effective construction program and will continue to do so in the future.

Vail Pass is a milestone in Colorado highway construction history by which all past and future highway projects will be measured.

ACKNOWLEDGMENT

I would like to give special thanks to Michael Tupa of Engineering District 6, Colorado Department of Highways, for his assistance in preparing this paper.

# Final Geotechnical Investigations on the Vail Pass Project

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**An overview of final geotechnical investigations on the 22.5-km (14-mile) Vail Pass I-70 alignment is presented. The geotechnical studies involved personnel of the Division of Highways, Colorado Department of Highways, and consultants from four engineering firms. The Interstate corridor traversed several areas where geologic conditions posed extreme difficulties. Unique and innovative designs were required to provide a stable, attractive, and environmentally compatible highway. Examples of the geologic problems encountered and the techniques used to surmount them are described. Approximately 30 person years of geotechnical expertise and 10 drill-crew years were required on the project. The geotechnical studies are estimated to have cost $2 million.**

Final geotechnical investigations on the 22.5-km (14-mile) Vail Pass I-70 project began in 1971 after completion of preliminary investigations by a consultant in engineering geology. The preliminary and intermediate phases included four years of investigation by engineering geologists of the Division of Highways, Colorado Department of Highways; consulting engineering geologists; soils engineers; and rock-mechanics engineers.

Preliminary and intermediate investigations identified many kilometers of potentially unstable areas where the routing of a four-lane Interstate highway could be adversely affected by rock, soil, and snow slides. It was recognized at the end of these early studies that the final alignment would have to be selected in conjunction with a final, comprehensive geologic investigation.

From 1971 to 1977, geologists and soils engineers worked closely with both consulting design engineers and Colorado Department of Highways design and construction engineers in selecting the optimum location for the roadway. This report describes some of the investigations conducted during that period and explains, from a geologist's viewpoint, how and why various alignment and alignment-related features were finally selected. The geologic symbols used in the figures are explained in Figure 1.

## GENERAL GEOLOGY

The final geotechnical investigations added the third dimension to the geologic maps assembled by Charles S. Robinson and Associates and R. V. Lord and Associates and provided soil and rock engineering properties and groundwater data. Aided by geologic maps; black-and-white, color, and infrared photography; and seismographs and drills, it was possible to create a complete picture of the recent geologic history of the Vail Pass corridor and to predict the impacts and consequences of various alignment alternatives.

It was discovered that a glacier had caused deposition of extensive silt, sand, and organic horizons high on the mountainside just east of Bighorn Creek at the west foot of Vail Pass (see Figure 2). It was probably this same glacier, originating from main Gore Creek, that dammed Black Gore Creek and caused widespread fine-grained lake sediments to be deposited on the hillsides for a distance of 1.6 km (1 mile) up Black Gore Valley.

Many of the bedrock failures were drilled, and much was learned about the failure machanisms. It was concluded that the failures originated on extremely weak, thin, fine-grained, silty, micaceous shale lenses within the predominately sandstone and siltstone bedrock. Bedrock failures on slopes as flat as 4° were observed. In the areas between stations 615 and 680 (Figure 2), both valley walls of Black Gore Creek have failed from near their crests to below the level of the creek—a vertical distance of more than 610 m (2000 ft).

Soils were sampled and tested for strength and permeability. The predominant soils at Vail Pass, classified AASHTO A-2-4 and A-4, were deceptively poor for highway purposes. These soils were highly micaceous and relatively impermeable. Although visually the soils all appeared to be similar, R-values determined by using the Hveem stabilometer varied from 5 to 82 (the stabilometer test is a measure of the relative stability of soils on a scale of 1-100), and triaxial tests indicated a wide variation in strength parameters.

## SELECTED GEOLOGIC PROBLEMS AND SOLUTIONS

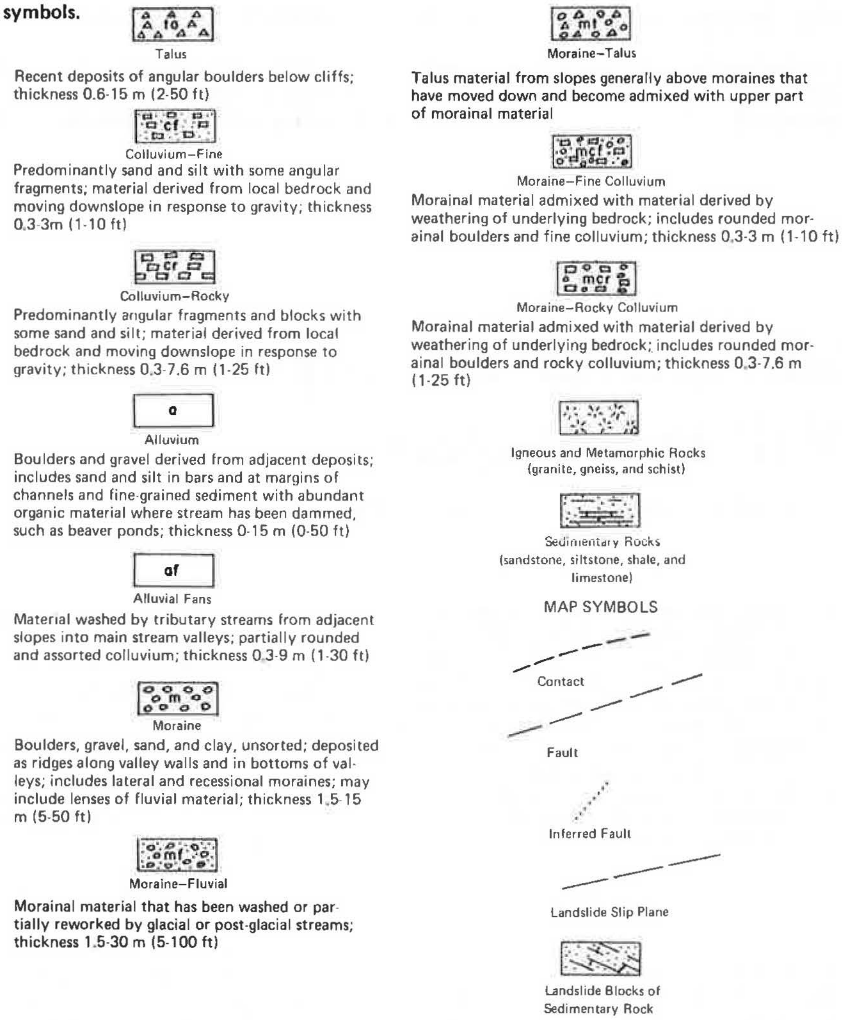
### Stations 310-340

The consultant's preliminary studies concluded that the hillside in the vicinity of stations 310-340 probably contained deep deposits of unconsolidated materials. Concern was expressed about the extensive side-hill cuts proposed for that area (see Figure 3).

In 1972, drilling was initiated in the area. The soils initially selected for drilling consisted of granite boulders as large as 0.9 m (3 ft) in diameter in a matrix of sand, gravel, and cobbles. Since these materials could not be penetrated by the standard rotary drilling equipment owned by the Colorado Department of Highways, they were drilled by a private firm using a percussion drill.

The drilling program verified that deep, uncon-

**Figure 1. Explanation of geologic symbols.**




solidated glacial deposits were perched on the mountainside above the proposed extensive cuts. These deposits consisted primarily of sand overlaid by gravel and boulders and contained organic horizons at various depths. The deepest deposit exceeded 43 m (140 ft), and the water table was near the surface. Had cuts been attempted in this area, extensive failures—probably involving more than 900 000 m (1 000 000 yd)—would have resulted.

Based on the drilling information, it was recommended to the designers that no cuts be made in this area. Two alternatives remained: a standard fill slope or a side-hill viaduct. Partly because of extensive condominium development and high land values, it was determined that a viaduct was the better choice.

During foundation investigations for the structure, it was discovered that the cross-slope safety factor was only 2.0 in the existing hillside in the vicinity of one of the piers. Precedents established on Interstate projects required a 3.0 safety factor on structure foundations. After considerable review, highway department management and personnel of the Federal Highway Administration agreed to accept a factor of 2.0 in this area.

## Station 425

In the area of station 425, geologic mapping and subsequent instrumentation by a geologic consulting firm defined an area in which two active landslides were moving from opposite sides of the valley and constricting Black Gore Creek. After intensive studies and analyses, it was decided that an extensive earth buttress would successfully stop movement in the slides. This buttress would fill the channel and floodplain of Black Gore Creek and cause the slides to push against each other (see Figure 4).

One major consideration in this decision centered around the fate of Black Gore Creek itself. Economically, it would have been desirable to place the creek in a large pipe under the embankment; that alternative, however, was deemed aesthetically unacceptable by practitioners of several disciplines.

Keeping the stream flowing on the surface posed major design problems. An impoundment was required upstream from the earth buttress and a run-down, or waterfall, was required on the downstream end. Because of the marginal stability of the area, design and construction procedures were quite complex. The technique chosen for the run-down included stepped gabion walls with deep foundations to act as cutoff walls and post-grouting to reduce permeability through the rock baskets.

## Stations 438-450

In the area of stations 438-450, subsurface investigations revealed the presence of a lake deposit consisting of silt and fine sand perched on the hillside where a balanced cut-and-fill alignment was planned. The lake sands were saturated, and cuts into these deposits would certainly be unstable. Predrainage was not a practical alternative. Neither was a standard all-fill template, since moving the alignment out to an all-fill section would have required extensive rechannelization of Black Gore Creek. A channel change was not recommended because of the presence of unstable, unconsolidated lake deposits on the opposite hillside. It was also aesthetically unacceptable.

The remaining choices were a high wall or a side-hill viaduct. Because of foundation conditions and cost factors, a reinforced earth wall was selected. Before construction of the wall, organic soils in the floodplain of Black Gore Creek were excavated and a drainage blanket was placed over most of the wall foundation area and on the slope behind the wall. Extensive under-drain systems were installed to further ensure continued adequate drainage (see Figure 5).

## Station 499

Instrumentation installed during the intermediate study identified an active landslide on the north side of Black Gore Creek in the area of station 499, a discovery that

Figure 2. Vail Pass location map showing elevation and stations along Black Gore Creek.

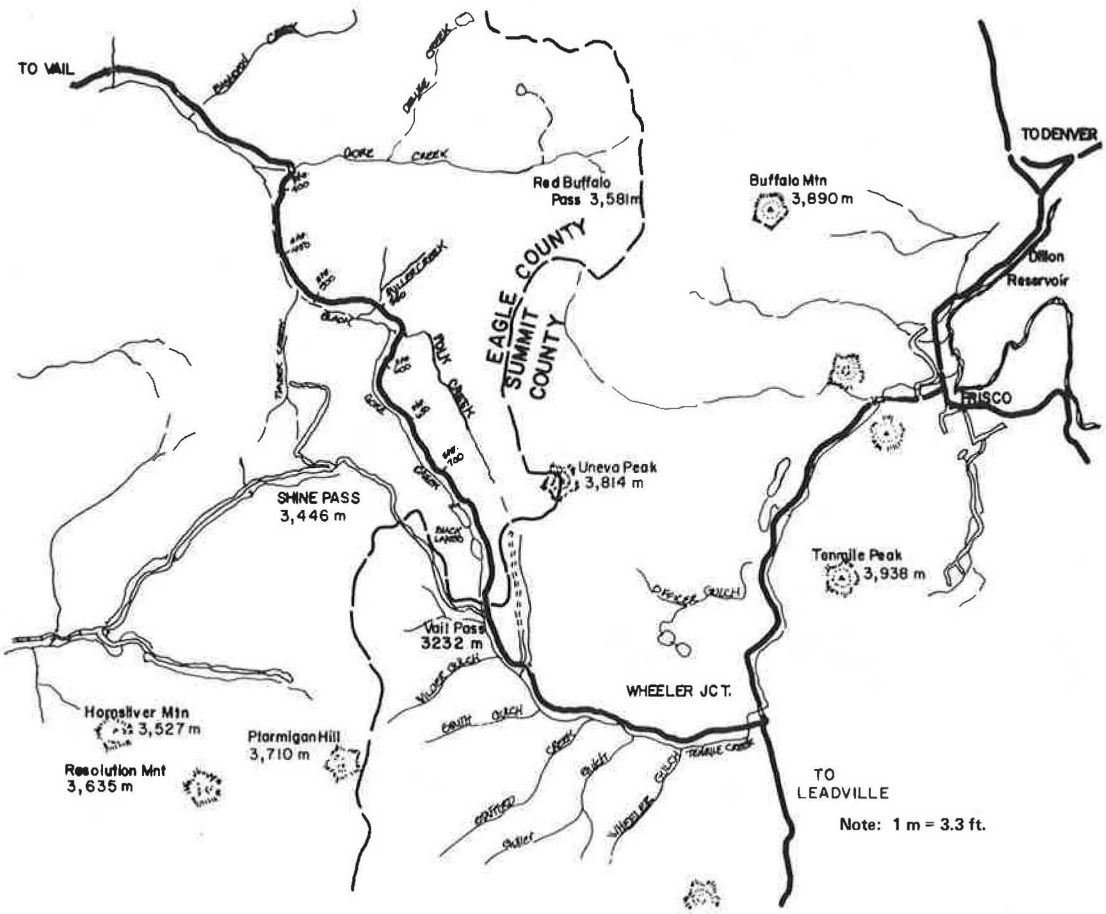



Figure 3. Geologic section: station 335.

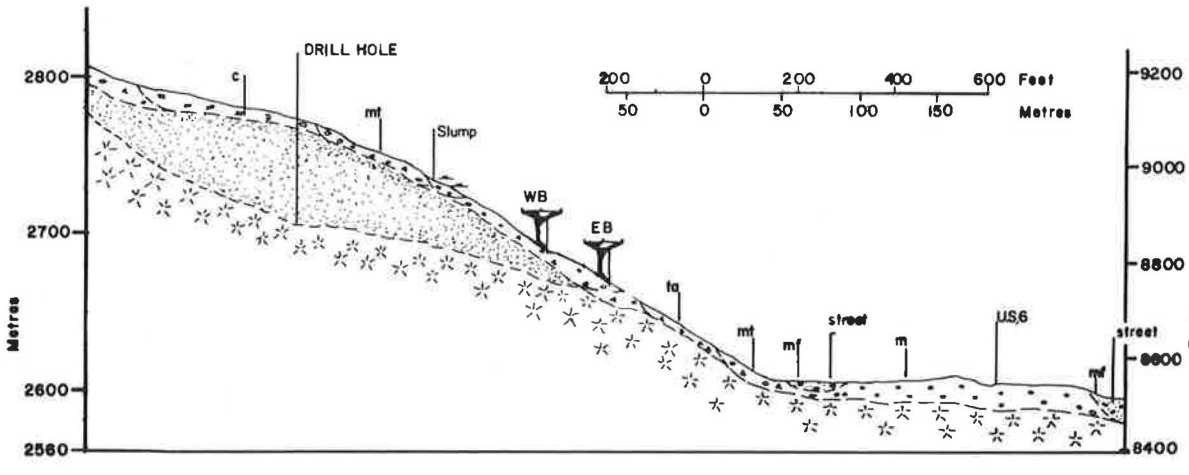


Figure 4. Geologic section: station 425.

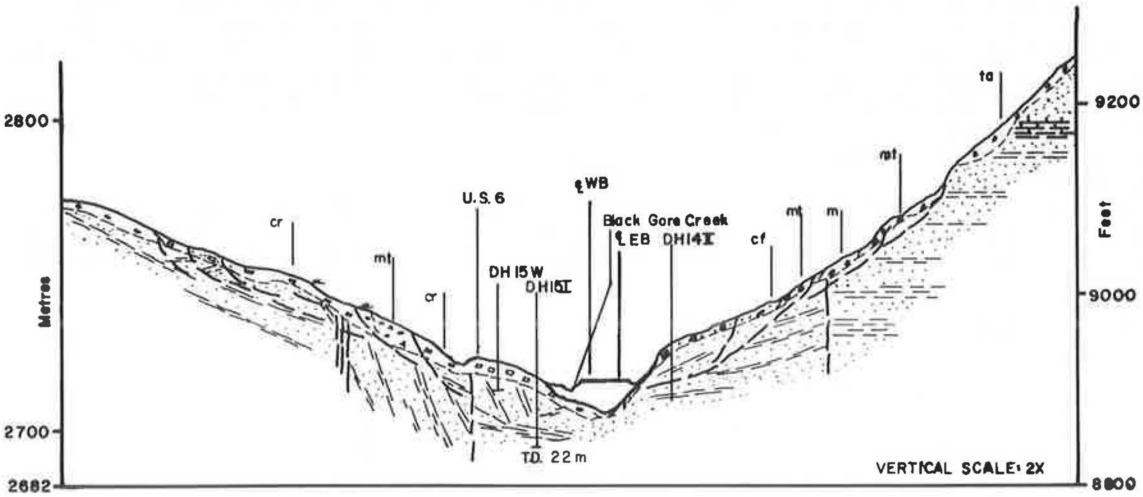



Figure 5. Geologic section: station 450.

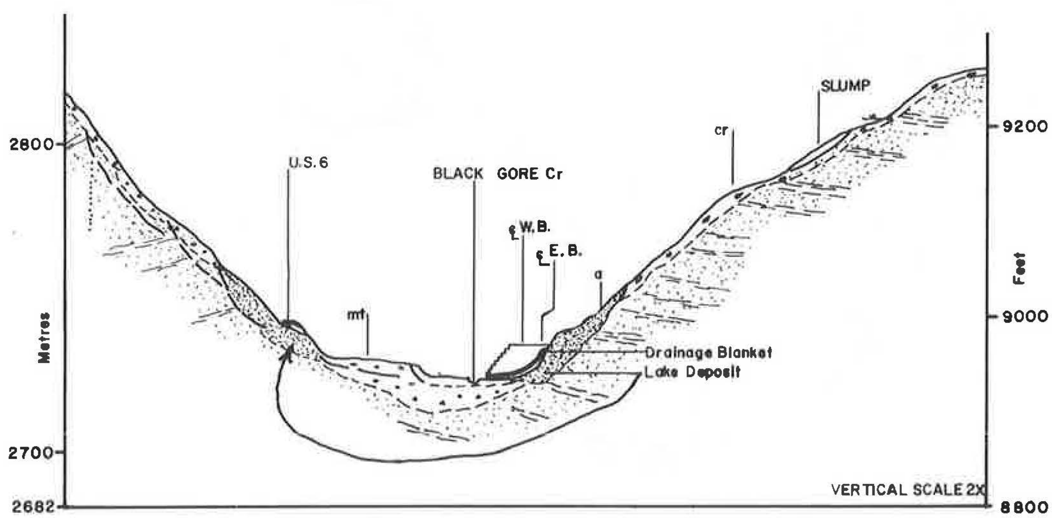
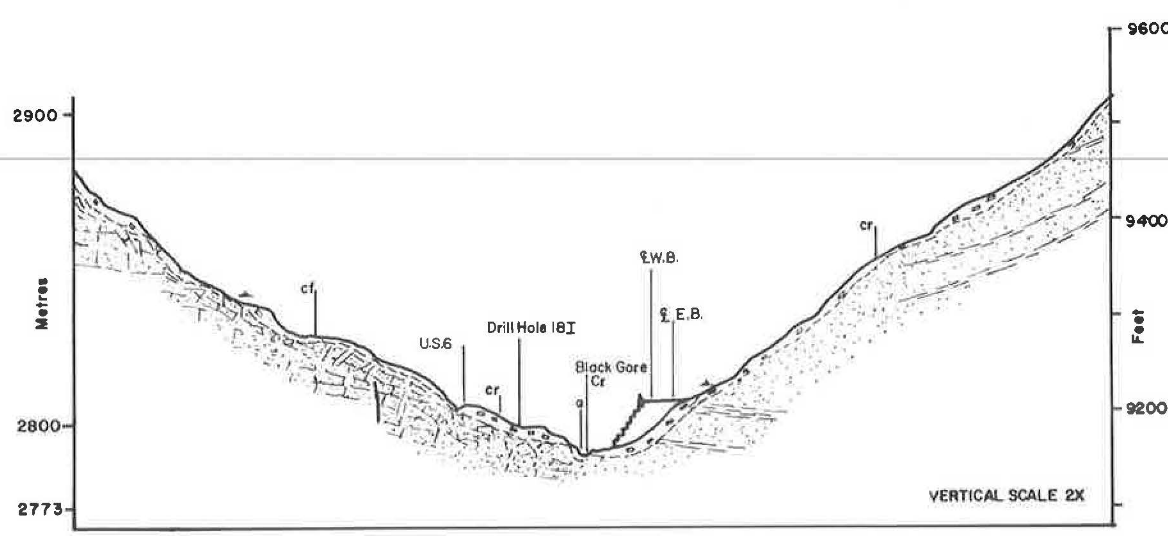



Figure 6. Geologic section: station 499.



resulted in the final alignment being placed totally south of the creek and requiring one high retaining wall. This retaining wall was the first precast tieback wall to be constructed on a highway project. There was a substantial amount of instrumentation on this wall, including load cells, inclinometers, and strain gauges. The wall system proved effective and efficient. Figure 6 shows the final cross section.

## Stations 545-555

The most difficult and challenging problem for geotechnicians at Vail Pass was in the area known as the Miller Creek slide. This slide is over 1 km (0.62 mile) long, almost 1 km wide, and about 46 m (150 ft) deep. The slide consists of up to 21 m (70 ft) of surficial silty soils overlying some 24 m (80 ft) of failed bedrock. Water levels were erratic, varying from surface ponds in some areas to dry in other areas.

During the earliest drilling, the lowest failure plane was thought to be at the soil-rock interface. Two core samples from 6 m (20 ft) below this contact indicated intact sandstone and siltstone dipping by 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 Board of Water Commissioners in conjunction with a proposed water-diversion tunnel. 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 occurred below this zone. The bedrock failure plane corresponded with the 4° bedrock dip in the lower reaches of the slide.

During the course of the drilling, a buried soil profile was uncovered in the toe area of the slide. Carbon-14 dating showed that a sample of organic material from the soil layer was 1000 years old, plus or minus 70 years, which indicated that the movement was relatively recent.

The topography of the Miller Creek slide area, shown in Figure 7, included a deep canyon at the toe of the slide into which the slide would spill during active periods. Severe grade restrictions, both up and down station from the slide area, dictated that the alignment must cross the toe of the slide. In other critical areas, it was usually possible to vary vertical and horizontal alignment to optimize alignment and geologic conditions.

The first alignment investigated included major fills ramping onto and off the toe area and the roadway in cut and fill across the toe area proper. Cuts of up to 15 m (50 ft) were required. Because relatively recent movement had occurred, it was decided that the risk was too great. The safety factor would have been reduced by about 25 percent in the immediate area.

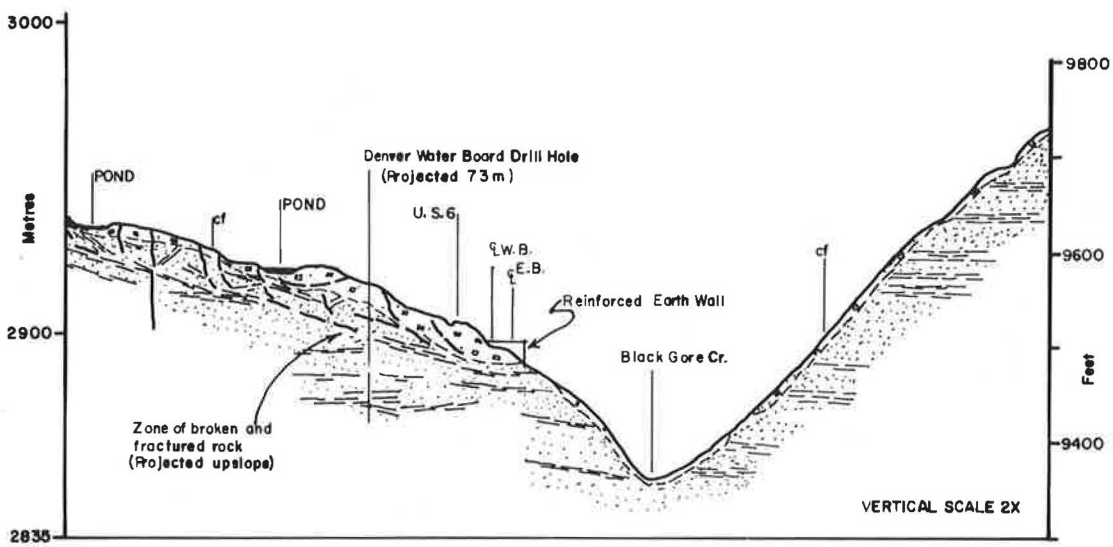
The second alignment investigated included twin bridges with caisson foundations into intact bedrock. In conjunction with this alternative, a consultant presented a European concept that included oversized, elliptical, concrete-lined caisson holes with free-standing caissons inside. This technique would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

The remaining alternative was a high, vertical, reinforced earth wall, the toe of which would be placed on the edge of the steep canyon wall and the base of which would have to be placed on intact bedrock. In place, this design raised existing safety factors insignificantly, but excavation of the toe of the slide to permit construction showed a mathematically significant reduction in the safety factor on the critical circle.

After a tentative decision was made to try the wall concept, a detailed geotechnical investigation was done. It was decided that, if the water table could be lowered about 11-17 m (35-55 ft) below the ground line as far as 91 m (300 ft) horizontally behind the excavation, it would be possible to construct the wall. Based on readings from several water-level-monitoring sites, 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. First, an attempt was made to construct an access road for the drilling equipment along the lower margin of the slide toe on the rim of the canyon. Because the area was saturated and not frozen [a 2.1-m (7-ft) snow cover had not allowed frost penetration], the dozer became helplessly mired. A second road was successfully built about 6 m (20 ft) higher. The minimum result

Figure 7. Geologic section: station 550.



expected from drainage holes at this level was that they would help to dry the lower area for a future drainage project.

The horizontal drilling proved successful and interesting. Some intact blocks of rock 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 into a shear zone. One such flow was measured at 757 L/min (200 gal/min). Most flows dropped off substantially in 30 min to 2 h.

Water levels were reduced by almost exactly 11 m (35 ft). Because of the low cost and the 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 15 m (50 ft) below the ground surface.

Before construction, three inclinometer holes were installed up the slope to monitor any movement that might result from the excavation. The construction plan included very stringent procedures to minimize the amount of excavation in the toe area of the slide that would be left unsupported at any given time. These restrictions, coupled with a winter work requirement, caused considerable consternation among both highway department construction personnel and the bidding contractors. Some painted a bleak picture of the plan's prospects of success.

An unusually dry autumn and a snow-free early winter in 1976 permitted construction to proceed as planned. The backfill was all large gravel [2.5-20.5 cm (1-8 in)] and cobblestones (tailings from early gold-dredging operations in the Blue River), which was workable at all temperatures; thus, achieving density on subzero days was not a problem. The high [21-m (70-ft)], steep (1:1) temporary slopes to the slide toe remained stable, and the inclinometers detected no movement.

### Stations 615-680

Bedrock failures were observed throughout the upper reaches of Black Gore Creek. One failure series was continuous from station 615 to station 750±. The alignment location was first split, the eastbound on the south valley wall and the westbound using the existing US-6 platform. This concept was abandoned when it was discovered that a bridge from the west valley wall would have to be located on marginally stable failed bedrock.

The remaining choice was to place both directions of travel on or near the existing US-6 alignment. The roadway template was minimized by using a median wall, which allowed a better fit on the steep hillside. Several attempts were made at defining critical failure circles and strength parameters through this area. None were felt to be entirely reliable, and the final design was accepted by geotechnical personnel on the basis that it caused the least disturbance—that is, the least reduction in safety factor on mathematical models. It was the consensus of opinion that the final alignment had a reasonable and acceptable probability of success (see Figure 8).

### Stations 700-715

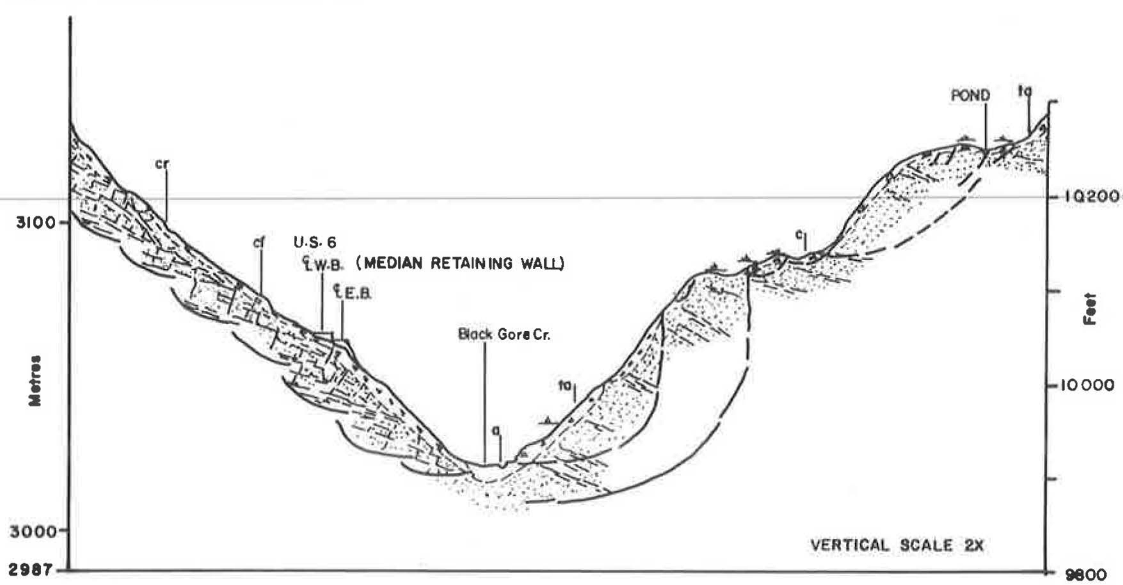
The alignment in the area of stations 700-715 traversed an area of extensive bedrock failure, a continuation of the failure that begins at station 615. Topographic restraints limited the choice of design to a through cut into failed bedrock or twin viaducts running above and parallel to the channel of Black Gore Creek. The cost for the cut section was estimated at $700 000 and that for the viaducts at $2 000 000. It was decided, for a variety of reasons, that the cut was also aesthetically and environmentally more acceptable. After geotechnical personnel estimated that any failure that might occur could be successfully controlled at less total cost and with less environmental damage than if the bridge alternative were used, the cut alternative was selected.

During construction through this area, two failed bedrock areas reactivated. They were stabilized by means of a grade adjustment, horizontal drains, and a rock buttress at a total cost of $810 000. Thus, the final cost was still approximately $500 000 less than that of the alternative that would have avoided failure.

## GEOTECHNICAL RESOURCE EXPENDITURES

During the period between 1968 and 1977, geotechnical

Figure 8. Geologic section: station 640.



investigations were virtually continuous. It is estimated that 30 person years of geotechnical expertise and 10 drill-crew years were involved. Approximately 6100 m (20 000 ft) of vertical drilling and 2400 m (8000 ft) of horizontal drilling were completed. Several hundred standard split spoon samples and about 50 thin-walled tubes were obtained. Over a hundred meters of penetrometer holes were also accomplished.

The instrumentation used consisted of 32 inclinometers, 11 borehole extensometers, 12 piezometers, 40 gloetzl soil pressure cells, 63 electrical strain gauges, and 2 shear strips. Geophysical studies were conducted by the Colorado School of Mines and by the Colorado Department of Highways with single and multichannel seismographs.

The total expenditure for geologic investigation on the Vail Pass project is estimated to be $2 million. According to the study by the Robinson and Lord firms, approximately 11.2 of the 22.5 km (7 of 14 miles) traversed by I-70 was unstable to marginally stable terrain. Only one unanticipated failure larger than 153 $m^3$ (200 $yd^3$) occurred as a result of construction. This occurred during the spring of 1978 and cost about $75 000 to repair.

## SLOPE DESIGN

Angles of cut-and-fill slopes across Vail Pass were based on geotechnical data and on experience in similar materials in similar climatic conditions. Many areas of Vail Pass required cuts and fills that approached critical heights.

At the highest cut, a 91-m (300-ft) high bedrock cut between stations 605 and 614, the highway department retained a specialist in rock mechanics to evaluate the slope design and to design and interpret an instrumentation system. In fact, all significant cuts and fills were reviewed by at least two geotechnical specialists in an effort to minimize stability problems during construction.

The final slope configuration, especially the molding and sculpturing effect now visible to the motorist, is the result of intensive joint efforts by landscape architects and construction and geotechnical personnel. Individual grading projects were staffed with landscape specialists who prepared conceptual plans for the final appearance of cuts and fills and other related features. These plans were reviewed by geotechnical personnel to ensure compatibility with on-site geologic conditions and with construction personnel to ensure that construction was practicable.

Slope design was thus yet another product of cooperation among practitioners of several disciplines.

## CONCLUSIONS

Geologic conditions at Vail Pass were generally unfavorable for the construction and maintenance of a four-lane highway facility. Severe constraints and limitations were placed on designers by extensive areas of soil and bedrock failures, steep topography, and a cold and wet climate.

Successful completion of a project of this magnitude can result only from the combined efforts and cooperation of several disciplines under strong and enlightened leadership. These elements were present throughout the Vail Pass project. Geologic constraints were given appropriate consideration at every level and during each phase of project development. All of the geotechnicians involved appreciated the opportunity to participate in so challenging an undertaking, in a cooperative atmosphere.

The substantial amount of money expended for geotechnical aspects of the project was justified by the project's successful and timely completion. From a geotechnical point of view, this project stands as a model for future major engineering efforts in difficult geologic conditions.

## ACKNOWLEDGMENT

Major contributors to the final Vail Pass geotechnical studies include J. B. Gilmore, R. P. Moston, and A. C. Ruckman of the Colorado Department of Highways; Charles S. Robinson and M. D. Cochran of Charles S. Robinson and Associates; Dwayne Nielson of International Engineering Company; Mike Bokovansky of Dames and Moore; and Horst Ueblacker of Ueblacker and Associates.

The figures presented in this paper are based on those in a Robinson-Lord joint-venture report and were modified by Jim Lance of the Colorado Department of Highways.

*Abridgment*

# Landscape Treatments on the Vail Pass Project: Slope-Design Procedures

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During the construction of I-70 over Vail Pass, many landscape techniques were used to stabilize highway slopes and to achieve visual compatibility with the surrounding forested mountainsides. Although slope beautification was an initial objective, successful landscape treatments were soon found to be those that imitated existing landscape elements. Methods of erosion control, slope stabilization, and revegetation eventually merged into a format that has proved to be successful in preserving scenic quality.

Stable highway slopes were always the basic consideration in any treatment. At no time could landscape treatments take precedence over the engineered stability factors necessary for an Interstate highway.

Highway safety was also an important consideration. Treatments were designed to ensure the safety of the