

Geological, Geophysical, and Engineering Investigations of the Loveland Basin Landslide, Clear Creek County, Colorado, 1963-65

Geological Investigations
Electrical Resistivity Investigations
Seismic Refraction Studies
Engineering Investigations
Compilation of Results of Geological, Geophysical,
and Engineering Investigations
Stability Analysis
Summary and Recommendations

GEOLOGICAL SURVEY PROFESSIONAL PAPER 673-A, B, C, D, E, F, G



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Engineering Investigations of the
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FRONTISPIECE.—Loveland Basin landslide, view northwestward (1965).

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Geological Investigations

By CHARLES S. ROBINSON *and* FITZHUGH T. LEE

Electrical Resistivity Investigations

By R. WOODWARD MOORE

Seismic Refraction Studies

By RODERICK D. CARROLL, JAMES H. SCOTT, *and* FITZHUGH T. LEE

Engineering Investigations

By JOHN D. POST *and* CHARLES S. ROBINSON

Compilation of Results of Geological, Geophysical, and Engineering Investigations

By CHARLES S. ROBINSON *and* FITZHUGH T. LEE

Stability Analysis

By ROBERT A. BOHMAN

Summary and Recommendations

By CHARLES S. ROBINSON *and* FITZHUGH T. LEE

GEOLOGICAL SURVEY PROFESSIONAL PAPER 673-A, B, C, D, E, F, G

*A study using several methods of investigation
to define the geometry and behavior of a
landslide*



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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

By CHARLES S. ROBINSON, FITZHUGH T. LEE, and others

INTRODUCTION

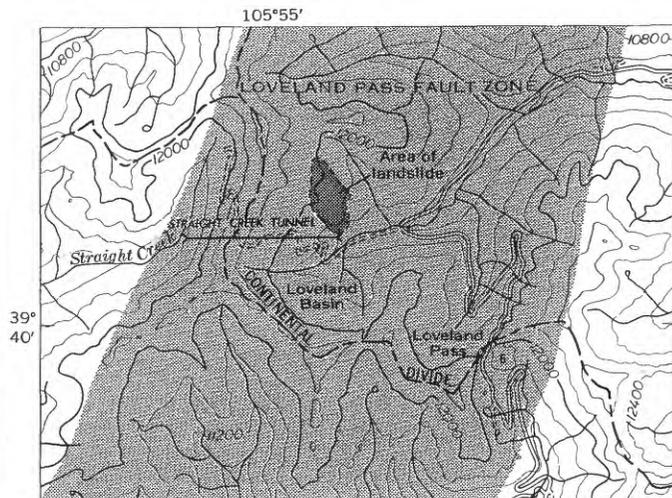
By CHARLES S. ROBINSON and FITZHUGH T. LEE

The Loveland Basin landslide that occurred in the Spring of 1963 was caused by an excavation made for the approach road to the east portal of the Straight Creek Tunnel. The site of the tunnel is about 55 miles west of Denver (fig. 1) on the north side of Loveland Basin and northwest of the Loveland Basin ski area.

A contract for the construction of 2 miles of Interstate 70 that will serve as the east approach road for the Straight Creek Tunnel was awarded in August 1962. In June 1963, much of the construction under this contract had been completed, and the cut for the east portal of the Straight Creek Tunnel was nearly finished. At that time, F. W. Fitzpatrick, project engineer for the Colorado Department of Highways, while examining the slope above the top of the cut, noted landslide scarps. He notified G. N. Miles, district engineer, who in turn notified the division personnel of the U.S. Bureau of Public Roads. The U.S. Geological Survey was alerted by these agencies because its personnel were then engaged in a research project for the Straight Creek Tunnel pilot bore, which had a direct bearing on conditions at the east portal.

Action had to be taken immediately to stabilize the landslide because the Colorado Department of Highways are planning to let a contract early in the fall of 1963 for the construction of the pilot bore for the Straight Creek Tunnel. The landslide was above and north of the east portal. Before remedial procedures could be planned, the volume and mass of the landslide had to be determined, and if possible, some evaluation of its rate of movement had to be made. Joint conferences of the Colorado Department of Highways, the U.S. Bureau of Public Roads, and the U.S. Geological Survey resulted in decisions to map the landslide geologically and to try to determine the thickness of the slide by geophysical methods. The authors believed that drilling the slide would be impractical because of the rela-

tive inaccessibility and the nature of the geology of the slide area. R. Woodward Moore, assisted by other members of the U.S. Bureau of Public Roads and by members of the Colorado Department of Highways, made the resistivity measurements in



Base from U.S. Geological Survey,
Loveland Pass, 1:24,000, 1958

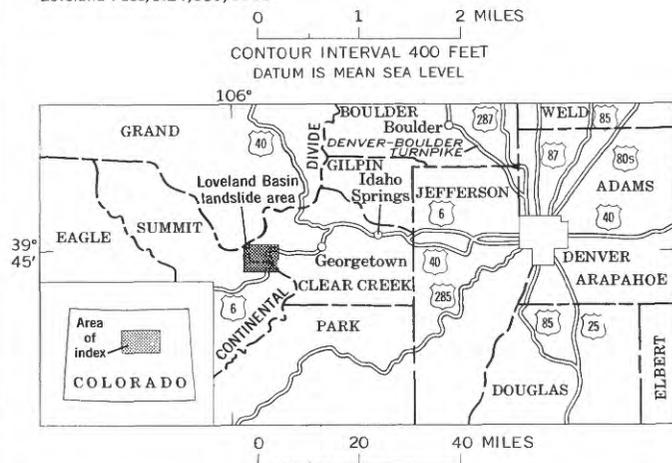


FIGURE 1.—Index maps of the Loveland Basin landslide area, Clear Creek County, Colo. Stipple indicates area of Loveland Pass fault zone.

the slide area. R. D. Carroll and J. H. Scott, assisted by other members of the U.S. Geological Survey, made the seismic measurements on the slide. C. S. Robinson and F. T. Lee mapped the geology of the slide area and surveyed the resistivity and seismic lines by planetable methods. Control for the planetable mapping was established by a transit survey around the slide by the Colorado Department of Highways. At that time the department also had a topographic map of the landslide area prepared from aerial photographs taken in 1961.

Approximately 3 weeks in July 1963 were required to obtain the geologic and geophysical data. During this time, the Colorado Department of Highways had two holes drilled in the plaza area at the bottom of the cut. (See fig. 2.) The department inserted plastic casing in these holes and probed the holes daily for any movement. It also established a first-order grid on 50-foot centers in the plaza area and kept records of the movement of these points.

The geological and geophysical data were compiled and interpreted, and a report giving the volume and mass of the slide was prepared and submitted by Robinson, Carroll, and Lee (1964) to the U.S. Bureau of Public Roads and the Colorado Department of Highways in September 1963. These data were then used by R. A. Bohman, U.S. Bureau of Public Roads, in analyzing the stability of the landslide and in calculating the load necessary to stabilize the slide. Late in September 1963, compacted fill material was placed at the base of the slide to serve as a buttress. The load was emplaced

by October 1963. Low-angle drainage holes were drilled into the slip zone at various locations near the western and southern perimeter of the slide, and these were lined with perforated casing. Additional drainage was accomplished by the construction of French drains and the installation of corrugated-metal pipe culverts. The contract for the construction of the Straight Creek Tunnel pilot bore was awarded late in October. For about 2 years, periodic observations of the slide area were made, mainly by John D. Post of the Colorado Department of Highways.

The authors have little firsthand information regarding the behavior of the landslide since 1965. The Colorado Department of Highways (oral commun., 1969) believes that the lower part of the landslide, northwest of the buttress, has deepened and that the northwest and northeast bordering scarps have lengthened during 1968 and 1969. Steel supports in the east portal of the Straight Creek Tunnel have tilted eastward near the crown line, possibly owing to landslide movement. The landslide remains a great concern to those involved in the safety and economic completion of the Straight Creek Tunnel.

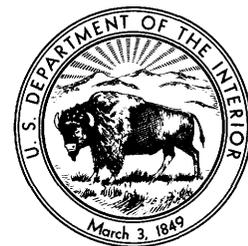
The purpose of this report is to present the methods used to investigate the Loveland Basin landslide during the period of June 1963 through June 1965 and the results of those investigations. The work is presented in seven topical chapters; each chapter is written by those persons principally responsible for the work.

Geological Investigations

By CHARLES S. ROBINSON *and* FITZHUGH T. LEE

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
1963-65

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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND
BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

GEOLOGICAL INVESTIGATIONS

By CHARLES S. ROBINSON and FITZHUGH T. LEE

ABSTRACT

The Loveland Basin landslide occurred in a high mountain valley near the Continental Divide about 55 miles west of Denver during construction of the east portal area for the proposed Straight Creek Tunnel. The landslide developed in Precambrian bedrock, consisting of Silver Plume Granite with inclusions of metasedimentary rocks that are chiefly biotite gneiss and schist. Thin deposits of surficial material, consisting of moraine and soil mixed with talus, overlie the bedrock.

The entire area is within the Loveland Pass fault zone. Individual faults within this zone are characterized by masses of breccia and gouge, ranging in width from less than 1 foot to 1,000 feet; these masses are separated by masses of less intensely sheared rock. The preexisting structure partly determined the shape of the landslide.

The landslide is outlined by distinct scarps. The upper two-thirds of the landslide is downdropped in relation to the surrounding rock. The lower one-third of the landslide has been thrust upward and outward over the surrounding topography. Within the landslide mass, movement is indicated by tension fractures, which are most numerous near the margins of the mass.

INTRODUCTION

Probably not all landslides can be prevented, even after the most careful examination and interpretation of the geology of an area in advance of construction. When a landslide occurs, it must be defined rapidly and stabilization methods must be designed. Construction activity was at a critical stage in Loveland Basin when the landslide occurred. The Colorado Department of Highways' construction schedule called for completion of the tunnel plaza (portal) area by midsummer 1963 so that construction of the Straight Creek Tunnel pilot bore could be started by late summer—or before the start of winter. The landslide had to be defined and stabilized to ensure the safety of men and equipment at the plaza, to forestall any effects of the landslide on construction of the pilot bore, and to

prevent rejuvenation of movement of the landslide by construction of the pilot bore.

The geologic studies of the Loveland Basin landslide, including a written report, were completed in approximately 2½ months. Persons concerned with the studies came to realize that more definitive results could have been obtained in a shorter period of time had the personnel and the necessary equipment been available on a full-time basis.

Knowledge of the geology, particularly the land-forming processes, is necessary for the prevention or analysis of a landslide. Geologic studies revealed the framework of the Loveland Basin landslide. The bedrock of the area consists of Precambrian igneous and metasedimentary rock that locally has been extensively faulted and altered. During the Pleistocene the stream valleys at the altitude of Loveland Basin were glaciated and their sides were steepened. When the glaciers retreated, they deposited morainal material in the bottoms of the valleys and plastered it along the oversteepened valley sides. Since the retreat of the glaciers, the morainal material locally has slid part way or completely down-slope, and the oversteepened bedrock walls have been retreating by weathering processes. As a result, many slopes in the high mountain valleys are not stable under present conditions, and changes of these slopes caused by construction may trigger serious landslides. When the slope of the bedrock in Loveland Basin was oversteepened, the rock failed along preexisting faults and moved with its load of surficial material toward the cut. The Loveland Basin landslide is but one of several landslides in the high mountains that are the result of failure of highly sheared and altered bedrock loaded with surficial material. Another such slide, but a much larger one, occurred about 1 mile west of the west portal of the Straight Creek Tunnel pilot bore along the west approach road.

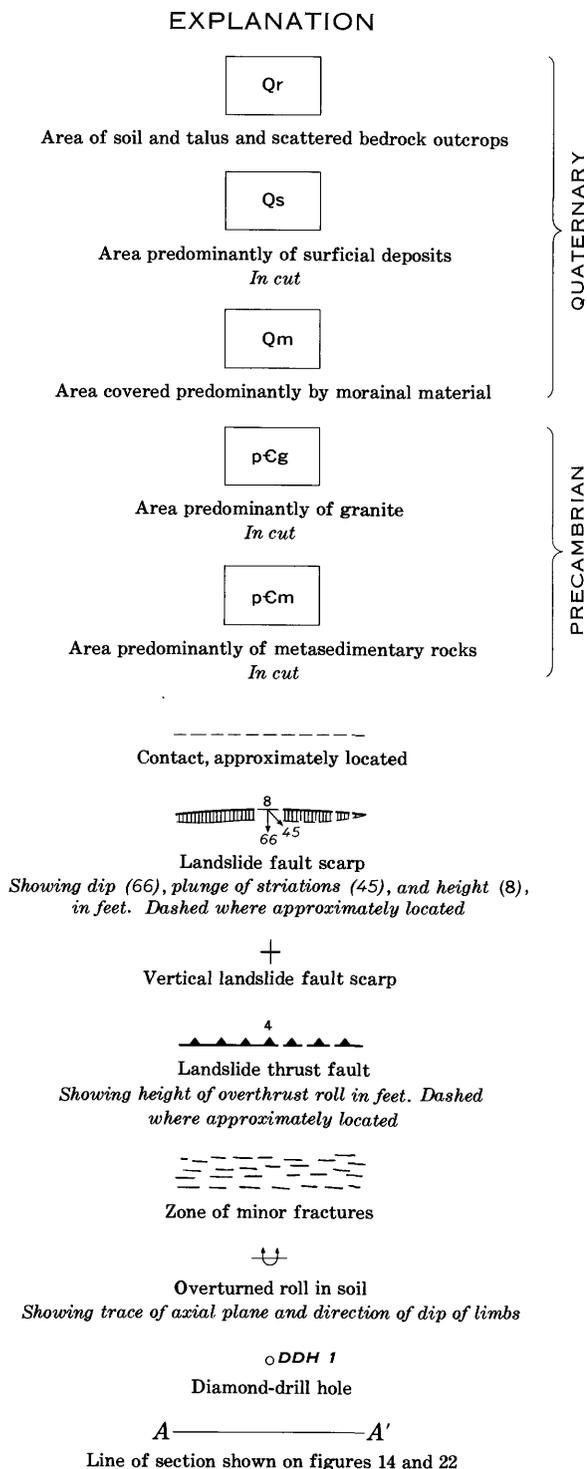
The extent of the Loveland Basin landslide was difficult to define. The slide area, particularly the upper part, was inaccessible to drilling equipment without the construction of a long access road. The authors believed that the surface of the slide should be disturbed as little as possible and that drilling should only be considered if alternate methods of delineation proved inconclusive. The construction of

roads across the slide would have required the removal of vegetation and forest debris, which would have increased the absorption of surface water and would have contributed to the instability of the landslide mass.

A more compelling reason for placing a low priority on a drilling program was the nature of the material involved in the landslide. Owing to the heterogeneous mixture of altered and sheared metasedimentary and granite bedrock and blocks of unaltered and unshaped metasedimentary rocks and granite, the material could not be expected to be cored and a slip plane identified. The alteration and gouge along preexisting faults and shears in the landslide mass and in the undisturbed bedrock below the landslide mass probably would have resembled the clay gouge commonly found along slip surfaces. The large blocks of relatively unaltered and unshaped rock in the landslide mass would have resembled that in the undisturbed bedrock. The authors concluded that a slip surface, or surfaces, could not be identified by drilling.

From these considerations, and because of the need for immediate information, participating agencies decided that the landslide could best be defined by careful surface geologic, geophysical, and engineering studies.

The principle considered in the use of geology and geophysics in defining a landslide is that landslides are natural phenomena controlled by certain natural properties. These properties can be identified, measured, and interpreted. By mapping the natural features—such as scarps, crevices, and rolls—that are associated with a landslide, the landslide can be defined in two dimensions. From a knowledge of the composition of a landslide mass, which can be determined from the regional geology, geophysical measurements can be interpreted to define the third dimension. A landslide mass has, in general, a greater porosity and permeability—and therefore lower density—than the surrounding material. This fact means that, depending upon hydrologic conditions, the landslide mass can be expected to exhibit an increased or decreased electrical conductivity and a decreased seismic velocity as compared to the surrounding stable area. The degree of accuracy of the geologic and geophysical interpretations can be checked by engineering calculations. For example, for a given landslide mass to be able to move at a known slope angle, it must have a cohesion and internal friction that can be estimated within certain mathematical limits. The driving force must be greater than the restraining force



and is limited by the observable dimensions of the landslide. Therefore, the interpretations of geophysical results are tempered by data or by estimates of geometric and mechanical properties of the landslide mass.

The geology of the Straight Creek Tunnel site, including the area where the Loveland Basin landslide developed, was mapped at 1:12,000 in 1962 by the authors as part of a research project on the pilot bore for the Straight Creek Tunnel. (See Robinson and Lee, 1962.) After the development of the landslide, the geology of the landslide area was mapped (fig. 2) by planetable methods at 1:1,200.

PHYSIOGRAPHY

The landslide occurred near the head of Clear Creek in the high mountains of the Front Range of Colorado. Clear Creek valley was occupied by glaciers during the Pleistocene and the valley walls were oversteepened by glacial erosion. Morainal and talus deposits are common on the valley walls, and talus is at the present time still accumulating along these slopes. The level of the principal glacial erosion in the valley extended to an altitude of about 11,500 feet. Above this level the slopes are more gentle and are covered by frost-heaved rock and the products of solifluction (fig. 3). The annual precipitation occurs principally as snow from January to April and yields about 25 inches of moisture. Thunderstorms, however, are common in the afternoons during the summer months. Timber, principally spruce, grows to an altitude of about 11,700 feet, but above this altitude vegetation is limited to mountain grasses and willows. The top of the landslide was approximately at timberline.

REGIONAL GEOLOGY

The bedrock of the landslide area consists principally of Silver Plume Granite that contains inclusions of metasedimentary rocks. The granite and the metasedimentary rocks are of Precambrian age. A few augite diorite dikes of probable Tertiary age occur about 1 mile west of the landslide.

The landslide is within the northeast-trending Loveland Pass fault zone. The fault zone, which is about 2 miles wide, consists of numerous faults and shear zones that are separated by relatively unsheared rock. Individual faults or shear zones range in width from less than 1 foot to 1,000 feet. The Loveland Pass fault zone is considered to be an area of Precambrian shearing and faulting that was re-

sheared during the Laramide orogeny in a manner similar to the reshearing of the faults and shear zones described by Tweto and Sims (1963).

BEDROCK

Before the excavation of the east portal area and the development of the landslide, outcrops of bedrock in the area of the landslide were few. Small knobs of granite and metasedimentary rock cropped out in the stream valley just west of what is now the central part of the landslide and in the low cliffs in what is now the central third of the northeast edge of the landslide. Construction of the cut near the toe of the landslide exposed granite in the upper part of the cut face (fig. 4) and to the east and south of the cut where an excavation had been made for the approach road. The center of the cut exposed altered metasedimentary rock. Geologic mapping in the vicinity of the tunnel site (Robinson and Lee, 1962) revealed that the bedrock in the tunnel area consisted of about 75 percent granite and about 25 percent metasedimentary rocks; the percentage of the two rock types probably is similar in the landslide area. The metasedimentary rocks occur as inclusions in the granite which range in maximum dimension from less than 1 foot to about 1,000 feet.

METASEDIMENTARY ROCKS

The metasedimentary rocks in the landslide area consist of several types of fine-grained biotite gneiss which are typically interbanded with granitic material. The most common varieties of the gneiss are biotite-quartz-microcline gneiss and biotite-quartz-plagioclase gneiss. The rocks have a distinct foliation as a result of the concentration and orientation of the minerals in layers. The layers range in thickness from about one-tenth of an inch to several inches. Layers of granitic material—principally quartz and microcline—of irregular thickness occur in the metasedimentary rocks. These layers commonly constitute 25 percent of the rock.

The metasedimentary rocks are commonly altered—more so than the granitic material. Most of the minerals, with the exception of quartz, in these rocks are altered to clay over a distance of less than 1 inch to several inches on each side of joints and faults. Where large bodies of metasedimentary rock are within a shear zone, the rock is commonly altered to masses of chloritic clay in which the foliation, though contorted, can still be recognized. The interlayered granitic material is generally less altered and forms layers of sheared, but relatively competent, rock within the clay mass.



FIGURE 3.—Unfinished cut near east portal of the Straight Creek Tunnel prior to development of landslide. Depth of cut is about 150 feet.

GRANITE

Petrographic analyses of samples of the granite collected during the investigation of the Straight Creek Tunnel site show that the granite typical for the area consists of about equal percentages of microcline, plagioclase feldspar (sodic oligoclase), and quartz and 1–15 percent biotite. Muscovite is a common accessory mineral. Other accessory minerals are apatite, zircon, magnetite, and pyrite. The size of mineral grains ranges from less than 1 mm to 10 mm; the average grain size is about 3 mm.

The microcline grains in the granite are commonly subparallel; this orientation gives the rock an indistinct foliation. Most of the biotite is not oriented; however, in biotite-rich varieties of the gran-

ite, the orientation is parallel to the microcline grains. The foliation of the granite is believed to be the result of flow at the time of the rock's emplacement. Microshearing of the grains locally imparts an indistinct cataclastic foliation. In the granite outcrops along a stream west of the landslide, feldspar grains have been cut and offset by many microfractures, and biotite grains are aligned parallel with the microfractures. These features give the rock a cataclastic foliation that is generally not parallel to the flow foliation.

The mineral grains of the granite are commonly altered adjacent to faults and joints, but their alteration is not as conspicuous in the landslide area as in other areas near the Continental Divide to the



FIGURE 4.—Cut near east portal of Straight Creek Tunnel after start of landslide.

west. The ferromagnesian minerals and the feldspars are commonly partially altered to clay minerals out from faults and joints to distances ranging from less than an inch to several inches. In this area there is no evidence of sulfide mineral deposition; therefore the authors believe that the alteration is primarily the result of the action of ground water rather than of hydrothermal solution.

SURFICIAL DEPOSITS

The landslide surface is mantled by a thin layer of morainal material, soil, and talus (fig. 2). Morainal materials occur along the west side, across the southeast end, and to the east of the landslide. The geophysical investigations indicated that the morainal material along the west side of the landslide was deposited, in part, in a preglacial valley. The morainal material ranges in thickness from less than 1 foot to about 15 feet and consists of an un-

sorted mixture of clay, sand, gravel, and boulders. In general, it is well compacted.

The higher parts of the landslide to the east are mantled by less than 1 foot to about 3 feet of soil mixed with talus. The soil is sandy and silty and, in the forested areas, is masked by an inch or so of decayed organic material and evergreen needles. Mixed with this soil in varying amounts are talus blocks ranging in largest dimension from less than 1 inch to about 3 feet. Talus forms a small cone below the steep slope on the northeast margin of the landslide between altitudes of 11,500 and 11,575 feet.

STRUCTURE

The extent of the Loveland Basin landslide is principally controlled by preexisting faults that form a wide shear zone. The shear zone in the area of the landslide was previously mapped by Robinson



FIGURE 5.—Landslide fault scarp at the northwest border of the landslide. Note striations.

and Lee (1962) as about 600 feet in width and about 4,000 feet in length. It trends about N. 15° E. and is terminated on the west by the fault that forms the western boundary of the landslide. This fault can be traced for more than half a mile and extends northwest of the northern end of the landslide as it existed in 1965 (fig. 2).

The northeastern boundary of the landslide was determined by two preexisting en echelon faults. The northern fault was not recognized before the development of the landslide. The scarp of the southern fault was exposed in the vicinity of the curve in the fault near the northeast boundary of the landslide (fig. 2). Before 1963, this scarp was about 6 feet high and the fault surface showed silicified slickensides with striations bearing S. 25° E.

and plunging 25°, indicating that the south side was downdropped relative to the north side.

The landslide was outlined by landslide fault scarps and thrusts as of 1965. The scarps occur where the landslide mass has been displaced downward. The thrusts occur where the landslide mass has moved up and over the preexisting land surface.

The landslide fault scarps are on the western and northeastern sides of the landslide. At the time of geologic mapping, the heights of the scarps were measured and are shown in figure 2 by the number adjacent to the scarps. The heights ranged from less than 1 foot to 11 feet. The dip of the scarps and the bearing and plunge of striations on the scarp surface were also measured and are shown in figure 2. For several years after the geologic map-

ping was completed in 1963, the heights of the scarps increased, and weathering removed the striations. The striations on the scarp surface near the northwest corner of the landslide (fig. 5) were well formed.

The landslide thrust faults occur principally along the southeast margin of the slide and in the interior of the slide near the southwest margin. The heights of the thrusts range from less than 1 foot to about 6 feet (fig. 2). Rolls in the soil with amplitudes of about 1 foot exist in the landslide mass adjacent to the thrusts. During the initial period of landslide movement the thrusts were connected to the scarps by open fractures that had little or no vertical displacement. These thrusts were the most spectacular discernible feature of the landslide during the early days of the investigation. At their margins flowers and small trees could be seen in the process of being overturned and covered by the mass of moving soil and rock of the landslide. A photograph of the thrust sheet near the southeast



FIGURE 6.—Landslide thrust fault at southeast edge of landslide. Note destruction of plant life as landslide advanced.

corner of the slide (fig. 6) shows a small tree being pushed over by the landslide mass.

During the early period of landslide movement (1963), the ground surface in the central part of the landslide showed little evidence that the mass was moving. However, near the west margin and upslope (northwest) from the main thrust, there developed some relatively small scarps and thrusts and some zones of small fractures that locally cut across the landslide mass. Along the southwest margin of the landslide a zone was formed that is bounded by a generally parallel line of scarps, open fractures, and a thrust. In this zone the landslide mass was considerably disturbed, and probably the greatest amount of movement has taken place there. Upslope from the southeastern marginal thrust is another zone of scarps and open fractures that are probably the result of tension caused by the development of the thrust. Associated with the other marginal scarps and thrusts, and locally in the middle of the landslide mass, are zones of minor fractures. The fractures in these zones range in length from a few inches to several feet and in width from less than 1 inch to about 3 inches. Few of these minor fractures show any relative vertical displacement.

The magnitude, orientation, and location of the scarps, thrusts, and fractures indicate that the landslide did not move as a single homogeneous mass, but rather as several units, each somewhat dependent upon the others. The principal movement was parallel to the long axis of the landslide, but the southwest side of the slide moved faster, or more, than the east side. The lower part of the east side probably moved more than the main mass, as indicated by the fractures to the northwest of the thrust. After completion of the geologic mapping, survey points were established by J. D. Post, of the Colorado Department of Highways, and records were maintained as to the relative amount of movement of different parts of the landslide. This work is discussed in chapter D.

Electrical Resistivity Investigations

By R. WOODWARD MOORE

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
1963-65

GEOLOGICAL SURVEY PROFESSIONAL PAPER 673-B



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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND
BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

ELECTRICAL RESISTIVITY INVESTIGATIONS

By R. WOODWARD MOORE²

ABSTRACT

A rapid subsurface survey, involving 26 earth-resistivity depth determinations, was made primarily on two longitudinal traverses along the Loveland Basin landslide mass. These measurements, involving two field parties and a 3-day period in July 1963, yielded preliminary information on the probable location of the active slip surface, or zone of slip surfaces. From this information a tentative estimate was obtained of the volume of unstable material; approximately 500,000 cubic yards of material was indicated either to be moving or to be potentially unstable. The data from this investigation, together with information from 12 subsequent resistivity depth determinations on a third longitudinal traverse, were used in the subsequent comprehensive geological, geophysical, and engineering landslide analysis.

INTRODUCTION

Landslides have plagued construction work wherever man has disturbed nature's balance by making excavations or by destroying stabilizing conditions. Determining the zone of slippage in an active landslide by means of borings and test pits can be time consuming and costly. Moving heavy equipment to drill sites for testing constitutes a difficult, if not insurmountable, problem in some areas, particularly when time is short. Use of light weight portable geophysical equipment, therefore, is generally advantageous in making a rapid reconnaissance of subsurface conditions. Use of the earth-resistivity method for subsurface exploration seems to have merit for such preliminary surveys of landslide conditions.

In demonstrating the usefulness of the resistivity technique the Bureau of Public Roads has made tests of 44 landslide areas in 19 States (Moore, 1954, 1957, 1958, 1961a, b; Trantina, 1963). For most areas only a single section taken through the approximate center of the moving mass was studied. However, at one location in Ohio in 1951, and

at two locations in Tennessee in 1961, more complete surveys of relatively large landslide areas were made. The successful results obtained in these investigations by the resistivity method prompted the use of that method at the Loveland Basin landslide.

Two resistivity parties were made up of Colorado Department of Highways and Bureau of Public Roads personnel. During 3 days, beginning on July 17, 1963, the two groups made resistivity-depth tests at 26 locations along two sections (A and B in fig. 7) spaced about 200 feet apart and passing through the slide area. Subsequently, an additional 12 measurements conducted by Bureau of Public Roads personnel were made along a third section (C in fig. 7).

THEORY AND INTERPRETIVE METHODS

The earth-resistivity measurement is made by passing electrical current through the ground and measuring the resistance of earth materials to current flow. This current flow is largely electrolytic and is dependent on the moisture and dissolved salts within soils and rocks for direction of its path (Tagg, 1964, p. 5).

A Wenner configuration, consisting of four equally spaced electrodes with the center of the system set at a fixed point (fig. 8), was used in this investigation. The outer two electrodes, c_1 and c_2 , pass current into the ground, and the difference in potential due to this current is measured between the inner two electrodes p_1 and p_2 (Howell, 1959, p. 371).

As the spacing of the electrodes increases, greater amounts of underlying material are influenced. The average measured (apparent) resistivity will trend toward the resistivity of the underlying layer. The resulting average resistivities are plotted

² Formerly highway research engineer, U.S. Bureau of Public Roads.

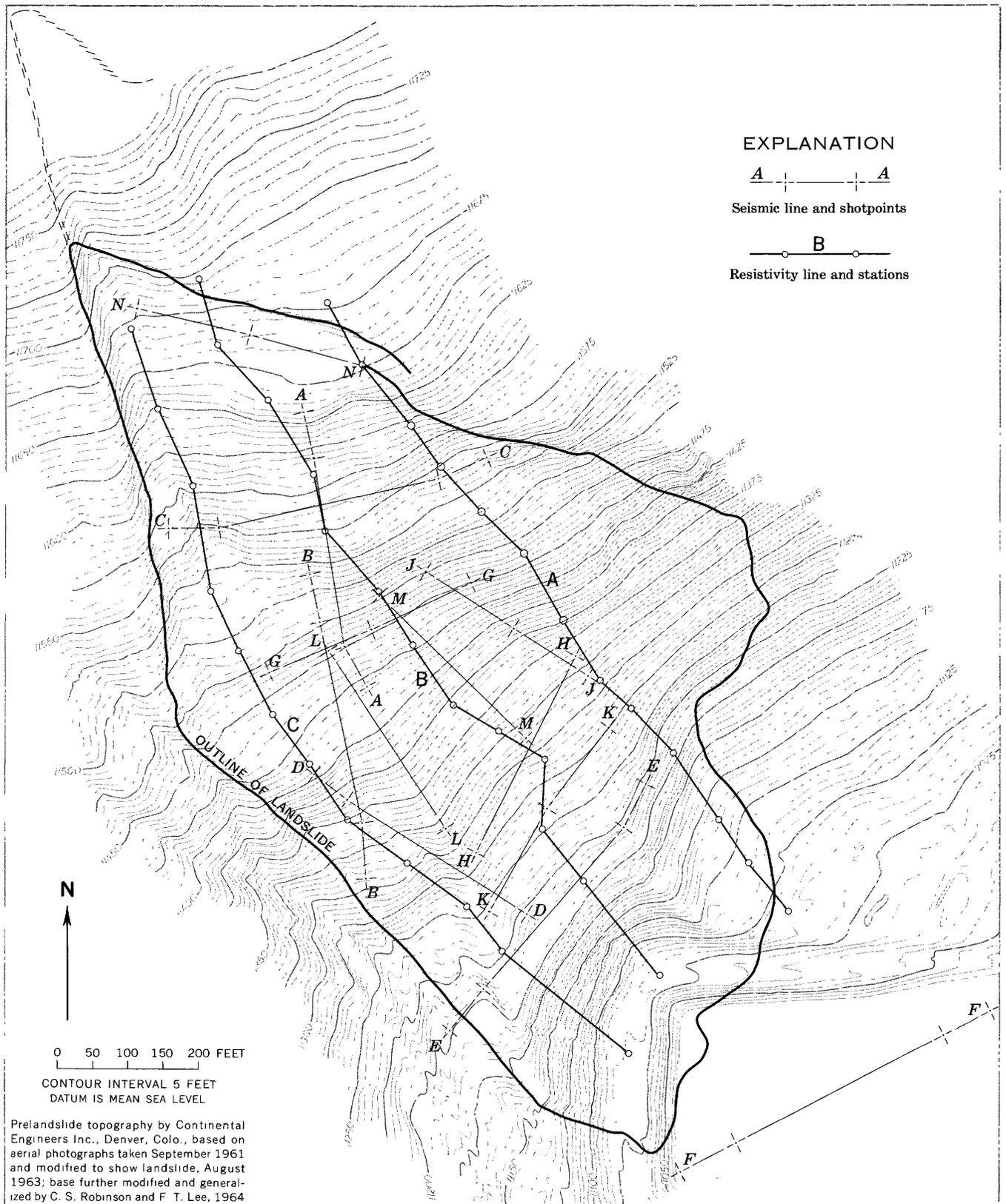


FIGURE 7.—The Loveland Basin landslide showing location of geophysical traverses.

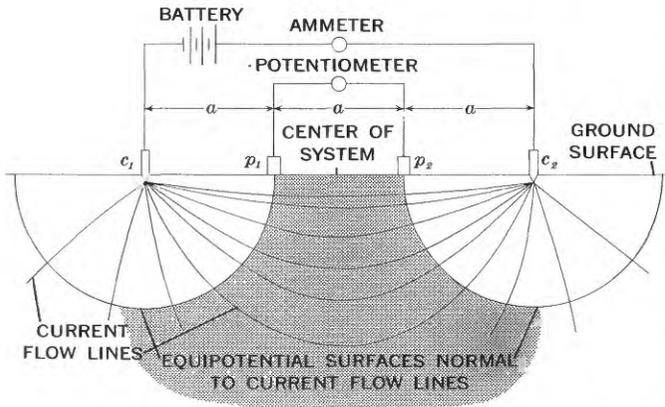


FIGURE 8.—Principle of earth-resistivity measurement using the Wenner configuration c_1 , c_2 , current electrodes; p_1 , p_2 , potential electrodes; a , electrode separation.

against the electrode separations, resulting in a curve. Figure 9 shows a typical resistivity curve which was obtained by expansion of the electrode system to include the depths shown.

From the earliest days of surveying by resistivity technique, the interpretation of the curves has been a matter of great concern and, indeed, of controversy. Two schools of thought exist regarding the interpretation of resistivity curves. The first of these schools is primarily empirical and is based partly on experience. The second is primarily based on theory and employs several methods which make use of theoretically derived depth curves.

Whatever the interpretive method employed, pertinent geological and additional pertinent geophysical data are necessary to explain changes in resistivity with depth. When calibration tests can be

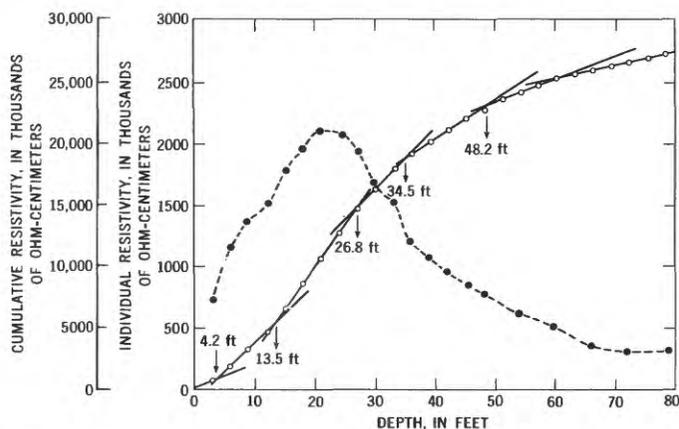


FIGURE 9.—Resistivity-depth curve (dashed line) for talus overlying weathered bedrock and cumulative resistivity curve (solid line) used to empirically determine depths to subsurface layers. Depth, in feet, of effective current penetration is assumed to equal the surface electrode spacing.

made over known subsurface conditions, resistivity changes can be conclusive. A major drawback with the theoretical analyses is that they assume ideal conditions; that is, uniform isotropic layers with interfaces parallel to the ground surface. The advantage in empirical interpretations is that they endeavor to treat the real, and always anisotropic, situation. The author attempted to use theoretical depth curves to analyze the data obtained from previous near-surface studies performed by the Bureau of Public Roads. The results were generally discouraging owing to the frequency with which the field conditions failed to be adequately described by the results obtained by use of the theoretical curves.

The author favors the empirical interpretation of depth curves on the basis of over 30 years' experience which yielded largely successful results; this method, however, also has shortcomings. Tagg (1964, p. 21) showed that for an ideally isotropic material the current penetration varies with depth but is always greater than the electrode spacing. However, for anisotropic heterogeneous material there is closer correspondence between depth of current penetration and distance of electrode spacing. As Barnes (1952, p. 31) pointed out, current flow follows an irregular path and is governed by the anisotropies of natural materials. Such irregular paths could shorten the effective depth penetration of current flow in anisotropic material. Thus the electrode spacing could nearly equal, if not equal, the depth of current penetration. Studies conducted to correlate electrode spacing with depth of current penetration indicated that, for depths above 100 feet, depth of current penetration is nearly equivalent to the distance between adjoining electrodes. Malott (1965) found that the results of 34,000 shallow resistivity soundings and of more than 4,000 correlation borings indicated that the electrode spacing is equal to the depth of investigation (current penetration).

In an attempt to show more clearly the geologic boundaries on depth curves the author (Moore, 1945) introduced the cumulative resistivity method. In the present report both the individual value curves and the cumulative curves are shown. The cumulative method has been most successful in areas containing a sand and gravel deposit having a shallow water table and overlying clay.

TEST RESULTS

Resistivity measurement is a logical technique for use in studies of landslides in which excess water is

likely to be associated with the zone of sliding. When porous talus material and sheared bedrock slide out over less pervious material such as undisturbed sheared bedrock, one would expect evidence in the plotted resistivity curve of the effect of the trapped excess water at the base of the disturbed material. The change in slope shown in the cumulative resistivity curve of figure 9 at a depth of about 35 feet is an example of the type of effect to be expected when the less pervious deeper material is less resistant to current flow than the overlying slide material (Moore, 1957). Although no confirming borings had been made in this slide area as of 1965, the 35-foot depth shown in figure 9 correlated well with the depths at which slope changes were detected in eight other depth tests made along a section through the center of the moving mass. In the tests to be discussed, such slope changes were taken as evidence of the probable slip zone.

Figure 10 shows three resistivity depth curves considered to be rather typical for the 38 depth tests that were made over the slide area. Slope changes are present in each curve at depths believed to coincide with the elevation at the top of the slip zone. Slope changes similar to those appearing in the three curves of figure 10 at depths of about 27, 25, and 16 feet were observed in most of the resistivity-depth curves plotted for the 38 tests.

The cross section shown in figure 11 is typical of the three sections plotted from the data obtained in the several tests, showing the relation of the existing ground surface to the inferred position of the slip surface. The three lines of tests from which the three sections were prepared were spaced at inter-

vals of 200–300 feet across the slide area from east to west.

Figure 11 shows all the significant subsurface resistivity changes along line A. The longer the dashed lines, the more laterally persistent were the resistivity layers. The most significant and the most persistent resistivity layers are shown by the heavier dashed lines. These heavier lines also indicate the location of the probable slip surface. The author believes that the subsurface conditions at the depths indicated by these lines are the weak zones that are the most continuous and therefore the most susceptible to movement. Near the toe of the slide two such zones or surfaces were defined from the resistivity measurements. Therefore, the author believes that probably two separate slip surfaces were present, one above the other, in this part of the slide.

These data were made available to the Colorado Department of Highways and the U.S. Geological Survey and were used by Robinson, Carroll, and Lee (1964) to prepare the maps of the landslide and to calculate the volume and mass of the landslide.

SUMMARY

The resistivity method has considerable merit for use in landslide studies. Its use in this study, as well as in many other tests made over other landslide areas, has served worthwhile purposes in providing, rapidly and economically, an approximation of the total quantities of material undergoing movement and in suggesting depths to which corrective measures must be made in attempts to control fur-

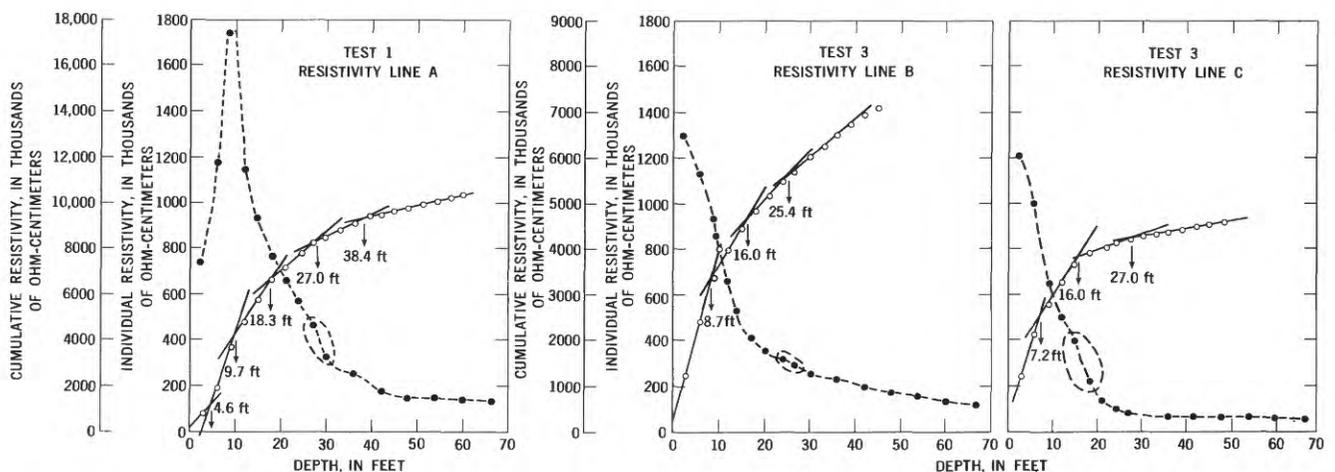


FIGURE 10.—Resistivity-depth curves (dashed lines) and cumulative resistivity curves (solid lines) typical of data obtained on Loveland Basin landslide area. Circled zone is believed to indicate effect of water just above slip surface. Depth, in feet, of effective current penetration is assumed to equal the surface electrode spacing.

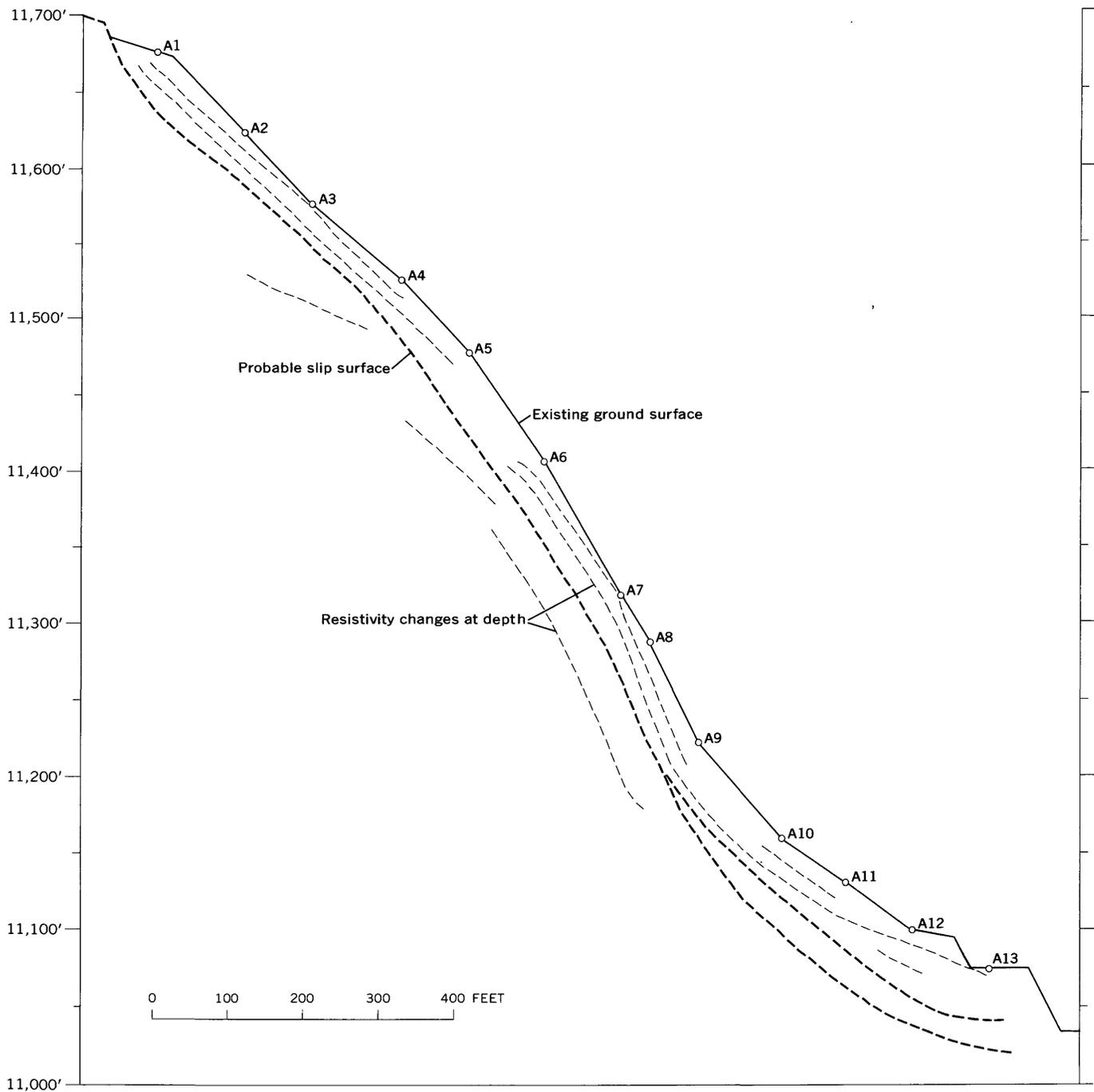


FIGURE 11.—Section through east part of sliding mass (resistivity line A) Loveland Basin landslide. A1–A13, resistivity-depth test locations. Line of section shown in figure 7. (Vertical exaggeration, $\times 2$.)

ther sliding. When used in conjunction with other geophysical techniques, such as the seismic refraction method, and with available geologic informa-

tion, the resistivity method helps to provide more complete subsurface data and to select places where boring results should prove most profitable.

Seismic Refraction Studies

By RODERICK D. CARROLL, JAMES H. SCOTT, *and* FITZHUGH T. LEE

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
1963-65

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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND
BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

SEISMIC REFRACTION STUDIES

By RODERICK D. CARROLL, JAMES H. SCOTT, and FITZHUGH T. LEE

ABSTRACT

Seismic refraction measurements were made in July 1963 along 13 lines in the area of the Loveland Basin landslide. Velocity layering detected on the landslide consists of a near-surface layer exhibiting compressional velocities from 1,000 to 2,700 fps (feet per second) and an overlying 3,000 to 6,500-fps layer. Thickness of the low-velocity layer ranged from 8 to 66 feet, tending to be greater in the central and western parts of the slide. The low-velocity layer probably represents the material above the top of the slip zone, the volume of which was 440,000 cubic yards. Severe energy attenuation and complex arrival times on several lines rendered the seismic interpretation uncertain at several locations when used without other criteria to aid in the depth determination.

INTRODUCTION

The U.S. Geological Survey undertook a seismic-refraction investigation of the Loveland Basin landslide in July 1963 to delineate the depth of unstable material. The seismic refraction method of investigation provided a third independent method of obtaining an estimate of the geometry and volume of the material involved in the landslide.

Published data on the delineation of landslides by seismic methods are meager. Trantina (1963) reported on two landslides in California that had depths to the slip plane of approximately 12 and 23 feet; these slides were successfully delineated by the use of a sledge hammer as an energy source and by means of seismic-refraction techniques. In these reported landslides the slip surface represented a well-defined geologic and seismic boundary between two different materials; this boundary enabled excellent resolution of arrival times. However, geologic conditions in the Loveland Basin landslide area differ in that the slip surface or slip surfaces are associated with very incompetent zones within an inhomogeneous and moderately incompetent

mass of material. Consequently, the delineation of the slip zone by the detection of seismic contrasts in the media can be expected to be strongly dependent on localized geologic conditions.

METHOD

Because of the high altitude, steep slopes, and inaccessibility to motor transport, portability was a major consideration in the selection of seismic equipment; consequently, a portable seismic instrument (Portaseis ER-75) was used. Seismic signals were recorded on Polaroid film. The entire recording apparatus was contained within a 24-pound package. Standard seismic cables and detectors were used and 12 geophones were spaced at 25-foot intervals. Measurements were made along 13 seismic lines in the landslide area (fig. 7). Shot-points were located at distances of 25 and 50 or 25 and 100 feet from the ends of the lines. On several lines a shotpoint was located at the midpoint of the line to obtain more information on variations in the overburden velocity.

Holes for the emplacement of the seismic charges were driven with a punch bar; and charges, averaging five sticks of dynamite (40-percent gel) per hole, were placed at depths of 1.5-3 feet. Energy input to the ground ranged from good to poor, depending on the nature of the near-surface material. A typical record obtained on line *N* is shown in figure 12. The aerated and disturbed condition of the overburden material above the slip zone yielded poor first arrivals on many lines. Several lines crossed fracture zones visibly open at the surface to depths of a few inches to several feet. The poor energy-transmission characteristics of the overburden would have made sledge hammer refraction methods unsuccessful.

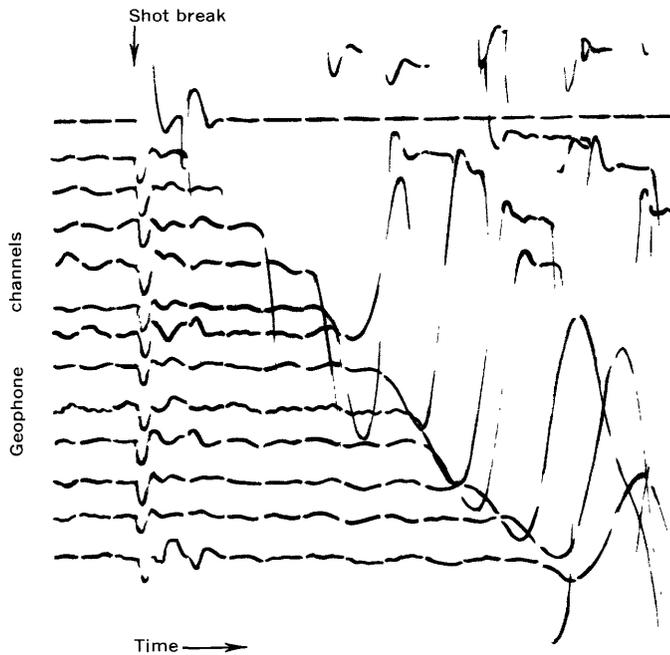


FIGURE 12.—A part of seismic record obtained on seismic line *N*. Charge was five sticks of dynamite at 3-foot depth. Interrupted segments of traces represent time intervals of 10 milliseconds.

Interpretation of the results obtained on several lines was hampered both by severe attenuation of high-frequency energy and by weak energy in the first arrivals. Interpretation was further complicated by the occasional presence of an irregular refracting surface as well as abrupt velocity variations within the near-surface layer. These two features were sometimes inseparable and caused considerable scatter of points on several travelttime plots. Two examples, typical of the extremes of difference in the travelttime plots, are shown in figure 13.

Travelttime curves for line *K* shown in figure 13 are such that a unique interpretation may be made of the thickness of the low-velocity layer in this area. The travelttime curves for line *D*, on the other hand, exhibit the effects of complex subsurface conditions. On several other lines, especially those parallel with the long axis of the landslide, travelttime curves indicated complex seismic ray paths. One possible explanation for these complex paths is the presence of relatively competent zones of high velocity within the low-velocity layer. These competent zones may possibly act as isolated buttresses within the sliding mass, inhibiting the development and movement of the slide.

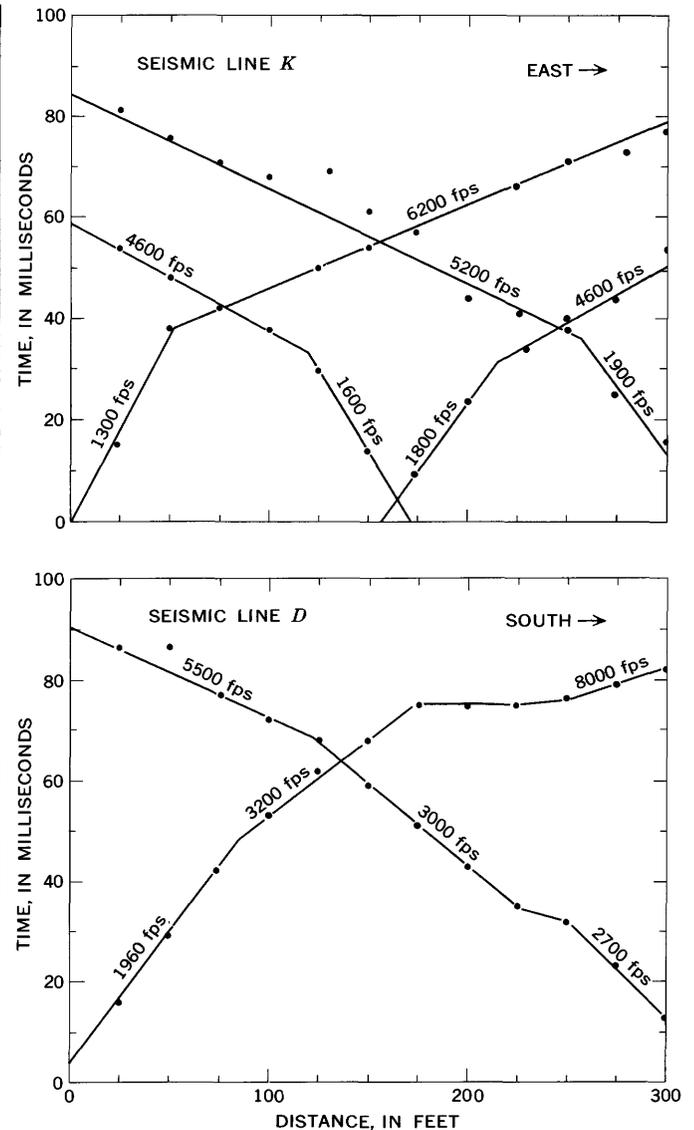


FIGURE 13.—Time-distance graphs obtained on seismic lines *K* and *D*. Location of lines shown in figure 7.

RESULTS AND CONCLUSIONS

The results of the seismic refraction survey indicate that, in general, two velocity layers exist within the landslide—an upper layer of soil and disturbed incompetent bedrock in which the velocity ranges from 1,000 to 2,700 fps (feet per second) overlying a layer which has a velocity ranging from 2,000 to 6,500 fps and which probably represents less disturbed but incompetent bedrock. An exception to this generalization occurred on line *F* in the plaza area. Velocities in the overlying material along this line ranged from 5,500 to 6,700 fps to a depth of about 30 feet. The material composing

the underlying high-velocity layer was characterized by velocities in the range of 14,000 to 15,000 fps; these suggest the presence of firm, granitic bedrock. Subsequent core drill data furnished by the Colorado Department of Highways substantiated this interpretation.

Thickness of the low-velocity layer of the slide ranges from 8 to 66 feet (fig. 14). This layer is thin in the eastern and southern parts of the slide and thick in the western and central parts. Interpretation of the seismic data in the vicinity of the old preglacial stream channel on the western margin of the slide was hampered by velocity variations in the subsurface bedrock and in the surficial material. The data suggest, however, that the low-velocity layer was thickest near the western edge of the slide.

The thickness of the low-velocity layer (fig. 14) is considered to represent the probable minimum amount of material involved in landslide movement.

The volume of this material was calculated to be 440,000 cubic yards. Data from other investigative methods used in the study of the landslide suggest that a greater volume of material may be involved than is indicated by the seismic data. At many locations the seismic data were in general agreement with estimates made by other means.

The results of this investigation indicate that the seismic refraction method is helpful in delineating landslides of a complex nature similar to the one discussed in this report. At some locations, however, the reliability of the interpretation of seismic data is doubtful without independent criteria for checking the thickness of the sliding mass. By using several independent criteria for estimating the geometry of the sliding mass—in this instance, seismic and electrical resistivity soundings coupled with detailed geologic mapping—a greater measure of confidence can be achieved in making calculations.

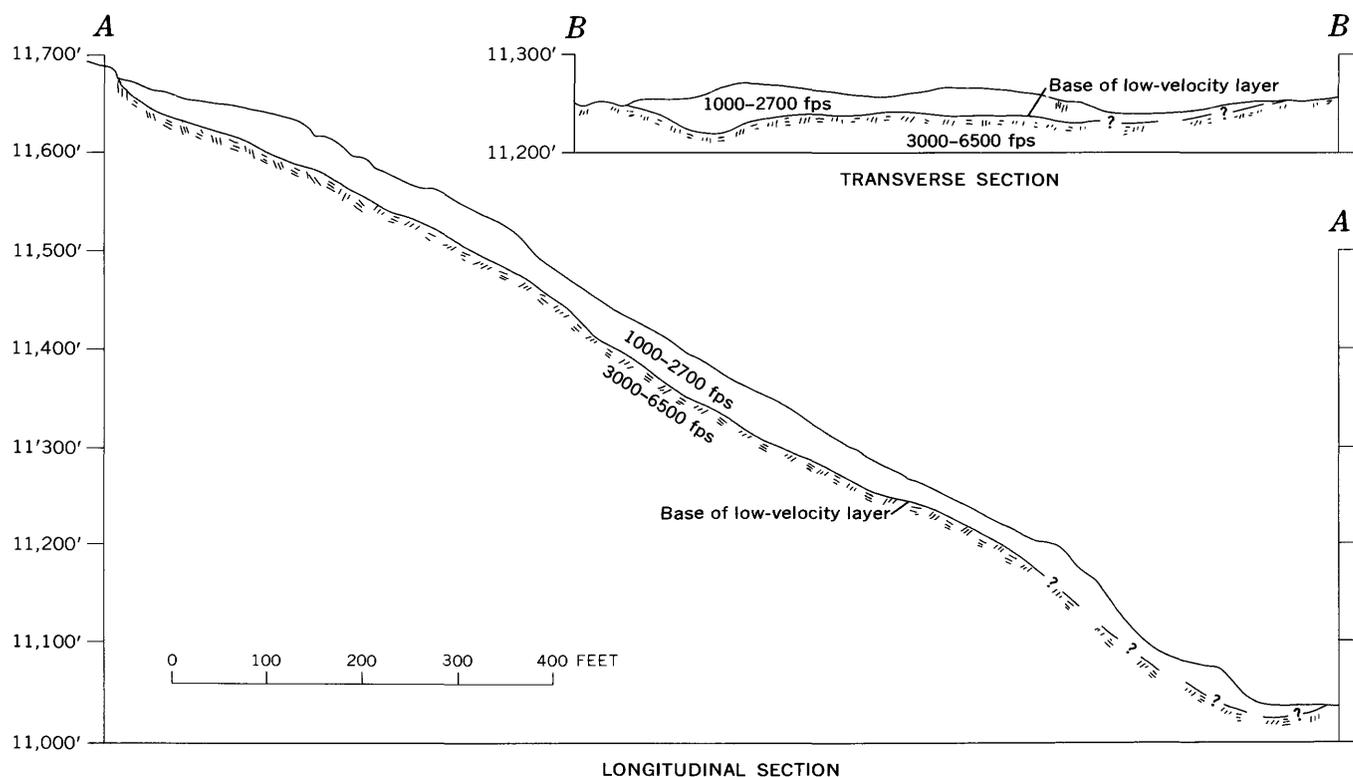


FIGURE 14.—Longitudinal and transverse sections of the Loveland Basin landslide showing thickness of low-velocity layer. Location of sections shown in figure 2.

Engineering Investigations

By JOHN D. POST *and* CHARLES S. ROBINSON

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
1963-65

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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND
BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

ENGINEERING INVESTIGATIONS

By JOHN D. POST⁴ and CHARLES S. ROBINSON

ABSTRACT

After recognition of the Loveland Basin landslide on June 28, 1963, the Colorado Department of Highways established control points around and on the landslide mass in order to observe the amount and rate of movement. Measurements between pairs of these points established that different parts of the landslide were moving at different rates and that the chief movement was during the summer months. To define the toe of the landslide and to measure its movement, a survey grid was established in the plaza area to the south of the landslide. The records of measurements of the grid showed that the toe of the landslide was restricted to the northwest corner of the plaza area.

Two core holes were drilled and logged south of the landslide to determine if a slip plane existed below the plaza area. Plastic pipe was packed into these holes and the alignment of the pipe was checked periodically.

To drain surface and ground water from the landslide, five near-horizontal holes were drilled in the west margin of the slide. The water from two springs was collected and piped off the slide, and an 18-inch pipe was placed in an area of unusually heavy snow accumulation to conduct the melt water from the landslide.

INTRODUCTION

The Colorado Department of Highways, upon recognition of the landslide and after conferences with the U.S. Bureau of Public Roads and the U.S. Geological Survey, established surveyed reference points on and around the Loveland Basin landslide to determine the amount and direction of movement of the landslide. Subsequently, two holes (fig. 2) were cored in order to detect any landslide movement near the site of the portal of the Straight Creek Tunnel pilot bore. In addition, efforts were made to drain the accessible parts of the landslide mass. Many persons of the Colorado Department of Highways participated in this work. In general, however, the department's work was under the direction of the senior author, who made periodic

records of the movement of the landslide through 1966.

SURVEYS

Early in July 1963, a transit and chain survey was run around the periphery of the slide. After this survey was run, 15 pairs of transit and tape measurement points were established at or within the slide margin, and 32 other points, to be observed periodically by transit, were established across the slide mass. The locations of some of the pairs of transit and tape measurement points across the breaks in the slide are shown in figure 15. Figure 16 shows records of the relative change in distance between the pairs of points between July 1963 and January 1965.

The measurement points were established after the major movement of the landslide had occurred; consequently, the total vertical change in distance between a pair of measurement points can only be approximated by adding the vertical change as measured since July 1, 1963, to the height of the scarps as shown in figure 2. A systematic schedule of observations of the measurement points could not be made throughout the year because of the deep winter snows. As a result of the movement and breakup of the landslide mass, some points were destroyed and had to be reestablished (new points, fig. 16). The records show, however, that both the horizontal and the vertical movements of the slide mass were greatest between July and November of 1963. Movement continued, principally during the summer months, in 1964 and 1965. A buttress load was placed on the toe of the slide in September and October 1963. Much of the slide mass was frozen during the winter and spring months (1963-65), and little movement took place. Noticeable movement occurred on the east and west sides of the slide in June and July 1964, and then decreased as the

⁴ Colorado Department of Highways.

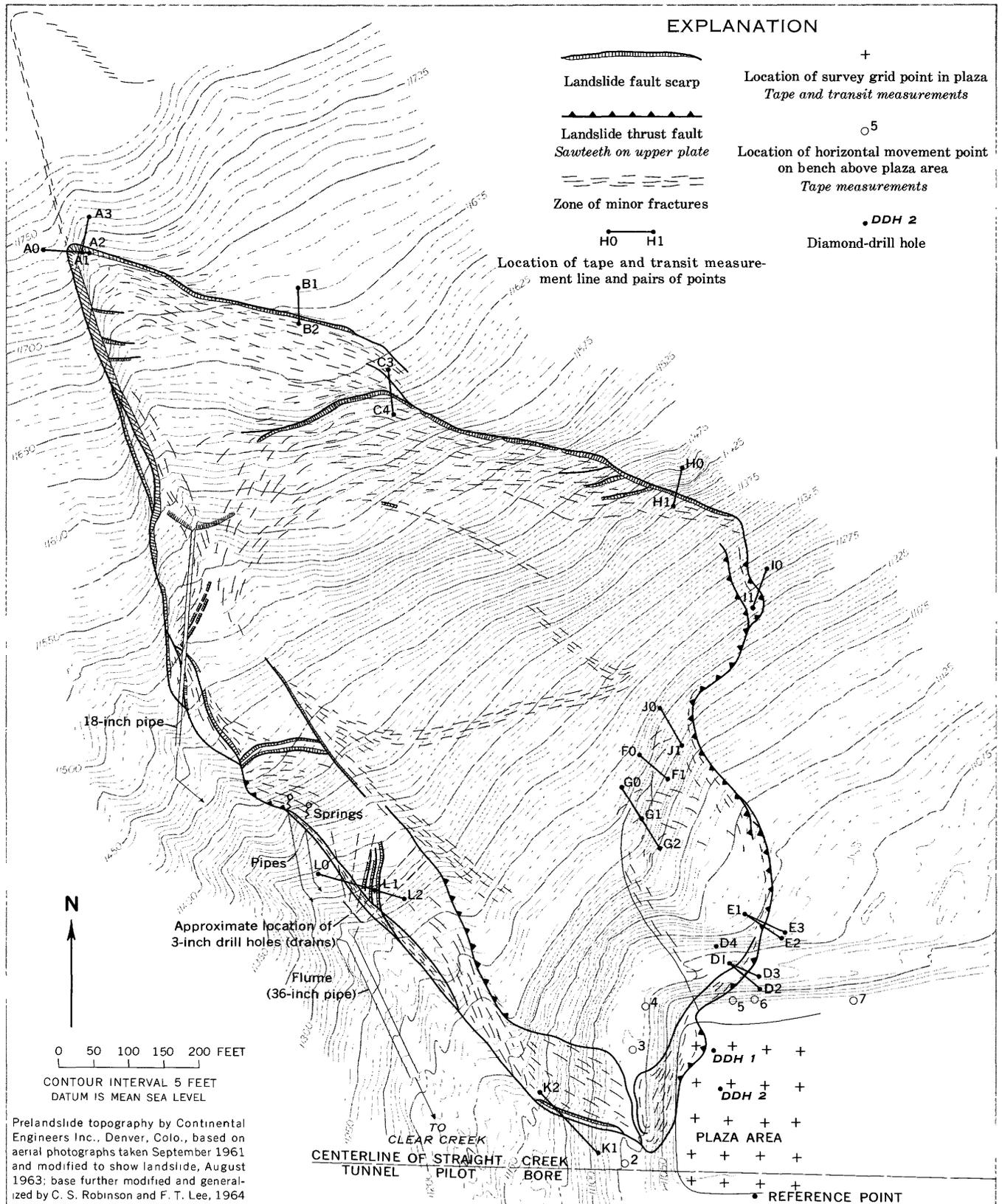


FIGURE 15.—The Loveland Basin landslide; the location of the measurement points at the margins of the landslide and the approximate locations of the surface and subsurface drainage pipes are shown. Prepared by J. E. Gay, Colorado Department of Highways.

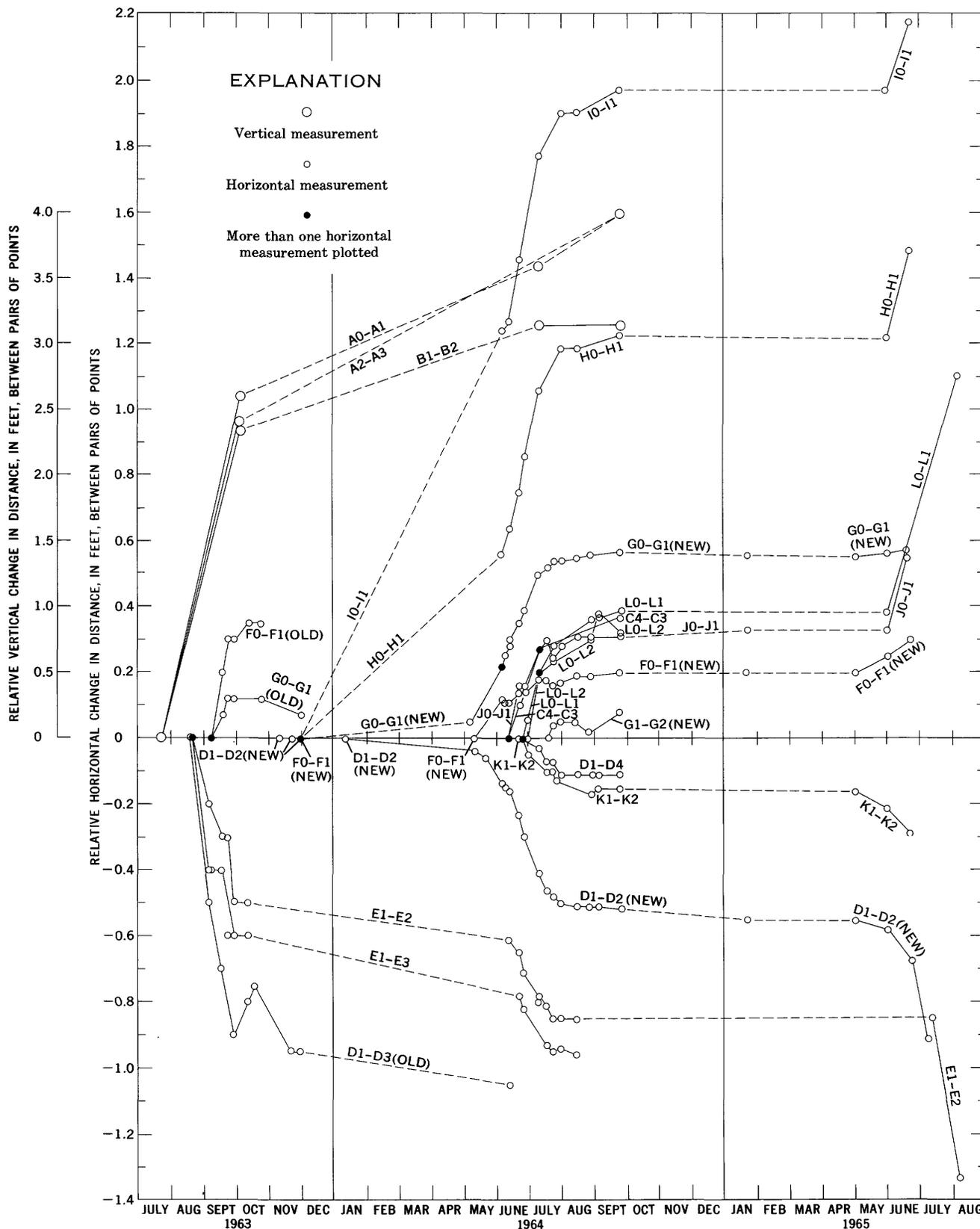


FIGURE 16.—The relative vertical and horizontal changes in distance between pairs of points outside and within the landslide mass. Locations of surveyed lines are shown in figure 15. Dashed lines indicate inferred change.

water table lowered. The central part of the slide did not move as much as the margins—which were deformed more during the original movement.

On July 11, 1963, six points were established on the first cut bench above the plaza area. The horizontal distances from these points to a reference point south of the landslide area were established with a 500-foot chain at a tension of 60 pounds. The approximate locations of these points are shown in figure 15. Measurements between these points and the reference point were made between July 11 and September 3, 1963. The total horizontal movements of these points during this period are shown in the following table.

*Horizontal movement of points on bench above plaza area,
July 11 to September 3, 1963
[500-ft chain under 60-lb tension]*

Point	Total horizontal movement (feet)
2	0.02
3	2.97
4	2.65
5	.05
6	.03
7	.02

These measurements show that the horizontal movement of the cut bench at the front of the landslide during this period was chiefly restricted to the northwest side (points 3 and 4) and that the maximum movement averaged about 0.06 foot per day.

At the start of the landslide investigation the location of the toe of the landslide in the plaza area was considerably in doubt. This uncertainty was partly because the contractor was continuing to lower the grade in the plaza area. A survey grid was established in the northwest corner of the plaza to determine the limit of the movement of the landslide so that the portal of the Straight Creek pilot bore could be relocated in stable ground. Figure 17A shows the location and original dimensions of the survey grid, which is shown in figure 15. The points of the grid were ×'s on steel caps threaded on 1 $\frac{7}{8}$ -inch steel pipes grouted into holes drilled into the bedrock of the plaza floor. The grid system was completed on August 12, 1963, and daily horizontal and vertical measurements (referenced to a point south of plaza, fig. 15) were made from August 12 to August 23, 1963. Figure 17 shows the results of the initial and final horizontal and vertical measurements of the grid system. It should be noted

that point M1, in the northwest corner of the plaza, moved upward 0.23 foot over a period of 11 days. Point O3 in the southwestern part of the plaza moved 0.24 foot upward in the same time.

CORE HOLES IN PLAZA

From August 5 to 10, 1963, two core holes were drilled and logged in the plaza area (figs. 2, 15). These holes were drilled to determine if a slip surface of the landslide existed below the plaza level or if a slip surface developed below the plaza with time.

The holes were drilled with an NX-size diamond bit. Hole 1 was drilled in competent granite to a total depth of 90.0 feet. It was cased, after reaming, for the first 10 feet with uncemented 4-inch casing. Hole 2 was drilled to a depth of 82.6 feet, also in competent granite, and was left uncased. (See chap. C, seismic line *F*.) No slip plane was recognized in either drill hole. Plastic pipe, 1 inch in diameter, was placed in each hole and the space between the pipe and the wall of the hole was packed with cinders.

The assumption was that, if any movement took place in the rock below the plaza area, the plastic tubing in the drill holes would be bent and eventually sheared. In order to test for bending of the plastic tubing, a 30-inch-long 3/8-inch-diameter rod was lowered on a nylon line to the bottom of the holes. These tests were made daily until mid-August, and then at various times until the start of construction of the rockfill buttress in the first week of September. During this time interval there was no indication that the plastic tubing had been bent.

DRAINAGE

In order to lower the water level in the landslide and increase stability, as much ground water as possible needed to be drained from the landslide mass and as much surface water as possible needed to be collected and diverted from the landslide. In mid-September 1963, five nearly horizontal 3-inch holes were drilled into the west side of the landslide from a point about 25 feet northeast of the flume west of the landslide (fig. 15). The flume had been installed during the construction of the cut to divert a small stream around the area of the cut. Following is a tabulation of the lengths, bearings, and inclinations of the drill holes and of the initial flows from each.

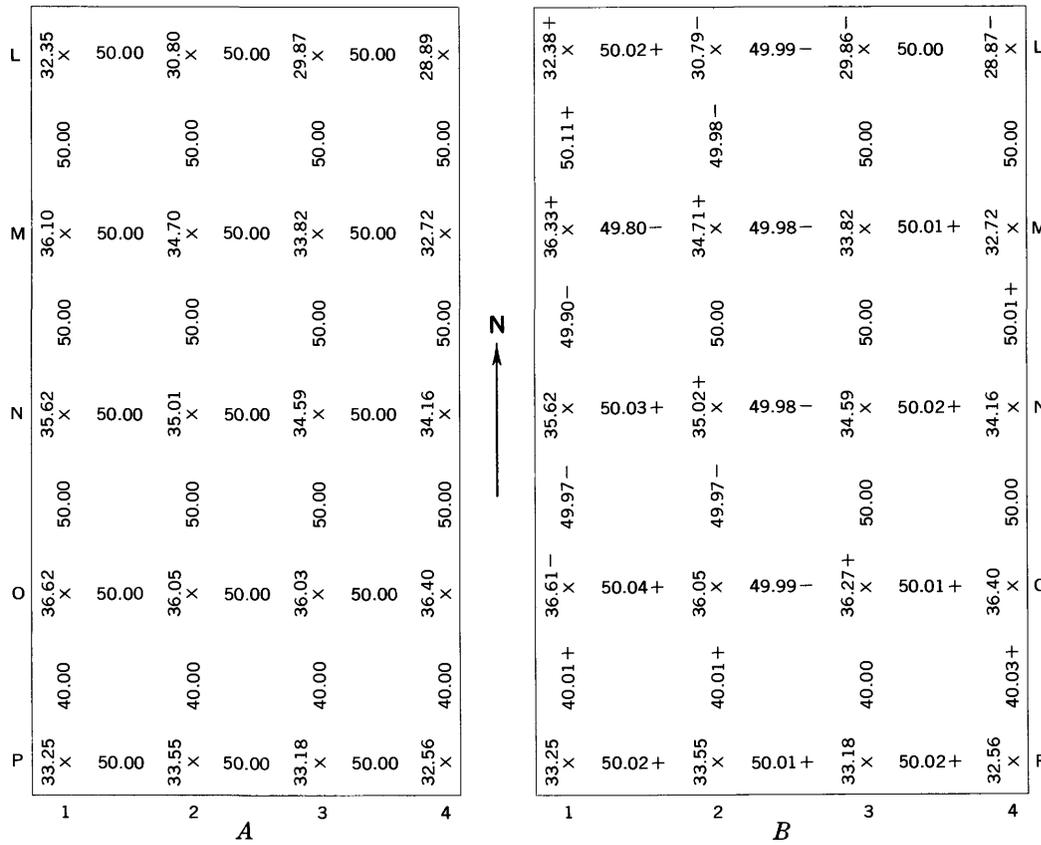


FIGURE 17.—The location and initial (A, Aug. 12, 1963) and final (B, Aug. 23, 1963) dimensions of survey grid established in the plaza area southeast of the landslide mass. Vertical measurements are shown to the left of the reference points (X). Horizontal measurements are shown between reference points. Units are feet. +, lengthened dimension; -, shortened dimension.

Lengths, bearings, inclinations, and initial flows from drill holes in western part of the Loveland Basin landslide

Hole	Length (feet)	Bearing	Inclination	Initial flow (gallons per minute)
1 -----	97	N. 25° W.	10° SE.	1
2 -----	168	N. 10° E.	5° SE. (?)	0
3 -----	50	N. 40° E.	5° SW.	(¹)
4 -----	60	N. 30° E.	12° SW.	0.5
5 -----	110	N. 16° E.	21° SW.	2

¹ Plugged.

The holes were drilled into bedrock and pipes were inserted into the holes for part of their length. In addition, the water from two small springs near the west margin of the landslide was impounded and conducted through pipes to the large flume west of the landslide. In July 1964, water was running from two of the drill holes, and one of the springs that had been tapped was flowing at an estimated rate of 8 gallons per minute.

In mid-October 1963, an 18-inch pipe was laid on the ground near the west margin of the slide (fig. 15) to divert surface runoff from snow which col-

lects during the winter months in a depression on and above the present head of the landslide. The snow accumulation amounts to as much as 21 feet, and the snow commonly persists into July of each year. The runoff from this area of snow accumulation was anticipated to be collected by the pipe and then to be diverted off the landslide to the large flume to the west of the landslide. Actually, very little water was diverted because the numerous fractures around the inlet caused the runoff to go underground.

CONCLUSIONS

The engineering investigations of the landslide were carried out under difficult conditions. The establishment of control points and the periodic observations of these points were hampered by topography and climate. The measurements made, however, proved to be valuable in defining the limits of the toe of the slide and in evaluating the effect of the buttress load on the toe of the slide. The at-

tempts to drain the surface and ground water from the slide were not as successful as had been hoped. Because of the topography, much of the slide area is not easily accessible to drilling equipment for drilling near-horizontal drain holes. Most of the precipitation on the landslide is snow, which accumulates to depths of as much as 250 inches. When the snow melts, very little of this moisture runs off the landslide; most of it melts from below and seeps into the landslide mass or evaporates. The data from results of engineering investigations and observations to September 1963 were available to R. A. Bohman and were used in his considerations of methods to stabilize the landslide.

Compilation of Results of Geological, Geophysical, and Engineering Investigations

By CHARLES S. ROBINSON *and* FITZHUGH T. LEE

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND
BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

COMPILATION OF RESULTS OF GEOLOGICAL, GEOPHYSICAL, AND
ENGINEERING INVESTIGATIONS

By CHARLES S. ROBINSON and FITZHUGH T. LEE

ABSTRACT

The compilation of data from the geological and geophysical investigations of the Loveland Basin landslide indicates that the movement of the landslide mass took place through a relatively wide zone in comparison to the total thickness of the landslide mass. Isopach and structure contour maps prepared from the geological and geophysical data indicate that the volume of the landslide above the slip zone is 470,000 cubic yards weighing 990,000 tons and that the volume of the landslide to the base of the slip zone is 770,000 cubic yards weighing 1,600,000 tons. It is believed that, in any sudden failure of the landslide, the volume and weight of material that would be involved would be in amounts between these figures.

INTRODUCTION

Geologic mapping and engineering measurements defined the shape, behavior, and composition of the Loveland Basin landslide at the surface. Geophysical investigations gave data on the possible composition of the landslide with depth and a probable thickness at several points. The determination of the remedial measures that should be taken to stabilize the landslide required a calculation of the mass and volume of the landslide.

The authors of this chapter made the final interpretation of the geological, geophysical, and engineering data and the calculations of the mass and volume of the landslide. A summary of the investigations and the results of the interpretation of the data were made available by Robinson, Carroll, and Lee (1964) about 4 weeks after the start of the investigations and were the basis for the engineering analysis of the landslide and the recommendations for the stabilization of the landslide that are presented in chapter F.

COMPILATION OF GEOLOGICAL AND
GEOPHYSICAL DATA

During the geophysical investigations, numbered stakes were placed for the resistivity work at the center of each spread and for the seismic work at each geophone location and shothole. At the time of the geologic mapping, these lines were located and plotted on the map (fig. 7). R. Woodward Moore furnished the authors with plots of the apparent resistivity and two interpretive sections along the resistivity traverses (lines A, B; fig. 7). Subsequently, the authors obtained from R. A. Bohman the plots of the apparent resistivities for the third traverse (line C, fig. 7). The authors furnished topographic profiles along each of the seismic lines (fig. 7), which were used to interpret the seismic data. On copies of the geologic map that showed the locations of all points where geophysical measurements were made, the authors plotted the depth information supplied by R. Woodward Moore and R. D. Carroll.

A controlling factor in the interpretation of the possible depth of the slip surface was the geometry of the landslide. From the geologic mapping and the engineering measurements, the outlines of the landslide mass and the relative rates of movement of different parts of the landslide were known. The dip of the scarps and the plunge of the striations on the scarp surfaces limited the depth of a possible slip surface and defined the direction of movement of the various parts of the landslide.

The geometry of different types of landslides has been well defined and illustrated by Varnes (1958). According to Varnes' classification, the Loveland Basin landslide is partly type IIA, a slump, and partly type IIB, a debris slide. Slump features are best displayed in the upper parts, whereas debris sliding and earthflows predominate toward the base.

Because of these features and because the Loveland Basin landslide consisted of several units moving at different rates, all of which were dependent upon each other, this landslide could best be described as a composite slide.

In order to determine the probable depth of the slip surface, the geophysical data, geological data, and geometry of the slide were carefully considered for each point of measurement. To aid in this consideration, longitudinal and transverse vertical sections were constructed that intersected most of the points of measurement. On these sections were plotted the geophysical and geologic data. At several points, the geophysical data had indicated more than one possible depth to the slip surface. From the plots of these data in section and because the landslide consisted of a central, virtually unbroken mass bordered—particularly on the north and west sides—by a zone of intensely sheared material, a conclusion was reached that the movement of the landslide was taking place throughout a wide zone, or on a series of closely spaced slip surfaces, rather than on a single slip surface. Resistivity data at several locations indicated subsurface changes at greater depths than the depths calculated from the seismic velocity data. The different possible depths for the slip zone as given by the geophysical data, it was presumed, indicated the possible top and bottom of the slip zone. From the comparison and evaluation of the resistivity and seismic data with the geologic information and the limiting geometry of the slide, a depth for the top and bottom of the slip zone was picked for each point of measurement.

CALCULATIONS OF VOLUME AND WEIGHT OF THE LANDSLIDE

The depths to the top and bottom of the slip zone were plotted on separate copies of the topographic and geologic map. With this control, construction of isopach maps was then possible for the thickness of the landslide above the slip zone and for the thickness of the landslide to the bottom of the slip zone. The two isopach maps so constructed are figures 18 and 19.

The volume and weight of the landslide above the slip zone and to the bottom of the slip zone were then calculated from the isopach maps. The volume was calculated by superimposing a 10×10 -divisions-per-inch grid over the isopach maps and then averaging the thickness for each square inch or part thereof and multiplying the thickness by the area in square inches. The weight was calculated by multiplying the volume, in cubic yards, by a value

of 2.1 tons per cubic yard. The weight per cubic yard was determined, assuming complete water saturation, from density and porosity measurements and density-of-fracture calculations previously made during the study of the geology of the Straight Creek Tunnel pilot bore (Robinson and Lee, 1962).

With the isopach maps plotted on the topographic base, structure contour maps of the top and the base of the slip zone could be constructed. These maps are figures 20 and 21. The greater detail of the contours on these maps as compared to the isopach maps is the result of the greater detail of the contours on the topographic base. The purpose of the structure contour maps was to give the approximate elevation of the top and bottom of the slip zone at any point within the landslide mass. These maps permit the rapid calculation of the necessary length and attitude of any holes drilled to intersect the top or bottom of the slip zone for drainage or any other purpose.

CONCLUSIONS

The volume of the landslide to the top of the slip zone was calculated to be about 470,000 cubic yards weighing about 990,000 tons. The total volume of the landslide to the bottom of the slip zone was calculated to be about 770,000 cubic yards weighing about 1,600,000 tons. The authors believed that, should the landslide fail suddenly, the volume and weight of material involved would be between these two sets of figures.

Figures 18 through 21 are modified from the original isopach and structure contour maps made available by Robinson, Carroll, and Lee (1964). The original maps were compiled on a topographic base with a 1-foot contour interval. The contour interval has been reduced to 5 feet on the maps included with this report. A 5-foot contour interval is more realistic in achieving accuracy in the calculated probable thickness of the landslide and the elevation of the top and bottom of the slip zone at any one point. These data on the maps are considered to be accurate to within 10 percent.

Longitudinal and transverse sections (fig. 22) were prepared from the structure contour maps to show the configuration of the slip zone and its relation to the entire landslide and the toe load. The central and the western parts of the slide, in general, are deepest. The changes in thickness of the slip zone and the sliding mass and the irregularities along the base of the slip zone are due mainly to variations in surface topography and bedrock competency.

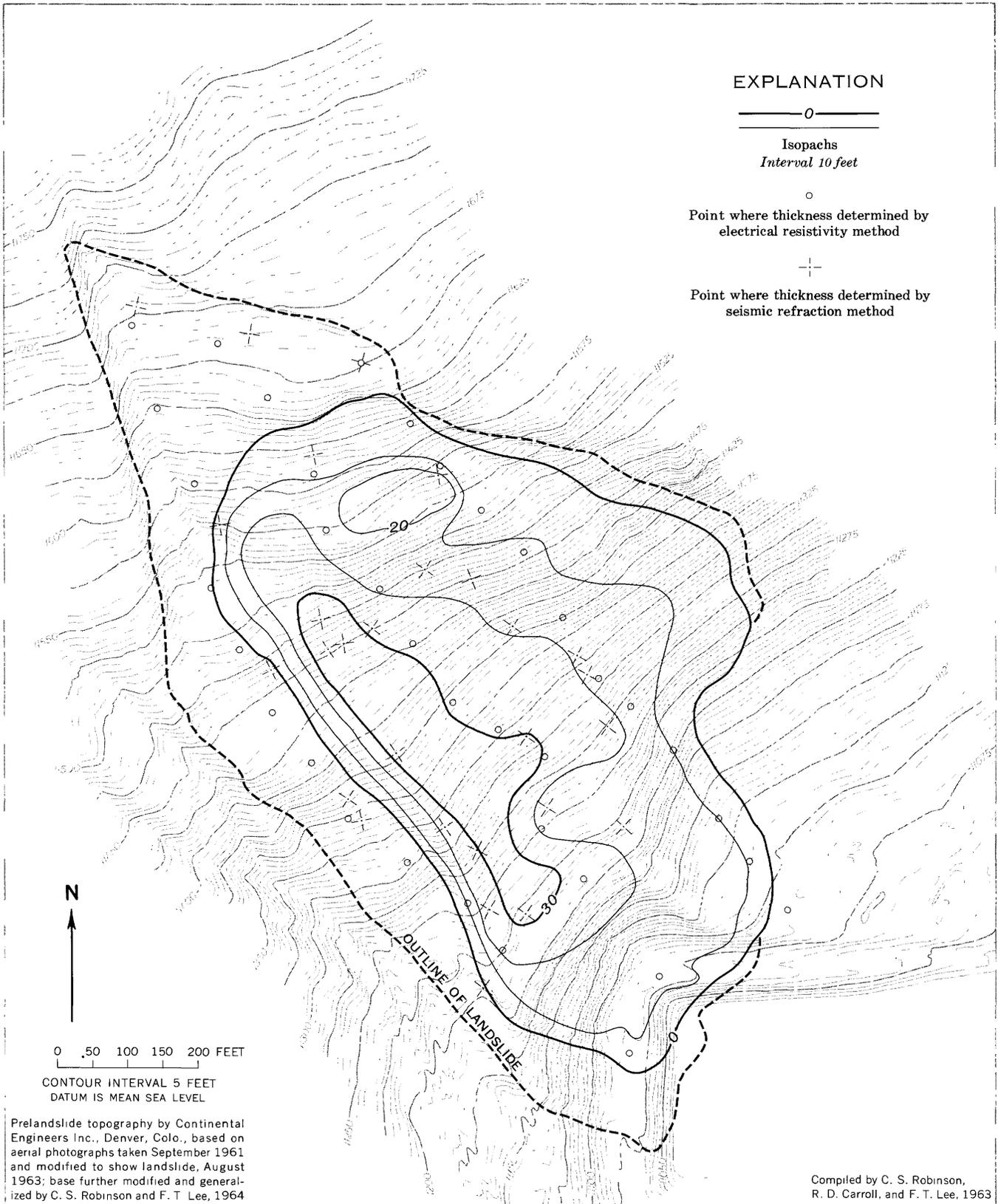


FIGURE 18.—Isopach map of the Loveland Basin landslide to the top of the slip zone.

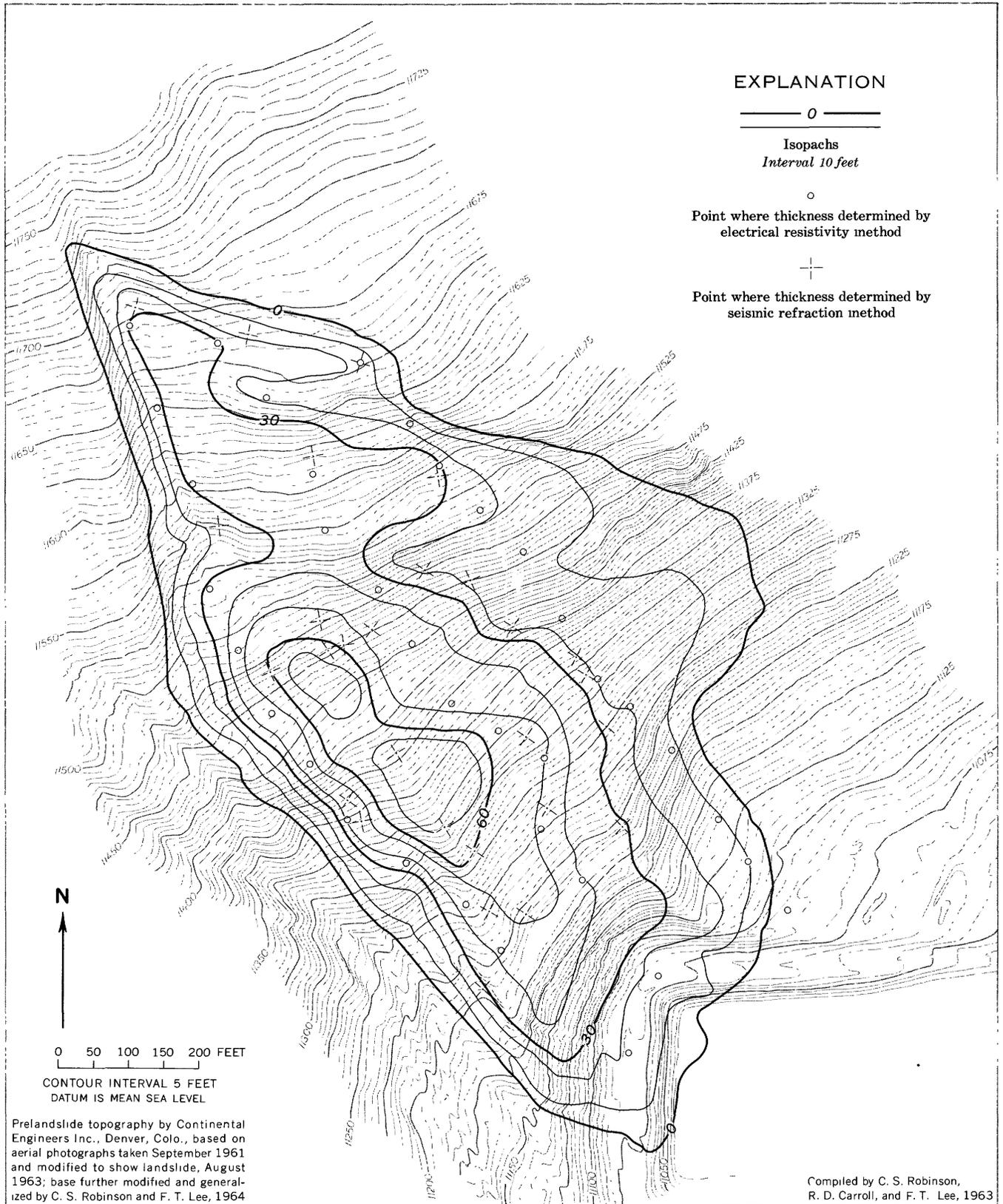


FIGURE 19.—Isopach map of the Loveland Basin landslide to the base of the slip zone.

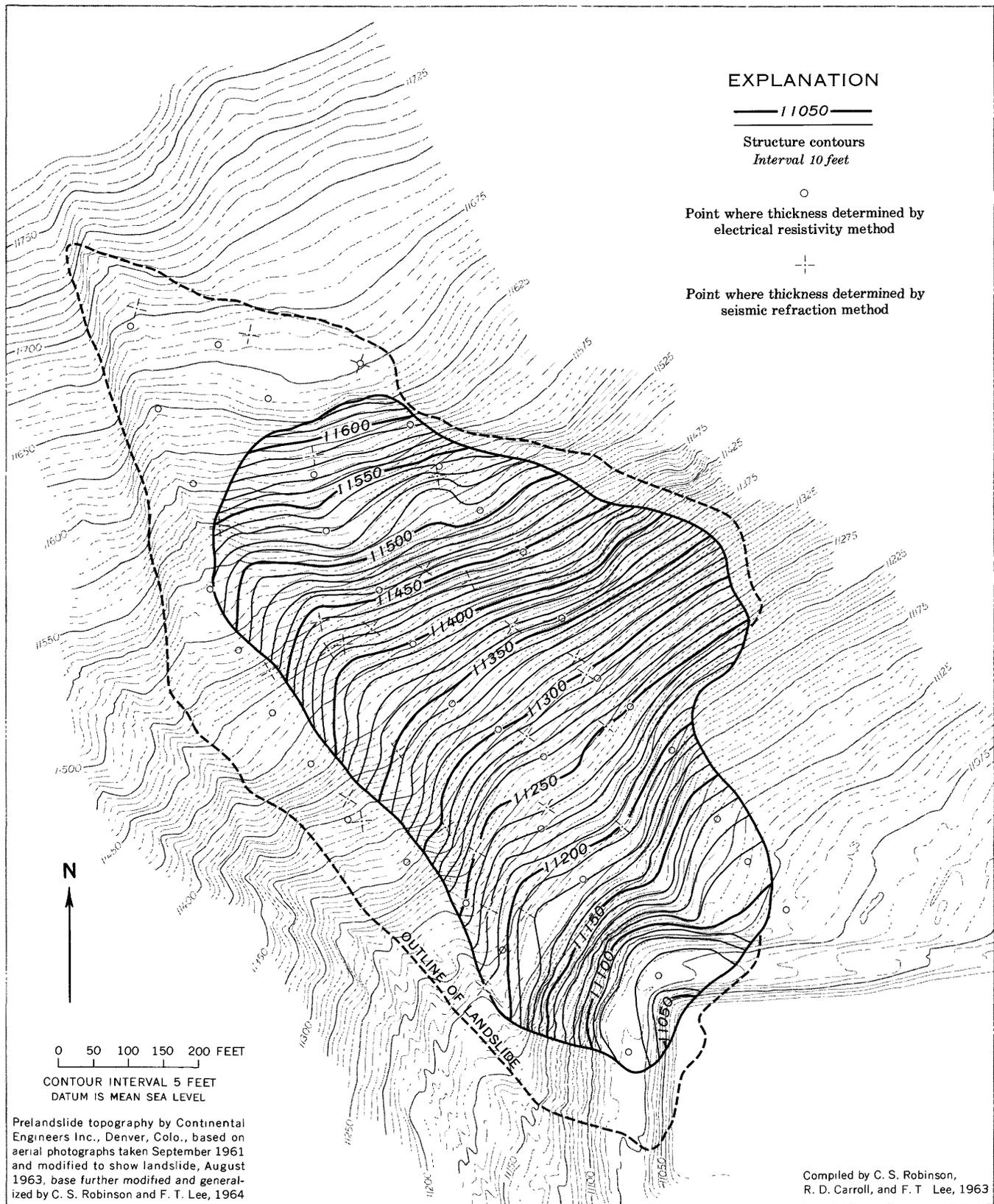


FIGURE 20.—Structure contours drawn on the top of the slip zone of the Loveland Basin landslide.

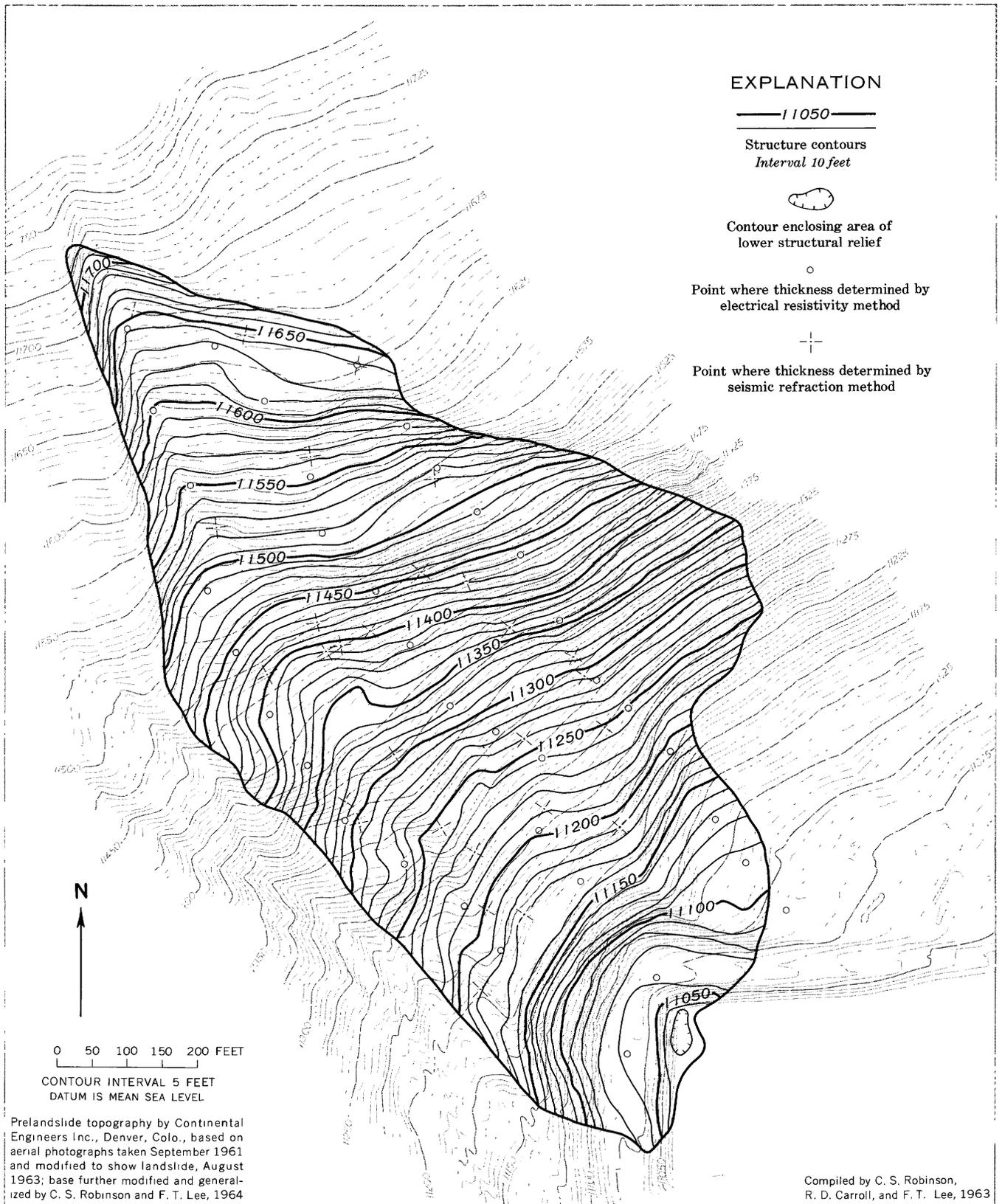


FIGURE 21.—Structure contours drawn on the base of the slip zone of the Loveland Basin landslide.

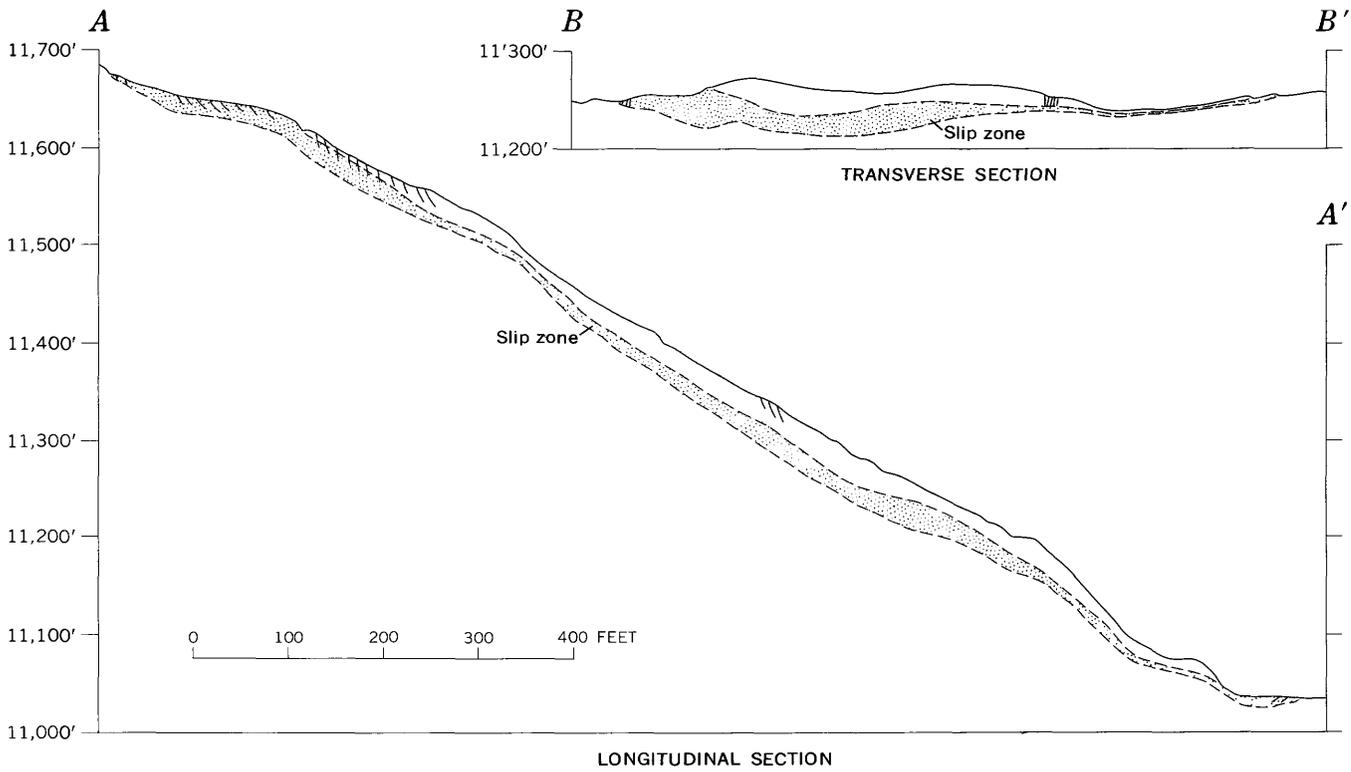


FIGURE 22.—Longitudinal and transverse sections of the Loveland Basin landslide. Lines of sections shown in figure 2.

Stability Analysis

By ROBERT A. BOHMAN

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
1963-65

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GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS OF THE LOVELAND
BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO, 1963-65

STABILITY ANALYSIS

By ROBERT A. BOHMAN⁵

ABSTRACT

A preliminary analysis of the Loveland Basin landslide indicated that a fill or buttress containing approximately 100,00 cubic yards of material properly placed in the plaza area at the toe of the landslide would provide a factor of safety against further movement of approximately 1.4. During September and October 1963, a buttress of 61,775 cubic yards of material was placed at the toe of the landslide. An analysis of this buttress indicates that the factor of safety against failure was about 1.1 at the base of the slip zone and about 1.2 along the top of the slip zone. The factor of safety against failure of the landslide mass directly above the level of the buttress in the cut slope was calculated as about 1.0. The effect of water was not considered because of the lack of hydrologic data.

The probability of failure of the landslide above the present buttress might be reduced by raising the level of the buttress. If the landslide should fail above the level of the present buttress, however, the landslide material would probably be retained on top of the present buttress and could aid in further stabilization.

INTRODUCTION

Prior to the preparation of the report by Robinson, Carroll, and Lee (1964), a stability analysis of the Loveland Basin landslide was made only from preliminary electrical resistivity data, measured dimensions, and assumed physical properties of the sliding material. This analysis indicated that a fill containing approximately 100,000 cubic yards of material properly placed in the plaza area at the toe of the landslide would provide a factor of safety against further movement along the full length of the existing slip zone of approximately 1.4. The fill was expected to act primarily as a buttress to receive pressure of the sliding material. The stability analysis of the buttress, after its construction, was based on the delineation of the slide and slip zone by Robinson, Carroll, and Lee (1964).

As a result of the analysis of all available data and after numerous meetings, field inspections, and

office projections, the decision was made to shift the position of the Straight Creek Tunnel pilot bore to the south, to lower the pilot bore and approach grade, and to shift the portal to the east of the location originally planned. The basic reasons for these changes were to place the portal of the pilot bore as far from the active landslide as possible and to develop all material possible for constructing the recommended buttress. The original plaza area, centerline locations, and other pertinent features are shown in figure 23. Also shown are the changes from the original plan.

The buttress was constructed in September and October 1963 and contains a volume of approximately 61,755 cubic yards of compacted material consisting primarily of shot and ripped rock obtained from the section immediately east of the pilot bore portal where the grade was lowered approximately 16 feet. This amount of material was all that was readily available at the time and was far less than that recommended for the buttress. A second analysis to be based on the actual buttress was considered essential; this analysis was made in 1964 and is presented here.

ANALYSIS

Data on the constructed buttress, together with the information presented by Robinson, Carroll, and Lee (1964), form the bases for the stability analysis presented in this report. The method of analysis is virtually that given by Baker and Yoder (1958).

Assuming the main driving force is generally parallel to the axis of the zone of slippage in a vertical plane, the available factor of safety against failure at the lower limit of the slip zone may be calculated as follows (fig. 24):

Approximate unit weight of landslide material =
150 lb per cu ft (pound per cubic foot).

F.S. (factor of safety) = 0.98. (The factor of safety

⁵ Assistant regional materials engineer, region 9, U.S. Bureau of Public Roads.

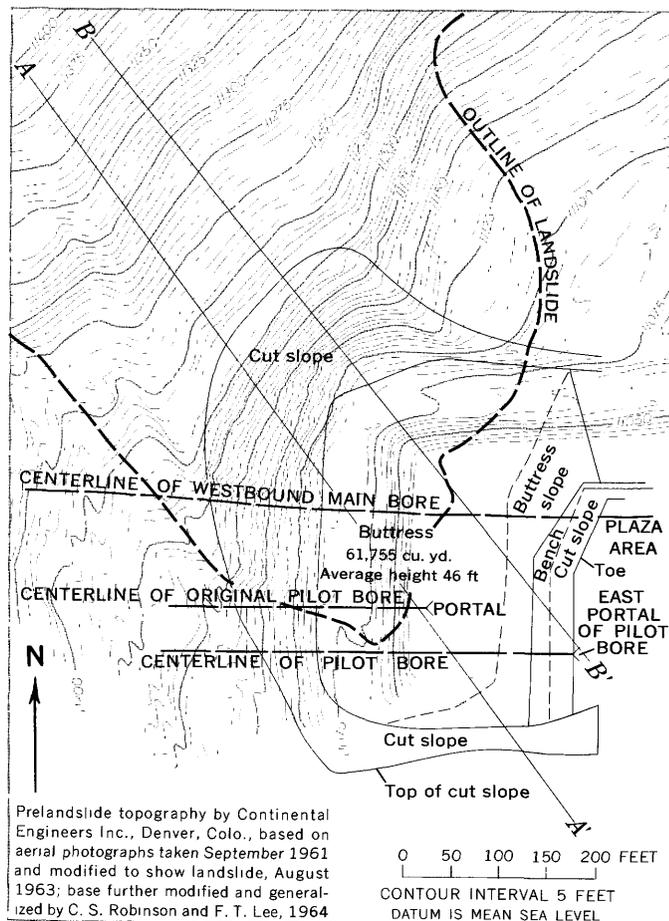


FIGURE 23.—The plaza area and buttress placed on toe of Loveland Basin landslide. A-A' and B-B', sections used to calculate factor of safety against failure at the lower limit of the slip zone; shown in figure 24.

of a slope just prior to failure is 1.0. A slightly more conservative value of 0.98, before placement of the buttress, is assumed here.)

θ (average slope angle) = 26° ($\sin \theta = 0.438$, $\tan \theta = 0.488$, $\cos \theta = 0.899$).

Volume of landslide to lower limit of slip zone:

Section A-A' = 61,800 cu ft, 1-ft-wide slice.

Section B-B' = 54,000 cu ft, 1-ft-wide slice.

W (weight of landslide):

Section A-A':

$$W = \frac{150 \times 61,800}{2,000} = 4,640 \text{ tons.}$$

Section B-B':

$$W = \frac{150 \times 54,000}{2,000} = 4,050 \text{ tons.}$$

T (driving force) = $W \sin 26^\circ$:

Section A-A':

$$T = 4,640 \times 0.438 = 2,030 \text{ tons.}$$

Section B-B':

$$T = 4,050 \times 0.438 = 1,770 \text{ tons.}$$

S (shear resistance) = $T \times \text{F.S. (assumed)}$:

Section A-A':

$$S = 2,030 \times 0.98 = 1,990 \text{ tons.}$$

Section B-B':

$$S = 1,770 \times 0.98 = 1,730 \text{ tons.}$$

Because no hydrologic data were available, this analysis omitted any specific or tangible allowances for the effects water undoubtedly has on the stability of the mass. The assumption was that the hydrologic conditions existing at the time of failure (F.S. = 1.0 or 0.98) would be repeated about annually and that this repetition would represent the worst hydrologic condition likely to be present for the ensuing few years. During this time further physical, geologic, and hydrologic data could be gathered, and the slide corrective measures could be reevaluated on the basis of new information.

Because of the lack of hydrologic data a possible drainage solution could not be evaluated. In addition, where no drainage solution is involved, no values of friction and cohesion for the sliding material in redesigning the failed slope or buttress need be determined.

It is, however, necessary to estimate the shear strength of the material used in the buttress. We assumed that the material would be granular material similar to shot rock, talus, or sand and gravel. We further assumed this material would have no cohesive strength but would have an angle of internal friction of about 34° or about a 1:1½ slope. This value is considered to be conservative but realistic. Therefore, for the buttress (fig. 24):

ϕ (angle of internal friction) = 34° . $\tan \phi = 0.67$.

Approximate unit weight of buttress material = 150 lb per cu ft.

W_B = estimated weight of a 1-ft-wide slice of buttress material on sections A-A' and B-B'.

$W_B = N \cos \phi$, $\phi = 0^\circ$, $\cos 0^\circ = 1$, N = force normal to surface.

T_B = (tangential component of W_B) = 0 (no driving forces):

Section A-A':

$$W_B = \frac{150 \times 5,480}{2,000} = 411 \text{ tons.}$$

Section $B-B'$:

$$W_B = \frac{150 \times 5,820}{2,000} = 436 \text{ tons.}$$

Factor of safety with buttress at lower limit of slip zone is:

$$\text{F.S.} = \frac{S + W_B \tan \phi}{T}$$

Section $A-A'$:

$$\text{F.S.} = \frac{1,990 + 411 \times 0.67}{2,030} = 1.11.$$

Section $B-B'$:

$$\text{F.S.} = \frac{1,730 + 436 \times 0.67}{1,770} = 1.14.$$

The factor of safety against failure along the upper limit of the slip zone may be calculated as follows:

Volume of landslide to upper limit of slip zone:

Section $A-A'$ = 36,300 cu ft, 1-ft-wide slice.

Section $B-B'$ = 36,400 cu ft, 1-ft-wide slice.

Section $A-A'$:

$$W = \frac{150 \times 36,300}{2,000} = 2,720 \text{ tons.}$$

$$T = 2,720 \times 0.4380 = 1,190 \text{ tons.}$$

$$S = 1,190 \times 0.98 = 1,170 \text{ tons.}$$

$$\text{F.S.} = \frac{1,170 + 411 \times 0.67}{1,190} = 1.21.$$

Section $B-B'$:

$$W = \frac{150 \times 36,400}{2,000} = 2,730 \text{ tons.}$$

$$T = 2,730 \times 0.4380 = 1,200 \text{ tons.}$$

$$S = 1,200 \times 0.98 = 1,180 \text{ tons.}$$

$$\text{F.S.} = \frac{1,180 + 436 \times 0.67}{1,200} = 1.23.$$

If we assume that the force of the landslide at the lower limit of the slip zone acts on the buttress and, in the plaza area in general, is a horizontal one, the factor of safety is raised slightly:

Horizontal component of driving force ($T_h = T \cos 26^\circ$):

Section $A-A'$:

$$T_h = 2,030 \times 0.899 = 1,820 \text{ tons.}$$

$$S_h = T_h \times \text{F.S.} = 1,820 \times 0.98 = 1,780 \text{ tons.}$$

$$\text{F.S.}_h = \frac{1,780 + 411 \times 0.668}{1,820} = 1.13.$$

Section $B-B'$:

$$T_h = 1,780 \times 0.899 = 1,600 \text{ tons.}$$

$$S_h = 1,600 \times 0.98 = 1,570 \text{ tons.}$$

$$\text{F.S.}_h = \frac{1,570 + 436 \times 0.67}{1,600} = 1.16.$$

From the appearance of conditions shown on sections $A-A'$ and $B-B'$, figure 24, shear failure through the landslide mass might possibly occur in a zone between the approximate altitudes of 11,080–11,150 ft. If we assume that the weak zone is at an altitude of 11,080 ft and that failure would most likely occur along a horizontal surface at the top of the buttress, then all material below this point, including the buttress, can be omitted from further analysis and the force components are as follows:

W_1 = Estimated weight of a 1-ft-wide slice of slide material on section $A-A'$ and $B-B'$ if we assume failure will most likely occur at an altitude of 11,080 ft.

Section $A-A'$:

$$W_1 = \frac{(61,800 - 2,700) \times 150}{2,000} = 4,430 \text{ tons.}$$

Section $B-B'$:

$$W_1 = \frac{(54,000 - 2,000) \times 150}{2,000} = 3,900 \text{ tons.}$$

The negative areas in the above equations are shown as crosshatched areas in figure 24.

Section $A-A'$:

$$T_1 = (\text{tangential component of } W_1) = W_1 \sin 26^\circ.$$

$$T_1 = 4,430 \times 0.438 = 1,940 \text{ tons.}$$

$$\text{Horizontal component of } T_1 = 1,940 \times 0.899 = 1,740 \text{ tons.}$$

Section $B-B'$:

$$T_1 = 3,900 \times 0.438 = 1,710 \text{ tons.}$$

$$\text{Horizontal component of } T_1 = 1,710 \times 0.899 = 1,540 \text{ tons.}$$

W_x = Estimated weight of 1-ft-wide slice of the resisting block of slide material (stippled areas in fig. 24) if we assume failure is most likely to occur at an altitude of 11,080 ft.

Section $A-A'$:

$$W_x = \frac{950 \times 150}{2,000} = 71 \text{ tons.}$$

Section $B-B'$:

$$W_x = \frac{600 \times 150}{2,000} = 45 \text{ tons.}$$

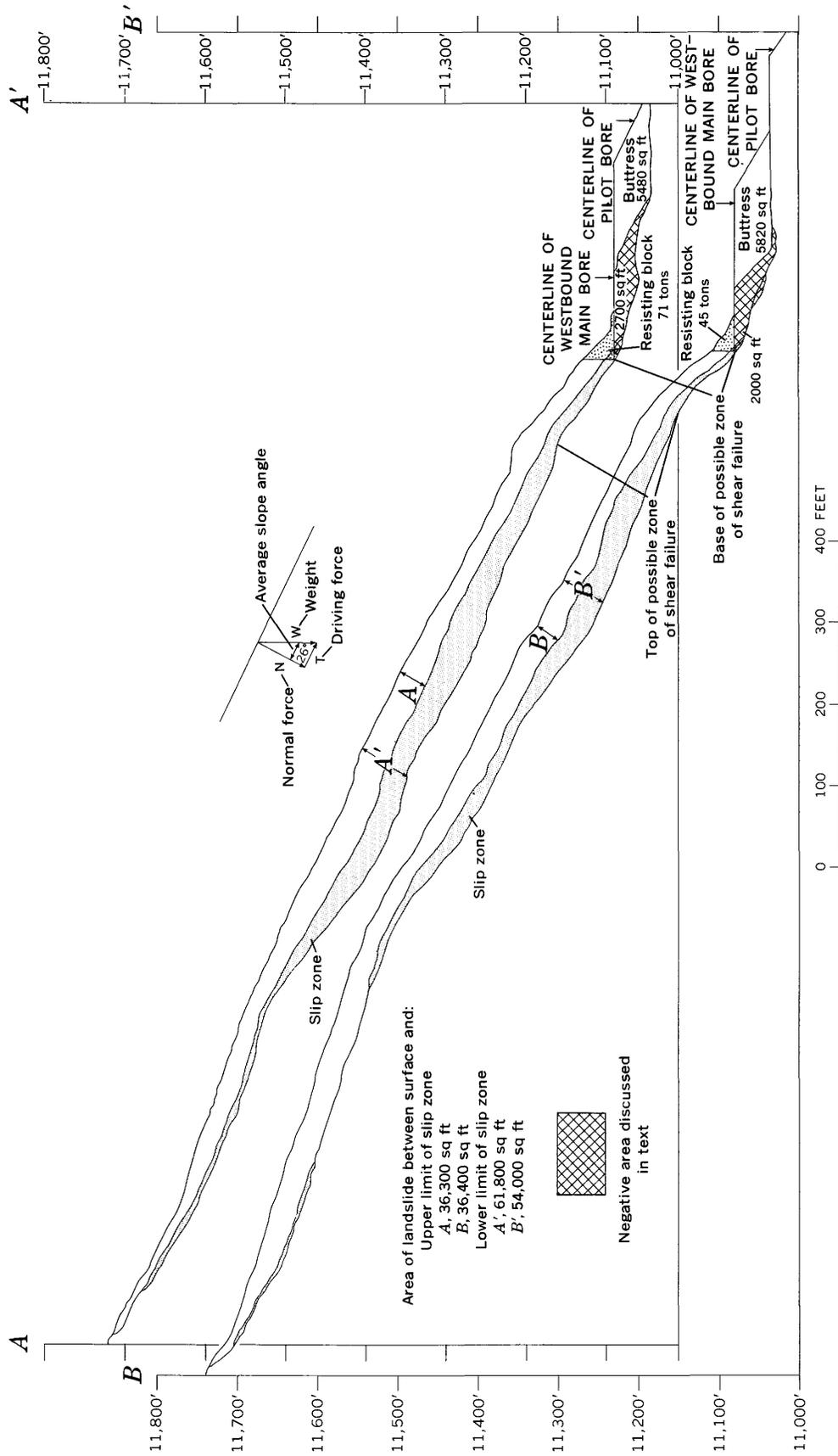


FIGURE 24.—Sections for analysis of the Loveland Basin landslide. Prepared by R. A. Bohman, U.S. Bureau of Public Roads, 1964.

T_{1h} (resisting forces) = horizontal component of
 $T_1 \times \text{F.S.} + W_x \tan \phi$:

Section A-A':

$$T_{1h} = 1,740 \times 0.98 + 71 \times 0.67 = 1,750 \text{ tons.}$$

Section B-B':

$$T_{1h} = 1,540 \times 0.98 + 45 \times 0.67 = 1,535 \text{ tons.}$$

The factor of safety, then, is:

Section A-A':

$$\text{F.S.} = \frac{1,750}{1,740} = 1.006.$$

Section B-B':

$$\text{F.S.} = \frac{1,535}{1,538} = 0.997.$$

These figures indicate that the factors of safety are only about 1 and that conditions are such that additional movement can be expected.

Areas used in all calculations were measured by planimeter. Calculations were made to slide-rule accuracy.

One method of minimizing the possibility of failure above the buttress would be the construction of another buttress on top of the existing one. The desirability of doing this was recognized and discussed during earlier stages of investigation and analysis. Lack of material and cost were the primary reasons given for not building an additional, second-story buttress during the period July-October 1963. Another reason was that the author believed that shear failure along a plane at an approximate altitude of 11,080 feet would not result in an instantaneous movement of the entire landslide mass that might override the buttress and spill into the new plaza or portal area. Rather, he believed that such shear failure would result in a slow movement of landslide material out onto the surface of the buttress, thereby forming a natural second-story buttress that would effectively resist further movement before it reached the edge of the buttress where it could spill over into the plaza area. Any movement of material onto the buttress would result in a decreased driving force and increased resistance to further movement.

No data are available for use in determining the effect of water on this slide. The analysis presented

makes no specific allowance for the effect of water, although water unquestionably has considerable influence on the overall condition of stability and should receive attention. The author believes that the most critical condition of stability will coincide with, or shortly follow, periods of rapid thawing or heavy rainfall.

Control points were placed and observed regularly by personnel of the Colorado Department of Highways to detect and determine magnitude and direction of movement (chap. D). A review of these data in July 1964 revealed that movement was still occurring in certain segments of the landslide, particularly near the head and eastern flank. The direction of movement near the head was generally parallel to the longitudinal axis of the slide. The movement along the east flank, however, was more in an easterly direction approximately parallel to the highway and tunnel alignment; this direction would indicate that natural buildup of resistance above the buttress and additional resistance provided by the buttress itself had shifted the point of weakness toward the east flank, which in turn resulted in a change of direction of the landslide movement away from the critical portal area.

In effect, the east flank was acting as a "relief valve," thereby reducing the stresses or driving forces assumed in this report to have been acting directly on the buttress and the part of the landslide immediately above the buttress.

CONCLUSIONS

Specific recommendations regarding additional landslide control measures or final location and design of the proposed vehicular tunnels are beyond the scope of this report. However, the author believes that any further studies supporting ultimate design of the vehicular tunnels and portal areas should not overlook the probable desirability of extending the height of the buttress and providing additional drainage for the landslide mass.

Further, the analysis contained herein is somewhat simplified and is based on many assumptions which may or may not be generally accepted. The aim here has been to reasonably evaluate the landslide conditions as they existed in 1963 and 1964 at the east portal of the Straight Creek Tunnel pilot bore.

Summary and Recommendations

By CHARLES S. ROBINSON *and* FITZHUGH T. LEE

GEOLOGICAL, GEOPHYSICAL, AND ENGINEERING INVESTIGATIONS
OF THE LOVELAND BASIN LANDSLIDE, CLEAR CREEK COUNTY, COLORADO,
1963-65

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SUMMARY AND RECOMMENDATIONS

By CHARLES S. ROBINSON and FITZHUGH T. LEE

SUMMARY

The Loveland Basin landslide occurred in an area of extensively faulted and altered Precambrian igneous and metasedimentary rock. During Pleistocene time, morainal material was deposited in valley bottoms and on glacially oversteepened valley walls. After the glaciers retreated, the morainal material locally moved downslope, and the oversteepened and incompetent valley walls began receding. Thus, before the Loveland Basin landslide occurred, the slope was in a weakened and potentially unstable condition. Slope failure occurred when surficial material and bedrock were excavated at the base of the slope, effectively decreasing the resisting forces and triggering movement along preexisting faults and joints toward the plaza cut.

The heterogeneity of the geologic framework was expressed in the nonuniform geometry of the landslide mass. Both slump and debris slide features developed indicating the composite nature of the slide. Engineering surveys established that the mass consisted of several parts that were moving at different rates and that each part was dependent upon its neighbor. The geometry of the landslide influenced estimates of depth of the slip surface. Geologic mapping established the outline of the slide, dip of scarps, and the plunge of striations on the scarp surfaces. This information helped to limit estimates of the depth to a slip surface and, with the engineering measurements defined the direction of movement of the various parts of the slide.

The probable depth to the slip surface was determined by considering—at each point of measurement—geophysical, geological, and engineering data. At many points the seismic data and, to a greater extent, the resistivity data indicated more than one possible depth to a slip surface. This information combined with the extremely nonuniform

character of the bedrock was interpreted as indicating that landslide movement was occurring nonuniformly throughout a zone rather than occurring on a single slip surface. Seismic depth interpretations were generally more shallow than resistivity depth interpretations. Although many seismic and resistivity depth determinations at similar locations were in good agreement, many resistivity plots indicated additional, deeper changes. These deeper changes helped establish the presumed lower limit of the slip zone. A depth for the top and bottom of the slip zone was obtained for each point of measurement by comparing resistivity and seismic data with the geologic information and the limiting geometry of the landslide.

By means of the most significant shallow resistivity depth observations, a volume of 500,000 cubic yards for the slide was calculated. The generally shallow seismic depth determinations yielded a calculated volume of 440,000 cubic yards. The relatively close agreement of results from these two methods, when compared with geologic and engineering measurements, resulted in a calculated minimum volume of 470,000 cubic yards. This represented the volume of moving material above the top of the slip zone. The deeper resistivity depth changes combined with the observed bounding scarps, particularly as displayed in the plaza cut, determined the base of the slip zone. From this information the total volume of the sliding mass was calculated as 770,000 cubic yards.

A compacted buttress load of 61,775 cubic yards was placed on the toe of the landslide. A stability analysis of this buttress was made on the basis of the above slide mass volumes. The load was calculated to provide a factor of safety against failure of 1.1 at the base of the slip zone and a factor of safety of 1.2 at the top of the slip zone. The factor

of safety against failure of that part of the landslide mass directly above the level of the buttress in the plaza cut slope was calculated as about 1.0. If the landslide failed above the buttress level, slide material would probably be retained on top of the buttress and could aid in the further stabilization of the landslide.

RECOMMENDATIONS

The frequency of landslides related to construction could be greatly reduced if adequate geologic and engineering investigations were made in advance of design. This procedure would not prevent all landslides related to construction because the prediction of landslides is far from an exact technique. When a landslide does occur, experienced personnel and necessary equipment must be available full time until a solution is reached.

The first essential in the study of a landslide is an accurate topographic base map. The base map used for the study of the Loveland Basin landslide was prepared from earlier photographs taken for another purpose and lacked pertinent definition of the topography. The authors recommend that upon recognition of a landslide, or potential landslide, ground control be immediately established, the area be photographed, and a topographic and photogeologic map be prepared. Several bench marks placed at the time of establishing topographic control on the landslide mass and on the adjacent stable ground would be useful for future reference. An overlay which could be prepared during photogrammetric compilation of the topography and photogeology and which could show the distribution of various types of vegetation for interpretation of soil and ground-water conditions and as an aid in planning geologic and geophysical work would be helpful.

The geological and geophysical work should be conducted simultaneously and the interpretation of the results should be done jointly by the geologist and geophysicist. Ideally, the geologist and geophysicist should have the opportunity of returning to the field after preliminary compilation of data to check anomalous results. For the investigations of most types of landslides, both electrical resistivity and seismic studies are recommended. The combination of the two types of data allows a more accurate definition of the landslide than do the results of only one method.

Any landslide study should include repeated and systematic measurements of the locations of accurately located survey points established on the landslide mass and on stable ground adjacent to the mass. These measurements would not only show the direction and amount of movement before remedial work but would indicate, if measurements were continued after completion of the remedial work, the degree of success of the remedies.

The conclusions from this study are that the combination of geological, geophysical, and engineering methods of investigations of the Loveland Basin landslide during its early history had several advantages over physical methods of investigation, such as drilling and trenching. In summary, these advantages are: (1) less investigation time, (2) lower cost, (3) less disruption of the landslide mass, and (4) a more complete definition of the sliding mass. Ideally, a combination of geological, geophysical, engineering, and physical investigations should give the best results, but if time is important and the area has only limited accessibility, good results can be obtained by geological, geophysical, and engineering methods.

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