Rockwood Embankment Slide Between Stations 2001+00 and 2018+00: A Horizontal Drain Case History

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The most troublesome interval in regard to landslides on Interstate 40 near Rockwood, Tennessee, was along the eastbound lanes between stations 2001+00 and 2018+00. During an extensive investigation of the slides, subsurface water was determined to be one of the major factors in the failures. At the time that alternatives were being prepared as remedial measures, economical methods for the installation of deep horizontal drains were not available. Initially, minimum stabilization of the slide was accomplished by an alignment shift and grade change. Stabilization of the slide to tolerable limits was finally accomplished by the installation of 3120 m (10 325 ft) of horizontal drains in 1976. Flow records from the drains give insight into the reasons for the success of the drainage.

"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" (1). Construction on this segment, which began in late 1967, was soon delayed by the failure of a massive fill in January 1968 along the eastbound lanes between stations 2001+00 and 2018+00. Ironically, this failure, although it was one of the more than 30 slides that had to be corrected before all four lanes could be opened to traffic in the late summer of 1974, was the last area to be stabilized to tolerable limits. Remedial measures for these slides included partial to total removal, minor grade and alignment changes, various restraint devices, and various drainage dewatering systems such as French drains, vertical wells, and horizontal drains.

By far the most troublesome interval on the Rockwood project was along the eastbound lane approximately between stations 2001+00 and 2018+00. This paper presents a history of this slide from the time of the initial grading in August 1967 through the initial attempts at stabilization and the later installation of the horizontal drainage system. Finally, it is looked at in its present condition.

GEOLOGIC SETTING

The alignment of Interstate 40 at Rockwood traverses the steep southeastward-facing slope of the Cumberland escarpment through approximately 244 m (800 ft) of elevation in the 6.4-km (4-mile) problem area (Figure 1). The escarpment lies at the juncture of two physiographic provinces, the Valley and Ridge provinces to the east and the Cumberland Plateau to the west (2).

The face of the escarpment, in which the strata dip northwest 30-60°, is composed of a rather distinct and identifiable yet highly complex assemblage of soil and geologic materials and conditions (1). The rock formations include sandstone and conglomerate cap rock of the Crab Orchard Mountains and Gizzard groups, highly variegated clay shale that has interbedded siltstones and sandstones of the Pennington Formation, and Newman limestone near the base of the escarpment (Figure 2). Along the escarpment the strata are severely faulted, folded, jointed, and weathered. These lithologies are overlaid by accumulations of up to 15 m (50 ft) of colluvium. The colluvium, or slope debris, consists of a silty heterogeneous mixture of sand and clays that has conglomeratic sandstone fragments that range from the size of a pebble to close to that of a boxcar.

FAILURE MECHANISM

The landslide between stations 2001+00 and 2018+00 involved both a failure zone along the interface between the colluvium and the weathered shale and a deeper failure zone that passed through sandstone, siltstone, shale, and limestone formations that have been severely folded, jointed, and fractured. Basically, the existence and interaction of clay shales, colluvium, and water underlie the principal reason for failure along the interface between the colluvium and the weathered shale. Accumulations of the colluvial veneer vary considerably in thickness over a relatively small area. This variance is due primarily to the troughs and lows filled with colluvium in the highly undulating and irregular subtopography of the area rather than to buildups of colluvium over a uniform surface. The colluvium, which is very permeable, permits ready access to water from both surface and subsurface sources. These sources are highly variable and unpredictable where springs and seep outlets are covered by colluvium. The underlying clay shale and residual clays are virtually impermeable. The difference in permeability allows the groundwater to collect and sometimes become trapped along the contact between the two materials. Trapped water along the interface reduces the shear strength, and failure occurs.

In an effort to locate the zones or strata in which water pressure was the highest and to define the groundwater conditions, piezometers were installed in 10 different borings. The piezometers were set at varying levels based on the relative porosity and permeability as shown by lithology, structure, and degree of weathering. Although the results obtained were quite inconclusive for defining zones of excessive pore pressure or direct sources of infiltration, quite erratic conditions in the groundwater were found. These were attributed to two factors--the varying permeabilities of the severely jointed and fractured lithologies and the weight of the fill, which closed or blocked drainage in the lower formations and tended to dam up the groundwater and produce higher groundwater levels.

The highly irregular groundwater levels made it difficult to establish the effect of groundwater quantitatively. However, the circumstantial evidence of the effect of water was confirmed by stability analyses that showed that the higher the groundwater level, the less stable the mass (1 m = 3 ft):

Water-Table Location	Safety Factor
6 m below fill surface	0.84
7.5 m below original ground surface	1.00
Below failure surface	1.39
At surface, fill removed	1.24

Considering the observed groundwater levels at their deepest, about 7.5 m (25 ft) below the original ground surface, the analysis showed a safety factor of approximately 1. Considering that the groundwater levels had been altered by the fill to be above the original ground surface, the safety factors were

Figure 1. General alignment of Interstate 40 across the Cumberland Plateau (shaded area).

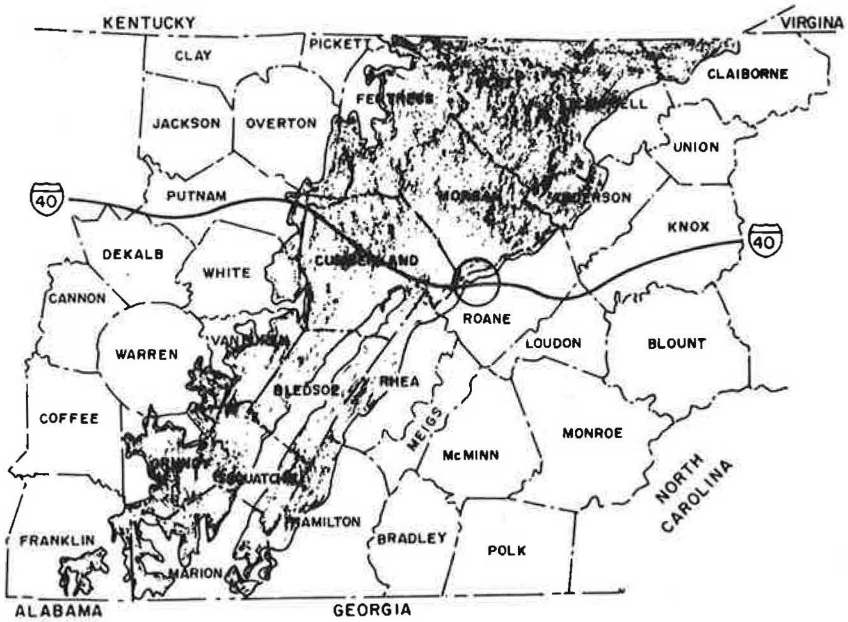
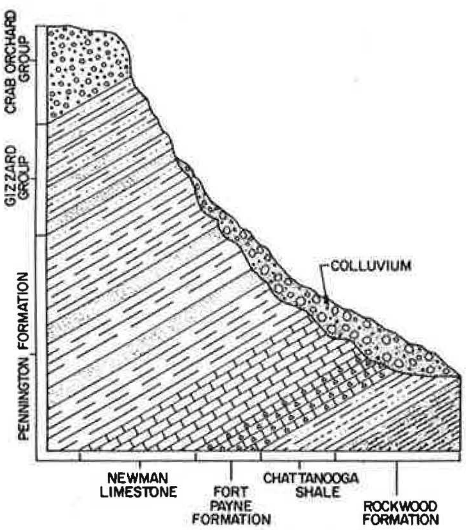


Figure 2. General sequence of geologic formations and material encountered along the plateau escarpment near Rockwood.



well below 1. The actual effective groundwater table was probably between these two limits, equivalent to a calculated safety factor of approximately 0.9 (Figure 3).

It was concluded that water that affected the stability was migrating along the strike of the steeply dipping beds and collecting in a colluvium-filled trough centered beneath the embankment. Movements along the relatively shallow interface--3-12 m (10-40 ft)--were correlated with average infiltration of water or conditions of a perched water table above the interface between the weathered shale and the colluvium. Movements in the deeper failure zone [18-30 m (60-100 ft)] apparently occurred during periods of heavy and prolonged infiltration. During these periods the high water table reduced effective stresses in the lower strata until failure occurred.

CHRONOLOGY OF EVENTS

Construction on the eastbound lane of Interstate 40 began in May 1967 by using clearing and grubbing. Grading of the 42-m (140-ft) high embankment began in August 1967. The initial failure occurred in January 1968 when the embankment was about 27 m (90 ft) below the proposed grade. Grading of the landslide area was halted in March 1968.

The installation of a drainage facility for the slide area was begun in April 1968. The facility consisted of shallow interceptor drainage trenches dug to the left (uphill) side of the embankment. Grading was resumed in mid-April 1968 and, when continuing movement was observed, an effort was made to alleviate excess pore pressures beneath the embankment. A line of vertical shafts 0.9 m (3 ft) in diameter and 10 m (36 ft) deep was drilled outside of the right (downhill) embankment toe and filled with crushed stone. Movement continued, however, and grading was again halted in June 1968.

After preliminary investigations by state highway personnel were performed in the spring of 1968, a consultant was retained to investigate the failure and develop remedial measures. After an extensive investigation in June 1969, several alternatives were developed and cost analyses and degrees of effectiveness were given (3). The alternatives proposed included total removal and replacement, deep drainage that used galleries and wells, drilled shaft restraint, relocation, bridging, and a minimal stabilization plan that included drainage, as shown below:

Alternative	Cost ($000 000s)	Effectiveness (safety factor)
Removal and replacement	4.0	1.4+
Deep drainage	1.75	1.4
Drilled shaft restraint	10.45	1.4
Relocation	1.5	-
Bridge	1.4	1.5
Minimal stabilization	0.6	1.0

Because of costs, the minimal stabilization plan was chosen even though the factor of safety was calculated to be only of the order of that of the original slope prior to construction. The design consisted of removing much of the weight of the existing embankment by a minor alignment change and by lowering the proposed grade about 21 m (70 ft). A drainage system that consisted of interceptor drains at both the uphill and downhill toes of the embankment was included to increase the safety factor to a low, but tolerable, level. This program would involve continued movement, especially during wet periods, and continued maintenance by means of periodic paving on each side of the slide area (Figure 4).

From 1969 to 1972 (when the redesigned embankment was completed) movements continued to occur that totaled 152-203 mm (6-8 in) vertically per year.

HORIZONTAL DRAINAGE SYSTEM

The first horizontal drains in the state were installed in 1972 as part of the remedial measures for several embankment failures on Interstate 75. Since the initial results from these drains appeared to be good, it was felt that additional stability could be gained by installing drains along the toe of the I-40 embankment. These would reduce the driving force of the slide by removing water from the perched water table in the embankment. During

Figure 3. Typical section of embankment at time of failure.

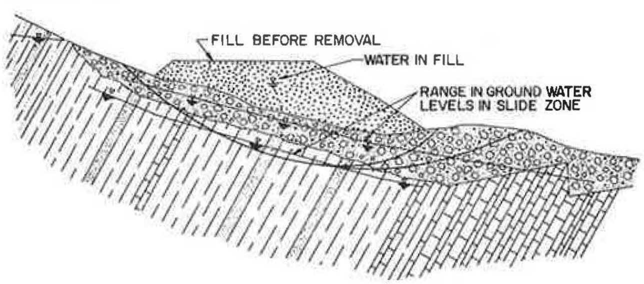
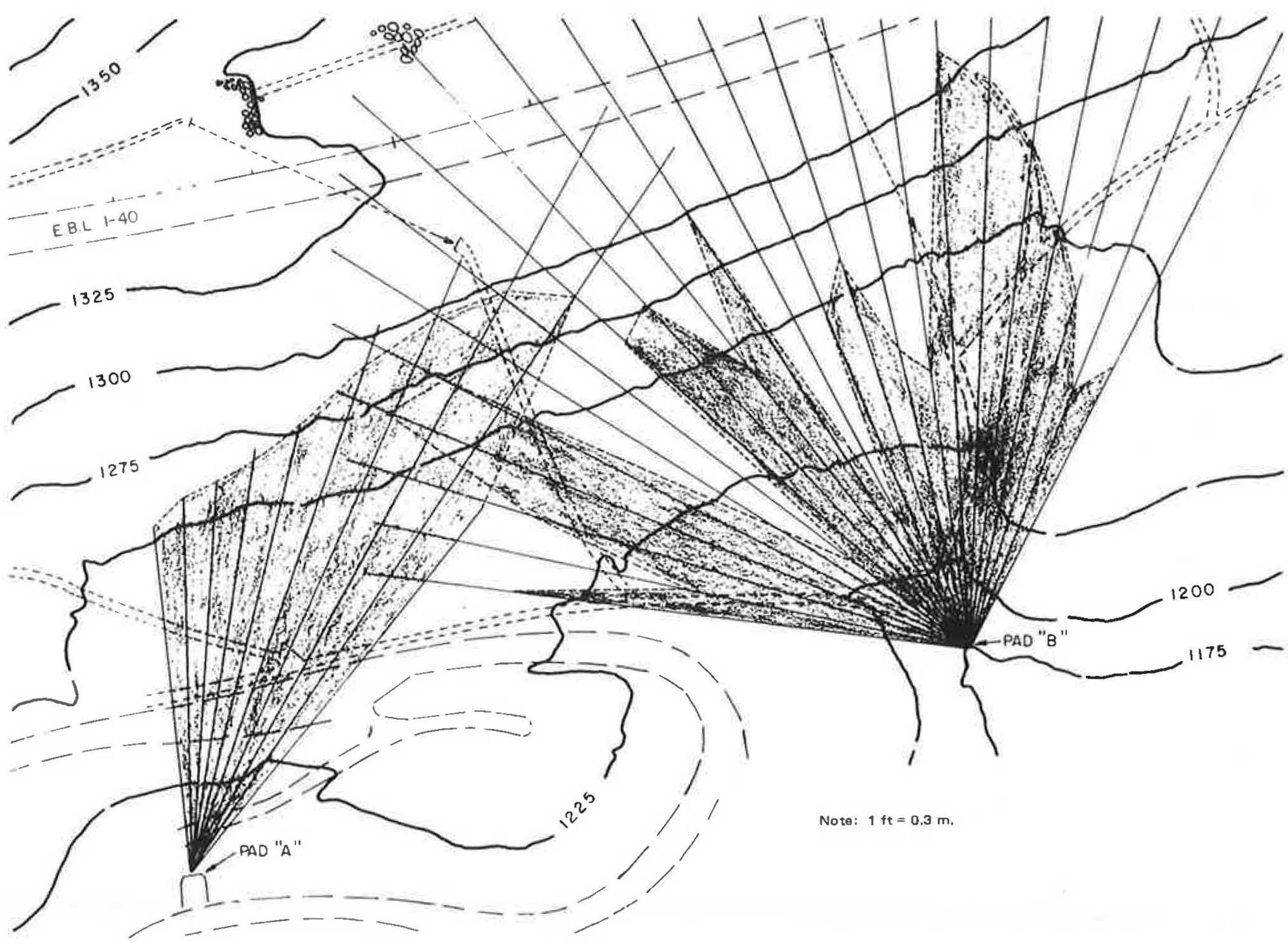


Figure 4. The Rockwood fill slide between stations 2001+00 and 2018+00 (paved areas delineate lateral limits of slide).

Figure 5. Horizontal drain layout at pad A and pad B, 2001+00–2018+00 slide.



the winter of 1972, 44 drains on 5-10 percent grades were installed to lengths of 23-46 m (75-150 ft). Although only approximately one-half of the drains were producers initially, most of them became active during wet periods (3).

Between 1972 and 1975 vertical movement was of the order of 76-127 mm (3-5 in) per year. Although this installation had reduced the amount of movement somewhat, it was felt that additional measures were necessary to further increase the stability of the embankment.

Because of the success of horizontal drains on the I-75 projects it was felt that a modified deep drainage system as proposed in June 1969 could be installed at a much lower cost by using horizontal drains. As a result, a project was begun in August 1975 to install horizontal drains in the two ravines below the fill. Along the slide between stations 2001+00 and 2018+00, 33 drains were to fan out from two centers, one in each ravine (shaded area, Figure 5). At these two centers (pad A and pad B), drilling platforms were excavated at an elevation of 367 m (1175 ft). The excavations into natural slopes of approximately 4H:1V provided a vertical face into which drilling progressed. Grades of 5 percent and lengths that ranged from 27 to 146 m (90-480 ft) were installed in the 10 holes at pad A and 23 holes at pad B. Lengths of up to 183 m (600 ft) were planned for several holes in order to provide drainage from beneath the embankment; however, because of the geologic conditions and the equipment and procedures available at the time, these lengths could not be achieved. Most of the holes were terminated after they had penetrated the heterogeneous colluvium and had been refused in relatively thick [2-5 m (6.5-16 ft)] zones of sandstone (4). Penetration rates were very erratic due to the variation of materials--clay, shale, sandstone--encountered by both drag and roller bits. Several holes were terminated due to lost bits or broken steel. Water used as the principal drilling medium was collected and recirculated during drilling.

Figure 6. Completed horizontal drain installation at pad B.

Schedule 80, type II polyvinyl chloride plastic pipe that had an inside diameter of 38.1 mm (1.5 in) was inserted in each hole as a liner. The pipe, supplied in 6.1-m (20-ft) sections, was inserted through the drill stem as it was extracted from the hole. Because of the variation in the grain size of the materials in the embankment, slot openings of 0.51 mm (0.020 in) were used. After all drains at a pad had been completed, the outer 3-6 m (10-20 ft) of the plastic pipe was encased in corrugated pipe 152.4 mm (6 in) in diameter and the slope reshaped by using shot rock (Figure 6).

Initial rates of flow ranged from drips at pad A to relatively high rates of 2.54-56.8 L/min (0.67-15 gal/min) at pad B. For the most part, although the rates varied considerably, the drains were very active after installation (Table 1). Total flow from the drains varied according to the influence of two factors. One of these--the season of the year--influenced the average total flow. During the wet season (November to June), corresponding to a high water table, flows were generally high. The dry season (June to November), when there was a lower water table, yielded lower flows. For example, average total flows during the wet season were 46.2 L/min (12.2 gal/min) on March 10, 1976, and 66 L/min (17.4 gal/min) on May 21, 1976. During the dry season flows averaged only 2.22 L/min (0.58 gal/min) on August 4, 1976, and 1.92 L/min (0.5 gal/min) on September 1, 1976. The table below presents the total flow rates for each pad throughout the year (1 L/min = 0.264 gal/min):

	Flow (L/min)		
Date	Pad A	Pad B	Total
3/10/76	1.2	44.6	45.8
3/30/76	14.4	106.7	121.1
5/20/76	2.5	63.2	65.7
9/04/76	0.53	1.67	2.2
10/01/76	0.45	1.44	1.89

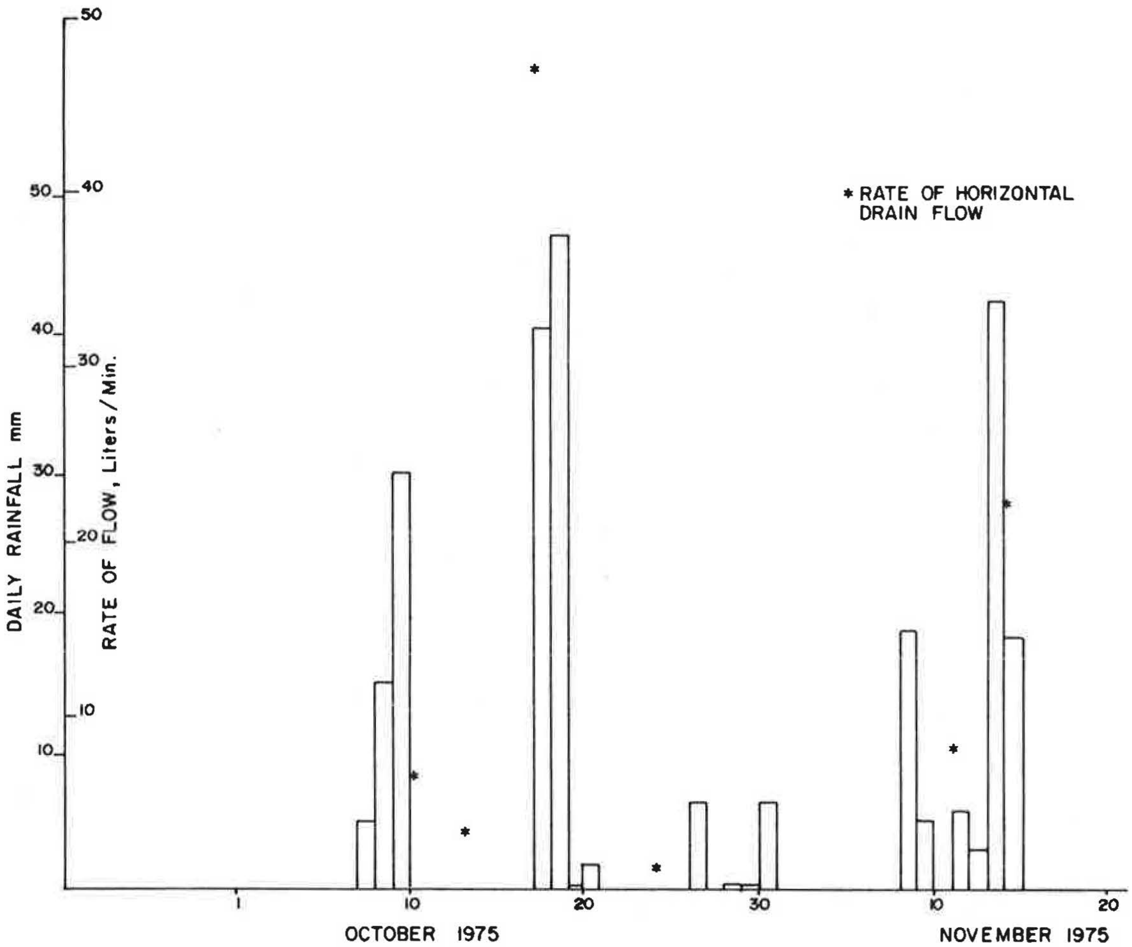
The second factor that influenced the total flow consisted of the almost-immediate response to short-duration heavy rainfall. Figure 7 shows the daily rainfall and the corresponding rate of flow for drain number 23 during October and November 1975. On both October 17 and November 13, the rate

Table 1. Rate of flow of horizontal drains at 2001+00–2018+00 slide, 1975.

Pad	Drain No.	Date	Flow (L/min)	Date	Flow (L/min)	Date	Flow (L/min)	Date	Flow (L/min)	Date	Flow (L/min)
A	5	12/5	0.19	12/8	0.19	12/12	0.38	12/22	0.3		
	6	12/4	1.25	12/5	1.32	12/12	0.08	12/22	0.08		
	10	12/5	0.08	12/8	0	12/12	0.08	12/22	0.08		
B	1	10/6	3.79	10/10	3.03	10/13	2.54	10/22	3.03	11/4	1.25
	3	10/6	7.57	10/10	3.41	10/13	2.65	10/22	3.79	11/4	2.27
	5	10/8	3.22	10/10	3.22	10/15	3.8	10/22	2.54	11/5	1.97
	6	10/8	2.54	10/10	2.54	10/15	2.08	10/22	2.08	11/5	1.25
	18	10/17	18.9	10/20	9.84	10/27	6.06	11/3	3.79	11/24	3.79
	20	10/15	49.2	10/16	49.2	10/23	3.79	10/30	2.84	11/24	0.95
	21	10/14	56.8	10/15	34.1	10/21	9.08	10/28	0.49	11/24	0.08
	22	10/10	3.79	10/13	3.79	10/17	27.3	10/24	1.21	11/10	1.7
	23	10/10	6.43	10/13	3.48	10/17	49.2	10/24	1.4	11/10	0.76

Note: 1 L/min = 0.264 gal/min.

Figure 7. Daily rainfall versus rate of flow at drain number 23.



of flow increased either during or immediately after heavy rainfall.

CONCLUSIONS

Of the 4842 m (15 885 ft) of proposed horizontal drains, 3120 m (10 325 ft) were installed at a total cost of $113 692. This amounted to more than $1.5 million less than the original deep-drainage alternative. Granted that the horizontal drainage system was not nearly so extensive as the original alternative, it has reduced movements at the slide to well within tolerable limits that can be maintained at a reasonable cost.

In the more than three years since the drainage installation was completed (March 1976), vertical movement measured at the scarp where it crosses the roadway (at approximately station 2004+00) has averaged approximately 7.5 mm (0.3 in) per year. When compared with the vertical movements of 152-203 mm (6-8 in) per year before the horizontal drains were installed, the results must be considered excellent.

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