

# Proposed Correction of Landslide for Relocation of Kentucky Highway 9 Using Deep Drainage Gallery

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The proposed relocation of Kentucky Highway 9 places the new alignment in a known landslide area. The geotechnical investigation for the proposed alignment required correction of the landslide problems. Deep drainage galleries were proposed to lower the groundwater tables in the landslide area. The deep drainage gallery consists of a series of interconnected vertical wells with intersecting horizontal drains. A 100-ft-long test section was built to confirm the practicability of constructing such a system, to test the effectiveness of the system in lowering groundwater levels, to see the effects of lowering groundwater on landslide movement, and to evaluate the costs of constructing such a system. Extensive monitoring of the test section was performed using observation wells and slope inclinometers. Groundwater tables were effectively lowered, and landslide movements were significantly reduced. The cost to construct the deep drainage gallery is expected to be 40 percent less than its alternative, a tied-back retaining wall. Results of the test section monitoring were implemented in the final roadway plans.

Recent industrial development in the Cincinnati area is prompting reconstruction of major and secondary roads in adjacent Northern Kentucky. The reason for relocating Kentucky Highway 9 (KY-9) in Campbell County, Kentucky, is to provide access to the Riverport Industrial Area. Existing KY-9 is a winding, two-lane road leading from Interstate 275 (I-275) north to Wilder, Kentucky, approximately 2.3 mi. Reconstruction of this section of highway will tie into the existing four-lane section of KY-9 north of Wilder and the new AA Highway (formerly KY-9), which is south of the I-275 interchange. Landslides along the existing alignment of KY-9 result in continual maintenance problems and expenses for the Kentucky Department of Highways (KYDOH). About 2,800 linear-ft of the new alignment will pass through a known landslide area.

Planning for this section of relocated KY-9 began in August 1984, and several alternative alignments were proposed. Factors considered in selecting the final alignment were (a) providing access to the Licking River, (b) resolving landslide problems that plague the area, and (c) avoiding relocating or disrupting facilities that currently exist in the area. The alternatives proposed were as follows:

- Place the alignment to the east of existing KY-9. The location would be on top of the hill in a more stable area and would not be in the influence of the Licking River.
- Follow existing KY-9 but lower the grade 9 to 10 ft.

- Place the alignment west of existing KY-9 closer to the Licking River. This alternative would provide the best access to the Riverport Industrial Area but place the roadway in known unstable areas.

Because the expressed intention of the project was to serve the industrial base of the area, the chosen alignment was west of existing KY-9. Therefore, the geotechnical investigation for this project required developing a solution to the existing landslide problems that occurred along the proposed alignment. Figure 1 shows a site map of the area showing the proposed alignment.

This paper describes the correction method proposed to facilitate construction of the new alignment, the construction of a test section to study the proposed correction method, the results obtained from instrumentation for the test section, and the incorporation of the instrumentation results into the design of relocated KY-9. The scheduled contract date for this project was March 1992.

## GEOLOGY AND TOPOGRAPHY

Subsurface investigation along the proposed alignment reveals alluvium, colluvium, and terrace deposits as well as bedrock from the Ordovician Age formations. The alluvium, colluvium, and terrace deposits consist of silty clays with minor sands and gravels. The Ordovician Age bedrock consists of the Kope formation with minor outcrops of the Point Pleasant formation. Figure 2 shows a cross section of a typical natural landslide on the Kope formation.

The Kope formation consists of shale (75 to 80 percent) and limestone. It ranges from medium gray to light bluish gray, and when weathered it becomes greenish gray to dark yellowish orange. The shale of this formation slumps readily when wet, and extreme care must be taken to provide adequate drainage for structures placed on the Kope formation (1). A weathered rock zone typically occurs between the shales of the Kope and the overburdened soils.

The shale of the Kope formation shows a very low permeability (2). Small fractures in the interbedded limestone can transmit additional water into the slopes in addition to the natural groundwater that is already seeping downhill in the weathered rock zone. The weathered rock zone is typically more permeable than either the underlying shale or the overlying soils, and landslides in the Kope typically occur in this zone.

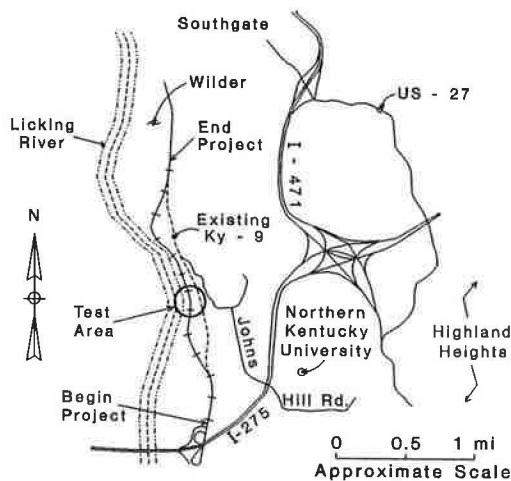


FIGURE 1 Project location map.

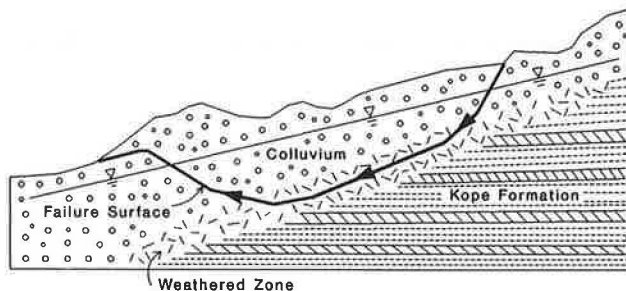


FIGURE 2 Sketch of typical landslide on Kope Formation—modified from Fleming (2).

Groundwater is usually not encountered in drilling until the weathered rock zone is reached. However, water levels measured by observation wells some time after drilling are typically much higher than those encountered during drilling. Therefore, a reasonable explanation for the slope failures is excess water pressures in the weathered rock zone. In addition, increased rainfall and its effect on the water table can drastically accelerate movement.

The Point Pleasant formation lies beneath the Kope and consists of limestone (45 to 65 percent) and shale. This unit is resistant but poorly exposed because of weathering and slumping of the upper-lying, shale-rich Kope formation (1).

Topography of the area is of a hilly nature. However, the existing ground surface in the test section area slopes downward to the west from the existing alignment of KY-9 on approximately a 1-vertical to 4- or 5-horizontal ratio (1V:4H or 1V:5H) toward the Licking River.

## SITE HISTORY

Landslide deposits extend along existing KY-9 from I-275 north to the C&O railroad bridge in Newport, Kentucky. This is evident by the distress of the road and the many pavement overlays required to maintain traffic on existing KY-9. Some of the winding nature of existing KY-9 is due to landslide movement displacing the alignment of the highway.

Several homes and other structures in the area have been abandoned or condemned because of structural damage resulting from landslide movement. Typically, the slopes both above and below the existing alignment are unstable. Movement appears to be of a creep type; however, in some areas the movement is more rapid than in others.

Many landslide investigations have been conducted in this area. In general, the sliding planes of the landslides were located along the weathered shale interface. In addition, excess pore pressures were present in the weathered rock zone. Sliding planes were generally determined by slope inclinometers, whereas observation wells were used to determine water table levels and pore pressures. In many cases, the landslides were not repaired because they did not extensively affect the roadway or because the expense of repair outweighed the costs to maintain the road in its current state.

Correction of a slide on KY-9 immediately south of the I-275 interchange involved construction of a shear key (3). Instrumentation installed at this site indicated that the failure surface was located in the weathered rock zone. Although large movements occurred during excavation for the shear key, this correction method has performed well since its installation in 1974. A perforated pipe placed in the bottom of the shear key collects groundwater and maintains a steady flow even during the typically dry periods of the year.

Slope inclinometers were installed along existing KY-9 alignment in the fall of 1979. These inclinometers were installed as part of a cooperative agreement between KYDOH and the Northern Kentucky Port Authority to provide subsurface information concerning industrial development along the Licking River and for possible relocation of KY-9. Readings were obtained from eight inclinometers from November 1979 to June 1980. As with other slides in the area, excess pore pressures as measured by observation wells were present in the weathered rock zone and failure surfaces were at the weathered shale interface. The slope inclinometers indicated the average rate of movement of the landslide at about 4 to 15 in. of movement per year with a mean rate of 8 in./year. Four of the eight inclinometer casings closed off in 90 days or less; these casings closed off during winter when movement is typically slow.

At the time of this writing a slide in a cut section on the new AA Highway south of the I-275 interchange was being investigated. Observation wells installed in the slide area indicated that excess pore pressures were present in the weathered rock zone. The proposed method of correction for this slide was dewatering using horizontal drains.

## PROPOSED STABILIZATION METHODS FOR NEW ALIGNMENT

The potential landslide-prone area affects about 2,800 linear-ft of the proposed alignment (stations 112+00 to 140+00). Construction in this area is further complicated by its proximity to the Licking River. Static water tables are high because of the proximity to the river, and excess pore pressures in the weathered rock zone are relatively large because of the large hillside recharge area above the area.

Consolidated-undrained triaxial tests were performed on the foundation soils to determine their peak shear strengths.

These tests indicated that the mean cohesion and friction angle were about 450 psf and 27 degrees, respectively. If the foundation soils were to actually exhibit these strengths, the proposed embankments would be stable even with the high pore pressures. However, KYDOH's experience indicates that the residual shear strengths of the foundation soils in this area are much less than the strengths obtained from triaxial tests. Using the slope inclinometer movement data, a failure plane for the test area was determined and backed-in soil parameters were calculated for a factor of safety of 1.0. The backed-in friction angle was calculated to be about 16 degrees (cohesion was arbitrarily set to zero). This value was consistent with the backed-in parameters for other landslide corrections in this area and in the Kope formation in general.

Excavation and replacement of the foundation soils (shear key) is not feasible. The depth of the foundation soils in this area is about 40 ft, and excavation of this material could create further landslides. These landslides would most likely close existing KY-9, which must remain open during construction. In addition, the proximity to the Licking River would make it difficult to keep the excavation dewatered during construction. Stone columns are an effective method of foundation soil replacement, but it would be difficult at this location to penetrate the failure surface (weathered rock zone) adequately in order to prevent sliding. In addition, some method of dewatering would be required for the stone columns to be reasonably effective.

KYDOH narrowed the alternatives to two methods: a tied-back retaining wall and a deep drainage gallery consisting of interconnected vertical and horizontal drains. A tied-back wall at this area would consist of either a soldier pile and lagging wall or a drilled caisson wall. Either type of wall would have been an effective method of correcting the landslide problems, but the cost of constructing such a wall was prohibitive. Some provision for lowering the groundwater table would still have been necessary to provide an adequate factor of safety. The estimated cost in 1989 for a tied-back wall with some provisions for subsurface drainage at this location was \$5,900,000.

The deep drainage gallery system is similar to that used by the California Division of Transportation on a landslide on I-80 near Pinhole, California (4). The proposed deep drainage gallery system is shown in Figure 3. The system proposed for this project consists of a line of drilled vertical wells placed on the uphill side of the new embankment. Each vertical well would have a diameter of 3 ft, be placed on 6-ft centers, and

be drilled at least 10 ft into the formation. The vertical wells would be interconnected either by overlapping belled bases or by interconnecting tunnels. Every third vertical well would then be fitted with horizontal drains that would intersect near the base of the vertical well. The wells would then be filled with free-draining material and capped. This interconnected system would serve as a net to catch and drain groundwater that is seeping downhill.

Because this method of correction had never been used in Kentucky, the KYDOH Division of Materials was reluctant to recommend its use without knowledge of its operations and effects. Without the benefit of this information, the Division of Materials would recommend retaining walls. As a result, approval was granted to construct a 100-ft-long test section, which was to provide the following information:

- The ability to construct the proposed deep drainage gallery system.
- The effectiveness of the deep drainage gallery in lowering the groundwater table.
- The effects of groundwater lowering in the area on landslide movement, and
- The costs of constructing a deep drainage gallery system.

#### TEST SECTION INSTRUMENTATION AND CONSTRUCTION

The area chosen for the test section construction was located near the center of the landslide area between Stations 124+50 and 125+50. Before construction, extensive drilling was performed in the test section area between Stations 124+00 and 126+00. A total of 30 borings were performed in this area, and either observation wells or slope inclinometers were installed in all but three of the boreholes. Monitoring of the instrumentation at the site began in October 1985. Figure 4 shows a plan view of the instrumentation locations.

Construction of the test section drainage gallery began April 1, 1986, and was completed April 30, 1986. Additional observation wells and slope inclinometers were placed in selected vertical wells. Construction of the embankment was delayed until May 29, 1986, because of utility relocation. The embankment was completed on June 28, 1986, with an additional 1 ft of fill being added on July 18, 1986.

Sixteen of the 38 observation wells installed either closed off or were destroyed during construction. However, the obser-

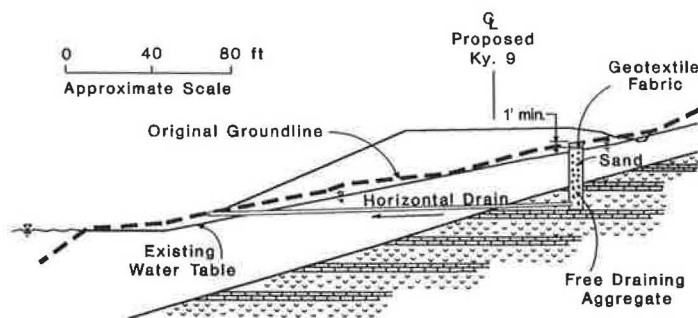


FIGURE 3 Deep drainage gallery system—Cross-section view.

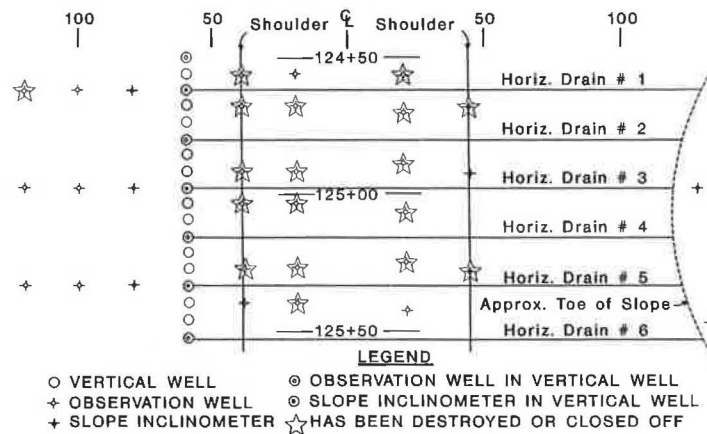


FIGURE 4 Instrumentation plan view.

vation wells that remained open showed an approximate drop in the water table in the drainage gallery area of 10 to 20 ft after construction. This drop corresponded with the water table being at or near the elevation of the horizontal drains. Figure 5 shows a typical cross section of the drainage gallery with anticipated water tables both before and after construction.

Drought conditions were common from the end of construction of the test section until spring 1989. From spring 1989 to present, rainfall amounts have been at or above normal levels. However, only minor fluctuations in the water table have been observed since the test section was completed. A typical plot showing the water table elevation versus days is shown in Figure 6; this plot shows Observation Well 24, Station 125+40 at ground surface elevation of 532.0 ft mean sea level (MSL). The initial reading was taken November 15, 1985.

Monitoring of the slope inclinometers in the test section area indicated a rate of movement of about 1 in./year before construction. Movement during construction was relatively large and was mainly due to dewatering, lateral squeeze from embankment construction, and consolidation of the foundation soils. From the end of construction to April 1991, the average rate of movement was about 0.35 in./year. No significant fluctuations in the rate of movement were noted as a result of precipitation amounts or pool elevations of the Licking River. Figure 7 shows a typical movement versus days

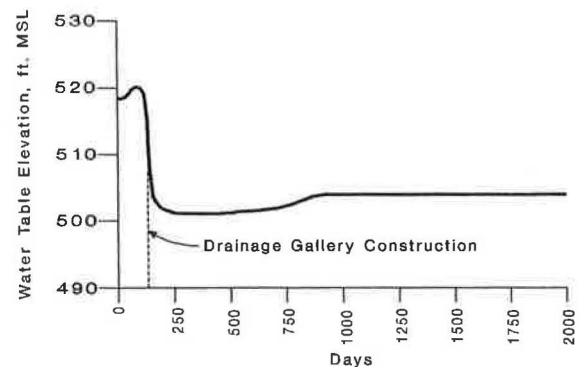


FIGURE 6 Typical water table elevation-versus-days plot.

plot for the test section area; it shows Inclinometer 4 in Campbell County. Depth was 6 ft, and initial reading date was December 3, 1985.

#### IMPLEMENTATION OF TEST SECTION DATA INTO FINAL ROADWAY DESIGN

Monitoring of the test section indicated that the deep drainage gallery was effective in lowering the groundwater level under

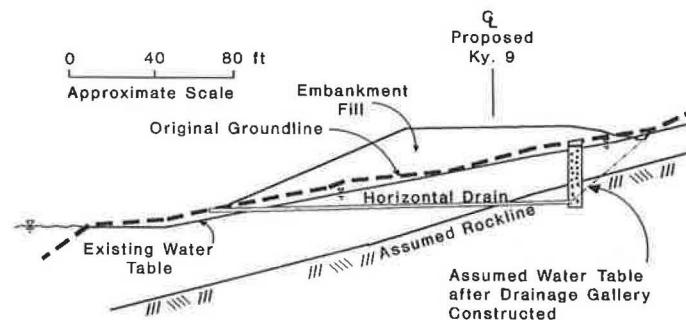


FIGURE 5 Anticipated water tables before and after drainage gallery construction.

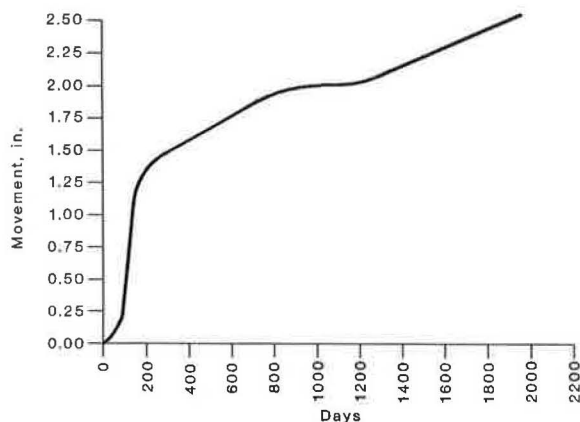


FIGURE 7 Typical movement-versus-days plot for drainage gallery test section.

the test embankment by 10 to 20 ft. The lowering of the groundwater table did not completely stop the deep creep movement of the hillside, but it apparently slowed the rate of movement and prevented some of the large rates of movement from occurring.

Additional drilling was performed along the proposed alignment of the vertical wells to define the depth of rock. Drilling was also performed at selected critical stations for the purpose of performing slope stability analyses. Embankment heights through the deep drainage gallery area averaged about 45 ft.

Stability analyses were performed on the assumption that the groundwater table would drop to the elevation of the horizontal drains. These analyses were conducted using the backed-in parameters and the failure planes determined from slope inclinometer movements (wedge analysis). These analyses indicated that marginal factors of safety ( $FS = 1.2$  to  $1.3$ ) exist for constructed embankments in the drainage gallery area. KYDOH's typical target factors of safety for roadway embankments are 1.4 to 1.8. Figure 8 shows a typical embankment stability analysis for the test section area. Stability analyses (rotational) using the strength data obtained from triaxial tests were also run to further check the stability of the embankments.

Some minor landslide and embankment stability problems are expected to occur during and after construction even though the analyses indicate factors of safety greater than 1.0. These problems are expected because of variations in construction practices, spatial variation in subsurface conditions, and so on. In addition, KYDOH's experience with embankments on the Kope formation with factors of safety less than 1.5 is that settlement, embankment squatting, and creep movements occur.

On the basis of the unit bid items from construction of the test area, the estimated cost for the drainage gallery is about \$2,250,000. Therefore, the expense of constructing a deep drainage gallery was estimated to be 40 percent of that of constructing a tied-back retaining wall. After considering the engineering and economic data, KYDOH decided to proceed with the deep drainage gallery.

Project specifications call for the installation of 441 vertical wells and 147 horizontal drains in addition to those already installed for the test section. Other embankments, cuts, and a reinforced concrete box culvert outside the landslide area were also a part of the scope of the project. Dewatering of selected cut sections will be performed by using horizontal drains. Provisions were included in the specifications for the department to install additional slope inclinometers throughout the proposed deep drainage gallery area so that long-term monitoring can be performed.

## CONCLUSION

KYDOH found that the deep drainage gallery system was effective in lowering the groundwater table at this location. Even with the lowering of the groundwater table, it is expected that long-term creep of the landslide will continue, and it is believed that the rate of movement will be such that typical maintenance practices can maintain the roadway. The average rate of movement for the test section area was 0.35 in./year compared with the average rate of 8 in./year as measured in 1979. Construction of the deep drainage gallery will be cost-effective compared with tied-back retaining walls.

Several questions remain unanswered. Most important is the effect that long-term creep movements will have on the deep drainage gallery. Damage to the drainage gallery could

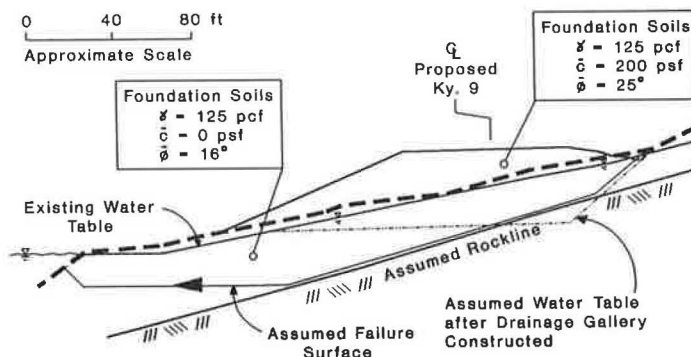


FIGURE 8 Typical embankment stability section (effective stress analysis: using existing water table,  $FS = 1.0$ ; using lowered water table,  $FS = 1.2$ ).



render it useless and result in massive movements. In addition, the test section is relatively short in comparison with the proposed area that is to be treated. It is expected that the edge effects of groundwater flowing around the drainage gallery will be less critical with a longer drainage gallery, and movements are expected to be less with a longer drainage gallery. Instrumentation will be installed to monitor construction of the drainage gallery and to observe its long-term effects.

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