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DISCUSSION

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8731 FAILURE OF COLLUVIAL SLOPE

KEY WORDS: Claystones; Colluvium; Cut slopes; Displacement; **Engineering geology; Landslides; Pennsylvania; Rock mechanics; Shear strength; Slope stability; Soil mechanics**

ABSTRACT: Interstate Route 279 crosses the Ohio River 9 miles northwest of Pittsburgh, Pa., and continues northward along Kilbuck Run tributary valley. One section of the highway passes through a zone of colluvium on the wall of the tributary valley. When construction of this section began in 1968, several slides were initiated along ancient landslide surfaces in the colluvium. The relationship among the landslides, a weak claystone zone in the stratigraphic dequence (the Pittsburgh Redbeds), and the valley wall topography are analyzed. One typical landslide which was studied in detail is described. Friction angles of 12.5° to 15.5° were calculated for limiting equilibrium of the failure mass and residual friction angles of 13.5° to 16° were determined from repeated direct shear tests on failure surface materials. It is concluded that the average shear strength mobilized along the failure surface material. This residual strength condition is attributed to the shearing displacement of the ancient landslide.

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FAILURE OF COLLUVIAL SLOPE

By James V. Hamel,[1] A. M. ASCE and Norman K. Flint[2]

INTRODUCTION

Colluvium is geological material which has moved downslope under the influence of gravity, i.e., landslide or creep debris. Colluvial slopes are natural slopes which have a geological history of landslides or creep, or both. The zone of colluvium along these slopes is potentially unstable because the shearing displacements associated with past movements have reduced the shear strength along the surface (or surfaces) of sliding or creep, or, both. When a cut is made in a colluvial slope, failure is frequently initiated along the existing surface (or surfaces) of sliding in the slope.

A section of Interstate Route 279 near Pittsburgh, Pa. passes through a zone of colluvium in the wall of a tributary valley of the Ohio River. When construction began on this section late in 1968, several slides were initiated along ancient landslide surfaces in the colluvium. These slides were investigated in a research project sponsored by the Pennsylvania Department of Highways and the United States Department of Transportation, Bureau of Public Roads, and they were described in a report by Hamel and Flint (4).

The location and geology of the slide site are described herein along with engineering and geological features of the slides. Shear strength parameters calculated for limiting equilibrium of a typical slide mass with the Morgenstern-Price method of slope stability analysis are presented and compared with shear strength parameters measured in laboratory tests on the

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[1] Asst. Prof. of Civ. Engrg., South Dakota School of Mines and Technology, Rapid City, S.D. and Principal, Hamel Geotechnical Consultants, Rapid City, S.D.

[2] Prof. of Geology, Univ. of Pittsburgh, Pittsburgh, Pa.

failure surface material. Conclusions are drawn concerning the level of shear strength mobilization in this typical slide.

DESCRIPTION OF SLIDE SITE

Location.—Interstate Route 279 (I-279) crosses the Ohio River 9 miles northwest of Pittsburgh on a bridge at Neville Island (see Fig. 1). This bridge carries the highway into the west wall of the valley of Kilbuck Run, a small stream which flows south into the Ohio River. The highway extends along the

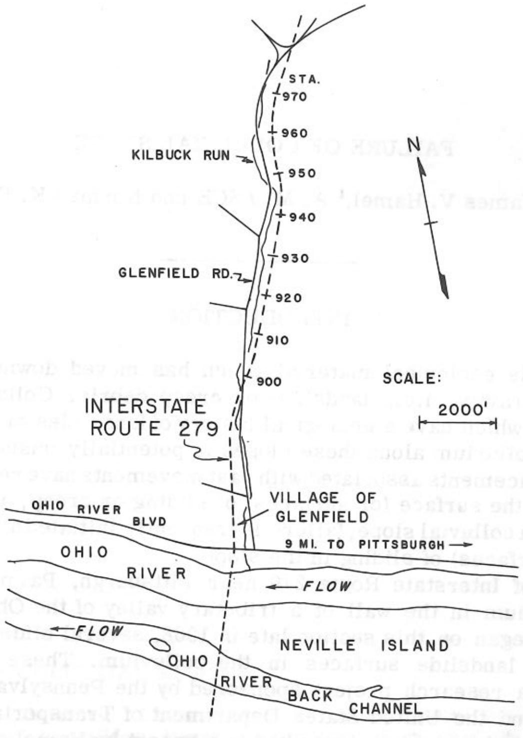

FIG. 1.—LOCATION MAP, INTEREST ROUTE 279

west wall of the valley above the village of Glenfield for approximately 0.9 miles north of the Ohio River where it crosses to the east wall of the valley for approximately 1.6 miles and then crosses back to the west wall on a third bridge. This highway alinement was chosen to avoid as much as possible the existing houses, roads, and stream on the valley floor.

Construction of this section of I-279 began in the autumn of 1968. Slides began soon after slope excavation commenced at several sidehill cut sections on the east wall of the valley between Station 899 and Station 955. Slide A, a typical slide described herein, was located between Station 906+50 and Station 909+50 (see Fig. 2).

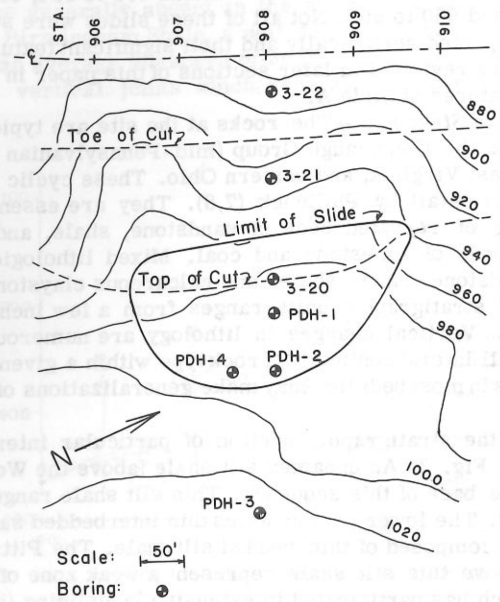

FIG. 2.—PLAN OF SLIDE A

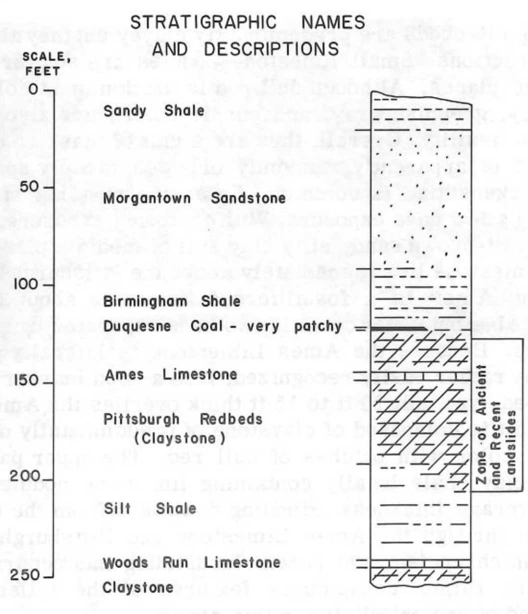

FIG. 3.—STRATIGRAPHIC SECTION

Other major slides occurred from Stations 899 to 904, 916 to 919, 920 to 922, 926 to 932, and 950 to 955. Not all of these slides were studied in detail, but they were inspected periodically and their significant features were noted. These features are reviewed in later sections of this paper in connection with the significant features of Slide A.

Stratigraphy and Structure.—The rocks at the site are typical cyclic sedimentary rocks of the Conemaugh Group (mid-Pennsylvanian age) of western Pennsylvania, West Virginia, and eastern Ohio. These cyclic sediments have been described in detail by Philbrick (7,8). They are essentially flat-lying strata consisting of repeated beds of sandstone, shale, and claystone with occasional thin beds of limestone and coal. Mixed lithologies such as silty shale, shaly sandstone, sandy shale, and calcareous claystone are common. The thickness of stratigraphic units ranges from a few inches to a few feet or tens of feet. Vertical changes in lithology are numerous. Even though there is an overall lateral continuity of rock type within a given stratum, there are local changes in most beds that may make generalizations of their lithology misleading.

The part of the stratigraphic section of particular interest at the slide site is shown in Fig. 3. An unnamed silt shale (above the Woods Run Limestone) lies at the base of this sequence. This silt shale ranges from 35 ft to 50 ft in thickness. The lower part contains thin interbedded sandstone layers; the upper part is composed of thin-bedded silt shale. The Pittsburgh Redbeds lying directly above this silt shale represent a weak zone of claystone 40 ft to 60 ft thick which has participated in extensive landsliding (both ancient and recent) in the study area. The failure surfaces of the landslides along I-279 all occurred at or slightly above the contact between these redbeds and the silt shale.

The Pittsburgh Redbeds are predominantly clayey but they also contain silt and fine sand fractions. Small limestone nodules are scattered through the redbeds in some places. Although dull red is the dominant color of the unit, pale green, gray, greenish gray, and purple colors are also common. The redbeds show no fissility. Overall, they are a unit of massive claystone containing a myriad of apparently randomly oriented, closely spaced fractures along which slickensiding is common. Extensive raveling of the claystone begins after only a few days exposure. With continued exposure, the claystone weathers into a red-brown sandy silty clay soil of medium plasticity.

The Ames Limestone lies immediately above the Pittsburgh Redbeds in the study area. The Ames is a fossiliferous limestone about 2 ft thick. It is generally a single bed but it may occur in two beds separated by a few inches of calcareous shale. Because the Ames Limestone is laterally persistent and because it can be rather easily recognized, it is a good marker bed.

Another redbed-type unit 10 ft to 15 ft thick overlies the Ames Limestone. This unnamed unit is composed of claystone of predominantly drab green and greeenish gray color with patches of dull red. The upper part grades into poorly bedded clay shale locally containing limestone nodules. The entire zone of 55 ft average thickness extending downward from the top of this unnamed claystone through the Ames Limestone and Pittsburgh Redbeds is a weak zone in which ancient and recent landsliding has occurred. Colluvial benches that are rather conspicuous features of the hillside topography indicate localities of ancient sliding in this zone.

The Duquesne Limestone and Duquesne Coal locally overlie this weak zone,

though they are generally absent in the study area. The weak zone is thus overlain by the Birmingham Shale, a unit of interbedded sandy shale and shaly sandstone with an average thickness of 30 ft. The Birmingham Shale contains well developed vertical joints which generally trend parallel to the valley walls.

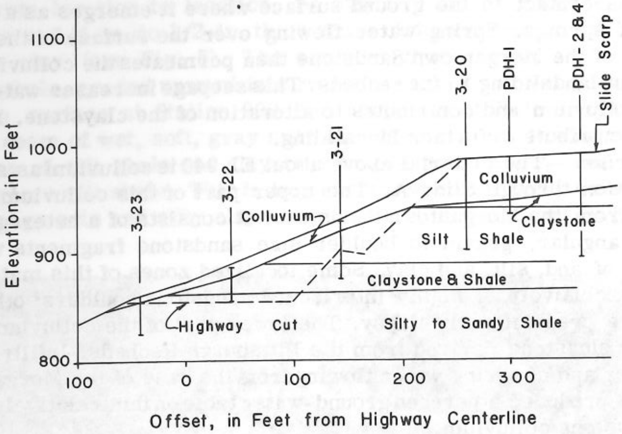

FIG. 4.—GEOLOGIC SECTION THROUGH SLIDE A

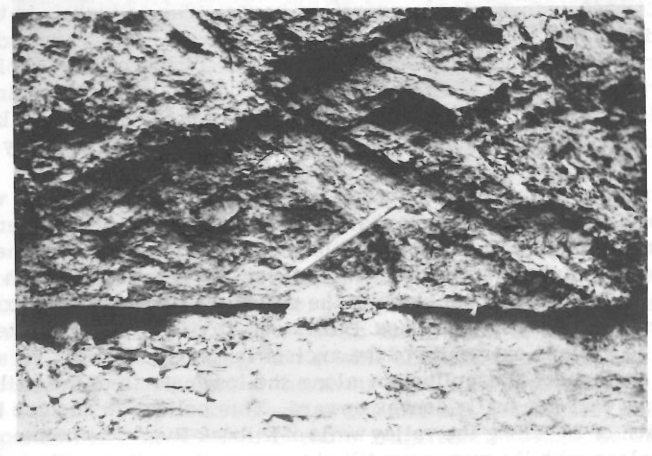

FIG. 5.—PHOTOGRAPH OF SHEAR ZONE AT STATION 928, MAY 20, 1969

The Birmingham Shale is overlain by the Morgantown Sandstone, a massive to thick-bedded unit whose base is disconformable. This irregular base truncates the Birmingham to different extents regionally so that in some places the base of the Morgantown Sandstone is in direct contact with the top of the previously mentioned weak zone.

The Morgantown Sandstone occurs at or near the level of ridge tops in the study area and, like the Birmingham Shale, contains numerous vertical joints. This is an ideal condition for the development of contact springs. Surface water infiltrates the Morgantown Sandstone and seeps down through it (and through the Birmingham Shale, where present) to the contact with the relatively impermeable claystone of the weak zone. The water then moves laterally along this contact to the ground surface where it emerges as a spring or as a line of springs. Spring water flowing over the surface of the slope below the level of the Morgantown Sandstone then permeates the colluvium produced by ancient landsliding in the redbeds. This seepage increases water pressures in the colluvium and contributes to alteration of the claystone. Both of these effects contribute to further landsliding.

Colluvium.—The material above about El. 940 is colluvium as shown in Fig. 4, a section through Slide A. The upper part of this colluvium was derived largely from the Morgantown Sandstone. It consists of a heterogeneous mixture of angular, gravel to boulder size sandstone fragments with variable amounts of and, silt, and clay. Some localized zones of this material consist almost exclusively of highly interlocked sandstone boulders; other localized zones are predominantly clayey. The lower part of the colluvium consists of clay and claystone derived from the Pittsburgh Redbeds. Infiltration of surface water and of spring water flowing from the base of the Morgantown Sandstone has produced a perched ground-water table on this relatively impervious clay-claystone colluvium.

The rocks below about El. 940 are in place. They consist of silty to sandy shales with some thin beds of claystone and sandstone. There was a relatively thin zone of colluvium and weathered rock along the valley wall below above El. 940 before slope excavation began. The colluvium in this zone is believed to have spilled over the edge of in-place shale at approximately El. 940 during the ancient landsliding. The boundaries between colluvium and weathered rock and between weathered rock and unweathered rock in the valley wall below about El. 940 were difficult to determine from the available boring information. The colluvium-rock boundary shown along the valley wall below El. 940 in Fig. 4 is therefore only approximate.

It should be noted that ground surface profiles along both walls of the valley of Kilbuck Run have the characteristic shape of landslide terrain. This is shown by the ground surface profile in Fig. 4. The surface of the colluvium near the top of the cut slope is nearly level. This colluvial bench or terrace is quite consistent on both sides of the valley. It occurs at approximately the stratigraphic level of the Ames Limestone (top of Pittsburgh Redbeds) and marks the upslope extremity of the ancient landslide masses.

The surface of the colluvium along the lower parts of the valley walls is hummocky and generally convex upward. This colluvium surface had a mean inclination of 23° along the valley walls of Kilbuck Run. The hummocky ground profile along with the numerous tilted trees on the valley walls indicates that surface creep is active in the colluvium.

DETAILS OF SLIDES

Shear Zones.—The failure surfaces of Slide A and other slides studied are located in clay-claystone colluvium at or slightly above the base of the Pittsburgh Redbeds. These failure surfaces are all believed to coincide with the failure surfaces of ancient landslides. Outcrops of these failure surfaces were studied in test pits excavated in the slope faces and in surface exposures.

Each of the failure surfaces was located in a shear zone from 1 in. to 12 in. thick. Most shear zones were located at the top of in-place silt shale and were generally overlain by claystone colluvium. The exact nature of the shear zone different from location to location. At Station 928, for example, the failure surface was a 1/4-in. to 1/2-in. thick seam of damp, medium-stiff, slickensided gray clay (see Fig. 5). The clay, which was underlain by weathered fissile silt shale, graded upward into relatively intact red and gray claystone. The failure surface at Station 909 (the north end of Slide A) consisted of a 2-in. thick seam of wet, soft, gray silty clay (see Fig. 6). It was underlain by a 3-in. zone of silt shale and claystone fragments in a silty clay matrix and then by in-place silt shale. This failure surface was overlain by above 6 in. of claystone fragments and silty clay and above that by fractured claystone.

FIG. 6.—PHOTOGRAPH OF SHEAR ZONE AT STATION 909, MAY 13, 1969

Most of the shear zones studied along this section of I-279 were similar to the one at Station 909. They had three definite parts. The actual surface of sliding was generally a 1/4-in. to 2-in. thick seam of damp to wet, soft to medium-stiff, gray silty clay with variable amounts of sand. This seam was usually located near midheight of the shear zone. The parts of the shear zone above and below the clay seam consisted of a mixture of silty clay and angular, sand to gravel size claystone and shale fragments. Though the thicknesses of these upper and lower parts varied considerably from place to place, they were typically 2 in. to 3 in. The platy shaped claystone and shale fragments in the upper and lower parts of the shear zones were commonly alined parallel to the direction of movement. This is considered a macroscopic manifestation of the parallel particle arrangement reported by Skempton (9) for residual strength behavior. The shear zones were usually damp to wet and frequently showed appreciable seepage.

The preliminary X-ray diffraction study of the minerology of shear zone materials indicates that they contain quartz, kaolinite, illite, and expandable-lattice clay minerals. These expandable-lattice minerals (possibly vermiculite and one or more minerals of the smectite group) occur preferentially along the failure surfaces of the slides. Ground-water flow through the relatively permeable shear zones may have caused geochemical changes that resulted in the formation or concentration of these expandable clay minerals, or both, along the failure surfaces. This possibility is being studied further.

Index properties were determined for samples of clayey failure surface materials obtained from 11 locations between Station 899 and Station 953. Ranges and average values of these index properties are given in Table 1.

TABLE 1.—INDEX PROPERTIES OF CLAYEY FAILURE SURFACE MATERIALS

Property (1)	Range of values (2)	Average value (3)
Natural water content, as a percentage	17-31	24
Liquid limit, as a percentage	27-41	35
Plastic limit, as a percentage	19-29	24
Plasticity index, as a percentage	8-13	11
Clay fraction (minus 2 μ), as a percentage	14-29	21
Specific gravity	2.74-2.80	2.77

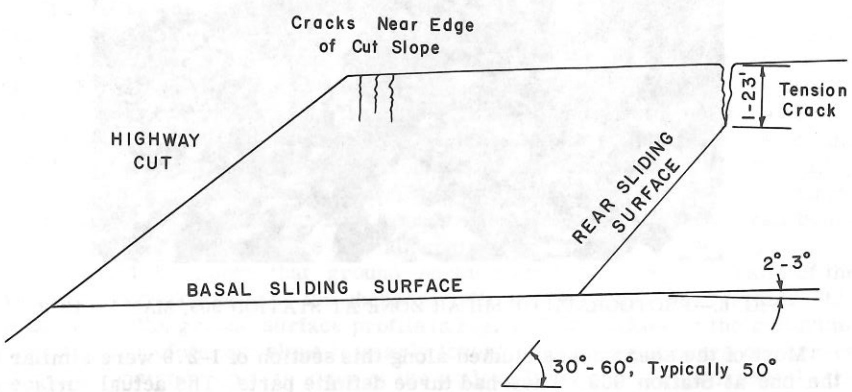

FIG. 7.—SCHEMATIC CROSS SECTION OF TYPICAL FAILURE MASS

The failure surface materials are well-graded mixtures of sand, silt, and clay size particles. There was considerable variation in particle size distribution from sample to sample. About half the particles in each sample were of fine sand and silt size; 40 % to 80 % of each sample passed the No. 200 sieve. It is difficult to classify these materials according to either the Unified Soil Classification System or the Revised Bureau of Public Roads Soil Classification System. As about half their particles are in the fine sand to silt size range, these materials generally fall at or near the border between the SM-SC and ML-CL soil classifications of the Unified System. They are A-4 or A-6 soils in the Revised Bureau of Public Roads System.

Geometric Details.—All slides observed along this section of I-279 were of the sliding wedge type. Each failure surface consisted of three parts as shown in Fig. 7. These parts are a basal surface of sliding, a rear surface of sliding and a tension crack at the ground surface. The basal surfaces of sliding typically dipped 2° or 3° and, as mentioned previously, probably coincided with ancient landslide surfaces near the top of in-place rock.

The rear surfaces of sliding dipped 30° to 60° along their upper portions where they crossed clay-claystone colluvium. Most of these rear sliding surface dips were on the order of 45° to 55°; a 50° dip is considered typical. There was commonly a 1/4-in. to 1/2-in. thick layer of soft to stiff, slickensided clay along these rear surfaces of sliding. It is not certain whether the rear sliding surfaces coincided with segments of ancient landslide surfaces, though it is suspected that some did.

The details of failure surface geometry at the intersections of the rear sliding surfaces with the basal sliding surfaces are not well known. It is considered likely that the rear sliding surfaces flattened somewhat or became curved above these intersections but this was not verified.

Tension cracks occurred at the rear of each failure mass observed along I-279. These cracks began to open in the early stages of sliding. The depth of tension cracks depended primarily on the nature of the colluvium near the ground surface. Deeper tension cracks generally formed in sand-sandstone colluvium than in clay-claystone colluvium. Tension crack depths of 1 ft to 10 ft were measured at the rears of slides in clayey colluvium; tension crack depths of 6 ft to 23 ft were measured at the rear of slides in sandstone colluvium.

Where the failure mass moved out into the highway cut, the toe of the mass was sometimes cantilevered as much as 1 ft over the edge of in-place rock. Where the toe material was relatively clayey and plastic, it sometimes bent under its own weight and flowed several inches down over the face of the cut slope before breaking off and falling to the bottom of the cut slope. Definite streamlines were observed in this flowing clay; rock particles in it were generally alined parallel to these streamlines. Nonplastic toe material simply tumbled over the edge of in-place rock and rolled or slid down the slope. This sliding over the edge of the cut induced flexural-type tensile stresses in the upper parts of the failure masses. Additional tension cracks then formed near the edge of the original cut slope as shown in Fig. 7.

ANALYSIS OF SLIDE A

History of Slide.—Excavation began in the vicinity of Slide A on November 26, 1968. The colluvium was excavated at an inclination of 1.25:1 or 39°. Tension cracks were reported above the cut slope between Stations 906 + 50 and 908 + 50 on December 4. This slide was first inspected by the senior writer on December 17 when excavation was down to about El. 920. On December 17 when excavation was down to about El. 920. On December 17, the slide extended from Station 906 + 50 to Station 908 + 50. There was a 3 ft to 5 ft vertical scarp about 100 ft back from the edge of the cut slope between Station 907 and Station 908 and horizontal movement of 3 ft to 4 ft had occurred. A relatively planar sliding surface was exposed at the base of the scarp at Station 908. This surface consisted of red clayey colluvium slickensided from the slide movement. The surface dipped 35° to 45° west (in the direction of the slide movement). There were also many open fissures in the top of the slide mass parallel to the edge of the cut slope. Fig. 8 is a photograph of the cut slope at Slide A.

Surface creep indications and new cracks were observed at the rear of the slide mass after a relatively warm period at the end of January, 1969.

FIG. 8.—PHOTOGRAPH OF CUT SLOPE AT SLIDE A, MARCH 22, 1969

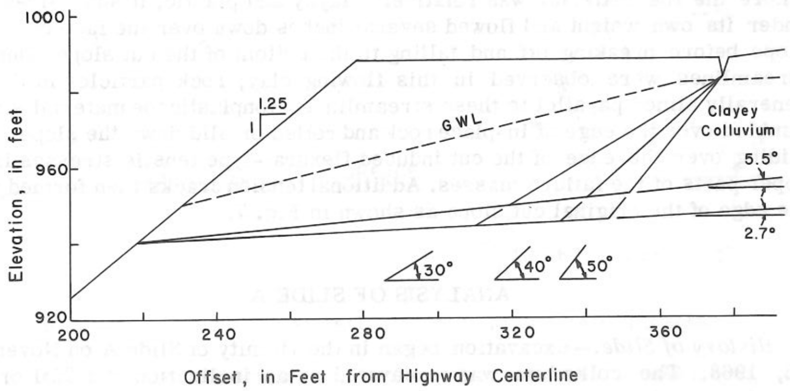

FIG. 9.—IDEALIZED CROSS SECTION THROUGH SLIDE A

The slide outline shown in Fig. 2 is that of February, 1969. Slide material began falling to the bench at El. 900 in appreciable quantities at that time. Movement of Slide A continued at least through November, 1969.

Idealized Slope Cross Section.—Before calculating sets of Mohr-Coulomb shear strength parameters for limiting equilibrium of the failure mass in Slide A, it was necessary to define an idealized cross section through the slide. This idealized slope cross section is shown in Fig. 9.

Two basal surfaces of sliding were assumed for the stability analyses. Both intersect the slope face at El. 940, the observed elevation of the failure surface outcrop. The lower basal surface of sliding has an inclination of 2.7°. It follows what is believed to be an ancient landslide surface at the base of the clay-claystone colluvium. The upper basal sliding surface has an inclination of 5.5°. It is located near the top of the clay-claystone colluvium at what may also be an ancient landslide surface.

A 5-ft tension crack was assumed to exist at the rear of the slide mass. Rear sliding surfaces of three different inclinations (30°, 40°, and 50°) were drawn from the bottom of this tension crack to the basal surfaces of sliding.

The colluvium in the failure masses analyzed for Slide A was assumed to have an average total unit weight of 145 pcf. Sandstone boulders and the relatively intact claystone in the colluvium typically have total unit weights of 155 pcf to 160 pcf; the more soil-like parts of the colluvium typically have total unit weights of 120 pcf to 130 pcf. The average unit weight of 145 pcf was therefore considered reasonable for the total failure mass.

The colluvium along each failure surface was also assumed to have the same average shear strength parameters. Inspection of samples from borings PDH—2 and PDH—4 and of exposures in open cracks indicated that the colluvium at the rear of Slide A was predominantly clayey. This clayey colluvium at the rear of the failure mass was, of course, derived from the same Pittsburgh Redbeds claystone material as the clayey colluvium along the base of the failure mass.

The ground-water level (GWL) shown in Fig. 9 is that corresponding to steady state seepage. It is based on springs observed at El. 950 in the slope face and the average water elevation of 982 observed in boring PDH-2 over the period from February to June, 1969. Boring PDH-2 pinched off near the failure surface of Slide A so water levels observed in it correspond to the perched water table above the failure surface. This ground-water level shown in Fig. 9 is considered a good estimate of ground-water conditions in the failure mass at the time of failure.

Calculation of Shear Strength Parameters.—The Morgenstern-Price (5,6) method of slope stability analysis was used to calculate effective stress Mohr-Coulomb strength parameters ϕ' and c' required for limiting equilibrium along each of the failure surfaces described in the previous section and shown in Fig. 9. The Morgenstern-Price method is a two-dimensional limiting equilibrium method of stability analysis which treats a failure surface of general shape and requires the failure mass to be in complete static equilibrium. Calculations were performed with the IBM 7090 computer at the University of Pittsburgh Computer Center using a Fortran II version (2) of the MGSTRN program developed by Bailey (1).

The values of ϕ' required for limiting equilibrium with $c' = 0$ and the values of c' required with $\phi' = 0$ were calculated along with the values of c' required with $\phi' = 10°$. These calculated values are given in Table 2. Values of tan ϕ' versus c' for the 2.7° basal sliding surface are plotted in Fig. 10. The tan ϕ' versus c' values for the 5.5° basal sliding surface are not plotted in that figure as they fall within the range of tan ϕ' versus c' plotted for the 2.7° basal sliding surface.

The effective shear strength of the failure surface material is believed to have been predominantly frictional in nature. The ϕ' values of 12.5° to 15.5°

calculated for limiting equilibrium with $c' = 0$ are therefore considered to represent the in situ strength of the failure surface material. It is impossible to calculate the exact value of friction angle mobilized in situ because of minor uncertainties concerning the geometry of the failure mass and the water forces acting on it. The writers believe, however, that the ϕ' value of 14° calculated for limiting equilibrium with the 2.7° basal sliding surface and the 40° rear sliding surface is a reasonable estimate of the friction angle mobilized along the failure surface of Slide A.

Comparison of Calculated and Measured Shear Strength Parameters.—A series of consolidated drained direct shear tests was performed on samples of material from the failure surfaces of ancient and recent landslides at 11 locations on the site. The index properties of these samples are summarized

TABLE 2.—STRENGTH PARAMETERS CALCULATED FOR LIMITING EQUILIBRIUM

Inclination of basal sliding surface, in degrees (1)	Inclination of rear sliding surface, in degrees (2)	Cohesion, c', in pounds per square foot (3)	Friction Angle, ϕ', in degrees (4)	Tangent ϕ' (5)
2.7	30	0	15.5	0.277
		320	10.0	0.176
		850	0.0	0.000
	40	0	14.0	0.249
		230	10.0	0.176
		770	0.0	0.000
	50	0	12.5	0.222
		150	10.0	0.176
		720	0.0	0.000
5.5	30	0	15.5	0.277
		300	10.0	0.176
		800	0.0	0.000
	40	0	14.5	0.259
		250	10.0	0.176
		750	0.0	0.000
	50	0	13.5	0.240
		200	10.0	0.176
		720	0.0	0.000

in Table 1. The direct shear test specimens were 2 in. square by 1/2 in. to 1 in. thick; they were sheared at a rate of 0.0045 in. per min. Each specimen was sheared repeatedly on the same failure surface using the shear box reversal technique suggested by Skempton (9) in order to obtain residual shear strengths. The sampling and testing of these materials were described in detail by Hamel (3) and Hamel and Flint (4).

The residual strengths of the failure surface materials were found to depend on the amount of sand size claystone and shale fragments present. Specimens containing appreciable quantities of these fragments had residual cohesion intercepts of 50 psf to 100 psf and residual friction angles of 20° to 25°. Specimens containing few sand size fragments had residual cohesion intercepts of essentially zero and residual friction angles of 8° to 18°. The residual friction angles of most of these latter specimens were in the range of

11° to 16°. Residual strength parameters $c' = 0, \phi' = 13.5°$ to 16° were determined from tests on three samples from the failure surface of Slide A.

The friction angles of 12.5° to 15.5° calculated for Slide A are in excellent agreement with the measured residual friction angles of the failure surface material (see Fig. 10). The fact that a residual-level strength was mobilized in the slide is consistent with the geologic history of the site. Movements in the ancient landslides were of sufficient magnitudes to reduce the strengths of the failure surface materials to residual levels. It is believed that the failure surfaces did not heal following the ancient landslides and that residual or near

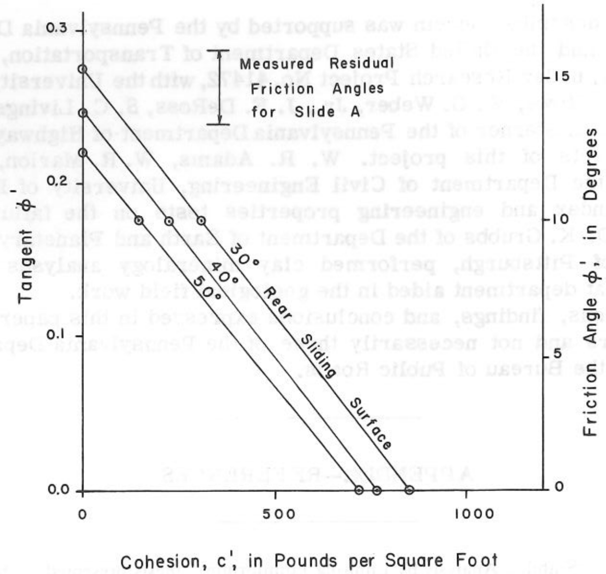

FIG. 10.—CALCULATED TAN ϕ' VERSUS c' FOR SLIDE A

residual strengths existed along the failure surfaces when slope excavation began.

CONCLUSIONS

Landslides encountered during construction to I-279 were caused by excavation of the toes of ancient landslide masses along the valley wall. This removal of toe support reactivated the ancient landslides. Both ancient and recent landslides occurred in a weak zone of the stratigraphic section composed primarily of the Pittsburgh Redbeds claystone. The ancient landslide masses were recognizable in the preconstruction topography of the valley wall.

Failure surfaces of the ancient and recent landslides were located at or near the base of the Pittsburgh Redbeds claystone. Movements in the ancient landslides reduced shear strengths along these failure surfaces to residual levels. The strength of the failure surface materials probably remained at or very near residual levels until the time of slope excavation.

The average residual shear strength parameters of the clayey failure surface material of a typical slide at the I-279 site are on the order of $c' = 0, \phi' = 13°$ to $16°$. These strength parameters were calculated for limiting equilibrium of the failure mass. They were also obtained from laboratory direct shear tests on samples of the failure surface material. It is likely that residual shear strength parameters of these same magnitudes were also mobilized along the failure surfaces of the other slides at the I-279 site.

ACKNOWLEDGMENTS

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The opinions, findings, and conclusions expressed in this paper are those of the writers and not necessarily those of the Pennsylvania Department of Highways of the Bureau of Public Roads.

APPENDIX.—REFERENCES

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FAILURE OF COLLUVIAL SLOPE[a]

Errata

The following corrections should be made in the original paper:

Information Retrieval Abstract, line 6: change "... stratigraphic dequence ..." to read "... stratigraphic sequence ..."

Information Retrieval Abstract, line 11: change "... the average shear strength mobilized along the failure surface material." to read "... the average shear strength mobilized along the failure surface of this typical slide was the residual strength of the failure surface material."

Page 168, Fig. 1 caption: change "Fig. 1.--Location Map, Interest Route 279" to read "Fig. 1.--Location Map, Interstate Route 279"

Page 172, paragraph 2, line 5: change "... and, silt, and clay." to read "... sand, silt, and clay."

Page 172, paragraph 3, line 4: change "... above El. 940 ..." to read "... about El. 940 ..."

Page 173, paragraph 1, line 4: change "... different from location to location." to read "... differed from location to location."

Page 173, paragraph 1, line 11: change "... above 6 in. ..." to read "... about 6 in. ..."

Page 173, last line: change "The preliminary X-ray diffraction study ..." to read "A preliminary X-ray diffraction study ..."

Page 174, line 4 below Fig. 7: change "... 40% to 80% of each sample ..." to read "... 50% to 80% of each sample ..."

Page 175, paragraph 2, line 6: change "... surfaces conincided with ..." to read "... surfaces coincided with ..."

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[a] February, 1972, by James V. Hamel and Norman K. Flint (Proc. Paper 8731).

Page 175, paragraph 4, line 7: change "... rear of slides ..." to read "... rears of slides ..."

Page 175, paragraph 6, lines 5 and 6: Delete second "On December 17 when excavation was down to about El. 920 ..."

Page 179, paragraph 2, line 1: change "... construction to I-279 ..." to read "... construction of I-279 ..."

Page 180, last paragraph, last line: change "... Pennsylvania Department of Highways of the Bureau of Public Roads." to read "... Pennsylvania Department of Highways or the Bureau of Public Roads."