VOL. 105 NO. GT7. JULY 1979

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

PROCEEDINGS OF
THE AMERICAN SOCIETY
OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

VOL. 105 NO. GT7. JULY 1979

JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

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PSP	=	pore size parameter;
q	=	volume flow rate;
R^2	=	coefficient of determination;
R_H	=	hydraulic radius;
$R_H^2 n$	=	hydraulic radius model pore size parameter;
r	=	pore radius;
S	=	hydraulic gradient;
S_s	=	specific surface area;
T_s	=	surface tension;
v	=	discharge velocity;
v_s	=	seepage velocity;
w	=	water content, as a percentage;
γ	=	unit weight;
δ	=	pore diameter corresponding to intrusion pressure;
θ	=	contact angle; and
μ	=	absolute viscosity.

Sample Code

D	=	dry of optimum water content;
H	=	high compactive effort;
L	=	low compactive effort;
M	=	medium compactive effort;
O	=	near optimum water content;
$O-D$	=	slightly dry of optimum water content;
S5	=	50% silt and 50% kaolin;
S7	=	70% silt and 30% kaolin;
S9	=	90% silt and 10% kaolin; and
W	=	wet of optimum water content.

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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

LANDSLIDE IN CLAYSTONE DERIVED SOIL

By Archie M. Richardson,[1] M. ASCE

INTRODUCTION

Landslides are one of the major natural disasters in the Pittsburgh area. Millions of dollars are lost annually through damages resulting from landslides and costs for corrective construction. The principal soil formations in which landslides occur are the deposits of colluvial soils on the valley slopes and at the transition from slope to flood plain. The geology of southwestern Pennsylvania has been described by others (9,10,12,24,25,26). Briefly, the region is characterized by relatively flat-lying sedimentary rocks in repeating sequences of coal seams and strata of sandstones, limestones, shales, and claystone. As erosional valley formation proceeds, the outcrops of these strata are exposed to weathering. The material removed by weathering from the rock outcrops is transported downslope by gravity and is subjected to additional weathering during downslope movement. The thickness of the soil material on the slope varies, and often a talus-type accumulation of material builds up at the toe of the slope.

Several of the strata of claystones that outcrop are poorly indurated, and some are, in fact, layers of hard clay. These formations are fissured and contain slickensides that may have been formed by slumping during deposition but most probably resulted from shear failures resulting from stress relief that occurred as erosion removed overburden and formed valleys (7,8). Soil deposits containing weathered material derived from such claystones are particularly landslide prone and are frequently found to contain preexisting planes of shear displacement (4). Landslides in such deposits sometimes occur spontaneously, especially where stream activity erodes material at the toe. More frequently, destructive landslides

Note.—Discussion open until December 1, 1979. To extend the closing date one month, a written request must be filed with the Editor of Technical Publications, ASCE. This paper is part of the copyrighted Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 105, No. GT7, July, 1979. Manuscript was submitted for review for possible publication on March 24, 1978.

[a]Presented at the April 24–28, 1978, ASCE National Spring Convention and Continuing Education Program, held at Pittsburgh, Pa. (Preprint 3200).

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occur when cuts have been made at the toe of such deposits or where fill has been placed on such slopes.

This paper describes a landslide that occurred in a soil deposit derived from the Pittsburgh Redbed Claystone when a cut was made at the toe. The topography before the slide was known and the geometry of the slide mass was established. Slope stability analysis indicated that the landslide could have been predicted

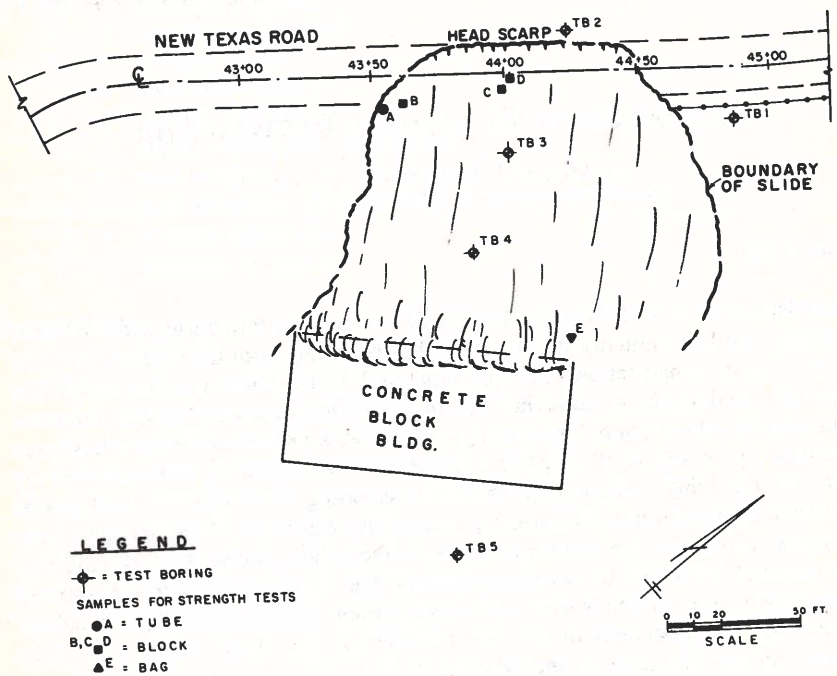

FIG. 1.—Plan: New Texas Road Landslide

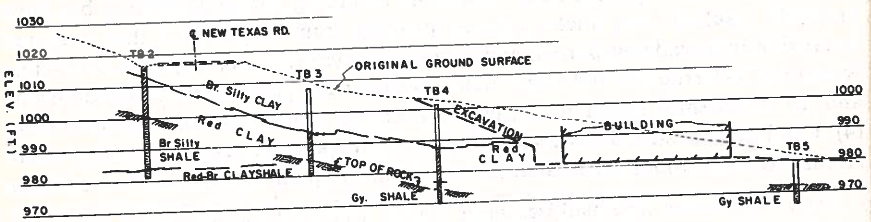

FIG. 2.—Section along Axis of Slide

using effective stress-shear-strength parameters of $\bar{\phi}$ = 12° to 13° and $\bar{c}$ = 0, in which $\bar{\phi}$ = the angle of shearing resistance; and $\bar{c}$ = the cohesion intercept in terms of effective stresses. Residual shear-strength parameters measured in large displacement direct shear tests on the soil varied from $\bar{\phi}$ = 6° and $\bar{c}$ = 120 psf (5.8 kN/m^2) to $\bar{\phi}$ = 12° and $\bar{c}$ = 400 psf (19.2 kN/m^2).

Pittsburgh Redbeds

The Pittsburgh Redbed Claystone stratum is the parent of much of the soil material involved in landslides in the Pittsburgh area and occurs about midway between the base of the Pittsburgh Coal and the top of the Upper Freeport Coal in the Conemaugh Formation. Thus, in the Pittsburgh area, it outcrops at elevations conducive to formation of downslope colluvial soils. This bed varies from approx 10 ft (3 m) to over 50 ft (15 m) in thickness and is essentially a hard clay material. The soil resulting from weathering of this unit typically contains 20%–25% finer than 2 μ, has a liquid limit of 30%–45%, and is classified as a CL soil in the Unified Soil Classification System (11,25,26). Other similar claystone and clay shale outcrops occur, and soils very similar to those resulting from weathering of the Pittsburgh Redbed unit are found over southwestern Ohio, West Virginia, northwestern Virginia, and elsewhere.

Published maps and other information indicating probable locations of landslide-prone material exist (1,2,14,17,19). Developments have often been sited without reference to published information, and many landslides might have

TABLE 1.—Soil Samples Shortly after Landslide

Sample number (1)	Elevation, in feet (2)	Type of sample (3)	Remarks (4)
A	1,006	driven tube	embankment material
B	1,006	block	red clay material
C	1,007	block sample	failure plane near headscarp
D	1,010	block sample	failure plane near headscarp
E	988	bag	failure plane near toe of slide

been avoided by changing grading plans that required cutting or filling on landslide-prone slopes. However, as land becomes more scarce it has become essentially impossible to avoid landslide-prone areas, especially for construction of transportation facilities.

Slides resulting from construction that involves cuts into or fills placed on landslide-prone soil often do not occur until after construction is complete. Construction is often accomplished during relatively dry periods, and a landslide occurs subsequent to completion of construction following a period of rainfall or a rise in the regional ground-water levels. Thus, buildings, highways, sewers, and other constructed facilities may be partially or fully completed and even occupied before being damaged by a landslide caused by site grading but triggered by water activity.

New Texas Road Landslide

The landslide on New Texas Road (LR 02159), Plum Borough, Allegheny County, Pa., occurred during the early part of September, 1975. The slide was about 120 ft (36 m) wide and about 130 ft (39 m) from the head to the toe of the slide. Fig. 1 is a plan of the slide area indicating the locations of test borings, test pits, and piezometers. A section on the center line of the slide is presented in Fig. 2. The original slope of the surface of the landslide was about 5 horizontal on 1 vertical (approx 12°). Site grading for a commercial building included cutting at the toe of the slide as shown in Fig. 2. The cutting was completed in mid July. The concrete block building was partially completed at the time of the slide and was destroyed by the landslide. A 100-ft (30-m) long section of New Texas Road was removed by the landslide, and the road was closed until reconstructed.

The slide activity was first noted on or about September 3. Movement accelerated during the second week of September, and the road was closed on September 10. The building collapsed on September 18. Relatively heavy rainfall occurred on August 29, 30, and 31.

Subsurface Investigation

Samples of soil were obtained at five locations shortly after the landslide occurred. Four samples were obtained close to the surface on which sliding

TABLE 2.—Observed Water Elevations, in feet

Test Boring number (1)	36 hr (2)	11 days (3)	31 days (4)
1	—	1,007	1,006
2	995	998	996
3	—	977	975

occurred, and one sample was obtained in the material above the zone of sliding that supported the roadway section. The locations of these samples are shown on Fig. 1, and the types of samples appear on Table 1. Descriptions, index properties, and shear strengths of these soils appear in a following section.

Five test borings totaling 125 ft (38 m) were drilled in December, 1975. These test borings were drilled to obtain information on the soil zones involved in sliding and on the depth and type of bedrock, to obtain soil samples for laboratory testing, and to install piezometers of the open standpipe type for long-term observations of ground-water conditions. Except for Test Boring 1, NX cores of bedrock were obtained in all test borings.

Soil Conditions

The soil zone in the landslide area consisted of two distinct layers at the test boring locations. Overlying bedrock was a 6-ft (1.8-m) to 14-ft (4.2-m) thick layer of a reddish soil, classified as CL according to the Unified Soil Classification System, with some silt near the head of the slide. This layer was absent in Test Boring 5, which was drilled beyond the toe, and may have been removed by excavation during site grading.

Above the red clay layer, a 6-ft (1.8-m) to 16-ft (4.9-m) thick layer of brown

clay and silt, classified as CL or CL-ML according to the Unified Soil Classification System, was encountered. This material was absent in Test Boring 2, which was drilled above the head scarp of the slide. A reddish-brown silt fill formed the roadway embankment. These soil zones are indicated on Fig. 2, which is a section on the axis of the slide as established in the field. Wet reddish soil was encountered at the interface of the red and brown zones in Test Borings 3 and 4. This wet soil zone was judged to be the zone in which sliding was occurring.

Bedrock Conditions

It is judged as probable that the soils at the site consist of colluvial soils formed from downhill transportation by gravity of material resulting from

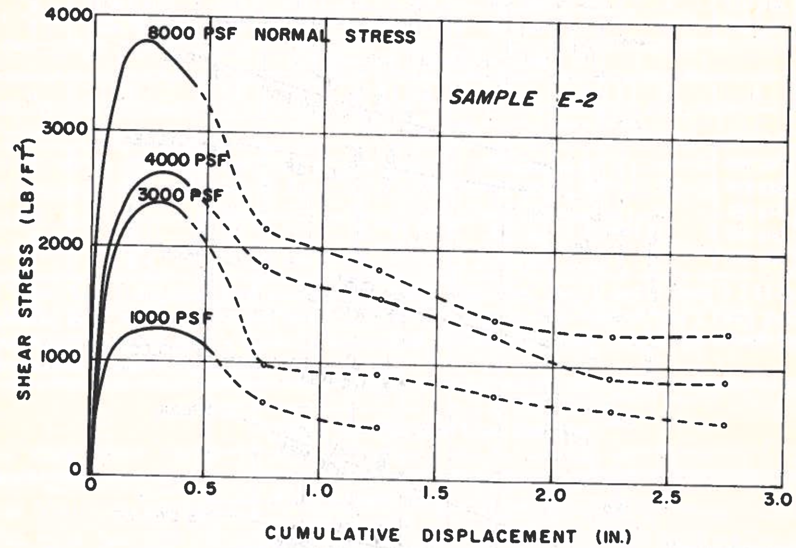

FIG. 3.—Shear Stress Versus Displacement

weathering or rock formations outcropping above the slide area. The red soil layer probably originated from an outcrop of the Pittsburgh Redbed Claystone Formation or a similar claystone formation outcropping just above the head of the slide zone, and the brown clay and silt layer probably originated from a clay shale stratum overlying the redbed formation. Downhill transportation probably resulted in the observed two-layer stratification.

Published regional geologic information (24,25) places the landslide stratigraphically in the Saltsburg Member of the Glenshaw Formation in the Conemaugh Group of rocks of Pennsylvanian Age. This means that the Pittsburgh Redbed Claystone Formation probably outcrops at and above the elevation of the slide. Bedrock types cored in the slide zone are indicated on Fig. 2 and were shales, clay shales, and claystones.

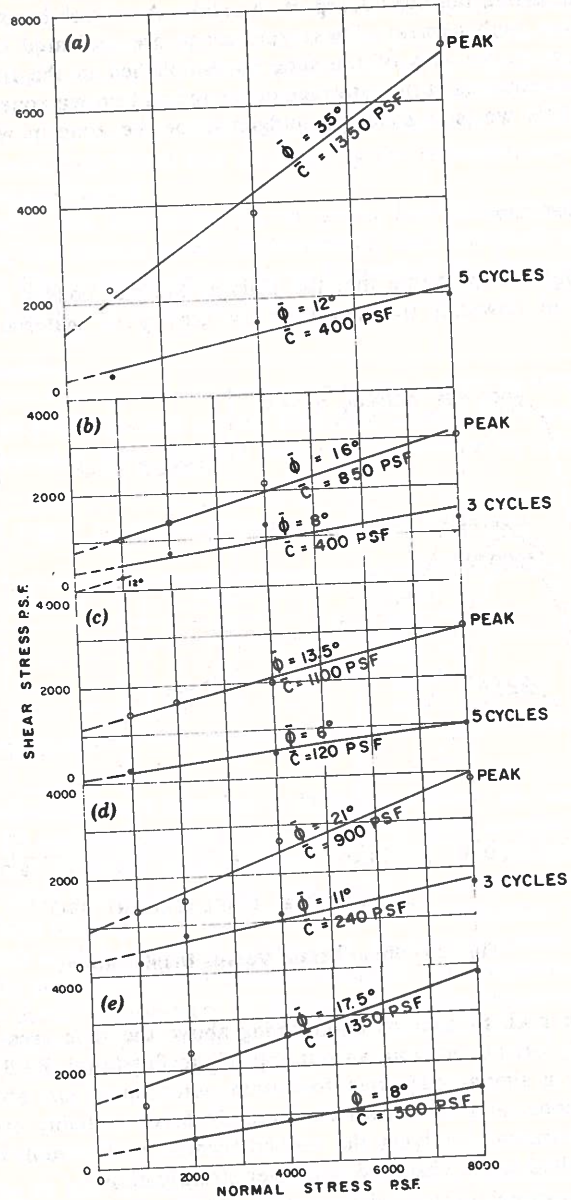

FIG. 4.—Strength Envelopes: (a) Sample B (Red Clay, Some Sand, and Shale Fragments); (b) Sample C (Red Clay); (c) Sample D (Red Clay); (d) Sample E-1; (e) Sample E-2

Ground-Water Conditions

Open standpipe-type piezometers sealed near the ground surface to reduce the risk of surface-water infiltration were installed in Test Borings 1, 2, and

TABLE 3.—Direct Shear Test Results

Test number (1)	Sample (2)	LL as a percentage (3)	PI as a percentage (4)	No. 200 as a percentage (5)	Clay size as a percentage (6)	Peak $\bar{c}$ (7)	Peak $\bar{\phi}$ (8)	Number of cycles (9)	Residual $\bar{c}$ (10)	Residual $\bar{\phi}$ (11)
(a) Brown Clay and Silt										
1	TB3 8.5 ft	—	—	—	—	100 psf	35°	3	15 psf	34°
2	TB4 8.0 ft	34	14	55	45	200 psf	25°	3	150 psf	24°
3	TB4 16.0 ft	30	9	35	25	150 psf	36°	3	115 psf	35°
(b) Roadway Embankment Silt										
4	A	36	11	67	24	0	40°	4	0	31°
(c) Red Clay										
5	B	34	11	61	21	1,350 psf	35°	4	400 psf	12°
6	C	44	20	95	39	850 psf	16°	3	400 psf	8°
7	D	40	15	96	36	1,200 psf	13°	5	120 psf	6°
8	E-1	33	12	74	27	900 psf	21°	3	240 psf	11°
9	E-2	33	12	74	27	1,350 psf	17.5°	6	300 psf	8°

Note: Test 1 was performed using a 2-in. square shear box, tests 2 and 3 using a 2.85-in. diam circular shear box and tests 4–9 using a 2.5-in. square shear box. Tests 6 and 7 were performed on samples trimmed from a relatively undisturbed block sample while tests 8 and 9 were prepared by remolding plastic bag samples.

TABLE 4.—Triaxial Test Results

Sample number (1)	$\bar{c}$, in pounds per square foot (kilonewtons per square meter) (2)	$\bar{\phi}$ (3)	Pore-pressure parameter A (4)
C	575 (27.3)	21°	0 to −0.06
D	1,000 (47.9)	18°	0 to −0.05

4. The readings of water levels in these piezometers are shown in Table 2. The observed water levels remained fairly constant during the period of observation and were at or below the approximate top of bedrock elevation.

Topography

The topography before excavation and before the landslide was obtained from a topographic survey performed for the developer of the property and from sections of LR 2159 obtained from the Pennsylvania Department of Transportation (PennDOT) The topography after the landslide was obtained from a survey conducted in December, 1975.

Laboratory Shear Testing

Shear-strength parameters were determined in the laboratory. Direct shear tests were performed in which conditions representing large displacements were obtained by displacing the two halves of the shear box in a forward direction to shear the sample in the normal manner, stopping forward motion and returning the halves of the shear box to the original position without releasing the normal load. The sample was again sheared in a forward direction in the normal manner, and the process was repeated until the strength attained on subsequent forward displacements generally fell to a repeatable value. The normal load on the sample was then increased, and the procedure was repeated.

Such tests have been used by others to estimate the residual strength of stiff fissured clays and have been termed multiple reversal direct shear tests (3,21). Evidence suggests that the stability of first-time slides in heavily overconsolidated clays can be predicted using the shear-strength parameters determined on remolded, normally consolidated samples (22,23) but that the residual shear strength as measured in multiple reversal direct shear tests can predict the stability of slopes in heavily overconsolidated clays with preexisting planes of failure (18,21). Residual shear-strength parameters slightly lower than those obtained in multiple reversal direct shear tests have been obtained using torsional shear devices (16). Attempts have been made to relate the residual strengths to the classification indices (6). In addition, two series of $C\bar{I}U$ triaxial tests were performed on the red clay material.

Multiple Reversal Direct Shear Tests

Fig. 3 presents the results of the series of reversal direct shear tests on Sample E-2 of the red clay soil. The tests reported were performed at a rate of displacement of 0.02 in./min (0.05 cm/min). A subsequent study of the effect of rate of displacement on the residual strength of the red clay soil indicated little effect at displacement rates, which varied from 0.02 in./min–0.005 in./min (0.05 cm/min) (R. Turgeon, personal communication, 1977, 1978). The entire shear stress versus displacement plot for the first forward displacement and the value at the midpoint of each subsequent displacement cycle is plotted for each normal load. This plot is typical of the results of the large displacement direct shear tests on the red clay. The peak shear stress values and the "residual" values of shear strength for each normal load were used to define a peak and residual angle of shearing resistance $\bar{\phi}$ and shear strength axis intercept $\bar{c}$ using the method of least squares to fit a straight line to the data points as shown in Fig. 4(*e*). Figs. 4(*a*)–4(*e*) are strength envelope plots for the direct shear tests on the red clay material. Peak and residual values of $\bar{\phi}$ and $\bar{c}$ for all the samples tested in direct shear are as shown on Table 3. Values of residual angles of shearing resistance for similar soil materials have been reported in the literature (4,10,11,13) and the values reported in this paper fall within the reported range.

Triaxial Compression Tests

Consolidated undrained triaxial compression tests with measured pore-water pressures ($C\bar{I}U$ tests) were performed. Specimens 2.85 in. (7.24 cm) in diameter were trimmed from Block Samples C and D of the red clay. All specimens were compressed to 20% axial strain and the deviator stress in all samples

TABLE 5.—Analysis after Excavation

Position of phreatic surface (1)	$\bar{\phi}$ required for FS = 1.0 (2)
At or below failure surface	12°
1 ft (30 cm) above failure surface	13°

TABLE 6.—Analysis before Excavation

Position of phreatic surface (1)	FS $\bar{\phi}$ = 12° (2)	FS $\bar{\phi}$ = 13° (3)
At or below failure surface	1.24	1.34
1 ft (30 cm) above failure surface	1.20	1.30

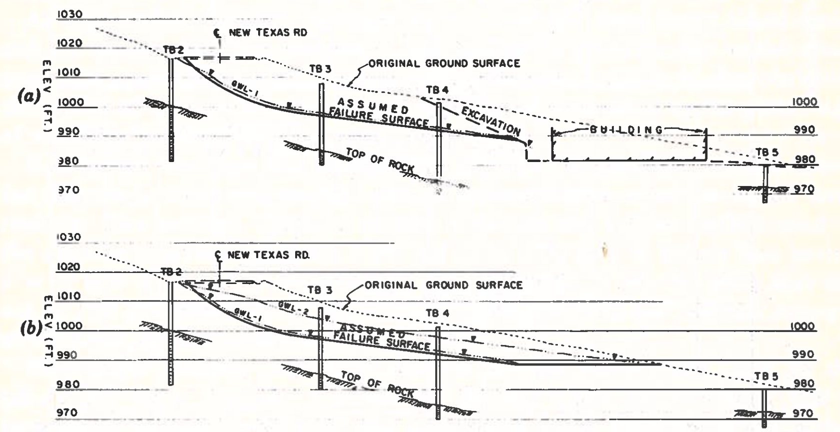

FIG. 5.—Failure Conditions Assumed for Analysis: (*a*) after Excavation; (*b*) before Excavation

was either increasing slightly or had leveled off at a strain level of 20%. Values of deviator stress and pore-water pressure at 10% strain were used to obtain the values found in Table 4.

Stability Analysis

Stability analyses using the Morgenstern-Price Method for noncircular failure surfaces were performed (19). The purpose of these analyses was to estimate the soil strengths in the field at the time of failure and at a time prior to making the cut.

The geometry of the landslide could be established reasonably well using survey information obtained before and after cutting and after the landslide occurred.

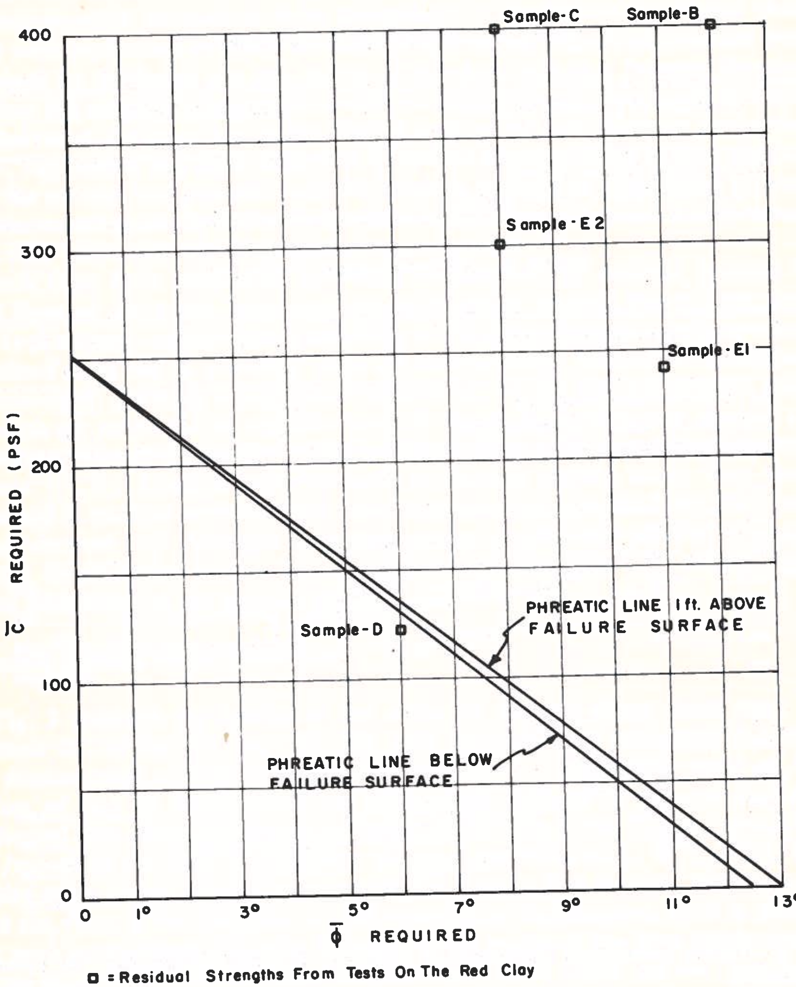

FIG. 6.—Shear Strength Parameters for FS = 1.0 after Excavation

The location of the failure plane on the axis of the slide could be observed at the head scarp and at the toe of the slide. The wet zones observed near the top of the red clay in Test Borings 3 and 4 were assumed to represent the failure plane and the failure surfaces shown on Figs. 5(*a*) and 5(*b*) were constructed by approximately connecting the head scarp, the toe, and the wet zones in Test Borings 3 and 4.

During the period of observation of ground-water levels in the piezometers, the ground-water level was near or below the top of rock, well below the assumed surface of sliding. The ground-water conditions at the time of the slide are not known. Analyses were performed for two ground-water levels, a ground-water level at or below the assumed failure surface, and a ground-water level approx 1 ft (30 cm) above the assumed failure surface. Based on the existence of the wet soil zones and calculations using consolidation theory of the time required for a rise in ground-water level of 1 ft (30 cm)—a time in excess of 4 days—the location of a ground-water level between the failure surface and 1 ft (30 cm) above the failure surface at the time of the slide does not seem unreasonable. The results of the stability analyses are presented in Tables 5 and 6.

Additional stability analyses were performed to determine combinations of $\bar{\phi}$ and $\bar{c}$ that would result in a calculated Factor of Safety (FS) equal to 1.0. The results of these analyses for the two positions of the ground-water level were used to plot Fig. 6.

Analysis

The values of residual $\bar{\phi}$ and $\bar{c}$ obtained from the multiple reversal direct shear tests are plotted on Fig. 6. It can be seen that all of the values obtained, with the exception of Sample D, would have resulted in calculated factors of safety greater than FS = 1.0 had they been used directly in stability analyses.

As indicated in Table 3, Samples B, E-1, and E-2 have lower liquid limits and are less plastic than Samples C and D. This is probably due to the much greater amounts of minus 200 sieve material in Samples C and D. In all probability, the results of tests on Samples B, E-1, and E-2 are influenced by the percentage of sand-size material in the samples.

If a failure envelope had been drawn through the residual strength points for Sample C as shown on Fig. 4(*b*), a curved failure envelope would have resulted. The average effective normal stress on the failure surface used in the stability analyses is approx 1,500 psf (71.8 KN/m^2). A failure line having $\bar{c} = 0$ and $\bar{\phi} = 12°$ is indicated on Fig. 4(*b*), and it can be seen that the use of this line would be in much better agreement with the residual strength at the lower normal pressures.

Subsequent research (13) using multiple reversal direct shear tests on the minus 200 sieve fraction of Pittsburgh Redbed Clay has suggested that a progressive degradation of grain size occurs with larger displacements at higher loads. This research is continuing, but it is felt that this phenomena can result in a curved failure envelope.

Conclusions

It is the writer's opinion that the landslide was caused by the excavation that removed lateral support near the toe of the slide. The landslide was probably triggered by a slight rise in ground-water level or a slight increase in total unit weight of soil in the sliding mass, or both, following the period of heavy rainfall. Based on the stability analyses described, the average in-situ value of the angle of shearing resistance in terms of effective stress $\bar{\phi}$ for the red clay soil most probably lies between 12° and 13° if the cohesion is assumed to be zero.

This range of values is for a position of ground water at the time of the landslide between the failure surface and 1 ft above the failure surface.

The use of the residual shear strength as obtained in multiple reversal direct shear tests of Sample D would have closely predicted the failure. This sample of soil was essentially silt and clay size. A factor of safety close to 1.0 would have resulted from stability analyses using a failure envelope for Sample C appropriate for the in-situ normal stress range. Sample C also consisted almost entirely of silt and clay-size material.

It can thus be concluded that the shear strength operating at the time of failure was closely related to the residual shear strength of the silt and clay-size fraction of the red clay soil.

Acknowledgments

The original investigation of the New Texas Road Landslide was performed by Ackenheil & Associates Geo Systems, Inc., for the Pennsylvania Department of Highways. The triaxial tests and multiple reversal direct shear tests on the red clay were performed by PennDOT, as were the tests on the roadway embankment silt. Robert Turgeon directed the laboratory testing performed in the PennDOT Bureau of Materials and Testing. The plots presented in the paper were developed from data given the writer by Turgeon. The writer appreciates the assistance given him during the investigation by R. Turgeon, U. Dash, and J. DeRoss of PennDOT. The writer is appreciative of the time made available on computer facilities at the University of Pittsburgh for additional analyses during the preparation of the paper. Ackenheil & Associates Geo Systems, Inc., provided the typing and drafting to prepare the paper and thanks are due J. Ratajczak and P. Tambellini. The interpretation of the data obtained and the conclusions presented in the paper are the writer's and complete responsibility for them is accepted by him.

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