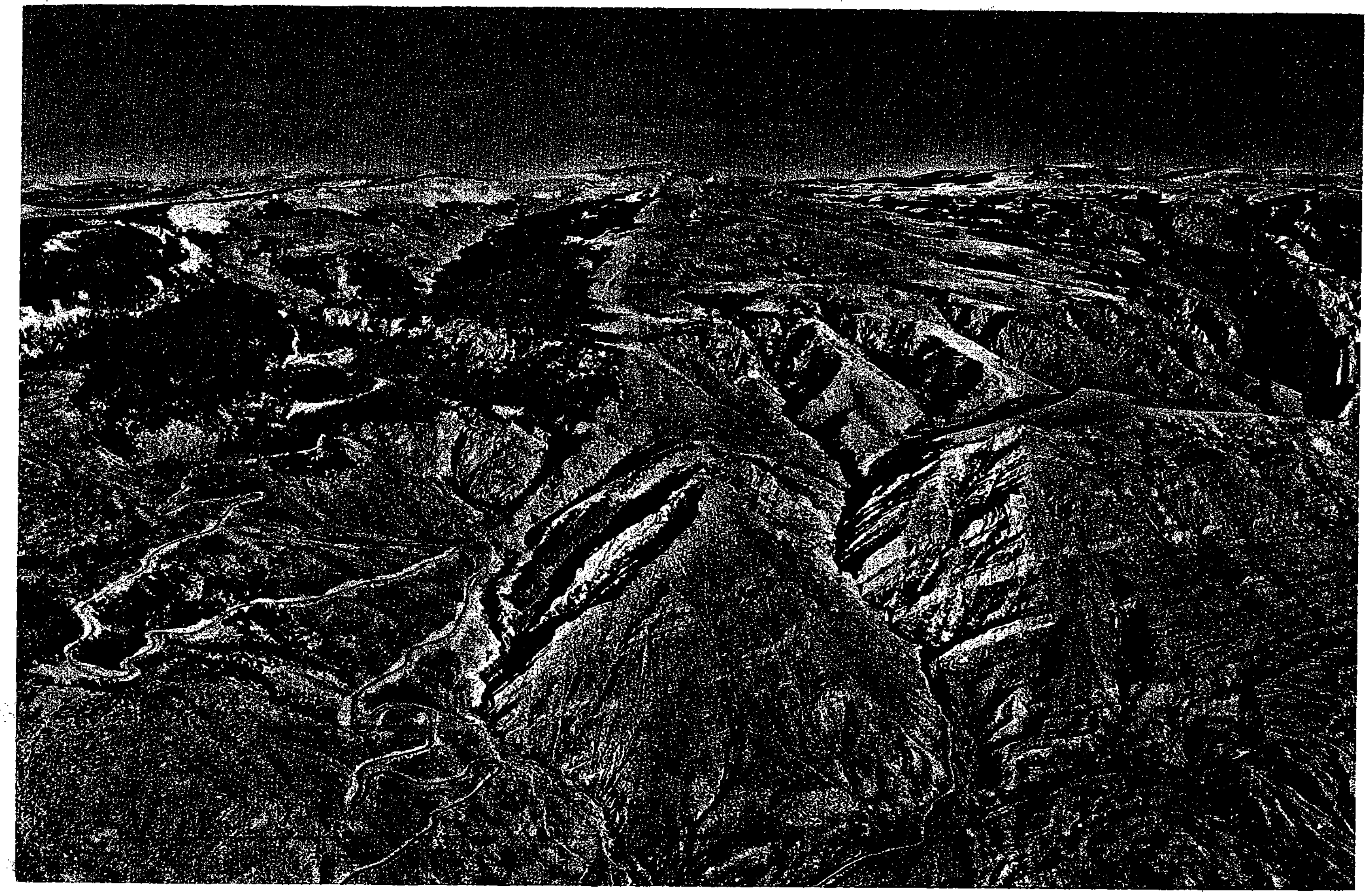

TWENTY-FOURTH ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Sheridan, Wyoming
August 9-10, 1973

CORRECTIONS TO THE PROCEEDINGS ARE AS FOLLOWS:

The title on figure 3, page 44 and the title on figure 5, page 57 should be reversed.

PROCEEDINGS OF THE 24th ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

SHERIDAN CENTER
SHERIDAN, WYOMING
AUGUST 9-10, 1973

Sponsored By
WYOMING STATE HIGHWAY DEPARTMENT

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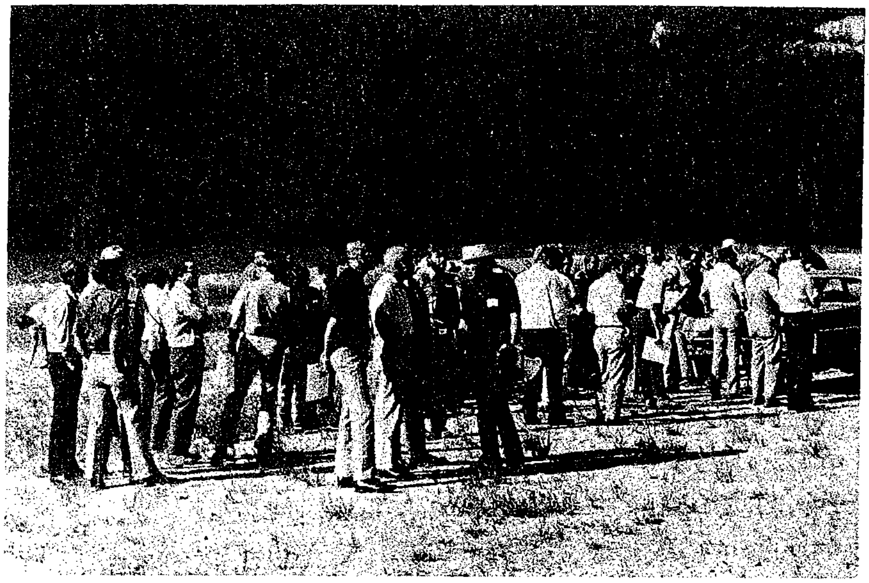
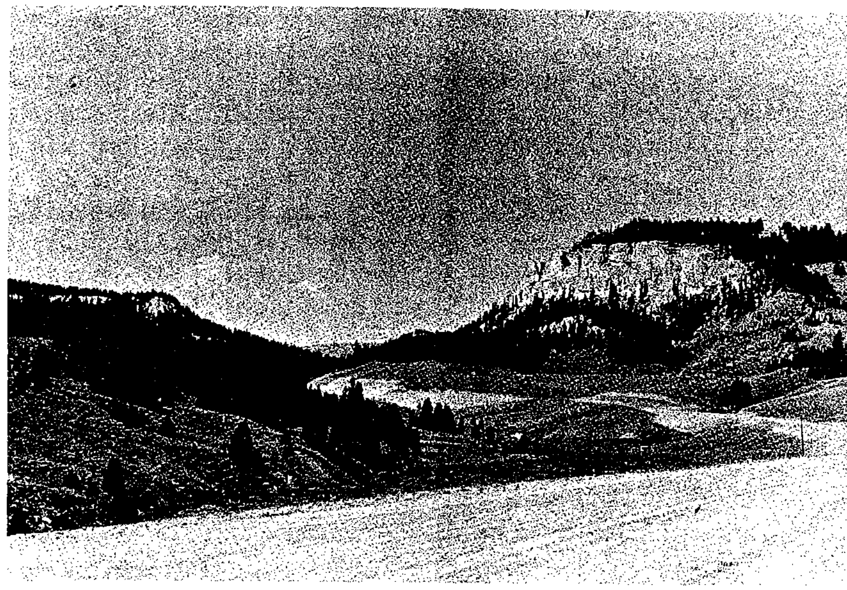


SERIOUS DISCUSSION



NEAR SUMMIT TOPOGRAHPY

WEST FLANK SWITCHBACKS

MOUNTAIN BUFFET

TABLE OF CONTENTS

WYOMING GEOLOGY

Daniel Miller
State Geologist
Wyoming State Geological Survey

Abstract

Wyoming encompasses nearly 10,000 square miles and supports a stable, dispersed population of less than 340,000 people. The State's affluent economy is related to its geology and is due largely to exploration for, and production of, its mineral resources, agriculture, ranch management, and tourism.

Much of the State's development is related to surface land conditions and to sources of water supply. The physiography controls local climatic conditions and hence development of soils and vegetation. Among the State's major geologic features are 10 mountain ranges and 10 structural basins, all of which are clearly defined at the surface, and accessible for investigation. From a practical standpoint, there is some representation within Wyoming of all ages of rock from Precambrian through Recent. In parts of the State the geologic record is remarkably complete.

Extensive deformation during the Laramide Orogeny, and subsequent erosion and deposition during the Paleocene has altered the State into more or less its present structural form.

Because of the State's general geology and the nature of certain kinds of deposits, Wyoming sustains a diverse minerals industry, which is rapidly expanding on all fronts. Wyoming ranks tenth in the U. S. in terms of the value of its total annual mineral production ($750,000±).

Editorial Note: The above listed paper was not delivered to the Symposium Committee for publication.

HIGHWAY LANDSLIDE PROBLEMS ASSOCIATED WITH THE ESCARPMENT AREAS OF THE CUMBERLAND PLATEAU IN TENNESSEE

David L. Royster
Chief, Division of Soils and Geological Engineering
Tennessee Department of Transportation

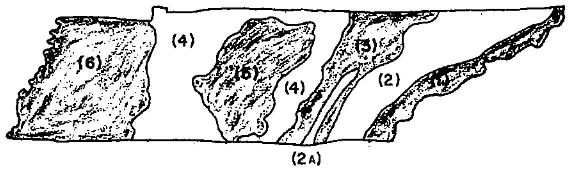
Abstract.--Throughout the years, many of the greatest problems associated with the construction and maintenance of highways in Tennessee have occurred along the eastern and western escarpments of the Cumberland Plateau, one of the six major physiographic provinces of the state.

These problems have resulted primarily from a rather complex, yet fairly distinct and identifiable assemblage of soil and geologic materials and conditions. The materials are clay shales, colluvium, and moisture. The conditions are a rugged surface configuration, a "sidehill" environment, 800'-1200' of total relief in the project areas, unpredictable subsurface drainage patterns, and highly irregular zones of weathering.

Various methods have been used to correct or control the slides. They include partial to total removal; alignment and gradient changes; the use of various types of support and restraining devices, such as rock buttresses, Gabion walls and reinforced earth platforms; as well as the usual means of de-watering slides through trenching, bored horizontal drains, and vertical wells equipped with submersible pumps.

INTRODUCTION

The Cumberland Plateau is the southern portion of the greater Appalachian Plateau that extends across Tennessee from New York to Alabama (Figure 1). The northern extension is referred to as the Allegheny Plateau, and the



MAJOR PHYSIOGRAPHIC PROVINCES OF TENNESSEE

LEGEND

(1) UNAKA AND BLUE RIDGE MOUNTAINS
(2) VALLEY AND RIDGE
(2A) SEQUATCHIE VALLEY
(3) CUMBERLAND PLATEAU
(4) HIGHLAND RIM
(5) CENTRAL BASIN
(6) GULF COASTAL PLAIN

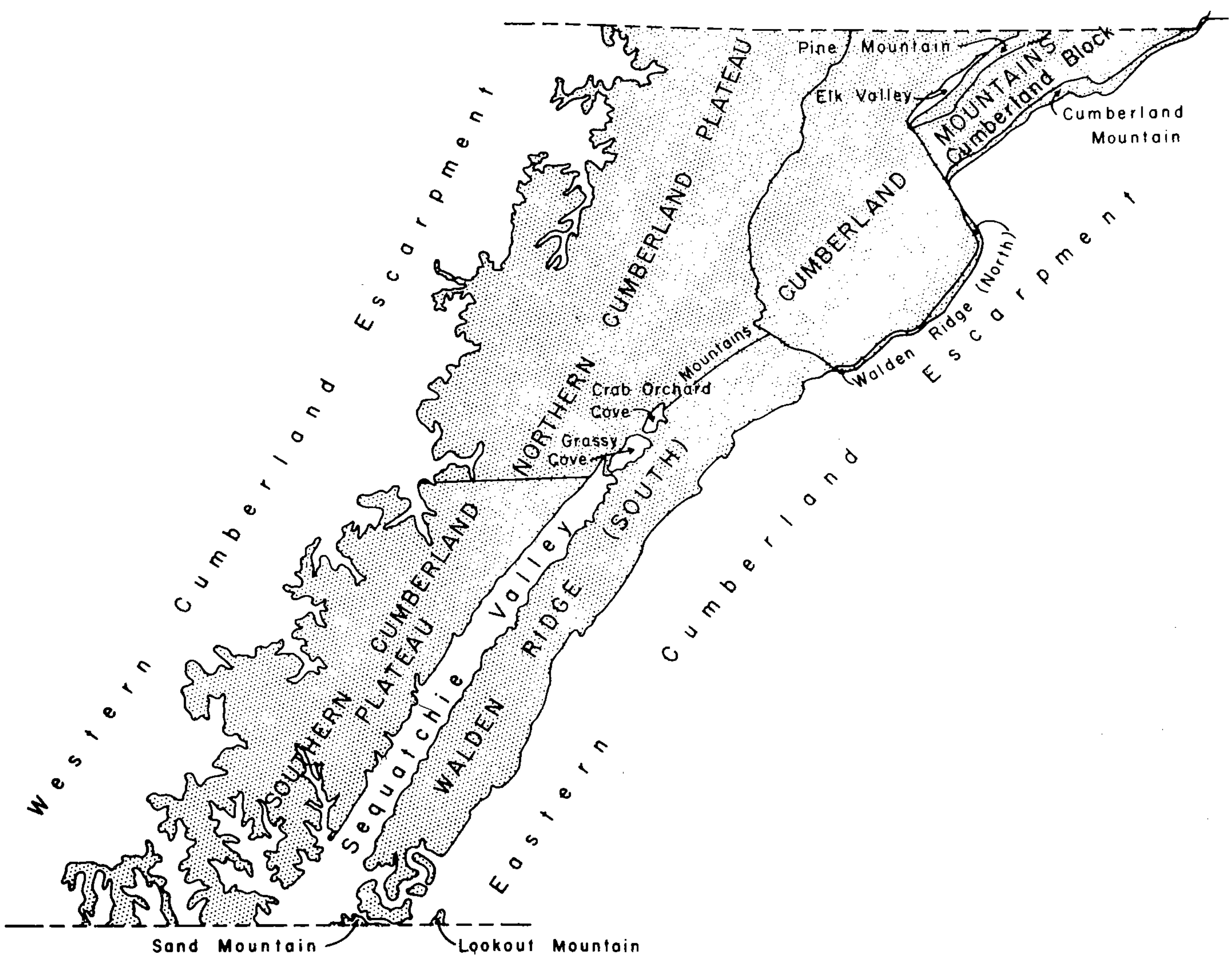


FIGURE 1

Kentucky River in Middle Eastern Kentucky is considered by most geologists to be the dividing line for the two sections.

The plateau in Tennessee encompasses an area of approximately 5400 square miles. The average elevation is about 2000 feet with higher elevations of around 3500 feet being reached in the Cumberland Mountains in the northeast portion.

The great majority of the problems have occurred not on the plateau itself but rather on the slopes of the escarpments that border the Ridge and Valley of East Tennesee, the Sequatchie Valley of Middle Eastern Tennessee, and the Eastern Highland Rim of Middle Tennessee (Figure 2). This paper will describe some of the more recent problems and the means that were used to correct them. It will also describe some of the things that may be done to minimize future problems.

GEOLOGY OF THE PLATEAU

Sandstones, conglomerates, siltstones, shales and occasional coal beds, all of Pennsylvanian age, constitute the surface and near-surface rocks of the Cumberland Plateau. Most of these rock types, being moderately to highly resistant to weathering, produce relatively shallow soils. Except for the faulted eastern front and the geologic structures associated with the Sequatchie Valley, Crab Orchard Cove, Grassy Cove, and the Elk Valley-Pine Mountain area, the formations are essentially horizontal.

Along the escarpment slopes, however, the materials and conditions are quite different. Here, one encounters shales and limestones of Mississippian age overlain by shallow to deep residual soils and, quite frequently, deposits of colluvium that range in thickness from 5 to 50 feet or more. It is this condition--colluvium overlying residual clay soils or shale in a sidehill environment--that creates most of the problems.

The colluvium in this area is a heterogeneous mixture of boulders--up to room size in dimension--rock fragments, and sand, silt, and clay particles. It is material that

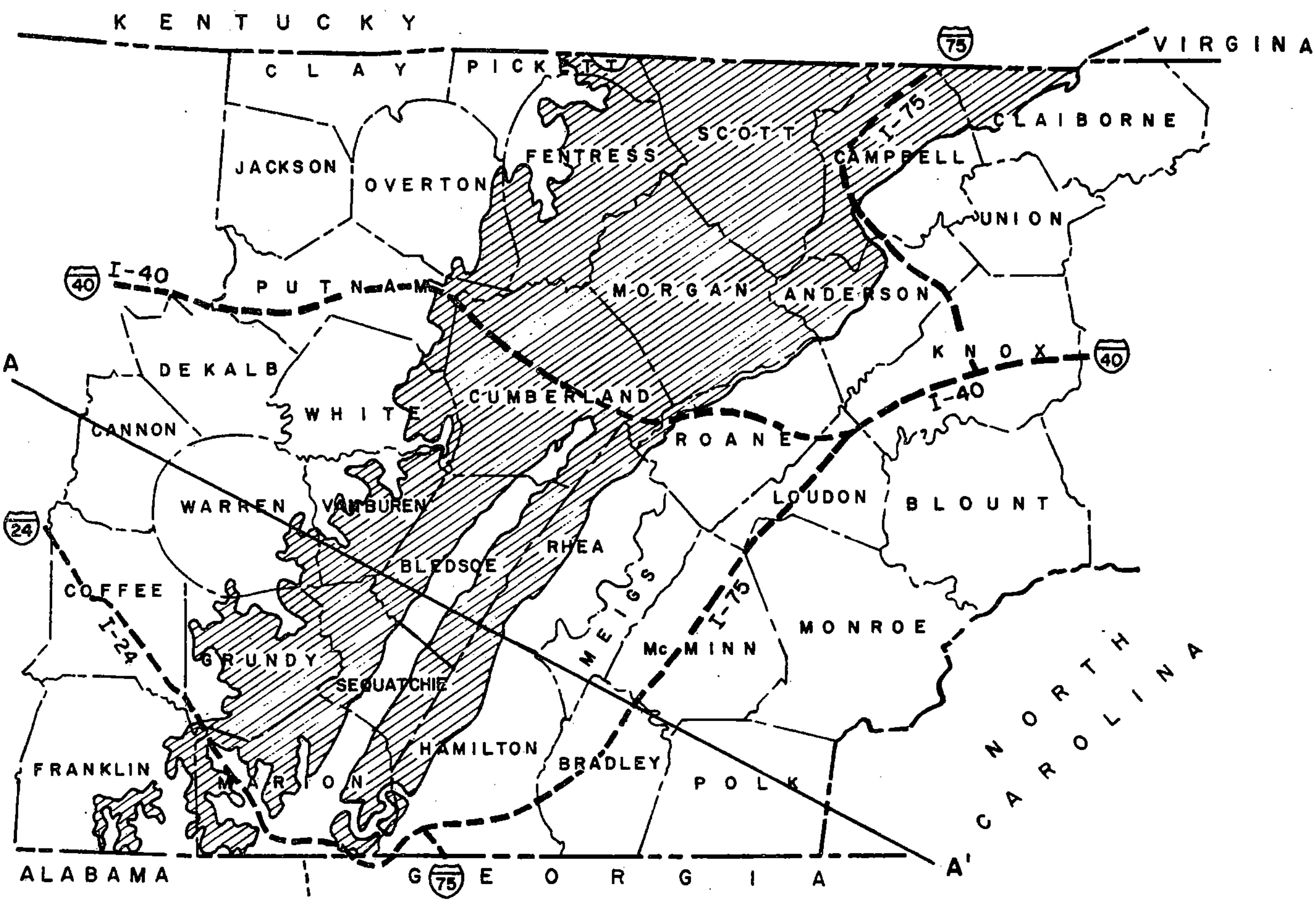
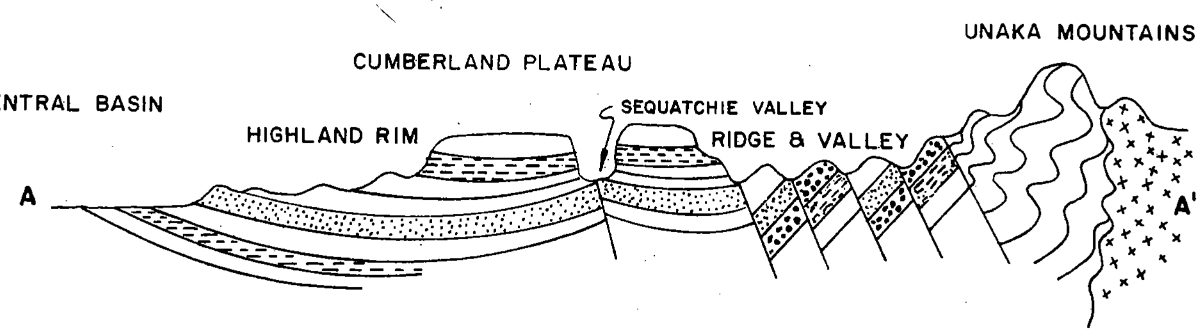


FIGURE 2. GENERAL ALIGNMENTS OF INTERSTATE ROUTES-24,-40, AND-75 ACROSS THE CUMBERLAND PLATEAU (SHADED AREA).



GENERALIZED NORTHWEST - SOUTHEAST CROSS SECTION ALONG A-A'
(Vertical scale highly exaggerated in relation to horizontal scale)

has moved downslope primarily under the force of gravity over a rather long span of geologic time. The silt and clay materials are products of weathering of the shales, siltstones, and limestones that underlie the sandstone which caps the escarpment. The sand, rock fragments, and boulders are derived, chiefly, from the "cap rock" itself. The colluvium lies as a veneer, draped, as it were, over the escarpment slope (Figure 3).

A HISTORY OF INSTABILITY

In most cases, especially where it is underlain by shale or residual clay soils, the colluvium is in a very delicate state of equilibrium; that is, the safety factor against failure in its natural condition probably is not much in excess of 1.00. Hummocky topography, as well as arched and deformed trees that have righted themselves in their growth processes through the years, are vivid testimonials to the instability of the escarpment slopes. Most failures occur along the contact between the colluvium and the underlying weathered shale or clay. The colluvium, as compared to the weathered shale and clay, is quite pervious. Soft or weak zones result in these underlying materials when water percolates down and moves along the interface. When a load is applied, as in the construction of an embankment, or support removed, as in the excavation of a cut, failure is almost inevitable. The large majority of roadway and slope failures along the escarpments over the years may be so attributed.

The failures vary all the way from creep and flow movements occurring over a relatively long span of time (Figures 4 and 5) to instantaneous collapses (Figures 6 and 7). Depending on conditions, these collapses may occur almost immediately and with the emplacement of only a few feet of embankment, or they may not occur for several years after initial construction. The latter failures are due primarily to pore pressure buildups resulting from a damming effect caused by deterioration and further consolidation of the shale materials from which the embankments are constructed, or simply damming by consolidation along the colluvium-clay/shale interface beneath the fills. These collapses often occur during or

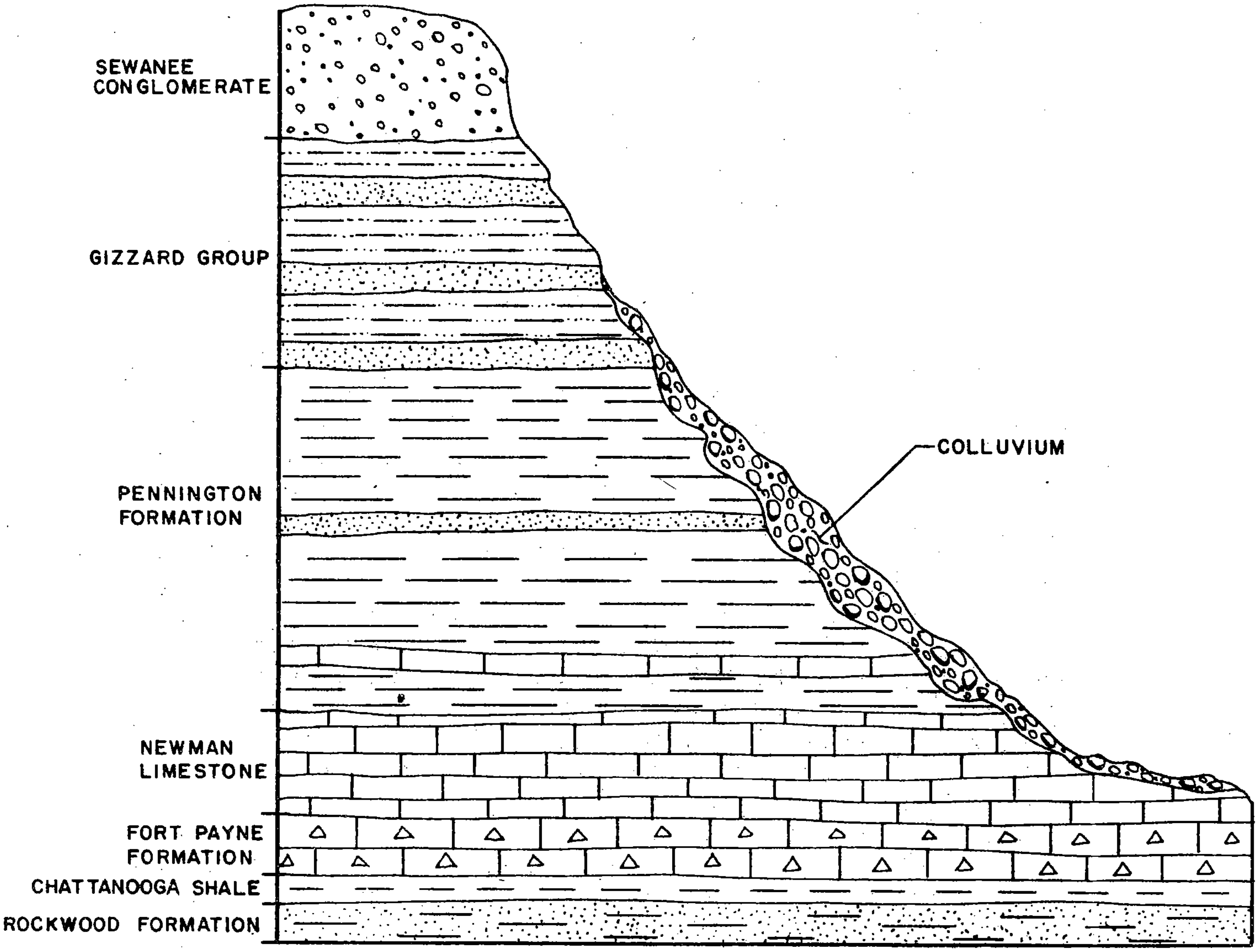


FIGURE 3. GENERAL SEQUENCE OF GEOLOGIC FORMATIONS AND MATERIALS ENCOUNTERED ALONG THE PLATEAU ESCARPMENT NEAR ROCKWOOD.

FIGURE 4. COLLUVIAL SLIDE ALONG I-40 NEAR CRAB ORCHARD IN CUMBERLAND COUNTY.

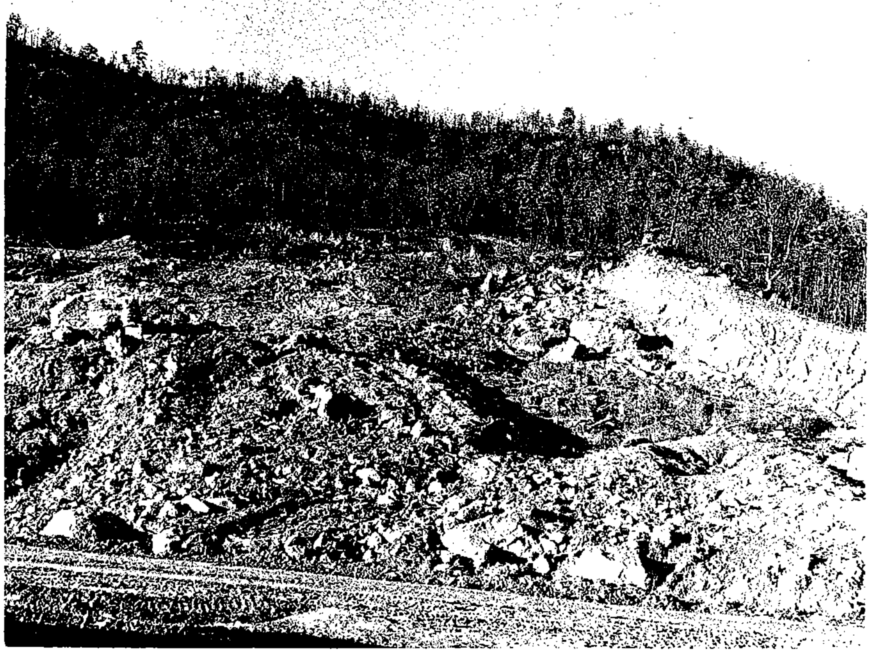


FIGURE 5. MASSIVE COLLUVIAL CUT SLIDE ALONG INTERSTATE-40 NEAR ROCKWOOD IN ROANE COUNTY.

FIGURE 6. EMBANKMENT FAILURE ALONG STATE ROUTE 30 JUST EAST OF DAYTON IN RHEA COUNTY.

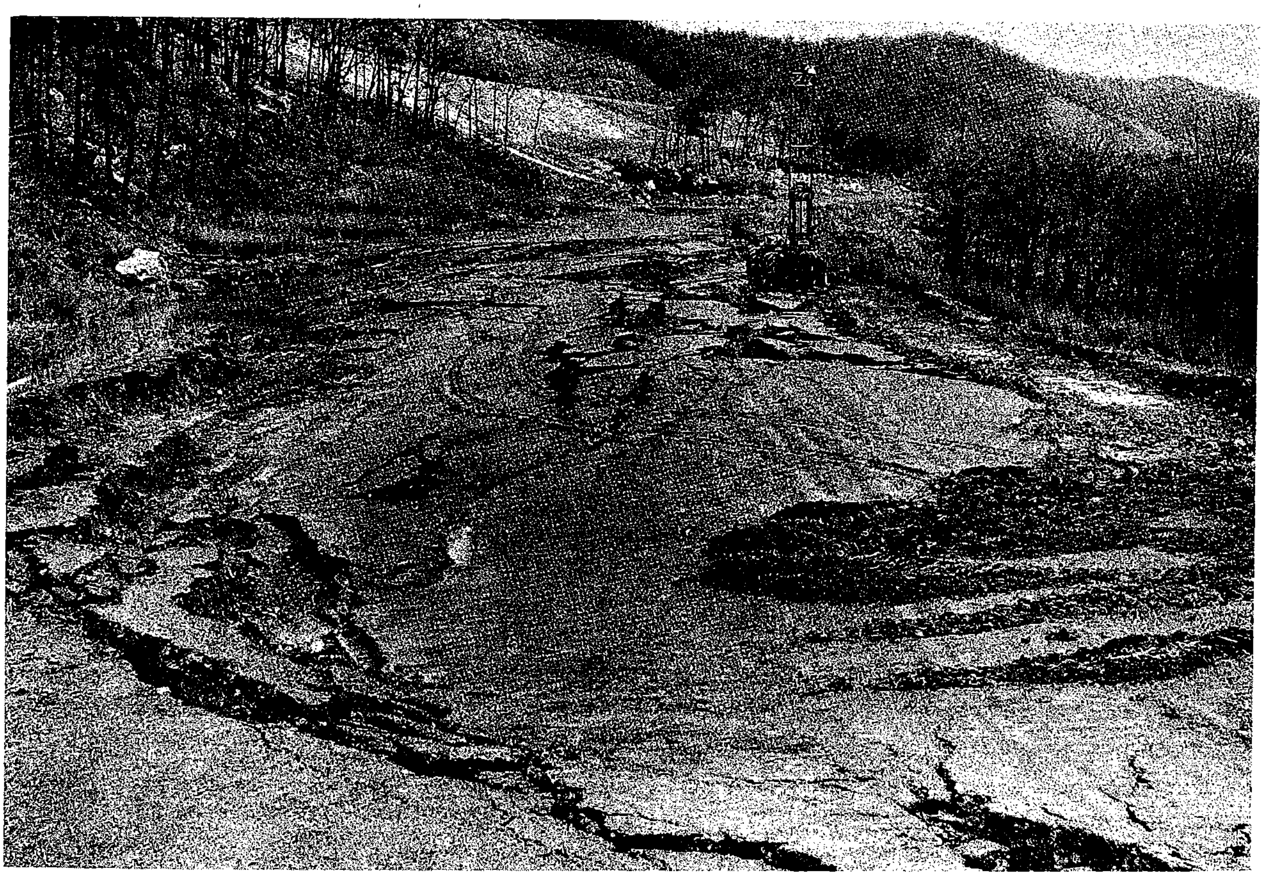


FIGURE 7. THIS FAILURE, ALONG I-40 AT ROCKWOOD, OCCURRED IN JAN., 1973 APPROXIMATELY FOUR YEARS AFTER CONSTRUCTION OF EMBANKMENT.

immediately following heavy or prolonged periods of precipitation.

While there is a rather long history of embankment and cut slope failures along roadways that traverse the plateau escarpments, the most recent and costly problems, i.e., since 1968, have involved major sections of Interstate-40 and -75 in Roane and Campbell Counties. Slides in Roane County have prevented the completion of the east-bound lanes of I-40 whose construction was begun in 1967. The west-bound lanes were only opened in late 1972. Since its completion in 1967, but primarily since the Spring of 1971, major fill failures have resulted in at least five separate lane closings along I-75 in Campbell County.

Since late 1967, approximately 35 separate slides have occurred in an interval of about four miles along that section of I-40 that traverses the escarpment just west of Rockwood. Of these, about 25 involved remedial costs in excess of $50,000.00 each, with several exceeding $250,000.00 to $300,000.00.

REMEDIAL MEASURES

While tremendously costly both in terms of time and money, the Rockwood slides have given the Department the opportunity to experiment with several different methods of treatment. Since the original alignment and gradient are being held, for the most part, the remedial measures being utilized consist of various forms of drainage, partial to complete removal, partial removal and restraint, or "total" restraint. In the case of restraint, three methods are being utilized: rock buttresses, gabions, and at least one reinforced earth wall.

Rock buttresses (Figures 8 and 9) are free-draining gravity structures consisting of large blocks of non-degradable sandstone or limestone. The gradation being used calls for 50% of the material to be greater than one cubic foot, with no more than 10% passing the #2 sieve. Because of its free-draining characteristics, the rock buttress works well in restraining colluvial slides. Movement is prevented by restraint while at the same time allowing

ROCK BUTTRESS

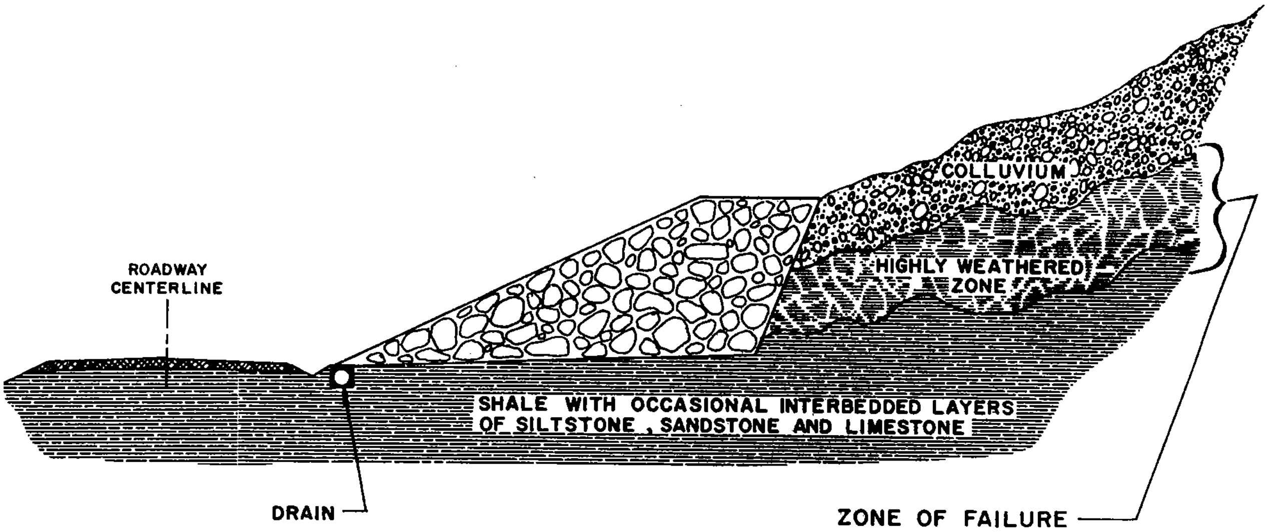


FIGURE 8.

FIGURE 9. BUTTRESS BEING USED TO CONTROL SLIDE ALONG THE EAST-BOUND LANE OF I-40 AT ROCKWOOD.

the large quantities of water usually associated with these slides to seep or flow through virtually unrestricted. This greatly reduces the likelihood of ponding and pore pressure buildup.

A second type of restraint structure being utilized at Rockwood is the Gabion wall (Figures 10 and 11). It is a free-draining, heavy, monolithic gravity structure consisting of wire mesh baskets (Gabions) filled with coarse, non-degradable rock. The Gabions are made of zinc-coated Number 11 steel wire and range in size from 3' X 3' X 3' to 3' X 3' X 9'. In the construction of the wall, each basket is secured to an adjacent basket with a tie-wire and then loaded in place. Gabion walls provide the flexibility needed to cope with the differential settlement so typical of the site conditions at Rockwood. Since they require considerably less lateral space than rock buttresses, they were used where space was somewhat limited. Another major advantage is that once the materials are on the construction site, the wall construction may continue even under very adverse weather conditions.

In August of this year, a third type of restraint structure will be constructed; that of reinforced earth (Figure 12). While most of the buttresses and Gabions have been used to control cut slides, the reinforced earth structure will serve as a platform to carry the east-bound lane across a fill area (see Figure 7). It was chosen in lieu of the rock pad--soil platform design used in a similar slide (Figures 13 and 14) because of economics.

Partial removal or total removal, of course, involves removing all or part of the unstable material from the failing cut or embankment area. At Rockwood, this has involved as little as a few hundred cubic yards up to 500,000 to nearly 1,000,000 yards for one slide. Where embankments are concerned, the removed material must be replaced by a more stable material up to the required grade. The greatest disadvantages to this treatment are in finding suitable waste areas and the fact that in cut sections removal often leaves rather large ugly scars that require several years for the development of new vegetation. A slide in the east-bound lane (Figures 13

GABION WALL

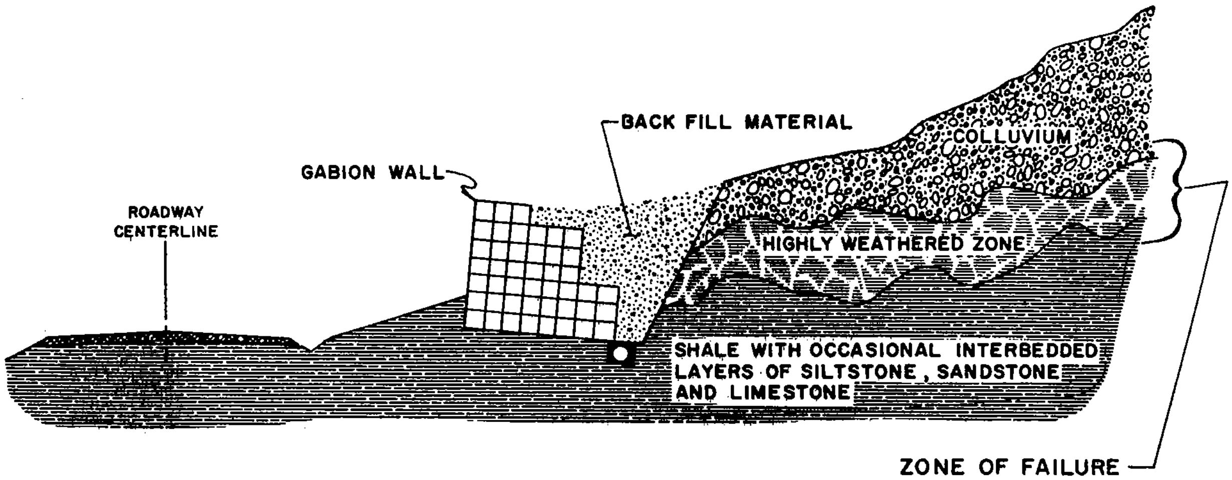


FIGURE 10

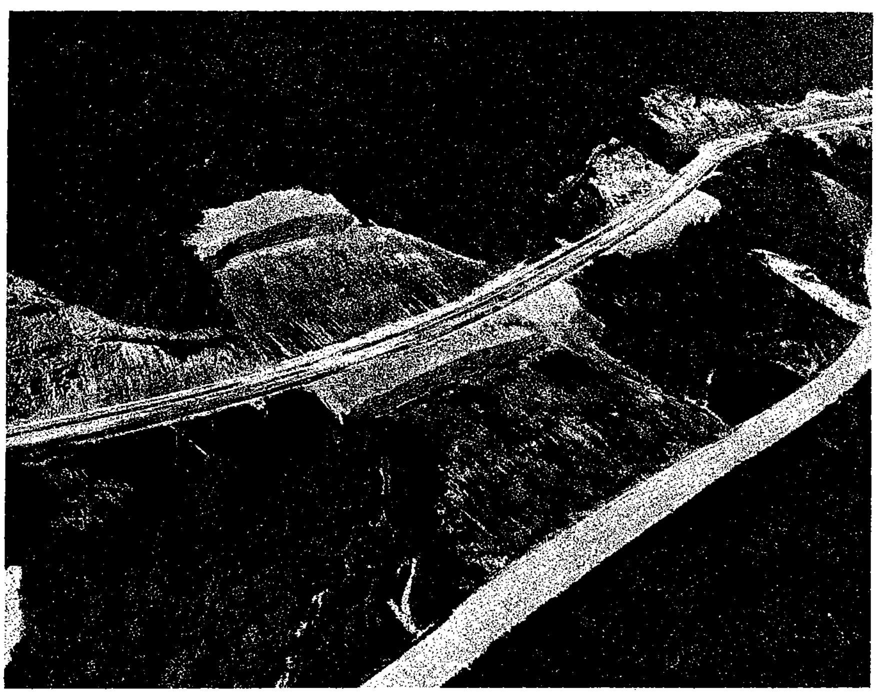


FIGURE 11. AERIAL VIEW OF TWO GABION WALLS BEING USED TO RESTRAIN SLIDES ALONG THE EAST-BOUND AND WEST-BOUND LANES OF I-40 AT ROCKWOOD.

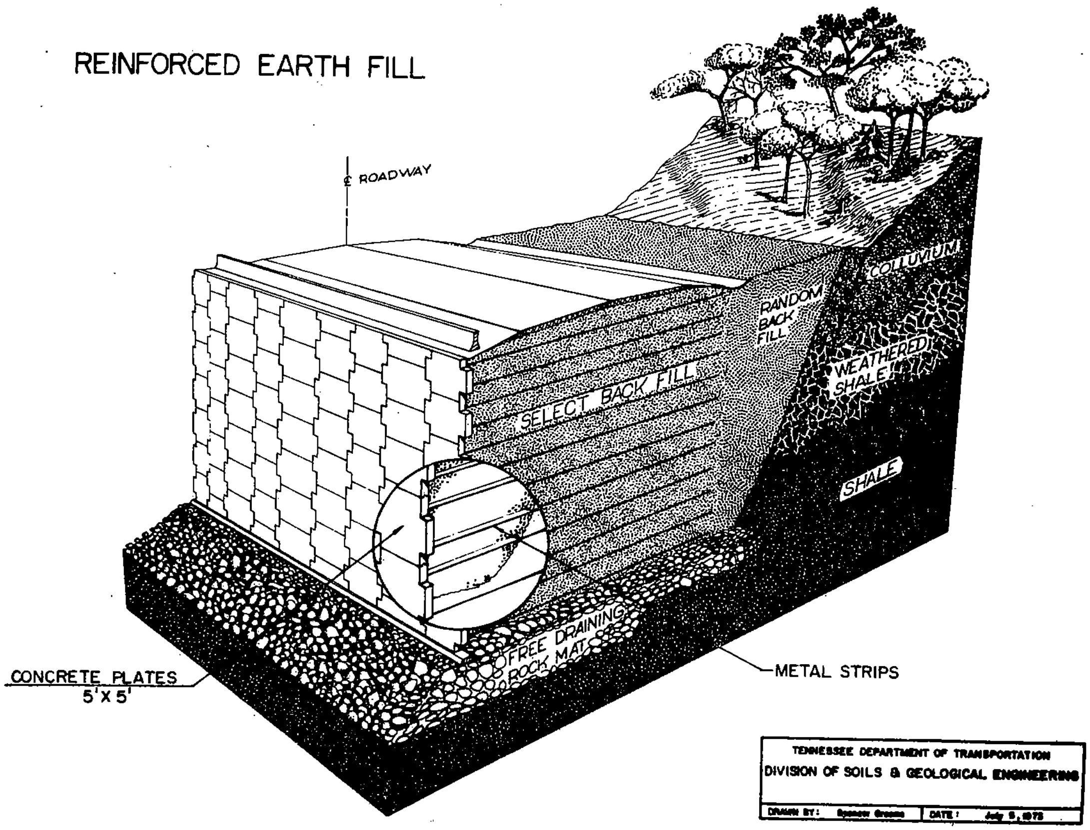


FIGURE 12. SCHEMATIC OF REINFORCED EARTH WALL BEING USED TO SPAN A SLIDE AREA ALONG I-40 AT ROCKWOOD (REFER TO FIGURE 7)

and 14) is an example of a fill failure in which the removal and replacement method has been used. In this case the rock pad, which is made up of buttress-type rock, serves as the foundation for the fill while at the same time serving as a drainage outlet and as a buttress against further upslope movement. The reinforced earth structure (Figure 12) was designed with the same principle in mind.

Comparisons of quantities and costs of typical examples of the various restraining methods just described are presented as follows:

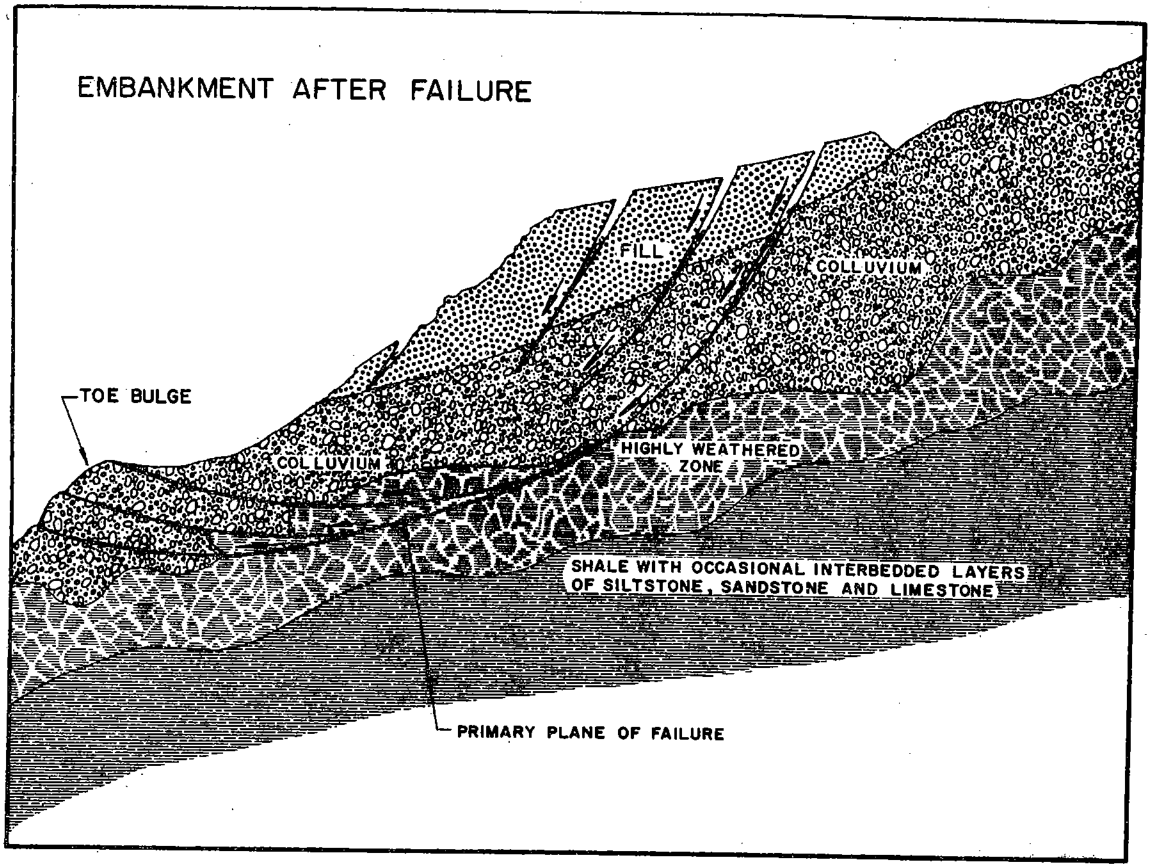


FIGURE 13. SCHEMATIC VIEW OF SLIDE 10 IN THE EAST-BOUND LANE.

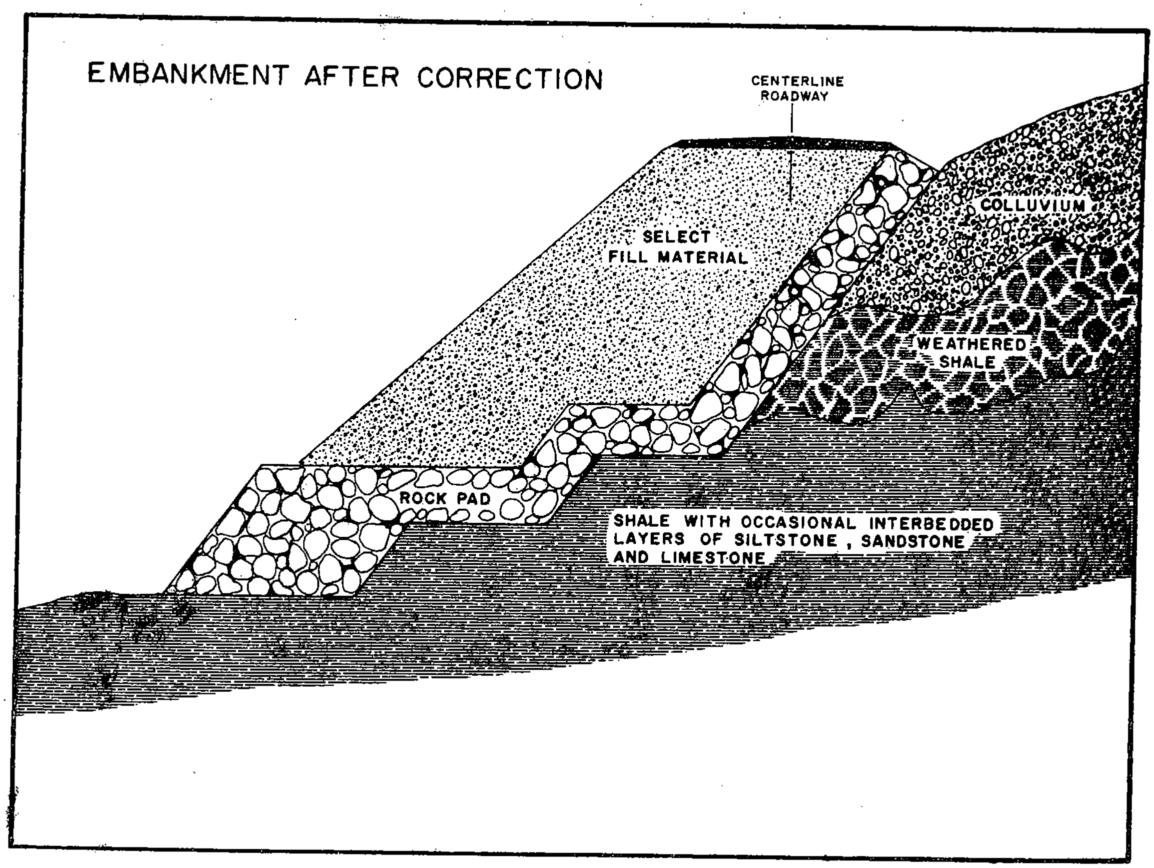


FIGURE 14. DESIGN USED TO REPAIR SLIDE 10.

Gabion Wall (Slide #4, West-Bound Lane)

Length: 268'
Height: 29'
Base width: 16'
Excavation: 41,750 cu/yds. at $1.97
Backfill (#57 stone): 12,488 cu/yds. at $7.00
Backfill cost: $87,416.00
Gabion quantities ("baskets", stone and erection):
3,451 cu/yds. at $44.00
Gabion cost: $151,844.00
Total costs: $321,507.50

Rock Buttress (Slide "A", West-Bound Lane)

Length (approximate): 700'
Excavation: 404,302 cu/yds. at $1.97
Excavation cost: $687,313.40
Buttress rock (supplied under 2 separate contracts):
(10,321 cu/yds. at $3.00 = $30,963.00)
(11,288 cu/yds. at $6.00 = $67,728.00)
Buttress rock cost: $98,691.00
Total costs: $786,004.40

Removal and Replacement (Slide #10, East-Bound Lane)

(See Figures 13 and 14)
Length: 600'
Excavation: 125,585 cu/yds. at $1.97
Excavation cost: $247,403.00
Rock pad: 29,073 cu/yds. at $6.00
Rock pad cost: $174,439.00
Soil replacement: 85,095 cu/yds. at $1.97
Soil replacement cost: $167,637.74
Total costs: $589,480.00

Reinforced Earth Wall (Slide 10-A, East-Bound Lane)

Length: 800'
Height (highest point): 35'
R/E materials and erection: 18,826 sq. ft. at $8.55
R/E cost: $160,962.00
Select backfill: 18,638 cu/yds. at $4.50
Select backfill cost: $83,871.00
Excavation: 69,475 cu/yds. at $1.97
Excavation cost: $136,865.75
Random backfill: 16,257 cu/yds. at $1.97
Random backfill cost: $32,026.39
Rock drainage pad: 6,244 cu/yds. at $6.00
Rock pad cost: $37,464.00
Total costs: $451,189.44

The problems at Rockwood have been frustrating to say the least and, worse, they are not over. The west-bound lanes were opened in December 1972, but the conditions there are not totally stable. Discounting further major difficulties, the east-bound lanes should be completed by the latter part of this year (1973). However, problems will no doubt continue to crop up at various points for many years to come. There are, for example, a number of fills similar to those illustrated in Figures 7 and 13 that, while apparently stable at present, are very likely to deteriorate to some point of instability within the next several years. Such instability, as indicated previously, is usually brought on by changes in ground water levels followed by further deterioration of the fill and foundation materials.

To reduce and minimize the problems that might be brought on by changing water levels, subsurface horizontal drains are being installed at the base of selected fills. This is being done with a special boring machine that uses water as drilling fluid and "knock-off" drag or roller cone bits attached to the drilling steel. Slotted plastic (1½" inside diameter) pipe is inserted as the drill stem is removed. The plastic pipes serve as a casing to prevent the holes from squeezing closed while at the same time allowing water to flow through the perforations, into the pipe, and on to the outside fill slope. Approximately 30,000 feet will have been installed by the time the project is completed. In addition, the Department's Division of Soils and Geological Engineering will install instrumentation in some of the more critical areas to monitor ground movements and water levels.

The remedial measures used along I-75 in Campbell County were somewhat different than those at Rockwood, but then the circumstances were different. Here the roadway had already been opened and traffic had to be maintained virtually through the construction. Furthermore, all of the failures involved embankments and all occurred four to five years after initial construction. These embankments had been constructed across moderately deep to deep ravines and drainage swells in such a manner that blocked or impeded the natural drainage. While most of the surface water was taken care of by flumes, culverts, and pipes, the subsurface drainage, such as wet weather springs and seeps, were very often sealed off by the embankments. This resulted in the fills acting as earth dams which blocked off flow and caused a slow, steady rise in the water table. As the water table rose, the fill material became essentially saturated, and since it was primarily shale, it began to deteriorate, lose strength, and fail (Figures 15, 16, 17, and 19). Such a process is quite slow; hence, the reason for the embankments not failing for as long as five years after construction.

Since alignment changes, by and large, were not feasible and since complete removal and replacement was not altogether possible, it was necessary to devise a concept that utilized three different treatments. First, essentially all of the failed material had to be removed; second, since the high water table and perched water zones were contributing to the instability, they had to be lowered; and third, those portions of the fills that had not failed, plus the replaced material, had to be supported or buttressed.

To lower the water table in three of the four slides, horizontal drains were used. Several of these holes were drilled as much as 600 feet in length, with the initial flow of some being up to 400 gallons per hour. Subsequent measurements (2 and 4 weeks later) showed flows up to 200 gph. Altogether on the three slides, approximately 50,000 feet of horizontal drains were installed (Figure 18).

To lower the water table at the 840+00 (south-bound lane) slide (Figures 16 and 19), the Department employed a different method. There, twelve 100 foot vertical wells were drilled on 25-foot centers along the median above the slide, and equipped with ½ hp submersible automatically actuating pumps. Horizontal drains could not be used because of the height, steepness and deteriorated condition of the outside slope. Since the 840+00 slide was to be repaired using the removal and replacement method, and since the removal of this material was expected to jeopardize the stability of the north-bound lane, it was determined that the median area had to be dewatered. This would, hopefully, result in a shear strength increase of the material that was to be exposed in the median. As it was, the 1:1 slope (Figure 20) remained well intact to the full removal depth. Those who were involved in the

FIGURE 15. AERIAL VIEW OF AN EMBANKMENT FAILURE THAT OCCURRED ALONG THE NORTH-BOUND LANE OF I-75 IN APRIL, 1972.

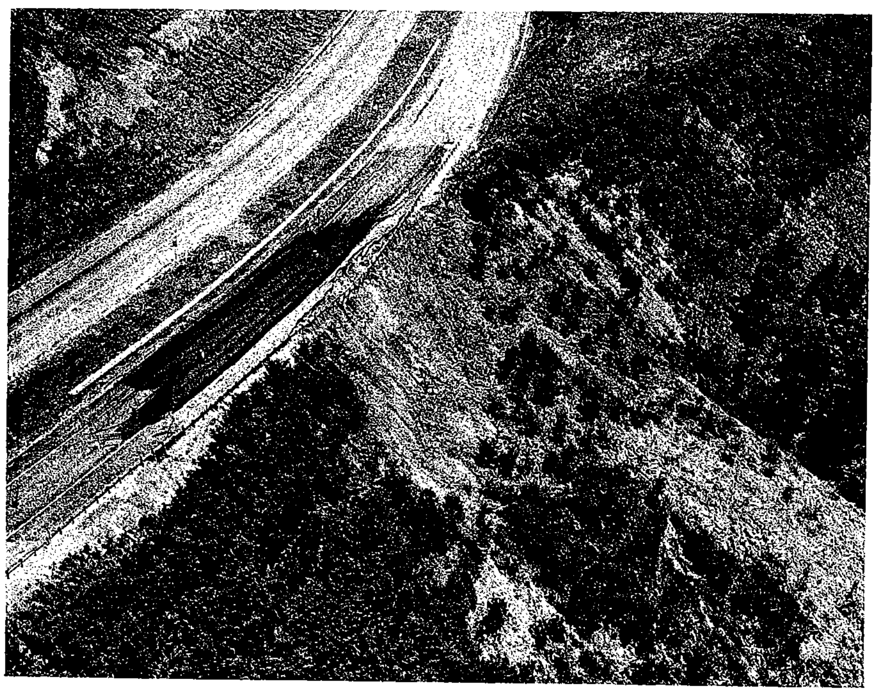


FIGURE 16. THE 840+00 (S.B.L.) FAILURE ALONG I-75 AS IT APPEARED IN APRIL, 1972.

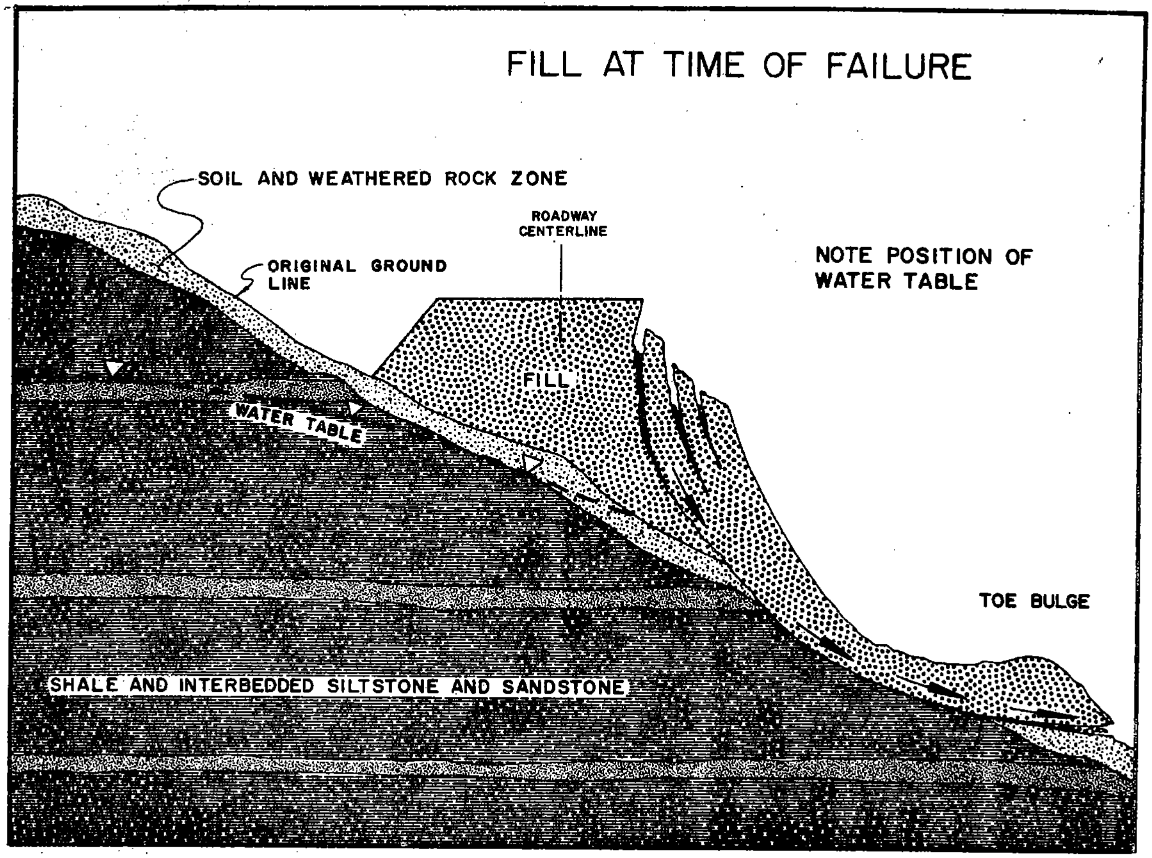


FIGURE 17. SCHEMATIC OF THE 1464+00 (N.B.L.) FILL FAILURE.

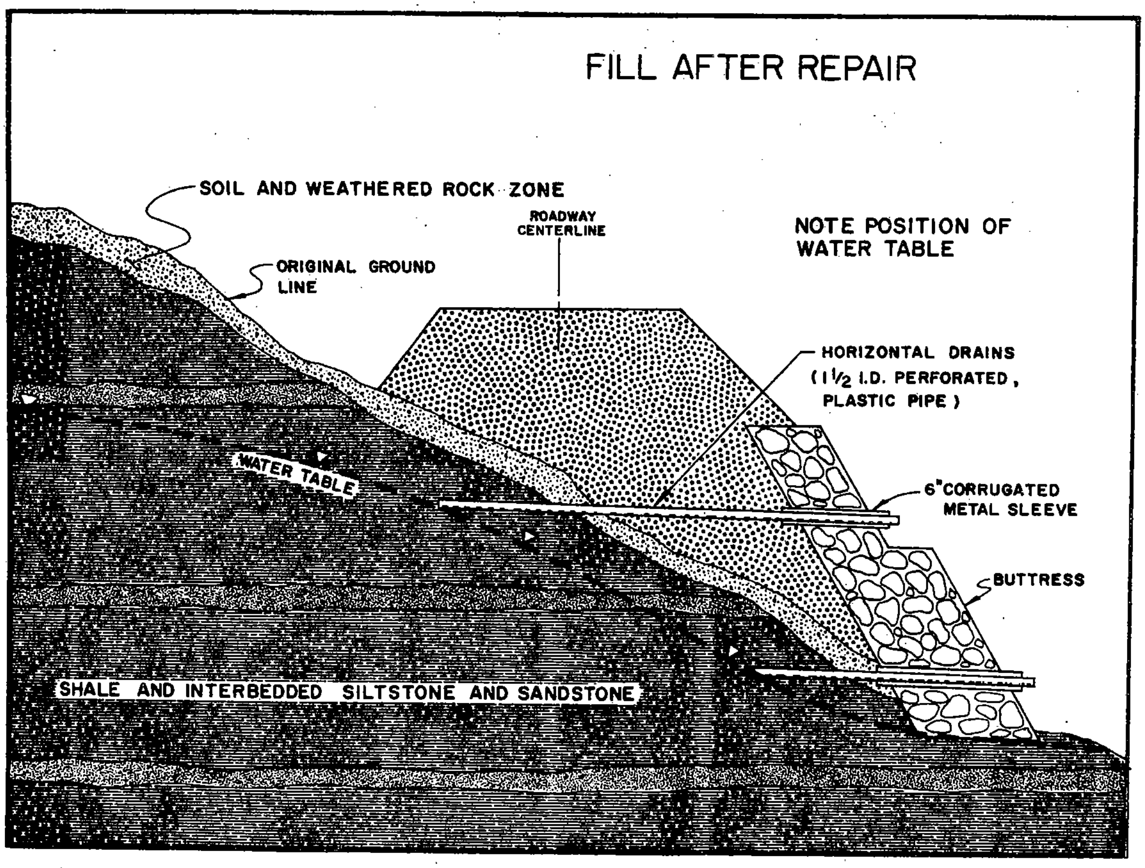


FIGURE 18. SCHEMATIC OF THE REPAIRED 1464+00 FILL.

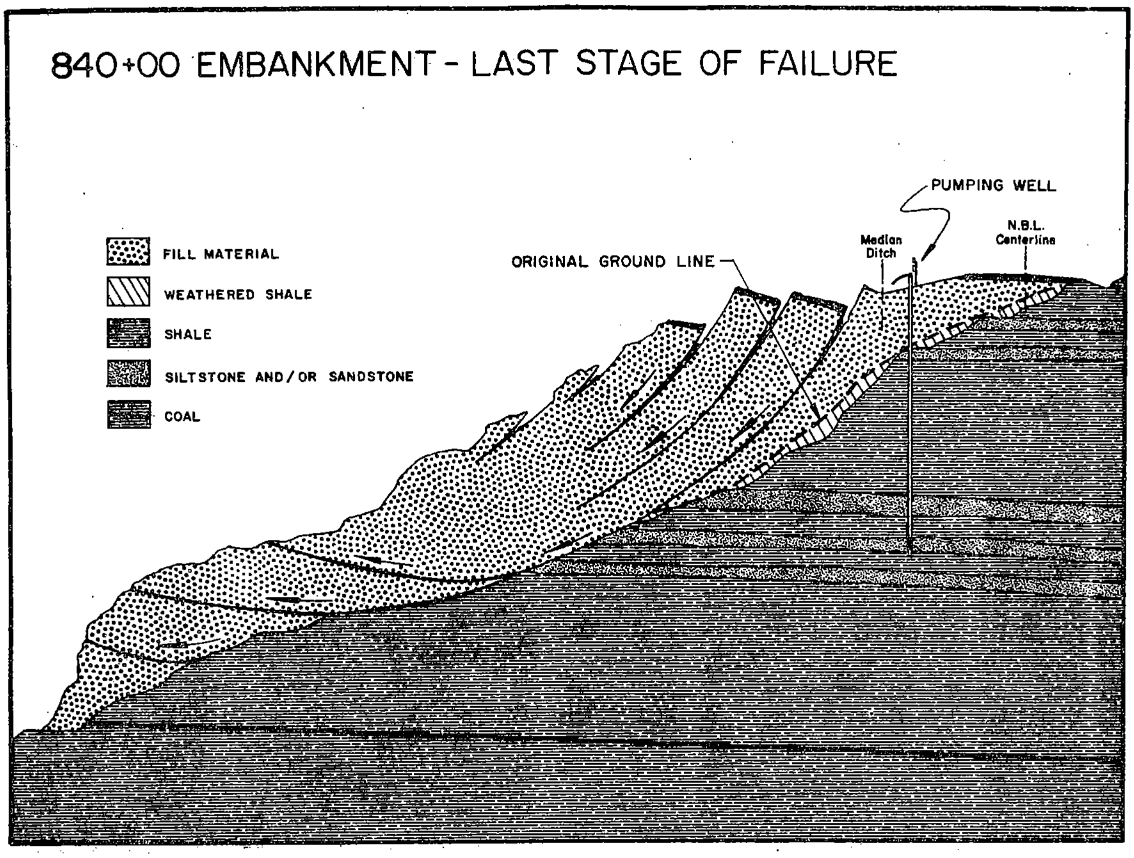


FIGURE 19.

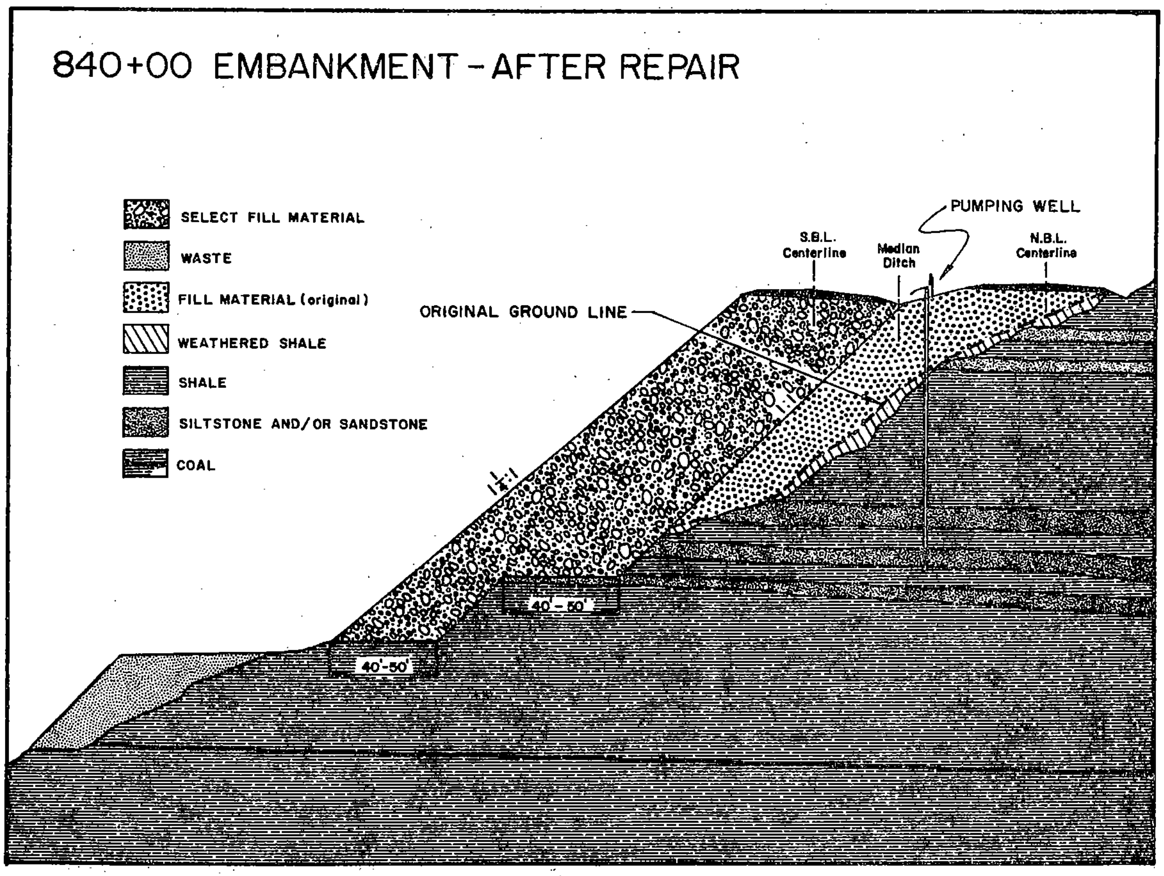


FIGURE 20.

construction of the 840+00 repair on a day-to-day basis unanimously agreed that the stability of the median slope had to be attributed to the dewatering operation.

CONCLUSION

There is no question but that the stability and costs of roadways constructed along the escarpment areas of the Cumberland Plateau in the future will depend on the significance placed on the soil and geologic conditions in the planning, location, and design stages. If these conditions are assigned a rather low priority in these stages, then we can expect the same sort of problems that we have encountered at Rockwood and in Campbell County. If, on the other hand, we have learned from our experiences, we will place a very high priority on these conditions. We might even be well advised to establish these conditions as <u>the</u> principal criterion in the selection of all future routes in these areas.

SLOPE STABILITY AND ENGINEERING CONSEQUENCES OF A STREAM CAPTURE

Bluff Creek, Northern California

By

Richard M. Wisehart[1]

J. Ross Wagner[2]

Bluff Creek drains 193 km^2 (74 $mi.^2$) of rugged mountainous terrain (1420 m (4650 ft.) maximum relief) near the town of Orleans in the Klamath Mountains of northern California. On the slopes in the lower part of this drainage basin are two forest roads and a state highway bridge which provide access to valuable stands of timber. The roads are being damaged and the bridge is threatened by slope failures related to a drastic alteration in the regimen of Bluff Creek.

On 22 December 1964, flood waters of an estimated 400-year flood overtopped then rapidly eroded a low divide of serpentinized peridotite separating Bluff Creek from the Klamath River. The resultant stream capture shortened the course of Bluff Creek by 2.0 km (1.2 mi.) and steepened the channel gradient to $\approx$ 33% at the newly established mouth. Downcutting of the channel started immediately and has since progressed rapidly upstream (2.4 km (1.5 mi.) in 5 years).

The rapid downcutting has initiated numerous debris slides which are damaging the roads in the lower part of the Bluff Creek basin. Of more concern, however, are the many large (> 0.4 km^2; 100 acres) dormant, rotational landslides which toe out in Bluff Creek. As downcutting proceeds, these slides may be reactivated causing much greater destruction to timber and roads. An attempt is made to evaluate this potentiality.

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SLOPE STABILITY AND ENGINEERING CONSEQUENCES OF A STREAM CAPTURE BLUFF CREEK, NORTHERN CALIFORNIA

by

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INTRODUCTION

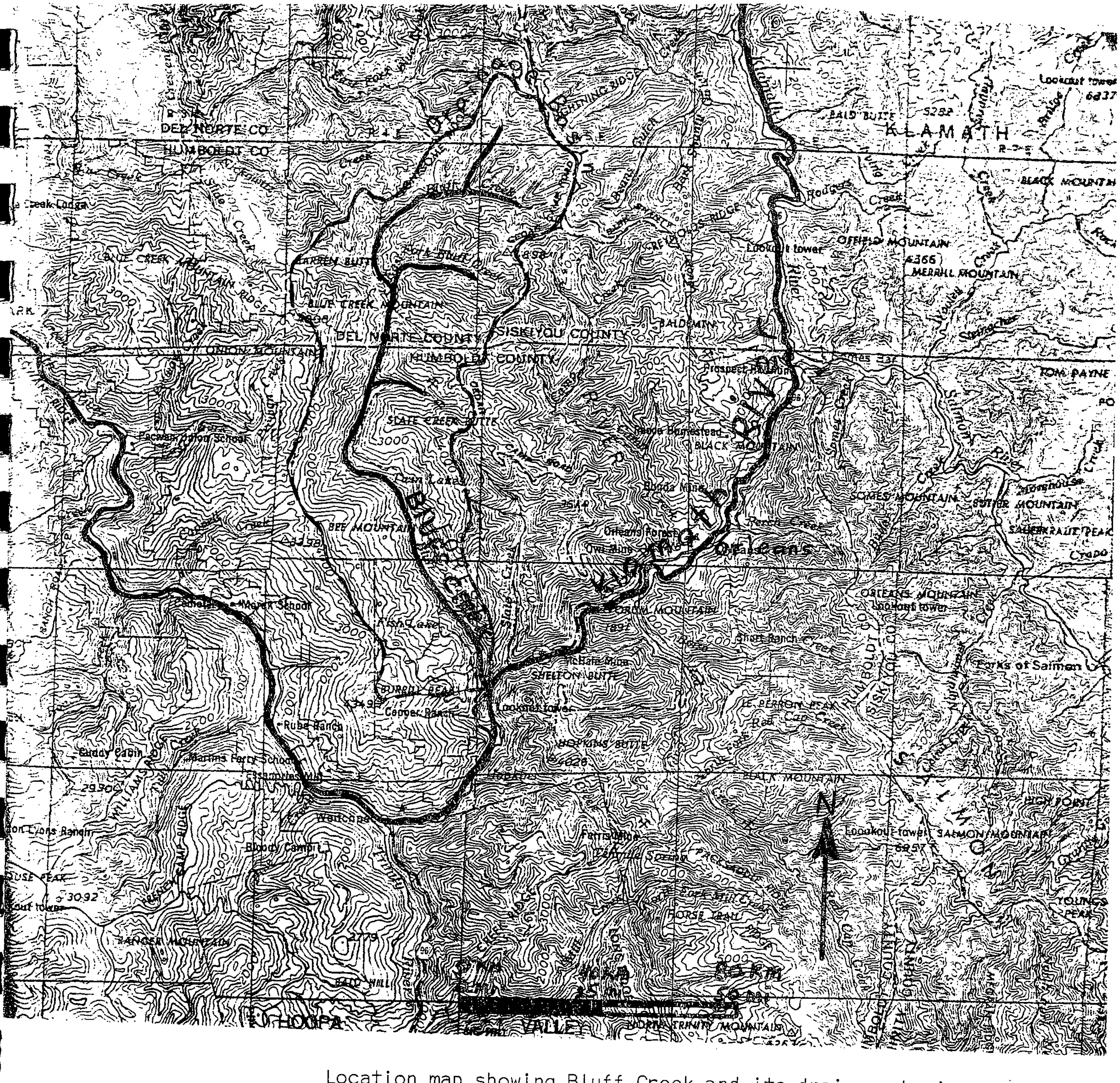
The Klamath Mountains of Northern California are a very rugged and imposing series of high peaks and deep valleys whose steep intervening slopes are covered with large quantities of valuable timber. The high peaks are the product of a regional uplift which began in the Pliocene and is still continuing. Uplifting is counteracted by vigorous fluvial erosion stemming from heavy seasonal rainfall. In excess of 1.3 meters (50 inches) falls during the winter months in this part of California.

Rapid downcutting of stream channels, caused by high relief and high rainfall, has recently been replaced locally by aggradation. The onset of aggradation appears from a series of aerial photographs spanning 30 years (1942-1972) to have occurred during the 1950's. This time coincides with an increase in logging activity in the Northern California area. It is speculated that large quantities of sediment derived from denuded logged slopes and logging access roads and delivered directly to adjacent streams is the major cause of the aggradation. (Work recently completed by Eugene Kojan of the U.S. Forest Service, and the authors, seems to confirm this speculation[2].)

The aggradation, in many instances, has either directly or indirectly caused severe damage to roads, bridges, and homes, by forcing streams out of their established channels. In one such instance, Bluff Creek, located near the town of Orleans, jumped its channel and was captured by the Klamath River. The stream capture resulted in serious damage to the slopes adjacent to the capture and to the roads and trees lying on those slopes. The damage to the roads and to the local environment has been substantial but it might have been avoided. Close geologic scrutiny of the area prior to the capture and the implementation of proper engineering controls based on that scrutiny, might have averted the capture and the resultant damage.

BLUFF CREEK

Bluff Creek is a 35 kilometer (22 mile) long north trending tributary of the Klamath River with a drainage area of approximately 193 km^2 (74 mi^2) (see Figure 1). Maximum relief within the basin is 1493 m (4890 ft). In its lower reaches, the channel gradient is about 3 percent.



Location map showing Bluff Creek and its drainage basin.

FIGURE 1

The rocks in the Bluff Creek drainage are approximately equally distributed between serpentinized peridotite or serpentinite and metasedimentary rocks and greenschist belonging to the Jurassic Galice formation (see Figure 2). The Galice formation lies stratigraphically over the serpentinite and the contact between the two is inferred from regional relationships to be a bedding plane fault. At the Klamath River, this contact, between serpentinite and greenschist, follows a northeast trend parallel to the river and has been mapped by W.R. Hansen and others[1] as the Boundary Fault.

The slopes on both sides of Bluff Creek have undergone considerable modification by large scale (>0.4 km^2; 100 acres) landsliding. The largest of these slides occur in serpentinite on the southwest side of Bluff Creek (see Figure 2). They are part rotational and part translational in character and at the present, none show any signs of recent activity and are therefore considered to be dormant.

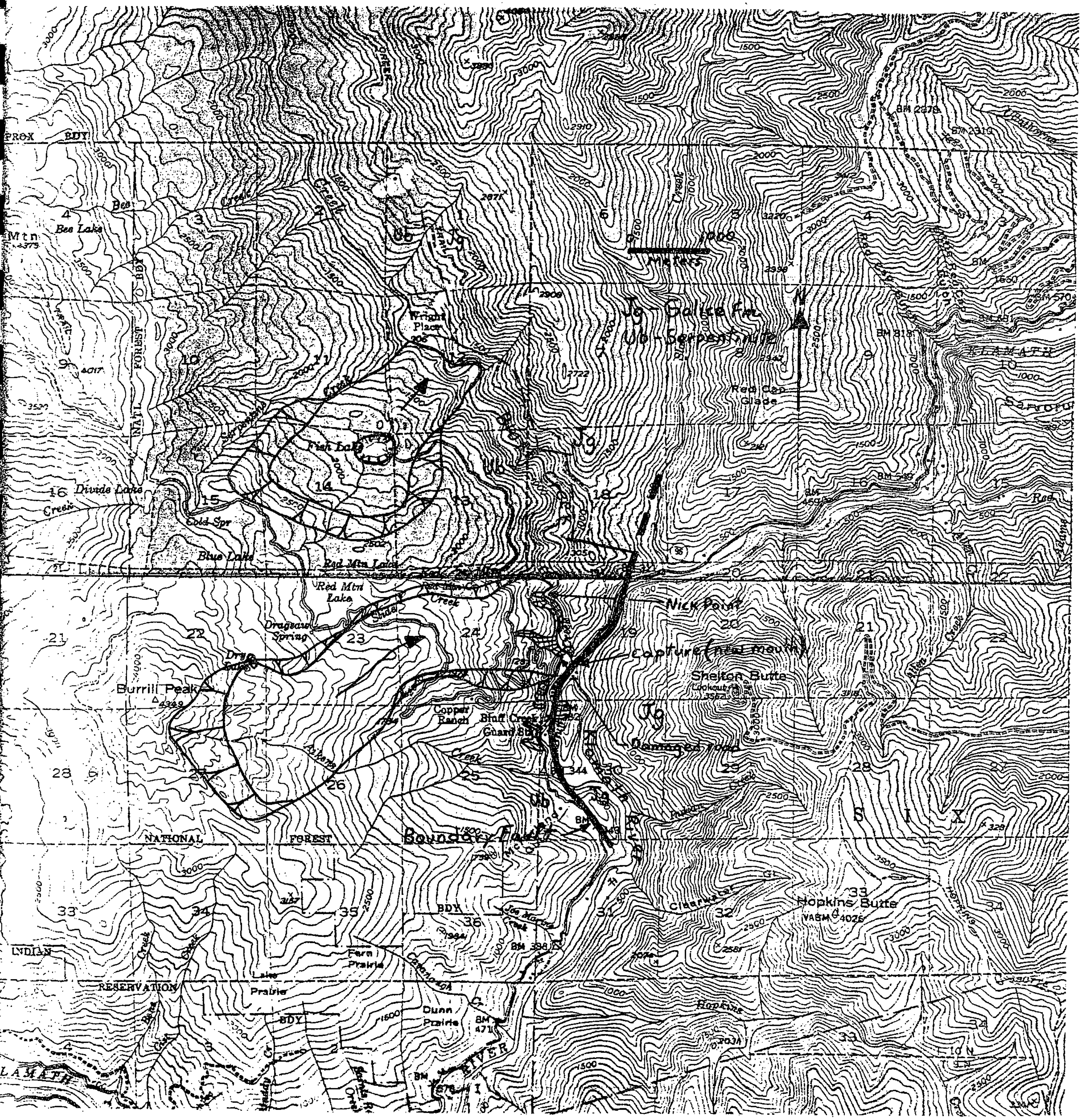
Immediately adjacent to Bluff Creek on both sides of the channel are many active debris slides. Most of these occur along the outside bank of bends in Bluff Creek, but near the present mouth they occur almost continuously along the channel walls regardless of stream orientation. Some of these slides are situated in the toes of the large rotational-translational slides where they are changing the mass distribution and subsequently reducing the stability of the large slides by removing material.

STREAM CAPTURE

The capture of Bluff Creek resulted from torrential rains and record flooding during December of 1964. More than 0.25 m (10 in.) of rain was recorded in the area during a 24-hour period on December 21-22. The stream gauge on Bluff Creek was destroyed by the ensuing flood, but slope-area measurements suggest the maximum flow was 770 cubic meters per second (27,000 cubic feet per second). This flow is estimated to represent a flood of 400-year frequency since it was approximately 33 percent greater than the previous record flood in the winter of 1955 and approximately 25 percent greater than the legendary flood of 1861-62[4].

On December 22, 1964, a low divide located 2.0 km (1.2 mi) up Bluff Creek from its junction with the Klamath River was breached by the record floodwaters. 1960 aerial photos show the divide separated Bluff Creek from the Klamath River at a point where the two streams were flowing parallel to and only about 70 m (220 ft) horizontally from each other. The channel of Bluff Creek at this point was about 50 m (165 ft) higher in elevation than the channel of the Klamath River and the crest of the divide stood only about 8 m (27 ft) above Bluff Creek's channel.

A cross section of Bluff Creek's channel exposed by the capture (see Photo 1) shows that apparently during the 1964 flood an additional 2 m (7 ft) of alluvium was deposited over the existing alluvium in Bluff Creek at the base of the divide. The grey color of this material in comparison with the red alluvium underlying it suggests that it is relatively fresh and probably derived from debris slides initiated upstream by the floodwaters. With the deposition of this material, the difference between the top of the divide and the channel of Bluff Creek was reduced to only about 6 m (20 ft).



Geologic and landslide map showing also the new mouth and nick point of Bluff Creek and damaged road.

FIGURE 2

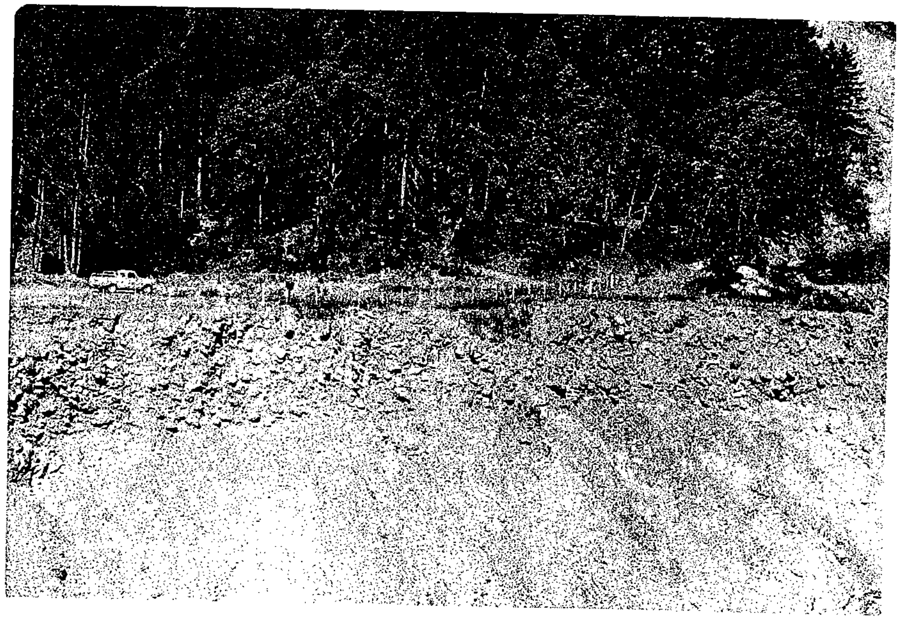


Photo 1

Alluvial deposits overlying serpentinite bedrock exposed in gorge cut in abandoned reach of Bluff Creek. Gray layer at the top represents material deposited during the 1964 flood.

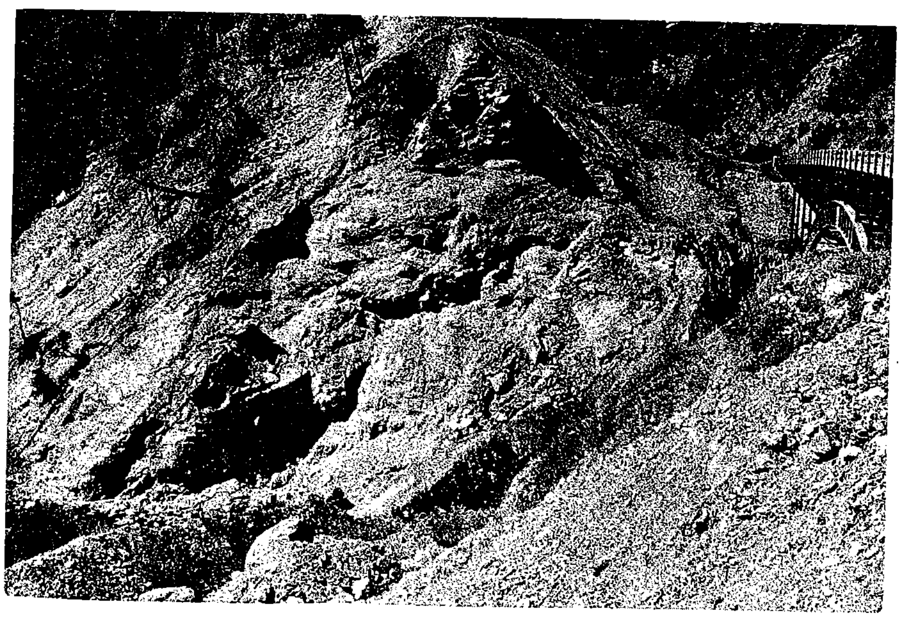


Photo 2

Boundary fault shear zone exposed in eroded divide. Greenschist overlies serpentinite. Note small rotational-translational slide in greenschist failing along shear zone.

Rough calculations using a channel configuration determined from aerial photographs taken in 1960 and an estimated stream velocity of 3 m per second (10 ft per second) were made to determine if the peak flow of 770 cms (27,000 cfs) was sufficient to overtop the 6 m (20 ft) high divide. These calculations indicate the maximum flow the channel could have accommodated without overtopping the divide was about 850 cms (30,000 cfs) or 80 cms (3,000 cfs) more than the estimated peak flow. In the 1955 flood, a local resident claimed the water in Bluff Creek came within 1.5 m (5 ft) of overtopping the divide. Although neither this personal account nor our crude calculations are conclusive, they do indicate that the flow in 1964 must have come very close if, in fact, it did not overtop the divide.

If overtopping was not responsible for breaching the divide, two other mechanisms may have been. One of these involves erosion by debris slides of the sides of the divide. Both the Klamath River and Bluff Creek made sharp bends at the divide and the outside of these bends were located at the base of the divide. Bank erosion was therefore quite vigorous at the base of the divide on both sides and the erosive force of both streams was undoubtedly at least partially responsible for destruction of the divide.

The other possible mechanism involves a prominent rock structure defect in the divide. The defect is a shear zone, representing the Boundary Fault, which is several meters (∿10 ft) wide in greenschist and serpentinite. From its orientation, as seen in the exposed cross section of the breached divide, it appears to have sliced through the divide, striking approximately parallel to it and dipping about 35° away from Bluff Creek, towards the Klamath River with greenschist overlying serpentinite (see Photo 2). In the divide, it apparently daylighted at the Bluff Creek channel on one side and on the other side at the Klamath River. A landslide along this shear zone could have removed the overlying greenschist, reducing the height of the divide by about 6 m (20 ft) or to the level of the Bluff Creek channel. A small landslide exposed on the south wall of the breached divide (Photo 2) and a larger landslide immediately north of the breach have both failed in a translational sense along this shear zone. They may be the remnants of a slide which removed the divide.

The evidence is inconclusive, however, as to whether overtopping or landsliding effected the breach. Perhaps it was a combination of both, a partial landslide facilitating overtopping. It is certain, in any event, that stream bank erosion on both sides of the divide set the stage for the breach and the capture of Bluff Creek.

The capture that occurred in 1964 was apparently not the first time Bluff Creek has been captured by the Klamath River. A relict channel of Bluff Creek lies about 30 m (100 ft) above and 0.5 km (0.3 mi) south of what used to be the mouth of Bluff Creek just prior to 1964. It also lies approximately along the strike of the Boundary Fault. The stream capture that formed this relict channel was probably an event quite similar to the 1964 capture.

CONSEQUENCES

The breach of the divide and consequent stream capture created a spectacular change in the regimen of Bluff Creek. Immediately, the length of the drainage was shortened about 2.0 km (1.2 mi). As a result, the decrease in channel elevation which previously occurred over 2.0 km (1.2 mi) at a 3 percent gradient was forced to occur over a horizontal distance of 70 m (220 ft) at a 75 percent gradient. This extreme gradient initiated rapid erosion of the underlying greenschist (assuming a landslide had not already removed it) and serpentinite as Bluff Creek began regrading its channel to its new base level.

An awesome gorge was cut into the serpentinite at the new mouth of Bluff Creek. In May 1973, the gorge was about 100 m (330 ft) wide at the top and was about 42 m (140 ft) deep measured from the level of the abandoned channel (see Photo 3). Most of the downcutting which formed the gorge apparently occurred very rapidly. A local resident claimed that all of the downcutting was accomplished during the first "few days" after the capture and that the mouth of the gorge has essentially not changed in appearance since. Upstream migration of the downcutting and gorge has continued, however, at an ever decreasing rate since the capture.

The gorge extends about 1100 m (3600 ft) upstream with the channel gradient at the bottom averaging about 7 percent. At the upstream end, the channel in the gorge intersects the old channel whose gradient, as yet unaffected by the regrading, is about 3 percent. This point is the nick point in the profile of Bluff Creek and it occurs where the stream crosses a resistant steeply dipping layer of unserpentinized peridotite. Regrading has temporarily halted at this point.

The total volume of alluvium and rock eroded from the gorge is estimated to be about 500,000 m^3 (700,000 yd^3). This volume equals 1.5×10^6 tons (2,000 lbs). Considering the average yearly sediment yield in the Klamath Mountains is estimated to be about 3,000 tons/km^2 (8,000 tons/mi^2)* the amount of material removed in the downcutting of Bluff Creek represents more than 2.5 times the yearly sediment yield of the entire Bluff Creek basin. In addition, debris slides precipitated by oversteepening of the canyon walls due to downcutting have contributed an estimated 1.6×10^5 tons of sediment or one quarter the yearly sediment yield of the basin.

Along the 1100 m (3600 ft) of newly downcut channel, 1800 m^2 (2200 yd^2 of the slopes are involved in active debris slides. This compares with 600 m^2 (700 yd^2) prior to the downcutting (see Figure 2) and it represents a 200 percent increase in debris slides. Along the abandoned reach of Bluff Creek on similar slopes, the increase in debris slides due to the 1964 flood is about 100 percent. This increase is attributed totally to sidewall erosion. The difference in the amount of increase along the downcut reach and that along the abandoned reach represents the debris slides caused by downcutting.

DAMAGE

The most immediate damage resulting from the capture of Bluff Creek was the destruction of the segment of State Highway 96 which crossed the breached divide. This damage was costly in terms of the interruption it caused in access to the town of Orleans. Several hundred people were isolated for several weeks while the road was closed. It was also costly in terms of

* This estimate is based on data provided in Water Supply Paper 1929[4].

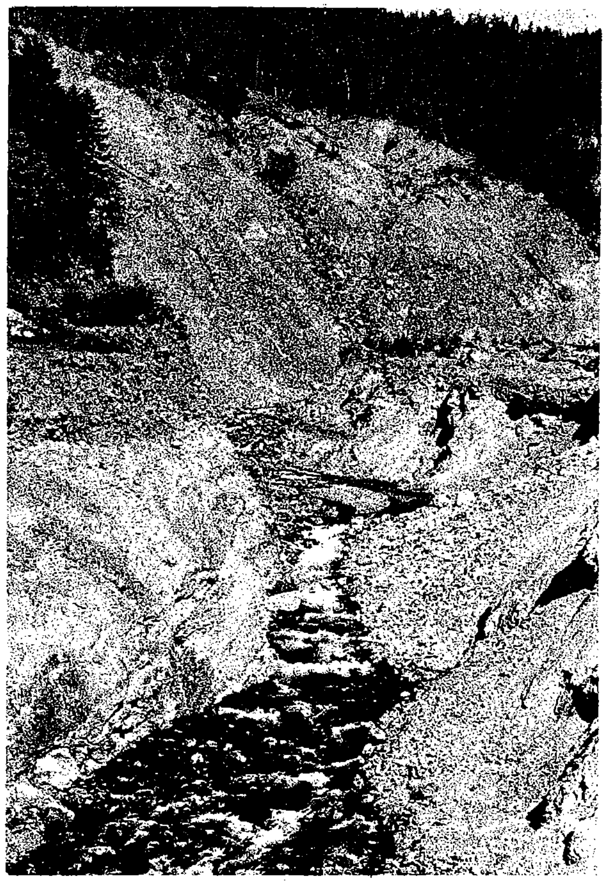


Photo 3

Gorge formed by rapid downcutting by Bluff Creek after December 1964 capture. The gorge is approximately 100 m wide at the top and 40 m deep.

repair to the road. A 122 m (401 ft) span steel arch bridge costing close to $1,000,000 was constructed to cross the chasm created by the stream capture.

The abutments for the bridge both lie about 15 m (50 ft) above Bluff Creek and are founded on highly jointed serpentinite. Joint sets on both sides of Bluff Creek dip into the stream and may pose a hazard to the abutments. At the present, however, the abutments appear to be secure, although rip-rap has been placed by the State Highway Department along Bluff Creek to protect the abutments from erosion.

In the gorge, a debris slide initiated by downcutting has damaged a portion of a Forest road lying on the slope above. This slide is located on the west side of Bluff Creek about 400 m (1300 ft) from the mouth of the creek (see Figure 2). It has removed a portion of that road and is threatening to remove some more. If the headward progression of this slide continues at the same rate aerial photos taken in 1970 and 1972 indicate, about 6 m/yr (20 ft/yr) average, the threatened segment of the road will be damaged in about 2 years. (Of course many factors control the enlargement rate of the slide, the most important of which is climate. Therefore, this prediction is heavily reliant on the premise that the next two years will have weather similar to 1970-72.)

Near this failure, a debris chute enhanced by the downcutting is causing erosion of a fill on the main Bluff Creek road (see Figure 2). Partly because of this problem and partly because of other road perils upstream, associated with Bluff Creek, abandonment of the road has been considered. This road is a major arterial in the Forest which provides access to millions of dollars of timber.

The large rotational-translational slides which toe out in Bluff Creek and are having their stability reduced by debris slide removal of material at their toes are potentially quite hazardous to the roads and trees which lie on them. The most immediate hazard is from the slide which toes out in the downcut reach of Bluff Creek where the most vigorous debris slide activity is occurring (see Figure 2). Several thousand dollars worth of timber and over 1 km (0.6 mi) of the main Bluff Creek road could be damaged if this slide is reactivated. Additional damage would be caused by the dumping of large quantities of sediment into Bluff Creek. As Bluff Creek continues to regrade its channel and downcutting migrates upstream, additional rotational-translational slides will be subjected to increased debris slide removal of their toes and subsequent reduction in stability.

At the new mouth of Bluff Creek, a delta formed by debris from the downcut channel has forced the Klamath River against its east bank. In so doing, bank erosion has increased and precipitated a slump in the slope above. This slump is supplying sediment to the Klamath River and is destroying the many trees which lie on its slope.

One more aspect to the damage caused by the stream capture has been the disruption of Bluff Creek to spawning salmon. As much as 40 percent of the salmon spawning grounds in the 30 km (20 mi) segment of the Klamath River from Somes Bar to Weitchpec occurs in Bluff Creek.[3]

CONCLUSION

The breach of the divide and resultant stream capture and all the damage it caused might have been prevented. The potential for breaching could have been determined; it certainly was alluded to during the flood of 1955. A cost-risk analysis probably would have shown that the amount of money required to prevent the breach would have provided a significant savings in comparison to the damage done.

Preventative measures would have been relatively inexpensive. A type of levee, such as an embankment, placed on top of the divide to increase the freeboard above Bluff Creek would have reduced the possibility of overtopping. A system, such as horizontal drains, to dewater the divide might have been used to lower any excess pore water pressure which could have facilitated landslide failure along the Boundary Fault shear zone. Rip-rap placed at the base of the divide on both sides might have protected the divide from the erosive attack of the Klamath River and Bluff Creek. Any or all of these measures could have been taken if the geologic and hydrologic hazards were first realized.

Although stream captures are fairly infrequent, any alteration in the regimen of a stream which initiates or increases downcutting and degrading of the channel will have an adverse effect on the stability of adjacent slopes. The erosional or depositional state of a stream, therefore, should be assessed and the potential for any natural "phenomena," such as stream captures, recognized before any roads or structures are placed near the stream. Careful geologic reconnaissance focusing on potential hydrologic hazards will lower the risk of stream caused road and slope failures.

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ENGINEERING GEOLOGIC MAP UNITS FOR HIGHWAY PLANNING - A QUANTITATIVE APPROACH

ABSTRACT

Soil and geologic maps prepared for highway planning often show established soil types or geologic formations as map units. A report or legend accompanying the map explains the engineering significance of the soil types or geologic formations. For highway planning, map units can be established which do not follow the conventional soil types or geologic formations. Statistical analysis of variation of physical properties of soils and rocks permits selection of map units which are significant in terms of some physical property or behavioral characteristic. These map units may be subdivisions or groupings of conventional soil types or geologic formations, and generally have consistant texture, grain-size distribution and mineralogic composition within specified limts of variation.

Through application of air-photo interpretation and photogrammetric procedures strip maps and profiles can be prepared of soil and rock units in terms of engineering properties or behavior.

INTRODUCTION

Geologists and many engineers are familiar with the use and interpretation of geologic maps for highway planning purposes. While being an invaluable tool for preliminary planning purposes, most general geologic maps are not detailed enough for design purposes. Furthermore, the map units of general purpose geologic maps, geologic formations, often do not represent a single or consistant lithology or material type. Many formations do not possess uniform physical properties necessary to define a material for engineering purposes.

Various types of soil maps, particularly those prepared by the Department of Agriculture, are also used and interpreted for highway planning purposes in some parts of the country where the weathered zone and soil layers are thick. In many cases agricultural soil maps are too detailed for highway engineering design purposes; and usually they describe the soils in detail to a depth of only about 3 feet.

While the general geologic map and the agricultural soil map describe their respective map units in their own standard terminology, engineering soils maps or engineering geologic maps describe earth materials more in terms of their physical properties, for example, grain size, plasticity, cementation, and so on. Usually in the material accompanying a standard engineering geologic map can be found a description of the engineering properities or behavior of the earth materials as shown in a typical example from the soil maps of Delaware (Table 1).

TABLE I. Qualitative evaluation of soil engineering properties from a typical engineering soil map. From Adams, Bogges and Coskery (1964).

Soil type[1]	Description	Origin	Engineering properties				Suitable compaction equipment
			In place		Disturbed[2]		
			Suitability as a subgrade[3]	Suitability as a wearing surface[4]	Suitability as embankment material	Compaction characteristics	
AM2	Nonplastic to slightly plastic, sandy soil.	Fluvial deposits of Pleistocene age.	Good	Excellent to good depending on binder present.	Good	Good	Rubber-tired equipment.
AM23	Nonplastic, generally poorly graded sandy soil.	Fluvial deposits of Pleistocene age.	Good to fair.	Excellent to good depending on binder present if surface is A-2. Fair if surface is A-3.	Good if predominant material is A-2. Fair if predominant material is A-3.	Good if predominant material is A-2. Poor if predominant material is A-3.	Rubber-tired equipment for soil which is predominantly A-2. Vibratory equipment for soil which is predominantly A-3.
AM24	Nonplastic to slightly plastic, sandy and silty soil.	Fluvial deposits of Pleistocene age.	Good if material left after grading is predominantly A-2. Fair if material left after grading is predominantly A-4.	Excellent to good depending on binder present if surface is A-2. Fair to poor if surface is A-4.	Good if predominant material is A-2. Fair if predominant material is A-4.	Good if predominant material is A-2. Fair if predominant material is A-4.	Rubber-tired equipment.
AM26	Nonplastic to highly plastic, sandy and clayey soil.	Fluvial deposits of Pleistocene age.	Good if material left after grading is predominantly A-2. Poor if material left after grading is predominantly A-6 or poorly drained A-2.	Good if surface is A-2. Poor if surface is poorly drained A-2. Very poor if surface is A-6.	Fair if predominant material is A-2. Very poor if predominant material is A-6.	Fair if predominant material is A-2. Poor if predominant material is A-6.	Rubber-tired equipment for soil which is predominantly A-2. Sheep's-foot rollers for soil which is predominantly A-6.
AM3	Nonplastic, poorly graded sandy soil.	Fluvial deposits of Pleistocene age.	Fair	Fair	Fair	Poor	Vibratory equipment.
AM4	Slightly plastic, silty and clayey soil.	Fluvial and possibly eolian deposits of Pleistocene age.	Fair to poor.	Fair to poor.	Fair to poor.	Fair to poor.	Rubber-tired equipment.
AR	Alluvial gravel, sand, silt and clay.	Alluvium of Recent age.	Variable	Variable	Variable	Variable	Variable.
MTM	Soil rich in organic material and subject to inundation by high tides. No definite profile.	Tidal marsh deposits.	Variable	Variable	Variable	Variable	Variable.
U	Urban areas where soil has been altered extensively by man.	Undetermined	Variable	Variable	Variable	Variable	Variable.
Z	Soil rich in organic material and frequently poorly drained. May be underlain at shallow depths by gravel, sand, or clay.	Swamp deposits of Pleistocene age.	Variable	Variable	Variable	Vairable	Variable.

[1]Two different soil types may be combined into a single map symbol (AM2/24), but the engineering characteristics of the individual soil types are described separately.
[2]For soil types designated by two-digit numbers, these columns refer to the composite soil.
[3]When not subject to frost action. Frost will affect soils that contain appreciable silt and clay and have a high moisture content.
[4]Untreated. Additives may aid in stabilization of the sandy soils and minimize dusty conditions.

This type of general qualitative description leaves much to be desired when the values of specific soil properties are required for highway design purposes. Liu and Thornburn (1965) at the University of Illinois made a significant advancement when they used statistical procedures to define the average value and confidence limits of some of the engineering index properties of agricultural soil types. Data for this analysis was obtained by a rigorous sampling and testing procedure applied over a large area encompassing several counties in Illinois.

Soil engineering maps have been prepared by highway engineers in South Africa showing the distribution of both soils and rocks (Kantey and Williams, 1962). Sampling and testing of the soils and rock units was performed throughout the mapping area. A statistical study was made of the results of the soil tests to provide reliable information on soil properties for preliminary evaluation of pavement requirements. The study defined the mean value, range and standard error of selected soil index properties including liquid limit, plasticity index, linear shrinkage, clay fraction and California bearing ratio. It was concluded that if the coefficient of variation for the soil properties did not exceed 30% that the soil properties for the map units could be used reliably for preliminary design purposes.

As an alternative to using conventional formations or agricultural soil units for highway materials maps, this paper discusses the selection of mapping units representing soils or rocks which are distinctive in terms of some particular physical property. Any quantitative property or group of properties may be used for the selection of significant mapping units. In this investigation the primary argument for the selection of map units was the pavement foundation condition, while grain size particularly with respect to construction material was the secondary argument. Hveem's "R" value for subgrade resistance was used as a measure of the foundation conditions, however the CBR value or any other value could have been used.

BASIC DATA AND DETAILED MAPPING

A ten mile section of proposed highway relocation from Kaycee to Barnum, Wyoming, was used as a study area. The general location is shown in Figure 1. Topographically the area consists of a number of sharp ridges and intervening valleys parallel to the front of the Big Horn Mountains. The ridges are cut by the flood plain of the Middle Powder River. The geology of the area consists of sedimentary rocks, predominately shales, shaley sandstones, siltstones and sandstones of Jurassic and Cretaceous ages as listed in Table II. Figure 2 shows the general geology and cross section of the area. Folding and faulting cause the repetition of the units several times throughout the study area. The lack of vegetation and thinly developed topsoil made the area an ideal study area. The many geomophic features of the area were important not only as guides for air photo interpretation mapping, but also they are significant with respect to the material types within the area. Geomophic features include bedrock ridges, strike valleys, flood plain and terraces, and the ever present coalesced alluvial slope deposits or colluvium.

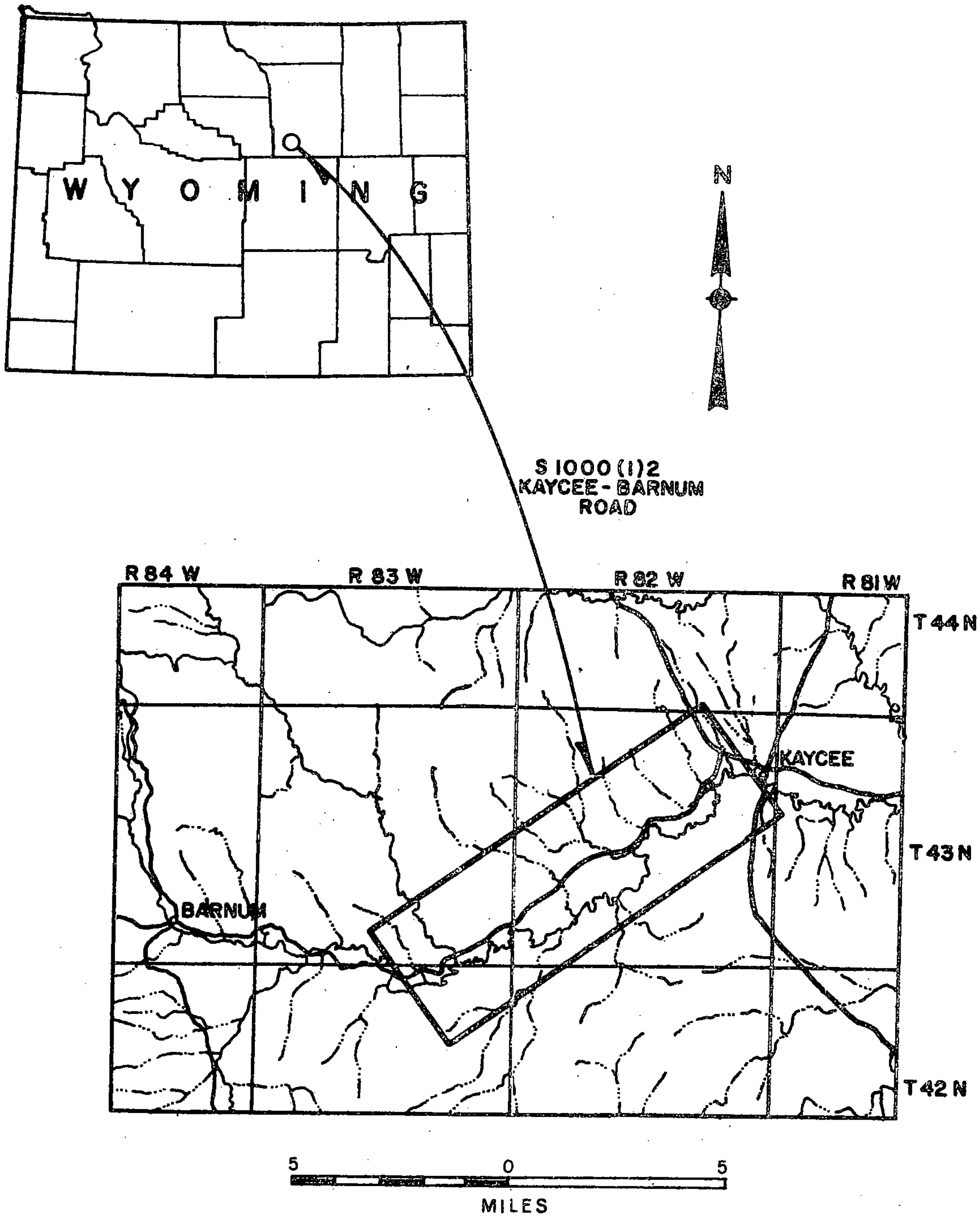


FIGURE 1. Location of study area, Kaycee-Barnum road.

TABLE II. Generalized geologic section of bedrock deposits adjacent to Kaycee-Barnum road.

AGE		FORMATION	MEMBER OR UNIT	THICKNESS	LITHOLOGIC DESCRIPTION
CRETACEOUS	UPPER CRETACEOUS	Cody Shale		3000'±	Light gray silt shale for upper two-thirds, light gray calcareous clay shale lower third. Topographically weak, forms bad land topography where severely eroded, often has piping erosion.
		Frontier formation	First Wall Creek Sand	50'	Tan crossbedded massive sandstone shaley toward base with carbonaceous shale partings in lower part. Calcareous cementing. Resistant topographically, forming sharp ridge.
			Upper shale unit	200'±	Sandy yellow and gray shale with bentonite zone in lower part. Weak topographically, forming valleys.
			Second Wall Creek Sand	80'±	Gray and tan fine-grained "salt-and-pepper" textured friable sandstone, conglomeratic with black chert pebbles in lower part. Resistant topographically, forming less sharp ridge than First Wall Creek sand.
			Lower shale unit	400'±	Alternating yellow, gray, black carbonaceous shales, sandy shales with bentonitic zones at top below Second Wall Creek sand and near middle. Weak topographically, forming valleys between strike ridges or soft hills where flat lying. Sandy zones form topographic breaks.
			Bentonite	5'	Yellow-green high-grade bentonite clay.
			Total Frontier	735'±	
	LOWER CRETACEOUS	Mowry Shale		200'	Dark gray to black fissile shale with siliceous cementation. Very bentonitic with 3' bentonite bed in upper 50', and numerous 1' beds, thinner seams and partings throughout. Very resistant topographically, exposed in steep slopes. Weathers to characteristic silver gray color.
		Thermopolis formation	Upper Thermopolis shale member	70'	Black, hard, chunky shale with numerous ironstone concretions and characteristic cone-in-cone structure. Slightly bentonitic in parts. Weak topographically, weathers to a black soil.
			Muddy Sandstone member	50'	Light gray very fine-grained, heavy-bedded friable sandstone. Exhibits ripple marks. Weathers to yellow-brown. Variable resistance, forms broken minor topographic ridge in some places, but in others quickly disintegrates to form a light brown sandy soil. Lower 40 feet is less resistant with numerous soft black shale seams, weathers rapidly to tan sandy soil.
			Lower Thermopolis shale member	120'	Gray to black, thin-bedded soft, flaky paper shale. Weathers quickly to a heavy black soil.
			Total Thermopolis	240'±	
		Cloverly formation		30'	Yellowish white small-pebble conglomerate and predominately gray-white fine grained massive sandstone, well cemented. Very resistant ridge forming unit which usually forms capping over Morrison formation. (In other areas considerably thicker with two main sand units and shale unit between.)
JURASSIC		Morrison formation		200'	Variegated maroon, olive green, gray, white, tan and red shales and some sandstones. One white fine-grained friable porous sandstone appears about 20 feet from the base, approximately 30'-40' thick forming a small ridge in otherwise weak shales.
		Sundance formation		10'±	Gray crystalline sandy coquinal limestone or calcareous sandstone. Resistant.
				100'±	Gray-green thin-bedded sandy shale, non-resistant except for the more sandy zones.
				80'±	White, buff, greenish gray, tan sandstones and siltstones, variable bedding from thin to heavy, loosely cemented, resistant topographically, forming ridges.
				60'-160'	Light gray to buff thin bedded sandy shale, weak topographically.
			Total Sundance	260-360	

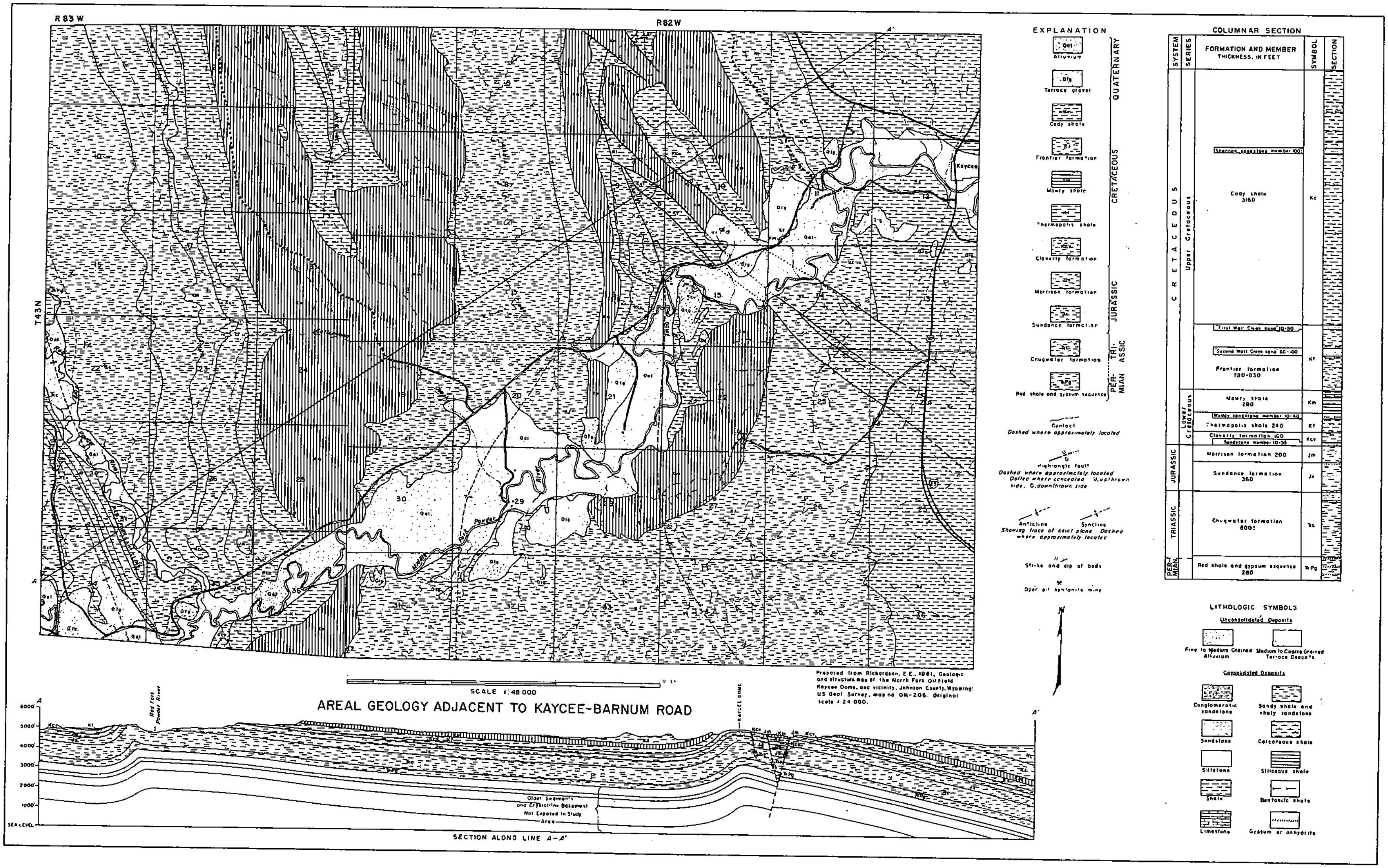


FIGURE 2. General geology map of the area along the Kaycee-Barnum road. Modified from Richardson (1961).

The entire study area in a band approximately one mile wide was mapped in detail using air photo interpretation. All soil and rock units having apparently differing textures or grain sizes, lithologies. compositions, outcrop conditions, and stratigraphic position were mapped. In all over 55 different units were identified and described. The photo mapping was transferred from the original aerial photos having a scale of approximately 1600 feet per inch to photo enlargements with a scale of 400 feet per inch which provided better visual reference than the original photos. An example of the photo maps is shown in Figure 3 reduced for publication.

The map units were then plotted to an exact scale of 400 feet per inch using a photogrammetric plotter in order to provide the overall planimetric accuracy required for the plotting of sample locations and analysis of data with reference to the map units determined from photo interpretation.

Data on the physical properties of the materials in the area was obtained from the soil profile survey performed by the Wyoming Highway Department along the centerline of the proposed highway relocation. Included in the data was grain size of the material sampled, liquid limit, plasticity index and Hveem's "R" value for subgrade supporting ability.

ANALYSIS OF MAPPED UNITS - EVALUATION OF SOIL DATA

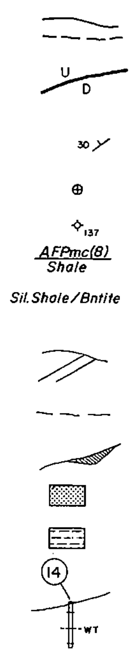
The object at this point was to evaluate the physical data with respect to the appropriate map units and subsequently to determine the optimum map units for highway engineering purposes which were (1) distinct from one another, and (2) free from excessive variation in physical properties. Additional considerations were that the map units be easily recognized, repetitive from one location to another, and at a level of detail practical for engineering design purposes.

The approach used to achieve this objective is outlined below. Several trial mapping categories were first selected based on preliminary considerations and opinions developed during the photo interpretation. These categories and corresponding map units are listed in Table III. The following steps in the analysis included:

(1) Plotting of geologic test hole locations on geologic base map using centerline station and offset distance;
(2) Preparation of overlays to geologic base map indicating map units of each mapping category based on a recombination or grouping of the original photo map units;
(3) Sorting of geologic test hole samples into groups according to location of each sample within map units indicated on overlays of each analysis category;

EXPLANATION OF SYMBOLS AND MATERIAL TYPES – SOIL SURVEY PLAN AND PROFILE SHEETS

MAP SYMBOLS



- Boundary between soil or rock units – dashed where approximate.
- Fault–letters indicate relative displacement up or down.
- Dip of tilted beds – short line indicates dip or direction of downward tilting of beds, number indicates degrees of tilting (dip).
- Indicates horizontal bedding.
- Location and number of test hole.
- Map unit designation – upper material type & depth – lower material type.
- Intermixed materials, design for poorer type.

PROFILE SYMBOLS

- Boundary between soil or rock layers.
- Boundary between unconsolidated deposits and consolidated bedrock.
- Existing fill material.
- Sandstone material which may require rock excavation.
- Siliceous shale which may require rock excavation.
- Location, number and depth of test holes. Lines indicate material changes in log of test hole. Water table indicated when observed.

* Low mean indicates values for poorer soils of group, high mean for better soil subgrade conditions.

DESCRIPTION AND ENGINEERING PROPERTIES OF SOIL SURVEY MATERIALS

Material Type	Designation on Plan and Profile	Material Description	*	Sieve Analysis – % Passing # 4	#10	# 40	#200	Liquid Limit	P.I.	"R" Value	AASHO Class.	Unified Class.	Remarks
Alluvium: Floodplain deposits, medium to coarse grained	AFPmc	Layered deposits of sand and gravel with irregular deposits of silt and clay. Includes river and stream floodplains and floodplain terraces.	Low Mean	90.47	83.27	73.99	52.76	34.40	18.01	25.74	A-6(6)	CL	Low mean values should be used where silt and clay deposits are near grade. Average value used elsewhere.
			Average	82.35	74.47	64.11	44.37	30.76	14.05	34.17	A-6(3)	SC	
			High Mean	74.23	65.67	54.23	35.98	27.12	10.09	42.50	A-4(0)	SC	
Alluvium: Terrace deposits, medium to coarse grained	ATmc	Older deposits of sand and gravel with silt intermixed. Deposits located on hillsides and hilltops above present floodplain.	Low Mean	90.47	83.27	73.99	52.76	34.40	18.01	25.74	A-6(6)	CL	Good sources of construction materials. Higher deposits tend to be more coarse; use highmean for these.
			Average	82.35	74.47	64.11	44.37	30.76	14.05	34.17	A-6(3)	SC	
			High Mean	74.23	65.67	54.23	35.98	27.12	10.09	42.50	A-4(0)	SC	
Alluvium: fine to medium grained	Afm	Varied deposits of silt, sand and clay on slopes and in minor stream valleys. Includes colluvial slope deposits, valley fill and talus.	Low Mean	97.83	95.13	91.67	57.94	31.08	14.67	22.93	A-6(6)	CL	
			Average	95.65	91.23	86.15	53.88	29.23	12.67	27.00	A-6(5)	CL	
			High Mean	92.47	88.33	80.63	49.82	27.38	10.71	31.07	A-6(3)	SC	
Sandstone	Sandstone	Predominantly fine-grained sandstone bedrock, not strongly cemented. Major units 50-80 ft. thick.	Low Mean	100	100	99.77	58.50	30.98	11.86	25.57	A-4(6)	CL	Some units will require rock excavation.
			Average	94.72	92.27	88.90	50.07	24.36	8.36	36.63	A-4(3)	CL	
			High Mean	85.62	82.29	78.03	41.64	17.74	3.86	47.69	A-4(1)	SC	
Shale	Shale	Fine-grained bedrock composed mainly of silt and clay.	Low Mean	98.06	94.71	89.70	65.17	40.14	23.39	15.32	A-7-6(11)	CL	Can be excavated by normal methods.
			Average	97.36	93.45	86.90	62.16	37.30	20.75	17.27	A-7-6(9)	CL	
			High Mean	96.66	92.19	84.10	59.15	34.46	18.21	19.22	A-6(8)		
Bentonitic Shale	Bnt.Shale	As above but containing a significant amount of the clay bentonite throughout.	Low Mean	98.60	94.97	90.47	73.95	78.63	59.92	5.75	A-7-6(20)	CH	Can be excavated by normal methods. Swells when wetted. Use average values except where pure bentonite.
			Average	95.54	90.31	84.04	66.73	64.31	46.04	6.54	A-7-6(18)	CH	
			High Mean	92.48	85.65	77.60	59.51	49.99	32.16	7.33	A-7-6(14)	CH	
Bentonite	Bntite	Layers of very pure bentonite clay up to 4 ft. thick, very plastic, highly swelling.	Low Mean										Must be subexcavated.
			Average	98.60	94.97	90.47	73.95	78.63	59.92	5	A-7-6(20)	CH	
			High Mean										
Siliceous Shale	Sil.Shale	Very hard cemented shale. Bentonite always found in association.	Low Mean										Requires rock excavation methods. Use values for bentonite above for pavement design.
			Average	98.28	77.71	54.14	37.59	40.14	19.85	35.42	A-6(3)	SC	
			High Mean										
			Low Mean										
			Average										
			High Mean										

FIGURE 3. A typical example of one of the air photo strip maps of the Kaycee-Barnum road showing the detailed map units. The original map scale was 400 feet per inch. From Edwards, 1972.

TABLE III. Categories of soil or rock units initially selected for analysis to find the significant soil-rock classification system.

Analysis Category	Map units comprising each analysis category
Combined samples	(no map units)
Soil samples grouped according to geologic formations on areal geologic map	Sundance formation Morrison formation Cloverly formation Thermopolis shale Mowry shale Frontier formation Cody shale Terrace gravels Alluvium
Soil samples grouped according to broad lithology	Sandstone Shale Alluvium
Soil samples grouped according to photo map units	

(4) Tabulation of the statistical analysis of the soil physical properties, percentage passing nos. 4, 10, 40 and 200 sieves (P4, P10, P40, P200), liquid limit (PLL), plasticity index (OPI) and Hveem's resistance value (R), to determine the maximum and minimum values, range, mean value, standard deviation and coefficient of variation for the samples within each map unit of each analysis category.

The standard deviation and variation were determined for all of the samples combined to determine an upper limit against which to compare the results of the analysis categories. The combined samples should in this manner represent the maximum variation. The areal geology map units were taken from the Areal Geology Map by Richardson (1961). The map was enlarged to a scale of 400' per inch and overlayed on the geologic base map to determine the samples falling with each geologic formation.

The statistical properties of each analysis category were compared using the hypothesis that map units representing valid engineering soil units will have the lowest possible standard deviation (S) and coefficient of variation (CV). Kanty and Williams (1962) in South Africa indicated a coefficient of variation on the order of 30% for valid engineering soil units.

Typical values for S and CV found by other authors are indicated in Table IV. The values in the first two columns are representative of values obtained for tests on several different samples of a particular soil type. Thus, they are representative of the natural variability within an identifiable soil type. The values in the next four columns are from several tests performed on material from the same sample. As such, they represent the amount of variability within the particular test, either repeatability by the same technician, or, reproducibility between technicians (see Mandel (1971) for a discussion of repeatability and reproducibility).

These values have been used as a guide in this study for determining the valid engineering soil units. The values in the first two columns of Table IV are considered to be the typical values of S and CV which may be expected for valid engineering soil units. The values in the next four columns are considered to be the minimum values of S and CV. The closer the engineering soil units can be grouped and defined, the closer their S and CV will come to these minimum values.

TABLE IV. Typical values of standard deviation (S) and coefficient of variation (CV) for some soil properties obtained by several authors.

SOIL TYPE	Natural	Natural	Natural	Controlled		
SAMPLES	Multiple	Multiple	Single	Single		
TESTS PER SAMPLE	Single	Single	Multiple	Multiple		
CLASSIFICATION	Agricultural Soil Types	Genetic - Textural	"R" Value Ranges	Sandy Silt	Silt	Silty Clay
SOURCE	Thornburn & Larsen (1959)	Kantey & Williams (1962)	Ames et al. (1971)	Liu & Thornburn (1965b)		
LIQUID LIMIT						
S	3.8 - 6.4	4.7 - 12.5		.36 - .67	.43 - .96	.52 - .59
CV	7.0 - 15.8	20 - 38		1.31 - 2.41	1.24 - 2.83	1.16 - 1.29
PLASTICITY INDEX						
S	2.7 - 5.5	3.0 - 6.2		.59 - .71	.79 - .94	.82 - .90
CV	13.4 - 34.0	29 - 43		5.15 - 6.20	7.20 - 7.81	3.37 - 3.78
CLAY CONTENT						
S	1.8 - 5.1	4.0 - 7.6				
CV	5.1 - 18.8	42 - 74				
"R" VALUE						
S			2.7 - 6.8			
CV						
Significance of indicated ranges	Values for different soils		Values for different soil technicians			

The primary purpose of the map units to be defined in this study is the indication of subgrade conditions. Therefore, in the analysis to determine valid map units, the emphasis is placed on achieving the lowest possible deviation and coefficient of variation for the "R" value of the soils.

With regard to the "R" value, Ames, Benson and Sundquist (1971) are in the process of determining a precision statement for the Hveem stabilometer test for "R" value. Table V indicates the standard deviation and expected range of results for their preliminary findings. Consistent with these findings is the results of a series of tests performed within the Wyoming Highway Department Materials Testing Laboratory (personal communcation from Patrick Nolan, Soils Engineer, Wyoming Highway Department). Five different soil technicians tested material from the same sample to determine the "R" value. Results ranged from about 23 to 32.

In consideration of these results, overall values of 6 for standard diviation and 30 for coefficient or variation were selected for use in this study.

The standard deviations and coefficients of variation of some map units for the initial categories listed in Table III were found to be somewhat higher than the acceptable values of 6 and 30. S ranged from an average of 11.5 to 15.5 for the categories and the properties LL, PI and R, while CV ranged as high as 80. Consequently, other analysis categories and units thought to be more represenative of engineering requirements were selected. These are indicated in Table VI.

The analysis category based on the geologic formation grouping of samples was modified to place samples into more correct groups as a result of observations in the field and on the aerial photography. More specific classifications of alluvial material than shown on the areal geology map were added and the Cody shale unit was deleted since field observations indicated the surface materials in this area to be colluvial deposits. A new lithologic classification category was devised having more detailed map units than the previous one yet attempting to keep the number of map units to the minimum. Tables VII and VIII list the standard deviations and coefficients of variation of the soil properties for the various trial map units.

The basic statistics of each map unit in the various analysis categories were calculated using a computer. The soil physical properties from the lab tests were punched on cards, one card per soil sample. For each analysis category the cards were sorted, placing the samples into their appropriate map units, then read by the computer to calculate the statistics for each map unit.

TABLE V. Standard deviation and acceptable range of two results for the Hveem stabilometer "R" value. These values are considered as the absolute minimum values for ideal engineering soil units. These preliminary findings are furnished by and cited with the permission of the authors (Ames et al. 1971).

R-Value Range	Standard Deviation	Acceptable Range of Two Results
0 - 30	4.43	12
31 - 69	6.78	19
70 and above	2.70	8

TABLE VI. Categories of soil-rock units selected for analysis subsequent to those in Table IX.

Analysis Category	Map units comprising each analysis category
Soil samples grouped according to accepted geologic formations as identified and located from field and photo observations	Sundance formation Morrison formation Mowry shale Frontier formation Terrace #4 Terrace #1 Flood plain Streambed Colluvium
Soil samples grouped according to lithologic classification, or parent lithology	Alluvium, fine to medium Alluvium, medium to coarse Sandstone Bentonitic shale Siliceous shale Shale Sandy shale/shaley sandstone

TABLE VII. Standard deviations of the soil physical properties for each of the analysis categories. Weighted averages of the standard deviations determined for each category for LL, PI and R are shown in parentheses.

ANALYSIS CATEGORY	Map Unit	P4	P10	P40	P200	LL	PI	R
Combined Samples		9.58	12.82	17.85	15.53	19.86	18.92	14.23
Broad Lithology	Sandstone	13.33	14.51	15.68	13.74	9.66	6.25	14.90
	Shale	5.81	10.80	18.01	16.09	24.97 (15.57)	24.10 (14.91)	11.54 (12.86)
	Alluvium	11.27	14.14	17.89	13.92	7.62	7.67	13.76
Formations from Geologic Maps	Sundance	8.53	13.07	17.65	19.24	12.93	9.80	20.98
	Morrison	4.52	6.15	9.07	10.72	4.28	5.59	13.15
	Cloverly	2.04	6.04	7.26	6.57	2.49	2.48	3.03
	Thermopolis	13.44	13.98	13.38	11.90	8.27	7.50	6.99
	Mowry	4.15	12.09	20.84	20.98	29.91 (11.29)	29.68 (11.28)	14.70 (12.13)
	Frontier	6.68	8.26	10.79	10.24	6.18	6.15	8.80
	Cody	1.52	3.53	5.01	12.38	4.18	5.01	6.69
	Terrace Gravel	15.78	18.04	21.10	15.09	6.02	5.96	15.02
	Alluvium	5.43	10.69	13.74	10.50	2.23	4.05	10.32
Formations Identified From Field and Photo Observation	Sundance	2.59	3.53	5.12	9.95	15.53	8.77	17.76
	Morrison	1.85	3.71	6.72	10.06	5.64	5.62	7.54
	Mowry	4.20	12.33	21.20	21.45	30.81	30.48	13.77
	Frontier	6.76	8.24	10.86	9.95	6.10	6.32	9.53
	Terrace #4	7.83	11.19	15.92	13.01	5.13 (12.11)	5.15 (11.96)	9.46 (11.51)
	Terrace #1	8.37	13.02	17.66	23.35	9.93	10.67	16.57
	Floodplain	2.47	3.33	3.55	6.76	1.97	4.05	4.57
	Streambed	7.36	14.78	17.54	7.98	2.68	3.69	10.17
	Colluvium	8.52	10.30	13.45	11.11	9.57	9.40	13.11
Photo Map Units	#5	1.45	3.45	4.86	12.28	3.96	4.77	5.47
	#7	2.53	4.23	6.16	7.33	2.61	4.23	7.57
	#10 *	0.02	0.02	0.99	6.29	3.99	3.00	2.49
	#13 *	1.99	2.99	3.99	1.69	5.99	4.99	0
	#20	14.91	17.47	21.01	17.96	9.05	8.90	16.92
	#20 - T1 *	8.37	13.02	17.66	23.35	9.93	10.67	16.57
	#20 - T4	7.93	11.54	6.69	15.29	8.72	8.16	10.18
	#21	7.48	9.38	12.77	17.98	41.04	39.39	3.52
	#30 *	2.06	3.93	6.04	11.21	6.25 (11.23)	6.18 (11.11)	7.69 (9.92)
	#32	9.38	10.71	13.74	12.98	10.46	10.24	14.67
	#33	5.23	13.93	19.34	14.18	19.18	13.44	16.27
	#39	7.08	7.97	9.59	8.05	5.56	5.03	4.74
	#40	1.69	1.82	2.40	6.40	3.86	4.46	7.15
	#41 *	6.01	7.69	8.88	6.17	3.13	3.84	12.44
	#49 *	2.32	3.55	4.64	6.10	3.09	5.24	3.77
	Rest of photo map units only one sample per unit.							
Lithologic Classification	Alluvium F-M	15.38	9.65	13.65	10.05	4.59	4.90	10.69
	Alluvium M-6	15.77	17.08	19.20	16.29	7.08	7.69	16.18
	Sandstone	13.58	14.88	16.18	12.58	9.86	6.71	16.48
	Bentonitic Sh.	6.90	1.55	15.97	17.90	32.32 (9.1)	31.17 (8.39)	1.96 (7.67)
	Siliceous Sh.	4.53	14.56	24.47	14.77	4.56	4.09	8.22
	Shale	2.00	3.23	8.06	8.30	9.34	7.85	3.69
	Sandy Shale	2.15	3.70	7.11	9.26	6.93	6.64	6.89

*Less than 5 samples per unit.

TABLE VIII. Coefficients of variation of the soil physical properties for each of the analysis categories. Weighted averages of the coefficients of variation determined for each category of LL, PI and R are shown in parentheses.

ANALYSIS CATEGORY	Map Unit	P4	P10	P40	P200	LL	PI	R
Combined Samples		10.16	14.43	21.97	28.07	51.84	88.95	62.52
Broad Lithology	Sandstone	14.23	15.96	17.96	26.54	40.45	76.31	41.87
	Shale	6.04	12.08	22.31	26.83	51.82 (37.97)	79.44 (65.96)	68.52 (58.68)
	Alluvium	12.18	16.21	22.27	26.82	24.18	51.07	52.31
Formations from Geologic Maps	Sundance	9.16	15.09	22.56	36.26	47.89	88.12	56.15
	Morrison	4.66	6.58	10.32	19.26	15.52	46.92	47.65
	Cloverly	2.11	6.71	8.69	10.57	7.69	13.62	17.67
	Thermopolis	15.27	16.94	17.90	20.58	20.43	31.44	33.71
	Mowry	4.27	14.06	28.89	36.71	48.61 (25.10)	70.49 (48.07)	87.29 (55.50)
	Frontier	7.02	8.85	12.00	17.11	16.78	30.75	51.02
	Cody	1.54	3.64	5.28	22.55	16.22	56.76	23.57
	Terrace Gravel	17.97	21.92	28.27	32.07	19.13	40.19	60.34
	Alluvium	5.66	11.62	15.64	17.92	8.00	35.46	32.61
Formations Identified From Field and Photo Observation	Sundance	2.63	3.76	5.85	15.69	58.61	67.49	66.42
	Morrison	1.88	3.97	7.81	15.33	19.86	40.15	42.39
	Mowry	4.31	14.27	28.89	36.90	49.33	70.54	90.11
	Frontier	7.07	8.78	11.93	16.13	16.06	30.40	55.74
	Terrace #4	8.70	13.29	20.95	26.48	16.11 (28.00)	32.54 (50.46)	42.53 (55.72)
	Terrace #1	9.89	17.67	28.95	51.56	33.12	88.97	40.11
	Floodplain	2.54	3.51	3.86	11.25	7.12	37.46	14.51
	Streambed	7.81	17.39	23.00	16.84	9.32	27.92	29.92
	Colluvium	8.99	11.34	15.81	30.45	29.43	59.38	50.77
Photo Map Units	#5	1.46	3.54	5.09	21.66	15.24	51.39	23.22
	#7	2.64	4.56	6.95	11.96	9.18	33.70	25.74
	#10 *	0.02	0.02	1.00	8.26	9.52	12.00	38.46
	#13 *	2.04	3.09	4.21	2.73	16.66	24.99	0.03
	#20	17.19	21.74	29.17	36.91	27.38	54.91	63.29
	#20 - T1 *	8.81	13.70	21.83	30.17	25.45	45.20	47.95
	#20 - T4 *	9.89	17.67	28.95	51.56	33.11 (26.01)	88.97 (48.68)	40.11 (45.40)
	#21	7.91	10.10	14.10	23.86	53.53	69.39	48.74
	#30 *	2.09	4.23	7.19	17.19	21.77	42.65	47.34
	#32	9.93	11.99	16.79	20.67	29.74	56.63	54.78
	#33	5.36	17.50	32.96	32.75	34.87	53.21	75.11
	#39	7.56	8.79	11.15	12.29	13.39	20.13	31.25
	#40	1.73	1.88	2.51	11.19	10.87	27.04	30.46
	#41 *	6.34	8.36	10.23	11.80	10.25	25.64	51.84
	#49 *	2.88	3.82	5.41	8.94	8.74	30.28	22.62
	Rest of photo map units only one sample per unit.							
Lithologic Classification	Alluvium F-M	5.63	10.58	15.84	18.66	15.73	38.64	37.37
	Alluvium M-6	19.16	22.94	29.95	36.72	23.02	54.72	47.35
	Sandstone	14.33	16.13	18.20	25.12	40.47	80.25	45.00
	Bentonitic Sh.	7.22	12.79	19.00	26.82	50.25 (27.88)	67.71 (49.57)	30.08 (35.07)
	Siliceous Sh.	4.61	18.74	45.19	39.30	11.36	20.64	23.21
	Shale	2.06	3.51	9.63	13.73	23.96	34.69	23.87
	Sandy Shale	2.21	3.90	7.88	14.47	19.54	35.45	35.91

*Less than 5 samples per unit.

Examination of the figures in Tables VII and VIII show a refinement and an overall decrease in the standard deviations and coefficients of variation going from the combined sample category to the lithologic classification. The decrease is particularly apparent when the weighted averages are examined as shown in parenthesis in the tables. For the lithologic classification the weighted average standard diviations and weighted average coefficients of variation are very close to the values of 6 and 30 for the liquid limit, plasticity index and "R" value.

Although satisfying the requirements for variation of the material with the map units, it was not readily apparent at this point whether the map units in the analysis categories were actually independent and valid units, although logically and intuitively they were independent. Various statistical tests are available to test differences. In this case it is desired to test the hypothesis that each map unit represents a population different from all other map units within the particular mapping classification system (analysis category).

In particular, the Mann-Whitney "U" test (Siegel, 1965) is well suited for this purpose. The calculations are relatively simple. But because of the number of calculations required. the computer was again used to perform the calculations.

Each map unit was compared with each other map unit in turn. The results are shown in Table IX. It can be seen that most of the map units were found to be significantly different from one another in at least 3 out of 7 physical properties.

Therefore, it was concluded that the litholoigc mapping category and its map units were valid quantitative map units for the study area in that they were all different and did not have excessive variation in terms of the engineering properties tested, specifically grain size, liquid limit, plasticity index and "R" value.

PREPARATION OF HIGHWAY SOIL PROFILE MAPS

With the quantitative map units selected it was then possible to proceed with the preparation of the final engineering geologic map for the highway project. The plan and profile type format was chosen with a drawing scale of 400 feet per inch. This enabled the maps to be prepared directly from the photogrammetric manuscript which had been compiled earlier. The technique of superimposing the geologic map on a half-tone reproduction of the topographic map added greatly to the visual quality. Figure 4 is an example of the engineering geologic map plan and profile of the same area as covered by the photo map in Figure 3. Again the scale has been reduced for publication.

In addition to the quantitative map units the plan also shows the location and number of all test holes and the geologic structure through the use of strike and dip symbols and fault lines.

TABLE IX. Results of Mann-Whitney test for significant difference of means of "R" value and other soil properties for the lithologic classification. An X in a column indicates significant difference between the two units being compared at a level of significance of 0.05.

	P4	P10	P40	P200	LL	PI	R	Total Differences
Alluvium F-M/Alluvium M-C	X	X	X	X				4
Alluvium F-M/Sandstone						X		1
Alluvium F-M/Bentonitic Shale				X	X	X	X	4
Alluvium F-M/Siliceous Shale		X	X	X	X	X	X	6
Alluvium F-M/Shale				X	X	X	X	4
Alluvium F-M/Sandy Shale				X	X	X	X	4
Alluvium M-C/Sandstone	X	X	X					3
Alluvium M-C/Bentonitic Shale	X	X	X	X	X	X	X	7
Alluvium M-C/Siliceous Shale	X				X	X		3
Alluvium M-C/Shale	X	X	X	X	X	X	X	7
Alluvium M-C/Sandy Shale	X	X	X	X			X	5
Sandstone/Bentonitic Shale				X	X	X	X	4
Sandstone/Siliceous Shale		X	X		X	X		4
Sandstone/Shale		X	X	X	X	X	X	6
Sandstone/Sandy Shale				X	X	X	X	4
Bentonitic Shale/Siliceous Shale		X	X	X	X	X	X	6
Bentonitic Shale/Shale					X	X	X	3
Bentonitic Shale/Sandy Shale					X	X	X	3
Siliceous Shale/Shale		X	X	X			X	4
Siliceous Shale/Sandy Shale		X	X	X			X	4
Shale/Sandy Shale		X	X					2

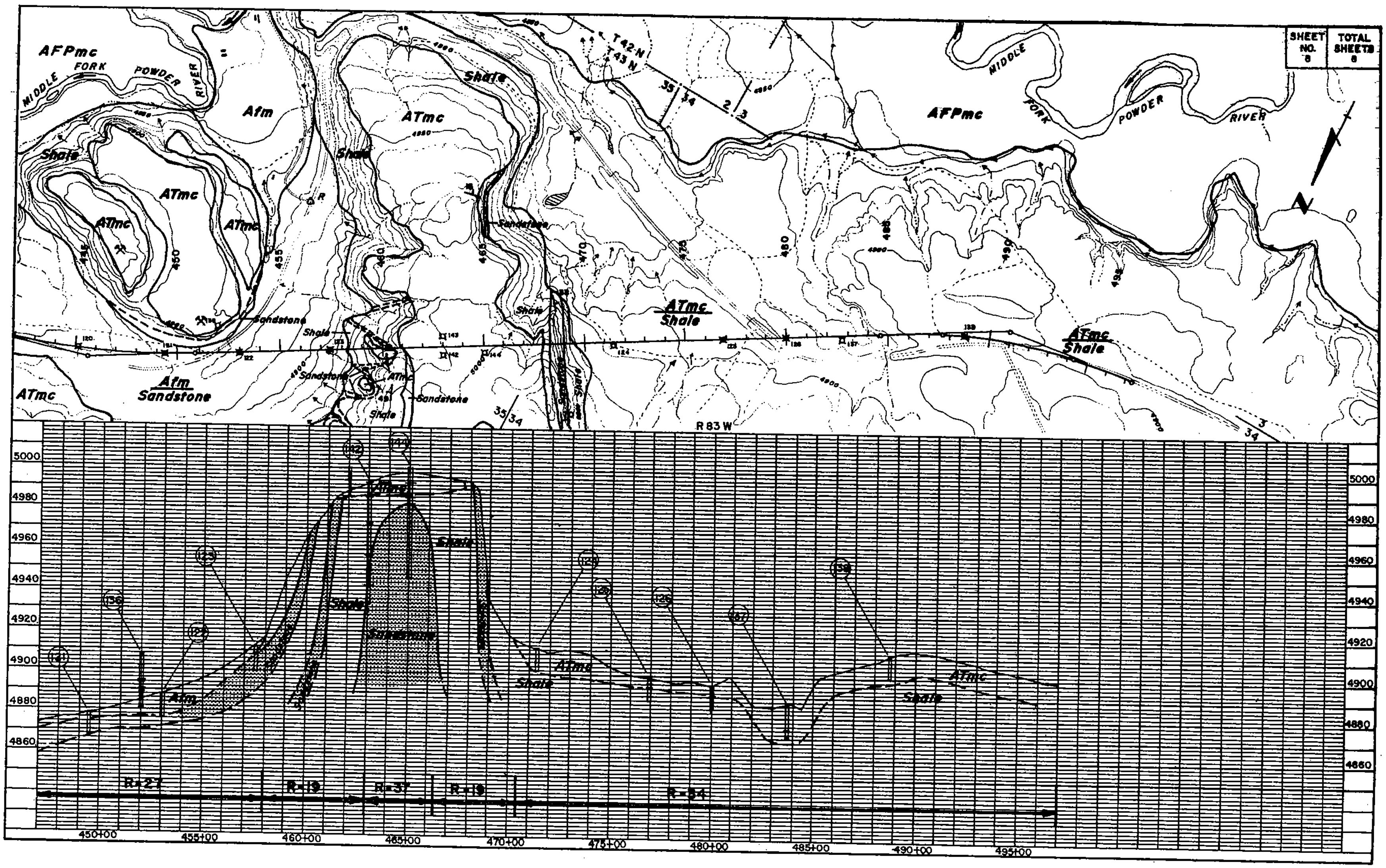


FIGURE 4. A typical example of the engineering geologic map and profile developed for highway engineering use. The original map scale was 400 feet per inch. From Edwards, 1972

The profile shows the ground line along the proposed highway centerline. The vertical scale is exagerated at a 10:1 ratio typical of engineering profiles. Through interpretation of the map and the drill logs an engineering geology-soil profile was prepared along the proposed highway centerline. Information on the geologic structure, the strike and dip of the beds, necessary to draw the profile conditions was obtained from spot elevation measurements made with the photogrammetric plotter on bedding surfaces seen on the aerial photographs. The strike and dip was determined from the spot elevations by the usual three-point method used by geologists.

Below the interpretive profile the "R" of the Subgrade is shown for the material at or just below the proposed highway grade line. The grade line is not included on the example shown in Figure 4.

Along with the engineering geologic map is included an explanation of symbols and material types (Figure 5). The most important part of this explanation is the description and engineering properties of the materials. Material types are listed according to the map units of the lithologic classification in Table VI, which was selected as the most valid quantitative system. Some modifications were made for the final mapping purposes. The map unit for sandy shale/shaley sandstone was dropped from the classification because it did not differ significantly from the quantitative properties of the material in the shale unit. The medium to coarse grained alluvial deposits were divided into two units, one for the coarse grained terrace deposits and one for the flood plain deposits. This was done because of the importance of the terrace deposits as sources of construction materials. Bentonite was added as a map unit because of its highly detrimental nature in highway subgrades and because numerous mappable layers from 1 to 4 feet thick were present throughout the study area.

Features of the table of the description and engineering properties of the materials include the designation of the materials include the designation of the materials as used on the plan and profile which is no more than a letter designation or abreviation of the map unit name. A brief description of the material follows. The physical properties of each unit are listed as determined from the statistical analysis of the data. The mean value of each property was determined from all of the samples falling within each map unit. The high mean and low mean shown represent the 95% confidence limits of the mean value. As such the high mean is taken to represent the better materials within the map unit and the low mean the poorer materials recalling that the initial criteria for the map units specified a 30% variation. From the mean values the AASHO and Unified Soil Classifications were determined and shown. Lastly, there is a provision for any additional remarks concerning the material which might be of importance.

FIGURE 5. Explanation of symbols and material types for the engineering geologic map and profile. From Edwards, 1972.

The symbols used on the maps and profiles and indicated in the explanation are relatively straight-forward and typical of geologic maps. The method for showing layered or mixed material types should be pointed out. This method follows that used by Kaye (1948) in his engineering geology map of the Homestead Quadrangle in Montana. Where material types are layered they are shown with the appropriate symbols one over the other Approximate thicknesses of the materials are shown in parenthesis. Mixed materials are shown with the appropriate disignations side by side separated with a slash.

CONCLUSIONS

As a result of the study and analysis it was concluded that the methods of geologic interpretation and mapping can be used successfully to present information with regard to the subsurface soil and rock conditions useful to the engineer in highway subgrade and surfacing design procedures. The engineering geologic soil units, when properly selected, have engineering properties which can be expressed quantitatively, rather than just qualitatively, in terms which can be used directly by the engineer. The units can be selected so that the average "R" value or any other property will have an acceptable standard deviation of approximately 6 and a coefficient of variation of approximately 30%, values which are not excessive for highway engineering purposes.

Air photo interpretation and mapping of a proposed highway route prior to field investigations can save considerably on the field costs of a highway design. Not only can the geologist plan his approach to the field work but he will go to the field with a good representation, if not a detailed geologic map, of the conditions in the area.

With prior knowledge of the engineering properities of recognizable and repetitive geologic units, the geologist using statistics can determine the number of test holes required, spacing of the test holes, and necessary type of samples to confirm the physical properties (see Kantey and Williams, 1962; Liu and Thornburn, 1965; and Deen, 1959). Statistical data and analysis of the map units for the study area indicate an average of 10 samples per map unit would have adequately defined the physical properties of each map unit. This would represent a reduction of 60% from the actual number of test holes drilled on this project.

It is reasonable to believe that a continuing program of air photo mapping and statisitical analysis of sample data can provide highway department staffs with a data bank of information on the soils within their region of operation. Computer processing, analysis and storage of data can lead to undetermined possibilities for future savings such automated route selection for the most economical highway route (Turner, 1963). However, quantitative engineering geologic map units are a first requirement.

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ACCELERATED POLISH TEST FOR COARSE AGGREGATE

by

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ABSTRACT

A laboratory test method recently implemented by the Texas Highway Department provides a rating system by which different coarse aggregates can be ranked according to an expressed frictional reading. The method, "Accelerated Polish Test for Coarse Aggregate," Test Method Tex 438-A, utilizes the British Accelerated Polishing machine to bring about the polishing action and the British Portable Tester to obtain frictional readings. The relative extent of polish reached by the aggregate as measured by the tester is expressed as the "polish value." Data from samples, representing both natural and synthetic aggregate sources from all parts of Texas, indicates that the test method provides a relative measure of an aggregate's susceptibility to polishing under traffic.

INTRODUCTION

For the past few years there has been an increasing concern over slippery pavements resulting from high-volume traffic. It has long been recognized that certain pavements tend to be less skid-resistant than others in the same length of time, and also that certain types of paving aggregates will polish more rapidly than others. In addition, it is generally accepted that the coarse aggregate exposed at the pavement's surface is the major determinant of the skid resistance provided by that surface. Although all of these features have been recognized, there has been no means by which different aggregates could be quantitatively measured in terms of their relative frictional characteristics. However, in an attempt to better understand the frictional properties of paving aggregates and hopefully be able to effectively differentiate the aggregate types which readily polish under traffic from those that do not, the Texas Highway Department has set out to evaluate a laboratory test by which aggregates can be rated as to their relative susceptibility for polishing. In doing so, insight as to why various aggregates polish differently under traffic may be better understood.

MATERIALS AND METHODS

Basically, the Accelerated Polish Test, mostly referred to as "The Polish Test," consists of three parts; (1) sample preparation, (2) polishing phase, and (3) frictional reading.

A. Sample Preparation. The test calls for a 30 pound sample representing production designated for highway use to be submitted for testing. Also, the material shall be clean, dry and graded so that only the -1/2" to #4 fraction will be tested. At least 7 specimens are cast in concave metal molds using a polyester for bonding. The aggregate particles are hand-placed as close as possible and Ottawa sand is used to fill the interstices (Figures 1 and 2).

B. Polishing. The rapid polishing is accomplished by a Wessex Accelerated Polishing Machine, designed by Road Research Laboratory of Great Britain (Figure 3). The test specimens are clamped around a specimen wheel which is brought to a speed of 320 rpm. As a loaded rubber tire bears against the specimen wheel, silicon carbide grit (#150) and water are continuously fed between the tire and specimens. After nine hours of polishing action, the specimens are removed and washed thoroughly to remove the grit.

C. Readings. After cleaning, the specimens are tested for surface frictional properties with the British Portable Tester (Figure 4) using a modified form of ASTM Designation: E 303. The British Portable Tester, (BPT), utilizes a swinging pendulum with a spring-loaded 1-1/4" wide rubber slider. Upon release of the pendulum, the slider "skids" across the curved test specimen pushing a pointer which is calibrated to indicate a quantity of energy loss brought about by the friction of the slider. The swinging pendulum is adjusted prior to each test to where the pointer reads "0" when the pendulum is allowed to swing free. Adjustments are also made with each test specimen to assure that the contact path for the rubber slider across the top of the specimen is three inches. The test specimens are flushed with water and a series of readings are taken. When consistent readings are obtained, this value is recorded. The average of these BPT numbers is expressed as the aggregate's "polish value."

RESULTS

After the completion of the feasibility and evaluation study of the test by the Research Section of the Highway Design Division and subsequent implementation by the Materials and Tests Division, polish-value data has been gathered on samples representing about 90 commercial and non-commercial sources throughout Texas and neighboring States. Several sources have been tested repeatedly over the past three years. For convenience, the sources, which include both natural and synthetic types, have been grouped into 5 major aggregate categories; namely, (A) Synthetics, (B) Sandstones, (C) Igneous, (D) Carbonates and (E) River Gravels (See Table I).

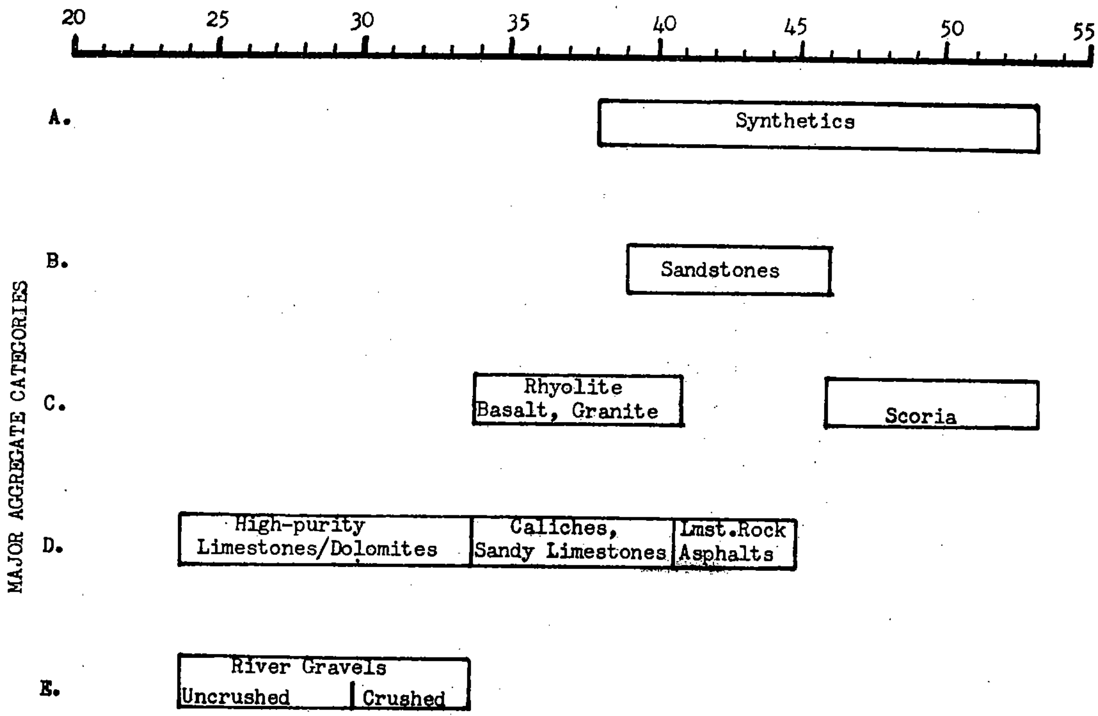
Within each of these major categories, the aggregate types can exhibit a wide range of polish values because of variations in microtexture. Extensive microscopic studies indicate that mineral content, hardness (Moh's Scale) and grain size have the greatest influence on textural characteristics and thus control the polishing behavior of aggregates (See Figures 5-10). With synthetic aggregates, the textural characteristics are controlled by the vesicular or bleb structure of the particles. To a lesser degree, the particle shape and orientation has some influence on the range of polish values for certain aggregate types.

DISCUSSION

As the table shows, scoria, a natural lightweight aggregate, and synthetic lightweight aggregate, exhibit the highest ranges of polish

T A B L E I

POLISH VALUE RANGES (BPT)



values. Scoria is a volcanic-igneous rock and was sampled from sites in Mexico and New Mexico. Slags from blast furnace operations are also synthetic, but exhibit polish values slightly lower than the lightweight types (expanded shale and clay).

In general, mineral hardness, grain size and cementing agents control the polish values of sandstones. The wide range of variability is found in the carbonate types. Limestones and dolomites occur with wide variations in composition, density and textural features. The relatively pure types polish readily, whereas, those with impurities (especially sand) have higher values. River gravels are typically naturally polished and will show low readings; however, crushing apparently has some effect on the siliceous types.

Duplicate testing, in addition to multiple samples from a single pit or quarry, indicate that the test is repeatable within a narrow range (1-3 points) as long as the rock type remains the same. Sources which have strata that vary in composition show wider ranges in polish values. In applying this test to paving aggregates, special provisions to specification requirements for the coarse aggregate in hot-mixed asphaltic concrete and aggregate for surface treatments require a polish value of not less than 35. Texas has adopted the value of 35 based on laboratory study and limited field observations. Field observations indicate that pavement skid resistance can be improved when material which exhibits a high frictional property is used along with proper construction practices. It should be noted that the polish test is not an indicator of wear.

There is some question as to just what the nine hours of accelerated polishing as specified in the polish test is equivalent to in terms of traffic applications. In the field it has been observed that under high volume traffic certain limestones and dolomites (high-purity types) tend to polish and become less skid resistant in less than one year, whereas, other aggregate types such as certain sandstones and synthetic materials have been capable of providing skid resistant surfaces for several years. It has been observed in the laboratory that the high-purity limestones and dolomites reach their maximum degree of polish in only 3-4 hours, whereas, other materials reach a stable polished condition at 7-9 hours. Most of the lightweight synthetics essentially never polish or loose any frictional property during the test (often they gain in frictional values). Without extensive field records it would be difficult to apply a numerical value of traffic applications that this accelerated polish test simulates.

Economics may restrict the use of certain materials to roadways with certain traffic volumes. It may not be economical to require a material with a high polish value to be used on a secondary road with very little traffic when a crushed stone that exhibits a susceptibility for polishing would never reach a hazardous slippery condition during its functional life under a low traffic volume. On the other hand, if an aggregate with a low polish value is used on a high volume expressway it may have to be replaced in just a few months due to its polished condition, much sooner than its expected functional life.

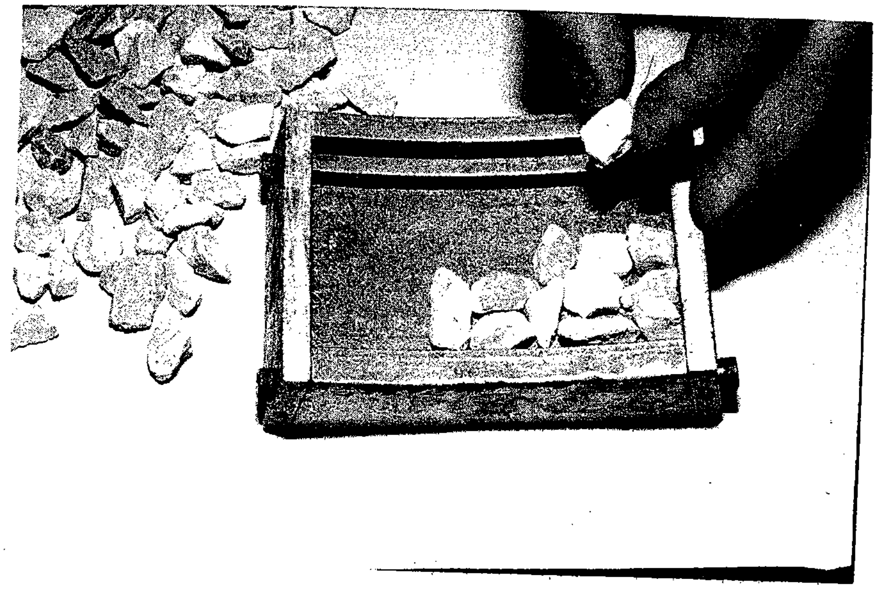


Fig. 1 Placement of aggregated in steel molds.

Fig. 2 Upper left: Disassembled steel mold.
Upper right: Aggregate placed and spaces filled with sand.
Lower left: Polyester poured and struck off.
Lower right: Cured specimens released from molds.

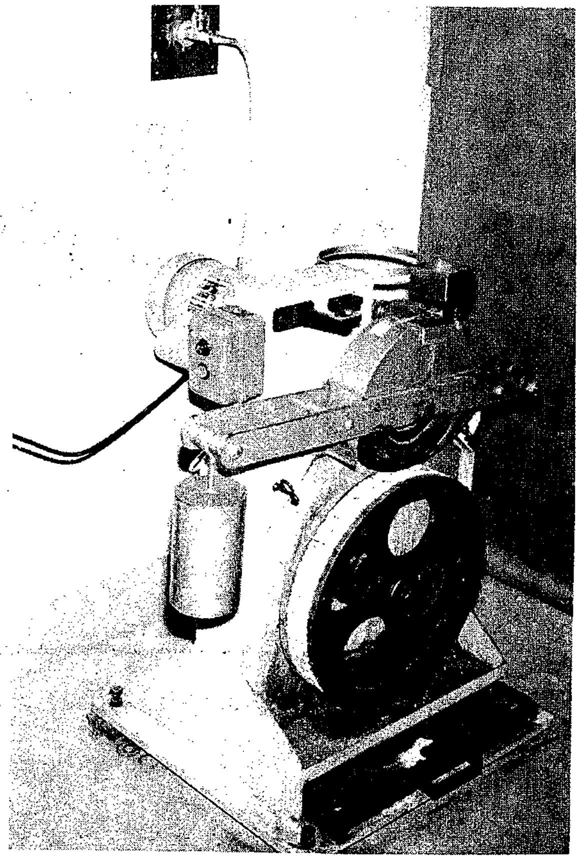


Fig. 3 British Accelerated Polishing Machine.

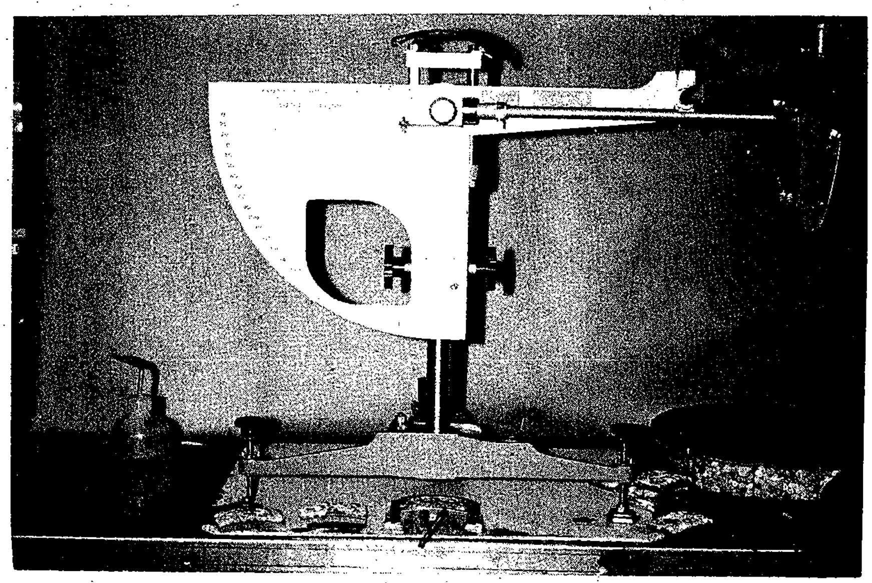


Fig. 4 British Portable Tester (BPT).

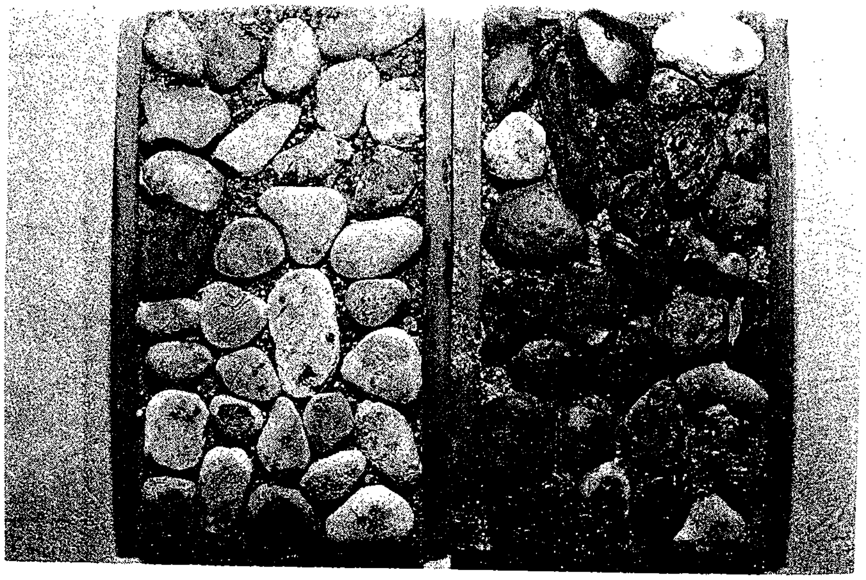


Fig. 5 Samples of well-rounded river gravel with polish values in the upper 20's. Limestone at left and siliceous gravel on right.

Fig. 6 Close-up of siliceous river gravel having no micro-texture. (Magnification 7X)

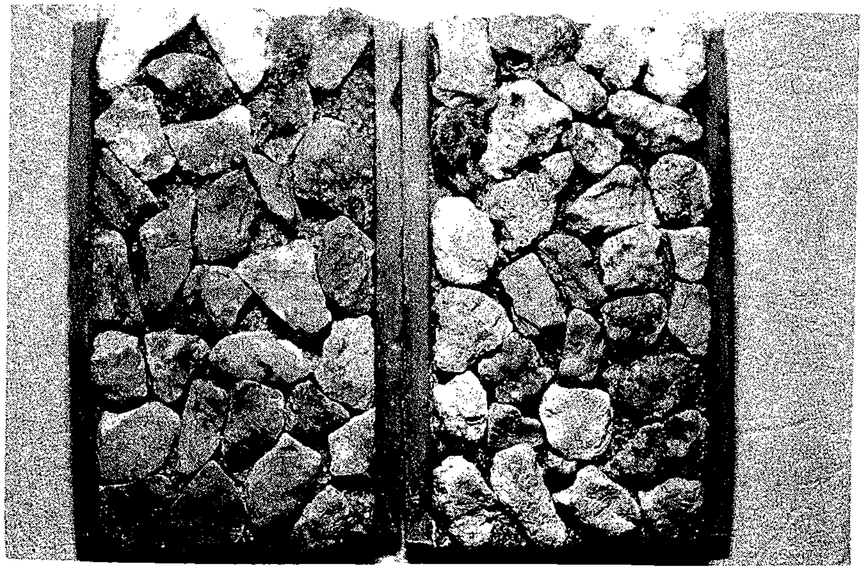


Fig. 7 Samples of crushed limestone which exhibit slight differences in microtexture as a result of mineral impurities. Sample on left measured a polish value of 28, sample on right measured a 33.

Fig. 8 Close-up of a limestone particle with no microtexture. (Magnification 7X)

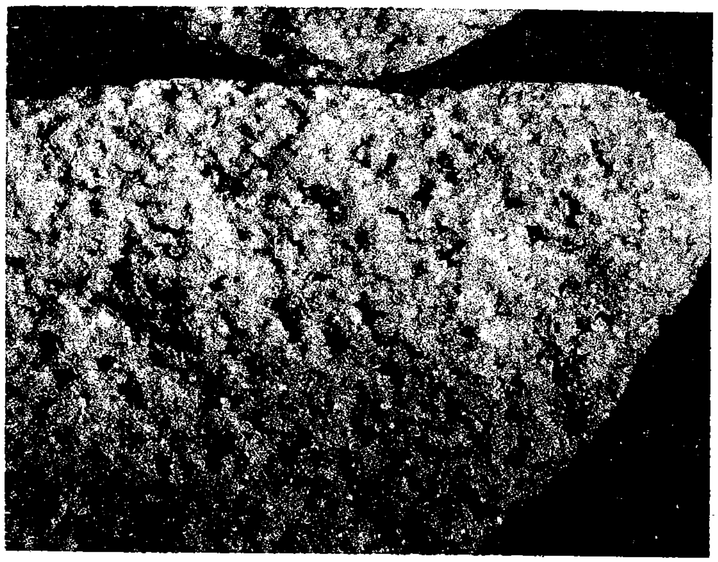


Fig. 9 Sandstone particle showing good microtexture as a result of mineral composition. The sand grains are quartz (Moh's Hardness = 7) and the cementing agent is calcite (H = 3). The polish value measured a 42. (Magnification 7X)

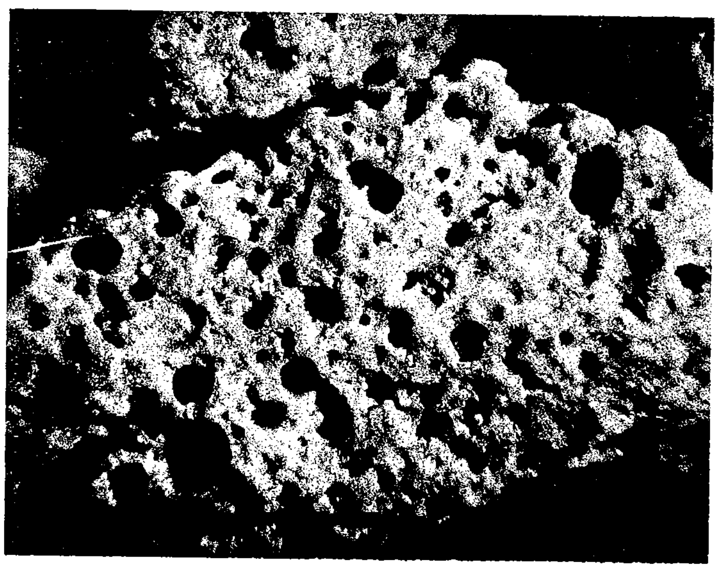


Fig. 10 Sample showing the vesicular nature many of the synthetic aggregates such as slag and expanded shale. Volcanic scoria has a similar texture. The polish value ranges in the 40's and 50's. (Mag. 7X)

REINFORCED EARTH FOR HIGHWAY APPLICATIONS

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Abstract

The technical and economic feasibility of Reinforced Earth for construction in highway retaining structures has been successfully demonstrated in over 150 applications in Europe, Canada and the United States. In this paper, the authors review some significant projects in the United States and present pertinent design and cost data. Also discussed in detail is the use of Reinforced Earth in remedial highway construction primarily slide stabilization and slab construction over cavernous limestone foundations. Several actual case histories are presented.

Editorial Note: The above listed paper was not delivered to the Symposium Committee for publication.

ENGINEERING GEOLOGY AND ROCK MECHANICS HELP TO DESIGN A HIGHWAY IN NORTHERN SPAIN

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ABSTRACT

A four-lane toll-highway is being built in Northern Spain between the city of Bilbao and the French border. It roughly follows the coastline of the Bay of Biscay, and has a total length of some 70 miles.

The area has a mountainous character with very unfavorable geologic and climatic conditions. Most of the area is underlain by thinly-bedded, intensively-folded, sedimentary rocks of Cretaceous and Tertiary age.

The designers of the road have minimized the construction costs by replacing bridges and tunnels with high embankments and deep cuts.

A team of experienced engineering geologists and rock and soil engineers assisted the designer during the feasibility studies, detailed design, and construction, where the most up-to-date field and analytical geotechnical methods were applied. The per-mile construction costs are lower than those on other European highways in comparable conditions.

INTRODUCTION

A relatively short four-lane toll-highway is being built in Northern Spain. The design and construction of this highway is certainly interesting from the engineering/geological point of view for several reasons. First, it is being built in extremely unfavorable topographic, geologic, and climatic conditions; and second, the owner wanted to build it at the lowest cost possible. The toll-highway is private; financed partly by Spanish and partly by foreign funds. It will be turned over to the Spanish government after 35 years.

To keep the total costs low, the designer reduced the total length of structures such as bridges and tunnels to an unusual minimum of less than 10 percent of the total highway length, compared with more common 30 to 40 percent on other European highways in similar conditions. Bridges were replaced by high embankments and tunnels by deep cuts.

The team of Spanish designers realized design and construction problems of a highway in such unfavorable conditions. From the very beginning of individual design phases, a team of experienced engineering geologists and rock and soil engineers assisted in the design of the alignment, cut-and-fill slopes, foundation of structures, location of borrow materials, etc. The team was composed of the designer's own geotechnical staff and Dames & Moore's professionals.

The highway goes from the industrial city of Bilbao, along the Atlantic coast to San Sebastian, and farther to the French border (Figure 1). Its total length is approximately 68 miles. The total construction costs are on the order of 252-million dollars at present exchange rates, which means approximately 3.7-million dollars per mile. This is considerably less compared with highways in comparable conditions in Austria, Switzerland, Italy, or France.

TOPOGRAPHY, GEOLOGY, AND CLIMATE

The area has a mountainous character, with great elevation differences and deep valleys with slopes up to 45 degrees. Most of the region is underlain by thinly-bedded, sedimentary rocks of Cretaceous and Tertiary age. Limestones, marls, sandstones, and shales are the most common rock types, and the bedrock has a character of flysch. Short sections of the highway are in massive limestones and volcanic rocks such as basalt and tuffs. All the area was intensively folded during the late Tertiary period, so that any orientation of bedding could be found along the highway. The bedrock is often weathered to depths exceeding 120 feet.

Numerous slides of various extent have developed in the area during the Pleistocene and recent periods, and often the highway had to be designed across these slides.

The climatic conditions of this portion of Spain are also unfavorable, with an annual rainfall over 50 inches. The rains are concentrated in the winter period of approximately 5 months.

HIGHWAY CONCEPTS

Since the earth movements comprise a considerable portion of the total highway costs, cross sections for both the cuts and embankments were carefully designed. Figure 2 illustrates one difficult highway section 19.2 miles long. The total length and percentage of cuts, embankments, and structures is evaluated for this section. It can be seen that more than 50 percent of this section is constructed in cuts. The cuts are divided according to their depth into three categories. Approximately 6 miles of cuts 65 to 130 feet deep, and 1.6 mile of cuts 130 to 250 feet deep, had to be designed and constructed within the section. The embankments on this section, 130 to 250 feet high, reached the total length of 0.7 mile. It can be seen on this figure that the total length of bridges and tunnels within this section is only 1.9 miles, which comprises less than 10 percent of the total section length.

It should be mentioned here that the construction of a highway which tries to eliminate expensive structures has one serious disadvantage. From the environmental point of view, the highway with deep cuts and high embankments significantly changes the character of the country. Highway designers in central European countries, such as Switzerland or France, usually prefer more expensive solutions with tunnels and bridges which are more acceptable from the environmental point of view. Numerous highways within the United States, however, are being built using similar design concepts such as on this Spanish highway. Interstate 70, recently constructed through the Rocky Mountains, is a typical example of such an approach.

Figure 3 shows typical cross sections of high embankments constructed in rough topographic conditions. Most high embankments were built with good-quality limestone rockfill.

Figure 4 is a series of basic design schemes for highway cuts in different geotechnical conditions. The cut slopes were designed individually for each cut. Since the soil cover in the area has a negligible thickness, nearly all cuts were excavated either in sound or weathered bedrock. Four typical highway cross sections can be seen on this figure. Figure A comprises a cut designed in structure-free bedrock such as basalt or massive limestone. All cuts were designed without intermediate berms, and a wide and deep berm was excavated always at the slope toe as a space to catch smaller rock falls. Wire mesh and rock bolts were used occasionally in rocks of worse quality. Figures B and C show the general approach to cut design in stratified rocks with different orientation of bedding. Cuts in these geological conditions have caused most troubles during the construction. Figure D is a case of a cut slope which undercuts a wedge of rock, limited by a bedding plane and a joint, or by a combination of two joints. These cases were frequent in the area of folded rocks; and wherever possible, the cut slopes were flattened along the intersection dip. In numerous cases, however, flattening was not feasible and deep anchors were designed as retaining forces.

ENGINEERING-GEOLOGICAL INVESTIGATION

A scheme of the geotechnical assistance during the individual phases of the design and construction, methods of study, and staffing are shown on Figure 5. It can be divided into three phases.

First, the investigation phase was basically a feasibility study which dealt with various route alternatives. It was based mainly on aerial photograph studies and engineering-geological mapping on a scale of 1:5,000. The second phase was a detailed investigation based on additional mapping on a detailed scale of 1:1,000; core drilling; collection of rock structure data, such as bedding and jointing; and finally, analyses of these data and design of cut-and-fill slopes.

Since the orientation of rock dividing planes, such as bedding and joint planes, is of utmost importance for cut stability analyses, data on them were collected for each individual cut and plotted in the form of polar diagrams. Figure 6 shows this method of representing the rock structure. Any plane in the space is plotted as a pole at the equal-area or Lambert's stereonet.

Rock structure data were used to design cut slopes in stratified rocks. We have been using only one analytical method, usually referred to as the wedge-and-block analysis according to Klaus W. John, who introducted it in 1969. An example of a wedge stability analysis using his graphical method is shown on Figure 7. Although there are few more analytical methods for the wedge stability available at present, John's method, which is fairly simple and does not require the use of a computer, is quite sufficient for most problems of the cut slope design.

CONSTRUCTION PROBLEMS

In spite of a very detailed second-phase geotechnical investigation, numerous problems occurred during the construction. They were solved during our third phase of study, which was basically technical assistance to the contractors. Cut slope failures were most important and interesting from the geotechnical point of view. Two examples of quite unusual rock slope failures are presented on Figures 8 and 9.

The first example (Figure 8) is a toppling-type failure of a 150-foot-deep cut, excavated in weathered flysch-type rock. The strike of the bedding is parallel with the slope, and the dip is into the slope. Except for bedding, joints normal to bedding could be observed in the rock mass. They are, however, never continuous, being limited only to one or two layers. The cut was designed at a slope of 45 degrees (1:1), and it failed immediately after the excavation. A series of test pits along the failure axis revealed that the failure had a character of toppling and the cut slope was flattened to an angle normal to bedding.

It should be mentioned that there are no reliable methods available to analyze this type of failure, and that the flattening of the slope was recommended basically according to an engineering judgement. With regard to the uncertainty of the solution and with regard to the water reservoir located close above the cut crest, three inclinometers were installed along the slope to monitor the slope performance. After one year of monitoring, no deformations were observed on the slope.

Toppling-type failures of the same type occurred at several other locations along the highway. Most of them happened in weathered rock and without presence of groundwater. To avoid failures of this type, cut slopes were flattened to angles normal to bedding, wherever possible.

The second example (Figure 9) is the design of a cut slope in the right flank of a large syncline structure. The bedrock is formed by sound shales where only bedding planes comprise potential failure planes. A cut slope of 2 (V) : 3 (H) was carefully designed using non-circular Spencer's stability analysis. The shearing resistance along bedding was estimated on the direct shear testing apparatus of the Imperial College in London.

During the cut construction, small failures started to develop along bedding planes. Their volume increased with proceeding excavation; and finally, the construction had to be interrupted and the cut slope re-designed.

Data on the shearing resistance along the bedding were re-evaluated using the back-calculation of a number of recent rock slides. The back-calculation showed that the actual shearing resistance along bedding is only 14 degrees (assuming zero cohesion) and 6 to 8 degrees lower than the laboratory testing results.

The safety factor of the originally-designed slope was lower than one, and the cut was re-designed as indicated on Figure 9. A sufficiently thick loading berm was left at the lower third of the cut slope. The middle portion of the cut was excavated along bedding.

FIG. I HIGHWAY VICINITY MAP

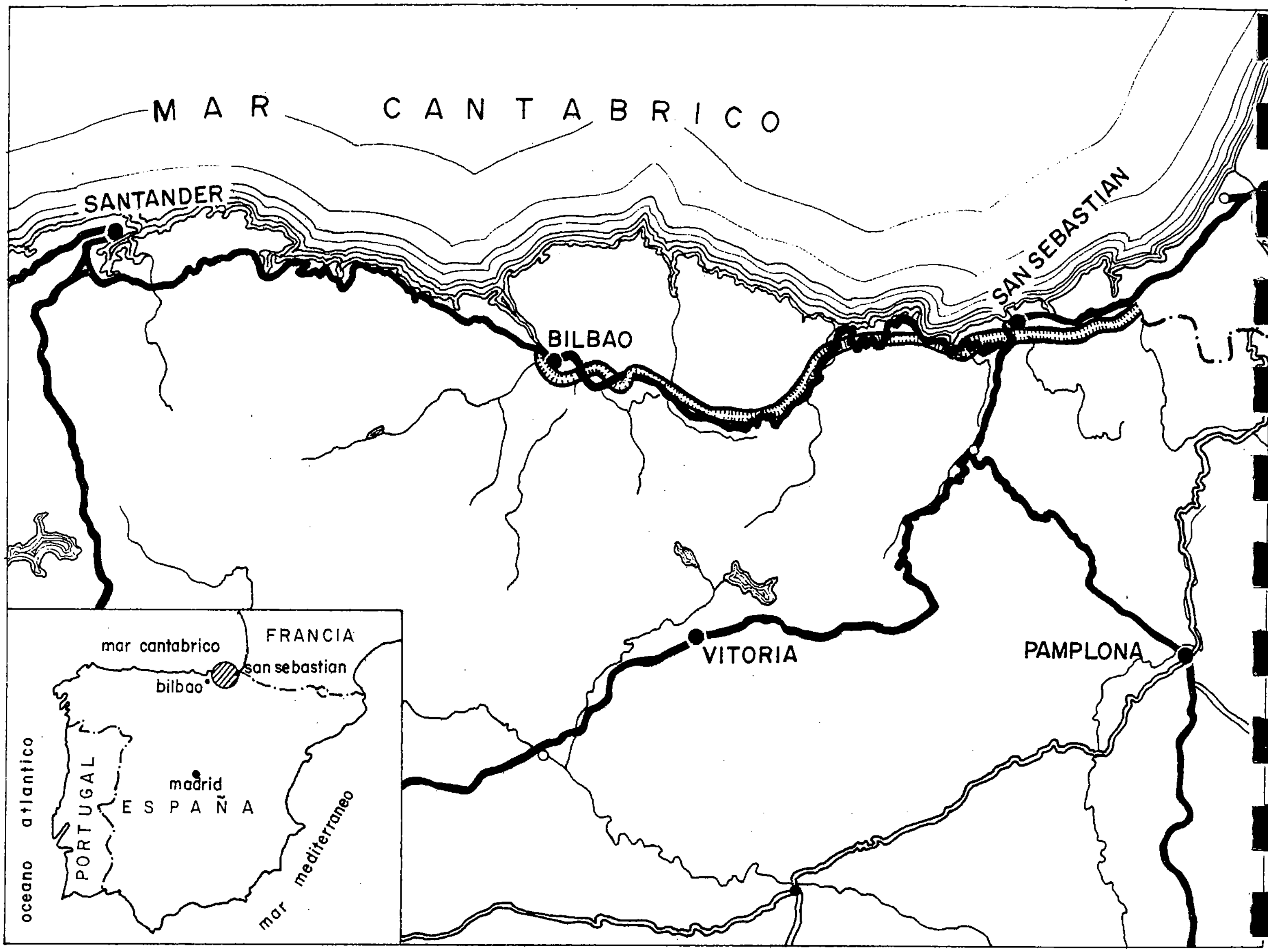


FIG. 2

CHARACTER OF A HIGHWAY SECTION (TOTAL LENGTH 19.2 MILES)

Cuts			Embankments			Structures (Tunnels & Bridges)
Number of Cuts	Height (ft)	Length (miles)	Number of Embankments	Height (ft)	Length (miles)	Length (miles)
26	0 H < 65	3.4	39	0 < H < 65	4.0	1.9
32	65 < H < 130	6.0	11	65 < H < 130	1.6	
6	130 < H < 250	1.6	5	130 < H < 250	0.7	
Length of cuts 11.0 miles. 57.3% of total length.			Length of embankments 6.3 miles. 32.8% of total length.			Length of structures 1.9 miles. 9.9% of total length.

FIG. 3

HIGHWAY EMBANKMENTS

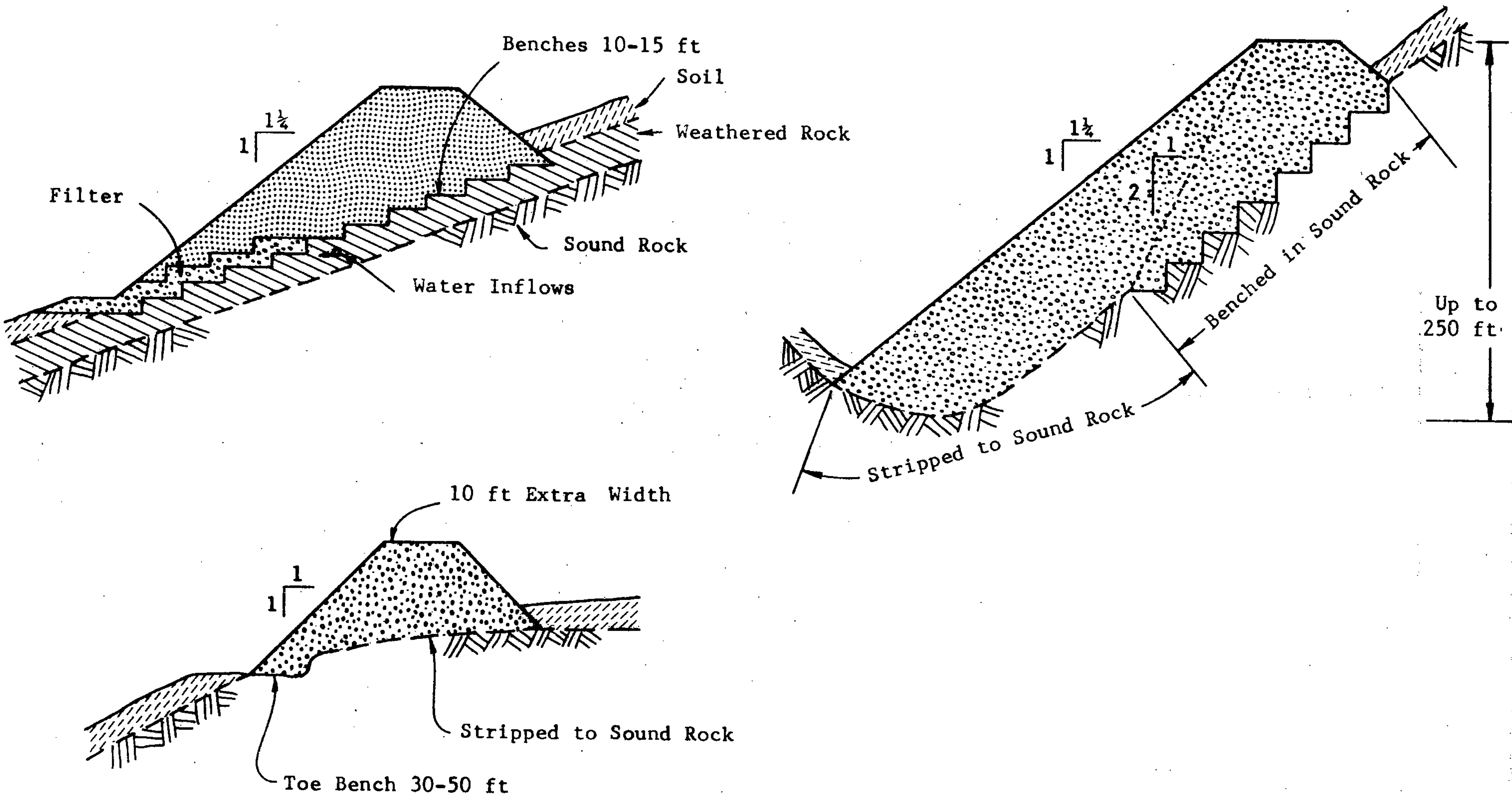


FIG. 4 HIGHWAY CUTS

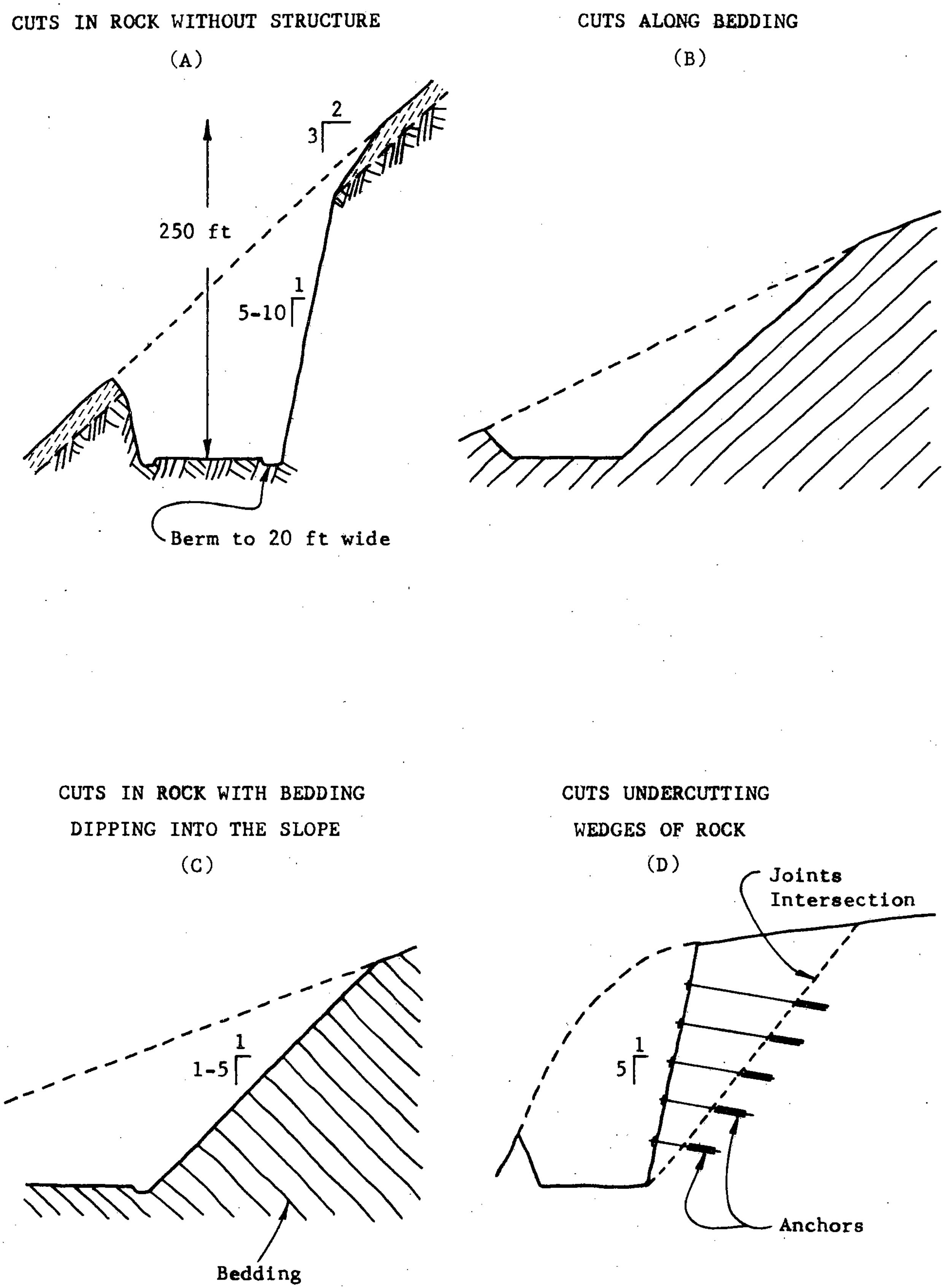
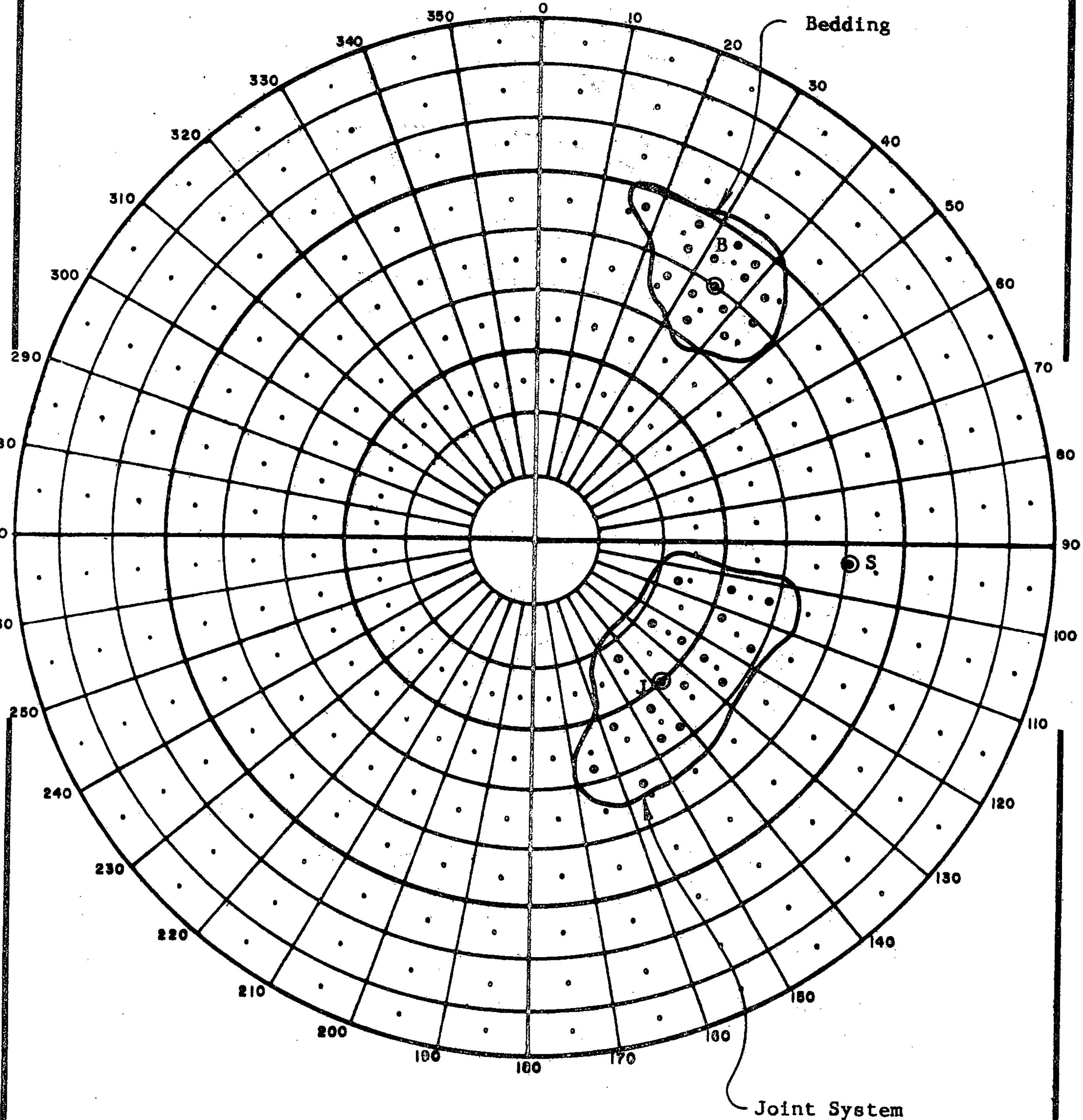


FIG. 5 GEOTECHNICAL STUDIES

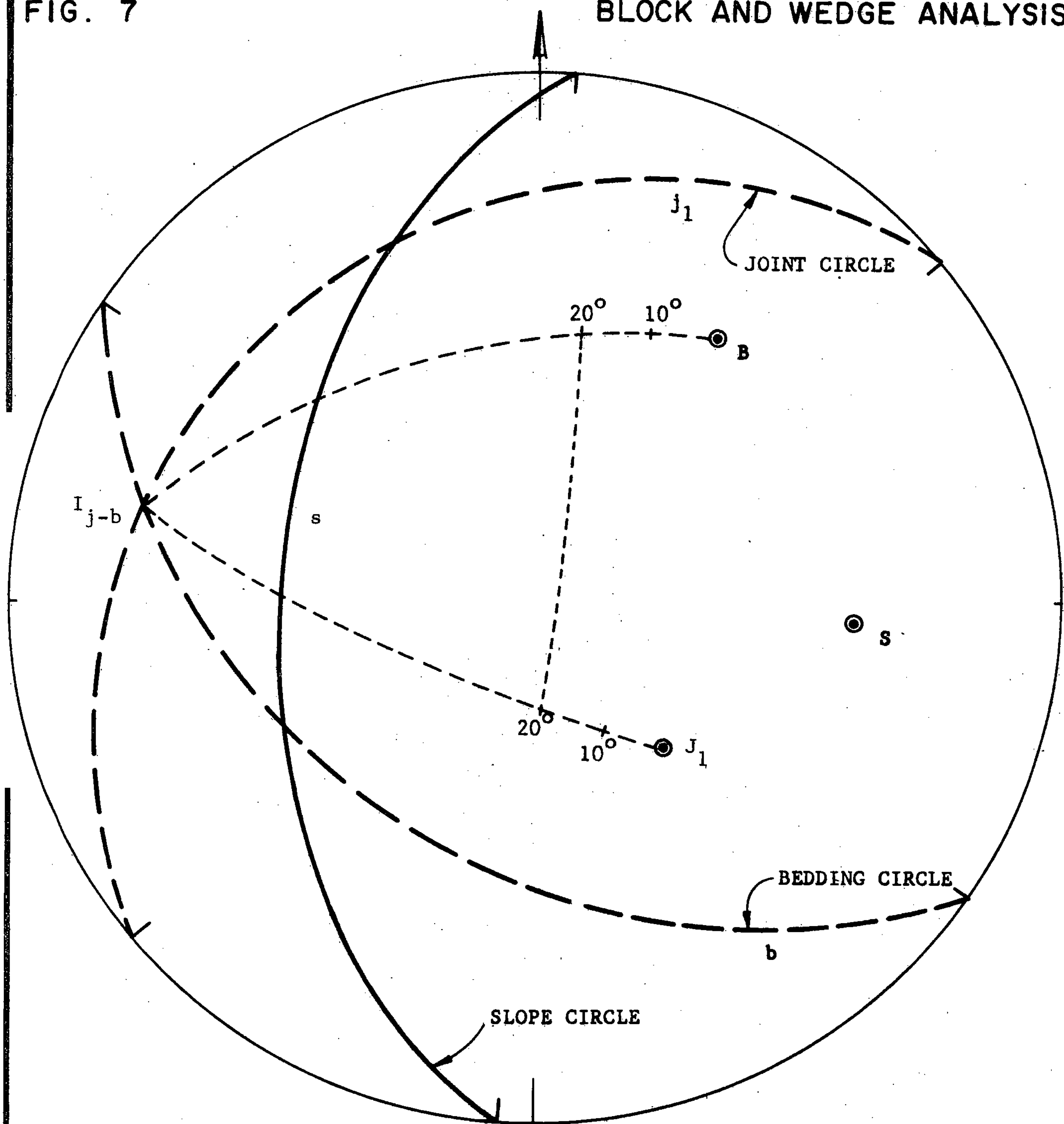
Phase	Purpose	Methods	Staffing
Feasibility Study	Choice of alignment	Aerial photograph study; engineering geological mapping, scale 1:5,000	1-2 engineering geologists
Detailed Study	Detailed design of alignment. Design of cuts and fills. Tunnels and bridges. Study material resources	Engineering-geological mapping, scale 1:1,000. Core drilling. Rock structure data collection, analyses	2 engineering geologists, 2 soils engineers, 2 rock engineers
Assistance for Construction	Changes of design. Solution of construction problems.	Geotechnical study of critical areas. Analyses	2 soils engineers 3 rock engineers

FIG. 6 ROCK STRUCTURE DATA



ROCK STRUCTURE SHOWING BEDDING, ONE JOINT SYSTEM, AND ORIENTATION OF CUT SLOPE

FIG. 7 BLOCK AND WEDGE ANALYSIS



ASSUMING FRICTION ALONG BEDDING AND JOINT
$\phi = 20^{\circ}$, WEDGE I_{j-b} IS NOT STABLE

FIG. 8

TOPPLING TYPE FAILURE

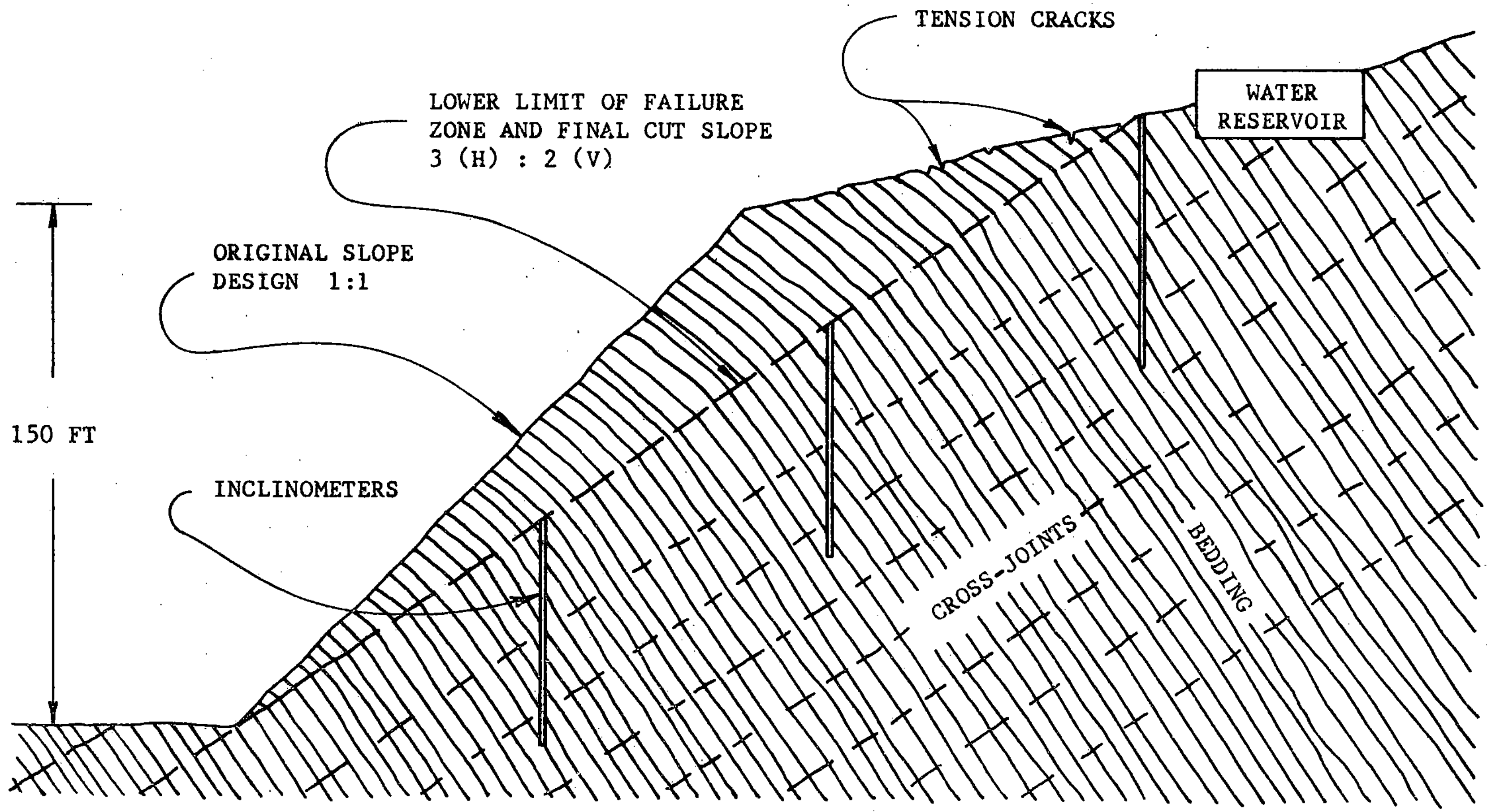
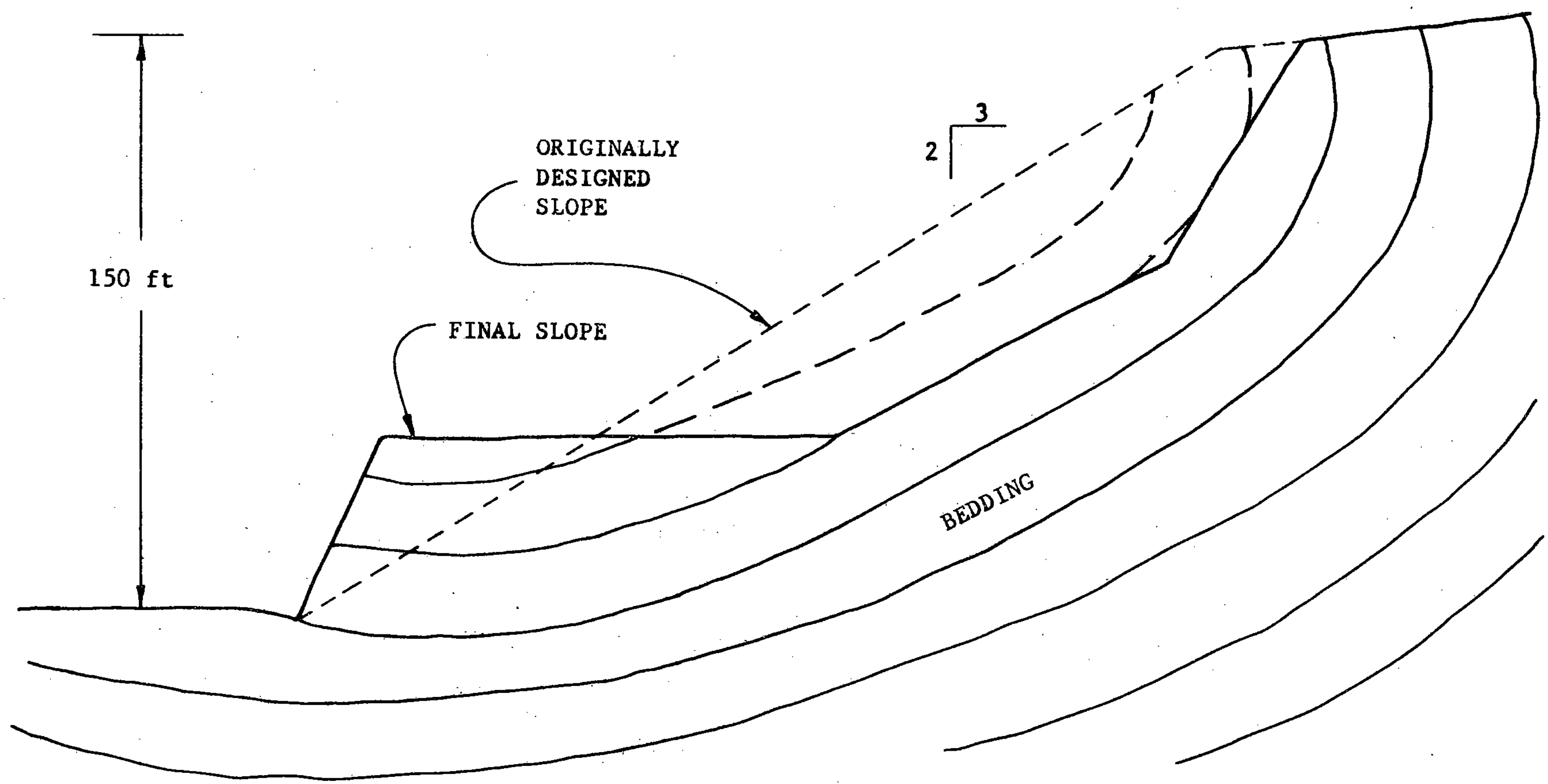


FIG. 9

DESIGN OF A CUT
IN A SYNCLINE



LANDSLIDE PROBLEMS ON THE WEST APPROACH TO EISENHOWER (STRAIGHT CREEK) TUNNEL, COLORADO

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Abstract.--Several large landslides occurred during construction of Interstate Highway 70 between the Blue River and the west portal of the Eisenhower Tunnel. Results of extensive geologic investigations showed that different conditions were associated with the various slides and as a result, different corrective measures were employed in individual cases. These measures included removal, drainage, slope flattening, roadway realignment, and buttresses composed of both rock and soil.

INTRODUCTION

The Eisenhower Tunnel carries Interstate 70 under the Continental Divide approximately 50 miles west of Denver. The west approach to the tunnel follows the north wall of the valley of Straight Creek from the Blue River valley to the west portal, and climbs from an elevation of 8700 feet to just over 11,000 feet along its eight mile extent. Construction of the road was started in the spring of 1963, utilizing cut slopes of 1:1, fill slopes of 1-1/2:1 and grades averaging 6 percent. In June of 1964, distress was noted in the backslope at several locations, and within a few weeks six large failures had developed along a two and one half mile interval near the 10,000 foot level. These failures were subsequently designated Slides A, B, 1, 2, 3 and 4, from east to west. Corrective work was accomplished between 1968 and 1972 and the road was opened to traffic early in 1973.

GEOLOGY

Bedrock in the area consists of schist and gneiss containing local intrusions of granite. The rock has been heavily faulted and fractured by tectonic activity, and extensive alteration of the bedrock to clay minerals is common along the faults. As a result, the bedrock consists of masses of competent rock surrounded by zones of weaker material. Pleistocene glaciation oversteepened the lower portions of the valley walls and as the ice melted, landslides developed in the areas of less competent rock. As the landslides progressed, surface drainage was disrupted in the affected areas and local swamps and internal drainage developed. Five of the six failures which later occurred during roadway construction were located in cuts constructed through these landslides.

Material in the slides consisted of rocky, micaceous colluvium containing occasional large masses of highly weathered, incompetent bedrock. Most laboratory samples were classified as A-1-a or A-1-b.

INVESTIGATIONS

Investigation of the slides was accomplished in two separate phases. The first was conducted by a private consulting firm during the summer of 1967 and included resistivity surveys, drilling, Menard Pressure Meter tests, water level observations, and classification, triaxial compression and compaction tests. Slide movements were monitored by means of observation points established on the surface of the slides. All six slides were investigated.

The second phase was conducted by the Department during the summer of 1971 and included seismic surveys, additional drilling, downhole installations of shear strips and pneumatic piezometers, and additional triaxial tests. These investigations were confined to Slides 1 and 2, with a minimal amount of additional work on Slide A.

SLIDE A

Slide A measured 1300 feet across the base and 1500 feet from toe to crown, with 600 feet of vertical relief from top to bottom.

The only one of the fix failures <u>not</u> resulting from distrubance of a pre-existing landslide, Slide A contained very little water and was triggered by undercutting of a relatively thin (0-25 feet) colluvial mantle resting on foliation surfaces which dipped toward the roadway.

The corrective measure used consisted of flattening the slope to 1-1/2:1. During the slope flattening, competent bedrock was encountered on the west side of the slide and the contractor extracted several thousand yards of rock which was crushed and later used for surfacing aggregate. Minor local failures have since occurred in the mantle material remaining on the eastern part of the slide, but all are located high on the slope and pose no threat to the roadway.

SLIDE B

Slide B measured 1000 feet across at the base but exhibited only 200 feet of vertical relief, with a slope distance of 400 feet from toe to crown. Depth to the failure zone ranged from 40 to 50 feet. Springs and small local swamps located above the scarp contributed water to the failure. Corrective measures included improving surface drainage above the slide and flattening the slope, which removed most of the material in failure. This area has been stable since the correction was completed.

SLIDE 4

The westernmost of the six slides, Slide 4 measured only 500 feet across the base but extended 1300 feet up the hillside and exhibited 500 feet of vertical relief. Depth of sliding material averaged about 20 feet. Natural drainage above the crown was confined to a small, narrow valley which permitted construction of an interceptor basin twenty feet deep to collect both surface and subsurface water. This water was then diverted down the hillside by means of a ditch constructed in stable ground along the west side of the slide. The drainage correction, in combination with slope flattening to 1-3/4:1, has stabilized the failure.

SLIDE 3

Slide 3 measured 800 feet across and 1200 feet along the slope from toe to crown, with 440 feet of total vertical relief. Maximum depth of slide material was about 35 feet. Corrective measures applied to this failure were somewhat more extensive than those used on the preceeding slides. Surface drainage was improved above the slide scarp and directed into ditches along both sides of the slide. The slope was flattened to 2:1 and a drainage blanket installed across the toe to facilitate internal drainage. The initial failure at this location encroached well into the westbound lane, as did the failures at Slides 1 and 2, and it was therefore decided to relocate the roadway approximately 40 feet out from the toes of the three slides. No movements affecting the highway have occurred at Slide 3 since installation of the correction.

SLIDE 1

Slide 1 adjoined the east side of Slide 2 but constituted a well-defined, separate failure measuring 1200 feet across at the base and 700 feet along the slope from top to bottom with 330 feet of vertical relief. Maximum depth of the central portion of the slide ranged to about 80 feet. This slide contained large amounts of water and surface drainage was an important part of the initial correction attempted, which also included flattening the slope to 2:1 and relocating the roadway as previously described. The correction was completed by 1970, but the slide continued to move and further investigations were conducted in 1971. Since the upper part of the slide was bounded by competent bedrock, it was recommended that the slide be almost completely removed and that the material developed be used to relocate the road an additional 60 feet out from the slide. The recommended relocation extended from the vicinity of Slide 3 to a point near Slide B and therefore affected both Slides 1 and 2. Since construction of the correction, minor failures have developed on the surface of the remaining material but have not affected the highway.

SLIDE 2

The largest of the six slides and the most difficult to control, Slide 2 measured 2200 feet across, 1500 feet from top to bottom, and exhibited a total vertical relief of 540 feet. Initial recommendations for correction included extensive alteration of surface drainage, together with subsurface herring bone drains, drainage wells and a subsurface drainage gallery driven parallel to the roadway in the bedrock below the failure zone. In addition, slope flattening to 2:1, installation of a drainage blanket across the toe and the 40 foot line change previously described were recommended. Construction of the drainage gallery was discontinued after it became apparent that the bedrock was not as highly fractured as anticipated, and that the desired drainage was not being accomplished. The balance of the correction was completed by 1970 but, like Slide 1, the mass continued to move.

The 1971 investigation led to a recommendation for construction of a shear key buttress of rock at the toe combined with surface drainage and the additional 60 foot line shift previously mentioned. Because of the depth of the trench required for the shear key, construction was scheduled for accomplishment during the winter, when the slide was least active. Construction was to proceed in increments, with rock emplacement immediately following excavation, both operations occurring simultaneously as the work proceeded from west to east across the toe. Several weeks after the start of construction, the buttress rock source was depleted and it became necessary to provide another alternative.

The next design considered was a shear key buttress composed of soil cement. Triaxial tests of 6 inch diameter specimens containing 4% cement showed values of $\phi = 36^{\circ}$ and $C = 50$ psi, permitting design of a somewhat smaller buttress than the rock buttress previously attempted. A rock blanket along the back side of the buttress was included to provide drainage. However, because of cost and other factors, this design was rejected. A caisson shear key was next considered but was also rejected because of cost.

Finally, computer analysis by the Department indicated that removal of material from the upper portion of the slide and compaction of this material into a counterweight buttress configuration at the toe, together with installation of a drainage blanket along the back side of the buttress, would give a safety factor of 1.32. This design was constructed during the summer of 1972 and is apparently successful, having shown no sign of distress at the present writing.

THE SHELL CANYON LANDSLIDES

A GEOLOGICAL ENGINEERING CASE HISTORY

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Rocky Mountain Region, U.S. Forest Service
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ABSTRACT

Massive landslides occurred during construction of a portion of US Highway 14 in the Big Horn Mountains in 1965. The geologic materials were the Cambrian age Deadwood Shale and overlying detritus aided by a very large volume of water. The terrain was historically unstable and the construction was the obvious triggering mechanism.

Two of eight major slides were in excess of a million cubic yards. One was avoided by relocation. Drainage was installed in the other following extensive studies.

A 12" diameter well with a pump set at 201 feet began operating in 1971. Initial water yields of 400,000 gallons per day have tapered off to less than 10,000 gallons per day at present. The slide is becoming stablized and construction will continue in 1973. Drilling of the well was unusually difficult and some novel equipment and techniques were employed.

Prepared for the 24th Annual Highway Geology Symposium
Sheridan, Wyoming
August 9-10, 1973

THE SHELL CANYON LANDSLIDES

A GEOLOGICAL ENGINEERING CASE HISTORY

Foreword. This paper recounts the history of the massive landslides which occurred along US Highway 14 in Shell Canyon, Bighorn National Forest, Wyoming in the spring of 1965 and the subsequent efforts at investigation and stabilization which have continued to date. It is typical of the situations which might be encountered when high standard highways are located across inherently unstable earth material in rugged topographic setting. The primary purpose of this paper is to supplement the field trip review of the site.

History. The Bighorn mountain range is an elongate dome about 100 miles in length and some 30 to 35 miles in width. Its long axis trends NNW to SSE. Pre-cambrian granite forms the core and is exposed in the relatively flat topped area along the crest of the range. Sediments overlie the granite. They are exposed as steeply dipping hogback ridges along the flanks of the range and relatively flat lying remnants along the crest of the ridge. There has been some glacial action in the higher elevations but in the Shell Canyon area the glacial front was above this project and glacial outwash deposits are absent.

There are few places where a transportation route can cross the mountains because of the very steep flanking slopes. The earliest wagon roads of the indian war period connected Fort McKenzie north of Sheridan to settlers in the Bighorn basin near Greybull. The route was generally near the present route, except that it is believed to have been south of Shell Canyon in descending the western slope. A trail did pass down Shell Canyon.

In the 1920's the wagon road and trail was enlarged into an auto road which was typical of the roads of that time. In the 1950's further improvement was undertaken and continues at present. It may be that serious consideration was never given to any other corridor for the most recent development because of the political considerations involved in bypassing established businesses, towns, etc.

The road constructed in the period before World War II was narrow and crooked but cuts and fills were fairly small at least in comparison to the present road. Even so, landslides were common. When the new road was in design the unstable terrain was recognized. However, the magnitude of the problem was underestimated and the urgency of it was not appreciated by the designers.

Complicating the problem was the wider road, flatter curves, and flatter grades required by modern high speed, high volume traffic. This required very large cuts and fills and a large excess of earthwork volume was developed. The solution for this was to steepen cut slopes and widen fills, in some cases adding climbing lanes and scenic turnouts. This is common practice but in this location was catastrophic since the cuts were made even more unstable and some of the fills added weight to the most sensitive area of the slides.

In some $3\frac{1}{2}$ miles under construction in 1965 no fewer than 8 major landslides occurred of which two were in the million yard class and are the primary subject of this paper. Several additional minor slides have occurred and the field trip saw some which have occurred recently.

Investigators. The early pre-design investigations were conducted by W. F. Fitzer about 1960. The primary work after the slide was begun by R. B. Sennett of the FHWA in 1965 and 1966. M. C. Everitt, then with the FHWA, continued that work and drilled two preliminary borings in 1967. In 1968 E. J. Bauer and J. Hale of the Wyoming Highway Department made 32 test borings. In 1969 J. M. Ellis of the FHWA performed pump tests on several test wells and in 1971 Ellis and T. W. Holland installed a permanent well in one of the slide areas. Fitzer and Sennett studied the entire canyon and Sennett proposed solutions to many of the problems which were sucessfully put into practice. The remainder of the work was directed at the largest and most difficult slide between Sta. 766 and Sta. 779.

Geologic Setting. The basement rock is pre-cambrian granite. In the project area it is a red or pink coarse crystalline rock which is fresh and unweathered in most exposures. Its surface is quite smooth and the joints are tight so that it is an effective water barrier. Shell Creek is cut into the granite in the subject area so that in general the road lies near the granite surface and occasionally cuts into it.

Overlying the granite is the Deadwood formation of Darton and Salisbury (1). This Cambrian Age formation is typically about 900 feet thick but locally in Shell Canyon it is 1500 feet thick. Durkee (5) has subdivided this into the units shown on the attached geologic sections. These differ slightly from the descriptions used in other papers presented at this meeting. The road lies near the bottom of the section in the Flathead Shale which is described as soft green and gray shale with thin sandstone and limestone interbeds. These interbeds are aquifers. The basal sandstone unit is generally absent or insignificant in the slide area. It was encountered for the first time in two slope indicator borings made in August 1973.

The massive limestone and dolomite of the Gallatin (Upper Cambrian) and Big Horn (Ordovician) formations form impressive cliffs at the canyon rims (Fig. 1). These are slightly cavernous and unquestionably are a major water source. The regional dip is 4^{o} to 6^{o} southwesterly into the canyon and road. This is a significant factor. The typical section sketch illustrates the general situation.

The surface is nearly everywhere covered with detritus derived from the shale and limestones. It is generally a stony lean clay of extremely irregular nature and thickness. It may vary abruptly from a few feet to over 100 feet in depth.

A striking feature is the nearly complete absence of forest cover on the north wall of the canyon as compared with the heavily forested south side. This is not due to the instability but to a forest fire before the turn of the century. Subsequent natural processes and grazing have prevented the return of timber.

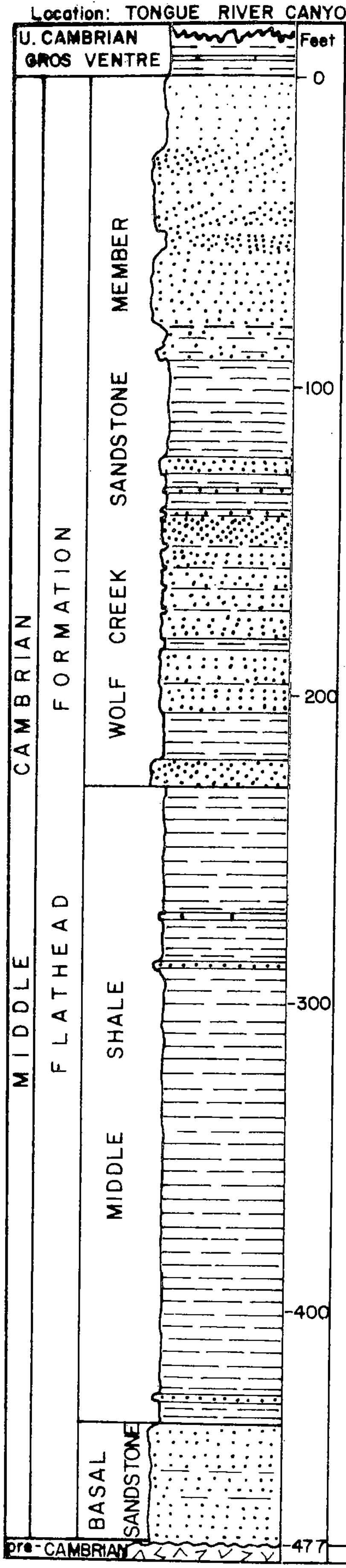
GENERALIZED GEOLOGICAL SECTION — NORTHERN BIG HORN MOUNTAINS

System	Formation	Thickness (feet)	Description
ORDOVICIAN	BIGHORN DOLOMITE	300±	Dolomite and Limestone buff, massive, basal sandstone, light gray to white, prominent escarpment
UPPER CAMBRIAN	GALLATIN LIMESTONE	50–200	Limestone, gray to tan, thin-bedded to flaggy, flat-pebble conglomerate, in places cherty.
UPPER CAMBRIAN	GROS VENTRE SHALE	500±	Shale, gray—green, glauconitic, calcareous, many intercalated thin-bedded sandstone and limestones, subject to landslides.
MIDDLE CAMBRIAN	FLATHEAD FORMATION — WOLF CREEK MBR.	140–229±	Sandstone, gray-white, buff to brown, fine-to medium-grained, thin-to thick-bedded, occasional cross-bedded and massive, cliff former, intercalated shale, gray, green, buff and maroon, fissile, soft
MIDDLE CAMBRIAN	FLATHEAD FORMATION — MIDDLE SHALE	175–250	Shale, gray—green, gray, soft, intercalated sandy shale, limestone and sandstone, subject to landslides.
MIDDLE CAMBRIAN	FLATHEAD FORMATION — B. S.S.		Sandstone, buff to yellow-orange, friable feldspathic
Pre-Cambrain			Granite, pink, gray.

"from E. F. Durkee, 1953"

- GEOLOGICAL SURFACE SECTION — NORTHERN BIGHORN MOUNTAINS

Location: TONGUE RIVER CANYON



Shale, gray—green, calcareous, intercalated Limestone and occasional flat—pebble Conglomerate.

Sandstone, gray—white, buff to yellow, brown, fine—to medium—grained, thin–to thick–bedded, in part massive and cross–bedded, friable to cemented, porous, (grades laterally into Limestone, tan to gray, thin—to medium—bedded, arenaceous), cliff former.

Shale, gray to green, slightly calcareous, glauconitic, fissile, soft.

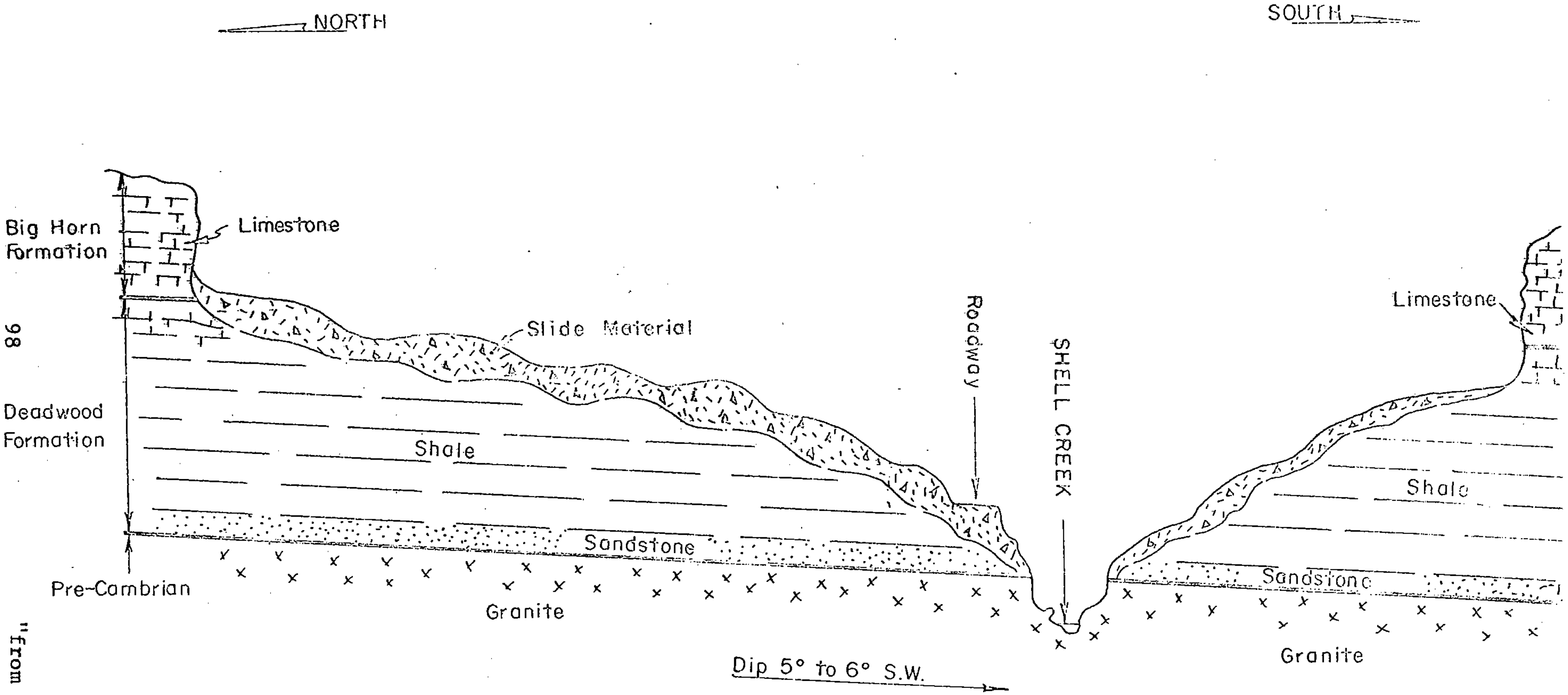
Sandstone, gray to white, red–brown to brown, fine–to medium–grained, in part massive and cross-bedded, friable to cemented, intercalated shale, buff to gray—green, maroon, fissile, slightly calcareous, soft.

Shale, gray—green, gray, soft, minor intercalated, thin Limestone, Sandstone, and sandy shale, mostly covered, subject to landslides.

Sandstone, buff to yellow—orange slightly calcareous, friable to cemented, feldspathic, coarse—grained, porous, minor sandy shale partings.

Granite, pink, gray.

"from E. F. Durkee. 1953"



Typical Section

No Scale

"from R. B. Sennett, 1967"

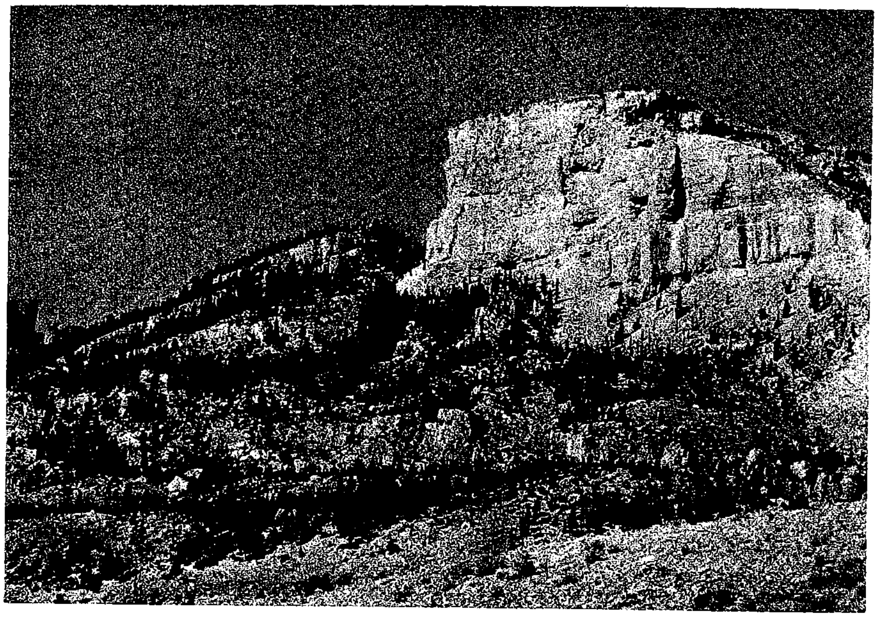


Fig. 1. Copeman's Tomb. Typical landslide terrain in Shell Canyon a short distance west of slides. Cliffs are in Gallatin and Big Horn Formations. Slope below is debris overlying Deadwood Formation.

Fig. 2. Oblique view of slide area Sta. 661-683. Note relict slide features nearly duplicating present slide and straight line of supposed joint or fault extending from the slide area to the skyline. Faint scar in middle area is location of a drain line intercepting springs above slide.

Aerial photography of the canyon displays nearly every recognizable feature of relict and active landslides. The magnitude of some of the past movements is startling and it is interesting to note that many of the current landslides have occurred within the recognizable scarps of earlier landslides.

There is a remarkable amount of water present as will be noted later. The sandstone and limestone interbeds in the shale are very good aquifers and water is available in the snow and rainfall of the high country which migrates down into the canyon. Relict slide planes and the generally permeable detritus provide ample storage and transport volume. The shale is incompetent and very sensitive to water.

Landslide, Sta. 661-683. In early March of 1965 a fill failure caused by overloading an unstable foundation developed around Sta. 682. The writer spent a week in bitter weather attempting to analyze it and on the day he left to return to Denver, a minor cut slope slump appeared near Sta. 667. Within a week or two this had developed into a million yard landslide extending from about Sta. 661 to Sta. 683 and the fill failure study became academic.

The area near Sta. 661± has always contained an active spring. This spring was a continual maintenance problem since it appeared at the roadside, softened the surface and caused severe winter icing problems. As the slide progressed, other wet areas appeared which had not been seen earlier.

Study of air and ground photography revealed that the present slide scarp almost exactly occupied the crescent scars of earlier movements. The photos also show a straight line, displayed as a green line of lush vegetation, extending all the way to the canyon rim (Fig. 2). This may be a fault or major joint. Along this line springs appear and disappear several times with continual surface flow only at the wettest seasons. The line intersects the crescent scars near their center. It is theorized that the water split and went both ways along the relict scarps to emerge as springs and seeps, with the most of the volume at Sta. 661. One of the first steps in controlling this slide was to trap as much of this water as possible where it was on the surface, some 1500 feet upslope from the head of the slide, and divert it through a pipeline into another drainage outside the slide area.

The remainder of the solution to this slide can best be called relocation or avoidance. The former road location is now covered by slide material and buttress earthfill. Shell Creek originally ran in a deep granite gorge about where the present road now lies between approximately Sta. 662 and Sta. 672. The stream bed was shifted about 100 feet south in a channel cut into the granite rock. At the same time the gradient was flattened to 2% so that a waterfall was created at the outlet (Fig. 3). The channel upstream was then filled to conform to the new gradient. This provided a buttress fill which controlled the large mud wave which had developed in the creek bed. It also provided space for a large volume of waste soil material and has made fishermen access and parking possible where none existed before.

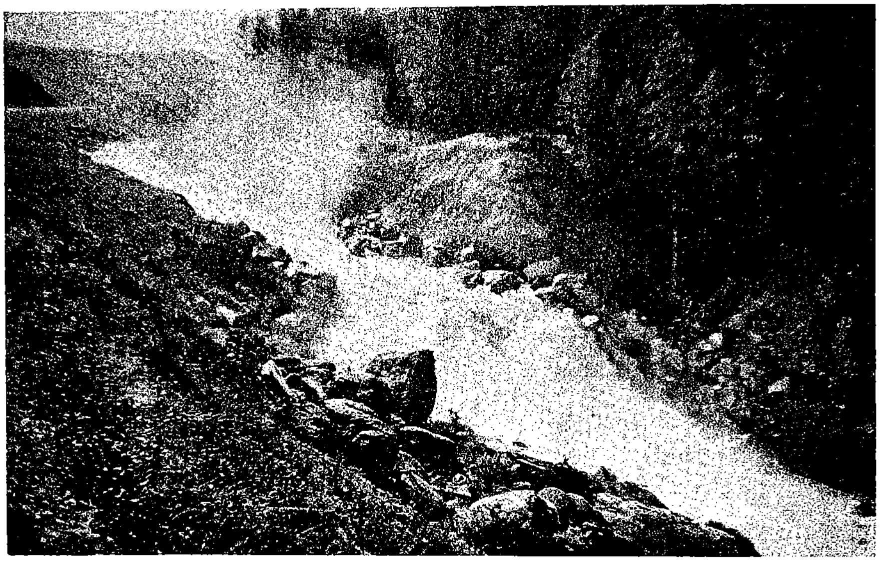


Fig. 3. Waterfall at the end of channel change below Sta. 660.

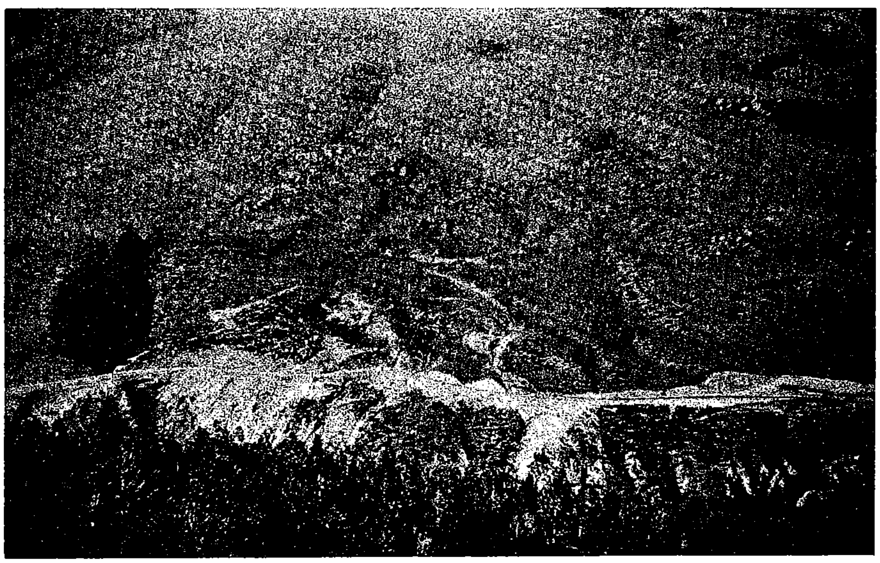


Fig. 4. Oblique view of slide area Sta. 766-779. Note relict scarps and other slide features. Slump portion is on the left, mudflow portion is almost in the center of the picture.

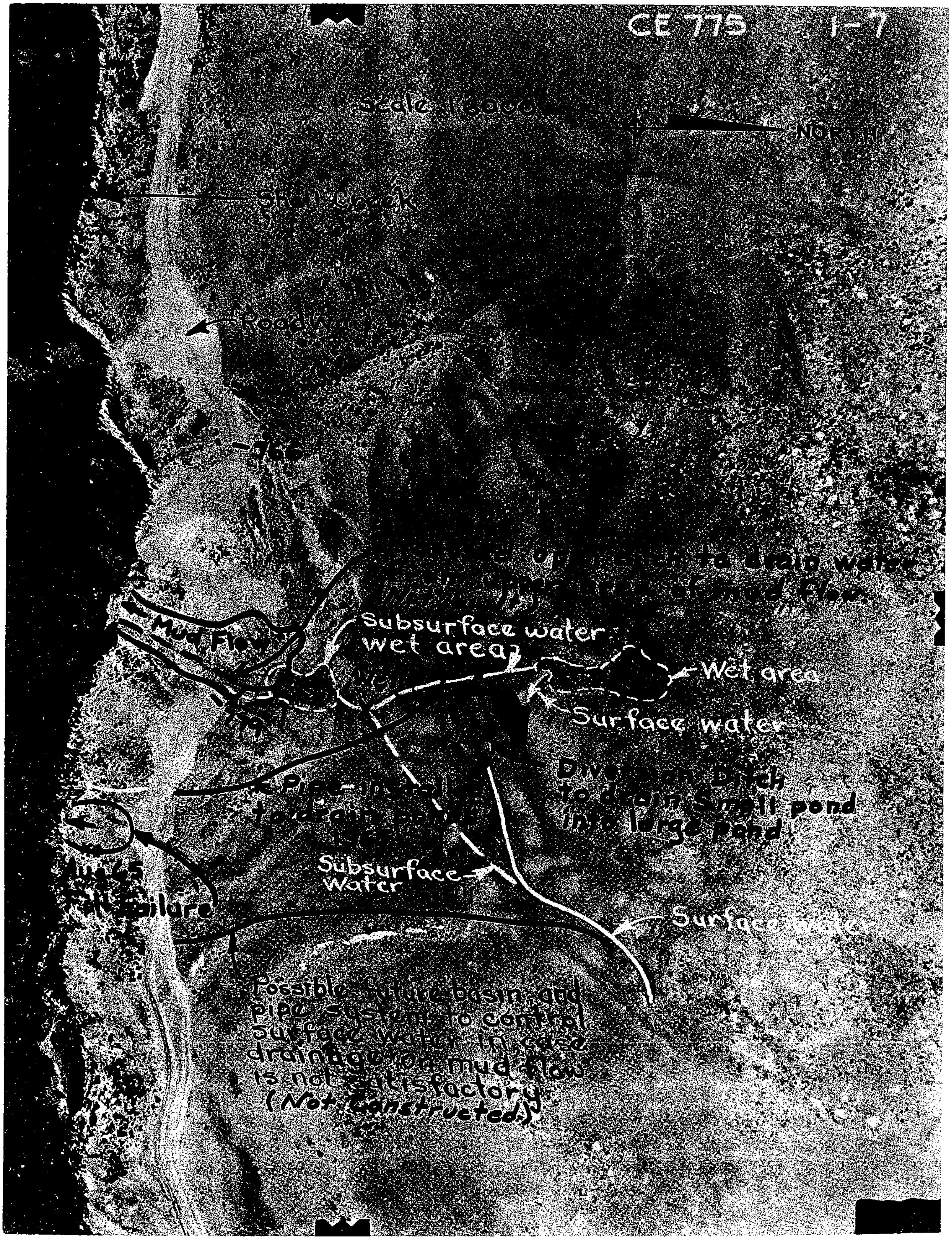


Fig. 5. After Sennett. Vertical air photo of major slide area showing several remedial measures proposed early in the studies. Eventual location of relief well was near the intersection of dashed lines above the head of the mudflow.

The road in this section is now stable. Some ravelling of the slide mass continues but it is unlikely to again encroach on the road.

Landslide, Sta. 766 to Sta. 779. This is really two slides acting simultaneously. The larger segment is a slump extending from Sta. 766 to about Sta. 776. A mudflow extends from Sta. 776 to Sta. 779 (Fig. 5). A large block of exposed flathead shale separates the heads of the two slides but below that they run together. The shale outcrop may be a detached block but has not been involved in the recent movements.

The two occurred at the same time and it is not known whether either might have started first and triggered the other. At its maximum, movement was estimated at around 3 ft. per minute and a section of the partially completed road was displaced, almost intact, about 200 feet downslope. The mudflow reached the creek and formed a dam which later was breached by the stream. A small pond remains and it is understood to be a good fishing spot.

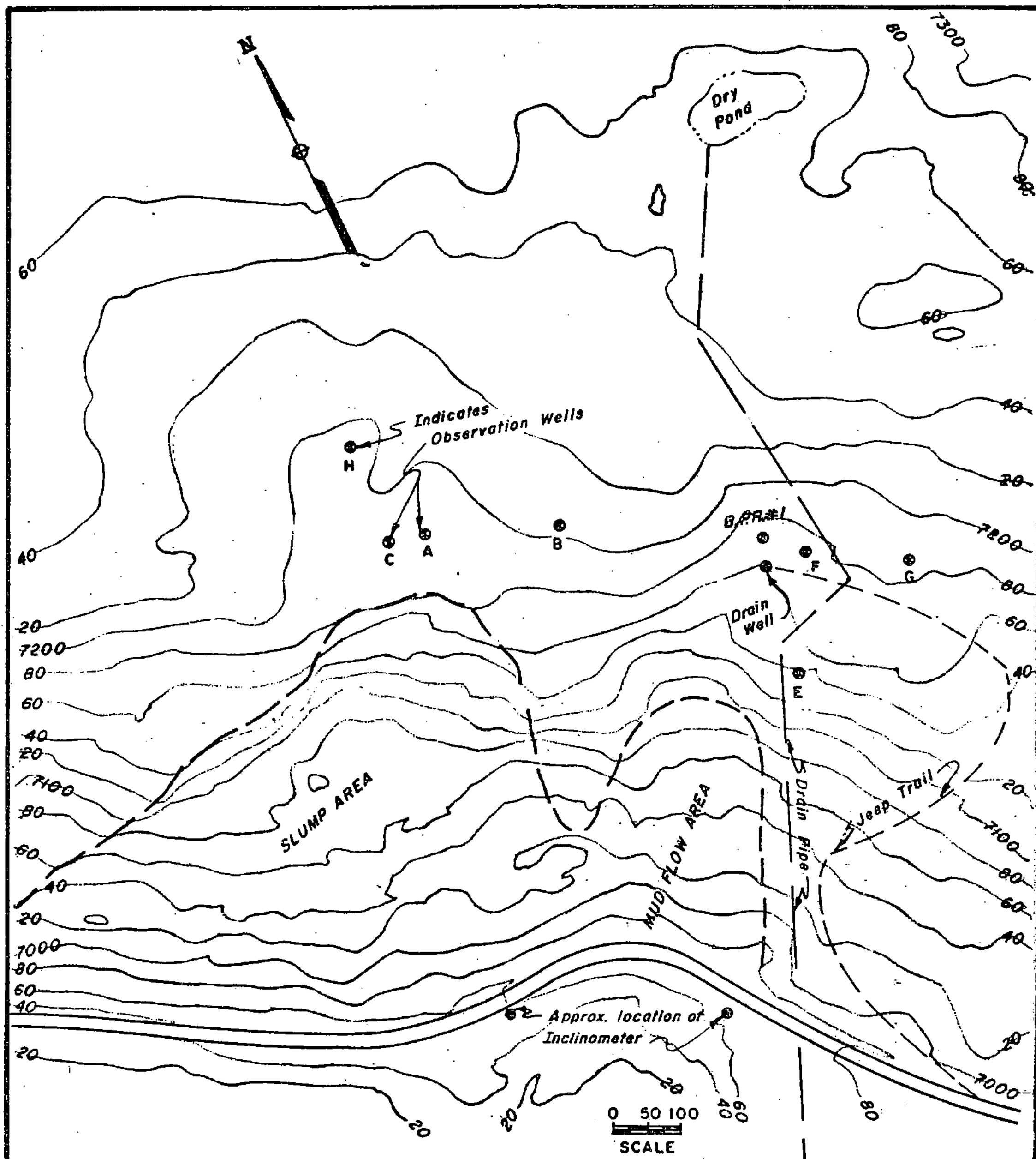
The toe of the slump also reached Shell Creek. The upper units of the slump area were dropped almost vertically about 60 feet and the head-scarp remains almost unchanged (Fig. 6). The upper units were nearly intact and contemporary photos show a long section of fence and several undisturbed trees on these units. The lower units were badly jumbled and subsequent adjustments have further reduced them.

Water was a striking feature. The mudflow was saturated for months and even after surface flow disappeared, one could hear water moving just under the surface. A large spring occurred near its head with flows roughly estimated at 30 to 50 gpm even in dry seasons.

In comparison, the head scarp and the body of the slump was dry in spite of permeable soil and ample water in a test boring less than 200 feet above the head scarp. Several explanations for this have been offered but none has satisfied all investigators.

Two small ponds formerly existed about 650 feet north (upslope) of the head of the mudflow. The smaller drained into the larger but the larger had no surface outlet even though it received considerable inflow from springs and rainfall. It fluctuated from about 5 feet to 15 feet in depth and was about 100 feet wide. It never completely dried up.

While there was initial opposition to draining ponds which served livestock and game and which were seemingly a long distance from the slide, this was done since it was abundantly evident that they contributed to the mudflow problem. The smaller was trenched into the larger pond. Spring boxes were placed in the larger pond and a buried pipeline took the water around the slide area into a roadside culvert where it was released. The ponds have never been allowed to refill though springs in the area were still active and the water diversion was not total, up to the start of pumping in the drainage well in 1971.



MAP OF "BIG SLIDE" AREA
SHELL CANYON, WYO.
(CONTOUR INTERVAL 20')

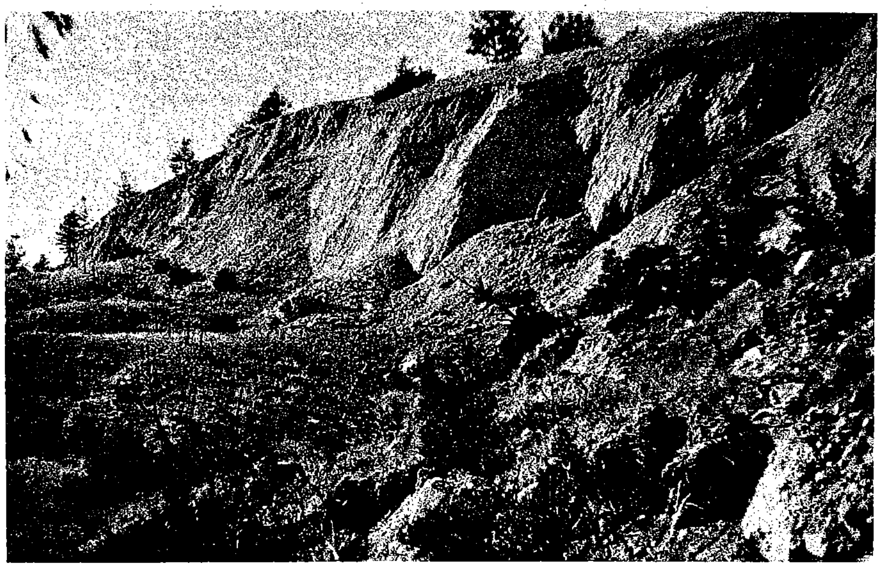


Fig. 6. Contemporary view of upper unit and headscarp near Sta. 770. Scarp is more than 60 feet high. Note undisturbed fence on slump block.

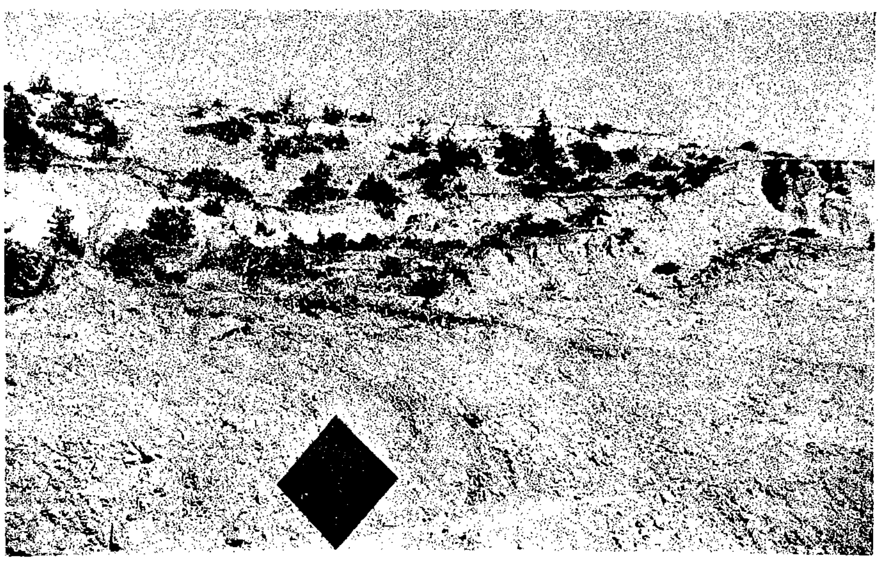


Fig. 7. View up from restored road near Sta. 776 showing face of major slump area.

It was concluded early that relocation of the road was impossible and restraint of the slide could only be attempted through control of the water. The first proposal involved horizontal drilled drains but it was accepted that no installation would be possible if it had to pass through the slide body which was expected to continue its internal adjustments for some time.

A drainage caisson, which would be big enough to admit a horizontal boring machine was proposed for a site about 80 feet above the head of the mudflow and on a line between the mudflow and the ponds. This would have been 14 feet in diameter and at least 105 feet deep. Radiating from it at one or more levels would have been arrays of lateral drains sloping at various angles toward the caisson and lined with slotted plastic tubing. The prospective installer claimed the ability to place the pipe at least 600 feet out from the caisson, in any direction.

The sewer leading out of the caisson was to be drilled out to daylight from the bottom of the caisson and lined with wire wrapped rubber hose which could stand considerable distortion without stopping the flow. Two outlets in different directions were contemplated to reduce the chance of failure. The whole installation would have been essentially maintenance free since no pumps were required. It would have cost in excess of $100,000 at 1967 prices. Later experience clearly showed that the proposed 105 feet depth would not have been enough to be fully effective.

Precedent for such caissons exists on Togwotee Pass where 2 or 3 such installations have been functioning successfully for many years but these are only about 35 feet deep.

With this caisson plan in mind a preliminary boring was made at the proposed caisson site in 1967 using a Mayhew 1000 rotary drill. It encountered water estimated crudely at 30 to 50 gpm beginning about 53 feet below the surface and apparently diminishing below depth 104 feet. The boring was stopped at 156 feet without reaching granite but a casing was left in place to 96 feet and this later became a test well.

A second boring about 600 feet northwesterly was abandoned at 94 feet when the hole collapsed repeatedly. No pipe could be set here but water was encountered and noted and a later observation well was drilled at this location. These two borings were not conclusive in themselves but were a valuable prelude to later work.

In 1968 E. J. Bauer and Jack Hale of the Wyoming Highway Department drilled 32 test borings on and around the slide area and made many observations of the ground water and stratigraphy which was valuable in further defining the materials involved and the structural situation.

In the fall of 1969 J. M. Ellis of the FHWA made eight additional borings, several of which were cased to serve as permanent observation wells. Pumping tests were conducted in the original FHWA boring and in one new boring. Drawdown and recovery were observed in the two pumped wells and the observation wells.

Using all of the data previously compiled, Mr. Ellis designed the single permanent relief well which was finally installed in 1971. This is a fairly normal water well in design but its construction was unique in many aspects and its performance has exceeded expectation. The well was located a short distance downslope from the first test boring and very near the originally suggested caisson site.

One of the problems encountered in drilling in colluvial material is the inability of a rotary rig to maintain circulation through the jumbled and broken slide debris. For this reason it was decided to place a surface casing through the colluvium to enable the rotary driller to hold circulation while drilling the bedded materials.

A twenty-inch surface casing was placed through the colluvial overburden to the top of bedrock (Flathead Formation). This surface casing was driven by the Becker Hammer Drill which is essentially a diesel pile driver driving a string of double wall drive pipe. When casing is to be placed, it is driven simultaneously with the drive pipe. A bit on the lower end of the drive pipe projects slightly ahead of the bottom of the casing and is centered in the bottom joint by flutes (Fig. 8, 9, 10). The drive head at the top of the string positions the top of the casing relative to the drive pipe. Compressed air is pumped through the annulus between the inner and outer walls of the drive pipe, water is pumped through the annulus between the drive pipe and casing, and cuttings are brought up through the center of the drive pipe. The cuttings are discharged through a cyclonic collector into a tub where they may be collected for sampling and examination (Fig. 11).

Operations commenced on August 31, 1971. The twenty-inch surface casing was driven to a depth of 44 feet through colluvial material composed primarily of green shale and limestone fragments. Some flat pebble conglomerate and sandstone fragments were noted. At a depth of 44 feet limestone boulders were encountered. At 58 feet, while attempting to drive through these boulders, the twenty-inch surface casing was bent and a tooth was broken off the drive bit. At this point it was decided to abandon this hole, move over 15 feet and start over again.

Drilling was suspended for 10 days to repair the equipment and build a new bit. The casing was abandoned in place. On restarting the hole, a rotary drill was available to drill a pilot hole whenever the casing encountered boulders.

Alternating drilling with the rotary using a 12-inch bit and driving with the Becker Hammer drill bottomed the surface casing in shale at 69 feet. A wet zone was noted from 50 feet to 68 feet, just above the contact between colluvial material and bedrock as exploratory drilling had previously indicated.

Rotary drilling in the Flathead Formation began at the 68 foot depth. Drilling was done with a composite bit consisting of a 9-inch pilot running 2 feet ahead of an eighteen-inch reamer. No drill collars or stabilizers were used.

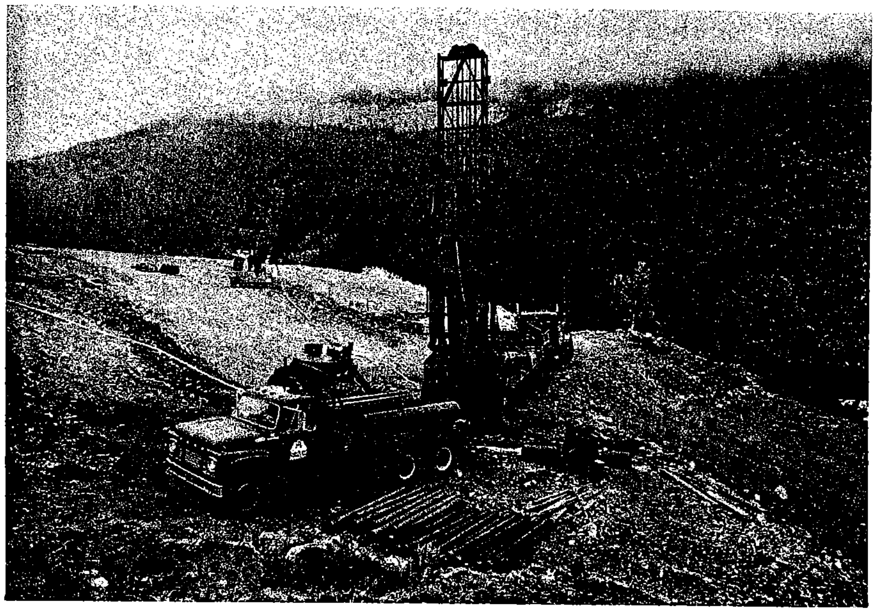


Fig. 8. Becker Hammer Drill set up on the drainage well site. Cyclone separator is partially visible behind pipe truck.

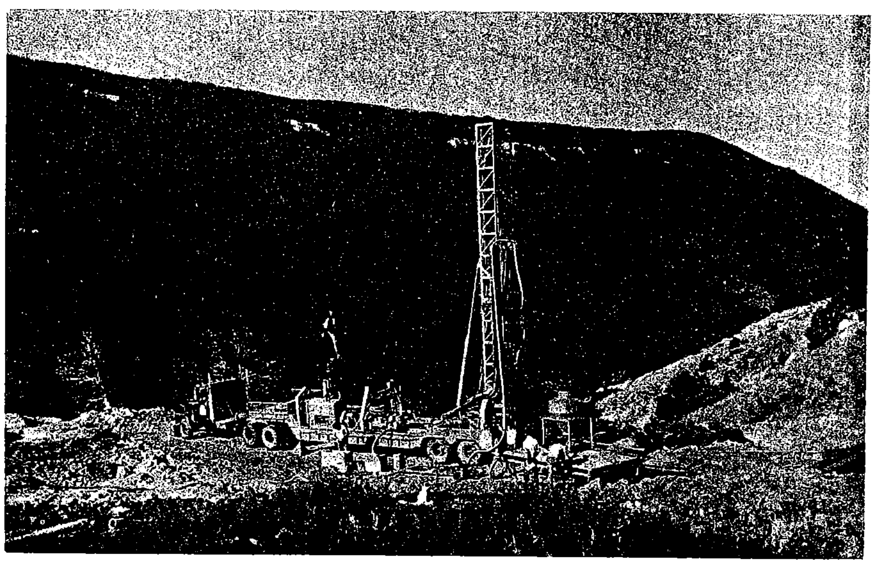


Fig. 9. Another view of Becker Hammer Drill.

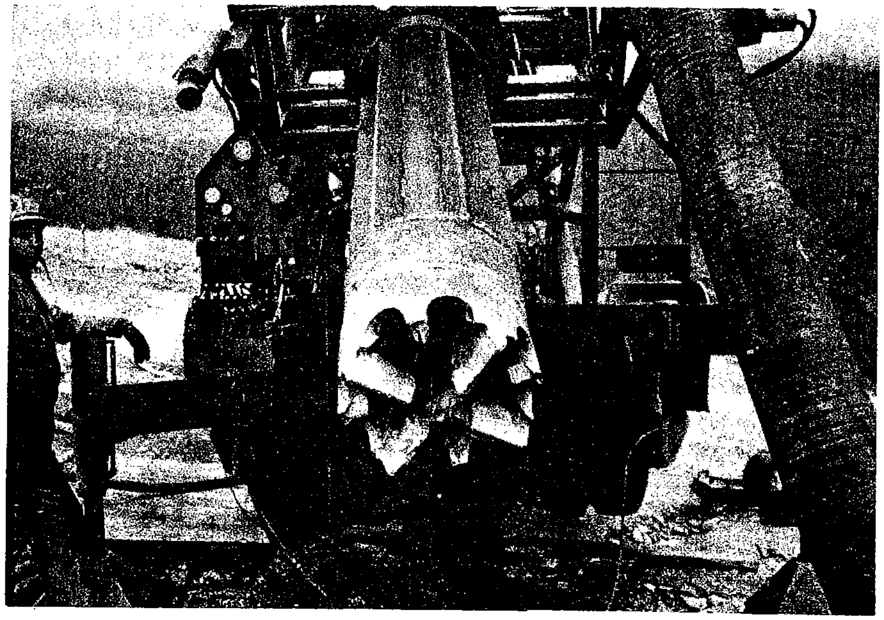


Fig. 10. 20-inch diameter bit used on Becker Drill.

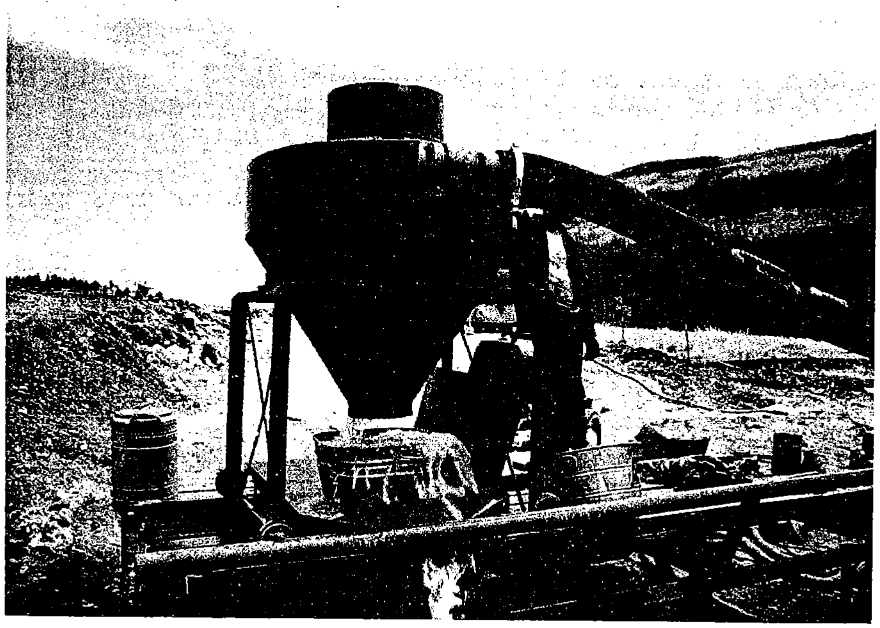


Fig. 11. Cyclone seperator extracting cuttings from the drilling fluid.

Bentonite drilling mud was used in the circulatory fluid to remove cuttings and hold the hole open. It was necessary to continually thin the mud during the course of drilling after the initial batch had been made up as the hole made its own mud from the silty shale being drilled.

The well was finally bottomed out at a depth of 212 feet on October 23, 1971. This was about six feet into the pre-cambrian granite which was first encountered the day before at 206 feet. The basal sandstone unit of the Flathead was absent.

The twenty-inch surfacing casing was jet perforated with thirty shots from 52 to 68 feet depth to facilitate drainage of the wet zone above the shale - overburden contact.

The original intent had been to place a temporary 18-inch casing from ground level to top of granite in order to hold the hole open while setting screens and permanent casing and to pull it back while installing gravel pack. Setting of the 18-inch casing began on October 29, but refusal was encountered at a depth of 77 feet; 8 feet below the bottom of the surface string. Refusal was apparently due to a slight angular deviation of the rotary hole from the alignment of the surface string. The next day the eighteen-inch casing was removed prior to further work. An exploratory probe with the eighteen-inch bit indicated no sign of sluff or caving and the decision was made to condition the hole, and set the well screens and 12" permanent casing without the use of the eighteen-inch temporary casing.

A Gamma-Ray Neutron log of the boring was made by Dresser-Atlas on November 2.

Five ten-foot sections of 12-inch diameter, #80 slot, Johnson stainless steel well screen were set in conjunction with the 12-inch permanent well casing at depths of 68 - 78', 86 - 96', 132 - 142', 148 - 158', and 188 - 198'. The screen settings were based upon observation of the drilling, cutting logs, and the radioactive logs.

During drilling small artesian flows had been noted at several depths indicating the magnitude of the water present.

An artificial gravel pack manufactured by Fountain Sand and Gravel of Pueblo, Colorado was placed in the annulus between the boring wall and the screens. This is almost an ideal gravel pack material for the aquifers encountered. The contractor had originally planned to place the pack by pouring it down a 2-inch plastic tremie pipe, but this method proved extremely slow. Halliburton Services was called in to pump the pack into place. Twenty-four thousand pounds of gravel pack (250 cubic feet) were placed in about fourteen hours of pumping.

The theoretical annular volume of the hole is 210 cubic feet. The excess of gravel pack indicates some cavitation in the hole, and also is indicative of a successful gravel pack.

Development of the well was initiated with the placing of 100 pounds of sodium hexameta-phosphate in the boring on the evening of November 8 and allowing it to remain in place overnight to "break" the drilling mud gel. The well was surged using a 900 cfm compressor for fourteen hours. Each screen was surged individually for as long a period as was needed to completely clear the water produced in that zone. Observation of the discharge water during development disclosed no removal of the pack material.

A concrete pump base was poured around and integral with the surface pipe. The pump which was installed is a Peerless eight - L B, eight-stage turbine pump rated at 250 gallons per minute, with bowls located at a depth of 201 feet. Sensors were set in the well to activate and deactivate the pump relative to the water level. The pump initially activated when the water level reached a sensor at the 85 foot level and ran until the water was drawn down to the 193 foot level. This normally took a few minutes of operation every 4 to 5 hours though initially the pump ran almost constantly. It is understood that the sensors have recently been adjusted and the present depths are not precisely known. The water is piped to Shell Creek. A signal light visible from the highway below was installed to warn of mechanical or electrical failure in the system.

The pump was placed into operation on November 19, 1971. Initial discharge was approximately 400,000 gallons a day, and after three days pumping, the rate had leveled off at approximately 72,000 gallons per day. It should be noted that observation well "F", located about 75 feet WNW of the drain well, was used as a water source during drilling operations. It was pumped an average of six hours a day for almost two months, and during placement of the gravel pack 150,000 gallons were obtained from it in 24 hours.

It is expected that the discharge from the relief well will vary considerably seasonally. The important fact is that the zone of influence, as depicted by water level measurements in the test borings, within the slide body, has developed a radius in excess of six hundred feet, which is being affected or dewatered by the pumping.

At this time the discharge is variable with a peak daily rate of about 40,000 to 50,000 gallons. New data indicates that it may have been as low as 7,000 gallons per day in the spring of 1973. The majority of the observation wells have completely dried up as have all of the springs on the slide body and near road. Some springs high on the relict scarp areas north of the well still yield a little water but much less than before. One which was about 1,000 feet northeasterly and some 200 feet higher than the wellhead has dried up completely. This effective radius of the well has greatly exceeded expectations and while seasonal fluctuations of the water may still occur, the slide mass near the road is effectively dewatered and is sufficiently stable to allow reconstruction of the road which is now underway.

The cost of the well was about $86,000 including construction of about 1 mile of electric transmission line. This does not include the cost of all of the explorations which preceded the well construction.

To date maintenance has been insignificant however, as equipment ages, maintenance will increase. It is understood that there have been some problems with the pump cycle adjustment. This well must be kept permanently in operation. If it is ever allowed to be inoperative for an extended period the ground water will recharge and landsliding problems at the road can be expected to resume.

In August 1973 two inclinometers were set on the shoulder of the existing roadway outside the construction limits to monitor any movement during completion of the roadway. Both holes drilled for installation of the inclinometers were completely dry. No significant data is available as yet (see sketch map).

This has been a challenging project to which many men have made signicant contributions. Everyone who participated can take pride in the successful and economical solution of many very difficult problems. We trust that the experiences of this section of the Shell Canyon Highway will guide future efforts further down the canyon so that these massive landslides will not be duplicated.

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APPLICATION OF GEOLOGY TO HIGHWAY CONSTRUCTION
IN MOUNTAIN TERRAIN, LOVELL-BURGESS JUNCTION, WYOMING

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Abstract

In construction of highway systems in mountainous terrain, a thorough understanding of geologic principles is fundamentally important, in as much as problems of slope stability are often more critical than in areas of low relief and less extreme climatic conditions.

The purpose of this report is to present a case history of highway construction in mountainous terrain where landslide susceptible formations and poor soils limit route selection and oblige the engineering geologist to critically examine all parameters for backslope design, fill height, and structure design to insure a minimum risk of road or structural failures.

Systematic investigations include three basic phases: 1) Corridor Study; 2) Location for Baseline; and 3) Final Soils Profile.

Introduction

The goal of the highway engineer is to design and build good highways economically so that the user may transport himself and his goods economically and safely. In civil engineering, geology is fundamentally important, in as much as roads are founded on geologic materials and built of geologic materials.

The purpose of this report is to present a case history of highway construction in mountainous terrain where engineering geology conditions are of critical importance because of the limitations in the route selection. Landslide susceptible formations and poor soils cannot always be avoided without unduly restricting the geometrics of the road design. Consequently, all facets of the engineering geology must be thoroughly investigated in order to establish parameters for backslope design, fill heights, and design of underdrains, bridges, and other structures in order to insure, within reasonable limits, a minimum risk of road or structural failures. Highway construction material sources must be established within short distances of the construction site to maintain economical operations, although in mountainous areas, environmental considerations often require that materials be obtained from distant sources.

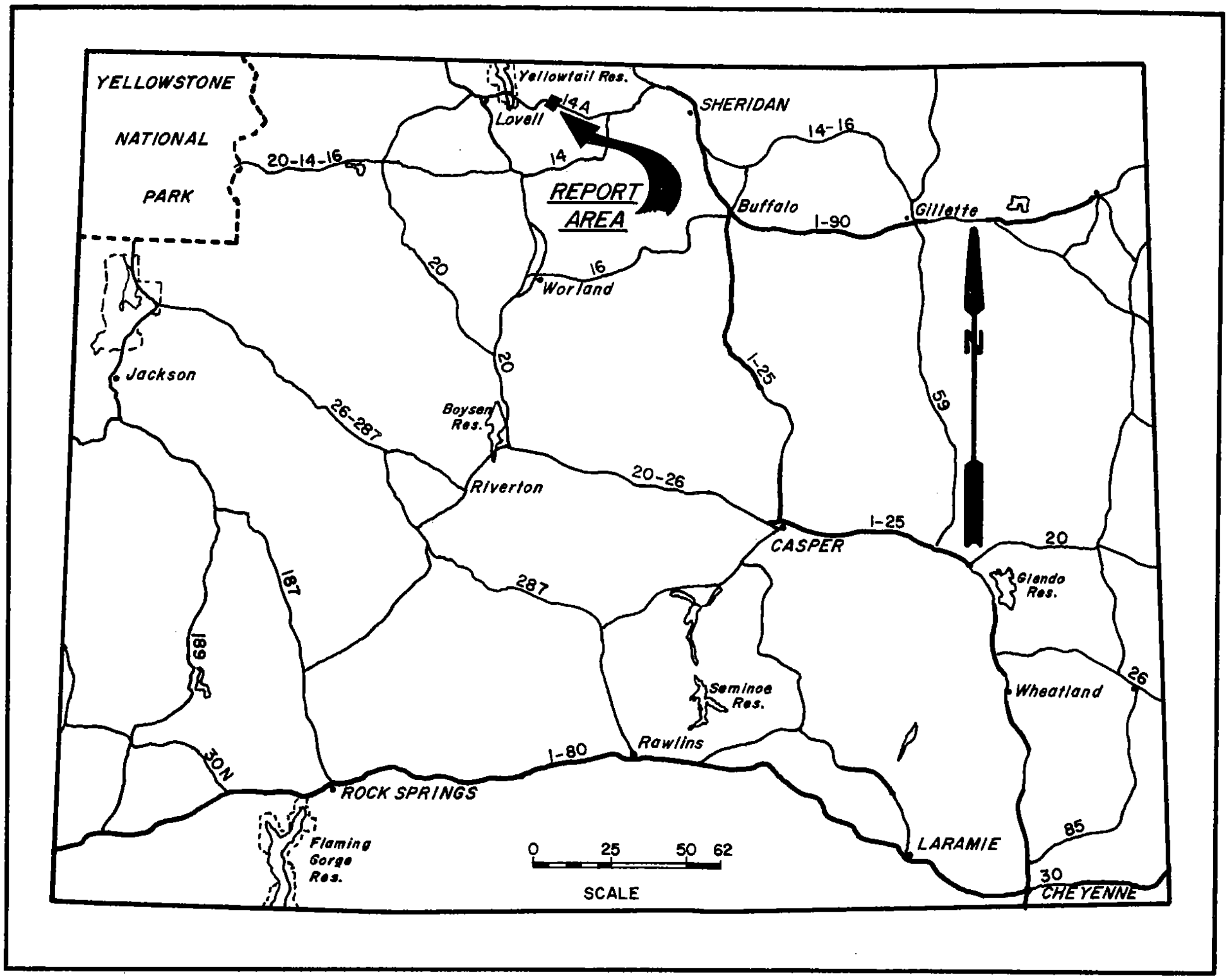


Fig. 1 - Location map. Report area is 70 miles west of Sheridan, Wyoming along Highway U. S. 14 Alternate

The report covers four construction projects included in the primary, secondary, forest road system and financed entirely with State funds.

Location:

The area of the report is located in the northwest part of the Big Horn Mountains, Wyoming, and along Highway U. S. 14 Alternate. The city of Sheridan is approximately 70 miles east, while the town of Lovell, in the Big Horn Basin is located 35 miles west of the report area.

Topography and Drainage:

The topography reflects an early mature stage of development. The average elevation along the highway in the report area is 8,500 feet above mean sea level, with the highest elevation of 9,270 feet being near Bald Mountain in the vicinity of the drainage divide between North Tongue River and Crystal Creek (fig. 4). The lowest elevation of 7,930 feet is near the western end of the new highway construction project (PSF 6484).

Fig. 2 Stereogram of a mountain valley slope in Gros Ventre shale, along Highway 14A. The symbol, Qc, represents Quaternary colluvium consisting of clay, silt, and rock fragments. Its relative stability is indicated on aerial photographs by the smooth texture and uniform tone.

The symbol, Ql, represents Quaternary Landslide consisting of clay and rock fragments, and often Gros Ventre shale. An old mud flow Qlmf, is clearly defined in the center of the stereogram. The instability of landslide areas is indicated on aerial photographs by the irregular surface texture and variable tone. Springs are often present (near cabins).

The principal water courses are North Tongue River and Beaver, Crystal, and Five Springs Creeks. Small tributaries, springs, and seeps are present throughout the area.

GENERAL GEOLOGY

Stratigraphy and Soils Classification:

Early Paleozoic and Precambrian age rocks are widely distributed in the Big Horn Mountains. Escarpments of light-colored limestone and dolomite of the Madison and Big Horn formations rise imposingly along the highway in the report area. Subdued slopes below the scarps are underlain by grey-green shale and thin-bedded limestone of the Gros Ventre and Gallatin formations. The Gros Ventre shale often lies directly on granite of the basement complex, but in places, thin-bedded Flathead sandstone forms the basal unit of the sedimentary rock section.

Alluvial silt and clay are present as thin deposits in the drainages. The Gros Ventre formation is covered generally by a variable thickness of colluvium or landslide debris composed of clay, silt, and rock fragments derived from the Gros Ventre shale and overlying limestone formations (fig. 2). Deep-seated landslides include the Gros Ventre shale because this unit is slide susceptible. Disintegrated granite soil is present frequently in areas where erosion has removed the Paleozoic rocks and exposed the granite to weathering.

The following soils classification is based upon laboratory analyses of samples taken from the outcrop, drill cuttings, shelby tubes, cores, and penetration probes. The ranges of California Bearing Ratios ("R" values) are also given.

TABLE 1 CLASSIFICATION OF SOILS DEVELOPED FROM BEDROCK

Soil Unit	Parent Material	AASHO Classification	Range of CBR Values
Alluvium	Limestone, sandstone, and siltstone	A-4 to A-6	11 to 33
Colluvium	Limestone, sandstone, and shale	A-1 to A-7	6 to 70
Landslide	Limestone and shale	A-4 to A-7	6 to 22
Remolded Fill	Gros Ventre Shale	A-4 to A-7	5 to 29
Remolded Fill	Flathead Sandstone	A-1 to A-2	18 to 75
Disintegrated granite	Precambrian Granite	A-1 to A-6	15 to 77

Structure and Engineering Geology Characteristics:

Paleozoic strata dip at an average of four degrees along Highway 14A in the report area. Variations in the directions of strike and dip are caused by gentle flexures in the bedrock that are only slightly discernible on aerial photographs because of the very shallow dips. Local steepening of dips is present near faults and in slump blocks. Bedrock attitudes reflect the vertical uplift of the Big Horn Mountains, as compared to uplift by compressional forces that generally result in steep dips in the crestal areas. Strata on the flanks of the Big Horn Mountains are, of course, steeply dipping and often overturned.

The stability of a slope on stratified rock depends, in part, on the orientation of the bedding plane with reference to the slope. If the bedding planes are horizontal, no bedding plane slides will occur. When inclined bedding planes dip toward the roadway centerline at angles smaller than the angle of friction at the bedding plane, the critical backslope is vertical. For bedding plane dips greater than the angle of friction at the bedding plane, the critical backslope angle is equal to the dip of the bed. Because of the shallow dip of the strata in the report area, bedding plane slips were not anticipated, nor did they occur after construction.

Fracture systems developed in the Precambrian rocks during incipient stages of mountain building and later in the overlying sedimentary strata are the primary concern in the relationship between structure and engineering characteristics of bedrock. Faults and joints serve as conduits for ground water that infiltrates slide susceptible, clayey formations and often produces unstable conditions especially where the water table is shallow.

Original joint sets in the Precambrian and Paleozoic rocks unaffected by the Laramide orogeny are shown in Table II. Joint spacing varies from a few inches to occasionally ten feet.

TABLE II ORIGINAL JOINT SETS IN THE NORTHERN BIG HORN MOUNTAINS (WILSON)

Average Strike	Average Dip
N 21° E	37° SE
E - W	Vertical to 54° S
N 48° W	Vertical
N 45° E	Vertical
N 83° E	56° N
N - S	Vertical to 56° E
N 5° W	49° W

These joint sets cross the dendritic drainage pattern; and consequently, the open fractures are constantly replenished with water from the streams. The problem of "turning off" the water in mountain highway construction projects in highly fractured rock can well be appreciated by the existence of many springs and seeps.

The fracture systems form many planes of weaknesses in normally competent rock types such as limestone, sandstone, and granite. Joints and faults control the critical backslope angle even when shallow dipping or horizontal bedding planes are present. In general in the report area, failure will occur along the joint planes in competent rock most often if the backslopes are steeper than 1/4:1, while the safety factor will increase as the backslope is flattened within the limits of the natural slope angle.

METHODS OF INVESTIGATION

Highway construction in mountainous terrain requires a detailed study of the geology of the region and the locale because of the great variability of bedrock and soils types, topographic conditions, and ground water surfaces within a relatively narrow route band. In comparison, generalized investigations often suffice in flatlands where soil and bedrock types may be uniform over extensive areas, where slope conditions may be gentle or sometimes non-existent, and where ground water may be a negligible factor in foundation stability. Rolling topography that is present in the basins adjoining the Big Horn Mountains presents many engineering geology problems similar to those confronted in mountainous terrain; and consequently, a considerable amount of overlap in the methods of investigation is necessary in these areas.

Systematic investigations in the report area include three basic phases (fig. 3), although, at times, overlapping of specific items of investigation did occur for various reasons, such as, a) availability of drilling equipment and field personnel, b) weather conditions, and, c) economic considerations.

1. Corridor Study Phase:

   The initial phase of investigation requires a regional study of geological and terrain conditions so that the most feasible corridor can be selected on the basis of optimum soil characteristics and road design geometrics. In recent years the impact of road construction on mountain environment has become an important consideration; although, in the past, concern toward the environment has been reflected by the need for adequate backslope and fill slope stability analyses, proper tunnel planning, materials locations, and other facets that influence the final alignment.

   Basic engineering geology considerations involve a thorough review of bedrock stratigraphy and structural features and types of soils derived from bedrock even though the soil mantle is often thin in

METHODS OF INVESTIGATION

BURGESS JCT.— LOVELL ROAD

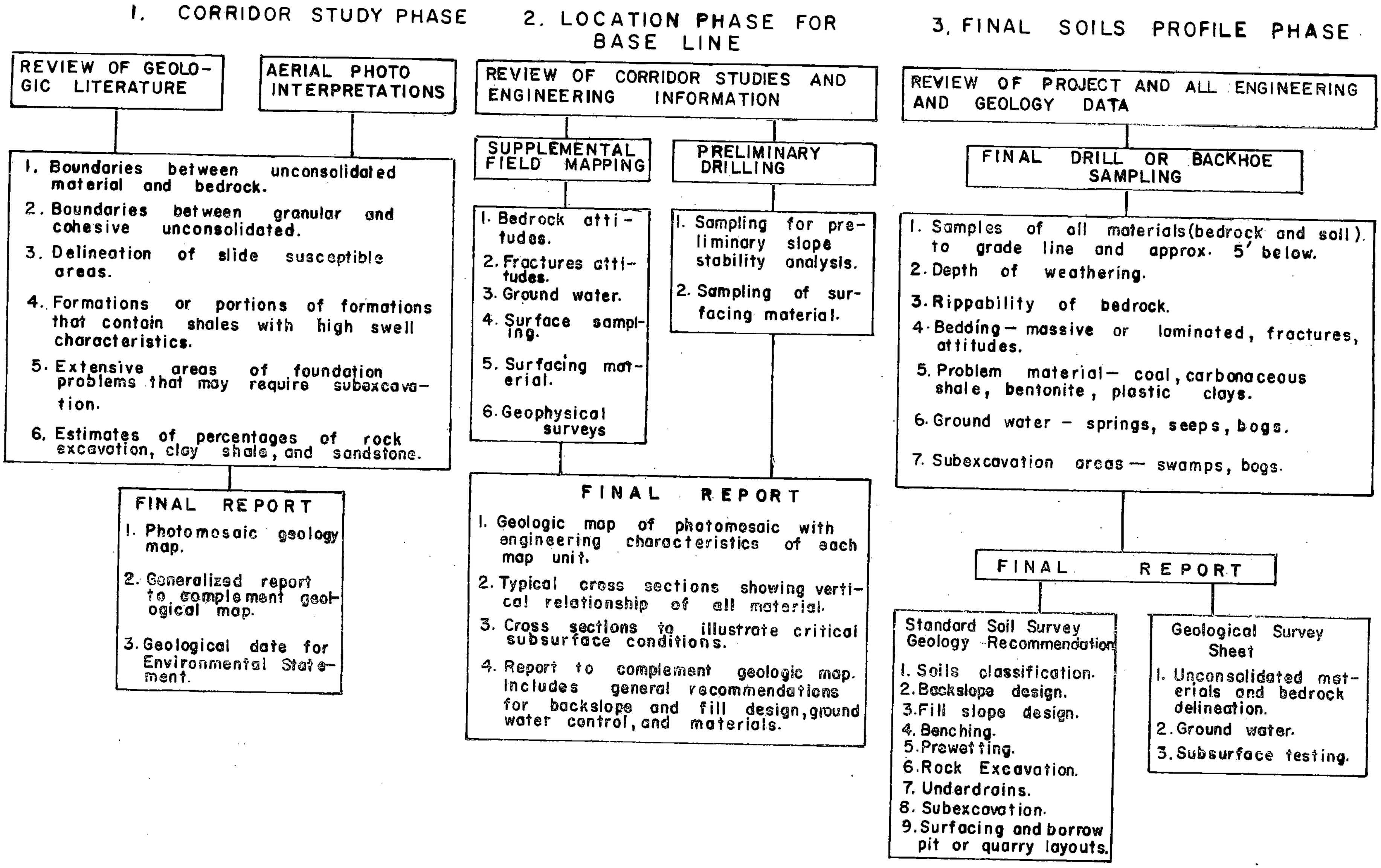


FIG. 3

mountainous terrain. The dominant concern then in geologic mapping in mountain provinces for fulfilling the needs of the highway engineer is the delineation of bedrock units because generally, the consolidated formations, not the soil mantle, control the stability of the terrain.

The generalized geologic map (fig. 4) becomes a useful tool in the corridor selection because it enables the engineer to determine the type of material on which he will construct a highway system. The engineer and geologist can also evaluate potential problem areas that will require subsequent investigations concerning, a) final route selection, b) road design parameters, c) remedial methods for resolving stability problems, and d) environmental considerations.

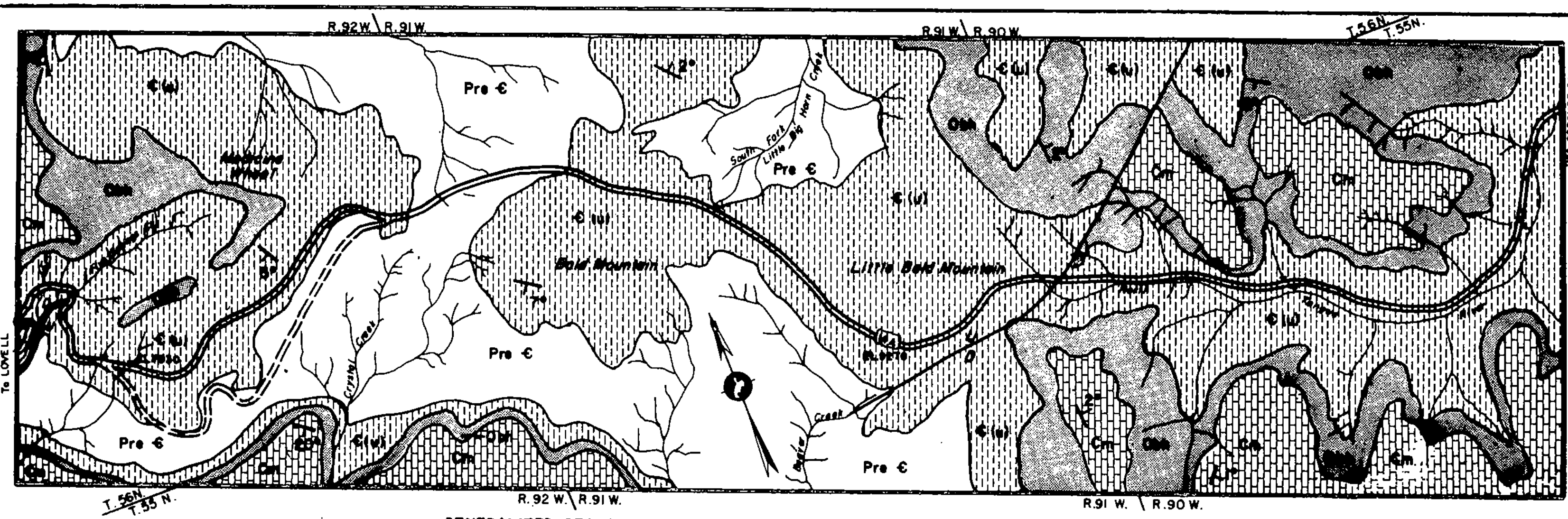
2. Location Phase for Baseline:

The purpose of this phase is to provide geologic data for the route band selection within the corridor. The major task to accomplish this will be detailed photogeologic mapping with supplemental field investigations and preliminary drilling if necessary. Geophysical surveys may be used in this phase.

Special interest in this phase for preliminary road design is directed toward obtaining information concerning, a) bedrock attitudes; b) fracture delineations; c) ground water indications, such as seeps, springs, and bogs; and, d) index properties of soils and also bedrock if applicable; although, a soils classification of bedrock drill cuttings is often misleading. Sampling for preliminary slope stability analysis in potential slide areas constitutes an important part of this phase of the investigation because small line shifts are critical along a narrow route band on steep topography, especially in drainages where nearby streams undercut the slopes.

Certain segments of the alignment in the area of project PSF 6205 of the report are bordered by the North Tongue River and steep, natural ground, and it is in places like this that slide analyses are important from the viewpoint of preventing slope failure or determining corrective measures in case slide failure occurs during construction.

The locations of surfacing material and borrow sources are determined at this time so that final sampling and pit delineation can be done in conjunction with the soils profile investigation; although, sufficient time should be allowed between the preliminary and final pit investigations in the event that additional material sources must be located as dictated by the following requirements; a) adequate material quantities with minimum overburden, b) satisfactory physical properties of the material, c) minimum haul distance, d) workability of the material in the pit, and e) environmental considerations.



GENERALIZED GEOLOGIC MAP OF A PORTION OF THE BIG HORN MOUNTAINS ALONG U.S. HIGHWAY 14A, BIGHORN AND SHERIDAN COUNTIES, WYO.

Scale

0 ½ 1 2 3 4 5 mile

GEOLOGIC FORMATIONS	DESCRIPTION	TOPOGRAPHY	ENGINEERING CHARACTERISTICS AND WORKABILITY	AASHO CLASSIFICATION OF SOIL DEVELOPED FROM BEDROCK
Madison	Thick-bedded Limestone	Escarpments and ridges	Stable, blasting required	Form immature soils of colluvium and Landslide debris. (A-1 to A-7)
Big Horn	Thick-bedded dolomite	Escarpments	Stable, blasting required	
Gallatin Gros Ventre Flathead	Thin-bedded Limestone, & shale Blocky and fissile shale Thin-bedded Sandstone	Steep talus slopes Subdued slopes Subdued slopes	Stable, rippable Unstable, rippable Stable, rippable	Forms rocky talus slopes A-4 to A-7 A-1
Pre-Cambrian	Disintegrated and/or unweathered granite	Subdued slopes, and escarpments	Stable, disintegrated is rippable; massive, blasting required	A-1, A-2, and A-4

SYMBOLS

- Highway 14 alternate
- Highway under construction (Project PSF 6484)
- U/D Fault
- 6° Strike and dip of bedrock

FIGURE 4

ENGINEERING GEOLOGY INVESTIGATION
BURGESS JUNCTION-LOVELL RD.
Projects: PSF 6484, 6204, 6205 and 5897
Wyoming State Highway Department
Cheyenne, Wyoming
March, 1973

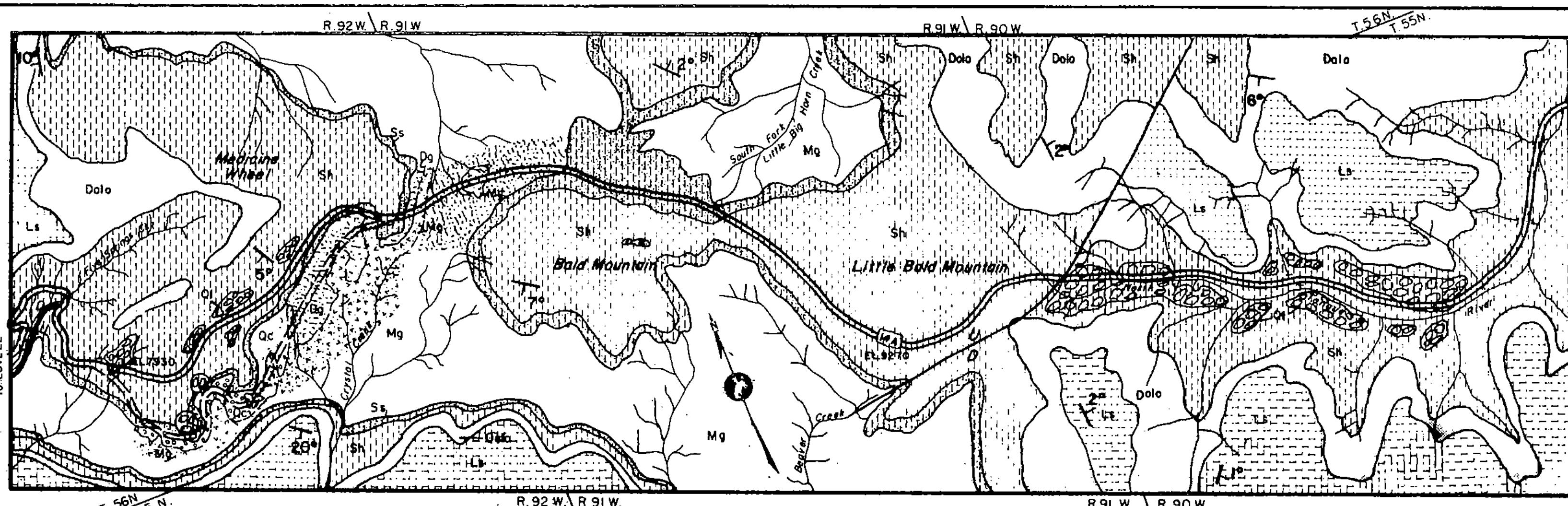
Investigation for quarryable surfacing material may require test blasting to determine the breakability of the rock and the presence of schist, phyllites, soft limestone, and vuggy dolomite. These materials are unsuitable for plant mix and can cause pavement failure in mountain roads in the presence of high moisture conditions and great temperature variations.

The final report for the location phase for baseline includes a geologic map (fig. 5) and cross sections (if necessary) showing individual lithic units based on their engineering characteristics; i.e., rippable shale, siltstone or sandstone, swelling type clay shale, or massive hard rock requiring drilling and blasting and other types of material with deleterious physical properties that may affect the integrity of the highway system.

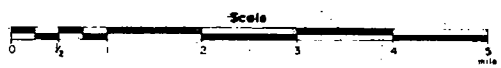
General recommendations are given in the written report for backslope design, fill heights, special excavation, and location of material sources. Ground water control is also discussed so that the design engineers become acquainted with the water problems that will require underdrains, or other methods of lowering the water table to maintain stable fill foundation and backslope conditions, and possible shifts in the alignment.

3. <u>Final Soils Profile Phase</u>:

Detailed drill or backhoe sampling is done in this phase to obtain information for final surfacing and road template design and design of underdrains if required. Profile data is also used by the field engineer during construction; although, soils classification of bedrock units can be misleading if soils and bedrock are not differentiated. Furthermore, the standard soil survey sheet (fig. 6a) does not show ground water which is a critically important factor in road construction in mountainous terrain. For these reasons, geological survey sheets (fig. 6b) are included with the final soils profile so that soils and bedrock limits and ground water level can be shown. In the example in figure 6b, the field engineer has an opportunity to determine potential problem areas and caution the contractor that landslide material will be encountered. The contractor, in turn, can proceed with his operations knowing that precautions must be taken when cutting or filling in this section of the project. The type of excavation (common or hard rock) is also shown on the geological survey sheet. The accuracy of the soil/bedrock limits on the geological survey sheet depends mostly on the drilling density. The amount of final drilling necessary to illustrate detailed subsurface conditions is determined after completion of the location phase for baseline in order to establish an economical and objective drilling program. Supplemental geophysical surveys may also be made.



GENERALIZED GEOLOGIC MAP OF A PORTION OF THE BIG HORN MOUNTAINS
ALONG U.S. HIGHWAY 14A, BIGHORN AND SHERIDAN COUNTIES, WYO.



SYMBOLS

- Highway 14 alternate
- Highway under construction (Project PSF 6484)
- U / D Fault
- 6° Strike and dip of bedrock

MATERIAL AND MAP LEGEND

SYMBOL	DEPOSIT	DESCRIPTION	ENGINEERING CHARACTERISTICS	AASHO CLASS	γ	C	Φ
Qc	COLLUVIUM	UNC. SILT, SAND, CLAY & ROCK FRAGS., COBBLES & BOULDERS.	MOD. STABLE; GOOD TO POOR DRAINAGE; EXCAVATION WITH DOZER.	A-1 to A-7	122–136	306–1640	16°–45°
Ql	LANDSLIDE	UNC. CLAY & ROCK FRAGS., GROS VENTRE SHALE.	VY. UNSTABLE IF SATURATED; EASILY SLIDES IN CUT SLOPES STEEPER THAN 1:1; EXCAVATION WITH DOZER.	A-4 to A-7	109–129	829–1520 w/S.F.= >1.	0°
Ls	LIMESTONE (MADISON)	THICK-BEDDED, HARD LIMESTONE, OCC. CAVERNS.	STABLE; ROCK FALL & SLIDES IN CUT SLOPES STEEPER THAN 1/2:1; BLASTING REQUIRED.	BEDROCK	160	NONE	45°
Dolo	DOLOMITE (BIG HORN)	THICK-BEDDED & MASSIVE DOLOMITE, VUGGY.	STABLE; ROCK FALL IN CUT SLOPES STEEPER THAN 1/4:1; BLASTING REQUIRED.	BEDROCK	155	NONE	35°
Sh	SHALE (GROS VENTRE)	LAMINATED & THIN-BEDDED CLAY SHALE.	VY. UNSTABLE WHEN SATURATED; IN CUT SLOPES STEEPER THAN 1:1; EXCAVATION WITH DOZER.	BEDROCK	124–135	1500–4000	25°
Ss	SANDSTONE (FLATHEAD)	THIN-MED.-BEDDED, POORLY-WELL-CEMENTED.	VY. STABLE; GROUND WATER RESERVOIR; EXCAVATION WITH DOZER & RIPPER.	BEDROCK	145–150	NONE	30°–45°
Dg	DISINTEGRATED GRANITE	LOOSE GRAVEL, OCC. VY. CLAYEY; HD. GRANITE BOULDER; IRREG. CONTACT.	MOD. STABLE; GOOD TO POOR DRAINAGE; EXCAVATION WITH DOZER & RIPPER.	A-1 to A-6	125–130	0–962	10°–25°
Mg	MASSIVE GRANITE	HARD, JOINTED; ZONES OF DISINTEGRATED GRANITE.	STABLE; ROCK FALL IN CUT SLOPES STEEPER THAN 1/4:1, BLASTING REQ.	BEDROCK	170	NONE	45°+

FIGURE 5

ENGINEERING GEOLOGY INVESTIGATION
BURGESS JUNCTION–LOVELL RD.
Projects: PSF 6484, 6204, 6205 and 5897
Wyoming State Highway Department
Cheyenne, Wyoming
March, 1973

STANDARD SOIL SURVEY SHEET — PROJECT: PSF 6205

LABORATORY No.	69-1356	69-1355	69-1354	69-1353						
FIELD NUMBER	G-40	G-26	G-23	G-22						
STATION SAMPLED	454+00	504+50	508+70	510+50						
MAXIMUM SIZE	1"	3/4"	3/4"	1"						
PASSING No. 4	74	68	79	82						
PASSING No. 10	69	55	66	75						
PASSING No. 40	66	49	59	68						
PASSING No. 200	52.7	45	52.3	60.1						
LIQUID LIMIT	38	42	39	42						
PLASTIC INDEX	20	24	22	23						
CLASSIFICATION	A-6(9)	A-7-6(5)	A-6(7)	A-7-6(10)						
"R" VALUE		5	6	6						

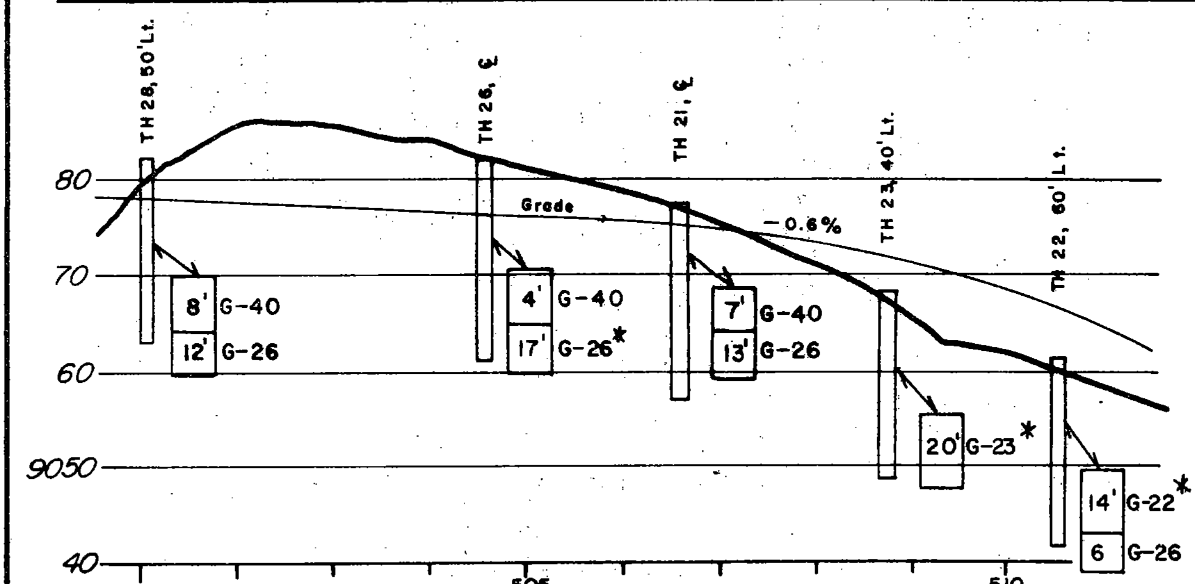


Fig. 6a

GEOLOGICAL SURVEY SHEET — PROJECT: PSF 6205

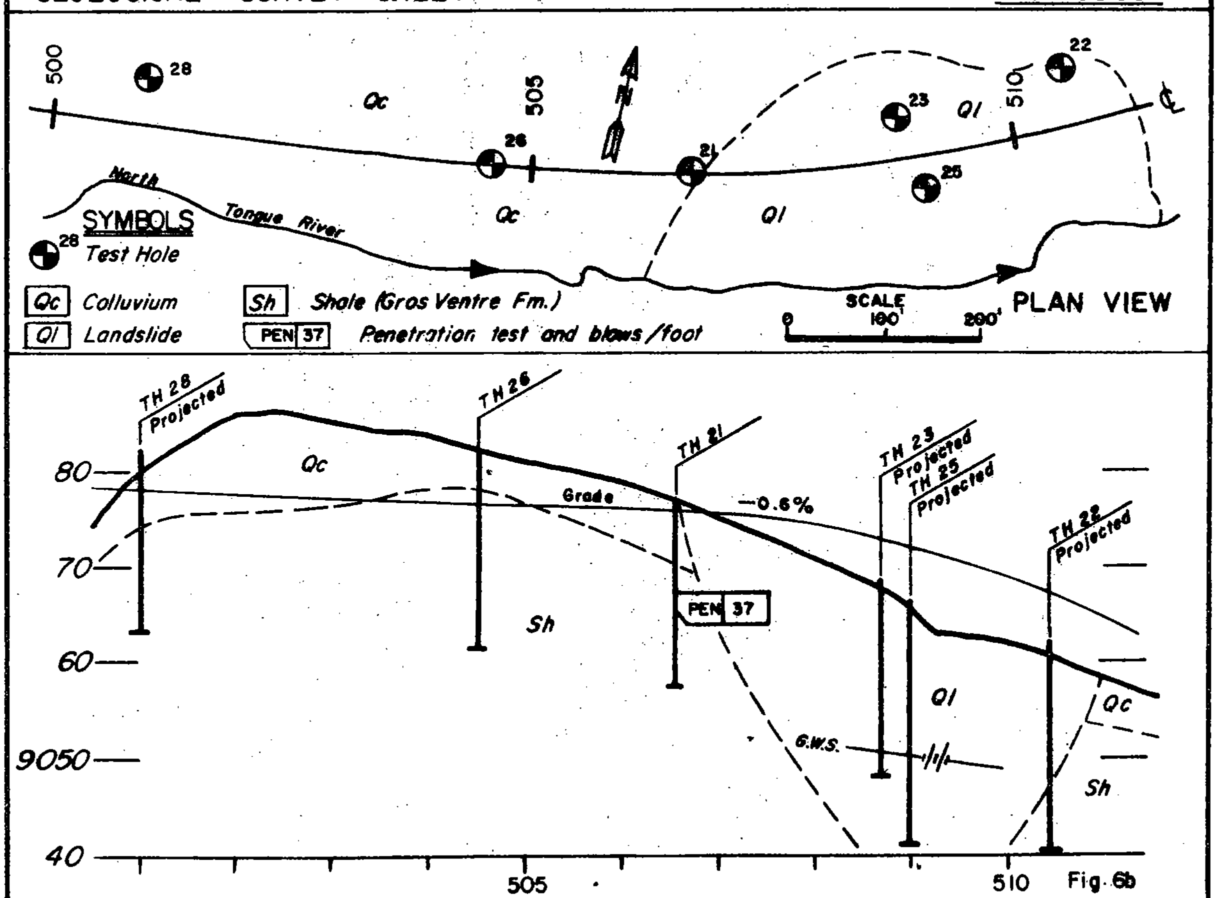


Fig. 6b

Figure 6

The final report furnishes the design engineers with all the information required for the final plans, concerning, a) backslope and fill slope design, b) benching design, c) underdrain design, d) subexcavation areas, and e) surfacing and borrow pit layouts. Special recommendations for moisture-density control (prewetting) and rock excavation are also included.

CONCLUSIONS

The end product of engineering geology investigations is a highway system that has been built to modern standards without sacrificing its usefulness by numerous road failures that impede traffic, risk lives, adversely affect the environment, and cause an undue strain on the maintenance budgets of highway departments.

The application of the methods of engineering geology investigations in mountainous terrain discussed in this report is no guarantee that road failures will not occur, but it does minimize the risk of failures in the highway system. An inherent problem in the analysis of slides is the difficulty in giving a value of shear strength to a mass of highly fractured clay rock, such as the Gros Ventre shale. Laboratory tests of this material do not always produce reliable results because the test specimen may have fewer fractures than the mass of inplace material. Thus, the shear strength obtained in the laboratory may be considerably higher than the actual shear strength of the mass of shale. Average shear strength in the shale from at least six penetration probes in a local area may be reliable providing this information can be checked with shear strengths obtained from cores. In general, shear strengths from penetration tests in fractured shale tend to be lower than shear strengths obtained by other methods.

The intensively jointed Gros Ventre shale is much less competent than unjointed shale, and even without the presence of ground water, the shale is susceptible to slumping on backslopes that exceed a design of 1:1 and in steep-sloped fill foundations if the fill height is excessive. General limitations in the fill height relative to the slope of the fill foundation in Gros Ventre shale are listed as follows:

Foundation Slope	Fill Height
Steeper than 2:1	8 feet or less
2:1 to 3:1	8 to 10 feet
3:1 to 3.5:1	10 to 15 feet
3.5:1 to 5:1	15 to 30 feet
Less than 5:1	30 feet or more

If fill heights exceed the above limitations, then safety factors would be less than the minimum of 1.5 used as the lower limit of risk of slide failures. For example, a 40 foot fill on a 3:1 slope would produce a safety factor of 0.43, well below the 1.5 value.

The degree of saturation is a variable factor that must be considered in the final analysis of slide failures because it is related to seasonal moisture conditions as well as joint intensity. The Gros Ventre shale is less slide susceptible during late summer, fall, and winter than in spring and early summer when moisture conditions are at maximum levels. Experience has shown that in mountainous terrain it is best to analyze slide characteristics of the Gros Ventre shale under worst conditions (lowest cohesive strength) obtainable in a given region in order to compensate for saturation variability. After the natural slope has been disturbed in cut sections, snow melt and rain water readily enters into fractures in the shale and rapidly lowers the cohesion of the material (generally at the toe of the slope) so that backslope failures can occur during or shortly after construction. Slide analysis under worst conditions in the design stage will necessitate the following remedial measures to lower the risk of slide failures; a) minimum backslope design, b) design of adequate underdrains and surface drains to lower the ground water table and minimize surface runoff onto cut slopes, and, c) shift of the alignment to avoid potential slide problem areas.

Geophysical Surveys:

Electrical resistivity surveys were made in the report area to determine the thicknesses of disintegrated granite and colluvium in lieu of detailed drilling. The results of the electrical surveys were satisfactory if sufficient moisture was present in the material to conduct an electrical current. Surveys in material with dry void spaces were entirely unreliable.

Resistivity methods in Gros Ventre shale slide areas produced promising results. The rotational slide shown in figure 7 developed during construction after the backslope was formed. Drilling on the slide block was confined to a small dozer trail cut in the lower part of the slump above the road. Electrical resistivity surveys were made as a supplement to drilling to delineate the slide plane.

Figure 8 is a cross section of the slide at stations 500-510, project PSF 6205. Resistivity surveys and test hole locations and data obtained during the investigation are shown on the cross section. Resistivity breaks are indicated in each survey. These anomalies were caused by electrically resistant media within the shale mass, and they may represent either fresh water zones, limestone beds, or possibly air-filled voids or fractures. The utilization of the data from the electrical surveys is an interpretative concern of the engineering geologist who must apply geologic knowledge and related phenomena in this interpretation. From field observations, the slide plane is defined at the surface along the escarpment of the upper perimeter of the slide and at the slide toe near the roadway centerline. Completion of the slide plane in the subsurface is attained by connecting the resistivity breaks, drill data and surface points as illustrated in figure 8.

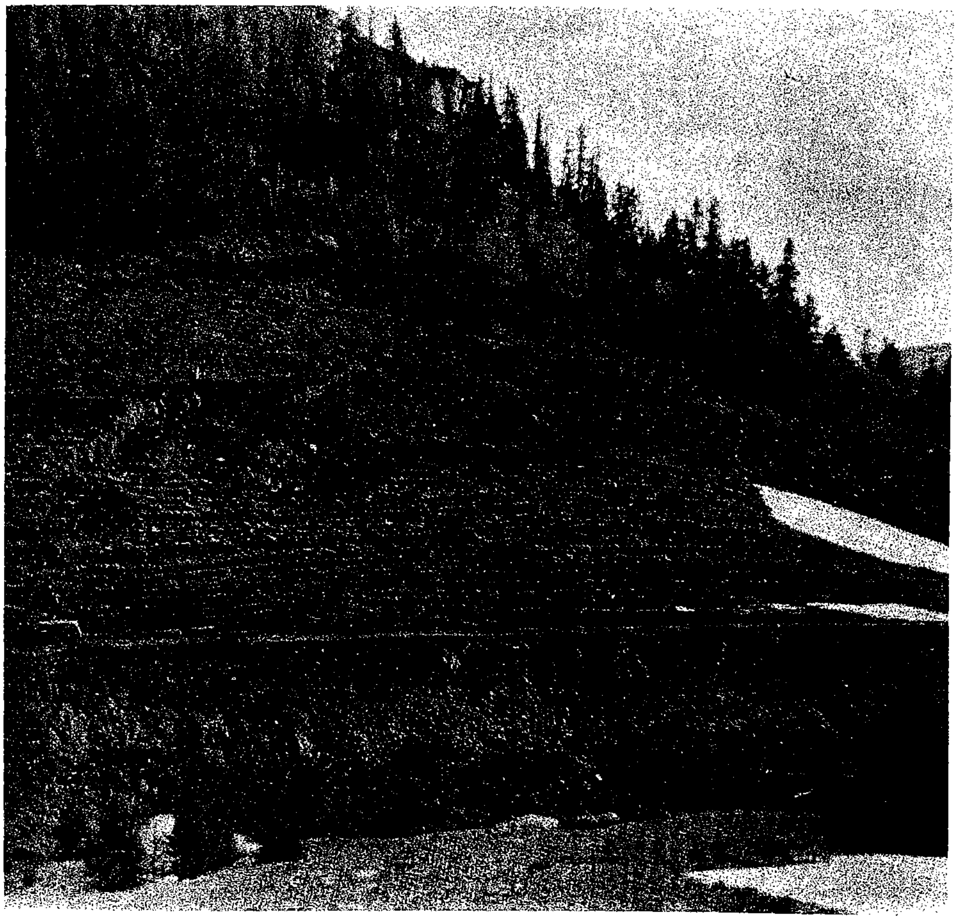
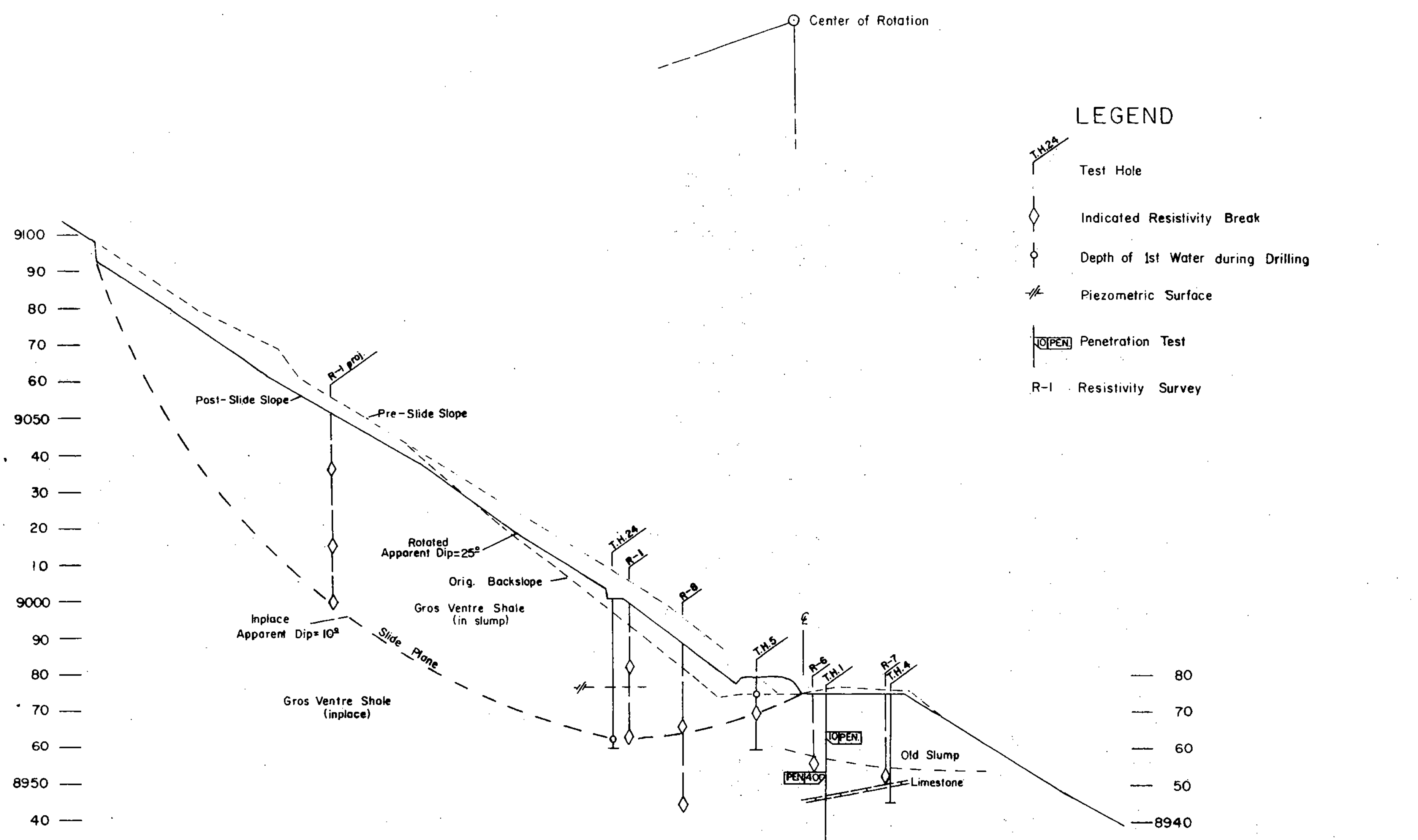


Fig. 7 Rotational slide in Gros Ventre shale at stations 500-510, Project PSF 6205

Seismic methods were not employed in the investigations. Seismograph surveys are useful to differentiate between rippable material and massive, hard rock that would require drilling and blasting.

Communications:

Construction of highways in mountainous terrain, more than anywhere else, requires a close working relationship between design and construction engineers and the engineering geologists. A breakdown in communications between these technical groups can lead to serious consequences especially if plans are finalized, contracts are let, and construction is started. Small line shifts and vertical changes in the alignment are critical where steep, natural slopes; poor soils; and ground water are all present. In these areas, all line adjustments require additional stability analyses and often final recommendations based on experience in nearby construction projects.



SLIDE INVESTIGATION
STATIONS 500-510
BURGESS JCT.-LOVELL
PROJECT PSF-6205
WYOMING HIGHWAY DEPARTMENT
FIGURE 8

Finally after construction, the entire project, especially problem areas, should be reviewed in detail so that knowledge gained can be applied to subsequent projects.

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INVESTIGATION OF FAILING CONCRETE IN HOUSTON, TEXAS, CAUSED BY UNSOUND CEMENT

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Texas Highway Department

Abstract

Physical, chemical and petrographic studies, together with on-site inspections, have been utilized in delineating the conditions present in structural concrete which has failed as a result of unsound cement. Concrete samples representing three structures at two separate highway projects in Houston, Texas, had irreparable damage and evidence of internal distress soon after being placed. Portland cement, routinely sampled from shipments supplied to these projects and submitted to the Materials and Tests Division for physical testing, failed to meet the soundness requirements. A total of seven samples, all produced at a portland cement plant in Houston, Texas, during April and May of 1972, exhibited expansion which ranged from 2.5 to 20 percent by the autoclave test. Chemical analysis showed that these unsound cements contained excessive amounts of uncombined calcium oxide. Additional testing demonstrated that the relative unsoundness of the cement decreased with aeration; however, briquettes cast with the cement which showed the highest range of expansion were found to be unsound by the autoclave and steam-bath tests even after 5 months of open-air storage. Petrographic studies on 4-month-old concrete found no free lime present; however, evidences of unaccommodative chemical reactions were observed along with an extensive system of microcracks in the paste.

I. SUBJECT

Several job samples of portland cement, produced at a portland cement plant in Houston, Texas, and submitted during April and May 1972 to the Materials and Tests Division Laboratories for routine physical tests, failed to meet the soundness test as outlined in ASTM Designation: C 150. In addition, failing concrete made with this unsound cement has been removed from highway projects located in the Houston area. The formal documentation of the studies made on the cement and samples of the concrete removed from structures is the subject of this report.

II. PURPOSE

The purpose of this report is to make available to departmental personnel the results of a series of laboratory studies, which include physical, chemical and optical as well as field observations, made on unsound cement and associated failing concrete from Texas Highway projects in Houston.

III. CONCLUSIONS

1. The extensively-cracked concrete observed on three structures at two separate highway projects in Houston failed as a result of using unsound portland cement.

2. A total of seven cement samples produced at a portland cement plant in Houston, Texas, contained uncombined calcium oxide and exhibited autoclave expansion results ranging from 2.5 to 20 percent.

3. Petrographic studies on the concrete removed from the structures identified extensive systems of microcracks in the paste fraction as well as exudations of secondary mineral compounds within the internal void system; however, traces of free calcium oxide could not be determined in the concrete by conventional optical methods.

IV. MATERIALS AND METHODS

A. Location, Field Observations and Samples.

The two construction sites from which concrete was removed and examined for this report are located within the city of Houston as indicated by the general highway map in Figure 1.

One site, a District 12 project, located in the western part of Houston on US 59 (C-27-13-65, PD0028) consisted of a foundation for a retaining wall situated at an approach to the West Belfort Overpass. The other site, a Houston Urban project (I-610-7(189) 782, 271-15-8) in the eastern part of the city consists of a center slab of a 3-span continuous unit on structure 351, ramp "A", at the multilevel interchange on Loop 610 and SH 225. Concrete samples taken from a pier on structure 346, ramp "C", of the same interchange were also collected for this study.

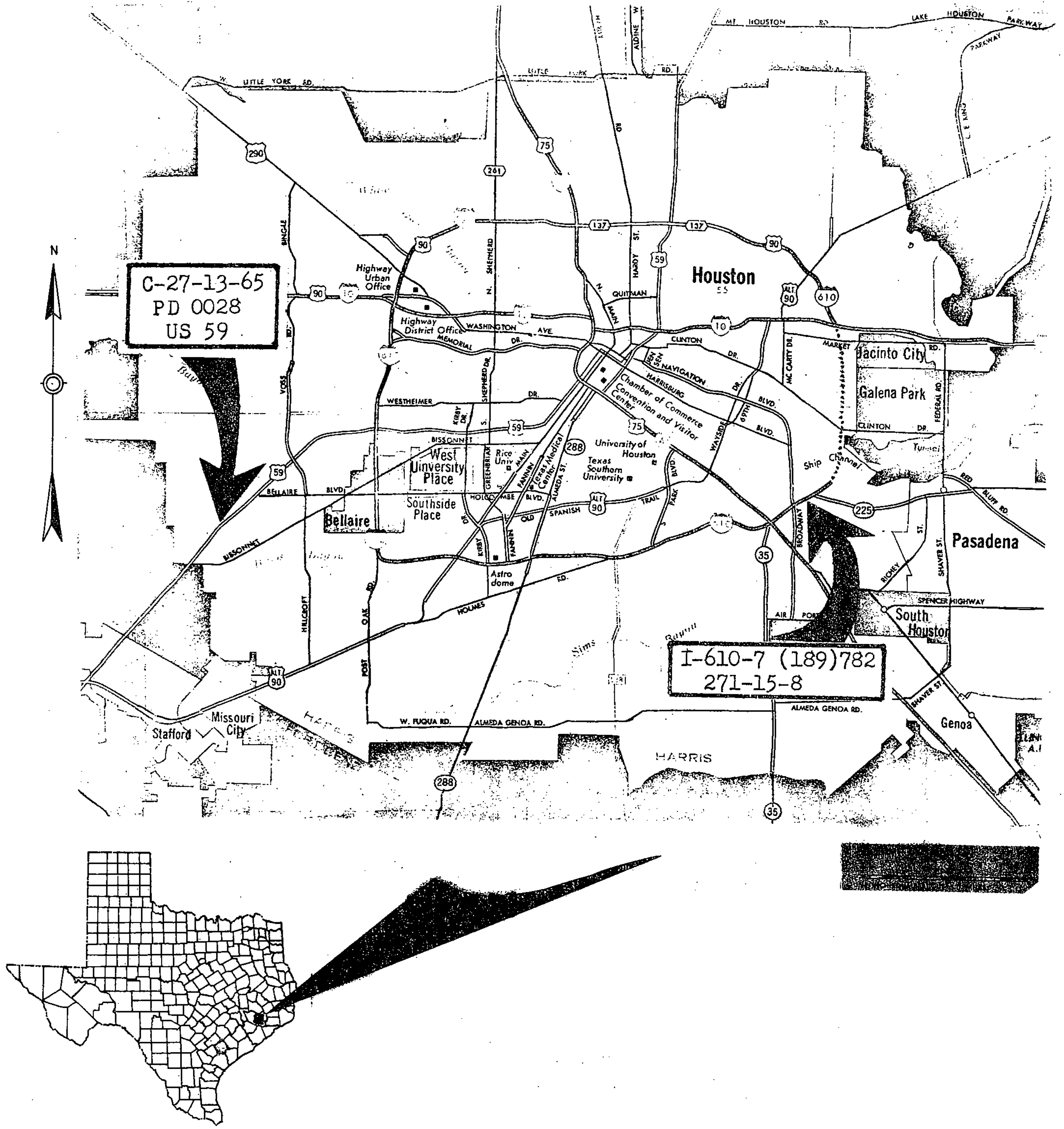

Fig. 1 General highway map of Houston showing location of structures examined for this investigation.

Field observations at the project sites were conducted by District 12 and Houston Urban personnel. Photographs and pertinent data, along with concrete samples removed from the retaining wall foundation and both cores and large pieces from the bridge deck, were submitted to the Materials and Tests Division for examination. Results of compression tests conducted by a commercial testing laboratory on cores taken from the bridge deck and on test cylinders, in addition to the Concrete Design Worksheet that was used in the batching operation, have been recorded and placed on file.

Portland cement samples taken according to standard procedures by departmental personnel and submitted for routine physical tests have been retained after testing by the Materials and Tests Division for extended physical testing, chemical analyses and petrographic studies. The cement samples taken from transport trucks by inspectors and shipped in 1-gallon buckets are identified under Laboratory Numbers 72-2321-D, 72-2349-D, 72-2420-D, 72-2423-D, 72-3137-D and 72-3603-D, all of which are Type I, and 72-2816-D, a Type III. Each were produced at a Houston portland cement plant during April 1972.

B. Chemical Analyses.

When routine physical tests indicated that a cement sample from a Houston cement company was failing to meet specification requirements for soundness, a representative amount was submitted to the Materials and Tests Division Chemical Section for chemical analyses. Sample 72-2321-D, which exhibited significant expansion in the Autoclave Test, and a "normal" sample from the same producer were chemically analyzed for the free lime content as outlined in ASTM Designation: C 114, "Chemical Analysis of Hydraulic Cement." In addition, the percent loss on ignition (LOI) and the percent magnesium oxide were determined; the first by the ASTM method, the latter by a Materials and Tests Chemical Section procedure (See Appendix II). The sulfur trioxide (SO_3) content of the unsound cement was determined by the Materials and Tests Cement Section by means of a Wagner turbidimeter (ASTM C 114). For comparative studies a duplicate sample was chemically analyzed at a Fort Worth laboratory by means of X-ray methods.

C. Physical Tests.

A number of routine physical tests are performed on all cement samples submitted to the Materials and Tests Division and, if a sample shows abnormal results, additional tests are often applied.

One of the basic routine tests performed on the subject cement samples, which are required on all cements proposed for use in concrete pavements or structures, is outlined in ASTM Designation: C 190, "Tensile Strength of Hydraulic Cement Mortars." This test employs the briquette specimen cast with the standard Ottawa sand.

Another routine test method utilized for this report is the "Fineness of Portland Cement by the Turbidimeter," ASTM Designation: C 115. This standard method covers the determination of the grind size as represented by a calculated measure of specific surface, expressed in square centimeters of total surface area per gram of cement, using the Wagner turbidimeter.

The third, and no doubt the most significant, physical test applied for this study, provides an index of potentially delayed expansion caused by the hydration of calcium oxide (CaO) and/or magnesium oxide (MgO). This method, outlined in ASTM Designation: C 151, "Autoclave Expansion of Portland Cement," involves subjecting a neat-cement prism to steam pressure at 295 psi (about 420°F) for 3 hours. The prisms are cast and placed in moist chambers for one day, then taken out and measured, after which they are placed in the autoclave and subjected to the test. After initial cooling, the pressure was released, the specimens were removed and placed in 194°F water, cooled gradually to 74°F, then dryed and measured. Calculations were determined and expressed as a percentage of length-change.

Two additional tests on the unsound cement were used for informational purposes; namely, the pat-boiler test and aeration. The former is actually a discontinued (Since 1947) ASTM Standard Method, "Soundness of Hydraulic Cement Over Boiling Water," ASTM Designation: C 189-44, and the latter is simply a technique utilized to gain information on relative hydration rates of cement samples containing unhydrated components. This observational test involved allowing the raw cement to be exposed to the air for various lengths of time and repeating some of the above listed tests and noting any differences in results.

D. Petrographic Studies.

A number of petrographic techniques and methods were utilized in examining the samples obtained for this investigation and each employed the use of a binocular stereoscopic microscope and/or a polarizing microscope.

Representative pieces of concrete from each examined structure were sliced, ground to a smooth finish and examined under a microscope by the linear traverse technique as outlined in ASTM Designation: C 457, "Microscopical Determination of Air-Void Content, Specific Surface and Spacing Factor of the Air-Void System in Hardened Concrete." During this examination, a survey of microcracks in the paste was taken in addition to noting the presence of bleed channels and secondary mineral deposits within the air voids.

Highly-polished sections processed from the concrete samples were examined at magnification ranges up to 1000X in order to determine the relative extent of cement hydration. The general condition of the aggregate-paste bond and other physical features of the cement paste such as hardness, granularity, porosity and color were observed from freshly broken pieces.

Powder mounts of the cement using immersion oils of different refractive indexes were devised for making additional observations with polarized light under high magnification. The cement was also examined by means of thin-sections of the concrete (ground to less than 25 microns), however, this technique was found to be inferior to the polished-section method of preparing sliced pieces of the concrete. The latter method included polishing the smoothed surface to a mirror finish for noting the composition, fineness, extent of hydration and the dispersement of relic cement particles. Transmitted light was utilized for examining thin-sections and oil-immersion slides; whereas, reflected light was necessary to examine the polished sections.

Additional powder mounts of the unsound cement were subjected to a "quickie-test" treatment as an additional optical verification of free lime. The test, described by Lea (1956)*, consists of placing a small amount of the cement on a glass microscope slide and adding a drop or two of White's reagent; a solution of 5 g of phenol in 5 cc of nitrobenzene to which two drops of water are added. After stirring, a cover-glass is pressed lightly onto the mixture. The slide is then examined under crossed polarized light at a magnification of about 125X. If free lime is present in the cement, long needles of calcium phenoxide form within a few minutes.

V. RESULTS

The condition of the deck surface and cores examined for this investigation, documented by personnel from the Houston Urban Office, can best be seen in the photographs taken during their on-site inspections during May 1972. The photos, shown as Figures 2-7, illustrate the distress cracks, which occurred in the unstable concrete as a result of the incorporated unsound cement, as well as show the condition of the cores removed from the deck. As can be seen, the crack patterns occur in localized areas a few feet across and sometimes show lineation when influenced by the underlying reinforcement.

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*Lea, F.M. "The Chemistry of Cement and Concrete," Edward Arnold Publishers LTD., London (Revised Edition). 637 pp., 1956.

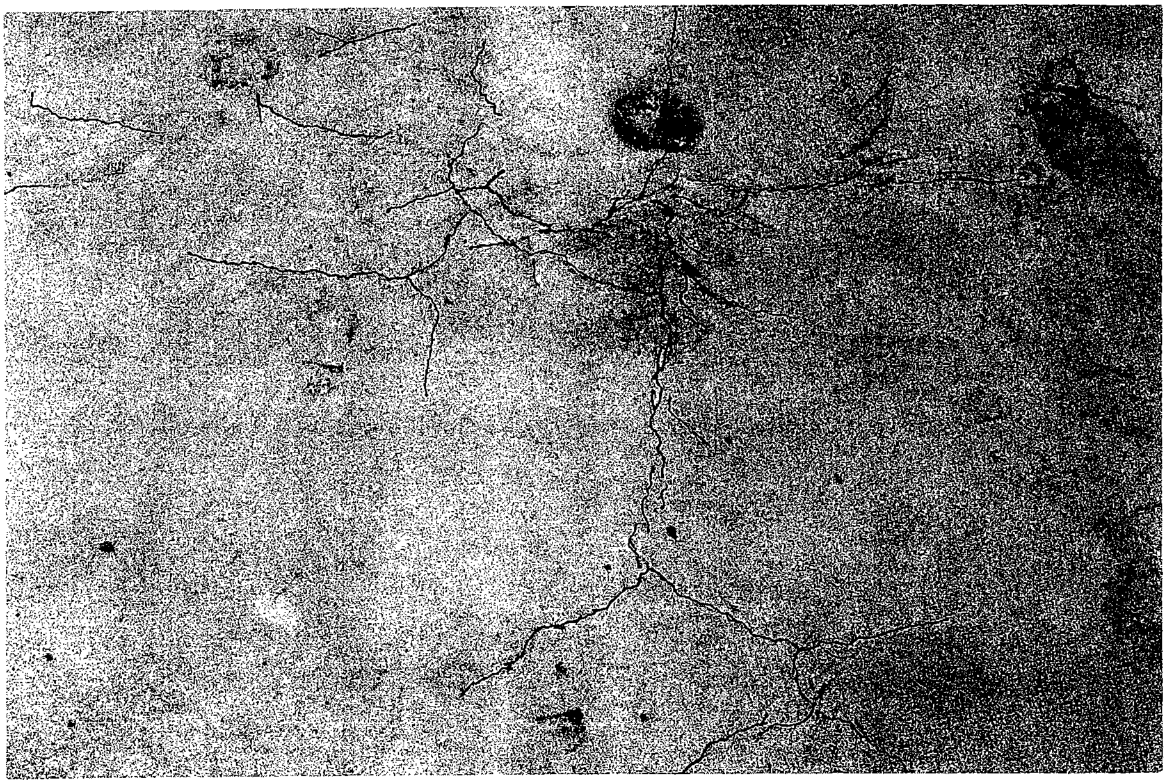

Fig. 2

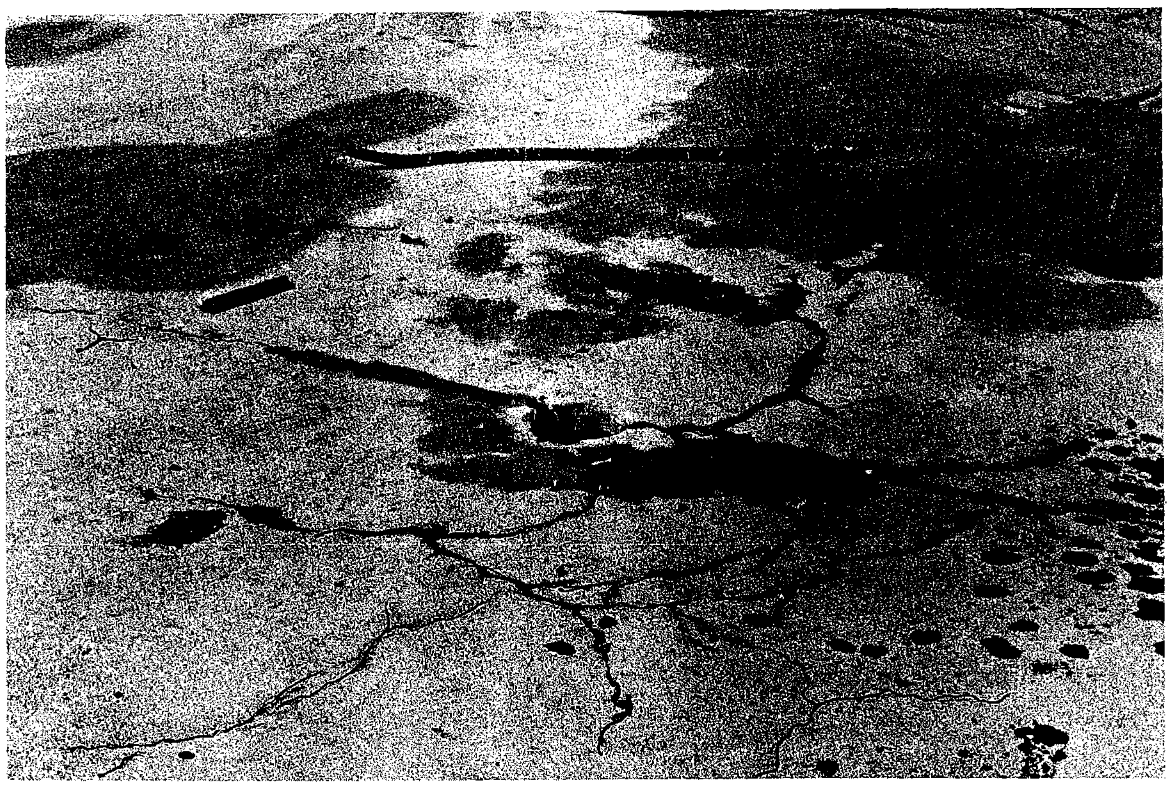

Fig. 3

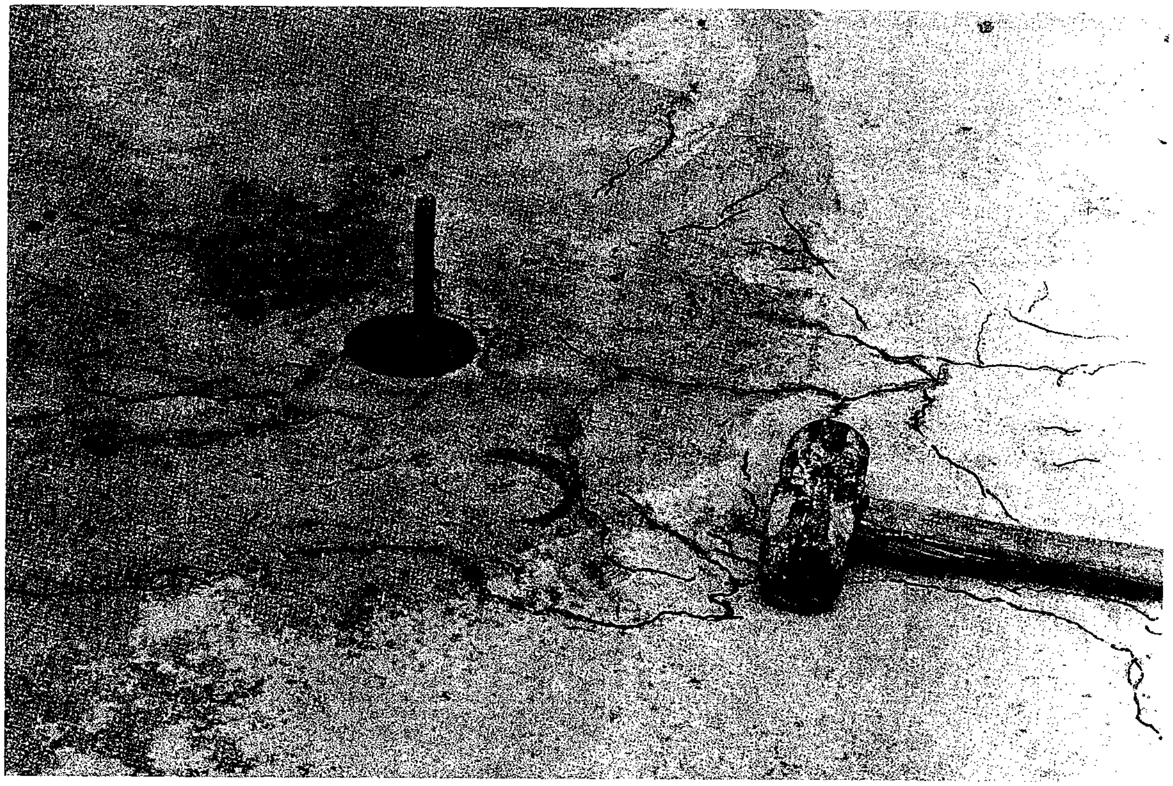

Fig. 4

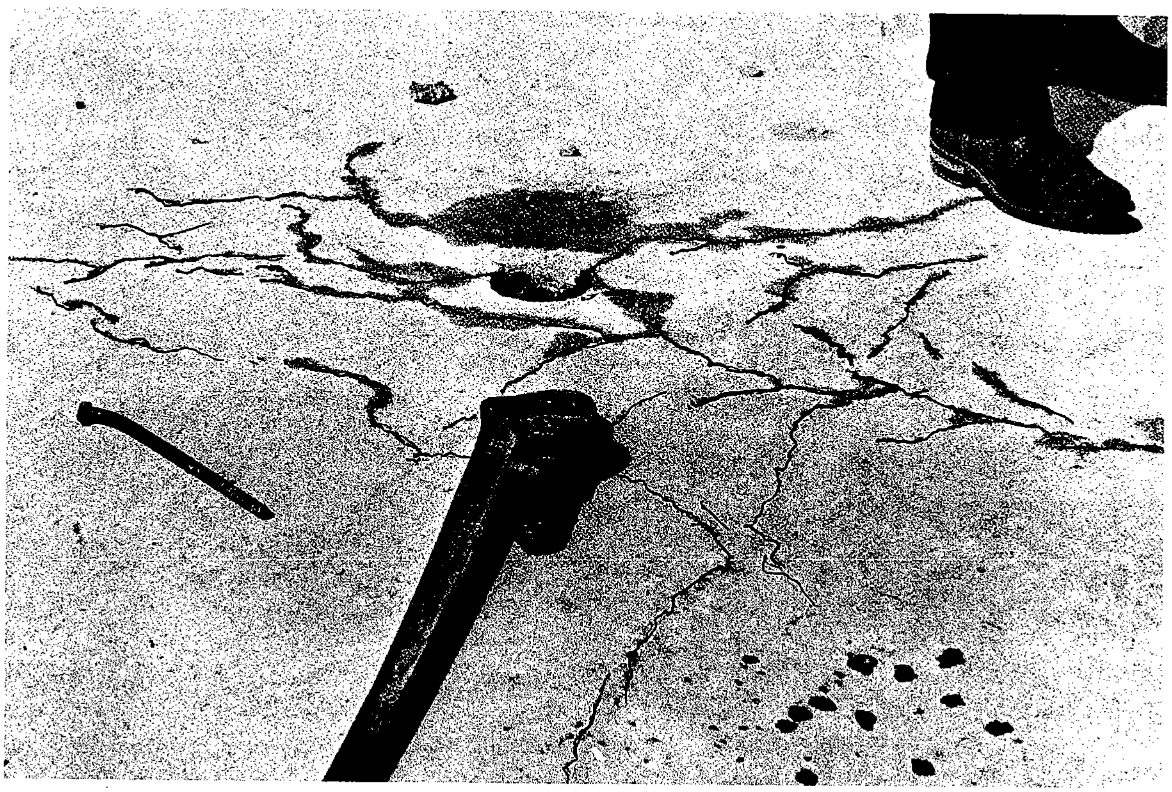

Fig. 5

Fig. 6

Fig. 7

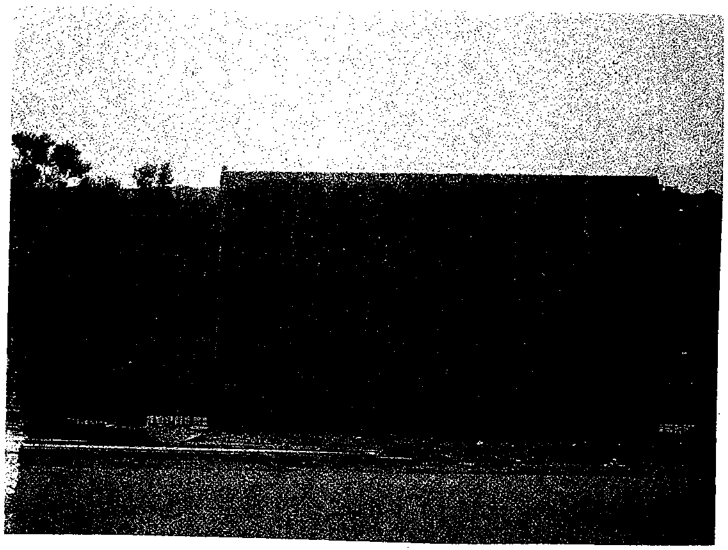

Fig. 8 View of retaining wall at the west Belfast overpass and US 59.

Fig. 9 View of retaining wall foundation before removal showing distress cracks. Scale is indicated by pocket knife.

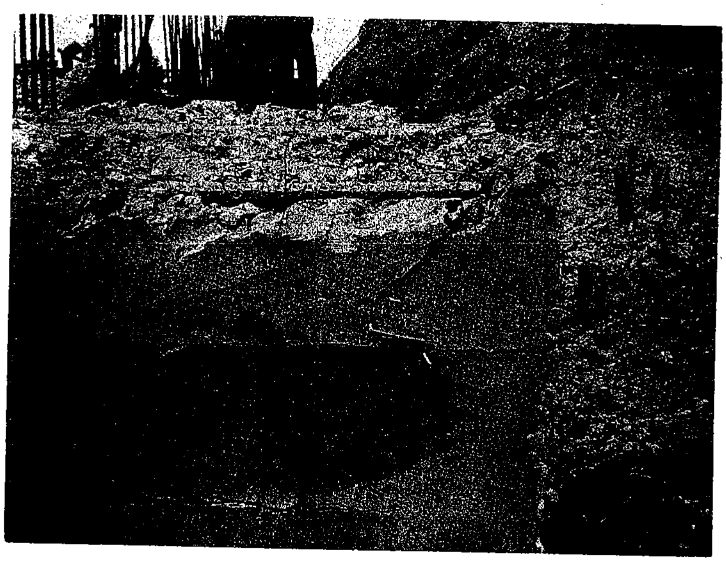

Fig. 10 End of foundation section which measured 24' long, 8½' wide and 2½' deep. Note distress cracks and lack of aggregate-paste bond. Scale is indicated by pocket knife.

Fig. 11

Photos showing retaining wall foundations also shown in Figures 9 and 10. Distress cracks are indicated. Scale is indicated by pocket knife.

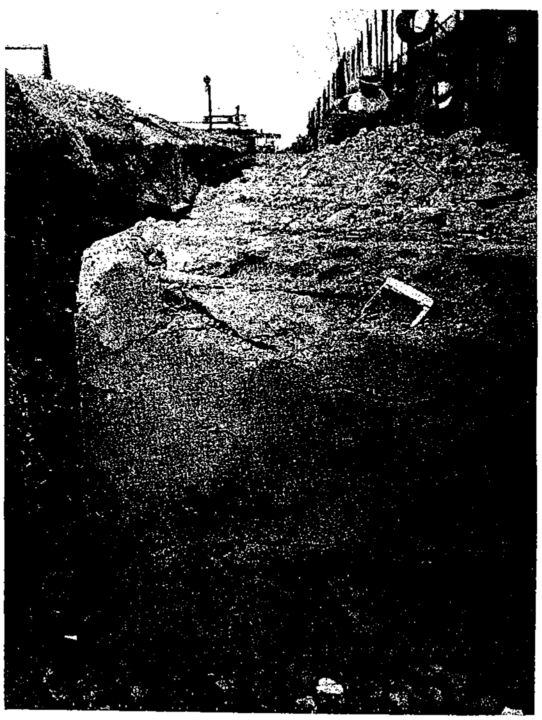

Fig. 12

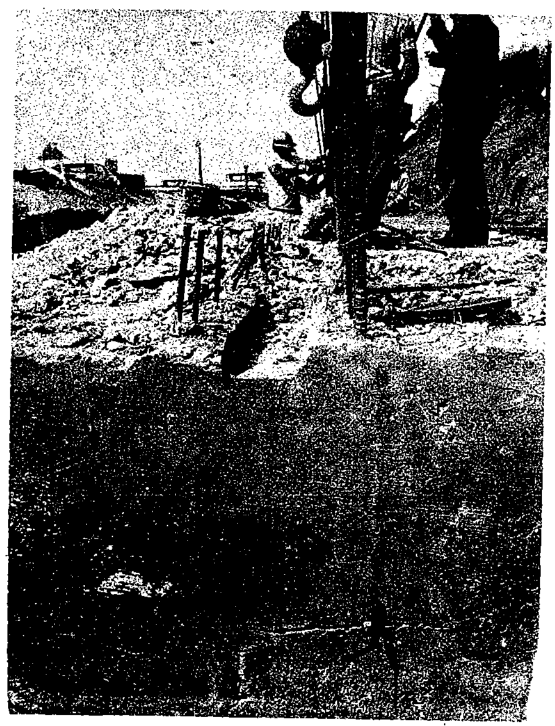

Fig. 13

Photographs of the affected retaining wall foundations at the West Belfast Overpass and US 59, taken by District 12 personnel, are shown as Figures 8-13. In addition to showing the cracked and spalled condition of the concrete, Figures 9, 10 and 12 show that a weak aggregate-paste bond is prevalent. Two of the 24' long sections cast with the unsound cement were removed soon after these photos were taken.

Photographs were not taken of the failing column at the I-610 interchange, however, visible cracks were observed when the samples were received in the Materials and Tests Laboratory. Details of the microscopic studies are included in the petrographic analysis.

Results of partial chemical analyses obtained on the initial cement sample (Lab #72-2321-D) which showed to be unsound by the autoclave expansion test are summarized in the following Table I. For comparative purposes, another sample of Type I cement from the Houston plant, which showed only 0.078% autoclave expansion, was analyzed and the results are also listed.

Table I

Sample #	72-2321-D	72-1827-D
Free Lime (CaO) - Run #1	8.31%	0.77%
Run #2	8.34%	0.77%
Magnesium oxide (MgO) - Run #1	1.08%	1.64%
Run #2	1.14%	1.64%
Loss on Ignition (LOI)	2.62%	0.08%
Sulfur Trioxide (SO_3)	2.41%	*

*Results Undetermined

Additional chemical data was collected when a portion of sample #72-2321-D was taken to a Fort Worth cement plant and analyzed by X-ray methods. The oxides of silica, aluminum, iron and magnesium were found to be normal; whereas, the calcium oxide content was found to be too high.

During the course of this investigation a total of seven cement samples from the Houston cement plant were found to be unsound. A number of routine physical tests were performed on these samples and the results are listed in Table II.

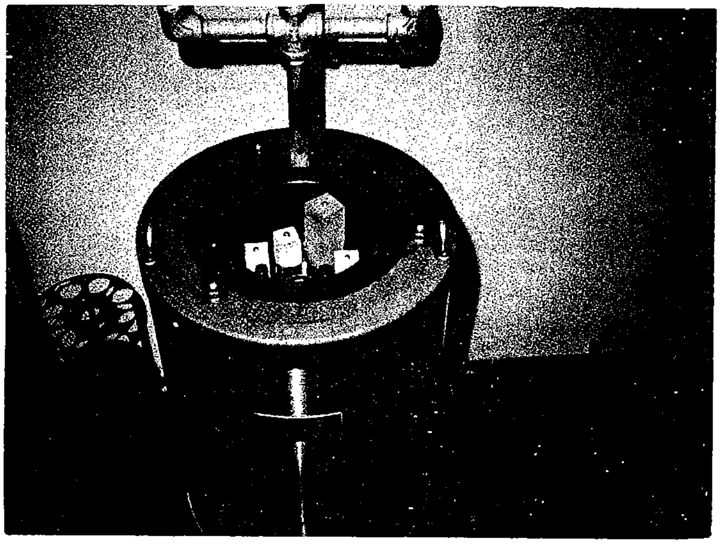

Fig. 14

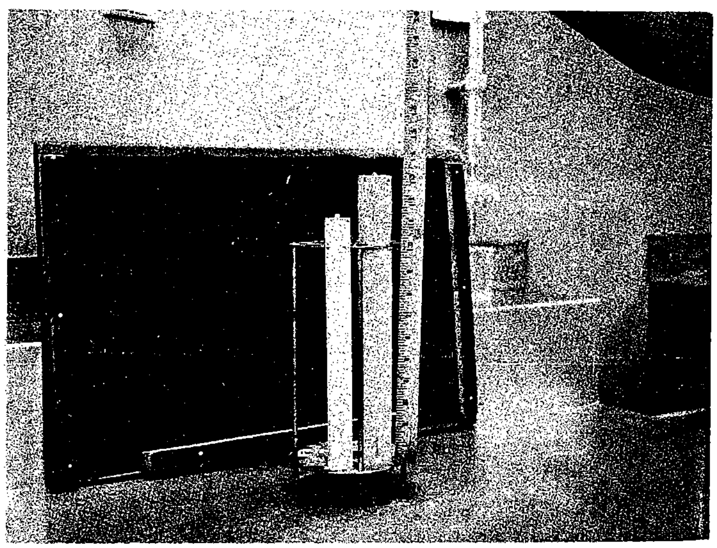

Fig. 15

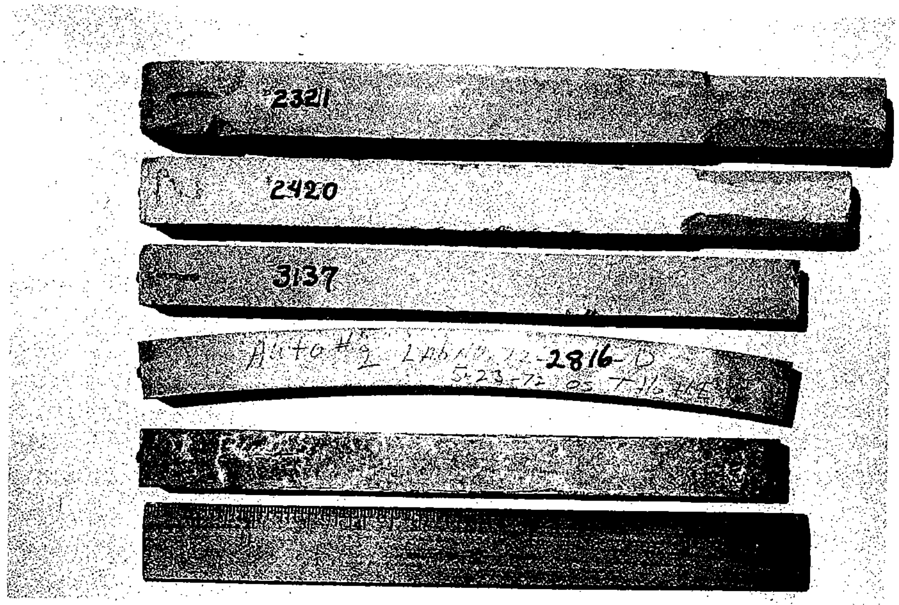

Fig. 16

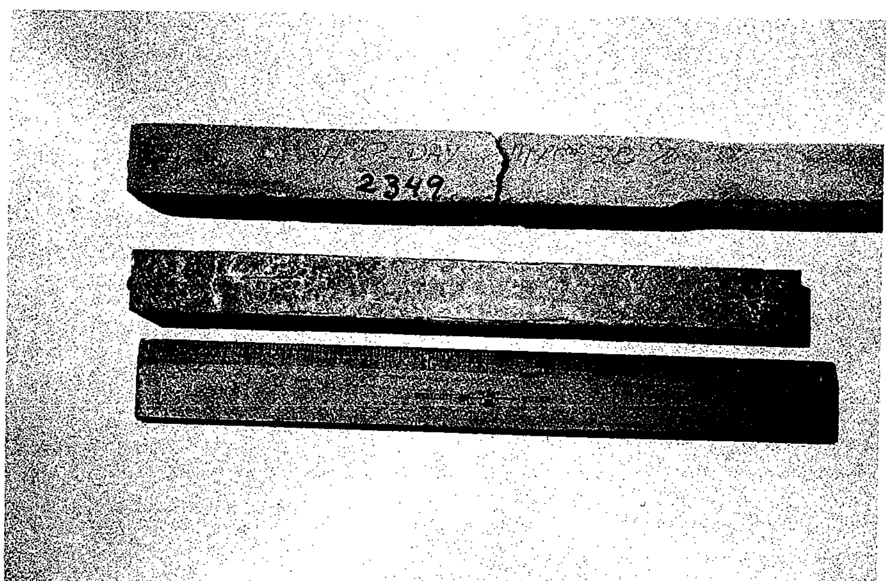

Fig. 17

Table II

	Sample No.						
	2321	2349	2420	2423	3137	3603	2816
Surface Area (cm^2/gm)	1910	1815	1855	1815	1915	1780	-
Tensile Strength (psi)							
1-day	-	-	-	-	-	-	323
3-day	187	173	260	263	303	290	413
7-day	273*	250*	337	343	400	410	-
Autoclave Expansion - %	20*	20*	10*	10*	3.0*	3.0*	2.5*
Sulfur Trioxide - %	2.39	-	-	-	2.56	-	-
Setting Time	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.	O.K.

*Failing Results
-No Results Determined

NOTE: Minimum 5-day strength 275 psi.
Maximum allowed expansion 0.8%.

Of the seven cement samples listed in Table II, two exhibited approximately 20% expansion when subjected to the autoclave test. Figures 14 and 15 show prisms cast with normal and the unsound cement (#72-2321-D) as observed in the autoclave and after removal. Figures 16 and 17 illustrate results of the autoclave test on a number of unsound prisms. The relative amounts of expansion can be easily compared with a "normal" prism. Figure 17 shows a prism made with cement from sample #72-2349-D which still exhibited 20% expansion even after the cement was aerated for three days.

Results from additional aeration on two separate cement samples containing the free lime are summarized in Table III. The two samples (#2349 and #3137) were placed in a pan, stirred frequently and exposed to the air (in the laboratory). Subsequent autoclave tests show that a loss occurs in the cement's expansive properties which reflects the chemical changes taking place as the free calcium oxide is converted to the hydrated form upon exposure.

Table III

Sample E	Aeration Time	Percent Expansion
72-2349-D	As received	20.0
72-2349-D	1-week	20.0
72-2349-D	4-weeks	0.1
72-3137-D	As received	3.0
72-3137-D	2-days in closed can	0.24
72-3137-D	1-day in open air	0.16

Results from samples being subjected to the pat-boiler test are best illustrated by Figures 18, 19, 21 and 22. A neat-cement sample made with the unsound cement (#2349) completely disintegrated when it was subjected to the pat-boiler (steam bath) for 5 hours as shown in Figure 18. The sample was cured in a moist chamber for 24 hours. A normal control sample which exhibited no expansion is also shown for comparison. Figure 19 shows exudations that were observed on the bottom of one of the neat-cement samples (#3137) after being subjected to the steam bath. Figure 20 shows a similar type of exudation observed on the surface of a piece of concrete (from the bridge deck) which had been subjected to the autoclave test.

Figures 21 and 22 illustrate the effect of 5 hours of steam on briquettes made with the unsound cement. An untreated briquette is shown on the far right for comparison in Figure 21. Photomicrographs taken at 4X show details of the crack pattern as observed on the curved sides of specimen #2321 in Figure 22; a close-up of the untreated sample showing a crack-free condition is illustrated as Figure 23. Briquettes containing the bad cement (#2321 and #2349) cured 7 days in water and left exposed in the laboratory for 5 months still showed to be unsound by the autoclave and steam-bath tests.

Other than the identification of the expected component phases of the cement, results from the petrographic studies were limited to indirect evidence of the presence of free lime. The major significant evidence that the failure of the concrete examined for this investigation was caused from internal instability comes from the association of secondary chemical exudations formed within the air-void system (as those shown in Figures 24 and 25) and the presence of a pronounced network of microcracks. Secondary compounds formed on the surface of neat-cement pats subjected to the steam-bath were similar to those observed

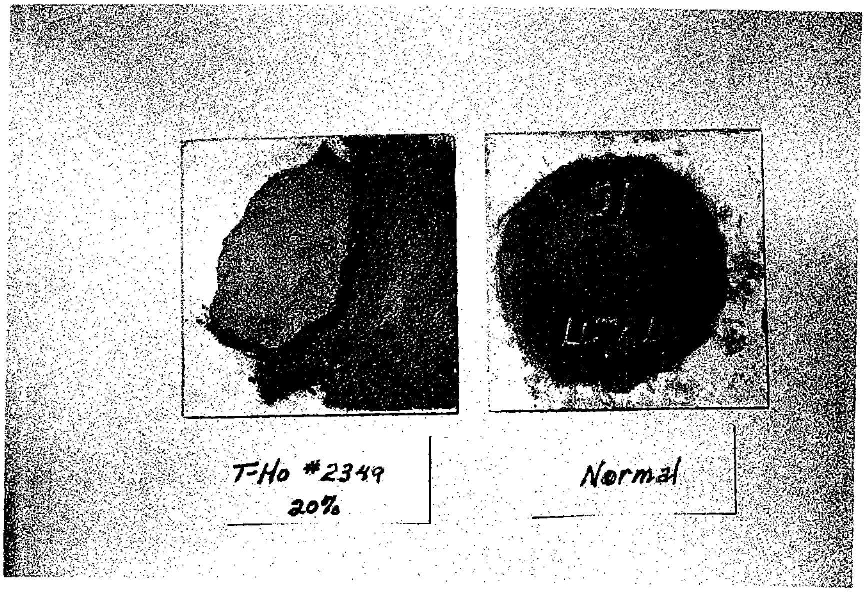

Fig. 18

Fig. 19
(Mag. 30X)

Fig. 20
(Mag. 20X)

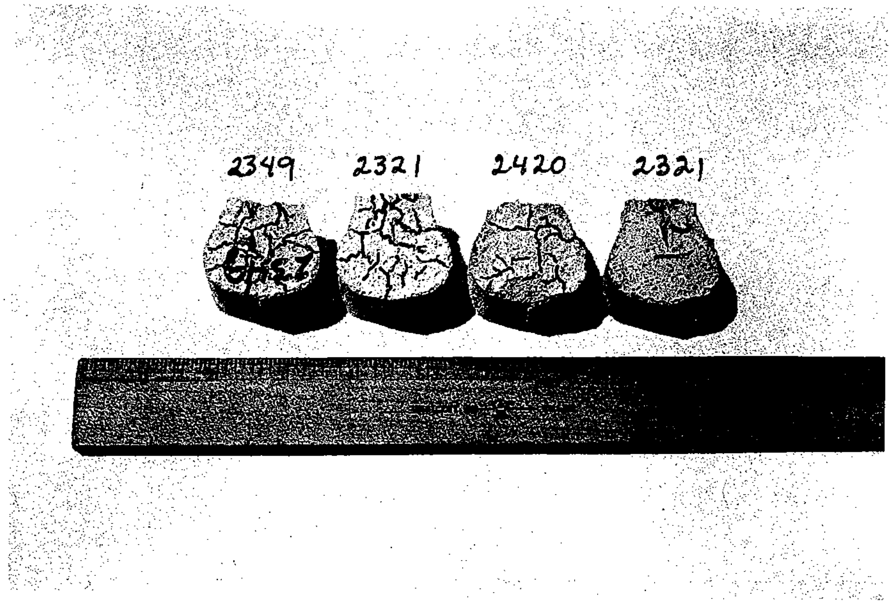

Fig. 21

Fig. 22
(Mag. 4X)

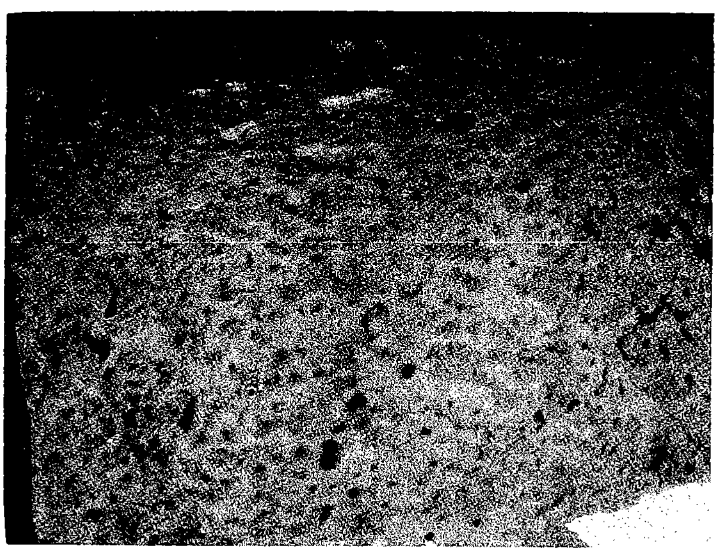

Fig. 23
(Mag. 4X)

Fig. 24
(Mag. 15X)

Fig. 25
(Mag. 60X)

within the air-void system of the concrete as well as those formed on the surface of concrete samples subjected to the autoclave test. Powder mounts of these secondary materials indicate that they are composed of calcium hydroxide being partially converted to calcium carbonate. The calcium hydroxide occurs in abundant quantities in concrete; however, these exudations suggest that an abnormal condition exists. These secondary deposits may very well indicate that "unaccommodative" chemical reactions have occurred as a result of the accelerated conditions (steam-bath and autoclave). Many of these exudations are in the form of circular mounds that are often cracked or collapsed as illustrated in Figures 24 and 25. Some are donut-shaped.

The crack patterns range in size from those shown in Figures 2-5 of the bridge deck, to hair-line cracks as those covering the entire surface of the column (Figures 26 and 27), down to the microcracks noted in the paste from all the examined structures. Especially, unusual are the "septa-like" cracks found in several air voids of the column. These unusual microcracks shown as Figures 28, 29 and 30 appear to have been formed before final-set of the paste and "healed" as calcium hydroxide solutions were forced into the void space and solidified. Microcracks were also noted throughout the retaining-wall samples. Large cracks visible on the top of Core VI-A removed from the bridge deck were easily traced to depths of about 10 inches. Several pronounced horizontal separation cracks were found some 6 inches from the bottom of the core, but were not associated with the rebar (Figures 31 and 32).

Extended efforts were made to identify any free calcium oxide component within the remaining unhydrated cement grains comprising the paste fraction of the concrete samples. Highly-polished concrete samples were examined at 1000X under polarized reflected light but failed to reveal any abnormal compounds. Although the primary cement components (C_3S, C_2S, C_3A and C_4AF) were easily identified, no positive determination of CaO was made in any of the samples examined. Relative density of cement particles was noted, however, and indications were that mix-water contents for the bridge deck and the column were normal but may have been too high for the concrete in the retaining wall foundation. It was also noted that the relative extent of carbonation was much higher for the latter concrete samples. This condition is indicative of higher porosity which is generally related to high water/cement ratios. Numerous bleed channels were also noted and the paste appeared chalky and powdery.

Linear traverse data was obtained from the bridge deck samples and the column; however, efforts were unsuccessful in preparing samples from the retaining wall for this type of examination. From the first two structures samples were analyzed for comparative purposes and the results are listed in Table IV.

Fig. 26
(Mag. 7.5X)

Fig. 27
(Mag. 7.5X)

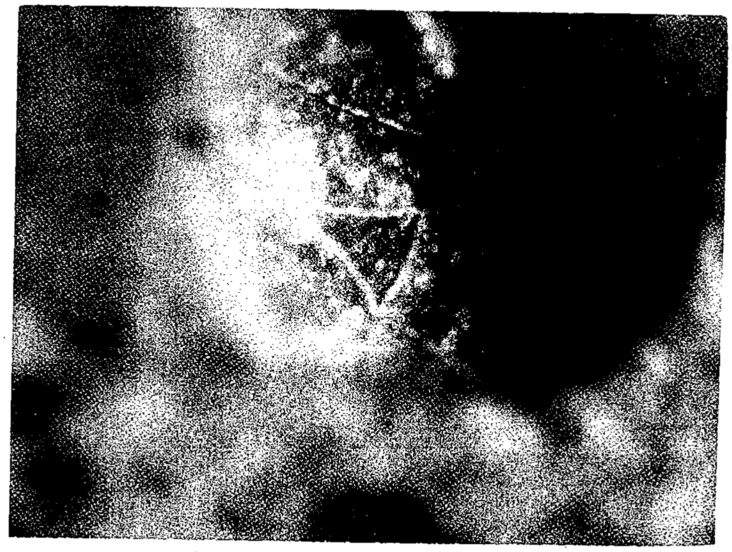

Fig. 28
(Mag. 50X)

Fig. 29
(Mag. 50X)

Fig. 30
(Mag. 75X)

Fig. 31
(Mag. 10X)

Table IV

Results of Linear Traverse

	Bridge-Deck Samples			Column Samples	
	A	B	C	A	B
Air Content (%)	6.38	7.42	6.98	2.75	2.11
Paste Content (%)	27.25	24.53	25.30	29.01	21.41
Surface Area (in^2/in^3)	570	541	597	412	391

Although powder mounts of the unsound cement proved to be inferior to polished sections of concrete for examining the cement, they did provide some information that suggests the presence of free lime more so than the latter technique. Under plane and polarized light at intermediate magnification ranges (200-400X) when emersion oils with refractive indexes of 1.71-1.73 were used, a number of "high relief" particles (those with refractive indexes which differ from the oil used) which resembled free lime were noted. However, these particles could not be differentiated from other cement components which also have a higher relief (about 1.83). On the other hand, when a small quantity of the cement was placed on a microscope slide with about 2 drops of White's solution, very delicate needle-like crystals of phenoxide appeared in 2-3 hours (Fig. 33). Many crystals were observed when the slide was observed a day later. Reportedly, this solution will react within a few minutes if appreciable quantities of free lime are present; however, the samples of cement examined (#2321 and #2349) had been retained in the lab for about 5 months before this observation was made. The delay in reaction time no doubt reflects a loss in reactive or uncombined free lime due to its gradual hydration. The same applies to the absence of free lime in the polished concrete pieces. By the time these pieces were prepared in the laboratory the original properties of the free lime had long disappeared.

VI. DISCUSSION

Departmental personnel concerned with the quality control of concrete and durability of structures have long recognized that volume stability is one of the basic desired characteristics of structural concretes. Regardless of how well the aggregates have been selected or the quality of entrained air, it cannot be expected to produce structural concrete that will be free from objectionable volume changes and cracking, unless the cement paste has a high degree of volume stability. Often these changes contribute to at least a gradual, and sometimes sudden, deterioration.

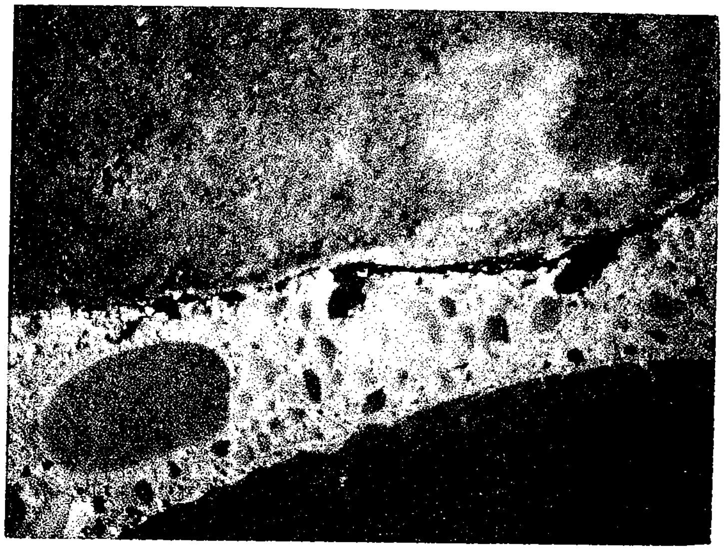

Fig. 32
(Mag. 7.5X)

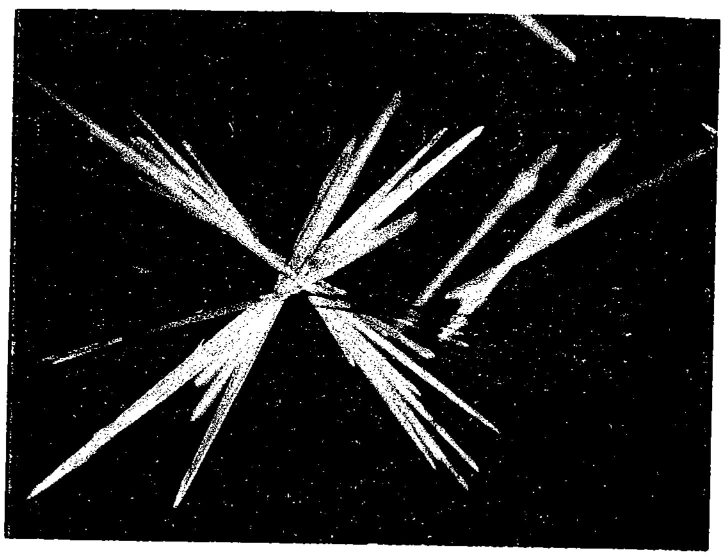

Fig. 33
(Mag. 500X)

During April and May 1972 several cement samples, produced at a portland cement plant in Houston, and shipped to the Materials and Tests Division laboratory in Austin, were found to be unsound as indicated by the autoclave expansion test. It was determined that these cements exhibited expansion ranges from 2.5 to 20 percent compared to the maximum allowable of 0.8 percent. Two samples, which had a 20 percent autoclave expansion, passed the 3-day tensile-strength test only by a narrow margin but failed the 7-day test. Project engineers both at the Houston Urban Office and District 12 were quickly alerted so that highway projects using the unsound cement could be closely watched and precautions made. Fortunately, use of the cement on highway projects was limited; however, it was found that three structures at two separate projects in Houston had severe damage and had to be removed.

Both uncombined calcium oxide and magnesium oxide are chemically unstable in portland cements. Although the chemical analysis of the cement samples revealed that the calcium oxide content was abnormally high, the percentage of the latter compound was found to be normal. In addition, results of the loss-on-ignition test were also high indicating that some of the free lime had converted to the hydrated form when the sample was tested. No doubt that when the cement was used on the Houston projects it contained excessive amounts of calcium oxide in the unhydrated form, and with an attendant volume increase of almost 100 percent when free CaO hydrates to $Ca(OH)_2$, the resulting concrete failed with irreparable damage.

Additional laboratory studies, which involved subjecting neat-cement pats and briquettes to the steam-bath test, showed that a decrease in potential reactivity occurred with aeration. The samples which exhibited 10 percent autoclave expansion or less showed a rapid loss in potential reactivity; whereas, the ones with 20 percent expansion still showed a 0.1 percent expansion even after 4 weeks. Extensive cracking occurred when these latter briquettes containing the "20%" cement were cured 7 days in water, stored 5 months in the laboratory and finally subjected to the autoclave and steam-bath tests.

The significant petrographic evidences for unstable cement in the examined concrete samples were exudations of calcium hydroxide within the air-voids and the associated network of microcracks. Cracks ranged in size from those shown in the photographs of the bridge deck and retaining wall foundation to microscopic in size as with those found in the paste of all samples examined. Exudations also occurred on the surface of neat-cement pats subjected to the steam bath and on concrete samples subjected to the autoclave test. Both the cement pat samples and briquettes showed varying degrees of disintegration when tested in the steam bath after extended periods of aeration. Microscopic examinations of those samples reveal a pronounced loss of bonding strength, loss of color and a change from a dense glassy appearing paste to one that looks porous and chalky. The concrete samples appeared the same way after treatment in the autoclave.

Thin-sections made of the concrete samples proved to be inadequate for examining the cement as compared to the successfully used polished sections. Although the polished sections provided a means of determining a number of features of the paste and cement particles such as the relative extent of hydration, no traces of free lime were found at the time of this study (about 4-5 months after the concrete was placed). However, powder mounts of the cement after being treated with a solution of phenol and nitrobenzine showed that minute traces of the free lime were present.

Microscopic analysis indicates that mix-water contents for the bridge deck and column were not excessive; however, the lighter-colored paste, higher porosity and the presence of bleed channels suggest that the concrete batched for the retainer wall may have had some extra water added. Linear traverse data shows that the air content of the deck concrete was about 6.5 percent and had a paste content of some 25 percent. Air content for the column was measured at about 2.5, indicating that no air-entraining admixtures were added to the mix. The aggregate-paste bond was so poor in the retaining wall samples that efforts were unsuccessful for analysis by linear traverse methods.

DETERMINATION OF MAGNESIUM OXIDE*

Procedure

Weigh out approximately 3 grams of the cement to be analyzed. Transfer the sample to a platinum dish and moisten with a little distilled water. Then add approximately 25 ml of 6 N. HCl and heat until the sample dissolves. Evaporate the sample to dryness by baking in 212°F oven and then redissolve in 25 ml of 6 N. HCl. Transfer the sample from the platinum dish to a 250 ml small-form beaker. Carefully add 15 ml of 20% ammonium hydroxide. Then, using the pH meter, adjust the pH to between 5 and 7 using 1 Normal NaOH. A precipitant of Aluminum and Iron hydroxides should form. After adjusting the pH, carefully wash off the probes so as not to lose any sample. To the sample, add 30 ml of the 20% Na_2SO_3 solution to precipitate calcium and any other heavy metals that might be present. At this point, the solution must be filtered. A quantitative filter paper, #42, should be used. The sample may be filtered directly into a 250 ml volumetric flask. After filtering out the precipitant, wash it several times with distilled water. The precipitant may be discarded at this point. The filtrate in the 250 ml volumetric flask contains the Magnesium which is to be determined titrimetrically. Add enough distilled water to the flask to bring the total sample volume up to the 250 ml mark. Mix the contents of the flask thoroughly, and pipet out a 50 ml portion into a 400 ml tall-form beaker. Add 10 ml of a pH 10 buffer and 10 ml of a 10% NaCN solution. Add distilled water to bring the total volume up to about 150 ml. Then add 0.100 to 0.150 gm Eriochrome Black-T indicator. When viewed with a light behind it, the solution should have a bright red color. Titrate the sample rapidly with a 0.01 molar solution of EDTA until the solution changes from red to a reddish blue color. Then add the EDTA slowly until the last trace of red disappears. This is the end point. Duplicate samples should be run. To calculate the Magnesium present in the sample as MgO:

$$\% \text{ MgO} = \frac{(\text{Ml.EDTA})(0.2017)}{(\text{wt. of Sample, gms})}$$

Note: The size of the original sample and the fraction of solution titrated were designed for a sample containing approximately 0.5 to 3.0% MgO. Sample size, dilution, and fraction titrated may be adjusted to compensate for MgO content outside these limits. In event this is done, the formula for calculating percent MgO must also be changed.

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* This method is primarily used for analyzing lime samples but can be appropriately applied to portland cements.

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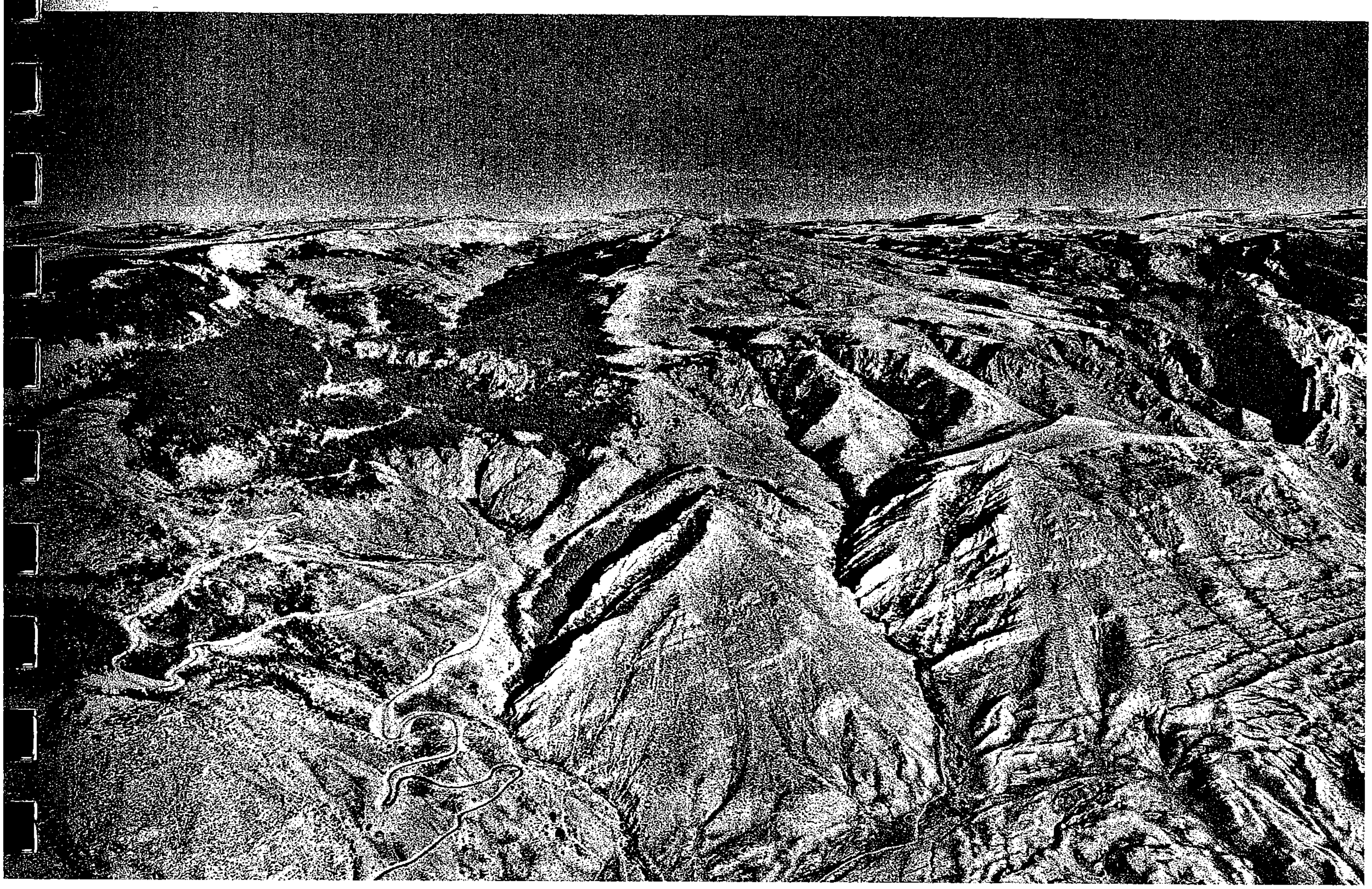
24th Annual

Highway Geology Symposium

Field - Trip Guidebook

Sheridan, Wyoming August 9, 1973

GEOLOGY AND ITS RELATIONSHIP TO HIGHWAY CONSTRUCTION BIG HORN MOUNTAINS, NORTHERN WYOMING



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GEOLOGY AND ITS RELATIONSHIP TO HIGHWAY CONSTRUCTION BIG HORN MOUNTAINS, NORTHERN WYOMING

Field-Trip Contributors

Wyoming Highway Department Geology Division Personnel
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TABLE OF CONTENTS

FOREWORD

On behalf of the National Steering Committee, the sponsors welcome you to Wyoming and the 24th Annual Highway Geology Symposium field trip.

The field trip, scheduled the first day of the Symposium, will emphasize primarily the geological engineering problems encountered in roadway construction in mountainous terrain. Anticipated engineering and geologic problems will also be discussed concerning future construction in this area. A minimum of four stops is scheduled on the trip; although, other stops may be included, time permitting.

We hope you will find the trip stimulating and informative.

Frank Morgando
Field Trip Chairman

WYOMING GEOLOGIC HISTORY IN BRIEF

By: Robert A. Lewis[1]

Wyoming: A state exhibiting a vast complexity of geologic structure remnant of eons of tectonic activity coupled with several stages of ancient migrating and retreating seas.

Below is a brief treatise concerning Wyoming geologic history sprinkled with noteworthy examples of typical Wyoming geology.

GEOLOGIC SETTING

Wyoming occupies a transitional zone between the stable cratonic regions of the great plains in the east and the mobile overthrust belt in the west, which is the beginning of the tectonically active Cordilleran Geosyncline. Consequently, Wyoming physiography is representative of several northwest-southeast trending mountain ranges surrounded by specifically named intermountain basins containing thick sequences of sedimentary rocks (figure 1).

Isolated highlands within the typical basin have a low profile and are usually composed of sedimentary rocks. Notable examples are the Rock Springs uplift in southwest Wyoming and uranium-rich Pumpkin Buttes in west central Powder River Basin. Still other highlands have granite cores such as the granite mountains of Central Wyoming that from a distance appear to be rocky islands rising from a sage brush sea.

GEOLOGIC HISTORY AND WYOMING GEOLOGY

Precambrian

Highly complex basement rocks chiefly composed of granite with minor occurrences of metasediments form the core of Wyoming's major

[1]Engineering Geologist, Wyoming Highway Department

mountain ranges. Little is known of Precambrian rocks as compared to Phanerozoic sequences that overlie this basement complex.

In years past, hundreds of base and precious metal mines were scattered throughout Wyoming in Precambrian rocks. Practically none of these remote, "one-horse" mines are in operation today.

Paleozoic

Seas migrating from the Cordilleran Geosyncline west of Wyoming are the major depositional agents in Wyoming from Cambrian through Mississippian period. Near the bottom of the sedimentary section is the highly landslide susceptible Cambrian, Gros Ventre shale, also home of the elusive trilobite.

Prominent above the Gros Ventre is Ordovician, Big Horn Dolomite, and Mississippian Madison limestone that form massive, near-vertical cliffs in the numerous mountain front canyons of western Wyoming.

Regional uplift and consequent erosion in early Devonian time has removed all the Silurian and most Ordovician sediments in Wyoming. Rocks containing early to late Silurian fossils have been preserved in diatrems located along the Colorado-Wyoming border southwest of Cheyenne. This coupled with other evidence in the Rocky Mountain region is indicative that occurring during Silurian was one of the most widespread of Paleozoic epeiric seas.

Total thickness of Cambrian through Mississippian rock aggregates about 2500' in the Wind River Range and thins to zero in southeastern Wyoming.

Pennsylvanian rocks, and to a lesser degree Permian, are characterized by marked facies changes over relatively short distances caused by mild, unstable, local tilting. Pennsylvanian sediments are the first to be present in all parts of the State. Characteristic of the Pennsylvanian facies change is the existence of cross-bedded sandstone of the Tensleep formation in west central Wyoming interfingering with the massive Hartville limestone of the southeastern region.

A sea transgressing from the Cordilleran Geosyncline in mid to late Permian deposited the commercially attractive phosphorites of the Phosphoria formation. As with Pennsylvanian sediments, the Phosphoria formation interfingers eastward into carbonates and nonmarine red bed facies.

Mesozoic

Triassic and Jurassic rocks in Wyoming are a part of a vast sequence of continental and marine rocks extending in an eastward thinning wedge from Canada to New Mexico. Wyoming is considered to be situated on the eastern shelf. During early Triassic, the Wyoming depositional environment was predominately marine consisting of very fine-grained red beds with evaporites as lenses (Chugwater and Spearfish formations). Upper Triassic sediments are mostly continental in origin, containing red beds of mudstone, sandstone, and conglomerate. A notable exception is the thin but conspicuous Alcova limestone at the base of the upper Triassic sequence.

In Wyoming, a gradational boundary exists between the Triassic-Early Jurassic (Absaroka) sequences, which reflects new marine invasion from the north and west. In the upper Triassic series the marine Sundance cycle composed of shales, sandstones, and siltstones grade upward to nonmarine variegated shales, freshwater limestone and sandstone of the landslide-prone Morrison formation.

During the Cretaceous period, Wyoming was apparently an asymmetrically subsiding basin collecting sediments from the uplifting Cordilleran regions of western Utah and Idaho. Cretaceous sediments are concentrated in southwestern and south central Wyoming and aggregate over 25,000 feet of beds. This unique depositional environment has double commercial value allowing for the coal measures of the Mesa Verde formation in the southwest to be time equivalent of the marine bentonite bearing Pierre shale of northeastern Wyoming.

Cenozoic

Roughly 55% (50,000 square miles) of Wyoming's land area is composed of surface-exposed Tertiary rocks concentrated in all intermountain basins. The source area for these continental deposits were the mountain ranges scattered throughout the State and to the west.

The basic structural trends existing presently in Wyoming were uplifted in late Cretaceous through Paleocene (Laramide Orgeny) and culminating with regional uplifting during Pliocene and early Pleistocene. This tectonic activity produced classic geologic structure such as the drape folding occurring in the northwest Big Horn Mountains and the Grand Tetons with their spectacular east-facing scarp.

Basins and Ranges of Wyoming

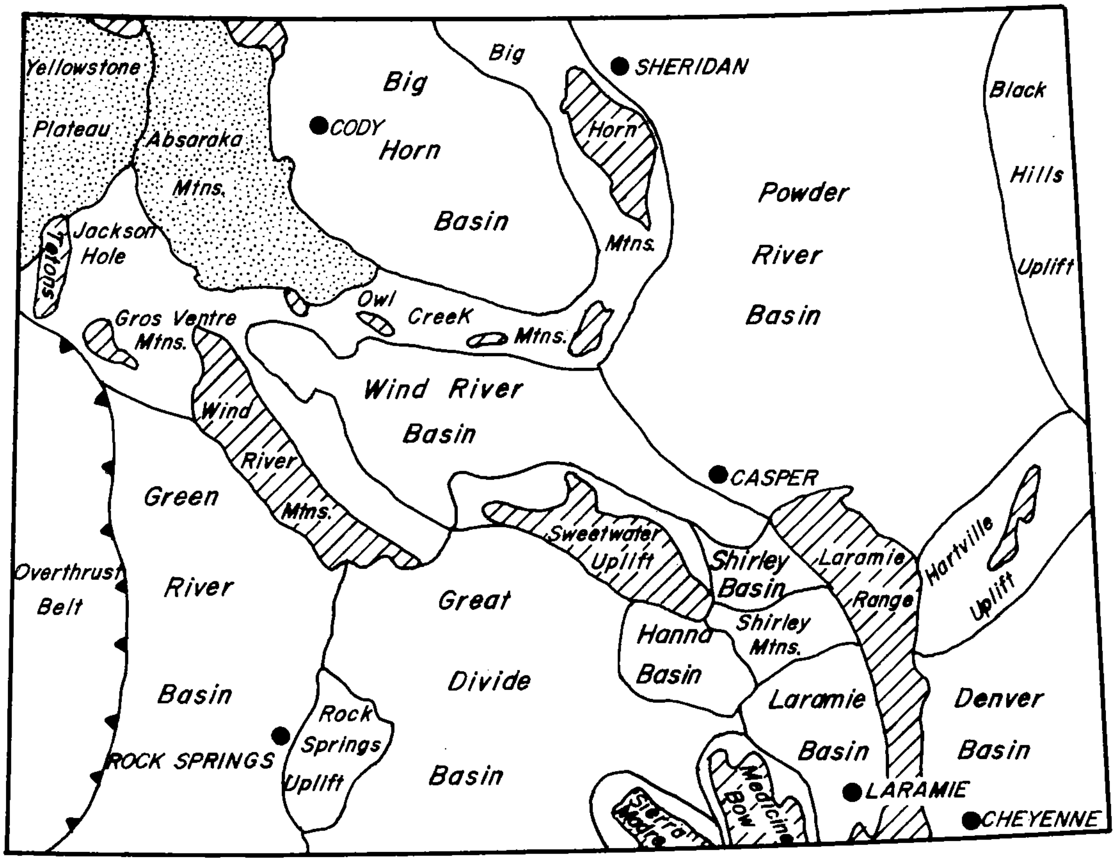

Fig. 1

Major Structural Features of Wyoming

Cross hatched areas are Pre-Cambrian Granite.
Stippled areas are Tertiary Volcanics.
Sawteeth on up thrown block of Overthrust Front.

THE FIELD TRIP AREA

By J. Hale[1]

Selection of the Big Horn Mountain area for the field trip was made because of the abundance of geologically associated highway problems and excellent rock exposures. Highway construction problems through this range can be divided into three groups; these include, past construction projects where the science of geology was not applied, past projects where engineering geology was utilized, and future projects to which geologic engineering principles will be implemented. These problems can be viewed from the standpoint of what preventative measures should have been taken, what has been taken, and how some of the problems can be prevented in future road building programs.

The Big Horn Mountains comprise the major portion of an arcuate shaped uplift which extends from south central Montana to central Wyoming. The "Big Horns" rise to elevations of 10,000 feet for most of the summits to slightly over 13,000 feet at the highest point. This is some 6,000 to 9,000 feet above the adjoining Powder River Basin to the east and Big Horn Basin to the west. The Big Horn River has severed the uplift to an elevation of 3,500 feet and demarks the north boundary of the "Big Horns" and south boundary of the Pryor Mountains. Bridger Creek separates the "Big Horns" from the east-west trending Bridger and Owl Creek Mountains to the south.

The "Big Horns", an outlier of the Rocky Mountains, were formed during the Laramide Revolution in late Cretaceous time. Folding and relatively minor faulting resulted in the uplift of the Pre-Laramide

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[1]Engineering Geologist, Wyoming Highway Department

sediments into a box fold several thousand feet high. As uplift occurred, the destructive processes of weathering and erosion began wearing away the elevated rock with the materials redeposited as post-Laramide sediments on the flanks of the range and in the adjacent basins. Subsequent isostatic adjustments rejuvinated erosive agents in the area at periodic intervals leading to the development and destruction of several pediment surfaces at the mountain front.

These processes have resulted in the exposure of an almost complete stratigraphic section from the Precambrian through the Tertiary. Most of these units are crossed by highways U. S. 14 and 14A (field trip routes). The stratigraphic section is shown on figure 3. A great variety of engineering problems is provided by the construction across this sequence, and several are to be brought into focus during the field trip.

These problems range from the spectacular and highly destructive mass gravity movements to inconspicuous roadway foundation problems which result in pavement breakup through swell, settlement, and frost heave. Due to the frequency of landslides resulting in major construction problems, the minor problems have been largely overlooked in mountainous construction.

The problems that can be associated with any given map unit (formations or beds) are dependent upon several factors which include: 1) parent material; 2) location with respect to underlying and overlying beds; 3) past deformation (fractures, etc.); 4) topographic location; and, 5) ground water.

For instance, the Cambrian system has presented more construction problems than any other geologic time unit in the area. The inherent weakness within this sequence of beds dates back to their origin; the deposition of weak, slippery, glauconite shale alternating with thin sandstone and limestone beds. These units were then overlain, during the Ordovician and Mississippian, by a thick carbonate section.

The above sequence, when finally exposed to attack by weathering and erosion, began to degrade extensively by mass gravity movement because of conditions described above, coupled with tilting and fracturing that occurred primarily during the Laramide Orogeny.

The mechanical development of landslide topography and subsequent slope failures during highway construction is as follows: The carbonate rocks, principally the massive Ordovician Big Horn Dolomite, is very resistant to erosion which tends to result in an over steepening of the underlying weak shale sequence. When the dolomite finally gives way to destructive processes, it provides a talus cover on the underlying shale providing a protective blanket which resists normal erosion. The numerous sandstone and limestone beds become aquifers when fed by precipitation and transmit water down dip, each creating a potential slide surface by lubricating the slippery bounding glauconite shale. In addition, after initial slippage, drainage of the water bearing beds is restricted, resulting in pore pressure buildup.

The above described processes have created a very delicately balanced topography which when only slightly altered by construction, begins to slide unless counter measures are employed. Failure to recognize and/or to adequately compensate for these conditions has resulted in extensive reconstruction of roads through these areas.

Problems associated with Ordovician and Mississippian carbonate rocks are of an entirely different nature and are due more to the inherent strength rather than the weakness of these units. The natural slope of these units is as steep as vertical and rock cuts are required with the possibility of blocks slipping on jointed or fractured surfaces. In addition, the long steep slopes associated with these units are often covered with talus. Attempts at cutting these deposits produce continual rock slide until the cut slope coincides with the angle of repose.

The remainder of the Paleozoic, Triassic, and Jurassic has not produced any persistant construction problems. Engineering geology can best be used through these units in designing optimum cut and fill slopes.

The dark marine shales of the Cretaceous contain numerous bentonite beds which are very volumetrically sensative to changes in moisture and pressure. Swell and settlement problems have been extremely difficult to cope with resulting in rough roadway where these beds are crossed.

These sediments are covered by pediment gravels where U. S. 14 (field trip route) crosses on the east flank of the mountains.

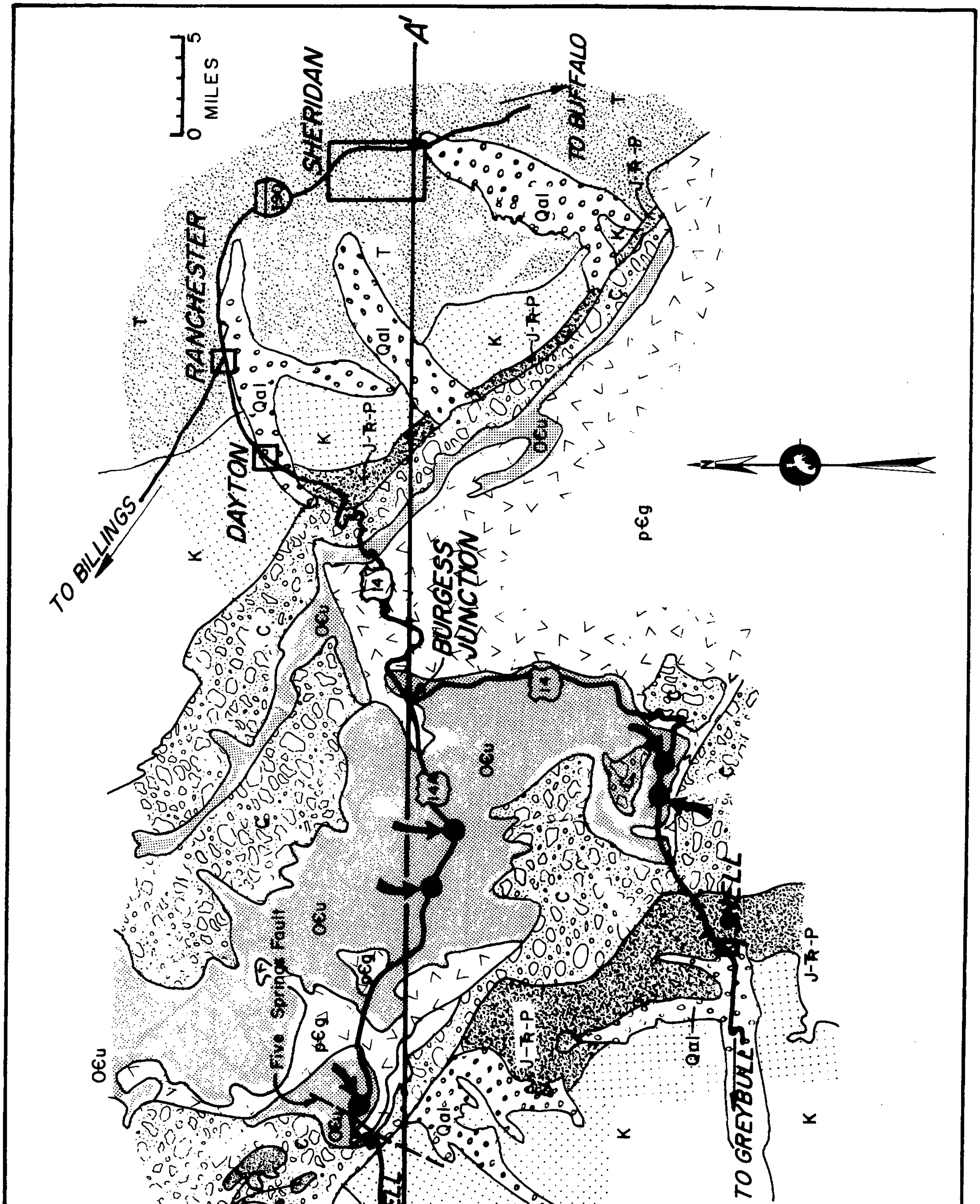
The Tertiary sediments, although they do not comprise mountain geology, are crossed by the field trip route, thus warranting some description. The Tertiary, though at almost the opposite end of the geologic time table, has essentially the same landslide susceptibility as the Cambrian. The Tertiary is similar to the Cambrian in that both contain weak shales with abundant aquifers which give rise to numerous slope stability problems.

The depositional history of the two groups is dissimilar with the Cambrian being a marine deposit as compared to terrestrial deposition for the Tertiary. The aquifers in the Tertiary are generally coal and lignite beds as compared to fractured limestone and sandstone for the Cambrian. Tertiary shales owe their inherent weakness more to lack of induration through pressure and time whereas the weakness of Cambrian shale is more the result of mineralogy.

Unstable Tertiary deposits are a conspicuous feature of the landscape around the Sheridan area and numerous roadway and backslope failures have occurred on roadway construction projects in the vicinity.

Quaternary deposits (primarily terraces) have been the major source of aggregate both for roadway and building construction in the area. Roadway construction problems in the Quaternary consist of soft subgrade and frost heave due to the organic topsoil developed on these sediments and associated high water tables.

The preceding is by no means a complete guide to problems within each time interval described, but is intended as a guide to repetitive problems which will almost certainly be encountered during highway construction across these units.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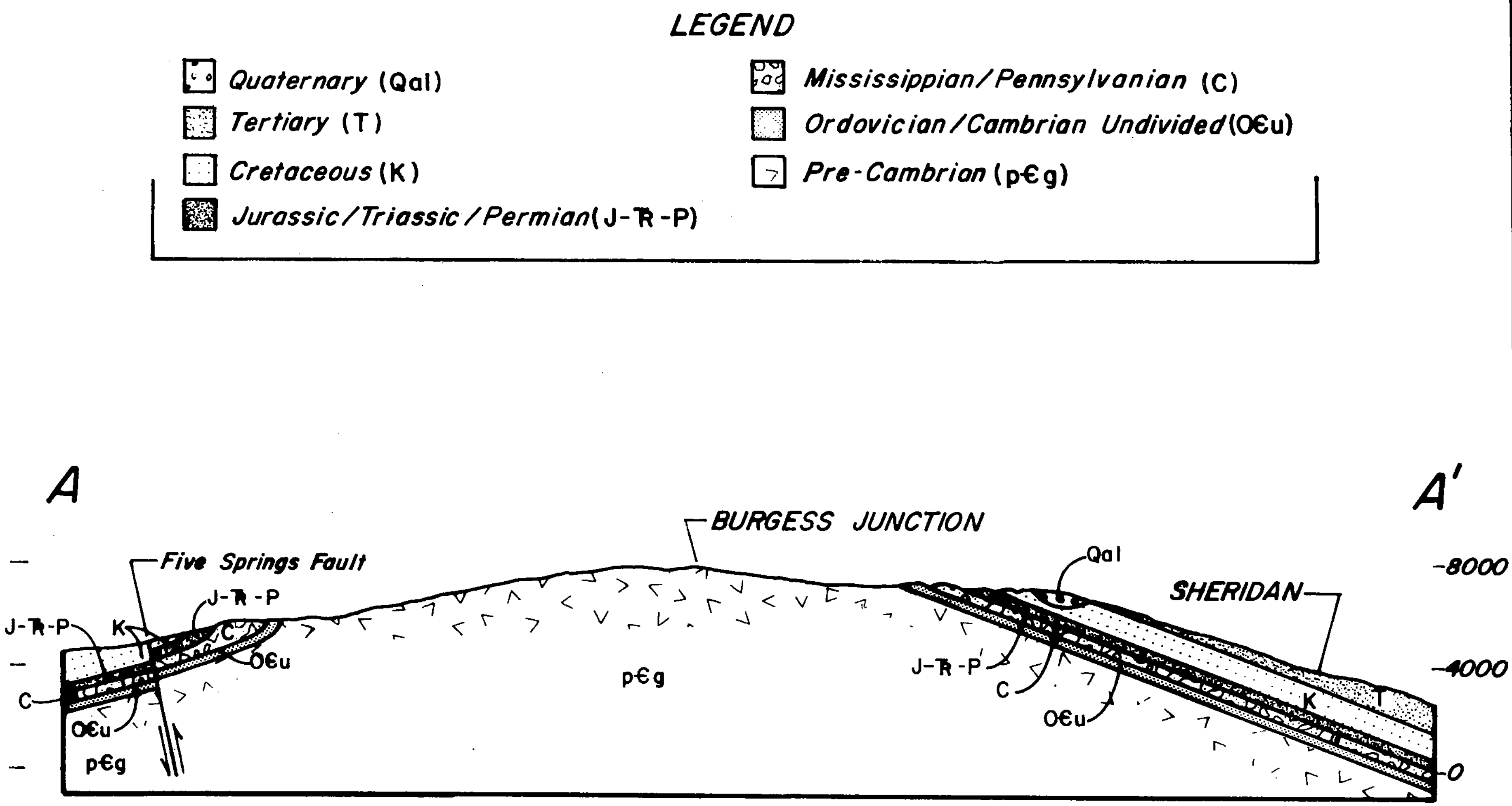


GEOLOGIC MAP
and
SCHEMATIC CROSS SECTION
of
FIELD TRIP AREA

FIGURE 2

ROCK SEQUENCE IN THE BIG HORN MOUNTAINS

Quat	Recent and Pleistocene		Gravels and Terraces	Gravel terraces or benches near major drainages.
Tertiary	Eocene		Willwood west flank	Willwood, drab to pastel shale, claystone, sandstone on flank. Forms badlands.
			Wasatch east flank	Wasatch is drab claystone, siltstone, sandstone, and shale containing numerous coal and lignite beds. Contains conglomerate member.
	Paleocene		Fort Union	Drab sandstone, mudstone, and coal beds. Some fossil leaves. General tan appearance on outcrop.
Cretaceous		Upper	Lance	Sand, shale, and few coals. Forms resistant ridges.
			Lewis	Massive white sandstones, and many thin shales and coals. Generally less resistant than formations above or below.
			Mesaverde Group	Sandstone, shale, coal. Forms resistant ridges. Outlines many anticlines in the basin.
			Cody	Gray, black shale with bentonite beds and few thin sandstones.
			Niobrara, Carlile	Forms an alternating series of sandstone ridges and shale valleys.
			Frontier	Forms rounded steep slopes with some resistant, siliceous bands. Silver-gray color on outcrop.
		Lower	Mowry, Thermopolis	Dark gray to black shale. Muddy Sandstone member, near center of section, generally poorly resistant.
			Muddy ss., Cloverly Group	Thin sands and shales, tan to brown, sideritic. Some conglomerate lenses in lower part.

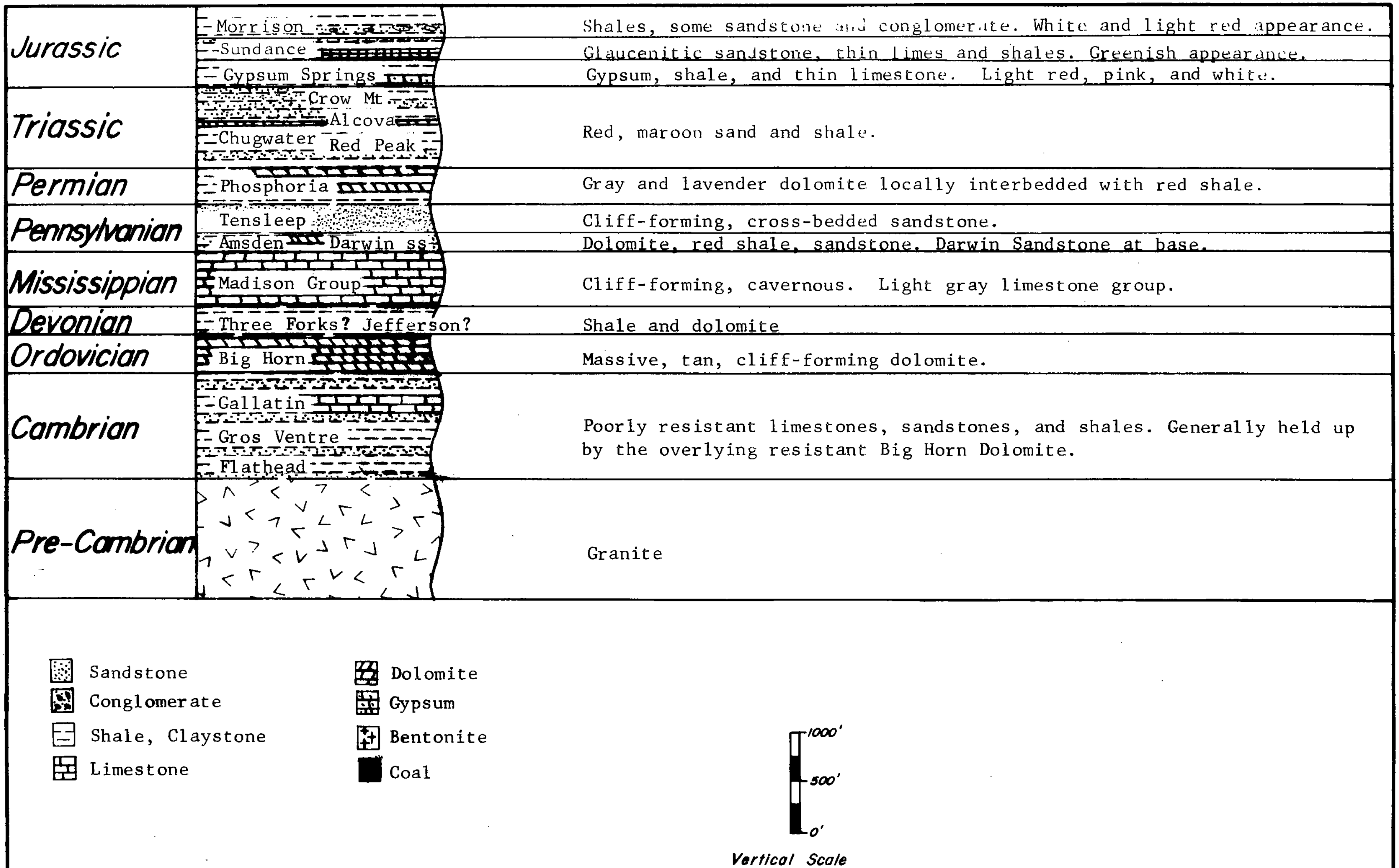
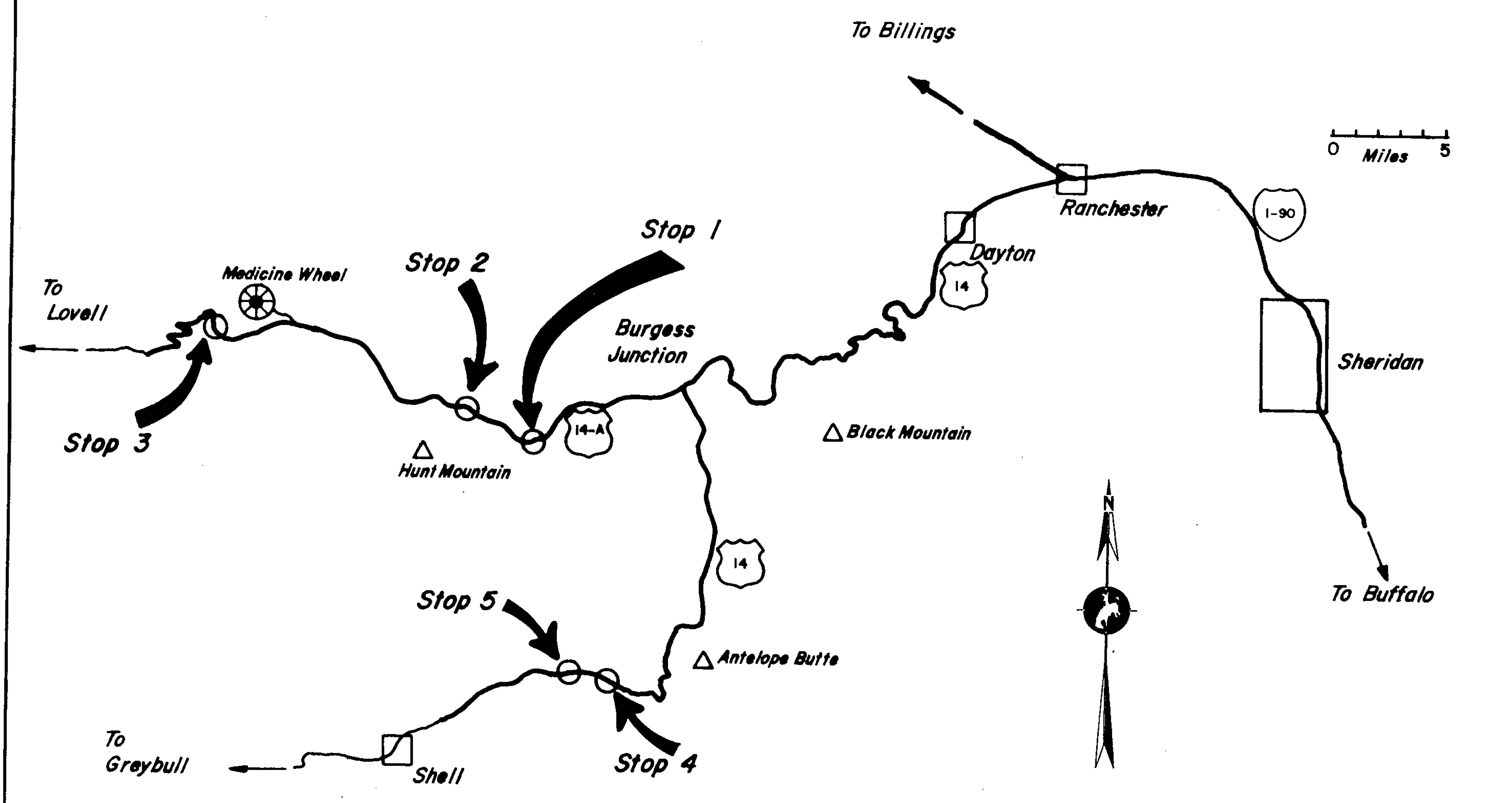


FIGURE 3

Adapted from the Wyoming Geological Association
Highway Geology of Wyoming, 1964



ROAD MAP of FIELD TRIP

Fig. 4

ROAD LOG

Start - Sheridan Center

0.0	Terminal point, Sheridan Center Inn, center of town,
2.0	elevation 3745. Population 10,856
2.0	Big Goose Creek - approximate, arbitrary contact between Fort Union and Wasatch formations - Roland Coal Member
5.0	
7.0	Cut section with underdrains in left ditch. Coal is primary aquifer.
1.0	
8.0	Coal mine subsidence area. Mine depths vary from 20-100'.
2.4	
10.4	Cuts exposing "scoria" reddish heat-altered shales and sandstones.
4.7	
15.1	Junction I-90 and highway 14.
.5	
15.6	Town of <u>Ranchester</u>, population 235, elevation 3775' - The hills on the north skyline are composed of the Tertiary Fort Union formation.
1.4	
17.0	Roadway is on Quaternary alluvium underlain by the Upper Cretaceous Lance formation.
0.7	Terraces of the Tongue River can be seen ahead (right). Gravel pits have been developed on these terraces.
17.7	Hills north of road are composed of the Cretaceous Lance formation.

20.8 East Dayton city limits, elevation 3926', population 350 approx.

.5

21.3 West Dayton city limits, for the next 2.0 miles we will pass over Cretaceous sediments. Carlisle shale to the Cloverly formation.

2.6 The highway 14 road scar can be seen along the mountain front.

23.9 Jurassic Morrison-Sundance, Gypsum Springs formations begin

1.1 at this point.

25.0 Road cut is in Jurassic Gypsum Springs formation.

1.9

26.9 Road cut to north in dolomites resting on Pennsylvanian Tensleep sandstone. Most all backslopes 1/4:1 - fractured. Weathering and bedding cause some slough.

.4

27.3 Tensleep Amsden formation contact.

.7

28.0 Old fill failure - rock fill on steep sidehill - occasional movement during spring of year - primary cause - surface water saturation of embankment foundation.

.2

28.2 Amsden-Madison Limestone formation contact.

4.2

32.4 Fallen City slide - Ordovician dolomite blocks in contact with wet Cambrian shales.

0.8

33.2 Contact Cambrian shales with Big Horn dolomite. Remainder of roadway to Burgess Junction is forested with Lodgepole pine and is located primarily on Precambrian granite and Cambrian shales.

8.3

41.5 Sibley Lake, a recreation area

4.9

46.4 View of Twin Buttes to northwest. Northwest dipping Ordovician Big Horn dolomite is the caprock.

1.0

47.4 Burgess Junction, Junction of highways 14 and 14A. Highway 14 Alternate to Lovell, Wyoming, Highway 14 to Greybull, Wyoming.

BURGESS JUNCTION TO LOVELL, Highway 14A

47.4 Junction Highways 14 and 14 Alternate
For the remainder of the field trip on the mountains, we will be traversing the Gros Ventre Formation of Cambrian age.
5.5 This formation consists of shale and thin, fractured limestone and sandstone beds. The valleys are capped by the Ordovician Big Horn dolomite which has been used for surfacing material on one section of roadway as has Precambrian granite.

52.9 Small slide area to the right - roadway was shifted away from hillside and grade was raised. No noticeable movement has
1.2 occurred since above corrections were made.

54.1 Large slide area to right and ahead - (Stop 1) History and correction of slide to be discussed. This slide as in the
4.0 others occurs within the Gros Ventre formation - shales and thin limestones - Note landslide topography and inherent problems throughout remainder of field trip.

58.1 (Stop 2) Backslope slide failure at right. History and correction of slide to be discussed. This portion of roadway was constructed in 1971.

3.8

61.9 Observation point (left) elevation 9,430 - Panoramic view of the Big Horn Basin - Little Bald Mountain north of highway. Across the basin to the northwest is the Bear Tooth Mountains, Absoraka Range to the west, and the Owl Creeks to the south.

7.4

69.3 Excellent view of slide topography - continual maintenance backslope and fill failures.

1.7

71.0 Observation point, view of Basin and road construction.

1.3

72.3 Lunch stop. Discussion of proposed roadway through canyon to south and related highway 14A slide problems. Rest room facilities available in F.A.A. building (Stop 3)

Buses will return to the Junction of Highways 14 and 14 Alternate.

ROAD LOG, JUNCTION 14A and 14 to GREYBULL

0 Burgess Junction - Highway 14 as on 14A will be traversing

1.2 Precambrian (granite) and Cambrian sediments as formerly discussed. Burgess Ranger Station to the north and Bear Lodge to the west.

1.2 Blue Spruce Campgrounds

6.8

8.0 Big Horn dolomite is the caprock in the ridge seen on the west skyline. Granite knobs are a conspicuous part of the topography, the Precambrian surface being above the road level.

1.5

9.5 Crossing Granite Pass, elevation 9,035

2.8

12.3 Antelope Butte ski area.

1.1

13.4 Smooth topography of valley to northwest is developed on Cambrian shales.

1.8

15.2 Slide area on the left -- Downed timber south of Shell Creek caused by a tornado.

2.7

17.9 Shell Slide (Stop 4) Discussion of geology, construction, and general history of this large slide.

A slide one mile ahead of this point can be generally discussed also.

2.6

20.5 Thompson Falls (Shell Creek) observation point. Good outcrop of the Flathead sandstone a basal Cambrian unit can be seen here.

Return to Sheridan, a stop of Fallen City observation point will be made, time permitting.

STOP 1

ENGINEERING GEOLOGY PROBLEM: Block Glide Backslope Failure

Preliminary studies indicated that a backslope failure condition would be created by this cut section as proposed. Elimination of the cut section by grade raise or a change in alignment would have also created additional problems due to the crossing of a then active slide in the adjoining proposed fill section. The final alignment was based also on the crossing of other potential failure areas.

The preliminary solution to the problem was to cut off the water flowing at the rubble shale contact in order to increase the strength of the soft shale sufficiently to resist the sliding forces. The method was to excavate that portion of the cut section to a point where the potential slide plane could be intercepted by an eight foot deep drain ditch. In order to maintain a relatively constant grade for a cut off drain, deeper excavation was required in central portion, and this was accomplished by means of a 1 on 1 slope trench to within eight feet of flowline (see figure D).

During installation of the underdrain, a backslope failure occurred which was caused by severing the slide plane with the underdrain ditch. The movement involved approximately one million cubic yards, with the scarp located 600-700 feet left of centerline.

When the failure occurred, 500 feet of the proposed 1200 feet of underdrain had been installed. Photographs of the area immediately after the slide took place are shown below.

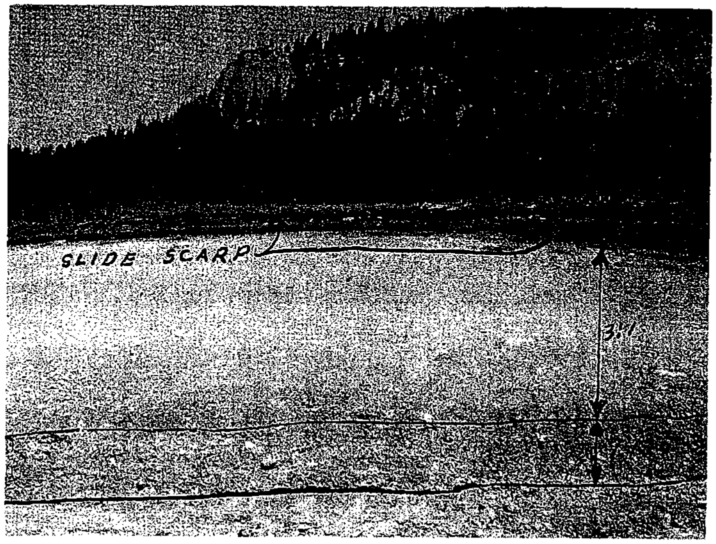


Figure A

Slide area showing scarp and cut slopes

Figure B

Note flat terrain of scarp area shown above

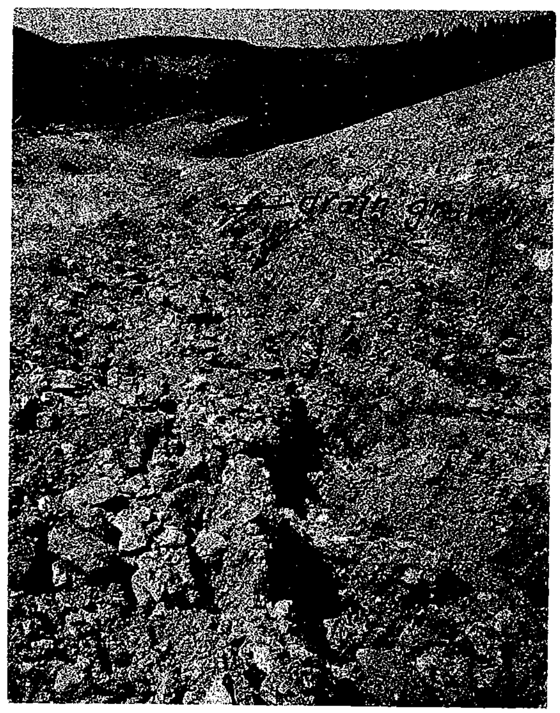
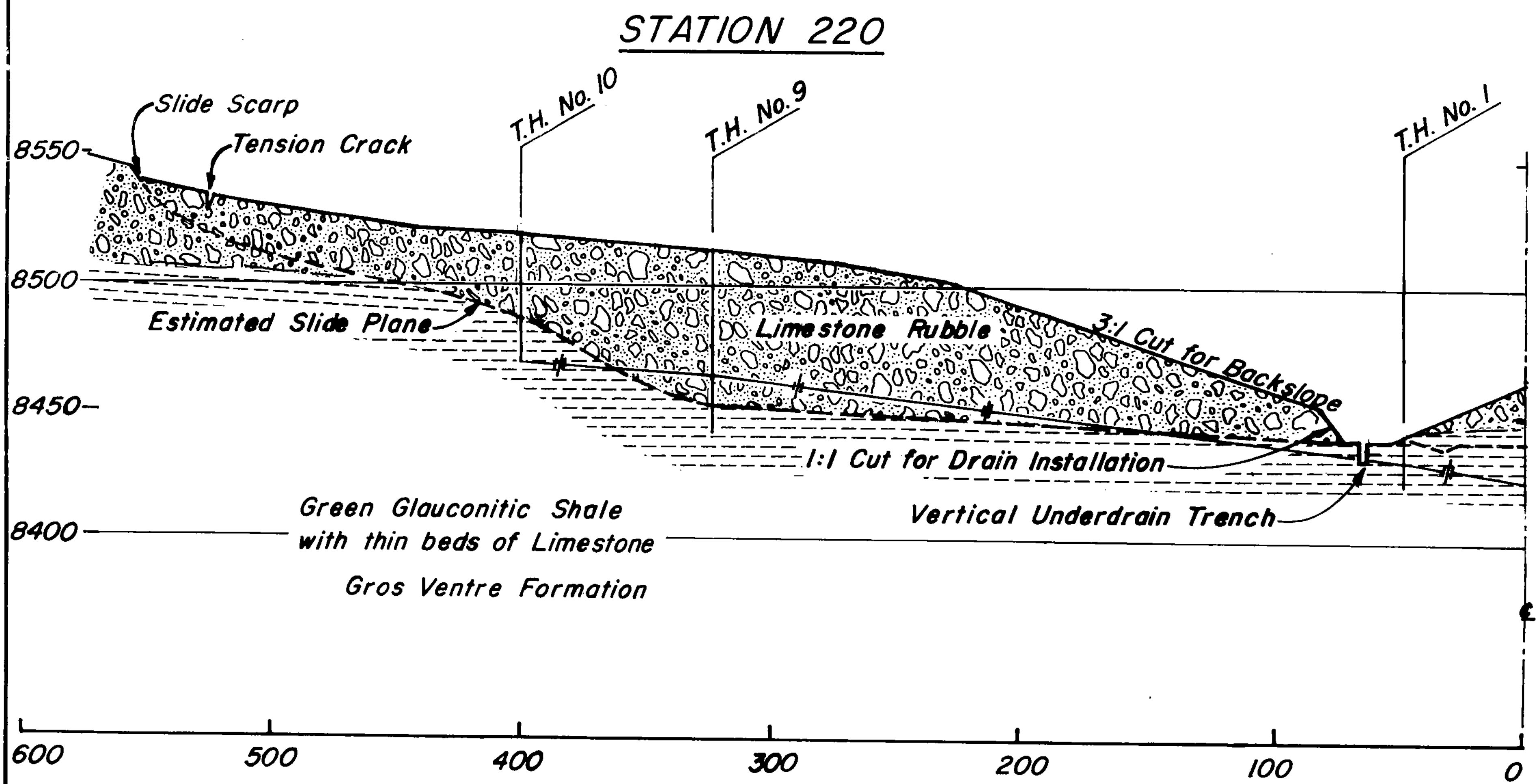


Figure C

Photograph above shows former location of underdrain trench closed by the slide. Drain gravel was forced up by compression in completed section.



CROSS SECTION THROUGH
CENTRAL PORTION OF SLIDE AREA

FIG. D

CORRECTIVE MEASURES: Three possible corrective measures were considered. These include: 1) installation of the remainder of the underdrain; 2) balancing the slide block by removal of additional material at the head of the slide; 3) a shift in the alignment coupled with a grade raise; and, 4) revision of the underdrain system. A decision was made in favor of the third alternate. The cut has been stable since completion in August, 1969.

STOP 2

ENGINEERING GEOLOGY PROBLEM: Backslope Slide in the Gros Ventre Shale

Backslope failure occurred during construction in the summer of 1971, after the road grade was established, with backslope design being 1 on 1. Material in the cut is highly fractured Gros Ventre shale with thin-bedded limestone. In place bedrock dips into the slope at about 10 degrees, while dips up to 25 degrees were measured in the slump block. Figure E shows the slide scarp and encroachment of the slide toe onto the roadway.

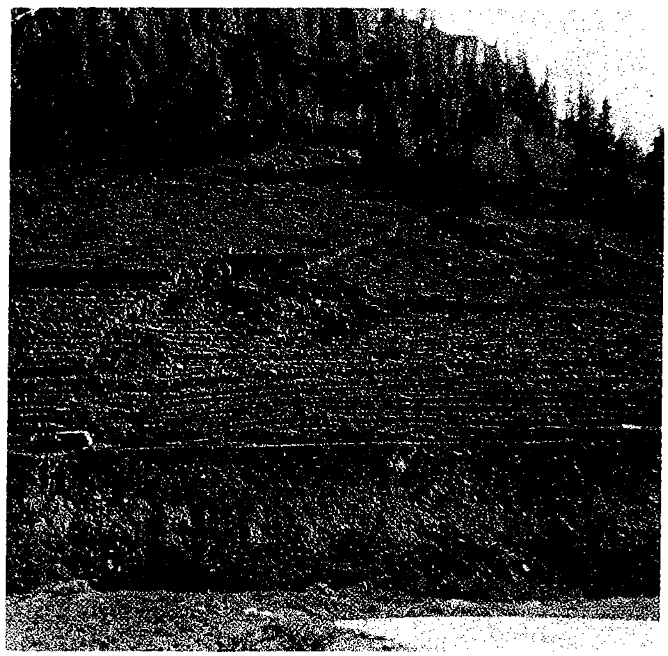


Figure E

Rotational slump in backslope. (A) Scarp, (B) Toe

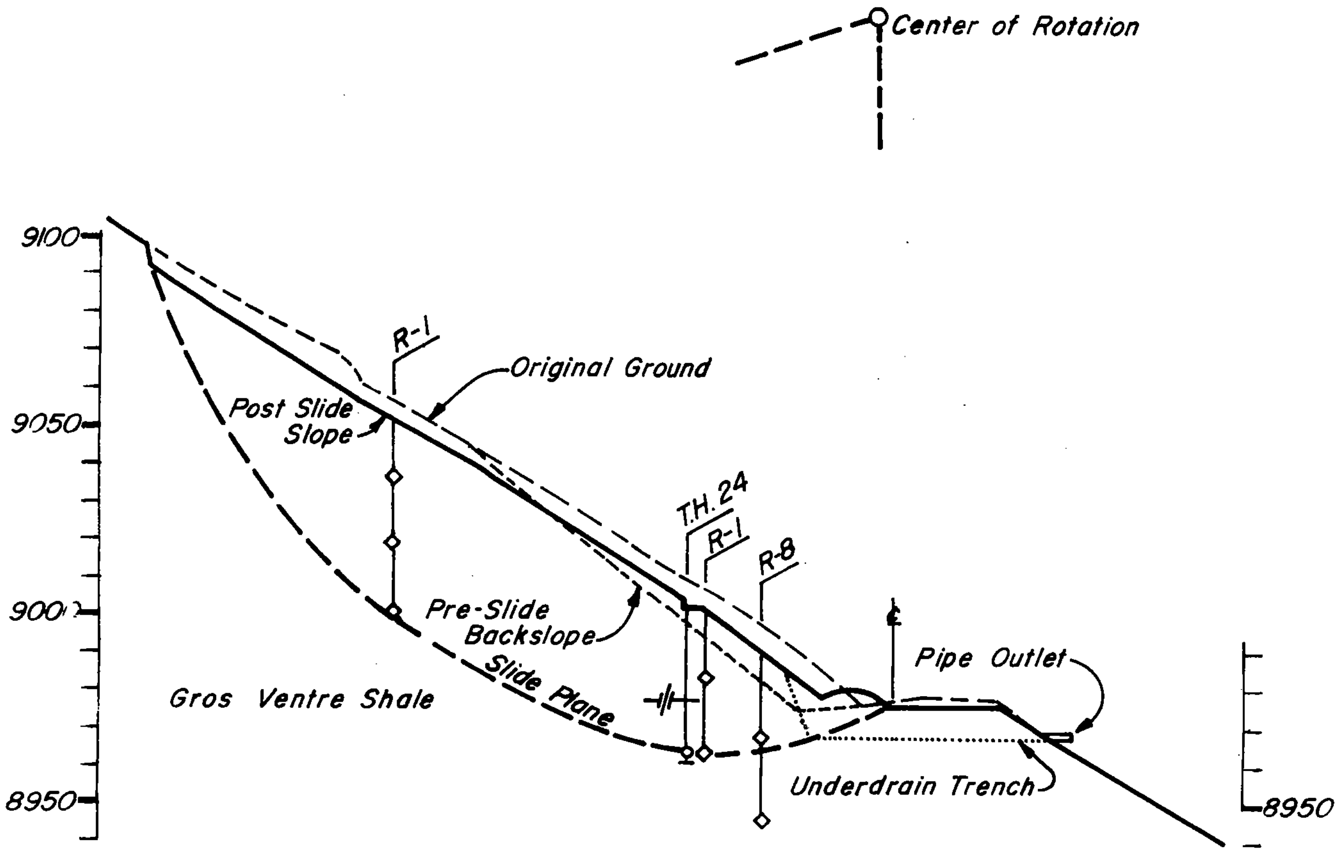
CORRECTION METHOD: Drilling and resistivity surveys defined the slump as illustrated in figure F as a rotational slide with the slide plane extending from the escarpment, downward approximately 50 feet into the ground, and then curving upward to the surface near the roadway centerline.

Ground water was encountered in test hole 24, with the piezometric surface at the time of the investigation near the level of road grade. Water was seeping from the slide toe at that time.

Prior to further road construction in the slide area, underdrains were installed to lower the water table and increase the cohesive strength of the shale, especially in the toe area of the slide. This was accomplished by placing four underdrains approximately 10 feet deep, perpendicular to the road, so that water would drain from the slump block to outlets right of centerline. Underdrains parallel to the road in the ditch section or backslope were not feasible because of the danger of caving while the drain trenches were being excavated.

The centerline of the roadway was shifted approximately six feet away from the backslope to minimize additional cutting of the backslope. However, after final dressing up of the backslope, a small slide movement reoccurred, and the ditch section was filled in as it is at the present time. The underdrains were installed in 1971, and apparently have dewatered the slump sufficiently to stabilize the slide.

Drilling will be done at the toe to determine the cohesive strength of the shale at the slide plane and ground water condition prior to reshaping of the ditch.



LEGEND

R-1	Resistivity Survey
T.H. 24	Test Hole
	Ground Water Surface
	Resistivity Break

FIG. F

CROSS SECTION OF ROTATIONAL SLUMP

STOP 3

ENGINEERING GEOLOGY PROBLEM

The proposed highway construction project viewed in part at this stop, is located on the west slope of the Big Horn Mountains. The new alignment avoids unstable slide areas to the north along the existing highway; although, problems concerning fill stability and backslipe design in jointed, hard rock (granite and limestone) arise in certain areas of the proposed alignment where line shifts to avoid these problems are essentially impossible (see figure G).

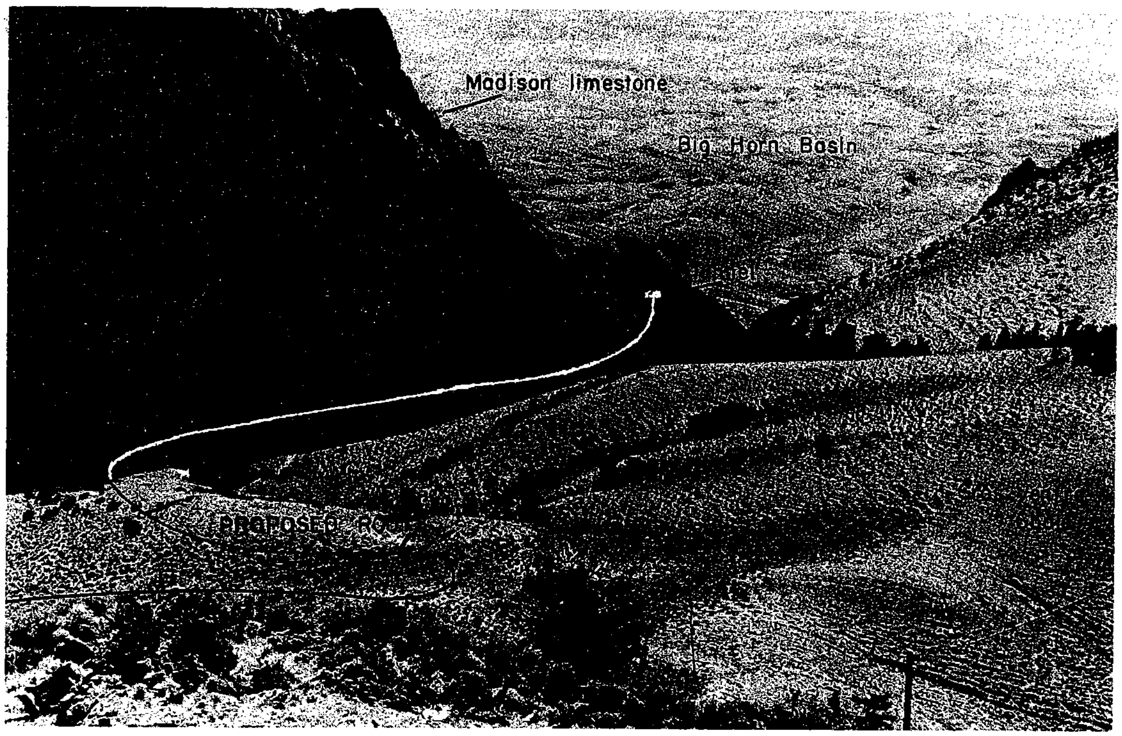


Figure G

View of proposed highway, looking west toward Big Horn Basin

Vertical or overturned limestone and dolomite of the Madison and Big Horn formations are complexly jointed. Principle joints or fractures are nearly perpendicular or oblique to the bedding planes. Many joints between the blocks of two adjacent layers are generally staggered at the boundaries between layers. Some movement has occurred along joint surfaces, and small displacement of joint blocks is evident (figure H).

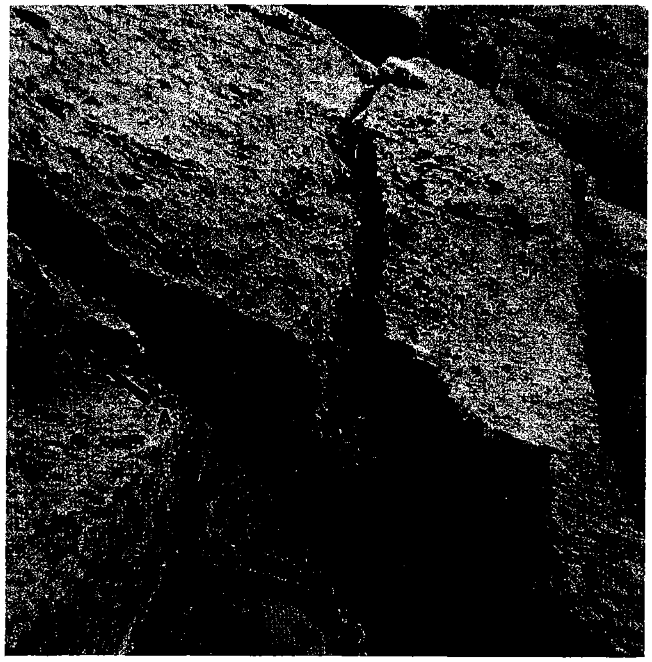


Figure H

Jointed Madison Limestone. Points A show displacement along fracture plane

The stability of backslopes on jointed rock in the vertical limestone section of the alignment depends more on the joint pattern than the bedding planes because the bedding is oriented at or near right angles to the slope. Of five prominent joint sets measured in the limestone, only one of these sets dips toward the roadway centerline, and its dip angle of 78 degrees controls the critical backslope angle whereby a 1 on 1/4 slope will have a safety factor of unity. Failure along the joint plane will occur if the backslope is steeper than 1 on 1/4, while the safety factor will increase as the backslope design is flattened within the limits of the natural slope angle.

Wedge-type sliding occurs when two joint planes form a line of intersection that dips toward the roadway centerline at an angle less than the backslope design. This type of failure is possible in the limestone rib section of the project where three separate lines of intersection dip toward the roadway at angles ranging from 47 to 70 degrees. Backslope design of no steeper than 1 on 1 would be required theoretically to avoid wedge-type slide failure. But, actual movement would depend on the continuity of joint planes and the undulations on the joint surfaces. Field observations and studies of aerial photographs of existing, steep, natural slopes up to 1 on 1/4 indicate that staggered joint planes and uneven surfaces contribute to the stability to the degree that wedge-type sliding would be minimal.

CORRECTIVE METHODS: Selective rock bolting of jointed limestone blocks in backslopes would further reduce the risk of rock slides and toppling of individual blocks. Preliminary rock bolt design is feasible based on the joint investigations already done. It will be desirable in the limestone rib area to place 15 feet long rock bolts in the backslope face approximately every 20 feet of lateral distance in the space of 15 foot lifts. Modification of the preliminary design may be necessary after the cut is opened, and on-the-job selection of rock bolt drill holes by the engineering geologist can be anticipated.

Because of the steepness of the natural ground in the interrib areas, standard fills are not feasible, as the fill slope would be steeper than 1 on 1, or if flattened, fill slopes would extend to the canyon floor of the South Fork of Five Springs Creek. Several solutions to establish a satisfactory roadway in the interrib areas include bridging or building fill retaining walls. Problems of constructing bridges, let along the cost of such structures, demanded a look at other solutions, such as bin walls or similar type materials. However, because of fill heights occasionally in excess of 40 feet, bin walls were not considered feasible.

The relatively new concept to our country of maintaining high fills with Reinforced Earth has been investigated quite thoroughly by the Wyoming Highway Department, and this method is considered to be the most practical solution to hold fills in areas of the project where fills must be placed on steep ground slopes (figure I).

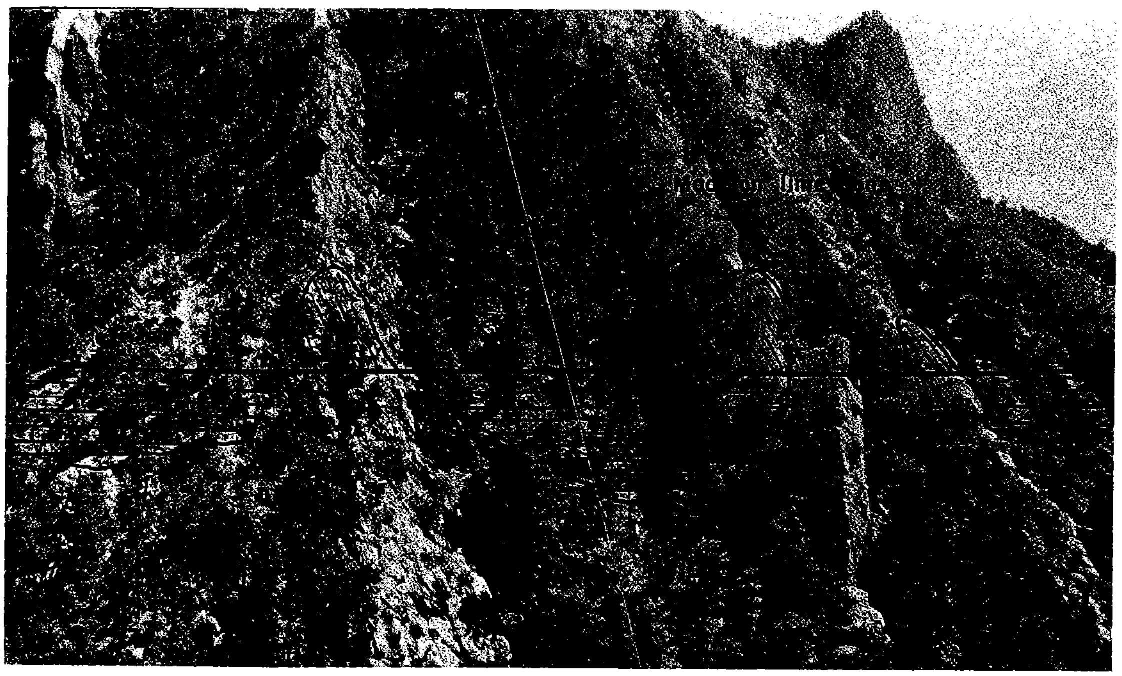


Figure I

Limestone rib area (east of tunnel, figure G) showing backslope cuts (oblique lines) in jointed limestone beds where rock bolts will be required. Intervening areas will be spanned by Reinforced Earth fill (horizontal lines).

SHELL CANYON SLIDE

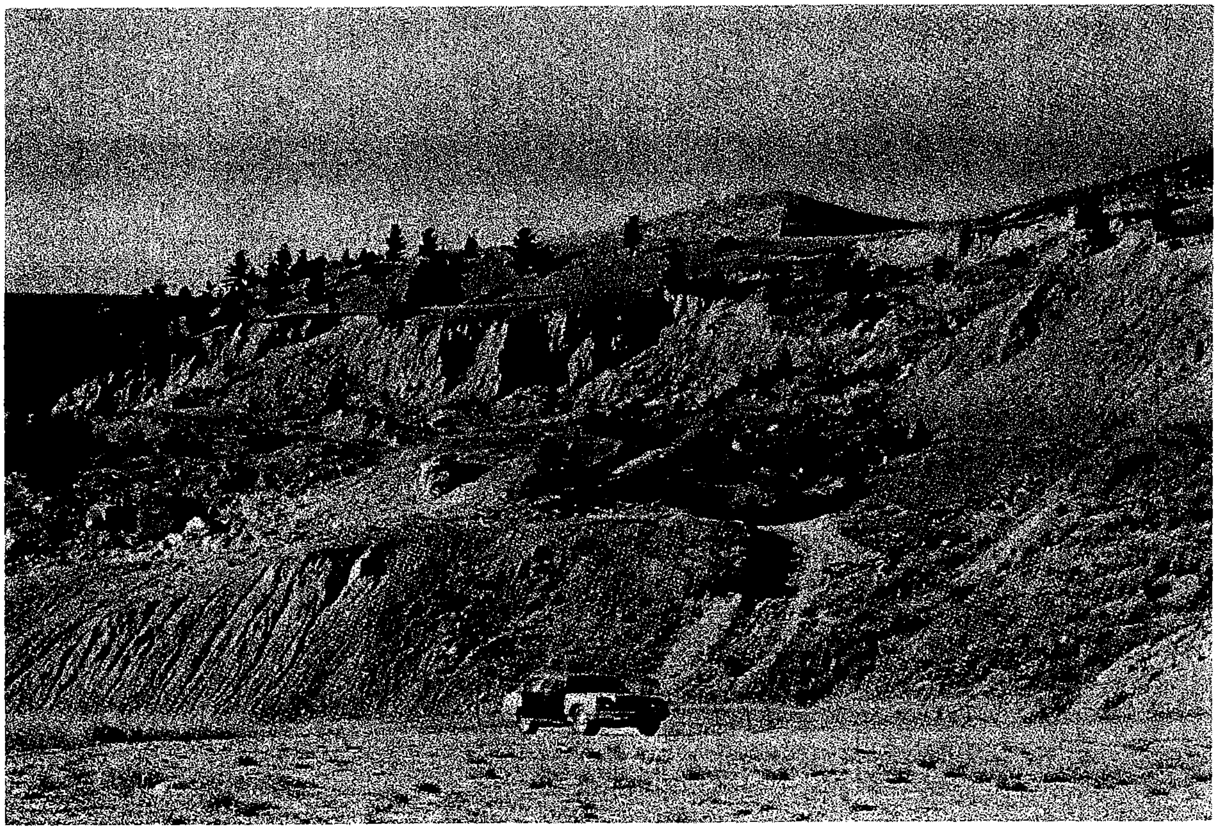


Figure J

Shell Canyon Slide Area

This stop affords a good view of the Shell Canyon slide area which is located on the southwestern flank of the Big Horns. The slide, which failed during road construction, lies within an older, very large slide zone. The formations involved are of Cambrian age as in other areas visited. The discussion at this stop will include the history and the remedial action taken during and subsequent to construction.

If time permits, another slide area approximately a mile southwest of this stop will also be discussed at this time.