at sampling those intervals were abandoned. The second problem would occur when coarse gravel clasts or cobbles were present in the material being sampled. These materials would bend the tube as it was advanced and caused poor recovery and very disturbed samples. These problems were most prevalent in RD-2, RD-3, and RD-4. No sampling was possible at depths greater than about 200 feet in boring RD-3 although several attempts were made. RD-1 yielded, for the most part, good quality Pitcher Barrel samples. Average Pitcher Barrel sample recovery was 88 percent for RD-1, 79 percent for RD-2, 72 percent for RD-3, and 61 percent for RD-4. Overall average recovery was 78 percent.

A modified California Drive Sampler was used to obtain blow count data as well as 2½ inch diameter, slightly to highly disturbed samples. Either a 140 lb or 300 lb hammer was used to drive the sampler depending on the hardness of the formation encountered. Average recovery for the California Drive samples using the 300 lb hammer was 73 percent for RD-3 and 63 percent for RD-4. Overall average recovery was 70 percent. Use of the 140 lb hammer yielded lower recoveries. Interpretation of blow count values fromthe modified California Drive Sampler are discussed under Field Testing.

In general, target sampling depths were chosen on the basis of geologic projection of shear planes observed in Trench B-3 and computer generated failure surfaces from preliminary stability analyses. Sampling was not, however, restricted to these depths since the field engineer tried to avoid sampling the very gravelly material which tended to result in poor to no recovery. Finer grained silty and clayey units were the preferred sampling horizons.

Samples were handled and transported carefully to minimize additional disturbance. After the sample was removed from the hole, it was logged, labelled, capped and sealed with tape and wax. Samples were transported to ESA's geotechnical laboratory in Palo Alto by ESA field personnel.

Field Testing

Standard Penetration Tests (SPTs) were performed using a standard 2-inch OD spoon with a split inner barrel. The tests were conducted by driving the sampler a distance of 18 inches using a 140 lb hammer and a 30-inch drop. The number of blows required to advance the sampler each of three successive 6-inch increments was recorded. The sum of the blows for the last two increments is the Standard Penetration Resistance (N_{STD}) in blows/ft. These values are plotted versus depth on Figure A-1.

A modified California Drive Sampler was also used to obtain penetration resistance data. This sampler consists of a 3-inch OD thick-walled barrel with three 2½-inch OD, 6-inch long steel liners. The liners facilitated handling, transport, and storage of recovered samples. Penetration tests using this sampler were conducted in the same way as the SPTs, except that a 300 lb hammer was used in very dense or hard materials. In order to compare the penetration resistance determined by these tests with the SPT values a correction factor was applied to the California Drive Sampler test results. The correction factor accounts for the difference in input energy when different hammer weights are used and for the difference in wall thickness of the samplers. The correction factor is calculated as follows:

$$C = \frac{B_{STD}}{N_{CAL}} = \frac{0.0005\ Wh}{D_o - D_i}$$ where,

C	=	correction factor, dimensionless
B_{STD}	=	STP - equivalent penetration resistance, blow/ft
N_{CAL}	=	field blow count for modified California Drive Sampler, blows/ft
W	=	weight of hammer, lb
h	=	height of drop, inches
D_o	=	sampler outer diameter, inches
D_i	=	sampler inner diameter, inches

The SPT-equivalent standard penetration resistance values from the modified California Sampler tests are plotted versus depth on Figure A-1, along with the SPT results.

Estimates of in-situ strength can be made based on the values of standard penetration resistance summarized on Figure A-1. Using published relationships of unconfined compressive strength to standard penetration resistance suggests that the undrained shear strengths of the fine grained soils tested in-situ is on the order of 5,000 to 10,000 psf to depths of about 100 feet (NAVFAC DM-7, 1971). Similar relationships between friction angle, ϕ , and standard penetration resistance indicate that the ϕ angle for the coarse grained soils tested to depths of about 40 feet is at least greater than 35 degrees and is likely signficantly higher (Lambe and Whitman, 1969).

Pocket penetrometer tests were performed on the finer grained materials whenever possible to obtain approximate values of undrained shear strength. In most cases the sample strengths exceeded the capacity of the penetrometer (4500 psf) or the sample was too gravelly to test. The results of the penetrometer tests are included on the boring logs accompanying this appendix.

Geophysical Logging

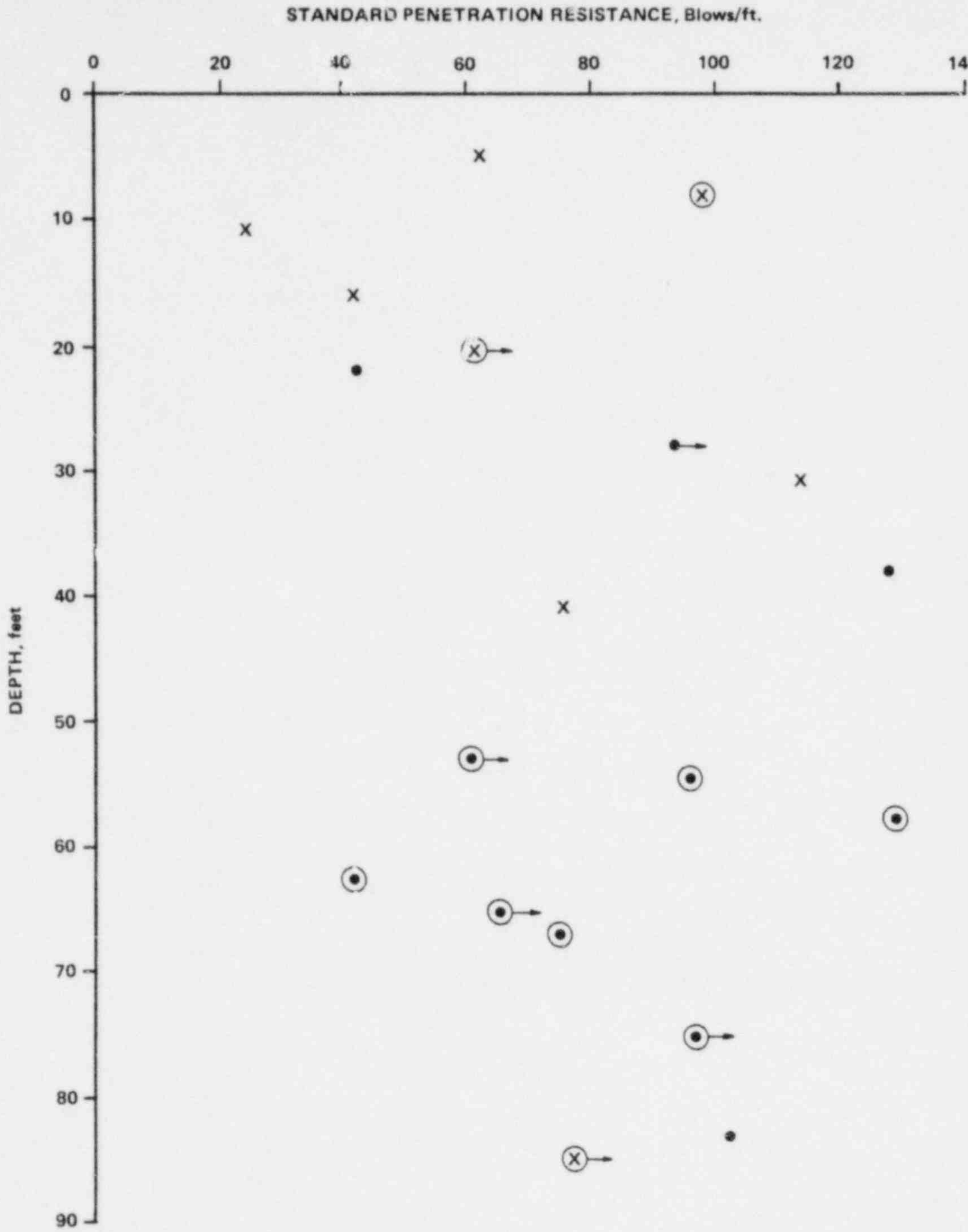
A suite of geophysical logs was run by ESA personnel in borings RD-2 and RD-3 using ESA's WIDCO 1200 Logger. Natural gamma radiation, spontaneous potential and resistivity surveys were run in RD-2. Gamma and resistivity logs were recorded in RD-3, but an equipment malfunction precluded recording spontaneous potential in that hole. Copies of the geophysical logs are included as Figures A-2 and A-3 in this appendix.

Piezometer Installation and Monitoring

Piezometers were installed in each of the boreholes to provide data on ground water conditions within the slope explored. Since perched water conditions were suspected, two well points were staged in RD-2 and a third deeper piezometer was installed in the adjacent boring RD-4. A typical piezometer installation consisted of a 3 foot long slotted PVC well point attached to the bottom end of a length of 1-inch diameter PVC piping. The well point was inserted to the desired depth in the completed borehole. Then $\frac{1}{4}$-$\frac{1}{2}$ inch diameter pea gravel backfill was packed around the piezometer tip to the desired depth of the interval to be monitored. Impervious bentonite seals were installed to isolate the desired monitoring interval. The seals were emplaced by pouring high density dry bentonite pellets into the borehole. The pellets expand on exposure to water in the borehole and form an impermeable seal within a few minutes. More gravel was used as backfill on top of the seals. Details of the piezometer installations in each of the borings are included on the drilling and sampling logs.

The completed piezometers were all flushed with compressed air in an attempt to remove any remaining drilling fluid. Water levels have been periodically measured since installation with an electric water probe.

Piezometer readings from borings RD-2 and RD-4 are plotted on Figure A-4 and those from RD-3 are shown on Figure A-5. The piezometer installed in boring RD-1 remained dry throughout the monitoring period shown on Figures A-4 and A-5. As seen on Figure A-4, the upper piezometer in RD-2 and the deep piezometer in RD-4 are recording very low heads and the fluid levels appear to be declining slowly with time. The readings in these piezometers may actually be residual drilling fluid which was not removed by the compressed air flushing. The readings in the deep piezometers in RD-2 are erratic, but also show a general decline in fluid level with time. During the blow-out operation, compressed air input at the bottom of the deep piezometer in RD-2 caused some fluid to be blown out of the top of the shallower piezometer. This indicates that some hydrualic connection exists between these staged piezometers in spite of the presence of two bentonite seals. A conservative interpretation of these data is that the fluid recorded by the deeper piezometer in RD-2 is due to a zone of perched ground water above the top of the piezometer in boring RD-4. The maximum head due to this inferred perched zone would be about 20 feet of water.



EXPLANATION

SPT • } clays, silts
Mod. Cal. ⊙ } clays, silts

SPT X } sands, gravels
Mod. Cal. ⊗ } sands, gravels

Arrow indicates refusal before 18 inches of penetration was achieved. →

Modified California values are converted to give equivalent SPT values.

Earth Sciences Associates
Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS

SUMMARY OF PENETRATION TEST RESULTS

Checked by Don Fodor Date 8/28/80	Project No.	Figure No.
Approved by [signature] Date 8/28/80	1886	A-1

Earth Sciences Associates

PROJECT NUMBER 1886 HOLE NUMBER RD-2

SCALE 1" = 10'(original) LINE SPEED 25 ft./min.

LOGGED BY CJP DATE 6/24/80

ELECTRIC LOG

HOLE DEPTH (MIN) 9.5 ft. (MAX) 118.8 ft.

SP 10 mv/division RES 10 ohms/division

REMARKS poor SP response

SPONTANEOUS POTENTIAL

RESISTIVITY →

BORING LOG*
DEPTH (FEET)

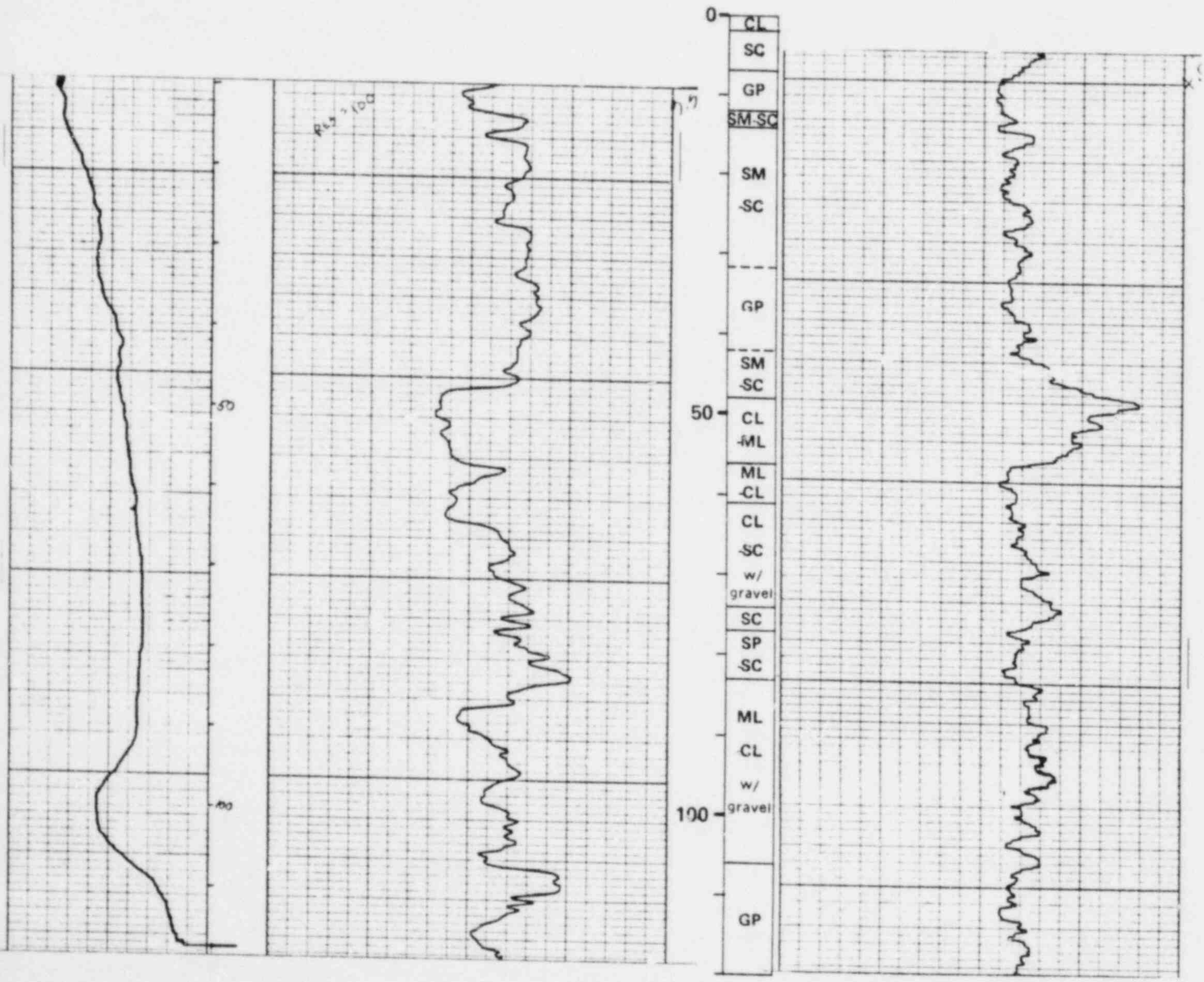
RADIOMETRIC LOG

HOLE DEPTH (MIN) 40 ft. (MAX) 119.1 ft

TC 3 sec.

REMARKS -------

NATURAL GAMMA RAY INTENSITY →



* See Drilling and Sampling Log in Appendix A for detailed descriptions.

Figure A-2

Earth Sciences Associates

PROJECT NUMBER 1886 HOLE NUMBER RD-3

SCALE 1" = 10' (original) LINE SPEED 25 ft./min.

LOGGED BY DMY DATE 7/8/80

ELECTRIC LOG

HOLE DEPTH (MIN) 44.1 ft. (MAX) 429.9 ft.

SP —— RES 5 ohms/division

REMARKS SP non-functional

SPONTANEOUS POTENTIAL

RESISTIVITY →

BORING LOG*
DEPTH (FEET)

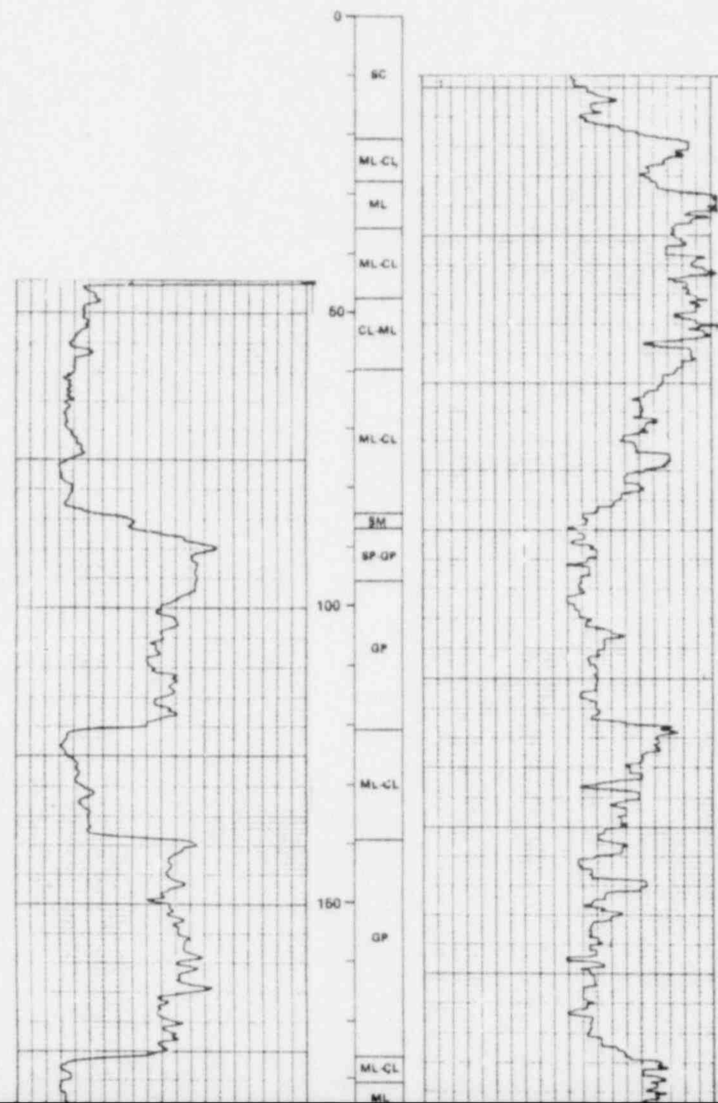
RADIOMETRIC LOG

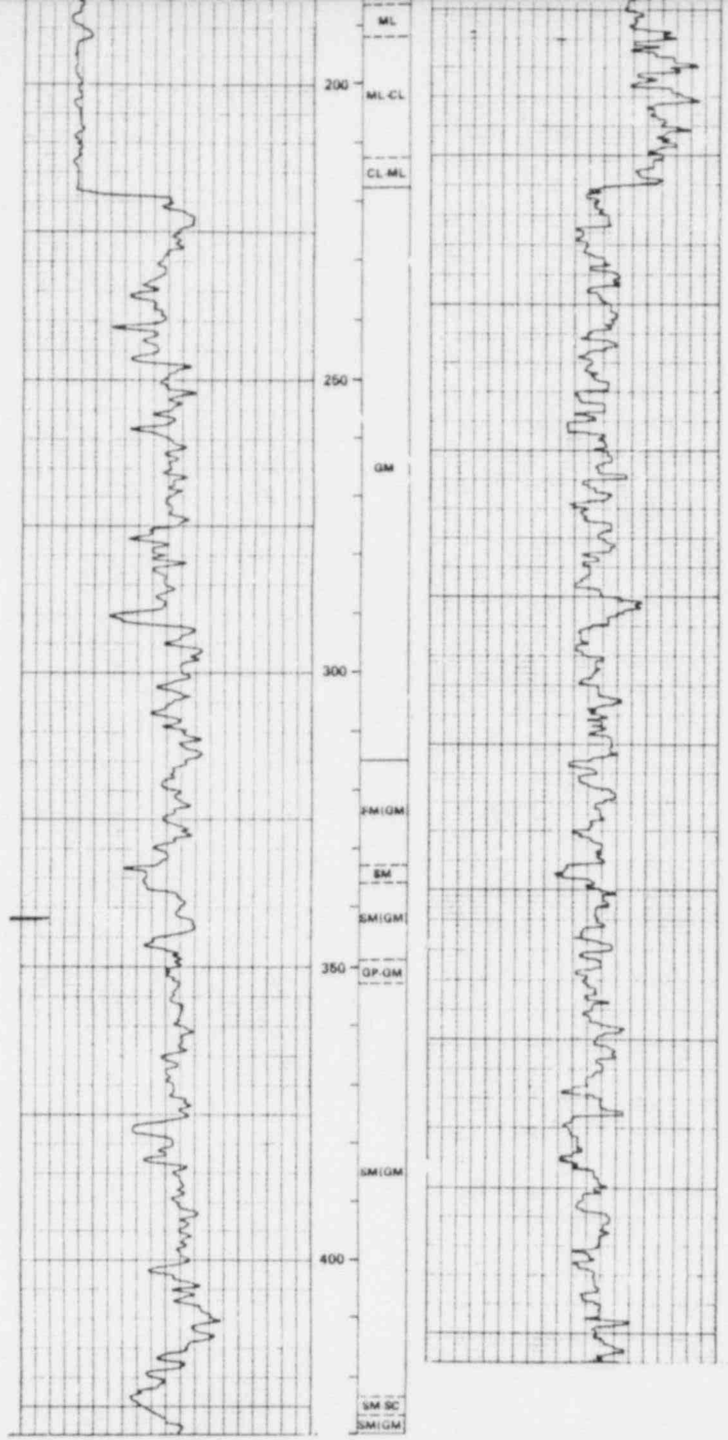
HOLE DEPTH (MIN) 10.0 ft. (MAX) 417.0 ft.

TC 1 sec

REMARKS ——

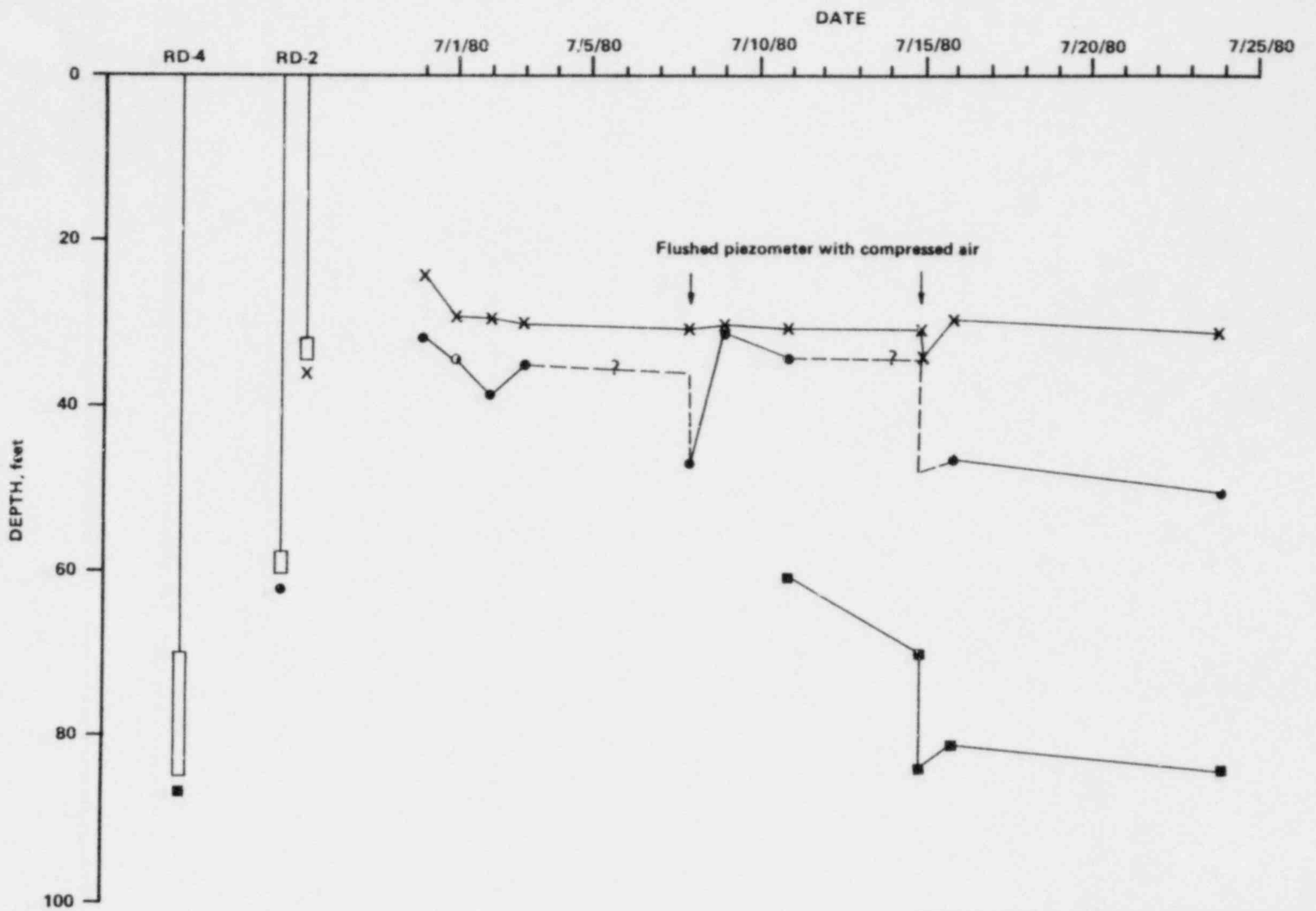
NATURAL [illegible]A
RAY INTENSITY →





See Drilling and Sampling Log in Appendix A for detailed descriptions.

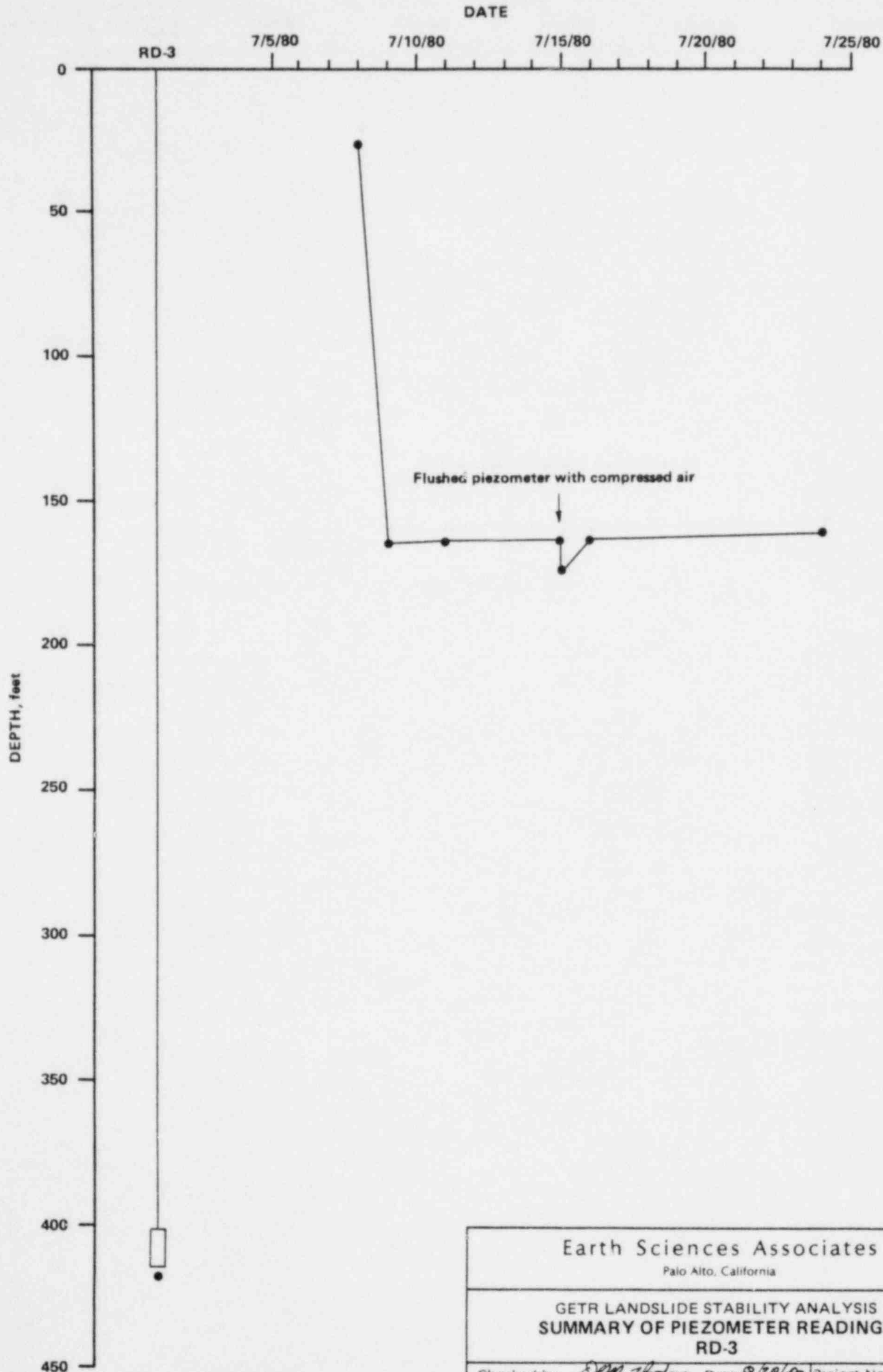
Figure A-3



Earth Sciences Associates
Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS
SUMMARY OF PIEZOMETER READINGS
RD-2 and RD-4

Checked by Dan Fordan Date 8/28/80		Project No	Figure No
Approved by R.L. Meehan Date 8/28/80		1886	A-4



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Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS
SUMMARY OF PIEZOMETER READINGS
RD-3

Checked by DM Hodan Date 8/28/80	Project No.	Figure No.
Approved by R.L. Meehan Date 8/28/80	1886	A-5

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT 1886 GETR - LANDSLIDE INVESTIGATION DATE DRILLED JUNE 25 1980 HOLE NO. RD-1

LOCATION TOP OF TRENCH 12-2 GROUND SURFACE ELEV. ~607 (TOPO)

DRILLING CONTRACTOR PITCHER DRILLING LOGGED BY CJP DEPTH TO GROUND WATER NA

TYPE OF RIG FAILING 1500 HOLE DIAMETER 3" PB HAMMER WEIGHT AND FALL NA

SURFACE CONDITIONS PAD CUT BY CAT ON HILLSIDE WEATHER WARM, CLEAR, BREEZY

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0–2.5	ML	SAMPLE ST-1: CLAYEY SILT: GRAYISH ORANGE (10 YR 7/4); 90% PREDOM SILTY FINES; SLOW DILATANCY; LOW TO MOD PLASTICITY; 10% FINE SAND; DAMP HARD; CALCAREOUS	ST-1	P 1.0/2.5	ON SITE & DRILLING 12 NOON. PUSHED SHELBY TUBE 0'-2.5'
2.5–5	CL	SAMPLE ST-2: SILTY CLAY: MODERATE BROWN (5YR 4/4); 50% HIGH TO MED PLASTICITY CLAY FINES; 40% SILTY FINES; 10% FINE SAND; NON-DILATANT, HARD, BRITTLE, SLIGHTLY CALC; MOIST.	ST-2	P 2.5/2.5	PUSHED SHELBY TUBE FROM 2.5'-5.0'
5–7.5	ML	SAMPLE PB-1: CLAYEY SILT: MODERATE BROWN; 90% PREDOM SILTY FINES, SOME CLAY OF MEDIUM PLASTICITY; 10% FINE SAND; FIRM, SLOW DILATANCY, SLIGHTLY CALC. MOIST.	PB-1	P 2.5/2.5	SET 4' OF STEEL SURFACE CASING, AFTER CLEANING HOLE W/ AUGER. CUT PITCHER BARREL FROM 5.0'-7.5'
7.5–10	CL	SAMPLE PB-2: SILTY CLAY: LT TO MODERATE BROWN (5YR 4/6); 90% PREDOM. MAINLY CLAYEY FINES; NON DILATANT, LOW PLASTICITY; 10% FINE SAND & OCC. CHUNKS OF GRAVEL; HARD; DAMP; SOME CALC STREAKS.	PB-2	P 2.1/2.5	CUT PITCHER BARREL FROM 7.5'-10.0'
10–12.5		PB-3: AS ABOVE	PB-3	P 2.3/2.5	CUT PITCHER BARREL FROM 10.0-12.5
12.5–15	ML	SAMPLE PB-4: CLAYEY SILT: MOD YELLOWISH BROWN; 90% PREDOM. SILTY FINES, SLOW TO MODERATE DILATANCY; LOW PLASTICITY, HARD, DAMP.	PB-4	P 2.3/2.5	CUT PITCHER BARREL FROM 12.5-15.0
15–17.5		SAMPLE PB-5: CLAYEY SILT: LT BROWN (5 YR 6/4) OTHERWISE AS ABOVE WITH NO SAND OR GRAVEL.	PB-5	P 2.0/2.5	CUT PITCHER BARREL FROM 15.0-17.5
17.5–20		SAMPLE PB-6: CLAYEY SILT: LT BROWN; LOW PLASTICITY PREDOM SILTY FINES & WEATHERED FRAGS OF GRAYWACKE, NON-CALCAREOUS, HARD, DAMP. SOME LARGE WEATHERED CLASTS.	PB-6	P 2.1/2.5	CUT PITCHER BARREL FROM 17.5-20.0'
20–22.5		SAMPLE PB-7: CLAYEY SILT: MOD YELLOWISH BROWN (10YR 5/4); 90% PREDOM MAINLY SILTY FINES, 10% FINE TO COARSE SAND; LARGE GREENSTONE CLASTS (>1").	PB-7	P 2.0/2.5	CUT PITCHER BARREL FROM 20-22.5'
22.5–25		PB-8: AS ABOVE, NO GRAVEL	PB-8	P 2.1/2.5	CUT PITCHER BARREL FROM 22.5'-25.0' $S_u > 4.5$ tsf
25–27.5		SAMPLE PB-9: CALCAREOUS CLAYEY SILT: MOD YELLOWISH BROWN W/ SOME GRAVEL	PB-9	P 2.2/2.5	CUT PITCHER BARREL FROM 25.0'-27.5'
27.5–30		SAMPLE PB-10: GRAVELLY CLAYEY SILT: LT BROWN W/ WEATHERED GRAYWACKE	PB-10	P 2.4/2.5	CUT PITCHER BARREL FROM 27.5-30.0'
30		BH = 30.0'			TERMINATED HOLE; SUFFICIENT SAMPLES OBTAINED. PIEZOMETER WELL PT INSTALLED AT 30'. SEE DEPTH COLUMN FOR PIEZOMETER DETAILS.

HOLE [illegible] BENTONITE SEAL [illegible]

SHEET 1 OF 1

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT 1886 GETR - LANDSLIDE INVESTIGATION DATE DRILLED JUNE 23-24 1980 HOLE NO. RD-2

LOCATION NW SIDE OF WATER TANK GROUND SURFACE ELEV. ~645' (TOPO)

DRILLING CONTRACTOR PITCHER DRILLING LOGGED BY CJP DEPTH TO GROUND WATER NA

TYPE OF RIG FAILING 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL NA

SURFACE CONDITIONS GRAVELLY, GRASSY ROAD SHOULDER WEATHER CLEAR, WARM, SUNNY

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0–2	CL	0'–2' SILTY CLAY: DUSKY YEL (5Y 6/4) TO LIGHT OLIVE BRN (5Y 5/6); FIRM TO STIFF, SLIGHTLY MOIST.		AD	ON SITE 9 AM DRILLING BY 10 AM AUGER TO 4 FEET, SET 4' OF SURFACE CASING. 1' STICK-UP.
2–7	SC	2-7 SILTY CLAYEY SAND: DUSKY YELLOW (5Y 6/4); ~25-40% low-MED PLASTIC FINES; 60-75% FINE SAND; FRIABLE, SLIGHTLY MOIST.		RD	DRILLING W/ BENTONITE TO SEAL TOP OF HOLE. CHATTERING IN GRAVELS, AT ~7' & 9'
7–12.2	GP	7.0'-12.2' SILTY SANDY GRAVEL: MATRIX IS DUSKY YELLOW, CLASTS MULTICOLORED; 10-15% SILTY CLAY BINDER W/ 10-15% V. FINE SAND. 70-80% COARSE TO FINE GRAVELS; CLASTS ARE ROUNDED TO SUB-ROUNDED, CONSIST OF QTZT, CHERT, GREYWACKE; INTERBEDDED COARSER & FINER LAYERS; FRIABLE; UNCEMENTED.	PB-1	PB 1.4/2.5	CUT PITCHER BARREL 10.0' - 12.5' H₂O. PRESSURE ~ 200-270 psi
12.2–14	SM-SC	12.2' – 14.0' SILTY SAND: DUSKY YELLOW (5Y 6/4); PALE OLIVE (10Y 6/2) NO GRAVELS; LOW PLASTIC FINES.		RD	
14–33	SM-SC	14.0' - 49.0' - GRAVELLY SILTY SAND: DUSKY YELLOW (5Y 6/4); 20% LOW PLAS. FINES; 20% COARSE SAND; 10% MED SAND; 10% FINE SAND; ~30% FINE GRAVEL - SUB-ANG TO SUB-ROUNDED CLASTS OF CHERT, QTZ, GREYWACKE; FRIABLE; UNCEMENTED. 19.0 - INCREASING GRAVEL ~50% 23.0' - SILTY SAND W/ NO GRAVEL. OCCURS AS LENSES OR V. THIN STRINGERS. 27.0' - MORE STRINGERS. 27.8' - GRADES INTO COARSER MATERIAL W/ SOME SMALL CARBONATE NODULES, GRAVELS ~ 30% 31.5 - 42.0' BECOMES COARSER		PB 0/2.5 RD PB 0/2.5 RD	CUT PITCHER BARREL 15.3' - 17.8' H₂O. PRES ~ 200 psi CUT A ROCK, NO RECOVERY 21.5' CUT PITCHER BARRELL 24.0' - 26.5 NO RECOVERY 29' CHATTER 32.0' 31.5' LOUD CHATTER
33–40	GP	GRAVELS ~ 50%; MAXSIZE 1/4" GRADES TO YELLOWISH GRAY (5Y 7/2) TO LIGHT OLIVE GRAY (5Y 5/2).		PB 0/2.5	CUT PITCHER BARREL 34.0 - 35.0 HYDR. PRES ~ 100 psi NO RECOVERY ADD MUD TO FLUID.

SHEET 1 OF 3

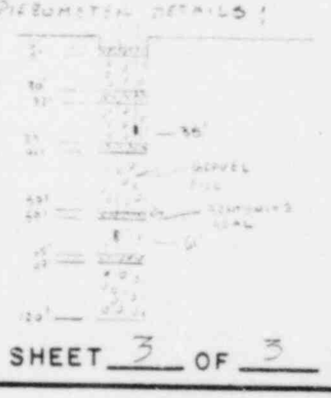
PROJECT 1886 GETR - LANDSLIDE INVESTIGATION DATE DRILLED JUNE 23 1980 HOLE NO. RD-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
40–42	GP	14.0' - 430' GRAVELLY SILTY SAND: (con't)		RD	41.5 GRAVEL SLOUGHING INTO HOLE.
42–48	SM-SC	42' - 43' GRADES LESS GRAVELLY; GRAVEL CONTENT ~30% OR LESS. FINES APPEAR TO BE BECOMING MORE PLASTIC.	PB-2	PB .8/2.5 RD	NO SLOUGH IN HOLE. CUT PITCHER BARREL 45.0' - 47.5 TUBE BENT
48–56	CL-ML	48.0' - 56.0' SILTY CLAY: LIGHT OLIVE GRAY (5 Y 5/2); 90% LOW TO MED PLASTICITY FINES; SLOW DILATANCY, MOD TOUGHNESS, 10% OR LESS V. FINE SAND; VERY CALCAREOUS (NUMEROUS WHITE STREAKS AS WELL AS PERVASIVE CARB ATE IN MATRIX), STIFF TO HAR DAMP; FRIABLE. $S_u > 4$ $\pm$ (POCKET PENETROMETER)	PB-3 PB-4	PB 1.0/2.5 RD PB	CUT PITCHER BARREL 48.5 - 51.0' TUBE BENT CUT PITCHER BARREL 53.0 - 55.5 (BENT) 4 PM, 55.5 QUIT FOR 6/23 ON SITE 7:15 AM 6/24 WATER IN HOLE AT 14'
56–61	ML-CL	56.0' - 61.0' SA SILT: LT OLIVE GRAY (5/2); 60% LOW PLAS. FINES; SLOW DILATANCY; 30% FINE SAND; 10% MED. SAND; HARD; MOIST.		RD	
61–74	CG-SC SC	61.0' - 74.0' SANDY SILTY CLAY - CLAYEY SAND: MODERATE BROWN (5 YR 4/4); 80% LOW PLASTICITY FINES, 20% FINE & MED. SAND; ANG TO SUB-ANGULAR GRAINS OF GREYWACKE & CHERT; HARD; MOIST. GRADES SANDIER - 35% FINE & MED SAND; 4-15% FINE GRAVEL; SOME LARGER CLASTS UP TO 3/4" DIAMETER. INCREASING SAND & GRAVEL	PB-5 B-1 J-1 PB-6	PB 2.4/2.5 RD PB 2.1/2.1 RD PB 1.2/1.5 RD	CUT PITCHER BARREL 60 - 62.5 TUBE BENT SLIGHT CHATTER ~61' CUT PITCHER BARREL 65 - 67.1 EXTRUDED & SAVED INTACT PORTION IN JAR CUT PITCHER BARREL 70 - 71.5 HARD TO CUT.
74–77	SC	74' - 77': CLAYEY SAND: 45% LOW PLASTIC CLAY & SILT FINES; SLOW DILATANCY, 55% SAND & FINE GRAVEL; HARD; S_u > 4.5 tsf MOIST.	B-2	PB 2.3/2.5	CUT PITCHER BARREL 75 - 77.5 TUBE BENT, SAMPLE DISTURBED.
77–83	SP-SC	77.0' - 83.0' SILTY GRAVELLY SAND: LT TO MOD BROWN (5 YR 5/4), 30% LOW PLAS FINES; 40% FINE TO COARSE SAND; 30% FINE TO COARSE GRAVEL W/ CLASTS UP TO 2" DIAM.; STRENGTH RANGES FROM 2.5 tsf TO >4.5 tsf FROM POCKET PENETROMETER TESTS ON FINE GRAINED MATRIX.	3-4	RD	
83–88	ML-CL	83.0' - 106.0' SANDY GRAVELLY SILT: MODERATE YELLOWISH BROWN (10YR 5/4) 50% MOD - LOW PLASTICITY FINES, SLOW DILATANCY, MEDIUM TOUGHNESS; 15% FINE TO COARSE SAND; ANGULAR TO ROUNDED CLASTS; 35% GRAVEL, ALL SIZES INCLUDING >2" DIAM.; HARD; DENSE; DAMP.	B-5 J-2	PB 1.5/2.5 RD	CUT PITCHER BARREL 84 - 86.5 TUBE BENT

SHEET 2 OF 3

PROJECT 1886 GEOR - LANDSLIDE INVESTIGATION DATE DRILLED JUN 23 - 24 1980 HOLE NO. RD-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
88	ML-CL	83.0' - 106.0' SANDY GRAVELLY SILT: (CON'T)		RD	
90				PB	CUT PITCHER BARREL 89.0' - 89.75' - TUBE BENT, NO RECOVERY
92			PB-8	PB 1.5/2.5	CUT PITCHER BARREL 90.0' - 92.5' TUBE BENT SLIGHTLY.
94				RD	
96			PB-9	PB 2.4/2.4	CUT PITCHER BARREL 95.0' - 97.5'
98			PB-10	PB 2.5/2.5	CUT PITCHER BARREL 97.5 - 100.0
100				RD	
102					
104					
106					105' LOUD CHATTER
108	GP	106.0' - 120.0' SANDY GRAVEL: INTERBEDDED SANDY & GRAVELLY LAYERS W/ SOME SILT/CLAY FINES. SAND IS COARSE & ANGULAR; GRAVEL IS PREDOM. WEATHERED GRAYWACKE & BREAKS DOWN INTO COARSE ANGULAR FRAGMENTS. SOME HARD GRAVEL CLASTS, (CHERT, QTZ, ETC.); GRAVEL SIZES ARE UP TO 3" OR LARGER.			108' LOUD CHATTER
110					
112					
114		PB-12 SAMPLE: SILTY GRAVELLY SAND; MOTTLED DK YELLWSH ORNG (10 YR 6/6) & MOD YELLWSH BRN (10 YR 5/4); LESS THAN 20% NON-PLASTIC SILTY FINES	PB-11	PB 1.5/1.5	CUT PITCHER BARREL 112.0' - 113.5'
116		30% SAND & GRAVEL, WELL GRADED IN SAND SIZES; GRAVEL CONSISTS OF CLASTS OF WTHRD GRAYWACKE OR METAVOLC., CHERT, GREENSTONE, ETC.; DENSE; DAMP; HARD; Su > 4.5 tsf BY POCKET PENETROMETER.		RD	
118					
120		T.D.H. = 120.0'	PB-12	PB 1.3/2.0	CUT PITCHER BARREL 118.0' - 120.0'
122					TERMINATED HOLE. SUFFICIENT DATA OBTAINED.
124					RAN SUITE OF GEOPHYSICAL LOGS (GAMMA / SP - RES)
126					6/25/80 INSTALLED 2 PVC WELL POINTS AT 35' & 61' DEPTHS.
128					PIEZOMETER DETAILS:
130					
132					
134					
136					



SHEET 3 OF 3

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT 1886 GETR - Landslide Investigation DATE DRILLED Jun 26 - 1980 HOLE NO. RD-3

LOCATION 150' from saddle along ridge GROUND SURFACE ELEV. ~883' (Topo)

DRILLING CONTRACTOR Pitcher Drilling LOGGED BY CJP DEPTH TO GROUND WATER NA

TYPE OF RIG Failing 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lbs, 30" unless noted

SURFACE CONDITIONS Cut pad on grassy sloping ridge WEATHER Clear, warm

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0	SC	0 - 21.0' Gravelly clayey sand: Dk yellowish orange (10YR 6/6); 30%-40% silt & clay low plasticity fines; 50% fine sand; 10-20% gravel consisting of weathered clasts of predom. graywacke; firm; damp; non-homogeneous, w layers of finer & coarser materials 1"-2" thick.		AD	Arrive at GETR 7:30AM Move equipment onto site & drilling by 8:30. Set 4' of surface casing. Using rock bit, reheat
4			B-1	SS 1.0/1.5	4.0 - 5.5' Drove std split spoon. 17/.5 23/.5 34/.5
6				RD	
8		Gravelly layers interbedded w/ sandy layers.	C-1, III	D .7/1.5	7.0 - 8.5 Drove modified 2.5" I.D. Calif. sampler w/ 140# hammer. 23/.5 52/.5 110/.5
10			B-2	RD	
12					Smooth quiet drilling
14					
16			B-3	SS 1.0/1.5	15.0'-16.5' Drove std. split spoon. 16/.5 20/.5 21/.5
18				RD	Chatter at 18.'
20					Chatter at 20'
22	ML-CL		B-4	SS 1.1/1.5	21.0 - 22.5 Drove std. split spoon 9/.5 13/.5 23/.5
24		21.0 - 28.0' Clayey silt: Grayish yellow green (5GY 7/2); 100% quickly dilatant predom. silty fines; v. homogeneous; hard; damp.		RD	
26		24.0' - Color changes to mod. yellowish brown (10YR 5/4)	PB-1	PB 2.4/2.5	25.0 - 27.5' Cut Pitcher barrel.
28	ML		B-5	SS 1.2/1.25	27.5 - 28.75' Drove std split spoon 26/.5 43/.5 50/.25
30		28.0 - 36.0' Sandy silt: Greenish gray (5GY 6/1); ~70% low plas. silt; quick dilatancy; 30% v. fine sand; v. hard, damp.		RD	Change to drag bit
32					
34		Graded less sandy (<15%) & becomes clayier			
36	ML-CL		PB-2	PB 2.2/2.5	35.0 - 37.5' Cut Pitcher barrel
38		36.0' - 48.0' Clayey silt: Mottled greenish gray & mod. yellowish brown (10YR 5/4); 100% low plas. silt & clay; mod. quick dilatancy; v. hard; damp; calcareous.	B-6	SS 1.0/1.5	37.5 - 39.0' Drove std split spoon. 37/.5 100/.5
40				RD	

SHEET 1 OF 10

PROJECT 1886 GETR - LANDSLIDE INVEST DATE DRILLED JUNE 23 1980 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
40 42	ML-CL	36.0' - 48.0' CLAYEY SILT: (con't)		RD	
44		GRADES SANDIER			
46 48		SAMPLE: SANDY SILT; LT OLIVE GRAY (5Y 5/2); 70-80% PREDOM NON-PLASTIC SILT FINES; QUICK DILATANCY; 20-30% V. FINE SAND, PREDOM. SUB ANG TO ANG QTZ; CALCAREOUS; HARD; MOIST	PB-3	PB 2.5/2.5	45.0 - 47.5 CUT PITCHER BARREL
50 52	CL-ML	48.0' - 60.0' SILTY CLAY: MODERATE YELLOWISH BROWN (10YR 5/4); ~100%, LOW TO MOD PLASTICITY PREDOMINANTLY CLAY FINES; SLOW TO NON-DILATANT; HARD; MOIST.		RD	LOUD CHATTER AT 48' CHANGE BACK TO ROCK BIT DRILLS QUIETLY.
54					53.5
56			PB-4	PB 2.2/2.5	55.0 - 57.5 CUT PITCHER BARREL
58			C-2, I	D .5/.5 RD	57.5 - 58.0 DROVE CALIF SAMPLER W/ 140# HAMMER 100/.5 TOO HARD TO DRIVE.
60		GRADES TO			
62 64	ML-CL	~60.0' - 84.5' CLAYEY SILT: MOD YELLOWISH BROWN (10YR 5/4); ~100% LOW PLASTICITY PREDOMINANTLY SILTY FINES; QUICK DILATANCY; CALCAREOUS; HARD; MOIST.			
66 68			C-3, I	D .5/.5 RD	65.0 - 65.5 DROVE CALIF SAMPLER W/ 300# HAMMER 50/.5 TOO HARD TO DRIVE FURTHER - AFRAID OF LOSING ENTIRE SAMPLE.
70 72 74					
76 78		SAMPLE: CLAYEY SANDY SILT: LIGHT OLIVE GRAY (5Y 5/2); 60% LOW PLASTICITY, MOD QUICK DILATANCY FINES; 40% V. FINE SAND	C-4, I	D .5/.5 RD	75.0 - 75.5 DROVE CALIF. SAMPLER W/ 300# HAMMER 75/.5
80 82		~77.0' COLOR CHANGE TO MODERATE YELLOWISH BROWN (10YR 5/4) MATERIAL IS LAYERED & VARIABLE IN COLOR FROM MOD. YELLOWISH BRN TO LT OLIVE GRAY. LAYERS ARE PREDOM SILT BUT VARY IN SAND & CLAY CONTENT.			
84					AT 5:30, 85. QUIT FOR 6/26/80
86	SM	84.5 - 87.0 SILTY SAND: LT OLIVE BROWN; 40% SILT, 60% FINE SAND, PREDOM QTZ; HARD; DAMP.	C-5, I	D .25/.5 RD	ON SITE 7:30 AM 6/27 85.0 - 85.5 DROVE CALIF SAMPLER 60/.5
88	SP-GP				

SHEET 2 OF 10

PROJECT 1886 GETR - LANDSLIDE INVEST. DATE DRILLED JUNE 26 1980 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
88	SP-GP	87.0 - 96.0 SILTY / GRAVELLY / SAND: 30% LOW PLASTIC SILTY / FINES; 40-50% MED FINE & COARSE SAND; 20-30% FINE - MED GRAVEL.		RD	LOUD CHATTER AT 87' CONTINUING
90					
92					
94					
96		GRADES COARSER & GRAVELLIER INTO:			
	GP	96 - 121 SILTY / SANDY GRAVEL: 30% LOW PLASTIC SILTY FINES; 10-20% SAND; 50-60% FINE TO MED GRAVEL CONSISTING OF QTZ, CHERT, GREENSTONE & GRAYWACKE; SUB-ROUNDED CLASTS UP TO 1/2 - 3/4".			
98					CHANGE TO ROCK BIT
100					
102					
104					
106		AMOUNT OF SAND, SILT & GRAVEL VARIES - LENSES & INTERBEDS ARE ~1' THICK			105.5'
108					
110					
112		GRAVEL CONSISTS PREDOMINANTLY OF GREENSTONE. MINOR AMOUNTS OF QTZ, CHERT, GRAYWACKE, ETC.			
114		GRADES SANDIER			
116		SAMPLE B-7: GRAVELLY SAND: COLOR HIGHLY VARIABLE FROM LT OLIVE GRAY (5 Y 5/2) TO DUSKY YELLOW (5 Y 6/4) LT OLIVE BROWN (5 Y 5/6) & DK YELLOWISH ORANGE (10 YR 6/6); SAMPLE IS NON-HOMOGENEOUS, LAYERED W/ MED SAND & SILT LAYERS; GRAVELLY LAYERS W/ WEATHERED GRAYWACKE CLASTS, CHERT, QTZ, ETC. LARGE (4") COBBLES OF GREENSTONE; FRIABLE; MOIST.	B-7	P3	116.0'
118				RD	CUT PITCHER BARREL FROM 116.0 - 116.75' CUTTING ROCK - A LARGE COBBLE STUCK IN TUBE END. NO RECOVERY IN TUBE. SAMPLE GRAVELS.
120					
122	ML-CL		PB-5	P 10/1.2	CUT PITCHER BARREL 122 - 123.2 V. HARD TO CUT
124		121 - 139.5 CLAYEY SILT: MOD. YELLOWISH BROWN (10 YR 5/4): 60% MOD. QUICK DILATANT SILTY FINES; ~30% LOW PLASTICITY CLAYEY FINES; 10% V. FINE SAND; V HARD; DAMP; SLIGHTLY CALC.		RD	
126					
128					126.5
130					
132					
134					
136		136' THIN LENSE OF GRAVEL			CHATTER AT 136.0'

SHEET 3 OF 10

PROJECT 1886 GETR. LANDSLIDE INV. DATE DRILLED JUNE 27 1980 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
136	ML-CL	121 - 139.5 CLAYEY SILT: (con't)		RD	
138		COLOR CHANGE AT 136.5' TO GREENISH GRAY (5G 6/1), VERY CALCAREOUS			
140	GP	139.5 - 176.5 SILTY SAND/GRAVEL AS BEFORE FROM 96' - 121'			RATTLING & CHATTERING BEGINS AT 139.5'
142					
144		GRAVEL GRADES COARSER & FINER - SMALL STRINGERS OF SILT AND/OR CLAY ARE PRESENT THRU-OUT THIS UNIT.			
146					
148					147'
150					
152					
154					
156					CHATTER
158					3:10 157'
160					
162					
164					
166					
168		SAMPLE: SILTY GRAVELLY SAND MED. BLUISH GRAY (5B 5/1); 50% PREDOM. FINE GRAINED SAND & MED SAND; FRIABLE; CEMENTED; DAMP.	PB-6	P / RD	3:40 167.5 CUT PB 167.5' - 168.5' QUIT FOR 6/27/80 ON SITE 7:30 AM 6/30/80 BRIDGE AT 40'
170					
172					
174					
176		176.5 - 181' CLAYEY SILT: DUSKY YELLOW (5Y 6/4); ~100% PREDOM. LOW PLASTIC SILTY FINES; NO SAND; COLOR CHANGES AT ~179' TO MED. BLUISH GRAY (5B 5/1).			
178	ML-CL				CHATTERING QUITS.
180					
182	ML	181 - 218 GRAVELLY SILT; ~30% GRAVEL; OTHERWISE, AS ABOVE.			~183 MUD RATTLING.
184					

SHEET 4 OF 10

PROJECT 1886 GETR DATE DRILLED 6/30/80 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
184	ML	181.0 - 218.0' GRAVELLY SILT: (con't)		RD	
186		186.5' - 192' GRADES TO:			RATTLING STOPS 186.0 - 186.9 CUT PITCHER BARREL - V. HARD.
	ML	SAMPLE: SANDY SILT: MED BLUISH GRAY (5B 5/1); ~ 85% PREDOM. SILTY FINES; <15% V. FINE SAND, V. HARD, DRY, CALCAREOUS	PB - 7	PB 6/9	
188				RD	150 - 320 psi HYDRAULIC PRESSURE.
190		COLOR VARIES TO MOD OR LT BROWN (5 YR 4/4).			190 - CHATTERING
192					
194					
196		BECOME MUCH LESS CALCAREOUS			
198					
200					
202					
204					
206		~206' STRINGER OF GRAVEL			
208		207 - 213.5 GRADES TO SANDY SILT AS AT 186.5 - 192 STRINGERS OF HARD, MOD BROWN SILT OCCASIONALLY.			SMOOTH, QUIET, DRILLING PROCEEDING SLOWLY & STEADILY. FORMATION IS V. HARD.
210					210'
212					
214	SC-ML	GRADATION MORE PLASTIC			
216					
218	GM	218.0 - 315' SANDY SILTY GRAVEL: ~ 30% PREDOM. LOW PLASTICITY SILTY FINES; ~ 25% FINE SAND; ~45% GRAVEL CONSISTING OF LARGE (1¼"-2") CLASTS OF META-VOLCANICS; CHERT, QTZ, GRAYWACKE AS MINOR CONSTITUENTS.			218' VIOLENT CHATTERING
220					3:30 220.5'
222					V. HARD, SLOW DRILLING
224					LOUD CHATTER CONTINUES
226					
228					
230					
232					3:50 231'

SHEET 5 OF 10

PROJECT 1886 GETR DATE DRILLED JUN 30 1980 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
232	GM	218.0 - 315' SILTY GRAVEL: (cont)			
234					
236					235' QUIT FOR 6/30/80 LEFT SITE 5:30 ARRIVE 7:30 AM & DRILLING AT 8 AM ON 7/1/80
238					
240					
242					8:50 / 9:00 241.5'
244					
246					
248					STEADY CHATTERING
250		Graded to include sandy STRINGERS.			
252					10:25 / 10:30 252.0'
254					255' VIOLENT CHATTERING
256					
258					
260		GRAVEL GRADES FINER			
262					11:55 / 12:00 262.5'
264					
266					
268					
270		GRADES SILTIER & COLOR CHANGES TO GREENISH GRAY (5G 6/1)			270' RATTLING HOLD/! ATTEMPT TO SAMPLE w/ PD TWICE - BOTH TIMES FAILED DUE TO ROCKS FALLING IN HOLE. DECIDED NOT TO ATTEMPT A THIRD TIME.
272					
274					3:00 273' RESUME DRILLING
276					
278					
280					

SHEET 6 OF 10

PROJECT 1886 - GETR - LANDSLIDE INVEST. DATE DRILLED 6/30/80 - 7/7/80 HOLE NO. RD - 3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
280	GM	218' - 315' SILTY GRAVEL; (CON'T)			
282					4:10 / 4:20 282'
284					
286					QUIET AT 285' VIOLENT CHATTER AT 286'
288					
290					290' - QUIT @ 5:30 7/1/80 ARRIVE AT GETR 7:30 7/2/80 WEATHER: RAINY, COOL ROAD UNPASSABLE, DUE TO RAIN
292		~292 LENS OF CLAYEY GRAVEL			292.5 WALKED IN TO SITE AFTER ARRANGING TO HAVE WATER HAULED IN BY CAT.
294					
296		~295 GRAVEL IS FINER, DRILLING IS QUIETER.			INTERMITTANT CHATTER.
298					
300		GRADES COARSER			V. SLOW DRILLING
302					11:12 / 11:50 - 302.5'
304					
306					
308					AT 311.5', PULLED BIT TO CHECK FOR WEAR & REPLACE IT. HOLE WAS SMALLER THAN NEW BIT SINCE OLD BIT WAS OUT OF GAUGE. REAMED HOLE TO 312.5'. QUIT AT 5:30. 7/2/80
310					
312					5:30 312.5'
314					7/3/80 7/3/80 - start into hole @ 7:45; hole bridged @ ~ 230'; sev thin bridges to bottom
316	SM (GM)	315-433' INTBDD. SILTY SAND AND SILTY GRAVEL: lt. d. gray (5Y5/2) w/red, green, white and grn.-blk. sand grains & gravel clasts; < 50% low plastic to non-plastic, silty fines; sand f. to c. grained; gravel variable size and amount, some lenses, stringers prob. w/ > 50% gravel, max. size prob. < 1-2"; uncemented, seems v. compact-dense from drill rate & down-pressure; water content unknown; beds/lenses vary 1-2" to 1-2' w/ sand predom.			DMY logging to T.D.
318					RD w/ ~ 200 psi hydraulic down pressure Occ. mod. chatter
320					313' - 8:50 315' - 8:58 317' - 9:07 321' - 9:22
322	?				RD w/100-150 psi hydraulics
324	SM (GM)	~322' grades more gravelly w/ clasts prob. > 1/2" common			323' - 9:30, 9:39
326		~323-333' some mod. brn silty clay cuttings; may be thin stringers or wash from sidewall			325' - 9:48
328					327' - 10:03 329' - 10:11

SHEET 7 OF 10

PROJECT 1336-55TR Landslide Invest. DATE DRILLED 7/3/89 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
328	SM (GM)	315- INTBDD. SILTY SAND AND SILTY GRAVEL : con't.		RD	smoother, quieter drilling w/occ. chatter
330		higher in hole ~326½' thin lens of coarse gravel			331'-10:18
332	SM	~332½' gravel lens ~2-3" thick			mod. chatter 332½' 333'-10:27-10:35
334		~333-336 SILTY SAND: mod. grn. gray (5G5/1); pred. f.-med. sand w/scattered f. gravel			smooth drilling 335'-10:40
336	SM (GM)	~336 - grades more gravelly w/thin intbds of silty sand, sandy, silt			lt.-mod. chatter 336½' 337'-10:48
338		~337½' gravel lens 3-4" thick			RD w/~200psi hydraulics 339' 10:54 , occ. lt. chatter
340		~338-344' still pred. silty sand but w/ more gravel than previously			340' 10:57 to T.D.
342					mod. chatter 342' 343' 11:15-11:22
344		~344-348.7' grades less gravel			quiet drilling 344' 345'-11:31, brief chatter
346		~346½' thin coarse gravel lens			347'-11:38
348	GP-GM	348.7-352.5' SANDY SILTY GRAVEL: coarser gravel than for unit as a whole; clasts pred. grn-blk. meta-volc., lesser chert (red, green, white); uncemented.			brief hd. chatter 348', hd. chatter 1-3" 348¾' 349'-11:46 grades to mod. chatter
350					351'-11:57
352	SM (GM)				quiet drilling 352½ 353'-12:12
354					brief lt.-mod. chatter 354' 355'-12:25 v. brief hd. chatter
356					
358		~359-360' gravel lens			358'-12:37 mod-hd. chatter 359-360'
360					360'-12:50
362					mod., occ. chatter
364		~363-368' grades generally more gravelly w/intbdd. silty gravelly sand and silty gravel lenses			363'-1:10-1:15 mod. to lt. chatter
366					365'-1:21 brief mod. chatter 366'
368		~367-367½' gravel lens; coarse clasts, to coarse sand at 367½'			367'-1:31 mod., steady chatter 367-367½'
370		~368-372' little gravel			quiet drilling to 368' lt. to occ. mod. chatter
372		~372' coarse sand to f. gravel stringer			372'-1:55, mod. chatter 373'-2:05-2:13
374		~374' thin coarse sand to f. gravel lens			mod.-hd. chatter 374' 375'-2:27
376					

SHEET 3 OF 10

PROJECT 1835 GETR Landslide Invest. DATE DRILLED 7/3,7/80 HOLE NO. RD-3

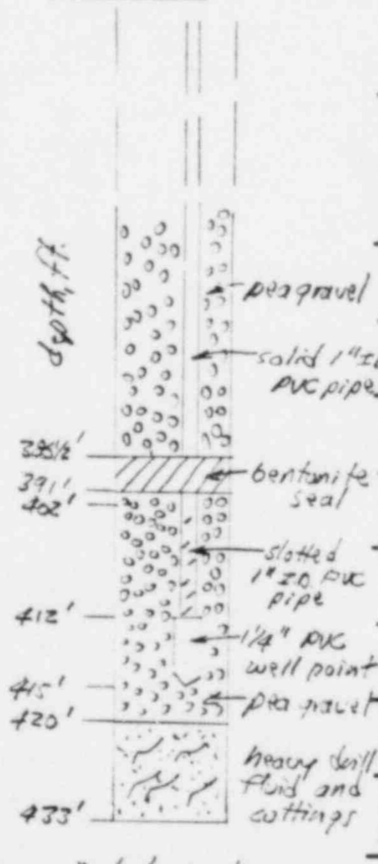
DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
376	SM	315.0-433.0' INTERBED. SILTY SAND AND SILTY GRAVEL:		RD	smoother, quicker drilling
378	(GM)	(con't)			378'-2:59
380		~375-381½' -less gravel w/a few thin c. sand, f. gravel lenses			mod.-hd. chatter 380'
382		~380' thin gravel lens ~381½' coarse sand lens			mod. chatter 381½' less chatter 382' 383'-3:04-3:07
384		~383½' thin gravel lens			brief mod.-hd. chatter 383½' v. lt. chatter 383½ to 387'
386					386'-3:24
388		~387' thin gravel lens			387'-3:29, brief hd. chatter
390		~389' thin gravel lens (1-2")			389'-3:40, hd. chatter (1-2") hd. chatter (390')
392		~392½' gravel lens (~6")			brief hd. chatter (392½') 393'-4:04-4:08
394					
396					395'-4:20
398					acc. lt.-mod. chatter
400		~398½' gravel lens (~4") grades w/some ol. gray to orangy mod. brn. sltst. rk. chip cuttings			398'-4:35 hd. chatter 398½' 400'-4:49, end shift 7/3/80 / 7/-/80
402		~401' thin gravel lens			start cleaning out hole @ 7:30, reach bottom @ 11:04; large cobble @ 84'; many bridges, cave & skas from ~180-273'
404		~402½' thin gravel lens			
406		~403' some cuttings grade slightly more clayey, low plastic, med. dk. gray (N4)			Attempt to sample @ 400'-couldn't get PB sampler past ~230' due to lg. cobbles wedging sampler; difficulty removing sampler; encountered ~2-3' of coarse gravel cave at B.H. after sampling attempt; abandon attempt to sample.
408		~404' gravel lens (3-4")			
410		~405' few gravel clasts ~406¼' scattered gravel			
412		~407 gravelly ~407½' thin gravel lens (~2")			411'-1:39 mod. chatter 404' lt. chatter 404¾' mod. chatter 405-405¼'
414		~408½' grades coarser sand w/ some f. gravel			413'-1:55-2:00
416		~410' gravelly			acc. mod. chatter 406'-422½' 415'-2:12
418		~412' scattered f. gravel ~413' scattered f. gravel			(see lith. description) 417'-2:27
420		~413½' thin gravel lens (1-2")			419'-2:35
422		~416½-417½' gravelly, binder still slightly clayey ~417½-422' scattered f. gravel			mod.-hd. chatter 422-422½' 423'-3:00
424	SM-GM	~422-422½' gravelly			

SHEET 9 OF 10

PROJECT 1885 GETR Landslide Invest. DATE DRILLED 7/7,30,31/81 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
424	SM-SC (SM)	315.0-433.0' INTERBEDDED SILTY SAND and SILTY GRAVEL: (con't.)		RD	424'-3:07
426					425'-3:12
	SM (GM)	~422½-426½' less gravelly w/ clayey silt binder			occ. H.-mod. chatter 426½-433'
428					
430		~426½-432' scattered gravel			
		~432' gravel lens (3-4")			hd. chatter 433 (1-2")
432		B.H. @ 433'			433-4:00
434					Attempt sample run @ 433'. have great difficulty coming out of hole - cobbles caught above bit; hole walls heavily disturbed coming out - abandon attempt to sample because risk of losing sampler/rods is judged too great. End shift at 5:00.
436					Terminate hole @ 433'
438					Start shift on 7/31/81 @ 7:30; drilling fluid level at ~120' after sitting overnight; hole bridged @ ~410-415' w/ cave to bottom of hole common
440					Reamed and flushed hole thoroughly
					E-logged hole w/ ESA (unreadable) 1200; ran SP-Res. and gamma probes (malfunction of SP)
					Set piezometer as shown in sketch opposite; attempted to set a piezo. higher in hole but gravel backfill bridged at ~60-73'; ended shift @ 5:00
					Start shift on 7/31/81 @ 7:30; attempted to get bridge at 60' w/ 3/4" PVC and mud pump; no success; abandoned attempt to set higher piezometer. Demobed from RD-3 @ 4:30

RD-3 Piezometer Details



not to scale

SHEET 10 OF 10

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT 1896 GETR - LANDSLIDE INVEST. DATE DRILLED JULY 9 1980 HOLE NO. RD-4
LOCATION 7' S 30° E OF RD-2, B/ WATER TANK GROUND SURFACE ELEV. ~645 (topo)
DRILLING CONTRACTOR PITCHER DRILLING LOGGED BY CJP DEPTH TO GROUND WATER ~31'
TYPE OF RIG FAILING 1500 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb, 30" UNLESS NOTED
SURFACE CONDITIONS GRAVELLY GRASSY ROAD SHOULDER WEATHER CLEAR, WARM

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0	ML	0' - 2' CLAYEY SILT: DUSKY YELLOW (5Y 6/4) TO LT OLIVE BROWN (5Y 5/6) W/ WHITE CALC. STREAKS; V. HARD; DRY.		AD	ON SITE AT RD-4 AT 9AM 7/9/80.
2	ML	2' - 6' GRADES SANDY & GRAVELLY			AUGERED DOWN 4' & SET CASING W/ BENTONITE. DRILLING W/ REVERT.
4				RD	
6	GP	6' - 14' SILTY GRAVEL: LT OLIVE BROWN (5Y 5/6); ~40% NON-PLASTIC SILTY FINES, < 10% FINE TO MED. SAND; 50 - 60% FINE TO COARSE GRAVEL CONSISTING OF CHERT, GRAYWACKE, META-VOLCANICS, ETC...; HARD; DAMP; SLIGHTLY CALCAREOUS.			CHATTER AT 6'
10			B-1	SS .5/1.5	DROVE STD SPLIT SPOON 10.0' - 11.5' 9/.5 10/.5 14/.5
12		FINES GRADE MORE PLASTIC; CONTAINS 10-15% CLAY.		RD	DRIVING ROCK IN SHOE.
14	SM-SC	14.0' - 28.0' SILTY SAND: DUSKY YELLOW (5Y 6/4); 30% NON-PLASTIC SILTY FINES; 40% FINE SAND; 30% FINE TO MED GRAVEL; FIRM; MOIST;			
18		19.5' - GRAVEL LENS			
20		20' - GRAVEL CONTENT DECREASES TO 15 - 20%; SAND CONTENT INCREASES TO 50 - 55%. GRAVEL IS GENERALLY FINER.	B-2	D 0/1.5	19.5' RATTLE. DROVE MODIFIED CALIF SAMPLER W/ 140# HAMMER 50/.5 50/.2 NO RECOVERY IN LINERS 0.2' IN SHOE.
22				RD	
28	GP	28' - 47' SILTY GRAVEL: DUSKY YELLOW (5Y 6/4); 40% NON-PLASTIC QUICKLY DILATANT SILTY FINES; < 10% FINE SAND; 50 - 60% FINE TO COARSE GRAVEL, SOME V. LARGE COBBLES; HIGH BLOW COUNTS DUE TO HARD BUT DRIVABLE GRAVEL CLASTS; MATRIX IS FRIABLE; MOIST.			
30			B-3	D 1.0/1.5	DROVE STD SPLIT SPOON 30.0 - 31.5 34/.5 44/.5 69/.5
32				RD	
34					
36					
38					
40					

SHEET 1 OF 3

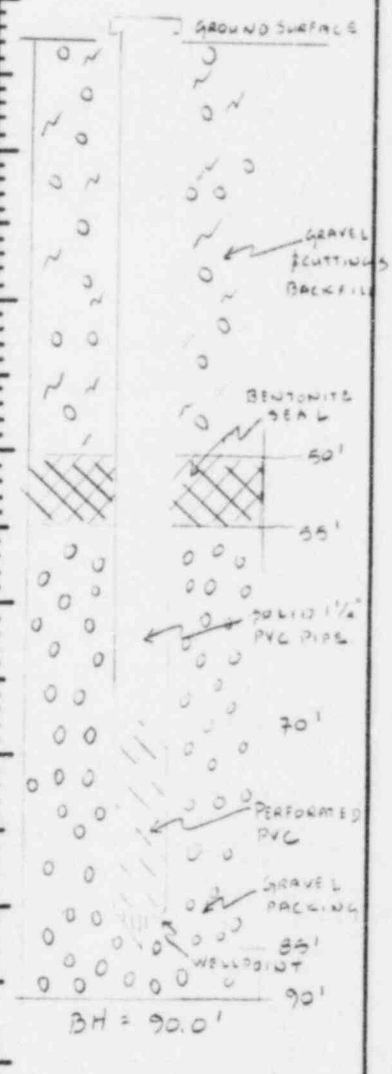
PROJECT 1886 GETR - LANDSLIDE DATE DRILLED July 9 1980 HOLE NO. RD - 4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
40, 42, 44, 46	GP	23.0' - 47.0' SILTY GRAVEL (con't) SAMPLE: SANDY SILTY GRAVEL: DUSKY YELLOW MOTTLED W/ DK. YELLOWISH ORANGE; ~7% NON-PLASTIC SILT FINES; ~10% FINE SAND; 60% COARSE TO FINE GRAVEL, ROUNDED - SUBROUNDED CLASTS OF GREYWACKE & GREENSTONE, QTZ & CHERT.	D-4	SS 08/1.5 RD	DROVE STD SPLIT SPOON 40.0' - 41.5' 33/.5 36/.5 39/.5
48, 50, 52, 54, 56	CL-ML	47.0 - 58.0 CLAYEY SILT: DUSKY YELLOW (5 Y 6/4) AND LT OLIVE BROWN (5 Y 5/6), ~100% SILT & CLAY, LOW PLASTICITY, MOD. QUICK DILATANCY FINES, NO SAND OR GRAVEL, HARD, DENSE, MOIST. SAMPLE: (PB-1) CLAYEY SILT: LT OLIVE GRAY (5 Y 5/2); 100% LOW PLASTICITY SLOW DILATANCY FINES; SOME LARGE GRAVEL CLASTS SCATTERED THROUGHOUT MAKING SAMPLING V. DIFFICULT.	PB-1, B-5, B-5, C-1, C-2	PB 1.3/2.0, D .6/1.3, D 0/1.5, RD	CUT PITCHER BARREL 50' - 52.0' H₂O PRES = 250 psi. DROVE MOD. CALIF SAMPLER 52.0' - 53.3' (140# HAMMER) 37/.5 50/.5 50/.3. DROVE MOD. CALIF SAMPLER 53.3' - 54.8' (140# HAMMER) 52/.5 74/.5 84/.5 ROCK IN SHOE - COUNTS INVALID. SLOUGH (?) IN C-2 - SAVED ANYWAY.
58, 60, 62	ML-CL	58.0 - 62.8 SANDY SILT: LT OLIVE GRAY (5 Y 5/2); 60% SILT & CLAY FINES; QUICK DILATANCY; 40% V. FINE SAND, PREDOM. QTZ; HARD BUT FRIABLE; DAMP.	PB-2	PB 10/2.6	CUT PITCHER BARREL 59.0 - 61.5 HYD PRESS = 270 psi
64, 66	ML	62.8 - 67.0 CLAYEY SILT: MODERATE BROWN (5 YR 4/6); 85% LOW PLASTIC, SLOW DILATANCY FINES; 15% FINE GRAVEL & SOME SAND; HARD; MOD. DRY STRENGTH; MOIST; CONTAINS CALCAREOUS STREAKS & BLEBS.	C-3, C-4, PB-3, C-5	D 1.3/1.5, RD, PB 2.2/2.2, D .4/1.0, RD	DROVE MOD. CALIF SAMPLER 61.8' - 63.0' w/ 300# HAMMER 11/.5 15/.5 17/.5. CUT PITCHER BARREL 64.0' - 66.2' HYD PRES = 170 psi CHATTERED WHILE CUTTING. DROVE MOD. CALIF SAMPLER 66.2 - 67.2 (300# HAMMER) 13/.5 45/.5 DRIVING ROCK.
68	ML	67.0 - 70.0: SILT: MODERATE BROWN ~100% SILTY, QUICK DILATANT FINES; LITTLE OR NO GRAVEL; FAST DRILLING; FIRM BUT FRIABLE.			
70, 72, 74, 76, 78	ML	70.0 - 79.0' CLAYEY SILT: AS ABOVE W/ SOME LARGE GRAVEL CLASTS. COLOR CHANGE TO DK YELLOWISH ORANGE (10YR 6/6)	PB-4	PB 1.0/2.5, RD	CUT PITCHER BARREL 71.0 - 73.5 HYD PRES = 120 psi
80, 82, 84, 86, 88	ML	79.0 - 90.0 GRAVELLY SANDY SILT: 60% LOW TO NON-PLASTIC SILTY FINES; 10-20% FINE SAND, 20-30% GRAVEL OF ALL SIZES - CLASTS UP TO SEVERAL INCHES; SAMPLE: GRAVELLY SANDY SILT: MOTTLED DK YELLISH ORANGE & MOD YELLISH BROWN (10YR 5/4); 60% NON-PLASTIC SILT; 10-20% FINE SAND; 30-40% ANGULAR TO SUBANG. GRAVEL UP TO 3" SIZE; V. FIRM; MOIST; DENSE.	B-7	D .8/.9, RD	CHATTERING @ 79.0' & CONTINUES TO BOTTOM OF HOLE. DROVE STD SPLIT SPOON FROM 83.0 - 83.9 42/.5 102/.4 DRIVING ROCK.

SHEET 2 OF 3

PROJECT 1886 GETR - LANDSLIDE INV DATE DRILLED JULY 9 1980 HOLE NO. RD-4

DEPTH	CLASS.		FIELD DESCRIPTION	SAMPLE		MODE	REMARKS
88		ML	79.0 - 90.0 GRAVELLY SANDYSILT:			RD	
90			(CON'T)				
			B.H. = 90.0				TERMINATED HOLE AT 90.0'
92							SUFFICIENT DATA / SAMPLES OBTAINED.
94							$1\frac{1}{4}$" PVC PIEZOMETER INSTALLED W/ WELL POINT AT 85.0'.
96							BENTONITE SEAL AT 50'-55'.
98							PIEZOMETER DETAILS:
100							



SHEET 3 OF 3

APPENDIX B
LABORATORY INVESTIGATION

Introduction

The purpose of the laboratory testing program was to evaluate the static strength characteristics of the various subsurface soils found in the vicinity of the GETR facility. To fulfill this objective, various laboratory tests were performed on selected samples of the subsurface materials obtained during the field investigation conducted as part of this study (see Appendix A). Moisture and density tests, grain size analyses and Atterberg limits tests were conducted to aid in the identification and correlation of the various soil types. Static shear strengths of the various materials were evaluated by performing consolidated-undrained triaxial tests with pore pressure measurements and consolidated-quick direct shear tests on relatively undisturbed Pitcher samples. In addition to the static shear strength tests on undisturbed Pitcher samples noted above, a series of consolidated-quick direct shear tests were performed on remolded test samples. To expedite the laboratory testing program, three triaxial tests and the direct shear tests were performed at Dames and Moore, San Francisco.

It should be noted that as part of previous investigations (Dames and Moore, 1960; Shannon and Wilson, 1973) a series of consolidated-undrained triaxial tests were performed. The previous triaxial tests were conducted using very low consolidation stresses which are not, in general, within the range of stress appropriate for the analyses conducted during this investigation. In addition, previous triaxial tests were performed to evaluate the total shear strength characteristics of the materials and therefore the test results could not be used to evaluate effective shear strengths. Although the previous triaxial test results are limited, they were used to the fullest extent possible to supplement the test results obtained during this investigation.

Material Types

As discussed in a previous section of this report, four material types were selected for testing. The description of each of these material types is summarized as follows:

Material Type 1 - Dark yellowish brown (10 YR 4/2) to light olive gray (5 Y 5/2); SANDY SILT to SILTY CLAY/CLAYEY SILT (ML to CL/ML); low plastic fines, some fine sand; pervasive carbonate with some hard cemented zones; very dense to hard; moist.

Material Type 2 - Moderate yellowish brown (10 YR 5/4); CLAYEY SILT/SILTY CLAY (ML/CL); slight to low plastic fines, some fine sand; very stiff to hard, moist.

Material Type 3 - Moderate brown (5 YR 4/4); SANDY CLAY TO GRAVELY CLAY (CL); medium to high plasticity; fine to coarse sand; fine to coarse gravel; very stiff to hard; moist.

Material Type 4 - Moderate yellowish brown (10 YR 5/4); CLAYEY GRAVELY SAND (SC) low to medium plastic fines; fine to coarse sand; fine to coarse gravel; very dense; moist.

Material types 1, 2 and 3 characteristically contained some randomly oriented slickensided fissures. Static triaxial tests performed on these materials generally failed along these pre-existing fissures.

Atterberg limits test results obtained for selected samples are plotted on Figure B-1 and indicate that the materials have a relatively wide range of plasticity. Gradation curves for selected samples are shown on Figures B-2 thru B-5 for material types 1 thru 4, respectively. Gradation tests were performed on most of the triaxial test samples in addition to a few samples which were not triaxially tested. Gradation and Atterberg limits test results obtained from triaxial test samples performed by other investigators (Dames and Moore, 1960; Shannon and Wilson, 1973) which were considered appropriate for the analyses performed for this investigation are shown for comparative purposes.

Sample Preparation

Samples designated for static triaxial testing were extruded with care from the sampling tubes to minimize sample disturbance. Sample tubes were cut to a length of approximately 6 inches using a slow speed motor driven horizontal bandsaw. The cut portion was then vertically extruded into a thin rubber membrane using a motor driven hydraulic extruder. Samples were then trimmed, weighed, and measured prior to placing them into the triaxial testing apparatus.

The soil samples that were triaxially tested were generally extruded with considerable ease. Some difficulty was encountered, however, when trying to extrude testable samples of material type 4. Because of its high gravel content, sampling of this material during the field exploration program was extremely difficult. Extruded samples were often disturbed or contained large voids or pieces of coarse gravel. It is for these reasons that only one testable sample was obtained for this material type.

Only one sample (RD-3/PB-3, material type 1) designed for triaxial testing was unintentionally disturbed during sample preparation. During trimming, the sample split down a pre-existing fracture plane and the sample had to be discarded.

Direct shear test samples of relatively undisturbed materials were extruded using the same procedures as the triaxial test samples. Samples were trimmed to a diameter of approximately 2.4 inches in order to fit into the direct shear testing apparatus. Some samples had to be slightly patched since some small voids were created along the sides of the sample during the trimming process.

Remolded direct shear samples were prepared by compacting the selected materials into 2 one-half inch direct shear testing rings to an estimated in-situ density and fully saturated moisture content. After the compaction process, the two direct shear testing rings were put together and placed into the direct shear testing apparatus. This procedure resulted in an artifical planar discontinuity within the sample along which shear displacement occurred during the test.

Static Triaxial Tests

A series of static triaxial tests were conducted on relatively undisturbed Pitcher samples (3-inch OD) of the various material types selected for testing. A total of 11 samples were tested. A summary of the static triaxial tests performed during this investigation along with pertinent testing and sample information is presented in Table B-1.

Prior to testing, all samples were fully saturated and consolidated isotropically to a range of effective stress conditions representative of those existing at considerable depth. It should be noted that in most cases the samples that were tested were obtained from relatively shallow depths (45.0 to 62.0 feet) and that the effective stresses to which the samples were consolidated are considerably higher than the stresses the samples had experienced in-situ. This procedure was used to determine the material strength characteristics over a range of stresses considered appropriate for the stability analyses.

Tests were run at strain rates ranging from 3 to 10 percent per hour to ensure reasonable pore pressure equilization throughout the sample. This procedure was followed to allow for the evaluation of effective strength parameters of the various materials even though the main objective of the testing program was to evaluate total strength parameters.

Most tests were continued until axial strains of approximately 20 percent had developed, however, some of the tests could only be continued to approximately 10 to 15 percent due to the mode of failure of the samples. Stress-strain relationships corresponding to each test are plotted on Figures B-6 through B-13. Photographs of the failed triaxial test soil samples are shown in Figures B-14 through B-17.

As can be seen by the stress versus axial strain relationships and the photographs of the test specimens, the various materials exhibit two distinct modes of bahavior during axial loadings. The first mode of behavior shows that the deviator stress ($\sigma_1 - \sigma_3$) reaches a distinct peak at relatively low axial strain (approximately 3 percent) at which point the deviator stress starts to drop off. This type of behavior was most prevalent in those materials which tended to fail along a pre-existing fracture planes (material types 1 and 2). The triaxial test performed on material type 1 at a confining stress of 7200 psf (Figure B-6) was the only exception even though a fracture plane was present in this sample. For this test the deviator stress increased with increasing strain, however, the soil strength as defined by the ratio of the effective vertical stress to the effective confining stress was essentially constant for axial strains greater than 9 percent. Dramatic drops in the deviator stress were recorded in the triaxial tests shown in Figures B-7 and B-9. These tests show a drop-off in the deviator stress of 20 to 25 percent over an axial strain increase of approximately 2 to 3 percent. This drop off in strength may be the result of breaking of cementation bonds between the pre-existing fracture planes. The triaxial test results shown in Figures B-8 and B-10 show a less dramatic drop-off in the deviator stress with a reduction of approximately 8 to 14 percent over the axial strain range of 3 to 12 percent. It should be noted that once the peak strength of these materials was reached, the strength measured at higher strains probably represents the residual sliding frictional strength along the failed fracture plane.

The second mode of behavior exhibited by the remaining triaxial test specimens was a general increase in deviator stress recorded during the entire test. This behavior was prevalent in material types 3 and 4 which tended to bulge during failure versus those that failed along a pre-existing fracture plane. The triaxial test results shown in Figure B-12 shows a decrease in deviator stress for strains greater than 4 percent, however, the photograph of this sample shown in Figure B-16, shows that this sample tilted and bulged to one side which may explain this sample's strength behavior.

Direct Shear Tests

A series of consolidated-quick direct shear tests were performed on both relatively undisturbed and remolded soil specimens of material types 1 and 2. Samples of material types 3 and 4 could not be tested in direct shear since they contained a large percentage of fine to coarse gravel. Undisturbed and remolded soil samples of material types 1 and 2 were prepared for testing as previously described in the section titled Sample Preparation. Each sample was submerged in water and consolidated to a specified normal pressure by applying incremental staged loadings. After consolidation , the samples were stressed to failure by applying a shearing load to the top ring of the direct shear testing apparatus.

The direct shear tests were performed at strain rates which would complete the test in approximately 10 to 20 minutes. The shear strengths derived from these tests represent the soil's total strength since the shearing rates were too fast to allow drainage of the developed excess pore pressures. Therefore, these direct shear test results can only be used to develop total strength parameters for the materials tested.

A summary of the direct shear tests performed during this investigation along with pertinent testing and sample information is presented in Table B-2. Applied shear force versus shear deflection for each test are plotted on Figures B-18 and B-19 along with axial deflection versus shear deflection. It should be noted that the area of the failure plane in a direct shear sample decreases as the deflection of the sample is increased. Therefore, the normal and shear stresses applied during the direct shear tests must be corrected to account for the changes in the area of the failure planes, particularly at higher sample deflections. A summary of the uncorrected and corrected direct shear test results is given in Table B-3 for both the peak shear force measured during the test and at a deflection corresponding to 10 percent shear strain.

The direct shear test results for material type 1 indicate that the strength of the undisturbed soil samples have a strength behavior which appears to be independent of the applied normal stress. This may be the result of the varying amounts of cementation found in this material type which may tend to mask any strength increase trends. The remolded test samples, on the other hand, show an increase in shear strength with increasing applied normal stress.

The direct shear test results summarized in Table B-4 for material type 2 indicate that there is an increase in shear strength with increasing applied normal stress for both the undisturbed and remolded test samples. The remolded samples, however, have a shear strength at 10 percent shear strain which is approximately 30 to 40 percent less than the strengths of the undisturbed samples.

Supplementary Test Data

Results of consolidated undrained triaxial tests performed by previous investigators (Dames and Moore, 1960; Shannon and Wilson, 1973) were reviewed and used to supplement the test data obtained during this investigation. Results from six consolidated undrained triaxial tests performed on materials similar to material types 3 and 4 were obtained from the previous investigations. A summary of pertinent sample and testing information is presented in Table B-4.

As previously discussed, these tests were conducted at relatively low consolidation stresses and therefore help define the soil strength at low to moderate consolidation stress levels. However, results of these tests can only be used to supplement the total strength of the materials since the pore water pressures developed during testing were not recorded.

Interpretation of Test Results

Effective Strength Parameters

Results of the consolidated-undrained triaxial tests with pore pressure measurements were used to develop effective strength envelopes for the materials tested. Effective strength envelopes are shown at the top of Figures B-20 through B-23 for material types 1 through 4, respectively, and correspond to an axial strain of 10 percent. This value of strain was chosen since the strength of the various materials, as defined by the ratio of the effective vertical stress to the effective confining stress (σ_1' / σ_3'), was always found to be less than the recorded peak strength.

The Mohr circles used in the development of the effective strength envelopes are also drawn in Figures B-20 through B-23. The Mohr circles corresponding to the highest effective confining stress for material types 1, 2, and 3 (Figures B-20, B-21, and B-22) seem to indicate that the strength of these materials were not greatly influenced by the effective confining pressure, that is the effective strength envelope at high principal stresses is extremely flat. However, it is our judgement that the results from triaxial tests performed at the relatively high confining pressures (test results shown in Figures B-8, B-10, and B-12) are probably not representative of the true strength of these materials. Published effective strength paramaters derived from tests performed on similar material types such as London Clay (Bishop et al., 1965) support this conclusion. Published data do indicate that the effective strength envelope should "flatten out" at relatively high confining pressures. However, the published strength envelopes do not flatten out to the same degree as is suggested by the limited number of triaxial tests performed for this investigation.

As was previously discussed, the triaxial tests performed on material types 1 and 2 all failed along an apparent pre-existing fracture plane. The peak strengths recorded for each of these tests probably represents the strength of these pre-existing fracture surfaces. In addition, photographs of the samples taken after each test show that the top cap of the triaxial test apparatus underwent an extreme rotation during testing. This would, in effect, reduce the normal stress acting on the failure plane of the sample which would result in a lower shearing force required to strain the sample. These observations help explain the unusually low effective strength envelopes derived from the triaxial tests performed for this investigation.

The effective strength envelope for material type 4 is shown in the top of Figure B-23. As was previously discussed, only one sample of this material type was found to be testable. The high percentage of gravel and cobbles present in this material made sampling extremely difficult. The strength envelope was developed using the results of the one triaxial test and published strength parameters for similar materials. It should be noted that the strength envelope has been flattened at high effective principal stresses. The rate at which the strength envelope was flattened was determined by findings presented by Leps (1970). It is our judgement that the effective strength envelope shown in Figure B-23 for material type 4 represents a conservative estimate of this material's strength. Pitcher tube samples from the same geologic unit from which the one testable sample of material type 4 was obtained generally contained a higher percentage of gravel. Boring logs obtained during the field exploration program indicate that the unit also contains scattered cobbles. Strengths of these materials should be significantly stronger than that of the tested triaxial test sample.

Total Strength Parameters

Total strength parameters were developed using triaxial test results obtained during this and previous investigations as well as the direct shear test results obtained from undisturbed and remolded test specimens. Total strength envelopes are shown at the bottom of Figures B-20 through B-23 for material types 1 through 4, respectively. The total strength envelopes were developed by plotting the undrained shear strength versus the normal stress on the failure plane at the end of consolidation (τ_{ff} vs. σ'_{fc}).

Total strength envelopes for material types 1 and 2 were developed for both undisturbed and remolded soil samples. The total strength envelopes developed for remolded soil samples have slopes 12 to 13 degrees less than the undisturbed samples for normal consolidation stresses generally less than 20 ksf. For normal consolidation stresses greater than approximately 20 ksf, the remolded total strength envelope was drawn parallel to the undisturbed total strength envelope.

Total strength envelopes of material types 3 and 4 were developed for undisturbed soil samples only. As previously discussed, direct shear tests were not performed on these materials and therefore, remolded soil strengths were not obtained. Triaxial test results obtained from previous investigators were used where applicable. The strength envelope for material type 4, Figure B-23 was flattened at high normal consolidation stresses in a similar fashion as the effective strength envelope shown in the top of the figure.

As in the case of the effective strength parameters, previously discussed, it is our judgement that the total strength parameters shown in the bottom of Figures B-20 through B-23 probably underestimate the true strength of these materials. The actual total strengths of the materials are probably much greater than those measured during testing since the total strength of the materials are subject to the same limitations as the derived effective strength.

TABLE B-1

SUMMARY OF TRIAXIAL TEST DATA

Material Type	Boring/ Sample	Depth of Sample Tested (Feet)	σ'_m (PSF)	Before Consolidation W/C (%)	Before Consolidation γ_{dry} (PCF)	After Consolidation W/C (%)	After Consolidation γ_{dry} (PCF)	Description
1	RD-2/PB-4	53.0-55.5	7200	20.7	109.2	20.1	109.7	Dark Yellowish Brown (10 YR 4/2) SILTY CLAY/CLAYEY SILT (CL-ML)
1	RD-3/[illegible]-3	45.0-47.5	15840	16.8	116.9	14.5	117.4	Light Olive Gray (5 Y 5/2) SANDY SILT (ML)
1	RD-2/PB-3	48.5-51.0	50400	21.1	106.2	19.3	117.5	Light Olive Gray (5 Y 5/2) CLAYEY SILT TO SILTY CLAY (ML-CL)
2	RD-3/PB-4	55.0-57.5	15840	14.9	116.3	19.2	116.8	Moderate Yellowish Brown (10 YR 5/4) CLAYEY SILT/SILTY CLAY (ML-CL)
2	RD-3/PB-4	55.0-57.5	50400	19.2	110.9	17.5	119.8	Moderate Yellowish Brown (10 YR 5/4) CLAYEY SILT/SILTY CLAY (ML-CL)
3	RD-2/PB-5	60.0-62.5	10800	11.7	119.9	15.6	120.4	Moderate Brown (5 YR 4/4) SANDY CLAY WITH GRAVEL (CL)
3	RD-2/PB-5	60.0-62.5	34560	18.6	111.0	17.3	117.3	Moderate Brown (5 YR 4/4) GRAVELY CLAY (CH)
4	RD-2/PB-11	112.0-113.5	165[illegible]	16.8	118.8	10.7	119.5	Moderate Yellowish Brown (10 YR 5/4) GRAVELY CLAYEY SAND (SC)

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TABLE B-2

SUMMARY OF DIRECT SHEAR TEST DATA

Material Type	Boring/ Sample	Depth of Sample Tested (Feet)	σ_n' (PSF)	Before Consolidation W/C (%)	Before Consolidation γ_{dry} (PCF)	After Consolidation W /C (%)	After Consolidation γ_{dry} (PCF)	Description
1	RD-2/PB-4	53.0-55.5	5760	28.1	94.5	29.3	99.7	Dark Yellowish Brown (10 YR 4/2) SILTY CLAY/CLAYEY SILT (CL-ML)
1	"	"	8640	26.5	98.1	28.7	101.9	
1	"	"	11520	27.1	98.1	27.1	102.6	
1*	"	"	5760	24.5	100.1	26.7	102.2	
1*	"	"	8640	24.1	100.2	26.0	102.4	
1*	"	"	11520	24.0	100.4	25.8	103.9	
2	RD-3/PB-4	55.0-57.5	5760	17.9	111.6	24.4	113.3	Moderate Yellowish Brown (10 YR 5/4) CLAYEY SILT/SILTY CLAY (ML-CL)
2	"	"	8640	17.8	110.8	24.3	114.4	
2	"	"	11520	17.6	111.6	22.1	114.8	
2*	"	"	5760	20.5	107.8	22.6	110.8	
2*	"	"	8640	20.4	108.8	21.4	112.4	
2*	"	"	11520	20.0	108.0	21.5	111.5	

* Test on remolded sample

TABLE B-3

SUMMARY OF DIRECT SHEAR TEST RESULTS

MATERIAL TYPE 1

Boring #/ Sample #	δ, in. (ϵ, %)	σ_n' psf	σ_n corr. psf	τ psi	τ corr. psf	δ, in. (ϵ, %)	σ_n corr. psf	τ psf	τ corr. psf
RD-2/PB-4	0.11 (4.5)	5760	6077	1920	2026	0.06 (2.5)	5914	2362	2425
"	0.24 (10.0)	5760	6624	1920	2208	0.24 (10.0)	6624	2165	2490
RD-2/PB-4	0.08 (3.1)	8640	8992	5052	5258	0.05 (2.1)	8871	3355	3445
"	0.24 (10.0)	8640	9936	4380	5037	0.24 (10.0)	9936	2875	3306
RD-2/PB-4	0.09 (3.5)	11520	12071	3540	3709	0.07 (2.7)	11866	3816	3930
"	0.24 (10.0)	11520	13248	3240	3726	0.24 (10.0)	13248	3446	3963

MATERIAL TYPE 2

Boring #/ Sample #	δ, in. (ϵ, %)	σ_n' psf	σ_n corr. psf	τ psi	τ corr. psf	δ, in. (ϵ, %)	σ_n corr. psf	τ psf	τ corr. psf
RD-3/PB-4	[illegible] (4.5)	5760	6077	5028	5305	0.07 (2.9)	5979	2597	2696
"	0.24 (10.0)	5760	6624	3636	4181	0.24 (10.0)	6624	2491	2865
RD-3/PB-4	0.14 (5.8)	8640	9331	4740	5119	0.05 (2.1)	8871	3534	3598
"	0.24 (10.0)	8640	9936	4440	5160	0.24 (10.0)	9936	3182	3659
RD-3/PB-4	0.12 (4.8)	11520	12211	7248	7683	0.16 (6.6)	12498	4066	4648
"	0.24 (10.0)	11520	13248	6876	7907	0.24 (10.0)	13243	4042	4648

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TABLE B-4

SUMMARY OF MISCELLANEOUS TEST DATA

Material Type	Boring/ Sample	Depth of Sample Tested (Feet)	σ'_m (PSF)	Before Consolidation W/C (%)	Before Consolidation γ_{dry} (PCF)	After Consolidation W /C (%)	After Consolidation γ_{dry} (PCF)	Description
1	RD-2/PB-4	53.0-55.0	—	21.4	103.5	—	—	Dark Yellowish Brown (10 YR 4/2) SILTY CLAY/CLAYEY SILT (CL-ML)
3	B-1A/S-1 Top[1]	20.0-22.5	2100	15.2	118.0	14.6	—	Brown Fine to Coarse SANDY CLAY (CL) With Trace GRAVEL
3	B-1A/S-1 Bottom	20.0-22.5	700	15.2	116.0	17.1	—	Brown Fine to Coarse SANDY CLAY (CL) With Trace GRAVEL
4	B-2/S-11[2]	50.0	6200	10.8	127.0	—	—	Brown SANDY CLAY With GRAVEL (CL)
4	B-2/S-5[1]	21.5-24.0	2800	12.3	128.0	10.6	—	Brown Clayey Fine to Coarse SAND AND GRAVEL to Fine to Coarse SANDY GRAVELLY CLAY (SC-CL)
4	B-1/S-7[1]	24.0-26.0	4220	12.0	126.0	11.8	—	Brown Clayey Fine to Coarse SAND AND GRAVEL (SC)
4	B-1/S-8[1]	27.0-29.5	4220	14.5	118.0	15.0	—	Brown Clayey Fine to Coarse SAND AND GRAVEL (SC)
4	RD-2/PB-8	90.0-92.5	—	12.2	125.7	—	—	Moderate Yellowish Brown (10 YR 5/4) CLAYEY SAND-GRAVEL (SC/GC)

1 Data from Shannon and Wilson, 1973.

2 Data from Dames and Moore, 1960.

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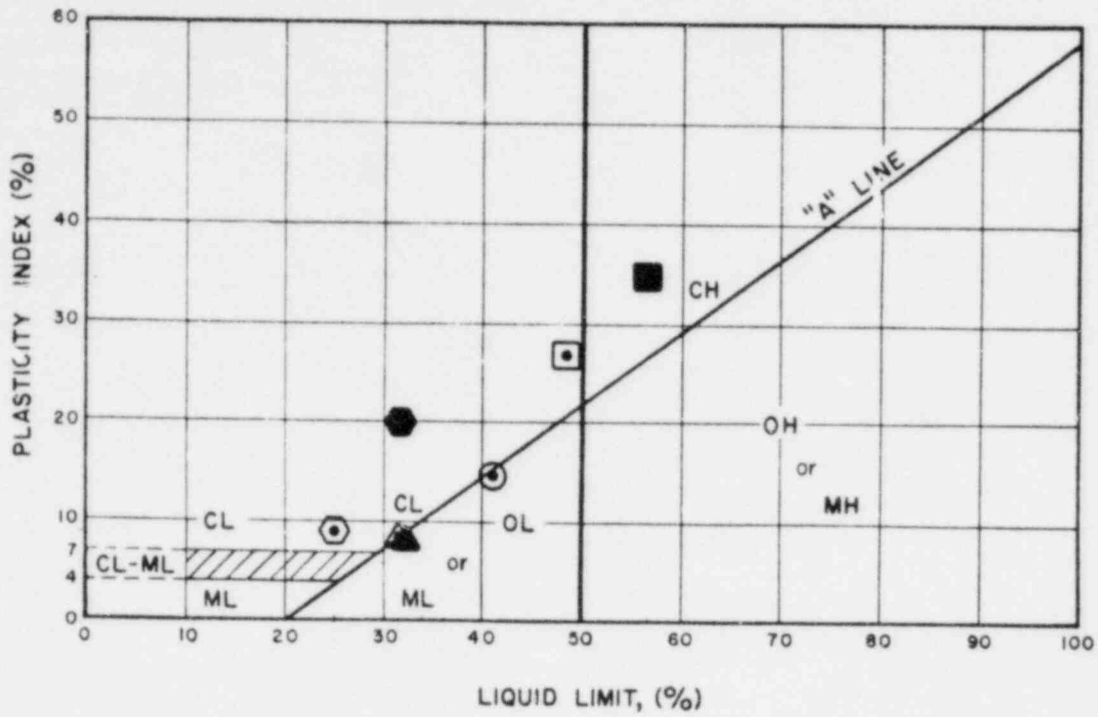

MATERIAL TYPE	SYMBOL	BORING NO.	DEPTH, FT.	LIQUID LIMIT, %	PLASTICITY INDEX, %	USC SYMBOL
1	△	RD-2/PB-4	53 - 55.5	31.5	8.6	CL/ML
1	▲	RD-3/PB-3	45 - 47.5	32.0	8.1	CL/ML
2	⊙	RD-3/PB-4	55 - 57.5	41.0	14.4	ML
3	⊡	RD-2/PB-5	60 - 62.5	48.5	26.5	CL
3	■	RD-2/PB-5	60 - 62.5	57.0	34.3	CH
4	⬡	RD-2/PB-12	118-120	25.0	9.0	CL
4	⬢[1]	B-1A/S-1	20-22.5	32.0	20.0	CL

1. Data from Shannon and Wilson, 1973.

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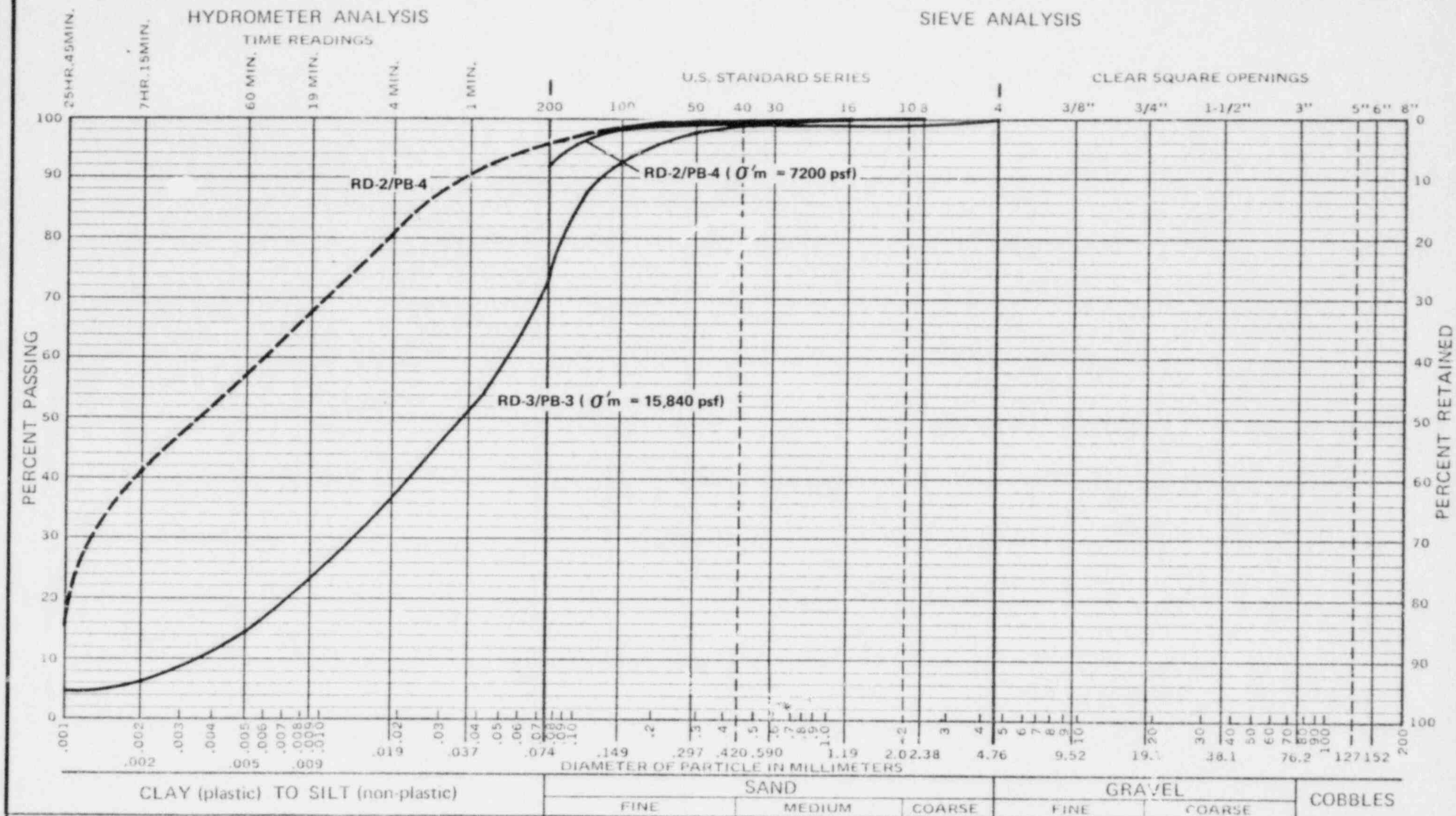
GETR LANDSLIDE STABILITY ANALYSIS

SUMMARY OF ATTERBERG LIMITS

		Project No.	Figure No.
Checked by D M Jacobs	Date 8/28/80	1886	B-1
Approved by R. L. Meehan	Date 8/28/80		



EXPLANATION

Boring number/sample number (σ'_m = mean effective confining stress used in static triaxial test)

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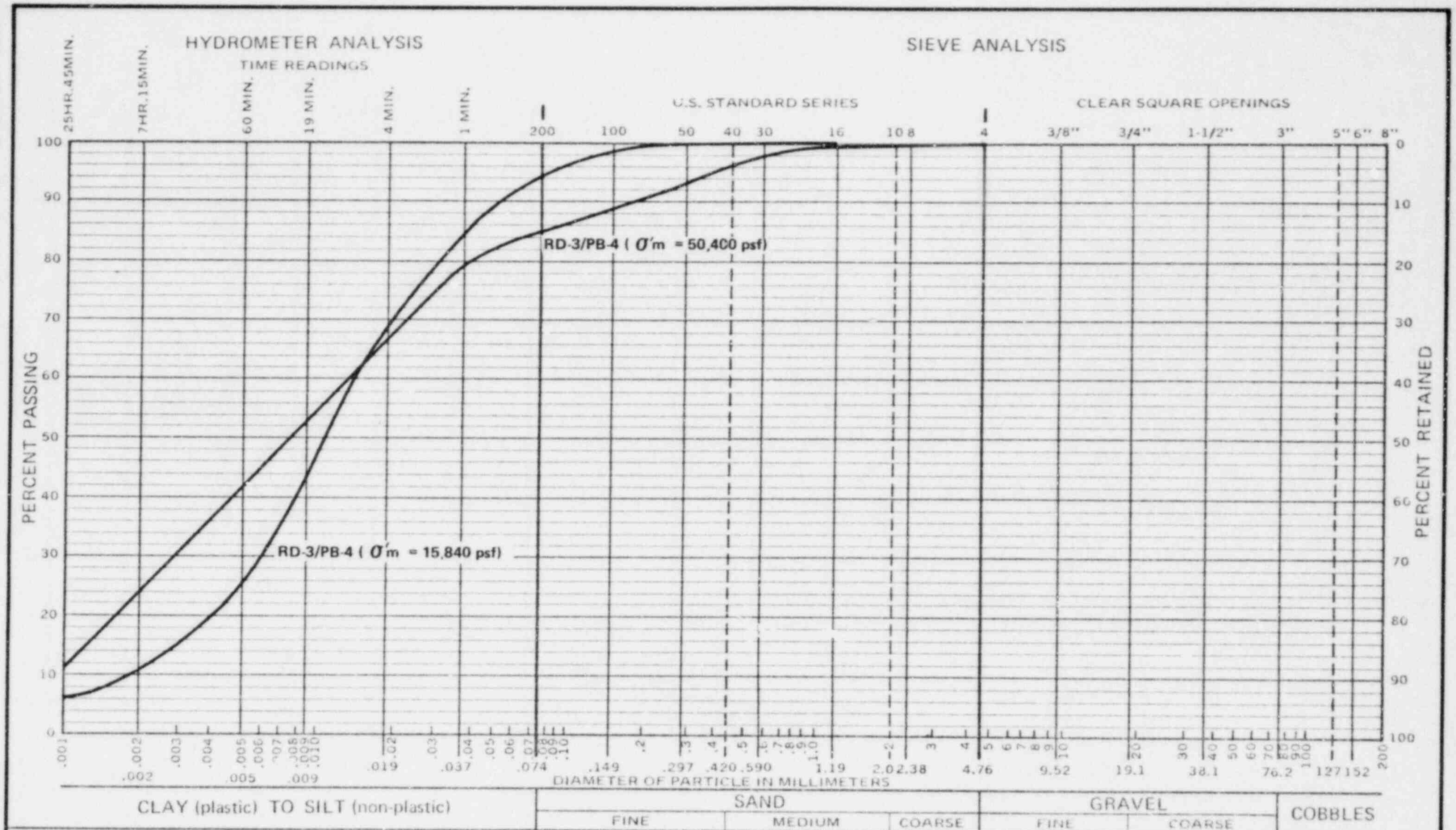
GETR LANDSLIDE STABILITY ANALYSIS

GRADATIONS – MATERIAL TYPE 1

Checked by	Approved by	Project No.	Figure No.
Date 8/28/80	Date 8/28/80	86	B-2



EXPLANATION

Boring number/sample number (σ'_m = mean effective confining stress used in static triaxial test)

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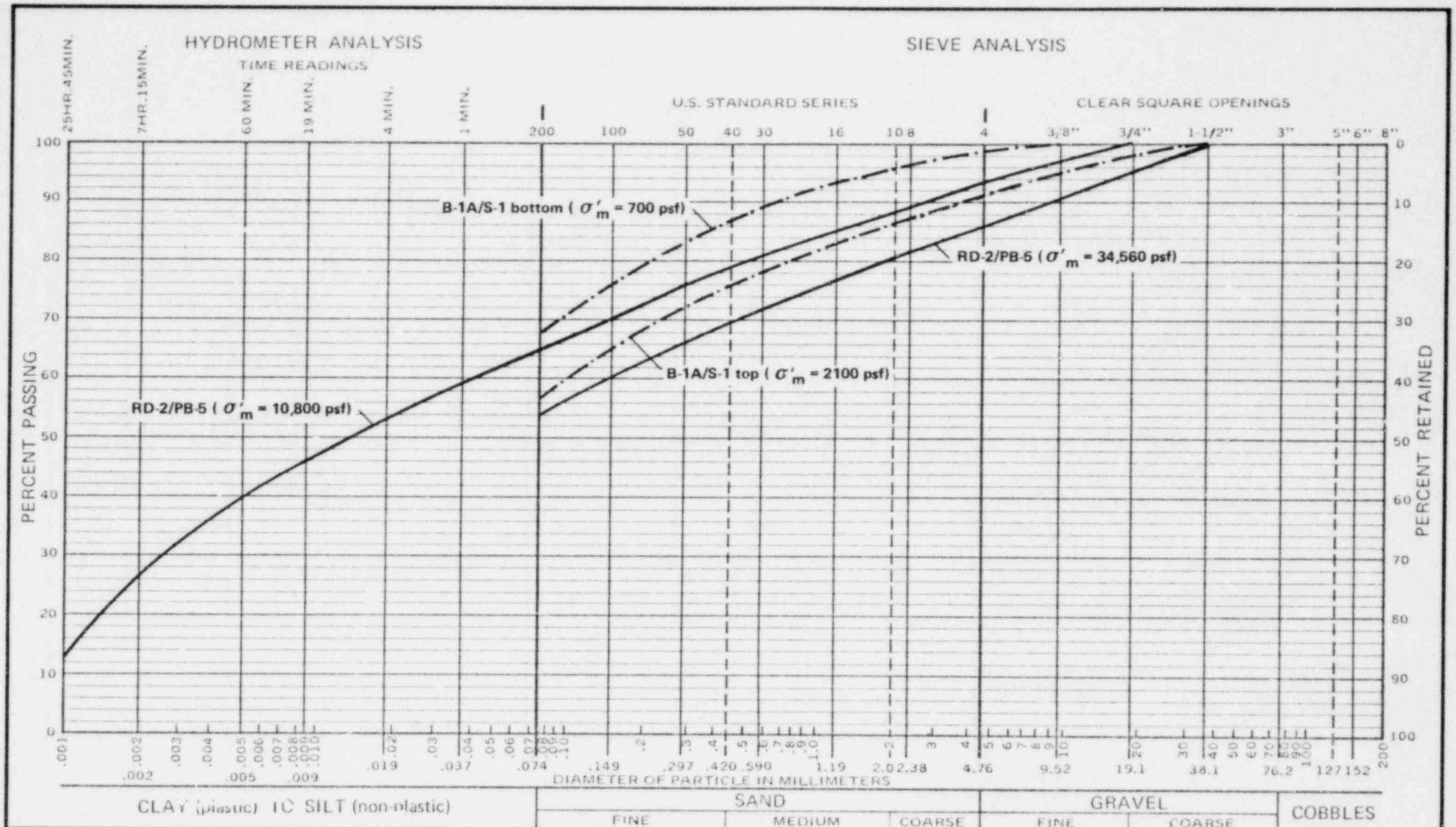
GETR LANDSLIDE STABILITY ANALYSIS

GRADATIONS – MATERIAL TYPE 2

Checked by DM Yoder	Date 8/28/80	Project No. 1886	Figure No. B-3
Approved by RC Nathan	Date 8/28/80		



EXPLANATION

Boring number/sample number (σ'_m = mean effective confining stress used in static triaxial test)

NOTE

Gradations B-1A/S-1 top and B-1A/S-1 bottom replotted from Figure 7, respectively, Shannon and Wilson, 1973.

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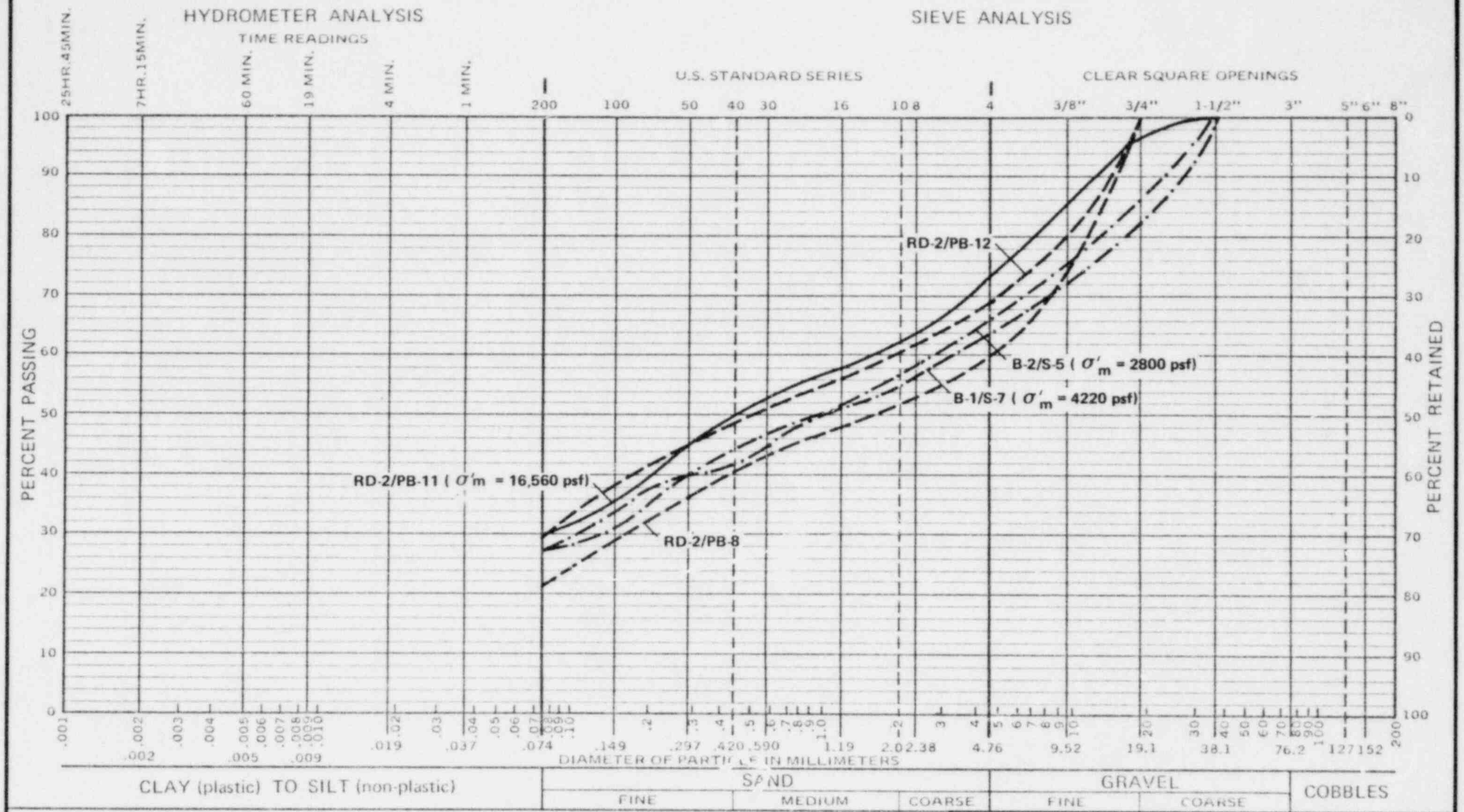
GETR LANDSLIDE STABILITY ANALYSIS

GRADATIONS – MATERIAL TYPE 3

Checked by *[signature]*	Date 8/28/80	Project No.	Figure No.
Approved by *[signature]*	Date 8/28/80	1886	B-4



EXPLANATION

Boring number/sample number (σ'_m = mean effective confining stress used in static triaxial test)

NOTE

Gradations B-2/S-5 and B-1/S-7 replotted from Figures 4 and 5, respectively, Shannon and Wilson, 1973.

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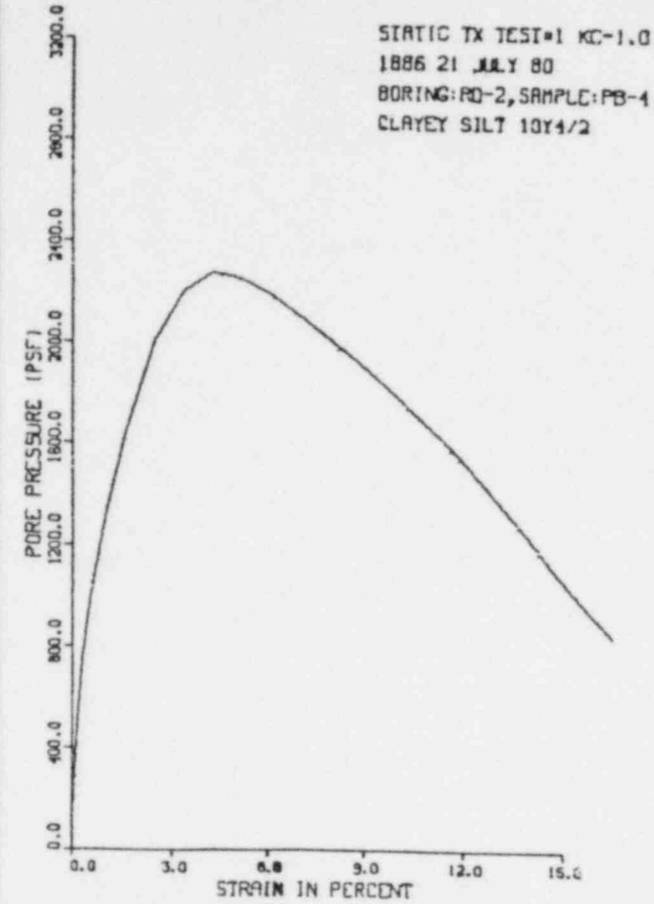
GETR LANDSLIDE STABILITY ANALYSIS

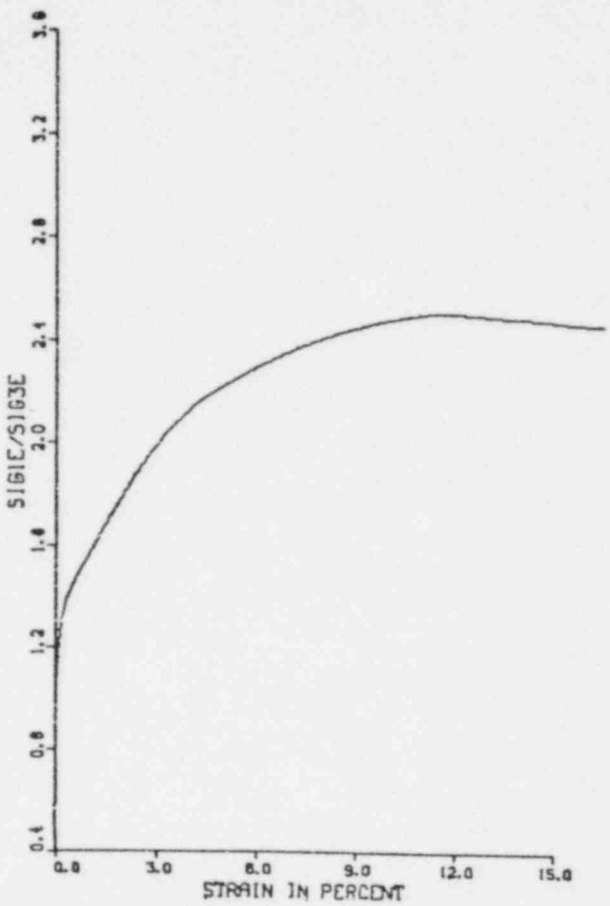
GRADATIONS – MATERIAL TYPE 4

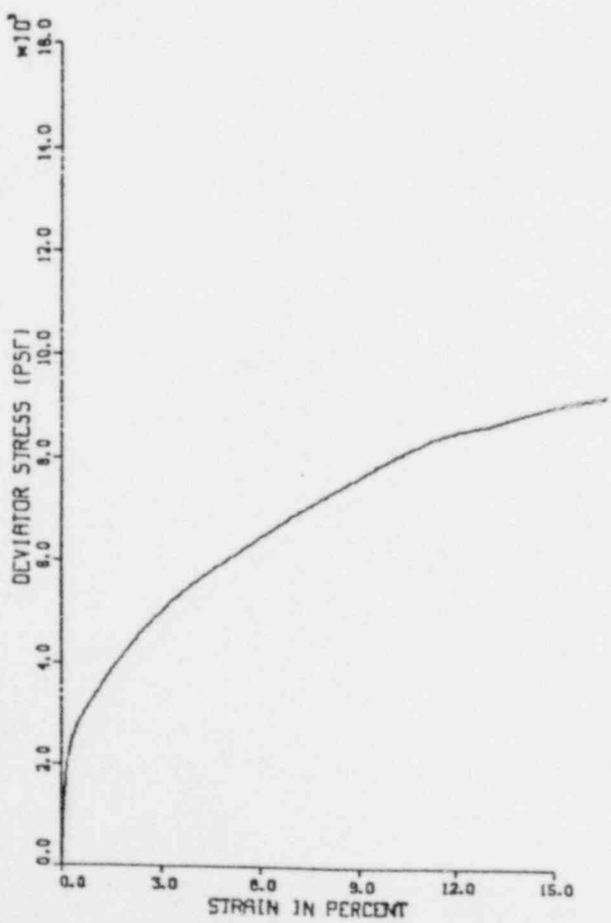
Checked by D.M. Yadon Date 8/28/80	Project No.	Figure No.
Approved by R. Hinton Date 8/29/80	1886	B-5

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

MATERIAL TYPE 1







ISOTROPIC CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENTS
1886L OETR LANDSLIDE INVESTIGATION TESTED 7/21/80 REDUCED BY BW

BORING:RD-2,SAMPLE:PB-4 DEPTH:53-55.5

CLAYEY SILT 10Y4/2

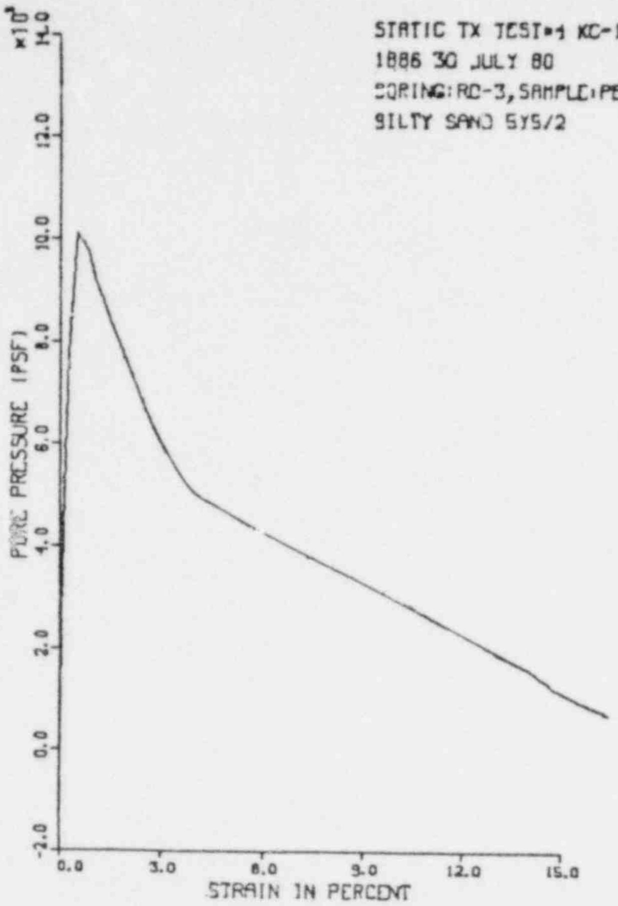
AT END OF CONSOLIDATION :
SAMPLE HEIGHT .... = 6.044 INCHES
SAMPLE AREA .... = 6.539 SQ. INCHES
EFFECTIVE CONFINING STRESS . = 7200. PSF
EFFECTIVE MAJOR PRIN. STRESS = 7200. PSF
PRINCIPAL STRESS RATIO ..... = 1.00

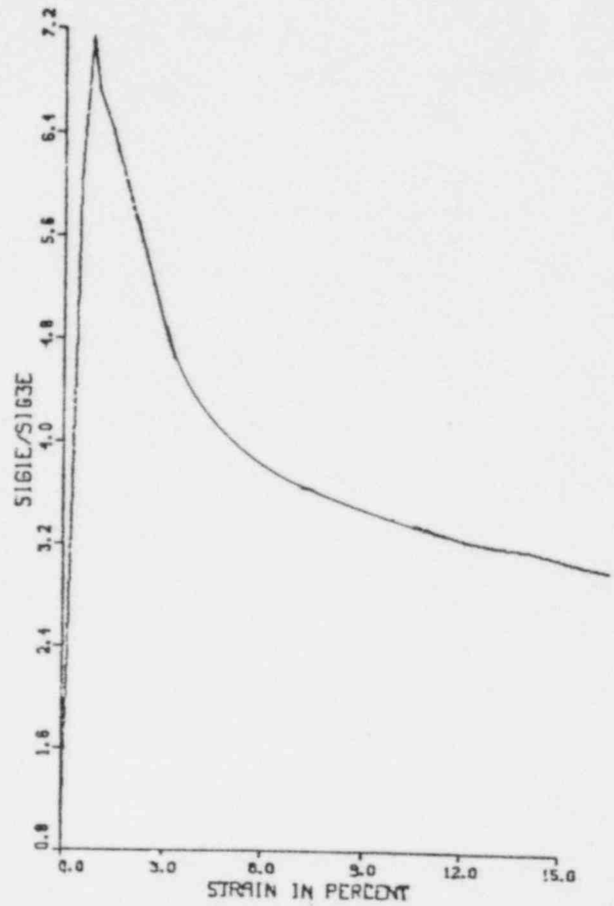
STRAIN PCT	SIGMA3E PSF	SIGMA1E PSF	RATIO SIG1E/SIG3E	PPRESS PSF	PBAR PSF	PTOT PSF	Q PSF
.0	7200.	7200.	1.0	0.	7200.	7200.	0.
.0	7056.	7221.	1.0	144.	7139.	7283.	83.
.1	6955.	7891.	1.1	245.	7423.	7668.	468.
.2	6696.	8674.	1.3	504.	7685.	8189.	989.
.3	6451.	8954.	1.4	749.	7702.	8451.	1251.
.5	6264.	9047.	1.4	936.	7655.	8591.	1391.
.7	6120.	9127.	1.5	1080.	7624.	87[illegible].	1504.
1.0	5875.	9253.	1.6	1325.	7564.	8889.	1689.
1.2	5774.	9353.	1.6	1426.	7563.	8989.	1789.
1.4	5674.	9430.	1.7	1526.	7552.	9078.	1878.
1.6	5573.	9507.	1.7	1627.	7540.	9167.	1967.
2.5	5198.	9869.	1.9	2002.	7534.	9535.	2335.
3.4	4997.	10231.	2.0	2203.	7614.	9817.	2617.
4.3	4925.	10637.	2.2	2275.	7781.	10056.	2856.
5.2	4954.	11061.	2.2	2246.	8007.	10254.	3054.
6.1	5011.	11527.	2.3	2189.	8269.	10458.	3258.
6.9	5098.	12004.	2.4	2102.	8551.	10653.	3453.
7.8	5184.	12420.	2.4	2016.	8802.	10818.	3618.
8.7	5270.	12851.	2.4	1930.	9060.	10990.	3790.
9.6	5371.	13258.	2.5	1829.	9315.	11144.	3944.
10.4	5472.	13639.	2.5	1728.	9556.	11284.	4084.
11.3	5573.	13984.	2.5	1627.	9778.	11406.	4206.
12.1	5688.	14259.	2.5	1512.	9974.	11486.	4286.
13.0	5818.	14506.	2.5	1382.	10162.	11544.	4344.
13.8	5947.	14797.	2.5	1253.	10372.	11625.	4425.
14.7	6091.	15107.	2.5	1109.	10599.	11708.	4508.
15.6	6221.	15379.	2.5	979.	10800.	11779.	4579.
16.4	6336.	15616.	2.5	864.	10976.	11840.	4640.
17.2	6451.	15843.	2.5	749.	11147.	11896.	4696.
17.9	6538.	16070.	2.5	662.	11304.	11966.	4766.
18.6	6638.	16266.	2.5	562.	11452.	12014.	4814.
19.0	6739.	17679.	2.6	461.	12209.	12670.	5470.

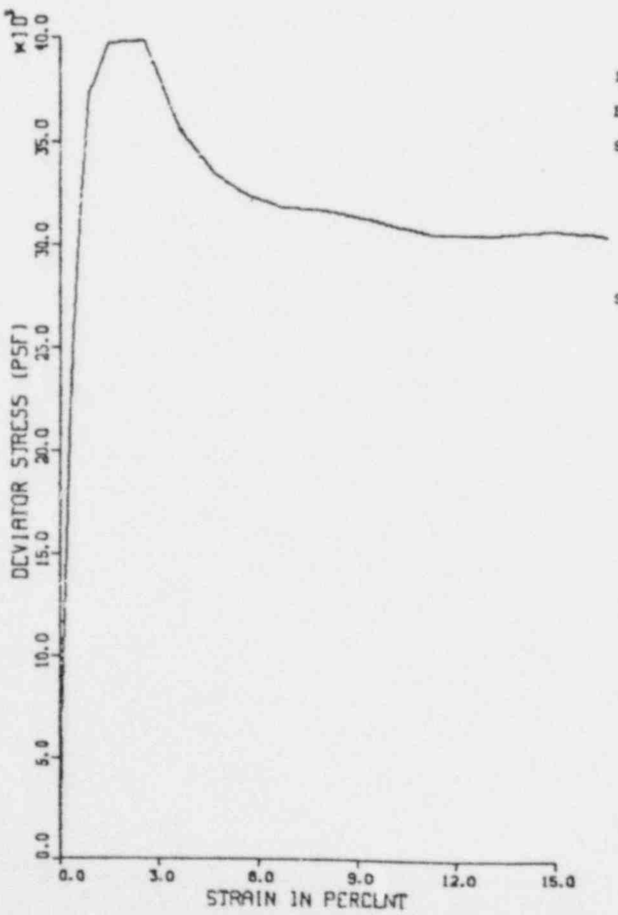
Figure B-6

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

MATERIAL TYPE 1







ISOTROPIC CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENTS
1886L GETR LANDSLIDE INVESTIGATION TESTED 7/30/80 REDUCED BY BW

BORING:RD-3,SAMPLE:PB-3 DEPTH:45-47.5

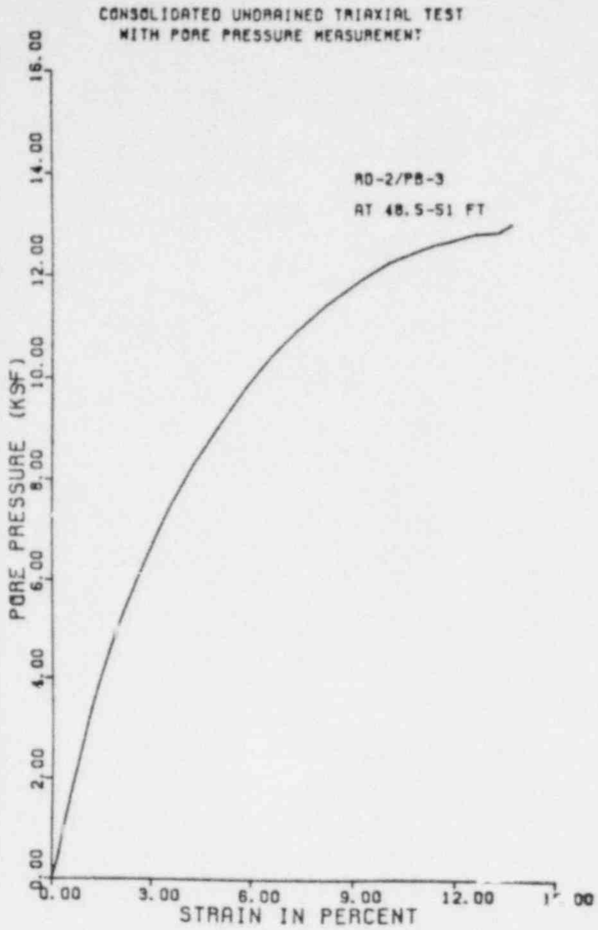
SILTY SAND 5Y5/2

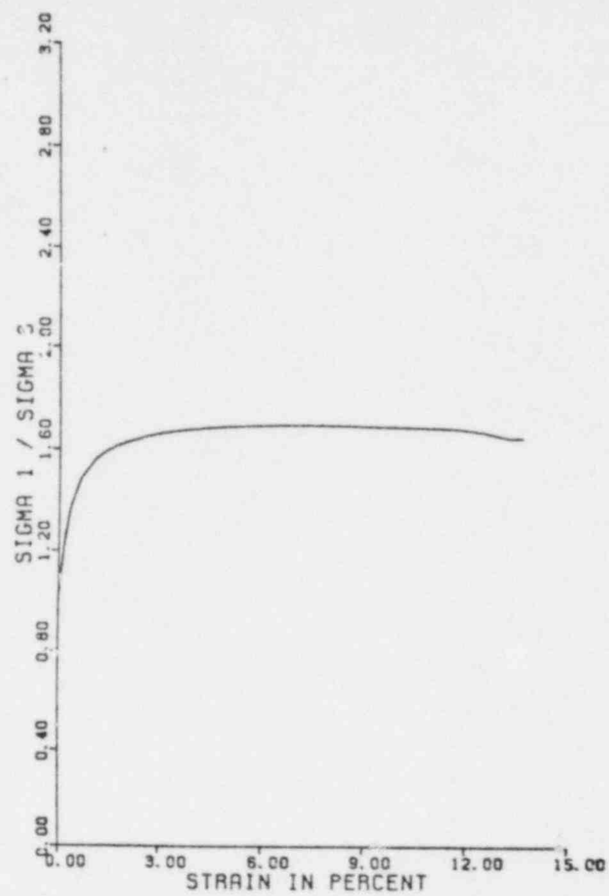
AT END OF CONSOLIDATION :
SAMPLE HEIGHT .... = 6.086 INCHES
SAMPLE AREA .... = 6.537 SQ. INCHES
EFFECTIVE CONFINING STRESS . = 15840. PSF
EFFECTIVE MAJOR PRIN. STRESS = 15840. PSF
PRINCIPAL STRESS RATIO ..... = 1.00

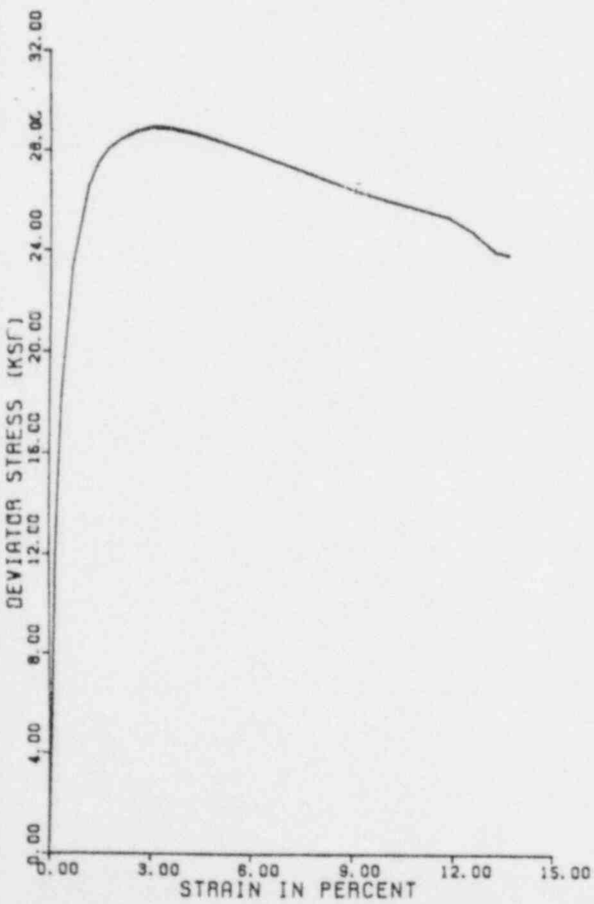
STRAIN PCT	SIGMA3E PSF	SIGMA1E PSF	RATIO SIG1E/SIG3E	PPRESS PSF	PBAR PSF	PTOT PSF	Q PSF
.0	15840.	15840.	1.0	0.	15840.	15840.	0.
.0	15178.	16444.	1.1	662.	15811.	16473.	633.
.1	13234.	19641.	1.5	2606.	16[illegible]37.	19043.	3203.
.1	10757.	22494.	2.1	5083.	166[illegible]6.	21709.	5869.
.3	8064.	27180.	3.4	7776.	17622.	25398.	9558.
.4	6926.	31592.	4.6	8914.	19259.	28173.	12333.
.5	5731.	34417.	6.[illegible]	10109.	20074.	30183.	14343.
.8	6091.	43422.	7.1	[illegible]749.	24756.	34505.	18665.
1.0	6624.	44494.	6.7	9216.	25559.	34775.	18935.
1.4	7243.	46759.	6.5	8597.	27001.	35598.	19758.
1.4	7344.	47011.	6.4	8496.	27177.	35673.	19833.
2.5	9130.	48961.	5.4	6710.	29045.	35756.	19916.
3.6	10915.	46433.	4.3	4925.	28674.	33599.	17759.
4.7	11218.	44662.	4.0	4622.	27940.	32562.	16722.
5.7	11520.	43892.	3.8	4320.	27706.	32026.	16186.
6.8	12182.	43956.	3.6	3658.	28069.	31727.	15887.
7.8	11966.	43680.	3.7	3874.	27823.	31697.	15857.
8.8	12326.	43775.	3.6	3514.	28051.	31564.	15724.
11.2	13291.	43829.	3.3	2549.	28560.	31109.	15269.
12.1	13709.	44232.	3.2	2131.	28970.	31101.	15261.
13.1	13954.	44460.	3.2	1886.	29207.	31093.	15253.
14.0	14227.	44947.	3.2	1613.	29587.	31200.	15360.
14.9	14645.	45458.	3.1	1195.	30051.	31246.	15406.
15.8	14933.	45610.	3.1	907.	30272.	31179.	15339.
16.6	15149.	45543.	3.0	691.	30346.	31037.	15197.
17.3	15437.	45017.	2.9	403.	30227.	30630.	14790.
19.0	15739.	42775.	2.7	101.	29257.	29358.	13518.
19.8	15494.	42928.	2.8	346.	29211.	29557.	13717.
20.7	15451.	43297.	2.8	389.	29374.	29763.	13923.

Figure B-7

MATERIAL TYPE 1







CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE ME SUREMENT

EARTH SCI. ASSO., 4755-007-03, AUG 13, 1980,
RD-2/PB-3, DEPTH 48.5-51 FT., GREENISH GRAY CLAYEY SILT TO SILTY CLAY

SAMPLE HEIGHT = 5.93 IN
SAMPLE AREA = 6.05 SQ IN
CONSOLIDATION PRESSU[illegible] = 50.40 KSF
INIT. MAX. PRI[illegible] ST[illegible]SS = 50.40 KSF
INITIAL PORE [illegible] = 49.8 PSI

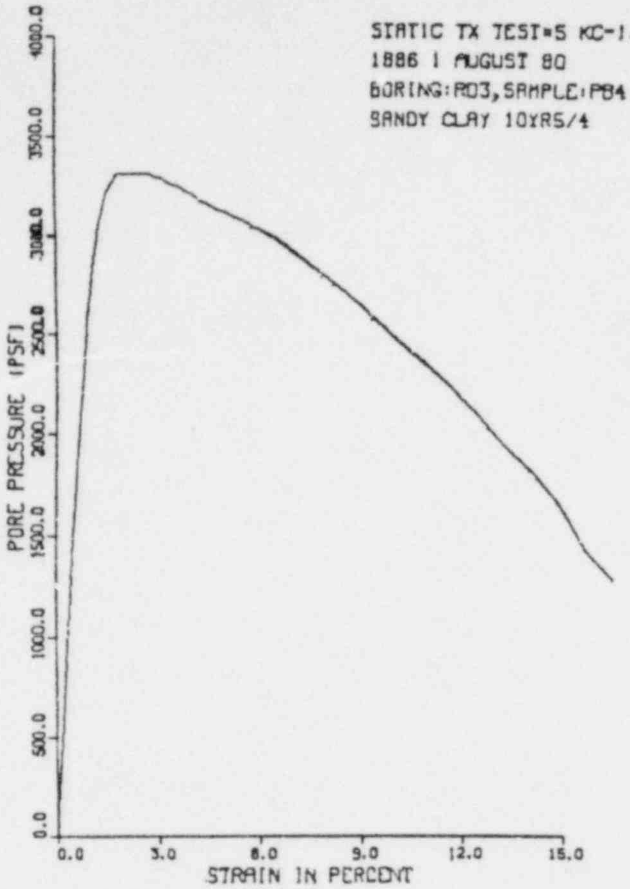
BEFORE CONSOLI[illegible] ION
DRY DENSITY = 106.2 PCF
WATER CONTE[illegible] = 21.10 PERCENT
AFTER CONSOL[illegible] ATION
DRY DENSIT = 117.5 PCF
WATER CON[illegible] NT = 19.30 PERCENT
BEAM LOAD F[illegible]CTOR = 19.02 LBS/PERCENT

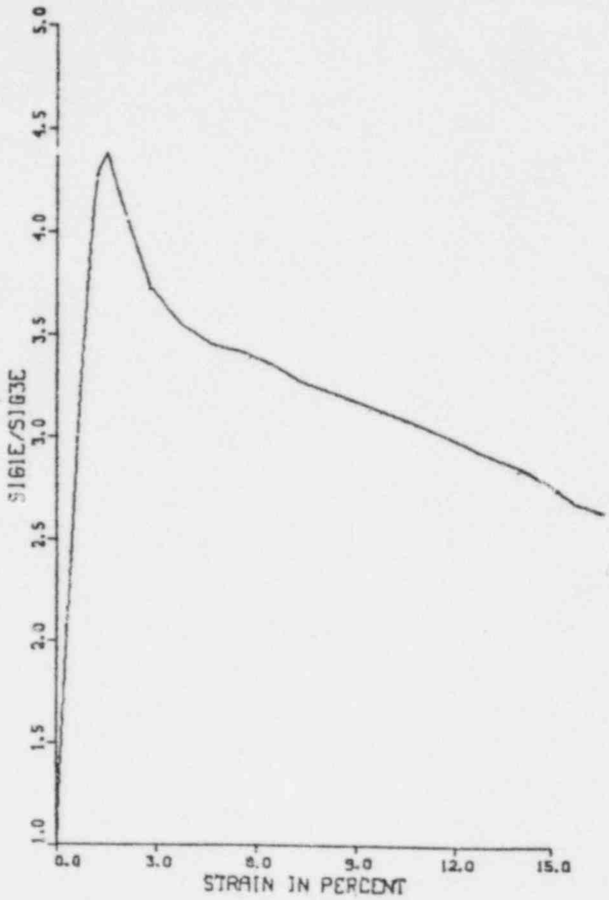
AXIAL STRAIN PCT	PORE PRESS RE KS	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.17	[illegible]	11.65	49.98	61.64	1.23	.04	55.81	5.83
.35	1.[illegible]	18.49	49.36	67.85	1.37	.06	58.61	9.24
.64	1.[illegible]	23.38	48.44	71.82	1.48	.08	60.13	11.69
1.10	3.[illegible]	26.54	47.17	73.71	1.56	.12	60.44	13.27
1.38	3.[illegible]	27.49	46.53	74.01	1.59	.14	60.27	13.74
1.69	4.51	28.07	45.89	73.96	1.61	.16	59.93	14.03
2.03	5.14	28.42	45.26	73.68	1.63	.18	59.47	14.21
2.48	5.89	28.77	44.51	73.28	1.65	.20	58.90	14.38
2.99	6.67	28.97	43.73	72.70	1.66	.23	58.22	14.49
3.51	7.42	28.90	42.98	71.89	1.67	.26	57.44	14.45
4.24	8.31	28.69	42.09	70.78	1.68	.29	56.43	14.34
4.98	9.07	28.38	41.33	69.70	1.69	.32	55.52	14.19
5.72	9.78	28.03	40.62	68.65	1.69	.35	54.64	14.01
6.50	10.41	27.67	39.99	67.66	1.69	.38	53.82	13.83
7.22	10.92	27.33	39.48	66.81	1.69	.40	53.15	13.66
8.08	11.42	26.91	38.98	65.89	1.69	.42	52.44	13.45
9.20	11.94	26.38	38.46	64.84	1.69	.45	51.65	13.19
10.01	12.25	26.06	38.15	64.21	1.68	.47	51.18	13.03
10.63	12.41	25.84	37.99	63.83	1.68	.48	50.91	12.92
11.31	12.59	25.60	37.81	63.42	1.68	.49	50.62	12.80
11.81	12.67	25.42	37.73	63.15	1.67	.50	50.44	12.71
12.51	12.80	24.86	37.60	62.46	1.66	.51	50.03	12.43
13.20	12.83	24.04	37.57	61.61	1.64	.53	49.59	12.02
13.60	12.99	23.92	37.41	61.34	1.64	.54	49.37	11.96

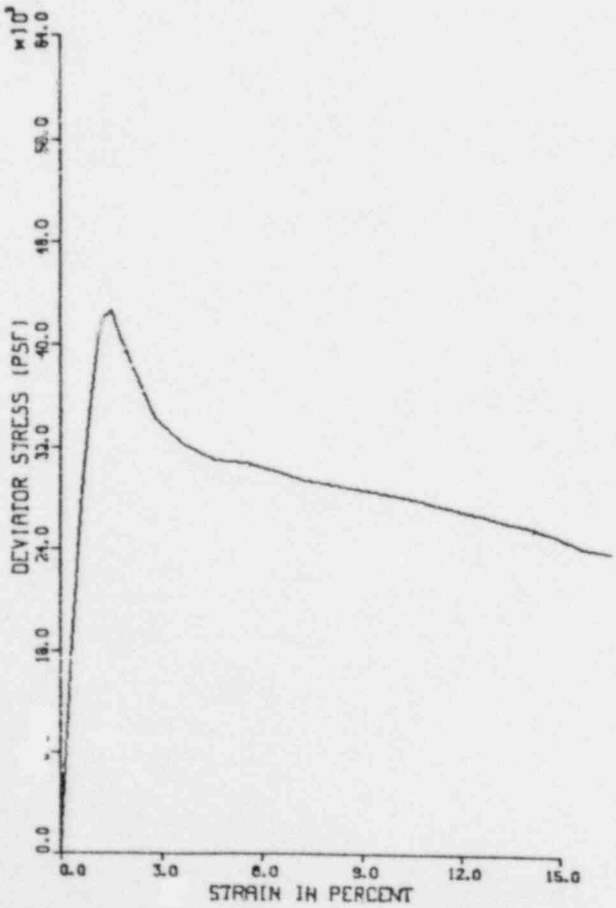
Figure B-8

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

MATERIAL TYPE 2







ISOTROPIC CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENTS
1886L GETR LANDSLIDE INVESTIGATION TESTED 8/1/80 REDUCED BY BW

BORING:RD-3,SAMPLE:PB-4,DEPTH:55-57.5

SANDY CLAY 10YR5/4

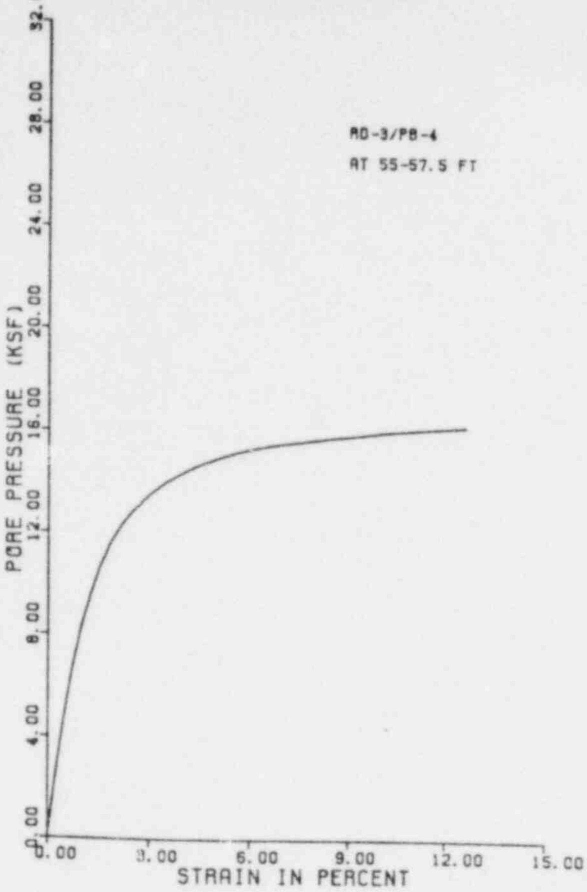
AT END OF CONSOLIDATION :
SAMPLE HEIGHT .... = 6.069 INCHES
SAMPLE AREA .... = 6.535 SQ. INCHES
EFFECTIVE CONFINING STRESS . = 15840. PSF
EFFECTIVE MAJOR PRIN. STRESS = 15840. PSF
PRINCIPAL STRESS RATIO ..... = 1.00

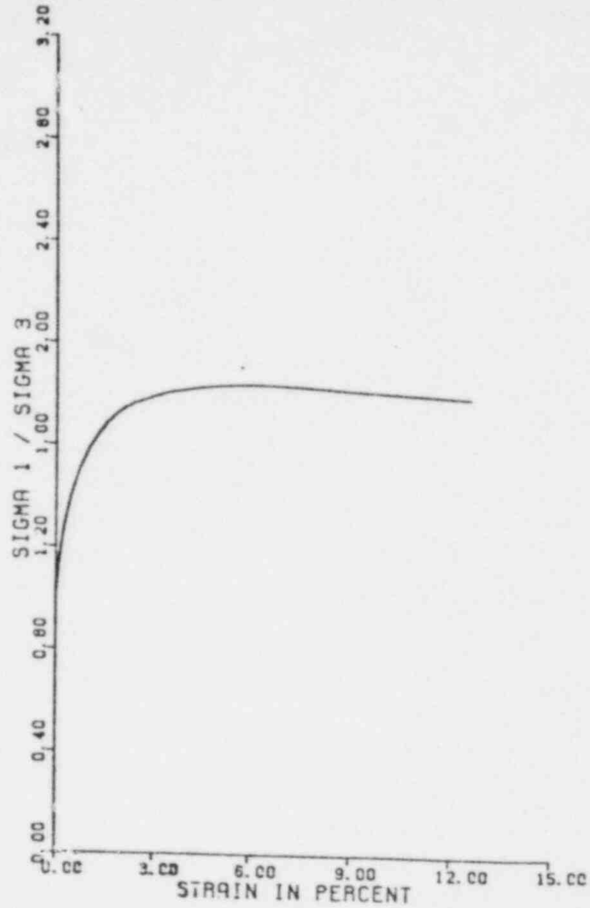
STRAIN PCT	SIGMA3E PSF	SIGMA1E PSF	RATIO SIG1E/SIG3E	PPRESS PSF	PBAR PSF	PTOT PSF	Q PSF
.0	15840.	15840.	1.0	0.	15840.	15840.	0.
.0	15768.	16021.	1.0	72.	15895.	15967.	127.
.0	15696.	17844.	1.1	144.	16770.	16914.	1074.
.1	15538.	20887.	1.3	302.	18212.	18515.	2675.
.2	15278.	24868.	1.6	562.	20073.	20635.	4795.
.3	14962.	29220.	2.0	878.	22091.	22969.	7129.
.4	14630.	33669.	2.3	1210.	24150.	25359.	9519.
.5	14256.	38150.	2.7	1584.	26203.	27787.	11947.
.6	13910.	42385.	3.0	1930.	28148.	30077.	14237.
.7	13608.	45921.	3.4	2232.	29764.	31996.	16156.
.9	13248.	49397.	3.7	2592.	31322.	33914.	18074.
1.1	12960.	53180.	4.1	2880.	33070.	35950.	20110.
1.3	12787.	54899.	4.3	3053.	33843.	36896.	21056.
1.5	12614.	55268.	4.4	3226.	33941.	37167.	21327.
1.8	12528.	52720.	4.2	3312.	32624.	35936.	20096.
2.8	12528.	46666.	3.7	3312.	29597.	32909.	17069.
3.7	12600.	44629.	3.5	3240.	28614.	31854.	16014.
4.6	12686.	43694.	3.4	3154.	28190.	31344.	15504.
5.6	12758.	43508.	3.4	3082.	28133.	31215.	15375.
6.5	12845.	42966.	3.3	2995.	27906.	30901.	15061.
7.3	12960.	42234.	3.3	2880.	27597.	30477.	14637.
8.2	13075.	41987.	3.2	2765.	27531.	30296.	14456.
9.1	13190.	41747.	3.2	2650.	27469.	30118.	14278.
9.9	13334.	41463.	3.1	2506.	27399.	29904.	14064.
10.8	13464.	41210.	3.1	2376.	27337.	29713.	13873.
11.6	13594.	40805.	3.0	2246.	27200.	29446.	13606.
12.5	13738.	40423.	2.9	2102.	27080.	29183.	13343.
13.3	13896.	40063.	2.9	1944.	26979.	28923.	13083.
14.1	14026.	39757.	2.8	1814.	26891.	28706.	12866.
14.9	14184.	39304.	2.8	1656.	26744.	28400.	12560.
15.7	14414.	38632.	2.7	1426.	26523.	27949.	12109.
16.5	14558.	38404.	2.6	1282.	26481.	27763.	11923.
17.2	14717.	37976.	2.6	1123.	26347.	27470.	11630.
18.0	14846.	37351.	2.5	994.	26099.	27092.	11252.

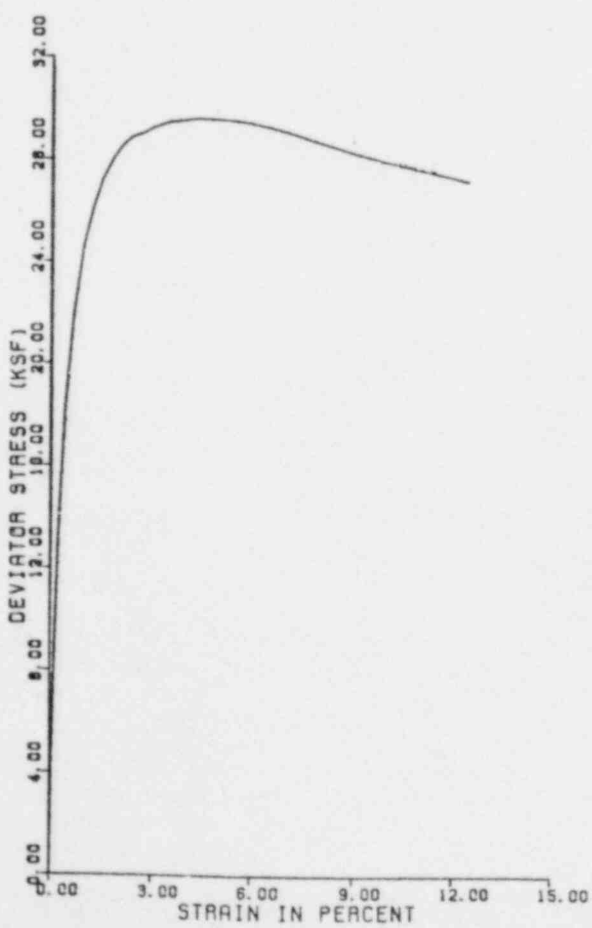
Figure B-9

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

MATERIAL TYPE 2







CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

EARTH SCI. ASSO., 4755-007-03, AUG 5, 1980
RD-3/PB-4, DEPTH 55-57.5 FT., MOD. YELLOWISH BROWN SILTY CLAY W/ TR. GRAVEL

SAMPLE HEIGHT = 6.60 IN
SAMPLE AREA = 6.14 SQ IN
CONSOLIDATION PRESSURE = 50.40 KSF
INIT. MAX. PRIN. STRESS = 50.40 KSF
INITIAL PORE PRESSURE = 68.3 PSI

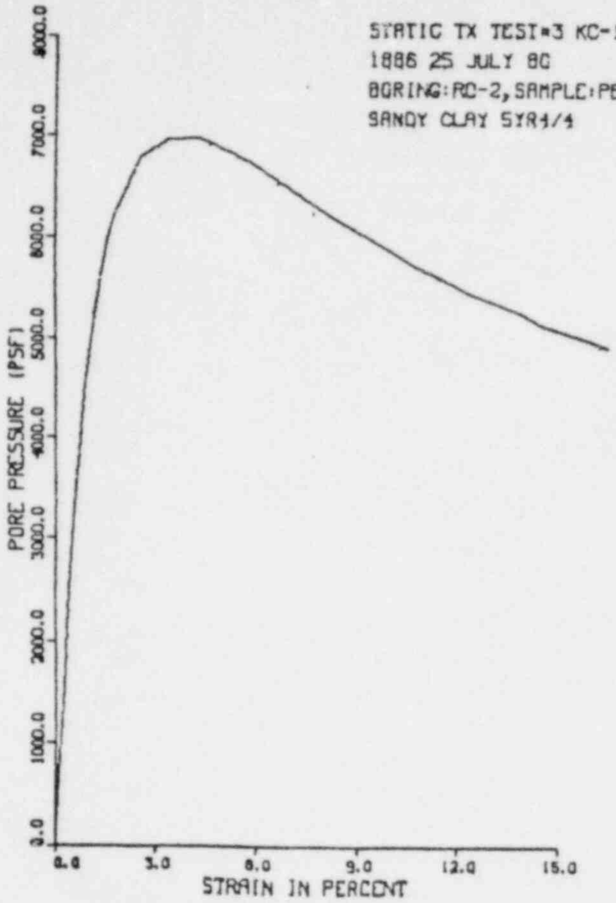
BEFORE CONSOLIDATION
DRY DENSITY = 110.9 PCF
WATER CONTENT = 19.20 PERCENT
AFTER CONSOLIDATION
DRY DENSITY = 119.8 PCF
WATER CONTENT = 17.50 PERCENT
BEAM LOAD FACTOR = 19.17 LBS/PERCENT

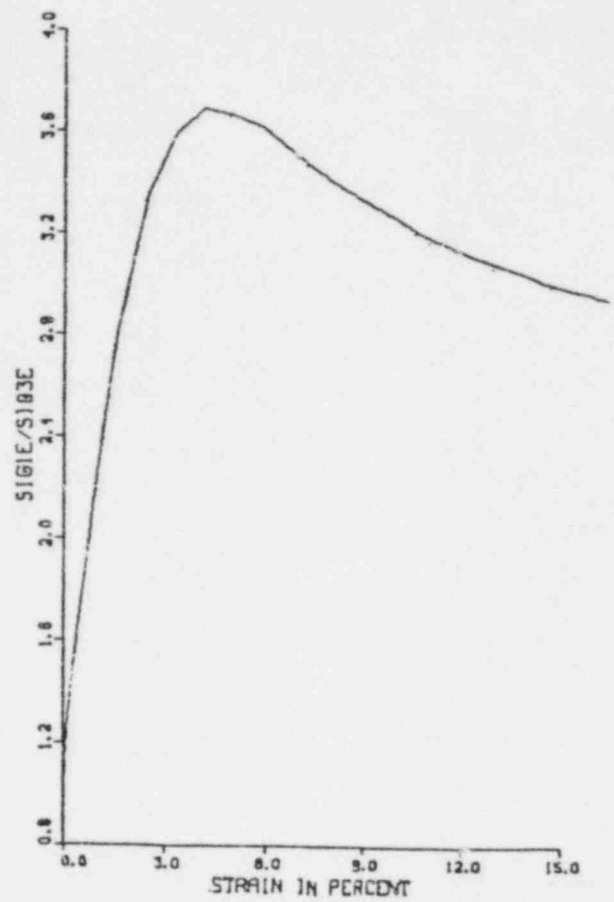
AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.08	.75	5.66	49.65	55.32	1.11	.13	52.48	2.83
.15	1.56	9.52	48.84	58.37	1.19	.16	53.61	4.76
.26	2.68	13.60	47.72	61.32	1.28	.20	54.52	6.80
.44	4.48	18.23	45.92	64.15	1.40	.25	55.04	9.11
.70	6.54	21.98	43.86	65.84	1.50	.30	54.85	10.99
.99	8.31	24.59	42.09	66.68	1.58	.34	54.39	12.29
1.24	9.53	26.04	40.87	66.90	1.64	.37	53.88	13.02
1.52	10.63	27.16	39.77	66.93	1.68	.39	53.35	13.58
1.8[illegible]	11.64	28.04	38.76	66.80	1.72	.41	52.78	14.02
2.12	12.24	28.53	38.16	66.69	1.75	.43	52.43	14.27
2.39	12.72	28.85	37.68	66.53	1.77	.44	52.11	14.42
2.73	13.20	29.01	37.20	66.21	1.78	.46	51.70	14.51
3.03	13.58	29.23	36.82	66.05	1.79	.46	51.43	14.61
3.49	14.04	29.44	36.36	65.80	1.81	.48	51.08	14.72
3.94	14.37	29.52	36.03	65.54	1.82	.49	50.79	14.76
4.38	14.66	29.60	35.74	65.34	1.83	.50	50.54	14.80
4.83	14.90	29.58	35.50	65.08	1.83	.50	50.29	14.79
5.40	15.15	29.54	35.25	64.79	1.84	.51	50.02	14.77
5.91	15.32	29.46	35.08	64.54	1.84	.52	49.81	14.73
6.52	15.48	29.31	34.92	64.23	1.84	.53	49.58	14.66
7.27	15.62	29.05	34.78	63.83	1.84	.54	49.30	14.53
7.88	15.72	28.80	34.68	63.48	1.83	.55	49.08	14.40
8.49	15.81	28.57	34.59	63.16	1.83	.55	48.87	14.29
9.25	15.90	28.27	34.50	62.78	1.82	.56	48.64	14.14
10.00	16.03	28.02	34.37	62.39	1.82	.57	48.38	14.01
10.79	16.11	27.77	34.29	62.06	1.81	.58	48.17	13.89
11.58	16.19	27.53	34.21	61.74	1.80	.59	47.98	13.76
12.56	16.27	27.22	34.13	61.35	1.80	.60	47.74	13.61

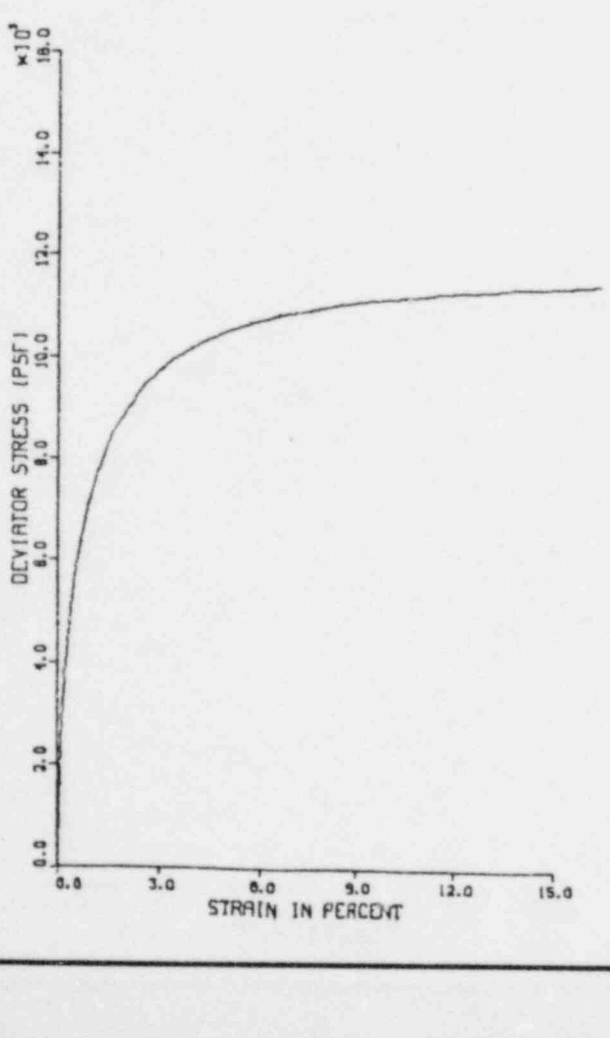
Figure B-10

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

MATERIAL TYPE 3







ISOTROPIC CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENTS
1886L GETR LANDSLIDE INVESTIGATION TESTED 7/25/80 REDUCED BY BW

BORING RD-2 SAMPLE PB-5 DEPTH 60-62.5

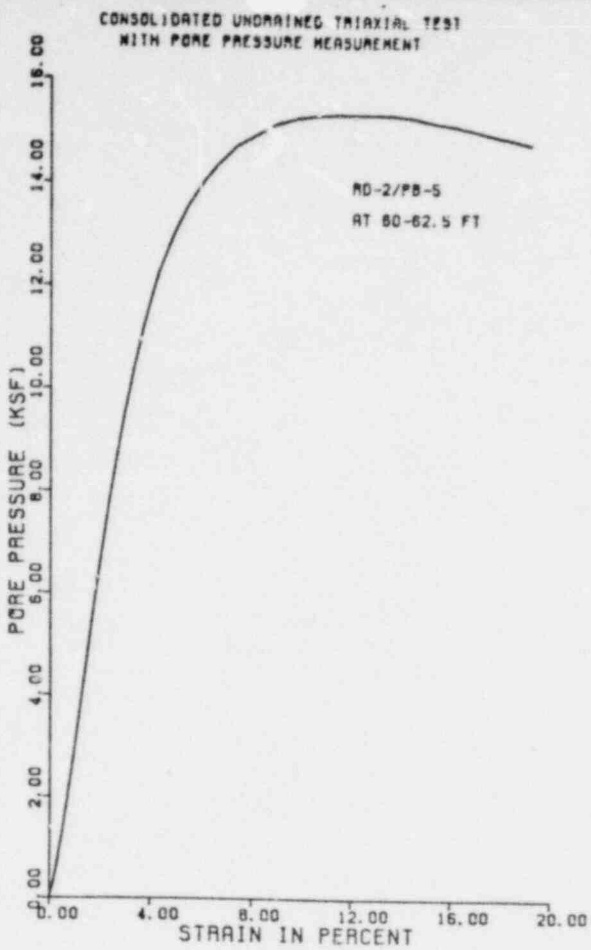
SANDY CLAY 5YR4/4

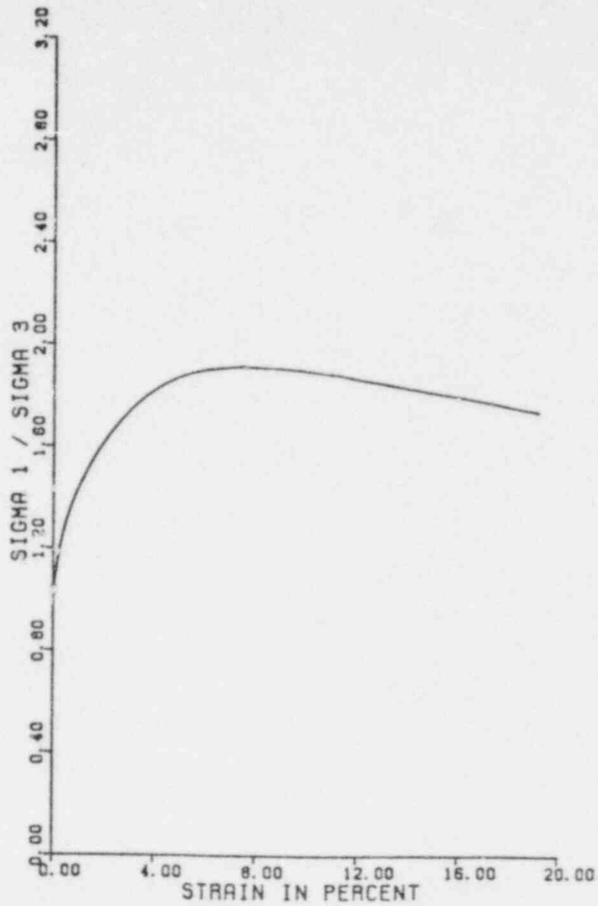
AT END OF CONSOLIDATION :
SAMPLE HEIGHT .... = 6.086 INCHES
SAMPLE AREA .... = 6.534 SQ. INCHES
EFFECTIVE CONFINING STRESS . = 10800. PSF
EFFECTIVE MAJOR PRIN. STRESS = 10800. PSF
PRINCIPAL STRESS RATIO ..... = 1.00

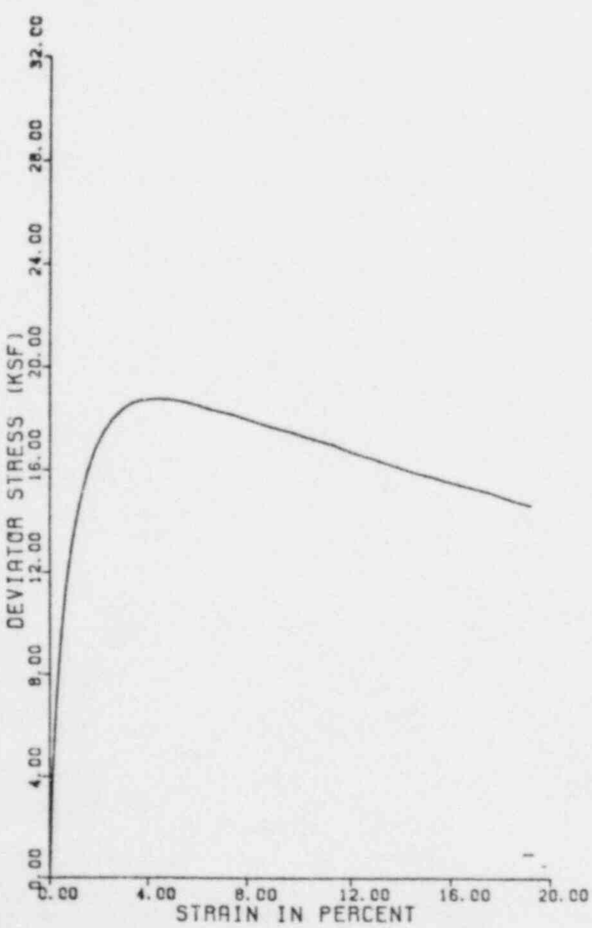
STRAIN PCT	SIGMA3E PSF	SIGMA1E PSF	RATIO SIG1E/SIG3E	PPRESS PSF	PBAR PSF	PTOT PSF	Q PSF
.0	10800.	10800.	1.0	0.	10800.	10800.	0.
.0	10742.	10963.	1.0	58.	10853.	10910.	110.
.0	10512.	11867.	1.1	288.	11190.	11478.	678.
.1	9936.	12819.	1.3	864.	11378.	12242.	1442.
.3	9058.	13234.	1.5	1742.	11146.	12888.	2088.
.4	8309.	13346.	1.6	2491.	10828.	13319.	2519.
.6	7502.	13354.	1.8	3298.	10428.	13726.	2926.
.7	6826.	13324.	2.0	3974.	10075.	14049.	3249.
.9	6264.	13286.	2.1	4536.	9775.	14311.	3511.
1.2	5587.	13256.	2.4	5213.	9422.	14634.	3834.
1.6	4810.	13237.	2.8	5990.	9023.	15014.	4214.
1.7	4608.	13260.	2.9	6192.	8934.	15126.	4326.
2.6	4018.	13476.	3.4	6782.	8747.	15529.	4729.
3.4	3845.	13807.	3.6	6955.	8826.	15781.	4981.
4.3	3830.	14139.	3.7	6970.	8985.	15954.	5154.
5.1	3960.	14503.	3.7	6840.	9232.	16072.	5272.
5.9	4090.	14809.	3.6	6710.	9450.	16160.	5360.
6.7	4277.	15119.	3.5	6523.	9698.	16221.	5421.
7.6	4464.	15405.	3.5	6336.	9934.	16270.	5470.
8.4	4637.	15672.	3.4	6163.	10155.	16318.	5518.
9.2	4795.	15913.	3.3	6005.	10354.	16359.	5559.
10.0	4939.	16117.	3.3	5861.	10528.	16389.	5589.
10.8	5112.	16338.	3.2	5688.	10725.	16413.	5613.
11.6	5242.	16525.	3.2	5558.	10883.	16441.	5641.
12.4	5371.	16697.	3.1	5429.	11034.	16463.	5663.
13.2	5472.	16830.	3.1	5328.	11151.	16479.	5679.
14.0	5587.	16994.	3.0	5213.	11291.	16503.	5703.
14.5	5688.	17106.	3.0	5112.	11397.	16509.	5709.
15.3	5774.	17220.	3.0	5026.	11497.	16523.	5723.
16.1	5861.	17332.	3.0	4939.	11596.	16536.	5736.
16.8	5947.	17432.	2.9	4853.	11690.	16542.	5742.
17.6	6062.	17552.	2.9	4738.	11807.	16545.	5745.
18.3	6134.	17644.	2.9	4666.	11889.	16555.	5755.
19.1	6221.	17749.	2.9	4579.	11985.	564.	5764.
19.8	6264.	17799.	2.8	4536.	12032.	568.	5768.
20.1	6293.	17824.	2.8	4507.	12059.	166.	5766.

Figure B-11

MATERIAL TYPE 3







CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

EARTH SCI. ASSO., 4755-007-03, AUG 5, 1980
RD-2/PB-5, DEPTH 60-62.5 FT., MODERATE BROWN GRAVELLY CLAY

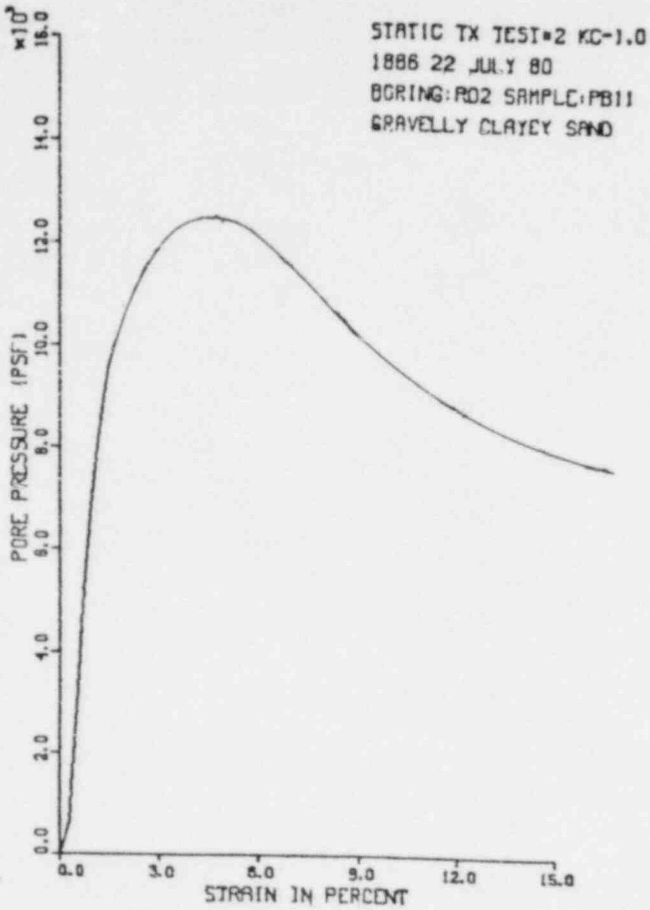
SAMPLE HEIGHT	=	6.29 IN
SAMPLE AREA	=	6.32 SQ IN
CONSOLIDATION PRESSURE	=	34.56 KSF
INIT. MAX. PRIN. STRESS	=	34.56 KSF
INITIAL PORE PRESSURE	=	49.4 PSI
BEFORE CONSOLIDATION		
DRY DENSITY	=	111.0 PCF
WATER CONTENT	=	18.60 PERCENT
AFTER CONSOLIDATION		
DRY DENSITY	=	117.3 PCF
WATER CONTENT	=	17.30 PERCENT
BEAM LOAD FACTOR	=	15.33 LBS/PERCENT

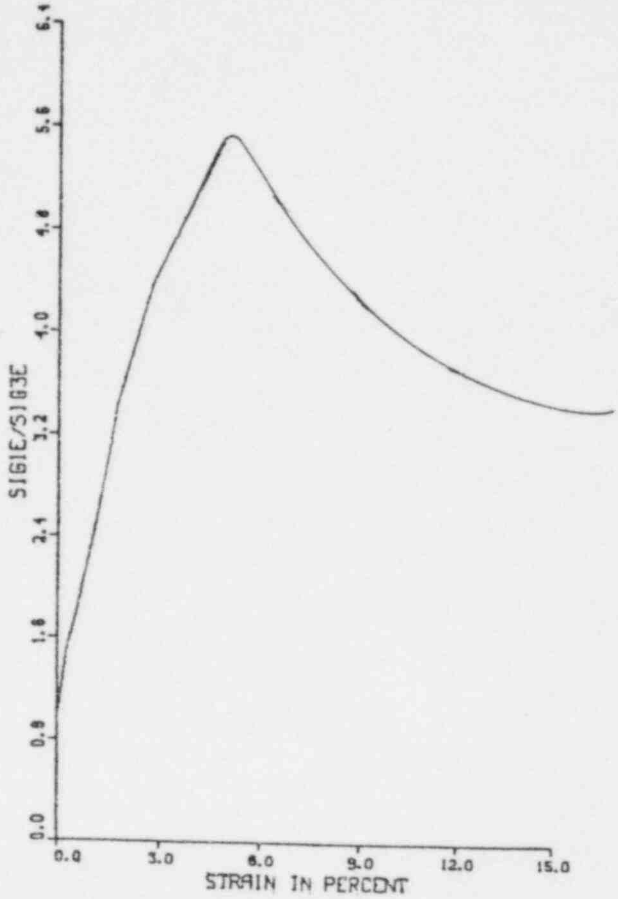
AXIAL STRAIN PCT	PORE PRESSURE KSF	DEVIATOR STRESS KSF	SIGMA3 EFFECTIVE KSF	SIGMA1 EFFECTIVE KSF	STRESS RATIO	ABAR	PBAR KSF	QBAR KSF
.10	.20	3.35	34.36	37.71	1.10	.06	36.03	1.67
.19	.42	5.33	34.14	39.47	1.16	.08	36.81	2.67
.29	.68	7.10	33.88	40.99	1.21	.10	37.43	3.55
.48	1.25	9.63	33.31	42.93	1.29	.13	38.12	4.81
.67	1.89	11.45	32.67	44.12	1.35	.16	38.40	5.72
.91	2.72	13.15	[illegible]	44.99	1.41	.21	38.41	6.57
1.18	3.72	14.53	30.84	45.37	1.47	.26	38.11	7.26
1.51	4.91	15.86	29.65	45.50	1.53	.31	37.58	7.93
1.81	5.99	16.70	28.57	45.27	1.58	.36	36.92	8.35
2.15	7.10	17.39	27.46	44.85	1.63	.41	36.16	8.70
2.54	8.31	17.93	26.25	44.19	1.68	.46	35.22	8.97
2.92	9.36	18.31	25.20	43.51	1.73	.51	34.35	9.15
3.32	10.32	18.57	24.24	42.80	1.77	.56	33.52	9.28
3.72	11.17	18.69	23.39	42.08	1.80	.60	32.73	9.35
4.32	12.21	18.74	22.35	41.09	1.84	.65	31.72	9.37
4.88	12.93	18.72	21.63	40.35	1.87	.69	30.99	9.36
5.43	13.51	18.64	21.05	39.69	1.89	.72	30.37	9.32
6.04	13.98	18.49	20.58	39.07	1.90	.76	29.82	9.24
6.53	14.28	18.33	20.28	38.60	1.90	.78	29.44	9.16
7.37	14.69	18.15	19.87	38.02	1.91	.81	28.94	9.07
8.93	15.11	17.65	19.45	37.10	1.91	.86	28.28	8.82
9.71	15.22	17.44	19.34	36.78	1.90	.87	28.06	8.72
10.54	15.28	17.18	19.28	36.46	1.89	.89	27.87	8.59
11.28	15.31	16.98	19.25	36.23	1.88	.90	27.74	8.49
12.06	15.31	16.67	19.25	35.93	1.87	.92	27.59	8.34
12.87	15.31	16.43	19.25	35.68	1.85	.93	27.47	8.21
13.60	15.29	16.19	19.27	35.45	1.84	.94	27.36	8.09
14.41	15.25	15.93	19.31	35.24	1.82	.96	27.28	7.96
15.27	15.15	15.71	19.41	35.12	1.81	.96	27.27	7.[illegible]
16.07	15.09	15.51	19.47	34.98	1.80	.97	27.23	7.7[illegible]
16.78	15.00	15.31	19.56	34.87	1.78	.98	27.21	7.66
17.56	14.90	15.11	19.66	34.77	1.77	.99	27.21	7.56
18.39	14.82	14.82	19.74	34.56	1.75	1.00	27.15	7.41
19.15	14.73	14.62	19.83	34.45	1.74	1.01	27.14	7.31

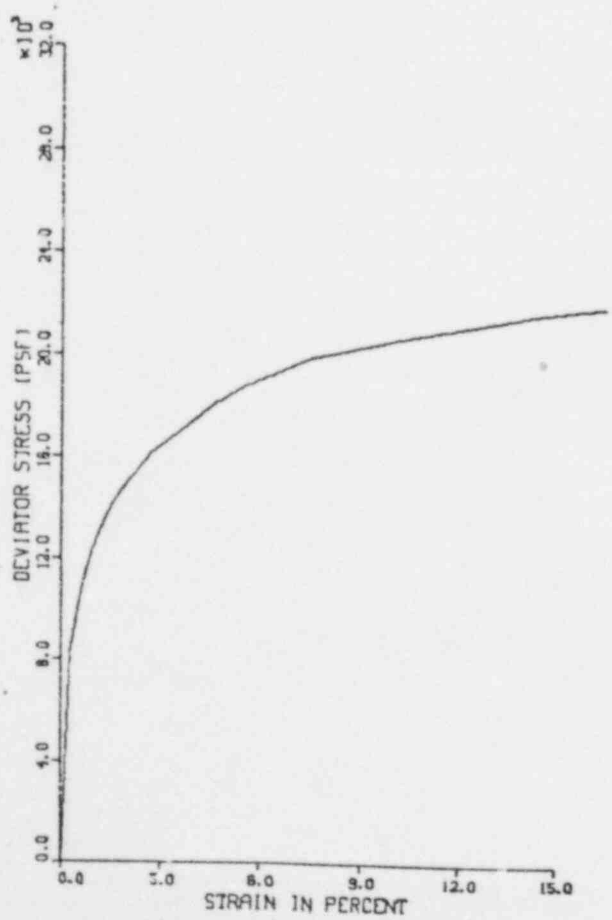
Figure B-12

CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENT

MATERIAL TYPE 4







ISOTROPIC CONSOLIDATED UNDRAINED TRIAXIAL TEST
WITH PORE PRESSURE MEASUREMENTS
1886L GETR LANDSLIDE INVESTIGATION TESTED 7/22/80 REDUCED BY BW

BORING:RD-2 SAMPLE:PB-11 DEPTH:112-113.5

GRAVELLY CLAYEY SAND 10YR5/4

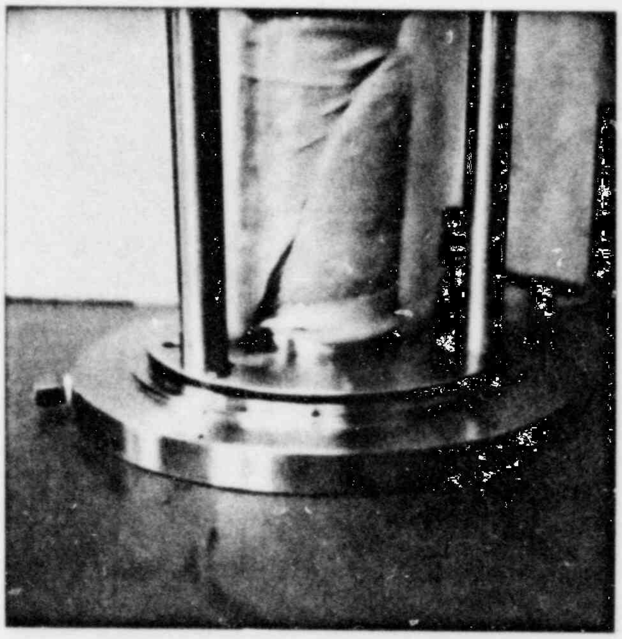
AT END OF CONSOLIDATION :
SAMPLE HEIGHT .... = 6.346 INCHES
SAMPLE AREA .... = 6.554 SQ. INCHES
EFFECTIVE CONFINING STRESS . = 16560. PSF
EFFECTIVE MAJOR PRIN. STRESS = 16560. PSF
PRINCIPAL STRESS RATIO ..... = 1.00

STRAIN PCT	SIGMA3E PSF	SIGMA1E PSF	RATIO SIG1E/SIG3E	PPRESS PSF	PBAR PSF	PTOT PSF	Q PSF
.0	16560.	16560.	1.0	0.	16560.	16560.	0.
.3	15941.	24243.	1.5	619.	20092.	20711.	4151.
.5	13306.	23513.	1.8	3254.	18409.	21664.	5104.
.7	11362.	22684.	2.0	5198.	17023.	22221.	5661.
.9	9720.	21976.	2.3	6840.	15848.	22688.	6128.
1.1	8410.	21379.	2.5	8150.	14894.	23045.	6485.
1.3	7402.	20994.	2.8	9158.	14198.	23356.	6796.
1.5	6595.	20721.	3.1	9965.	13658.	23623.	7063.
1.7	5990.	20570.	3.4	10570.	13255.	23825.	7265.
2.8	4781.	20912.	4.4	11779.	12846.	24625.	8065.
4.7	4061.	22206.	5.5	12499.	13134.	25633.	9073.
5.7	4536.	23329.	5.1	12024.	13932.	25956.	9396.
7.6	5198.	25041.	4.8	11362.	15120.	26481.	9921.
10.0	7301.	27895.	3.8	9259.	17598.	26857.	10297.
14.2	8280.	29899.	3.6	8280.	19090.	27370.	10810.
15.0	8770.	30527.	3.5	7790.	19648.	27438.	10878.
15.7	8827.	30693.	3.5	7733.	19760.	27493.	10933.
19.0	8928.	31149.	3.5	7632.	20038.	27670.	11110.
20.0	9317.	31562.	3.4	7243.	20439.	27682.	11122.
20.3	9691.	31905.	3.3	6869.	20798.	27667.	11107.

Figure B-13

MATERIAL TYPE 1

RD-2/PB-4 (σ'_m = 7200 psf)

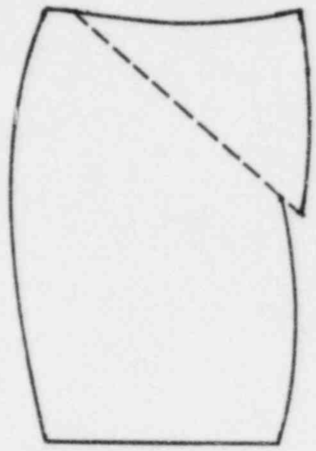


RD-3/PB-3 (σ'_m = 15,840 psf)

RD-2/PB-3 (σ'_m = 50,400 psf)

Figure B-14

MATERIAL TYPE 2

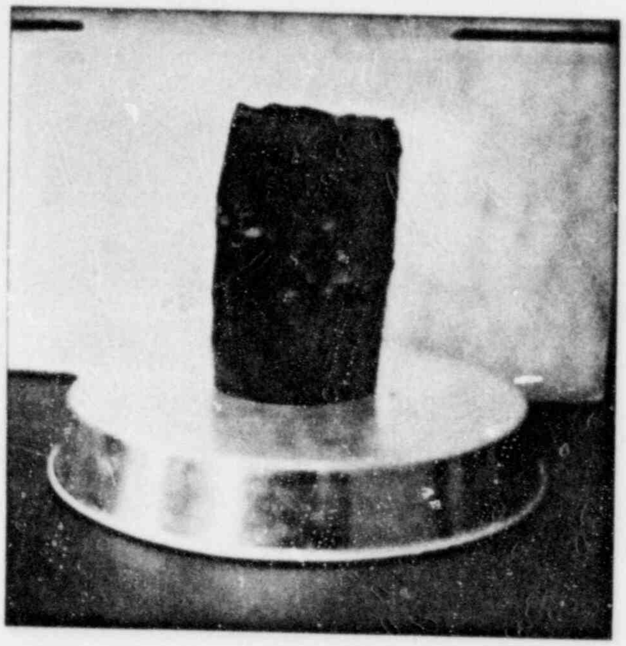


RD-3/PB-4 (σ'_m = 15,840 psf)

RD-3/PB-4 (σ'_m = 50,400 psf)

Figure B-15

MATERIAL TYPE 3



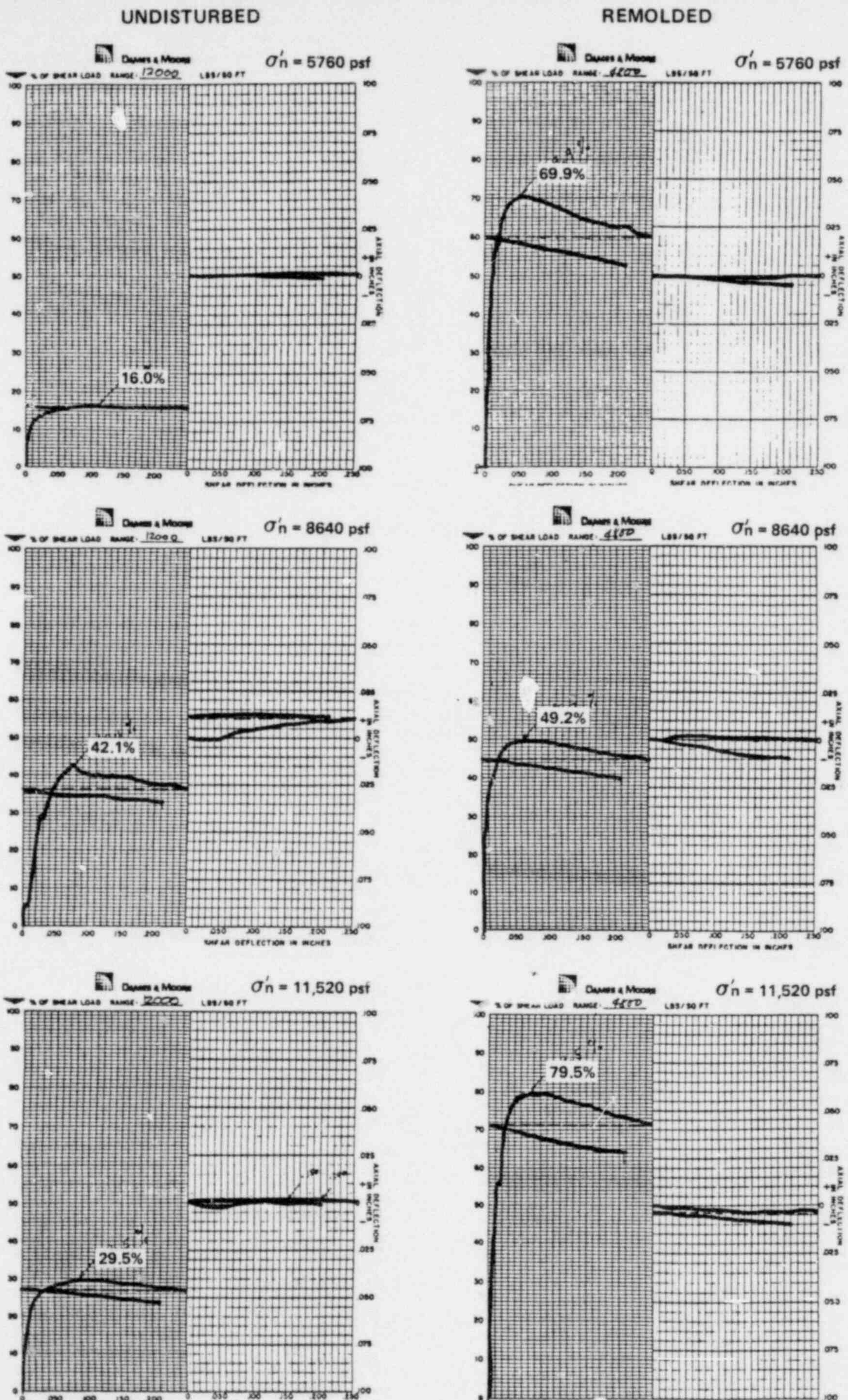
RD-2/PB-5 (σ'_m = 10,800 psf)

RD-2/PB-5 (σ'_m = 34,560 psf)

Figure B-16

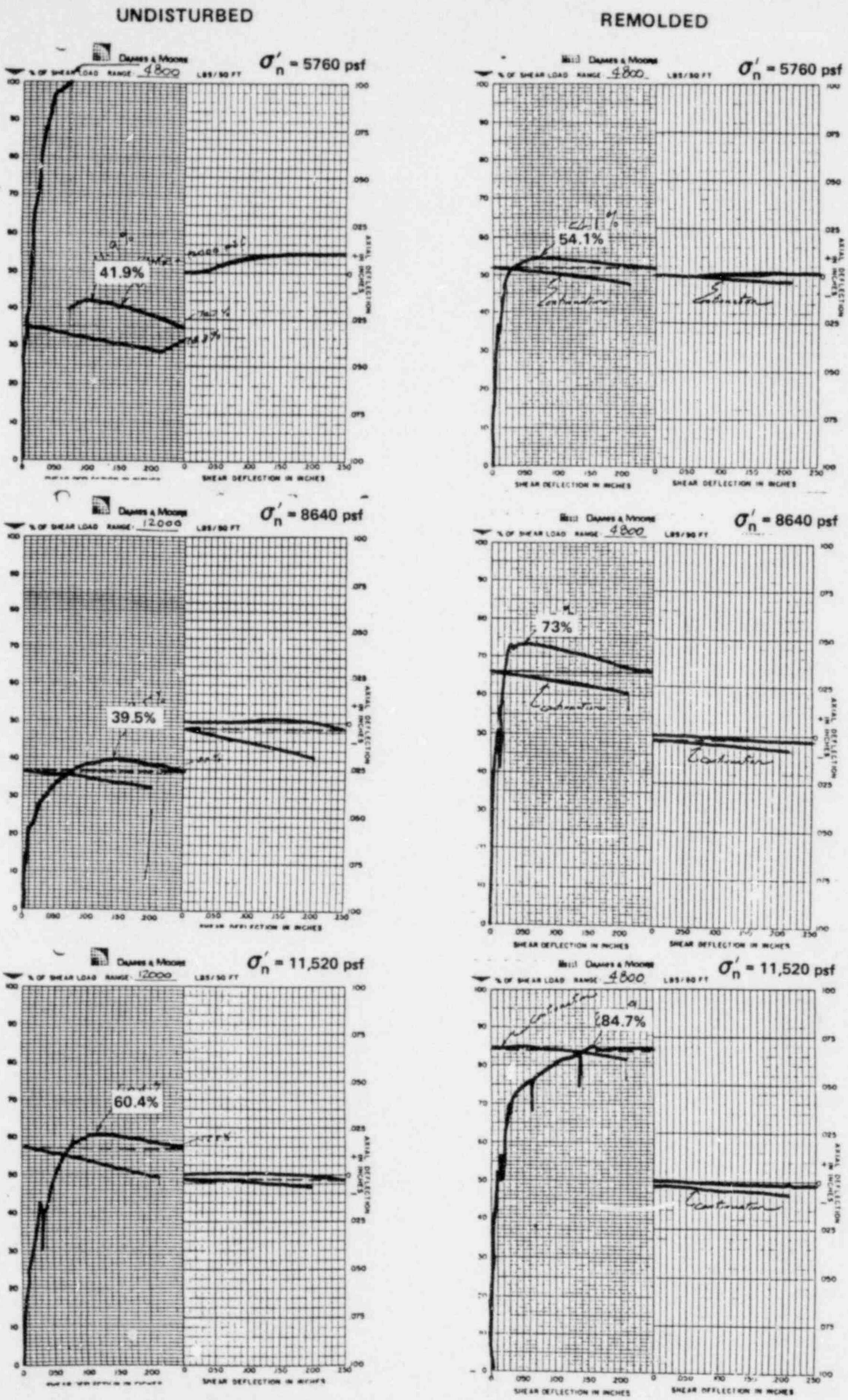
MATERIAL TYPE 4

RD-2/PB-11 (σ'_m = 16,560 psf)



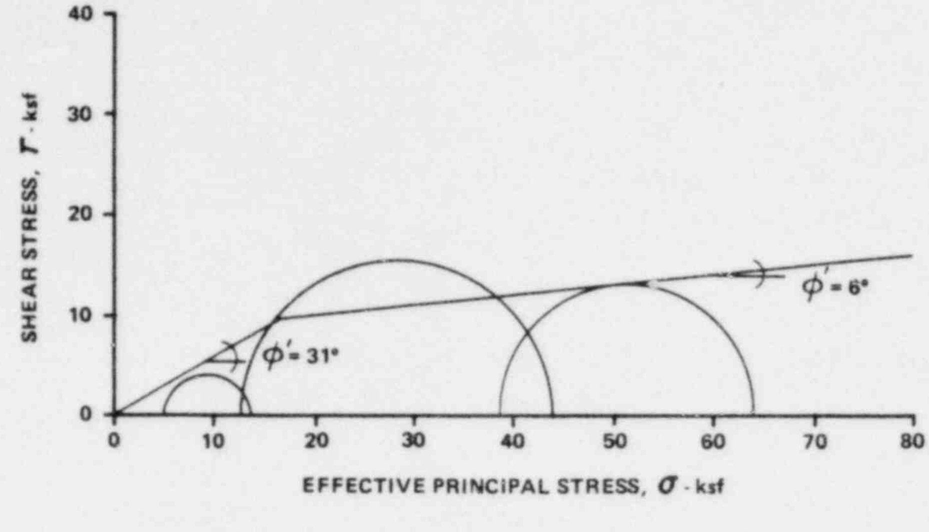
CU Direct Shear Test Results - Boring RD-2, Sample PB-4
Material Type 1

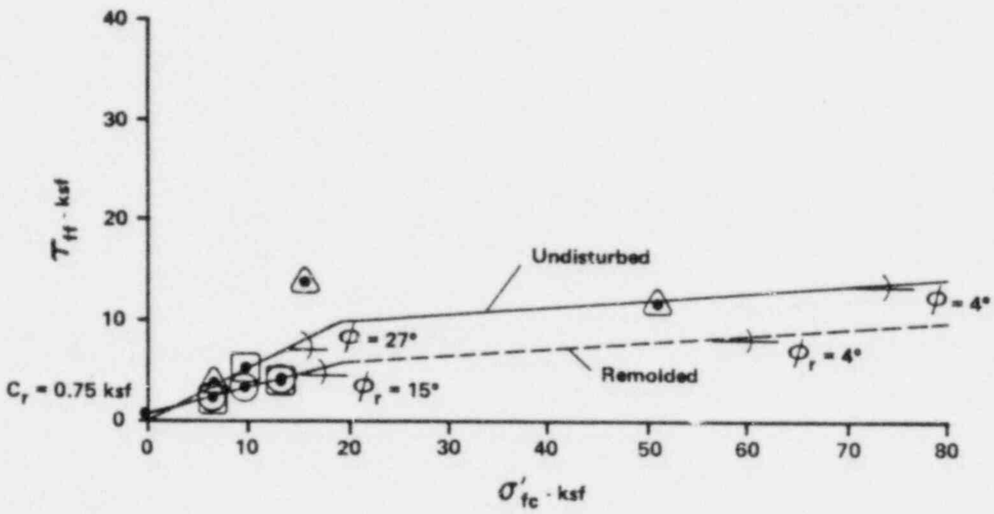
Figure B-18



CU Direct Shear Test Results - Boring RD-3, Sample PB-4
Material Type 2

Figure B-19





EXPLANATION

CU triaxial tests.

CU direct shear tests-undisturbed samples.

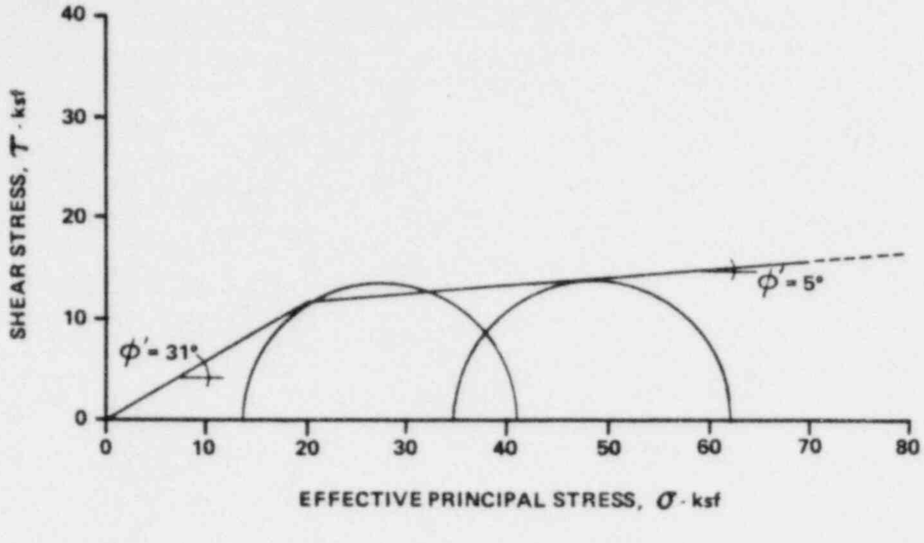
CU direct shear tests-remolded samples.

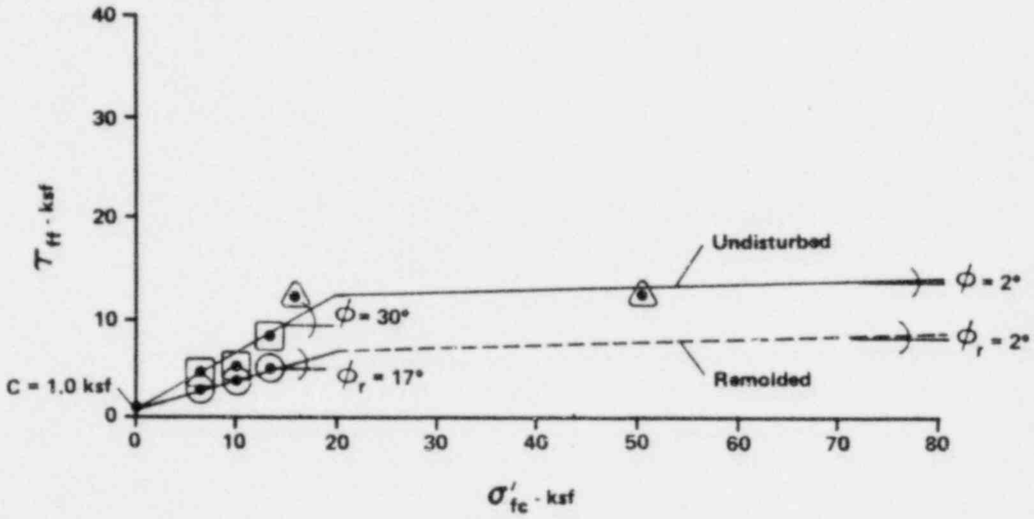
Earth Sciences Associates
Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS

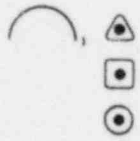
STRENGTH ENVELOPES AT 10% STRAIN
MATERIAL 1

Checked by / Approved by	Project No.	Figure No.
Checked by E.M. Forbes Date 8/28/80 Approved by RLMcNeill Date 8/29/80	1886	B-20





EXPLANATION



CU triaxial tests.

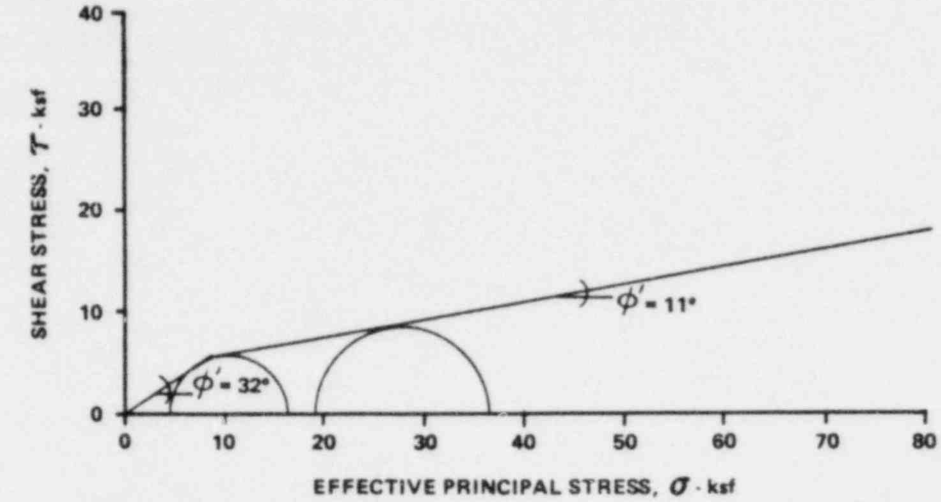
CU direct shear tests-undisturbed samples.

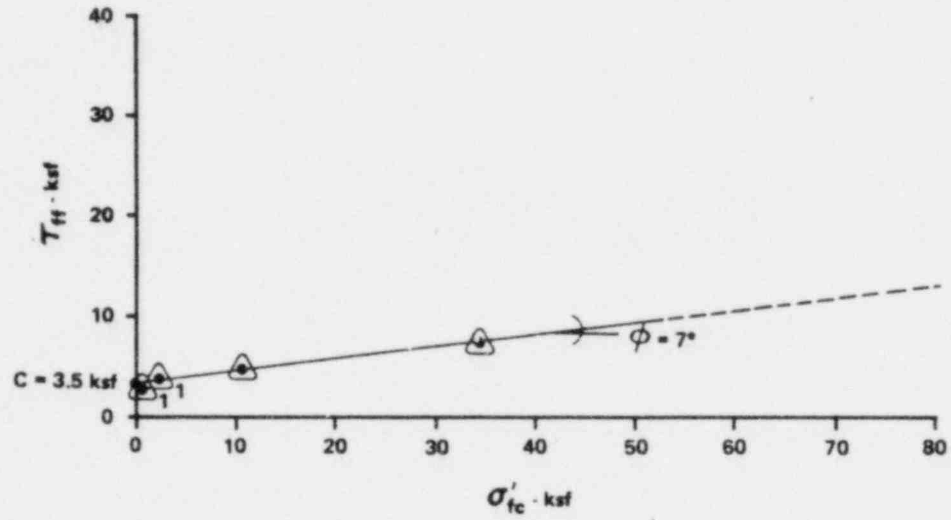
CU direct shear tests-remolded samples.

Earth Sciences Associates
Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS
STRENGTH ENVELOPES AT 10% STRAIN
MATERIAL 2

Checked by B.M. Jodar Date 8/28/80	Project No.	Figure No.
Approved by R.L. Merhan Date 8/28/80	1886	B-21





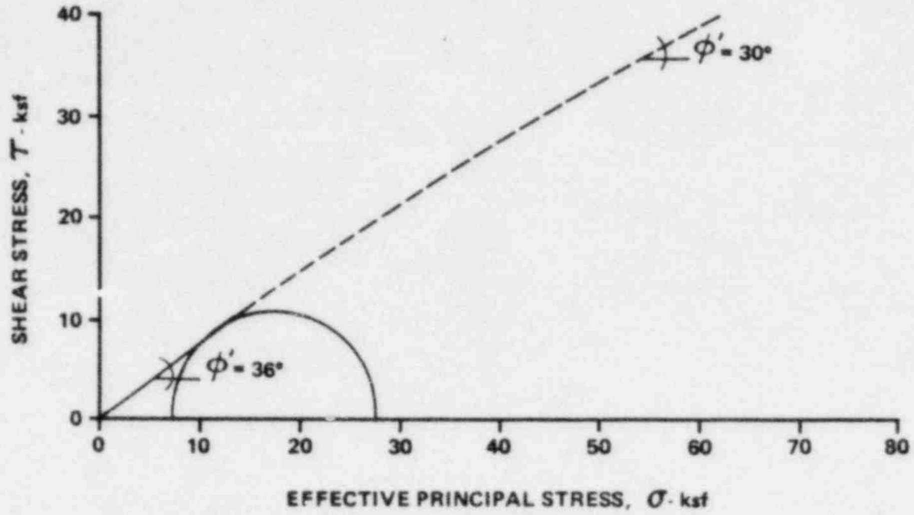
EXPLANATION

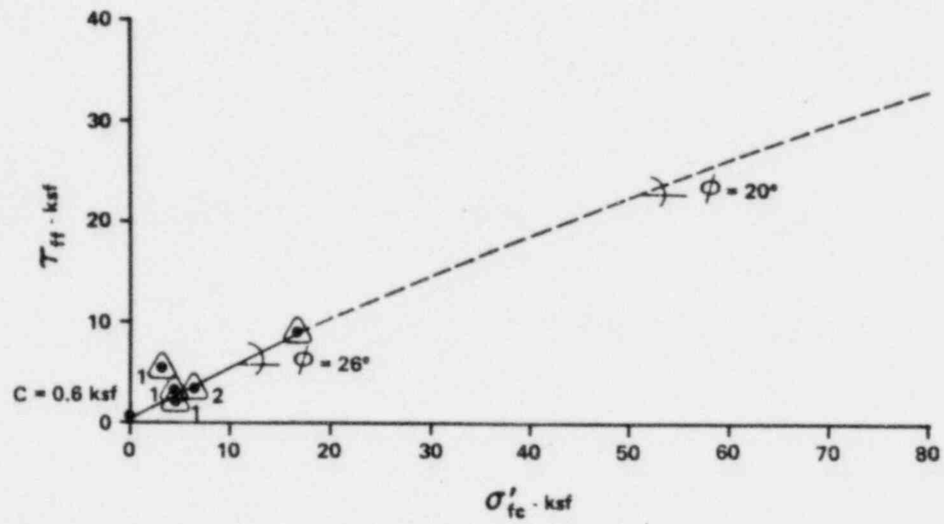
$\triangle$, $\triangle_1$ CU triaxial tests; subscript 1 indicates tests by Shannon and Wilson, 1973.

Earth Sciences Associates
Palo Alto, California

GETR LANDSLIDE STABILITY ANALYSIS
STRENGTH ENVELOPES AT 10% STRAIN
MATERIAL 3

Checked by D M Galar Date 8/28/80 Approved by R L Rahn Date 8/28/80	Project No. 1886	Figure No. B-22





EXPLANATION

CU triaxial test; subscript 1 indicates tests by Shannon and Wilson, 1973; subscript 2 indicates test by Dames and Moore, 1960.

Earth Sciences Associates Palo Alto, California		
GETR LANDSLIDE STABILITY ANALYSIS **STRENGTH ENVELOPES AT 10% STRAIN** **MATERIAL 4**		
Checked by [illegible] Date 9/28/80 Approved by [illegible] Date 9/28/80	Project No. 1886	Figure No. B-23

APPENDIX C
STABILITY ANALYSIS

Introduction

The procedures used to analyze the stability of the landslide complex in the hills north of the GETR are presented in this appendix. The results of these analyses are presented here and are also summarized in Section IV.C. of the main report. The analyses described were aimed primarily at assessing the behavior of the landslide complex under earthquake loadings compatible with the design seismic event required for structural analysis of the GETR (Newmark and Hall, 1980).

Landslide Model

The landslide complex in the hills north of GETR was modelled on a two-dimensional transverse section in which plane strain conditions apply. The section chosen for analysis is shown on Figures 1 and 2 as section X-X'. This particular section was selected for two main reasons. First, the geologic units and structure, as exposed near the ground surface, are well documented along almost the entire length of the hillslope portion of this section (see Section III.A.) and second, this section is representative of the major portion of the landslide complex as shown on Figure 1.

The generalized distribution and character of the geologic units and the daylight locations and near-surface attitudes of shears along this section were described previously (see Section III.A.; Figure 2). Because of adverse drilling conditions, the coarse-grained nature of much of the section, and the great depths involved, it proved infeasible to determine the location of failure surfaces at depth by means of subsurface exploration. However, the extensive near surface data and geologic interpretation provided a rational basis on which the geometrical models of the inferred existing failure surfaces shown on Figures C-1 and C-2 were developed. The toes of both of the modelled failure surfaces daylight at the location of the B-1/B-3 shear shown on Figure 2. From this point the surfaces dip back under the hills with an initial inclination of 15 degrees to the horizontal, the average observed near surface dip of the B-1/B-3 shear. The two failure surfaces then project down dip so that they stay within a weaker fine grained unit for as far as possible. The heads of the failure surfaces daylight upslope at the two locations where evidence of tensional, high angle shearing was found in Trenches G-6 and G-9 (see Figure 2). From these points the shear surfaces project downslope with an intial inclination of 60 degrees to the horizontal, the observed near surface dip of the shears. The head and toe portions of the failure surfaces thus defined are connected by a smooth curve to complete the modelled shear surfaces. The resulting model of the landslide complex thus consists of one block which involves nearly the full height of the existing slope and a second block which involves the lower half to two thirds of the slope, both toeing out along the same basal shear surface. This model of the landslide complex was analyzed as described in the following sections.

Seismic Stability

The stability of the modelled landslide complex was analyzed for earthquake loading conditions. The analytical procedure followed is based on the approach recommended by Makdisi and Seed (1978) for estimating earthquake-induced deformations in dams and embankments. The work by Makdisi and Seed represents, in turn, modifications of, and improvements on, earlier work by Newmark (1965), Ambraseys and Sarma (1967), Sarma (1975), and Franklin et al. (1977). Although the case at hand involves a natural slope rather than an engineered embankment, the basic physical and conceptual similarities in the two cases are judged sufficient to justify use of the Makdisi and Seed approach.

The analytical approach utilized in this investigation consisted of the following steps:

1. A value of yield seismic coefficient (k_y) was established for each failure surface; this value is an index of the resistance of the failure surface to deformation.

2. An appropriate value of maximum seismic coefficient (k_{max}) acting on a given landslide mass as a result of the design seismic event was established; this value represents the driving force tending to cause deformation.

3. Permanent displacement (u) was then estimated based on a relationship between u and the ratio k_y/k_{max}.

Step 1

The first step in the analysis was accomplished by performing pseudo-static total stress analyses on the landslide model described previously using total strength parameters appropriate to the inferred existing failure surfaces. This method of analysis is generally considered applicable for materials which do not suffer signficant loss of strength due to cyclic loading (Seed, 1979; Makdisi and Seed, 1978), as is judged to be the case for the materials tested during this investigation.

The total strength values appropriate to various portions of the failure surfaces were selected from the envelopes of normal effective stress on the failure plane at consolidation (σ'_{fc}) versus undrained shear strengths on the failure plane (τ_{ff}) shown on Figures B-20 to B-23. For purposes of the analysis, the failure planes were modelled as discrete segments to account for differences in average consolidation stress and material type along the surfaces. The in-situ consolidation stress on a given segment of the failure plane was taken as being equal to the average effective vertical overburden pressure. Effective overburden pressures were calculated based on average values of the total unit weights summarized on Tables B-1 through B-3 and the piezometric surface shown on Figure 2. The choice of the strength envelope to use for any given segment of the failure plane was based on the geologic units (as shown on Figure 2) within which that segment occurred. Remolded total strengths were used in the units characterized by fine grained material types 1 and 2. These lower strength values were used to account for effects such as pre-existing slickensided failure surfaces, lack of cementation, and remolding due to large shear displacements which one might postulate as applicable to a pre-existing landslide failure plane in fine grained materials. Where the failure planes pass through the unit characterized by material types 1 and 3 (see Figure 2) the remolded total strength of material type 1 was used unless the total undisturbed strength of material type 3 was lower at the given confining stress, in which case the lower of the two values was used. In the units containing material types 3 and 4, the lower total strengths of material type 3 were used throughout. This value is judged to be quite conservative because much of those sections are known to be predominantly coarse-grained from the trench exposures. The total undisturbed strength was used where the failure planes pass through sections characterized by material type 4. These gravelly to cobbly coarse-grained soils would not be expected to show any significant strength reduction within the failure zone.

The pseudo-static total stress analyses were performed utilizing a general purpose slope stability program, STABL, developed at Purdue University (Siegel, 1975; Boutrup, 1977). The program has been written for the general solution of slope stability problems using a two-dimensional limit equilibrium method. Calculation of the factor of safety against instability of the slope is performed by a method of slices based on Janbu's simplified method of slices for shear surfaces of general shape. This method satisfies overall moment and vertical force equilibrium, and assumes horizontal interslice forces.

The horizontal seismic coefficient (k_h) used in the analyses was varied in a trial and error procedure to converge on the value of k_h which resulted in a factor of safety of unity (FS = 1.0) for the given failure surface. This value is termed the yield seismic coefficient (k_y). The results of these analyses to establish k_y for the two failure surfaces considered are shown graphically on Figure C-3.

Step 2

The next step in the analysis requires that an appropriate value of maximum seismic coefficient (k_{max}) for the failure masses be established. The maximum seismic coefficient is the earthquake-induced simultaneous seismic coefficient acting on a mass bounded by a given failure surface and is a function of the peak crest acceleration for an embankment configuration. For the purposes of this analysis, the maximum effective peak acceleration of 0.75 g specified for structural analysis of the site (Newmark and Hall, 1980) is taken as the maximum ground acceleration ($\ddot{u}_{max}$) at the crest of the hills north of the GETR. The appropriate value of the maximum seismic coefficient corresponding to this maximum crest acceleration is estimated from the relationship of "maximum acceleration ratio" ($k_{max}/\ddot{u}_{max}$) to the normalized depth of the sliding mass (y/h) as shown on Figure C-4. For the two failure surfaces under consideration the ratio y/h is equal to one. Thus, a conservative estimate of k_{max} from Figure C-4 is given by:

$$\begin{aligned} k_{max} &= 0.45\ \ddot{u}_{max} \\ &= 0.45\ (0.75) \\ &= 0.34 \end{aligned}$$

It is in fact doubtful whether such a seismic coefficient would act simultaneously over the full lateral extent of the slide which is on the order of a mile, or at least several wavelengths, long. The so-called "tau" reduction effect would probably reduce the 0.34 to a lower value.

Step 3

The final step in the analysis procedure is to estimate the amount of permanent deformation (u) that might be expected to occur as a result of the design seismic event. The amount of permanent deformation on failure surfaces subjected to seismic loadings from various magnitude earthquakes have been calculated for a wide range of yield and maximum seismic coefficients (Makdisi and Seed, 1978). For a design earthquake of M 7.5 the relationship of u to k_y/k_{max} is as shown on Figure C-5. The ranges of permanent deformation for the two failure surfaces analyzed, as estimated from Figure C-5, are summarized on Table C-1.

Static Stability

In normal practice an effective stress analysis would be used to evaluate the long term stability of a natural slope. However, for this investigation a total stress analysis was used to estimate the static stability of the landslide complex at the GETR site. This approach was chosen in order to utilize the results of the reduced shear strength envelopes developed from the consolidated-quick direct shear tests performed on the remolded samples. These strengths were used to represent strength that might be applicable to pre-existing failure planes in the fine grained units. Effective strength parameters for these materials were not established during the laboratory testing program due to the emphasis placed on the seismic loading case analyzed in this investigation.

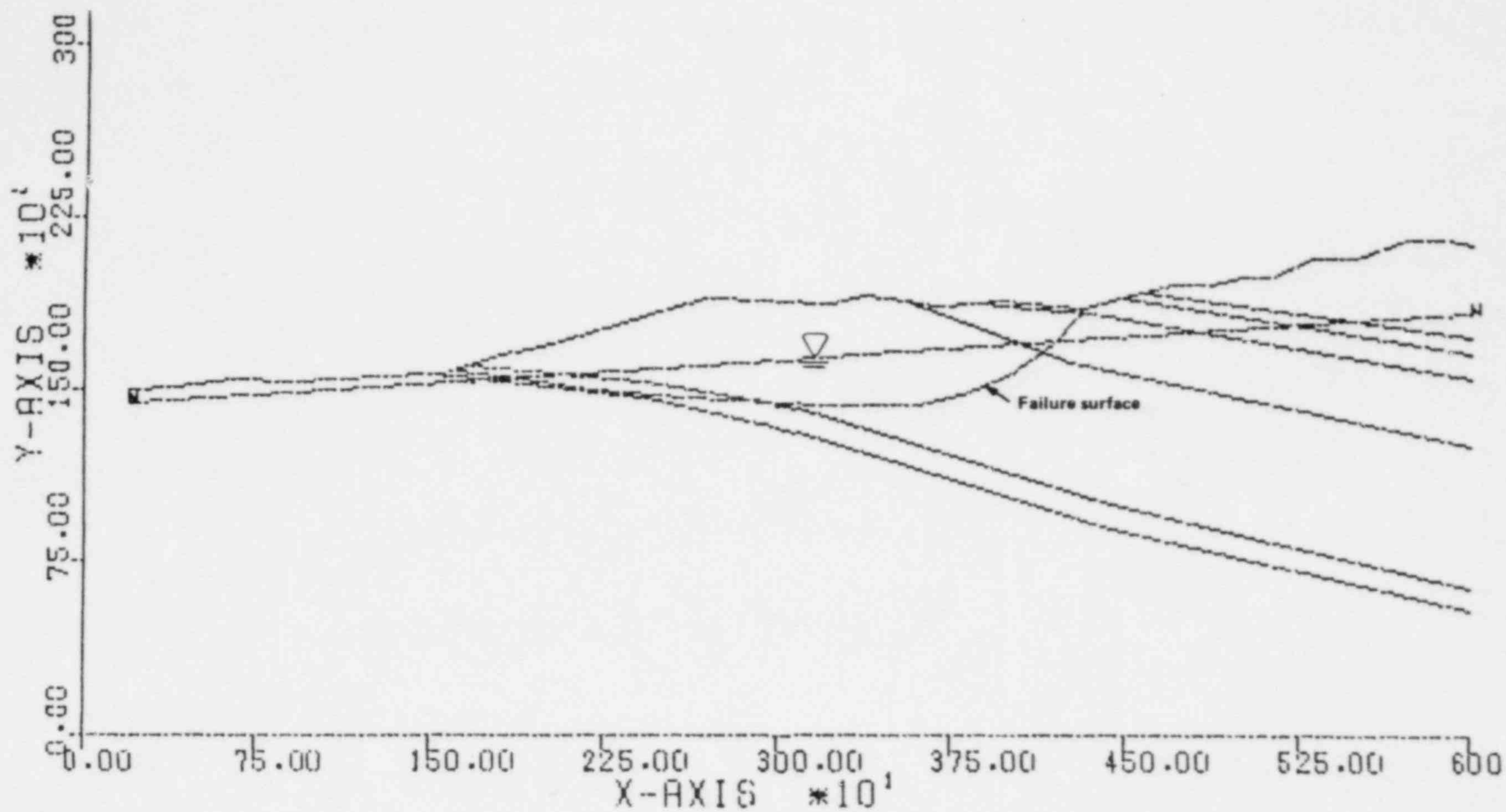
The static total stress analysis was performed using the same computer model and total shear strengths as used in the seismic stability analysis previously described. The only difference was that a horizontal seismic coefficient was not applied. The factors of safety calculated for the two failure surfaces analyzed are shown on Table C-1 along with the results of the seismic stability analyses.

For purposes of comparison, two additional analyses were performed using the failure surfaces previously described and effective and total strength parameters developed for the undisturbed soil samples. The factors of safety calculated for these two analyses differed by only 20 percent, with the effective stress analyses yielding higher factors of safety. Based on these findings, the use of total strength parameters yields factors of safety which are conservative. Therefore the use of the remolded total strength parameters in a total stress analyses to evaluate the static stability of the landslide complex is justified and probably conservative.

TABLE C-1

RESULTS OF SEISMIC STABILITY ANALYSIS

Failure Surface	Static Factor of Safety	Yield Seismic Coefficient, k_y (g)	Range of Yield to Maximum Seismic Coefficient Ratio k_y/k_{max}	Range of Estimated Permanent Displacement, u (cm)
Full-slope	2.9	0.18	0.53	3 - 18
Mid-slope	4.4	0.22	0.65	1 - 9



NOTES

1. See Figure 2 for description of units.
2. Scaled axes are in feet.
3. No scale exaggeration.
4. Location of scale origins is arbitrary.

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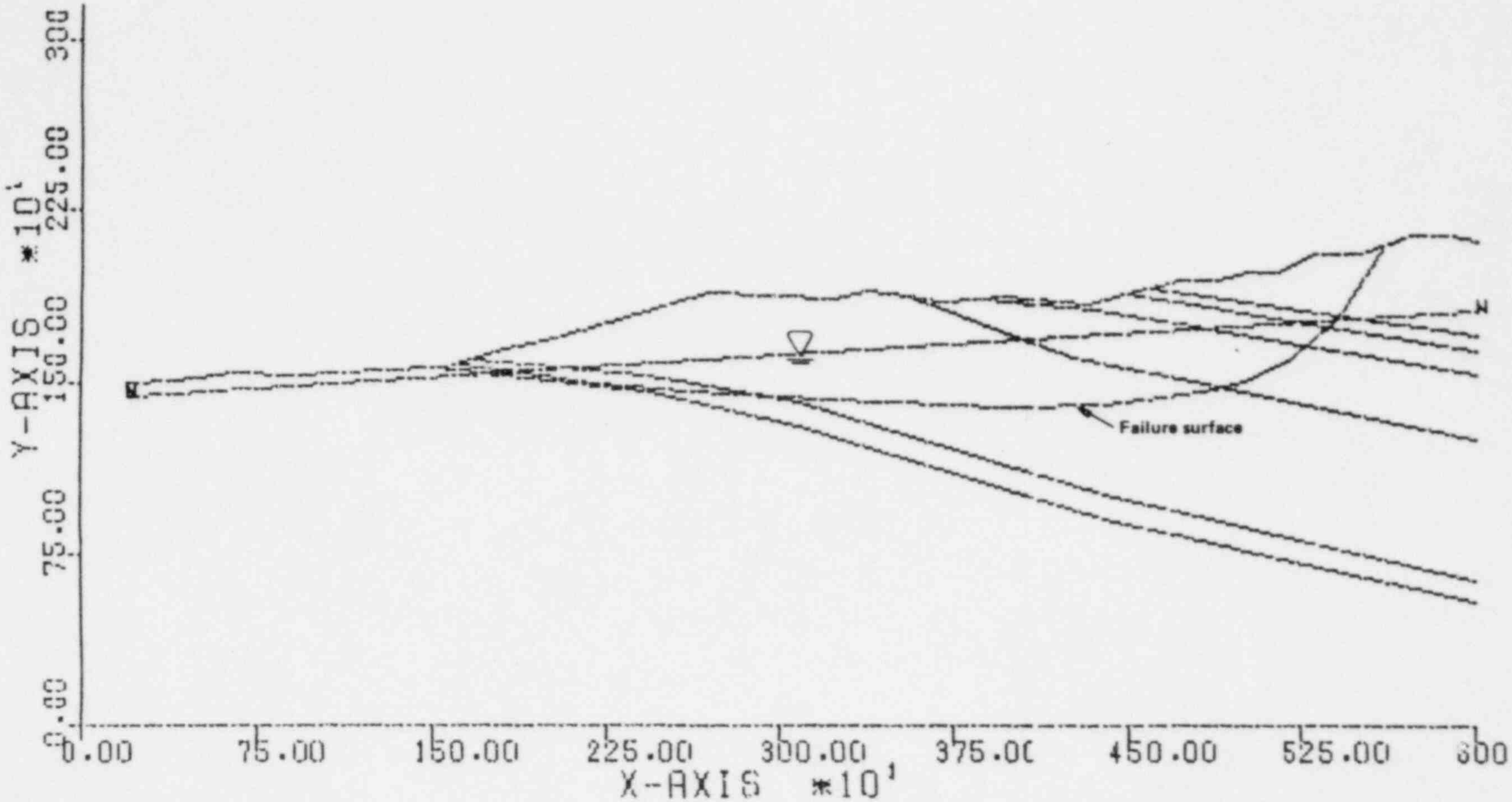
GETR LANDSLIDE STABILITY ANALYSIS
COMPUTER MODEL
EXISTING MID-SLOPE FAILURE SURFACE

Checked by [illegible] Date 8/28/80	Project No.	Figure No.
Approved by [illegible] Date 8/26/80	1886	C-1



NOTES

1. See Figure 2 for description of units.
2. Scaled axes are in feet.
3. No scale exaggerat on.
4. Location of scale origins is arbitrary.

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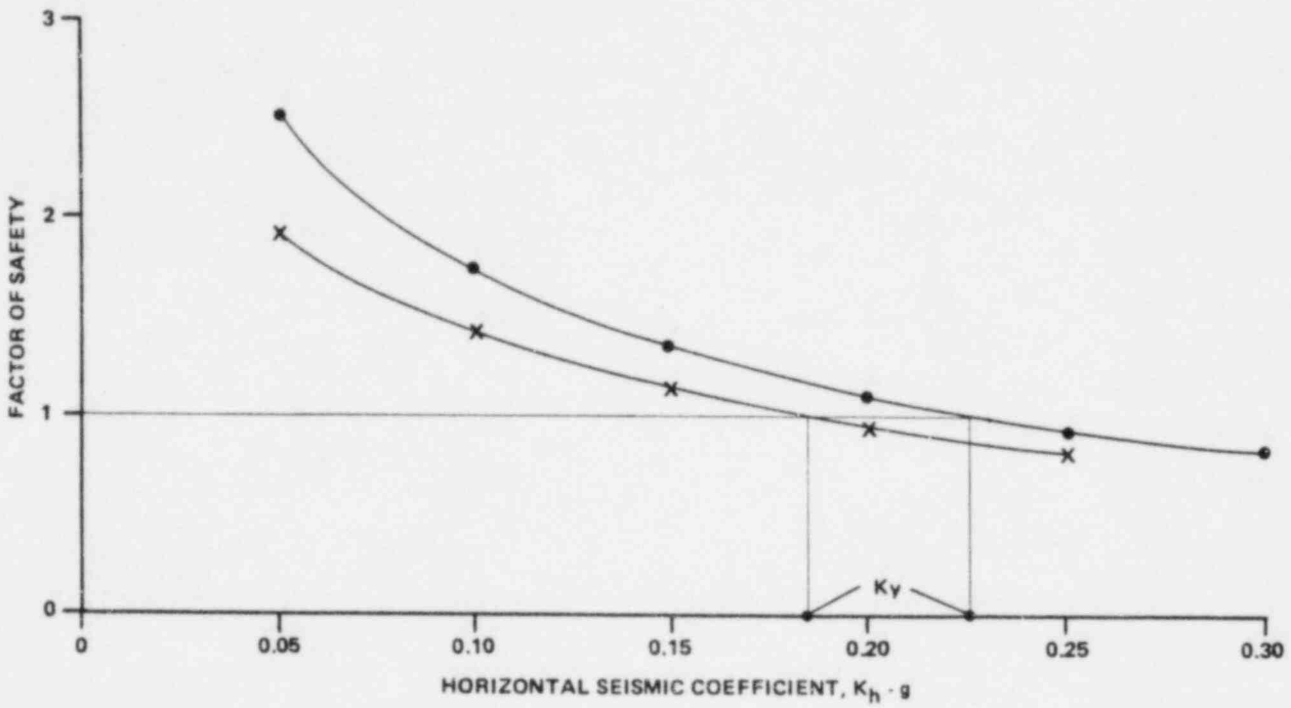
GETR LANDSLIDE STABILITY ANALYSIS **COMPUTER MODEL** **EXISTING FULL-SLOPE FAILURE SURFACE**		
Checked by DM Jordan Date 8/28/80 Approved by R L Mahan Date 8/28/80	Project No. 1886	Figure No. C-2

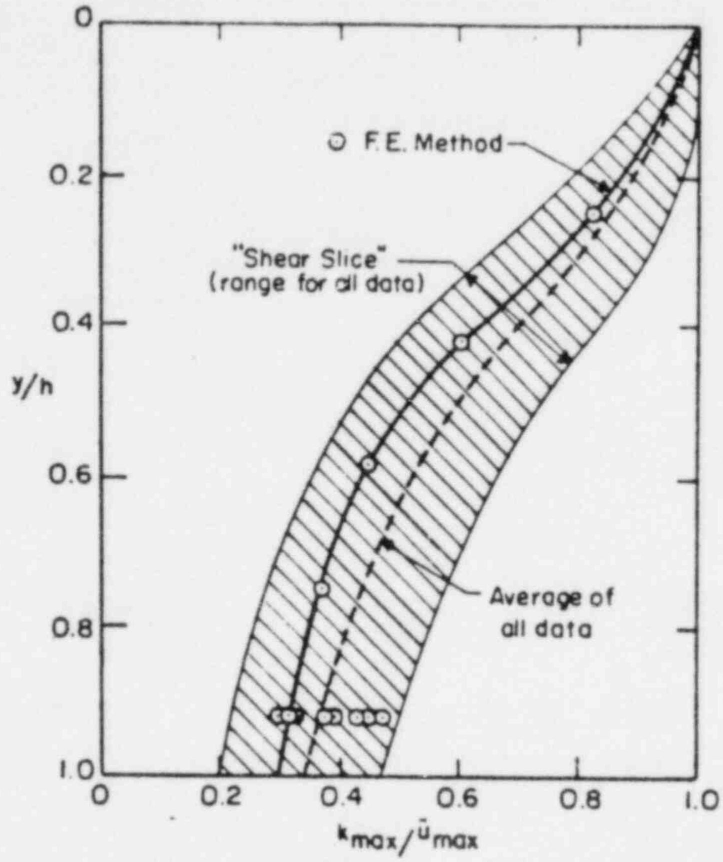


● Existing mid-slope failure surface

X Existing full-slope failure surface

Ky Yield seismic coefficient

Earth Sciences Associates Palo Alto, California		
GETR LANDSLIDE STABILITY ANALYSIS **VARIATION OF FACTOR OF SAFETY WITH HORIZONTAL SEISMIC COEFFICIENT**		
Checked by D.M. Yadon Date 8/28/80 Approved by RL Wuhlan Date 8/28/80	Project No. 1886	Figure No. C-3



(from Makdisi and Seed, 1978)

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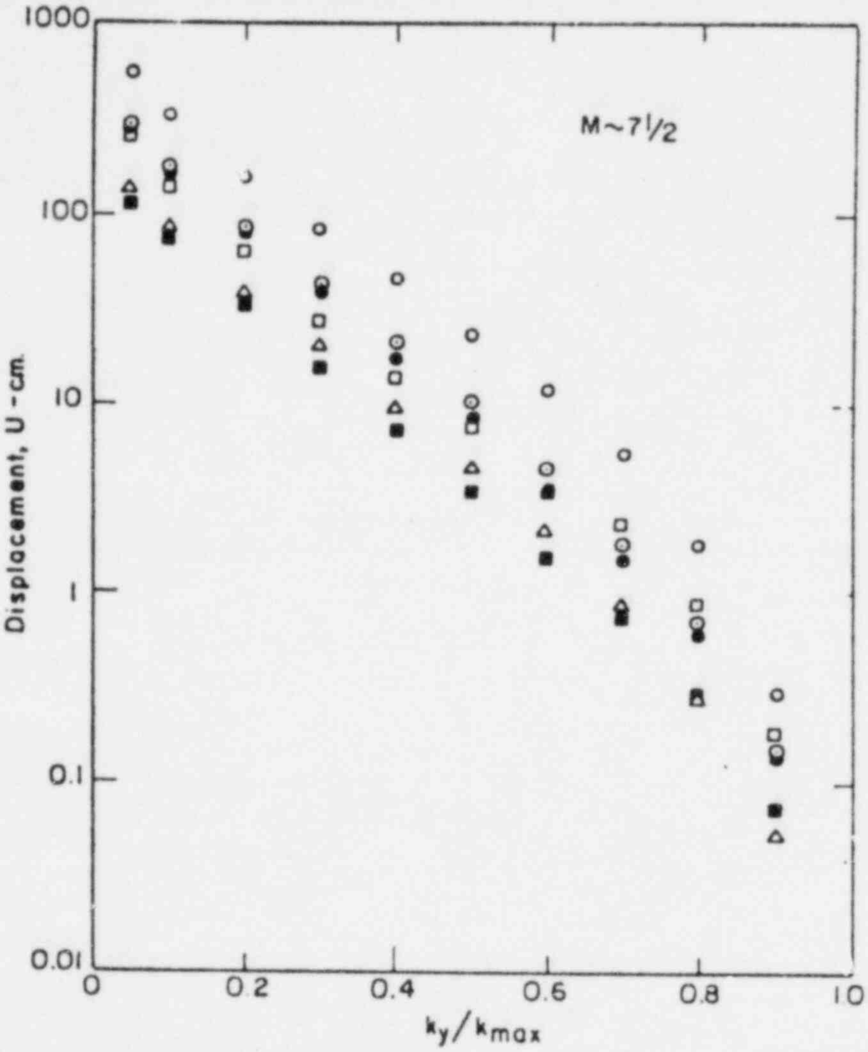
GETR LANDSLIDE STABILITY ANALYSIS
VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

Checked by D.M. Saylor	Date 8/29/80	Project No. 1886	Figure No. C-4
Approved by R.L. Matheson	Date 8/29/80		



(from Makdisi and Seed, 1978)

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GETR LANDSLIDE STABILITY ANALYSIS
VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION-MAGNITUDE 7.5 EARTHQUAKE

Checked by D.M. Jagon	Date 8/28/80	Project No. 1886	Figure No. C-5
Approved by DC Marhan	Date 8/28/80		