The Sherman Landslide: A Case History

BERKE L. THOMPSON AND BARNEY C. STINNETT

The lower part of a massive colluvium landslide located along the Ohio River near Ravenswood, West Virginia, has affected railroad traffic for an undetermined time. The railroad had unsuccessfully attempted to stabilize it by using timber piles and railroad rails. The roadway above the railroad had not required significant maintenance before 1970. After the pool level of the Ohio was increased by the U. S. Army Corps of Engineers from elevation 551.4 ft to 560.0 ft between 1969 and 1971, the landslide movement became more active. The subsequent repair for the roadway had progressed beyond routine maintenance by 1975 with subsidence of the roadway and a near-vertical 2-ft scarp at the edge of the pavement. In a geotechnical study in 1976 to develop a correction, it was found that the movement at the roadway and railroad was but a small part of the total instability, which included the entire mountainside. The 1,000 ft of movement extended from the top of the mountain to below the pool level of the river. The methods of correction considered were a piling wall, a counter berm, and a relocation. The methods of failure ranged from shallow to deep-seated circular failures, flows, topples, and planar failures. The final correction, proposed in 1976 but constructed in 1984, that satisfactorily solved all of the instability problems was a major relocation of the roadway into the hillside.

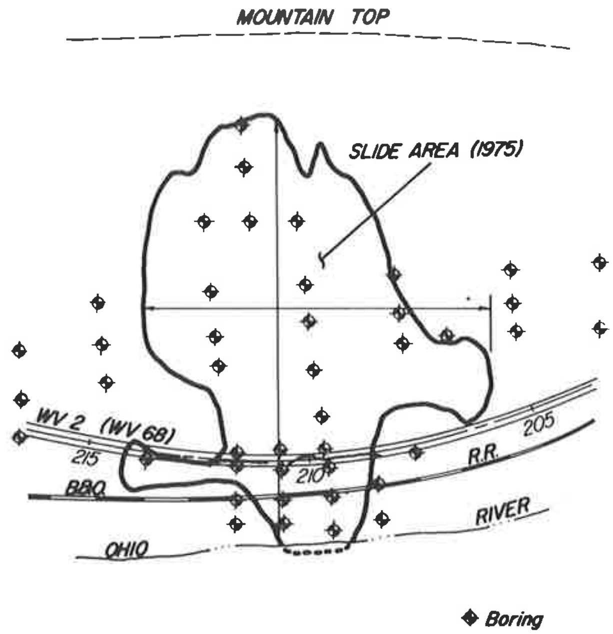

FIGURE 1 Plan view of Sherman landslide.

The Sherman landslide, so named because of a small community nearby, is located approximately 4 mi north of Ravenswood in Jackson County, West Virginia. It became a problem for the West Virginia Department of Highways in the early 1970s. A serious safety hazard resulted when a 2-ft vertical scarp occurred at the edge of the pavement. This scarp became larger as time progressed. A sag of 8 in. was apparent in the roadway, with several longitudinal tension cracks in the pavement (Figure 1).

The highway where the landslide is located was a two-lane expressway (WV-2) in the early 1970s. This highway parallels the Ohio River from Chester in the north to Huntington in the south. WV-2 is an important commercial route because of the industrial plants along the Ohio River. The entire length of approximately 235 mi has always been plagued with rockfalls and landslides that have hampered traffic flow. Because of the nearby mountainous terrain there is often no close alternative route for a detour if a portion of the route is closed.

The failures are due to weak shale bedrock and accompanying poor soil, along with fluctuating groundwater levels and high seepage forces. In addition, the pre-1950 construction of a high percentage of the roadway was by the cut-and-cast method without drainage layers.

This failure, which was surveyed in 1975 for the investigation of the landslide, involved only about 600 ft of roadway. The length of the overall failure zone transverse to the highway was almost 1,000 ft. Although the roadway had not experienced any significant problems before, the situation changed radically in the 1970s. The movement in the embankment next to the river progressed from the initial movement to such a degree that the status and size of the landslide caused it to be considered a critical one. This was one of the reasons that a 37-mi section of WV-2 containing this landslide was rerouted to Interstate 77. The classification of the bypassed section was then reduced from an expressway to WV-68, a feeder route.

The Baltimore and Ohio Railroad (B&O) has a single track 70 ft horizontally and 17 ft vertically downslope from the roadway. After the Racine Locks and Dam were built, the edge of the Ohio River at normal pool was 110 ft horizontally and 34 ft vertically downslope from the railroad. This location had apparently been a problem for the railroad for several years before 1975. During this time the railroad had tried several times unsuccessfully to stabilize their track, once with timber piles and once with railroad rails on both sides of the track. Although the highway surface had required some maintenance before 1970, the pavement thickness did not indicate that any significant leveling courses had been placed in the area before the problem in the early 1970s. The landslide activity at the roadway appeared to occur subsequent to a significant increase in the normal pool elevation of the Ohio River at this location. After the Racine Locks and Dam had been completed, approximately 20 mi downriver, the pool level was raised in stages so that the low lift wicket dams above the dam could be removed and river traffic could still be maintained. Two of these wicket dams affected the pool level at the landslide location—Number

West Virginia Department of Highways, 312 Michigan Avenue, Charleston, W. Va. 25305.

22 below and Number 21 above this location. The pool level was increased from 551.4 to 557.0 ft mean sea level (MSL) on September 20, 1969, and from 557 to 560 ft MSL on August 26, 1971, according to the Corps of Engineers, for a total increase at this site of 8.6 ft.

Although the original investigation of the slide was completed in September 1976 and a correction had been proposed, no action was taken to implement the correction until 1983 because of the cost of the correction and the lack of funds for this classification of highway.

The landslide affecting the roadway was in the lower one-third of the overall instability and continued to take more of the roadway during the 5.5 years after 1976. During this period several minor shifts of the roadway were made into the hillside in an attempt to keep it open to two-way traffic. The landslide movement by early 1982 had failed through the original alignment and had closed one lane of the relocated roadway. Any more shifts into the hillside would have required removal of a significant amount of soil, which could have precipitated a landslide into the roadway. To keep traffic moving through the failure area without any removal of soil, traffic lights were installed for one-way traffic through the landslide.

In 1983, because of continued failure, it became apparent that a permanent correction would have to be constructed to keep the roadway open. The field check in 1984 of the 1976 plans revealed that the lateral limits of the landslide had extended downriver and the instability had become deeper upslope, with much more movement near the mountaintop. The plans were modified as quickly as possible without additional geotechnical field investigation. Although some risk was inherent in this decision, the activity of the landslide and options left to maintain traffic did not allow further delay. Bids for this project were opened on October 30, 1984, and the project was awarded to Lang Brothers, Inc., of Bridgeport, West Virginia.

INVESTIGATIONS

Because of the workload of the department's Geotechnical Section in 1975, it was decided to utilize one of the geotechnical consultants who had a yearly contract to perform this type of investigation for the department. Approval by the department was required before development of any final corrective design by the consultant.

Law Engineering Testing Company was selected to perform the work. The consultant visited the site and prepared an outline of the work to be performed. This included 28 borings, laboratory testing, and a geotechnical report recommending a corrective design. Included in the 28 borings were 6 cased borings to survey subsurface movement. The department authorized the consultant to proceed with the work on September 17, 1975.

Borings

After 25 percent of the borings had been completed adjacent to the roadway, it was found that the soil depths both above and below the roadway were more than 60 ft. The consultant had originally estimated shallower depths for the soil and as a result of this initial boring data, some of the early suggested corrective methods being considered, such as piling, were found to be impractical. The consultant revised the scope of work on November 12, 1975, from 28 to 19 borings, with the approval of the department, in an effort to stay within the original approved engineering costs and still accurately define the failure planes. By January 29, 1976, it was believed that the movement data were representative of the actual landslide movement planes.

Four conclusions were reached by the department and the consultant on January 29, 1976, at the scheduled review:

1. On the basis of the subsurface data, two failure planes are located beneath the highway (Figure 2, Station 210+00). This conclusion was based on indicated movement zones in Borings C-12 and C-11. Boring C-12 above the roadway had a surface elevation of 610.6 ft and Boring C-11 below the roadway had a surface elevation of 608.6 ft. There was only one zone of movement in Boring C-12 at a depth of 19 to 22 ft. There were two zones of movement in Boring C-11, an upper zone at a depth of 9 to 13 ft and a lower zone at 25 to 30 ft. The rate of movement also varied for the two zones of movement. The upper zone, which was in Boring C-11 but not in C-12, had a

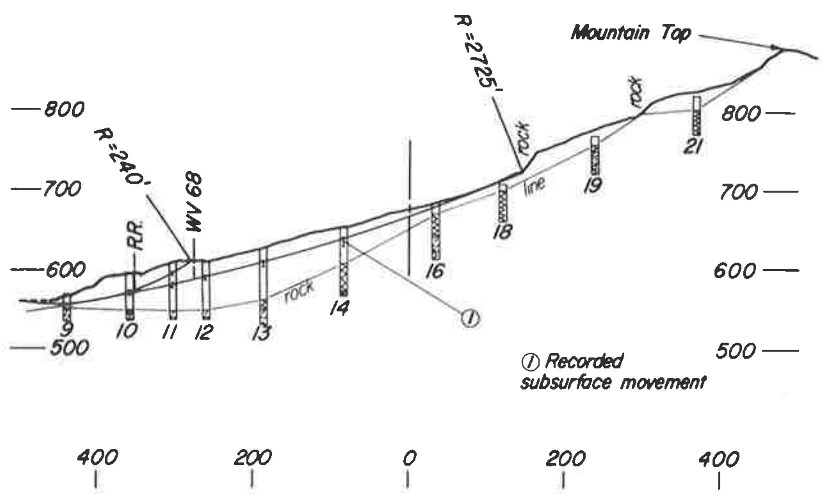

FIGURE 2 Boring locations and failure planes.

measured movement of 1.5 in. in 81 days. The lower zone in C-11 and the zone in C-12 moved 0.25 in. in 81 days. Using the movement data from Borings C-10, C-11, C-12, C-13, and C-14, it was found that the lower deep-seated movement had a scarp 400 ft to the right (east) of the roadway. The toe for this movement was below normal river pool elevation instead of above the roadway as the consultant had previously thought.

2. A berm (buttress) either between the roadway and railroad as proposed by the consultant or in the vicinity of the edge of the river, preferred by the department, will not stabilize the deep-seated lower failure zone. This conclusion was based on the evaluation of the long failure plane. The buttress would control the lower part of the failure, but a new toe for the failure could surface upslope in the vicinity of the roadway.

3. The only feasible correction is a major relocation of the roadway into the hillside and on the bedrock. This would also improve the alignment.

4. Sufficient borings should be completed to design the relocation, including the mountainside cut slope.

As a result of this review, 24 borings were added and all 43 borings were completed by April 19, 1976.

Testing

The consultant performed testing and soil analyses on 38 samples from 10 of the 43 borings completed in this area (Law Engineering Testing Company, unpublished data). The following tests were performed according to ASTM on undisturbed Shelby tube samples and disturbed split spoon samples:

1. Standard Proctor, undisturbed density and natural moisture;
2. Sieve and hydrometer analysis, specific gravity, and Atterberg limits; and
3. Triaxial shear tests (consolidated-undrained, consolidated-drained, and unconsolidated-undrained).

The soil strength values from the triaxial tests were as follows (ϕ = angle of friction; c = cohesion):

	Colluvium Peak	*Colluvium from Failure Zone*	*Alluvium*
ϕ (degrees)	20–24	12	19
c (psf)	300	125	125

The soil plasticity of the colluvium was high. Using the data for consolidated triaxial tests, the permeability was estimated to be 5.1×10^{-5} ft/day.

Geology

The bedrock at the landslide site is from the Permian Period (*1*), Lower Dunkard Series, and is some of the youngest bedrock in West Virginia. It is composed of nonmarine cyclic sequences called cyclothems (*1*) of sandstone and red and gray shales.

A cyclothem is a repetitive cycle of rock types, arranged in a particular sequence (Figure 3). Normally the coal units would

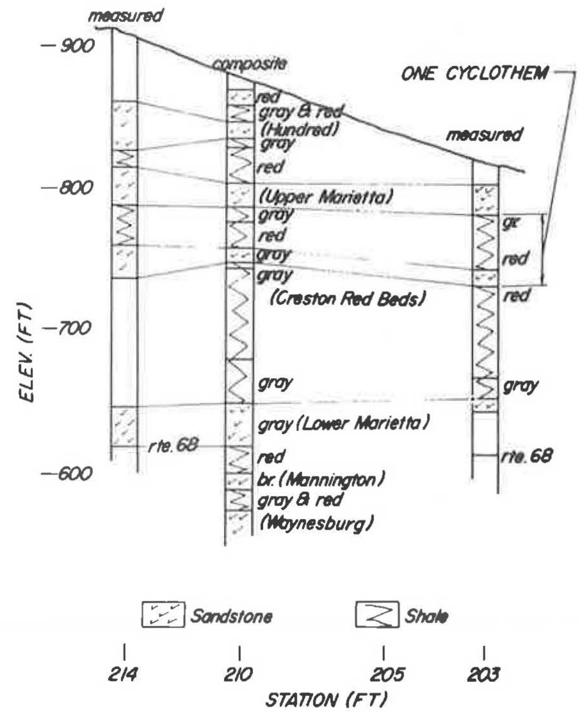

FIGURE 3 Stratigraphic columns.

be used as the boundaries for each cycle. Because no coal units were present, the bases of the sandstone units were arbitrarily used for the boundaries. As with cyclothems in general, and also at this site, all the rock units in a particular cyclic sequence may not be present because of erosion or environmental conditions. This is believed to be due to the variable depositional and erosional conditions at specific sites. The theory of a cyclothem is that the bedrock members that are present will be in the same relative position in all cycles to the other members in a cycle. At this site there are at least six cycles.

The bedrock units along the face of the slope have a significant variance in thickness. Measured sections and a composite boring log for several borings were used to evaluate the bedrock for slope design of the relocated roadway. Three sections are compared in Figure 3. In the center of the 1,100-ft horizontal distance along the centerline of the project, the thickness of the sandstone units decreases by as much as 50 percent. The corresponding shale units are thickened by the amount lost from the sandstone units.

The effect of the thickening of the shale units in the cyclothems in the center of the landslide is fourfold:

1. The colluvium is more plastic;
2. The colluvium has significantly lower strength, particularly with increased moisture;
3. The quantity of colluvium is larger because of a greater mass wasting the weaker shale units of the slope; and
4. A larger weak shale unit occurs below each sandstone unit, resulting in less stability of the sandstone unit.

The well-developed joint system in the sandstone, which was opened by the erosion of the Ohio River Valley, contributes in three ways to the instability of the slope:

1. The joints provide excellent avenues for groundwater,
2. The open-joint system in the sandstone results in free-standing blocks of sandstone (some of significant size), and
3. The underlying shale units are subsequently critically weakened as a result of excess moisture and near-surface weathering, resulting in reduced foundation support for the sandstone blocks.

As a result, large blocks of sandstone become a part of the colluvium because the supporting shale is sheared off or because the blocks topple from lack of support.

Although the Parkersburg syncline is only 6.4 mi east and is almost parallel with the face of the mountainside, it does not significantly affect this location. At the site of the landslide the strike is north 7 degrees east and the dip is 0.4 degree to the southwest. The dip of the bedding plane can be considered as nearly flat and has little effect on groundwater flow or slope failure.

Soil Characteristics

There are two types of soil below the roadway (Figure 4). From the surface to a depth of approximately 30 ft is colluvium, composed of brown clayey silt and silty clay with shale and sandstone fragments.

The soil below the colluvium is alluvial, 25 to 30 ft thick, and extends to bedrock. The alluvial soil, a silty clay with no shale or sandstone fragments, is a terrace deposited by the Ohio River. The elevation of the alluvial soil indicates that it is probably the remnant of the first terrace and was deposited during or shortly after the glacial period (2).

The colluvium extends continuously up the mountainside for a horizontal distance of 630 ft from the river's edge and for a vertical height of 156 ft above the river. Upslope from this elevation, the mountainside has two exposures of weathered shale and sandstone. The remainder of the upper slope consists of a colluvium mixture of soil, weathered shale, and sandstone boulders. The slopes are in various stages and types of failure from the mountaintop down to the continuous colluvium overburden:

1. In the shale, wedge failures, flows, and shear failures;
2. In the sandstone, toppling and wedge failures; and
3. In the colluvium, shearing and flows.

This movement eventually reaches the head of the lower continuous colluvium slope that extends into the river. As it flows or shears over the lower colluvium material, it loads the upper reaches of this mass. The failure of the upper slopes thus keeps the deep-seated lower failure plane active. The rate of failure of the lower deep-seated landslide is thus somewhat dependent on the rate of failure of the upper slope.

Weather Data

Weather data for 1965 through 1984 were obtained from the National Weather Service (NWS) for Parkersburg, West Virginia, which is about 21 straightline miles north of the landslide. This is the closest weather station to the failure that maintains continuous records. Although the daily and monthly amounts of rainfall at the landslide site may or may not compare exactly with the Parkersburg data, the yearly average should be in fairly good agreement. The data were collected for 10 years before the 1975 investigation and for 10 years after the investigation (Table 1).

The data were averaged on 10-year cycles because that was one of the cycles that the NWS indicated that they had used. However, different beginning and ending years were used in this study. The NWS indicated that a variance from the average of 10 in. would be within the normally expected yearly variance from the average. There are no yearly rainfall amounts that would be considered abnormal, although the amount in 1972, 1975, and 1983 was high and that in 1981 was low.

ANALYSIS

A review of the U. S. Geological Survey topographic maps found a significant topographic expression on the river floodplain at the site of the landslide (Figure 5), which had been in evidence in all of the topographic surveys reviewed. It was apparent that this expression was made up of the continuous colluvium slope. The subsurface movement data indicated

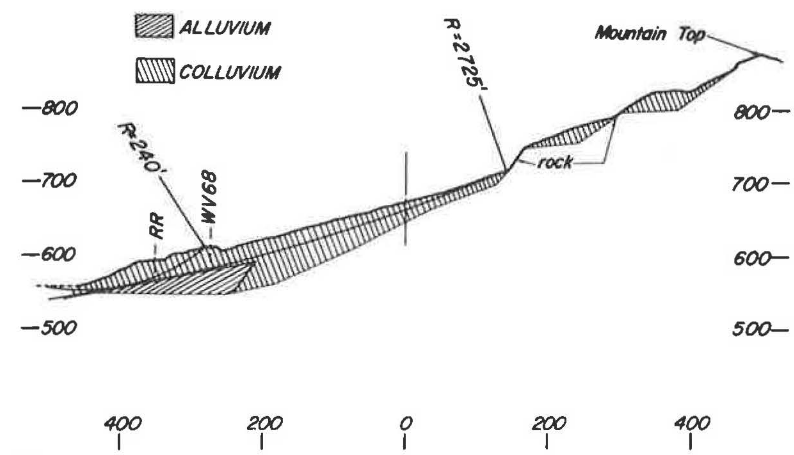

FIGURE 4 Soil types and failure planes.

TABLE 1 NATIONAL WEATHER SERVICE DATA FOR PARKERSBURG, WEST VIRGINIA

	Rainfall		
Year	Total (in.)	Avg (in.)	Departure from Avg[a] (in.)
1965	30.58	37.42	6.84–
1966	32.61	37.42	4.81–
1967	35.04	37.42	2.38–
1968	43.37	37.42	5.95+
1969	31.88	37.42	5.54–
1970	38.32	37.42	0.90+
1971	34.69	37.42	2.73–
1972	47.37	37.42	9.95+
1973	38.89	37.42	1.47+
1974	41.40	37.42	3.98+
1975	45.51	37.35	8.16+
1976	33.77	37.35	3.58–
1977	32.08	37.35	5.27–
1978	36.87	37.35	0.48–
1979	41.04	37.35	3.69+
1980	37.35	37.35	0.00
1981	28.68	37.35	8.67–
1982	36.70	37.35	0.65–
1983	45.16	37.35	7.81+
1984	36.31	37.35	1.04–

[a]A normal departure from average for this site is 10 in.

that the deep-seated (lower) failure plane was located at or near the base of the colluvium just above the alluvium–colluvium interface and was probably the cause of this expression. The shallow (upper) failure from the roadway to the river was also in the colluvium (Figure 4). The safety factor (SF) for the upper failure plane was analyzed by the consultant using the ICES LEASE I Simplified Bishop Method. (See Figure 2 for the location of the upper failure plane.)

An SF of 1.05 for a failure plane generated by the program was in the general area of the upper failure plane. The strength values used were ϕ = 12 degrees and c = 125 psf from the consolidated-undrained strength values of an undisturbed sample taken 18.6 to 19.1 ft deep in the colluvium in Boring C-12. The groundwater (piezometric surface) elevations used were measured at least 24 hr after the borings were complete. A check of the computer-developed plane was performed by using a large-scale model. The data for the model consisted of an SF of 1, the surface limits of the landslide, and the subsurface movement survey depths. The strength values determined were ϕ = 12.5 degrees and c = 125 psf; thus, the computer-developed failure surface indicated excellent agreement with the actual failure surface.

The consultant used an infinite slope analysis to determine an SF of 0.99 for the lower, deep-seated failure plane. The input values used were a depth (d) of 25 ft on an angle of movement (B) of 17 degrees.

No further analysis of the lower slopes was necessary for the relocation correction. The excavation for the relocation at the top of the continuous colluvium slope did, however, remove some of the driving forces from the head of the lower, deep-seated failure plane. This failure plane should become less active after construction of the realignment. The relocation is not expected to create any significant improvement in the stability of the upper failure zone from the existing roadway to the river.

The department analyzed the possible reasons for the increased landslide activity after 1970. Using the ICES LEASE I Simplified Bishop Method, failure surfaces through the areas of movement indicated on the consultant's subsurface surveys were analyzed.

The analysis was conducted by using Borings C-10, C-11, C-12, C-13, and C-14 (see Figure 2), which is a cross section through the center of the landslide. A radius of 2,725 ft was used for the lower deep-seated failure plane and a radius of 240 ft was used for the upper shallow failure plane. The strength values from Boring C-12 at the depth of the failure in the colluvium were used in the analysis. The same strength values were used in the upper shallow failure (Figure 6).

The piezometric surface before 1969 to 1971 was 8.6 ft lower at the river's edge than the 1975 piezometric surface. The two surfaces were assumed to converge at the bedrock at a distance 200 ft right of the centerline of the existing roadway. This analysis indicated a reduction in SF of 10 percent for the upper shallow failure and a 5 percent for the lower, deep-seated failure plane after the pool level was raised 8.6 ft. The weight of the water in the river was considered in each analysis.

The analysis did not consider erosion of material by the river at the toe because the department could not establish whether this had occurred and, if it had, how much material had been removed.

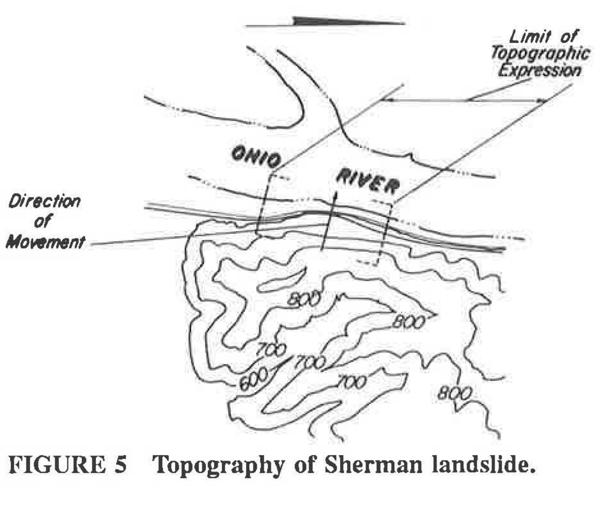

FIGURE 5 Topography of Sherman landslide.

CONTRACT PLANS AND EARLY CONSTRUCTION PROBLEMS

The consultant's final submission in 1976 provided a cut-slope design that removed only a part of the colluvium in the upper slopes. The roadway template based on borings was located on sound bedrock except at each end of the landslide. The consultant had analyzed the hardness and percentage of recovery of the bedrock core. The percentage of core recovery for both the sandstone and the shale varied from 8 to 100 percent. The sandstone generally had higher percentages of recovery than the shale. The rock quality designation (RQD) percentages were generally less than 30 percent. The recovery and RQD values indicated that the bedrock strength was poor with many

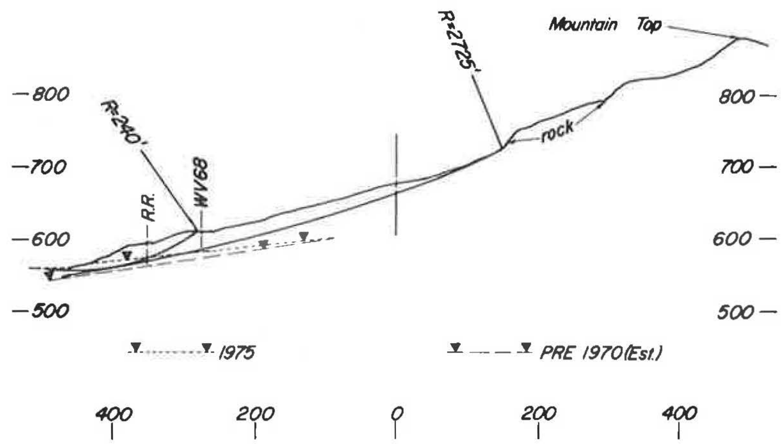

FIGURE 6 Piezometric levels and failure planes.

discontinuities. It was anticipated that there would be a considerable amount of sloughage from the design slopes in service unless gentler slope ratios were used.

The consultant's design was modified without additional boring data for the upper limits and the downriver lateral limits. The consultant's and the department's design consisted of a system of benches and backslope ratios tailored to the designer's opinion of the quality and durability of the material in the slopes. After observing the deterioration of the exposed shale and sandstone outcrops, the department decided to alter the consultant's design to reduce the risk of slope failure (see Figure 7 for the design at Station 210+00). The department's design consisted of five benches with widths from 20 to 30 ft. The backslope ratios were generally 1:1 (H:V) to 3:1 (H:V). These benches and ratios were also used for the slopes in the extended limits of the landslide. The revised design was intended to remove all unstable soil and bedrock. The first bench was located 5 ft above the roadway ditch line to reduce ditch-line maintenance. The controlling factors in the location of the benches above the first bench at 5 ft were as follows:

1. Benches were placed at the base of the sandstone units to provide good foundation support and to ensure against undercutting, and
2. Benches were placed in the shale units at maximum intervals of 40 to 45 ft for storage of sloughage from weathering of the shales. The stored colluvium should cover the shale units and reduce future weathering.

The redesign increased the excavation quantity from 441,290 yd^3, which was the amount estimated by the consultant, to 616,438 yd^3 for the department's redesign, which removed all unstable soil and bedrock, including the lateral and upslope extension of the landslide (see Figure 7).

Several slope failures occurred in the shale units and in one sandstone unit during construction. These failures were the result of the weak and highly weathered shale and the jointed sandstone supported by this shale. Failures of the 1.5:1 and 2:1 (H:V) slopes in the shale required a change of bench widths and a change in several instances to 2.5:1 (H:V) slopes in the shale. These changes increased the excavation by 26,000 yd^3.

The contract documents stated that the contractor was not to cast material downslope during the excavation. This requirement was generally adhered to. However, in constructing a haul road, which involved adding a small amount of material to the upper portions of the continuous colluvium, a landslide was created in material at the southern end of the project that had previously not shown any evidence of movement. This movement on two occasions sheared through the highway and railroad alignment with a total movement of 10 ft horizontally and

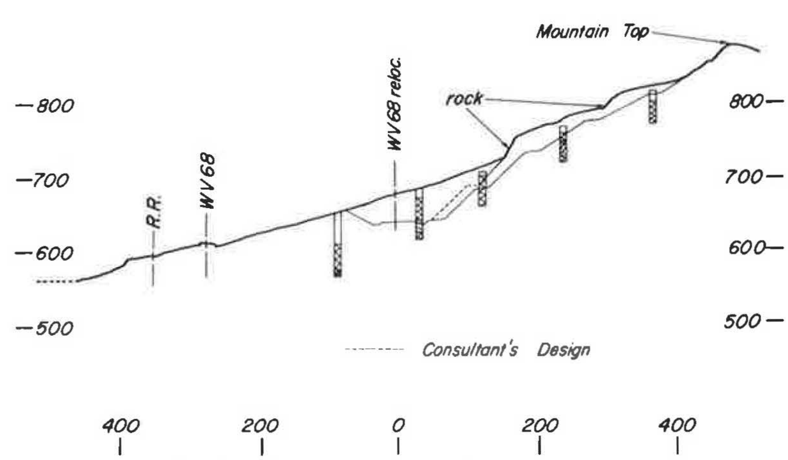

FIGURE 7 Construction cross section (1984).

1 to 2 ft vertically. A subsurface survey indicated movement at a depth of 28 ft, corresponding to the deep-seated failure plane. A toe was in evidence in the river just beyond the riverbank. Traffic has been maintained by excavation near the scarp and leveling of the roadway with aggregate.

CONCLUSIONS

This landslide exemplifies the delicate balance of the stability of colluvium soils on the mountainsides in West Virginia, and especially along the Ohio River.

The increase of the piezometric level within a landslide often lowers its stability to a critical level. The data concerning the rainfall for this general area indicate a somewhat higher amount during the early 1970s and a lower amount during the late 1970s. The yearly amounts during this time were not considered abnormal by the National Weather Service. The activity of the shallow failure surface remained nearly constant during this period. It continued to fail progressively during the 10 years after the investigation, with retrogression upslope, through the original roadway, and finally into the relocated roadway.

Analysis of the landslide indicates that the stability could have been reduced from equilibrium to below equilibrium by the change in the piezometric level as a result of the increase in the river pool elevation. Other factors may have contributed to the instability, such as some removal of material at the toe by the river. However, no man-made changes were made to the surface topography and no other major changes in conditions occurred other than the Ohio River pool change. The analysis indicates that the change in the pool level could have been the major factor causing the increase in the movement of the shallow landslide and probably affected the lower larger failure surface as well.

In studying a landslide of this magnitude, it is highly desirable to understand the mechanisms that have created the instability. This ensures that time and monies are not lost trying to develop corrective methods that will not be adequate now or at some future time.

The department's geotechnical staff recognized the delicate balance of the colluvium and bedrock failure in this landslide. As a result district personnel were advised to not make any significant changes in the topography over the years after the detailed investigation conducted in 1975 and 1976. Otherwise, the roadway and railroad might have been closed, as almost happened during the early construction phase. It was also realized that the construction work in the area could jeopardize the stability. If the landslide did endanger the highway or railroad, it was believed that construction equipment was on hand to relieve the problem.

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