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Use of Horizontal Drains: Case Histories from the Colorado Division of Highways

ROBERT K. BARRETT

Horizontal drains have been used on western Colorado highways for several years, in both preconstruction and postconstruction applications. In specific cases, horizontal drains have proved to be cost-effective in preventing and correcting failures in cut-and-fill slopes. The use of horizontal drains has been limited to specific locations at which subsurface mapping and sampling indicates that groundwater is a major detrimental factor and that it can be effectively collected for a required period of time. This report describes four case histories, all located on Interstate 70 in the Vail area of west-central Colorado. At site 1, horizontal drains were used as a postconstruction alternative to a deep underdrain trench that would have cut across an Interstate roadway. At site 2, horizontal drains were used to improve drainage behind a cut slope. At site 3, the drains were used to improve slope stability prior to construction of a major reinforced-earth wall. At site 4, horizontal drains were used for correction of a major cut-slope failure that occurred during construction.

Horizontal drains have been used by the Colorado Division of Highways (CDOH) for several years, in both preconstruction and postconstruction applications. Experience has shown that, in specific instances of slope instability or potential slope instability caused by free water, drainage by drilled holes that have drains installed at an angle that will permit gravity flow can improve slope stability.

In common use, "horizontal drain" refers to flat or low-angle drilled holes that may or may not be permanently cased. Horizontal drains incorporated by CDOH have been limited to small-diameter boreholes that have a 3.7-cm (1.5-in) diameter slotted polyvinyl chloride (PVC) casing installed for permanent drains.

The case histories selected for this paper are located on Interstate 70 in the Vail area in west-central Colorado (Figure 1). I-70 through this area used the US-6 corridor, and in many areas I-70 closely parallels or replaces US-6.

SELECTED CASE HISTORIES

Site 1: Interstate 70 West of Edwards

An unusual groundwater problem developed in the cut slope north of the westbound lanes of I-70 around station 470, about 3.2 km (2 miles) west of Edwards in west-central Colorado. The westbound cut and the eastbound fill were constructed during the summer of 1969. Nothing was observed during preliminary soils and geologic studies to indicate a high water table, and the cut remained dry during construction (see Figure 2).

Each autumn following construction, the cut slope became progressively wetter yet remained dry during the spring and summer. During late summer and fall of 1976, the water problem expanded further to include the median section. It was feared that, should the trend continue, the eastbound embankment could become saturated and fail. An investigation was begun to determine causes for the seasonal groundwater occurrence.

Geologic conditions on the project were fairly uncomplicated. I-70 traverses the lower portion of a major alluvial fan throughout the problem area. The practice papers concerning horizontal drilling and horizontal drains. There are also apparently no textbooks that cover the subject to any significant degree. But in spite of this sparsity of information, it would appear that the use of horizontal drains is increasing. This is especially true in the eastern half of the United States, in which use in the past has been far below that in the western states. As confidence is gained and as information is disseminated about successful installations, the use of horizontal drainage will no doubt increase as a principal alternative for consideration in the repair and prevention of many types of landslides.

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Figure 1. Location map of I-70 across Colorado.

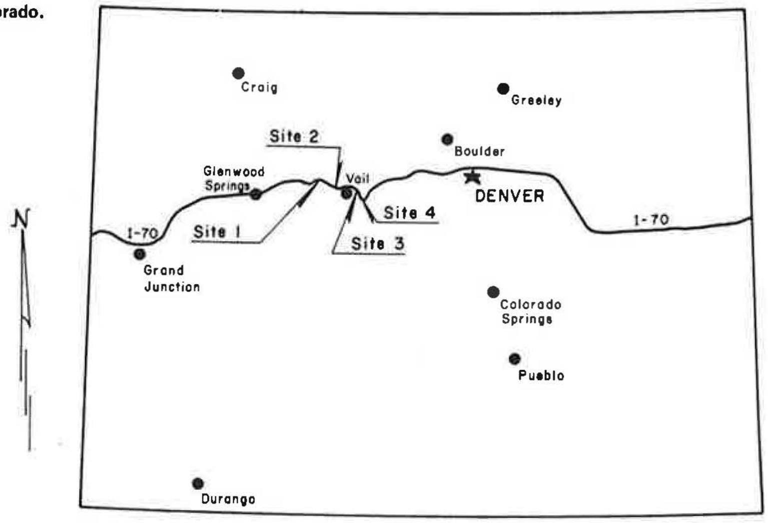
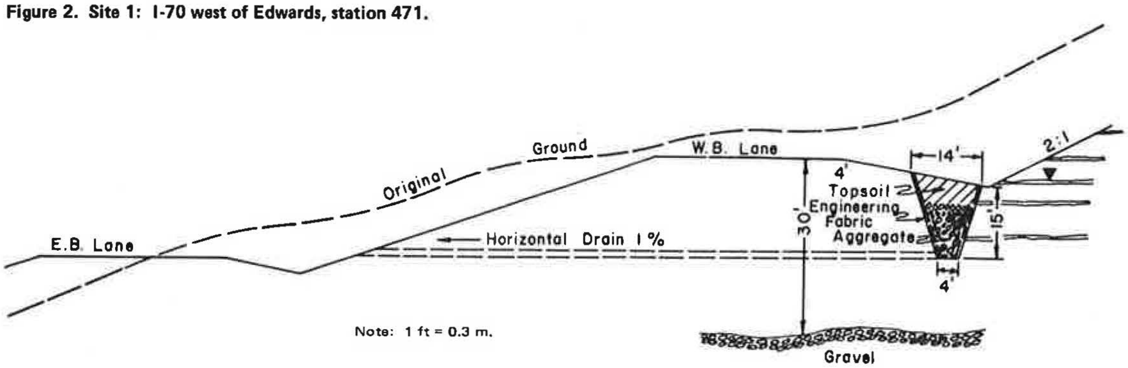


Figure 2. Site 1: I-70 west of Edwards, station 471.



12-m (40-ft) westbound cut through the fan exposes typical alluvial outwash consisting of a locally derived mixture of Eagle Valley Evaporite Formation and Maroon Formation clay, silt, sand, flat pebbles, and cobbles. The eastbound lanes are a fill section that covers most of the lower toe area of the fan.

Vertical drill holes indicated that the alluvial-fan deposit continued in depth about 9 m (30 ft) below the westbound ditch line and lay on an Eagle River gravel deposit. The gravel rests on the Eagle Valley Evaporite Formation. Saturated soils extended only 4.7 m (15 ft) below the ditch. The water that seeped in the lower portion of the westbound backslope exhibited horizontally linear patterns. These patterns appeared to follow lenses of coarser-graded materials, which are probably old stream channels that were buried during depositional (mudflow) events. After such an event, a new channel would temporarily develop on the fan.

It was concluded from the investigation that the seasonal groundwater flow was related to irrigation and snow melt. Water surfaced in the westbound cut from buried stream channels. The eastbound fill acted as a dam as well as compressing underlying soils. These two actions reduced permeability and caused more water to be available to the westbound ditch. Each annual cycle further developed water courses to the westbound ditch area, which caused a greater flow each year at the expense of other, slower paths.

It was concluded that the progressive development of these water courses could lead to sufficient saturation of the eastbound embankment and underlying soils to cause failure and that corrective action was warranted.

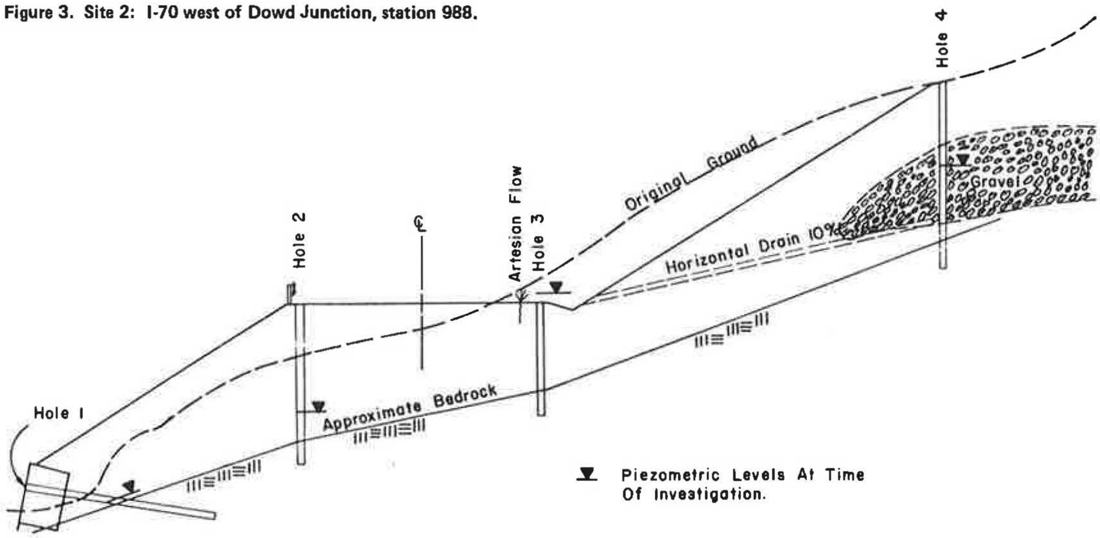
An attempt was made to drain the water into the gravel layer below the westbound ditch line. In semiarid climates, it is not uncommon for fan deposits in steep terrain to be more permeable horizontally than vertically, and improving vertical permeability can alleviate drainage problems in some cases. The gravel deposit proved to be quite impermeable, and none of three vertical drill holes into the gravel would accept a flow of water.

Next, a cost estimate was developed for a typical underdrain correction that included placing 183 m (600 ft) of underdrain under the westbound ditch line. The system would drain into the median inlet of the culvert at station 471+00 and would require cutting a trench about 6 m (20 ft) deep across the westbound lanes. Estimated cost for this work was $53 580.

Construction personnel were reluctant to cut a trench across the westbound lanes. Traffic problems would be considerable, and the trench would be difficult to recompact, which would result in a permanent bump in the road. Several other alternatives were considered, and it was finally decided to try a somewhat unorthodox method that used five small-diameter horizontal drains drilled from the median ditch line and a collection trench lined with engineering fabric in the westbound ditch.

Collection of the water was facilitated by means of the 183-m trench, which was dug in the westbound ditch to a depth of about 4.7 m (15 ft). The trench was lined with a synthetic nonwoven engineering filter fabric and filled to within 1.5 m (5 ft) of the surface with free-draining screened aggregate. After the engineering fabric had been folded over

Figure 3. Site 2: I-70 west of Dowd Junction, station 988.



the aggregate, the ditch was filled with common material and regraded.

The horizontal drain outlets in the median were collected into an open-graded gravel French drain and piped via a short underdrain into the median culvert inlet.

This correction avoided the traffic-control problems. It was constructed at a total cost of \$44 344.66, a substantial saving over the conventional underdrain method (1).

Site 2: Interstate 70 West of Dowd Junction

I-70 traverses a geologically complex area as it leaves the Eagle River and turns up Gore Creek, the west drainage of Vail Pass. The Dowd Junction interchange is located on a landslide of several million cubic meters that has moved more than 0.6 m (2 ft) in the past 12 years and has resulted in continuing maintenance operations to relevel the roadway and readjust bridge girders. Immediately west of that slide, a major cut-slope failure occurred during construction and was corrected by using a rock buttress and slope flattening. Immediately west of these two problem areas, artesian flow began from the eastbound lanes in 1970, two years after construction had been completed.

The roadway template in the subject area consists of a narrow-median, sidehill cut-fill section that has a 1.5:1 cut 17 m (50 ft) high for eastbound lanes and a 1.5:1 fill 17 m high retained by a metal bin wall 7 m (22 ft) high located on the shoulder of old US-6 (see Figure 3). When the water was observed, a field inspection was made of the entire area. The inspection revealed that the bin wall was exhibiting distress. Three vertical members were overriding and crushing at the mid-height connections. Total downward movement at mid-height had amounted to about 5 cm (2 in).

It was feared that the westbound fill had inhibited groundwater flow in the underlying soil and that a pore-pressure buildup had resulted that could overturn the wall. A drilling program was launched immediately to determine piezometric levels in the area.

Several holes were drilled from the roadway template--vertically to develop a cross-sectional view and horizontally through the wall to determine soil and groundwater conditions immediately behind the wall. Groundwater tables, subsurface saturation, and piezometric pressures did not approach critical levels. The soil behind the wall was not saturated and appeared to still conform to original design specifications.

Drilling was then extended upslope above the 17-m 1.5:1 cut. A totally unsuspected major gravel deposit was discovered--a buried channel of the Eagle River. This feature was not detectable on aerial photographs and had not been encountered during subsurface investigations during the preliminary engineering phase. Slope-design borings had not extended upslope far enough to encounter the channel.

The springtime artesian phenomenon in the pavement was determined to be related to snow-melt storage in the gravel. Construction of the fill and removal of a significant loading by cutting probably influenced preexisting, preferred drainage paths for the water.

The problem was resolved by installing a series of horizontal drains from the cut-slope ditch line upward into the base of the gravel formation. By the time the drains were actually installed, the artesian flow had ceased. Small amounts of water were initially encountered in the gravel, but this flow also ceased in the autumn. During the following spring, a considerable flow was observed from the drains, and no water appeared in the pavement.

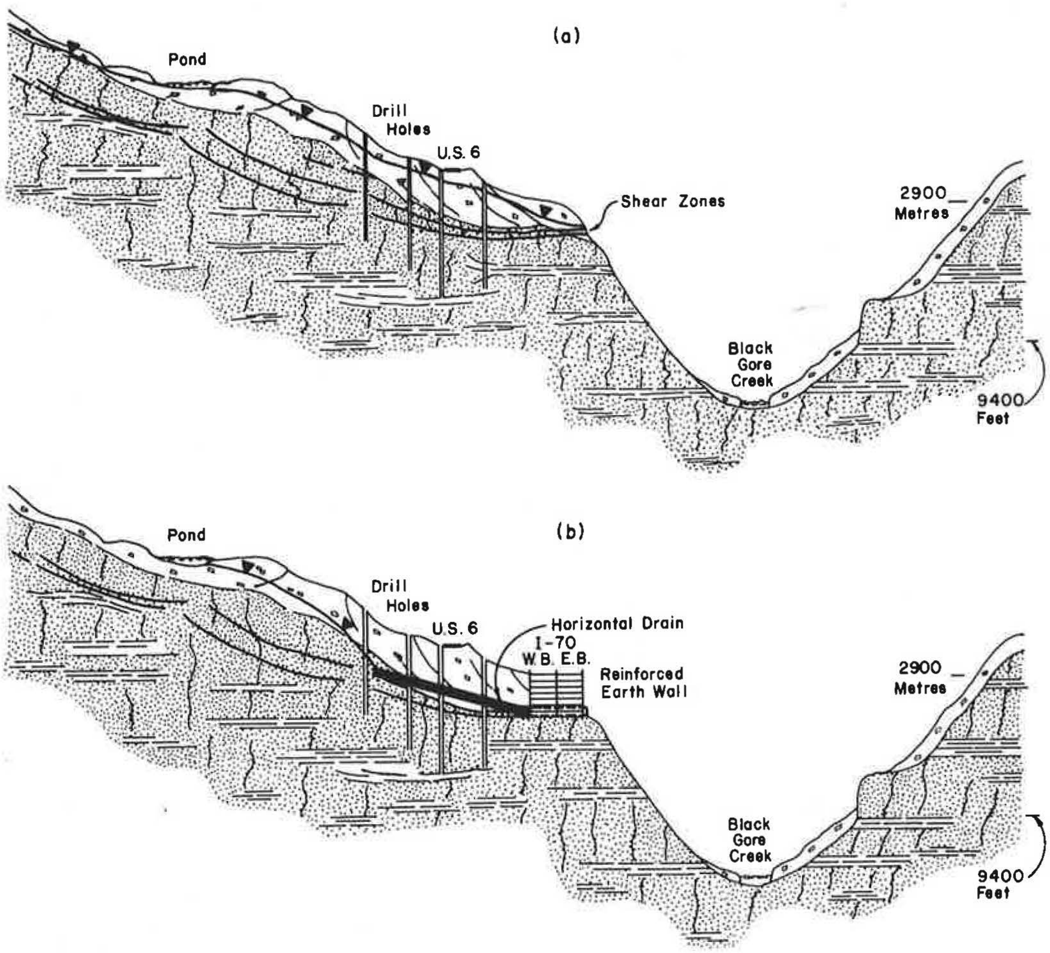
It was determined that movement or readjustment in the bin wall had ceased. The wall was instrumented and monitored for two years; there was no indication of significant vertical or horizontal movement. A likely reason for the distress was some readjustment and settling of backfill material within the bins, which could pull the metal downward and produce overriding at joints in vertical members.

Site 3: Interstate 70, Miller Creek Area, Vail Pass

I-70 over Vail Pass traversed both active and dormant landslides and bedrock failures for 11 of the 23 km (7 of the 14 miles). Geotechnical studies on that section of Interstate 70 encompassed 10 years and involved about 20 person years of geotechnical expertise (2).

The most difficult and challenging problem for geotechnicians on Vail Pass was in the area known as the Miller Creek slide. Maximum dimensions of the slide area are more than 1 km (0.75 mile) long, almost 1 km (0.50 mile) wide, and about 46 m (150 ft) deep. The slide consists of up to 21 m (70 ft)

Figure 4. Site 3: I-70, Miller Creek area, Vail Pass, station 550.



of surficial silty soils that overlie about 24 m (80 ft) of failed bedrock. Water levels varied; in some areas there was water standing in escarpment ponds while other areas were dry (see Figure 4a and 4b).

During the earliest drilling, the lowermost failure plane was thought to be at the interface between soil and rock. Two core drill holes that extended 6 m (20 ft) below this contact indicated intact sandstone and siltstone that dipped about 4°. This matched the geology in the surrounding area. It was found, however, that this did not match a deep core hole drilled previously by the Denver Water Board in conjunction with a proposed water-diversion tunnel (3). A third hole was drilled to 46 m (150 ft) and several obvious shear zones were found. Additional drilling revealed areas of completely disrupted bedrock. Competent, intact bedrock was below this zone. The bedrock failure planes were dipping only 4° in the lower reaches of the slide.

During the course of the drilling, a buried soil profile was discovered in the toe area of the slide. Carbon-14 dating indicated that a sample of organic material from the soil layer was 1000 ± 70 years old, which would indicate that relatively recent movement had occurred.

The topography in the Miller Creek slide area includes a deep canyon below the slide toe into which the slide spills during active periods. Severe grade restrictions, both up- and down-station from the slide area, dictated that the alignment must cross the slide toe. In other critical areas, it was usually possible to vary vertical or horizontal alignment, or both, to optimize alignment and geological conditions. Here, options were few.

The initial proposed alignment included major fills that ramped onto and off the toe area and the roadway in-cut and fill across the toe area. Cuts as long as 15 m (50 ft) were required and, due to the relatively recent movement that had occurred, it was decided that the risk was too great. The safety factor would have been reduced about 25 percent in the immediate area.

A second alignment investigated included twin bridges that had caisson foundations into intact bedrock. In conjunction with this alternative, a consultant presented a European concept that included oversized, elliptical, concrete-lined caisson holes that had free-standing caissons inside. This technique would keep horizontal pressures from acting on the caissons and would allow monitoring of any creep that might develop. Evaluations of existing safety factors, long-term monitoring, and maintenance costs and of the consequences of failure ruled out this alternative.

The alignment ultimately chosen required a high vertical reinforced-earth retaining wall, the toe of which would be placed on the edge of the steep canyon wall and the base of which would necessarily have to be placed on intact bedrock. In place, this design raised existing safety factors by an insignificantly small number; however, excavation of the toe of the slide to permit construction showed a mathematically significant reduction in the safety factor on the critical circle.

A detailed geotechnical investigation followed a tentative decision to try the wall concept. It was decided that, if the water table could be lowered about 11-17 m (35-55 ft) below the groundline for 91 m (300 ft) horizontally behind the excavation, it would be possible to construct the wall. Based on water-level monitoring, it was also decided that the wall would have to be constructed during the winter months--the period of lowest groundwater levels and least groundwater recharge.

In March 1974, a series of horizontal drainage holes was drilled in an effort to lower the groundwater table. A drill access road was attempted along the lower margin of the slide toe along the rim of the canyon. The area was saturated and not frozen. A 2.1-m (7-ft) snow cover had not allowed frost penetration and the bulldozer became helplessly mired. A second road was successfully built about 6 m (20 ft) higher vertically that allowed access of drill equipment. Minimum expectation from drainage holes at this level was to help dry the lower area for a future drainage project.

The horizontal drilling proved successful and interesting. Some intact blocks of rock were encountered that were as large as 24 m (80 ft) across, and almost invariably great bursts of water would flood from the drill holes when the bit broke through and into a shear zone. One such flow was measured at about 575 L/min (200 gal/min). Most flows dropped off substantially in 30 min to 2 h after the aquifer had been tapped.

Water levels were reduced almost 11 m. Due to the low cost and great success of the drainage program, it was decided that it would be worthwhile to drill another series of holes at a lower elevation in conjunction with the construction project. When the second series of drain holes was completed more than two years later, the toe area had, in the interim, dried substantially. The second series lowered water levels to about 17 m below the ground surface within the roadway template.

Prior to construction, three inclinometer holes were installed upslope to monitor any movement that might result from the excavation. The construction plan included very stringent procedures to minimize the amount of slide-toe area excavation to be left unsupported at any given time. These restrictions, coupled with a winter work requirement, caused considerable consternation both to CDOH construction personnel and to the bidding contractors. Some painted a bleak picture for success of the plan.

An unusually dry autumn and snow-free beginning of winter in 1976 permitted construction to proceed as planned. The backfill for the retaining wall was all coarse [2.5-20 cm (1-8 in)] gravel and cobblestones obtained from tailings from early gold-dredging operations in the Blue River. This material was workable at all temperatures, and achieving density even on subzero days was not a problem. The high [21-m (70-ft)] steep (1:1) temporary slopes in the slide toe remained stable and no movement was detected by means of the inclinometers.

This project was completed the following summer and opened to traffic that fall. The inclinometers were read on a weekly basis through the first spring and monthly for the following three years. The drain outlets were observed and the wall was inspected at least bimonthly for two years after completion of the project. No movement has been detected by the inclinometers, the drains are continuing to function, and no distortion can be observed in the wall.

Site 4: Interstate 70 West of Vail Pass Summit

The first major Vail Pass construction contract was awarded in 1979. During the fall of 1974, construction had proceeded to the 33-m (100-ft) level of a through-cut at station 712 planned to be 46 m (150 ft) deep when water began to flow from the floor of the cut. By the following day, tension cracks had appeared 91 m (300 ft) upslope. The contractor was ordered to cease work in the immediate area. An investigation was initiated immediately, which included drilling and instrumentation (see Figure 5a and 5b).

Based on geologic information gathered during the slope-design phase of project development, this cut was being constructed in a failed bedrock slope (4). In fact, these failures extended up and down the valley continuously for more than 3.2 km (2 miles). Drill information indicated that the bedrock (composed of red micaceous sandstone and siltstone and some shale) had been reduced to rubble in many areas. No groundwater table had been observed in the immediate area of the incipient failure.

Although it was recognized that cutting into this slope carried an element of risk of failure, several factors were considered that suggested this risk was acceptable. The bedrock failures were all quite old and apparently had occurred at approximately the same time. The failures had reduced the bedrock to rubble in many areas, which had destroyed the thin shale slip planes. Study indicated that the area was significantly more stable now than at the time of the initial failure.

Alternatives to the cut were not attractive. Avoidance of the cut would have required a twin viaduct in the narrow Black Gore Canyon. This alternative was explored, and the cost was $2 million more than the cut. In addition, the viaducts would have significantly affected Black Gore Creek proper. Placing Black Gore Creek in a culvert and filling the canyon was not even suggested due to environmental concern.

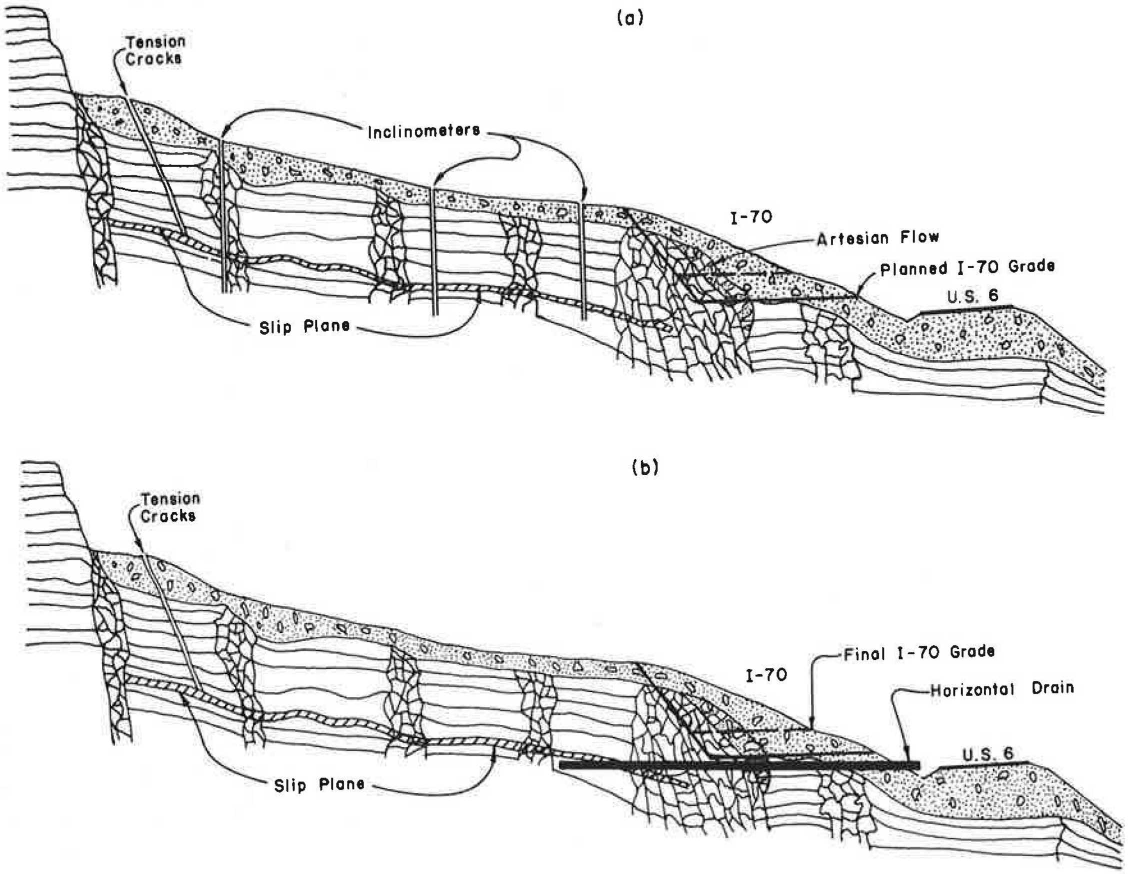
The investigation revealed that the failure was a reactivation of an original failure along a very thin, weak clay shale seam that underlay a sandstone-siltstone sequence. This dipping slip zone was impermeable to the extent that groundwater trapped underneath was under artesian pressure. Exit paths that appeared in the cut floor were quite selective, even though the cut proper was in a highly rubblized zone. It was apparent that a vertical drill hole could have missed the groundwater. The single upslope slope-design drill hole was also drilled in a shear zone. Hence, it was felt at that time that a 1.5:1 slope would have a reasonable probability of stability. Drilling and instrumentation subsequent to the failure located failure planes and piezometric levels in the slide. It was apparent that high piezometric levels were the cause of the failure.

Installation of horizontal drains began some three weeks after the problem was first observed. By that time, total movement exceeded 2 m (6 ft) in some areas, and the failure had spread both upslope and laterally until about 152 000 m^3 (200 000 yd^3) were involved. Drain holes were drilled below the slip plane from US-6, which was about 12 m (40 ft) vertically below final grade and about 18 m (60 ft) below the slip plane.

The initial holes produced spectacular results. Flows that exceeded 378 L/min (100 gal/min) were encountered in the vicinity of the contact between the slip plane and the shear zone. Water in the grade quickly dried and piezometric levels dropped substantially over the lower portion of the slide. Movement slowed and ceased shortly thereafter.

The final correction included a grade raise of 6 m (20 ft) along with provisions for positive, long-term flow capability from the various drains. Total cost of the slide correction was $785 000, due

Figure 5. Site 4: I-70 west of Vail Pass Summit, station 712.



in part to the earthwork imbalance caused by the grade raise. (No cost was assigned to the adverse grade that was introduced.) Thus, even with the failure, the cut alternative was less expensive than the viaduct.

CONCLUSIONS

Water is a major contributor to practically all stability problems on Colorado highways. Drainage is one of the obvious choices for correction of cut-and-fill failures; however, it is a method that seldom can be used alone with confidence. The cases described herein were all successful, but none of the CDOH failure corrections to date rely solely on drainage. Drainage has been used without additional measures for prevention, however, as at sites 1 and 2 discussed in this paper.

There is a tendency to be overly conservative when choosing drainage as one feature of a failure correction. Buttresses and other features are frequently designed for maximum hydrostatic loads on the assumption that drainage installations may cease to function. This redundancy is sometimes unwarranted and expensive. However, there are enough uncertainties associated with long-term reliance on drainage alone so that the redundancy can sometimes be justified.

Some type of drainage is often required to complete the construction phase of an instability correction. Whether or not this drainage is considered permanent and incorporated into the overall design is a question that must be considered carefully. Consequence of failure of the correction is often a guide to the degree of reliance on a drainage system.

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