TRANSPORTATION RESEARCH RECORD

482

Formerly issued as Highway Research Record

Landslide Instrumentation

5 reports prepared for the 53rd Annual Meeting of the Highway Research Board

subject areas
61 exploration-classification (soils)
62 foundations (soils)
63 mechanics (earth mass)

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
Washington, D. C., 1974

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TRR 482
ISBN 0-309-02268-1
LC Cat. Card No. 74-6879
Price: $1.80

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CONTENTS

FOREWORD

The importance of a knowledge of landslides in the development of transportation systems was early recognized by the Highway Research Board by establishment of the Committee on Landslide Investigations in 1951. This committee prepared and published the first major book in the English language on the recognition, analyses, and design of control methods for landslides (Landslides in Engineering Practice: Highway Research Board Special Report 29, 1958). It was a timely publication, because with the development of the Interstate Highway System the required standards for width, curvature, and grade increased the size of cuts and fills, which resulted in an increase in the number of landslides, and the probability of slope failure, along the highway system. The early analyses and design of remedial measures were based primarily on field mapping, drilling, and laboratory analyses of samples. In recent years methods have been developed through instrumentation for the measurement of properties of landslide materials in situ and for monitoring the behavior of landslides, or areas of potential slope failure, with time.

The papers in this RECORD are the result of recent developments in the study of landslides. The Engineering Geology Committee—successor to the Committee on Landslide Investigations—requested leaders in the field of the study of landslides from industry and state and federal agencies to present papers on modern methods of instrumentation of landslides. Five papers were selected for presentation at the 1974 Annual Meeting and for publication. The papers were selected to give as wide coverage as possible of different types of instrumentation in different geographic areas and with different geologic conditions.

The types of instrumentation described are the use of survey grid systems, slope indicators, borehole extensometers, piezometers, and in situ shear-test devices in California, Colorado, Oregon, Minnesota, and the southeastern United States. The types of landslides instrumented include bedrock failures along joint surfaces, reactivation of prehistoric landslides, bedrock and surficial deposit cut slope failures, and embankment failures. These papers present typical case histories and illustrate the importance of instrumentation in the investigation and definition of the extent and physical characteristics of landslides, the collection and interpretation of instrumentation data for the design of remedial measures, the importance of instrumentation during construction in unstable areas, and the monitoring of slope behavior after construction.

—Charles S. Robinson

THE ROLE OF FIELD INSTRUMENTATION IN CORRECTION OF THE "FOUNTAIN SLIDE"

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The reliability and usefulness of field instrumentation in providing design data for landslide correction are clearly illustrated in the case history of the Fountain landslide. The area known as the Fountain Slide affects an approximately ¾-mile (1-km) section of I-80N just north of Mt. Hood about 45 miles (72 km) east of Portland, Oregon. This section of highway is flanked on one side by a railroad and the Columbia River and on the other side by the unstable and ancient landslide, which extends for several thousand feet (more than 1 km) to a sheer rock face some hundreds of feet (more than 200 m) above the lower slopes. Construction of I-80N required cutting into the unstable area a substantial distance and resulted in increased slide activity. Since the construction of the Interstate, several efforts have been made to stabilize the Fountain Slide. None have been totally successful, largely because the cause of the failure was not fully understood. In 1968 field instrumentation was installed that made it possible to carefully define the failure surface and actual groundwater conditions. This information was used to design a slide correction scheme. Construction of a $3.2-million correction scheme, which was scheduled to start in the early spring of 1973, has been delayed by further areal extension of the slide mass. The instrumentation program has been expanded to include the new unstable area. The feasibility of relocating the highway into the Columbia River or a do-nothing alternative with continued maintenance and a warning system are also being studied. The field instrumentation is being continually monitored to ensure proper design input for whatever remedy is selected.

•HIGHWAYS that are built through old landslide areas require much from the engineer. He must determine the effects of his proposed construction on an already marginal condition and show that his design will ensure adequate safety against future instabilities. This degree of precision calls for development of field instrumentation to monitor ground movements in order to define design constraints.

Many field instruments that do exactly this have been developed. Two types of instruments utilized in the analysis of a particular landslide problem are the slope inclinometer and piezometer. The slope inclinometers installed were the Slope Meter and Slope Indicator types. The piezometers were of the pneumatic type.

The area in question is located along the Columbia River in Oregon about 45 miles (72 km) east of Portland. It has been a known landslide area for many years. The first roadway was constructed over the slide in the early 1920s, and the present highway, which is part of Interstate Route 80N, the major east-west highway in Oregon, was completed in 1968. Figures 1 and 2 show the area as it existed in 1968 and in 1973. The photographs illustrate the progressive type of landslide movement that is so common throughout the Pacific Northwest.

During construction, a significant amount of material was removed to widen an existing facility to a 4-lane divided highway. This involved cutting into the unstable hillside a substantial distance. As a result, movement became a much bigger problem. This required the removal of 160,000 yd^3 (122,000 m^3) of material to reduce the driving forces and attempt to stabilize the slide activity that was taking place.

In 1968 the first set of slope inclinometers was installed to define the failure surface and groundwater conditions. During the initial field instrumentation installations, two types of slope inclinometers were used—the Slope Indicator and the Slope Meter. The only available Slope Indicator casing at this time was approximately 3.5 in. (9 cm) outside diameter, and this required the drilling of two holes at each Slope Indicator location. The reason was that the subsurface exploration required the use of a core drill that is of a smaller diameter than the 3.5 in. (9 cm) required for the Slope Indicator. The Slope Meter requires a $1\frac{3}{4}$-in. (4.4-cm) square steel casing, and this eliminated the need for additional larger diameter holes. "H" series wire line drill tools were used, through which the square steel casing could be placed after completion of the core drill hole. A substantial drilling cost savings was realized during this initial work by switching to the Slope Meter. The later development of the smaller Digit-Tilt by Slope Indicator would have eliminated the need for the extra drilling.

Unfortunately, slide activity since completion of the project has required extensive maintenance work on the highway and railroad grade. During periods of movement, shifts in the railroad grade or alignment were corrected on a day-to-day frequency. Figure 3 shows the many inclinometers that have been installed because of the magnitude of movements and the scarp as it exists now along with the landslide limits of 1968. The present total area of sliding extends along the highway approximately $\frac{3}{4}$ mile (1 km).

SUBSURFACE CONDITIONS

Subsurface investigation work began in June 1968, and inclinometer casing was installed at each of the test boring locations so that water levels could be measured and slide movement monitored using Slope Meter and Slope Indicator equipment. Transit traverses were also run along the existing highway and railroad to measure surface movements. The slide mass is primarily a talus-like material ranging from a vesicular coarse-grained andesite to a fine-grained basalt. Matrix material varies from silts and sandy silts to sandy silty clay. Lenses or zones of siltstones, claystones, sandy-pebbly-rocky siltstones and sandy-pebbly-rocky claystones occur throughout the sliding mass. As shown in Figure 4, the degraded claystones and siltstones have been grouped together under the term "clay" or "mudstone". Weathering of this "mudstone" resulted in a weak zone within the subsurface profile. The instrumentation indicates that the slide movement is occurring within or along the surface of the clay-mudstone layer. Shear zones were evident on all the core samples obtained from the clay-mudstone layers.

The slide is moving along generally uniform slip planes with inclinations ranging from 4 to 16 deg from the horizontal. As Figure 4 shows, the slope inclinometers indicate a classic example of planar movement. The depth of movement varies throughout the landslide area, with the deepest movements 200 ft (61 m) at boring hole number SM-7. Lateral movement was quite restricted vertically in many of the inclinometer casings. Most of the bending took place on a 2 to 4-ft (0.6 to 1.2-m) interval that corresponded to the thickness of the clay-mudstone layer. The movement was generally in a northerly direction toward the Columbia River.

INSTRUMENTATION DATA

To date a total of 63 slope inclinometer casings have been installed. Many graphs have been prepared from Slope Indicator and Slope Meter data relating lateral movement to time and rainfall. Because of the excessive amounts of these movements, most of the slope inclinometer casing installed during the initial exploration has been sheared off. Prior to 1970, measured movement was at a rate of 2 ft (0.6 m) per year horizontally and 1.5 ft (0.5 m) vertically. Resurfacing of the existing pavement required a horizontal line shift of up to 5 ft (1.5 m) in some areas. This resulted in removal of some of the existing rock buttress and explains the increased rate of movement that occurred shortly thereafter. As can be seen from Figure 6, the rate of movement differed considerably throughout the area of sliding.

Figure 5 is representative of the lateral movements that have taken place from 1968

Figure 1. 1968 aerial photo showing existing landslide limits.

Figure 2. 1973 aerial photo showing existing landslide limits.

Figure 3. Plan of Fountain Slide showing instrumentation locations and limits of landslide. Contour intervals are 50 ft (15.24 m). SM = Slope Meter or Slope Indicator; OW = observation well; VD = vertical drain; DH = drill hole.

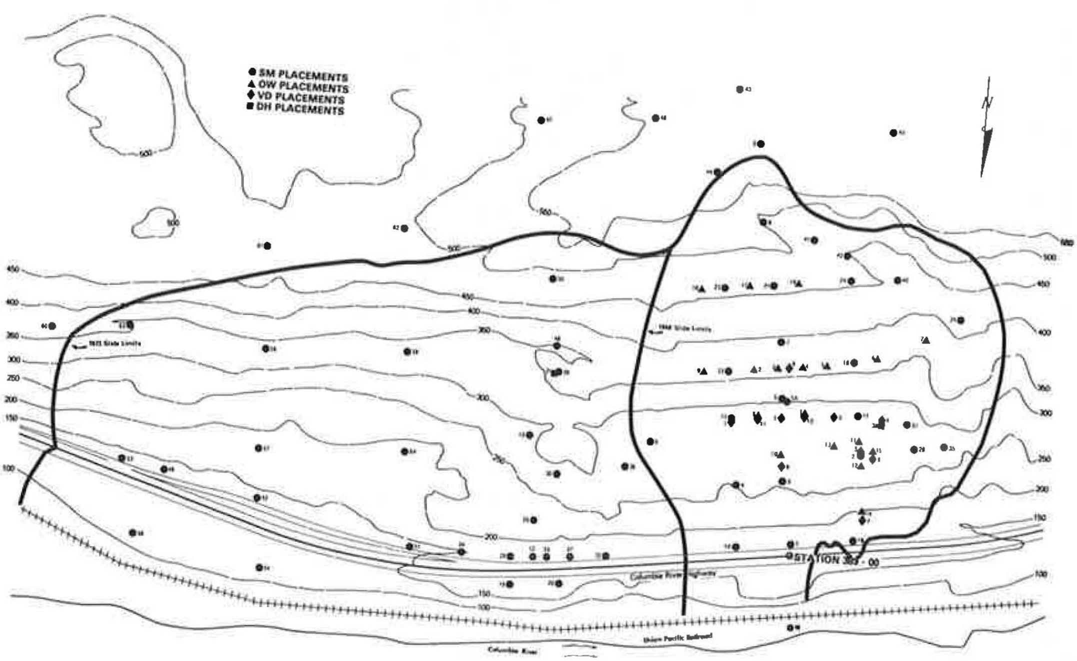
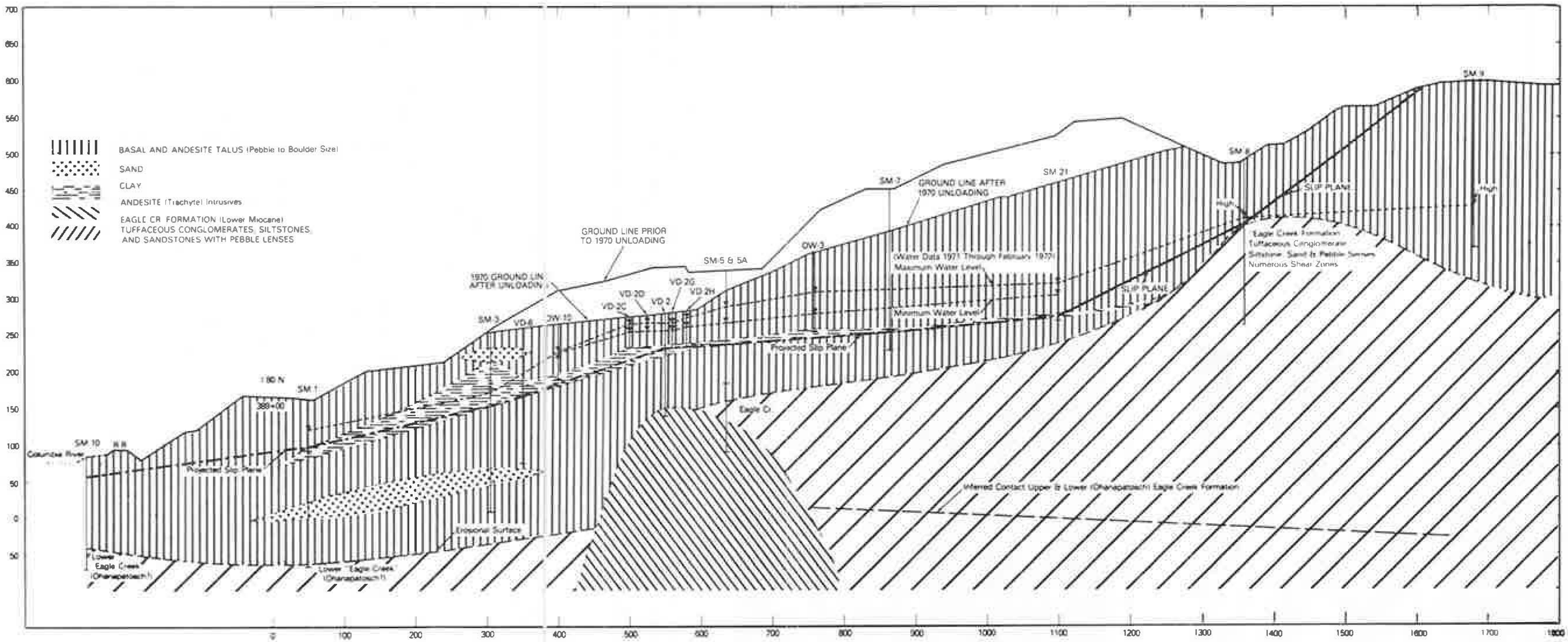


Figure 4. Cross section of Fountain Slide showing soil profile and slip plane as defined by slope inclinometers. Horizontal and vertical scales are in feet (1 ft = 0.3 m). SM = Slope Meter or Slope Indicator; OW = observation well; VD =vertical drain.



to 1972. Several slope inclinometer plots have been superimposed on the same graph to show the different types of plots from different areas of the slide. An observation that can be made from the plot is that when the movement was within a 0.5-in. (1.3-cm) range, the movement was pretty well confined to the soft zone, but once the magnitude of movement went beyond that range, the mass above the sliding plane would show uniform movement equal to the movement at the soft zone.

Measurements of horizontal and vertical displacement along the existing highway and railroad were also made and correlated with the field instrumentation data. During a 1-year period (1968-69), a horizontal displacement of approximately 2 ft (0.6 m) along the highway and 1.5 ft (0.5 m) along the railroad occurred. Vertical displacement along the highway and railroad for the same period of time varied from 0.68 to 1.56 ft (0.21 to 0.48 m) respectively. Figure 6 shows the horizontal movement that occurred on a section of the Interstate from February 5, 1968, to January 20, 1969. As would be expected, the areas showing the greater horizontal movement were the areas where the slip plane inclination was greater. It was generally established that slip plane inclinations greater than 12 deg from the horizontal showed a definite higher movement rate. These are the areas where the railroad grade and alignment must be corrected on a day-to-day basis during the rainy season when the rate of movement is high.

Figure 7 correlates the rainfall and the corresponding movement in the slope inclinometers. The graph was put together by assembling and correlating monthly slope inclinometer readings for several instruments for the period between 1970 and 1972; therefore, the plots are not actual for any one year. They indicate what has always been obvious and what many engineers have stated in the literature: The increase in sliding movements is a direct function of increased rainfall activity. Groundwater conditions throughout the slide mass do not conform to any uniform pattern; therefore, groundwater conditions varied throughout the landslide area for stability analyses. Seasonal fluctuations at the boring locations have been very erratic, which gave an indication of the variable permeability throughout. There are many zones of trapped or perched water tables through the sliding mass.

REMEDIAL DESIGN

Since completion of the Interstate Highway in 1968, several attempts have been made to stabilize the sliding mass. In December 1969, a contract was awarded for the removal of 1,700,000 yd^3 (1,300,000 m^3) of the sliding soil matrix, plus provision for drainage by installation of horizontal drains (Fig. 2). This work was completed in the fall of 1970 at a cost of approximately \$1.7 million. Eleven slope indicators were installed to monitor and assist in evaluating the stabilizing effect of the unloading work.

Instrumentation data showed that a movement pattern then developed in which ground movements occurred in the fall and winter during the rainy season and stopped during the dry summer months (Fig. 5). Movement had occurred both summer and winter prior to the 1970 unloading, which indicated that the unloading work had some stabilizing effect. Field instrumentation detected continued ground movements, and a decision was made to further attempt to drain the slide. The horizontal drains that were installed earlier gave very poor results; therefore, vertical test wells were proposed before proceeding with a well system to drain the slide. Four vertical test wells were installed during June 1971. Eight piezometers were installed around each of the vertical test wells to monitor groundwater fluctuations. Boring data had indicated that permeable talus material existed below the impermeable soft clay material (Fig. 2). The idea was to provide an escape for the trapped water through the impermeable clay into the permeable talus material below the slip plane.

An 8-in. (20-cm) unperforated casing was installed to the soft clay unit. A perforated 6-in. (15-cm) casing was then installed through and below the impermeable clay unit. Pump and bail tests were run on all the wells, and the results were as follows: well 1, bail test, 1 gpm (0.004 m^3/min); well 2, pump test, 45 gpm (0.17 m^3/min); well 3, bail test, 1 gpm (0.004 m^3/min); and well 4, bail test, 1 gpm (0.004 m^3/min).

Based on these test results, it was concluded that the material in the areas of wells 1, 3, and 4 was too impermeable for the wells to function effectively. The piezometers

Figure 5. Typical slope inclinometer plots (1 in. = 2.54 cm).

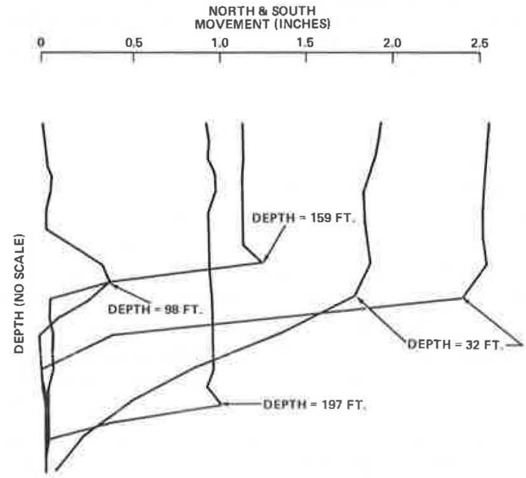


Figure 6. Highway horizontal movement.

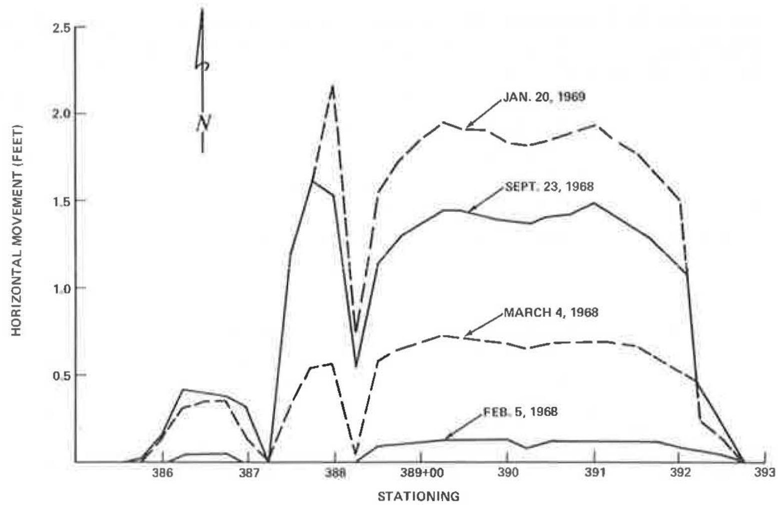
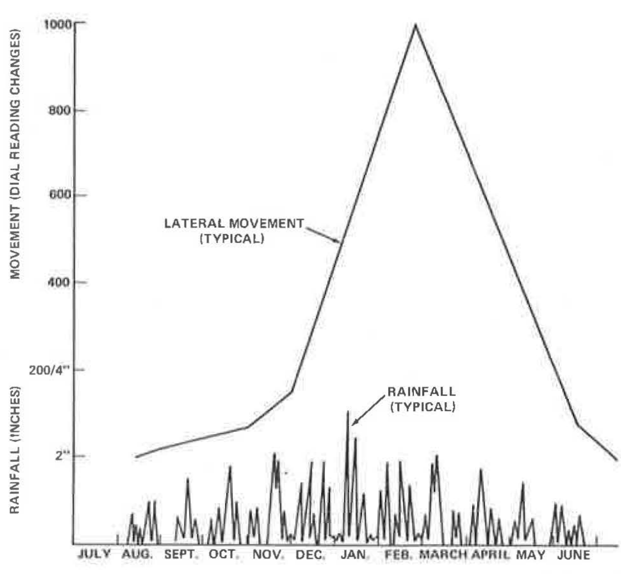


Figure 7. Rainfall versus movement.



in the area of well 2 indicated that the water level was lowered some 8 ft (2.5 m). Based on this one good result, it was decided to install 7 additional wells before a decision to abandon this method of drainage could be made. This drainage work was completed in October 1971 and included a total of 11 vertical wells, 48 piezometers, and 11 more slope inclinometers. Except for a few wells, the drainage worked poorly because of the very low permeability of the slide debris. Therefore, the number of wells that would be required to dry up the slide would be many more than was originally anticipated. Other disadvantages to the well system were that the annual maintenance on the pump system would be high and further movement could destroy the wells. This alternative was therefore abandoned.

During the period from November 1971 to February 1972, the slope inclinometer movement ranged from 2 to 7 in. (5 to 18 cm), depending on the location of the tube. This movement clearly defined the slip plane, as shown in Figure 4. With the location and angle of the slip plane now clearly defined, a stability study was undertaken and an elaborate remedial scheme proposed. This scheme proposed to (a) reduce the driving forces at the head of the slide by unloading where the inclination of the failure surface was greater than the angle of internal friction and (b) construct drainage trenches perpendicular to the highway centerline. The purpose of the trenches would be to improve drainage and accommodate construction of a rock buttress. The cost for this remedial scheme was estimated at $3.2 million, and construction was scheduled to begin in June 1973. During March 1973, surface observations indicated slide activity approximately 2,000 ft (600 m) east of the slide limits that were used for the remedial cost estimate. This newly observed areal extension of the landslide limits approximately doubled the sliding mass. Fifteen slope inclinometers were installed during April to June 1973 in the new area of sliding to monitor the movements and determine the slip plane. Because of the size of this sliding mass, a decision was made to wait on the proposed remedial work until the new area of sliding could be fully evaluated.

Three alternatives are under study by the Oregon State Highway Division. The alternatives are as follows:

1. Follow the remedial scheme as described above at a cost of $3.2 million and plan for remedial work on the new area of sliding once the field instrumentation data have been evaluated.
2. Consider the feasibility of moving the highway and the railroad location north into the Columbia River. The cost of this alternative is quite high (approximately $10 million) and, with the new environmental policies, this alternative seems unfeasible. The state is also checking into the possibility of relocating to the south; the estimated cost is also high and the route would be through other potential slide areas.
3. Consider the feasibility of maintaining the present location with no remedial work. An instrumentation system is under study whereby a warning signal would be given at a predetermined movement rate to warn the motorist of any impending danger. Maintenance cost figures are under study for a cost comparison with the other alternatives. If further study and monitoring of the instrumentation determine that the slide is in an advanced stage of development and some prediction may be made on when it will develop into a condition of equilibrium, this alternative may turn out to be the direction to follow.

CONCLUSION

In this paper, we have presented a case history that demonstrates the use of field instrumentation to study the progressive movements of the Fountain landslide. Data from the field instrumentation were used to define the failure surface and give a better understanding of the cause of the failure. The situation is now a matter of evaluating the three possible alternatives and making a decision on the remedial scheme. A total of 63 slope inclinometer casings have been installed.

The following observations have been made: In landslide areas such as this, where the major cause for movement is "trapped water" caused by underground streams and fractures in the bedrock, the remedial design is extremely difficult. With the present state of the art on subsurface exploration, the only means of locating this trapped water is by vertical borings. In areas as massive as the Fountain Slide, this becomes a nearly impossible task. Some drainage was very effectively placed as a result of being carefully located by borings, but many more areas of trapped water still exist, as is evident by the continued movement. If all the drainage paths providing water to the sliding mass could be located and adequately drained, the slide would undoubtedly be stabilized.

Landslide corrections are difficult to design and costly to correct; however, without proper means of identifying the depth and extent of such movements as well as the various soil, rock, and water parameters, a remedial method having any degree of reliability would be improbable.

One must always remember that instrumentation used in soil and rock mechanics is best utilized if it is simply constructed and easily observed. Very sophisticated approaches to instrumentation often lead to breakdowns during installation or difficulty in calibration during operation. It is indeed a challenge to seek better, but simpler, approaches to field instrumentation methods.

SLOPE INSTRUMENTATION USING MULTIPLE-POSITION BOREHOLE EXTENSOMETERS

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An unstable slope encountered during construction posed a serious hazard to men and equipment in the lower dam area of the Cabin Creek hydroelectric complex in Colorado. Following a series of rock slides, remedial work was undertaken to reinforce and stabilize the slope. As a part of the remedial program, multiple-position borehole extensometers were installed to permit the measurement of slope behavior and to provide safety monitoring for the protection of activities and facilities in the downslope construction area. The extensometers were read continuously when men and equipment were in the downslope area and intermittently at other times. All readings were immediately scanned to identify unusual or accelerating slope deformation. The readings were later processed by computer to permit a more detailed analysis of the nature, distribution, rate, and acceleration of displacements in each drill hole and an interpretation of the general behavior of the slope in the unstable area. Early in the Cabin Creek program, a method was developed for the comparison of displacement and displacement acceleration information by means of which surficial slumping and rebound influences could be isolated from other responses. The same comparisons were then used to identify periodic or continuing events related either in time, in space, or in both time and space. A final detailed interpretation of overall slope behavior was made on the basis of these space-time relationships. The Cabin Creek instrumentation illustrates the use of an economical and reasonably unsophisticated measurement program in support of other engineering activities in the diagnosis and solution of a serious construction problem.

THE Public Service Company of Colorado Cabin Creek Project is located in the Rocky Mountains about 35 miles (56 km) west of Denver. The project, constructed in 1964-1966, consists of a 325-MW power generating-pumping plant, located at an elevation of about 10,000 ft (3050 m), and two embankment dams impounding reservoirs at about 10,000 and about 11,200 ft (3050 and 3400 m) (Fig. 1). The upper reservoir and the generating-pumping plant are connected by a 4,300-ft (1280 m) pressure tunnel excavated in rock. The generating-pumping plant discharges directly into the lower reservoir.

During periods of peak power demand, power is generated by the release of water from the upper reservoir, through the generating-pumping plant, into the lower reservoir. During periods of low power demand—usually late at night—the turbine-pumps are reversed and water is pumped out of the lower reservoir and back into the upper reservoir. The pumping operation requires about 356,000 horsepower, which is provided by Public Service Company of Colorado's steam generating plants in the Denver area.

The lower Cabin Creek dam is an earth and rock fill embankment with a sloping, compacted, impervious core and an impervious upstream blanket. Crest length is 1,180 ft (359 m), and height above bedrock is a maximum of about 80 ft (24 m). The dam provides about 1,800 acre-feet of usable storage. The upper dam, also an earth and rock fill embankment, impounds approximately 1,400 acre-feet of storage. Crest length is 1,490 ft (454 m), and maximum height above bedrock is about 200 ft (60 m). The interconnecting pressure tunnel averages about 14 ft (4 m) in diameter and is concrete lined, with an additional steel liner in the lower section. Net maximum head in the upper reservoir-tunnel-generating plant system is 1,190 ft (363 m).

GEOLOGY AND GEOGRAPHY

The Cabin Creek complex is located in one of a prominent series of glaciated, north-south trending valleys near the west edge of the Colorado Front Range. The lower dam, reservoir, and generating-pumping plant are located on the floor of the valley. The upper dam and reservoir are located in a small hanging valley tributary to the main drainage. The main valley—the valley of the South Fork of Clear Creek—is moderately U-shaped in profile and ranges from a few hundred feet to perhaps 2,000 ft (approximately 75 to 600 m) in width in the general vicinity of the project. Several old rock slides are evident, at least two of which temporarily blocked the valley, forming lakes in which varying sequences and thicknesses of stream sediments, lakebed sediments, and organic sediments were deposited. The small hanging valley—the valley of Cabin Creek—enters the main valley from the southwest at a point more than 1,000 ft (305 m) above the valley floor.

Bedrock in the project area consists primarily of Precambrian quartz monzonite and hornblende gneiss, cut by scattered pegmatite dikes. The rock is hard and durable except where subject to local hydrothermal alteration. The altered zones, which are few in number and small in areal extent, are characterized by a general softening of the rock and by the development of clayey textures in the most intensely altered materials—generally along particular fractures and at complex fracture intersections. In addition to the landslide and lake sediments previously noted, the valley floor and walls are covered, from place to place, by varying thicknesses of outwash sediments and glacial moraine.

Each valley wall is paralleled by a pronounced set of joints, which exercise a broad control over the attitude of the slopes. These joints, attributed to stress relief occasioned by the melting of the valley glaciers, appear to become more widely spaced—and finally to disappear—at depth behind the valley walls. Other identifiable joints include a set striking parallel to the valley and dipping steeply toward the west, a set striking parallel to the valley and dipping gently toward the west, and a set striking more or less east-west—across the valley—and standing vertically. Foliation is pronounced in the gneiss, with laminae predominatly oriented north-south, dipping 60-80 deg west.

ROCK SLIDES IN THE LOWER DAM AREA

One of the first steps necessary in the construction of the lower dam was the relocation of an existing county road. The road was moved from the valley floor to a new alignment about 75 ft (23 in) above the floor on the west side of the valley. During the relocation, several small rock slides were encountered in a small area near the west abutment of the lower dam and almost directly upslope from the intake portal of the lower dam spillway tunnel. The slides were minor in extent and did not appear to reflect larger movements. The hillside was graded to an apparently stable 3/4:1 slope, and construction of the lower dam and spillway tunnel proceeded as planned.

The spillway tunnel, which is approximately 14 ft (3 m) in diameter and 700 ft (210 m) long, was excavated in competent rock with a minimum of difficulty. Following the completion of excavation, work was begun on a reinforced concrete intake structure at the upstream tunnel portal. During construction of the intake, a substantially larger rock slide occurred without warning. The slide, which started directly upslope from the relocated county road, was approximately 250 ft (76 m) wide, extended about 200 ft (61 m) up the slope from the road, and contained an estimated 85,000 yd^3 (65 000 m^3) of rock. The slide overflowed the county road and covered the spillway intake structure with approximately 40 ft (12 m) of debris.

Immediately after the slide an engineering investigation was undertaken to evaluate the implications with respect to the entire project. Had such a slide occurred in the same or a comparable location when the reservoir was full, a serious safety hazard would undoubtedly have resulted in the downstream area. Consequently, it was necessary to completely review the overall safety aspects of the location of the lower dam, the design of the spillway intake structure, and the alignment of the relocated county road.

As a result of the investigation, a remedial program was developed consisting of seven principal steps:

1. Removal of the slide debris and reshaping of the slope to the most favorable possible attitude.
2. Installation of rock bolts to varying depths in the slope, the bolting to be accomplished as the slide debris was removed and the slope reshaped.
3. Diversion of surface drainage away from the slope, particularly in the area immediately above the slide. This was accomplished by the relocation of one small stream to drain to the downstream side of the dam, by the construction of paved drainage ditches upslope and downslope from the slide, and by drain holes drilled in the slope itself.
4. Redesign of the spillway intake structure to withstand greater earth loads and to protect valve controls and operators from falling rock.
5. Relocation of the inlet to the spillway structure 150 ft (46 m) out into the reservoir. This was accomplished by using two 6-ft (2-m) diameter steel pipes.
6. Construction of a load surcharge at the toe of the valley slope below the slide area. The surcharge was formed by wrapping the upstream side of the dam around the spillway structure and extending it for a short distance upstream from the slide area. The surcharge also served to cover and protect the inlet pipes leading to the intake structure.
7. Development of a monitoring program to evaluate the stability and safety of the slope both during and after the remedial program.

The main features of the remedial work are shown in Figure 2. The remainder of this paper is a description of the measurement and monitoring program and of the principles developed for evaluation of the stability and general safety of the slope area both during construction and during the subsequent operation of the completed Cabin Creek facility.

INSTRUMENTATION

The principal instruments used in the Cabin Creek instrumentation program were wire-type multiple-position borehole extensometers. Generically, borehole extensometers are instruments designed to measure rock mass, soil mass, and structural deformation by means of the sensitive and reasonably precise measurement of changes in the lengths of drill holes or sections of drill holes. Extensometers are available for use in drill holes of virtually any diameter and of depths as great as several thousand feet (1 km or more). "Single point" instruments are used to measure changes in the total length of a drill hole. "Multiple point" instruments measure changes in the lengths of a selected number of drill hole segments as well as changes in the total length of the hole. Extensometer sensitivity is on the order of less than 0.001 in. (0.02 mm) to several thousandths of an inch (0.1 mm). Useful instrument range commonly varies, for different types of extensometer, from less than 1 in. (25 mm) to as much as several feet (1 m or more).

The type of multiple-position borehole extensometer used at Cabin Creek consists of an instrument "head" (Fig. 3) mounted at the drill hole collar and eight in-hole anchors, each secured tightly in position at a selected depth in the drill hole. Each in-hole anchor is attached by means of a high-strength, stainless-steel wire to an individual transducer in the instrument head. As the rock, soil, or structural mass is deformed, changes in the length of the drill hole are registered by the series of transducers in the instrument head and converted into either mechanical or electrical impulses, which may be read manually or recorded automatically for subsequent reference. Readout may be accomplished either at the instrument head or remotely, depending on

Figure 1. Cabin Creek pumped storage hydroelectric project.

Figure 2. Generalized section of project.

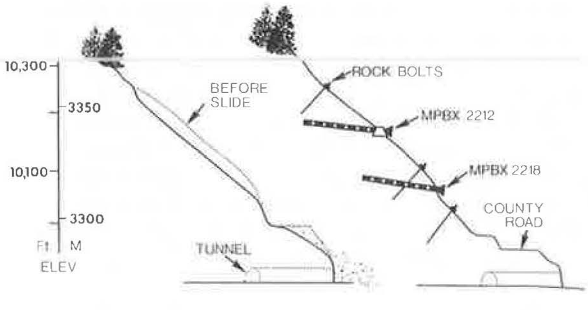
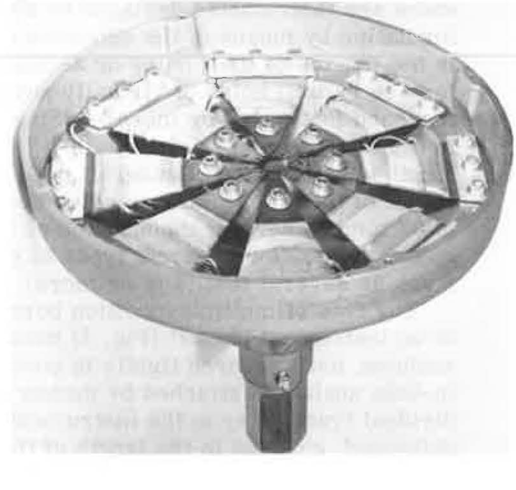


Figure 3. Instrument head, F-2 multiple-position borehole extensometer.



the circumstances of individual installations. In the Cabin Creek program, the instruments were read electronically from remote points well clear of the area of known instability.

The Cabin Creek extensometer holes were $2\frac{1}{4}$ in. (57 mm) in diameter, percussion-drilled, and of the lengths and attitudes listed in Table 1.

Inasmuch as an extensometer measures in a plane containing the length of the drill hole, the holes are customarily oriented to provide for measurement of the most favorable component of deformation in the plane or planes in which such deformation is most likely to occur. Other instruments are available for other applications.

In the Cabin Creek program, some or all of the extensometers were read continuously whenever men or equipment were present in the downslope construction area. The readings were made manually by an engineer or qualified instrumentation technician, who also was responsible for making an immediate sight interpretation of the data and for maintaining a continuous visual surveillance of the slope and adjoining areas. The engineer or technician was charged with the responsibility for immediately sounding an alarm in the event of a departure—however slight—from an acceptable pattern of instrument or visual responses. All instrument readings were later processed by computer to permit a more detailed analysis of the nature, distribution, rate, and acceleration of all measured displacements in each drill hole, in order to obtain a more definitive interpretation of the behavior of the slope in the unstable area.

REPRESENTATION OF EXTENSOMETER DATA

In slope investigation applications, the extensometer instrument head is most frequently located in the unstable area and thus undergoes more displacement than do the successively deeper in-hole anchors. The deepest in-hole anchor, usually situated beyond or outside the principal zone of deformation or displacement, thus constitutes a point relatively "fixed in space" toward which—or away from which—the instrument head is displaced. The intermediate anchors are also displaced away from, or toward, the deepest anchor in proportion to their individual locations in the deforming zone. It is sometimes convenient to visualize the instrument head as a yo-yo attached by its string (the measuring wire) to the deepest in-hole anchor or to some intermediate anchor.

The yo-yo analogy is also useful in understanding the format of the standard multiple-position borehole extensometer displacement plot (Fig. 4), in which the zero ordinate (of displacement) is taken, by definition, as a straight line representing the trace of the deepest in-hole anchor. All changes in distance between the instrument head and the deepest anchor are, therefore, attributed to displacement of the instrument head rather than to displacements involving a change in the position of the deepest anchor in space.

Figure 4 is a displacement-acceleration representation of data obtained from MPBX 2212, MPBX 2218, and MPBX 2219 during the Cabin Creek instrumentation program. For MPBX 2218, the zero displacement ordinate is the trace of the deepest 98-ft (30 m) anchor. Each of the other traces is labeled with a notation of the interval it represents; i.e., the 98-80 ft (30-24 m) trace defines the displacement of the 80-ft (24-m) anchor relative to the 98-ft (30-m) anchor. The 98-0 ft (30-0 m) anchor, by this token, defines the displacement of the instrument head at 0 ft relative to the 98-ft anchor. The abscissa is elapsed time (months) since instrument installation.

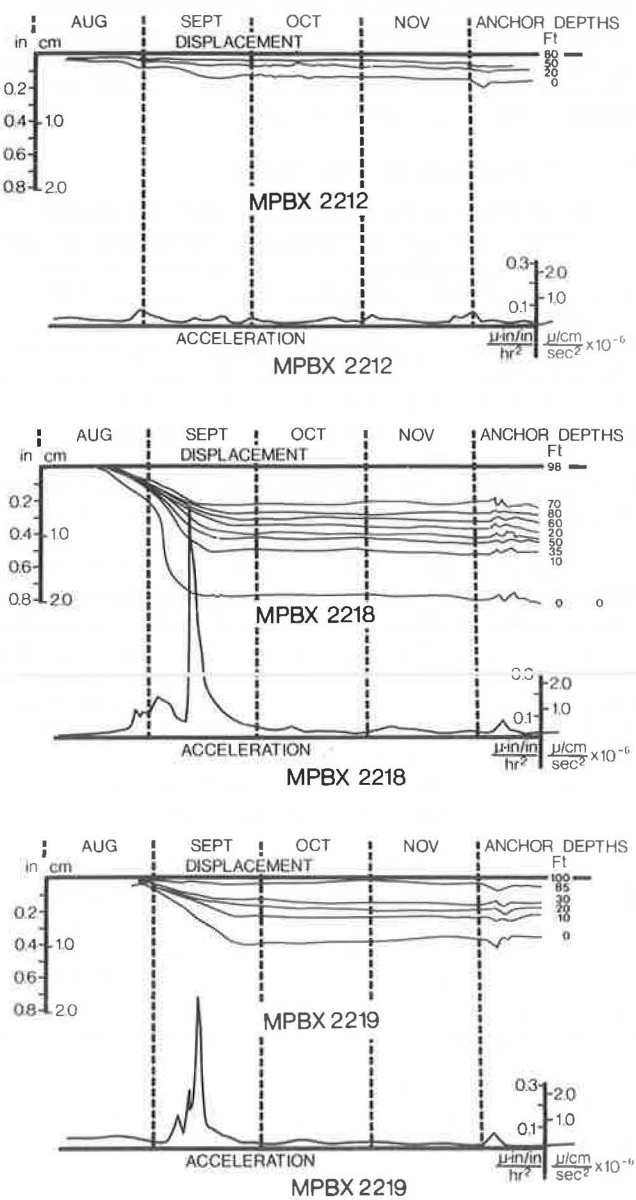
In the companion curve, the acceleration of displacement is represented versus elapsed time for the single interval 98-0 ft (30-0 m) comprising the entire length of the MPBX 2218 drill hole. This "average acceleration" curve is a useful adjunct to the displacement representation in defining the development of potentially adverse deformation in time. An inspection of the displacement representation will show that, although all values are plotted as functions of time, the factors that emerge most clearly in that particular format are those relating primarily to the spatial distribution of deformation in the drill hole. The use of both formats, therefore, permits the ready identification and analysis of deformation in both space and time in one drill hole. By combining the information with equivalent information from other extensometers in other drill holes, the space-time analogy can be extended in another plane to comprise all or part of the area under investigation.

Table 1. Multiple-position borehole extensometer data.

MPBX S/N	Hole Depth (ft)[a]	Inclination (deg)	Anchor Depths From Hole Collar (ft)[a]
2212	80	+5	10, 20, 30, 40, 50, 58, 70, 80
2213	77	+5	10, 20, 30, 40, 50, 60, 70, 77
2218	98	+7	10, 20, 35, 50, 60, 70, 80, 98
2219	100	+4	10, 20, 30, 85, 100
2220	150	+9	30, 50, 70, 90, 110, 130, 150
2221	125	+7	10, 25, 45, 65, 85, 97, 115, 125
2221-A	93	+15	5, 15, 25, 40, 53, 70, 85, 93

[a]1 ft = 0.3048 m.

Figure 4. Displacement and acceleration graphs.



The MPBX 2218 representations are cited as an informative example because of an unusually complete pattern of responses initiated by a heavy 4-in. (100 mm) rainfall to which the slope was exposed prior to completion of the remedial program. Referring first to the displacement representation and then to the average acceleration curve, these responses were more or less as follows:

1. Displacement—The first active reading (FAR) of MPBX 2218 was made at an elapsed time (ET) of approximately 21 days. Subsequent early readings showed a pattern of continuing displacements, proceeding at a reasonably constant rate, which were attributed to rebound induced by the slide and by the additional removal of superincumbent load during stripping and shaping of the slope. The displacement rate was about 4.0×10^{-4} in. per hour (3.0×10^{-4} μm per second), equivalent, in a 98-ft (30-m) hole, to a strain rate of 0.3×10^{-6} in. per in. per hour (10 μm per cm per second).

At an ET of approximately 8.0 days, the slope received over 4 in. (100 mm) of rain in a 24-hour period. Shortly thereafter, at ET 11.9 days, displacements rapidly accelerated, accelerating still further at about ET 13.1 days. Soon after the onset of the accelerated displacements, some surficial slumping and considerable rolling rock could be seen on the slope. By this time men and equipment had already been removed from the downslope area, the judgment having been made on the basis of the instrument data at about 12.3 days ET.

The displacement rate reached a maximum at about 14.5 days, thereafter subsiding gradually until about 16.9 days, at which time a relatively constant low gradient was reestablished. The new gradient was at an appreciably lower rate than the gradient measured prior to the rainfall. By about ET 31.2 days, displacements in the vicinity of MPBX 2218 had essentially ceased. The perturbations in the curves at about 85.4 days resulted from ice accumulation in the instrument head following an interruption of electric power to the heated instrument enclosure and hence do not reflect real data.

Note the irregular, saw-toothed pattern in all of the displacement curves at about ET 29.1 days (mid-September). This reflects an uncorrected temperature response for a daily temperature fluctuation of about 55 F (13 C), which was left in the data for reference and calibration purposes. The calibrated response of the Cabin Creek extensometers to temperature variations was slightly more than 0.02 percent per degree F (0.04 percent per degree C).

A quick inspection of the displacement curves from MPBX 2218 indicates that about half of the total displacement measured in the MPBX 2218 drill hole was localized in the interval between the 98-0 ft (30-0 m) and 98-10 ft (30-3 m) traces; i.e., in the surface 10 feet (3 m) of the hole, a condition strongly suggestive of surficial slumping in the disturbed slope material.

The presence of an appreciable interval between the zero ordinate and the 98-80 ft (30-24 m) and 98-70 ft (30-21 m) traces indicates that the deep part of the drill hole, at least to the location of the 80-ft (24 m) anchor, is <u>not</u> beyond the zone affected by the slope deformation. The progression of these deep displacements from ET 2 days appears, however, to be generally consistent with rebound rather than with the accelerating pattern of displacements registered by the shallower anchors after the 4-in. (100 mm) rainfall.

2. Average acceleration—Shortly after installation of MPBX 2218, a succession of discrete accelerations, each greater than the preceding one, was registered. The pattern cast some doubt on the "rebound" hypothesis, and the acceleration representation was watched closely for further developments.

After the rainfall, a succession of accelerating displacements was registered, the largest occurring at ET 14.6 days. The acceleration at ET 14.6 days was equivalent, in a 98-ft hole, to a strain change rate of 8.25 microin. per in. per hour per hour (6.35×10^{-5} μm per cm per second per second). Thereafter, the acceleration decreased gradually, approaching zero at about ET 23 days.

In considering the relationship between displacement magnitude, rate, and acceleration, several fundamentals are worth reiterating:

1. If the meaning of a "failure" is interpreted in its broadest sense; i.e., as an unexpected or unforeseen event constituting either a safety threat or an economic blow, it is apparent that neither displacement magnitude nor displacement rate are of much value in predicting when—or if—a failure might occur. Depending on the particular circumstances of a project and the types of materials involved, very substantial amounts of deformation can occur, at very appreciable rates, without constituting a failure in the broad sense.

2. Acceleration, on the other hand, is definitive. Acceleration, if continued, is certain to lead to a failure, whatever the sense in which failure is defined.

3. A preoccupation with acceleration, however, is not a simple answer. Large magnitudes of deformation, developing at high deformation rates, can occur without evidence of acceleration in instances in which the deformation is developing either at a reasonably constant rate or at a rate that is increasing in a series of spasmodic or periodic bursts of short duration with intervening periods of constant displacement at the progressively elevated rates.

Fortunately, the dual plotting formats previously outlined provide a ready and convenient means for evaluating each of the three major parameters of displacement, rate, and acceleration. Displacements and accelerations are, of course, measured directly from their respective curves. Rate can be readily determined by either measurement or inspection of the slope of the displacement (magnitude) curve.

From the foregoing discussion, it is apparent that different perspectives may be required in order to evaluate different situations and address different problems. It is also apparent that different sorts of information can be developed and slighty different conclusions drawn on the basis of different representations of the same fundamental information. The implications of these considerations are developed, for the specific Cabin Creek application, in the following sections.

INTERPRETATION OF EXTENSOMETER DATA

Figure 4 shows displacement and average acceleration measurements by three representative multiple-position borehole extensometers. Each of the sets of curves is arranged according to a single common time base in order to facilitate the comparison of data from instrument to instrument and the comparison of measured events with various other construction activities.

The displacement curves alone are quite ambiguous, particularly in regard to the events following the rainfall. One instrument, MPBX 2218, registered the very appreciable displacements already described. Another, MPBX 2212, registered little activity. The remaining instrument, MPBX 2219, indicated an intermediate low level of displacement. Although the curves develop a considerable amount of useful information in regard to possible rebound adjustments, possible surficial slumping, and the stability or instability of specific local areas of the slope—those areas in which each instrument was specifically located—the general progression and nature of the data offers little basis for evaluation of the probable stability of the entire slope. The best that can be said is that, in point of displacement magnitude and distribution in space, there is no definitive indication of the concerted movement of a single block of any appreciable size. The suggestion is, however, that the various parts of the slope responded to the same stimuli at about the same times and that only the magnitude of the response differed from hole to hole. This would imply that, given an adequately intense stimulus, the entire slope might respond in a violent or hazardous manner.

The acceleration curves, on the other hand, provide a substantially different view of the same events. On the basis of these curves the faintly ominous displacement events are seen to be spread in time. Neither the distribution nor sequence of the acceleration peaks—at least those associated with the rainfall—suggests a systematic dislocation of a block of any appreciable size. In other words, the overall response of the slope with respect to both displacement and average acceleration was substantially less disturbing than the displacement curves alone would have suggested.

A less reassuring aspect of the acceleration curves is a pattern of accelerations registered in late August and early November by MPBX 2212 and MPBX 2218 and at other times by other combinations of instruments. These accelerations, although slight, did appear to be aligned in time and thus may reflect influences potentially adverse to larger areas of the slope. For the specific events noted, reference to the displacement curves failed to reveal any evidence of appreciably increasing displacements. Nevertheless, any events common among groups of instruments with regard to either time or space should be watched closely for indications of potentially adverse long-term trends.

MPBX 2211-A

MPBX 2211-A was a special-purpose extensometer installed at the toe of the slope to measure the response of the toe area to the construction of the safety surcharge previously noted. The response of MPBX 2211-A, and lesser responses of MPBX 2219 and 2220 at about the same time, reflected a general compaction of the toe under the surcharge load.

SUMMARY AND CONCLUSIONS

The conclusions drawn from the extensometer information were as follows:

1. A slope disturbed by a slide and by the remedial measures occasioned by the slide undergoes a sequence of adjustments that include rebound and a certain amount of surficial sloughing and slumping. These adjustments, in and of themselves, have little structural significance.
2. With respect to the Cabin Creek slope, specifically, the remedial measures applied after the May 13, 1965, slide did effectively improve the stability of the slope, to the extent that it withstood a reasonably heavy rainstorm—before the remedial program was completed—with only superficial slumping and without indications of the instability of any block or area of appreciable or even identifiable size.
3. Insofar as the indicated safety factor of the slope after the rainfall and prior to completion of the remedial work was obviously greater than 1.0, the safety factor of the slope at the conclusion of the remedial program was considered to be substantially greater than 1.0 with respect to deformation of a structurally meaningful nature.

In the 8 years since the May 13, 1965, rock slide, no additional slope problems have been encountered.

The Cabin Creek instrumentation is an excellent example of an economical, practical, and reasonably unsophisticated program applied—in support of other engineering activities—in the evaluation and solution of a serious and hazardous construction problem. The data developed by the instrumentation program were used to guide and evaluate the remedial treatment of a known unstable slope, to reduce the hazards to men and equipment in a vital construction area, and in general to materially reduce the usual uncertainties in engineering judgments of rock mass and soil mass stability.

Perhaps an important final concept is that of practical, applied, instrumentation as a useful tool for engineers and engineering geologists, rather than as a separate—frequently obscure—technology wrought by practioners who are neither quite fish nor quite fowl. Extending this concept to more varied applications and more complex problems can only contribute to the further control of hazards, reduction of costs, and improvement of maintenance in both the construction and operation of engineering works.

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EXPERIENCES WITH LANDSLIDE INSTRUMENTATION IN THE SOUTHEAST

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During the past 4 years, investigations were made of several large cut and fill landslides along Interstate highways in Tennessee and North Carolina. Instrumentation played an important role in all investigations, both in determining the nature of the earth movements and in determining in situ soil properties. The types of instrumentation installed or used included inclinometers, piezometers, surface survey grids, Menard Pressuremeters, and in situ shear test devices. This paper discusses some of the problems and the resulting solutions found relating to instrumentation installation, data collection and interpretation, and use of the data in analysis.

•SINCE 1968, Law Engineering Testing Company has assisted the state highway departments of Tennessee, North Carolina, and West Virginia in investigating more than 20 separate large earth slides in cut and fill sections along highways in these states. The areas of study in Tennessee were on I-40 near Rockwood (40 miles west of Knoxville), on I-75 near Caryville (35 miles north of Knoxville), and on I-40 near Hartford (40 miles east of Knoxville). In North Carolina, a long segment of I-26 south of Hendersonville (20 miles south of Asheville) was studied. The West Virginia work involved a single slide on I-79 about 25 miles northeast of Charleston. During these investigations, various instrumentation techniques were employed to monitor slide movements, measure water levels, and determine in situ strength properties of soils. This paper reports the experience with various types of instrumentation, emphasizing the strength determination work, and describes general guidelines for effective use of the instrumentation that evolved during the investigations. Examples of the uses are drawn from case histories.

MEASURING SLIDE MOVEMENTS

The basic instrumentation methods for measuring slide movements consist of inclinometers and survey grid systems. Both methods were used in our investigations, generally in combination. Survey methods are used to measure surface movement patterns, whereas the inclinometers measure movements below the ground surface. Several inclinometer systems are available; our investigations used the Slope Indicator (Digitilt and Microtilt Models) manufactured by the Slope Indicator Company of Seattle. The equipment proved to be dependable, with few operating problems.

Survey Grids

Survey grids are an effective means of defining slide limits and patterns of movement, measuring growth of the area involved in movement, determining areas of bulge and subsidence, and providing a base grid for mapping of scarps, ground cracks, and seeps. The data obtained from survey grids can also be used to compare slide activity with rainfall occurrence and to guide in locating exploratory borings or supplementary instrumentation. The use is best suited to slow, creep movements; rapid failures that stabilize after initial movement are not subjects for survey grid measurement.

There are several requirements if a survey grid is to provide meaningful information. First, the grid system is laid out to provide at least three lines in the slide area, with the maximum grid size limited to about 100 ft (30 m). Grid lines are normally mutually perpendicular for ease of surveying, and the grid system is generally oriented with one of the lines paralleling the expected axis of slide movement.

The reference base lines for the grid, of course, must be set well outside the area involved in sliding. Reference hubs should be set so they will not be easily disturbed during other investigative activities. The grid should extend two or more lines outside the area of visible movements so that growth of the slide zone can be detected and measured.

For maximum effectiveness, the grid should be established so that surface movements can be measured in two directions and so that elevation data can be obtained. Elevation data are not always amenable to interpretation, particularly at any given grid point, but a pattern can sometimes be deduced from all the data considered together.

The grid system should be surveyed on a time schedule tailored to fit the rate of slide movement. If the rate of movement is unknown, an arbitrary measurement period can be used with the period being increased or decreased until measurable changes are reflected by the grid. Frequently, because movements are generally accelerated by rainfall, grids are surveyed within a week after an exceptional period of rainfall.

Figure 1 shows two ways of presenting the results of survey grid monitoring. The preferred method, shown in Figure 1b, depends on having grid movements measured in two directions so that a vector representing the resultant movement of the grid point can be drawn to a scale. Examining this presentation gives a much clearer picture of the trend of slide movements than does the presentation shown in Figure 1a, which is derived from a grid measuring movements only in one direction.

A variation of the survey grid is a simple arrangement of two surface monitor points to measure relative movement such as across a ground crack. One of the monitor stakes (a secure tree could be substituted) is set in ground considered not to be moving. The two stakes are generally set 12 to 16 ft (4 to 5 m) apart, and a board or pipe is rigidly attached to the stake thought to be in unsliding ground, extending to contact (but not fastened to) the second stake in the slide mass. A reference mark with the data noted is made with an indelible pen on the day of installation where the board contacts the moving stake. On subsequent visits, additional marks are made and vertical and horizontal movements measured to establish relative movements and rates. No surveying is required, and measurements are made by one man using an engineer's scale or pocket ruler. Even small surface movements are immediately obvious by this simple, inexpensive method.

Inclinometers

Inclinometers, used to locate zones of movement at depth in slide masses, are perhaps the most commonly known form of instrumentation. Most state highway departments in the Southeast have at least one inclinometer, and their use is becoming more and more routine. Previously published literature contains well-documented case histories illustrating the method of operation and benefits of inclinometers in landslide investigations. With rare exceptions, Law Engineering Testing Company uses inclinometers in all landslide investigations, small or large. To develop useful inclinometer data, the installation must be planned and the data interpretation must follow set procedures.

Planning Installations—Inclinometer locations are selected to best define sliding surfaces. All available visual, air reconnaissance, and survey data on slide cracking and bulging are reviewed to locate inclinometers where they are most likely to be useful. Arbitrary locations on a preset pattern can result in not having an inclinometer where the sliding surface is deepest, resulting in underestimation of the size of the moving mass. Where access is possible, it is best to have at least three inclinometers on each line parallel to the slide axis so that failure surfaces can be adequately described. It has also been found useful to install an inclinometer just behind the slide scarp to detect the direction of strain movements prior to failure.

Planning for the inclinometers also involves selecting the depth of installations and deciding if the inclinometers are to be used as water-level observation wells. Inclinometers can be used as water-level observation wells if a section of inclinometer casing is slotted to permit in-flow. Also, the top of the casing backfill must be sealed against infiltration of surface water, and the remaining backfill must be properly zoned to expose the desired water-bearing stratum. The water-level data obtained from inclinometer installations must be reviewed with some caution because the casing may permit connection of normally separate water-bearing zones. Caution is advisable when slotting the inclinometer casing, as any infiltration of sand or soil may cause irregularities in the casing and thus affect the data obtained for evaluating movements.

It is absolutely essential to have the bottom of the inclinometer casing seated into stationary material, preferably rock. For cases where sound rock is far below the expected movement zone, the bottom of the casing should be extended at least 20 ft (6 m), but preferably 40 ft (12 m), below the expected lowest limit of movement and grouted into place for necessary restraint. When in doubt, it is always better to carry the installation deeper than to risk obtaining meaningless data. The casing joints and bottom should be carefully sealed if grouting is to be done to avoid intrusion of the grout into the casing. It is good practice to clean and flush the inside of the casing with a brush attached to a small-diameter pipe before the grout sets up. Another potential problem is that the fluid grout may leak out undetected into surrounding porous soil before setting up, thus leaving the casing laterally unstable.

The annular space between the casing and the boring is generally filled with sand to stabilize the casing. When the casing is to be installed in rock fills or relatively porous soil, the sand backfill may be washed out into the porous fill by seeping groundwater or rainwater entering the casing-boring annulus from the top. Frequently, when backfill is washed out from the middle of the casing installation, its loss is not noticed because the upper sand backfill arches over the gap and does not settle at the ground surface. We have experienced several instances where questionable inclinometer data were later shown to result from loss of sand backfill and subsequent lateral instability of the casing. To reduce loss of the backfill, we use a fine gravel as casing backfill when porous soil or rock fill is encountered. This gravel can be mixed with a small amount of dry cement prior to backfilling to further solidify the mass. The amount of cement should be kept minimal to avoid creating a rigid concrete column around the casing, although this would not be a problem if large slide movements are expected.

When setting the plastic casing, it is advisable to use both pop-rivets and glue, particularly if casing is to be set at depths greater than 50 ft (15 m). The tendency of the field crews to allow insufficient setting time for the plastic glue has caused loss of several inclinometer installations at depths greater than 50 ft (15 m).

When installing the casing in a landslide where the direction of movement seems apparent, we attempt to orient one of the casing grooves in a direction parallel to the apparent axis of the slide movement. This is advantageous in data reduction.

Data Gathering and Use—Inclinometer data should be gathered only by a well-trained person familiar with the principles of inclinometer operation who can interpret oddities in the readings that may indicate machine malfunctions or improper casing installation.

It is important to obtain the compass bearing of the designated casing reference groove at the time of the initial data reading and to mark the reference groove by notching the casing. Without notching, different personnel making subsequent readings may use an incorrect reference groove. It is also good practice to mark the boring number and the depth of the casing on the casing cap. It is likewise good practice to provide the casing cap with a locked cover if there is a chance of vandalism. It seems that the temptation to drop pebbles or dirt clods down any open hole is great enough to warrant this relatively minor expenditure to assure the security of the expensive installation.

Initial readings taken immediately after installation and backfilling of the casing are often not suitable as reference for later changes in casing position because settlement of the sand backfill can occur in the first day or so after backfilling, and thus the initial position of the casing may change significantly. When a second set of readings in an inclinometer casing shows significant variations throughout the entire casing length, and later readings show little change, it is advisable to disregard the first set of readings and use the second set as reference for casing movements. Survey location of the tops of the inclinometers is frequently used as a check on the validity of the data-reduction process.

For each individual slide, the time interval between casing readings is best determined by trial. An initial frequency of every other day for the first week is a good practice, with later time intervals being adjusted to avoid accumulating many sets of essentially similar data. An exception to the time interval should be made to obtain a reading shortly after periods of heavy rainfall, when slide movements are probably accentuated.

Some data reduction should be done in the field by the person making the readings so that the frequency of readings can be adjusted as needed and so that results of the inclinometer measurements can be used together with exploratory work.

Reduction of the inclinometer reading data is tedious, and the data must be studied thoroughly by an experienced person to detect discrepancies. Data reduction procedures are described in the instruction manuals furnished with the various instruments. Figure 2 shows one way of presenting inclinometer data. Computer programs can be used to reduce and plot the data with little difficulty.

WATER-LEVEL MEASUREMENTS

Water is generally involved in many landslides. Investigation of an active or potential landslide or expected landslide location would not be complete without installing some sort of water-level observation device. Unfortunately, determining the groundwater conditions in the residual and/or colluvial soils of the Southeast is not always simple. While the soils themselves do not require sophisticated piezometers to gather data, interpretation of the results and obtaining meaningful results is a problem. Casagrande-type standpipes have been used in our investigations and have performed adequately.

Perched water levels are common in residual and colluvial soils of the Southeast. Groundwater often flows along old joint paths remaining in the residual soil. It is practically impossible to detect accurately all the perched levels or water channels in a slide area. The best approach is to use water-level wells sealed at different elevations and to map carefully the surface seepage zones. Any aerial photos of the slide area taken during construction or at different times of the year should be carefully studied for seasonal surface seepage locations. If possible, the slide area should be inspected carefully after one or more periods of rainfall.

Water-level monitoring should be done throughout the course of an investigation (and throughout remedial construction if groundwater is a causative factor). In critical or representative locations, one or more continuous automatic water-level recorders should be used. A recorder often used by Law Engineering Testing Company is manufactured by Leupold and Stevens Instruments, Inc., of Beaverton, Oregon. This instrument uses a float and counterweight coupled to a large pulley wheel. Variations in the water level as small as 0.001 ft (0.3 mm) can be monitored by this device. Such an installation is suitable for measuring any rapid fluctuations in water levels (hydroseisms) that might be related to seismic activity, occurring either locally or otherwise, or to sudden shear in the aquifer.

IN SITU STRENGTH DETERMINATIONS

One of the major purposes of a landslide investigation is to determine the soil shear strength for use in analyzing the forces resisting movement and in evaluating the benefit of alternative remedial schemes. The usual procedure is to take "undisturbed" samples and perform conventional laboratory tests, either triaxial or direct shear. Although laboratory tests are needed, the relationship of laboratory to in situ strength is often questioned. There is sample disturbance and samples are often not representative, particularly of soil-rock fill or gravel- and boulder-filled colluvium.

In situ strength measuring instruments such as the Menard Pressuremeter, Iowa Bore-Hole Direct-Shear Device, vane shear device, and static cone penetrometer are being used with increasing frequency. Methods of operation and illustrative case his-

Figure 1. Two ways of presenting survey grid data: a. (top) data from grid measuring movement only in one direction; b. (bottom) data from grid measuring movement in two directions.

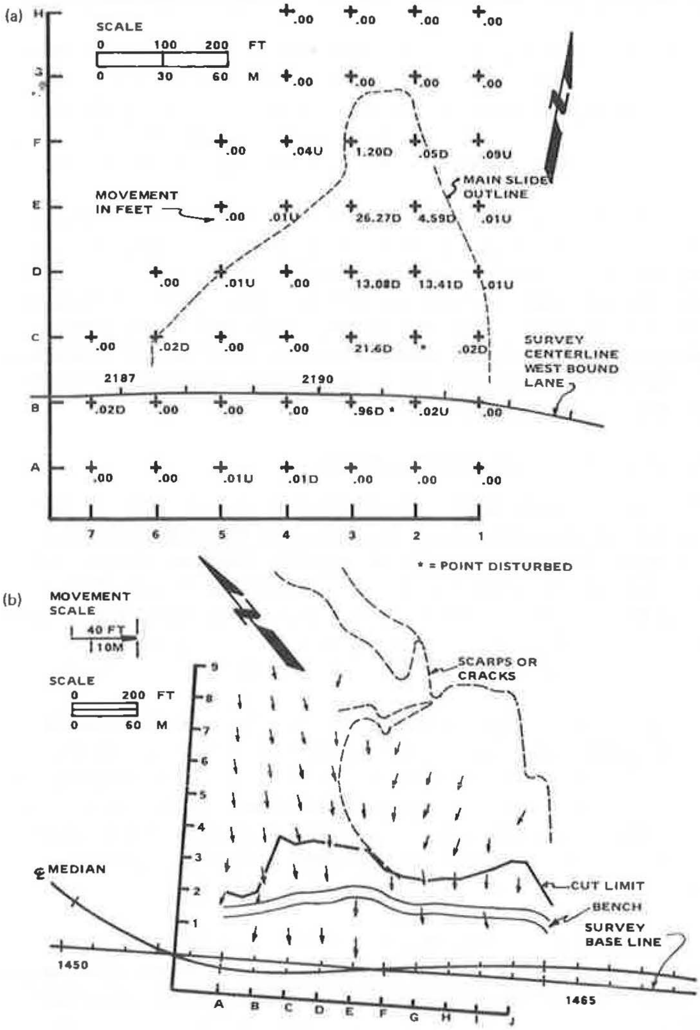
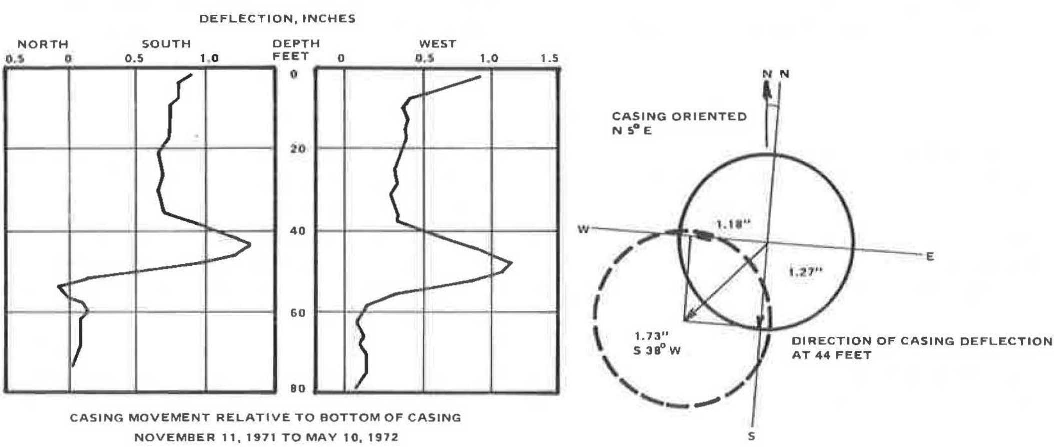


Figure 2. Typical presentation of inclinometer data.



NOTE: 1 in. = 2.5 cm
1 ft. = .3 m

tories have been reported in the literature for these devices, and therefore they will not be discussed here.

Law Engineering Testing Company has relied mainly on the Menard Pressuremeter, the Iowa Bore-Hole Direct-Shear Device, and large-scale field direct-shear tests to determine in situ strength characteristics. The remainder of this paper discusses the uses of these devices.

Because it is relatively expensive and time-consuming, in situ testing should be thoroughly planned. It is useful to have several borings with standard penetration testing for reference in in situ test locations so that soils can be zoned for testing and strata of suspected weak soils can be identified. Many times, inclinometer installations are monitored prior to selecting depths for in situ testing so that test depths corresponding to zones of movement can be selected.

Preliminary soil data also aid in selection of the in situ device to use. It would be pointless to attempt vane shear tests in a mixed soil and rock fill, but the Pressuremeter or Bore-Hole Direct-Shear Device would be appropriate. In cohesionless soils, the static cone penetrometer is a combined testing and investigatory device. In slide zones involving cohesive soil, vane shear, bore hole shear, and static cone penetrometer are appropriate testing devices. The speed with which these tests can be performed often allows their use in early borings without as much regard to planning locations and depth as for the Pressuremeter. Four case histories will be presented, two illustrating Law Engineering Testing Company's use of the Pressuremeter, one illustrating use of the Bore-Hole Direct-Shear Device, and one illustrating the use of a large-scale field direct shear test.

I-75 Fill Failure

North of Knoxville, Tennessee, the route of I-75 rises up the northwest side of Little Cumberland Mountain. Construction through this area involved numerous side-hill cuts and fills. Fill material was obtained from adjacent soil and rock cuts. The natural terrain was so steep that one lane was often in cut while the other lane was filled.

The geologic formations in the area are primarily flat bedded Pennsylvanian Shales, siltstones, and sandstones. On exposure to air and water, the shales and siltstones decompose rapidly into small, finger-sized pieces.

During the first 5 to 6 years after construction, several fills exhibited signs of distress (continual pavement cracking and subsidence), although visible movements in the fill base were not always noted. Also, several fills experienced failures requiring extensive repairs. An investigation of 10 fill areas was conducted in 1971-1972 to evaluate the possible causes of distress, the likelihood of failure, and possible corrective measures. Four of the investigated areas experienced some degree of failure before the investigation was completed. A typical case is described in the following.

Figure 3 shows a plan and section view of the fill at station 840. Original ground surface slopes varied from 2:1 to 1.5:1; thickness of the fill varied from 10 ft (3 m) at the median centerline to approximately 35 ft (10.5 m) at the outside shoulders. Fill slopes were constructed to 1.5:1. Soil test borings (all instrumented with inclinometers doubling as water level observation wells) indicated the fill was a heterogenous mixture of decomposed shale and siltstone containing gravel, cobble, and boulder-sized sandstone fragments.

Figure 3 shows the general stratification within the fill and the natural ground. Groundwater was relatively high—a fact verified by observed seepage zones on the face of the fill. The seeps increased after rains and decreased during dry weather, indicating a possible relation to surface water inflow from the uphill side of the fill. The decomposition of the shale layers in the fill would create semi-impermeable zones across which surface water could flow through the fill.

Because of the difficulty in obtaining "undisturbed" samples, a Menard Pressuremeter was used in one boring to gather strength data more representative of the hetereogenous fill. A metal-sheathed probe was used because the fill contained large quantities of rock fragments. Tests were taken at approximately 5-ft (1.5-m) intervals.

The critical stage in Pressuremeter testing is in making the hole for the probe. A neat, close-fitting hole is required for successful testing. We have obtained the best results with a hole about $\frac{1}{16}$ in. (1.6 mm) greater in diameter than the probe. Coring can be used where a large amount of rock fragments is present. Prior standard penetration tests and drilling experience are the best guide to determining the proper hole-making procedure.

Figure 4 shows a comparison of standard penetration test results, Pressuremeter strengths, and results of laboratory unconsolidated-undrained triaxial tests with confining pressure equal to the overburden pressure. The Pressuremeter strengths were obtained using the methods given by Menard. As can be seen in the figure, the laboratory tests showed large scatter whereas the Pressuremeter tests showed more consistency. The Pressuremeter strengths indicate a relatively low strength at 34 ft (10.5 m), in the stratum of badly weathered gray shale. The inclinometer data also indicated movement at this depth.

For analysis, the fill was assumed horizontally stratified as shown in Figure 3 and the strength parameters for the layers were selected based primarily on results of the Pressuremeter tests. Circular arc analyses yielded a minimum factor of safety of 0.95.

During the analysis, a failure occurred in this fill near the pavement edge encompassing most of the upper portion of the fill face, generally near the calculated failure arc. The Pressuremeter strength data and the movements indicated by the inclinometers were used to evaluate the stability of the remaining fill so that steps could be taken to prepare a traffic detour if necessary. Analysis indicated that further movement involving the entire southbound lane was probable; this movement eventually occurred, but not before appropriate measures had been taken to detour traffic around the failure area.

After the second movement, vertical wells with submersible self-actuating pumps were installed in the median behind the scarp to intercept the flow of water into the slide area and prevent further extension of the slide zone. These wells proved very effective; they were later incorporated into the repair treatment.

The repair measures consisted of removing the failed soil and reconstructing the southbound lane portion of the fill. The vertical pumped wells were incorporated in the remedial scheme to remove water entering from the natural ground uphill from the fill. The strength data from the Pressuremeter tests were again used to determine the recommended slope of the excavation to remove the failed soil. The repaired fill was placed in service on April 28, 1973, and has performed well to date.

Cut Slope Slide, Cocke County, Tennessee

Pressuremeter tests can often provide strength data when conditions eliminate any other approach. In 1971, a slow creep slide on I-40 east of Knoxville was investigated. The slide was causing vertical displacement of one lane that required a regrading and paving every 3 to 6 months. Exploratory drilling indicated a profile as shown in Figure 5. Inclinometer monitoring indicated movement in a badly weathered, fractured rock zone. Due to the rocky character of this zone, no "undisturbed" tube samples could be obtained. Core recoveries were less than 20 percent, even with extremely careful coring procedures. In the worst zones, the recovered core indicated that a large percentage of the weathered zone would be characterized as rock fragments in a soil matrix. The Pressuremeter was the only device suitable for reasonably estimating the shear strength of the soil-rock complex.

Pressuremeter testing was performed in one boring to provide an estimate of overall shear strength in the weathered rock soil zone. The results, when used in an arc-tangent-arc stability analysis, yielded a factor of safety of 1.1, which was believed to be in reasonable agreement with the nature of the slide movement. Effects of various remedial schemes were analyzed using the Pressuremeter strength. Because of the topography and the large mass of the slide area, remedial measures involving regrading, restraint, or road relocation were judged more costly than maintenance. Vertical wells with submersible pumps were installed along the shoulder through the problem area. Although these only provide a slight reduction in the water level, the reduction

Figure 3. Plan and section of side-hill fill failure, I-75, Campbell County, Tennessee.

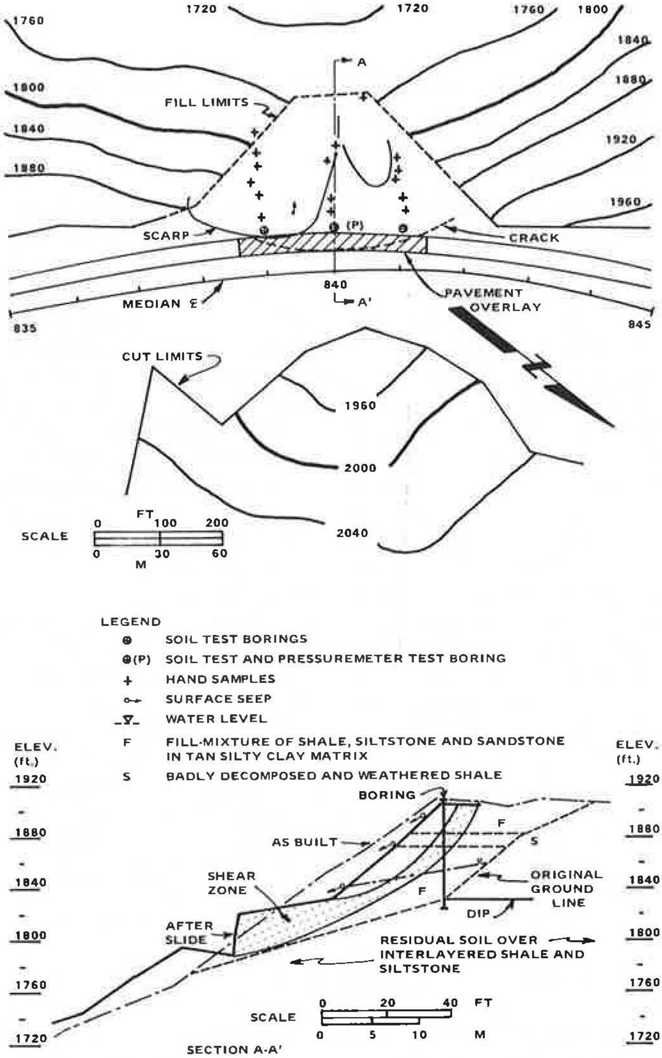
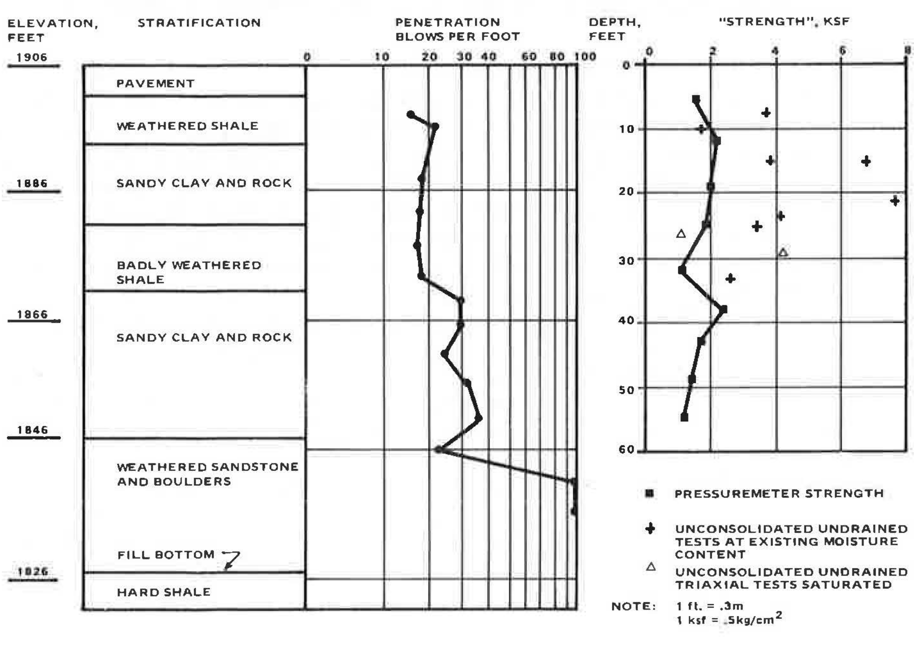


Figure 4. Comparison of laboratory test with field test results, side-hill fill slide, I-75, Campbell County, Tennessee.



has been sufficient to greatly decrease the slide creep. The wells have been operating since early 1973 and have proved effective in reducing the pavement uplifting.

Cut Slope Slides, Polk County, North Carolina

Interstate 26, to run from Asheville, North Carolina, to Spartanburg, Greenville, and Charleston, South Carolina, is complete and carrying traffic except for a section near Columbus in Polk County, North Carolina. The 1.2-mile (1.9-km) long problem section of the highway climbs 450 ft (140 m) along the southern slope of Miller Mountain. Numerous cut slope failures of varying size plagued the construction, and work had to be halted with grading incomplete to allow engineering studies of the problems.

The rock formations at the site are primarily gneiss. The rocks in the slide area are jointed and have abundant schistose mica concentrations of varying thickness. The rocks have a northeast trend in outcrop pattern and a southeast trend in dip, toward but slightly skewed from the highway alignment, and toward a large northeast trending syncline situated to the southeast. The rocks have weathered in place to saprolite and residual soil. Close inspection revealed that the weathering products have been disturbed to varying degrees by ancient natural slides, producing a colluvial cover. Some of the ancient slides have moved large blocks of the jointed saprolite, making the distinction between disturbed and in-place saprolite difficult.

A number of the slip surfaces were sampled and tested by standard direct shear tests in the laboratory. However, the foliation in the gneiss saprolite was "wavy" due to undulations produced by local folding. This and other considerations led to the decision to supplement the laboratory tests with a large-scale direct shear field test.

The test was performed on a micaceous foliation zone using an 18-in. (45-cm) square split shear box and three values of normal pressure (which required three different test setups). The test procedure was basically the same as the laboratory tests except that the field test specimens were not fully saturated. The vertical normal load was applied with hydraulic jacks reacting against a bulldozer. The shearing load was applied to the upper half of the box with hydraulic jacks reacting against a bracket attached to the bottom half of the box. Figure 6 shows the essential features of the shear box and jacking arrangements. The test results are shown in Figure 7. The field direct shear test results correlated very well with the laboratory results, producing a shearing strength intermediate between laboratory tests parallel to the foliation and those perpendicular to the foliation. The field direct shear test results, in conjunction with inclinometer data and the laboratory tests, proved very useful in the analysis of a massive slide involving material similar to that tested. The analysis yielded factors of safety that were compatible with the observed slope behavior, giving confidence in use of the strength parameters in the studies of remedial measures.

Waste Fill, Polk County, North Carolina

An area on the downslope side of I-26 was used by the contractor for disposal of approximately 400,000 yd (305 000 m^3) of excess material excavated from the roadway and some of the cut slope slides. The original ground slopes were 1.2:1 and 2.0:1. The surface of the fill, which was constructed by end dumping, was left as steep as 1.4:1 in places. Fill depths reached as much as 55 to 65 ft (17 to 20 m).

The fill stability was important because of private property and a local access road below the site and for a proposed outward alignment shift for I-26 near the top of the fill. An investigation was undertaken to assess the stability of the waste fill.

Borings disclosed that the fill material included significant amounts of gravel to boulder-size pieces of rock. The predominant soil types were silty sands and very sandy silts. Standard penetration tests in the fill, measured 2 to 2.5 years after the material was placed, ranged from typically 4 to 10 blows per foot, except in some cases of much higher blow count because of the rock fragments. The rock fragments hampered undisturbed sampling by bending or disturbing many of the tubes. Because it was felt that an insufficient number of high-quality undisturbed samples was available to adequately represent the fill material, it was decided to perform in-place Iowa Bore-Hole Direct-Shear tests in holes drilled adjacent to two of the completed test borings.

Figure 5. Plan and section of cut slope creep slide, I-40, Cooke County, Tennessee.

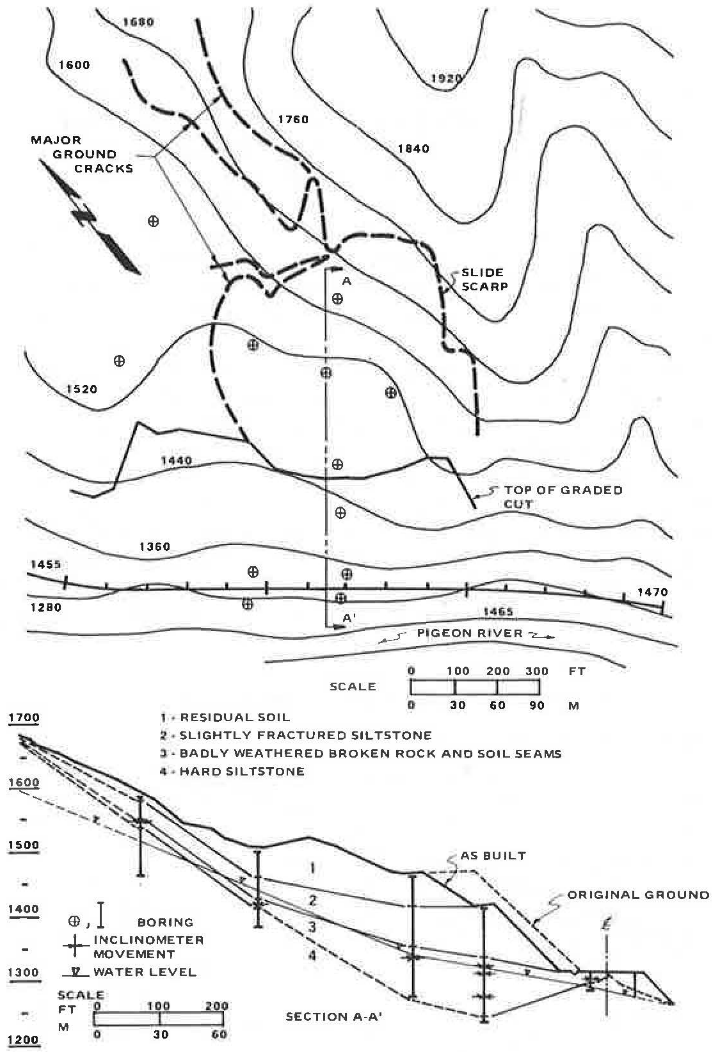
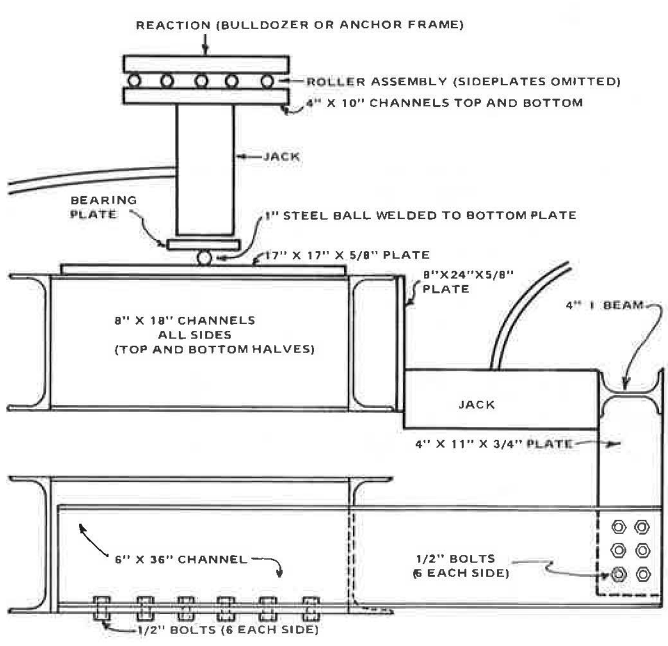


Figure 6. Large-scale direct shear device.

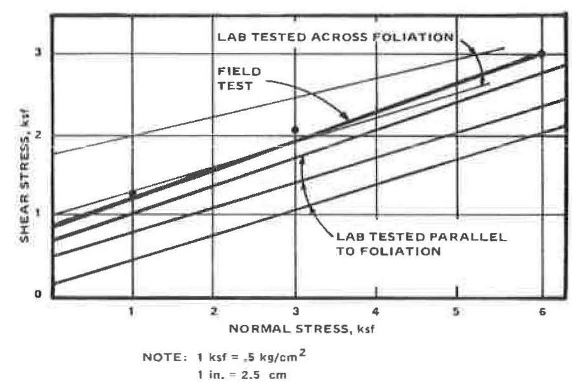


The Bore-Hole Direct-Shear test method involves drilling a smooth hole (which can be the hole used for split tube sampling, penetration testing, and undisturbed sampling), inserting an expandable shear head to the desired depth in the hole, and expanding the two opposing plates against the walls of the hole. The test is simple and like a direct shear test in concept. The force of the plates against the soil is regulated with CO_2 gas pressure. After allowing a predetermined time for consolidation (3 to 5 minutes in this particular case), the expanded shear head is pulled axially within the hole, and the pulling force is measured. The maximum pulling force divided by the contact area of the two plates represents the soil shear strength corresponding to the applied expansion pressure.

The test sequence is repeated at the same depth at successively higher expansion pressures. The expansion force divided by the contact area of a single plate is the normal stress and is used in plotting results of shear tests. The shearing strength versus the normal stress is then plotted to give a shear failure envelope. The slope of the line best fitting the points represents the angle of shearing resistance, and the intercept at zero normal stress is the cohesion parameter. The test data are available for interpretation and use immediately after completing the field work.

The Bore-Hole Direct-Shear test data are summarized in Figure 8. For comparison, the laboratory triaxial shear strength tests performed on the fill are also shown. The strength parameters from the Bore-Hole Direct-Shear test compare well with the

Figure 7. Comparison of field and laboratory direct shear test results.



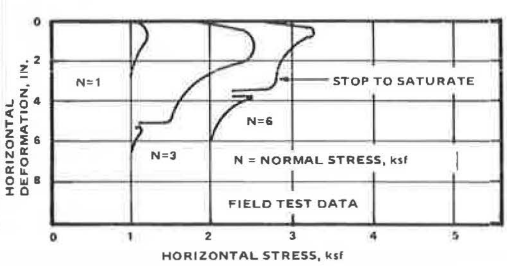
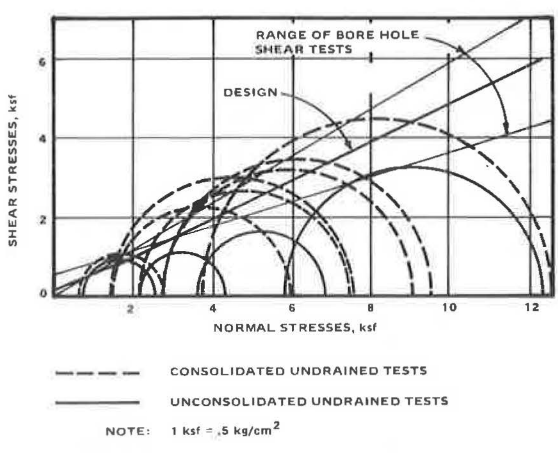


Figure 8. Comparison of Bore-Hole Direct-Shear results with laboratory triaxial test results.



available laboratory triaxial data and plot intermediate between undrained and consolidated undrained laboratory values.

Subsequent stability analyses of the waste fill using the average cohesion value ($C = 200$ psf $= 1000$ kg/m^2) and a lower quartile value for the angle of shearing resistance ($\phi = 26$ deg) from the Bore-Hole Direct-Shear data produced results indicating that the waste fill was only marginally stable. These overall results were confirmed by small movements subsequently detected at depth by inclinometers placed in the borings. It is significant to note that interpretation of the available laboratory test data for the waste fill would not have yielded the same strength parameters. Because a greater number of Bore-Hole Direct-Shear tests could be made, additional confidence was gained concerning the shear strength of the fill and the assessment of its stability.

CONCLUSION

It has been the experience of our firm that instrumentation to measure surface and subsurface movements is a necessity for any landslide investigation. The instrumentation may vary from simple crack width monitors to sophisticated electronic inclinometers. Careful planning of instrumentation location is required if the maximum benefits of instrumentation are to be realized.

In situ tests to measure soil strength parameters are often useful in the variable soil and rock conditions involved in landslides in the southeastern United States. Such tests are required in cases where samples suitable for laboratory testing cannot be obtained. The type of in situ test used must be compatible with the soil, rock, or soil-rock system about which information is desired. Even where in situ tests are used to supplement laboratory tests, the results can increase confidence in the design of remedial measures for landslide problems.

ACKNOWLEDGMENTS

The work described in this paper was performed by Law Engineering Testing Company for the state highway departments of Tennessee and North Carolina. The cooperation and assistance of personnel in these departments throughout the course of the investigation is gratefully acknowledged.

LANDSLIDE INSTRUMENTATION FOR THE MINNEAPOLIS FREEWAY

Stanley D. Wilson, Shannon and Wilson, Inc., Seattle

In 1967 a landslide developed along a section of freeway under construction in Minneapolis. The following procedure was undertaken to ensure the stabilization of the landslide and completion of the project: Instruments were installed to detect the depth and rate of movement, and exploration was undertaken to determine the properties of the material in the failure zone; a temporary buttress was placed to control the movements while corrective treatment was being designed; additional instruments were installed to monitor movements during construction of the permanent treatment; and, after completion, all instruments were maintained and additional instruments were installed to monitor post-construction movements, if any. The corrective treatment consisted of a series of slit-trench concrete buttresses anchored into limestone below the failure plane. Details of exploration, instrumentation, testing, and design are included in the paper.

FIELD instrumentation has become a working tool of the civil engineer faced with the problem of designing corrective treatment to control a landslide. The case history of the landslide described in this paper is a classic example of the essential role of instrumentation in all stages of investigation, design, construction, and post-construction monitoring.

No two landslides are identical in all respects. Likewise, no two installations of field instrumentation need be identical. Nevertheless, there are certain things that one needs to know about each landslide, e.g., the depth of sliding, the rate of movement, the distribution of the movements, the pore water pressure (or lack of it), and the properties of the material in the movement zone.

In 1967 a landslide developed along a section of freeway under construction in Minneapolis. The lines and grades were already established by new overpasses at each end, the materials appeared to be competent, and the design slopes were conservative. It was essential that adjacent property owners be protected and the freeway project be completed on a timely basis. Therefore, the following orderly sequence of events was initiated to bring about the stabilization of the landslide and completion of the project:

1. Instruments were installed to detect the depth and rate of movement, and explorations were undertaken to determine the properties of the materials in the failure zone.
2. A temporary buttress was placed to control the movements while corrective treatment was being designed.
3. Additional instruments were installed to monitor movements during construction of the permanent treatment.
4. After completion, all instruments were maintained, and additional instruments were installed to monitor post-construction movements, if any.

BACKGROUND AND SITE INFORMATION

Location and Description

The landslide occurred along a 1,100-ft (330-m) section of Interstate Highway 94 on the east side of Minneapolis (Fig. 1) bounded on the northwest by a railroad bridge and on the southwest by the Franklin Avenue Bridge.

At this site Interstate 94 was being constructed in a cut, and all of the previously existing streets and the railway crossed the highway on overpasses at close to their original grades. In the vicinity of the site, the original ground slopes to the west on an average grade generally flatter than 6 horizontal to 1 vertical. This gentle westward slope extends for approximately another ½ mile (0.8 km) to the east side of the Mississippi River. The area located immediately uphill from the right-of-way line is developed for single-family dwellings and a city park.

There were no significant stability problems in the general area prior to the start of construction, although some groundwater problems were encountered during the excavation of footings for bridges and retaining walls.

Construction Activities

To depress Interstate 94, it was necessary to make cuts more than 50 ft (15 m) deep. Most of the structural work was completed in the area prior to the start of roadway excavation. The railroad bridge adjacent to Prospect Park was essentially complete by June 19, 1967, and was founded on excavated shafts bearing on limestone. The Franklin Avenue Bridge and the other existing retaining walls in the area are founded on driven steel H piles.

The roadway excavation was started on June 19, 1967. By August 1 the eastbound roadway was complete and excavation was started on the westbound roadway, beginning at the Franklin Avenue Bridge. However, on August 11 all construction and excavation in the area were stopped because of the upheaval and cracking noticed on and below the cut slope in the vicinity of station 167 (Fig. 2). For approximately one month, all construction in the area was shut down while it was decided what course of action was to be taken.

During August the movements continued with increasing development of offsets and scarps upslope. The decision was made by the Minnesota Highway Department to engage a consulting firm with experience in both soil mechanics and instrumention, and the firm of Shannon and Wilson, Inc., was selected. The first site inspection was made on August 30, and a detailed site investigation, including installation of field instruments, was begun on September 2.

Within a few weeks the depth of the movements was identified and, although a minor amount of excavation was resumed in mid-September, the previously excavated subcut was subsequently backfilled with compacted granular base material. During late October, some slope flattening was accomplished below Prospect Park and to the southeast as far as station 170. This was done in an attempt to reduce the amount of earth movement that was occurring in this area. During the early part of November a perforated drain and storm sewer was installed along the toe of the slope to provide drainage for that area prior to the winter freeze.

By early November it was determined that enough information had been obtained to proceed with detailed engineering studies. Also, it was apparent that the earth movement was continuing even with the previously accomplished slope flattening. Therefore, in mid-November a sand berm was constructed about 5 ft thick and 40 ft wide (2 by 12 m) extending along the toe of the slope for most of the distance between the railroad bridge and the Franklin Avenue Bridge. Subsequent field observations showed conclusively that all of the earth movement in the area was essentially stopped by the berm by December 1, 1967.

General Geology

The Twin Cities of Minneapolis and St. Paul lie in a shallow basin intersected by the Mississippi River, which has cut a substantial valley into the underlying sedimentary rocks. In general, this is an area of gently rolling glacial topography dotted with numerous lakes. This glacial mantle consists primarily of tills and outwash materials. The sedimentary rocks that underlie the recent glacial deposits are of the Ordovician Period of the Paleozoic Era.

During this investigation, two of the sedimentary formations were encountered. The upper of these is the Decorah formation. In general, this is a blocky, blue-green to blue-gray shale with occasional limestone beds irregularly distributed throughout. In Minnesota, this formation varies in thickness from 20 to 80 ft (6 to 25 m). This formation is underlain by the Platteville formation, which varies from a light-gray to gray limestone to a calcitic dolomite. For all practical purposes, the rocks within the project are are flat lying. In addition, the limestones are of very good quality, with little evidence of jointing, faulting, or groundwater solution.

FIELD INVESTIGATION AND SUBSURFACE INSTRUMENTATION

The highway department had accomplished numerous borings in the area around Prospect Park prior to construction, including auger borings without samples and rotary borings where "undisturbed" samples were obtained using a Denison sampler. However, sampling was particularly difficult because of the presence of limestone stringers in the Decorah shale. Following the initial site inspection on August 30, an expanded field investigation was recommended that included additional borings, instrumentation, and other field explorations. Initially, six borings with Slope Indicator observation wells were recommended. Subsequently, the state drilled four additional borings in which Slope Indicators were installed and five additional holes with piezometers. Later, as a result of the field measurements, the investigation was expanded to include large-scale torsion shear tests and a program of test pits.

Borings

A well-planned program of exploratory borings provides a reasonably efficient and economic means of studying the subsurface conditions at a particular location. Specifically for this stability study, 15 borings were made; the depth of boring depended on its location and surface elevation. In addition, borings were located to serve the dual purpose of providing subsurface soil information as well as a hole for the installation of field instrumentation. Two sections in the vicinity of the slide were instrumented rather extensively.

An attempt was made to obtain good quality, undisturbed soil samples by using a Pitcher sampler. With this tool the sample is recovered in a 3-in. (76-mm) diameter thin-walled tube that is pushed behind the cutting edge. It was found that this sampler was able to cut through the thin limestone lenses with relatively little difficulty. In general, good sample recovery was obtained in the Decorah shale. However, it was not possible to obtain good samples of the material sandwiched between thin lenses of limestone. After recovery the tube samples were sealed with wax to preserve their natural moisture content and were then shipped to the laboratory for classification and testing.

Subsurface Instrumentation

The type of field instrumentation selected depends on the problem. In this instance it was obvious that movements of some type were taking place below the elevation of the subgrade, but it was not known whether these movements were at the shale-limestone interface or at some intermediate level, whether they were along a horizontal bed or a circular arc, whether they were uniformly distributed or progressive, and whether they were induced by high pore pressures or by a weak interbed.

An inclinometer was chosen because it is the best instrument to detect the depth, rate, and distribution of horizontal movements below the surface. Because of its proven record of performance on similar projects, and because of the immediate availability of both the sensing instrument and the casing, Slope Indicators were selected. This instrument is lowered down a grooved aluminum casing that controls the orientation of the instrument in a predetermined direction. Inclination readings are taken at frequent intervals of depth and are subsequently converted to displacement. Consecutive readings at predetermined depth intervals and at periodic time intervals are used to determine the depth and rate of ground movement. The 3-in. (76-mm) diameter Slope Indicator casing was installed by lowering it into a 6-in. (152-mm) boring, which was kept open with the use of drilling mud. The open space between the walls of the boring and the outside of the aluminum casing was subsequently filled with a weak cement grout. Initially readings were taken daily but later the reading interval was spread out to approximately three per week. Over the winter and after the remedial construction was accomplished, the reading intervals were further stretched out to about one per month.

Several of the Slope Indicators detected movement within a few days, and within a month it was apparent that slope failure was occurring as a result of stretching along a near horizontal plane at elevation 798 ± 1.0 ft. Figure 3 shows the geologic profile and the detailed movements as recorded by one of the Slope Indicators, and Figure 4 shows the distribution of these movements along a cross section, as well as the location of the failure plane.

Between September 1967 and January 1968, movements were as shown in Figure 4. The data indicated that the slide was of a progressive nature and that further upslope deterioration might be expected. Since the movements were not slowing down but on the contrary were increasing, a temporary buttress of sand and gravel was placed at the toe of the slope to stop the movements while remedial measures were studied. Figure 5 shows how the sand berm placement caused the movements to stop almost immediately at three of the Slope Indicators.

To evaluate the groundwater conditions on the site, seven Casagrande-type piezometers were installed. The two initial piezometers were installed at a relatively high elevation in the glacial drift material and in the upper zone of the underlying shale. The other five piezometers were installed so that the tips were set in the failure zone material at about elevation 798, and periodic observations were made to detect any change in water level with time. The data obtained during two months of observation showed that there were several perched water tables but none with high uplift pressures.

Water levels also were observed in the ten Slope Indicator casings. However, this was not always a reliable means of groundwater observation because grout may tend to seal water either in or out of the aluminum well casing.

The data from the Slope Indicator wells demonstrated conclusively that slope failure was occurring along a nearly horizontal plane only 10 to 15 ft (3.3 to 4.5 m) below the bottom of subcut elevation, and, since the borings had failed to detect any unusual material at this depth, it was decided that a test pit program would be useful in observing the materials along the failure surface.

Eight test pits were dug, varying in depth from 10 to 18 ft (3.3 to 5.5 m). During test pit inspection, it was possible to observe, sample, and test the soil within the failure zone. This was found to be a thin seam, averaging less than 1 in. (2.5 cm) in thickness, of a sodium bentonite. This material had been removed by the wash water during sampling and therefore had not been detected previously. Pocket penetrometer tests were conducted in most of the test pits to determine the in-place shear strength. In addition, Torvane [a 1-in. (2.5-cm) diameter hand-operated vane shear test apparatus designed for in situ testing of soft to medium stiff clays] and in-place torsion shear tests were conducted in one of the test pits.

The first test was conducted on a saturated shale at elevation 800. The second test was run on a moist shale at elevation 804. An attempt was made to run a third test in the soft, bentonitic clays at elevation 798, but because of adverse water conditions and difficulty in preparing the working surface it was not possible to complete this test.

LABORATORY TESTING

The need for an extensive laboratory program was precluded because of the mechanisms of sliding and the character of the soils and rocks involved. Hence, the testing program as such was developed primarily to establish the basic index and engineering properties of the representative samples of the materials encountered. In this respect, water content, Atterberg limits, grain-size gradations, and strength properties were obtained on selected samples.

Laboratory strength characteristics were measured in triaxial compression tests that were performed under conditions closely resembling the in situ condition. Unconfined compression tests were also performed on representative samples of cohesive soils. In addition, several small-scale torsional shear tests were performed in the

Figure 1. Vicinity map.

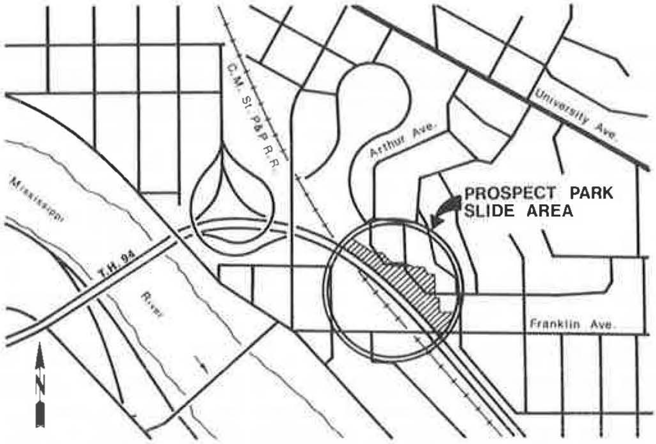
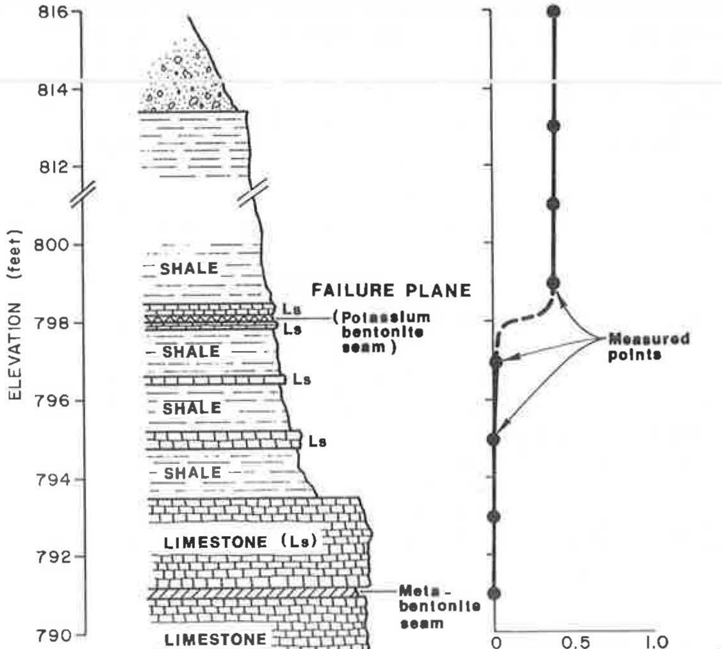


Figure 2. At Slope Indicator no. 4 looking toward railroad bridge and park.

Figure 3. Geologic profile and Slope Indicator observations at failure plane.



laboratory on undisturbed chunk samples obtained from the failure zone. The laboratory tests were designed to obtain an understanding of the shear strength properties of this clay, since knowledge of these properties was important to the design of the recommended remedial measures for this project.

All samples of the Decorah shale and the bentonitic clay were visually inspected and classified in the laboratory.

Atterberg limits were determined on representative samples, and the results showed most of the cohesive soils from the project area to be classified as clays of high plasticity (CH). The bentonitic clay from the failure plane had a liquid limit of 60 and fell 10 points above the A-line on the Casagrande plasticity chart.

In order to determine the shear strength properties of the bentonitic clay found at elevation 798, three small-scale laboratory torsion shear tests were conducted. The laboratory tests were performed in a torsion ring $3\frac{1}{2}$ in. (9 cm) in diameter, having a knurled or roughened face. Each specimen was cast in plaster of paris, fixed in a consolidation apparatus, and trimmed to expose a level test surface. A normal load was applied using dead weights on a loading frame. A rotational shearing force was then applied parallel to the failure surface through a system of cables, pulleys, and dead weights. The relationship between the normal stress and the shear strength was then determined using a procedure similar to that for the field shear test.

The results of the laboratory torsion shear tests are summarized in Figure 6. The results of numerous pocket penetrometer tests and Torvane tests conducted in the various test pits are also shown in Figure 6, as well as the average of results of six unconfined compression tests.

Four unconsolidated-undrained triaxial compression tests (R) were conducted on representative samples of the Decorah shale, and representative core samples from the Platteville formation were also tested. Visual inspection indicated that this latter material was a relatively sound rock with few joints or solution cavities. The results are not included here.

X-ray diffraction tests were conducted at the University of Minnesota on representative samples of the Decorah shale and clay. These tests are used to determine the clay mineralogy of various types of materials, and the results indicated that the shale located above the failure surface is composed almost entirely of illite. In contrast, the soft, gray-brown clays found at elevation 798 were found to contain approximately 50 percent illite and 50 percent mixed layers of montmorillonite clays. These soft clays appear to be similar to the potassium bentonites found in areas to the south of the Twin Cities. In any event, the clay in the failure zone is significantly different from that found either above or directly below the failure surface.

SUMMARY OF SUBSURFACE CONDITIONS

Generally, it was found that the subsurface conditions were quite uniform over the project area, although within various strata there were some complex or disturbed conditions. The subsurface soils are described in general terms in the following paragraphs.

Glacial Drift

Prior to excavation the entire area was mantled with a variable thickness of glacial drift. This material varies considerably in thickness and in consistency over the project area. Within the drift there is a considerable amount of brown, tan, or reddish-brown, silty, gravelly sand, which is believed to be glacial till. However, there are zones or lenses of relatively clean sand, gravel, and cobbles, which may represent outwash material associated with either the advance or retreat of the glacial ice. At various elevations within this project the drift is quite clayey, which might suggest that there were interglacial periods of lacustrine deposition.

Shale

Underlying the glacial drift is the Decorah formation. This formation generally

Figure 4. Typical section showing failure plane and movement distribution.

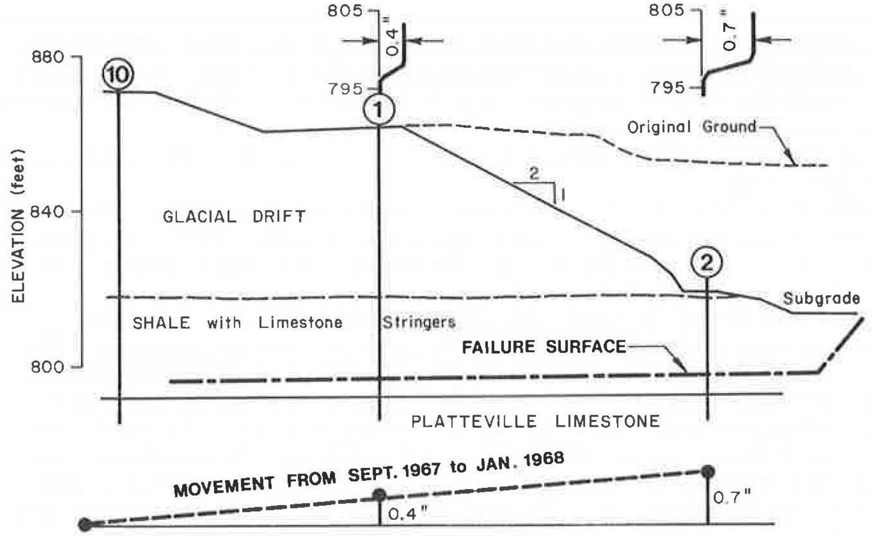


Figure 5. Slope Indicator dial changes with time.

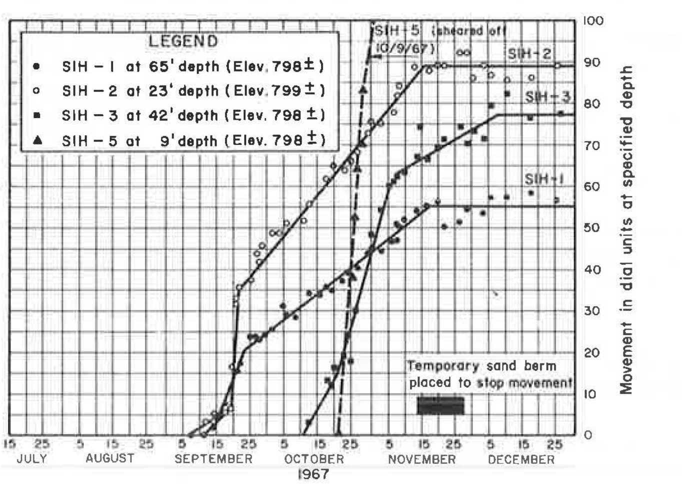
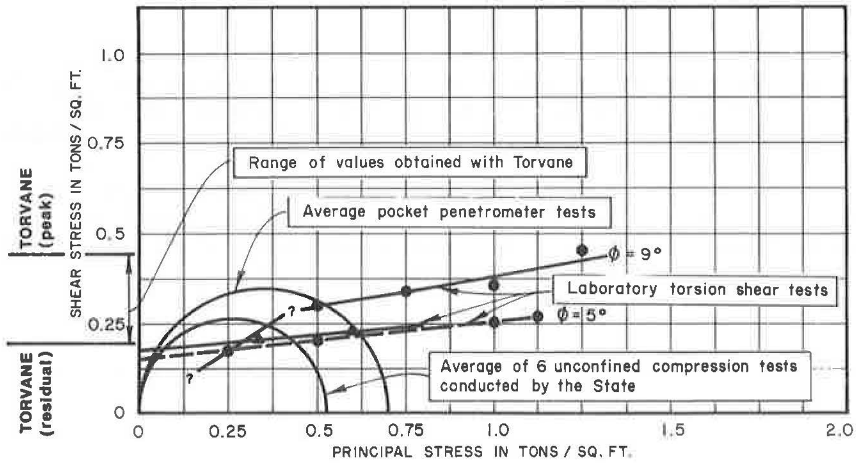


Figure 6. Summary of strength tests on failure zone materials.



consists of a gray-green, blocky, clay shale with thin interbedded (1 to 4-in., 2.5 to 10.0-cm) lime stone stringers. Locally, the formation appears to be quite disturbed. Within the project area, this formation is essentially horizontally bedded. The top elevation varies considerably because of glacial or pre-glacial erosion; however, the lower contact is found at elevation 793.

During the test pit program, a thin horizontal layer of soft, gray-brown, bentonitic clay was discovered in the Decorah shale at elevation 798. The clay was sandwiched between two of the thin limestone stringers. This material probably was not discovered earlier because of the difficulty in obtaining good undisturbed samples in borings under these conditions. As previously stated, the clay mineralogy of this material was found to be significantly different from that of the surrounding shales. It is believed that this bentonitic clay was originally deposited as a volcanic ash in a shallow sea.

Limestone

Underlying the Decorah shale is the Platteville formation. This consists of about 30 ft (9 m) of horizontally bedded gray limestones and dolomites. A thin bentonitic seam was also found in the Platteville formation. This was located approximately 2.5 ft (75 cm) below the top of the Platteville formation and was about 3 in. (7.5 cm) thick. Since the field instrumentation indicated that no movement was occurring in this zone or in the Platteville as a whole, this "metabentonite" zone was not extensively explored or tested.

Below the Platteville formation, sedimentary rocks extend to great depths, but these have no practical significance with regard to this investigation.

To some degree, groundwater was observed in most of the materials explored. Groundwater affecting the area of this study comes from two primary sources: surface runoff and deep aquifers from unknown sources. These water conditions undoubtedly influenced the stability of the cut slope, but relatively little preconstruction groundwater data were available. However, water levels in a few of the older borings and in an adjacent excavation indicate that the water table in the vicinity of the roadway centerline was considerably higher than the bottom elevation of the subcut. Therefore, the water trapped beneath the impermeable shale may have developed artesian pressures with relation to the excavated ground surface.

Primary emphasis in this investigation was directed toward an evaluation of the field, rather than the laboratory, properties of the soils and rocks encountered. An indiscriminate application of the laboratory strength properties in evaluating the stability problems might have been misleading. Because of the character of the materials that control the slope behavior and the mode of failure involved, the laboratory properties have served primarily as a means for relating quantitatively the soils existing throughout the site with those soils on which strength tests were performed. In addition, they have provided a basis for comparing the behavior of the Minneapolis materials with experience gained from other areas and with experience reported in the literature. Table 1 gives a summary of geology and engineering properties as used in subsequent failure analyses.

EVALUATION AND CORRECTIVE DESIGN

Slope stability computations were conducted on three representative cross sections where the limits of sliding were clearly defined by Slope Indicator data and by field observation of the heaving and cracking. In addition, the geology was more completely mapped at these locations.

Since the movement developed along a horizontal plane surface, the sliding wedge type of analysis was used. Hydrostatic pressures were assumed, based on available piezometric data. The location of the failure plane was determined from geologic observations and from the Slope Indicator data.

The parameters used for analyses, the general method of analysis, and the summary of results are shown in Figure 7. No seismic loadings were used in performing the stability analysis for this project.

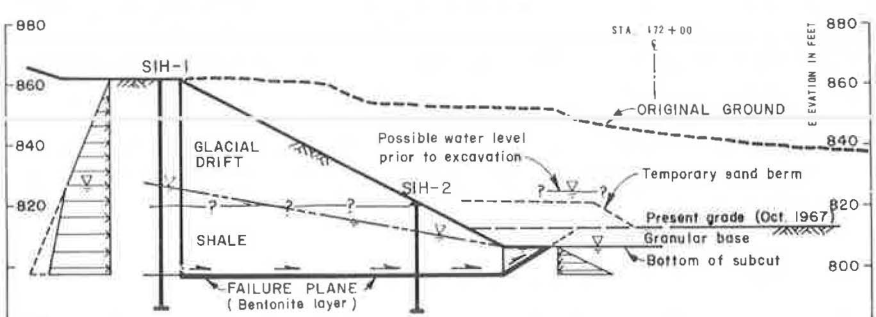
It was determined from testing that the strength of the bentonitic clay along the

Table 1. Description of materials encountered.

Material and Description	Properties	Testing Methods[a]
Recent deposits: Primarily glacial drift mantle (till and outwash materials); generally, brown, tan, or reddish-brown silty sand with varying quantities or lenses of gravel and cobbles	Unit weight (moist) = 130 pcf (2.0 kg/m^3) Shear strength ϕ = 35 deg c = 0	M, A
Decorah formation: Primarily a blocky, gray-green, clay shale with occasional thin limestone stringers	Unit weight (moist) = 130 pcf (2.0 kg/m^3) Shear strength ϕ = 9.5 deg (saturated) = 21.5 deg (moist) c = 0.22 tsf (0.22 kg/cm^2) (moist or saturated)	R, F, P, U
Failure zone: Within the Decorah at the elevation of the failure surface there is a 1- to 4-in. (2.5- to 10-cm) thick layer of soft, gray-brown clay (potassium bentonite)	Shear strength ϕ = 6 deg c = 0.2 tsf (0.2 kg/cm^2) residual = 0.4 tsf (0.4 kg/cm^2) peak	U, T, L, P
Platteville formation: Varies from a hard, gray to light gray limestone to a dolomitic limestone to a dolomite; the limestone has a relatively insignificant number of solution cavities.	Unit weight = 165 pcf (2.6 kg/m^3) Unconfined strength = 9,000 psi (600 kg/cm^2) min Modulus E = 5,000,000 psi (2500 kg/cm^2) min	S
Metabentonite layer: 0.25 ft (10 cm) thick, located 2.3 ft (0.6 m) below the top of the Platteville formation	Not tested during this investigation	

[a] Methods of determining average engineering properties:
M = Average of values supplied by state agency
R = Triaxial compression tests
U = Unconfined compression tests
T = Torvane
F = Field torsion shear
L = Laboratory torsion shear
A = Assumed values
S = Schmidt impact hammer
P = Pocket penetrometer

Figure 7. Stability analysis.



STATION	FACTORS OF SAFETY: AFTER ROADWAY EXCAVATION Aug. 1967: PEAK STRENGTHS AT REST PRESSURES	FACTORS OF SAFETY: AFTER ROADWAY EXCAVATION Aug. 1967: RESIDUAL STRENGTHS ACTIVE PRESSURES	FACTORS OF SAFETY: WITH PROPOSED SLIT-TRENCH BUTTRESSES AND CANTILEVER RET. WALL
166 + 70	0.84	0.91	>1.50
170 + 00 EXCAVATION TO SUBCUT LEVEL	0.85	0.87	—
170 + 00 WITH SUBCUT BACKFILLED	0.94	0.99	1.49
170 + 00 WITH TEMPORARY SAND BERM	—	>1.15	—
172 + 60	1.05	1.00	>1.50

failure surface was reduced as movement progressed. Therefore, a peak strength value of c = 0.4 tsf (0.4 kg/cm^2) was assigned to the clay for the condition immediately prior to the start of movement, but once movement remolded the clay, the strength dropped to the residual value of c = 0.2 tsf (0.2 kg/cm^2).

With a horizontal failure surface, the only available driving forces are those produced by the hydrostatic forces and the lateral earth pressure. Prior to roadway excavation, lateral earth pressures existed but were balanced so that no movement could occur. Excavation unbalanced these to the point where the lateral earth pressures, along with other factors, caused strain along the failure surface. Prior to excavation, the lateral earth pressure was in the at-rest condition. To cause failure, it was determined that the coefficient of lateral earth pressure at rest (K_o) would have to have been equal to 0.43 in the glacial drift and 0.76 in the Decorah shale. Once movement started, the lateral earth pressure dropped to a minimum value equal to the coefficient of active earth pressure (K_a). This value was determined to be K_a = 0.26 in the glacial drift and K_a = 0.50 in the Decorah shale. The results of these studies indicate that the reduction in lateral earth pressure from the at-rest condition to the active condition has a greater effect than the reduction in shear strength in the clay from the peak value to the residual value. Therefore, the slide did not accelerate with time.

The highway department desired that any structural scheme for hillside stability provide a factor of safety of approximately 1.5. By working backwards through the stability computations, it was determined that there would have to be an additional resisting force of 70 kips per lineal foot to provide the desired factor of safety. Because of this conservative factor of safety, it was recommended that the remedial structures be designed for ultimate stresses with no superimposed factor of safety.

In addition, it was determined that a retaining structure should extend at least 10 ft (3 m) above the final roadway elevation. The weight of the backfill behind this wall would prevent the possibility of a slope failure, which would come over the top of a lower structure. Further, it was considered desirable that all schemes be keyed or tied into the Platteville limestone at or below the elevation of the "metabentonite" seam at elevation 791.

Based on these design criteria, six different remedial schemes were studied, as summarized in the following paragraphs.

Slope Flattening

Although slope flattening is occasionally feasible, there were disadvantages:

1. Slope flattening is not a positive means of stopping movement, and a creep-type movement could continue for an indefinite period of time.
2. This scheme would require the purchase of private and public land and homes in a well-established neighborhood.
3. Even if landscaped, a large flat slope probably would not esthetically fit in with the design of adjacent sections of the highway.

Cylinder Pile Retaining Wall

Another scheme provides for stabilizing the cut slope by means of drilled, cast-in-place, reinforced-concrete cylinder piles. In essence, a cylinder pile is a cantilever member that is embedded deep below the failure surface and designed to resist the design loads. These are usually constructed as a continuous wall.

There are several significant advantages in the use of cylinder piles:

1. They provide a positive means of stopping the movement.
2. They may be constructed with a minimum of excavation, prior to the removal of the temporary sand berm.
3. Even under wet conditions, the drilling and casting of cylinder piles is a relatively simple construction procedure.

However, the estimated cost of this scheme was found to be excessive.

Anchored Cylinder Piles

A modified cylinder pile wall was considered that would reduce the quantity of drilling and the quantity of reinforcing steel. Batter tension or compression piles would be used to tie back the top of the drilled-in cylinder piles. However, the cost of preboring and driving the piles offset the savings in reduction of drilling and reinforcing steel.

Braced Soldier Piles

In a scheme that utilizes methods commonly used for making braced cuts for large trenches and for building foundations, the first step would be to drive a continuous row of soldier piles to the top of the Platteville elevation. These soldier piles probably would be steel H piles. Then, in short segments, a continuous wale would be constructed in front of the soldier piles. Finally, at regular intervals, reinforced concrete struts would be cast in narrow trenches excavated under the future roadway. With this system, the design loads would first be transmitted into the soldier piles, which in turn would distribute them into either the continuous wale or into the shale formation below the failure surface. That portion of the load carried by the wale would then be transmitted to the struts. In turn the struts would act essentially as long, horizontal friction piles. Any load not transferred out of the struts by friction would be carried by end bearing of the strut on the opposite side of the roadway. A 10-ft (3-m) cantilever wall, required to prevent slope failure over the top of the soldier piles, would be founded on the soldier piles plus any other piles required for stability.

As in the case of the other alternate schemes, this would be a positive means of stabilizing the hillside; it has the advantage of eliminating the expensive rock drilling required for cylinder piles. The primary disadvantage would be the need to excavate parallel to the toe of the slope during construction of the required wale. If not done properly, this excavation would trigger additional movement which might damage the soldier piles. In addition, there was a possibility that pile-driving vibrations could cause damage to existing structures or renew slope movement.

Anchored Slab

The anchored slab scheme essentially replaces a portion of the vertical load previously removed by roadway excavation with a new vertical load developed by tensioning prestressed vertical anchors. The Slope Indicator observations proved that a relatively small sand berm stopped the movement. A prestressed, anchored slab would provide the equivalent of a 15-ft (5-m) high berm over the entire roadway. To construct the anchored slab, it would be necessary first to remove small sections of the existing temporary berm prior to the casting of the slab. Anchor holes would then be drilled and steel anchors grouted into place. The final step before paving would be to stress the anchors to provide a 1.0 tsf (1 kg/cm^2) area load on the slab. The 10-ft (3-m) high cantilever wall along the toe of the slope would be pile-supported, as in most of the other schemes.

Although considered feasible, there are no significant advantages to this scheme as compared to the others. The main disadvantage would be the need to excavate areas of the temporary berm prior to the casting of the slab. In addition, anchors would have to be protected against the possibility of long-term groundwater corrosion.

Slit-Trench Buttresses

The remedial scheme finally recommended and adopted utilized reinforced concrete buttresses cast in narrow slit trenches excavated normal to the roadway centerline (Fig. 8). The design loadings on the buttresses are also shown in Figure 8. These buttresses were 3 ft (1 m) thick and spaced 12 ft (4 m) center to center and were designed as gravity sections. Each buttress was founded on the top of the Platteville limestone but was keyed into the Platteville with two 3-ft (1-m) diameter concrete filled calyx holes, heavily reinforced for shear. The horizontal load was transferred to the upper layer of the Platteville limestone, which was assumed to act as a start, and

Figure 8. Typical section of recommended remedial slit-trench buttresses.

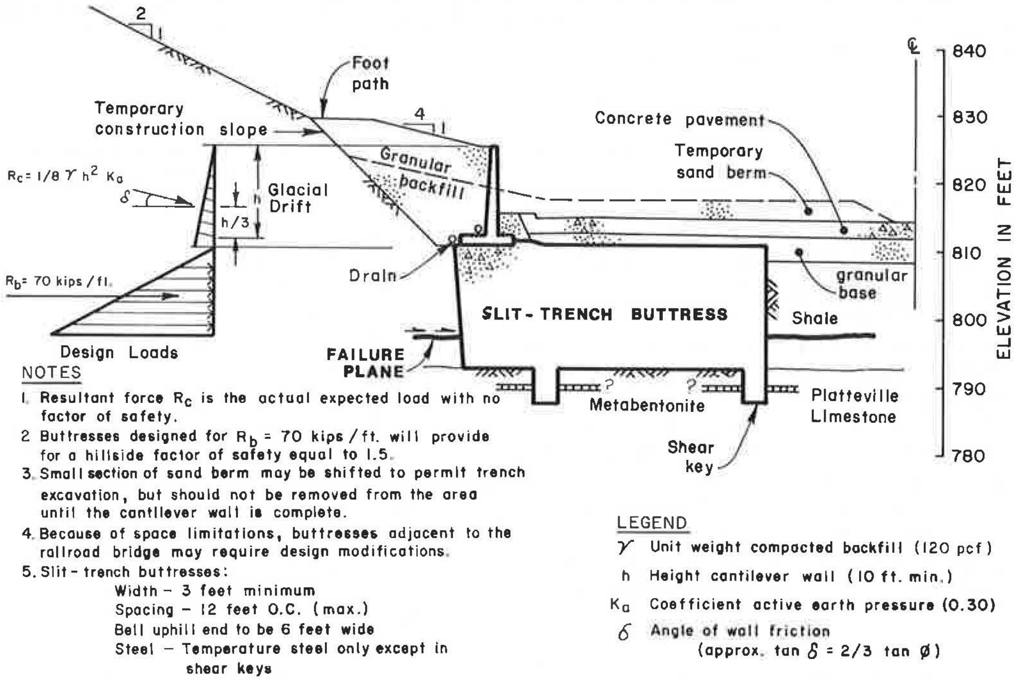


Figure 9. During construction, May 4, 1968.

the vertical load was transferred into the limestone below the metabentonite seam.

The retaining wall constructed above the slit-trench buttress was designed as a conventional cantilever retaining wall except that the footing was designed to span the 9-ft (3-m) distance between adjacent buttresses.

CONSTRUCTION

An essential feature of the recommended remedial design was that construction not further endanger the stability of the existing hillside. Therefore, all phases of construction were closely inspected and Slope Indicators and piezometers located uphill from the work area were read at least every other day during construction.

It was anticipated that at least three buttress trenches could be under construction at any one time, without significantly reducing the stability of the area, so long as there was at least 50 ft between adjacent excavations. Figure 9 is a view of construction in progress, showing the many activities going on at any one time. After a few initial problems, construction proceeded in an orderly, systematic fashion.

The temporary sand berm was left in place during buttress and retaining wall construction. However, to facilitate the construction operation, local portions of it could be temporarily moved. It was specified that, whenever possible, buttress construction was to be accomplished as one continuous operation. This meant that, once excavation had started on a particular buttress, it was not to stop until the final concrete was poured. During the actual construction, the importance of excavating the slit trench and pouring the concrete as soon as possible was confirmed. Several of the trench excavations were left open for 24 hours or longer. As a result, the trench sides caved in during the concrete pour, resulting in delays. This caving occurred even with bracing.

The Slope Indicator observations reflected the construction procedures. As areas of the berm were removed and the trenches excavated, especially where more of the berm was removed than for one buttress, the Slope Indicators showed the resumption of movement. As the buttresses were completed, the movement ceased. No movement has been detected since the buttresses have been completed. A total of 89 buttresses were constructed.

Although the construction took place during a very wet summer, groundwater was not a major problem. Sump pumps were able to dewater the keyways and the bottom of the excavation without any problems.

With completion of a series of adjacent buttresses, wall construction followed closely behind. Thus, the Franklin Avenue end of the wall was completed before the buttresses toward the railroad end were complete. This procedure enabled the contractor to complete and open the job to traffic on schedule on December 9, 1968.

ACKNOWLEDGMENTS

The detailed design of the remedial measures described herein was accomplished by the Minnesota Department of Highways Bridge Section. The staff of F. C. Fredrickson, Materials Engineer, was particularly helpful in handling details of the exploration program and in installing and reading the field instrumentation. Ray Adolfson maintained continuous liaison.

This investigation was under the direct supervision of the author, with the able assistance of E. D. Schwantes.

AN EMBANKMENT SAVED BY INSTRUMENTATION

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A case history of an incipient embankment failure that was averted is described. Plastic flow of a weak layer of soil in the lower portion of the embankment was responsible for the problems encountered. A soils investigation, slope indicators, and survey lines provided the information required to establish the movements taking place within the embankment. This information led to an understanding of the problem and design of proper corrective measures that allowed construction to proceed. After the movements were stabilized, the embankment functioned satisfactorily.

•THE California Department of Transportation makes extensive use of instrumentation to monitor the performance of embankments, embankment foundations, and cut slopes. Instrumentation is invaluable for providing information needed to analyze a landslide and to design corrective measures. The most valuable use, however, is in monitoring incipient landslides and in providing warning signals before complete failure occurs, allowing redesign of the facility at a minimum of cost and risk. This report gives a case history in which slope indicators and survey lines signaled the need for redesign of an embankment and also showed that complete removal and rebuilding of the embankment were not necessary.

PROJECT DESCRIPTION

The embankment in question was part of a highway project being built on US-101 on the north coast of California near the community of Orick. At this location, the alignment traverses a deep ravine (Fig. 1) requiring an embankment about 110 ft (33 m) high. The bottom of the ravine is composed of 15 to 30 ft (4.5 to 9 m) of weak organic silts and clays underlain by dense sand and decomposed shales and schists. The design called for the organic silts and clays to be removed, a 3-ft (1-m) blanket of permeable material and 8-in. (20-cm) diameter underdrains to be placed at the bottom of the stripped area to remove groundwater, and then construction of the embankment with material from excavation. The weak organic material was used to construct a 30-ft (9-m) high by 50-ft (15-m) wide strut on the downhill side of the embankment to provide additional stability.

The material used to build the embankment came from nearby excavations and consisted of a mixture of weathered shale, sandstone, and schist. Because extensive groundwater was found in the cuts, all the material exceeded the optimum moisture content for compaction and could not be suitably dried because of the foggy coastal weather. The contractor was still able to meet the 90 percent relative compaction requirements by judicious blending of the wet and some drier materials found in the cuts.

PROBLEM DESCRIPTION

Construction of the embankment began during the summer of 1971. By the end of October the embankment had reached a height of about 50 ft (15 m) when some lateral movement was noticed on the strut. An inspection of the 42-in. (1.1-m) corrugated steel pipe (CSP) installed in the fill (Fig. 2) revealed that several of the joints had pulled apart, the locations of which are shown by the circled numbers in Figure 2. The amount of displacement is given in Table 1.

Figure 1.

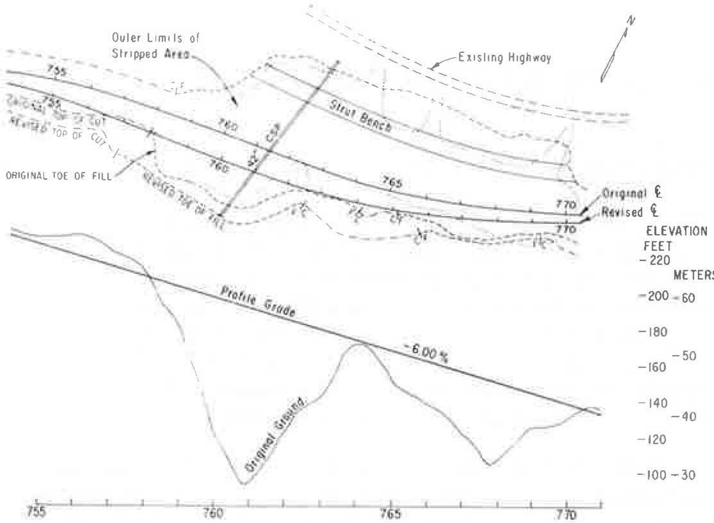


Figure 2.

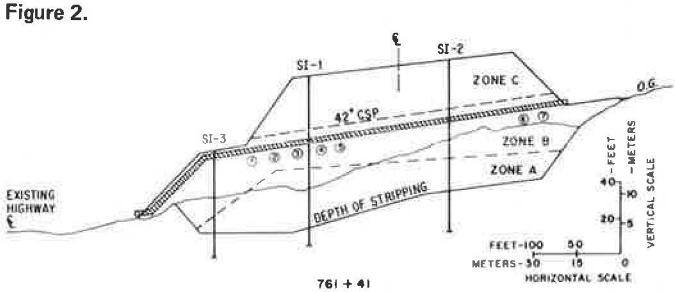


Table 1. Amount of joint displacement.

Joint	Displacement (in.)	Joint	Displacement (in.)
1	3	5	8
2	6	6	½
3	20	7	½
4	25		

Note: 1 in. = 2.54 cm.

Table 2. Embankment properties.

Zone	Cohesion[a] (lb/ft^2)	Relative Compaction (percent)	Wet Density (lb/ft^3)	Moisture Content (percent)
A	2,000	94	143	15-20
B	1,200	91	130	15-25
C	2,000	95	135	10-15

Note: 1 lb/ft^2 = 47.88 Pa; 1 lb/ft^3 = 16 kg/m^3.

[a]Unconsolidated undrained triaxial shear tests.

Because of the large amount of movement, construction was immediately suspended and an investigation was begun. Fortunately, the winter rainy season was just beginning so that a construction delay was not necessary. Construction records were reviewed, undisturbed samples were obtained from several borings, three slope indicators were installed as shown in Figure 2, and several survey lines were established on top of the embankment, on the strut, and on original ground at the toe of the strut to measure vertical and horizontal movement.

RESULTS OF INVESTIGATION

Compaction tests taken during construction of the embankment and laboratory tests on the undisturbed samples correlated fairly well and gave a good indication of the soil properties. They showed that the embankment could be characterized by three zones, A, B and C, as shown in Figure 2. The average soil properties of these zones are given in Table 2.

Slope Indicator SI-2 (Fig. 4) and survey lines on top of the embankment showed that a maximum of ½ in. (12 mm) of irregular lateral movement had occurred on the uphill (right) side of the embankment. Slope Indicators SI-1 and SI-3 (Figs. 3 and 5) revealed that movement was occurring in the weaker soil of Zone B on the downhill (left) side of the embankment. Survey lines on the strut indicated that the movement was primarily occurring between Stations 760 and 761. Vertical settlement of the embankment was a maximum of 0.20 ft (6 mm). No lateral movement was detected by the survey line between the toe of the strut and the existing highway.

An analysis of these movements and of available soil properties suggested that plastic flow was occurring in the weak soil of Zone B rather than a classical circular type of movement. A stability analysis assuming a circular failure surface and using the soil properties listed in Table 2 resulted in a safety factor of 1.1 for existing conditions and a safety factor of about 0.8 if the embankment had been completed without any corrective action. This indicates that a circular failure arc probably would have developed had construction continued.

CORRECTIVE MEASURES

Several methods of correcting the problem were considered. Removing the embankment and reconstructing it were rejected for reasons of cost and construction delay. Widening and raising the elevation of the existing strut would have provided stability. This alternative, however, would have required removing a portion of the existing strut and extending the stabilization trench toward the left to support the strut. Stability calculations indicated that the safety factor would be lowered to an unsafe level during the stripping operations, and therefore this alternative was rejected. Lowering the profile grade through the embankment area was not seriously considered because the roadway was already on the maximum 6 percent grade (Fig. 1).

The alternative finally selected was a line shift to the right. Although a shift of 100 ft (30 m) or more would have been desirable, it would have required taking additional right-of-way from a state park. The maximum line shift attainable within the existing right-of-way was 68 ft (21 m). Because of the fear that completion of the embankment might cause a deep-seated failure and endanger the existing highway, the outer 68 ft (21 m) of the completed embankment above the strut was removed (Fig. 6) to reduce the total load on the foundation. Later events showed that this precautionary measure was unnecessary because deep-seated movements did not develop. Stability analyses of the redesign showed a safety factor of 1.2 immediately after construction using undrained strengths and a safety factor of 1.3 using effective strength parameters.

FURTHER PROBLEMS

With the end of the rainy season, construction was resumed in May 1972 on the revised alignment. The fill was widened 68 ft (21 m) on the right while 68 ft (21 m) of the existing fill on the left was removed (Fig. 6). As fill height increased, bulging was

Figure 3.

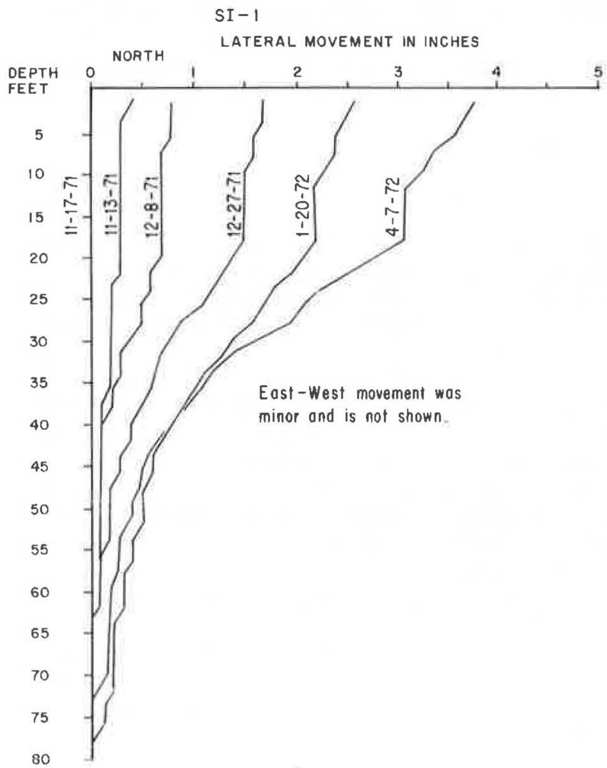


Figure 4.

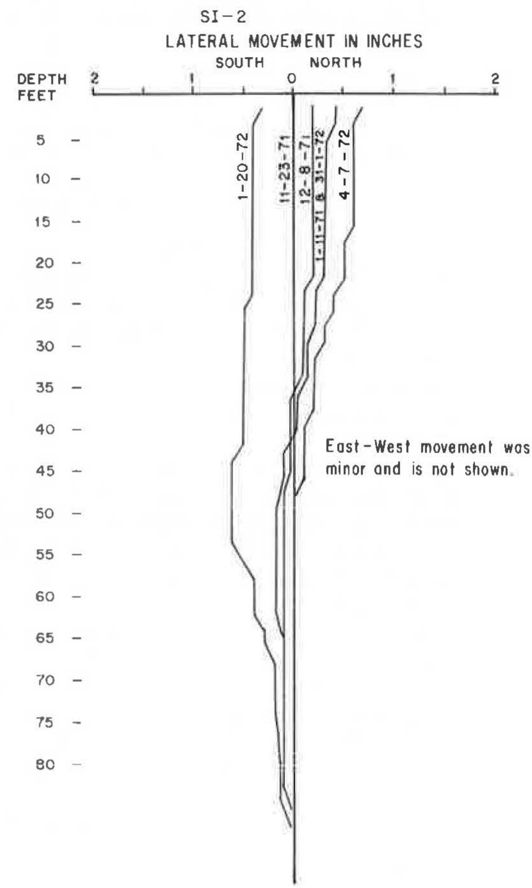


Figure 5.

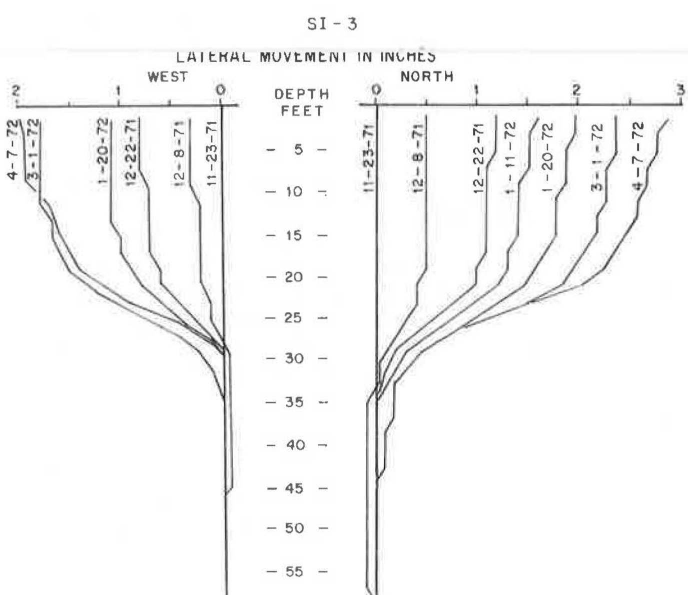


Figure 6.

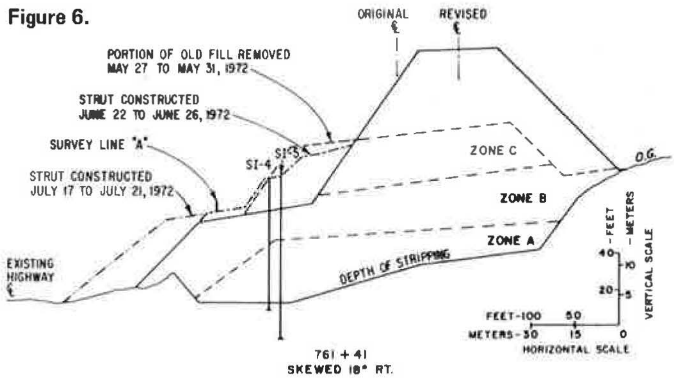


Figure 7.

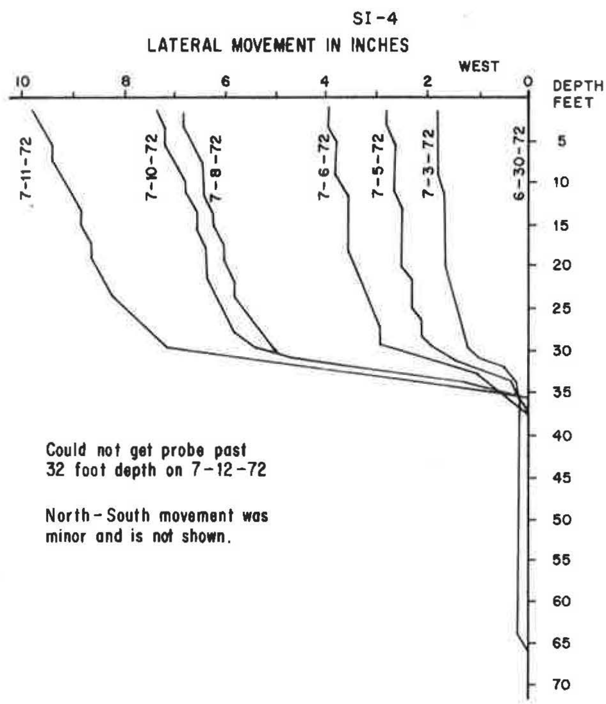


Figure 8.

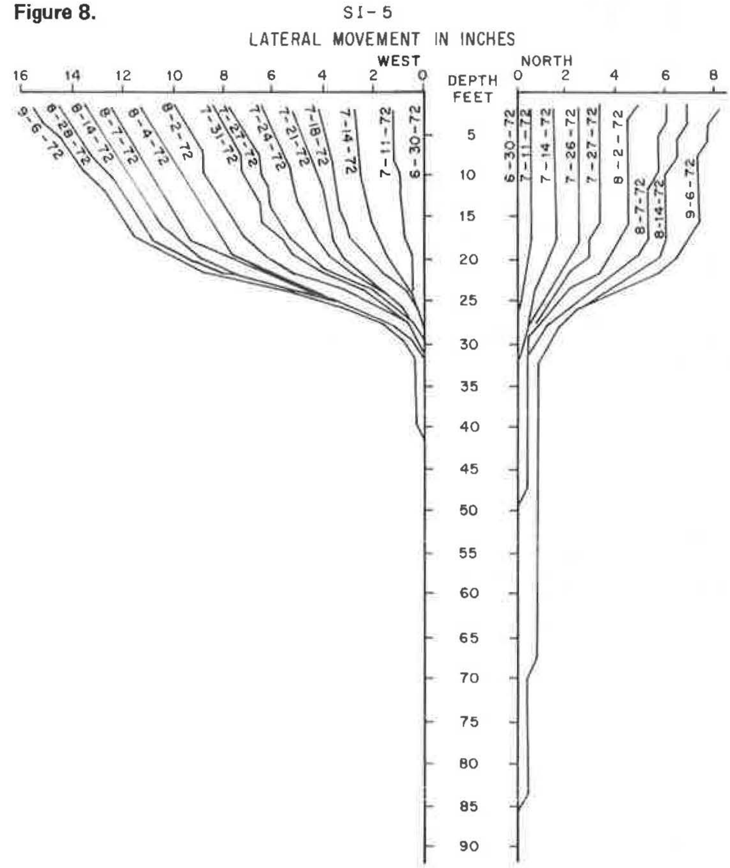


Figure 9.

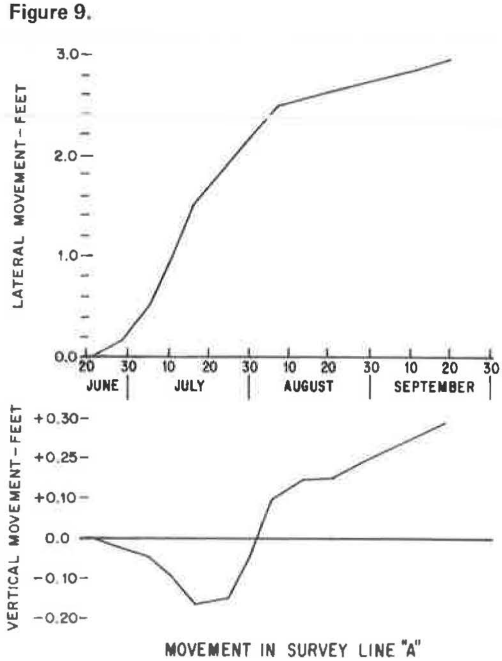


Figure 10.

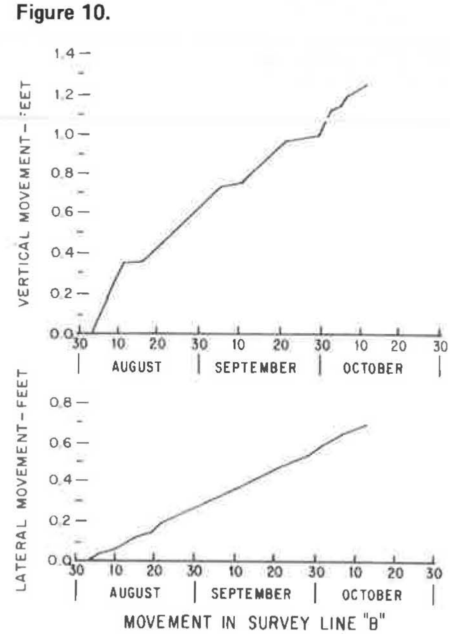


Figure 11.

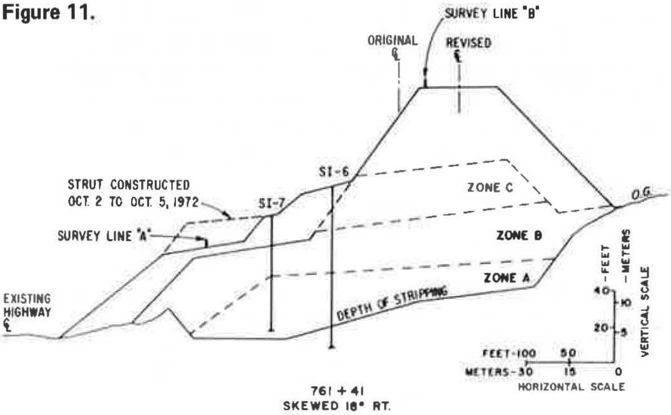


Figure 12.

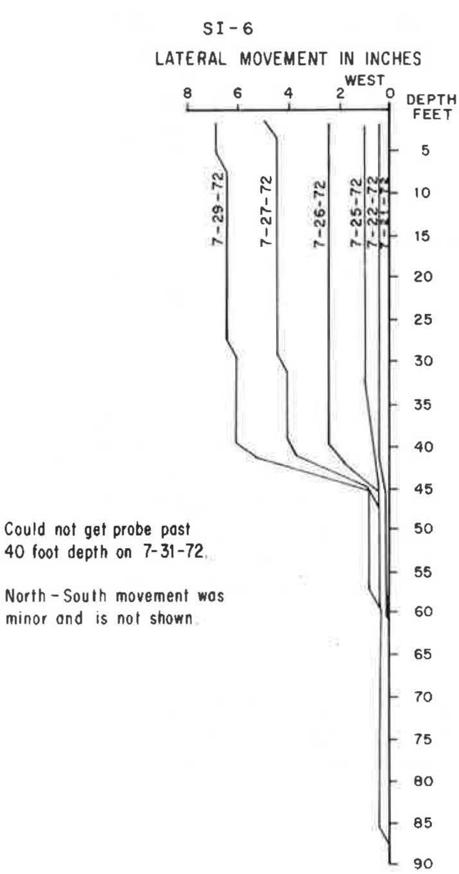
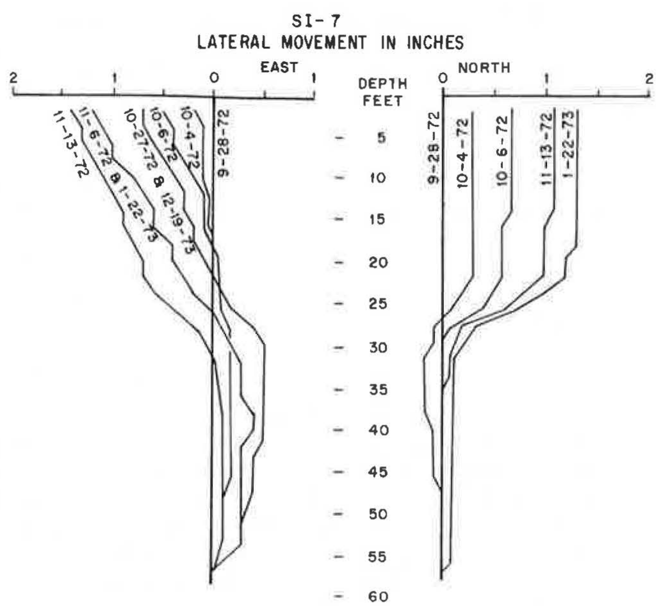


Figure 13.



seen in the left side of the fill above the strut, indicating that further movement was taking place. No vertical or horizontal movement was measured on the survey line adjacent to the existing highway, indicating that movement was confined to the embankment and therefore not deep-seated. In late June, additional material was placed on top of the original strut to replace that portion of the embankment removed in May (Fig. 6). At the same time, a controlled rate of loading of 1 ft (0.3 m) per day was imposed on the embankment construction.

Slope Indicators SI-4 and SI-5 (Figs. 7 and 8) were installed in this new strut to monitor the movement as construction continued. With the completion of the embankment in the second week of July, SI-5 continued to show an alarming rate of movement in Zone B. Survey line "A" (Figs. 6 and 9) on the new strut revealed that lateral movement and settlement were accelerating. This led to the decision to widen the original strut about 80 ft (24 m) to the edge of the existing highway. Lateral movement in Zone B continued but at a slightly slower rate. Also, survey line "A" stopped settling and began to rise, indicating that the movement had been shifted to a higher level in Zone B.

In the middle of September crescent-shaped cracks started appearing in the top of the embankment. Survey line "B" (Figs. 10 and 11) on top of the embankment showed continuing settlement and a lesser rate of lateral movement. Since survey line "A" was rising in elevation, it was felt that additional strutting at the location of line "A" would control the movement. This was constructed in the first week of October. Slope Indicator SI-7 (Fig. 13), installed at the same time, showed that lateral movement was effectively stopped even though settlement of the embankment continued.

Horizontal drains were then installed through the outer strut in order to accelerate the drainage and thus the strength gain of the soft saturated soil in Zone B. Although six drains were installed, only one produced a slight amount of water. Lateral movement was nil at this time, and vertical movement gradually ceased in the next few weeks. The embankment has functioned satisfactorily since.

A continuous record of the lateral movement of the embankment was not obtained. The movements that were measured, however, total over 9 ft (3 m) in the downhill direction.

CONCLUSIONS

A thorough soils exploration and instrumentation program provided the information needed for analysis of the problem and led to the design of proper corrective measures that allowed construction to proceed. The settlement and cracks in the top of the embankment and heaving of survey line "A" that occurred during September and October of 1972 would surely have been interpreted as the beginning of a slide requiring major reconstruction except for the information provided by one Slope Indicator, SI-7. It showed that no lateral movement was taking place, and therefore expensive reconstruction was not necessary.

ACKNOWLEDGMENTS

The construction of the embankment was done under the supervision of Transportation District 01 personnel. Particular thanks are extended to District Materials Engineer M. O. Clemens and Resident Engineers H. A. Davis and J. A. Mouat, who provided invaluable information for this paper.

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