Review of Research for Disaster Prevention, No. 132, December 1988

551. 432 : 551.311.235 :
624. 131.54 : 614.8(237.2)

Landslides in the Appalachian Region

Field Trip Records and Proceedings of
the 3rd International Conference and Field Workshop on Landslides,
Eastern United States

Edited by

Masaki Tominaga
Norio Oyagi

National Research Center for Disaster Prevention,
Tsukuba, Ibaraki, 305, Japan

Preface

The 3rd International Conference and Field Workshop on Landslides (ICFL) was convened and conducted by Mr. Donald R. Nichols, U. S. Geological Survey, May 1-13, 1983. The field trip was held in the Appalachian region of the eastern United States from May 2-9, and the conference was held in the National Center of the USGS in Reston, Virginia, on May 10 and 11.

The Japan National Research Center for Disaster Prevention, in order to make available the results of the field trip and conference to their researchers, proposed that the Center publish the proceedings as a Research Note of the NRCDP. Mr. Nichols invited all participants in the conference to prepare their papers for publication and to forward their manuscripts to the editorial committee of the NRCDP. The editors compiled a record of the field trip, based on field trip guides, handouts, referenced published papers, and photographs taken during the trip. Much of the delay in publication of these proceedings was caused by the need to compile the many published papers discussing the sites visited and to select those portions of the papers that accurately summarize the views of the authors on the geologic and engineering features of the sites and the remedial measures undertaken. For the convenience of the readers, the original papers are referenced in each paragraph.

Three formal papers were submitted to the editors for publication in these proceedings. One, by Mr. John S. Pomeroy, is of a presentation he gave to participants on May 3 during the field trip. Two, by Professor David M. Cruden and by Dr. Hans Kienholz, record their presentations during the conference on May 11 ; these papers follow the record of the field trip in the first part of the proceedings. We regret the delay in publication of these papers.

We wish to express our thanks to Mr. Donald R. Nichols for providing us with the opportunity of publishing the proceedings, and to Messieurs Richard E. Gray, James V. Hamel, Peter Lessing, Robert W. Fleming, Arvid M. Johnson, James E. Hough, David L. Royster, Harry L. Moore, and their colleagues too numerous to list, who provided well-prepared explanations of the landslides and the remedial measures undertaken at each site visited during the field trip.

February 20, 1988
Editors

The Field Trip

May 2—9, 1983

in the Appalachian Region, Eastern United States

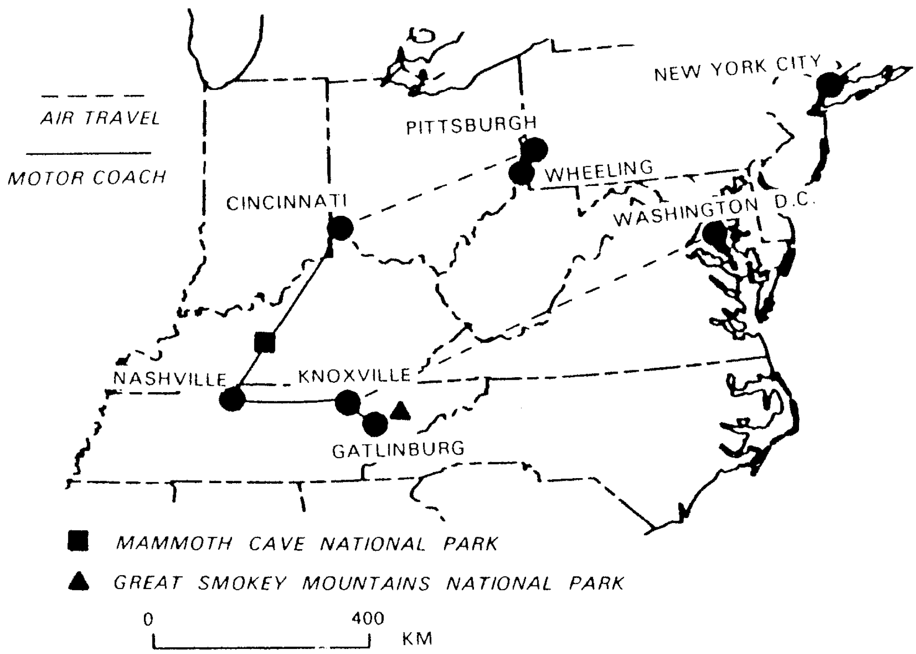


Fig. 1 Route Map of the field trip. (from the field trip guide)

May 2, 1983

Pennsylvania and West Virginia Portion

Introduction (from the field trip guide)

This portion is located in the Appalachian Plateau - one of the most active areas for landsliding in the United States. The Appalachian Plateau region of Pennsylvania and West Virginia is a maturely dissected plateau with deep valleys, moderate to steep slopes, and local relief on the order of 120-150m. Rocks of the region are generally flatlying, interbedded Paleozoic age claystone, shale, and sandstone with a few limestone and coal seams. Most of the region was not glaciated during the Pleistocene Epoch ; during that time the major rivers entrenched their valleys, then filled them with glacial outwash or lake silt and clay deposits. Valley wall stress relief and joint development accompanied downcutting of the rivers and soil and rock masses presumably slumped from valley walls under periglacial conditions. Weathering of rocks in valley walls, plus downward creep and sliding of the weathering products, particularly those derived from claystone and shale, have continued to the present. As a result of these processes, colluvial masses of various thicknesses and lateral extents exist in states of marginal equilibrium on many, if not most, slopes in the region.

Deep-seated rockslides are relatively rare. Where such slides occur, they typically involve excavated slopes in which large wedges of rock, separated from the valley walls by near-vertical stress relief joints, slide along weak claystone or shale beds. Water pressures in the slopes are usually significant contributing factors in such slides.

Rockfalls are common where rock strata are exposed in slopes. Differential weathering and erosion remove low-strength claystone and shale leaving unsupported ledges of stronger shale, sandstone, and limestone which ultimately fall. Typical rockfall volumes are small; e.g., on the order of 100 m^3 or less.

Most of the present slope stability problems in the region involve slump-type slides or slow earthflows of colluvial soil. The precarious equilibrium of these masses is frequently upset by man's activities; e.g., removal of toe support, loading the slope, or changing surface and subsurface drainage. Abnormally high precipitation also initiates movement of colluvial masses. Pomeroy (Ref. 1) discusses landslides of the region in detail.

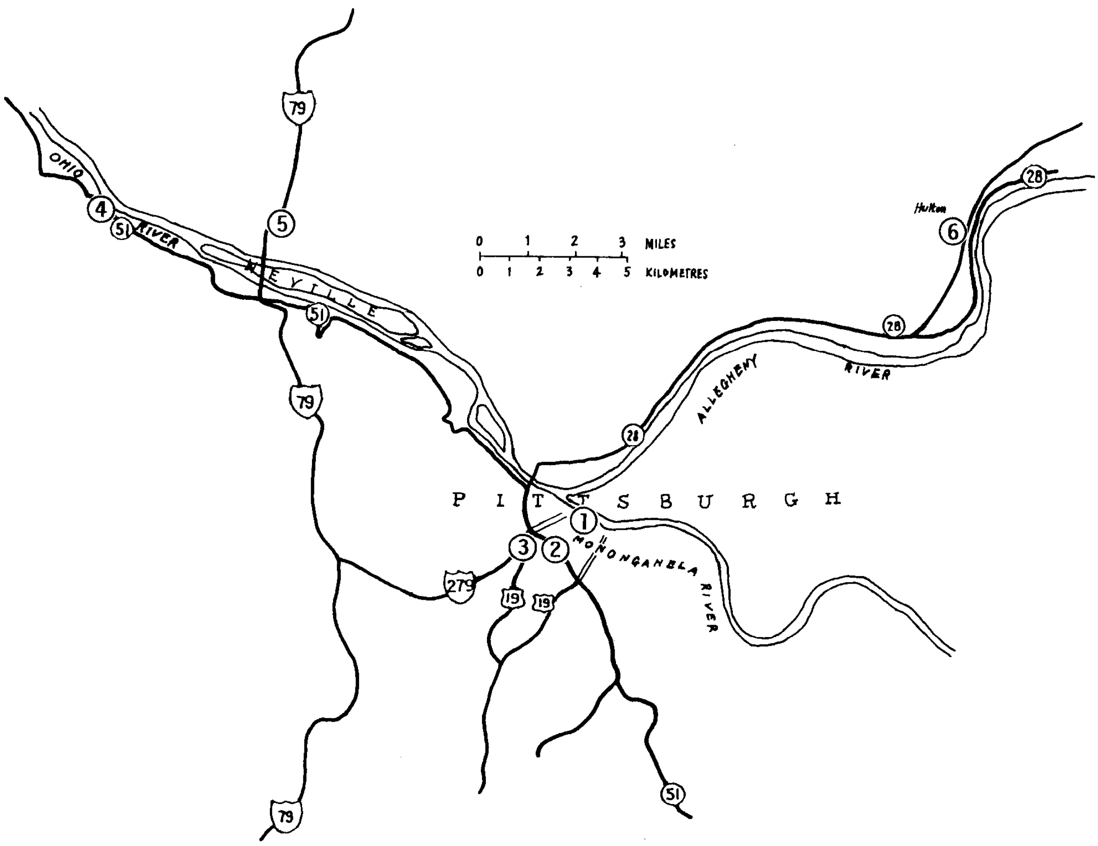


Fig. 2 Route Map, Pittsburgh area. (from the field trip guide)

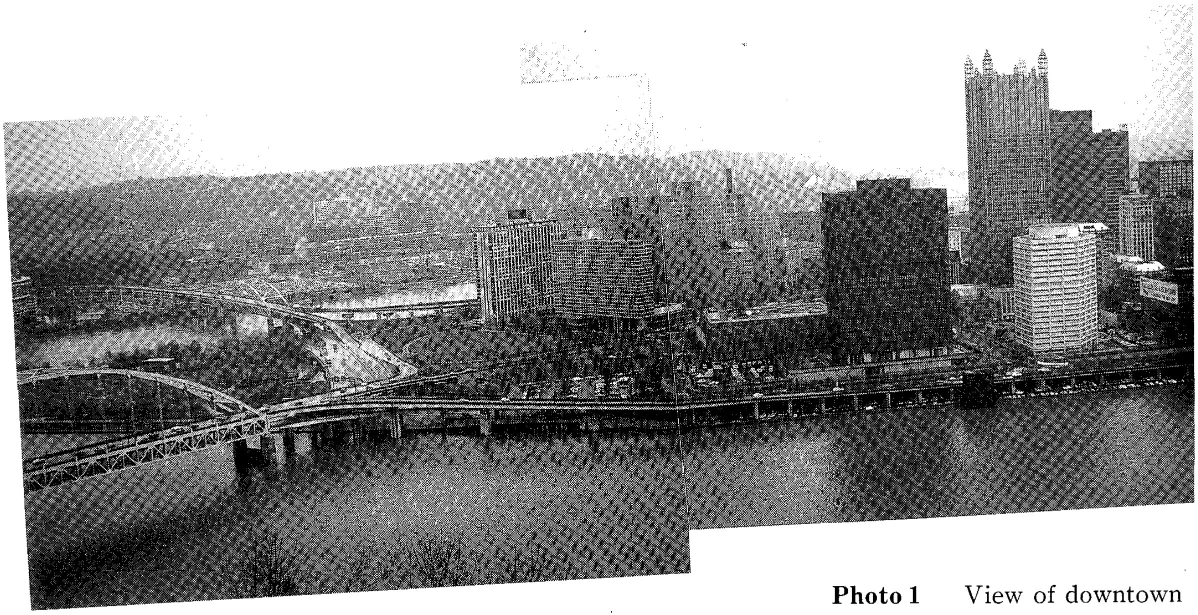


Photo 1 View of downtown

stop 1

View of Pittsburgh from Mt. Washington.

Major George Washington, the first President of the United States, selected the forks of the Ohio as the site of a fort in 1753. Due to available river transportation and coal resources, Pittsburgh grew as an industrial center and is now the third largest corporate center in the United States. (from the field trip guide.)

stop 2

Saw Mill Run Boulevard Rockfall (February 16, 1983).

Differential weathering resulted in undercutting of the massive Morgantown sandstone. During excavation of the slope in February 1983, a rockfall killed two people. See Reference 3 and handout by Miller and Mullarkey. (from the field trip guide.)

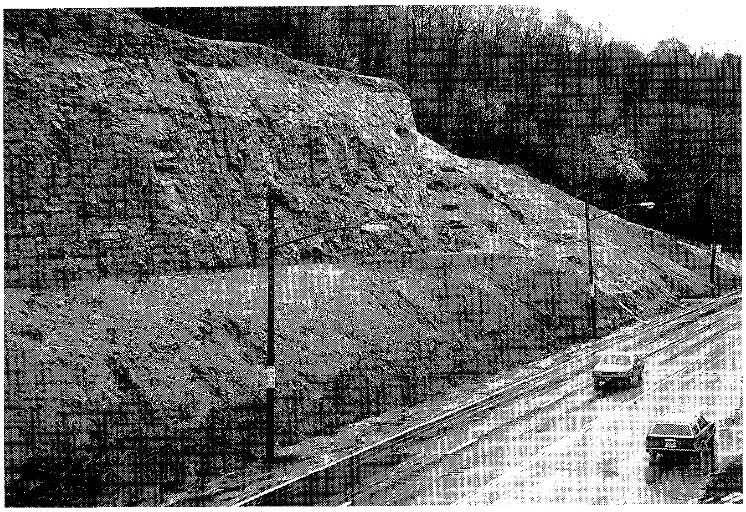


Photo 2 After removal of hanging rock.

Pittsburgh from Mt. Washington.

GENERAL CHRONOLOGY OF EVENTS (Miller & Mullarkey, 1983)

The slope that was envolved in this landslide has progressed through a series of stages that can be identified as follows:

- A natural slope composed of various sedimentary layers.
- This slope was steepened during the road cut operations necessary for construction of route 51 over half a century ago.
- A weathering sequence began on the more openly exposed claystone. The progression is well described by Gray in a publication of The Pennsylvania Geological Survey as follows:

  The interbedded nature of rocks in the area also results in small but dangerous rock falls. Cuts containing either hard sandstone or limestone seams and underlain by relatively weak shales or claystones are common throughout western Pennsylvania. (see attached cross section of the route 51 slope, Fig. 3). Weathering produces relatively rapid decompositions and spalling of the softer rocks so that eventually unsupported ledges of sandstone or limestone result. In time, weathering progresses to the point where the rock can no longer sustain the stress developed by its over hanging weight and the ledge falls.

In this specific example, a layer of massive sandstone approximately 25 feet thick was directly underlain by interbedded red claystone and limely claystone which are locally referred to as "Pittsburgh Redbeds". An overhang due to weathering of the redbeds caused the undercut sandstone to move into a meta-stable condition. During a construction operation to widen the road which involved blasting and clearing of the subsequent debris, a large section of the face toppled out across the roadway.

- As a final note, remedial operations have been carried out to bring the slope into its present configuration (note the "final proposed grade" on the attached cross section, Fig. 3). Although the slope was cut back quite a bit it appears that more work may be required.

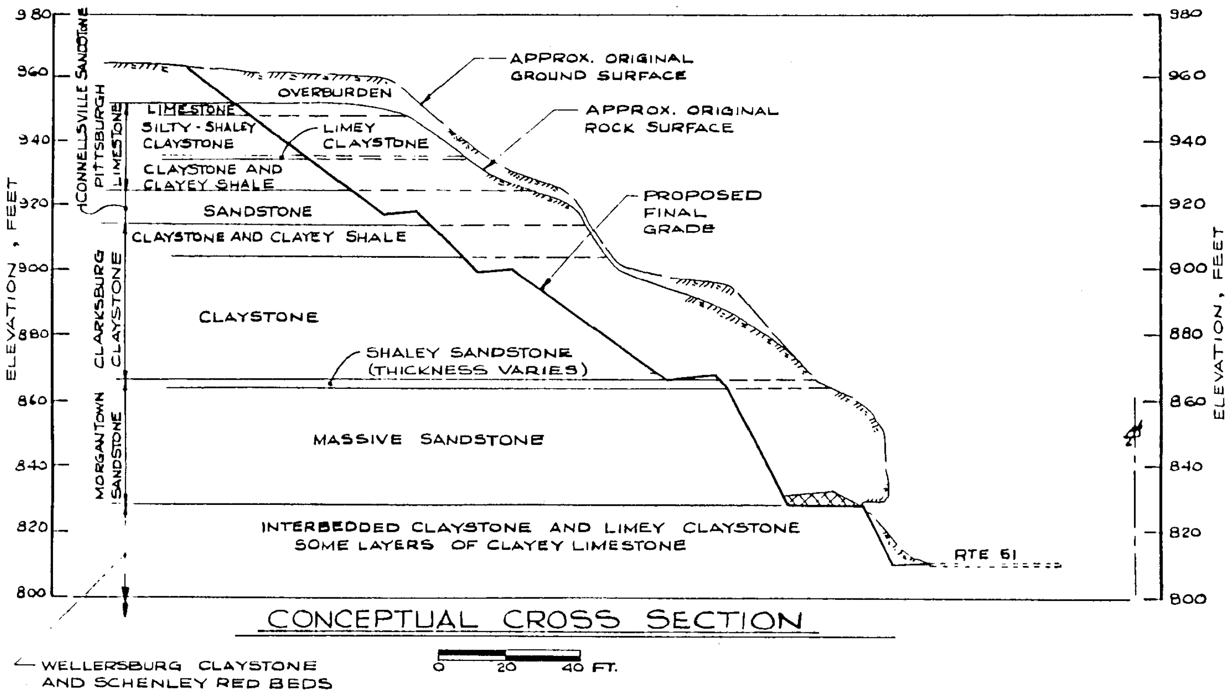


Fig. 3 Conceptual cross section. (Miller & Mullarkey, 1983)

stop 3

Parkway West Gabion Wall and Tieback Wall for Truck Escape Ramp.

A gabion wall was used to eliminate a problem of differential weathering in a critical section of roadway. A tieback wall was selected to permit construction of a truck escape ramp in a conjested traffic area. See Reference 4. (from the field trip guide.)

Photo 3 Tieback wall for truck escape ramp.

Green Tree Hill Truck Escape Ramp (Carrier, 1982)

- **Geotechnical Cause : Landslide, made space.**
- **Green Tree Hill :**

The hill on I-279 just south of the Fort Pitt Tunnel is a mile and a half long and has a 5% grade. Banksville Road merges from the right at the base of the hill just prior to the Fort Pitt Tunnels. Four lanes merge into a two-lane tunnel. Two exit ramps exist in the short space between Banksville merge point and the Tunnel. In short, the length and grade problems were compounded by the difficult merge that frequently stopped the traffic conditions at the bottom of the hill. The average daily traffic at the Tunnel is 77, 000 vehicles per day, half in each direction. The turck percentage is 6.8% thus accounting for 2,600 trucks per day descending the hill. Since the truck runaways were numerous, it was recommended that an escape ramp be built. Only place to build was in a landslide prone area on the side of a hill.

- **Job :**

Build a ramp to allow runaway trucks to exit out of the main flow of traffic along the paved shoulder and then into a gravel pile via a 229' long paved approach.

- **FHWA Design Criteria :**

Because of the proximity of Banksville Road merge ramp, insufficient space was available to make the ramp as long as the FHWA design criteria called for. A crash barrier was placed across the end of the gravel pile. It consists of 10-14" steel H-beams driven vertically at 3' centers with a steel beam batter and bracing system. A series of 31 sand filled plastic barrels are placed in front of the H-beams. This system of "positive restraint" is designed to keep the runaway vehicle from going through all the gravel and landing on top of Banksville Road standing traffic queues. The completed sandpile has a typical warning sign leading to the pile. The chevrons are placed on the pile for their target value. Concrete deadman anchors with hooks for wreckers were placed in the approach ramp to facilitate removal of the runaway trucks.

- **Failure :**

The failure of the soil slope below the timber lagging was primarily due to the location of the slag below the underdrain and the saturation of the steep (near 1 : 1) slope.

- **Extra Work:**

October 26, 1981 to November 20, 1981. Four additional rock anchors were placed along with timber lagging, geotextile filter, drainage systems, the excavation of soil in the slide area and the placing of a bituminous curb on the approach area.

- **Maintenance :**

Three maintenance needs have been experienced to date. Reshaping of the pile after each use. Periodic salting to keep the pile loose during the winter and periodic litter pickup after blowing litter has accumulated on the pile. The open graded clean gravel was chosen to facilitate natural drainage. Calcium chloride is hand broadcast over the surface periodically during the winter by tunnel employees stationed nearby. Experience through one winter, 1980-81, has shown that the snow does not melt and that only a 3-4" crust of frozen stones occurs.

• **Conclusions :**

Excellent location for the ramp. Eight trucks have used the ramp since 1980. During construction the existing shoulders dropped about two inches. The roadway pavement did not settle, however, there were voids under the pavement slab. The voids were grouted and the bituminous concrete pavement surface was milled. The settlement of the shoulder and voids under the pavement can be attributed to the vibration from the pile driving operation.

Stop 4

Route 51-Anchored Retaining Wall.

The roadway supported by a wall was completed in 1971. Cracks were noted on the roadway shoulder in 1975. A system of soldier beams, sheetpiling and walers with anchors was designed and installed to restrain wall movement. See Reference 5.

(from the field trip guide.)

PERFORMANCE MONITORING OF A TIEBACK WALL (Dash & DeRoss, 1979)

A continuous sliding movement of a reinforced concrete retaining wall, on Pennsylvania Route 51 north of Pittsburgh, was stopped by the installation of 130 rock anchors. Thirteen load cells and nineteen slope inclinometers were installed to continuously record the anchor loads and the wall and soil movements. In addition, survey readings were taken to measure vertical and horizontal movements of the wall as well as the movements in the highway pavements above the wall. The results from the field testing of rock

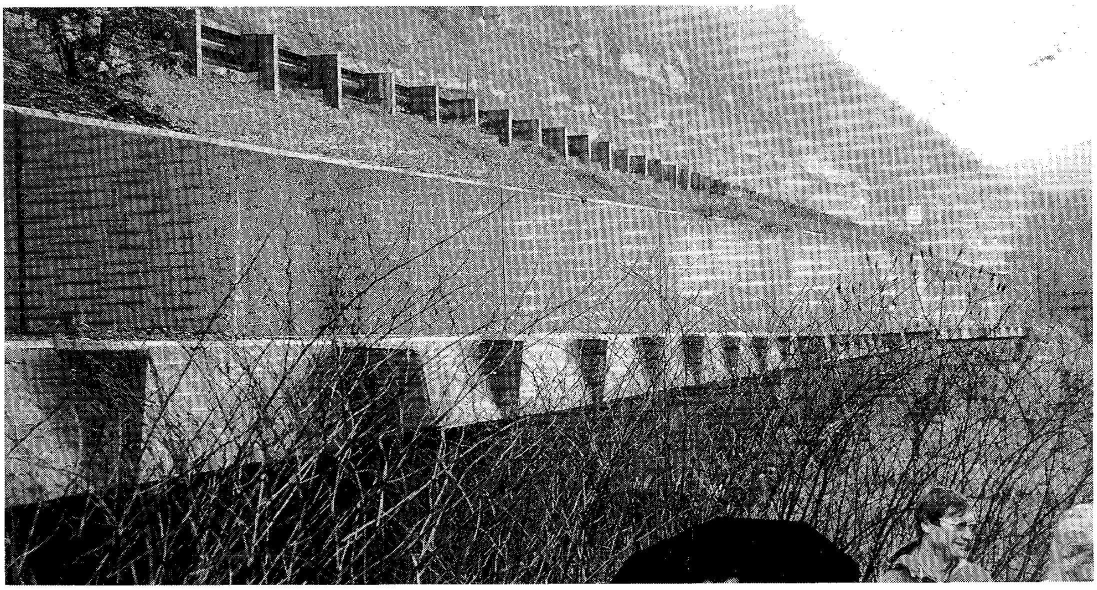


Photo 4 Retaining wall.

anchors, evaluation of the performance of Telemac Load Cells, the Remote Systems Load Cells, field data on load deformation, and time-versus-anchor load behavior, are presented. The following conclusions were made:

A significant amount (one-half inch to an inch) of permanent set was necessary for the seating of the rock anchors. The inclination (30°) of ROW-B and ROW-C anchors appeared to be excessive, causing downward movements on parts of the wall. An inclination of 15 degrees would have been more desirable. As a nominal load (10 percent of the design loads) on the ROW-C anchors had stabilized, it appeared that ROW-C anchors were not carrying large earth pressure loads and that these anchors probably provided an additional margin of safety. Telemac and Remote System Load Cells appeared to function well once the calibration was stabilized, but these were well below the desired one-half precent accuracy.

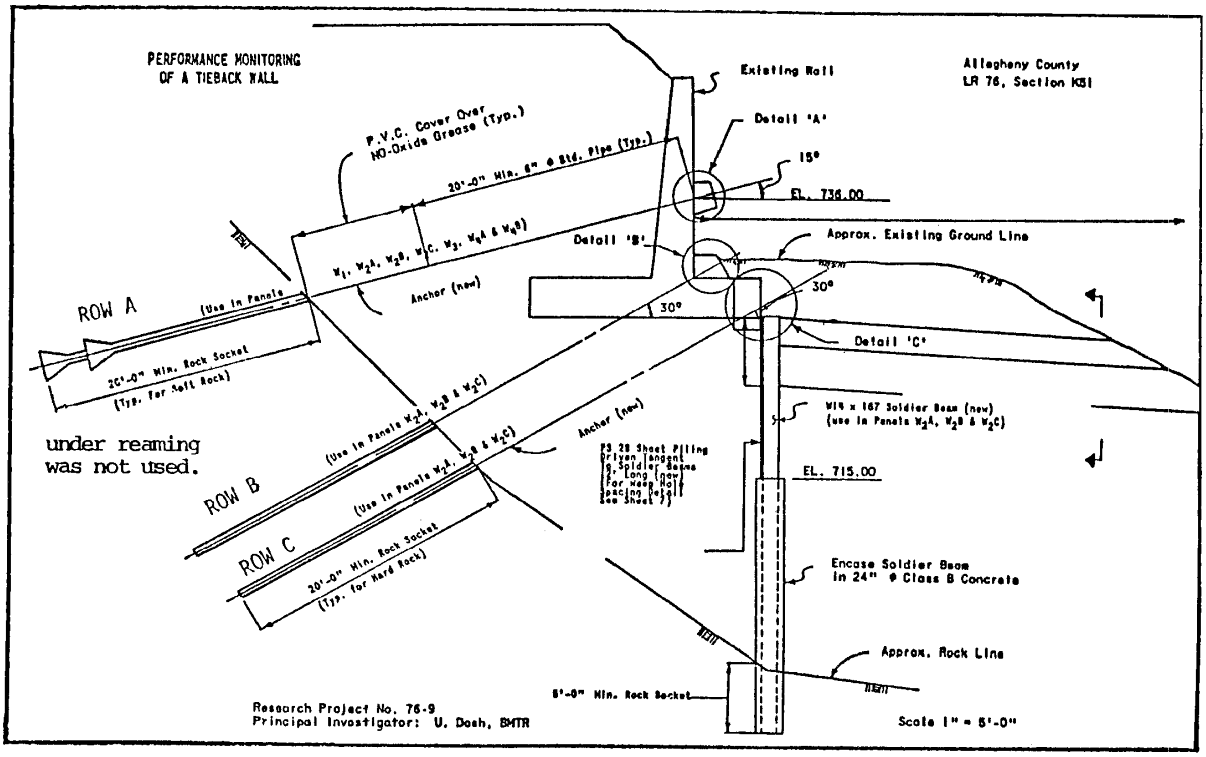


Fig. 4 Typical cross section of the tieback wall showing rock anchors. (Dash & Deross, 1979)

Stop 5

I-79 Slide.

Extensive sliding occurred during construction due to undercutting of old slide masses at the stratigraphic level of the Pittsburgh Redbeds (a claystone). See References 6, 7, and 8 (Case History 4). (from the field trip guide.)

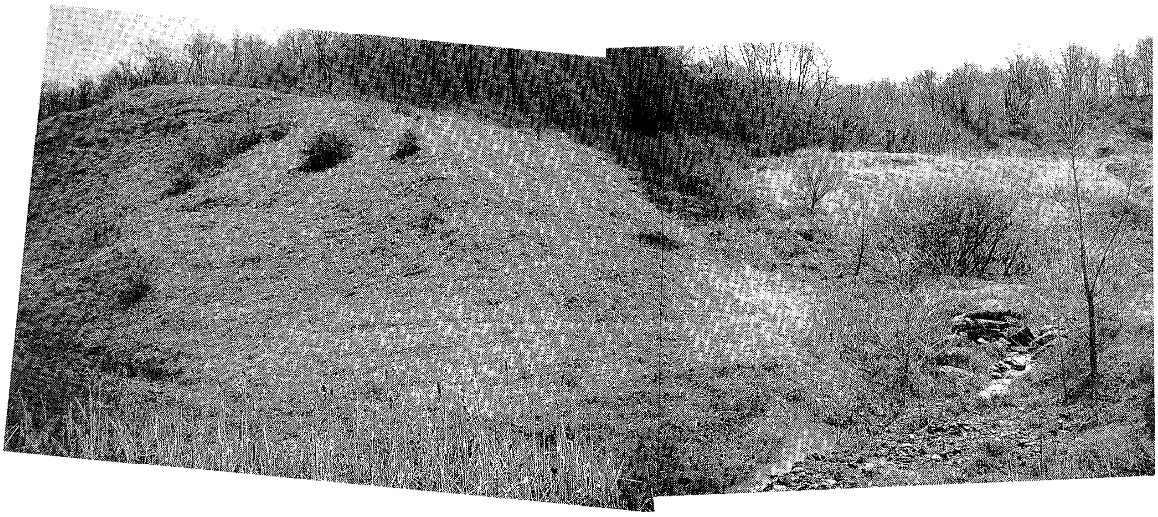


Photo 6 Valley stress relief joint.

Photo 5 Interstate-79 slide.

Location. (Hamel & Flint, 1972)

Interstate Route 279 (I-279) crosses the Ohio River 9 miles northwest of Pittsburgh on a bridge at Neville Island (see Fig. 5). 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 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. 6).

Shear Zones. (Hamel & Flint, 1972)

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 differed 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. 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. 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 about 6 in. of claystone fragments and silty clay and above that by fractured claystone.

Geometric Details. (Hamel & Flint, 1972)

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. 9. 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.

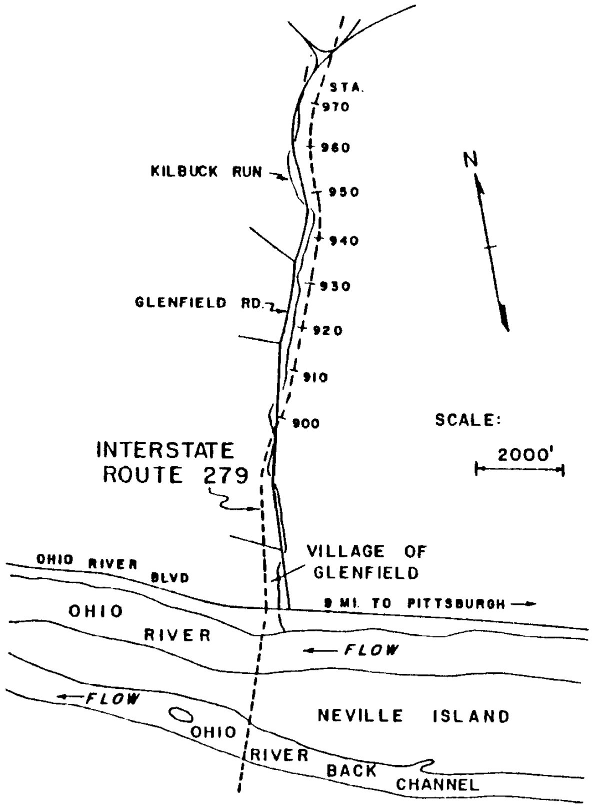


Fig. 5 Location map, Interstate Route 279. (Hamel & Flint, 1972)

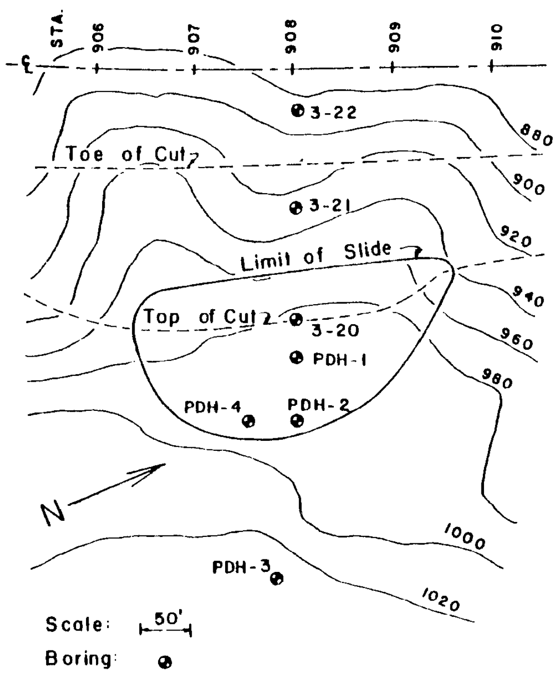


Fig. 6 Plan of slide A. (Hamel & Flint, 1972)

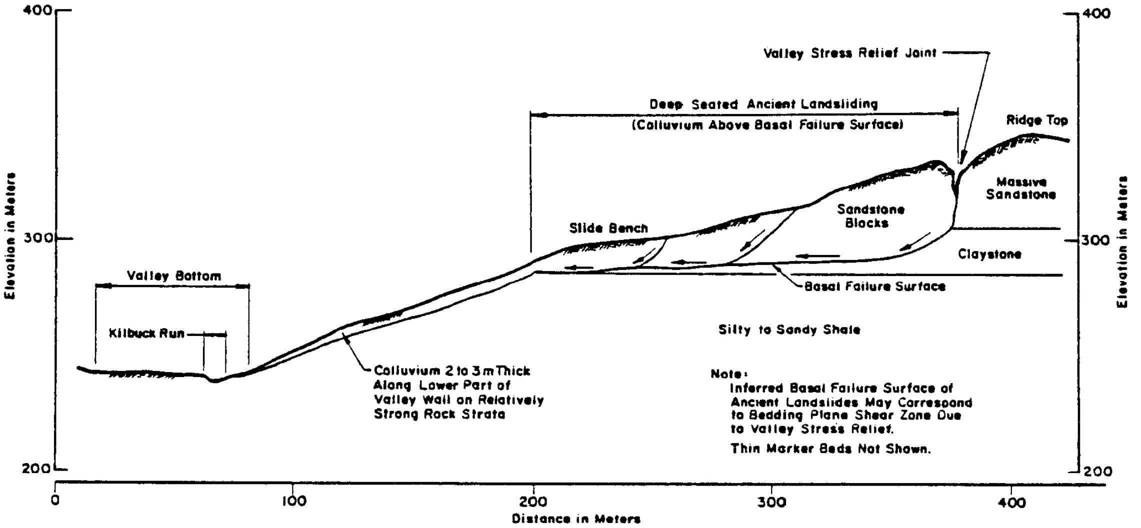


Fig. 7 Generalized slope cross-section, Sta. 908. (Hamel & Adams, 1981)

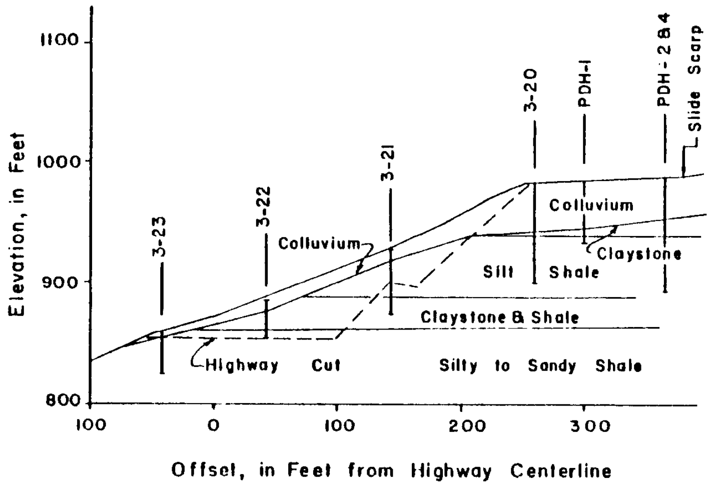


Fig. 8 Geologic section through slide A. (Hamel & Flint, 1972)

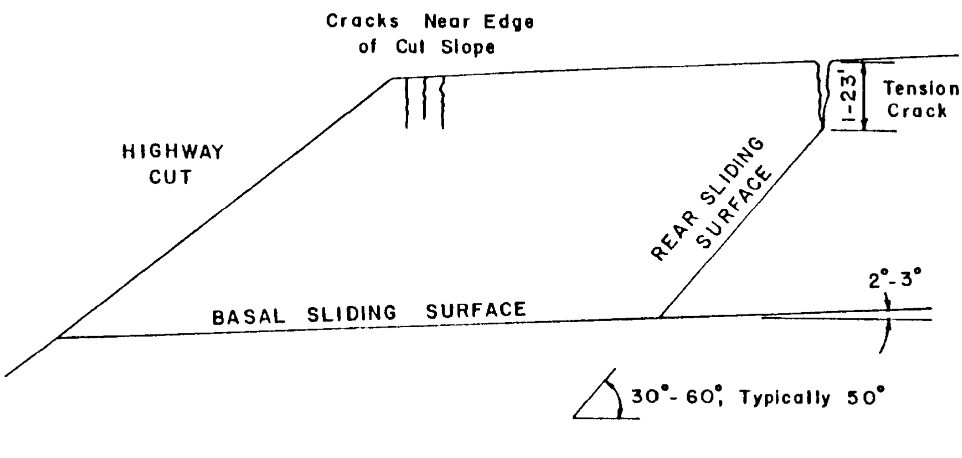


Fig. 9 Schematic cross section of typical failure mass. (Hamel & Flint, 1972)

Stop 6

Hulton Cut-Allegheny Valley Expressway.

This 400-foot high cut exposes the middle portion of the approximately 600-feet thick Conemaugh Group. (See Fig. 10). (from the field trip guide.)

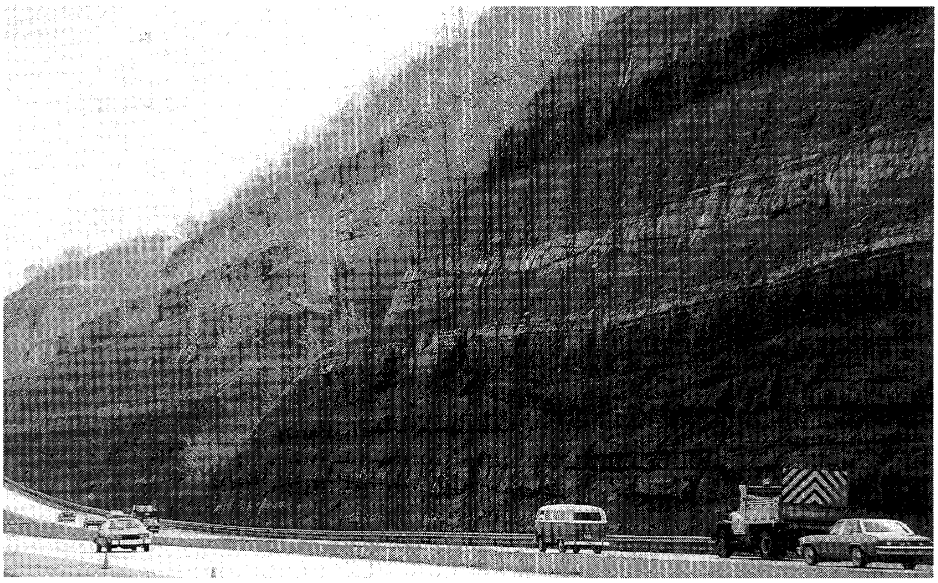


Photo 7 Hulton Cut.

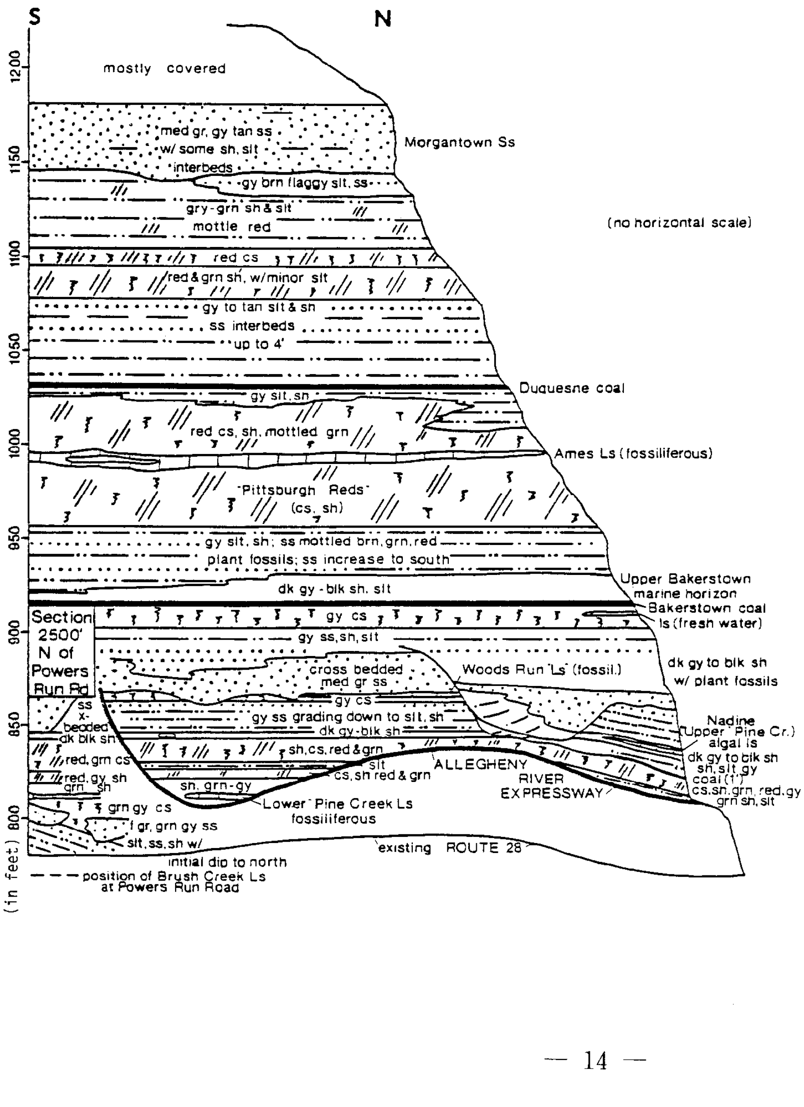


Fig. 10 Sketch of the stratigraphy of the outcrop at Hulton Bridge along the Allegheny River Expressway. (after D. R. Kelley, W. R. Wagner, R. Lund, 4/71, 45th Annual Field Conference of Pennsylvania Geologists, Guidebook, Pittsburgh, Pa., October 1980.)

May 3, 1983

Wheeling Area, West Virginia.

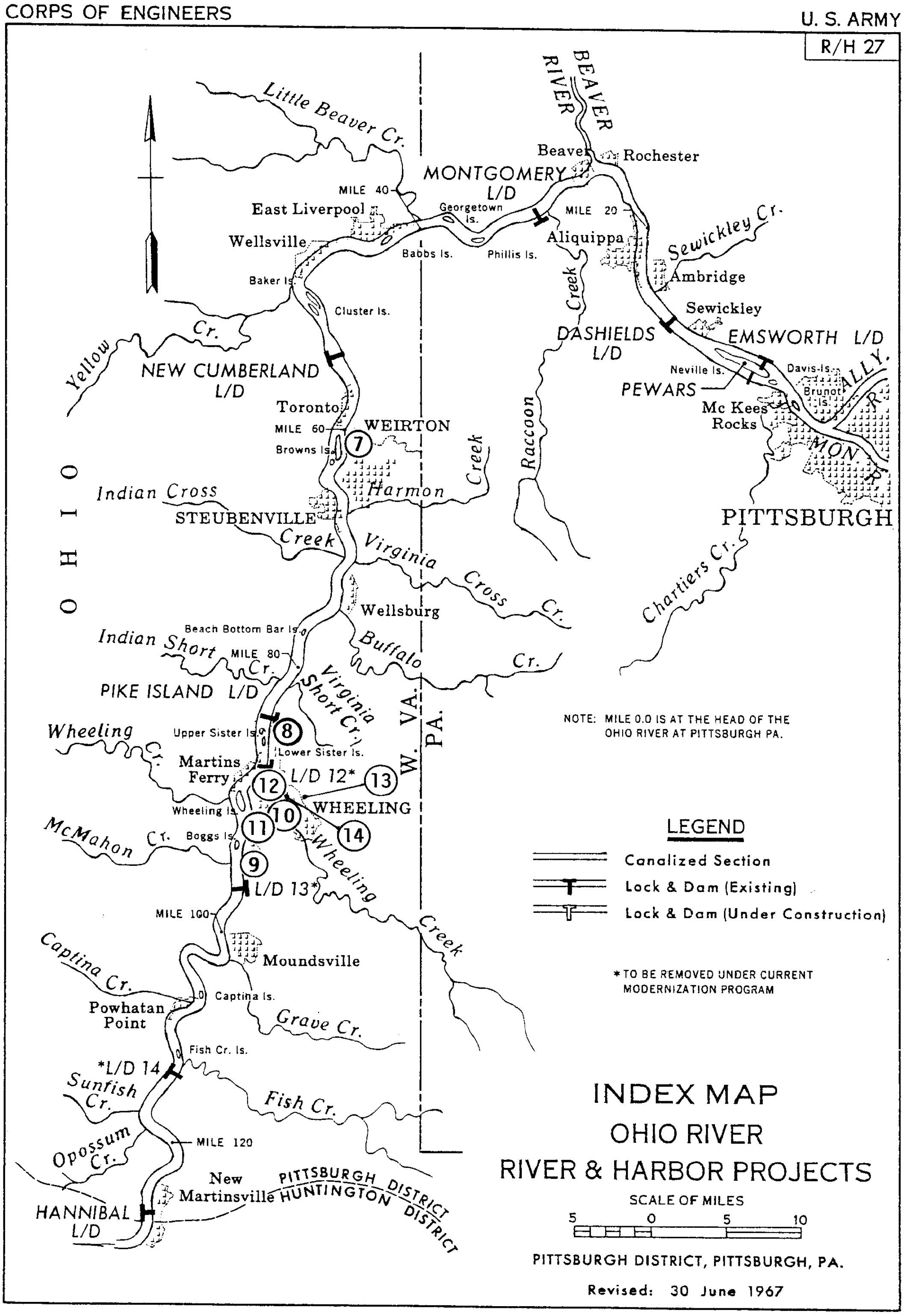


Fig. 11 Route Map, Wheeling area. (from the field trip guide)

Stop 7

Weirton Slope.

Exploration for expansion of a steel mill in 1964 revealed the hillside was a large, colluvial mass. Measures taken to prevent reactivating slope movements included a sheetpile wall with tension ties, two six-foot diameter drainage tunnels, and a deep cutoff trench to intercept ground water before it moved into the slide mass. See Reference 9. Case History 5 of Reference 8 describes a stability problem on this same valley wall approximately 2000m northwest of this site. (from the field trip guide.)

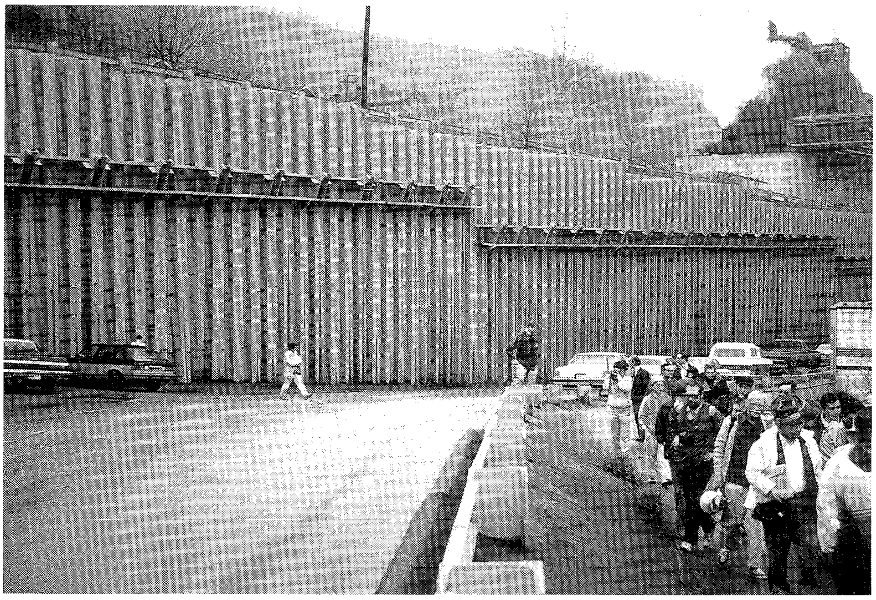


Photo 8 Sheetpile wall.

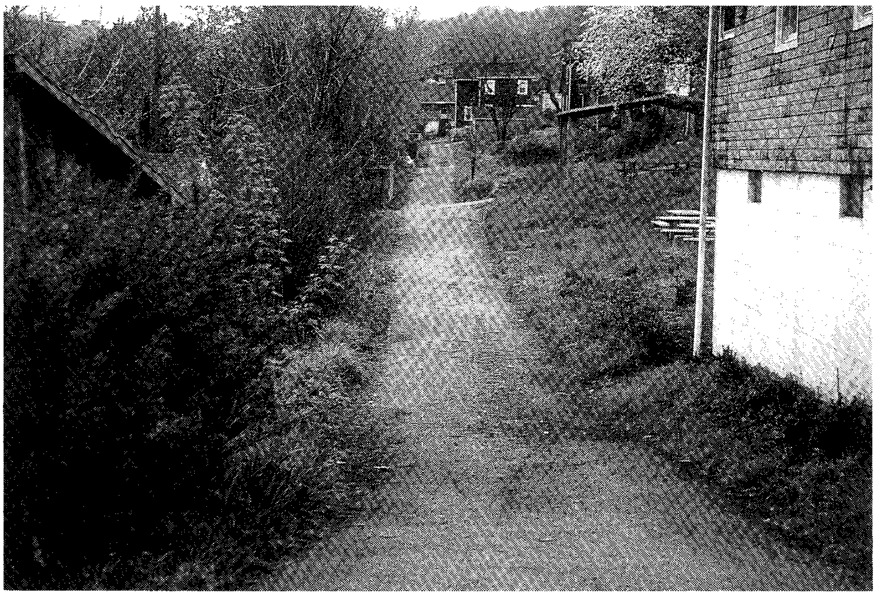


Photo 9 Road in residential area.

Photo 10 Cut off trench.

Case history 5 (Gray, et al., 1979)

Slope instability which resuted from excavation of the toe of a slope at Weirton, West Virginia exemplifies stability problems associated with colluvial slopes throughout the region. In 1956, the lower portion of the Weirton slope (Fig. 12) was excavated to develop a coal storage facility. This excavating initiated movement upslope in an area occupied by public roads, private property, and buildings. After initial sliding was observed above the excavation, remedial measures were undertaken, including sealing of tension cracks to prevent surface runoff from entering the slide mass and construction of a sheetpile wall just above the top of the excavation. A coal stockpile was then placed as a buttress to prevent further solpe movement. The amount of coal removed from the stockpile was later governed by visual observation of lateral displacement of the upslope sheetpile wall. When the lateral movement of the sheetpile wall became excessive, coal removal was halted. Extensive investigation of the slope was initiated in 1968 because the coal stockpile was to be removed for construction of another facility.

The slope is located on an abandoned meander channel about 30 m above the Ohio River. The soil beneath the slope consists of colluvial material interbedded with coarse- and fine-grained alluvium.

Based on the subsurface exploration program and the geometry of the slide mass, the failure surface was located in the upper colluvium or at its interface with the fine-grained alluvium (Fig. 12). The water table was midway between the ground and the failure surface.

Consolidated drained direct shear tests of the colluvium indicated shear strength values as follows:

	Peak	Residual
Angle of internal friction (degrees)	20	16
Cohesion (kN/m^2)	8	0

Analysis revealed that the colluvium's peak shear strength was exceeded as the toe of the slope was removed during the 1956 excavation. This caused movement in the vicinity of the toe, reducing the original peak or near-peak strength to a residual or near-residual strength. The calculated safety factor against sliding prior to excavation was about 2.0

based on peak strength values and 1. 3 based on residual strength values. After excavation in 1956, the safety factor for the entire slope based on peak strength was reduced to 1. 4. However, movements at the toe reduced peak shearing strength values to near residual values ; this weakening of the toe resulted in progressive failure upslope (Bjerrum, 1967). Eventually the shearing strength of the entire slope was reduced to the residual value (giving a calculated safety factor of 0. 9) thus leading to subsequent failure. Construction

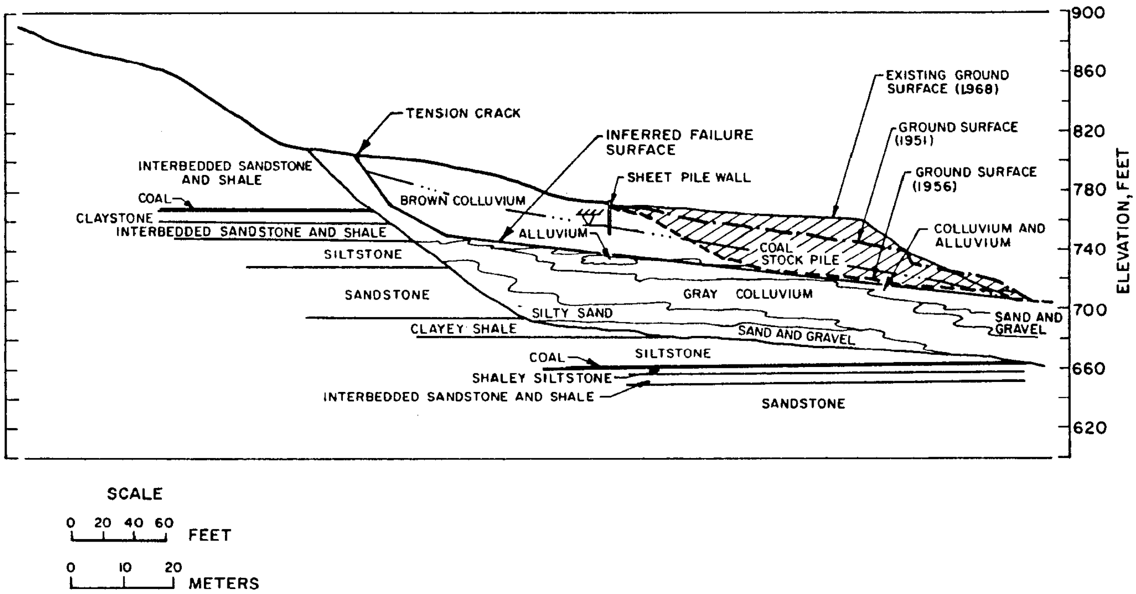


Fig. 12 Generalized cross-section, Weirton slope. (case history 5, (Gray, et al., 1979))

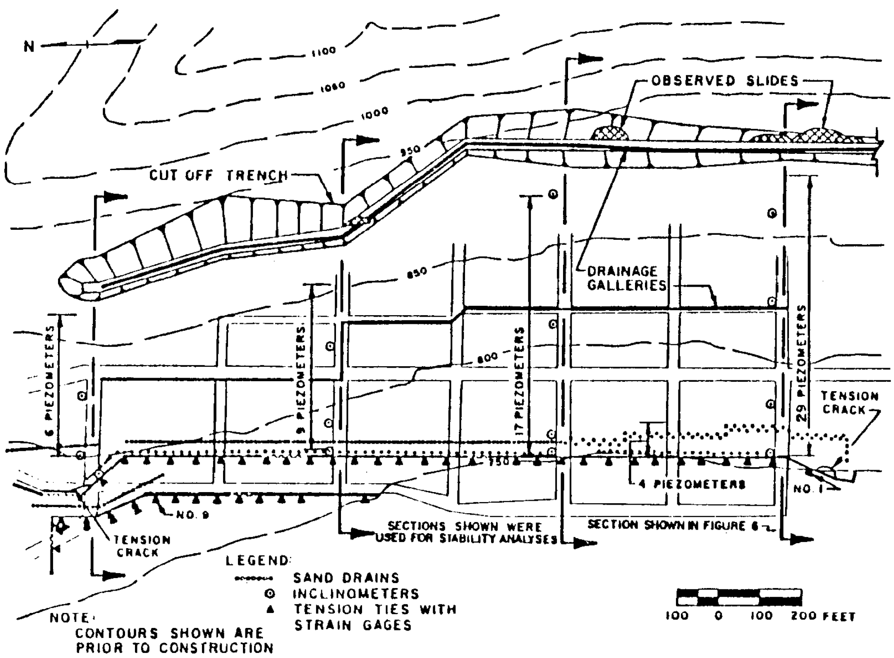


Fig. 13 Plan of site. (D'Appolonia, et al., 1967)

of the coal stockpile which acted as a buttress raised the calculated safety factor to 1.3, based on residual strength, and movement halted. When sufficient coal was removed from the stockpile to lower the factor of safety to less than 1.0, movement of the sheetpile wall occurred and coal would be added to the stockpile. Understanding of the slopes' response to the excavation in 1956 and its periodic movement when coal was removed from the stockpile permitted design of a grading scheme to stabilize the slope.

The coal was replaced by a slag fill on which the planned facilities were constructed. Removal and replacement of the coal was accomplished in a period of several months by limiting the unsupported width of excavation to 15 m at any time.

Stop 8

Pike Island Slide Repair.

In 1960, during construction of the Pike Island Locks, an old slide mass (approximately $1.5 \times 10^6 m^3$) was reactivated. Stabilization involved excavation to rock and construction of a fill keyed into rock to carry the railroad. See Case History 6 of Reference 8.

(from the field trip guide.)

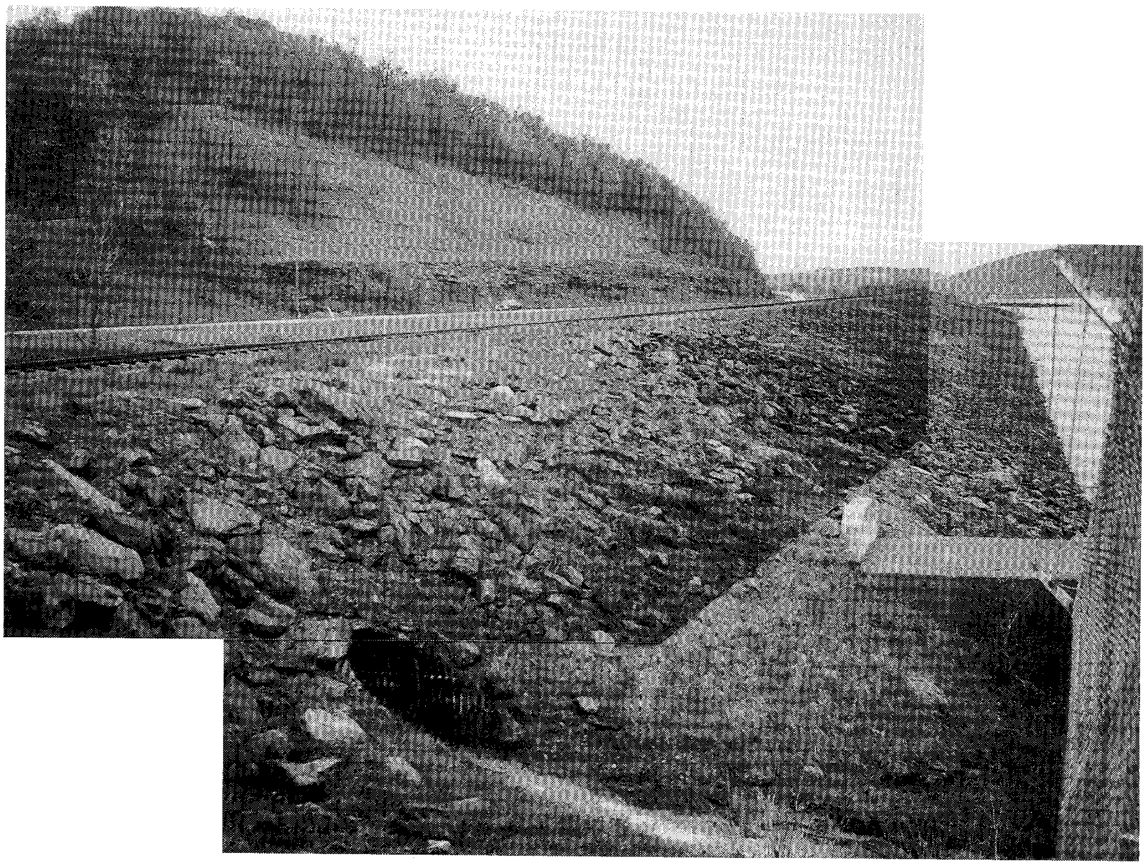


Photo 11 Pike Island slide repair.

Case history 6 (Gray, et al., 1979)

In May 1960, a massive landslide occurred on the landward side of the lower approach of the Pike Island Locks, then under construction. The Pike Island Locks and Dam project is located on the Ohio River near Wheeling, West Virginia, about 128 km downstream from Pittsburgh, Pennsylvania. This project was designed by and constructed under the supervision of the U. S. Army Engineer District, Pittsburgh, from 1959 to 1964.

West Virginia State Route 2 and a branch line of the Penn-Central Railroad crossed the slide area parallel to the river. The slide mass, approximately 1.2km long, 82 m wide, and 18 m deep, involved nearly $1.5 \times 10^6 m^3$ of colluvial and alluvial material. The slide was described in a Design Memorandum by the U. S. Army Engineer District, Pittsburgh (1963).

In the slide area, the Ohio River Valley was entrenched in the early Pleistocene Epoch, then was filled with generally coarse-grained glacial outwash. More recently, additional, generally finer-grained sediments have been deposited during flood periods.

In the early stages of slide movement, dewatering wells were drilled into vertical fractures of the massive Grafton Sandstone landward of the slide in an attempt to stabilize it and there was some slowing of the slide motion. Horizontal drains (Fig. 14) were later drilled through the slide mass and approximately 12 m into the base of the sandstone to intercept water in the vertical fractures. They were installed on 15-m centers and were equipped with flap valves to prevent entrance of water during high-river stages. These drainage measures also slowed the slide movement but did not stop it.

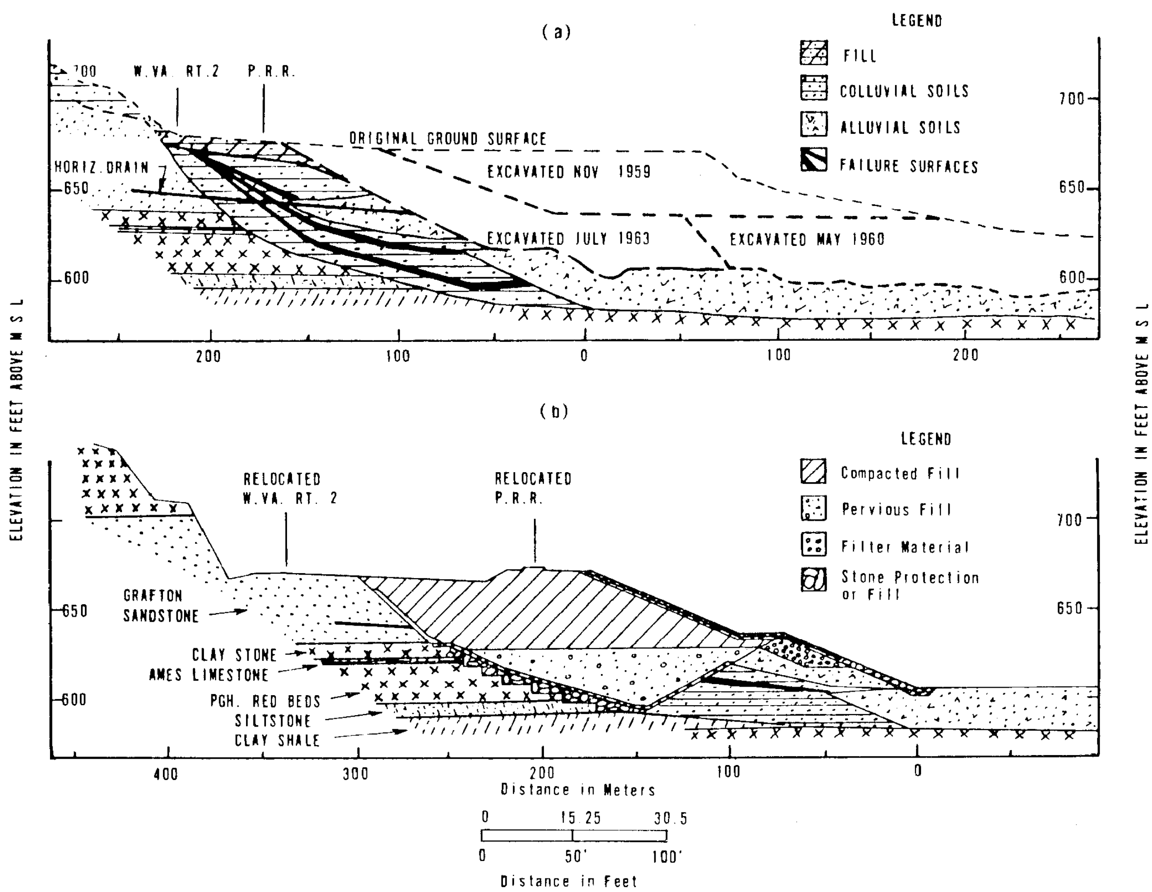


Fig. 14 Generalized cross-section, Pike Island slide. (case history 6, (Gray, et al., 1979))
(a) excavation history, (b) reconstructed slope.

Slide movement continued from 1960 to 1963 at variable rates depending on construction activities. Movement from June to November 1962 was 76 mm with the rate increasing to about 5 mm per day with flooding of the cofferdam at the end of construction in early November 1962. With this increase in motion, the head crack of the slide extended downstream. A sand and gravel berm was then placed in the river to buttress the toe of the slide. This berm also slowed, but did not stop, slide movement.

In March 1963, flood waters raised the river level 7 m above normal pool and caused the slide movement to accelerate to rates on the order of 125-150 mm per day. Until that time, the slide mass had been essentially intact but, with increased movement in th spring of 1963, the mass broke into a series of segments, some of which rotated slightly toward the river. It was then decided that stabilization and preservation of the slope could only be accomplished by relocation of the highway to intact rock and by providing support of the railroad on new fill keyed into rock with a riverward buttress and good under-drainage (Fig. 14). This fill was constructed and the highway and railroad were relocated in 1964. The reconstructed slope has remained stable up to the present time.

Stop 9

McMechen Slide.

Slide movements in 1975 damaged 56 homes and threatened many more. The colluvial mass has a maximum depth of 13m and a volume of approximately $1.5 \times 10^6 m^3$. Stabilization measures are based on preventing additional damaging movements and minimizing disruption of the community. See References 10 and 11.

(from the field trip guide.)

Photo 12 Upper slope, not sliding.

Photo 13 Surface drainage along debris bench.

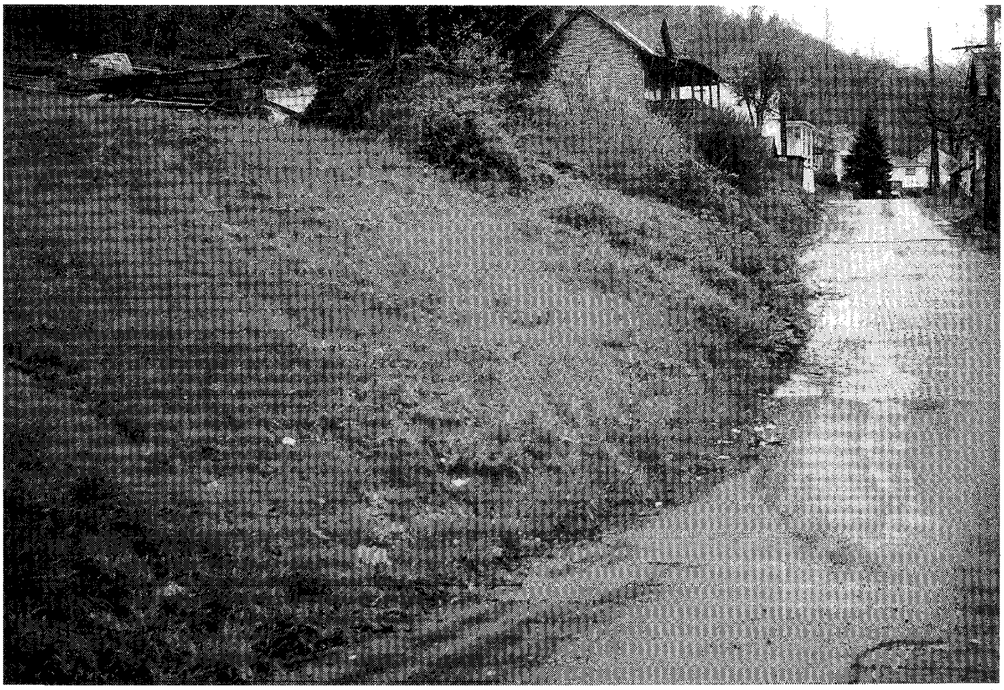


Photo 14 Locust street.

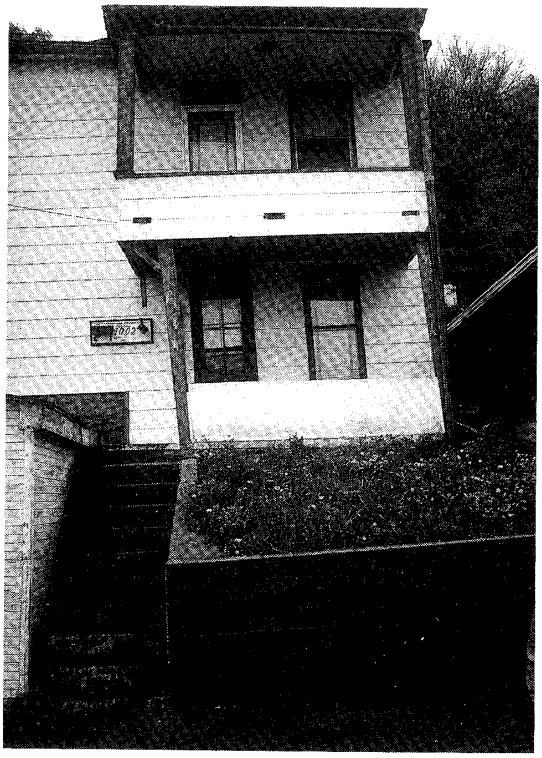


Photo 15 Damaged house.

ABSTRACT (Gray, et al., 1980)

Slide movements in March 1975 affected a significant portion of a small town in the Ohio River Valley, U. S. A. Subsequent investigation revealed the movements were due to reactivation of a colluvial slide mass. In addition to the economic restraints normal to most engineering projects, development of a stabilization scheme required consideration of social impacts. Excavation of the upper portion of the colluvial mass to reduce driving forces was restricted by the community's interest in minimizing relocation of undamaged homes. Drainage measures were limited to public streets. The adopted stabilization design was a compromise between technical measures and socio-economic considerations and resulted in a calculated factor of safety of 1. 0 for the worst post-construction case.

Landsliding

In March 1975, landslide movements damaged 56 of 120 homes located on the colluvial mass. Based on property damage and slide scarps, the movements were primarily in the upper, steeper portions of the colluvial mass. Numerous sliding masses of variable size and magnitude, moving predominantly as rotational slides and slow earth flows were observed. The most severe damage was in concentrated areas just below swales that extend upslope from the main colluvial mass. The area exhibiting movements had dimensions of 1000 meters along the hillside and 2000 meters across the slope. This colluvial mass has a maximum depth of 13m and a volume of approximately $1.5\times10^6 m^3$. Fig. 16 shows a section through the slide area.

There had been no recent excavation or filling to disturb the slope prior to the damaging movements in 1975. Therefore, the movements are attributed to increased precipitation. Fig. 17 shows precipitation data at McMechen from 1945-1978 in terms of deviation from the normal. Precipitation was generally well below normal for 12 years prior to 1974, with 1972 having the only significant above normal precipitation in that time period. In 1974, precipitation was well above normal for six straight months (April–September) culminating in September with the third highest monthly precipitation (11.7 cm.) since 1945. Precipitation was slightly below normal in October and November. From December 1974 through March 1975, when extensive landsliding was reported, precipitation was again above normal. Since 1975, rainfall has been slightly above normal. The long period of abnormally low precipitation, followed by a period of high precipitation and ground saturation in 1974 and early 1975, is believed to be the triggering mechanism for the sliding.

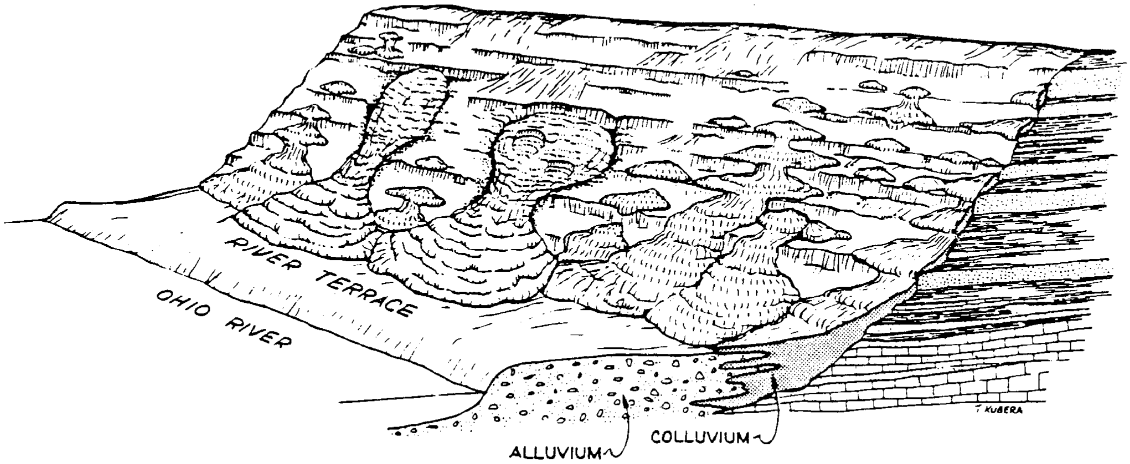


Fig. 15 Idealized diagram of colluvial slope development. (Gray & Gardner, 1977)

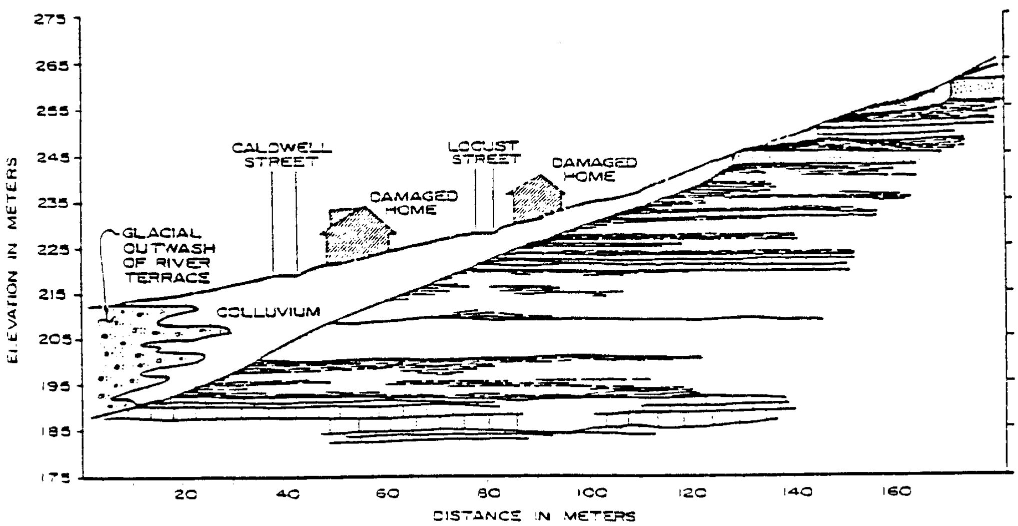


Fig. 16 Section showing lower third of the hillslope. (Gray, et al., 1980)

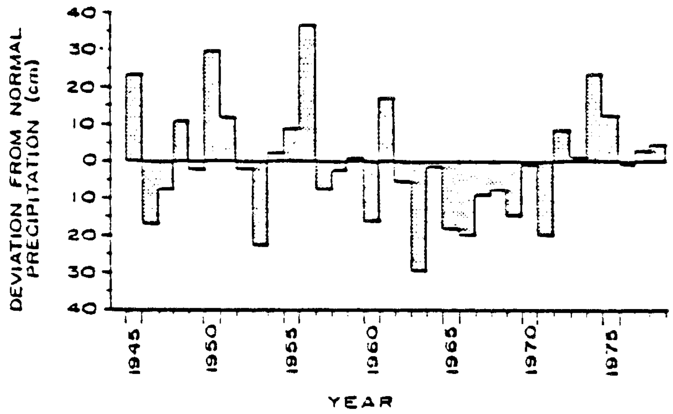


Fig. 17 Yearly deviation from normal precipitation. (Gray, et al., 1980)

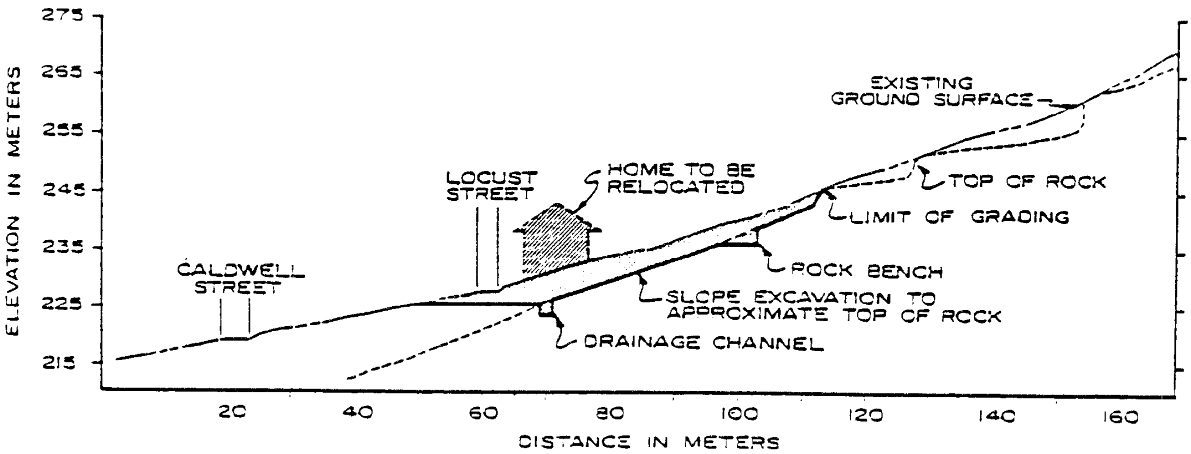


Fig. 18 Section showing excavation scheme. (Gray, et al., 1980)

May 4, 1983

Wheeling Area, West Virginia.

Stop 10

Reinforced Earth Wall.

Large concrete wall constructed on I-470. Discussion by Barney Stinnett, West Virginia Department of Highways. (from the field trip guide.)

REINFORCED EARTH WALL, WHEELING, WEST VIRGINIA (Thompson, et al., 1978)

I-470 in West Virginia crosses the Ohio River in South Wheeling and intersects I-70 in the northeast corner of the city.

The area, which we visited during the field trip, is the area of I-470 near the interchange (approximately station 156+00 to station 183+00) near the bridge at the end of the project at I-70.

This area had been mined, and spoil had been wasted on the project and adjacent areas. Some of the soil associated with the spoil was used in the embankment. The embankment had been constructed from approximately station 166+00 to the bridge at station 183+00. On May 11−12, 1974, a very heavy rainfall occurred on the project. On

Photo 16 Reinforced earth wall, distant view.

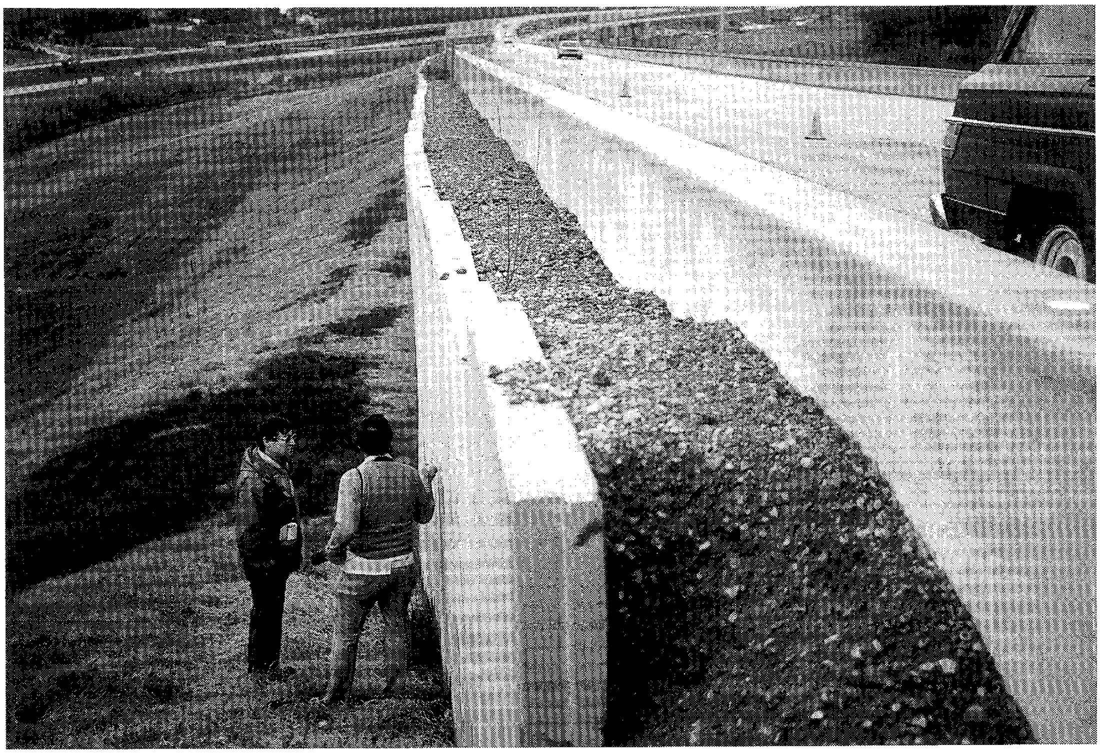


Photo 17 Reinforced earth structure.

May 12, 1974, a large embankment failure occurred approximately between stations 168+00 and 172+00.

Six types of corrections were considered for the two areas.

1. Buttress
2. Excavation and backfill
3. Bridging
4. Piling
5. Reinforced Earth
6. Root Piles (Fondedile)
7. Combinations of these types

After comparing all of the possible corrections, the most economical design was a combination of reinforced earth through the areas that had not been constructed (Station 156+54 to station 166+89), and a buttress design for the area where the landslide occurred.

Stop 11

Damaged houses, street, and sidewalk in Mozart.

This slide is 4 to 5 years old and has caused one house to be abandoned. Another damaged house has been reoccupied during the last year. Slope 40%, Westmoreland soil, Dunkard bedrock,with risks of *high* (31.6/14.1=2.24) (see reference 16 for explanation of terms), *moderate* (47.1/42.7=1.10) and *moderate* (35.5/35.2=1.01), respectively.

(from the field trip guide.)

Photo 18 Ground failure.

Photo 19 Damaged house.

Stop 12

Mrs. Fry's house - Waddles Run. (Photo 20)

This slide is a debris avalanche and occurred in December 1980. The slide stopped 20 feet (6m) in back of the house and Mrs. Frye moved out. During the next 6 months, the slide moved against the house. Little or no help was available from State or county personnel. Slope 44%, Westmoreland soil, Monongahela bedrock, with risks of *moderate* (10.9/19.7=0.55), *moderate* (47.1/42.7=1.10), and *moderate* (12.3/11.5=1.07), respectively.

(from the field trip guide.)

Stop 13

Landslides in Coal Waste. (Photo 21)

An example of coal waste from deep mining of the Pittsburgh coal seam. Large landslides exist at the top of the hill on left and small slips can be seen in the coal waste (see Fig. 58 in Reference 12). Also visible is acid-mine drainage with associated "yellow boy" ($FeOH_3$). Slope 35%, Westmoreland soil, Monongahela bedrock, with risks of *high* (31.6/14.1=2.24), *moderate* (47.1/42.7=1.10), and *moderate*. (12.3/11.5=1.07), respectively. Presently being reclaimed. (from the field trip guide.)

Stop 14

Gabion Drainage. (Photo 22, 23)

The Wheeling hospital was flooded due to 5"/24 hour thunderstorm soon after occupancy. New drainage systems, concrete walls, and gabions have been installed to divert flood waters from the creek and stabilize slides. Slope 32%, Westmoreland soil, Monongahela bedrock, with risks of *high* (31.6/14.1=2.24), *moderate* (47.1/42.7=1.10), and *moderate* (12.3/11.5=1.07), respectively. New slides occur to the right of hospital.

(from the field trip guide.)

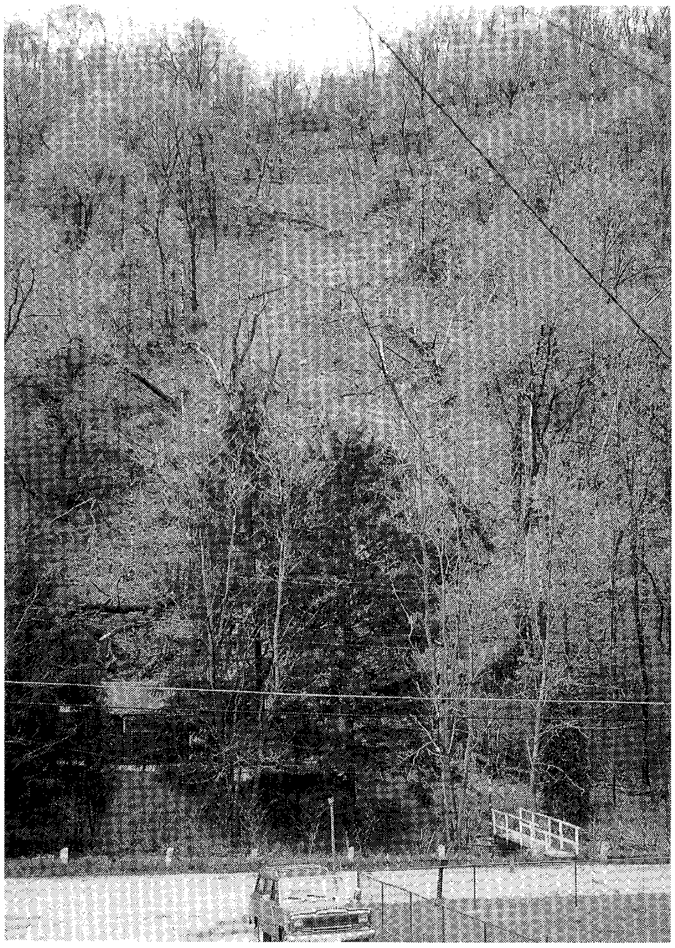


Photo 20 Mrs. Fry's house.

Photo 21 Slides in spoil bank of strip mine.

Photo 22 Gabion ladder.

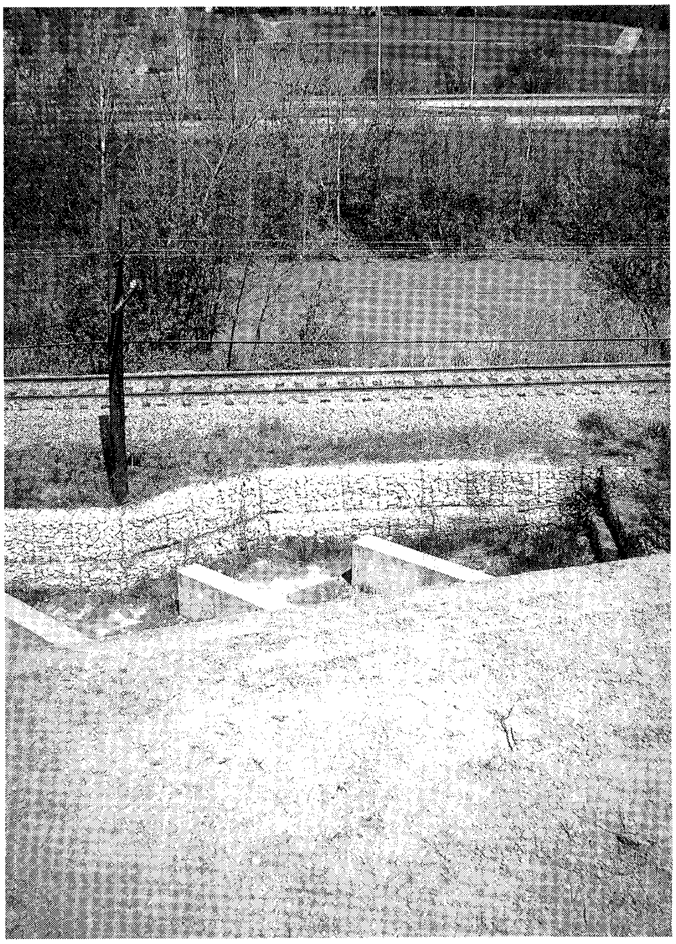


Photo 23 Gabion and control dike.

May 5, 1983

Cincinnati Area, Ohio.

Topography and geology (Fleming, Johnson & Hough, 1981, Reference 18)

The topography of the area is characterized by a dissected upland surface, hillslopes along the Ohio River and major tributaries, and flood plains and terraces of the rivers. Maximum relief in the area is about 160 m. The sharp, local relief provided by the Ohio River and its tributaries separates the metropolitan area into a large number of distinct communities or neighborhoods.

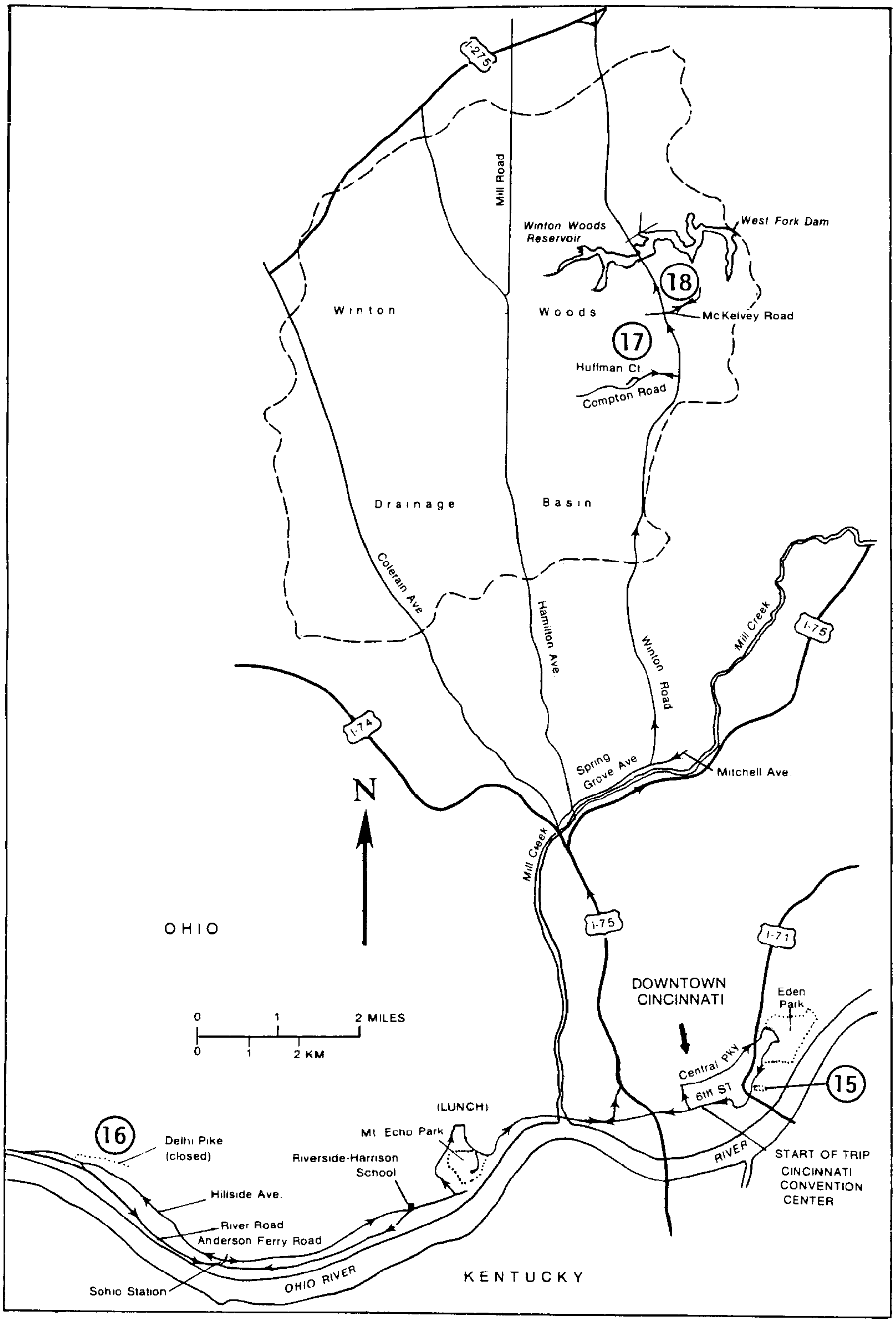


Fig. 19 Route Map, Cincinnati area. (Fleming, et al., 1981)

The geology of the area is best described in two parts:
(1) the bedrock of the area and the colluvial deposits developed on the bedrock, and (2) the glacial deposits. Units related to both the bedrock and the glacial deposits present engineering problems, particularly landslides.

Landslides in colluvium.

There are at least two distinct types of landslides associated with the colluvium. Where the colluvium is thin and occurs on relatively steep slopes, the entire thickness of colluvium fails and moves downslope by a combination of sliding and flowing, exposing the bedrock along the failure surface. Such failures typically occur during early spring, before the vegetation has sprouted, after the last frost, and during periods of intense rainfall.

The second type of failure typically occurs in thicker colluvium in the lower parts of slopes in the Kope Formation and Miamitown Shale or in colluvium that is loaded by artificial fills. These failures have the morphology of a slump with open, cresent-shaped cracks in the crown area, and bulges and transverse ridges in the downslope parts of the slides. Both of these types of failures are in the description of conditions at Stop 16.

Landslides in glacial deposits.

The landslide problems are associated with both the till and the lake clays. The till is present on the dissected upland and caps the terraces and valley fill of lake clays. Landslides in the till are common in cuts and fills but rare in natural slopes. The failures have the morphology of a typical slump.

The lake clays, which consist of laminated silts and clays are particularly troublesome. The locations of the deposits are poorly known because of the large drainage changes produced by the Illinoian glaciation. At Stops 17 and 18 of the field trip we examine landslides in the lake clays capped by till.

Stop 15

Mount Adams.

Junction of Oregon Ave. and Monastery St. Depart bus and walk east along Oregon Ave. to end and back to bus along Baum St. (from Reference 18)

Photo 24 Slide 1 in Fig. 20.

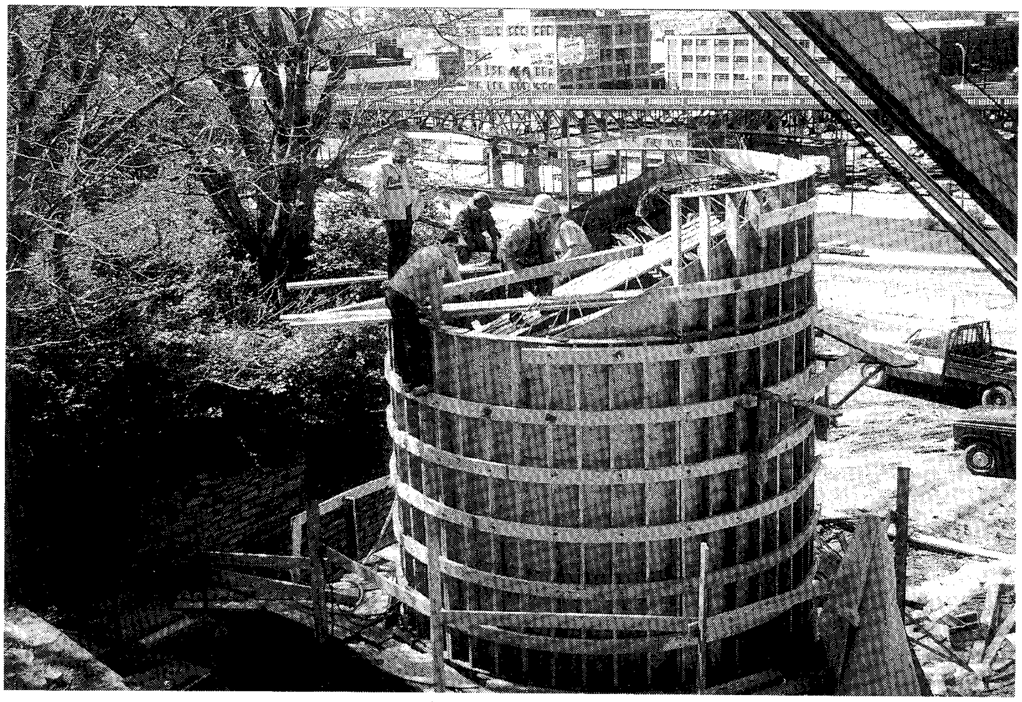


Photo 25 Entrance to tunnel.

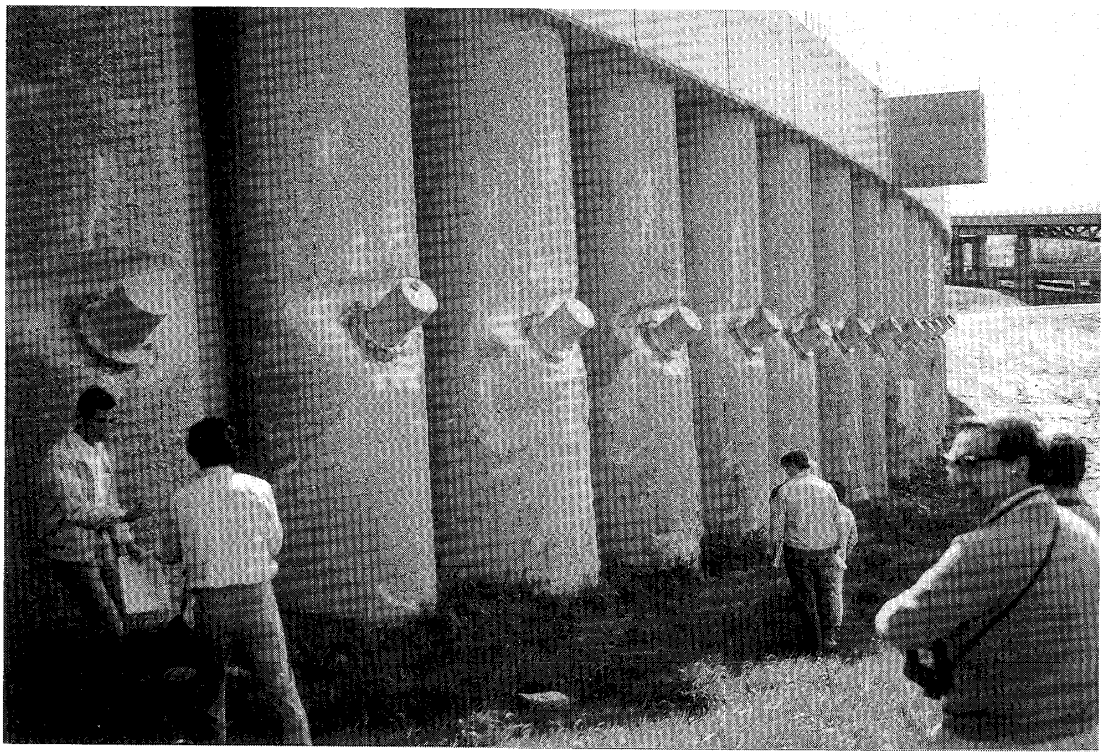


Photo 26 Pier wall with ties into tunnel.

Photo 27 Pier wall, distant view.

MOUNT ADAMS LANDSLIDE (Fleming, in Reference 18)

The landslides on Mount Adams (Fig. 20) began during November 1973, at the time construction was underway for ramps for a new highway, I-471. Piles had been driven and timber lagging installed coincident with about a 2.1-m-deep excavation along Kilgour Street. Shortly thereafter, utility lines failed and cracks appeared in sidewalks, streets, and houses along Baum Street (slide 1, Fig. 20). During the late spring of 1974, an excavation near the west end of Kilgour Street was followed by movement that disrupted utilities and cracked streets near the western end of Baum Street (slide 2, Fig. 20). Since then, both areas involved in the sliding have enlarged and include most of the property between Baum and Kilgour Streets.

The failure apparently did not involve bedrock. About 10m downslope from the head, the failure surface is in colluvium at a depth of about 5m. About 10m upslope from the toe, the failure surface is in colluvium at a depth of about 8 m. No information was available on the depth of the failure surface between these points, but it is probably close to the contact of colluvium with the bedrock along much of the reach.

When completed, stabilization of the Mount Adams landsliding will have cost in excess of \$ 22 million. A pier wall along the toe of the slope will contain more than 160 piers extending into bedrock. The piers will be tied back into the slope with steel tendons anchored in a 2.7-m-diameter tunnel in bedrock under the slope. (see inset in Fig. 20)

Remedial measures were under construction during 1980. In November 1980, movement along Baum Street prompted city officials to evacuate people from some of the structures.

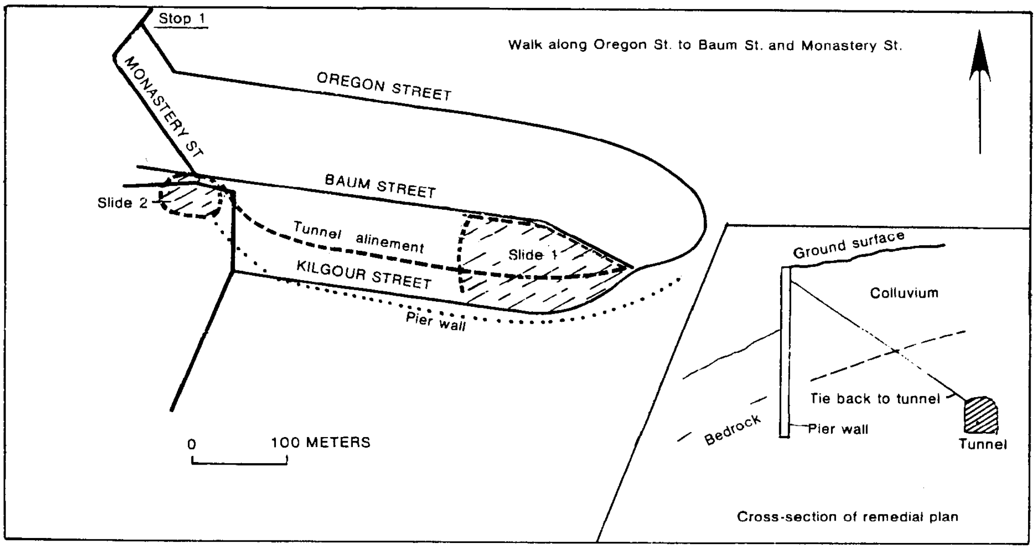


Fig. 20 Location of landslides along Baum and Kilgour Sts. that began during late 1973 and spring of 1974. This is Stop 15 of field trip. We will walk along Oregon St. to Baum and Monastery Sts. to view landslide damage. (sketch map adapted from a map in Cincinnati Enquirer (Kaufman, 1980).) (Fleming, et al., 1981)

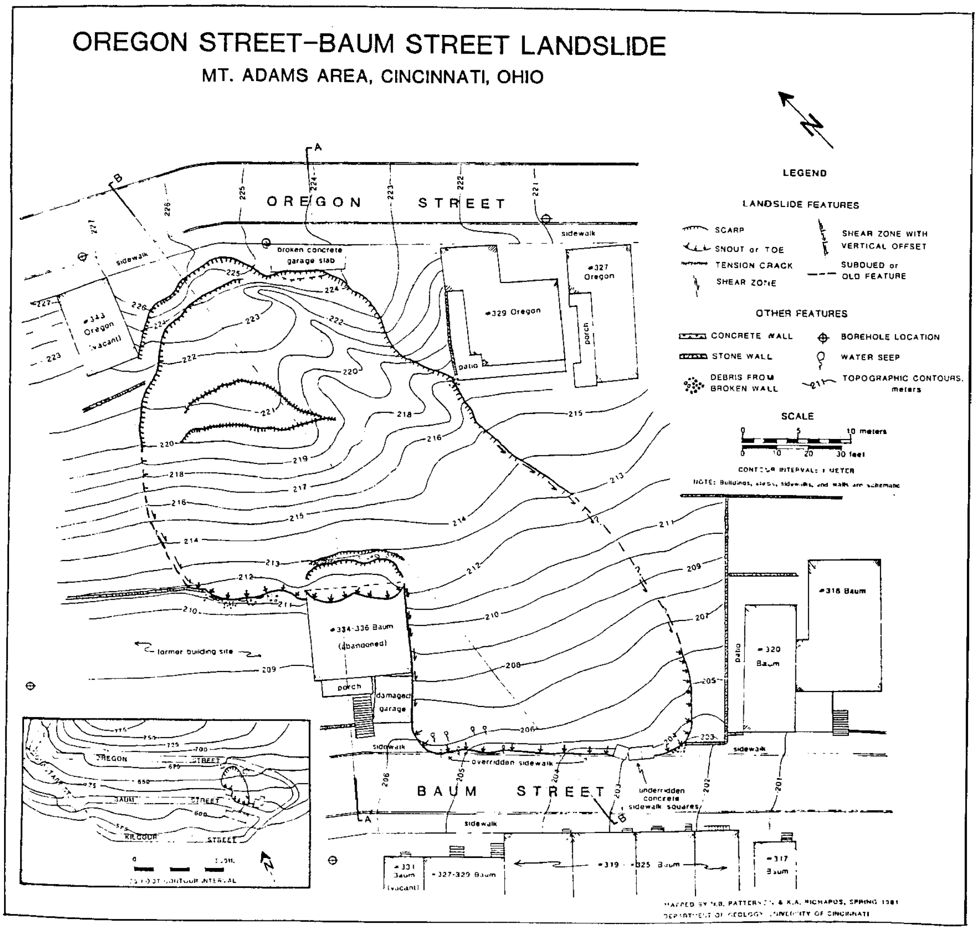


Fig. 21 Oregon Street-Baum Street Landslide. (Richards, et al., Reference 19)

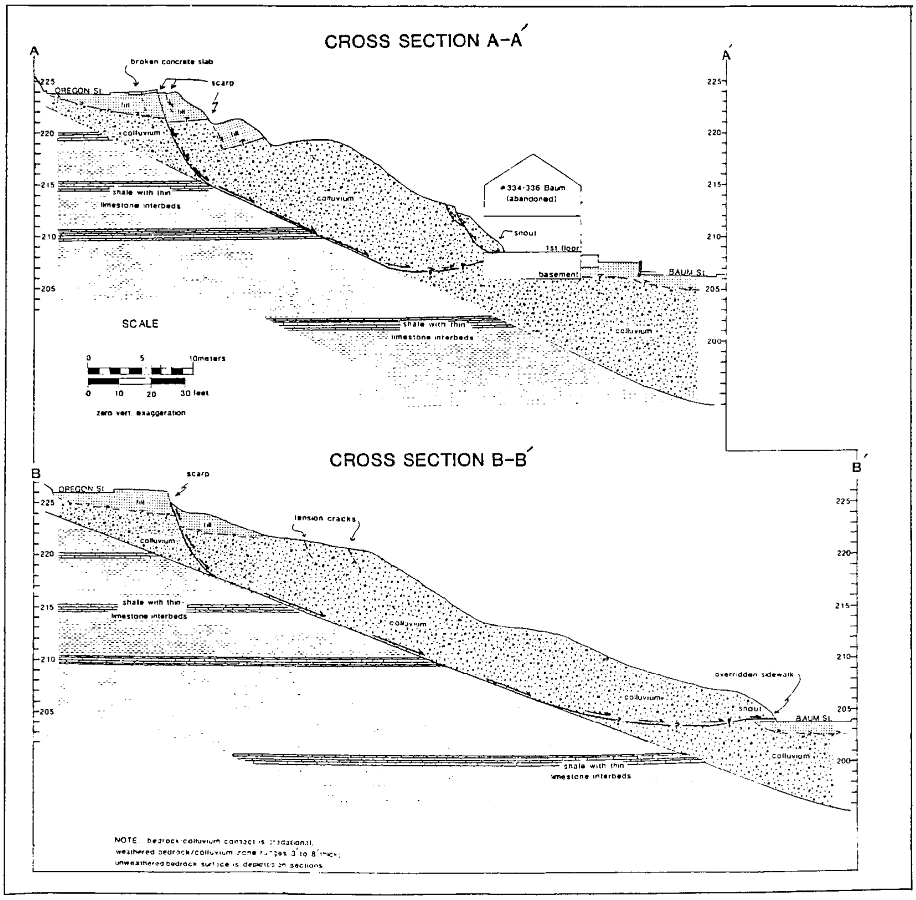


Fig. 22 Oregon Street-Baum Street Landslide, cross sections. (Richards, et al., Reference 19)

Stop 16

Hillside Avenue and Delhi Pike.

This is a lengthy stop where we will examine materials involved in slope failures, typical problems, corrective measures, and data obtained from an array of field instruments. (from Reference 18)

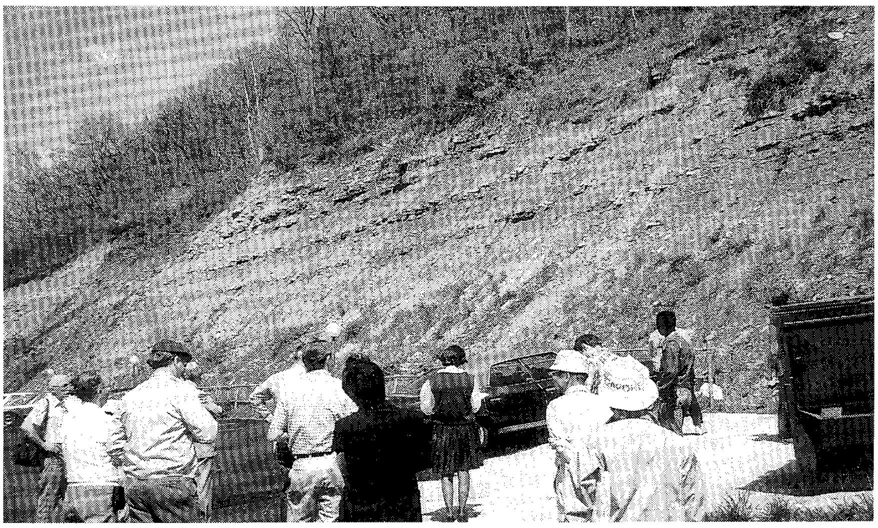


Photo 28 Colluvial slope.

Photo 29 Slide along Delhi Pike.

Photo 30 Damaged wall.

Landslides in colluvium (Fleming, et al., in Reference 18)

Delhi Pike was constructed on the hillside before 1912 by making a small sidehill cut and fill. Landslides above and below the road exhibit two different styles of failure. Upslope from Delhi Pike, the landslides are thin, translational slope failures that moved over the upslope edge of the road but apparently did not displace it. Downslope from Delhi Pike, beginning near the cut-fill bounday under the road, the landslides are deeper and most contain small rotational components of movement. Both types of movement are described below, beginning with the thin landslides above Delhi Pike.

Thin landslides

In the western part of the area upslope from Delhi Pike (Fig. 23), low, discontinuous scarps and toe bulges suggest a series of smaller landslides within a large landslide complex. A trench (No. 3) in one of the small landslides revealed steep to vertical cracks at the head and a failure surface that follwed the colluvium-bedrock contact.

To the east (Fig. 23), a more active landslide complex cuts across the pattern of low scarps and toe bulges. The landslide complex is about 90m long and 28m wide. The overall slope is 44% (24°). The lateral margins of the landslide complex are well defined in the upper part and merge into the pattern of low scarps and toes in the lower part.

Observations of this complex, and others involving thin colluvium on bedrock, point to a very limited time of activity during any one year. Reports of landslides of this type are common between late March and early May but are very rare throughout the rest of the year. This period, which corresponds to the time between the last hard frost and the leafing of the vegetation, was the only time when significant water was recorded in the piezometers. Further, the water level changed in direct response to precipitaion during the period. It is during this period of rapid rises in water level that the landslide is active.

Thicker landslides

The thicker landslide complex that was studied is immediately downslope from the upper, thin landslides (Fig. 23). The complex contains one prominent scarp represented by en echelon cracks in Delhi Pike. Downslope, three toes represented by bulges are identifiable. The lowermost toe produced a prominent bulge on the surface of Hillside Avenue near the junction with Darby Road (Fig. 23). The depth of failure, as measured by two inclinometers, was about 4.7m in the midpart of the landslide and 2.5m near the toe. The failure surface appears to follow the contact between bedrock and colluvium except where a failure surface exits as a toe.

This particular landslide complex appears to have formed progressively, beginning at Delhi Pike. According to the property owner (Mr. Dale Schmale, oral comm., October 1979), cracks were well developed in Delhi Pike, the upper rock wall was distorted, and houses farther east were destroyed before there was any indication of damage to his house, which was located farther downslope between borings 19 and 14 (Fig. 23). The uppermost toe is about 13m downslope from the scarp on Delhi Pike. The next toe, located about 20m downslope from the scarp is upslope from the site of the house.

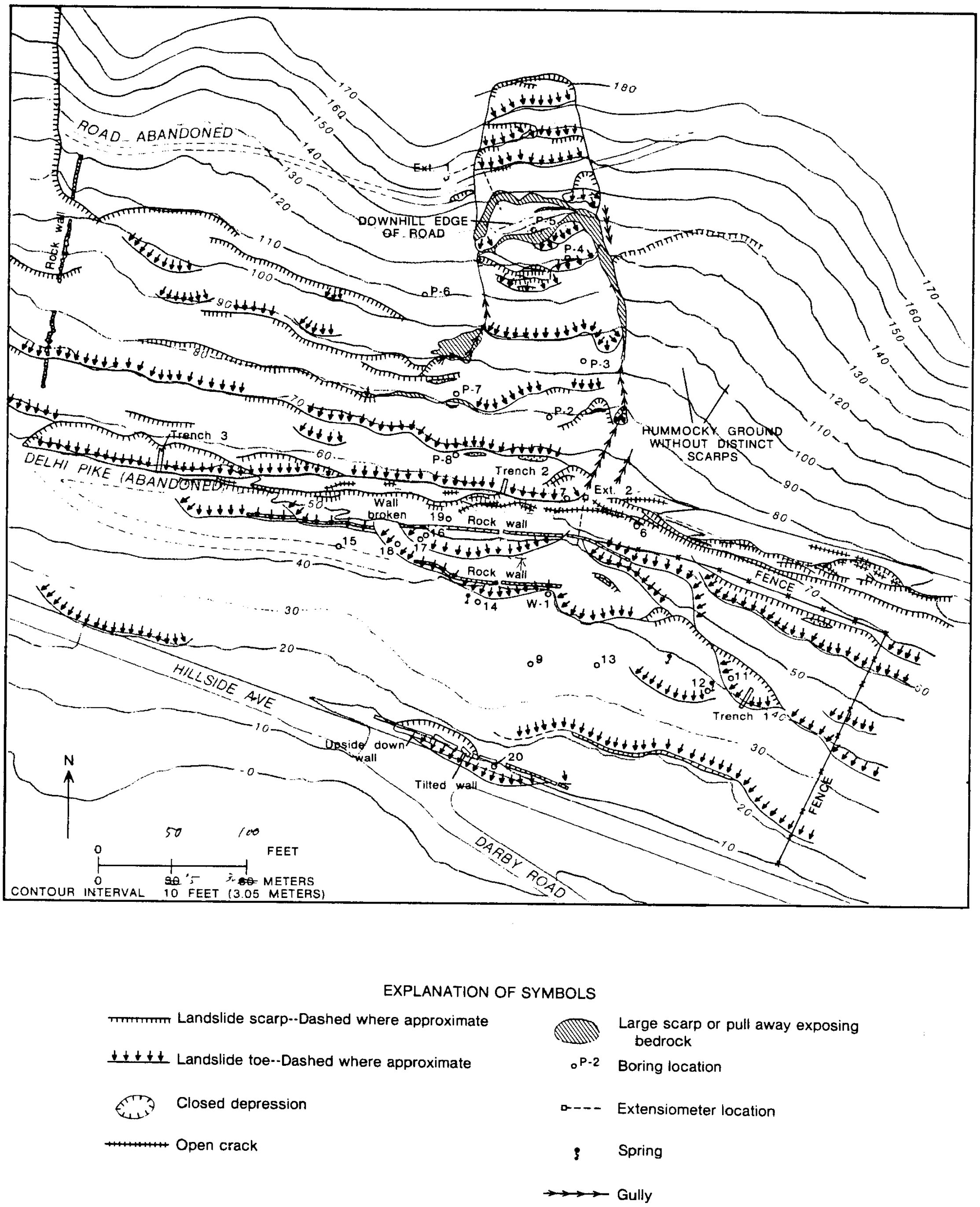


Fig. 23 Map of landslide features in Hillside Ave.-Dehli Pike area of Hamilton Co., Ohio. (Fleming, et al., 1981)

Stop 17

Huffman Court landslide complex.

Turn right into Huffman Ct. We will leave the bus here for a short walk to examine some of the problems posed by active landslides in glacial deposits. (from Reference 18)

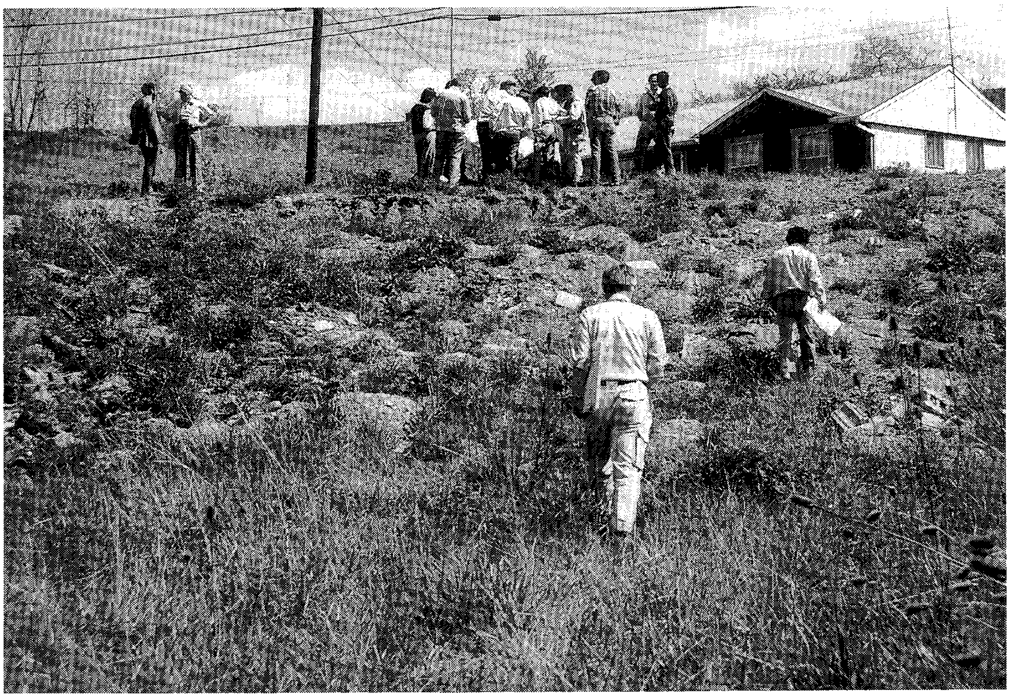


Photo 31 "early 1975" landslide, northeast edge of Huffman Court.

Photo 32 Looking southwest along Huffman Court.

Landslide in Glacial Lake Clays, (Gokce, et al., in Reference 18)

HUFFMAN COURT LANDSLIDE COMPLEX

Country Lanes Subdivision is a subdivision of low-cost housing underwritten by the Cincinnati office of the U.S. Department of Housing and Urban Development. It is centered on Huffman Court, which is in Springfield Township, on the north side of Compton Road and about one kilometer west of Winton Road (see Fig.19). The subdivision has 26 single-family residences that were constructed during 1971 and 1972. The residences

Photo 33 Outcrop of lake clay.

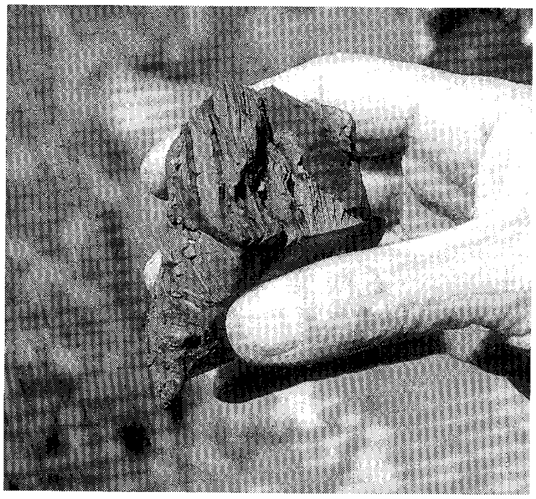


Photo 34 Lake clay.

are one-story, of slab construction, and have a one-car garage but no basement. Downspouts from houses on the south side of Huffman Court drain into the gutter of the court, whereas those from houses on the north side of Huffman Court discharge onto the slope north of the houses (Fig. 24).

Grading of the site was initiated in 1961, and consisted of a cut-and-fill operation on one side of a steeply sloping ridge. The side of the ridge between Compton and Huffman roads was excavated, and the hillside between Huffman Court and the small creek on the north side of the property was filled. The grading project was only partly finished by the end of 1962, when it was abandoned. Granding was reinitiated in 1971, by a different developer.

The landslide complex described here is not the only evidence of activity in the Country Lanes Subdivision. There is evidence of landsliding in the swale at the western end of Huffman Court on aerial photographs taken in 1938 and 1948. According to Scheper (1973), minor landsliding along the length of Huffman Court is also evident in aerial photographs taken during the early stages of grading, during 1961 and 1962, and a large slide developed within two lots at the northwest side of the cul-de-sac of Huffman Court at about that time. Another large landslide developed in the swale at the western end of Huffman Court sometime during 1971, after grading had been completed and during construction of the houses. The swale was graded with about 7.6 m of fill shortly before the landslide developed. In August of 1971, much of that fill was removed, the slide area was regraded, and drainage was installed. Within two weeks the slope failed again (Scheper, 1973) in the area identified in Figure 24. That landslide mass was also regraded. In early 1975, a large landslide developed in the same general area (marked "early 1975" in Fig. 24), and extended headward to within a few meters of the house at 1036 Huffman Court. In the spring of 1980, the landslide enlarged laterally, toward the east, to incorporate part of the lot of the house at 1024 Huffman Court (Fig. 24).

Landsliding also occurred at the eastern end of Huffman Court. In the winter of 1971, a small slide developed in the fill slope between the house at 980 Huffman Court and the county access road along the creek (H.C. Nutting Co., 1979). In early 1975, a larger landslide had begun to develop (Norman Hall, oral comm., 1979 ; H.C. Nutting Co., 1979), damaging the house at 980 Huffman Court and the sidewalk and driveway at 984 Huffman Court. By 1980, the scarp of that landslide had developed to the extent shown in Figure 24, involving lots at 980, 984, 988, and 992 Huffman Court. To date there have been no corrective measures to halt this sliding.

Other parts of the face of the fill have slid at various times (Fig. 24), but the slides seem to be shallow. There has been a series of shallow landslides in the cut slope, between Compton Road and Huffman Court, and some of these have caused minor damage to houses on the south side of Huffman Court (Fig. 24).

Lake clay.

Most test borings in the Huffman Court area indicate that the bedrock surface is overlain by lake clays. A few borings suggest a thin veneer of till locally between the bedrock and lake clay. The lake clay is laminated and ranges in color from brownish gray at the top to bluish gray at the bottom.

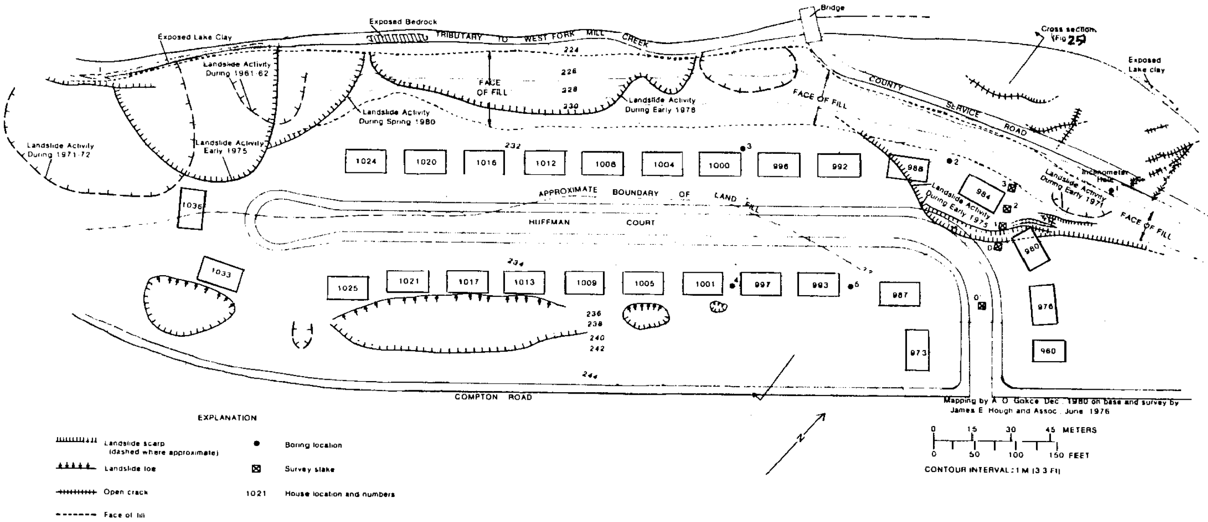


Fig. 24 Map of Huffman Ct. landslide complex. (Fleming, et al., 1981)

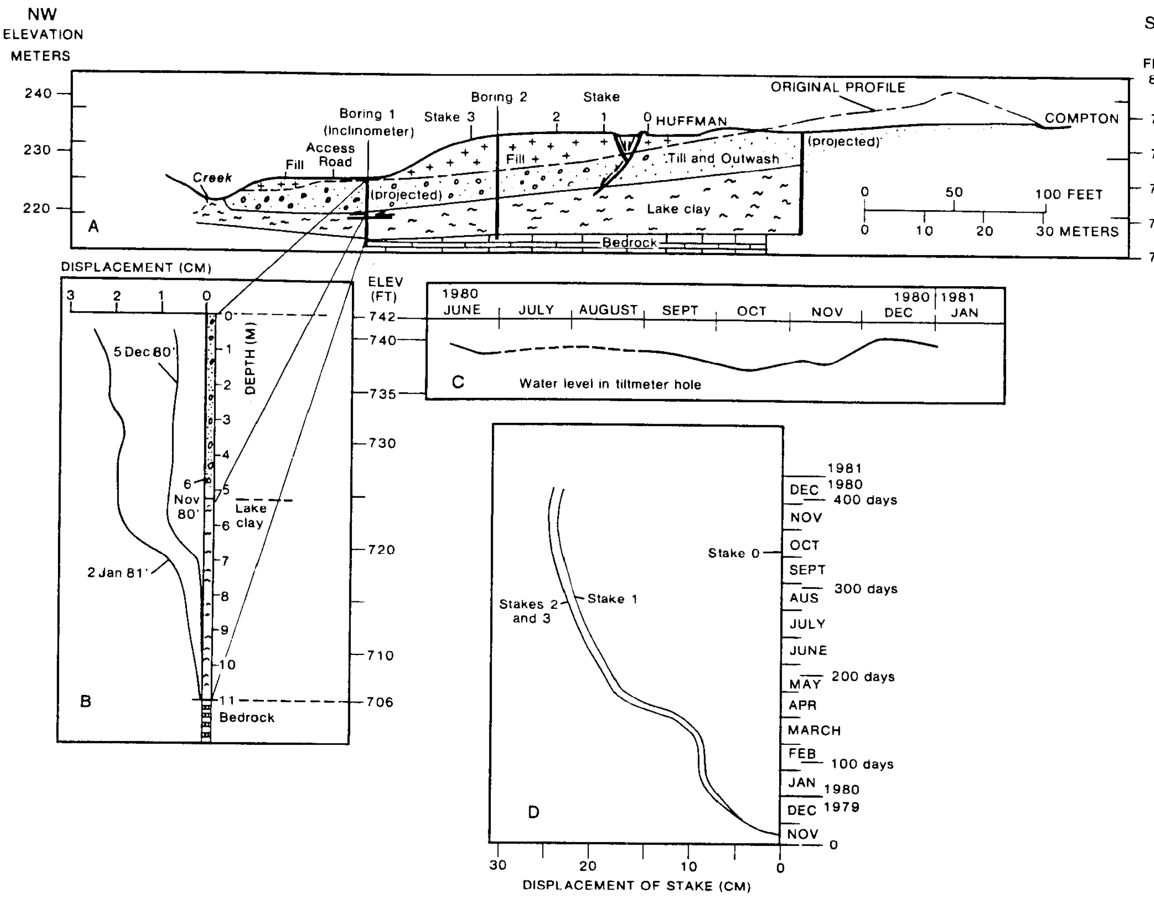


Fig. 25 Geologic cross section and displacement and water-level data for landslide on east end of Huffman Ct. (Fleming, et al., 1981)

Stop 18

McKelvey Road Landslide complex.

Stop at Paul farm where we will examine landslide features in lake clays.

(from Reference 18)

MCKELVEY ROAD LANDSLIDE COMPLEX, (Gokce, et al., in Reference 18)

McKelvey Road is 300-600 m east of Winton Road in Springfield Township. The location is shown in Figure 19. The geologic setting is essentially the same as that at Huffman Court, although the details of the buried valley of lake clays are not as well

Photo 35 Walking down to creek.

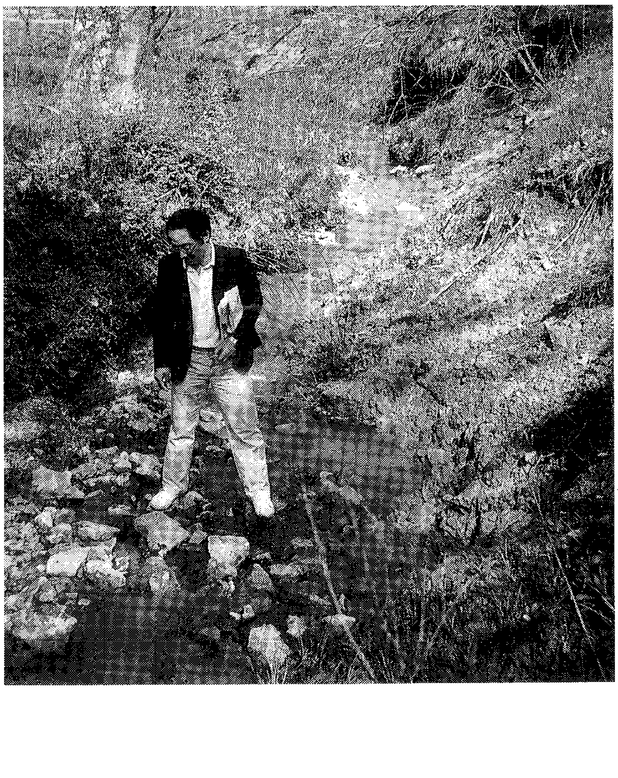


Photo 36 Outcrop of lake clay in the creek.

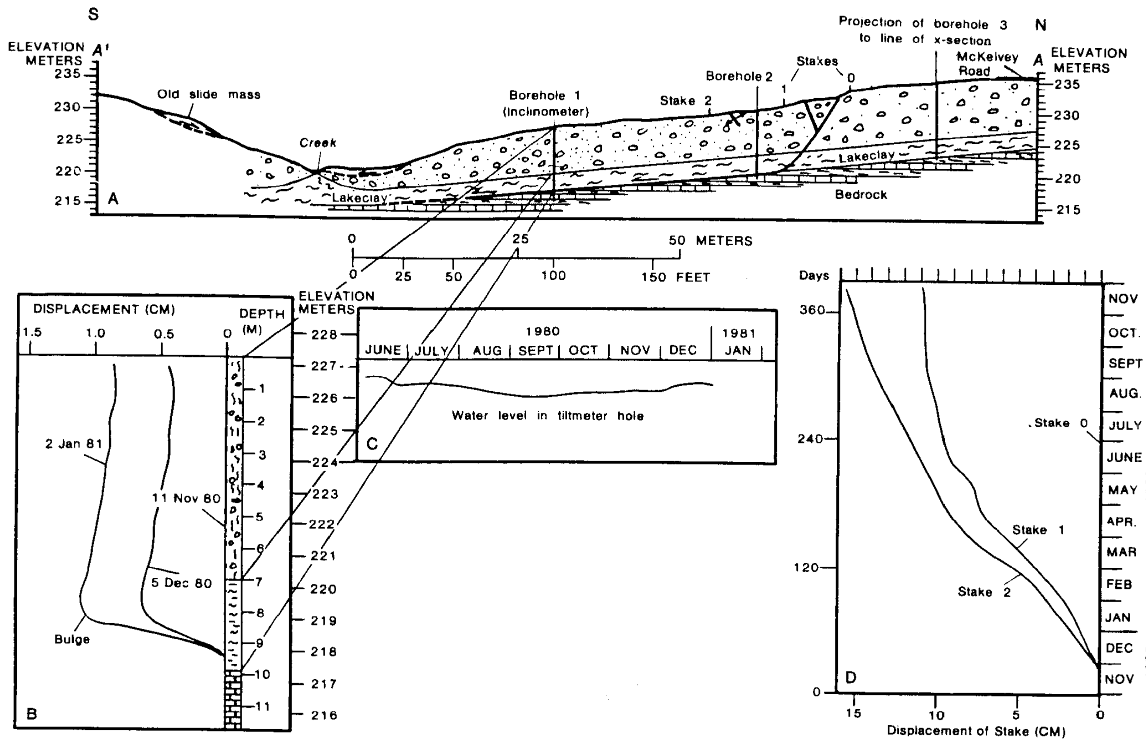


Fig. 26 Geologic cross section and displacement and water-level data for western part of Mckelvey Rd. landslide complex. (Fleming, et al., 1981)

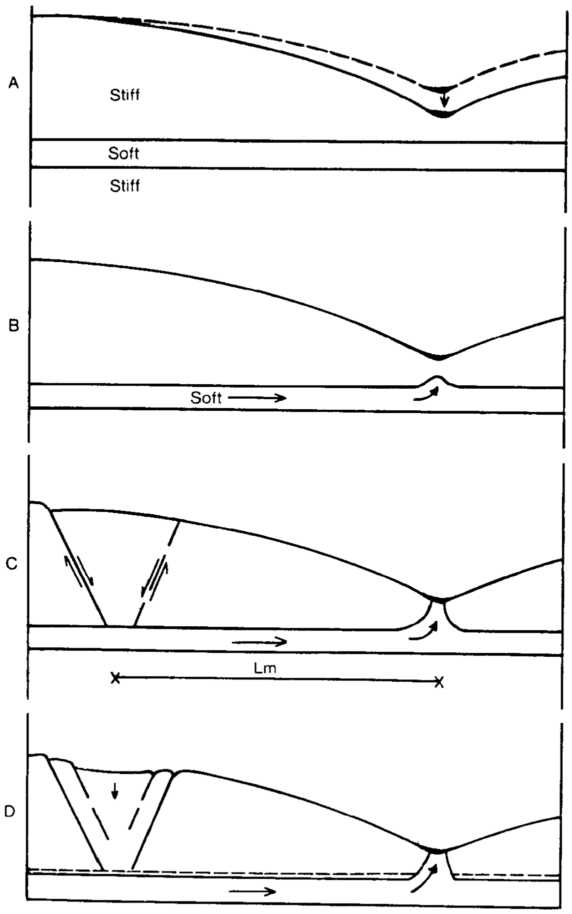


Fig. 27 Sketch of extrusion model. A - soft layer between stiff layers is unloaded by valley cutting. B - soft layer begins to extrude to valley floor. C - overlying stiff layer moves as a rigid block and graben forms at distance L_m (see text) from valley floor. D - continued movement produces more complex deformation in graben. (Fleming, et al., 1981)

known. Whereas the development of the Huffman Court area involved a large grading operation, that at McKelvey Road involved only minor grading for basements of houses and, in most instances, small fills for parking areas. The largest grading operation was the construction of an earthfill dam on the small creek about 100 m south of McKelvey Road. Another earthfill dam was constructed across a tributary creek in the extreme southeast corner of the study area.

Landsliding along McKelvey Road almost certainly is prehistoric. Topographic forms suggestive of old landsliding are present at scattered locations along the creek. Landsliding is known to have damaged structures by 1973, when a house, constructed in 1960, began to develop cracks in a floor slab and in foundation walls. Another house was damaged and repaired several times until January 1980, when disruption became so severe that repairs became impractical and the house was abandoned.

The hillsides along McKelvey Road are underlain by about 6 m of glacial drift, which, in turn, is underlain by about 4 m of glacial lake clays. The lake clays rest on bedrock of interbedded limestones and shales.

May 8, 1983

Roane County, Tennessee.

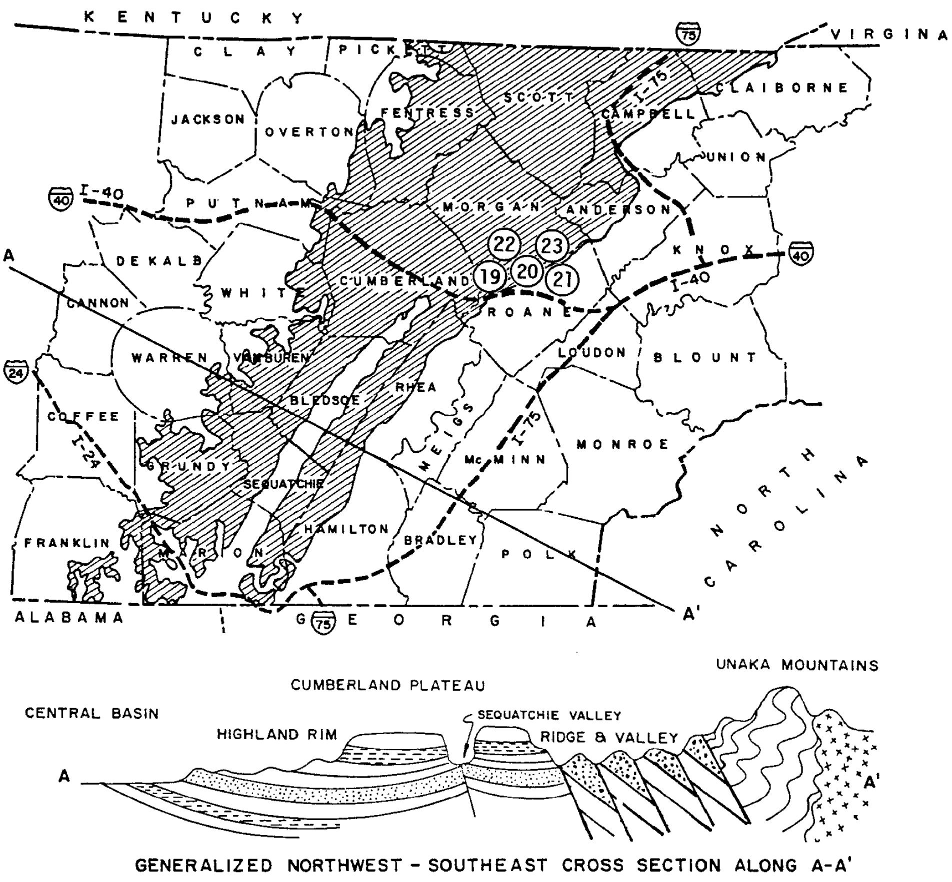


Fig. 28(a) Route Map, Roane County, Tennessee; Stops 19 - 23 along Interstate 40. (Reference 31)

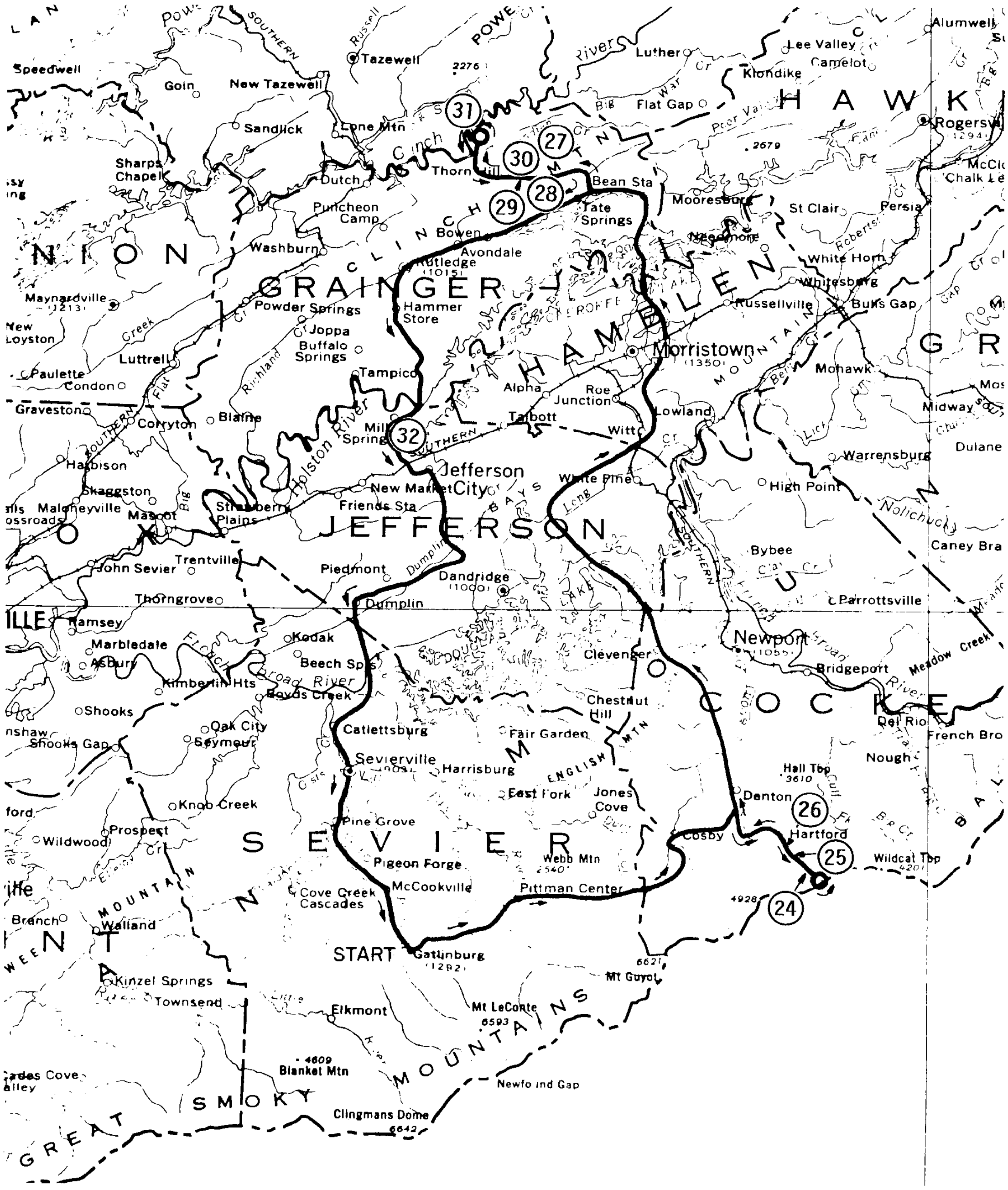


Fig. 28(b) Generalized map showing field trip route and approximate locations of stops 24 - 32. (Reference 26)

Interstate-40, Roane County : (Royster, 1977)

Without question, the most complicated and frustrating landslide problems in the history of highway construction in the State of Tennessee have occurred along a 6.4 km (4-mile) section of Interstate-40 near Rockwood (Royster, 1973). Construction on this segment began in late 1967 and, beginning with the failure of a massive fill in January 1968 along the east-bound lanes between stations 2003+00 and 2018+00, more than 30 slides had to be corrected before all four lanes could finally be opened to traffic in the late summer of 1974 (the west-bound lanes were opened to two-way traffic in December 1972). Remedial measures included partial to total removal ; minor grade and alignment changes ; various restraint devices, such as rock buttresses, gabion walls, a Reinforced Earth structure, and soil berms ; as well as various dewatering systems such as French drains, vertical wells equipped with automatically actuating pumps, and horizontal drains.

Stop 19

Fill Slide in Colluvium, Rockwood.

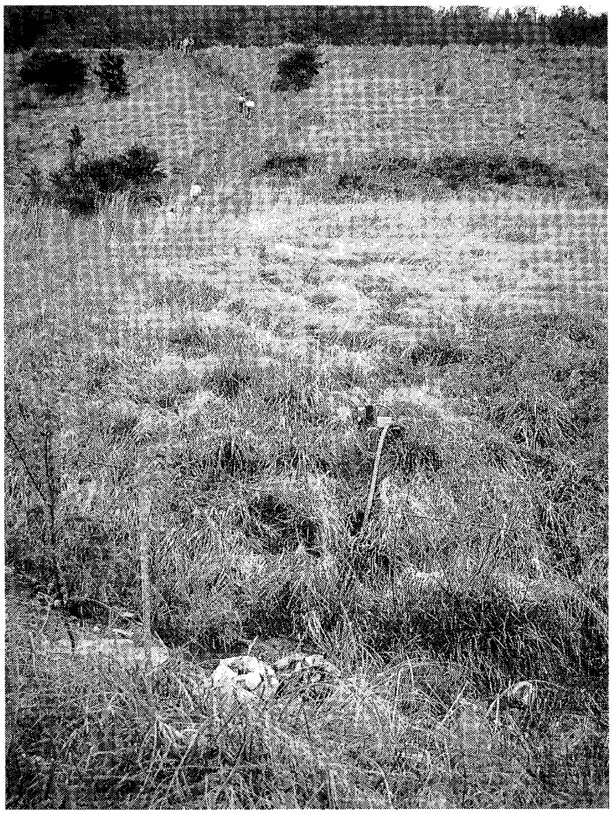


Photo 37 Looking up remedial slope of fill slide in colluvium.

Photo 38 Horizontal drainage.

Interstate-40, Roane County (Royster, 1977)

The most troublesome interval on the Rockwood project has been along the east-bound-lane between approximately stations 2003+00 and 2018+00. Construction began in this area in May 1967 with clearing and grubbing. Grading began in August 1967. Initial failure occurred in January 1968 with the embankment about 27m (90') high or 15m (50') below planned grade. Some adjustments were made in the alignment and gradient and shallow interceptor drainage trenches were constructed in an effort to stop the movement. With continuing movement into the spring 1968, however, a consultant was retained to investigate the failure and develop remedial measures. Several alternatives were developed and each analyzed as to cost and degree of effectiveness. The alternatives proposed included total removal and replacement, deep drainage with galleries and wells, drilled shaft restraint, relocation, and a minimal stabilization plan that included drainage trenches above and below the fill. Because of costs, the minimal stabilization plan was chosen even though the factor of safety was calculated to be only on the order of that of the original slope prior to failure. This meant that movement would probably continue, especially during wet periods, but it would be within tolerable limits and could be maintained by periodic patching of the pavement on each side of the slide area.

Stop 20

Reinforced Earth Construction.

Introduction (Royster, 1974)

In October 1973, approximately two months after the initial row of concrete panels was set, the first Reinforced Earth structure to be constructed along a highway project in the Southeast was completed. The structure, which is 831 feet long and 39 feet high at its highest point, spans one of the many landslide problem areas along Interstate 40 near Rockwood .

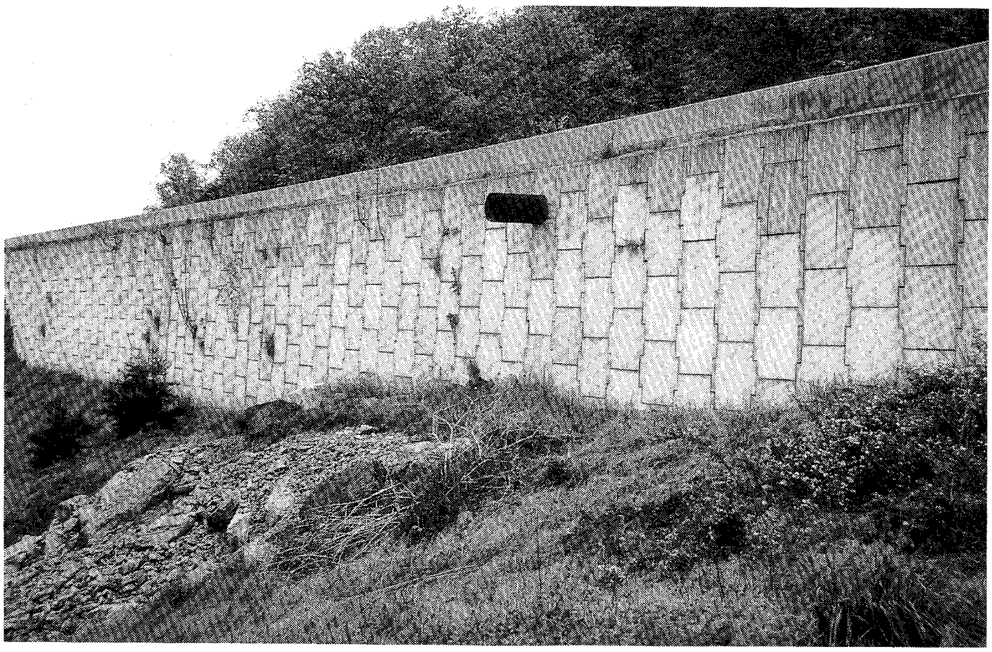


Photo 39 Reinforced earth wall.

The side, which occurred in January 1973, involved a section of sidehill fill between stations 2034+00 and 2042+00 along the east-bound lane that had been in place for approximately four years. Fortunately, this particular section of roadway had not been paved at the time of failure, Sliding was attributed to pore-pressure buildup in a zone that extended from the base of the soil fill into the foundation material, which consisted of colluvium overlying highly weathered shale. The weight of the fill over the years no doubt increased the consolidation along the colluvium-residuum interface. This resulted in a constriction of the drainage channels in this zone which impeded percolation, thereby creating the slow buildup of pore-pressure that ultimately produced the failure.

Slope inclinometers installed at about the center of the slide (station 2038+00) revealed the failure plane to be at a depth of 38 feet near the roadway shoulder, 22 feet at the toe of the fill, and 24 feet approximately 100 feet below the toe of the fill. As in virtually all the other slides investigated along this alignment, failure occurred along the colluvium-residuum contact (Royster, 1973).

Similar failures had occurred at three other locations along the project. These were corrected by removing the failed mass to below the shear zone, replacing the lower portion with a rock pad, and then carrying the fill to grade with soil. The rock pad, which is made up of buttress-type rock, acts as the foundation for the fill while at the same time serving as an outlet for subsurface drainage and as a buttress against further up-slope movement. This method, though quite effective, proved to be rather expensive, mainly because of the tremendous excavation quantities.

This prompted the Department to look at other alternatives.

In addition to Reinforced Earth, the alternates considered were relocation and a gabion wall.

As a result of a preliminary design and cost estimate prepared by The Reinforced

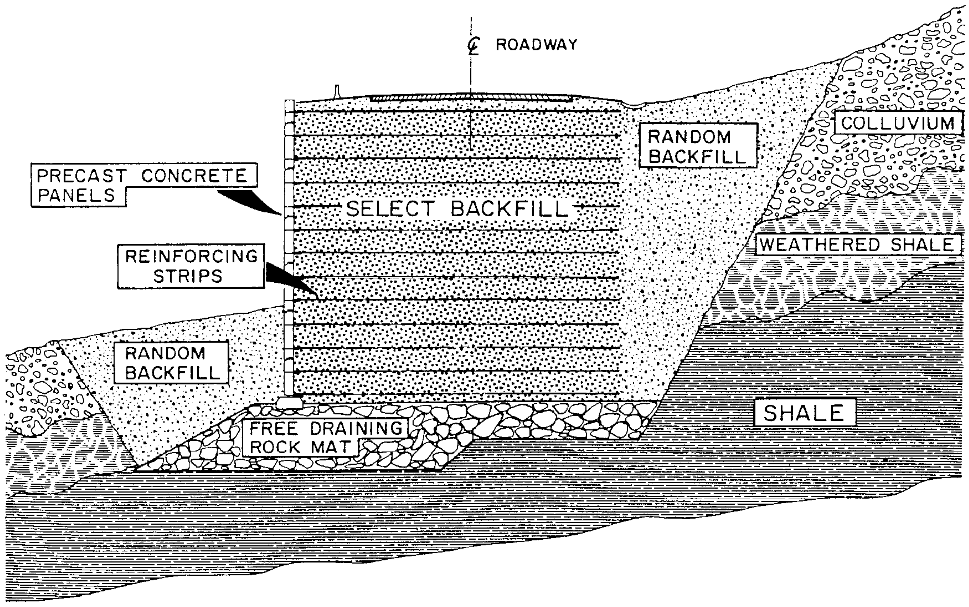


Fig. 29 A section view of the reinforced earth structure shown in Photo 39. # (Royster, 1982)

Earth Company, the Department decided that Reinforced Earth would be a feasible approach to the problem if a nearby source of granular material could be located. As it turned out, the Division of Soils and Geological Engineering was able to locate a very excellent supply of material just off the right-of-way within approximately one mile of the project.

Stop 21

Buttress and Horizontal Drainage.

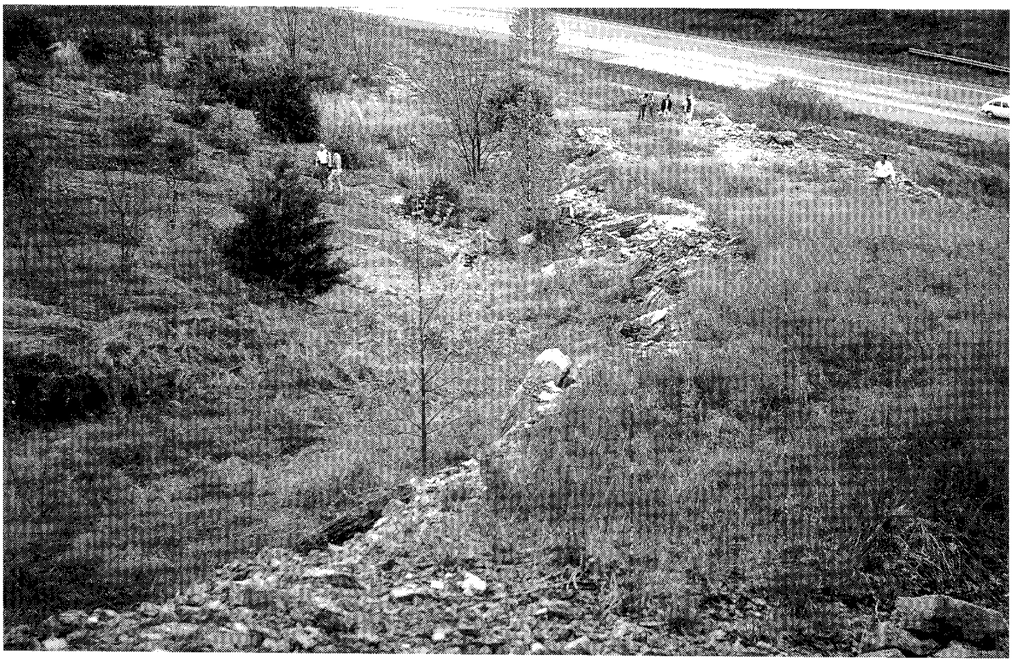


Photo 40 View from the right flank.

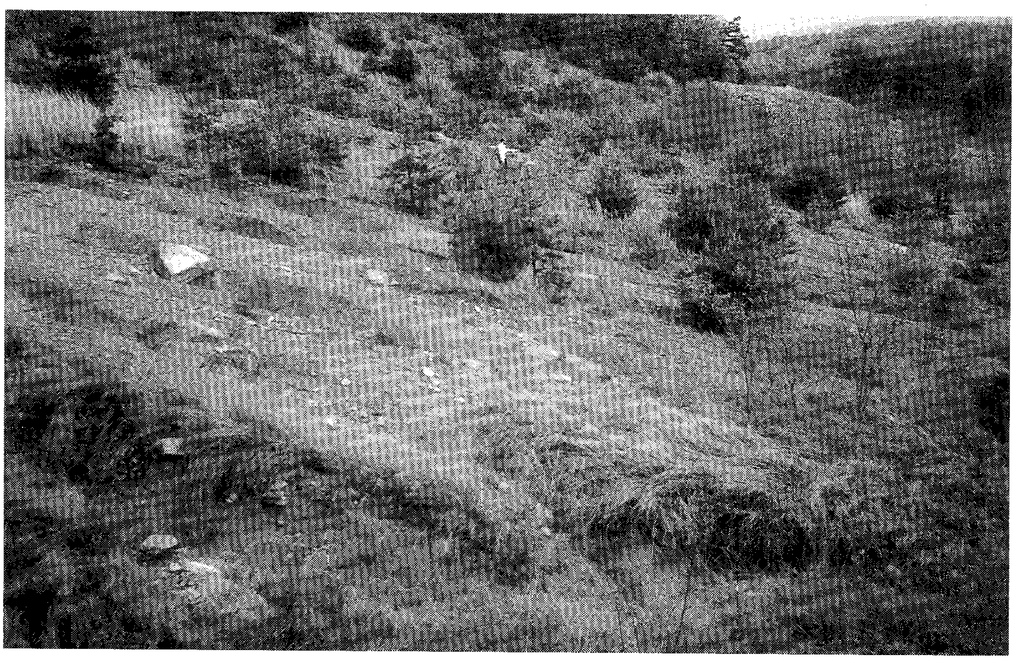


Photo 41 Moving zone.

Interatate-40, Roane County (Royster, 1977)

Horizontal drains were also used in an attempt to stabilize the right flank of a large horseshoe-shaped slide in which all of the slide material between the outermost scarps had been removed during repair in the fall of 1972. During the heavy spring rains in 1973, portions of the right flank (as viewed from the crown) began to break and flow laterally (Figures 30). As in most of the Rockwood slides, the failure involved colluvium moving over weathered shale. A total of 20 drains were installed to depths of 46 m (150') on 10% to 20% grades in the three slumped areas. The holes were randomly spaced with the locations of most being determined by the results from three or four test holes. Initial flows, while not spectacular, were on the order of .006 liters/sec to .019 liters/sec (.1gpm to .3gpm). The right flank has essentially stabilized over the years, and while it cannot be stated with certainty that the increased stability was totally a result of the drainage, it surely must have been a contributing factor.

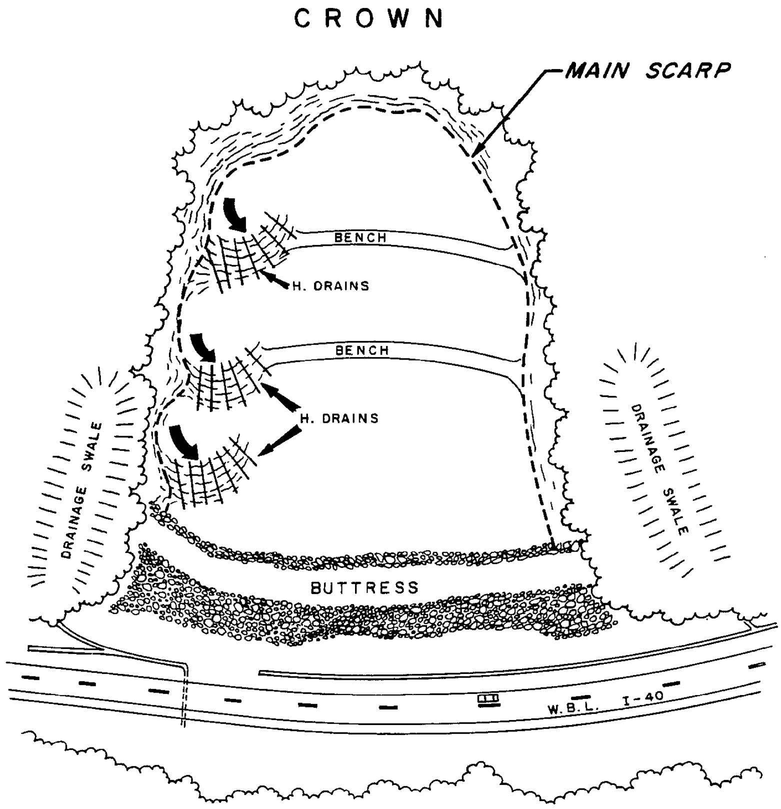


Fig. 30 Horizontal drains were used in an attempt to stabilize the right flank of this major slide at Rockwood. (Royster, 1977)

Stop 22

Gabion and drainage.

Photo 42 Gabion wall.

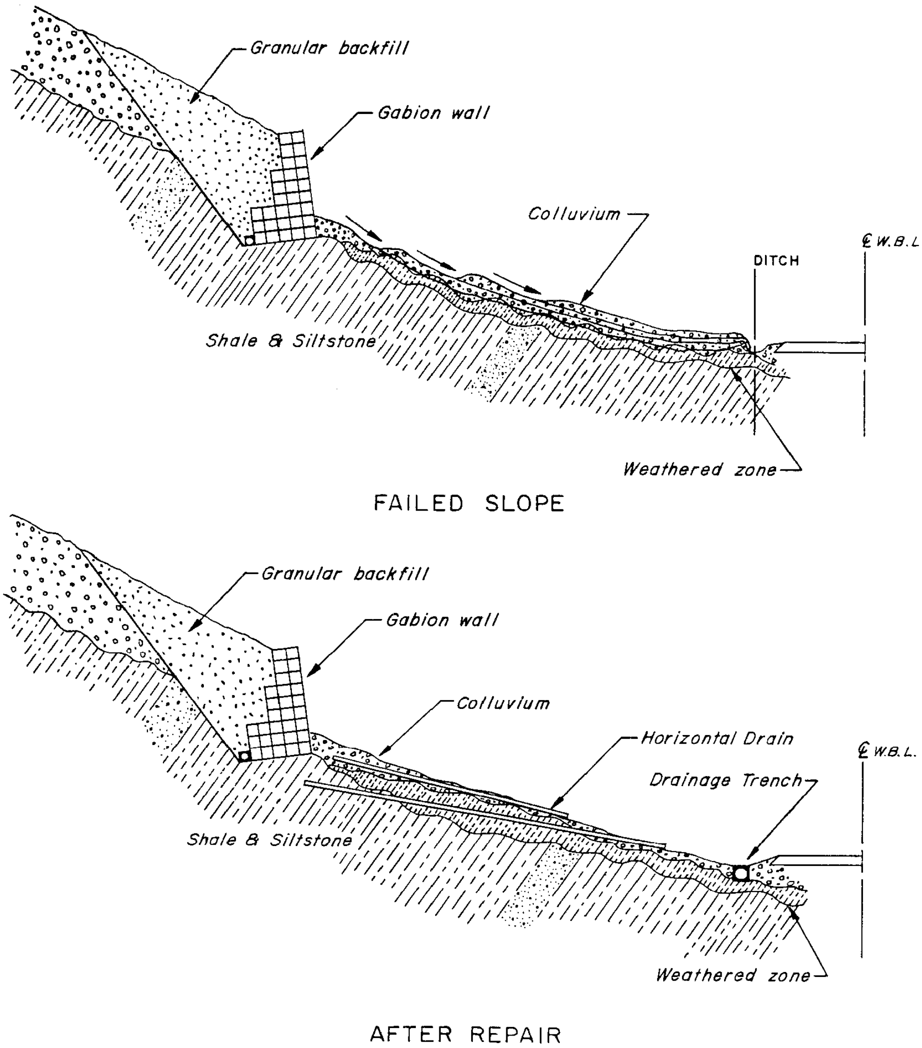


Fig. 31 Schematic views of slide 4-W at Rockwood before and after repair. (Royster, 1977)

Stop 23

Gabion and Fill to stabilize Edge of Road.

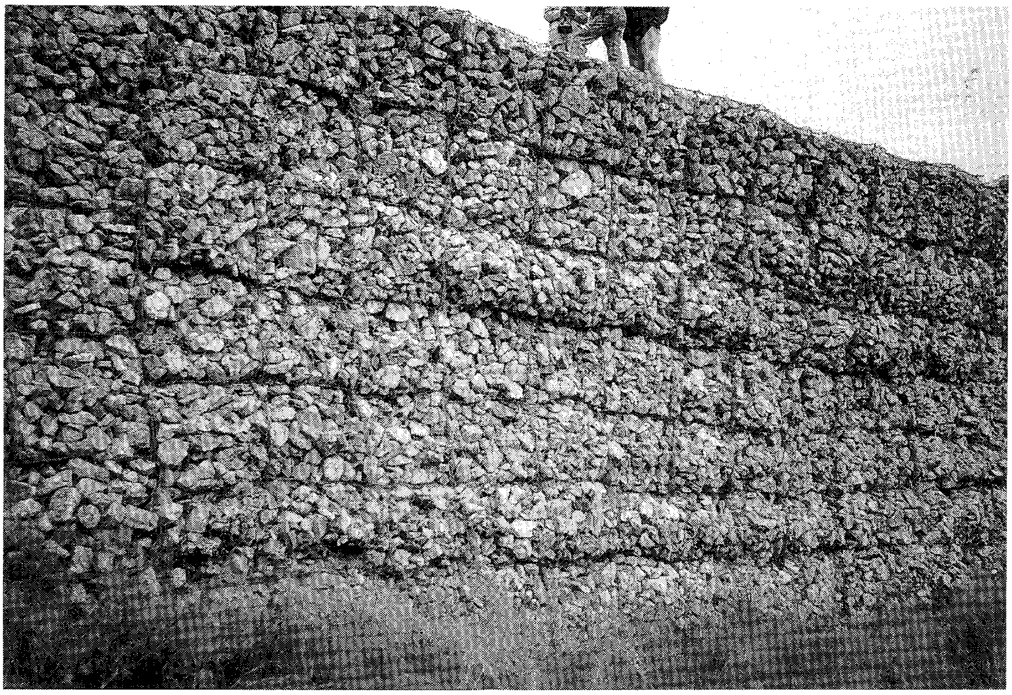


Photo 43 Gabion wall.

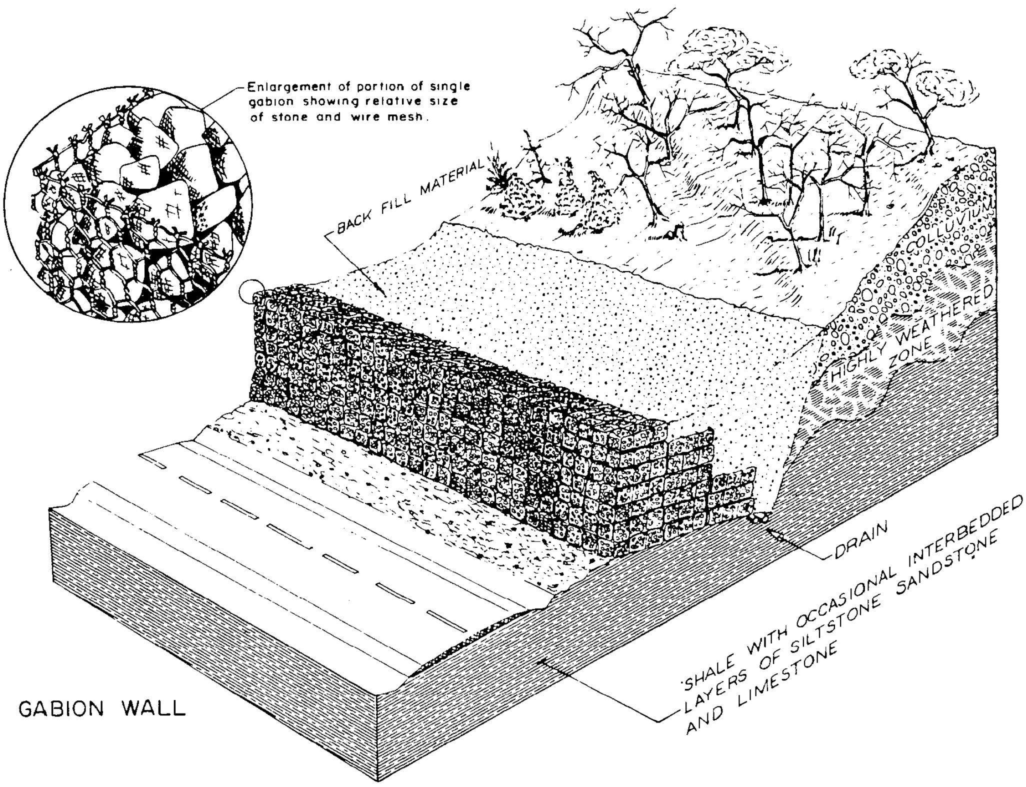


Fig. 32 Schematic of a gabion wall. (Royster, 1982)

May 9, 1983

Roane County, Tennessee.

Stop 24

Removal of soil, Waterville.

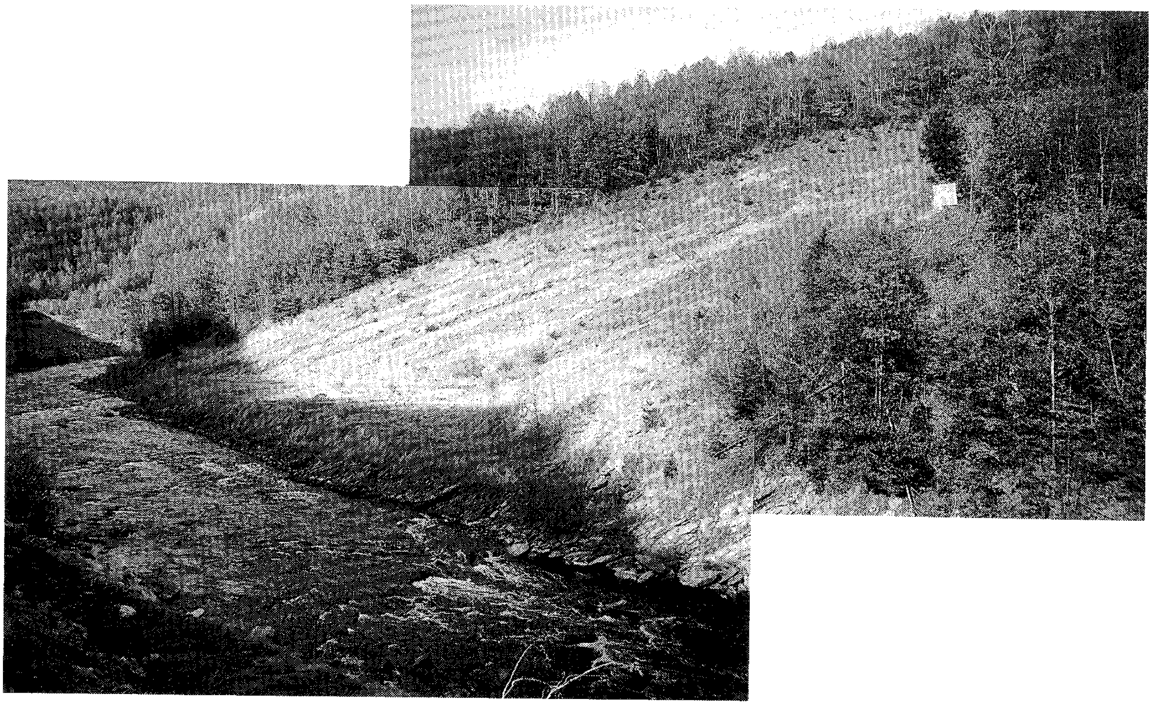


Photo 44 Removal of soil, Waterville.

Remedial Measures (Royster, 1982)

One of the larger slides occurred near the Waterville Interchange in the spring of 1972. It was actually the reactivation of a slide that had begun to develop in 1968 when the river channel was widened in connection with the placement of slope pavement. The initial failure developed during excavation of the river channel when the toe of a wedge-shaped deposit of soil and weathered rock was undercut near the river level. Minor movements, resulting in a slow but perceptible migration of material into the river, continued from 1968 through the winter of 1971-72. Then, in the spring of 1972, heavy rains caused a large segment of the stream to be diverted into the fill slope on the opposite side. This resulted in a lengthy section of the slope pavement being washed out. While this was being repaired in the fall of 1972, another flood occurred that caused further movement and the loss of most of the remaining slope pavement. In 1973, another contract was awarded to remove the slide, repair the damage to the fill slope, and to replace the slope pavement with boulder rip-rap. Restraint was not considered feasible because of the large quantity of material that would have had to be supported and bacause of the poor foundation conditions along the river's edge. Drainage was ruled out because of the infeasibility of controlling surface infiltration, as well as the improbability of removing enough water from the failure zone with horizontal drains to prevent further movement during periods of heavy rainfall. Approximately 124,000 cu/meters (162,000 cu/yds.) of slide material was removed at a cost of $2.88 per cu/meter ($2.20 per cu/yd.). The total project cost, including the buttress stone and boulder rip-rap used for fill slope protection, amounted to $2.25million. No additional problems have developed since completion of the project in June 1974.

Stop 25

Wedge failure.

Photo 45 Wedge failure.

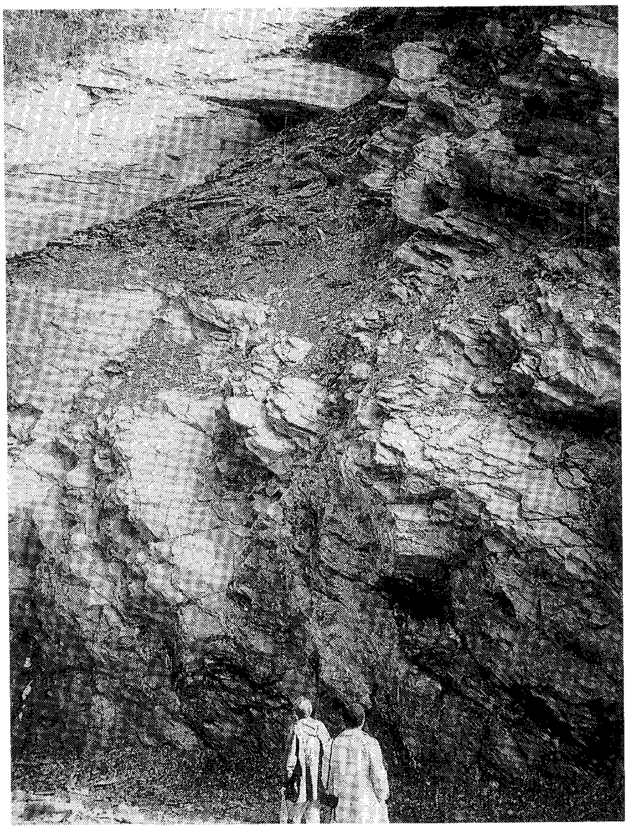


Photo 46 Wedge failure.

Road Log (Reference 26)

At this stop we will briefly review an area plagued with numerous rockfalls. In the past at least five rock slides have temporarily closed all or parts of all the traffic lanes of I-40, sometimes for up to 14 days.

Initial cut slopes along this section of I-40 undercut and exposed numerous unstable wedges of rock. These wedges are defined by pattern resulting from the intersection of bedding, joint and cleavage planes. After periods of freeze and thaw, heavy rains usually activate these masses of unstable rock resulting in rock and debris slides. These slides are classically known as wedge failures (Fig. 33).

These slides are developed in metasandstone and metasiltstone of the Roaring Fork Sandstone Formation. The rock units strike N 780 E with an average dip of 420 SE. As can be seen, cleavage is well developed in these rock units. The cleavage results in complex structural features which adds to slope instability.

In March of 1976 the Tennessee D. O. T. employed the services of Dr. D. R. Piteau, a rock slope engineering consultant, to make remedial recommendations for three specific slide areas along this section of I-40 in Cocke County. In addition, to Dr. Piteau's work, further study and detailed analysis of all the rock slopes in this section of I-40 was carried out by the Tennessee D. O. T., Division of Soils and Geological Engineering.

Appropriate remedial design concepts have been incorporated with photomosaics of each cut section. These photomosaics show clearly the location of specific design concepts.

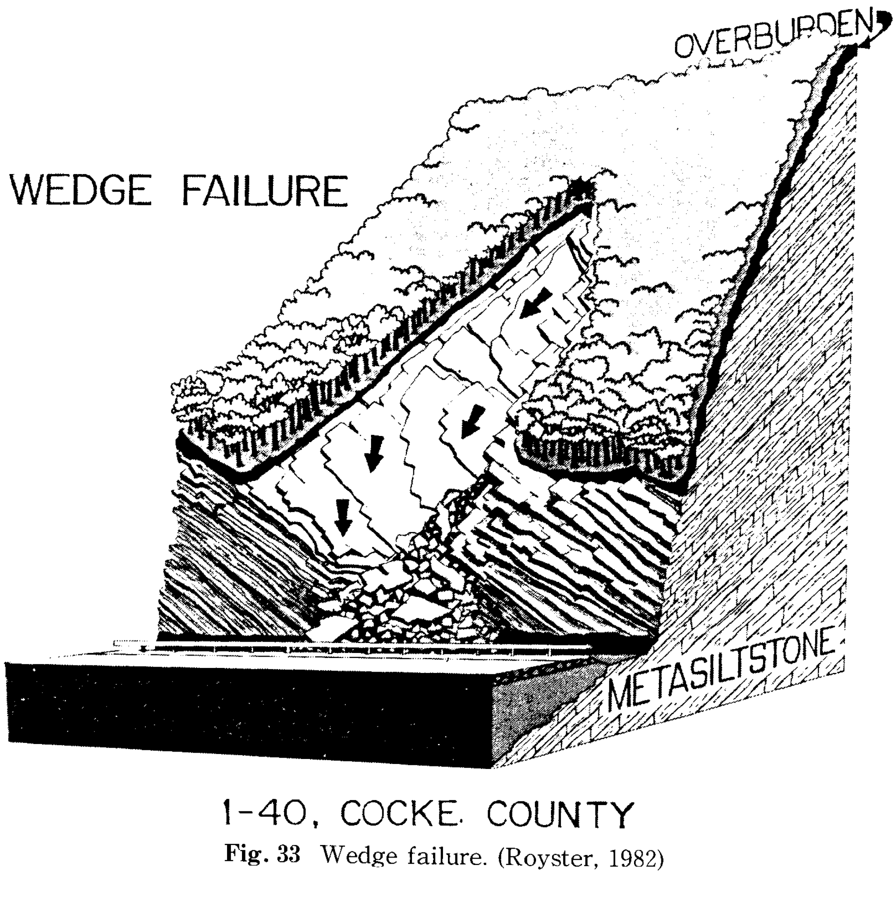


Fig. 33 Wedge failure. (Royster, 1982)

At present, detailed remedial workup is complete and includes such concepts as rock slope scaling & trimming, rock bolting, catchment fences, wire meshing, grouting, horizontal drains, and catchment ditches. Construction plans are in the offing with hopes of letting a construction project by late 1981.

Stop 26

Hartford Slide.

Hartford slide and horizontal drain installation ; Pigeon River Gorge on left.

Photo 47 Drainage pipe, Hartford slide.

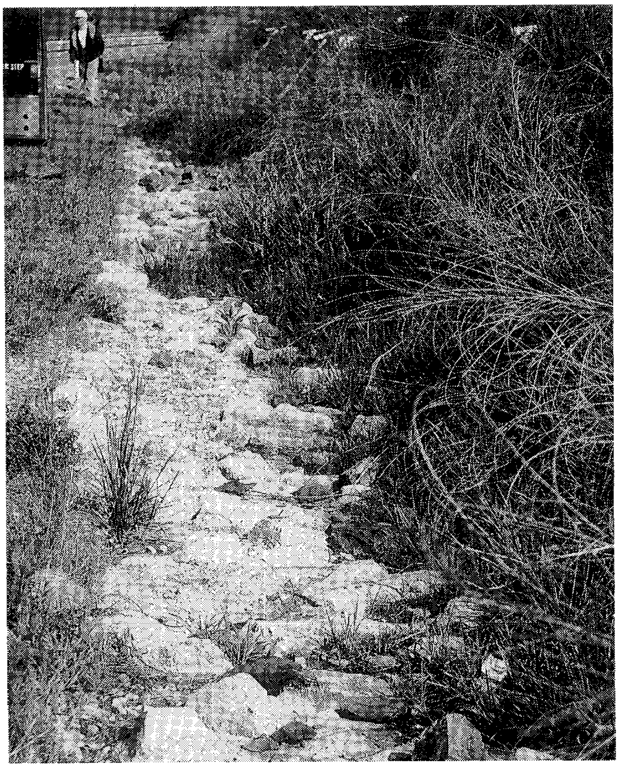


Photo 48 Drainage trough.

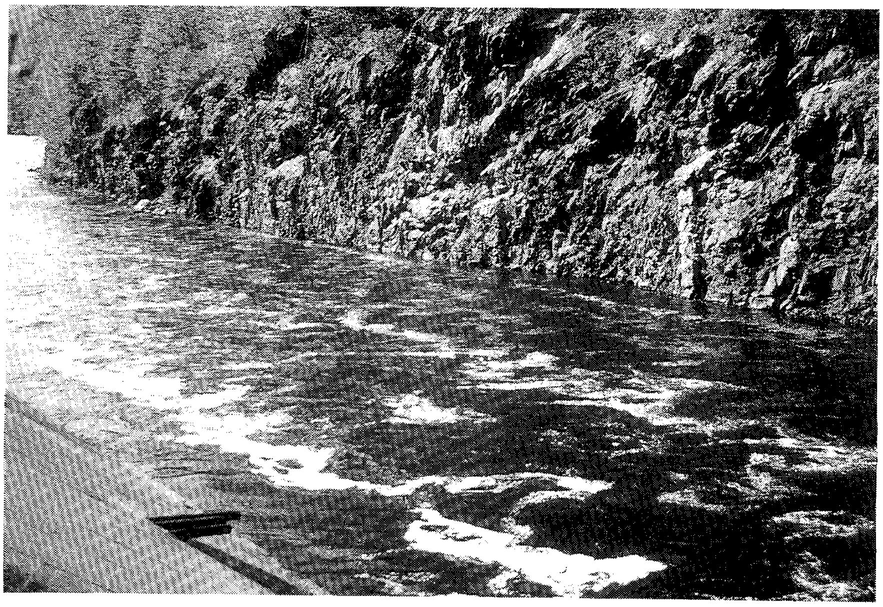


Photo 49
Horizontal drainage to the Pigeon River.

Road Log (Reference 26)

The Hartford slide, approximately one half mile east of the Hartford Interchange, is something of an anomaly in that, so far, only the westbound lanes have suffered damage and that relatively minor ; yet an estimated 2.25 million cubic yards of disturbed material are involved in the slide mass.

There is considerable evidence that the slide is a very old creep type movement that predates highway construction. The slide involves rock strata mapped by Hadley (1963) as Roaring Fork Sandstone and consists mostly of metasiltstone and metasandstone which are extremely broken and fractured.

Deep core borings show evidence of weathering to a depth of 180 feet indicating extremely broken or shattered strata possibly as a result of faulting. Movement is occurring at depth in a thick zone of badly weathered rock and soil (Fig. 34). A rock line that rises beneath the roadway in an east-west direction confines the slide mass to a "V" shaped trough. The solid rock line also rises toward the Pigeon River to a point near the surface beneath the median. This accounts for the previously perplexing fact that the eastbound lanes were not disturbed.

When highway construction began in 1962, the cut adjacent to the westbound lanes experienced toe movement during initial excavation. In June of 1962 work was temporarily disrupted by a slide that dumped material into the Pigeon River. Shortly after the highway was brought to grade, in 1964, the westbound lanes at the cut where toe movement occurred earlier began to rise vertically. Since 1964, continous maintenance involving regrading and resurfacing of the disturbed interval has been necessary to keep the westbound lanes operable.

Since the slide mass is confined, it is believed that water pressure plays a leading role in uplifting or "blistering" the roadway surface. Hydrostatic pressure is generally conceded to be the single most important activating agent.

In the fall of 1976 plans were finalized for the installation of 24,650 feet of horizontal drains. These drains were to be installed at three different levels within the slide (Fig. 35), and were scheduled to penetrate from 200-300 feet.

By early 1977 a contract was let and drilling began. Problems in drilling through the broken rock zone necessitated canceling the contract. In January of 1979 a new contractor was secured to complete the horizontal drain installation. By August of 1979, all drains had been installed.

Initially, most of the drains produced very heavy flows, especially those along the river. It is felt that this intensive dewatering has greatly aided in stabilizing the slide.

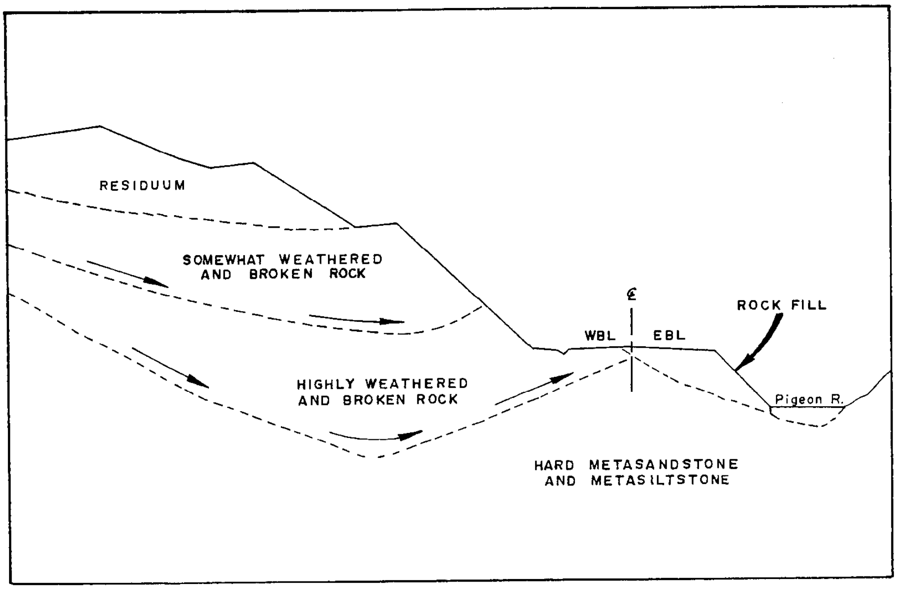


Fig. 34 The translational slide near the Hartford Interchange along Interstate-40 involved highly weathered and broken rock moving over essentially unweathered meta sediments. (Royster, 1982)

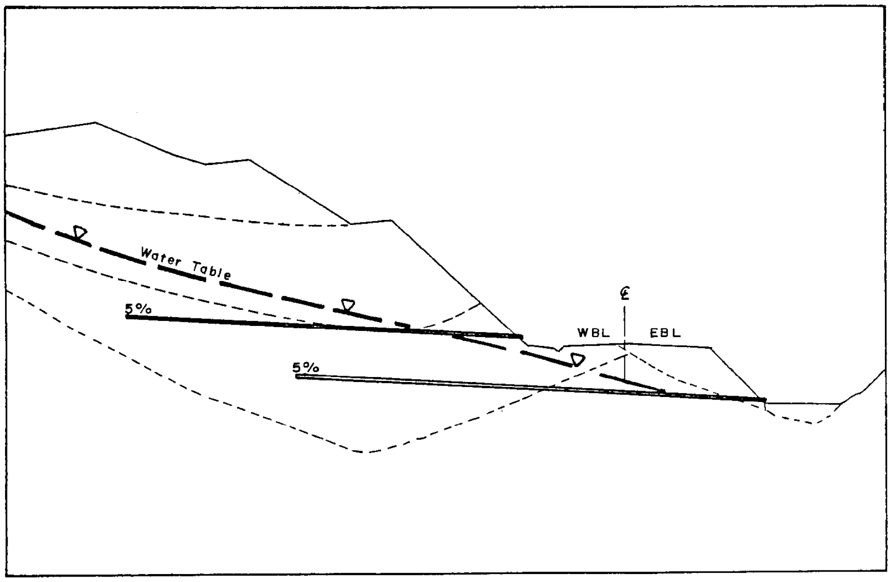


Fig. 35 Schematic showing the drainage levels in the Hartford slide. (Royster, 1982)

Stop 27

Rock Buttress remedial concept on right.

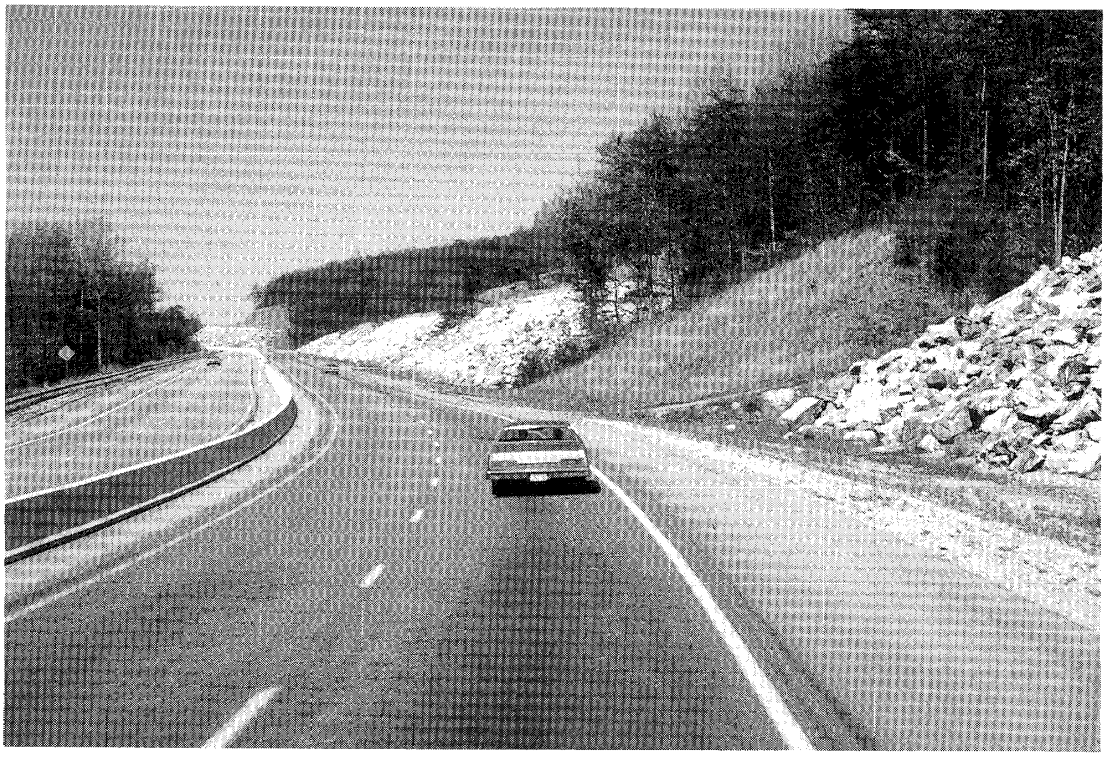


Photo 50 Rock buttresses.

Photo 51 Rock buttresses.

Road Log (Reference 26)

At this stop, we will review the geological problems and the associated remedial concepts employed for the construction of Appalachian Corridor "S" along the south side of Clinch Mountain.

For the entire 5.46 km (3.39 miles) section of the roadway on this side of the mountain, the alignment is typically in sidehill cut and fill in which the rock strata dip into the roadway on the uphill side of the alignment. This condition poses a serious stability problem in both cut and fill sections throughout the southeast slope. The massive Clinch Sandstone predominates in the interval Station 272+00 to 373+00 (south side of mountain up Beans Gap), but interbedded shale and clay seams present planes of instability throughout this interval (Fig. 36).

Fig. 38 illustrates the geotechnical design for embankment construction and Fig. 39 defines the cutslope design as recommended for the south slope of Clinch Mountain.

Construction on the south side of Clinch Mountain began September 13, 1976. The colluvium in ravines was stripped and replaced with free draining select sandstone from cut excavations. Toe benches were secured in stratified material. Rock drainage pads were added in the shale sections where none were scheduled. Shale embankment slopes were flattened as much as the right-of-way would permit (from 1.5 : 1 to 1.8 : 1). Cut slopes in sandstone units were pre-split vertical with a 12.2m (40') wide in-slope bench located 5.18m (17') above grade. For the larger cuts, a 15.25m (50') lift with a 9.15m (30') wide bench at the top continues above the first bench. Above the 9.15m (30') bench a vertical lift is carried through the natural slope intersect. Finally, a 3.05m (10') wide catchment area at grade is incorporated into the design.

During the winter of 1977-1978, planar block glide failures occurred in two of the largest sandstone cuts on the project (Fig.40). By the spring of 1978 planar failures of some magnitude were precipitated in virtually all shale backslopes and in several of the sandstone backslopes.

In some cases slope flattening was a viable remedial measure for the cut failures, but for most of the failures, either a rock buttress or a shot-in-place rock buttress was required for stabilization of the backslope (Fig. 41). Fill sections have not shown any movement to date. The project on the south side of Clinch Mountain was completed July 10, 1980, at a cost of approximately \$10,500,000.00.

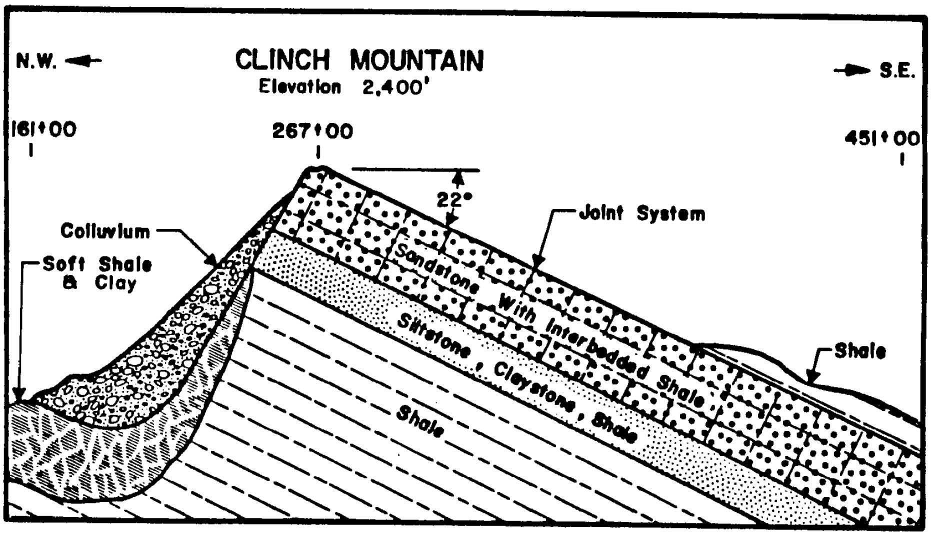


Fig. 36 Schematic drawing through subject section of State Route 32 - Appalachian Corridor "S" across Clinch Mountain showing conditions requiring compensatory design for unstable conditions - colluvium overlying soft shale, steep slopes, adverse dips, high angle joint systems, weak shale seams interbedded with thick more competent sandstone strata. (Aycock, 1978) (Reference 26)

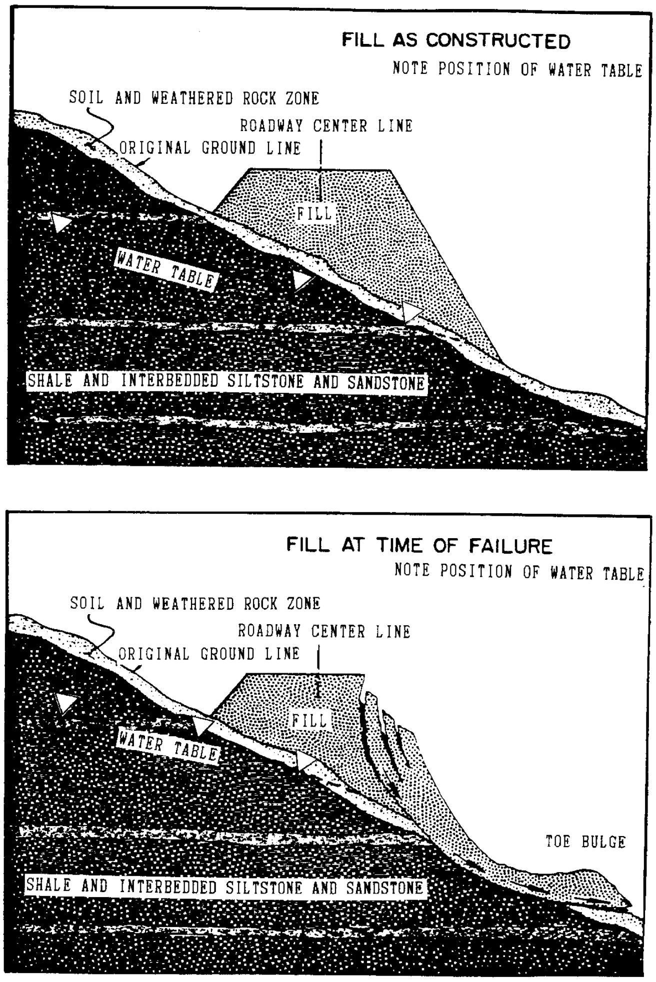


Fig. 37 Conditions leading to the failure of a highway embankment such as occurred along I-40 in Roane County and I-75 in Campbell County. Similar conditions exist along the north side of Clinch Mountain. (figures from Royster, 1973) (Reference 26)

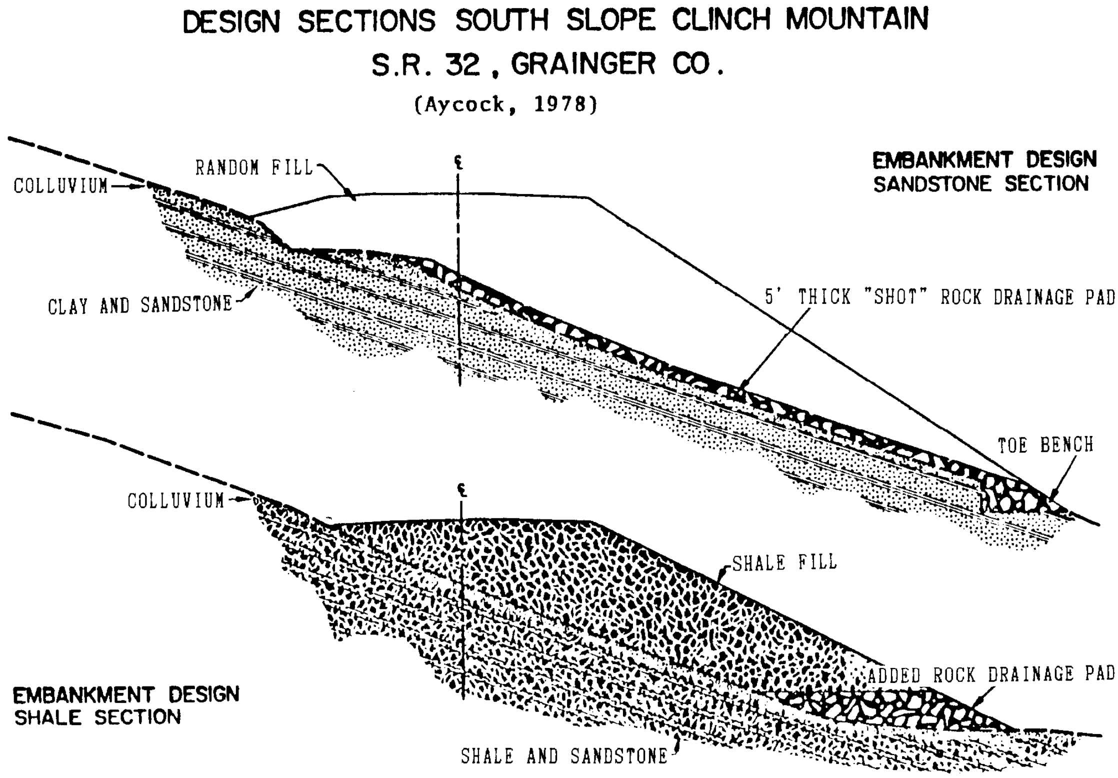


Fig. 38 Design sections south slope Clinch Mountain. (continued) (Reference 26)

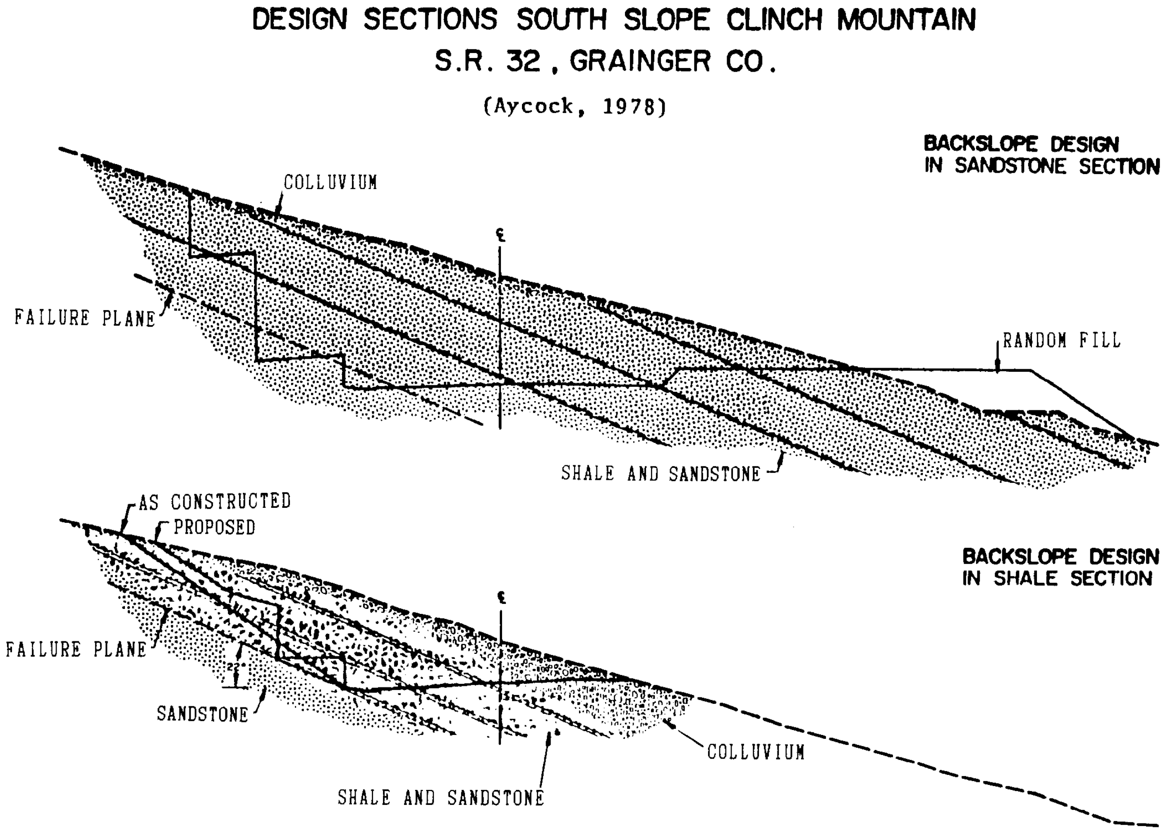


Fig. 39 Design sections south slope Clinch Mountain. (Reference 26)

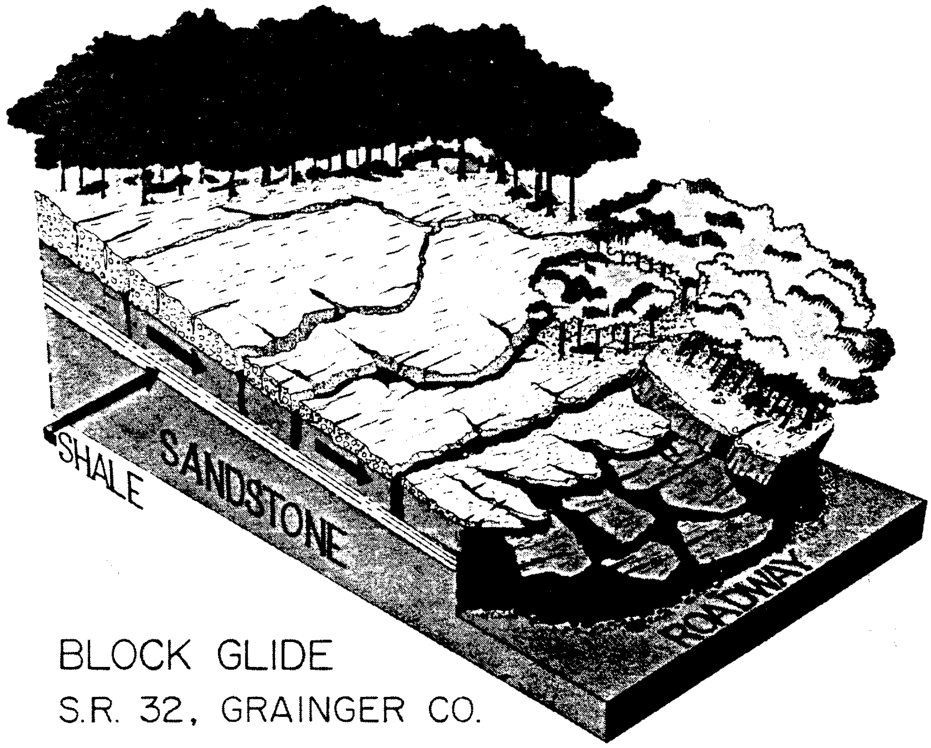


Fig. 40 Block slide. (Royster, 1982)

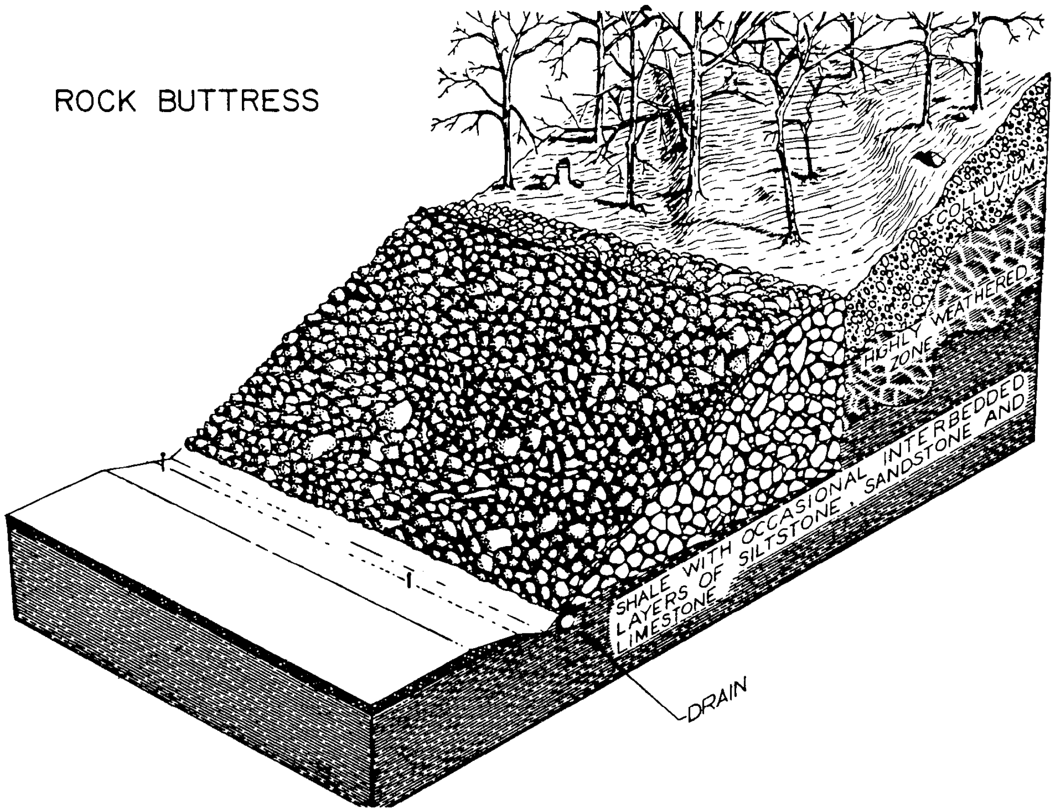


Fig. 41 Schematic of a rock buttress. (Royster, 1982)

Stop 28

"Shot-in-place" Buttress.

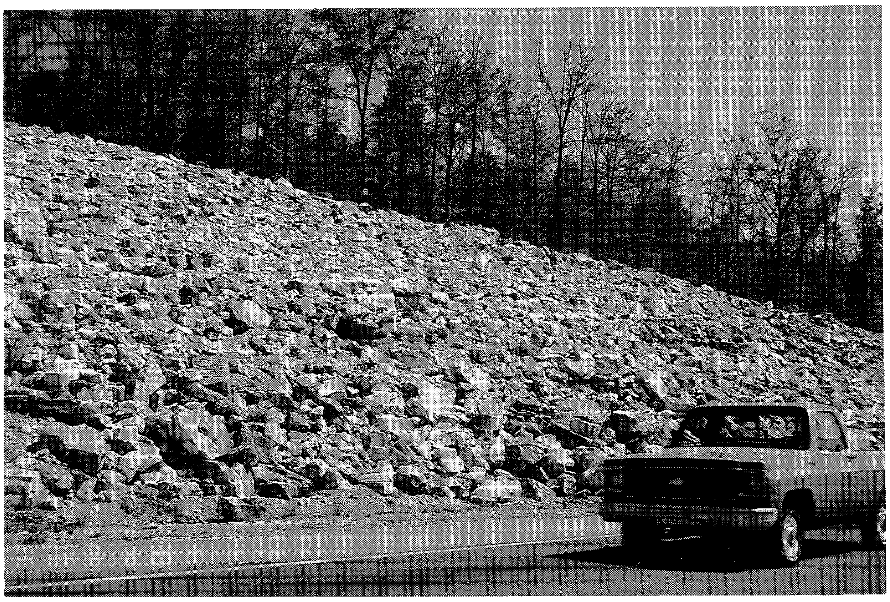


Photo 52 "Shot-in-place" buttress.

Remedial Measures (Royster, 1982)

Something that was tried in the stabilization of several of these failures (Fig. 42 and 43) is what might be termed a "shot-in-place" buttress. The idea was to "relax the slope" by breaking up the bedding planes along which sliding was taking place and use the "shot rock" to act as a buttress against further upslope movement. Shot holes were drilled on staggered 2 m (6') centers to varying depths, none of which exceeded depths that extended below the lowermost strata exposed in the ditch. This was done to minimize the amount of surface and subsurface water that might enter bedding planes and other discontinuities extending below the roadway. The depths of the shot holes were varied to prevent the development of "construction planes" along which additional sliding might occur (fig. 44 and 45). The size of the "shot-in-place" buttress was determined in the same way as that of a placed buttress. It was designed to provide a factor of safety against upslope movement of approximately 1.30. The ϕ of the "shot" material was determined to be 38°, with a unit weight of 2243 kgs/cu meter (140/pcf). Cohesion of the "shot" mass was considered to be 0. The 20% ± increase in volume ("swell") was used as buttress material for similar failures in shale cuts at other locations on the project. The initial results of this treatment look good, but only time will provide the true test.

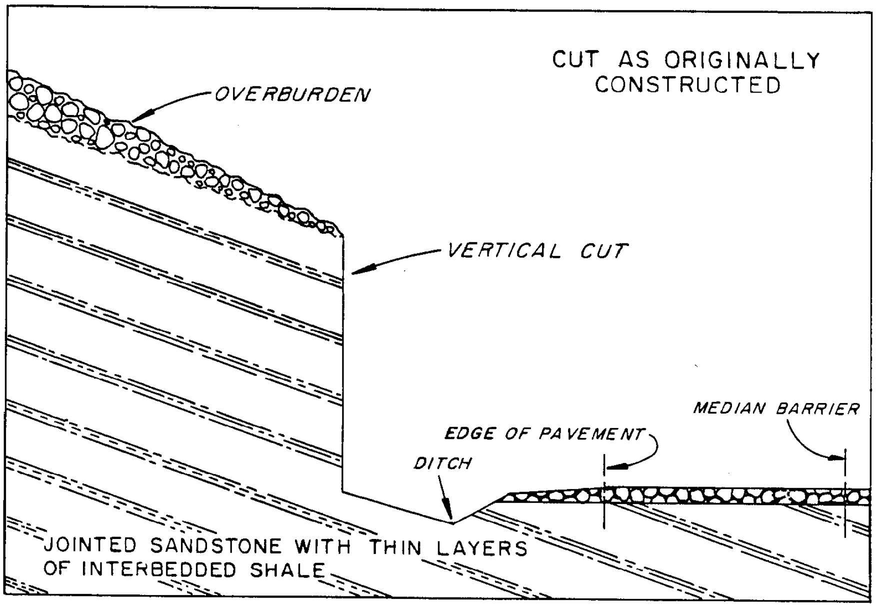


Fig. 42 Cut as originally constructed. (Royster, 1982)

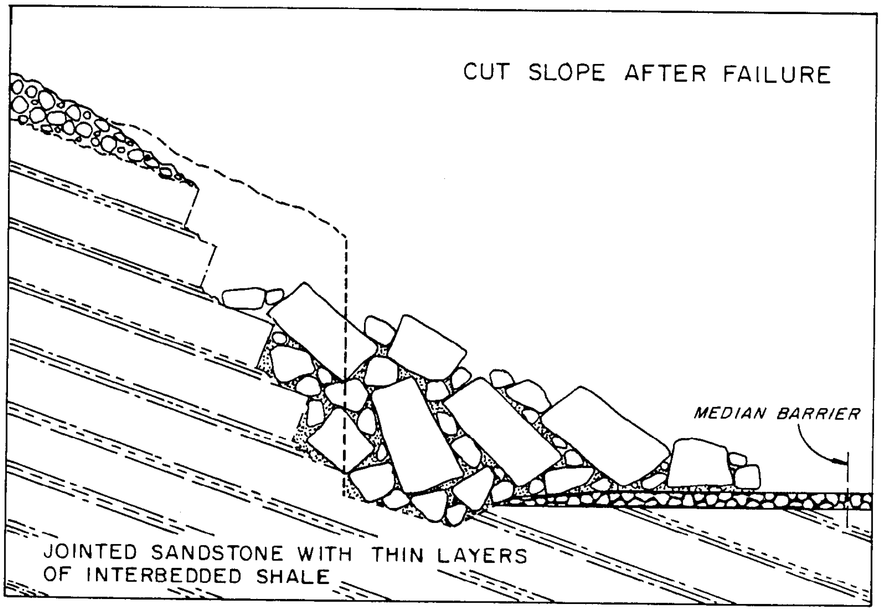


Fig. 43 Cut slope after failure. (Royster, 1982)

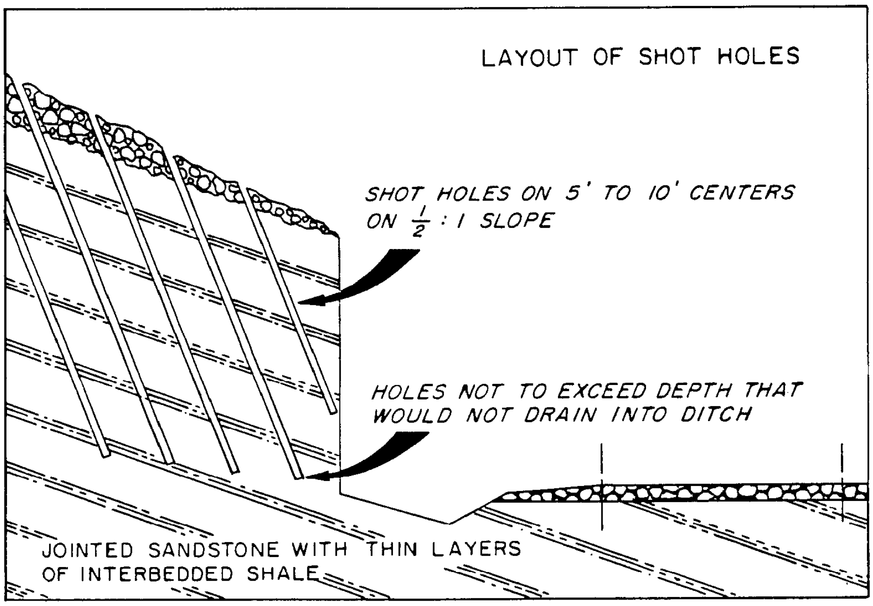


Fig. 44 Layout of shot holes. (Royster, 1982)

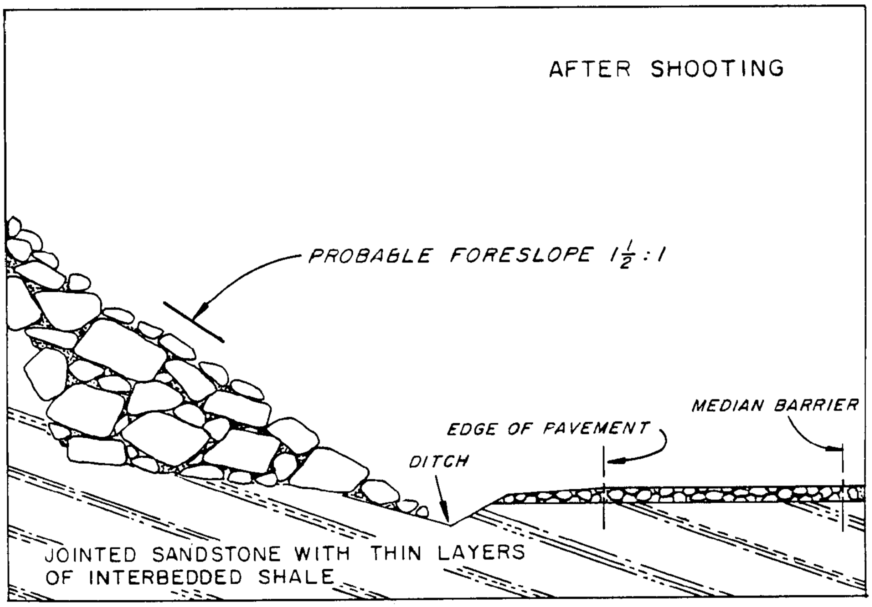


Fig. 45 After shooting. (Royster, 1982)

Stop 29

Scenic overlook of Appalachian Mountains.

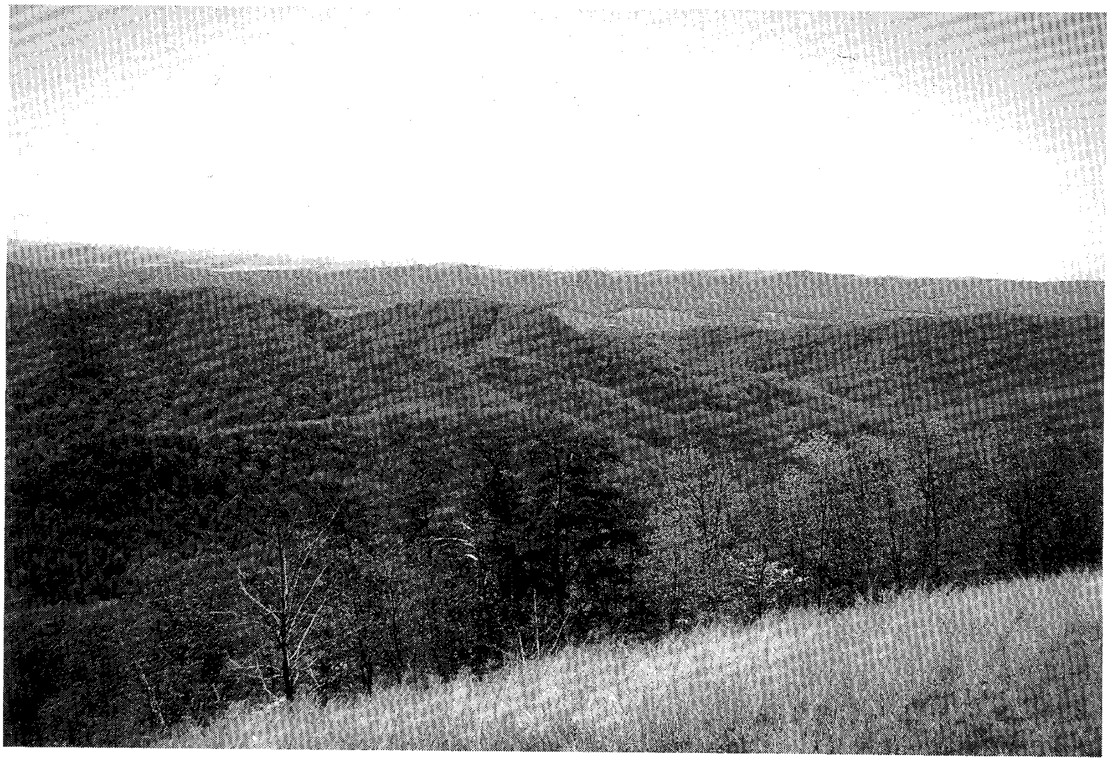


Photo 53 View of Poor Valley.

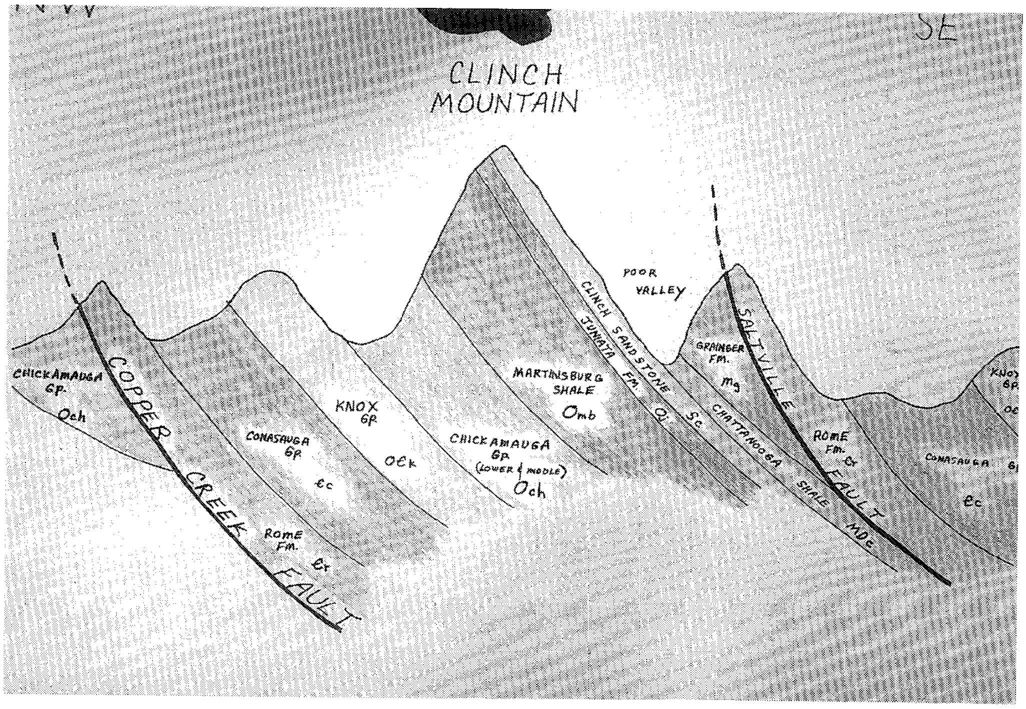


Photo 54 Schematic cross-section of structure of Clinch Mountain. (by Harry Moore)

Road Log (Reference 26)

At this site we can view, weather permitting, a section of the Ridge and Valley predominantly underlain by folded and faulted Cambrian and Ordovician strata.

The valley just below the overlook, locally known as Poor Valley, is underlain by the Devonian-Mississippian Chattanooga Shale. This formation weathers to a very thin, acid rich, unproductive residual soil giving rise to the name Poor Valley.

Stop 30

Beans Gap.

Photo 55 Beans Gap.

Road Log (Reference 26)

Across the road from the scenic overlook is Beans Gap located at the crest of Clinch Mountain. The new roadway excavation has exposed strata of the Clinch Sandstone, Juniata Formation and the Martinsburg Formation. These exposures illustrate the Ordovician and Silurian contact.

The initial design and construction of the cut interval at Beans Gap provided for 0.5 : 1 pre-split backslopes with in-slope benches at 20' to 30' vertical intervals. After initial excavation of the cut section, the rock strata, which is well jointed and badly weathered, began to break up in small wedges of rock that were precipitated onto the subgrade surface. Due to the serious rockfall problem that had developed, the backslope was redesigned and constructed on a straight 1 : 1 slope with a 40' wide fallout area at the ditchline.

Besides being arthrophycus-bearing, the Clinch Sandstone exposed at Beans Gap also contains "trilobite resting tracts" in the basal units. These can be observed behind the restaurant at Beans Gap (permission is suggested).

Photo 56 Faulted Martinsburg Strata.

Stop 31

Profile of faulted Martinsburg Strata. (Photo 56)

Massive exposure of faulted Martinsburg strata. (Reference 26)

Stop 32

Cherokee Dam.

Expansive view of Cherokee Lake.

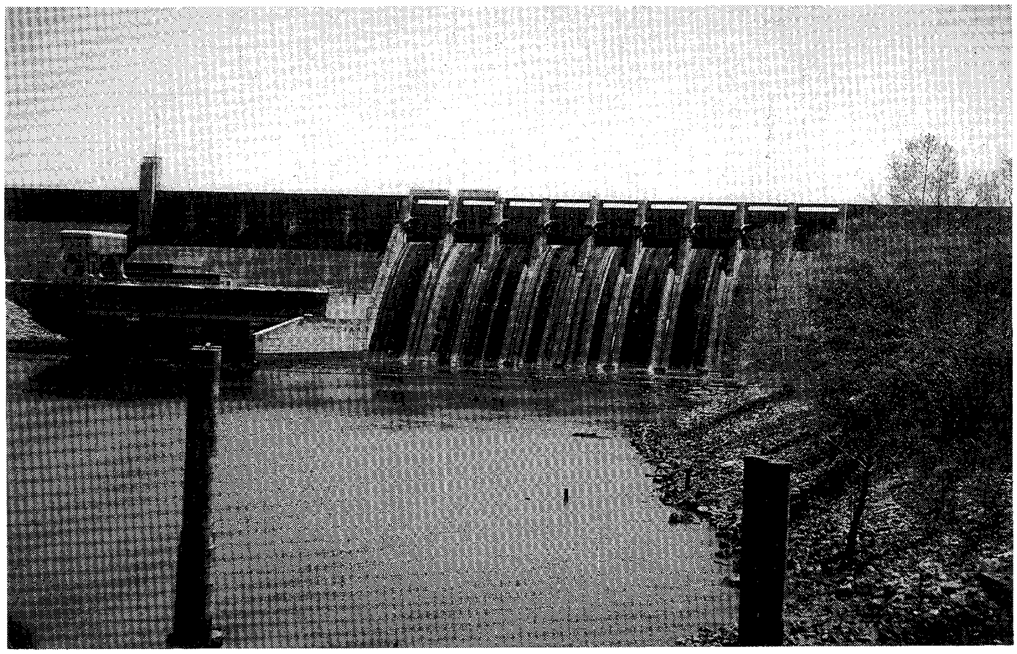


Photo 57 Cherokee Dam.

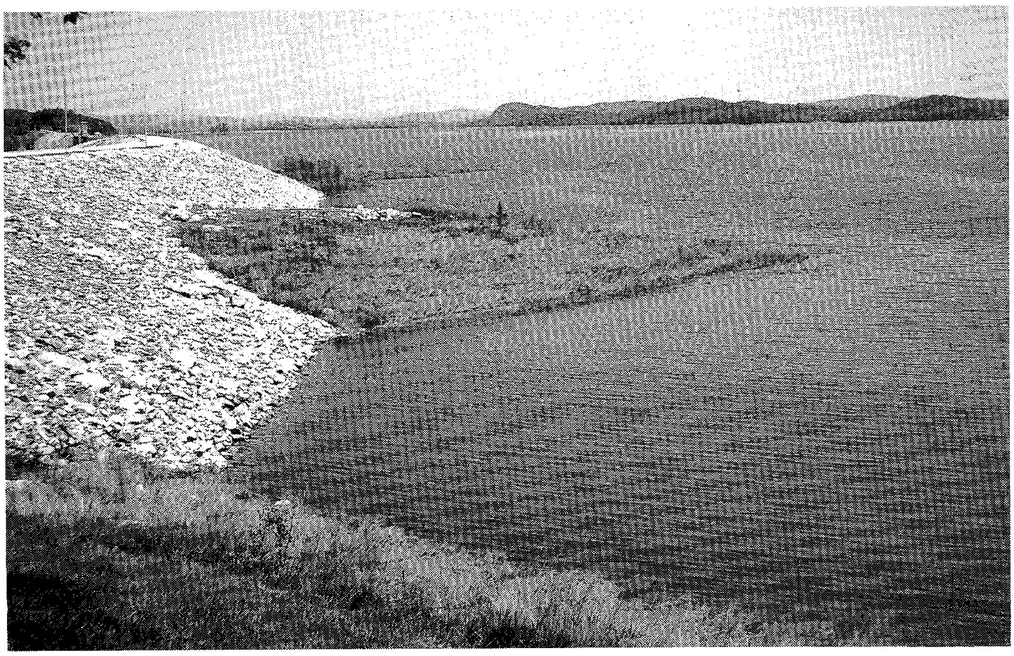


Photo 58 Cherokee Lake.

References

1) Pomeroy, J. S. (1982): Landslides in the Greater Pittsburgh Region, Pennsylvania. U.S. Geological Survey Professional Paper 1229, 48p.

2) Miller, S. A., and Mullarkey, P. W. (1983): Route 51 Landslide, February 16, 1983, Pittsburgh, Pennsylvania. Department of Civil Engineering, Carnegie Institute of Technology, Carnegie-Mellon University, Pittsburgh, Pennsylvania.

3) Gray, R. E. (1975): Design of rock slopes, in Subgrades, foundations, embankments, and cut slopes. Section 15 of Baker, R., ed., Handbook of highway engineering, New York, Van Norstrand Reinhold Co., pp. 457/460.

4) Carrier, R. E. (1982): Geotechnical problems in the Pittsburgh area. in Construction excavation proceedings, Harrisburg, Pennsylvania, April 1982, American Society of Civil Engineers, Central Pennsylvania Section, pp. 104/105.

5) Dash, U., and DeRoss, J. (1979): Performance monitoring of a tieback wall. Pennsylvania Department of Transportation Research Project 76-9, Report FHWA-PA-79-002, 60 p., available from U.S. Department of Commerce National Technical Information Service, Springfield, VA 22161.

6) Hamel, J. V., and Adams, W. R., Jr., (1981): Claystone slides, Interstate Route 79, Pittsburgh, Pennsylvania, USA. of Akai, K., Hayashi, M. & Nishimatsu, Y., eds., Weak rock: Soft, fractured & weathered rock - Proceedings of the International Symposium Tokyo, September 21-24, 1981, pp. 549/553. available from A. A. Balkema, P.O. Box 1675, Rotterdam, Netherlands.

7) Hamel, J. V., and Flint, N. K. (1972): Failure of colluvial slope. Proceedings of the American Society of Civil Engineers, Journal of Soil Mechanics and Foundations Division, vol. 98, SM 2, pp. 167/180.

8) Gray, R. E., Ferguson, H. F., and Hamel, J. V. (1979): Slope stability in the Appalachian Plateau, Pennsylvania and West Virginia, U.S.A. Chap. 12, in Voight, B., ed., Rockslides and avalanches, vol. 2, Engineering sites, Developments in geotechnical engineering 14B, New York, Elsevier Scientific Publishing Company, pp. 447/471.

9) D'Appolonia, E., Alperstein, R., and D'Appolonia, D. J. (1967): Behavior of a colluvial slope. Proceedings of the American Society of Civil Engineers, Journal of Soil Mechanics and Foundations Division, vol. 93, SM 4, pp. 447/473.

10) Gray, R. E., and Gardner, G. D. (1977): Processes of colluvial slope development at McMechen, West Virginia. Bulletin of the International Association of Engineering Geology, No. 16, pp. 29/32.

11) Gray, R. E., Gardner, G. D., and Wimberly, P. M. (1980): Stabilization of a colluvial slope at an urban site. Proceedings of the International Symposium on Landslides, New Delhi, April 1980, 4 p.

12) Lessing, P., Kulander, B. R., Wilson, B. D., Dean, S. L., and Woodring, S. M. (1976): West Virginia landslides and slide-prone areas, with map (Wheeling 7.5' quadrangle, scale 1: 24,000). West Virginia Geological and Economic Survey, Environmental Geology Bulletin, No. 15, 64p.

13) Lessing, P., and Erwin, R. B. (1977): Landslides in West Virginia. Reviews in Engineering Geology, Geological Society of America, vol. 3, pp. 245/254.

14) Erwin, R. B. (1969): (map) Geologic Map of West Virginia.

15) West Virginia Geological and Economic Survey (1980): (map) Geologic map of the Wheeling 7.5' quadrangle, Ohio and Marshall Counties, West Virginia. West Virginia Geological and Economic Survey Open-File Report OF801Q, scale 1 : 24,000.
16) Lessing, P., Messina, C. P., and Fonner, R. F. (1983): Landslide risk assessment. Environmental Geology, vol. 5, No. 2, pp. 93/99.
17) Thompson, B. L., chairman, Langen, R., field trip leader, and Adams, R., field trip manual (1978): Reinforced earth wall, Wheeling, West Virginia. West Virginia Department of Highways, Field Trip Manual of the 10th Annual Southeastern Transportation Geotechnical Engineering Conference, Wheeling, West Virginia, October 11, 1978, 8 p., 19 figs.
18) Fleming, R. W., Johnson, A. M., and Hough, J. E., with contributions by Gokce, A. O., and Lion, T. (1981): Engineering geology of the Cincinnati area (field trip 18). in Roberts, T. G., ed., GSA '81 Cincinnati field trip guidebooks--Gemorphology, hydrology, geoarcheology, engineering geology, vol. 3, American Geological Institute, pp. 543/570.
19) Richards, K. A., and others (1981-1982): (maps) Plate 1, Engineering geologic map of Mt. Adams and parts of Walnut Hills & Columbia Parkway, Cincinnati, Ohio; Plate 2, Relative stability map of Mt. Adams and parts of Walnut Hills & Columbia Parkway, Cincinnati, Ohio; Plate 3, Map and cross-sections of Oregon street-Baum street landslide, Mt. Adams area, Cincinnati, Ohio; Plate 4, Map and cross-section of landslide complex in thin colluvial soils, Eden Park, Cincinnati, Ohio. University of Cincinnati Department of Geology.
20) Brockman, S., Lion, T., and Riestenberg, M. (1980): (map and cross section) McKelvey slide. University of Cincinnati Department of Geology.
21) Lion, T. E. (1982): (maps) Plate 1, Engineering geology for part of Springfield Township, Hamilton Co., Ohio; Plate 2, Bedrock surface contours; Plate 3, Relative stability of ground for hillside development in part of Springfield Township, Hamilton County, Ohio. University of Cincinnati Department of Geology.
22) Lion, T. and Rauf P. (1981): (map) Partridge Hills landslide.
23) Nethero, M. F. (1982): Slide control by drilled pier walls. Proceedings of the Sessions on Application of walls to landslide control problems, American Society of Civil Engineers, annual meeting, Las Vegas, Nevada, April 26-30, 1982. pp. 61/76.
24) Riestenberg, M. M. and Sovonick-Dunford, S. (1983): The role of woody vegetation in stabilizing slopes in the Cincinnati area, Ohio. Geological Society of America Bulletin, vol. 94, pp. 506/518.
25) Sili Dur North American Company (1982): (engineering manual) Loffelstein wall. Sili Dur Co., P. O. Box 1043, Elyra, Ohio 44036.
26) Royster, D. L., chairman (1981): U. S. Landslide field trip, Appalachian region, Tennessee section. (32nd Annual Highway Geology Symposium field trip guide book, 60p.)
27) Royster, D. L. (1974): Construction of a reinforced earth fill along Interstate 40 in Tennessee. 25th. Annual Highway Geology Symposium, Raleigh, North Carolina, May 24, 1974, 11p.
28) Royster, D. L. (1975): Tackling major highway landslides in the Tennessee mountains. Civil Engineering, ASCE, Special Issue, September, 1975, pp. 85/87.
29) Royster, D. L. (1980): Horizontal drains and horizontal drilling # - # an overview. Rock Classifications and Horizontal Drilling and Drainage, Transportation Research Record, No. 783, pp. 16/20.
30) Trolinger, W. D. (1980): Rockwood embankment slide between stations 2001+00 and 2018+

00 # - a horizontal drain case history. Rock Classifications and Horizontall Drilling and Drainage, Transportation Research Record, No. 783, pp. 26/30.
31) Royster, D. L. (1977): Some observations on the use of horizontal drains in the correction and prevention of landslides. Tennessee Department of Transportation, 54 p. (prepared for presentation at the 28th Annual Highway Geology Symposium, Rapid City, South Dakota, August, 1977)
32) Royster, D. L. (1982): Landslide remedial measures (revised). 3rd. Biennial International Landslide Symposium and Field Trip, Gatlinburg, Tennessee, September 1982, Tennessee Department of Transportation, Publication Authorization No. 1011, 84 p., (originally published in the Bulletin of the Association of Engineering Geologists, vol. 16, No. 2, 1979.)
33) Moore, H. L. (1984): Geotechnical considerations in the location, design, and construction of highways in karst terrain-'The Pellissippi Parkway extension', Knox-Blount Counties, Tennessee. Proceedings of the First Multidisciplinary Conference on Sinkholes, Orlando, Florida, October 15-17, 1984, pp. 385/389.
34) Moore, H. L. (1985): Wedge failures along Tennessee Highways in the Appalachian Region -their occurrence and correction. Tennessee Department of Transportation.
35) U. S. Geological Survey (1982): Goals and tasks of the landslide part of a ground-failure hazards reduction program. U. S. Geological Survey Circular, No. 880, 49p.
36) Bjerrum, L. (1967): Progressive failure in slopes of over-consolidated plastic clay and clay shales. Proc. Am. Soc. Civ. Eng., J. Soil Mech. Found. Div., 93(SM5), pp. 1/49.
37) Scheper, R. J. (1973): Report of a landslide investigation [grad. student rep.], Univ. of Cincinnati, Geol. Lib., 20p.
38) Nutting, H. C., Co. (1979): Report of geotechnical investigation of existing residences, Huffman Court, Country Lanes Subdivision, Hamilton County, Ohio. U. S. Dep. Hous. Urb. Devel., 6 rep., 41 p. with append., unpubl.
39) Royster, D. L. (1973): Highway Landslide Problems Along the Cumberland Plateau in Tennessee. Bulletin of the Association of Engineering Geologists, Vol. X, No. 4, pp. 255/287.
40) Hadley, J. B. and Goldsmith, R. (1963): Geology of the eastern Great Smoky Mountains, North Carolina and Tennessee. U. S. Geological Survey Professional Paper 349B, p. 118.
41) Kaufman, B. L. (1980): Mount Adams isn't going anywhere! Cincinnati Enquirer, p. B-1, June 22.
42) Aycock, J. H. (1978): Construction Problems Involving Shale in a Geologically Complex Environment, State Route 32-Appalachian Corridor "S", Grainger County, Tennessee. A paper presented at the 37th Annual Sashto Convention, Nashville, Tennessee, October, 1978.

(received September 16, 1988)

The Symposia

May 10—11, 1983

at National Center of the USGS Reston, Virginia

Review of Research for Disaster Prevention, No. 132, December 1988

Geologic Factors in Landslide Susceptibility, Greater Pittsburgh Region, Western Pennsylvania, U.S.A.

by
John S. Pomeroy

U.S. Geological Survey
Reston, VA 22092

INTRODUCTION

The Greater Pittsburgh region of Pennsylvania **(Fig. 1),** which is within the Appalachian Plateau physiographic province of the eastern United States, is underlain primarily by cyclothemic Pennsylvanian (Carboniferous) and Permian sedimentary rocks **(Fig. 2),** In this region, the potential for landsliding is high (Pomeroy, 1982a, b). Based on reconnaissance mapping (Pomeroy and Davies, 1975; Pomeroy, 1978a) more than 3,000 recent and at least 12,000 older slides are recognized in Allegheny and Washington Counties alone.

Recent slope movements in the Greater Pittsburgh region are generally less than 60m in maximum horizontal dimension and less than 3 m thick. Most slides take place in silty-clay to clay soils; slides in bedrock are uncommon and are restricted to steep valley walls along major drainages. Slumps, earthflows, debris slides, and rockfalls are common. Complex forms of landslide types are abundant, and the distinction among the various

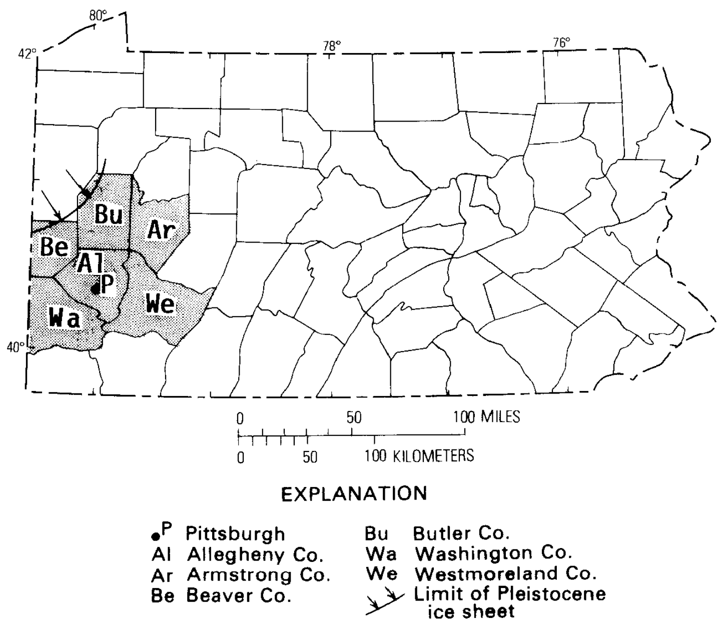


Fig. 1 Index map of Pennsylvania showing the Greater Pittsburgh region (from Pomeroy, 1982a).

METERS 200, 100, 0 — FEET 600, 300, 0 — CARBONIFEROUS

SYSTEM	GROUP	FORMATION	APPROXIMATE THICKNESS, IN METERS	LITHOLOGY Shale ....Sh Mudstone ....Mu Claystone ....Cl Siltstone ....Sl Sandstone ....Ss Limestone ....Ls Coal C	KEY HORIZONS	RELATIVE SUSCEPTIBILITY OF DERIVATIVE SOILS TO LANDSLIDING
QUATERNARY	Alluvium, terrace, and glacial deposits		0–30	Unconsolidated clay to boulders		Nil to low in terrace and glaciofluvial deposits moderate to moderately high in silty clay till
PERMIAN	Dunkard	Greene	145	(Cyclic) Mu, Sl, Ss, Ls, Sh, Cl, C		Moderately high to severe
		Washington	60	(Cyclic) Ls, Mu, Sl, Ss, Sh, Cl, C	Washington coal	
PERMIAN AND PENNSYLVANIAN		Waynesburg	42	(Cyclic) Mu, Ss, Sl, Ls, Sh, Cl, C	Waynesburg coal	Moderate to high
PENNSYLVANIAN	Monongahela	Uniontown	30	(Cyclic) Mu, Sh, Ss, Sl, Ls, Cl, C		Low to moderately high
		Pittsburgh	91		Pittsburgh coal	
	Conemaugh	Casselman	190	(Cyclic) Sh, Mu, Sl, Ss, Cl, Ls, C	Clarksburg red beds	Low
					Morgantown sandstone	Moderately high to severe
					red beds	Low
					Ames limestone member Pittsburgh red beds	Moderately high to severe
		Glenshaw			red beds red beds	Low to moderately high
	Allegheny	Freeport	85	(Cyclic) Ss, Sh, Mu, Sl, Cl, Ls, C	Upper Freeport coal	Moderate
		Kittanning				Low to moderate
		Clarion				
	Pottsville	Homewood	51	Ss, Sh, C		Low
		Mercer				
		Connoquenessing				
MISSISSIPPIAN		Mauch Chunk	60	Sh, Ss		Slight to low
		Pocono		Ss, Sh		

Fig. 2 Chart showing age, thickness, lithology, key horizons, and relative susceptibility to landsliding of geologic formations in the Greater Pittsburgh region (from Pomeroy, 1982a).

types is often arbitrary. However, most landslides in the Pittsburgh region are slump-earthflows as defined by Varnes (1978). Slumps in fill are numerous within the more densely populated areas. Soil creep contributes heavily to property damage.

Slope stability in the region is affected by slope steepness and configuration, precipitation, presence of old landslides, and the oversteepening of slopes by stream erosion. Equally significant natural factors affecting slope stability are the physical character, the composition, and the distribution of bedrock and soil types.

Rocks susceptible to sliding

The rocks of the greater Pittsburgh region are cyclic, and hence, of a heterogeneous character ; slope-stability problems are largely related to underlying incompetent rock types wherever they are present in the section. (Fig. 2). Particularly susceptible to slumping and earth flowage are deeply weathered slopes underlain by fine-grained redbeds of the Conemaugh Group (Pomeroy, 1979 ; Pomeroy and Davies, 1975) and by fine-grained, non-red rock units, particularly in the Dunkard Group (Pomeroy, 1978a).

Red mudstone, claystone, and shale units in the Conemaugh Group are thickest and most widespread near the top of the Glenshaw Formation (Pittsburgh redbeds), but they also are present at other horizons lower in the Glenshaw Formation and in the basal and upper middle parts (Clarksburg redbeds) of the Casselman Formation. These redbeds and derivative soils are of primary concern not only because they are widespread and susceptible to sliding, but also because they underlie the more densely populated areas such as Allegheny County, northwestern Westmoreland County, southern Butler County, and southeastern Beaver County (Fig. 1).

A second group of rocks whose weathered products are susceptible to sliding includes non-red mudstone and claystone, particularly those in the Dunkard Group in southern Washington County (Fig. 1). Although these rocks are of more limited areal extent than the redbeds of the Conemaugh Group, the density of slides in areas underlain by the 240 m thick Dunkard Group is higher than that on slopes underlain by the Conemaugh Group. Landslides are abundant anywhere in the Dunkard terrane and are not limited to specific sequences, although they are more common in the upper part of the section, which is dominated by the non-red mudstone units.

Figure 3 shows the area's relative susceptibility to landsliding (Pomeroy, 1982a). Area 1 is underlain by the Dunkard Group. Much of Area 2 is underlain by the Conemaugh Group. Areas 3 and 4 are underlain by more competent rock types.

Slides have also taken place on slopes resting on underclay (particularly in the Upper Freeport coal zone of the Allegheny Group), and above other non-red claystone, mudstone, and shale units in the Pottsville, Allegheny, Conemaugh, and Monongahela Groups (Fig.2).

Glacial till (Illinoian) in a small area of the northwestern part of Butler County is also susceptible to landsliding. Slumped material derived from the till typically is a relatively homogeneous bluish- to brownish-gray clay (Pomeroy, 1978b).

Although rock falls are not restricted to any particular stratigraphic unit, they are most common where massive sandstone or limestone overlies weaker rock ; such massive sandstone and limestone units are exposed in cliffs in the Allegheny, Conemaugh, and Monongahela Groups.

Landslide-prone soils

Throughout the Greater Pittsburgh region, rocks are normally not well exposed, but rather are masked by a fine-grained soil mantle or regolith. The regolith is relatively thin on upper slopes but increases in thickness to a maximum of about 30 m near the toes of slopes. Soils weathered from red and non-red mudstone and claystone are sensitive to mass movement. Clayey to silty soils are friable and relatively low in weight per unit

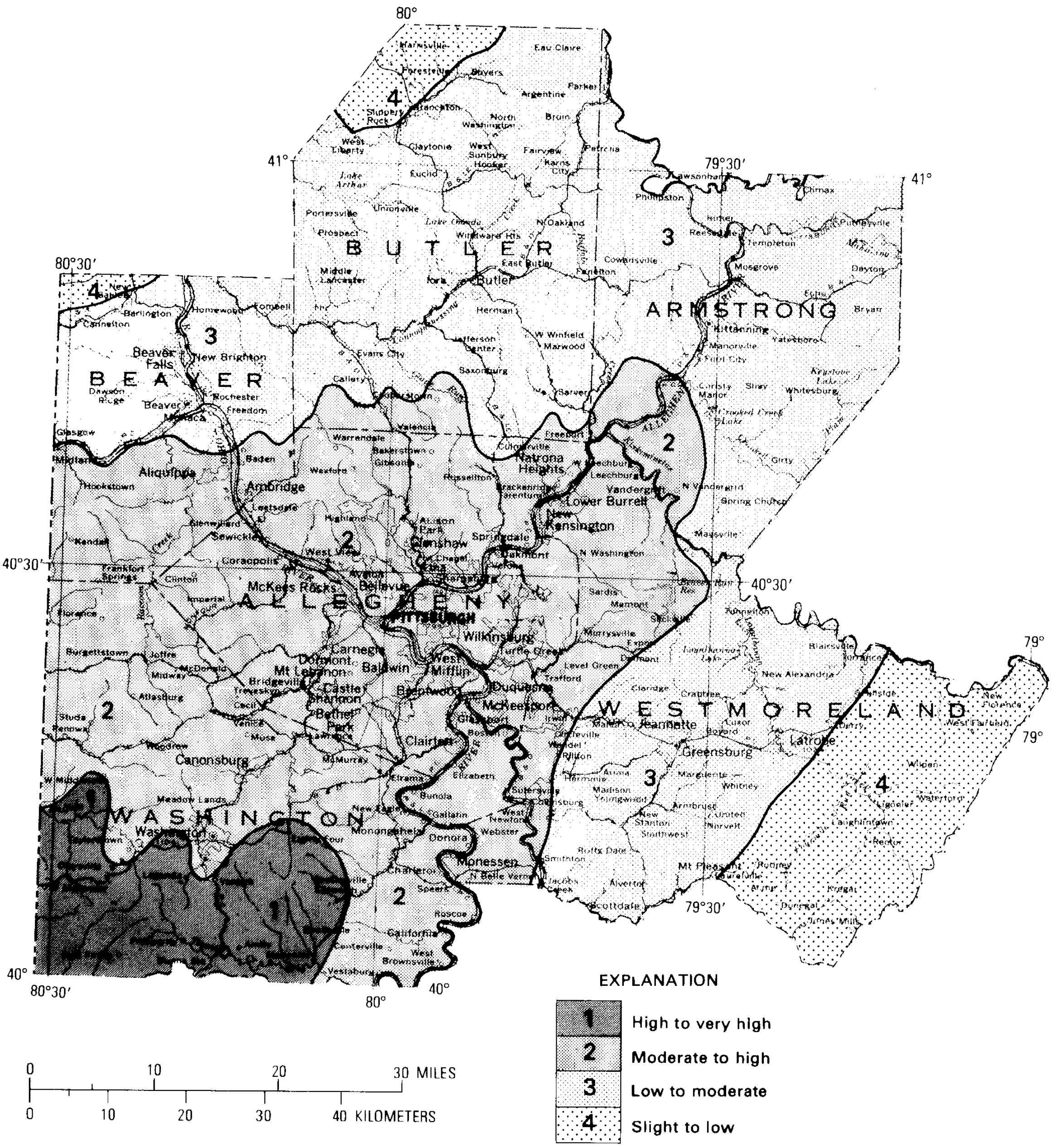


Fig. 3 Map showing relative susceptibility to landsliding, Greater Pittsburgh region (from Pomeroy, 1982a).

volume when dry ; however, they retain water, becoming heavier and more plastic, hence more susceptible to downslope movement. An extrapolation of U.S. Soil Conservation Service (1974a, b) figures for soil types and acreage in Washington County reveals that landslide-prone soils (that is, soils having a slope of at least 8 percent and moderate to high shrink/swell volume ratios) occupy approximately 75 percent of the total area ; they occupy only 18 and 20 percent of Allegheny and Beaver Counties, respectively. Landslide-prone soils include smaller percentages of the other three counties.

Physical properties of rocks and soils

Cyclic sedimentary rocks possess widely differing physical properties that affect their stability. Sandstone is at least five times as strong in compression and at least three times as strong in shearing strength as claystone (Philbrick, 1953). Sandstone from the Conemaugh Group has a bearing capacity more than four times as great as that of the Pittsburgh redbeds (McGlade and others, 1972).

Slaking-Many red and non-red claystones and mudstones in the region slaked within an hour to a few hours after immersion in water. The samples that I tested included weathered red claystone derived from the Pittsburgh and Clarksburg redbeds (Fig. 2) of the Conemaugh Group and weathered gray claystone derived from the Dunkard Group. Weathered red shales (fissile) from the Conemaugh Group, however, did not slake.

Atterberg limits-Atterberg limits indicate that soils derived from the Dunkard Group possess a slightly higher plasticity index than those derived from Conemaugh Group rocks (Pomeroy, 1982a). The expansion potential of a sample is regarded as moderate if the plasticity index is 5 to 25 and high if the plasticity index is greater than 25. The expansion potential of samples from both stratigraphic groups ranged from moderate to high.

Weathering and abrasion -In a study of the weathering of the Pittsburgh redbeds, Kapur (1960) concluded that each cycle of freezing, thawing, drying, and saturation produces a loss in strength and an increase in moisture content in the weathered rocks. The rates at which rock strength is lost and moisture content is increased diminish as cycling continues. Bonk (1964) stated that the size of the weathered particles from redbeds definitely decreases as the number of weathering cycles increases.

Younger Pennsylvanian and Permian non-red mudstones and claystones from Washington County have been subjected to weathering and abrasion tests that demonstrate the effects of compactive forces and repeated wetting and drying cycles upon disaggregated samples (Berryhill and others, 1971). The results showed that the effect of a single compactive force (hammer test) is almost equivalent to four cycles of wetting and drying.

Permeability and porosity-Both permeability and porosity are relevant to the landslide process because susceptibility to sliding is increased when excessive pore-water pressure in clay decreases its shear strength. Rocks and soils are most likely to be saturated by water in zones where permeable materials overlie relatively impermeable materials.

The red clayey soils derived from the Conemaugh Group have a relatively high porosity (as much as 40 percent), but their permeability is relatively low and as little as 1 to 5 percent of the pore water is drained by force of gravity (Subitzky, 1975).

The clay content of soil samples studied from landslides overlying Dunkard Group rocks is slightly higher than that of samples derived from Conemaugh Group rocks. The clayey soils have relatively high porosity and low permeability similar to those of the red clayey soils of the Conemaugh Group. The clayey soil material in both groups have the capacity to absorb and retain copious quantities of water, resulting in soil movement.

Ground water seeps are common at the bases of cliff-forming sandstone ledges in contact with poorly exposed mudstones and shales that underlie more moderate slopes. Movements in the regolith commonly originate below the seep along slopes of low permeability.

Mineralogy-X-ray diffraction studies of Dunkard Group samples indicate that the clay fractions of these samples consists of illite, vermiculite, kaolinite, and mixed-layer minerals in decreasing order of abundance. The clay mineralogy is similar to that of soils derived from the Conemaugh Group except that the soils derived from the Dunkard Group tend to have a slightly greater proportion of expandable minerals. Similar clay-mineral data were obtained from the same units in the Greater Pittsburgh region by Ciolkosz and others (1979). The moderate to high shrink-swell potential of most soils derived from the rocks of the Dunkard Group and of soils derived from certain rocks of the Conemaugh Group is attributable to both the relatively high clay content and the moderately high expandable-mineral content.

In a study of *equivalent* rock units in southeastern Ohio, Fisher and others (1968) found that clay-mineral suites in unstable redbeds consist dominantly of illite degraded by the leaching of potassium ions. These authors concluded (p. 79) that "simultaneous deposition of ferric iron with degraded illitic clay precluded reabsorption of the bonding potassium ion in the depositional environment. The continued presence of iron has greatly inhibited the reconstitution of the clay throughout diagenesis and later geologic time." Fisher and others (1968) indicated that degraded illite and montmorillonite react similarly in the presence of water except that expandability is not as great in the illite.

Summary

Landslides in the Greater Pittsburgh region are commonly thin and take place in silty-clay to clayey soil. Most landslide forms are slump-earthflows. Slope stability is affected by geologic factors related to the physical character, composition, and distribution of the bedrock and soils that underlie the slopes.

The highest density of mass movement is in southern Washington County along slopes underlain by mudstone of the Dunkard Group. In Allegheny County and adjacent areas, soils derived from red mudstones of the Conemaugh Group are responsible for most slope-stability problems.

Slide-prone soils derived from both the Conemaugh and Dunkard Groups have a relatively high porosity and low permeability. These soils slake within an hour to a few hours after immersion in water. Samples from both stratigraphic groups possess a moderate to high expandability.

Lithologic factors have a controlling influence on the origin and distribution of slope failures. Ground water seeps are common at the bases of sandstone ledges in contact with poorly exposed mudstones and shales that underlie more moderate slopes. Movements in the regolith originate below the seeps along slopes of low permeability.

References

Berryhill, H. L., Jr., Schweinfurth, S. P., and Kent, B. H., 1971, Coal-bearing Upper Pennsylvanian and Lower Permian rocks, Washington area, Pennsylvania, pt. 1, Lithofaces ; pt. 2, Economic and engineering geology : U.S. Geological Survey Professional Paper 621, 47 p.

Bonk, J. G., 1964, The weathering of Pittsburgh redbeds : Pittsburgh, University of Pittsburgh, M. S. dissertation, 39 p.

Ciolkosz, E. J., Petersen, G. W., and Cunningham, R. L., 1979, Landslide-prone soils of southwestern

Pennsylvania : Soil Science, v. 128, no. 6, p. 348-352.

Fisher, S. P., Fanaff, A. S., and Picking, L. W., 1968, Landslides of southeastern Ohio : Ohio Journal of Science, v. 68, no. 2, p. 65-80.

Kapur, Chandra, 1960, The effect of weathering on the Pittsburgh red shales : Pittsburgh, Pa., Carnegie Institute of Technology, M. S. thesis, 18 p.

McGlade, W. G., Geyer, A. R., and Wilshusen, J. P., 1972, Engineering characteristics of the rocks of Pennsylvania : Pennsylvania Geological Survey, 4th ser., Environmental Geology Report 1, 200 p.

Philbrick, S. S., 1953, Design of deep rock cuts in the Conemaugh Formation, *in* Morris Harvey College and West Virginia State Road Commission, Proceedings of the 4th Symposium on geology as applied to highway engineering, Morris Harvey College, Charleston, West Virginia, February 20, 1953, p. 79-88.

Pomeroy, J. S., 1978a, Isopleth map of landslide deposits, Washington County, Pennsylvania - A guide to comparative slope susceptibility : U.S. Geological Survey Miscellaneous Field Studies Map MF-1010.

——, 1978b, Map showing landslides and areas most susceptible to landsliding, Butler County, Pennsylvania : U.S. Geological Survey Miscellaneous Field Studies Map MF-1024.

——, 1979, Map showing landslides and areas most susceptible to sliding in Beaver County, Pennsylvania : U.S. Geological Survey Miscellaneous Investigations Map I-1160.

——, 1982a, Landslides in the Greater Pittsburgh region, Pennsylvania : U.S. Geological Survey Professional Paper 1229, 48 p.

——, 1982b, Mass movement in two selected areas of western Washington County, Pennsylvania : U.S. Geological Survey Professional Paper 1170-B, 17 p.

Pomeroy, J. S., and Davies, W. E., 1975, Map of susceptibility to landsliding, Allegheny County, Pennsylvania : U.S. Geological Survey Miscellaneous Field Studies Map MF-685-B.

Subitzky, Seymour, 1975, Heavy storm precipitation and related mass movement, Allegheny County, Pennsylvania : U.S. Geological Survey Miscellaneous Field Studies Map MF-641-D.

U.S. Soil Conservation Service, 1974a, Volume 1, Soil survey interpretations for Greene and Washington Counties, Pennsylvania : Pennsylvania Department of Environmental Resources, State Conservation Commission, 103 p.

——, 1974b, Volume 2, Soil survey map for Washington County, Pennsylvania : Pennsylvania Department of Environmental Resources, State Conservation Commission.

Varnes, D. J., 1978, Slope movement types and processes, Chap. 2 *in* Schuster, R. L., and Krizek, R. J., eds., Landslides-analysis and control : National Academy of Sciences, Transportation Research Board Special Report 176, p. 11-33.

Experience With Landslide Zoning in Canada

David. M. Cruden
Department of Civil Engineering
University of Alberta
Edmonton, Alberta, Canada, T6G 2G7

for presentation at the
Symposium on Landslide-Hazard Zonation and Mitigation

May 11, 1983
United States Geological Survey
Washington, District of Columbia
U.S.A.

Landslides in Sensitive Clays in Eastern Canada

The major landslide problem in Eastern Canada is with post-glacial marine clays which are sensitive, the clays also extend into the north-eastern United States. The Gulf of St. Lawrence is seismically active. In 1663, an earthquake of Richter magnitude 7.7 caused many earthflows, the largest, with an area of 22 km^2 occurred in the Saguenay and is shown on the air photograph. An air photo reconnaissance has found 686 similar scars in the province of Quebec. Settlement of Quebec has involved the development of areas prone to movement, leading to damage to property and loss of life. On the average a mudflow of more than 1 hectare occurs every 2.4 years in urban or farming areas. Deaths due to mudflows have occurred recently in 1955, 1962, 1963, 1971 and 1978.

In 1979, the Quebec Legislative passed a law "On planning and development" that required regional municipalities to draw up a Plan. The Plan must include, among other things:

"An identification of zones where the occupation of the ground is subject to particular constraints for reasons of public safety. The constraints include zones of flooding, erosion, landslides and other natural catastrophes". (L'Editeur Officiel du Quebec, 1979, Projet de loi 125, Rough translation of Chapter 1, Clause 5: 4).

To assist this planning effort, the Quebec government has begun mapping zones exposed to the risk of ground movement at a scale of 1 : 20,000. The choice of map areas was based on a reconnaissance of large old landslide scars at 1 : 250,000. The maps will show 5 risk zones (R. Bergeron et al., 1981), which may be roughly described in the following way:

"High risk zone (red on the zoning map) indicates that signs of instability have been observed on the ground and that geodynamic processes contribute constantly to the deterioration of the already precarious stability of the ground......

Medium risk zones (orange on the map) indicate that the geometry and the topography of the terrain suggest potential instability while no sign of instability has been observed during unmapping. Geodynamic processes are not contributing actively to the deterioration of ground stability but construction or development may provoke movements......

Slight risk zones (in yellow) show that no signs of instability were observed during mapping but the geotechnical properties of the ground and the local geological situation show that the site could suddenly be involved in an extensive slide......

Hypothetical rise zones (in yellow crosshatched with green) show no signs of instability were observed during mapping but the geotechnical properties of the ground and the local geological situation show that a huge landslide could be produced on the site if exceptionally unfavourable national conditions happened to occur—for example, an earthquake of Richter magnitude 7 or more with an epicentre close to the site......

Zones of no risk (white) show that no sign of instability had been observed on the terrain and that considering the local geological situation and the geotechnical properties of the materials the risk of ground movement is nil. This doesn't include risks of settlement which could follow heavy construction......"

The first of these maps will be published towards the end of 1983. The maps are expected to channel development into safer areas.

Postglacial marine clays also extend onshore around Hudson's Bay and on a number of the Arctic Islands. On the Pacific Coast of Canada, the mountains of the Coast Range and the nature of isostatic rebound from the recent glaciation have restricted the extent of clay deposition. A full review is given by Viberg (1984).

Site Specific Problems in Western Canada

The first case in Canada in which private development was affected by official fears of slope movements occurred in 1911, when a Royal Commission stopped the extension of 2 mines under the North Peak of Turtle Mountain in the Crowsnest Pass region of southwestern Alberta.

The oblique air photograph **(Photo 1)** shows the debris of the Frank Slide, a rockslide-avalanche of about 40 million cubic metres of Palaeozoic limestone which had destroyed the surface workings of the mine in April, 1903. The underground workings were almost undamaged, however and mining resumed southwards in the same year following the strike of the almost vertical coal seam by horizontal drifts.

A shaft mine was opened in the same seam to the north of the debris. The southerly expansion of the shaft mine towards the drift mine was vigorously opposed by the then Director of the Canadian Geological Survey, R.W. Brock. Brock considered the Frank Slide "was caused by the mining of coal ... there is no doubt in my mind" (Brock, 1910a). Again, ... "mining is too dangerous to be continued .... A large slide would cut off all railway communication and close the mines west of Frank, it might permanently close the pass. The town of Frank would be wiped out with a fearful toll of life" (Brock, 1910b).

A complex series of events led to a Royal Commission to arbitrate between the coal company and the Federal and Provincial Governments. The members of the Commission, R.A. Daly then a professor at Massachusetts Institute of Technology, G.S. Rice from the United States Bureau of Mines and the Ontario Geologist, W.A. Miller supported Brock's views that continued mining immediately north of the slide debris threatened the town.

They also emphasized the need to leave sufficient coal in the drift mine to support the South Peak of Turtle Mountain.

As a result the Provincial Government supported the relocation of the town to a new site owned by the mining company but further progress of the shaft mine was halted by water inflows. The drift mine removed a further 1/2 million tons over the next six years before a fire which had been burning in the mine for 8 years caused its closure. Extraction ratio from the seams was probably about 50%. In 1931, B.R. McKay, who had been in the Survey party assisting the 1911 Commission, returned to Turtle Mountain in the course of his duties as a Survey Coal geologist. He noticed changes in the South Peak area "the most unstable portion of the Mountain at the present time ..... almost completely surrounded by fissures." As a result of further investigations on behalf of the Provincial Government by John Allan (1933) from the University of Alberta (who studied under Daly at MIT), residences and the highway under South Peak were moved to the east outside a danger zone empirically drawn by Allan. Allan also set up a monitoring program for the South Peak.

The coal industry declined in the Crowsnest Pass. It was not until the mid 1970's that there was again pressure for the redevelopment of the bottom land and terraces between Bellevue and Hillcrest. At that time, the Slide Area was declared a Historic Site and a Restricted Development Area. In 1980, the monitoring program established by Allan was considerably expanded to determine which, if any, parts of the South Peak are moving. Instrumentation for more precise measurements of displacements across cracks has been installed (Cruden, Prosser and Sneddon, 1982), low-level photography flown fo- photogrammetric estimation of displacements over a wider area (Fraser, 1982) and EDM survey stations and reflectors located for remote monitoring of the slide mass. This new program should provide the information for land-use planning for the eastern slopes of Turtle Mountain.

Other Sites

A number of other sites in the Cordillera have been investigated in detail recently in connection with engineering projects (Moore, Matthews, 1978 ; Piteau, Mylrea, Blown, 1978). Some regional mapping of landslides has been carried out (Bayrock, Reimchen, 1980 ; Howes, 1981 ; Ryder, 1981). Some large historic and prehistoric natural movements have been described (Bruce, Cruden, 1980 ; Clague and Souther, 1982 ; Cruden, 1976, 1982 ; Eisbacher, 1977, 1978).

In Western Canada, generally, public and professional awareness of slope hazards is reasonable and the present population density allows hazardous sites, once they are recognized, to be avoided. The future may see increasing pressure for the development of such sites and zoning may then be an appropriate response. Cruden (1984) has recently reviewed these problems.

References

Allan, J. A., 1933, Third report on the stability of Turtle Mountain, Department of Public Works, Edmonton, Alberta, 28 p.

Bayrock, L. H., Reimchen, T.H.F., 1980, Surficial Geology, Alberta Foothills and Rocky Mountains, Alberta Research Council, 6 maps.

Bergeron, R., Desforges, P., Lebuis, J., Mackay, P., Maranda, R., Robert, J.-M., Rouleau, S., Thibault, G., 1981, Une politique d'intervention pour les zones exposees aux mouvements de terrain, Ministere de l'Environnement, Quebec, 105p.

Bruce, I. G., Cruden, D. M., 1980, Simple Rockslides at Jonas Ridge, Alberta, Canada, International Symposium on Landslides, New Dehli, Vol. 1, pp. 185-190.

Brock, R. W., 1910a, Letter to the Deputy Minister of Public Works, Edmonton, Alberta, 26 March, Alberta Provincial Archives, File 13048.

Brock, R. W., 1910b, Letter to the Deputy Minister of Public Works, Edmonton, Alberta, November 3, Alberta Provincial Archives, File 13048.

Cruden, D. M., 1976, Major rockslides in the Canadian Rockies, Canadian Geotechnical Journal, Vol. 13, pp. 8-20.

Cruden, D. M., 1984, Landslide Problems in the Canadian Cordillera, Proceedings 37th Canadian Geotechnical Conference, Toronto, pp. 1-21.

Cruden, D.M., Prosser, D., Sneddon, D. T., 1982, Monitoring the South Peak of Turtle Mountain, Proceedings 4th Canadian Symposium on Mining Surveying and Deformation Measurements, Canadian Institute of Suveying, Ottawa, pp. 335-349.

Eisbacher, G. H., 1978, Observations on the streaming mechanism of large rock slides, Northern Cordillera, GSC Paper 78-1A, pp. 49-52.

Eisbacher, G. H., 1977, Rockslides in the Mackenzie Mountains, District of Mackenzie, Rept. of Activities, A, Geol. Surv. Can. Paper 77-1A.

Fraser, C. S., 1982, The potential of Analytical Close-Range Photogrammetry for Deformation Monitoring, Proceedings 4th Canadian Symposium on Mining Surveying and Deformation Measurements, Canadian Institute of Surveying, Ottawa, pp. 183-196.

Howes, D. E., 1981, Terrain inventory and geological hazards, Northern Vancouver Island, Assessment and Planning Division, Department of Environment, Victoria, British Columbia, Bulletin 5, 105 p.

Moore, D. P., Mathews, W. H., 1978, The Rubble Creak landslide, S.W. British Columbia, Canadian Journal of Earth Sciences, Vol. 15, pp. 1039-1052.

Piteau, D. R., Mylrea, F. H., Blown, I. G., 1978, The Downie Slide, Columbia River, British Columbia, Chp.

Ryder, J. M., 1981, Biophysical resources of the East Kootenay area - terrain, Assessment and Planning Division, Department of Environment, Victoria, British Columbia, Bulletin 7, 152 p.

Viberg, L., 1984, Landslide Risk Mapping in Soft Clays in Scandinavia and Canada, Proceedings 4th International Symposium on Landslides, Volume 1, pp. 325-348.

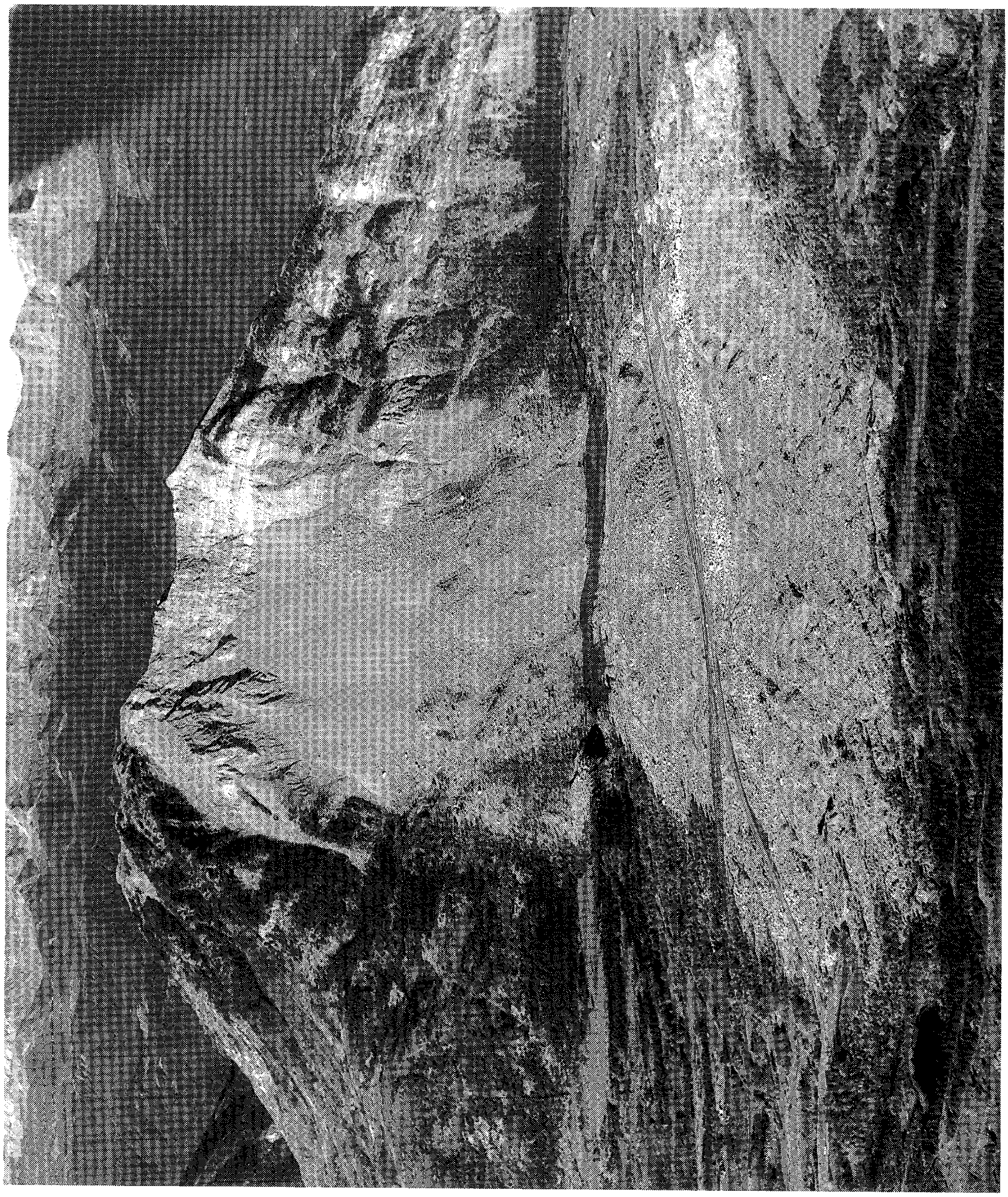


Photo 1 Oblique aerial view of the Frank Slide from the northeast. Photo by C. Beaty, 1976.

Review of Research for Disaster Prevention, No. 132, December 1988

Landslide Zonation
A Method for Landslide Hazard Assessment

HANS KIENHOLZ

University of Berne, Geographical Institute,
CH-3012 BERNE (Switzerland)

The levels of hazard assessment

The hazard assessment procedures used by the group "Natural Hazards and Geomorphology" at the University of Berne are divided into three levels. Each succeeding level results in a more detailed, in-depth analysis of hazardous processes (cf. **Fig. 1**).

First level of Assessment

recognition of the location and type of hazard	→	HAZARD-INDEX MAP

Second level of assessment

analysis and assessment of the hazard (probability, intensity, cause)	↗ →	HAZARDS MAP

Third level of assessment

in depth, site-specific analysis and assessment of the hazard (i.e. investigation of ground conditions at a building site)	↗ →	DETAILED ASSESSMENT

Fig. 1 The levels of hazard assessment

---

*) With support by the Swiss Mobiliar Insurance Company

The first level of hazard assessment

At the first level of hazard assessment, we are concerned with the recognition and localization of hazards and not yet with hazard assessment in the strictest sense. Relatively surficial and time-saving methods are used, in particular the interpretation of stereographic imagery supplemented by interviews with local inhabitants and by scattered field checks. The use of time-saving and surficial methods has an effect, however, on the accuracy of the results and on the degree of detail (KIENHOLZ, BICHSEL, 1982, GRUNDER, 1980 and GRUNDER, LANGENEGGER, 1983).

A cartographic illustration containing the information from this first level of assessment is called a *hazard-index map*. Such maps are accurate up to a scale of 1 : 20, 000.

The hazard-index map shows :

1) whether an area is endangered,
2) whether the hazard is confirmed, potential or only presumed,
3) and the kind of hazard responsible for the danger.

Hazard-index maps are used in regional and national planning.

The scale and content of hazard-index maps may differ greatly depending on the purpose of the map as well as on the amount of time invested and the method used.

The second level of hazard assessment

At the second level, the hazard is analysed and assessed. The cause, intensity, and probability or frequency of the hazard as well as the hazard process and its effects are investigated. Also, the endangered area is exactly delimited (KIENHOLZ, 1977).

Primarily settlement areas are examined and statements are made about the endangerment of specific sites and pieces of property. A map containing the information obtained from the second level of assessment is called a *hazards map*. Accurate maps can be made within a scale of 1 : 10,000 to a scale of 1 : 1000.

In Switzerland, a hazards map which is designed for local planning purposes should show : whether an area has been investigated, whether it is endangered and, if so, the types of hazards. It indicates the expected magnitude and, possibly, the expected frequency of a hazard at a particular site.

In practice, contrary to some laws which require that every kind of hazard be mapped, most hazards maps only show avalanche hazards. However, there are a few complex hazards maps.

Much more work than at the first level is required to obtain relatively accurate data about the endangerment of a specific size. The strive for accuracy, however, should not be overdone, otherwise, considering the large size of the assessment area, the time-expenditure and the cost become too great. If a justifiable amount of research is not sufficient to establish a definite degree of hazard for a specific site, we have to resort to designating the site as being "endangered to an unknown extent".

Each kind of hazard and each hazard site requires an individual combination of methods. Nevertheless, certain general guidelines can still be set up for the assessment procedure.

The third level of hazard assessment

At the third level of assessment, hazards and their effects are examined in greater detail (e.g. for a single building site) so that more precise statements can be made about the degree of hazard. I will not go into more detail about the third level of assessment here because is goes beyond the scope of this paper.

The basic methods of hazard assessment

We distinguish three basic methods of hazard assessment (KIENHOLZ, 1978):

1. "HISTORICAL": analysis of oral and written data about former events,
2. "FIELD": analysis of the geomorphology, geology, vegetation and the "silent witnesses" of former events in the field,
3. "MODELLING": simulation, experimentation, calculation. (Because simulation and experimentation are rather time-consuming, such methods are used mainly for the third level of hazard assessment only.)

Full assessment of any hazard requires that more than a single method be used and preferably all three.

Special problems of landslide hazard assessment

Some hazards occur more or less regularly in exactly the same place. Identical mechanisms are involved and the degrees of damage are comparable. This is particularly true of snow avalanches and of mountain torrents which are fed almost exclusively with fresh debris ("Jungschutt-Wildbäche" after STINY, 1931).

In these cases the hazard has been recognized and the assessment can be based almost exclusively on the first basic method, on the historical method.

Where landslides and rockfall are concerned, the situation is completely different because these processes often occur only once in the same way in a particular place.

The term landslide refers to so many different processes that it is difficult to make a list that is generally valid for all the problems involved in their assessment.

It seems useful to divide landslides into the categories shown in **table 1**:

Cases 1a) and 4 are usually relatively easy to deal with because earlier or actual events give us an indication of the kinds of mechanisms involved. We have to remember, however, that there might be a change in the mechanisms in the future. Generally, we can just proceed according to the second basic method, analysis of the landscape and of the silent witnesses, without encountering any difficulties.

In cases 2 and 5 it is decidedly more difficult to make an assessment of the situation or to make a prediction. The amount of effort necessary to arrive at a sufficiently accurate assessment must be weighed against the benefits that are to be gained from such an assessment. The question is how much time and money can be invested.

In case 1b) we are dealing with unique events which can be predicted neither historically nor through single geomorphic analysis. However, a prediction may be possible if an exact analysis of the geologic structure *and* the water regime as well as other more comlicated technical investigations are made.

LANDSLIDE CLASSIFICATION FOR PURPOSES OF HAZARD ASSESSMENT AND MAPPING
****
(Table 1)

LANDSLIDE TYPES	CHARACTERISTICS	SPECIFIC ASSESSMENT PROBLEMS
1. LARGE LANDSLIDES, ACTIVE		
a) slow, long duration	- slow, steady creeping - often quick secondary small landslides - mostly deep	- fact of movement normally well known - usually clearly visible because of silent witnesses
b) spontaneous, fast or slow	- mostly deep - often transition to type 1.a)	- prediction must be made while the situation is still type 1.a), 2. or 3.
2. LARGE LANDSLIDES, DORMANT	- silent witnesses of former movement - no actual movement	- predicting of the reactivation of the whole mass or large parts of it is very difficult - secondary small landslides can be predicted from the geomorphological analysis
3. LARGE LANDSLIDES, POTENTIAL	- no silent witnesses of former movement	- assessment very difficult - by hydrogeological, geological and geomorphological investigations - by stability analysis (soil and rock mechanics)
4. SMALL LANDSLIDES, ACTIVE	- mostly fast sliding on relatively steep slopes - often transition to a flow process - often surficial	- open scars clearly visible
5. SMALL LANDSLIDES, DORMANT	- silent witnesses of former movement - no actual movement - no exposed scars	- in most cases renewed sliding can be expected at any time
6. SMALL LANDSLIDES, POTENTIAL	- no silent witnesses of former movement - no actual sliding	- assessment very difficult - by hydrogeological, geological and geomorphological investigations - by stability analysis (soil mechanics)

Such expenditures only seem to be economically justified where, for example, the location and the construction of expensive buildings are concerned.

In case 6 we are also confronted with difficult problems. Hillsides with a slope angle of about 25-30° that are located in till deposited on tertiary sandstone, conglomerates and marl indicate a landslide hazard. There are so many such slopes in the Swiss Midlands that large areas of the region would have to be designated as being endangered by landslides if this were the criteria.

Such an encompassing and conservative demarcation of landslide hazard zones may be feasible in sparsely populated areas. However, in Switzerland, which is generally densely populated and where building land is scarce, such conservatism is neither justifiable nor politically realizable. Consequently, when assessing landslide hazards, we must try, even in cases 3 and 6, to arrive at the most correct site specific assessment possible.

A method of landslide hazard assessment for hazard-index maps and hazards maps

As I already mentioned, landslide zoning has been rather neglected in Switzerland. However, several state institutions and many private companies are tackling the problems of slope stability. The methods they use, for example, to examine soil and rock mechanics, are usually very complicated and generally serve to determine the suitability of a building site. Such investigations are quite specific and belong to the third level of hazard assessment. Here, we are interested in procedures that can be used at the first and second level of hazard assessment. Research in this area has been going on for quite some time at the Geographical Institute of the University of Berne.

The following procedure for assessing landslide hazards is applied at the first and second level of hazard assessment (Fig. 1). It is based on the simplified categorization of landslides presented in table 1.

This landslide check is an attempt to arrive at the most accurate results possible, in any case, at an objective recognition or assessment of hazards, within a reasonable amount of time. The landslide check serves only as a guideline for the assessment procedure, leaving the elaborator enough room for using his own judgement.

This is the only way to ensure that each landslide and each potential landslide site receives individualized treatment.

Before starting the landslide check, the area to be assessed is divided into units. The boundaries of each unit area should be determined with regard to the terrain. The following criteria, which have to be adjusted to suit the local conditions and the problem at hand, are used to delimit the unit boundaries.

Priority	*Criterion*
1	border between depressions of streams/creeks and "open" slopes
2	accumulation of certain types of damage
3	change in lithology
4	change of general slope angle
5	change in land use
6	change of slope direction (of more than 90°)

Ideally, each unit area should be between 2 and 50 hectares in size depending on the degree of detail needed for the assessment and on the terrain.

The landslide check is applied to each unit area.

The checklist consists of 5 forms (cf. appendix):

1. the main routine check
2. the checklist for assessing the potential hazard of small surficial landslides on slopes
3. the subroutine "historical data" checklist
4. the subroutine "detailed field checks"
5. the examination of "climatic conditions"

For the production of index maps, forms 1, 2 and 5 are used at the first assessment level. At the second assessment level, that is for the actual hazard mapping, all 5 forms are used.

The actual procedure that is followed for each unit area will be described next.

Climatic conditions

The most important climate data has to be compiled for each region. If possible, the data of one or more weather stations in the region should be analysed.

Data about the intensity and duration of precipitation, factors that can trigger landslides, are entered on **form 5.** With regard to snow-melt conditions, the frequency and the intensity of delayed snow-melt are of interest. Data about the frequency of fall winds with dry adiabatic warming such as the Föhn or the Chinook, snow accumulation in autumn, and the wetness of the snow are also relevant.

Recognition of landslide hazard

The second step consists in recognizing landslide hazards. Using the flow diagram on **form 1,** we decide whether landslides are visible or whether there is historical evidence of former landslides.

If landslide hazards have been confirmed, the evidence that led to the confirmation is to be noted. Such kinds of evidence are: exposed landslides, traverse ridges, depressions with accumulation, graben, reverse slopes, tension/shear cracks or historical evidence.

The next step is differentiating between large and small landslides and between active and dormant landslides. The criteria for deciding whether a landslide is dormant or active are the condition and approximate age of the vegetation cover in the landslide area or on scarps and fissures and any other indications of the time range of any movement. If the last movement occurred more than 100 years ago and if the vegetation cover in the slide area is well regenerated, we designate the landslide as being dormant, otherwise it is considered to be active.

Consequently, we divide confirmed landslides into the categories: "large, active", "large, dormant", "small, active", and "small, dormant".

If landslides are neither visible nor historically evident, we examine the area for choppy relief or wet areas.

If we find such areas and if the choppiness and the wetness are not solely caused by the bedrock structure, large landslides could occur and we characterize the area as being endangered by "*potential large landslides*".

If we do not find any choppy relief or wet areas, we look for indications of surficial creeping such as outcrop bending, tilted or curved trees, buckled roads or displaced grass cover.

If there are indications of creeping, we think that small landslides could occur, and we designate the area as being endangered by "*potential small landslides*".

When no indications of creeping are visible, we make one more test by going through the "checklist for assessing the potential hazard of small surficial landslides on slopes" **(form 2).** Eight questions are to be answered and each answer recieves a certain number of points. After totaling the number of points, we proceed to label A or B on form 1.

If the total number of points is high, it is an indicator of the possibility of small landslides, and we also designate the areas as being endangered by "*potential small landslides*". If the total number of points is low, one can suspect that there is no danger of small landslides and we choose the category "*probably no landslides*".

After having checked the unit area for landslides in this way, we have fulfilled the requirements of the first level of assessment, and we are in a position to make a landslide hazard-index map (see Fig. 1).

It is important to emphasize that the procedure for registering information indicated by the forms and the checklists does not prescribe which methods are to be used. At all levels of assessment, we work with aerial photographs whenever possible. However, wherever it is necessary, we make field-checks and we interview local inhabitants. The time expenditure is usually less than half a day per square kilometer.

The analysis and assessment of landslide hazards

At the second level of assessment and for the production of the actual hazards map, we need data that is more precise and we have to be able to make exact statements. The goal is not only to indicate whether or not an area is endangered but also to assess the degree of hazard and to specify its exact location.

To accomplish this, we continue to follow the steps in the flow diagram on form 1. Whenever a large or small, active or dormant landslide is confirmed or supposed, we investigate the historical data and make extensive field checks. With the help of **form 3,** the form "subroutine historical data", the data is arranged synoptically and evaluated.

The main points of interest concerning each event are:

— the source of the data,
— the type of process,
— the moved cubature,
— the location,
— the course of events,
— the velocity,
— the effects on man and property,
— the causes and the trigger, and
— if applicable, countermeasures.

If a particular unit area has been affected by landslides several times, we try to group events with similar processes and effects into categories. Based on earlier events, we attempt to differentiate between different degrees of endangerment.

We also try to get information about earlier conditions in the unit areas potentially affected by landslides. Written sources usually are not available so that we have to limit ourselves to the information given by local inhabitants.

Extensive field checks as outlined on **form 4** provide information about:

— the type of existing landslides (already documented on form 1 for the first level of assessment),
— geology/geomorphology,
— additional observations, and
— the type of damage to be expected if the hazard occurs.

The comprehensive field checks should be executed according to the guidelines on form 4. However, the actual assessment of landslide hazards cannot and should not be made according to a fixed scheme, and it is necessary for the elaborator to rely on his own judgement and experience.

In the column entitled "specific importance" on form 4, the elaborator is to weight the importance of each answer. The reasons for categorizing an answer as "highly important" or of "no importance" are to be noted in the column "remarks".

The answers in column 1, 2 and 3 only indicate an increasing tendency of danger but not the actual degree of endangerment:

— 0 indicates no tendency of danger
— 1 〃 〃 a low tendency of 〃
— 2 〃 〃 a tendency of 〃
— 3 〃 〃 a high tendency of 〃

Section E of Form 4

Checking types of existing landslides only serves to recall the data from the first level of assessment that was recorded on the first part of form 1 and to make it available for the hazard assessment. As a rule, active landslides should be considered more dangerous than dormant landslides. However, there are exceptions to the rule!

Section G of Form 4

An investigation of the geology and the geomorphology of an area provides us with information about certain bedrock and surface conditions that we know either indicate or affect the possibility of landslides. The structure and the lithology of the bedrock are of particular interest.

The relationship of the strike and dip of the stratification or fracture planes to the exposure and inclination of the slope has an effect on landslide tendency. When the strata run parallel to the slope, the situation shown in column 3, there is a high tendency of danger.

The nature of the bedrock affects the probability of large landslides and of small surficial landslides. Sometimes, large landslides originate in the bedrock. Depending on the composition of the weathered bedrock, the probability of small surficial landslides increases. To classify the loose material, we use the USCS system of classification. The categorization is based on the plasticity index I and on the angle of internal friction ϕ. When there is a change in the water content of the soil, the lower the value of the plasticity index is, the more rapidly the state of the soil is affected. The smaller the angle of internal friction is, the less stable the slope is.

Silty material is generally prone to landsliding. Complicated investigations of the hydrogeological and hydrological situation go beyond the scope of the second level of assessment because of the high time expenditure involved. We try to get information about the situation indirectly by observing and interpreting hydrological features on the surface of the slope. Springs indicate saturation zones, and such zones can act as potential shear surfaces or imply a high degree of water seepage in the ground soil. Landslides are often observed on slopes below wet, flat areas.

Slopes with an angle of 30–40 degrees are the most prone to landsliding. The morphology of a slope may indicate former ground movement. Research has shown that landslides happen very often below or near scarps on a slope. Some plants, like alnus, tussilago, and equisetum, are indicators of humid conditions. Curved and tilted trees can be a sign of mass movement although they do not have to be. Such features should be interpreted with caution.

Section A of Form 4

Here the elaborator's attention is drawn to observations that could be relevant. Experience has shown that, for example, below roads with a bad or with no drainage system, landslides have happened on slopes that had been stable up to that point.

Section D of Form 4

In the field, we should consider all eventualities as well as how and where damages are to be expected.

We differentiate between cases where a house or other property acts as an obstacle to the moving mass and cases where a house or other property is situated on the moving mass.

In both cases we have to try to judge whether an early warning and evacuation of people are possible. Where it appears to be impossible, no residential dwellings should be built. Where an evacuation seems feasible, houses should be built solidly and an alarm system should be installed. Buildings such as churches and schools in which often many people are assembled should not be built in such areas.

In the cases where the house or property acts as an obstacle, we have to check if the mass movement, for example, a small landslide that turns into a debris fiow, could endanger houses or property lying downslope.

At this point, we go back to the second half of form 1, to the main routine landslide check. The second half contains a synopsis of the preliminary assessments and of the final assessment.

Under "Synopsis of Assessment", we enter the provisional results of the individual checks:

— Indications of a very dangerous situation are given 3 points,
— 〃 〃 〃 dangerous 〃 〃 〃 2 〃
— 〃 〃 〃 less dangerous 〃 〃 〃 1 〃 and
— "no indication of danger" is designated with 0 〃

If there is a doubt about the significance of a hazard indicator, the probable danger span is given. For example, the span between degree 1 and 3 is written 1–3, meaning that the indicators show a degree of hazard between less dangerous and very dangerous.

Particularly in large unit areas, we sometimes encounter a rapid change in the degree of hazard within a small area. In such cases we write, for example, 1+2, which means that the indicators show a rapid change from less dangerous to dangerous.

In the course of the assessment, the data in the synopsis are to be weighted differently depending on their significance. The most important factors are the geomorphic and geologic analysis and the historical data.

The actual hazard classification of a unit area is undertaken in the final assessment. Through the evaluation of the data in the synopsis

— the minimal degree of hazard for the unit area as a whole is determined and

— local higher degrees of hazard are indicated along with their location.

The final assessment fulfills the requirements of the second level of assessment (see Fig. 1) and enables us to produce a hazards map for local planning purposes.

Where there is any doubt about the classification of an area, it is clearly shown on the hazards map, for exampe, by screening. If construction is planned in such an area, additional, more detailed investigations are necessary. Such local assessments fall under the third level of assessment because they are more time-consuming and expensive.

Although it is impossible to obtain absolutely accurate results with this landslide check, our results are satisfactory. In particular, we have reached a high degree of objectivity in the assessment process while still allowing for the unique circumstances of each case and while leaving the elaborator some freedom of judgement.

The landslide check was only recently systematically compiled as a whole. However, its individual components have proven themselves to be effective in the past.

References

Grunder, M., 1980: Beispiel einer anwendungsorientierten Gefahrenkartierung 1: 25000 für forstliche Sanierungsprojekte im Berner Oberland (Schweiz). Interpraevent 1980, Tagungs-publikation, Bd. 4, pp. 353-360, Forschungsgesellschaft für vorbeugende Hochwasser-bekämpfung, Klagnefurt.

Grunder, M., Langenegger, H., 1983: Beispiel einer anwendungsorientierten Gefahrenkartierung 1: 25000 für integrale Sanierungsprojekte im Berner Oberland. Schweizerische Zeitschrift für Forstwesen, 134. Jg., No. 4, pp. 271-282.

Kienholz, H., 1977: Kombinierte geomorphologische Gefahrenkarte von Grindelwald. Geographica Bernensia, G4, 204 pp., Geogr. Inst., Univ. of Berne.

Kienholz, H., 1978: Maps of Geomorphology and Natural Hazards of Grindelwald, Switzerland: Scale 1: 10000. Artic and Alpine Research, Vol. 10, No. 2, pp. 169-184. Boulder (Colorado).

Kienholz, H., Bichsel, M., 1982: The Use of Air Photographs for Mapping Natural Hazards in Mountainous Areas: A Study based on the Colorado Rocky Mountains, USA. Mountain Research and Development, Vol. 2, No. 4, pp. 349-358, Boulder (Colorado).

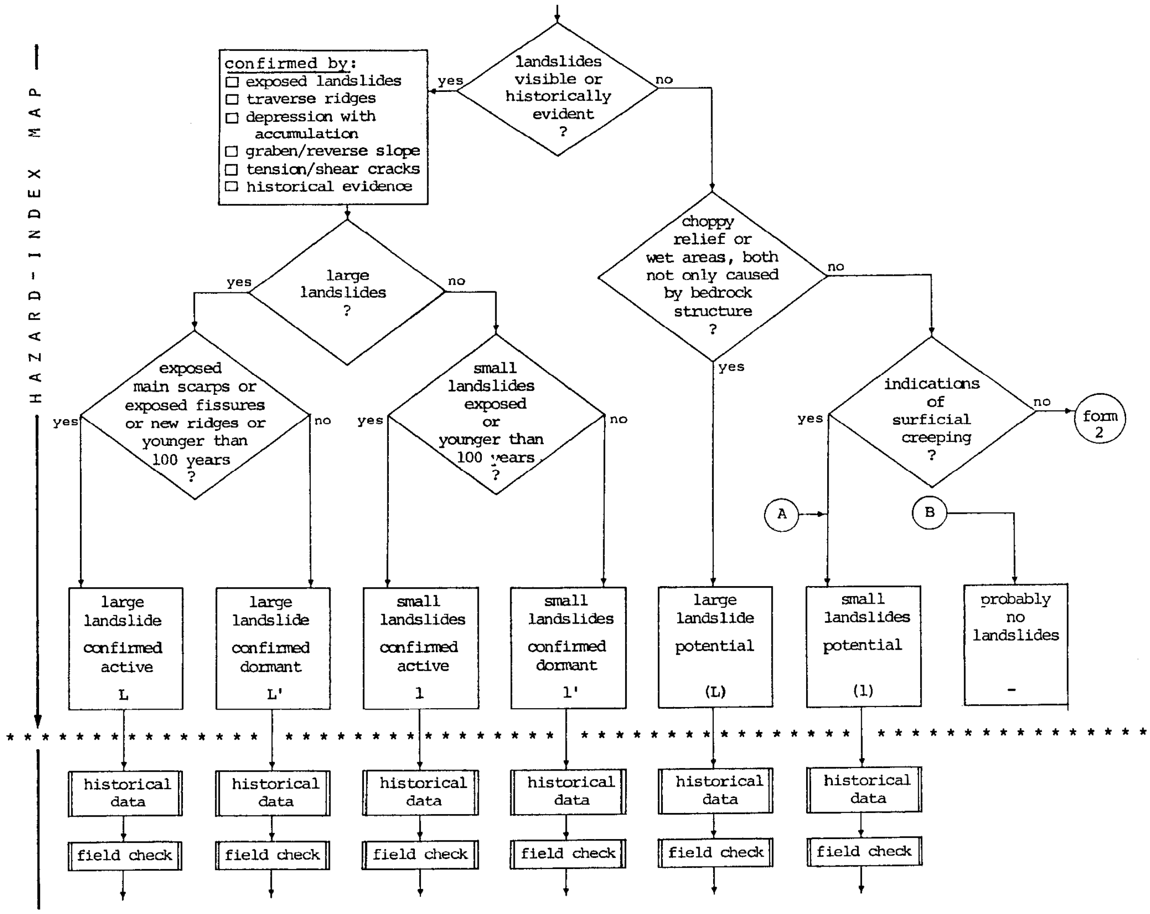
Stiny, J., 1931: Die geologischen Grundlagen der Verbauung der Geschiebeherde. Springer, Wien.

L A N D S L I D E   C H E C K

FORM 1

AREA: _______________

M A I N   R O U T I N E



HAZARD MAP

SYNOPSIS OF ASSESSMENTS			
put applicable number(s)		CLIMATICAL CONDITIONS	C ____
very dangerous : 3	from subroutine	HISTORICAL DATA	H ____
dangerous : 2	from subroutine	DETAILED FIELD CHECKS:	
less dangerous : 1		- existing landslides	E ____
no known danger : 0		- geologic / geomorphic analysis	G ____
min. danger = 1, max. danger = 3 : 1-3 etc.		- additional observations / facts / features	A ____
danger partly = 1, partly = 2 : 1+2 etc.		- type of damages to be expected in case of hazard	D ____

FINAL ASSESSMENT: types of landslides: L 1

degree of hazard:

- minimum for the whole area : __
- local : __ localities: ____________________________
- local maximum : __ localities: ____________________________
- areas below : __ localities: ____________________________

L A N D S L I D E   C H E C K

FORM 2

****

CHECKLIST FOR ASSESSING THE POTENTIAL HAZARD OF SMALL SURFICIAL LANDSLIDES ON SLOPES

No.	QUESTION	ANSWER	POINTS
1	slope angle	30°–40°	2
		20°–30° or 40°–60°	1
		0°–20° or 60°–90°	0
2	linear drainage	none	2
		partial	1
		existent	0
3	type of bedrock (influences the composition of the weathered layer)	rocks with variable friability	2
		solid rocks	0
4	stratification (changing layers etc.)	parallel to the slope surface	2
		disturbed, outcrop bending	2
		other	0
5	subsoil near the surface	Ø of largest components (d_{90}) / depth of the loose material	
		< 20 cm / > 1-2 m	3
		< 20 cm / < 1-2 m	2
		> 20 cm / > 5 m	2
		> 20 cm / < 5 m	1
		no loose material, only soil	0
6	variability of the soil depth (indication by uneven surface)	very uneven	2
		even	0
7	vegetation cover on the slope in question	grass, shrubs, coniferous forest	2
		mixed or deciduous forest	0
8	vegetation cover in the areas above the slope in question	grass on less permeable soil	2
		grass on permeable soil	1
		forest	0

10 - 17 points
9 points (answer to the first question = 2 points) } small landslides, potential ⟶ FORM 1 A

9 points (answer to the first question ≠ 2 points)
0 - 8 points } probably no landslides ⟶ FORM 2 B

FORM 3

L A N D S L I D E   C H E C K

****

SUBROUTINE : HISTORICAL DATA

DATE	SOURCE 1)	TYPE OF PRO-CESS 2)	MOVED CUBA-TURE (m^3)	LOCATION	COURSE OF EVENTS	v (m/s)	REMARKS	EFFECTS TO MAN AND PROPERTY	CAUSES/ TRIGGERS	MEASURES	REMARKS	CATE-GORY 3)

1) SOURCE

code type specification

written
- W 1 official ____________
- W 2 insurance company minutes ____________
- W 3 newspaper/journal ____________
- W 4 chronicles ____________
- W 5 other: ____________

oral
- O 1 expert ____________
- O 2 local official ____________
- O 3 other local ____________
- O 4 other: ____________

2) TYPE OF PROCESS

code	
1	large landsl. rot.
2	" " transl.
3	small landsl. rot.
4	" " transl.
5	debris flow
6	mudflow
7	rockfall
8	other: ____________

3) CATEGORISATION OF THE EVENTS

CATE-GORY	CHARACTER	LOCATION	EFFECTS	RECUR-RENCE INTERVAL	PRELIM. HAZARD DEGREE

RETURN

LANDSLIDE CHECK

FORM 4

SUBROUTINE : DETAILED FIELD CHECKS

INDEX INDICATION	0	1	2	3	IMP*)	remarks
E TYPE OF EXISTING LANDSLIDE	no	-	L' or l'	L or l		1/2/3/4/5/6/7/8 **)
G GEOLOGY / GEOMORPHOLOGY						
1. BEDROCK						
1.1. structure: orientat. / degree / spacing; stratific. .... / ...... / ......; fracture 1 .... / ...... / ......; fracture 2 .... / ...... /	....	....	....	....	..	
1.2. nature (classify and insert the specific rock types of the region under consideration)	solid rocks: ----	solid rocks: conglomerate, limestone	rocks with variable friability: sandy limestone	rocks with variable friability: clayey slate, slaty sandstone, sandy slate, sandstone	..	
2. LOOSE MATERIAL						
2.1. nature (general terms)	coarse rubble	rubble < 20 cm gravel, sand	clayey material	silty material	..	
2.2 nature (plasticity index I) (angle of internal friction) (USCS-classification)	cobbles and/or boulders constituting a framework	$\phi' > 30^{\circ}$ GW,GP, SW,SP	$20\% > I > 10\%$ or $30^{\circ} > \phi'$ GC,SC, CL,CH,OL,OH,MH	$10\% > I$ GM,SM, ML	..	
3. HYDROLOGICAL FEATURES	no spring, linear drainage at the surface		episodic spring	permanent spring dense drainage pattern, wet and flat area above the slope	..	
4. SLOPE ANGLE	$< 10^{\circ}$ $> 60^{\circ}$	10°–20°	20°–30° 40°–60°	30°–40°	..	
5. MORPHOLOGY OF THE AREA						
5.1. situation in the relief	slope	inclined ridge	inclined depression	scarp / step in the slope	..	
5.2. latitudinal partition of the slope	no or little	weak		strong	..	
5.3. longitudinal partition of the slope	no or little	weak		strong	..	
6. INDICATIONS FROM VEGETATION	no		alnus,equisetum,tussilago	bended/ tilted trees !	..	

*) IMP = specific importance: high = 3, medium = 2, low = 1, no = 0

**) specify: 1 = L/L' rotational, 2 = L/L' translational, 3 = l/l' rotational, 4 = l/l' translational, 5 = debris flow, 6 = mudflow, 7 = rockfall, 8 = other: ....

A ADDITIONAL OBSERVATIONS / FACTS / FEATURES	EFFECTS
- roads (good drainage system) ______________	______________
- roads (poor drainage system) ______________	______________
- other constructions: ______________	______________
- water conduits: ______________	______________
- condition of forest: ______________	______________
- ______________	______________
- ______________	______________
- ______________	______________

D TYPE OF DAMAGES TO BE EXPECTED IF HAZARD OCCURS	WITHIN THE CONSIDERED AREA	BELOW THE CONSIDERED AREA
HOUSE/PROPERTY AS OBSTACLE TO THE MOVING MASS		
- early warning/evacuation of people impossible	3	3
- " " / " " " possible	2	2
HOUSE/PROPERTY SITUATED ON THE MOVING MASS		
- early warning/evacuation of people impossible	3	
- " " / " " " possible (damage within hours/days)	2	
- " " / " " " " (" " months/years)	1	

RETURN

L A N D S L I D E   C H E C K

FORM 5

****

CLIMATICAL CONDITIONS

1. Determine the type, intensity and duration of precipitation in the area under consideration; distinguish between confirmed and supposed data:

STATION 1: ____________________________________

altitude: ______ m a.s.l.   coordinates: ________ / ________

source: ____
____

quality: ____

remarks: ____

intensity / duration	supposed	confirmed	recurrence interval
50-100 mm/h, few minutes			____________________
20- 60 mm/h, some hours			____________________
2 mm/h, days			____________________

STATION 2: ____________________________________

altitude: ______ m a.s.l.   coordinates: ________ / ________

source: ____
____

quality: ____

remarks: ____

intensity / duration	supposed	confirmed	recurrence interval
50-100 mm/h, few minutes			____________________
20- 60 mm/h, some hours			____________________
2 mm/h, days			____________________

2. Snow melt conditions:

____
____
____
____

source: ____
____

ESTIMATION OF THE PROBABILITY OF LANDSLIDES BEING TRIGGERED
in spite of the precipitation and/or snow melt conditions:

high = 3
medium = 2
low = 1

要　旨

米国東部アパラチア地域は，米国の中でも最も地すべりの運動が活発な地域のひとつである。ペンシルベニアからウエストヴァージニアに広がるアパラチア台地は大体平坦な古世代の粘土，頁岩，砂岩層から成っており，その中に石灰岩および石炭の層をはさんでいる。台地は開析され比較的急な斜面が形成されている。この地域は更新世の氷河による侵食は受けていないが，その時期の開析谷には氷河によるアウトウォッシュや氷縞粘土などが堆積している。この地域の地すべりは露岩の風化と，粘土岩や頁岩の風化生成物による下方へのクリープなどに起因しており，現在も動きは継続している。テネシーでは，このような基岩の性質に加え，ハイウェイの建設に伴なう斜面末端部のカッティングによりさまざまなタイプの地すべりが発生している。

一般的に見て，米国ではその国土の広さと，それゆえの交通網の整備の必要性とから，日本のような集中的な対策工事の必要性がなく，かつ資本的に不可能といえるであろう。そのために，各地の地すべり現場では「現場主義」ともいえるアイデアに富むさまざまな対策工法が見られる。本資料は調査と対策工事が行なわれたアパラチア各地の地すべりの例を，できるだけ出版された報告に基づき整理したものであるが，この中から米国のEngineering Geologist 達のレベルと活躍の様子をもうかがい知ることができる。

地質的には日本とは全く異なる地域の地すべりの性質を知り，それに対する彼等のさまざまなアプローチの過程と最終的な（「現時点での」）対策を学ぶことは，我々日本の地すべり関係者のみならず資本に余力のない地域の関係者にも資することが多いと考えられるので，その面からの利用も期待している。

本資料で取りあげた地すべり地は，1983 年の第 3 回国際地すべり研究会議（The 3rd International Conference and Field Workshop on Landslides, ICFL）の際に行なわれたフィールド・トリップに基づいている。この会議には初回から当センターの研究員が関与しており，とくに当会議の主催者である USGS のドナルド・ニコルス博士は当センターと「日米非エネルギー分野における科学技術協力協定」を通じて協力関係にある。このような背景があるので，当協定の地すべり分野の日本側コンタクトパーソンをつとめる国立防災科学技術センターが，本資料をまとめることを打診したところ快諾されたのである。