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AN ABSTRACT OF THE THESIS OF Susanne L. D'Agnese for a Master of Science in Geology presented May 19, 1986.

Title: The Engineering Geology of the Fountain Landslide Hood River County, Oregon.

APPROVED BY MEMBERS OF THE THESIS COMMITTEE:

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Ansel G. Johnson, Chairman

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Marvin H. Beeson

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Richard E. Thoms

The Fountain Landslide located along I-84, five kilometers east of Cascade Locks, Oregon has moved periodically for over thirty years. Aerial photographs taken prior to recorded movement of the landslide show the headscarp of a large preexisting landslide. In 1952 a cut was made into the toe of the landslide to straighten Highway 30. The recorded movement history begins at this time. Stabilization procedures in the late 1950's focused

on dewatering the slide mass. Movement had nearly stopped by 1957. A deeper cut was made into the toe of the landslide in 1966 to widen the highway to the four-laned I-80N (later renamed I-84). Accelerated movement resulted. The Oregon State Highway Division removed 264,000 cubic meters of material from the head of the movement zone. Accelerated movement continued. The Oregon State Highway Division then began intense research of the landslide. Research included core logs, slope inclinometers, and the ground water data. The western portion of the slide mass was unloaded more extensively in 1970 (1.2 million cubic meters). This later unloading slowed down the movement, but it continues periodically.

The oldest unit found in the area is a volcaniclastic unit. It is found only in core logs in the SW portion of the slide. The basalts of the Columbia River Basalt Group are found intact and as talus in the study area. Quartz diorite intrusives younger than the Columbia River Basalt Group is found at the surface and at depth along the entire length of the toe of the landslide. Wind River Lava crossed from Washington, dammed the Columbia River and was deposited within the study area.

The slide mass consists primarily of Columbia River Basalt Group talus and Wind River Lava talus. The slip plane consists primarily of rocky mudstone. The ground water table is elevated over the intrusive at the toe of the landslide and over the volcaniclastic unit at the head. Surface cracks and scarps indicate that the slide mass moves northward, drops at the head and heaves at the toe.

A slope stability analysis of the Fountain Landslide showed that the instability here is the result of elevated groundwater and the removal of material at the toe for highway construction. It also showed that the eastern portion is more stable than the western portion. The differences in the stability result of the addition of fill at the toe and a lower ground water table in the eastern portion. The development of the prehistoric slide resulted when the dam of Wind River Lava was removed and lateral support for the deposit was lost.

This study shows that it is essential to have adequate geologic information prior to construction or remedial design for any preexisting landslide to avoid stability problems.

THE ENGINEERING GEOLOGY OF THE FOUNTAIN LANDSLIDE HOOD RIVER COUNTY, OREGON

by

SUSANNE L. D'AGNESE

thesis submitted in partial fulfillment of the requirement for the degree of

MASTER OF SCIENCE
in
GEOLOGY

Portland State University

1986

TO THE OFFICE OF GRADUATE STUDIES AND RESEARCH:

The members of the Committee approve the thesis of Susanne L. D'Agnese presented May 19, 1986.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

FIGURE PAGE

FIGURE PAGE

INTRODUCTION

The Columbia River Gorge cuts through the Cascade Range between Troutdale and The Dalles, Oregon (Figure 1). It is noted for its spectacular scenery. This spectacular topography has been shaped by a variety of geomorphic processes. One of the important geomorphic processes seen in the Columbia River Gorge is mass wasting or landsliding.

LOCATION

This report discusses a relatively small landslide located about five kilometers east of the town of Cascade Locks, Oregon named the Fountain Landslide (Figure 1). It covers approximately 0.5 square kilometers on an irregularly sloping hillside below cliffs of the Columbia River Basalt Group. During its most active period a little over one kilometer (0.75 miles) of the I-84 freeway was involved with its movement. The slide mass is approximately 19 million cubic meters (25.5 million cubic yards) of talus with interlayers of mudstone.

PURPOSE

The purpose of this study is to understand those factors which influenced the development and continuing movement of the Fountain Landslide and to examine from an

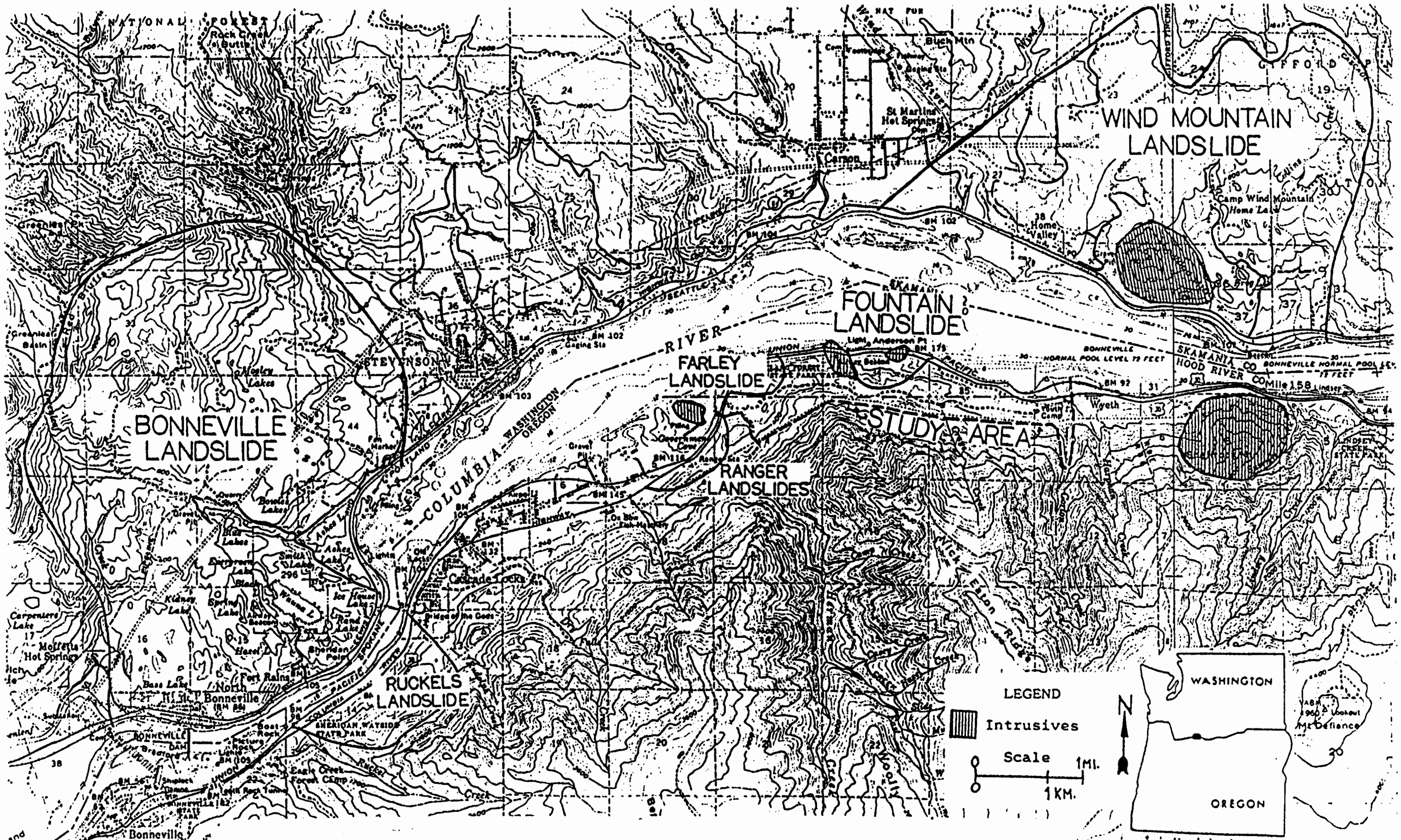


Figure 1. Location map. From USGS 15' Quadrangle, Bonneville Dam & Hood River, 1957

engineering geological standpoint why past interpretations have not sufficiently explained the Fountain Landslide.

One interpretation proposed in 1973 by A.C. Waters likens this area to landslides along portions of the Panama Canal, where a thick, heavy pile of competent, pervious material (Columbia River Basalt Group); overlies a weaker layer (Eagle Creek). When lateral support is lost the weak unit squeezes out.

The Army Corps of Engineers (1971a) in investigations of other landslides along the Oregon shore of the Bonneville reservoir, found that the impervious Eagle Creek formation perches the ground water table. Water runs through the pervious talus above, then flows along the contact, creating high pore water pressures.

The Oregon State Highway Division's preliminary analysis of the Fountain Landslide was that it was a planar slide with an elevated ground water table. Why this landslide has not been stabilized after three decades of research and various stabilization attempts, when other landslides along I-84 in this area have been, still poses an interesting geologic question.

SCOPE

The area included in the study is shown in Figure 2. It is bounded by Gorton Creek to the east, the fire lane to the west, the Columbia River Basalt Group cliffs to the

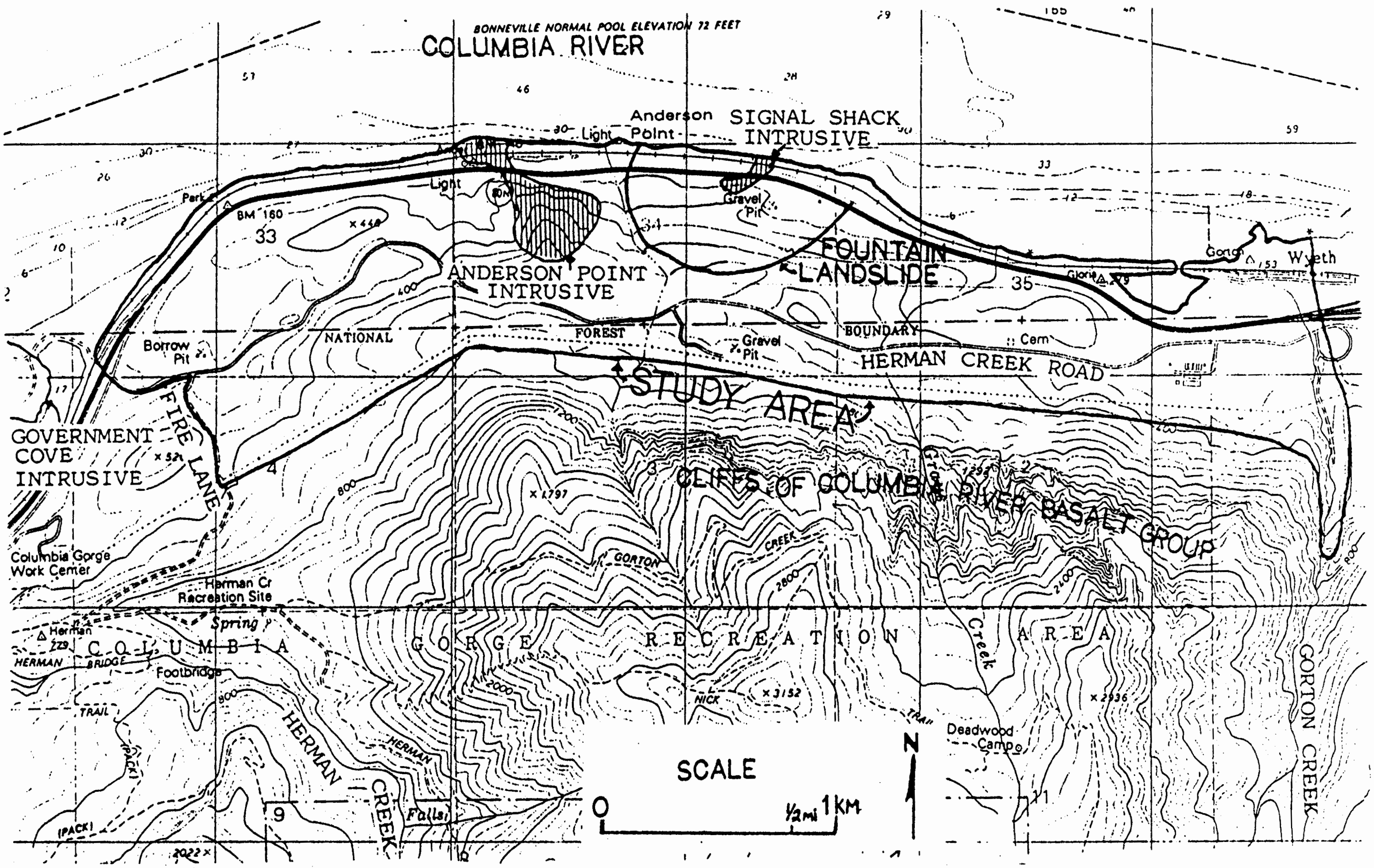


Figure 2. Study Area boundaries. Base map USGS Carson, WA 7½' Quadrangle, 1979.

south and the Columbia River to the north. An in-depth search of Oregon State Highway Division reports and data files on the Fountain Landslide was made. Basic field geologic techniques were used to establish the stratigraphic section and an engineering geologic map was prepared for the study area. Aerial photographic analysis was used to interpret the geomorphology and extent of movement. A survey grid was set up to describe the current surface movement.

A groundwater analysis was done to determine its potential effect on the landslide. This included: 1) locating the streams and springs in the area, 2) recording flow rates, and 3) the use of preliminary Oregon State Highway Division well data for ground water interpretations.

Three cross sectional computer models were made using the Modified Bishop Method in order to describe the factor of safety for the landslide, analyze past stabilization procedures, and examine the effect of three standard types of stabilization.

PREVIOUS WORK

The Fountain Landslide has been studied extensively by the Oregon State Highway Division because of its damage to the I-84 freeway. The details of their research and remedial engineering will be presented in the "Recorded

History" portion of this report. The geology and stratigraphy of this area has been studied by many, beginning in 1874 with the reconnaissance geology of Oregon by Condon, and has included Barnes and Butler (1930), Allen (1932), Hodge (1938), Wise (1970), Waters (1973), Hammond (1980) and Tolan and Beeson (1984). Free (1976) studied the stratigraphic relationships and geochemistry of the diorite intrusions in this region including Wind Mountain, Shellrock Mountain, Government Cove and the intrusions within the study area; Anderson Point and Signal Shack intrusions (see Figure 1). Palmer (1977) described many landslides in the Columbia River Gorge, including the Fountain Landslide. The Army Corps of Engineers (1971a) also reviewed the landslides affecting the shores of the Bonneville Reservoir which includes the Fountain Landslide.

GEOLOGIC SETTING

The Columbia River Gorge was formed by the down cutting of the Columbia River through the Cascade Range. Its canyon walls expose up to 1,200 meters of rock. Figure 3 shows a generalized stratigraphic section for this area (Wise, 1970). The oldest rock exposed in this region consists of a volcaniclastic unit of Eocene to Oligocene age. Wise (1970) identified this unit in the Wind River, Washington area (north of the study area) as the Ohanapecosh Formation. Although there have been no studies

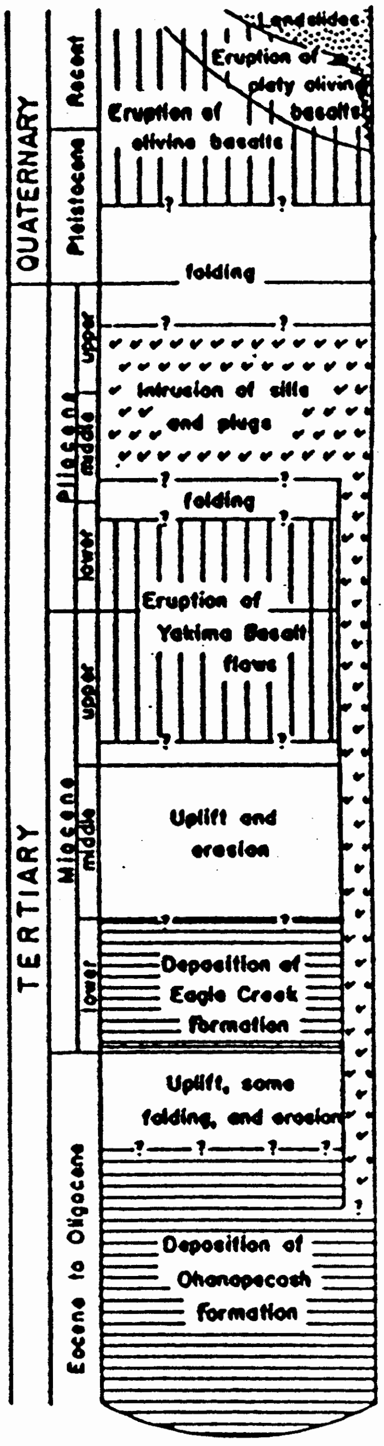


Figure 3. Generalized stratigraphic section for the region. (from Wise, 1970).

to extend this formation south of the Columbia River, the volcaniclastic unit of this age exposed in Oregon has generally been accepted as being the Ohanapecosh Formation (Beaulieu, 1977). When the Ohanapecosh Formation was being deposited andesitic volcanoes were common and a thick sequence of volcanic and sedimentary derived subaqueous shales, claystones, and pebble conglomerates accumulated in submerged basins. Rocks from the Ohanapecosh Formation are highly weathered to the point that matrix and rock fragments often break together. Zeolites are also common in these rocks. Free (1976) included some debris from the Fountain Landslide in his trace element analysis and concluded that some Ohanapecosh type material is included in the slide debris.

In late Oligocene time the ancestral Cascade Range began up-lifting and rising above water level (Wise, 1970). In early Miocene, volcanoes were still common in this region and their debris was deposited subaerially as epiclastic deposits. The volcaniclastic unit produced in this episode is younger than the Ohanapecosh Formation. It is named the Eagle Creek Formation (Chaney, 1918) in the Columbia River Gorge. The Eagle Creek Formation is much like the Ohanapecosh Formation but is characterized as coarser grained, and less weathered and altered. An outcrop of the Eagle Creek Formation has not been found within the study area but is found 3 kilometers to the west at Herman Creek (Free, 1976) (see Figure 1).

As uplift continued in the Miocene a low gap was located in the Northern Cascades region of Oregon (Tolan and Beeson, 1984). It is through this trough that the flood basalts of the Columbia River Basalt Group flowed westward, from mid Miocene through early Pliocene time. Flows of the Columbia River Basalt Group lie immediately above the volcaniclastic units of the Eagle Creek Formation and the Ohanapecosh Formation and occur both as intracanyon and flood basalt flows (Tolan and Beeson, 1984).

Volcanism continued in the Cascade Range and approximately six million years ago diorite intrusions and sills became emplaced along the axis of the Cascades. Radiometric dates on Wind Mountain and Shellrock Mountain (see Figure 1), two of the intrusions found in this part of the Columbia River Gorge, are 6.6 +/-0.7 and 5.7 +/-0.6 m.y.b.p. respectively (Tolan, 1985). Free (1976) found Columbia River basalt assimilation in the Wind and Shellrock Mountain intrusions which also dates the intrusions as younger than the basalt of the Columbia River Basalt Group. Intrusions crop out at two locations within the study area, the Anderson Point and the Signal Shack intrusives (see Figure 2). After the intrusives were emplaced the Cascade Range began uplifting.

In the Quaternary, volcanism remained common in this region forming large stratovolcanoes and smaller basalt volcanoes (Wise, 1970). Olivine basalts from shield volcanoes, in this region often termed High Cascade Lavas, occur as small flows and intracanyon flows. At least one of these intracanyon flows originating from Trout Creek Hill (Wise, 1970) nineteen kilometers (12 miles) north of the study area in Washington, crossed and dammed the Columbia River. From a single radiometric date on one Trout Creek Hill Lava the age of the High Cascade Lava found in Oregon is 338,000 years ago +/- 75,000 (Berri and Korosec, 1983). Stratigraphically, these lavas lie above the intrusive rocks and above the Columbia River Basalt Group talus.

Gigantic floods during the Pleistocene glaciation called the Bretz Floods, poured through the Columbia River Gorge. These floods occurred between 15,000 to 12,800 years ago (Allen, 1979). The flood waters reached a maximum elevation of 144 meters (800 feet) in the study area (Allen, 1979). Besides extensively eroding the Gorge walls, the floods locally deposited sands and some gravels.

The most recent deposits in the Columbia River Gorge are landslide debris, fluvial deposits, High Cascade Lavas and talus from the basalt cliffs. A very large landslide approximately 9.5 kilometers west of the study area in Washington occurred approximately 700 years ago, damming the Columbia River (Lawrence and Lawrence, 1958). This is the origin of the Bridge of the Gods legend and it is called the Bonneville Landslide (see Figure 1).

STRUCTURAL SETTING

The neotectonic map for this region by Bela (1982) shown in Figure 4, shows the structural trends. The folding in this region generally trends northeast, thrust faults trend east-west and the strike-slip faults trend northwest (Beaulieu, 1982). There is a regional southerly dip of 5 to 20 degrees.

Figure 4. Regional neotectonic map, from Bela, 1982.

RECORDED HISTORY

There are two basic types of landslides in the Columbia River Gorge. On the Washington side, all are large stratigraphically controlled landslides (Palmer, 1977). The regional southerly dip elevates the incompetent saprolite contact between the Ohanapecosh and the Eagle Creek Formations north of the river. On top of this unit the heavy flows of Columbia River Basalt Group act as a driving force. Erosion removed the lateral support, creating ideal conditions for large-scale landsliding. The landslides occurring along the Oregon banks of the Columbia River Gorge, while being smaller, are the result of several different controlling forces. A description of the landslides occurring in the Columbia River Gorge can be found in Palmer (1977).

The recorded history of the Fountain Landslide began as most landslides do, with the first construction project on them. In the Columbia River Gorge the first construction project was the Union Pacific Railroad in the 1870's. The first highway was constructed in the 1920's.

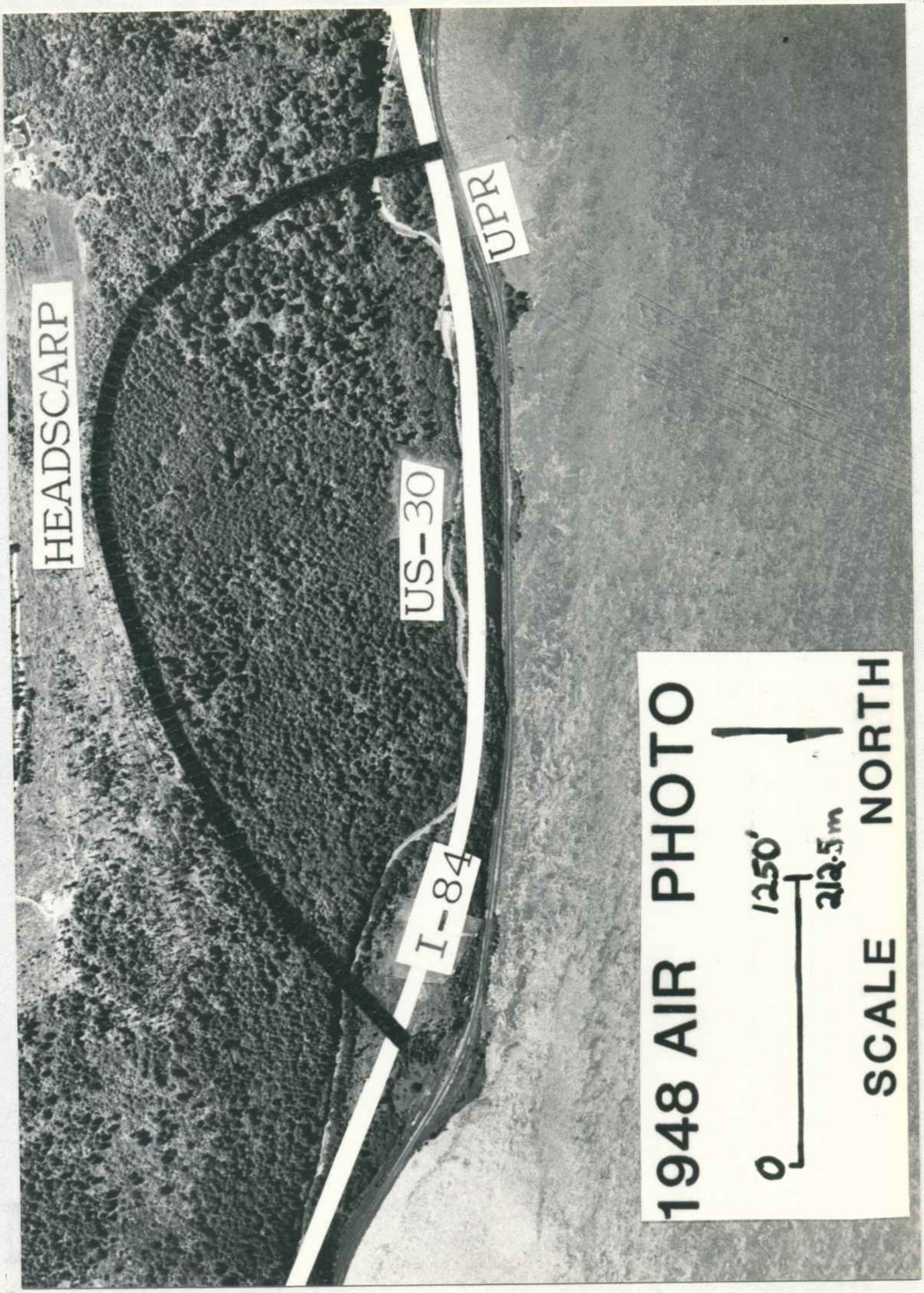
The Fountain Landslide has a detailed pictorial history due to its proximity to Bonneville Dam. The Army Corps of Engineers has regularly recorded the reservoir shores with aerial photographs beginning in the late

1930's.

Movement of the Union Pacific Railroad bed or of the highway by the Fountain Landslide did not begin with the first construction of the railroad or the highway. The Ruckels and Farley Landslides (see Figure 1) were more of a problem than the Fountain Landslide at this time. These two slides have since been stabilized.

The 1948 aerial photograph shown in Figure 5, shows no apparent movement of the highway or slope. The trace of an older headscarp can be seen. The present location of the I-84 freeway and the railroad are marked on the photo for reference.

According to Mr. Frank Dennis, the Union Pacific Railroad Engineering Inspector for this section of track from 1927 to 1971 (personal communication, 1983), there were no problems with track maintenance until 1952 when the Oregon State Highway Division started to realign US Highway 30. This construction cut into the hillside above the railroad tracks, into the toe of the Fountain Landslide and reactivated its movement. As excavation approached subgrade, the pavement raised 3.6 meters (12 feet) overnight (Johnsen, 1975). The railroad also experienced movement of the tracks at this time and the ballast was realigned at least once. The Union Pacific Railroad was awarded a contract for payment by the Oregon State Highway Division for all extra expenses for track maintenance and



<u>Figure 5.</u> 1948 aerial photograph. (U.S.C.O.E. 1"=1250')

repair. The amount eventually paid to the Union Pacific Railroad was estimated from limited data files at the Oregon State Highway Division to exceed $2000 and was probably much higher. Track movements were eventually stopped when the Oregon State Highway Division drove two sets of timber piling buttresses (personal communication Mr. Dennis, 1983), shown on Figure 6.

Other remedial measures taken at this time by the Oregon State Highway Division to stabilize the highway focused on drainage of the water from the slide mass. Dewatering was found to be effective in stabilizing other landslides occurring along US Highway 30 in the Columbia River Gorge such as the Ruckels Landslide (Palmer, 1977) (see Figure 1). In the early 1950's dewatering included digging test pits (large holes in the ground) to find the groundwater table, drilling shallow 15 to 30.5 meter (50 to 100 feet) horizontal drains and manually excavating one horizontal drainage tunnel in the toe. The progress and effectiveness of the excavation of the drainage tunnel were hindered because of the abundant water and slip planes found in the slide mass (Johnsen, 1975). Because the slide mass consists of talus which has almost no cohesive strength, running water and slip planes caused the walls of the tunnel to collapse.

Dye tests using a solution of fluorescein sodium were made in February of 1956 on two streams which disappear

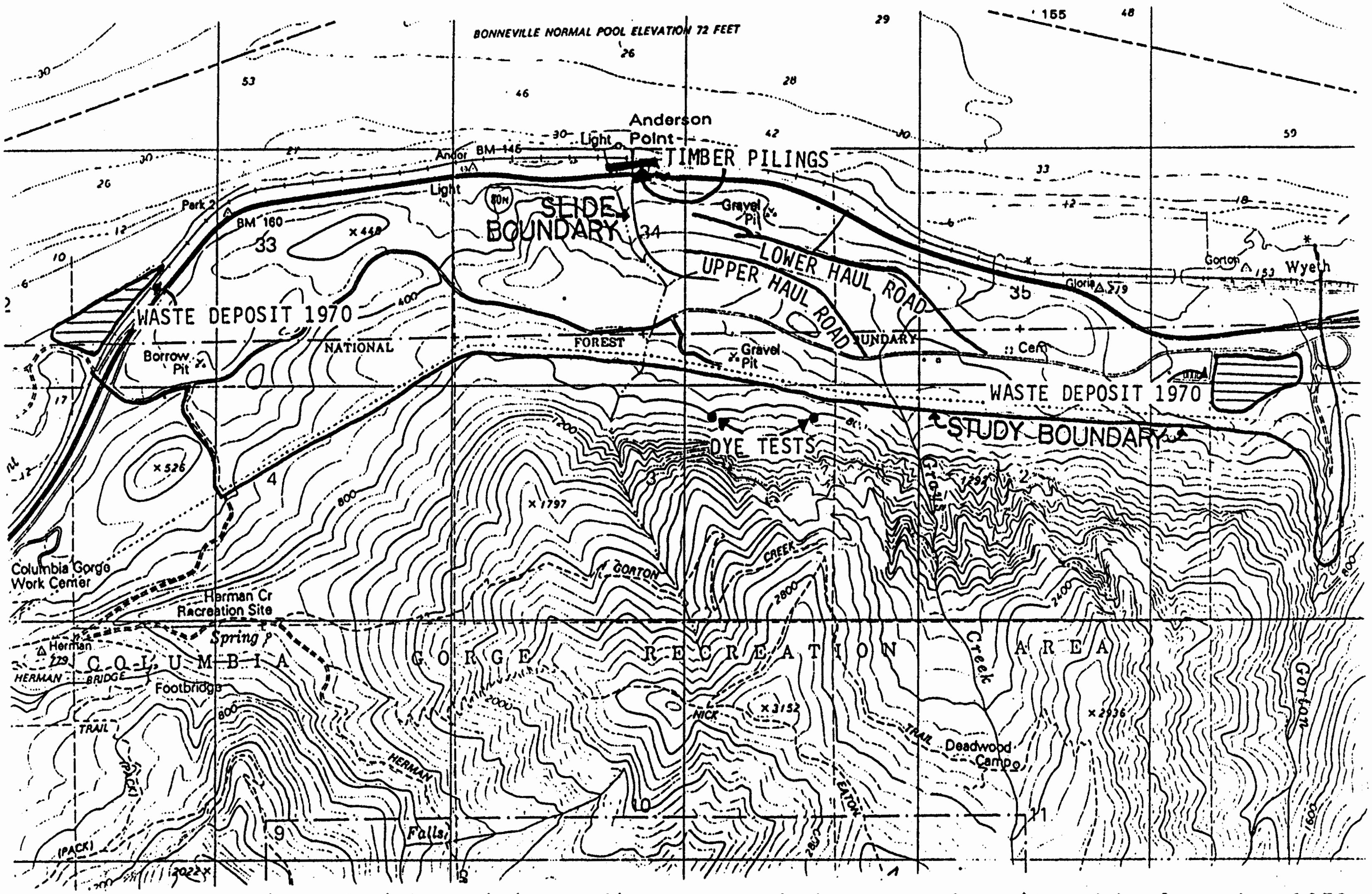


Figure 6. Location of timber pilings, die test, and the waste deposit and haul roads, 1970.

into the talus south of the landslide (see Figure 6 for test locations). No trace of the dye was ever found. The test results were inconclusive as to where this water traveled.

The 1957 aerial photograph shown in Figure 7, shows how US Highway 30 was straightened by the early 1950 construction. In this aerial photograph the different colors of asphalt and the soil exposed in the hillside above the highway express the extent of active movement.

For the next ten years there were no problems with railroad maintenance, but the highway experienced a minor amount of movement. This movement seemed inconsequential when, in 1965 construction began again across the Fountain Landslide. US Highway 30 was upgraded to interstate standards. This included widening it to the four-lane I-80N (later renamed I-84). A deeper cut into the toe of the landslide was necessary. It was during this construction period that the Fountain Landslide began accelerated movement, far beyond the capabilities of the Oregon State Highway Division to maintain it. At one point during construction the pavement was estimated to raise one to two meters (4 to 6 feet) per week for several weeks (Johnsen, 1975). The headscarp had moved much farther up than the previously delineated limits of the landslide. The Oregon State Highway Division contracted to have 264,000 cubic meters (350,000 cubic yards) of debris removed from the

Figure 7. 1957 aerial photograph. (U.S.A.C.O.E. 1"=883')

head of the landslide. The 1968 aerial photograph shown in Figure 8 shows the completed construction, the location of I-80N (I-84), the extent of the unloading, and the area where the debris was deposited immediately to the east, within the boundaries of the old headscarp.

The accelerated movement did not stop after the unloading, and in 1968 the Oregon State Highway Division initiated an extensive investigation of the Fountain Landslide. Initially ten slope inclinometers were installed to measure the depth, direction and amount of movement. The elevation of the ground water table was also monitored. Figure 9 shows the location of the drill holes, the direction of movement from September to December 1968, and the locations of the cracks. This data along with ground water table and slope inclinometer data were presented in a preliminary Oregon State Highway Division progress report dated March 19, 1969. The report showed with a series of graphs that: 1) The rate of movement was directly proportional to the amount of rainfall. Figure 10 combines two of their graphs presented in the progress report and typifies the relationship between rate of movement and amount of rainfall. 2) The ground water table in this area is erratic. Figure 11 shows that there appears to be no consistent relationship between the amount of rainfall in the area to the elevation of the ground water table. 3) The width of the slip plane is relatively

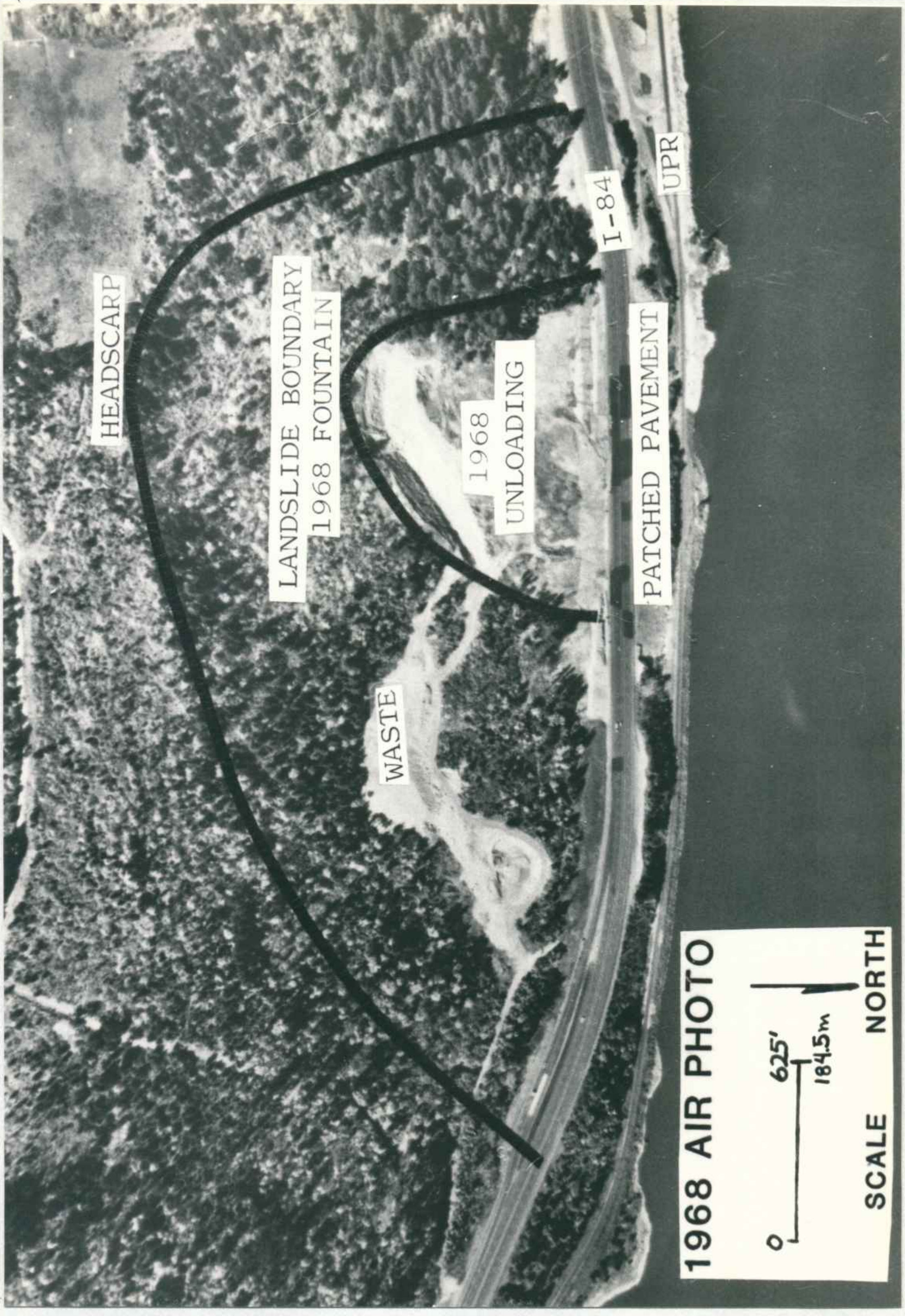


Figure 8. 1968 aerial photograph. (O.S.H.D. 1"=500')

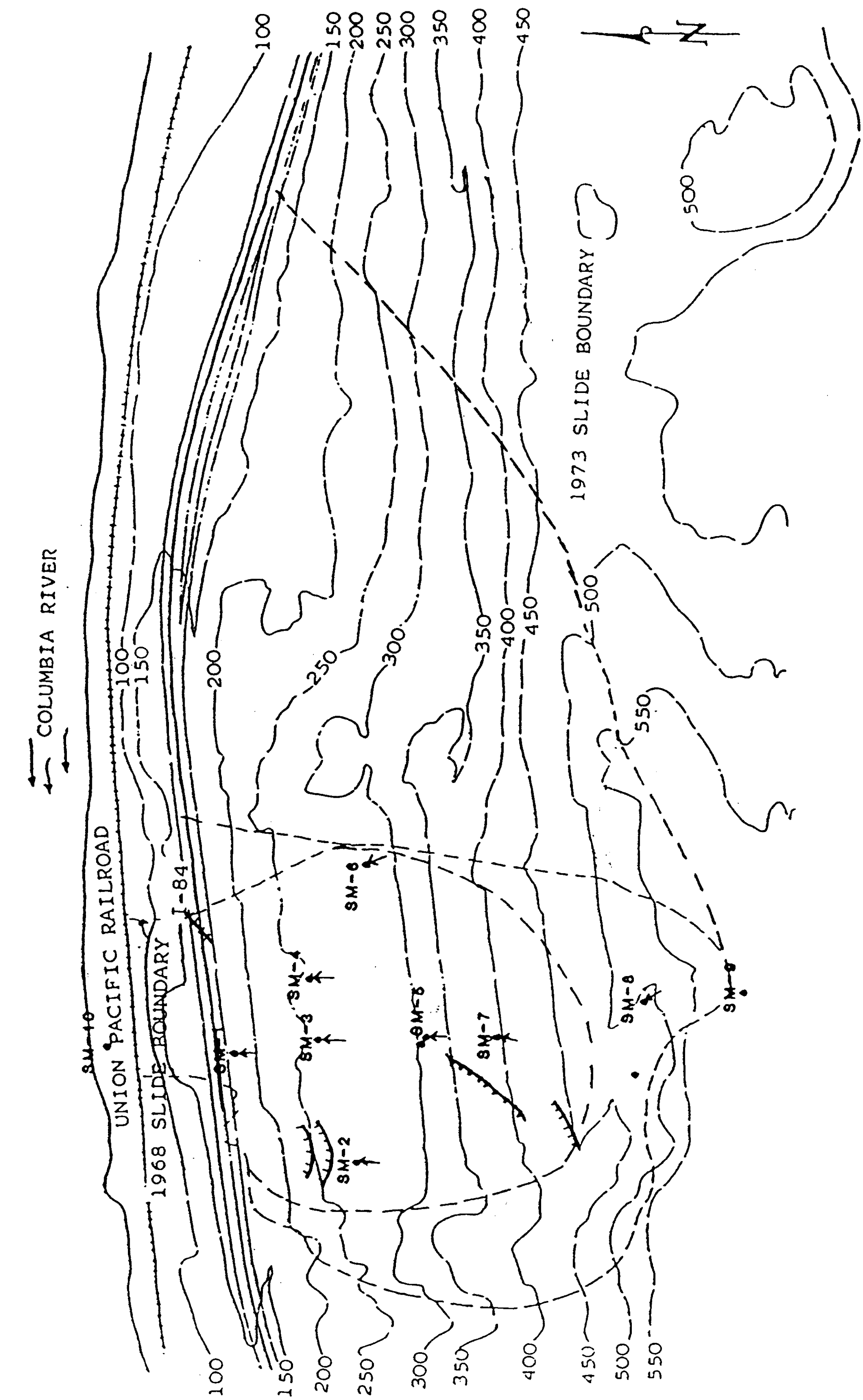


Figure 9. Locations of drill holes, direction of movement and cracks as of 1968.

narrow as shown in Figure 12. 4) The rate of movement was greater in the northern portions of the landslide. 5) Transit surveys along the highway and the railroad showed approximately 0.6 meters (2 feet) of horizontal movement of the highway and 0.45 meters (1.5 feet) for the railroad. The vertical movement was approximately 0.2 meters (0.68 feet) for the highway and 0.5 meters (1.56 feet) for the railroad.

From undefined laboratory determinations, the material within the shear plane had a cohesion value ranging from 1008 P.S.F. to 3124 P.S.F. and an angle of internal friction ranging from 5 to 9 degrees. Using this information a stability analysis was made by the Oregon State Highway Division. It was determined that 1.15 million cubic meters (1.53 million cubic yards) of material needed to be removed to gain a factor of safety of 1.38 (Gano, 1969).

The data presented in the March 19, 1969 progress report led the Oregon State Highway Division to take bids for the removal of 1.6 million cubic yards of landslide debris and any incidental freeway reconstruction. The contract was awarded to S. S. Mullins and was completed in the fall of 1970. The extent of the unloading can be seen in the 1971 aerial photograph shown in Figure 13 and in the cross section through the middle of the unloaded portion shown in Figure 14. The unloading left three terraces on

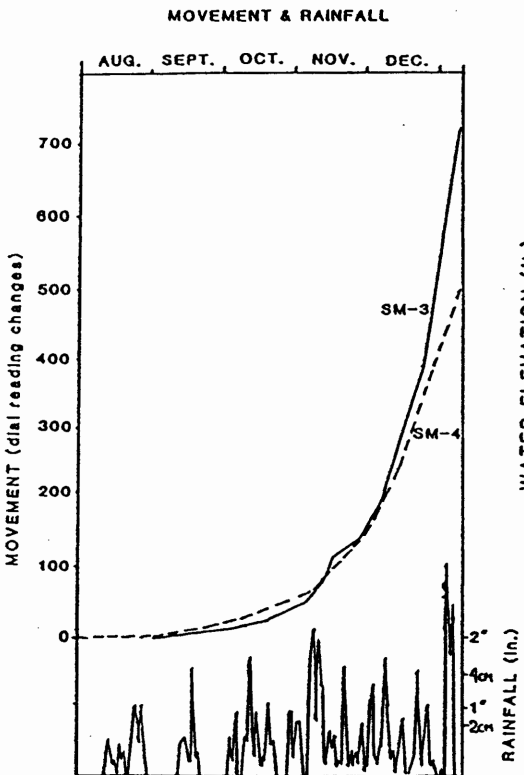


Figure 10. Movement and rainfall, (from Gano, 1969)

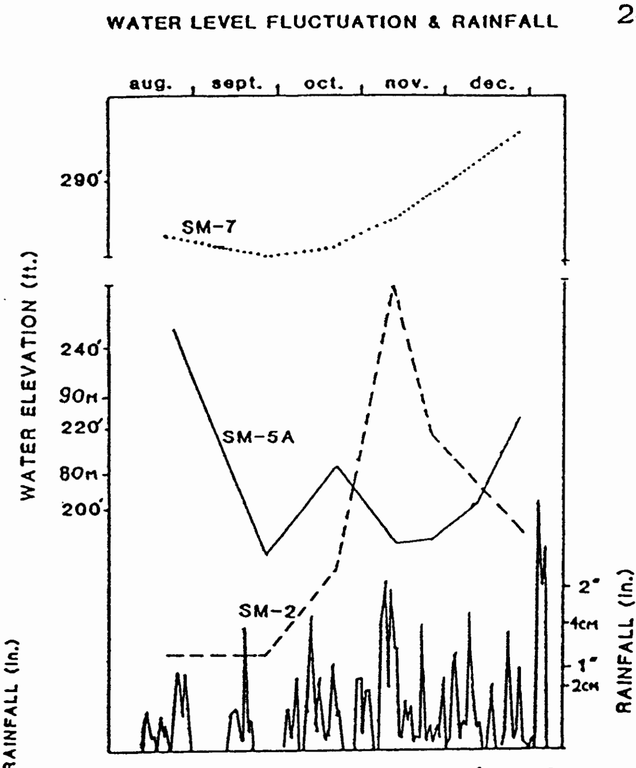


Figure 11. Changes in the ground water table and rainfall, (from Gano, 1969)

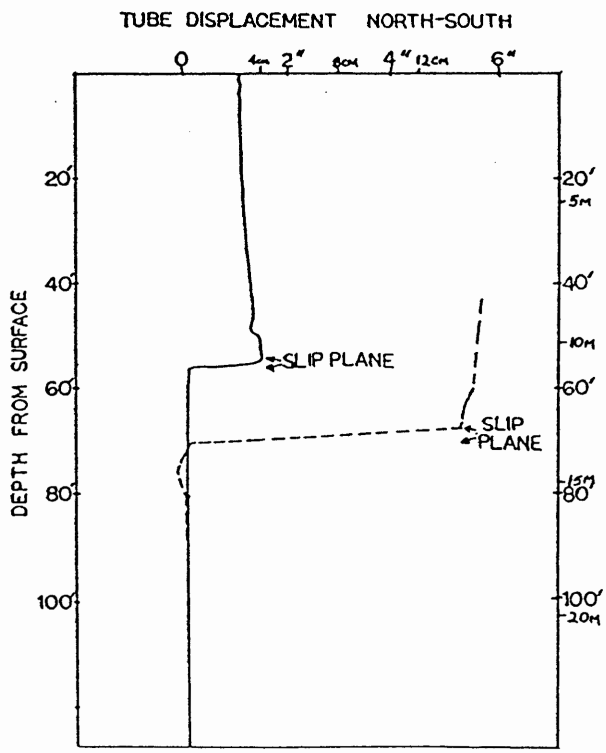


Figure 12. Width of slip plane, (from Gano, 1969)

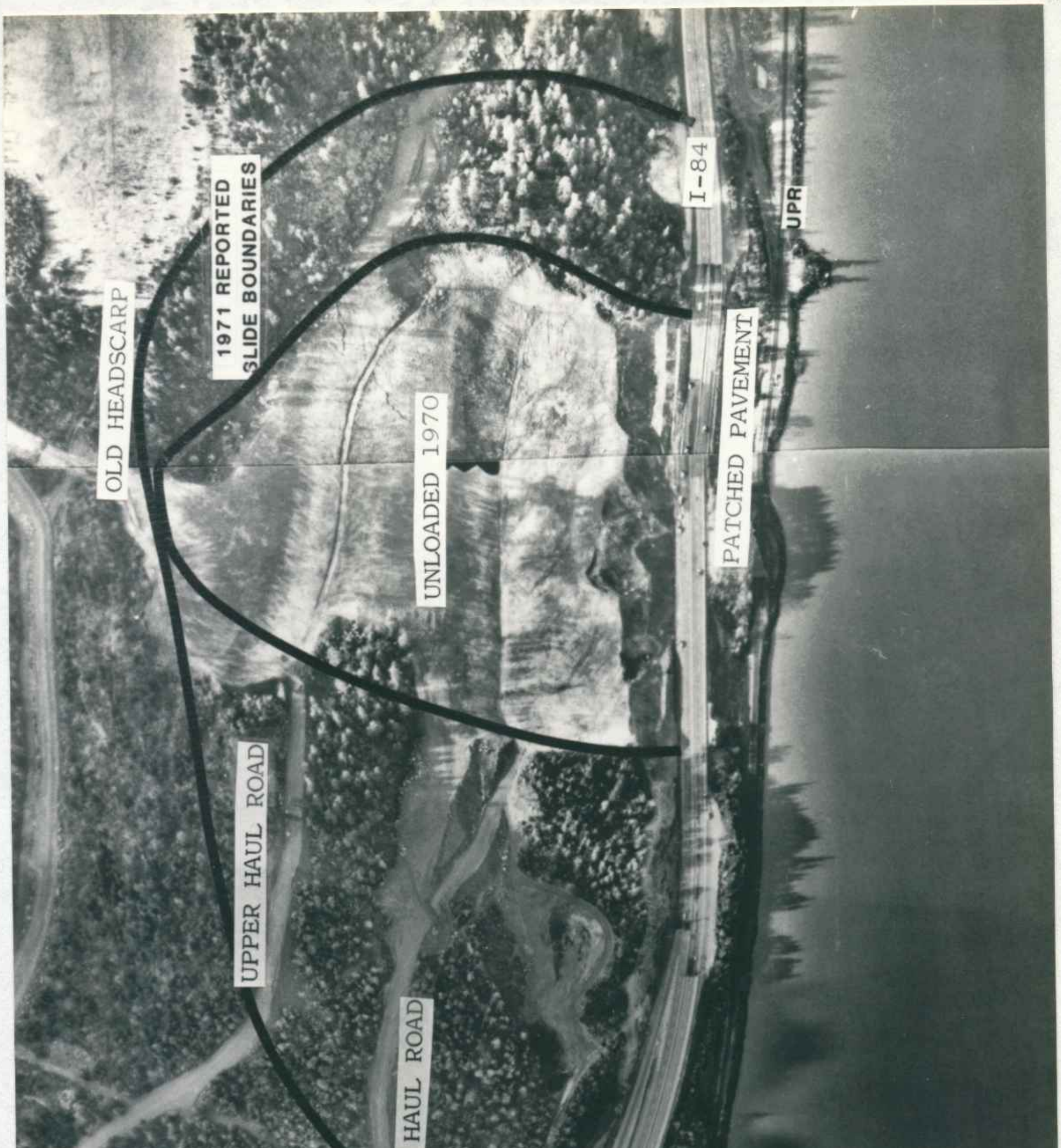


Figure 13. 1971 aerial photograph. (O.S.H.D. 1"=500')

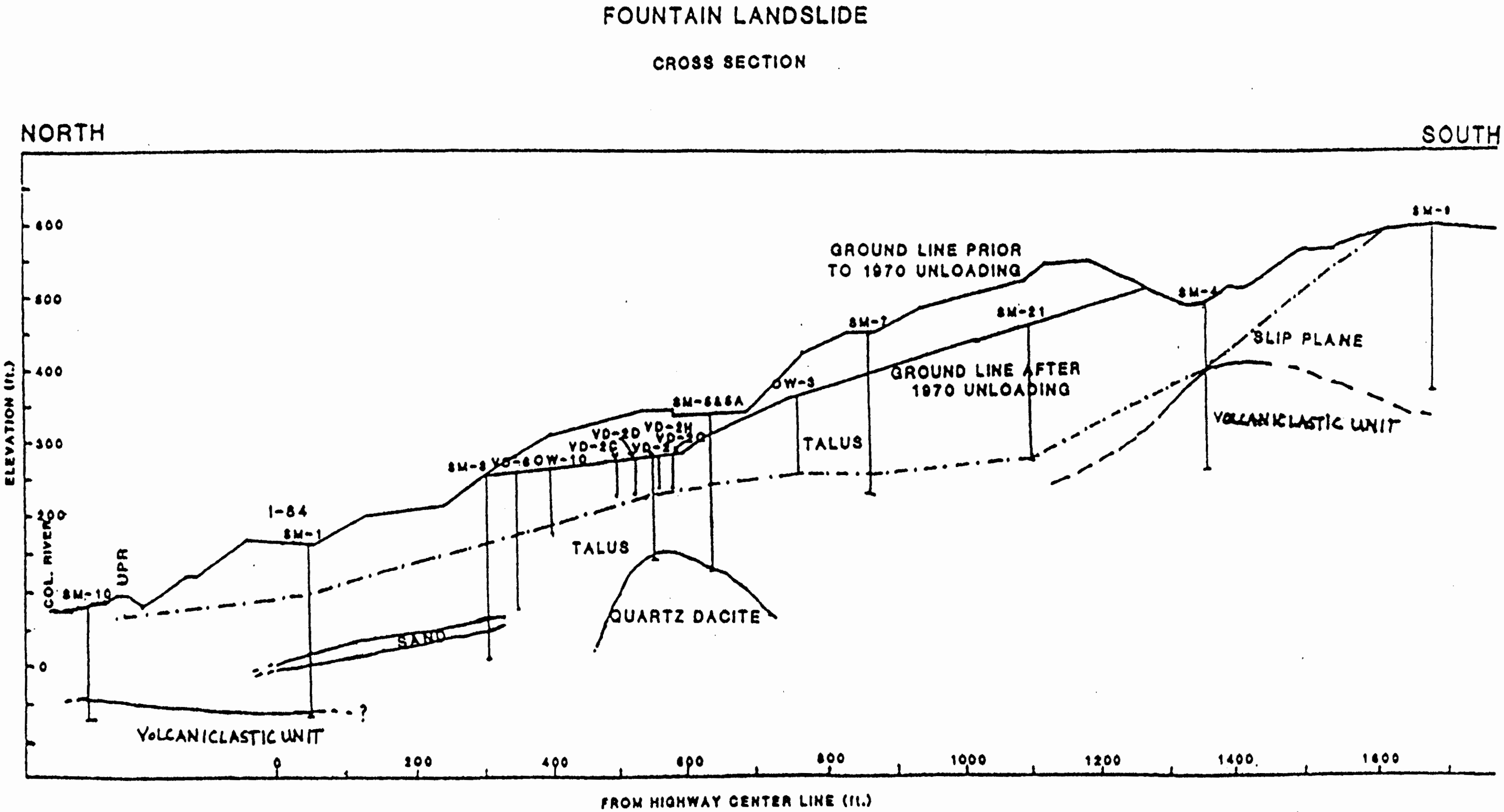


Figure 14. Generalized cross section for the western portion of the Fountain Landslide, from Johnsen, 1975.

the hillside. The waste was deposited in two locations off the landslide and two haul roads were constructed (see Figure 6 for their locations).

Upon completion of the unloading three more slope inclinometers were installed. These, along with the slope inclinometers still functional showed movement had stopped until November 1970. Movement began again along the highway in early December 1970 and continued until the spring. Severe raising of the highway along the western margin of the landslide was noticed. The movement was at a slower rate, approximately 0.3 meters (1 feet) per year, instead of 0.6 meters (2 feet) per year the previous winter. It was also found that the actively moving area enlarged as shown in Figure 15. Springs were observed during the winter near the back of the lower terrace also shown in Figure 15. The fact that movement stopped for a period of time (October to December 1970) indicated that the unloading favorably affected the landslide, but that possibly more remedial work was needed (Gano, 1971a). In the Oregon State Highway Division progress report of January 29, 1971 a suggestion was made to study the ground water condition and if possible drain trapped water.

The Union Pacific Railroad had few if any track adjustments due to slide activity after the 1970 unloading. The Union Pacific Railroad contract with the Oregon State Highway Division had expired by this time. It is assumed

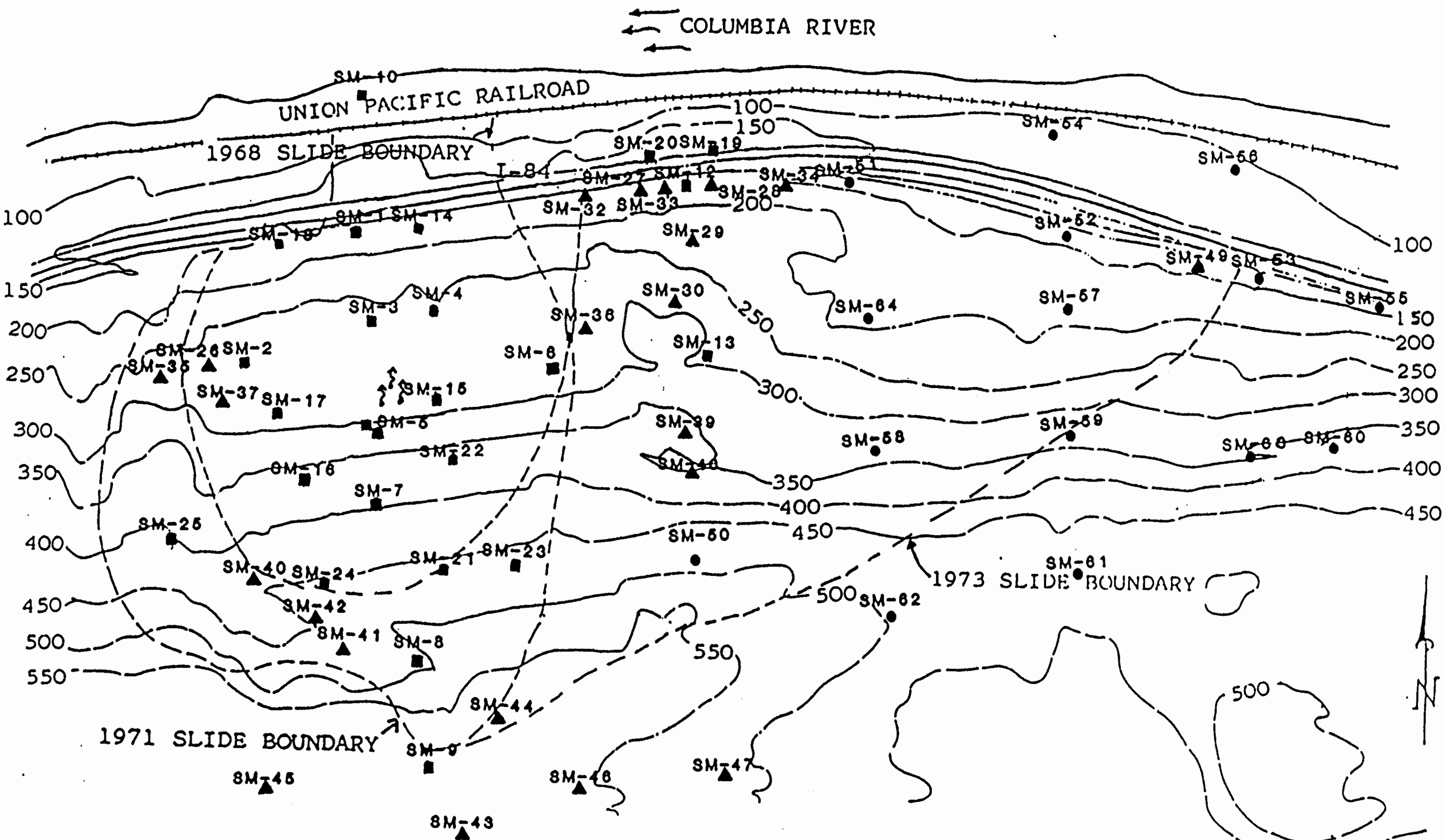


Figure 15. Drill hole locations. Those installed before 1971 - ■. 1971-1972 - ▲. 1972-1973-●.

they had no further track maintenance problems since they did not initiate proceedings to extend its term.

Many of the slope inclinometers had been sheared off or made unserviceable by the continued movement. In order to continue the research on the subsurface conditions, eleven more slope inclinometers were installed during the 1971 construction season. Their locations are shown in Figure 15.

Since previously installed horizontal drains gave poor results, four vertical drain wells were installed and tested in June, 1971. Sixteen observation wells were placed 7.6 and 15.2 meters from each vertical drain to monitor the results (Figure 16). The purpose of the vertical drains were to drain the perched ground water below the confining layer. The results of the four vertical drains were reported in a progress report of July 6, 1971 (Gano, 1971b). The results using pump and bail tests were:

V.D. 1 - Bail test - 1 gpm
V.D. 2 - Pump test - 45 gpm
V.D. 3 - Bail test - 1 gpm
V.D. 4 - Bail test - 1 gpm

Only vertical drain 2 gave satisfactory results. The other wells were in areas much too impermeable for the drains to function effectively. The one favorable result led to the installation of seven more vertical drains and thirty-two observation wells (Figure 16). The results were presented in the March 10, 1972 progress report. Only

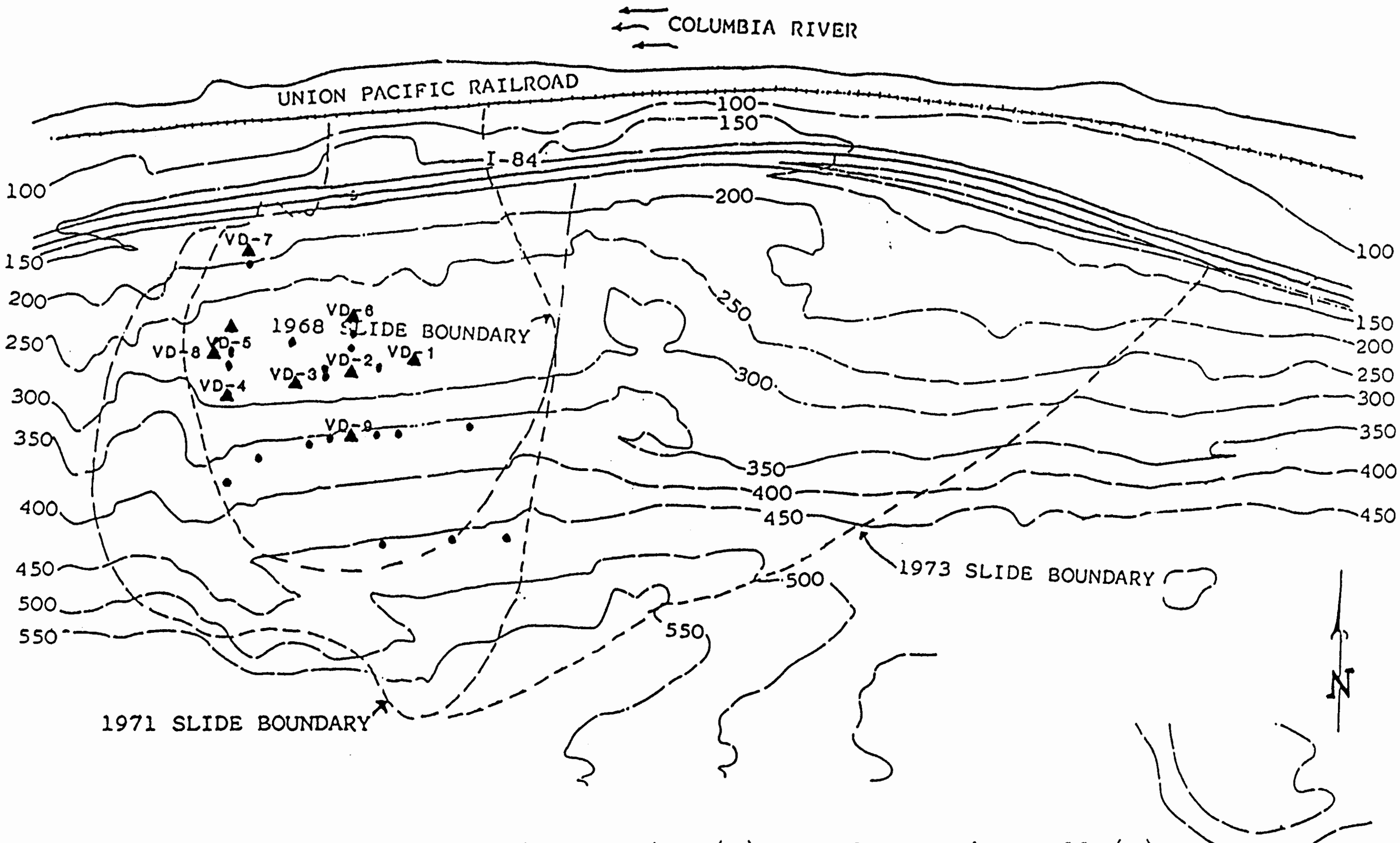


Figure 16. Location of vertical drains (▲) and observation wells(•).

vertical drain 5 and vertical drain 2 functioned effectively with 52 gpm and 45 gpm respectively. These generally poor results caused this approach to be abandoned.

The length of highway affected by slide movement enlarged to the east beginning in January 1971 (Gano, 1971a). After examining the area the Oregon State Highway Division found the boundary to be enlarged to the south and the east (see Figure 15). The enlarged boundaries were defined by the presence of 0.3 to 0.65 meter (1 to 2 foot) scarps across the haul roads constructed for the unloading process. A photograph of the the scarp on the upper haul road is shown in Figure 17.

Twenty-four more slope inclinometers were installed in 1972 and fourteen more were installed in 1973 (see Figure 15). A total of sixty-four slope inclinometers were installed in the Fountain Landslide. The information obtained from the last fourteen slope inclinometers was reported in an Oregon State Highway Division progress report in late 1973. The amount of movement at these locations was no more than could be attributed to measurement errors. However, movement continued on the highway.

To divert water from entering the landslide above the headscarp a culvert was constructed over the landslide. The location of the culvert is shown on the 1975 aerial

Figure 17. Photograph of scarp in upper haul road. (1971) (from OSHD files)

photograph (Figure 18). Many alternatives for stabilizing the movement were examined by the Oregon State Highway Division from 1972 to 1975. The alternatives included: 1) ion exchange (Scott, 1973), 2) continued unloading of the head (Gano, 1972), 3) excavating a drainage trench tangential to the direction of movement (Gano, 1972), 4) construction of a rock buttress (Gano, 1972), and 5) realignment of the freeway either around the toe of the landslide into the Columbia River or above the headscarp (Gano, 1975). None of these alternatives were used. The Oregon State Highway Division decided to continue maintenance of I-84 as needed.

A warning system was installed in 1975 to provide for motorist safety. The system included settling tubes and tensometers installed below the surface of the highway monitored by computers. The computer could activate changes in the three part warning sign shown in Figure 19, and notify the authorities. This system was never operational due to many system failures.

Since 1975 the Oregon State Highway Division has had to annually maintain the highway across the Fountain Landslide. When the foreman for that section of I-84 finds the condition of the freeway becoming unsafe he initiates the needed repairs. According to Mr. Harry Woodward, the Oregon State Highway Division Region 2C District Maintenance Supervisor, (personal communication, 1984)

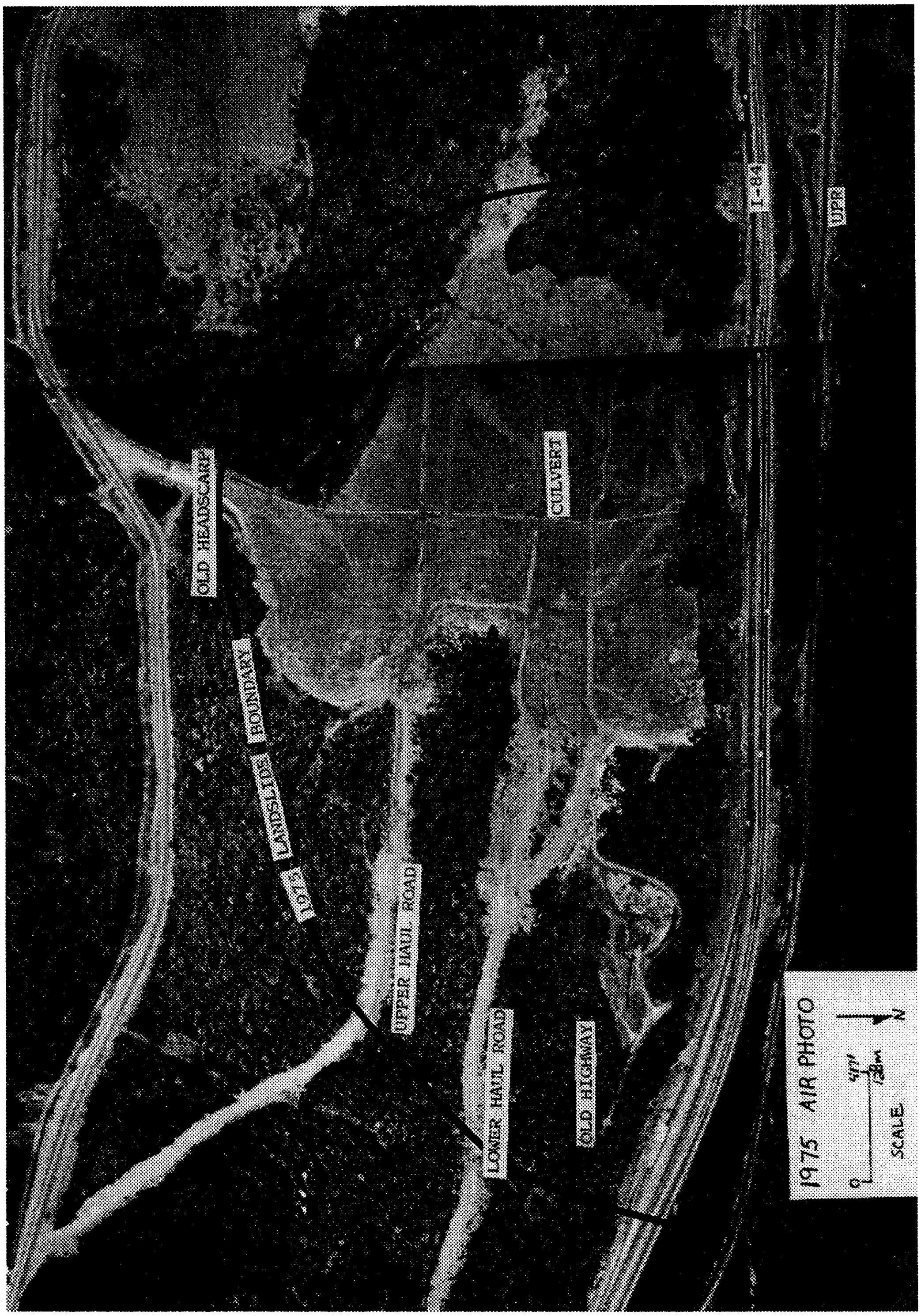


Figure 18. 1975 aerial photograph. (USACOE 1:5000)

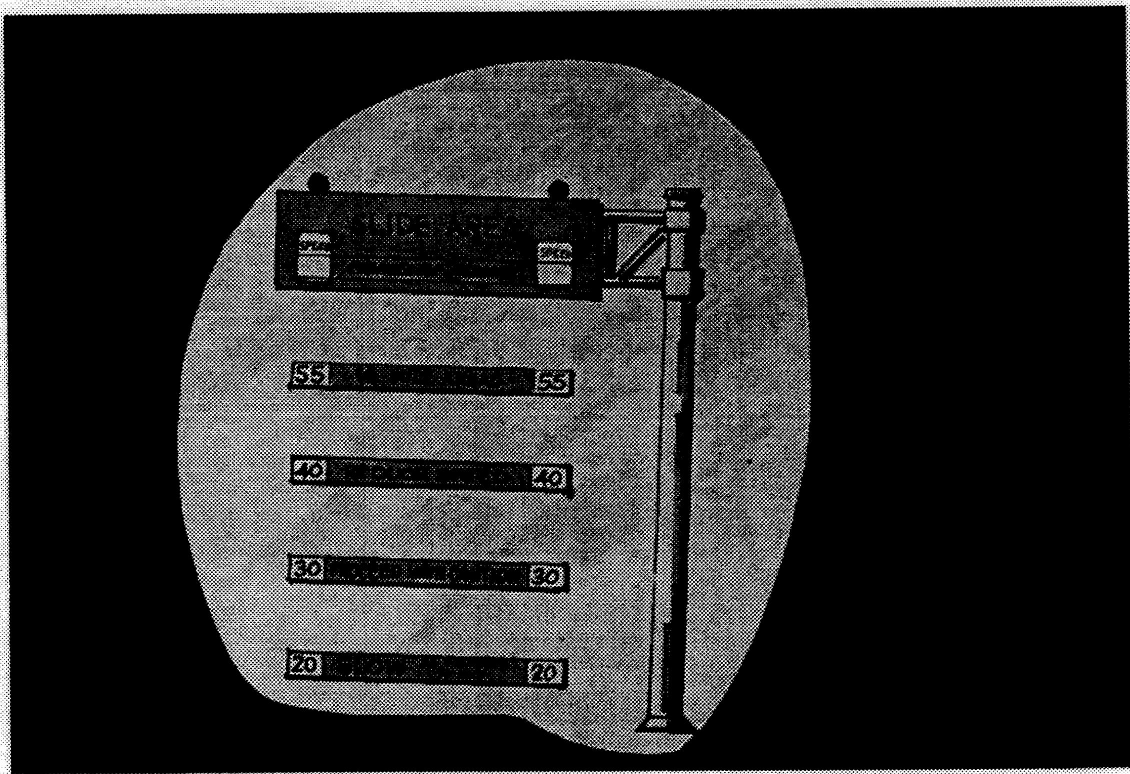


Figure 19. Drawing of the warning sign installed. The computer activated alternate messages shown below it. (from OSHD files)

nearly every spring some work is needed to repair the highway along the western portion of the Fountain Landslide. Generally the type of repairs consists of planing the highway surface and/or addition of an average of 350 tons of asphalt. Other maintenance needed periodically due to movement includes realigning and replacement of the center guard rail and straightening of the light posts.

In 1984 the metal guard rail on I-84 along the Fountain Landslide was replaced with a cement guard rail, except for a 100 meter section. This 100 meter section is located along the western portion of the landslide where the highway movement is still too great for emplacement of a cement guard rail.

GEOLOGY

The most important part of any landslide study is understanding the local geology. It is integral to interpretation of the cause and defining the type of landslide. Presentation of the studies completed for this report will begin with those that aid in the interpretation of the geology. These include: 1) an aerial photographic interpretation of the geomorphology, 2) description of the geologic units including their distribution and stratigraphic relationships, and 3) description of the structural features.

PROCEDURES

Field studies were made during the summer and fall of 1983 and in the spring of 1985. Basic field methods and sampling procedures were used to make an engineering geological map and stratigraphic section of the study area. A Fluxgate Magnetometer was used to determine remanent magnetic polarity in the basalt of the Columbia River Basalt Group. Preliminary Oregon State Highway Division reports, core logs and data files of the Fountain Landslide were also reviewed.

AERIAL PHOTOGRAPHY

An historical description of the aerial photography for this area, beginning in 1932, has already been presented in the "Recorded History" section. The geomorphology will be discussed here. The most noticeable feature seen in all the aerial photographs is a prominent headscarp. It traces the boundary of the Fountain Landslide in all the aerial photographs. This indicates the Fountain Landslide was a preexisting slide predating construction in this area.

The relief is extreme in the Columbia River Gorge in this area. Figure 20 shows a generalized cross section for this area. Elevations range from 770 meters (2500 feet) at the top of the Columbia River Basalt Group cliffs to 23 meters (75 feet) at river level. The basalt cliffs are 457 meters (1500 feet) high. Below the cliffs is a talus slope of basalt boulders. The talus slope begins at roughly 300 meters (1000 feet) and continues down to 150 meters (500 feet) at an angle of repose of approximately 29 degrees. Between the talus and the river there is a small hill resulting in a valley or saddle. The Fountain Landslide is located on the river side of the small hill. To the west of the landslide the surface is hummocky and is composed mostly of boulders. To the east of the landslide the ground surface is more rounded and smooth. A smaller landslide is seen just west of Anderson Point (Figure 21)

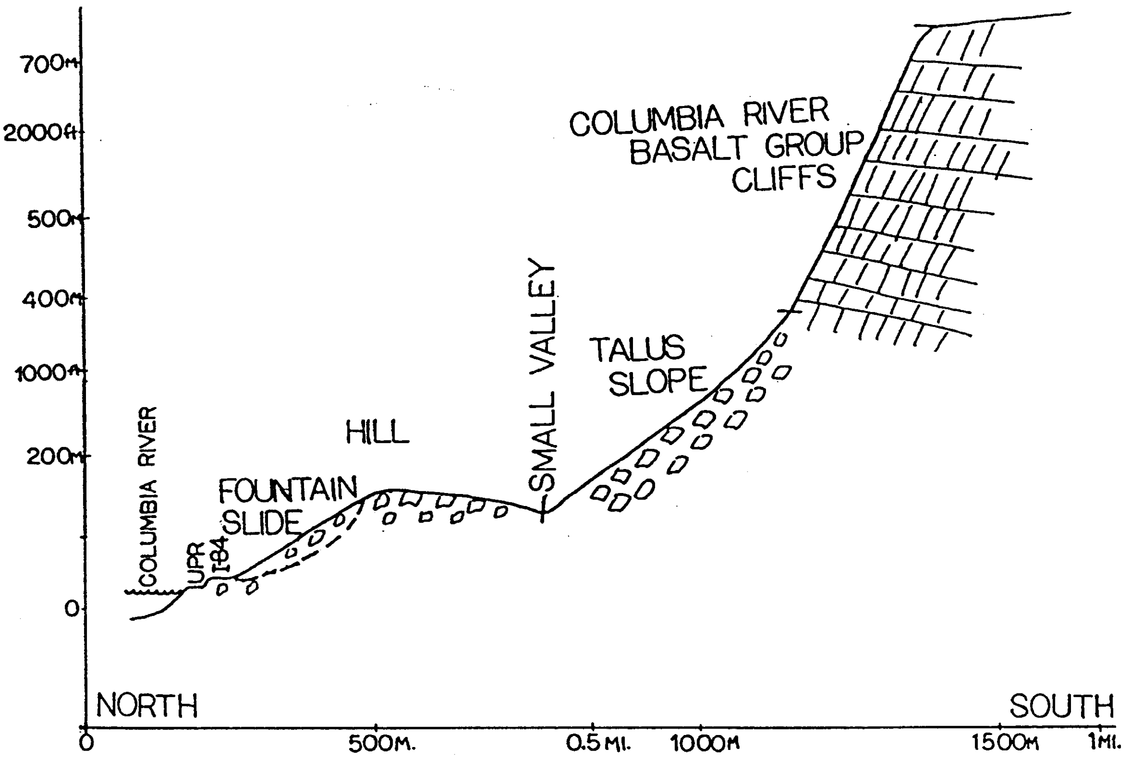


Figure 20. Generalized cross section of the topography.

and the Farley Landslide is included in the study area. Erosionally resistant highlands are found at two location within the study area, Anderson Point and 1.2 kilometers (0.75 miles) east of it along the Union Pacific Railroad. All the drainage within the study area is northward, into the Columbia River. All the streams are intermittent, flowing only in the spring and fall. In two instances the streams flow into the talus.

The geologic map for the study area is shown in Figure 21. The road accessibility is very good, but off the roads and trails accessibility is difficult due to the heavy vegetation of deciduous trees and brush. The heavy vegetation also reduced the number and condition of the outcrops. The units found within the study area include basalts of the Columbia River Basalt Group, quartz diorite intrusive, Wind River Lava, Columbia River Basalt Group talus, glacial flood deposits, alluvium, landslide deposits, and manmade fill. The distribution of these units are shown in Figure 21. Talus covers most of the ground surface within the study area.

STRATIGRAPHY

The geologic units within the study area include: (1) a volcaniclastic unit (found only at depth in core logs), (2) the Grande Ronde Basalt of the Columbia River Basalt Group (CRB), (3) quartz diorite intrusions, (4) Wind

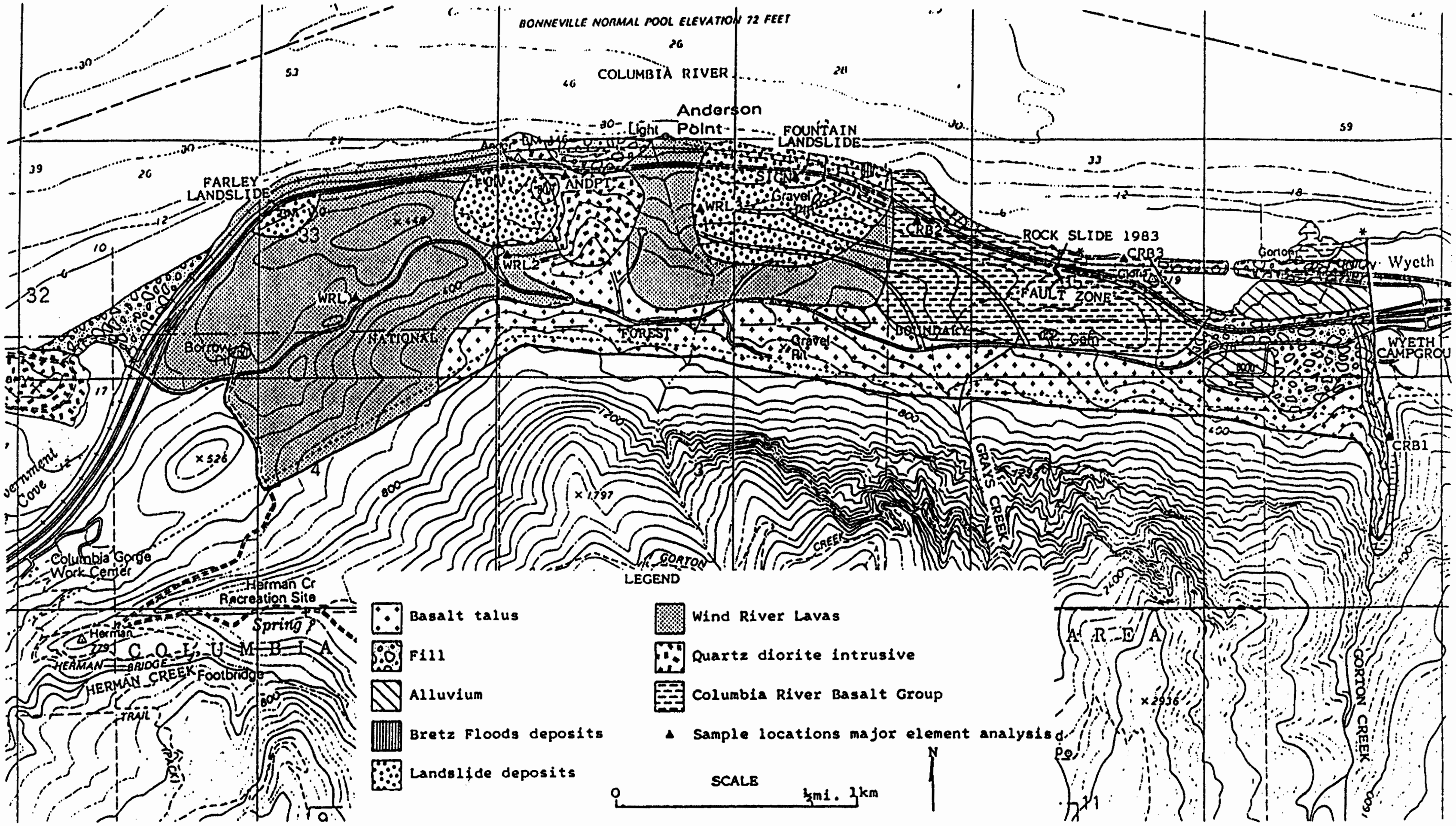


Figure 21. Geologic map of the study area. From USGS 7½' Quadrangle Carson, WA.

River Lavas (WRL), (5) Bretz Flood deposits, and (6) alluvium, landslide debris and fill.

One hundred and eight drill holes were made on or near the Fountain Landslide by the Oregon State Highway Division. These include those used for slope meters, vertical drains, and observation wells. All of the drilling was logged by the Oregon State Highway Division. Appendix A shows the logs for all the slope inclinometers. Since the Oregon State Highway Division study was done for engineering purposes, the core logs only delineate changes in engineering materials. Descriptions of these materials were generally sufficient to determine which geologic unit was present. The core samples from some of the drill holes were still available at the Oregon State Highway Division in 1983. Detailed logs of three drill holes are given in Table I.

Volcaniclastic Unit

The volcaniclastic unit is described in Oregon State Highway Division core logs as volcanic tuffs, shales, sandstones and some interbedded gravels. From samples seen in SM23 and SM24 (Table I) located near the head of the landslide (see Figure 15) the volcaniclastic unit consists of sub-angular sands and conglomerates with some siltstones and organic material. Many shear planes were seen throughout the core samples of the volcaniclastic unit especially in bedding planes. The samples seen were well

TABLE I

THREE LOGS MADE FROM AVAILABLE CORE AT OSHD FEBRUARY 3,1983.
SEE APPENDIX A FOR OSHD PRELIMINARY LOGS.
SEE FIGURE 15 FOR DRILL HOLE LOCATIONS.

SM23
ELEV. 462 FT

DEPTH (in ft.)	
0 - 102	Pink Wind River Lava Talus
102-103	CRB Talus
103-138	Pink WRL Talus
138-147	CRB Talus
-------	Not Avail.
180-181	Grey-green Breccia friable
181-184	Grey-green Breccia cemented
184-190	Tan Sandy Conglom.
190-191	Vol. Congl.
191-192	Sandstone
192-193	Slip plane Siltstone w/ brecciated rock frag.
193-196	Very fine grained organic mudstone
196-198	Fragmental organic mudstone
198-200	Volcanic silty sandstone w/ organic layers

SM24
ELEV. 460 FT

DEPTH (in ft.)	
50 -70	Pink WRL Talus
70 -71	Pebbly Mudstone
71 -72	CRB Talus
72 -90	WRL Talus
90 -106	CRB Talus
106-107	Tan Clay was soft when wet
107-112	Grey-green Breccia
112-113	Siltstone Slip planes
113-118	Volc. Congl.
118-120	Coarse sand w/ WRL frag.
120-126	Very Fine grained Claystone w/ Slip Planes
126-127	Coarse gr. Sandstone
127-129	Sub-rounded Vol. Congl.
129-130	Siltstone
130-131	Contact dip 10 Slip Planes Black Vol. Sandstone
131-133	Slip planes Siltstone w/ rx frag
133-136	Volc. Congl
136-137	Siltstone
137-138	Pebbly Sandstone
138-139	Sandstone grading to Siltstone

SM36
ELEV. 264 FT

DEPTH (in ft.)	
9 - 14	Pink WRL Talus
14 -27	Silt (was soft and wet) w/ CRB cobbles
27 -64	CRB Talus w/ abundant Pebbly Silt Matrix
64 -65	Siltstone w/ Slip Planes
65 -67	Pink WRL Talus
------	No recovery
69 -82	Quartz Dacite

indurated, and moderately weathered and altered. An apparent dip of 10 degrees was seen in the vertical core samples in SM24 (Table I). The thickest section was 43.5 meters (142 feet) measured in SM8 (see Appendix A).

Columbia River Basalt Group

The Columbia River Basalt Group (CRB) is exposed at four locations within boundaries of the study area (see Figure 21): 1) along I-84 east of the weigh station, 2) just below the highway along the railroad tracks, 3) along Gorton Creek beginning at an elevation of about 106.5 meters (350 feet), and 4) in the gravel pit located behind the weigh station.

Basalt is found in several core logs located to the south and east of the landslide. In SM46 (Appendix A) south of the slide (see Figure 15 for location) intact basalt was found for a thickness of at least 51.5 meters (170 feet). Basalt was also seen in logs SM47, SM60, and SM63 (Appendix A).

The largest exposure of CRB is seen along I-84 and the railroad. The outcrop is 0.8 kilometers (0.5 miles) long. The basalt consists of at least two flows with a total thickness of 98 meters (326 feet). Both CRB flows have blocky jointing spaced 8 to 12 centimeters apart. Both flows are black, fine grained, microphyric dense rock with vesicules seen near the upper and lower contacts. Both flows have reversed remanent magnetism. A uniform 0.5

meter thick, buff colored clay layer, that at the base has been silicified, separates the two flows at both outcrop locations. Along I-84 it is exposed near the western portion of the outcrop, and along the railroad it is found a little farther east due to the strike and dip (N60E 22SE) of this inter-bed. Jointing patterns seen in two outcrops near the Union Pacific Railroad bridge at Wyeth also suggest that the flows are tilted toward the south. A major element analysis (Table II) of samples from the flow above (CRB2) and below the clay layer (CRB3) was done (see Figure 21 for sample locations). Both flows were of the low magnesium type Grande Ronde Basalt (M. H. Beeson, oral communication 1985).

TABLE II

MAJOR ELEMENT ANALYSIS USING X-RAY FLUORESCENCE.
BY PETER HOOPER, 1985 WASHINGTON STATE UNIVERSITY.
SHOWING PERCENTAGES OF MAJOR ELEMENTS.
SEE FIGURE 23 FOR SAMPLE LOCATIONS.

SAMPLE	CRB1	CRB2	CRB3	WRL1	WRL2	WRL3	ANDPT	SIGN
SiO_2	53.93	55.31	55.15	51.63	51.79	51.84	62.25	60.25
Al_2O_3	15.23	14.97	14.96	17.10	17.10	17.21	18.27	18.47
TiO_2	2.08	2.14	2.15	1.41	1.44	1.42	1.02	1.10
Fe_2O_3	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00
FeO	10.01	9.88	9.96	7.53	7.83	7.74	3.03	3.76
MnO	0.19	0.18	0.19	0.16	0.17	0.16	0.09	0.10
CaO	8.15	7.26	7.51	9.40	9.26	9.38	6.10	5.55
MgO	4.51	3.74	4.02	7.72	7.24	7.50	2.49	3.21
K_2O	1.05	1.48	1.21	0.15	0.16	0.13	0.95	1.00
Na_2O	2.57	2.76	2.59	2.72	2.82	2.44	3.59	4.36
P_2O_5	0.27	0.27	0.27	0.17	0.18	0.18	0.22	0.21

The CRB flow examined in Gorton Creek is the lowest flow exposed in that area. It is also black, fine grained, microphyric dense basalt with reversed remanent magnetism. Major element analysis (Table II) of this flow (CRB1) (see Figure 21 for sample location) shows it to be of the high magnesium type Grande Ronde Basalt (M. H. Beeson, oral communication 1985).

The basalt seen in the wall of the gravel pit behind the weigh station is highly weathered to tan and has closely spaced joints, 5 to 10 centimeters, with up to 1 centimeter of fine grained silt filling the joints. The CRB here has been more highly weathered. This is the only CRB outcrop found within the slide boundaries.

Intrusive

The intrusive unit is stratigraphically younger than the CRB, but usually found at a lower elevation. It crops out at Anderson Point, at the signal shack on I-84, and just below the signal shack along the Union Pacific Railroad (Figure 21). It is often seen at depth in the core logs immediately below the slide plane near the toe of the landslide (Appendix A). Figure 22 is a contour map of the elevation of its upper surface. The thickness of the intrusive is unknown. Its maximum outcrop thickness, seen at Anderson Point, is 105 meters (360 feet). In outcrop the intrusive has a crude columnar jointing pattern spaced 20 to 38 centimeters (8 to 15 inches). The intrusive seen in core log SM36 (Table I) was similar to the previous descriptions, but it also had a narrow zone of 2 to 3 millimeter sized vesicules spaced approximately 5 centimeters apart with some zeolite linings.

The intrusion is buff to grey, fine grained, holocrystalline quartz diorite. The major minerals are plagioclase, quartz and pyroxene. At Anderson Point it is moderately weathered with a lighter colored weathering rind one centimeter wide. It is more highly weathered at the other two outcrops, where it is lighter in color with no weathering rind and has zeolites commonly found in the joints and cracks. Free (1976) lists the primary minerals for the intrusive as plagioclase, augite, hypersthene, and quartz with fine grained magnetite and ilmenite common. He has referred to both outcrops of the diorite found within this study area as sills and noted that in thin section they have very strong flow structure.

A major element analysis of the intrusive was made (Table II). The two sample locations were Anderson Point (ANDPT) and the signal shack (SIGN) (see Figure 21). The results showed that the intrusive was distinctly different from the basalts sampled in the study, with a much higher SiO_2 content, and that the two diorite samples have similar compositions.

Immediately above the intrusive the core logs suggest either of two conditions may exist: 1) a thin sandy layer followed by a thicker mudstone layer (e.g. SM13, SM30, SM54

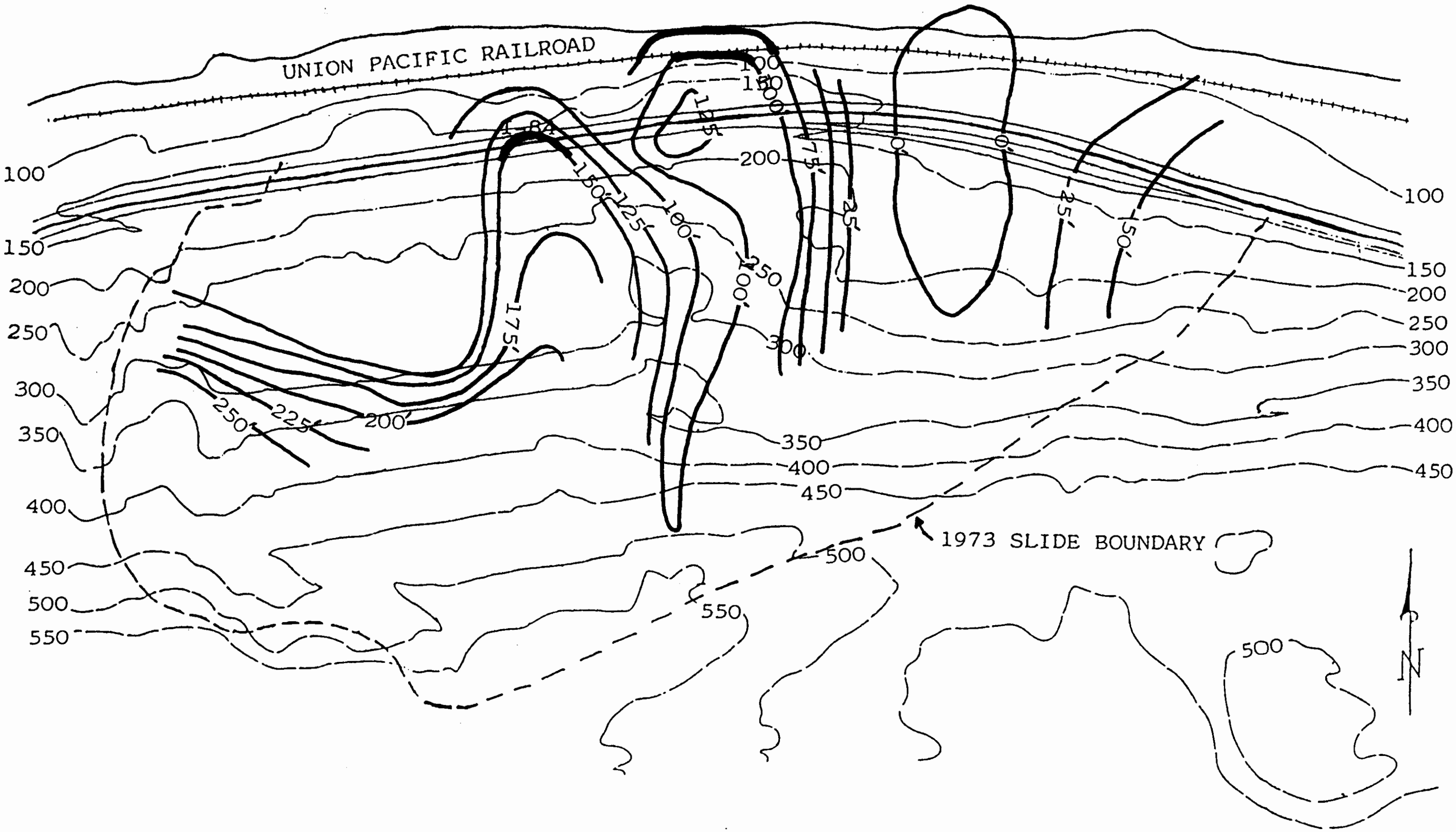


Figure 22. Contour map of the elevation of the upper surface of the quartz diorite intrusive. Heavy lines denote outcrops. Data from Appendix A.

and SM57, see Appendix A) or 2) Wind River Lava talus (e.g. SM4, SM14, SM12 and SM20, see Appendix A). The mudstone in condition 1) does not have bedding features and is found only in the eastern portion of the landslide. Wind River talus in condition 2) is only found in the western portion of the landslide.

Wind River Lavas

The next unit deposited in this area is the Wind River Lava basalt. The term Wind River Lavas (WRL) is used locally to describe the intracanyon High Cascade Lava flows in the Wind River Valley. The Wind River Lava (WRL) found in the study area occurs as loose boulders or talus. Figure 23 is a photograph of the western margin of the landslide showing the bouldery surface of the WRL. The boulders range from several centimeters up to 4 meters in diameter. Even in the core logs, WRL is always referred to as talus. The engineering term "talus" describes a deposit which consists mostly of boulders, and does not necessarily imply that the boulders fell into place. Figure 24, a photograph of three WRL samples found in the study, shows that it may be either massive or vesiculated, and colored light grey, dark grey or pink. The source of the WRL found within the study area has been established as Trout Creek Hill north of Carson, Washington (Wise, 1970).

Wise (1970) described the Trout Creek Hill lavas as "dark-grey olivine basalts with glomeroporphyritic olivine

Figure 23. Photograph of the surface condition of the Wind River Lava. Looking west at the western boundary.

Figure 24. Photograph of samples of the Wind River Lava.

clots." The WRL found in the study area matches that description. The clots of olivine are up to one centimeter in diameter and there are abundant large (5 millimeters) tabular plagioclase phenocrysts. In thin section its mineralogy is plagioclase, pyroxene, olivine (with iddingsite replacement) and magnetite. The degree of weathering of the olivine to iddingsite is varied. The differing colors represent the different degrees of weathering. The least is seen in the light grey samples, the most in the dark grey samples, and intermediate in the pink samples. The groundmass consists of fine grained plagioclase and very fine grained pyroxene. The magnetite occurs in the groundmass as both extremely fine grains included in the pyroxene and the olivine, and as larger euhedral grains with inclusions of pyroxene.

A major element analysis (see Table II) was made on three different colored samples of the WRL. The different textures and colors seen in the field seemed to suggest that more than one flow was deposited in the study area. Major element data from Table II shows no distinct elemental differences between the three samples indicating that there was probably only one WRL flow deposited in the study area. The distribution of the different colored WRL samples can be seen in Figure 25. The WRL was deposited up to an elevation of 183 meters (570 feet) in the study area. At Carson, Washington (see Figure 1) the maximum elevation

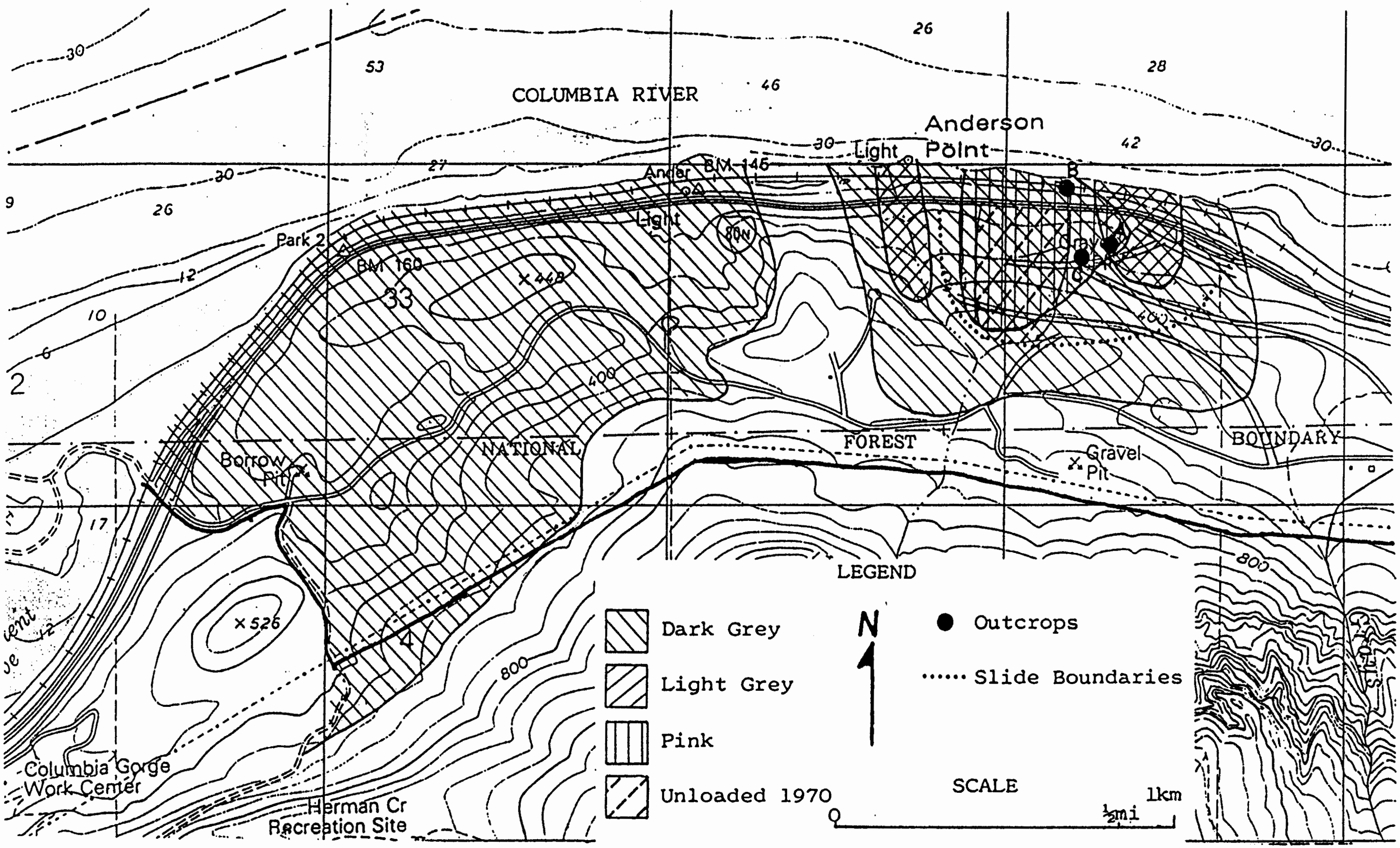


Figure 25. Distribution of the Wind River Lava.

of the WRL is only 161 meters (530 feet).

A relatively intact outcrop of the pink colored WRL is found in the the gravel pit (outcrop A in Figure 25). Figure 26 is a photograph of this outcrop and shows a zone below the pink colored WRL where the color changes downward in soil from pink to purple, to red, to orange and finally tan colored basalt. The same pattern is seen elsewhere in the gravel pit, higher to the north of this outcrop and lower to the south. The apparent dip of the zone is 22SE.

The same pattern of color changes is seen at outcrops B and C, along the Union Pacific Railroad and at a road cut made in the lower haul road (Figure 25). At these locations the color changes occur in unconsolidated silty sand and are 1.5 meters wide. The apparent dip at B is 60NW and is to the southwest at C.

Bretz Flood Deposits

Bretz Flood deposits were deposited next in the study area. These river deposits are found well above present river level at 146 meters (480 feet) at two locations within the study area (see Figure 21). Both are located in the small valley or saddle at the base of the talus slope. They are elliptical shaped deposits, with CRB under and around the eastern outcrop, and WRL in a similar relationship in the western outcrop. Both have been quarried for their sand and gravel. Observing the changes in shape and thickness of the excavated faces the cross

Figure 26. Photograph of outcrop at the gravel pit.

sectional shape of the deposits appear to be lenticular.

The eastern outcrop consists of a medium grained sub-rounded sand. It contains quartz, feldspars, volcanic rock fragments, mica and a few dark colored minerals. It is weathered, and clays are seen on the grain surfaces. It has little cohesive strength when dry. Its highest excavated face is the southwestern face which is 7 meters (25 feet). No bedding features are seen in this outcrop.

The western outcrop, at the borrow pit, is similar to what is seen at the other outcrop, but it has pebble and gravel lenses. These gravel lenses suggest the bedding dips westward. The gravels are well rounded and are composed mostly of basalt clasts with quartz common and a minor amount of crystalline rock. The majority of the outcrop consists of sand of the same type as found at the eastern outcrop. Because the sand at the borrow pit holds a higher excavated cliff, up to 12 meters (40 feet), it appears to have a higher cohesive strength.

Alluvium, Landslide Deposits, and Fill

The most recent deposits found in the study area are landslide deposits, alluvium and fill. The landslide debris consists of WRL and CRB talus and rock fragments with abundant fine grain material filling the voids. Alluvium is found where tributaries enter or have entered the Columbia River such as at Gorton Creek. Highway construction, as well as unloading of the Fountain

Landslide have created fill deposits within the study area.

STRUCTURE

Faulting and folding are common to the region (see Figure 4). Evidence for the presence of tectonic features within the study area include the breccia zone seen in the basalt outcrop along I-84 and breccia found in core logs SM23 and SM24 (see Table I). The breccia zone along I-84 (see Figure 21), is in the same location as a small rock slide which occurred in March of 1983. A photograph of the breccia zone is shown in Figure 27. It is 15 meters wide and consists of highly fractured CRB. The rock surfaces of the CRB fragments have turned greenish tan. The fractures are filled with a thin (less than 1.5 centimeters) silt layer. The fractures and the contact between the breccia zone and CRB are nearly vertical and trend N25W.

The tectonic breccia found in the core samples SM23 and SM24 (Table I) was not logged as such by the Oregon State Highway Division. It is therefore not known if it occurs at depth elsewhere in the area (Appendix A). In the core samples the breccia does not have much cohesive strength, especially when dry. A small piece from SM23 is shown in Figure 28. The breccia is light green in color with angular fragments ranging from silt up to 5 centimeters (2 inches) in diameter. Some of the original larger, darker minerals can be seen with indistinct grain

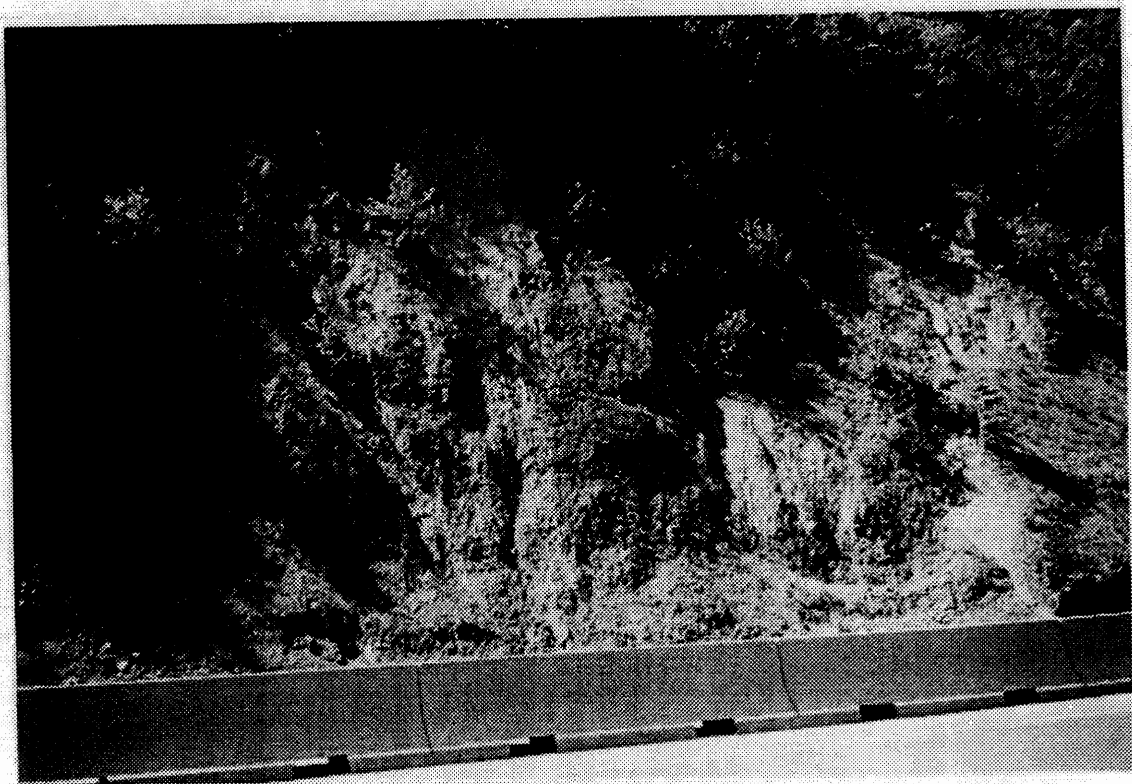


Figure 27. Photograph of fracture zone east of the weigh station. The zone is approximately 15 meters wide.

margins. Generally, very little of the original rock textures can be recognized.

The discordance in the attitude of the CRB flows in the cliffs south of the study area is additional evidence for tectonism in the area. The discordance is seen just east of the study area behind Wyeth Campground between Gorton and Harphan Creeks (see Figure 1). A photograph of this block is shown in Figure 29. This block appears to be tilting east while the surrounding cliffs appear nearly horizontal. A lineament map (Figure 30) suggesting locations of other possible faulting for the area was made using Bela's, 1982 neotectonic map as a base map.

DISCUSSION OF THE STRATIGRAPHY AND STRUCTURE

A stratigraphic section for the study area is shown in Figure 31. The explanation for Figure 31 will begin with the oldest unit, the volcaniclastic unit seen only at depth in core logs. This unit consists of volcanically derived sand, shales and some gravels. It probably belongs to either the Ohanapecosh or the Eagle Creek Formation; which can not be determined from the data. The upper contact of the volcaniclastic unit is generally a weathered erosional surface or slip plane.

Stratigraphically above the volcaniclastic unit are the flows of the Columbia River Basalt Group. The type and position of the contact in the study area between the two

Figure 28. Photograph of core sample of the tectonic breccia. Sample taken from SM24. See Table I.

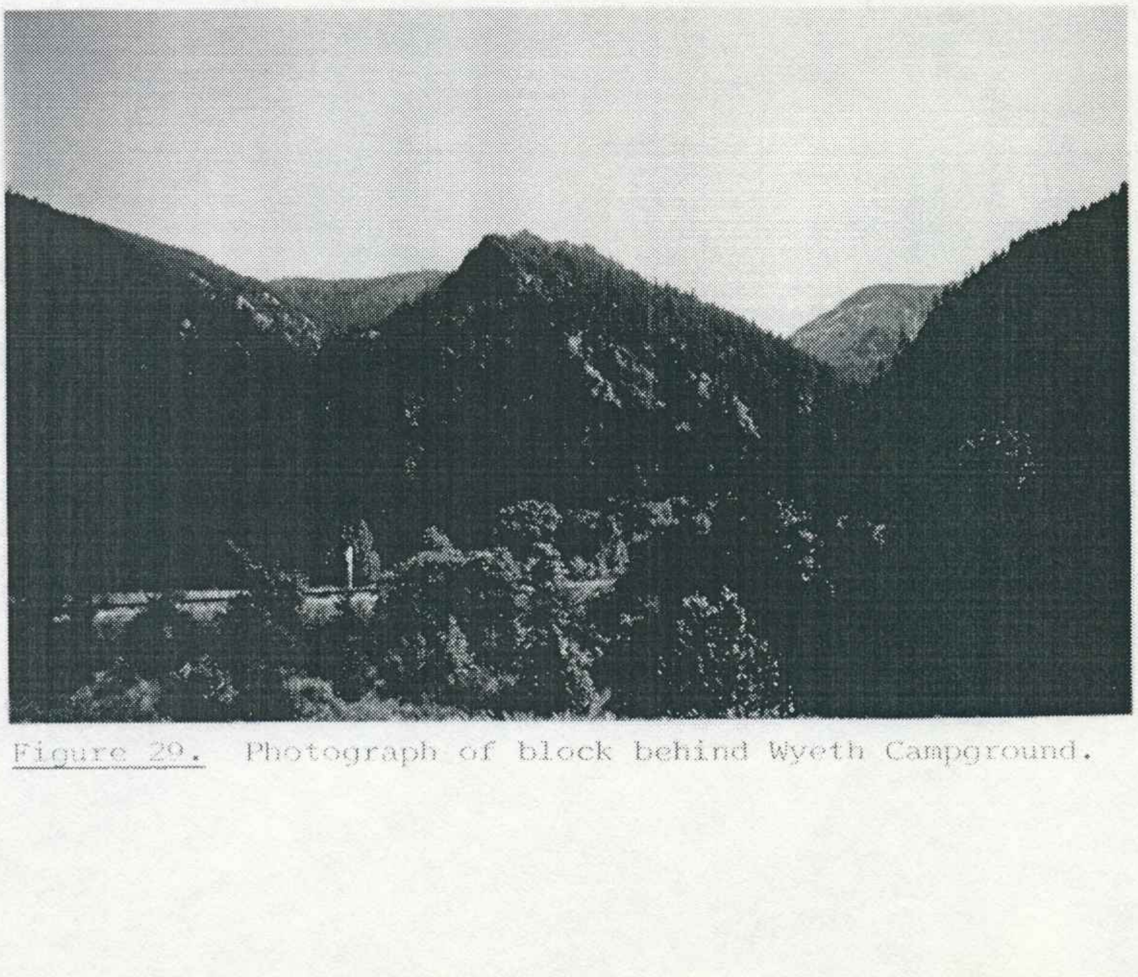


Figure 29. Photograph of block behind Wyeth Campground.

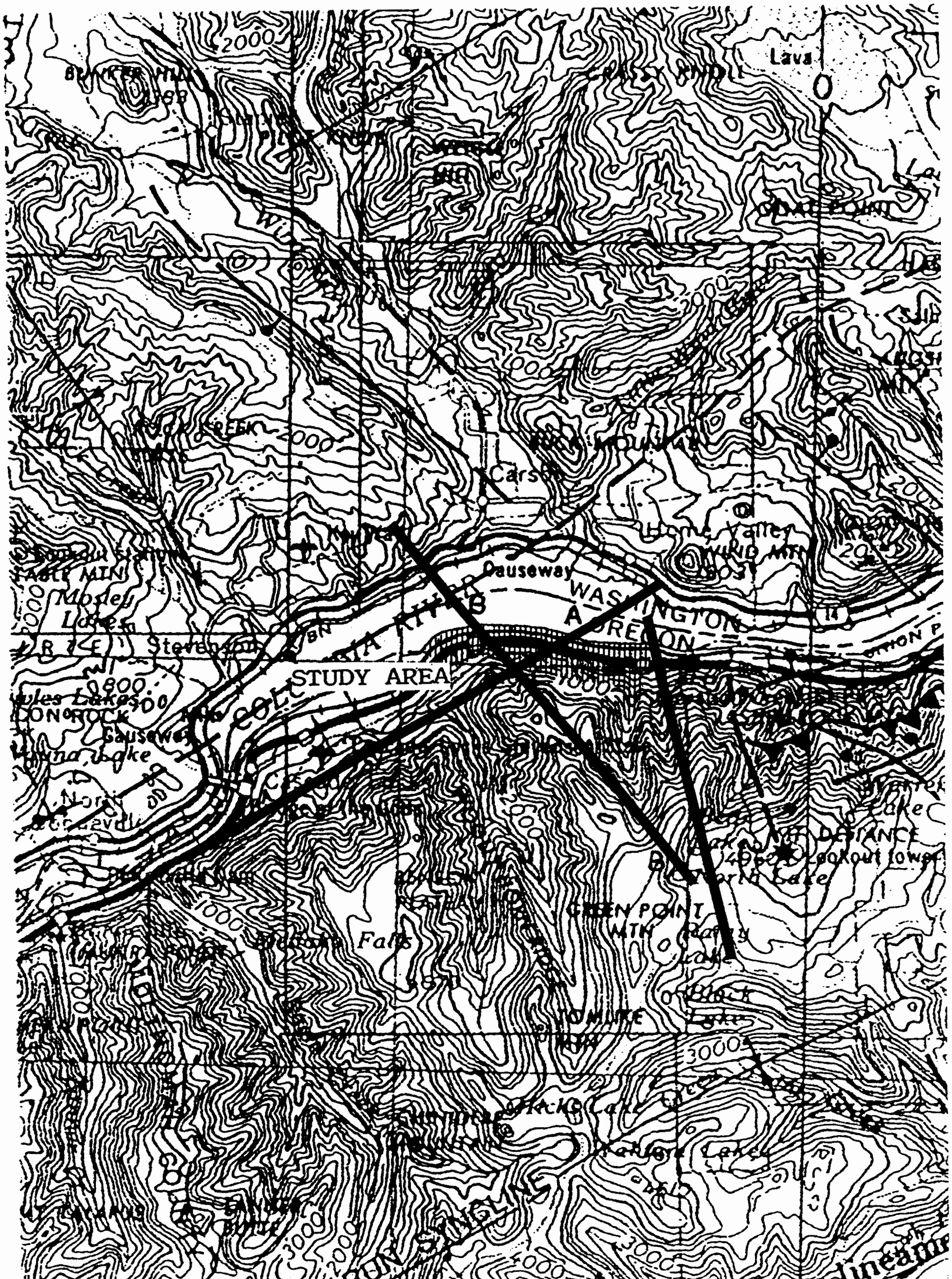


Figure 30. Lineament map, base map from Bela, 1983.

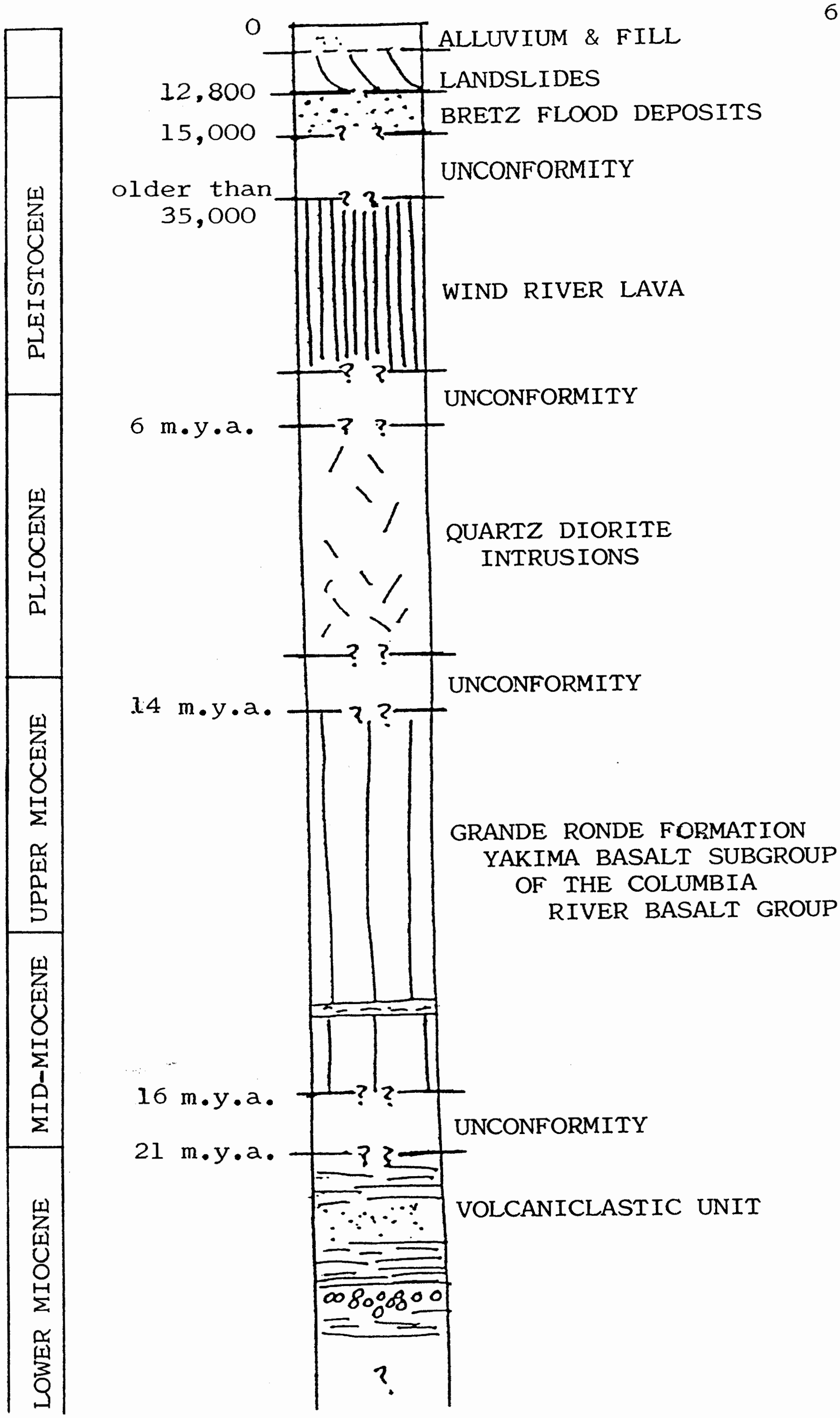


Figure 31. Stratigraphic column for study area.

units is not well understood. From the data presented in this study, three possible explanations for the type of contact emerge: 1) the CRB has slid into place from the cliffs to the south, 2) the CRB is the lower part of a conformable sequence with the cliffs to the south, or 3) the CRB has been faulted into place.

The contact between the CRB and the volcaniclastic unit in possibility one must be at or above river level. This would allow for sufficient erosion and undercutting of the softer volcaniclastic unit and create conditions for a large slide. The southerly dip in the flows of the CRB and the absence of lateral continuity of the flows support this possibility. If this hypothesis is correct, the size of the intact block that slid would have been at least 0.80 kilometers long (0.5 miles), 0.40 kilometers (0.25 miles) wide and 98 meters (326 feet) thick. Although it is possible the block could have slid a short distance, it seems unlikely that both the fault zone and the interflow clay layer remained intact. It would be necessary to do an additional study of the CRB stratigraphy in the cliffs behind the landslide to absolutely determine if the flows exposed along the highway and railroad slid into place from the cliffs above. Another possibility would be for the slide block to originate from the north side of the river. This possibility would be even less likely since the intact block would have slid almost two kilometers.

The second alternative is the CRB flows within the study area are stratigraphically conformable with the CRB in the cliffs. This would make the unconformable contact with the volcaniclastic unit occurring at approximately river level to the east of the landslide, and 101 meters (335 feet) to the west. If this is the case the pre-CRB topographical surface would have dipped approximately 55 degrees here. The first flows of the CRB into this region encountered an erosional surface with some topographical relief (Wise, 1970) however, it is unlikely that it dipped this steeply.

The third possibility that the CRB within the study area was faulted into place is supported by the evidence of tectonism in the study area. If faults exist within the study area they would most likely be located south of the landslide trending N30E (fault A, Figure 30) or bisecting the landslide trending N20W (fault B, Figure 30). The necessary relative movement on fault A would be the south side down and on fault B the north side down. Fault B alone could explain the stratigraphic relationships seen in the study area, but fault A alone can not.

After the deposition of the flows of the Columbia River Basalt Group, the intrusions were emplaced approximately 6 million years ago. Because the basal contact of the intrusive is not seen in any outcrops within the study area it is not certain whether they are sills according to Free (1976) or plutons. A study of the paleochannels of the Columbia River (Tolan and Beeson, 1984) indicated the youngest age the uplift of the Cascades began was as late as 2 million years ago. The necessary depth of emplacement of the intrusives (Brownlaw, 1979) suggests the uplift of the Cascades began after the intrusives were in place. The Columbia River began cutting the present gorge during this uplift (between 6 to 2 m.y.b.p.) first through the CRB then into the softer volcaniclastic unit. The intrusives restricted the erosional progress and formed exposed highlands.

As uplift and erosion approached its present elevation the Wind River Lava was deposited about 338,000 years ago (Duncan, 1985). Both the river and the CRB found <u>within</u> the study area were in their present position when the Wind River Lavas were deposited. This relationship is seen at the gravel pit with the WRL directly overlying highly weathered CRB, in core logs showing the same relationship and by WRL float resting on the intact CRB.

When the intracanyon WRL poured into the Columbia River it dammed the river as verified by the 35 meter (150 foot) thick delta deposit east of Carson, Washington (Williams, 1916). In Oregon the flow covered the intrusive at the signal shack and surrounded it at Anderson Point. Its base was oxidized as seen in the localized pink coloration zones. The channeling features found to the south of the landslide (the saddle) were most likely created during the initial erosion of the WRL dam. As the lava became solid it may have contracted and shrank especially in the middle of the channel, the thickest part. Erosion then continued through the middle of the channel. As erosion removed the dam many slides occurred in the WRL forming the present bouldery appearance of its surface.

During the Bretz Floods the dam was further eroded and sands were deposited in the saddle. Although some movement of the Fountain Landslide may have occurred before the glacial flooding the appearance of an easily recognizable headscarp in the early aerial photographs implies that movement has occurred after the Bretz Floods.

ENGINEERING GEOLOGY

The engineering geology studies made for this report describe the surficial stress features, the slip plane, the subsurface materials, the material properties, the ground water table, and the stability of the slide mass. The purpose of the engineering geology study is to aid in understanding the slide and interpreting possible stabilization procedures.

SURFACE INDICATIONS OF SLIDING

The type, distribution and attitude of the surficial stress features were mapped at a scale of 1 : 300. This map is shown in Plate 1. The historic movement of the landslide is partially shown in the aerial photographs Figures 7, 8, 13 and 18. These show the extent of movement on I-84 and the actively moving areas.

Description and Distribution

The engineering geology map (Plate 1) shows the distribution and the attitude of the surface indications of sliding such as, tension cracks, scarps and depressions. The majority of these features are found near or at the slide boundaries. The time period for most of the features was determined by their effect on the portion of the landslide unloaded in 1970 and the roads built for the unloading. A time frame for the remaining features could not be determined.

Along the western boundary, many tension cracks (labeled A, Plate 1) are seen. A majority of these cracks are short (about 2 meters long) and are up to 2 meters deep. A photograph of one of the larger cracks (labeled A1) is shown in Figure 32. The scarps found along the western margin of the landslide labeled B on Plate 1, indicate that it is dropping relative to the rest of the slide mass.

Along the eastern margin of the unloaded area are a series of scarps (labeled C on Plate 1). The scarps all have the eastern side down, indicating there is a relative lowering of eastern part of the slide mass. Other surface indications of sliding include the depression (labeled D on Plate 1), near the toe of the landslide. Near the headscarp are two south side down scarps (labeled E on Plate 1).

Most of the cracks and scarps are rounded and have vegetation covering them as shown in Figure 33. Except crack A1, (Figure 32) which has angular rims and no vegetation on it. Those features which are angular and not vegetated indicate recent activity, the others are older features. Two other recent cracks are, 1) the long crack along the lower haul road (F on Plate 1), and 2) the small

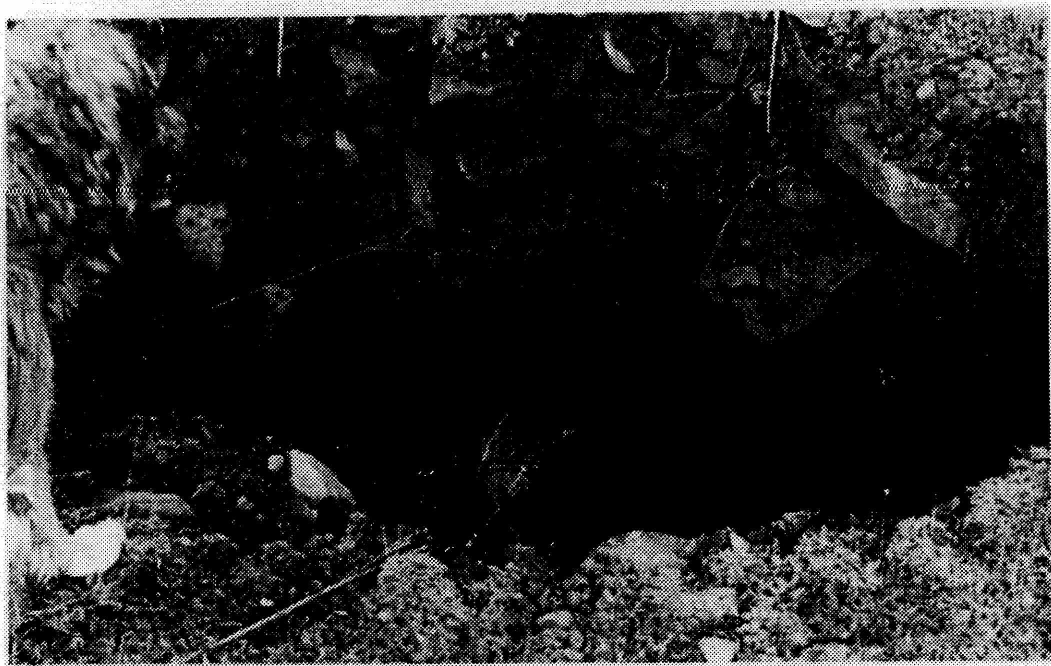


Figure 32. Photograph of crack A1. Located along the western edge of the landslide.

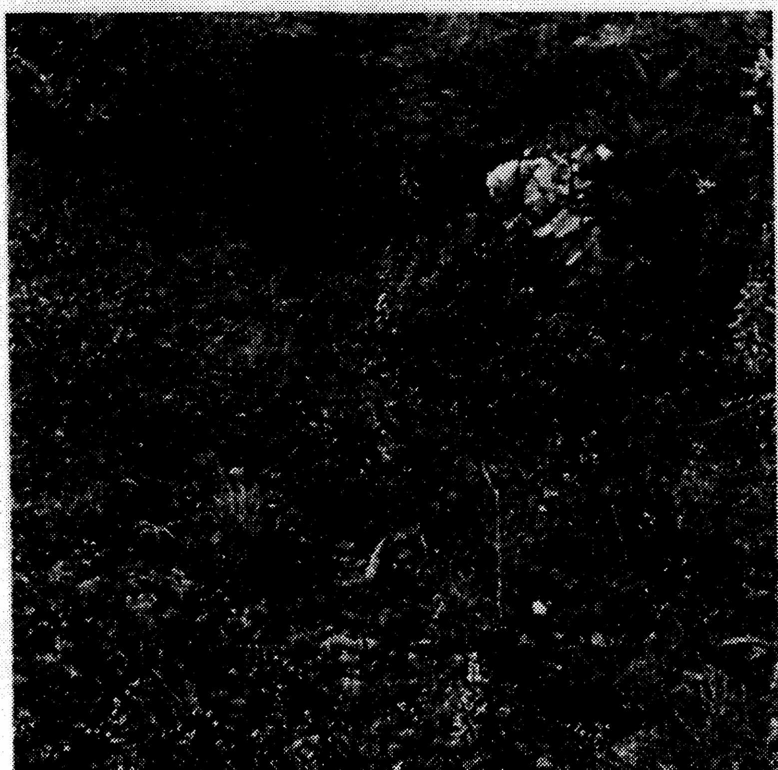


Figure 33. Photograph of grass covered scarp. Located near the eastern boundary of the unloading.

headscarp (Figure 34) located east of the mapped area in the lower haul road near its intersection with Herman Creek Road.

Discussions and Interpretations

The major movement directions of the slide mass can be determined from the distribution and attitude of surface cracks, scarps and depressions. Figure 35 diagrams the major movement direction of the landslide. The slide moves northward, the entire head of the slide drops, the western margin is dropping, and the toe heaves. There are graben features also seen at the head as the slide pulls away from more stable surroundings. The eastern edge of the unloaded portion has raised relative to the western slide mass, making the western portion of the slide appear to rotate with respect to the eastern portion. Most likely the relative rotation of the slide masses expresses the change in the equilibrium between the eastern and western portion of the slide from the unloading.

Upon analysis of the stress features the boundaries of the Fountain Landslide were modified from Oregon State Highway Division preliminary findings to include the features at the head of the landslide. The inability to find many surface indications of sliding along the eastern boundary makes precise determination of the eastern slide boundary difficult. The scarps across the two haul roads and the maximum extent of movement of I-84 describes the

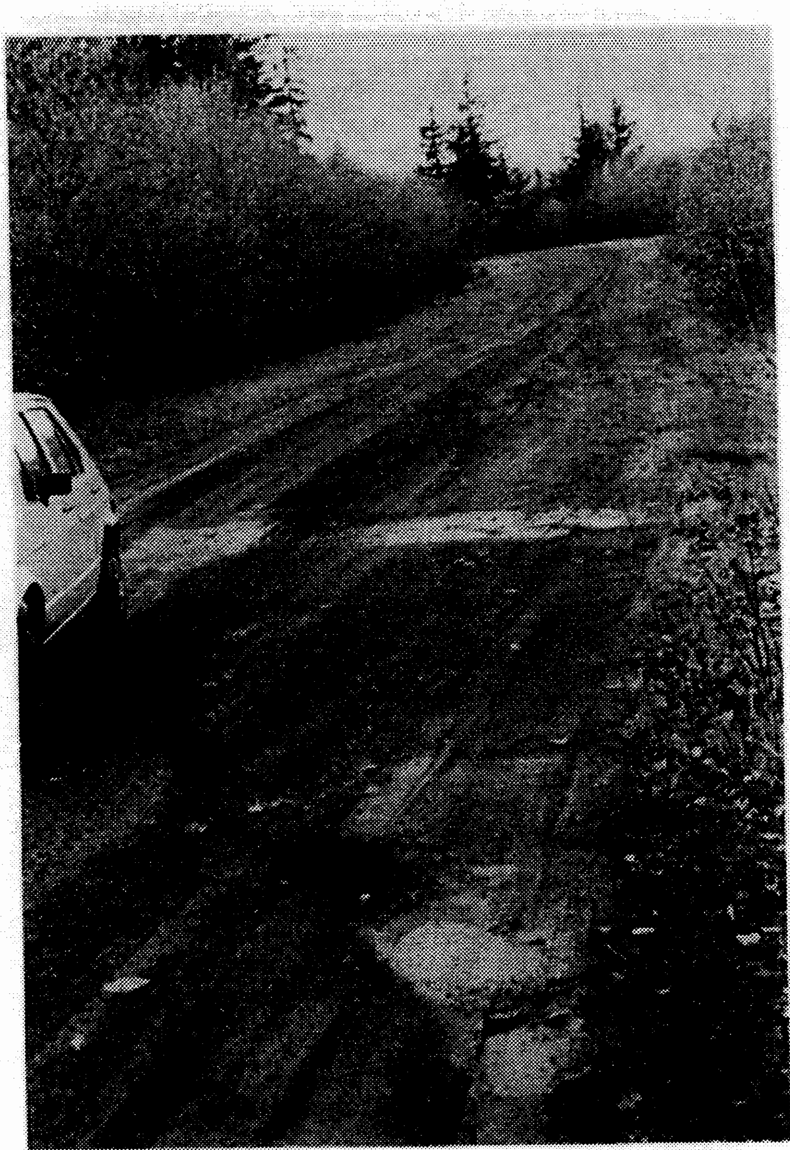


Figure 34. Photograph of arc-shaped crack in the lower haul road. Located near the intersection with Herman Creek Road.

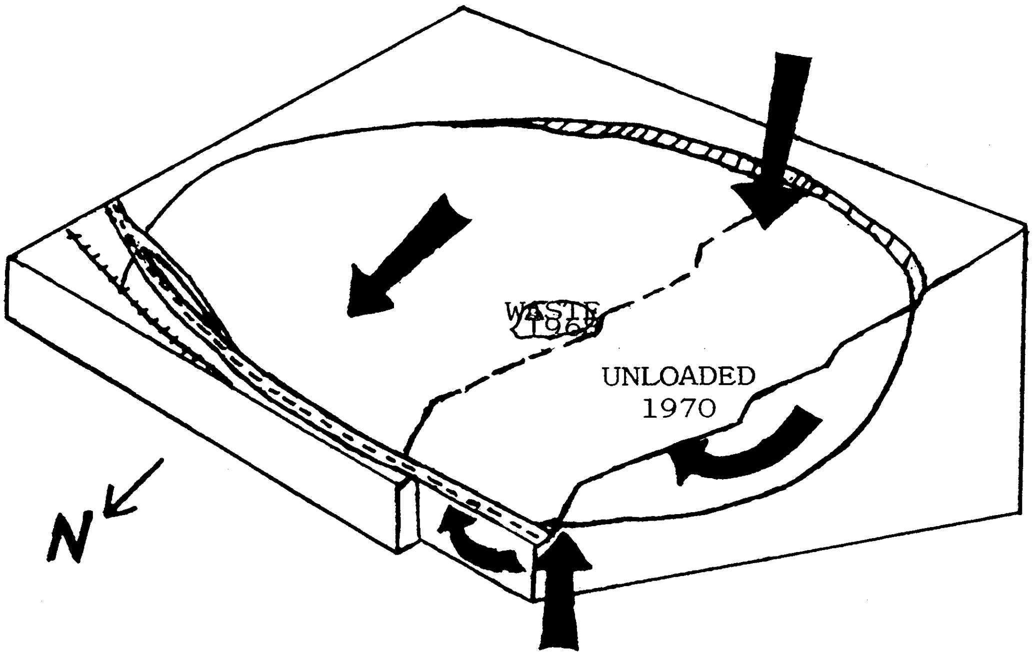


Figure 35. Relative movement of the landslide.

extent of movement there.

CROSS SECTIONS AND THREE DIMENSIONAL MODEL

The purpose of the cross sections and a three dimensional model was to aid in the interpretation of the distribution of the subsurface units, the shape and depth of the slip plane, and the depth of the ground water table.

Description of the Cross Sections

Three cross sections of the landslide were made (Figure 36A, B, C) using preliminary Oregon State Highway Division core logs (Appendix A) (see Plate 1 for the location of each cross section). The depth of the slip plane was determined from slope inclinometer data and verified by the presence of a mudstone layer in the corresponding core log. Inclinometers can only measure the shallowest major slip plane which shears off or distorts the tube so that the instrument can not be lowered any deeper within the tube. When talus was described in the core logs as black or basalt it was interpreted to be CRB talus. When it was described as either pink, scoria, andesite or grey it was interpreted to be WRL talus (using the engineering definition of talus). The depth to the WRL talus was found marked in some logs. This was interpreted to approximately express the depositional surface of the Wind River Lava. The depth to the ground water table was determined from the ground water table contours on Plate 1.

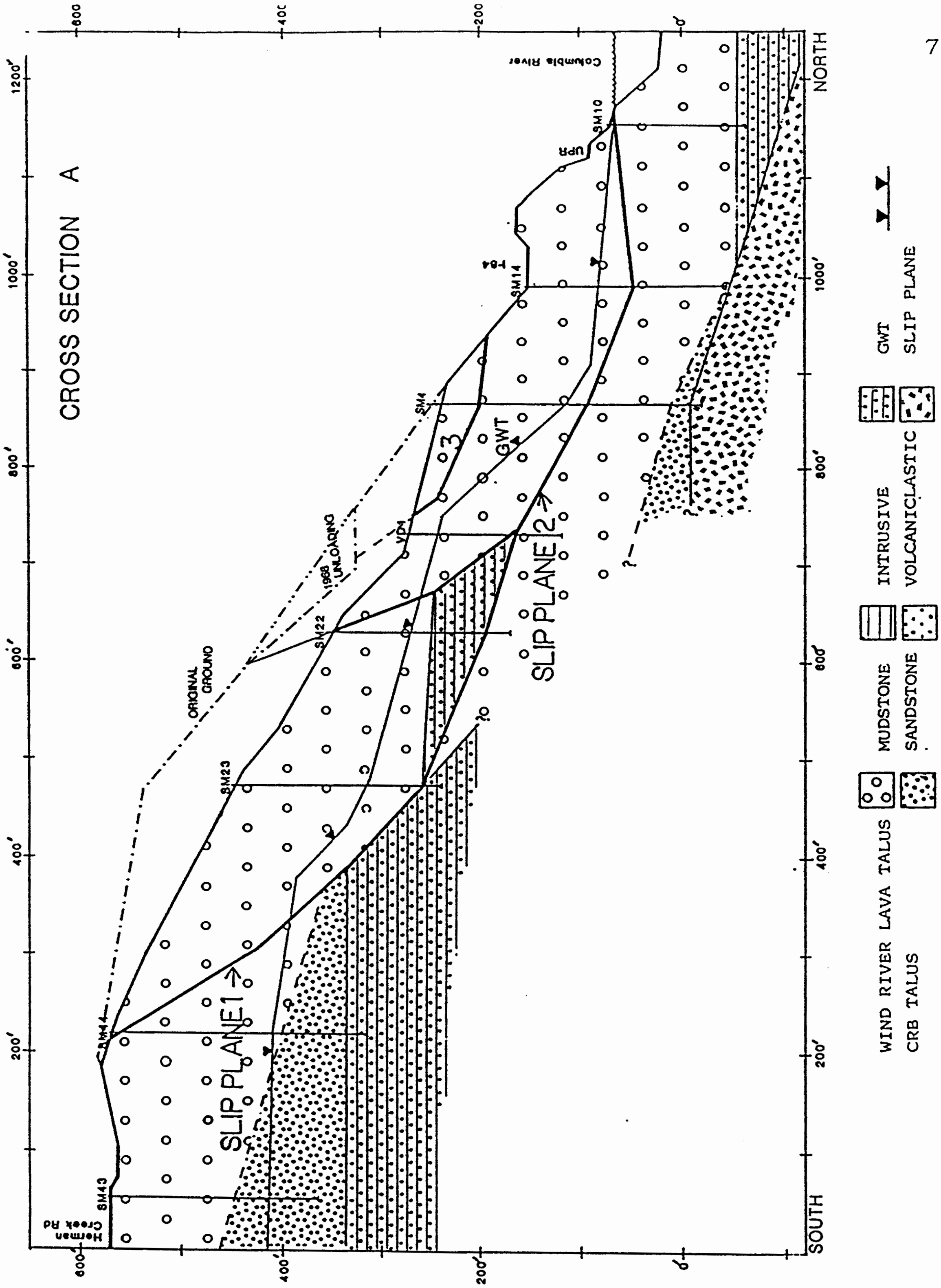


Figure 36A. Cross section A. For location see Plate 1.

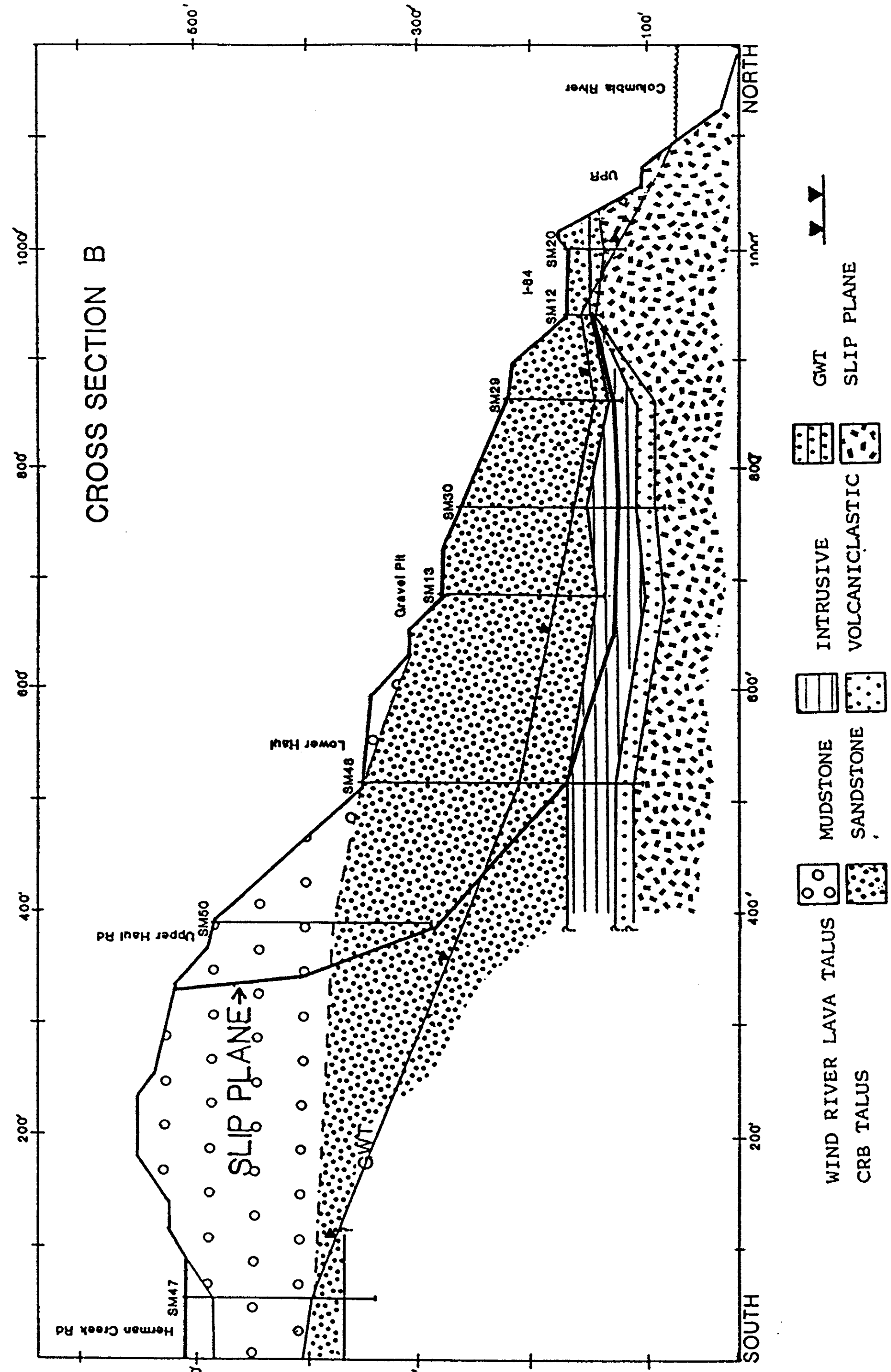


Figure 36B. Cross section B. For location see Plate 1.

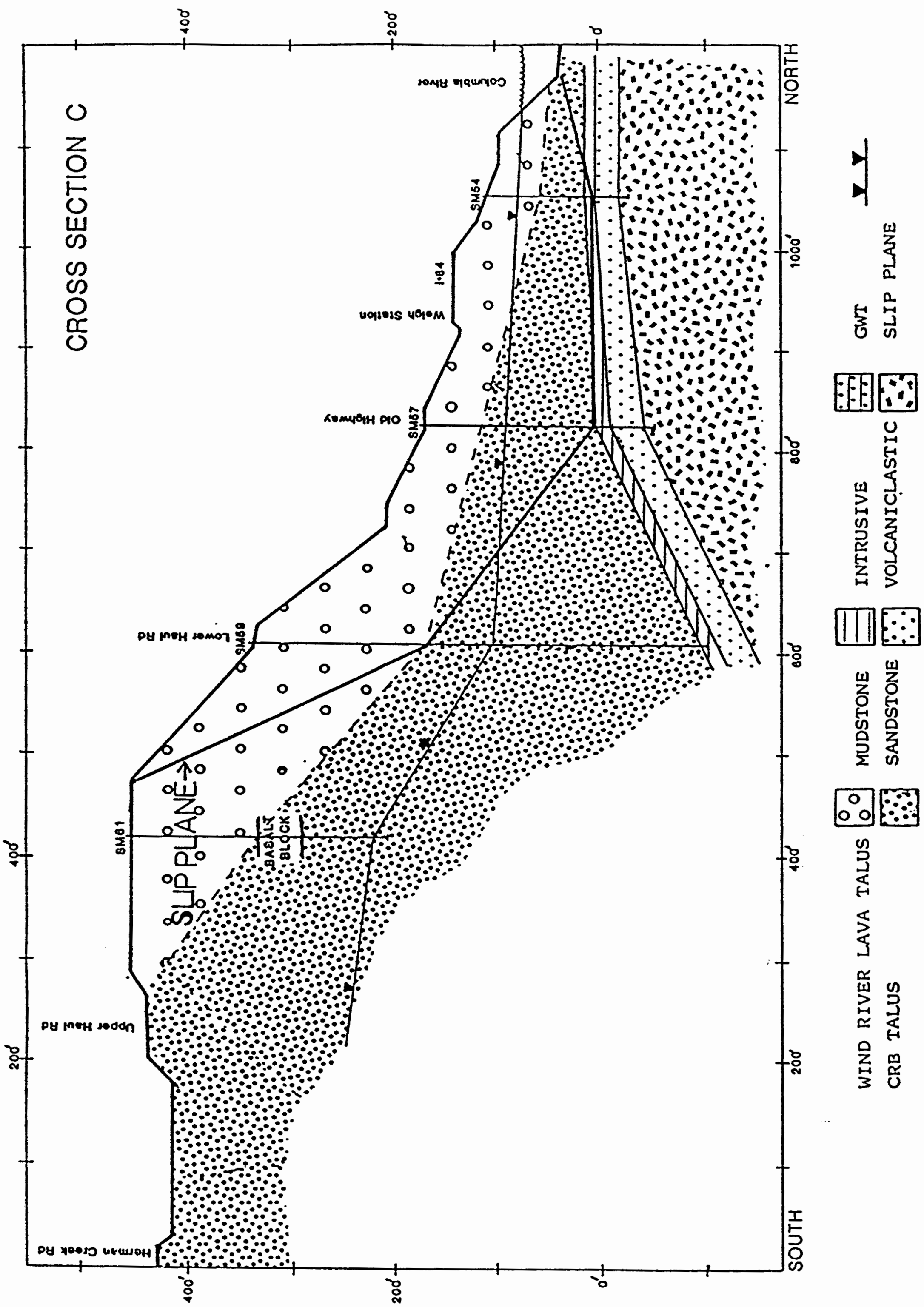


Figure 36C. Cross section C. For location see Plate 1.

Figure 36A shows the cross section through the unloaded portion of the landslide. Two major slip planes are seen in this cross section. The upper slip plane (1) is truncated at the toe by the lower slip plane (2). A smaller surface slide is marked by the slip plane number (3). The ground water table is perched over the volcaniclastic unit near the toe of slip plane (1). The slide mass is almost entirely Wind River Lava and Columbia River Basalt Group talus with silts and sand filling the voids. The two slip planes are confined to a 0.6 to 1.8 meter (2 to 6 foot) rocky mudstone layer. A sample of this material from core SM36 is shown in Figure 37.

The cross section seen in Figure 36B through the middle of the landslide, has only one major slip plane. The ground water table is lower than in cross section A, except at the toe where it is slightly perched over the intrusive. The slip plane occurs in an approximately 12 meter (40 foot) thick mudstone layer above the intrusive.

The cross section in the eastern portion of the landslide is shown in Figure 36C. It has one slip plane. The slope inclinometer and core log data only define the slip plane well at the toe. In drill hole SM59 the slope indicator data recorded only minor tilting of the tube and the core log had no interlayer of mudstone in the talus (Appendix A). The slip plane could occur below the level of the drill hole; SM59 may not be involved with sliding;

or the slide plane is not confined to a single zone. The location of the slide boundary in Plate 1 was used to delineate the head of the slip plane for this cross section. The ground water table in this cross section is much lower than the other two cross sections.

Description of the Three Dimensional Model

A three dimensional string model was made using data from core logs (Appendix A). The photograph in Figure 38 shows the constructed model. A dowel represents each hole. The top of the doweling marks the elevation of the ground surface. The elevations of the slip plane and of the upper contact of the subsurface units were also marked. The vertical exaggeration of the model is 2 : 1. Colored yarn represents the surfaces of the different geologic units and of the slip plane. The slip plane is marked with red yarn, the intrusive unit with blue, the volcaniclastic unit with green and the black yarn marks the basalt. Most of the slide mass consists of talus which was not marked. It fills in the unmarked areas of the three dimensional model.

Discussion of the Cross Sections and Three Dimensional Model

The cross sections show that the slide mass is almost entirely made up of Wind River Lava and Columbia River Basalt Group talus and that the slip plane occurs in some type of mudstone unit. Other than these similarities there

Figure 37. Photograph of the slip plane material. $2\frac{1}{2}$" core sample taken from SM36, see Table I.

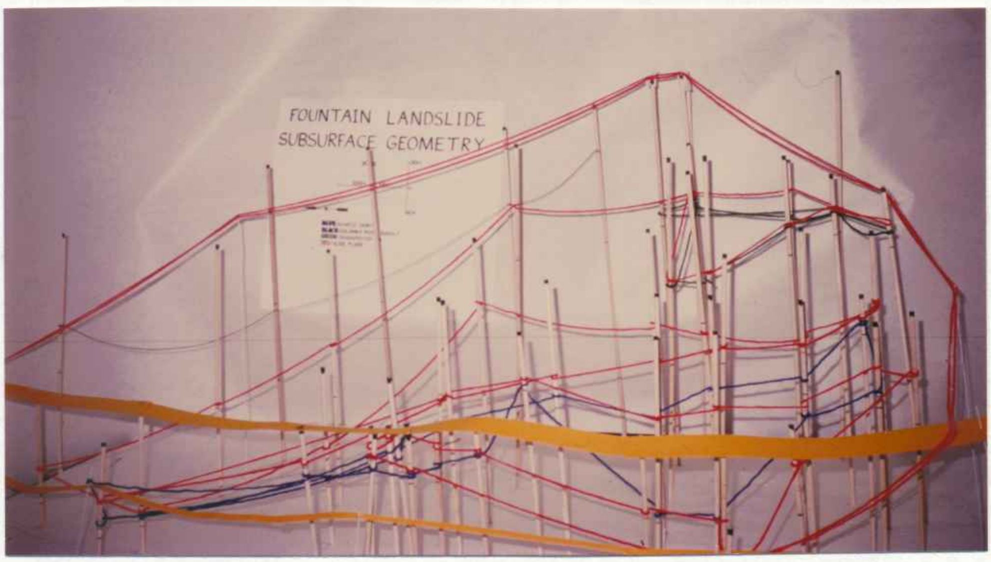


Figure 38. Photograph of the three dimensional model.

are significant differences between the three cross sections which suggests the presence of two slide masses, the western portion and the eastern portion. The boundary between the two is approximately the eastern edge of the 1970 unloading. The differences are: 1) the western portion (Figure 36A) has two slip planes and the eastern portion has only one, 2) the western portion cuts through a talus slope and the eastern portion is mostly in the incompetent mudstone layer above the intrusive (basal failure). These differences indicate that the two portions have differing mechanics, stabilities, and behaviors.

Some of the geologic history for the study area can be determined from the cross sections and core logs. When the Wind River Lava was deposited in the study area the intrusion was exposed in the western portion of the landslide. Behind the intrusion was a slope of talus and the volcaniclastic unit. In the eastern portion the Wind River Lava was deposited on a layer of CRB talus. A mudstone layer is often found in the core logs at the basal contact of the Wind River Lava. This suggests that there was sufficient time for a soil zone to develop on the talus prior to the deposition of the Wind River Lava. A soil zone or weathered zone appears above the intrusion in the eastern portion of the landslide.

GROUND WATER

The ground water condition influences the stability of landslides. This relationship is seen (see Figure 10) at the Fountain Landslide where the greatest amount of movement occurs during November to June, Oregon's rainy season.

Description of the Ground Water Table

Problems in determining an accurate ground water table for the Fountain Landslide were encountered. They were due to the inconsistent monitoring of water levels. Interpretations were difficult because of the known inconsistent responses of the ground water table throughout the landslide (see Figure 11). To get an accurate view of the ground water table it would be necessary to have all the measurements taken over a relative short period of time. This was done only during the testing of the vertical drains in 1971 and 1972 for the western portion of the landslide. During this period the highest ground water elevation was measured on January 21, 1972 (Figure 39) and the lowest on October 11, 1971 (Figure 40). A distinct flattening in the central portion of the slide is seen in both figures. This is where the ground water table is elevated over the volcaniclastic unit (Figure 36A).

A ground water table was estimated for the remaining portion of the landslide (Plate 1) from spotty water level

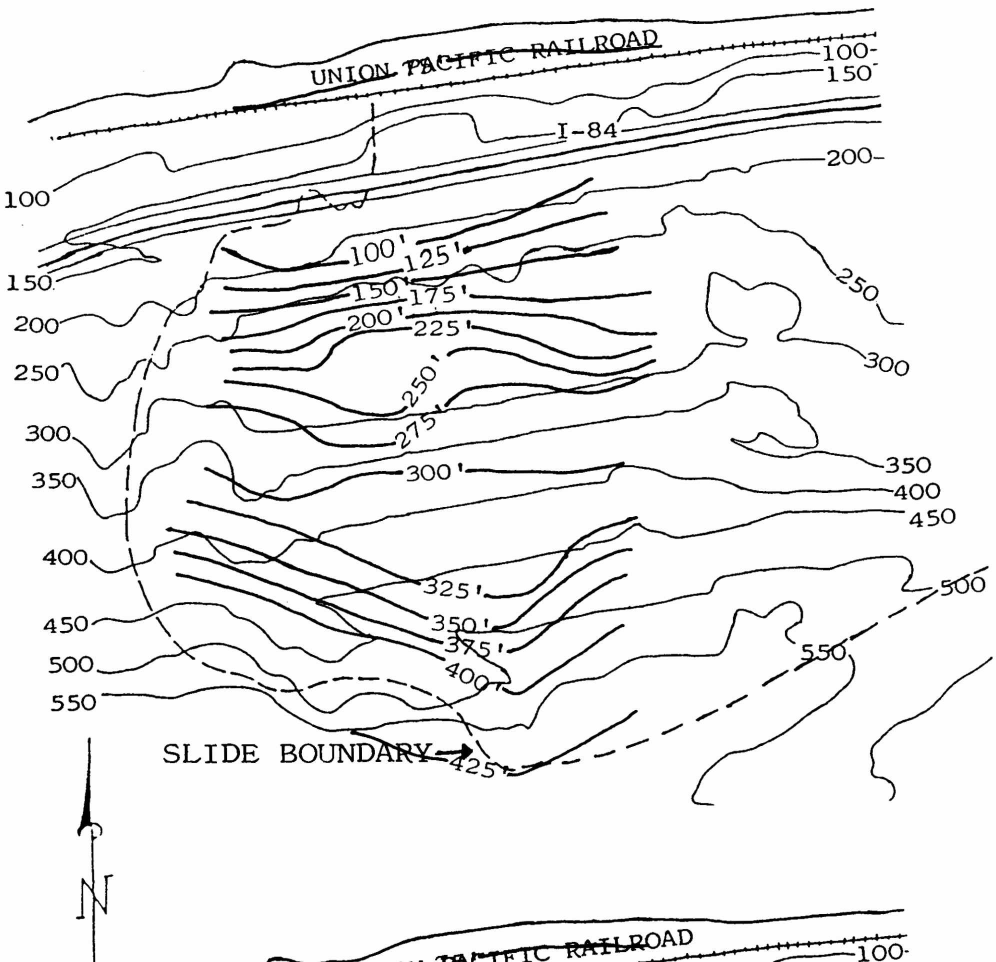


Figure 39. The elevation of the maximum measured ground water table, 1/21/72.

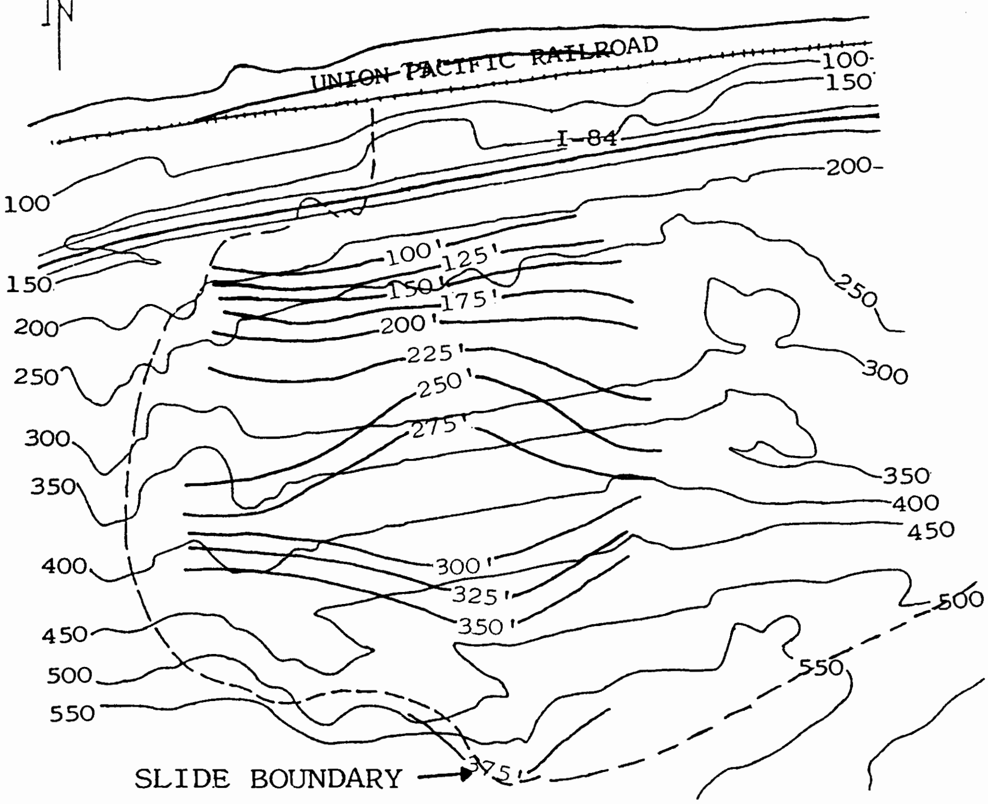


Figure 40. The elevation of the minimum measured ground water table, 10/11/71.

observations. The ground water table in Plate 1 corresponds to the January 11, 1972 measurement (Figure 39). The estimates were made by proportionally increasing the elevation according to the month it was taken. This has probably resulted in an underestimate of the water table for the eastern portion of the landslide.

Description of the Spring Flow Rates

Springs flow on the landslide during wet periods (see Plate 1 for locations). One spring is located just east of the unloaded portion on the lower haul road and the others are all located in the depression labeled D in Plate 1. The occurrence of springs indicate that at these locations the water table is at times, at or near the surface. However, the maximum measured ground water table contours (see Figure 39) do not place the ground water table near the surface in the vicinity of the springs.

A study was done of the flow rates of two of the springs located near the depression labeled D on Plate 1. The procedure for determining the flow rates included measuring the time it took each spring to fill a five gallon (19 liter) bucket. One spring was located 3 meters (10 feet) south of VD2 (see Plate 1) and the other spring flows from the observation well VD2E.

The spring at VD2E was easily measured. The PVC piping used for well casing extended about 0.5 meters above the ground and when a notch was cut into one side it formed a spigot. At VD2 it was necessary to dig a trench below the spring greater than the depth of the bucket and channel the water into the bucket.

The flow rate for each spring and the cumulated weekly rainfall from records of the U.S. Department of Commerce, (1983-1984), NOAA, Climatological Data at Bonneville Dam are shown on Figure 41. The spring at VD2 had a maximum flow rate of 8.9 gallons/minute (34 liters/minute) and at VD2E it was 7.8 gallons/minute (29 liters/minute). The springs did not flow during the summer or winter months.

When Grays Creek (see Figure 21 for location) is flowing it becomes ponded behind the lower haul road fill and then disappears into the talus. A culvert was not installed for Gray's Creek to flow under the lower haul road during construction of the haul road. Either the stream bed was dry at that time or possibly Gray's Creek has since changed channels. This ponding is therefore a relatively new feature.

Discussion

The elevation of the ground water table in Plate 1 is high in the western portion of the Fountain Landslide where it is elevated over either the volcaniclastic unit (Figure 36A) or the intrusive unit (Figure 36B). By comparison the eastern portion has a low ground water table. The

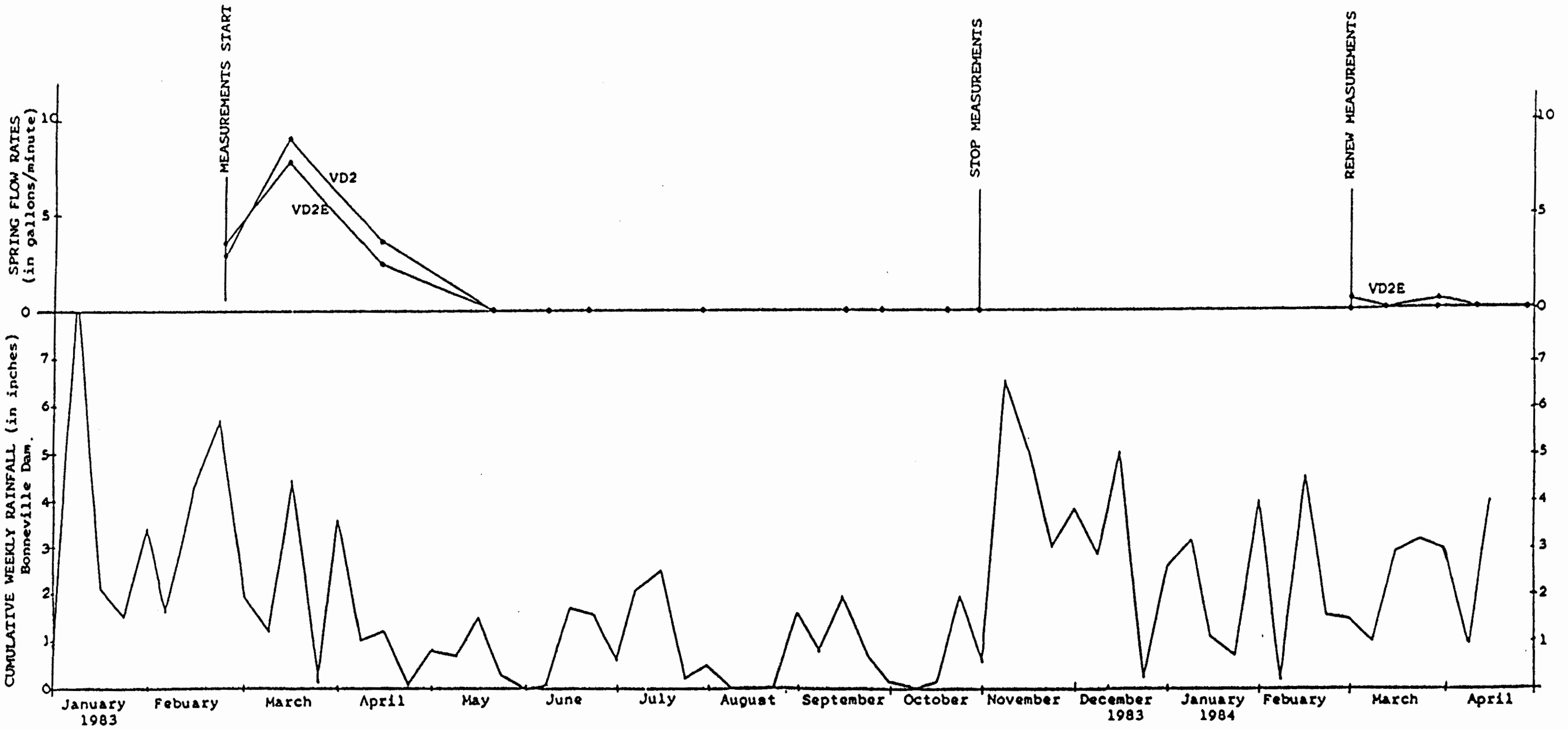


Figure 41. Spring flow rates and rainfall. See Plate 1 for spring locations. Rainfall data from US Dept. of Commerce, (1983-1984), NOAA, Climatological data for Bonneville Dam. Dots represent data points.

elevation of the ground water table appears to be controlled by the presence of the impermeable layers.

The maximum measured ground water table (Figure 39 and Plate 1) probably does not represent the highest annual ground water table, since it is not near the surface and does not reflect the flowing springs. The high water pressures which occur seasonally at the spring at VD2E, most likely are the result of high seepage pressures from the steeply inclined ground water table in this area.

The flow rate data from January 1983 to April 1984 shows that flow rates were affected by high monthly, not weekly rainfall. High weekly rainfalls only had a small effect on the flow rates while the previous monthly rainfall had a greater effect on flow rates. As the monthly rainfall decreased the flow rates slowly receded to zero. This is a slower response than expected for talus, indicating that it has a relatively lower permeability than would be expected. Localized frost heave was observed during extended cold periods with ice crystals up to 3.5 centimeter (1.5 inches).

The ground water table does not respond consistently throughout the slide mass to changes in rainfall. As shown in Figure 11 wells relatively close together can respond quickly to changes in rainfall, slowly, or oppositely.

SLOPE STABILITY ANALYSIS

To evaluate the stability of the Fountain Landslide three cross sectional stability models were made of the present slope. The analysis of each model consists of determining the failure surface with the lowest factor of safety. The factor of safety is the ratio of the factors which resist movement to those that encourage movement (resisting forces/driving forces). At failure the factor of safety is equal to one. The purpose of calculating the factor of safety is to aid in designing stability procedures, to place a value on the relative stability of the model, to possibly identify the conditions which create instability and to assess the effect of various design procedures. Specific designs for stability of the Fountain Landslide are beyond the scope of this study, therefore only a general discussion of some of the possible stability procedures will be presented in this report.

Factor of Safety Calculations

The presentation of the slope stability factor of safety calculation procedures was derived from Bell (1980), Chowdhury (1978), National Academy of Science, Transportation Research Board, (1978), and Terzaghi and Peck (1967). Refer to these references for a more in detailed discussion of the slope stability analysis.

To describe the calculations of the factor of safety a simplified model will be used (Figure 42). In this model the slope and soil are uniform, the center (A) and the radius (AB) of the critical circle (the failure arc with the lowest factor of safety) are known and the soil is saturated. The driving force (T) is the summation of that part of the weight of the slice acting parallel to the slip plane.

$$T = \sum_{i=1}^{8} W_i \times \sin\alpha_i$$

As is seen $\sin\alpha_i$ can have both positive and negative values. The resisting force is the amount of shear strength acting along the length of the slip plane. The shear strength (s) is defined by the Mohr-Coulomb criterion,

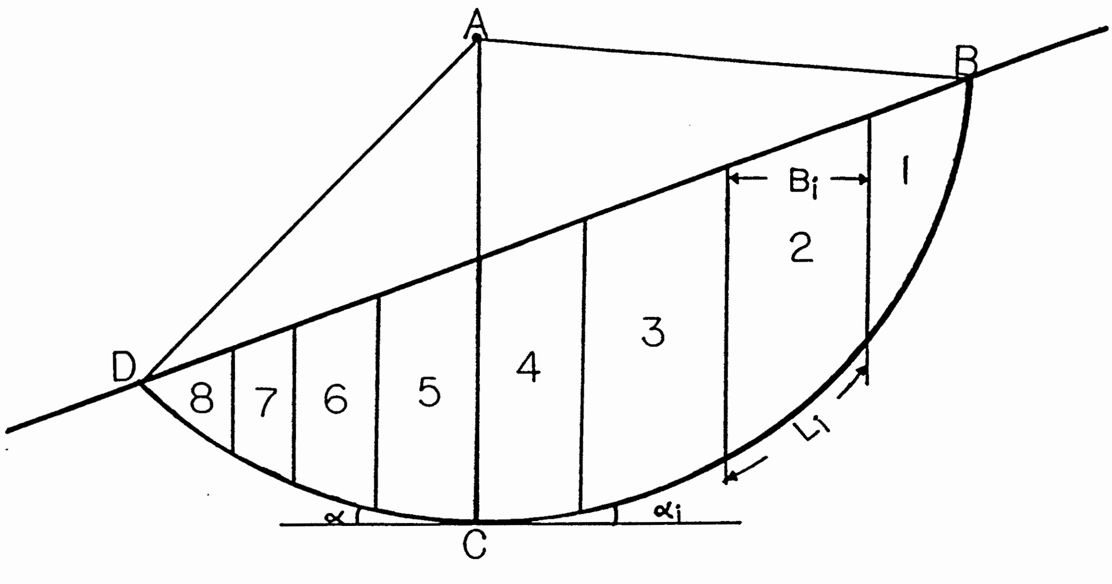
$$s = c + N \times \tan\phi$$

which relates the cohesive strength (c) of the material plus the normal stress (N) on the slip plane times the tangent of the angle of internal friction of the material (ϕ). The total resisting force is found by the summation of the shear strength times the length of the slip plane ($\sum s \times L_i$). Combining these two equations the factor of safety (F.S.) is,

$$F.S. = \left(\sum_{i=1}^{8} s \times L_i\right) / \left(\sum_{i=1}^{8} W_i \times \sin\alpha_i\right)$$

This formula represents the basic premise of all slope stability analyses. The differences arise by attempting to more accurately depict the natural conditions. Most slopes are not completely saturated.

FIGURE 42. CROSS SECTION OF SIMPLIFIED SLOPE STABILITY ANALYSIS



They have a ground water table. Because of the buoyant effect of water on the normal force across the slip plane it is important to evaluate the shear strength using the effective stress (N') for the material below the water table instead of the total stress (N). The effective stress is defined by,

$$N' = N - u$$

where (u) is the pore pressure defined as the unit weight (γ) of water times the height of water (h), $u = \gamma \times h$. The effective stress has a lower value than the total stress and when it is used instead of the total stress the resistance to shear is less.

Computer Model

The computer program used for modeling the Fountain Landslide is based on Bishop's Modified Method or the ordinary method of slices and was originally written by Guy Lefebvre of University of California, Berkely in 1971 and was later modified by S. Chirapuntu in 1972 and 1974. It was possible to evaluate the factor of safety by either the ordinary method of slices (also called the conventional method, the Swedish Slice Method or Fellenius Method) or, by Bishop's Method. The formulas for these two methods are given in Table III.

The differences between the two methods are in the manner in which the effective stress (N') is calculated and the assumptions made for the inter-slice forces. In the

TABLE III

EQUATIONS FOR SLOPE STABILITY ANALYSIS

ORDINARY	$F.S. = \dfrac{\Sigma\, cL + (W \cos\alpha - uL)\tan\phi}{\Sigma\, W \sin\alpha}$
BISHOP'S	$F.S. = \dfrac{\Sigma [cB + (W - uB)\tan\phi](1/M_\alpha)}{\Sigma\, W \sin\alpha}$
	$\left(M_\alpha = \cos\alpha + \dfrac{\sin\alpha \tan\phi}{F.S.}\right)$

summation of the equilibrium of forces on each slice, the ordinary method calculates the reactive normal force on the slip plane equal to the weight component normal to the slip plane (Chowdhury, 1978). Bishop's Method uses the total normal force. The ordinary method assumes all inter-slice forces equal to zero. Bishop's Method considers the vertical forces to be at equilibrium and the horizontal force are defined with respect to the factor of safety. With the factor of safety occurring on both sides of the equation in the Bishop's Method iterative calculations are made with successively better approximations of the inter-slice forces. The initial value of the factor of safety for the iteration is the factor of safety found by the ordinary method multiplied by 1.1. The differences between the two methods results in: 1) the ordinary method underestimating the factor of safety up to 20 percent (Attewell and Farmer, 1976), 2) the Bishop Method only underestimating the factor of safety up to 7 percent. A large error is found using the Bishop Method when the failure arc is deep (Bell, 1980) or steeply inclined (Chowdhury, 1978). This is the because when α is large the M_{α} term (see Table III) becomes large. The inter-slice forces become unreasonably large when this occurs and the factor of safety is greatly underestimated. When the failure arc is deep or steeply inclined Bishop's Method should not be used. For further explanations of the differences between various methods of calculating slope stability see Bell (1980) or Chowdhury (1978). Under most circumstances Bishop's Method is more accurate. This method was used in this study.

Procedures

The information the program needs to evaluate the minimum factor of safety includes the slope and material geometry, the position of the ground water surface, the material properties including the unit weight (γ), the cohesive strength (c), the internal angle of friction (ϕ) and the specifics on how to search for a critical circle.

Describing the geometry of the slope and the slope material was accomplished by establishing a coordinate system on the cross sections and establishing slice widths to correspond with changes in either the slope or subsurface geometry. The end points of the unit boundaries were assigned values. The lines between these points are assumed to be straight. This, along with size limitations of the program, resulted in the simplification of the cross sections. The geometry of the models A, B and C is shown in Figure 43, the original cross section were shown in Figure 36.

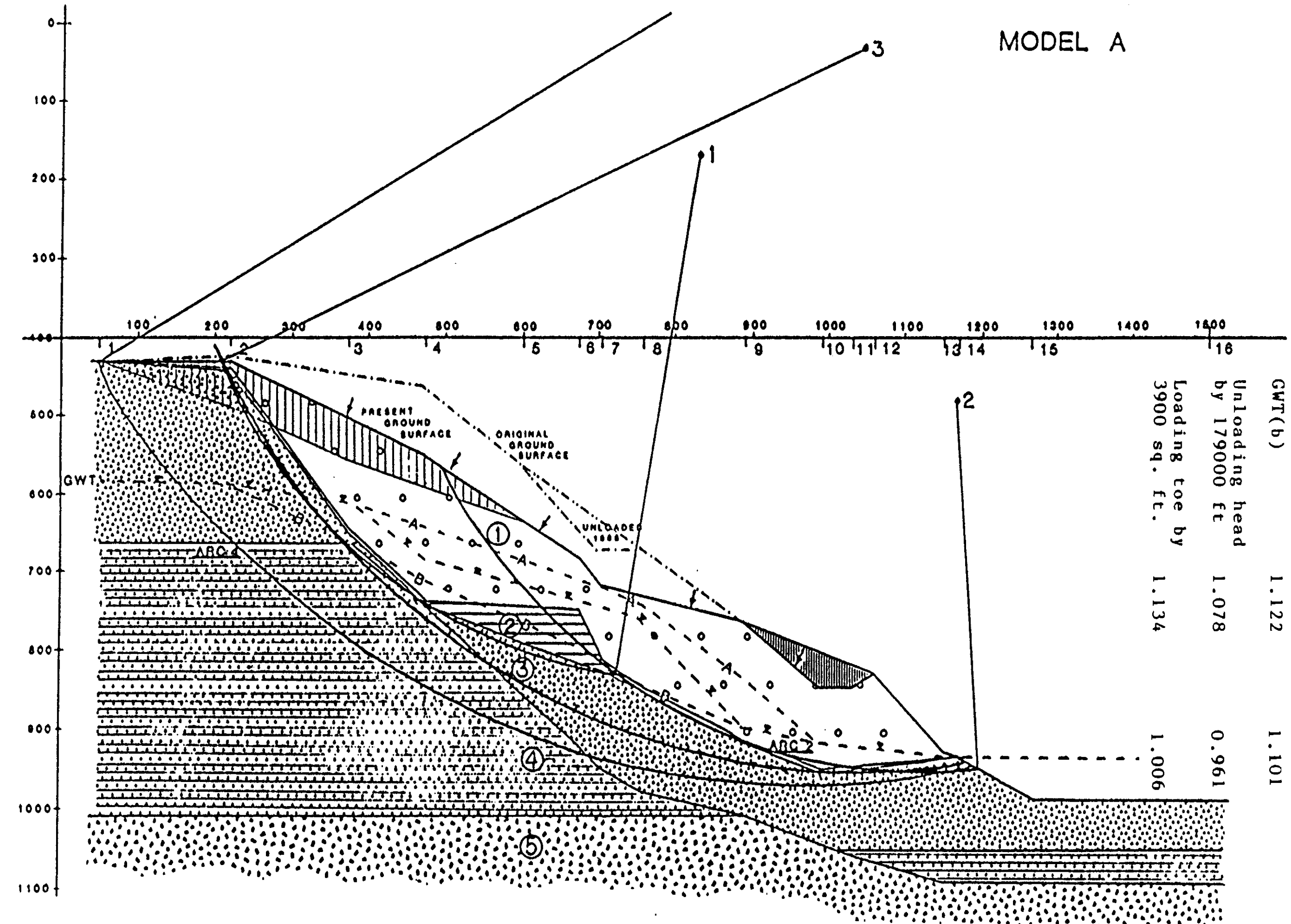
Each material unit was described by the cohesive strength (c), the internal angle of friction (ϕ) and the unit weight (γ). Within natural materials the value for each of these properties vary, so a range or average value must be estimated. The initial property values for each unit were derived four ways: (1) from field measurements, (2) laboratory measurements, (3) common usage and (4) tables. Field measurement of 29 degrees for the angle of repose for the CRB talus slope was used to approximate the angle of internal friction for the talus. The undefined laboratory determinations by the Oregon State Highway Division of the slip plane material were used. From these tests the cohesive strength ranged from 1008 to 3124 P.S.F. and the angle of internal friction ranged from 5 to 9 degrees for the slip plane material. Specific gravity was determined by pycnometer tests for the mudstone unit (132 P.C.F) and the volcaniclastic unit (135 P.C.F.). Standard laboratory determinations for the specific gravity of the intrusive (170 P.C.F.) and the WRL (113 P.C.F.) were used. Common usage by the Army Corps of Engineers for other landslides in the Columbia River Gorge comprised of CRB talus was used for the remaining property values for the CRB and WRL talus. Table values from Palmer (1982) were used for the following property values: sandstone ($c = 0$, $\phi = 25$ degrees, $\gamma = 130$ P.C.F.), volcaniclastic unit ($c =$ 2000 P.S.F., $\phi = 20$ degrees), and the intrusive ($c = 4500$ P.S.F., $\phi = 45$ degrees). In most slope stability analyses when failure has occurred more accurate material properties can be back calculated because it is known that the factor of safety is equal to one by definition at failure. This was done for each model. The back calculated values for each unit in each model are given in Figure 43 A, B, C.

The computer program can calculate the effective stress in one of three ways. 1) By entering the ground water table as a boundary in the geometry portion and including the submerged unit weight of the material below that boundary. 2) By entering the depth of the ground water table. 3) By using flow nets to enter the equal pressure lines of the pore water pressures. The ground water table was estimated from Plate 1 and method 2) was then used by the computer to calculate the effective stress on the slip plane.

The program either searches for the critical circle or calculates the factor of safety for any specified failure arc. In most cases the critical circle search was used to find the minimum factor of safety for each model. Calculation of the factor of safety for a specified arc had to be used to model cross section A. Since the geometry of the slip plane was well defined by slope inclinometer data, the accuracy of the specified arc calculations not significantly less than the critical circle search

Modeling

Three cross sections of the Fountain Landslide were modeled (see Plate 1 for locations). Each model represented a different portion of the landslide. For each model a search was made for the critical circle. The



	ARC 2 c (lb/ft²)	ARC 2 φ (°)	ARC 2 γ (lb/ft³)	ARC 3 c (lb/ft²)	ARC 3 φ (°)	ARC 3 γ (lb/ft³)
Unit 1	0	32	115	0	33	115
Unit 2	1000	6	130	1200	9	130
Unit 3	0	32	115	0	33	115
Unit 4	2400	13	135	2400	13	135
Unit 5	4500	45	170	4500	45	170

Changes in F.S. with:	ARC 2	ARC 3
GWT(a)	0.982	0.899
GWT(b)	1.122	1.101
Unloading head by 179000 ft	1.078	0.961
Loading toe by 3900 sq. ft.	1.134	1.006

Figure 43A. Cross sectional model A. See Figure 36A for original cross section.

WRL talus | Mudstone | Intrusive | Proposed unloading
CRB talus | Sandstone | Volcaniclastic | Proposed loading

Elevated GWTa —A— — — Lowered GWTb—B— — — GWT from Plate 1— ⊻ — ⊻

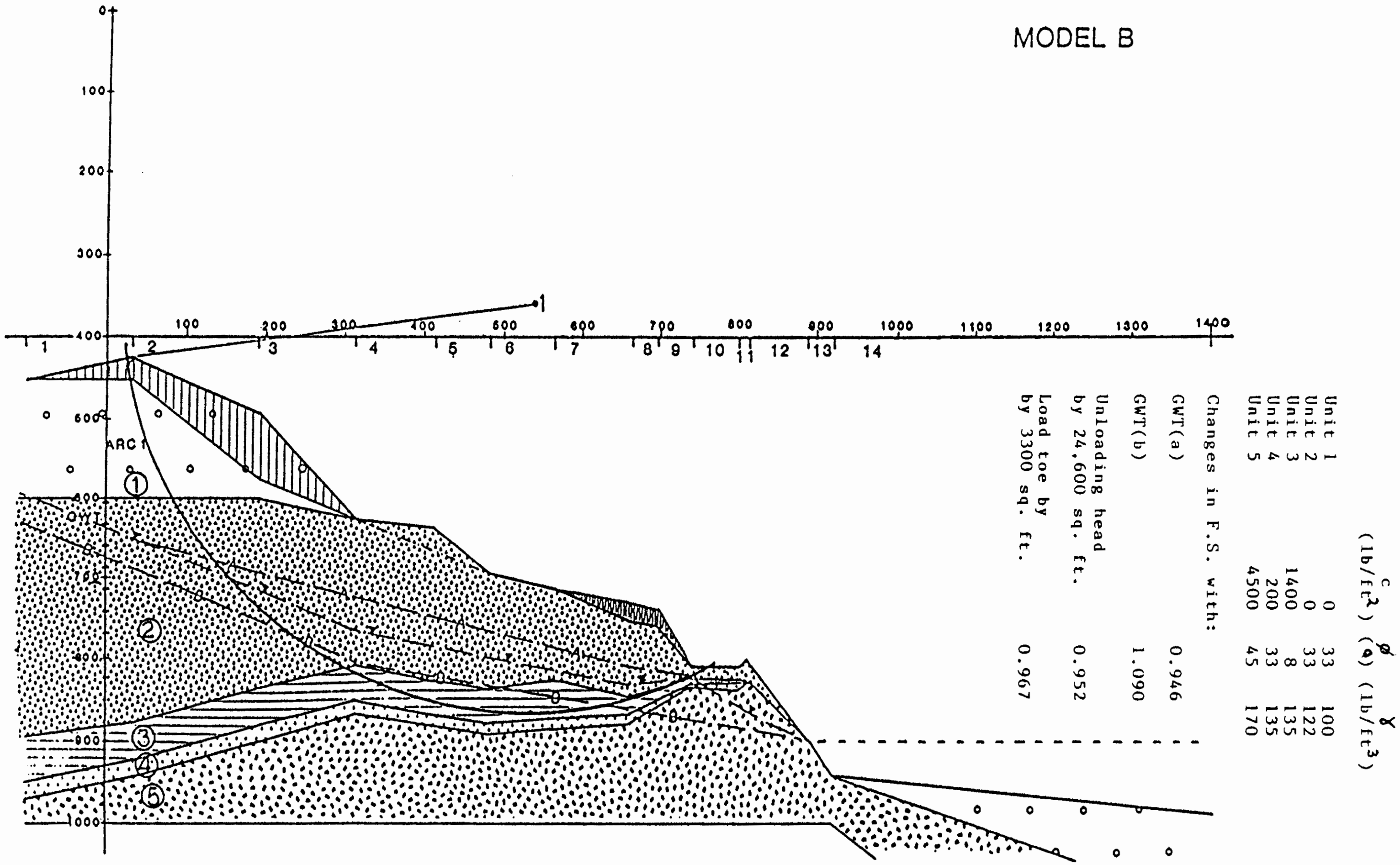


Figure 43B. Cross sectional model B. See Figure 36B for original cross section.

WRL talus, CRB talus, Mudstone, Sandstone, Intrusive, Volcaniclastic, Proposed unloading, Proposed loading

Elevated GWTa —A— — — Lowered GWTb —B— — — GWT from Plate 1 — ▼ — ▼

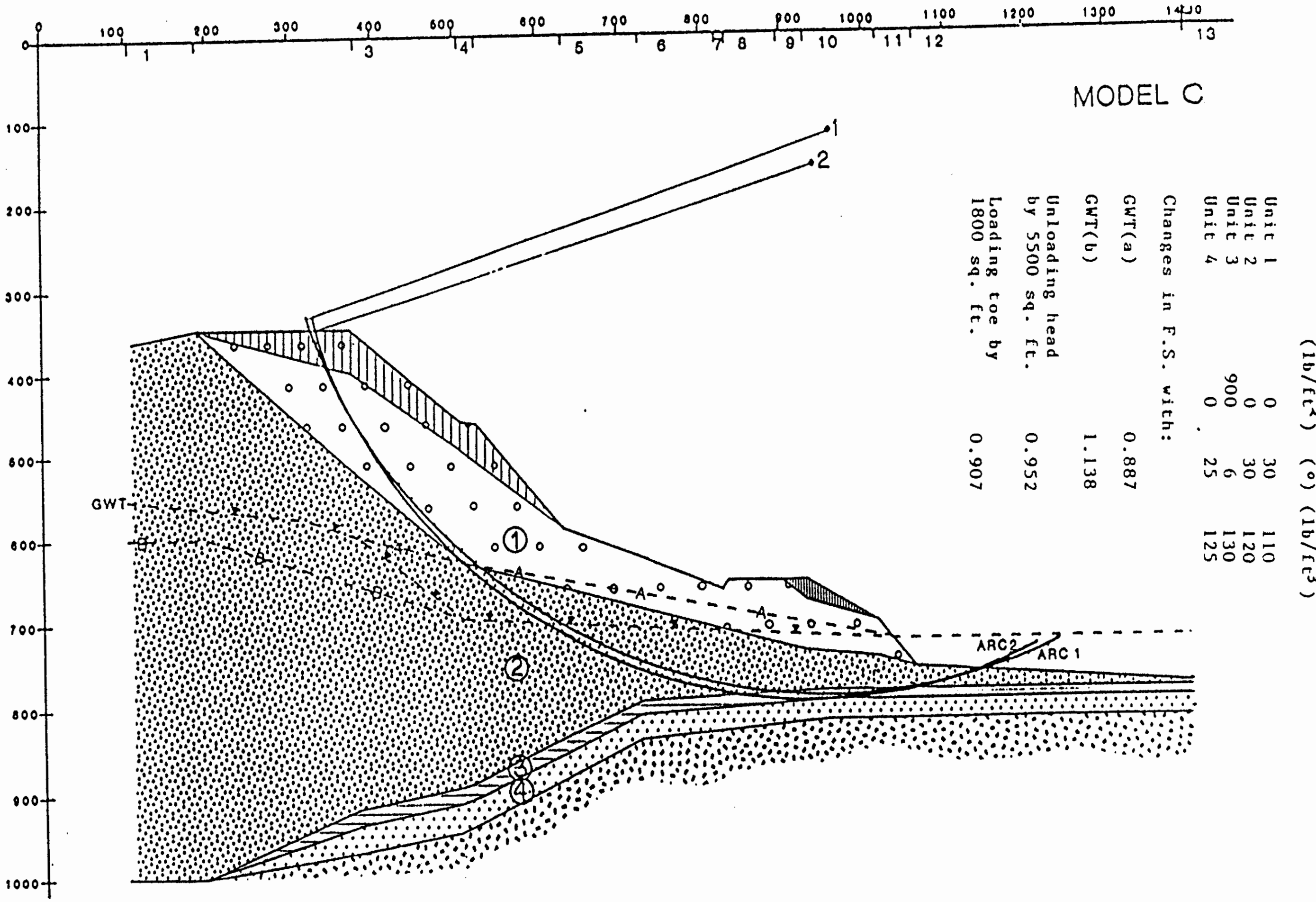


Figure 43C. Cross sectional model C. See Figure 36C for original cross section.

WRL talus Mudstone Intrusive Proposed unloading

CRB talus Sandstone Volcaniclastic Proposed loading

Elevated GWTa —A— — — Lowered GWTb—B— — — GWT from Plate 1— ⊻ — ⊻

search proceeded from the top of the intrusive unit upward until a minimum factor of safety was found.

The unit boundaries were continued laterally without supporting core data because the program allowed for boundaries to coalesce but not to be discontinuous. The extrapolations are not meant to represent the actual subsurface conditions, nor do they significantly effect the calculation of the factor of safety.

The ground water table shown in Plate 1 does not reflect the yearly high ground water table on the slide, therefore a water table up to 50 feet higher, labeled GWTa, was described for each model to compare the effect on the factor of safety. The criteria for the elevation of GWTa in each model was known surface springs and the subsurface geometry of the ground water table and the material units.

<u>Model C</u>

Modeling began with C (Figure 43C) through the eastern portion of the landslide because it had the least complex geometry. The original surface on cross section C has been altered by the addition of fill near the toe of the landslide during highway construction. Figure 43C shows the number and width of the slices along the bottom of the horizontal index, the present ground surface, the depths and thicknesses of the WRL talus, CRB talus, mudstone unit and the sandstone unit. The critical circle ARC 1 is marked, along with its center and radius. The ground water table (GWT) from Plate 1 is marked along with the higher (GWTa) and lower (GWTb) ground water tables.

A second critical circle, labeled ARC 2 on model C, was used because it fit the slope inclinometer data better than ARC 1. ARC 2 is a little deeper than ARC 1 and only increased the factor of safety from 1 (ARC 1) to 1.01 (ARC 2). To improve the fit of ARC 2 to the slope inclinometer data, the talus was divided into two parts. This was done so that the unit weights could be varied to reflect the lighter unit weight of the WRL (110 to 120 P.C.F.). The better match in this model led to the same separation of the talus for the other models.

The back calculated material properties (Figure 43C) for model C are given for ARC 1. Model C had the lowest back calculated material properties of the three models. The lower shear strength indicates that model C is the most stable. When the ground water table was elevated to GWTa (Figure 43C) the factor of safety was decreased to 0.887.

Model B

Model B is through the middle section of the Fountain Landslide. Changes in the original surface include the cut in the toe for highway construction, the waste deposit from the 1968 unloading, and material removed at the gravel pit. The depth of the critical circle for model B was found to occur within a very narrow range. The critical circle in this model as in model C, fits the slope inclinometer data best at the toe. The critical circle for both models B and C extends farther south than surface features and the slope inclinometer data suggest.

The back calculated material properties values (Figure 43B) at failure for model B were higher than model C. The factor of safety was only slightly lower (0.946) when the groundwater table was raised to GWTa, the result of only a 35 foot increase.

The addition of the waste from the 1968 unloading slightly increased the factor of safety from 1 to 1.016.

Model A

Model A passes through the unloaded western portion of the Fountain Landslide. This cross section has had the greatest movement during and after construction of the highway. The survey results (Appendix B) indicate that it is still moving during the wet months. The factor of safety for the slide therefore must be equal to one at least during the wet months. Model A was the most difficult to model because of the presence of at least two slip planes and the complicated subsurface geometry (Figure 43A). The critical circle search could only be made on the lower slip plane ARC 2. The search could not be completed for the upper slip plane (ARC 1) and the single larger slip plane (ARC 3). The steepness of ARC 1 and depth of ARC 3 resulted in extremely large errors in calculating these two factors of safety. To remedy this, both ARC 1 and ARC 3 were calculated using a specified slip plane. It was still necessary to use the ground surface before the 1970 unloading to prevent factor of safety of ARC 1 from equalling zero, when reasonable material property values were used. The specified critical circles in both instances resembled the slope inclinometer data.

There were four parts in modeling cross section A. First, since the accuracy of the initial property values for the volcaniclastic unit were uncertain, back calculated values were needed. The limitations of the program made it necessary to use the ground surface before the highway cut was made. Increasing the weight at the toe of the landslide avoided the large errors associated with deep failure arcs. The critical circle for this model (ARC 4) is shown on Figure 43A. The material property values used for the other units were derived by averaging the values back calculated for models B and C. Using this method, the material properties for the volcaniclastic unit were, c = 2400 P.S.F., ϕ = 13 degrees, and γ = 133 P.C.F.. These property values were used for the remaining slope stability analyses for cross section A.

The next step in modeling was searching for a critical circle for ARC 1. As was previously stated this was not possible. A specified arc resembling inclinometer data and the original ground surface was used to model ARC 1. The back calculated material property values at failure for ARC 1 were the same as those calculated for ARC 3 (Figure 43A).

The third part of modeling section A was modeling the lower slip plane (ARC 2). This part of modeling included a search for a critical circle. The critical circle, ARC 2, did not fit the slope inclinometer data well, which would be expected, since not all of the slope forces are included in ARC 2. The material properties at a factor of safety of one are given on Figure 43A. The elevated ground water table (GWTa) did not greatly effect the factor of safety of ARC 2, since it was elevated in only a small part of its slide mass.

The last step in modeling A was then to model both ARC 1 and ARC 2 as a single larger ARC 3. The failure surface was specified so as to best resemble slope inclinometer data. An approximation was made using the boundary at the head and the depth of movement at the two lower slope inclinometer data points. This arc probably best represents the slope stability for model A since the entire slope is modeled. The material properties back calculated at failure were slightly higher than ARC 2. This indicates that ARC 3 is less stable than ARC 2. The elevated ground water table (GWTa) significantly decreases the factor of safety for this failure arc from 1.0 to 0.899.

Modeling for Design Purposes

For each model the effects of lowering the ground water table and lowering the driving forces were investigated. The changes used for each model are given in Figure 43 A, B, C. Lowering the driving force may be done by removing material near the head of the slide or adding material to the toe. There are several ways in which the resisting forces can be increased such as dewatering the slide mass, installing piles or anchors, chemical treatments, electroosmosis, and thermal treatments. In this study increasing the effective stress by lowering the ground water table will be examined. Similar increases of the resisting forces using one of the other methods would result in similar increases in the factor of safety.

The effects of unloading the head of the slide, or adding material to the toe (simulating buttressing) was investigated and is shown in Figures 43 A, B, C. It is important to note that the elevated ground water table GWTa was used for these design considerations, to simulate the worst possible design condition. The factor of safety for GWTa given for each model (Figures 43 A, B, C) should be used as reference for the magnitude of change in the factor of safety.

The effect of lowering the ground water table (GWTb) on the factor of safety was investigated. The results are given in Figure 43 A, B, C. As can be seen in each model small changes in the ground water table greatly influences the factor of safety; much more than unloading or buttressing the slide mass. The increase in the factor of safety by lowering the water table ranged from 12 to 22 percent; the other two methods generally increased it from 1 to 16 percent. The restrictions upon the amount of material removed or added, greatly influences the amount of increase in the factor of safety. Ideally, dewatering the slide mass would therefore be the preferred method of stabilization.

Discussion

The back calculated material property values for the same units express the different stabilities of each model. The lower strengths required to get stability indicate a higher intrinsic stability as in cross section C. The least stable model was cross section B which required the highest material property values to obtain a factor of safety of 1. The range of values from Figures 43 A, B, C for each unit are:

Unit	c (P.S.F.)	ϕ (degrees)	γ (P.C.F)
WRL talus	0	30-33	100-115
CRB talus	0	30-33	120-130
Mudstone	900-1400	6-9	130-135
Sandstone	0-200	25-35	130-135
Volcaniclastic	2400	13	135
Intrusive	4500	45	170

The small range of values increases the confidence in the values used for the analyses and also indicates the units have consistent properties throughout the slide.

The slope stability analyses showed that the differences in stability between the eastern and western portions of the landslide are the result of: 1) the cut in the toe of the western portion, 2) the emplacement of fill in the eastern portion, 3) the higher ground water table in the western portion, 4) lower ground water table in the eastern portion, and 5) the buttressing effect of the intrusive.

The critical circle searches in models B and C and the specified arc in model A, do not precisely match the slope inclinometer data. To fit the critical circle closer to the slope inclinometer data the unit weight of the talus was varied. The poor final fit indicates that the natural slope material is not isotropic and homogeneous and the slip planes are not precisely circular. The slip planes in B and C resemble a basal failure arc (National Academy of Science, Transportation Research Board, 1978) which results from the lower shear strength of the mudstone unit and the higher strength of the talus. The slip planes are more complex in model A, most likely the result of several episodes of movement. The critical circles and the specified arc used for the analyses resulted in an underestimation in the factor of safety in all cases. More material at the head was included within the slide mass for the models than the surface indications show.

Cross section A was difficult to model because its slip plane geometry was incompatible with the method of calculation. By not modeling each slip plane separately the complex interactive force between the two can only be generalized and not quantified. In effect the lower slide mass acts as a buttress for the upper slide mass. The upper mass adds a horizontal force to the lower mass. Therefore the upper mass is trying to push the lower mass into failing and the lower mass is trying to hold the upper mass up. The 1970 unloading favorably affected the stability of model A as a whole, but its effect on either ARC 2 or ARC 1 separately does not show this effect. This indicates that the inter-mass forces between these two are high. For design purposes ARC 3 would best describe the slope stability of A.

The effects of three stabilization procedures on the factor of safety were investigated for each cross section. It was shown that lowering the ground water table has a greater effect than unloading or buttressing had.

The following problems are associated with additional unloading or buttressing: 1) the Fountain Landslide is a large landslide, and 2) the highway restricts the areas suitable for construction. The entire slide would have to be addressed to avoid problems similar to those which occurred after the two previous unloadings.

Previous attempts to dewater the Fountain Landslide by the Oregon State Highway Division (tunnel excavation and vertical drains) gave poor results, mostly due to the complex nature of the ground water system. The tunnel excavation did not extend deep enough into the slide mass to reach the area where the ground water table was high. Since the western portion is moving only when the ground water table is high, a factor of safety equal to one is reached only at that time. Dewatering the slide mass to prevent the elevation of the ground water table during times of high recharge appears to be one of the best methods of stabilization. If dewatering is attempted, additional studies of the permeability of the layers in the slide would be necessary.

DISCUSSION AND CONCLUSIONS

The purpose of this study was to examine: (1) which of the previously given hypotheses for development of the slope instability for this area best fit the data, (2) the causes for, and type of slope instability, and (3) why past measures taken to stabilize the slope did not work.

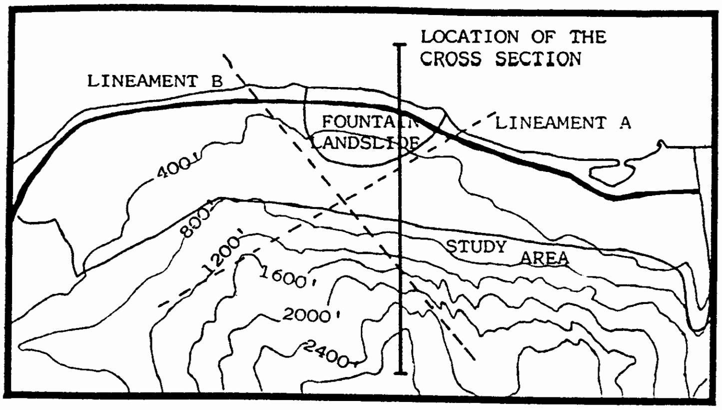
The two hypotheses presented in the introductory section of this report were by: 1) A. C. Waters (1973), and 2) the Army Corps of Engineers (1971a). Waters (1973) describes the instability in this area as the result of an incompetent layer squeezing out beneath a heavy pile of stronger material. Waters hypothesis does not describe the conditions at the Fountain Landslide. Supporting this conclusion is: (1) the volcaniclastic unit is found at depth in drill holes in only a small portion of the landslide, (2) when found the volcaniclastic unit is described as having intact bedding features, (3) the ground water table is perched in the western portion of the landslide, (4) The intrusive would act like a buttress to the squeezed out material restricting its progress, and (5) instead of the thin soil like material found above the intrusive, a layer of the volcaniclastic unit would be expected.

A better description of the conditions at the

Fountain Landslide was given by the Army Corps of Engineers. The Army Corps of Engineers (1971a) finds that the instability here is caused by perching of the ground water table in the permeable talus above the impermeable volcaniclastic unit. The perching at the Fountain Landslide is the result of impermeability of both the volcaniclastic unit and the intrusive. An argument against the U.S. Army Corp of Engineers hypothesis is that the ground water table is not perched throughout the entire landslide. This suggests that additional factors are involved with the mechanics of the Fountain Landslide.

I propose a different hypothesis for the development of the Fountain Landslide. The development I propose is described in the cross sections presented in Figures 44 A through F. The cross sections reconstruct episodes in the geologic history of the slide. The cross sections are located (unless otherwise specified) in the eastern portion of the landslide (see cross section C in Figure 36). Figure 44A begins just prior to the Cascadian uplift. As shown on Figure 44A, at this time (6 m.y.b.p.) the intrusion is in place as are the multiple flows of the Columbia River Basalt Group. For reference on this figure the location of the present ground surface, river level, and river position are marked. The hypothetical ground surface 6 m.y.b.p. is above the present ground surface.

Slide development could be influenced by the manner



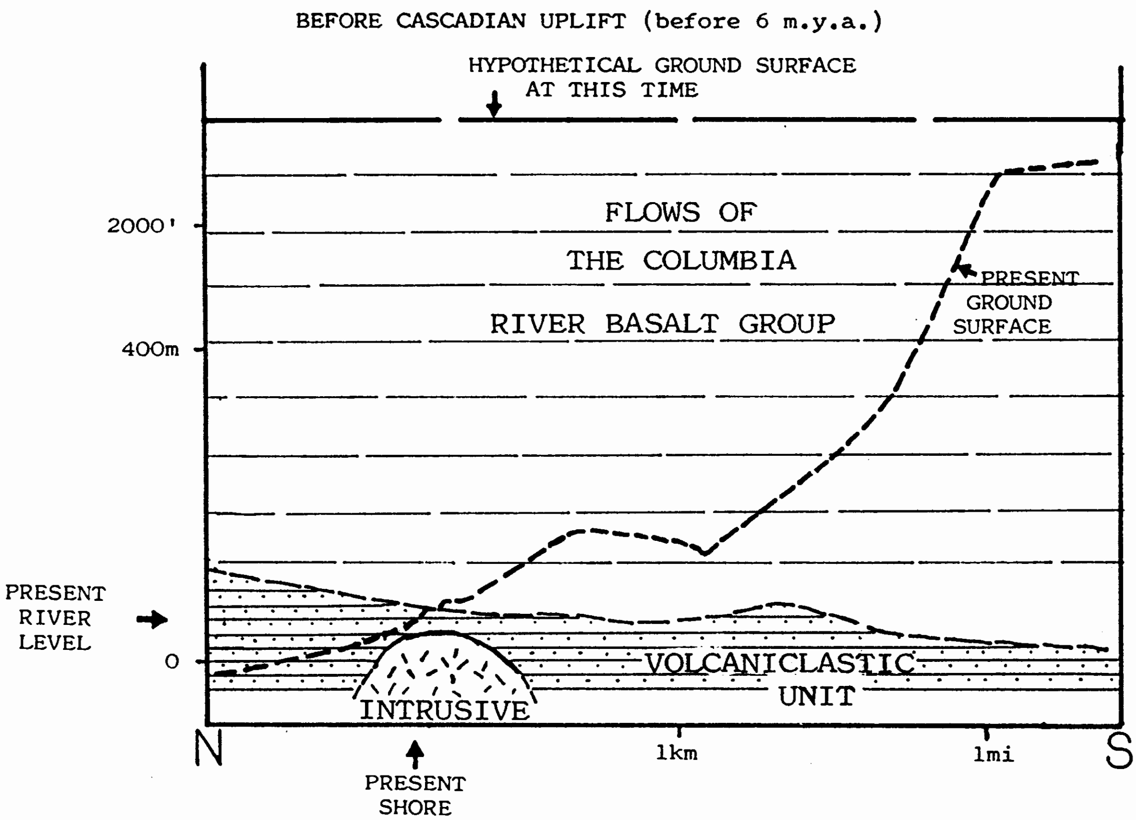
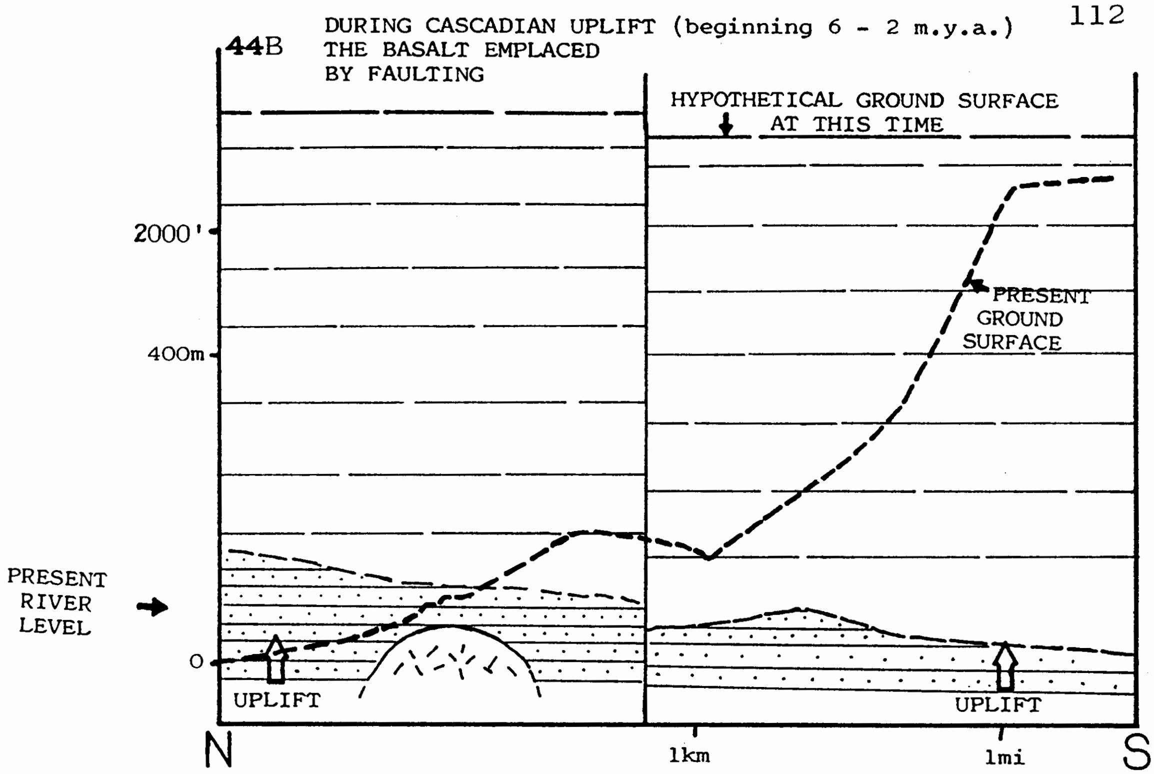
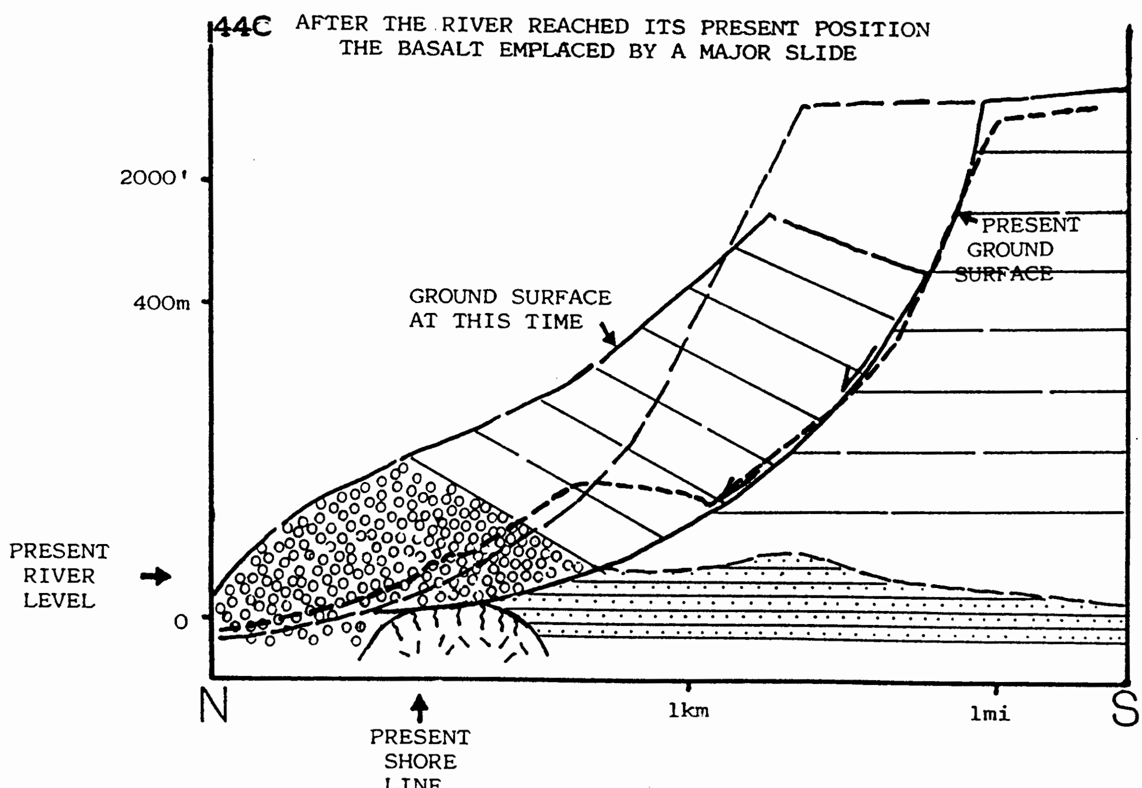


Figure 44A. The geologic history of the development of the Fountain Landslide.

in which the CRB within the study area was emplaced. The three hypotheses presented in the "Discussion of the Stratigraphy" section of this report for the emplacement of the CRB within the study area were: (1) it slid into place from the cliffs to the south, (2) it is from the lower most part of a conformable sequence with the cliffs to the south, or (3) it was faulted into place. The data is not sufficient to undeniably support or refute any of these, but 1) and 3) seem most probable. If the basalt was faulted into place the development of the landslide may have begun during the Cascadian uplift (Figure 44B). If the basalt slid into place within the study area (Figure 44C), it could not have occurred until after the river reached its present level (before 338,000 years ago) in order for sufficient erosion of the lateral support and undercutting slope to occur. In either case, (Figures 44B and C) the intrusive had to be exposed and weathered for the soil layer to have developed. Basalt talus from the cliffs to the south was then deposited over the soil layer (Figure 44D). Next the Wind River Lava dammed the Columbia River and was deposited in the study area (Figure 44D) about 338,000 years ago. Erosion of the Wind River Lava dam began first at the south contact with the talus slope (Figure 44D). Contraction and sagging of the Wind River Lava in the middle of the channel occurred as the lava cooled, which encouraged the river channel to shift to the





Figures 44B and 44C. The geologic history of the development of the Fountain Landslide.

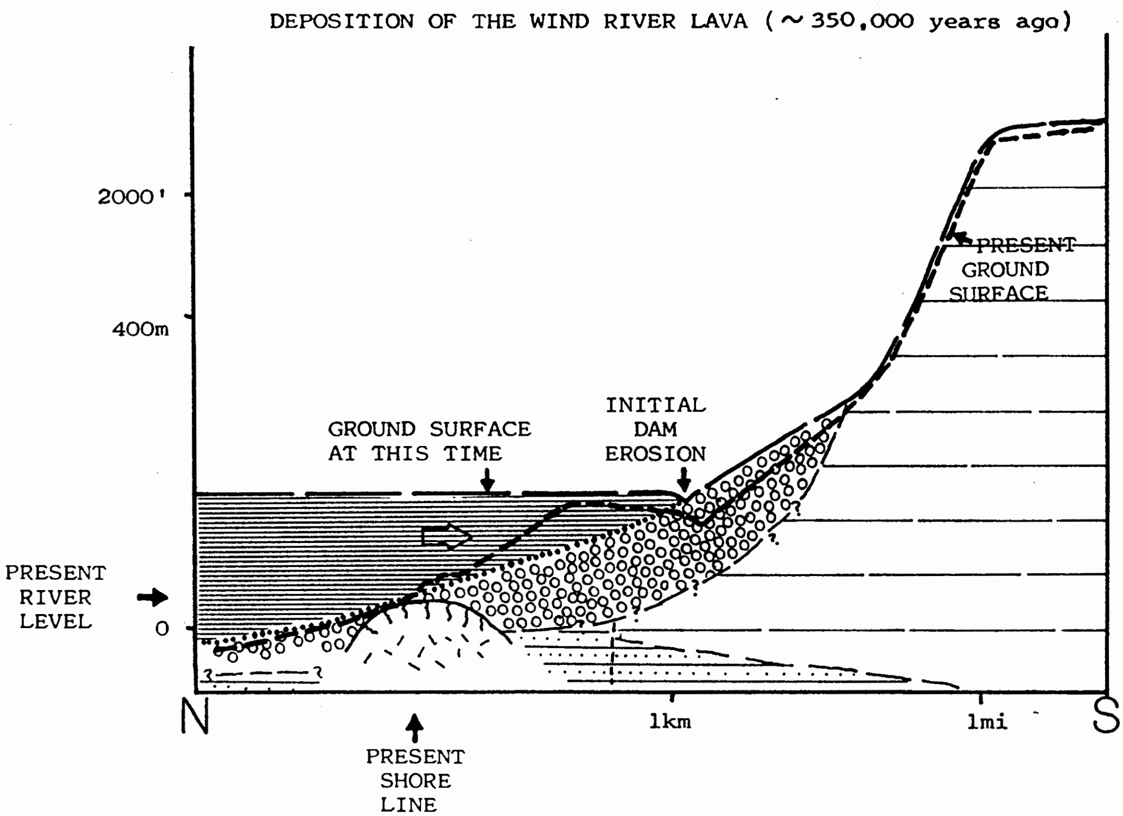
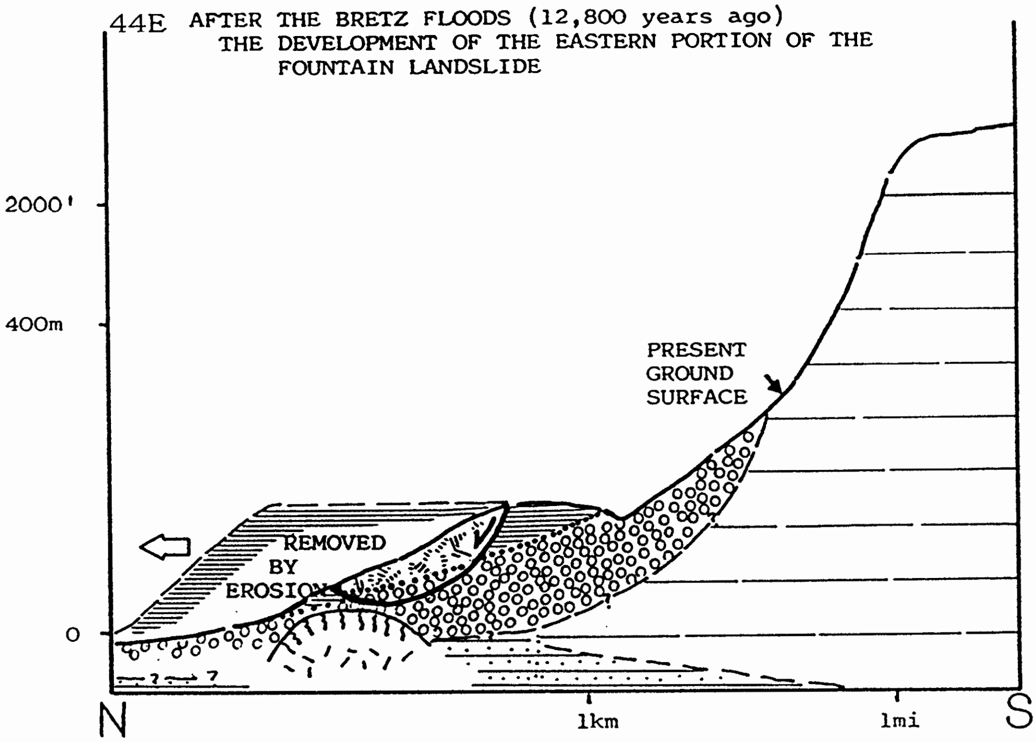
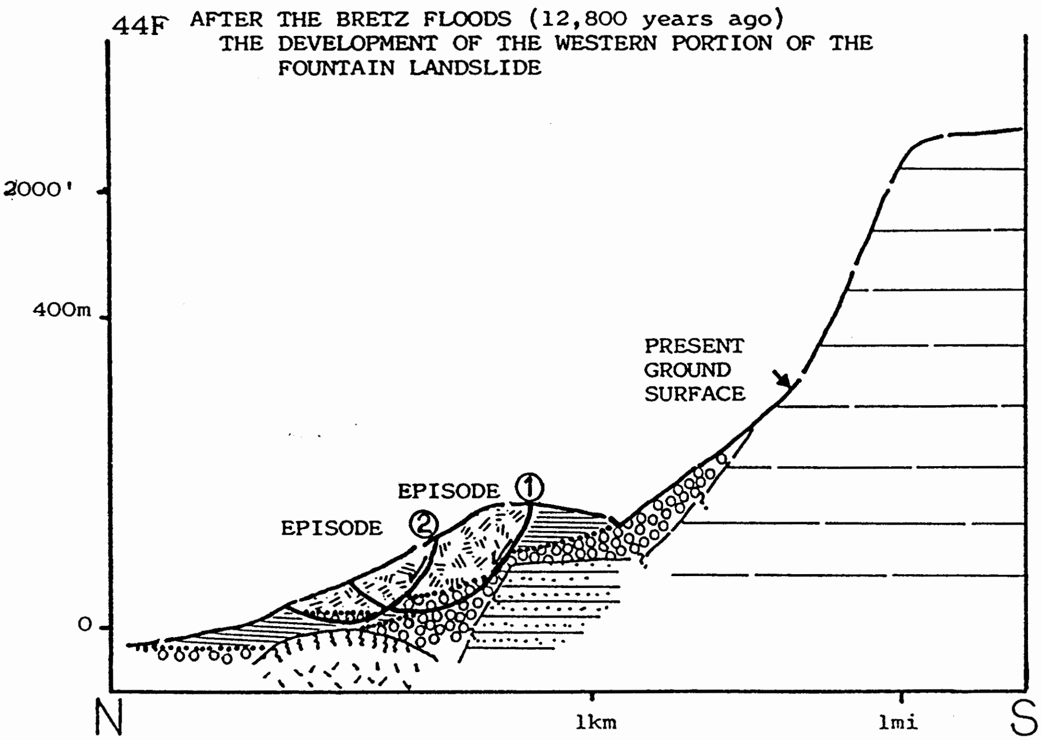


Figure 44D. The geologic history of the development of the Fountain Landslide.

now lowest part of the dam. Erosion of the dam continued through the middle of the channel. As the lateral support for the heavy pile of WRL lava on the existing talus slope was removed the present sliding began (Figure 44E). The differences between the eastern and western portions appear to be the result of differing subsurface conditions prior to the deposition of the Wind River Lava (Figure 44F). There was a steeper depositional surface in the western portion, which created higher instabilities and resulted in more episodes of movement and more failure surfaces.

The instability of the present Fountain Landslide was examined in the engineering geology section of this report. There appears to be two independently moving slide masses, the eastern and western portions. This is probably due to the intrusion confining the base of the slip plane in the central portion of the slide (see cross section B in Figure 36). Both appear to be rotational slides with a nearly circular slip surfaces. Although the two masses move independently, the 1970 unloading showed that any reduction or increase in the stability of one portion of the landslide effects the remaining portion. Movement presently occurs in the western portion when the ground water table is high. By modeling the slope stability it was shown that the cut into the toe for freeway construction severely reduced the stability of the slope. The modeling also suggests that theoretically, it would be





Figures 44E and 44F. The geologic history of the development of the Fountain Landslide.

possible to enact effective stabilization procedures.

There are several reasons why the remedial measures taken by the Oregon State Highway Division did not stop the movement of the Fountain Landslide. The major problem was that there was insufficient geologic information about the location of the boundary of the preexisting landslide. The efforts were designed for a much smaller landslide than actually existed. This was most clearly seen in 1968, when material from the middle of the western portion of the slide was removed and redeposited in the middle of the central portion. The horizontal drains and tunnels were not sufficiently deep to drain the entire landslide, again designs were for a smaller slide. The unloading in 1970 increased the stability of the slide and resulted in slower movement but did not stabilize it. Any of these stabilization procedures may have stabilized the Fountain Landslide if the designs would have considered the larger slide mass.

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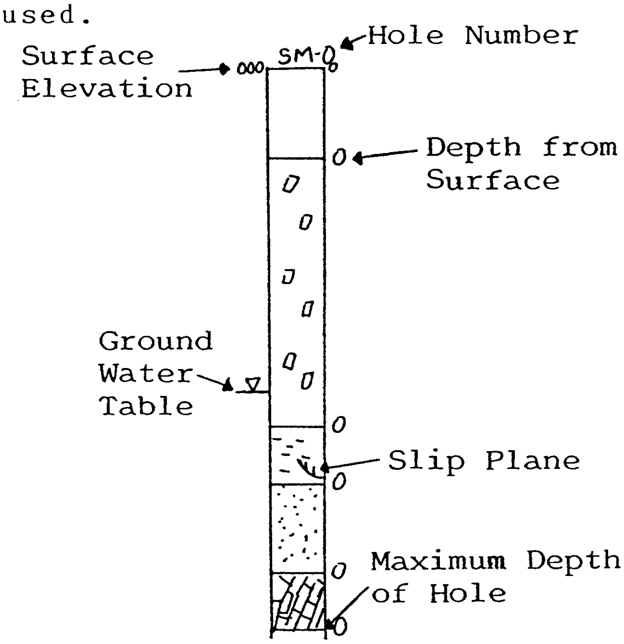
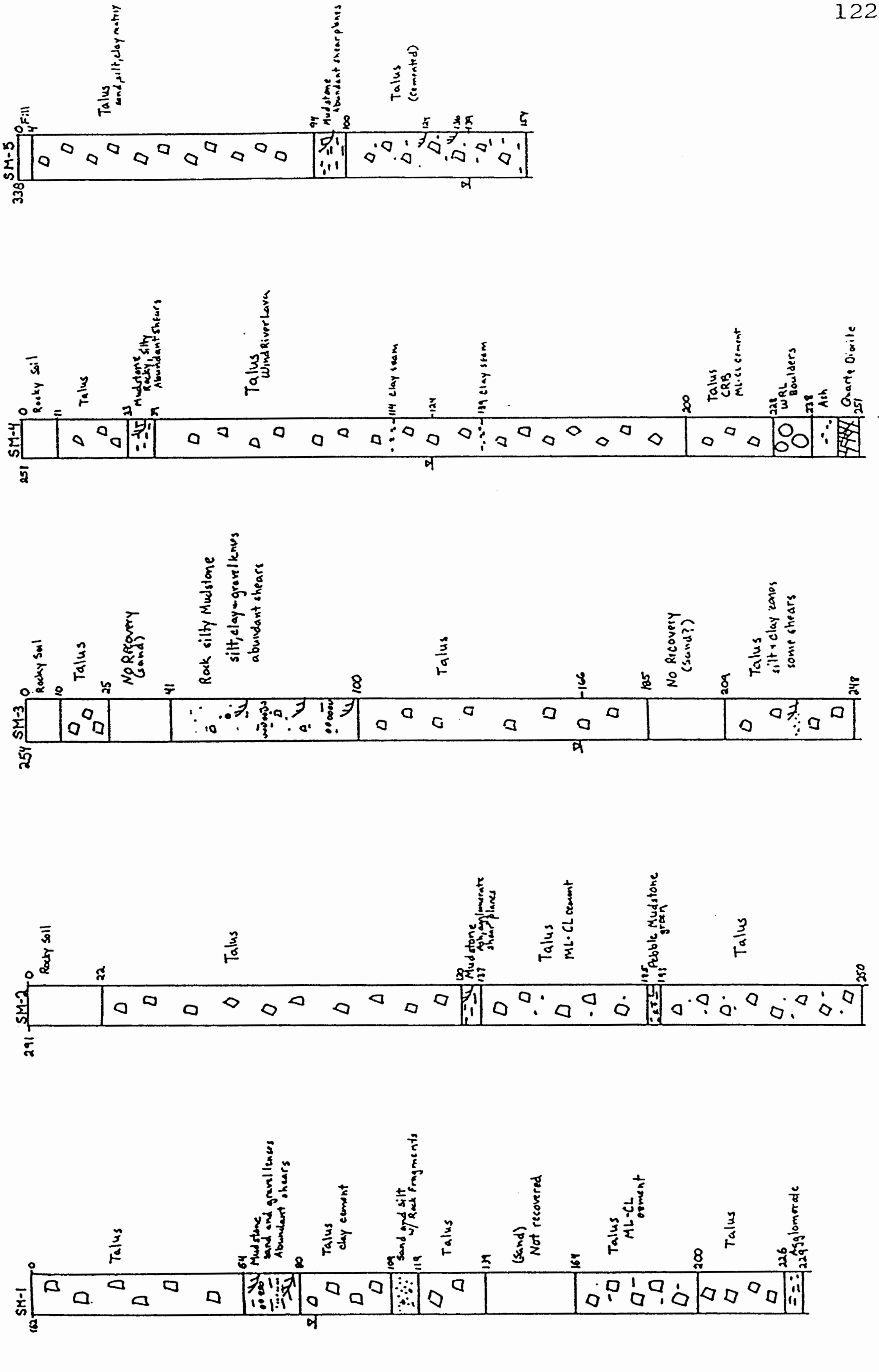
APPENDIX A.

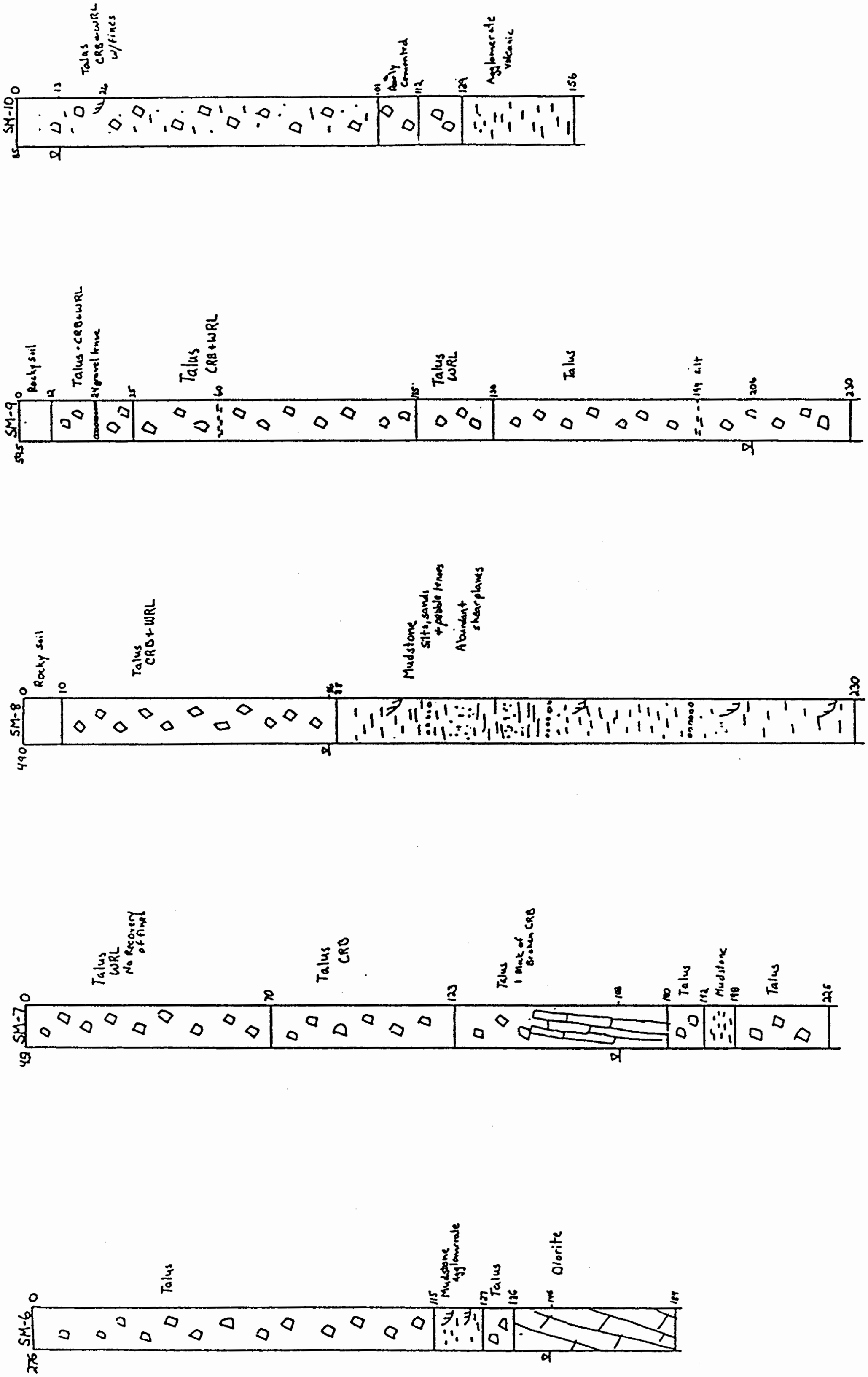
OREGON STATE HIGHWAY DIVISION PRELIMINARY CORE LOGS SM1 TO SM64. SEE FIGURE 15 FOR LOCATIONS

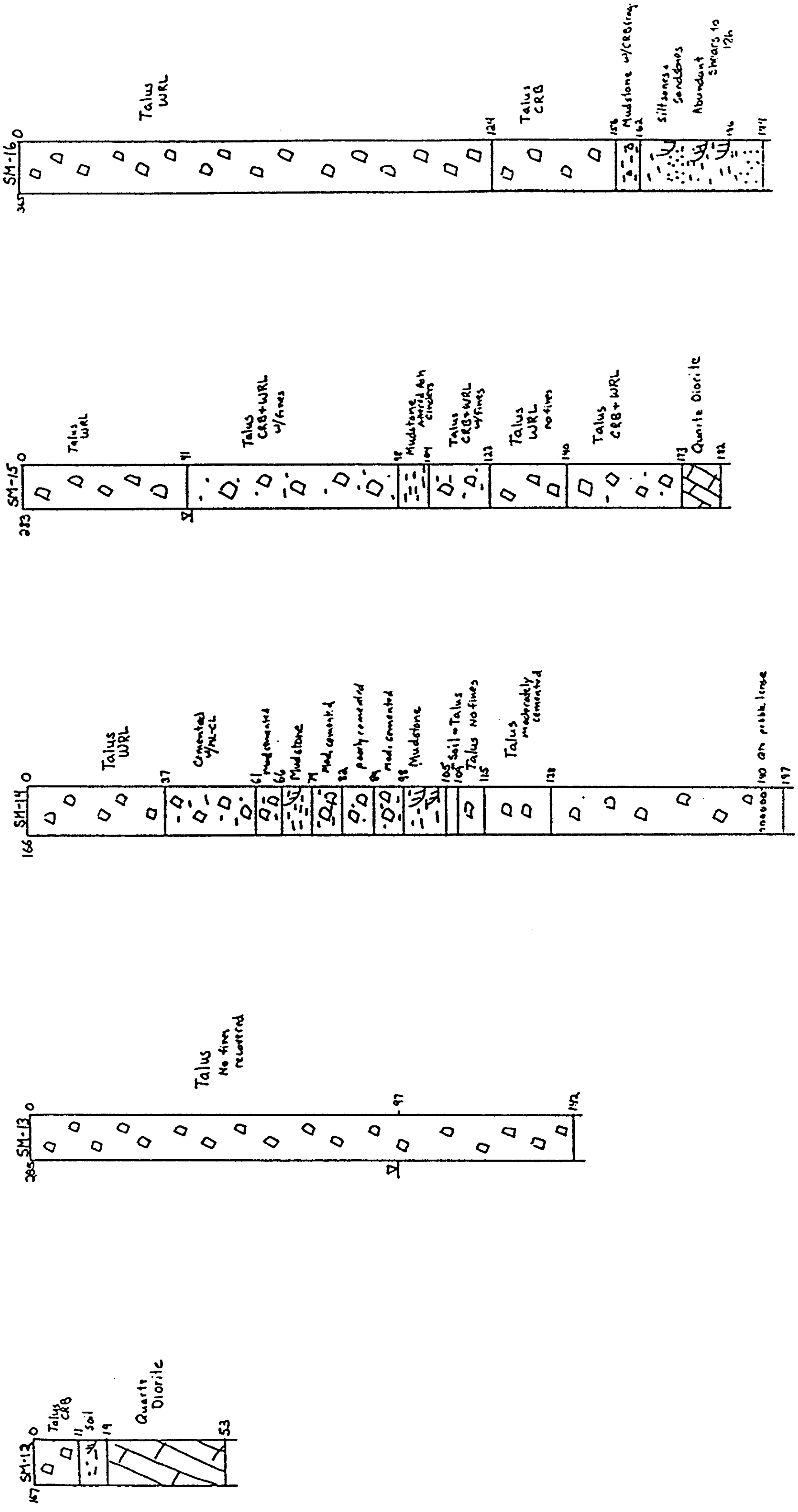
The logs presented here represent the material divisions marked by the Oregon State Highway Division. All unit boundaries are in feet and were rounded to the nearest whole number. The following interpretations were made: 1) black or basalt talus is called CRB talus, 2) pink, scoria, or andesite talus is called WRL talus, 3) bedrock, buff rock, or andesite intrusive is called quartz diorite intrusive, and 4) sandstone, shales, ash, conglomerate, or agglomerate was called the volcaniclastic unit. When it was not clear which unit was described the description in the Oregon State Highway Division preliminary logs were used.

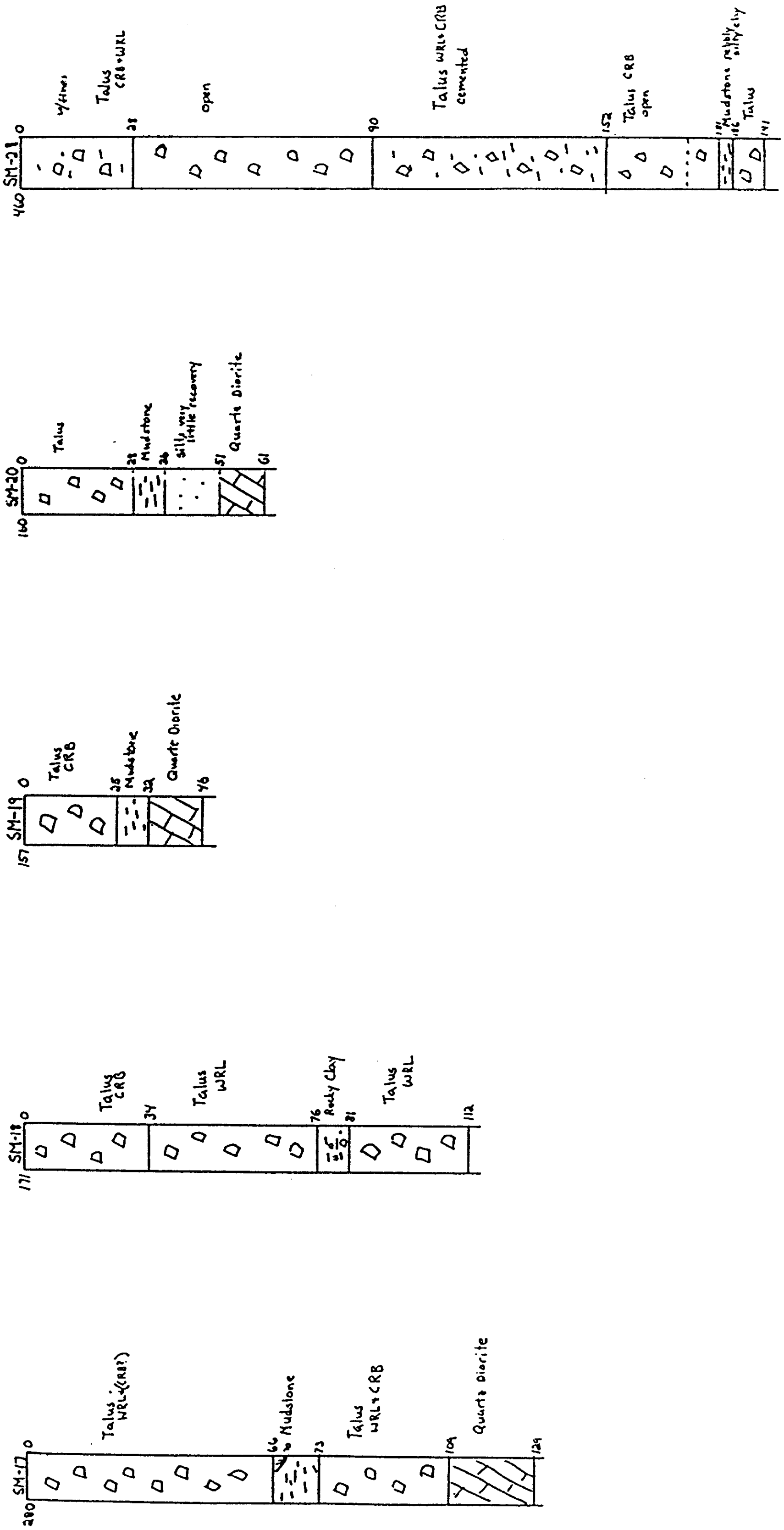
EXAMPLE:

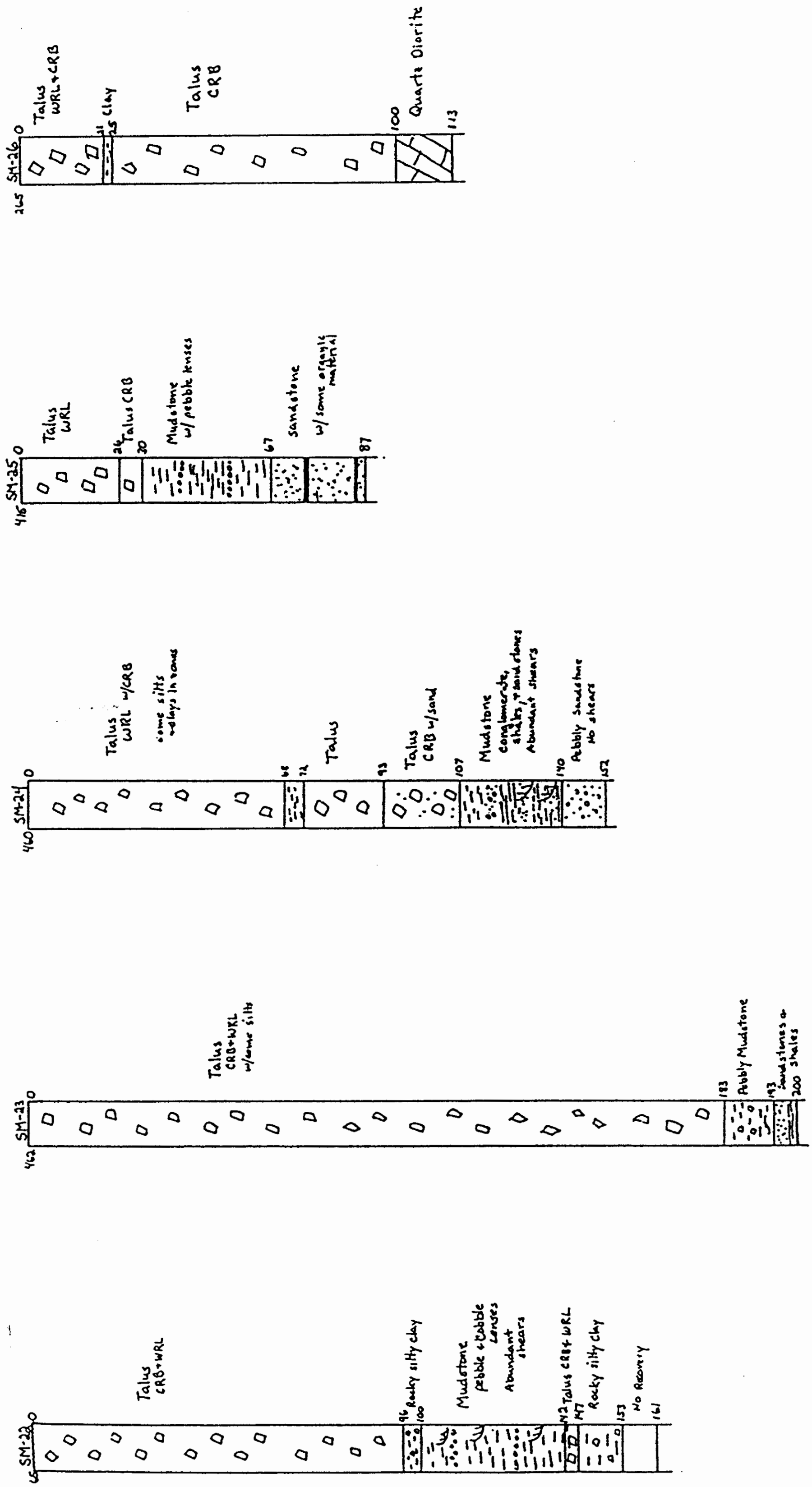


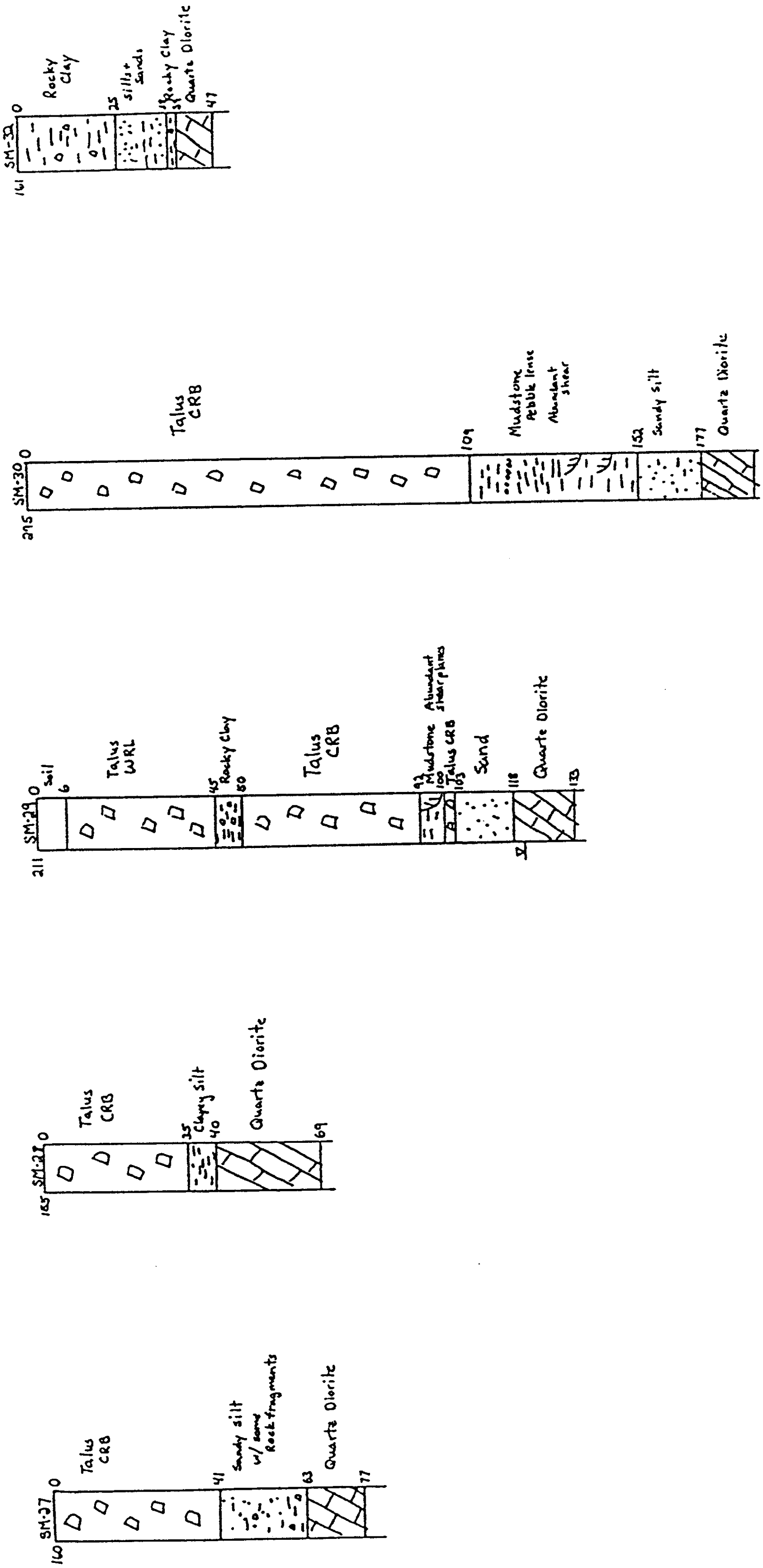

SM-5
338
0
4' Fill
Talus
sand, silt, clay matrix
94
Mudstone
abundant shear planes
100
Talus
(cemented)
124
136
139
159
SM-4
251
0
Rocky Soil
11
Talus
23
Mudstone
Rocky, silty
Abundant shears
29
Talus
Wind River Lava
114 clay seam
124
139 clay seam
200
Talus
CRB
ML-CL cement
228
WRL
Boulders
238
Ash
Quartz Diorite
251
SM-3
254
0
Rocky Soil
10
Talus
25
No Recovery
(sand)
41
Rock silty Mudstone
silt, clay - gravel lenses
abundant shears
100
Talus
166
185
No Recovery
(sand?)
209
Talus
silt + clay zones
some shears
248
SM-2
291
0
Rocky Soil
22
Talus
130
137
Mudstone
shear planes
Talus
ML-CL cement
185
191
Pebble Mudstone
Talus
250
SM-1
162
0
Talus
64
Mudstone
sand and gravel lenses
Abundant shears
80
Talus
clay cement
109
Sand and silt
w/ Rock fragments
119
Talus
139
(Sand)
Not recovered
169
Talus
ML-CL
cement
200
Talus
226
Agglomerate
229

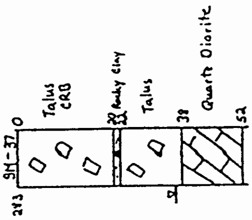

SM-10
Talus
CRB+WRL
w/fines
Agglomerate
volcanic
SM-9
Rocky soil
Talus - CRB+WRL
Talus
CRB+WRL
Talus
WRL
Talus
SM-8
Rocky soil
Talus
CRB+WRL
Mudstone
Silts, sands
+ pebble lenses
Abundant
shear planes
SM-7
Talus
WRL
No Recovery
of fines
Talus
CRB
Talus
1 block of
Broken CRB
Talus
Mudstone
Talus
SM-6
Talus
Mudstone
agglomerate
Talus
Diorite

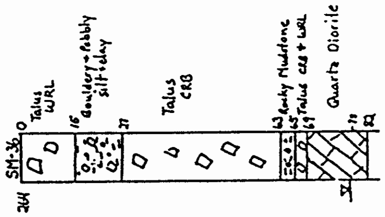

SM-16
Talus WRL
Talus CRB
Mudstone
Siltstones + Sandstones
Abundant
SM-15
Talus WRL
Talus CRB+WRL w/fines
Mudstone
Talus CRB+WRL w/fines
Talus WRL no fines
Talus CRB+WRL
Quartz Diorite
SM-14
Talus WRL
Cemented
Mudstone
Mod. cemented
poorly cemented
Mod. cemented
Mudstone
Soil + Talus
Talus No fines
Talus moderately cemented
SM-13
Talus No fines recovered
SM-12
Talus CRB
Soil
Quartz Diorite

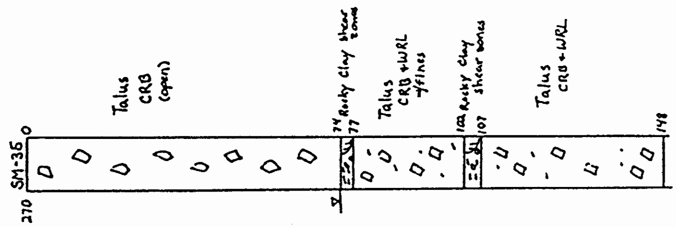

SM-21
460
0
w/fines
Talus CRB+WRL
28
open
90
Talus WRL+CRB cemented
152
Talus CRB open
Mudstone
pebbly silty clay
Talus
SM-20
160
0
Talus
28
Mudstone
36
silt, very little recovery
51
Quartz Diorite
61
SM-19
157
0
Talus CRB
25
Mudstone
32
Quartz Diorite
46
SM-18
171
0
Talus CRB
34
Talus WRL
76
Rocky Clay
81
Talus WRL
112
SM-17
280
0
Talus WRL+(CRB?)
66
Mudstone
73
Talus WRL+CRB
109
Quartz Diorite
129

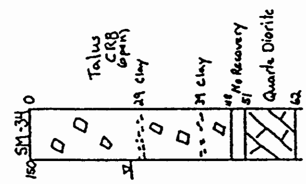

SM-26
245
0
Talus WRL+CRB
11
15
clay
Talus CRB
100
Quartz Diorite
113
SM-25
416
0
Talus WRL
26
Talus CRB
30
Mudstone w/ pebble lenses
67
Sandstone
w/ some organic material
87
SM-24
460
0
Talus WRL w/CRB
some silts + clays in zones
68
74
Talus
93
Talus CRB w/sand
107
Mudstone conglomerate, shales, + sandstones Abundant shears
140
Pebbly Sandstone No shears
152
SM-23
462
0
Talus CRB+WRL w/some silts
183
Pebbly Mudstone
193
Sandstones & shales
200
SM-22
0
Talus CRB+WRL
96
100
Rocky silty clay
Mudstone pebble + cobble Lenses Abundant shears
142
147
Talus CRB+WRL
Rocky silty clay
153
No Recovery
161

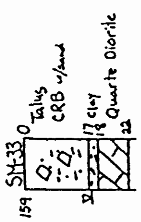

SM-32
161
0
Rocky Clay
25
silts + sands
Rocky Clay
Quartz Diorite
47
SM-30
275
0
Talus CRB
109
Mudstone
Pebble lense
Abundant shear
152
Sandy silt
177
Quartz Diorite
SM-29
211
0
Soil
6
Talus WRL
45
Rocky Clay
50
Talus CRB
92
Mudstone
Abundant shearplanes
100
Talus CRB
103
Sand
118
Quartz Diorite
133
SM-28
165
0
Talus CRB
35
Clayey silt
40
Quartz Diorite
69
SM-27
160
0
Talus CRB
41
Sandy silt w/ some Rock fragments
63
Quartz Diorite
77

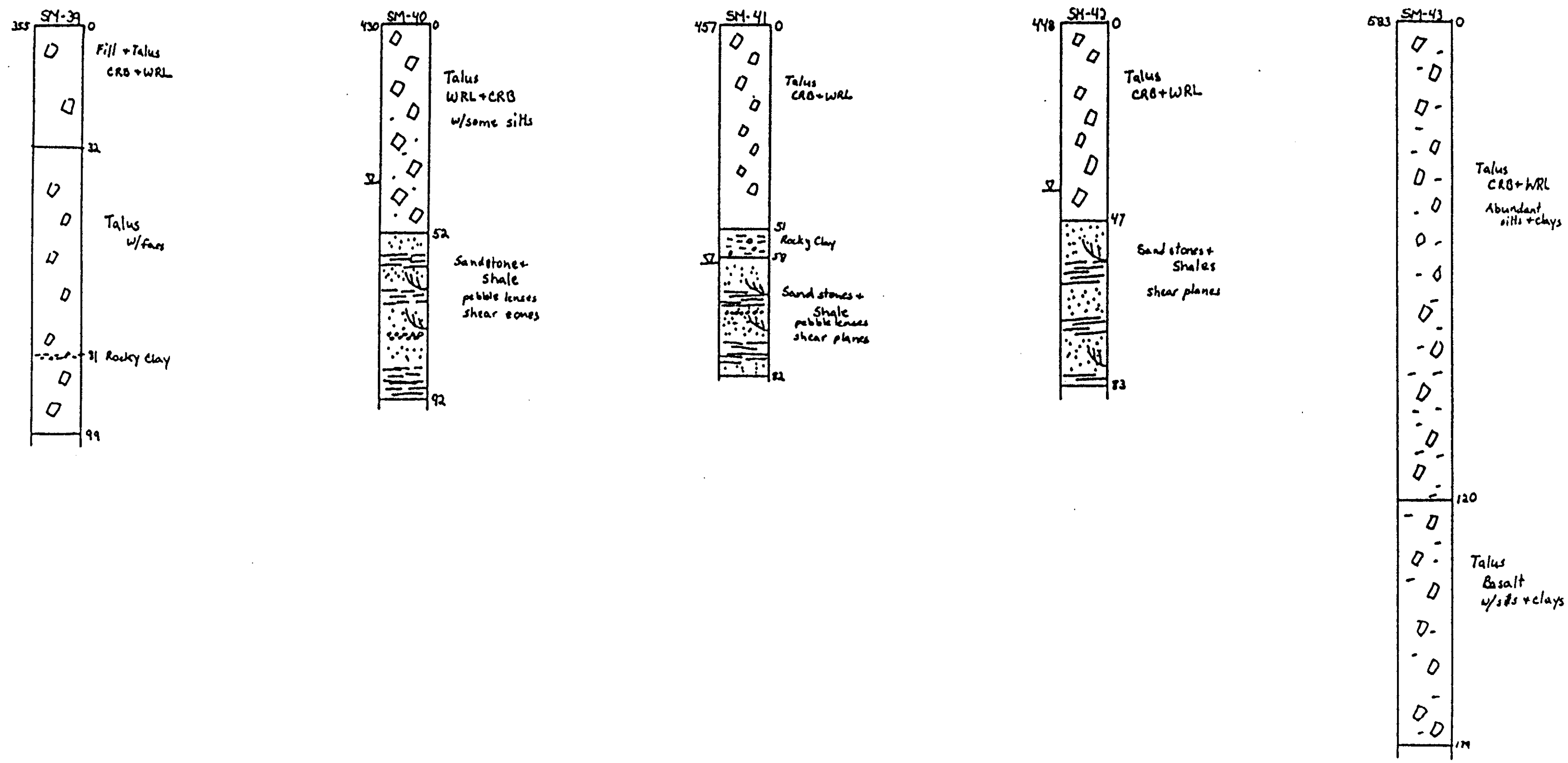

SM-37
243
0
Talus
CRB
Rocky clay
Talus
38
Quartz Diorite
52

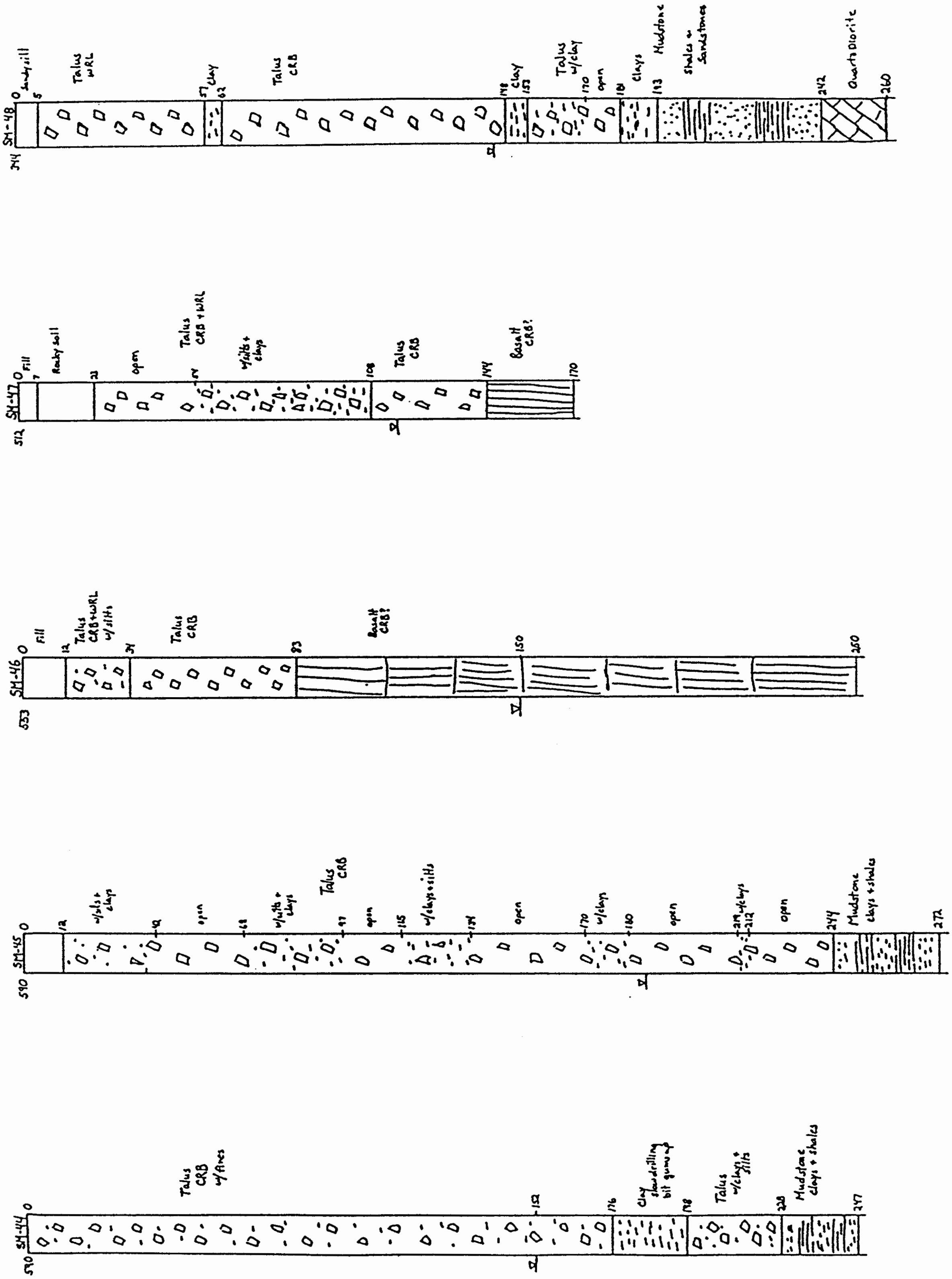

SM-36
244
0
Talus
WRL
16
Boulders + pebbly silt + clay
27
Talus
CRB
63 Rocky Mudstone
65 Talus CRB + WRL
69
Quartz Diorite
82

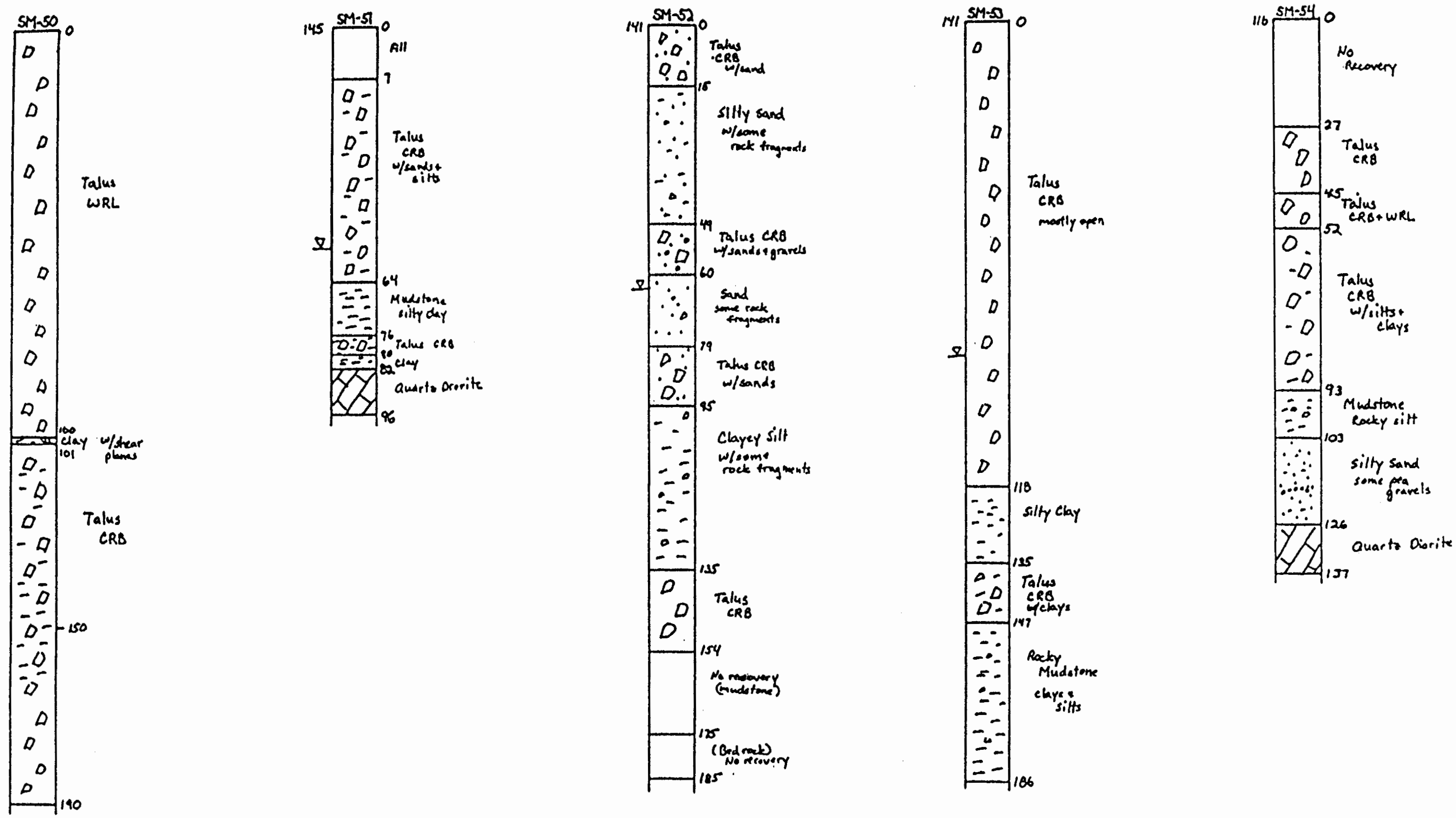

SM-35
270
0
Talus
CRB
(open)
74 Rocky clay shear zone
77
Talus
CRB + WRL
w/fines
103 Rocky clay
shear zones
107
Talus
CRB + WRL
148

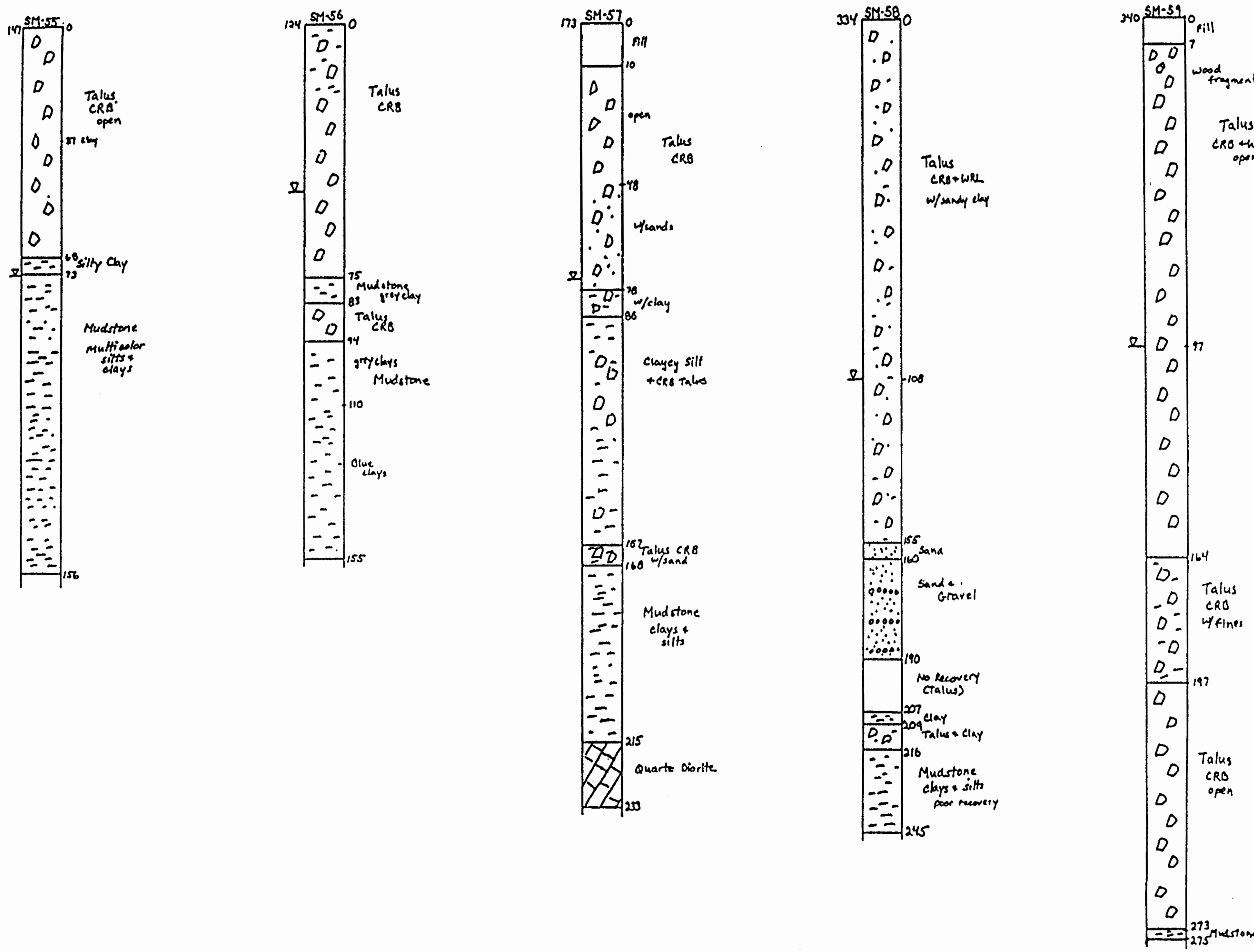

SM-34
150
0
Talus
CRB
(open)
29 clay
39 clay
48 No Recovery
51
Quartz Diorite
62


SM-33
159
0
Talus
CRB w/sand
17 clay
18
Quartz Diorite
22

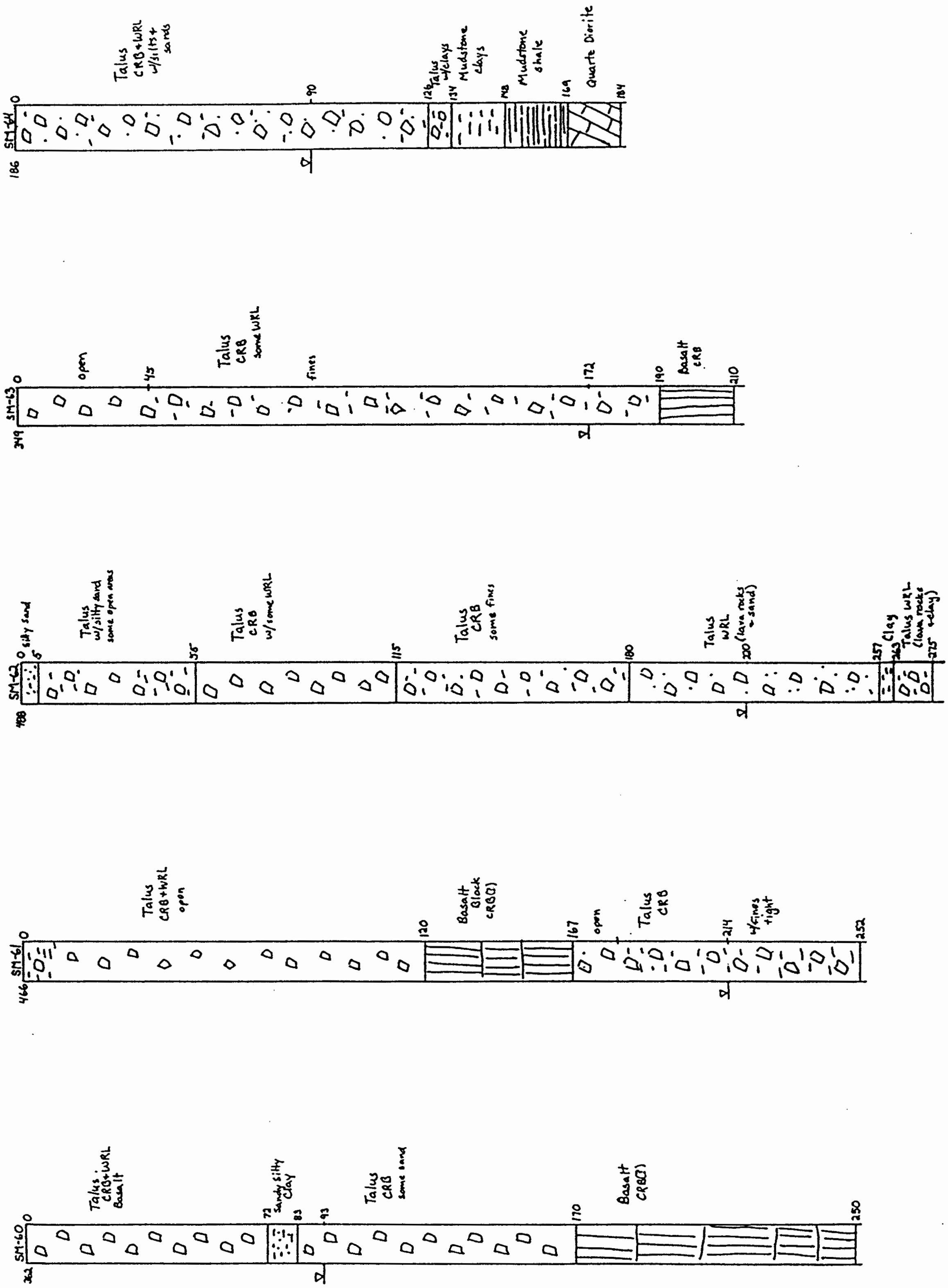

SM-39
355
0
Fill + Talus
CRB + WRL
32
Talus
w/fans
91 Rocky clay
99
SM-40
430
0
Talus
WRL + CRB
w/some silts
52
Sandstone +
Shale
pebble lenses
shear zones
92
SM-41
457
0
Talus
CRB + WRL
51
Rocky Clay
58
Sand stones +
Shale
pebble lenses
shear planes
82
SM-42
448
0
Talus
CRB + WRL
47
Sand stones +
Shales
shear planes
83
SM-43
583
0
Talus
CRB + WRL
Abundant
silts + clays
120
Talus
Basalt
w/silts + clays


SM-48
SM-47
SM-46
SM-45
SM-44
Talus CRB
Basalt CRB?
Mudstone
Quartz Diorite
Fill
Rocky soil
open


SM-50
0
Talus
WRL
100
clay w/shear planes
101
Talus
CRB
150
190
SM-51
145
0
Fill
7
Talus
CRB
w/sands+ silts
64
Mudstone
silty clay
76
Talus CRB
80
clay
82
Quartz Diorite
96
SM-52
141
0
Talus
CRB
w/sand
15
Silty sand
w/some
rock fragments
49
Talus CRB
w/sands+gravels
60
Sand
some rock fragments
79
Talus CRB
w/sands
95
Clayey Silt
w/some
rock fragments
135
Talus
CRB
154
No recovery
(mudstone)
175
(Bedrock)
No recovery
185
SM-53
141
0
Talus
CRB
mostly open
118
Silty Clay
135
Talus
CRB
w/clays
147
Rocky
Mudstone
clays+
silts
186
SM-54
116
0
No
Recovery
27
Talus
CRB
45
Talus
CRB+WRL
52
Talus
CRB
w/silts+
clays
93
Mudstone
Rocky silt
103
Silty Sand
some pea
gravels
126
Quartz Diorite
137



SM-55
147
0
Talus CRB open
87 clay
68 Silty Clay
73
Mudstone multicolor silts + clays
156
SM-56
124
0
Talus CRB
75 Mudstone greyclay
83 Talus CRB
94
greyclays Mudstone
110
Olive clays
155
SM-57
173
0
Fill
10
open
Talus CRB
48
w/sands
78 w/clay
86
Clayey silt + CRB Talus
152 Talus CRB w/sand
160
Mudstone clays + silts
215
Quartz Diorite
233
SM-58
334
0
Talus CRB + WRL w/sandy clay
108
155 Sand
160
Sand + Gravel
190
No Recovery (Talus)
207 clay
209 Talus + clay
216
Mudstone clays + silts poor recovery
245
SM-59
340
0 Fill
7
wood fragment
Talus CRB + WRL open
97
164
Talus CRB w/fines
197
Talus CRB open
273
275 Mudstone


SM-64
186
Talus CRB+WRL w/silts + sands
Talus w/clays
Mudstone clays
Mudstone shale
Quartz Diorite
0
90
126
134
148
169
184
SM-63
349
open
Talus CRB some WRL
fines
Basalt CRB
0
45
172
190
210
SM-62
488
silty sand
Talus w/silty sand some open areas
Talus CRB w/some WRL
Talus CRB some fines
Talus WRL (lava rocks + sand)
Clay
Talus WRL (lava rocks + clay)
0
5
56
115
180
220
257
263
275
SM-61
466
Talus CRB+WRL open
Basalt Block CRB(?)
open
Talus CRB
w/fines tight
0
120
167
214
252
SM-60
362
Talus: CRB+WRL Basalt
Sandy silty Clay
Talus CRB some sand
Basalt CRB(?)
0
72
83
93
170
250

APPENDIX B.

SURVEY

To determine the type of surface movement on the Fountain Landslide a survey grid was made. To measure the movement, monuments were set up on the landslide in a grid pattern and reoccupied at regular intervals.

The survey grid was designed to cover the actively moving western portion of the landslide. The survey was limited to this area based on Oregon State Highway Division data which indicated that it was the area on the landslide moving. Eleven stations were marked on the landslide using 60 to 120 centimeter lengths of one-half inch galvanized pipe. Figure 45 gives their locations. The elevations and distances to these stations were determined by surveying from two base stations also marked with pipes. It was assumed that base station 1 (BS1) and base station 2 (BS2) were stable and not involved with the slide. The actual elevations of the base stations were surveyed from a Bench Mark on Anderson Point (Figure 45) 906 meters (2973 feet) west of station 2ndL at the toe of the landslide.

Data files at the Oregon State Highway Division indicated that the amount of movement recorded by the survey over one winter could be expected to be less than

Figure 45. Location of survey stations.

N

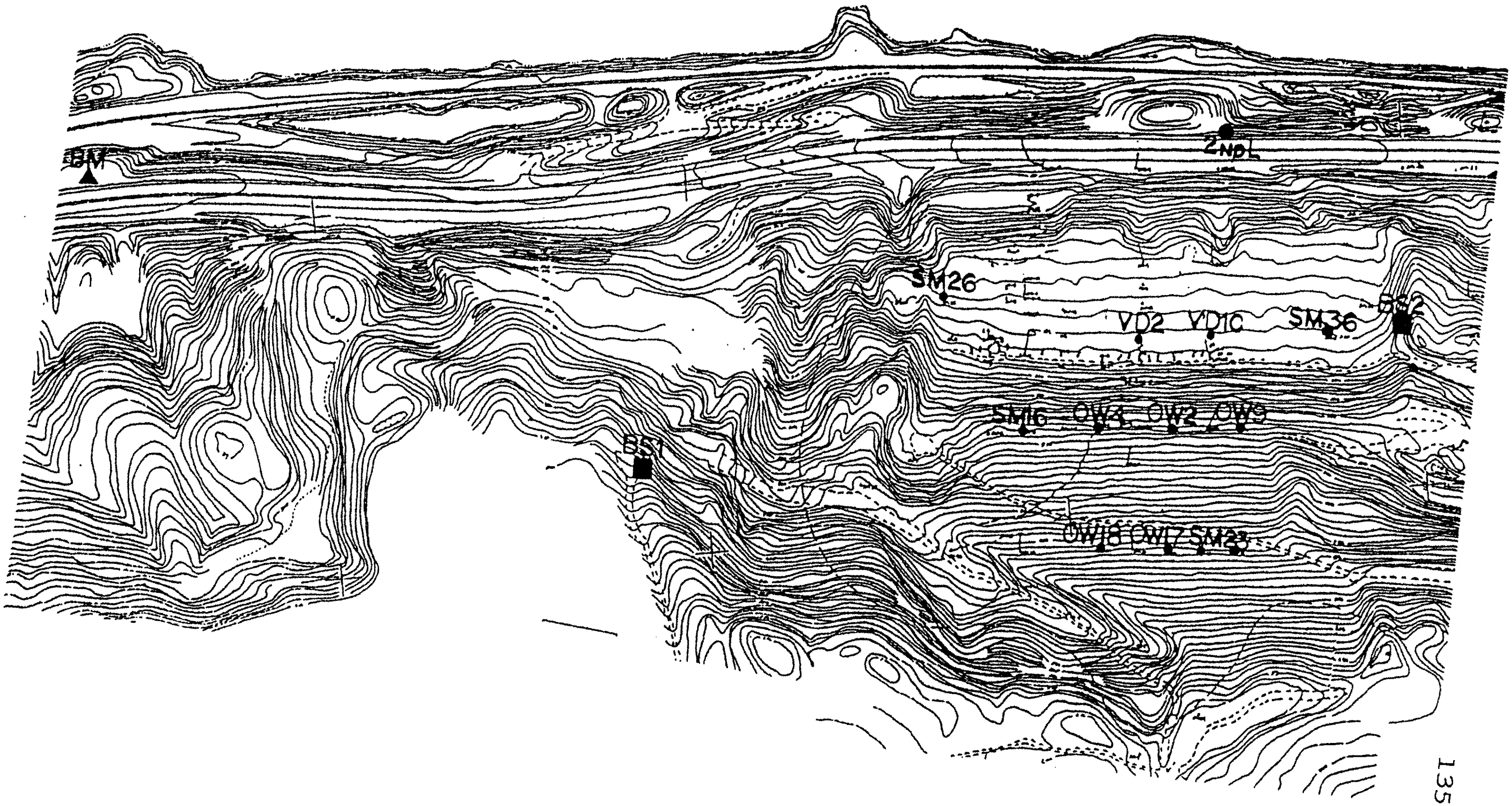
Bench Mark

Base Stations

Other stations

0 100'

30.5m



ten centimeters. The equipment needed (and available) to achieve measurements of that accuracy was a two second theodolite and an electronic distance meter (EDM). The survey method used to attain the desired degree of accuracy was the plunge and the reverse technique (Pugh, 1975).

Procedures

The plunge and reverse method requires that each angular measurement be taken with the vernier in both the face right and face left positions. This was done for all surveys except the initial survey in January 1983 where only the horizontal angle was recorded in both positions.

The field procedures in each of the surveys varied due to the use of different equipment and discoveries made while using the equipment. The initial survey was made January 16 and 22, 1983. The equipment used was a Wilde T-2 theodolite (PSU) and Philadelphia rod and a Topcon DMC-2 EDM (rented) with a single mount prism. The procedure used for this survey included:

1) take vertical and horizontal angle readings to each station with the theodolite and rod.

2) dismount the theodolite from the tripod and replace it with the EDM.

3) read the distance to each station using the EDM and the prism.

Dismounting the theodolite and the EDM from the tripod added an indeterminable and an unacceptable amount of error to the data. To remedy this situation when the survey was repeated on May 28 and 29, 1983 a five second Lietz SDM3E total station and a single prism (rented) was used. This equipment eliminated the need for switching the theodolite and the EDM. Since both the distance and the angles are shot to a single target only one set-up was required at each station. This greatly improved the accuracy and simplified the data reduction. The procedures using this equipment included:

1) take vertical and horizontal angle measurements and read the distance.

When the survey was repeated on May 26, 1984 the total station was not available. A two second Lietz TM85 theodolite mounted with a Lietz Red-2 EDM (rented) was used. Using this equipment required a single prism with an attached sighting target. The procedure using this equipment was different and included:

1) adjusting the distance between the target and the prism to equal the distance between the sighting lines of the theodolite and the EDM.

2) take horizontal and vertical angle measurements with the theodolite and read the distance with the EDM.

The survey was repeated on April 27, 1985 and a Lietz SDM3E total station was used again and the same procedures as in May 1983 were used.

Data Reduction

The data collected using the surveying equipment does not give the elevation difference or the horizontal distance to the station directly. To reduce the raw data the author wrote a short computer program (see Appendix D). For each station the program produced the difference in elevation, the horizontal distance and horizontal angle for both positions of the vernier using the plunge and reverse method. The two sets of numbers were then averaged. All averaged data from March 1983, March 1984, and April 1985 surveys is presented in Appendix C.

Error Determination

The error associated with each survey measurement was then calculated separately. To determine the amount of error associated with the survey data a total error was found for each reading. To determine the total error it is necessary to determine the amount of error associated with each piece of equipment used to attain the result. Table IV lists the equipment and the precision of each.

Table IV

THE SURVEY EQUIPMENT USED AND THEIR DEGREE OF ACCURACY

COMPONENT	ACCURACY
Theodolites	
Wilde T-2	2 seconds
Lietz SDM3E	2 seconds
Lietz TM85	2 seconds
EDM	
Topcon DMC1	1.0 mm
Lietz SDM3E	1.0 mm
Lietz Red2	1.0 mm

Height measurements
Philadelphia rod 3.048 mm
Tapes 1.588 mm

In order for each survey to have the same degree of accuracy and to avoid having to weigh the values when different equipment was used the accuracy of the least precise instrument was used for the error calculations. The error associated with the angle reading is proportional to and most heavily influenced by the horizontal distance to the target. From these numbers a total error was found by the square root of the sum of the squares (Kissam, 1971).

TOTAL ERROR ELEVATION =

$$\sqrt{EDM^2 + PRM\ HGT^2 + INST\ HGT^2 + (THEOD \times DIST)^2}$$

This type of calculation approximates the error when several measurements are used to produce one result. The error associated with each station reading can be found in Appendix C.

Using the Philadelphia rod greatly reduced the precision of the elevation determination of the first survey (Table IV). The random errors of the horizontal angle measurements were large in the first survey due to dismounting the theodolite. These two factors resulted in the decision to disregard its findings and only use the data from last three surveys for use in movement determinations.

Survey Data

From the reduced data listed in Appendix C, the actual amount and direction of movement of each station with respect to the instrument station was graphed and listed in Appendix E. The graphs in Appendix E were drawn assuming that the instrument stations were all stationary.

The movement graphed at some of the stations does not appear to be reasonable such as from BS2 to SM16 (see Figure 45 for their locations) which appears to move up hill (Appendix E). It was generally found that the movement graphed for one station such as OW4 did not have the same magnitude or direction from both BS1 and BS2. While some of the discrepancies could be attributed to operator errors and vandalism, it was found that trends emerged when the base stations were not assumed to be stationary. Therefore, the graphs found in Appendix E only represent the relative movements of each station with respect to the instrument station the reading was taken from.

When discrepancies in the data were attributable to operator errors or vandalism it was generally obvious due to the magnitude of the error. The suspected data is noted as such in Appendix C. When possible (e.g. transcribing errors) corrections to the data were made, but generally it was not possible and the data was disregarded. The graphs in Appendix E illustrate this final data. To determine the actual movement it is necessary to determine how the base stations have moved.

Discussion

An analysis of some of the possible movements of the base stations was made from the survey data. The assumptions for this were: 1) the bench mark at Anderson Point is stationary, 2) the stations may heave, 3) the stations can not move uphill, 4) the distance and elevation measurements may have a large error, and 5) the errors are small for the horizontal angle measurements. These assumptions are based on Oregon State Highway Division survey and slope inclinometer data along with the surface indications of sliding which indicate that the landslide moves northward and heaves at the toe.

To simplify interpretation, only four stations were used for the determination. The stations included the bench mark (BM), the second light post (2ndL), base station 2 (BS2), base station 1 (BS1) and SM36 (Figure 45). The relative locations of these stations were graphed (see Figure 46).

A summary of the known movements could be determined from the limited data and the assumptions, and is shown in Figure 46. They are: 1) station 2ndL (Figure 46) rose approximately 0.8 meters and moved approximately 0.014 meters closer to the BM, 2) BS2 has dropped approximately 0.3 meters and moved either 0.9 meters further from or 0.01

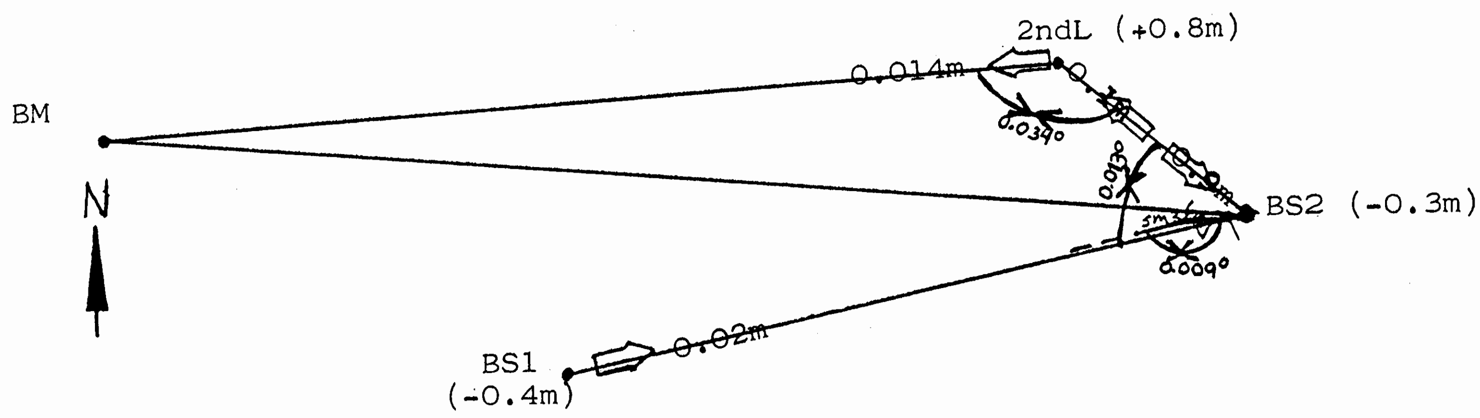


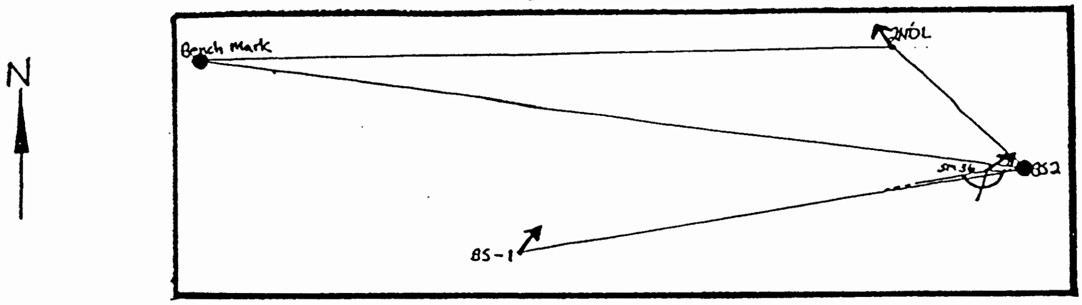
Figure 46. Movement direction of four survey stations. Arrows indicate direction of movement. (+) or (-) indicate relative change in elevations.

meters closer to the 2ndL, 3) BS1 has dropped approximately 0.4 meters and has apparently move closer to BS2 approximately 0.02 meters, and 4) SM36 has dropped approximately 0.4 meters. The differences in the horizontal angles measured are given in Figure 46.

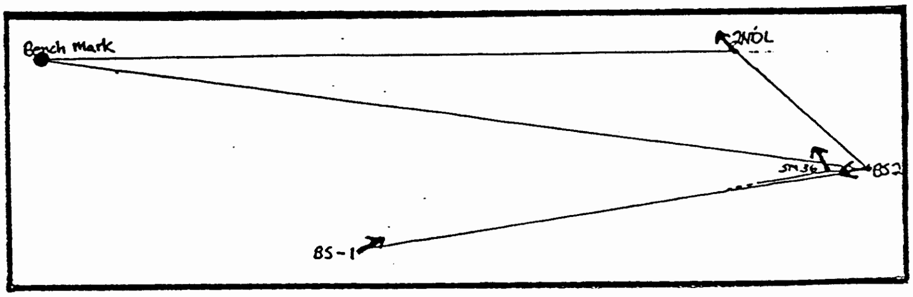
Since all recorded movements are relative the true direction and magnitude each station moved can not be determined directly from this graph. Of the stations surveyed the movement at 2ndL was described best since one leg had not moved, but the direction of movement was still not precisely defined. Therefore, it can only be generalized as moving some angle from west to northwest by an amount from 0.014 meters west to 0.107 meters northwest. The interpretation of the movement of the Fountain Landslide begins at station 2ndL by tracing the effect these two possible movement directions have on the other three stations. Beginning with the possibility of 2ndL moving to the northwest the possible movements of BS2 are shown in Figures 47A, B, C, D along with the effect on the other stations.

When 2ndL moves only westward there must be an error present in the data either in the horizontal angle measured between the Bench Mark and BS2 or the distance measured between the 2ndL and BS2 (Appendix C). Considering the possibility that an error does exist, the possible movements at BS2 are shown in Figures 48A, B, C, D along

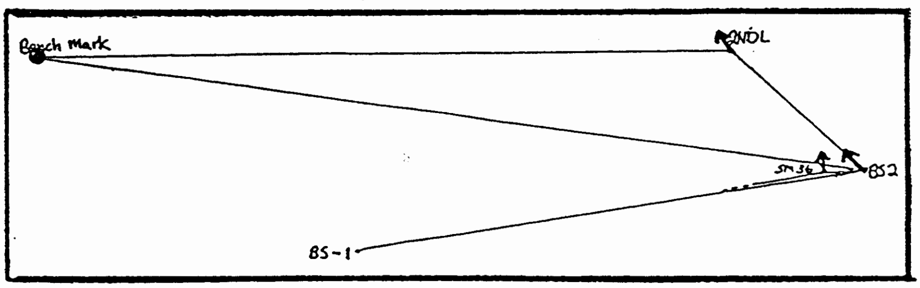
47A. BS2 stable (2ndL moves ~ 0.107m NW)



47B. BS2 moves west (2ndL moves more than 0.107m)



47C. BS2 moves northwest (2ndL moves ~ 0.107m more NW)



47D. BS2 moves north (2ndL moves much more than 0.107m)

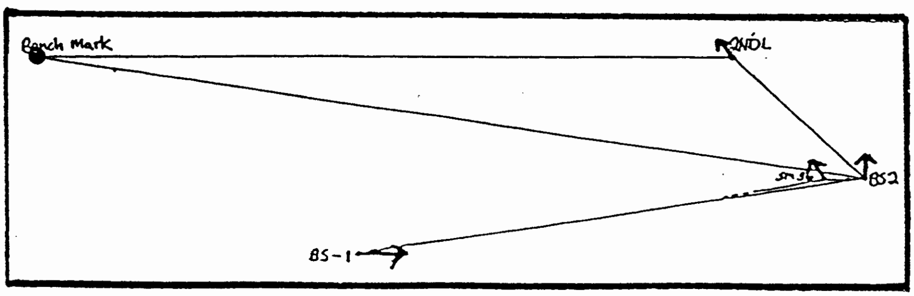


Figure 47. Possible interpretation of movement, condition 1. Arrows indicate directions of movement not magnitude. Slashes mark where errors must exist. Dots (●) indicate stable stations

with the effect on the other stations.

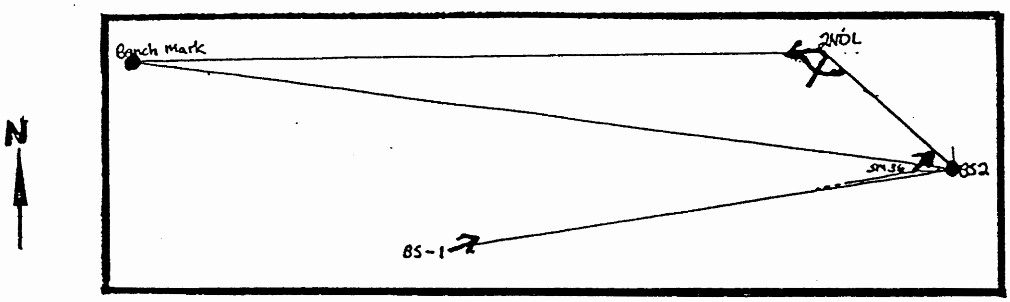
Which of these possible interpretation best describe the movement of the Fountain Landslide is found by first eliminating the possibilities which are not reasonable. For example, the possibility that BS2 remains stationary (Figures 47A and 48A) appears to be inconsistent with all measurements which indicate that BS2 has dropped in elevation and if it is stable there are considerable and systematic errors in several horizontal angle measurements. Therefore, 47A and 48A can be eliminated. Which direction BS2 has moved will be explored further, but first some other interpretations can be eliminated.

If BS2 is not stable, it is unlikely that the nearby SM36 has remained stable. This is supported by the lack of surface evidence of differential movement between the two stations such as new cracks or scarps. No cracks or scarps exist between the two (see Plate 1) therefore, differential movement has not occurred between BS2 and SM36 this eliminates the possibility 47D.

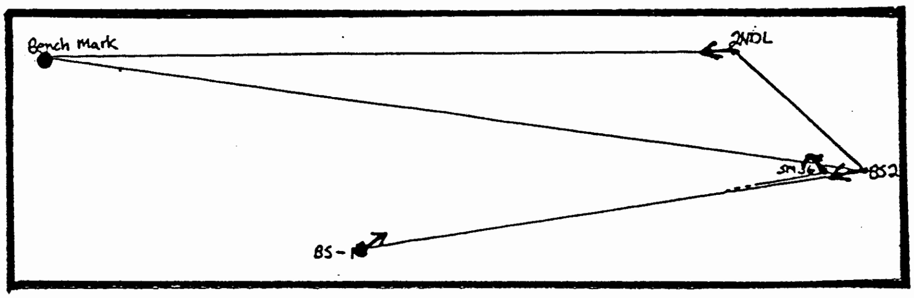
Of the remaining possible movement 47B, 47C, 48B and 48C, 48C could also be eliminated since they would require an unacceptable amount of error in several horizontal angle measurement.

To examine the possibility that errors in the data existed trigonometry was used to eliminate other possibilities. Suspected data was found in the distance

48A. BS2 stable (0.009° error in horz. angle)



48B. BS2 moves west (0.1m error in distance)



48C. BS2 moves northwest (error in both distance & angle)

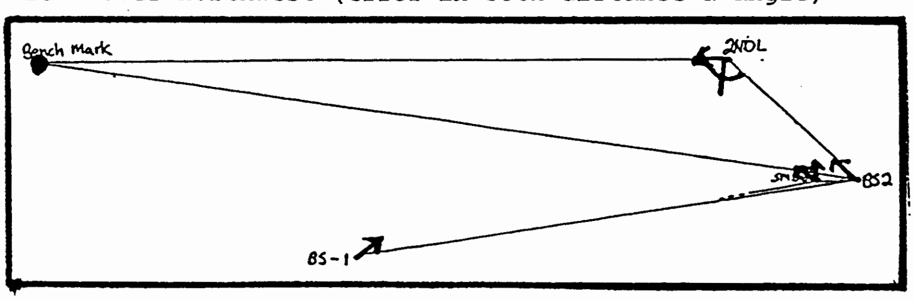


Figure 48. Possible interpretation of movement, condition 2. Arrows indicate directions of movement not magnitude. Slashes mark where errors must exist. Dots (●) indicate stable stations.

measured from BS2 to 2ndL in 1985 (Appendix C), where it is 0.10 meters shorter than when it was measured from 2ndL. Assuming the horizontal angle measurements are accurate, BS2 must have moved 0.10 meters closer to the Bench Mark. The effect of this on the distance between the 2ndL and BS2 was a decrease of 0.10 meters or the distance equalled 239.647 meters, the exact distance measured from BS2 to 2ndL. This indicates that alternative 48B can not be eliminated.

This then leaves alternatives 47B, 47C, along with 48B. Both 47B and 47C can be substantiated trigonomically so neither can be eliminated. Which one of these best describes the movement on the Fountain Landslide? Each show the expected northward movement. The cross sections and stress features would also support any and all of these alternatives. It was found that the precise determination of the movement of the Fountain Landslide using a survey grid is limited due to the instability of the base stations. The survey grid does shows that movement still occurs in the western portion of the Fountain Landslide and consists of heaving at the toe and a general northward movement of all the station survey.

Discussion of the Survey

The effectiveness of the survey grid was hindered because the base stations were not stable. It is very important to have the base stations stationary in a survey grid to determine accurately the surface movement. It was also found that it is difficult to find stable ground near a landslide without an initial detailed geological study. To find stable ground extensive field work would be necessary prior to beginning the survey. This was not done for this survey. Because of the vegetation and topography in the study area finding suitable stable ground proved to be very difficult. The rapid vegetation growth in the area impaired readings. In two years some stations visible during the first survey were obscured, even when an eight foot (2.44 meter) extension was added to the target. If there is not sufficient time or data to determine the stability of the base stations it would be necessary to determine accurately their movement for the duration of the survey.

One method of determining the movement at the base stations for this survey would have been to establish two or possibly three easily identifiable stationary triangulation points across the Columbia River. Other improvements to this survey would have been to extend the survey to the east and to the north in order to cover the entire landslide. This would have provided information as to the location of the slide boundary and whether the eastern portion of the landslide is presently moving.

APPENDIX C.

Reduced Survey Data with range of error. The horizontal angle measurements for each survey had different initial angle position. Data printed in Boldface represents vandilized stations, <u>Underlined</u> data are identifiable and corrected errors and data in "Quotation Marks" are errors in which the data has been disregarded.

REDUCED SURVEY DATA FROM 2ND LIGHT POST

STATION		DISTANCE	ELEVATION	HORIZONTAL ANGLE
BS2	'83	239.655m	41.536m	0
	'84	"239.156"	"41.349"	126.658
	'85	239.747	**40.973**	217.469
	ERROR	±0.003	±0.004	±0.002m
BM	'83	906.481	-8.705	0
	'84	"906.551"	"-9.290"	269.075
	'85	906.467	**-9.522**	360.082
	ERROR	±0.009	±0.009	±0.009m

REDUCED SURVEY DATA FROM BS2

STATION		DISTANCE	ELEVATION	HORIZONTAL ANGLE
BS1	'83	664.264m	**50.824**m	24.379
	'84	0	0	0
	'85	664.239	51.173	1.385
	ERROR	±0.006	±0.007	±0.006m
2NDL	'83	239.659	-41.389	70.758
	'84	239.622	-41.431	205.019
	'85	239.647	-40.994	47.749
	ERROR	±0.003	±0.004	±0.002m
SM36	'83	42.766	-11.830	27.195
	'84	42.758	-11.828	161.476
	'85	42.753	-11.822	4.200
	ERROR	±0.002	±0.003	±0.0004m

VD1C	'83	183.131	-10.415	29.205
	'84	183.135	-10.410	163.487
	'85	183.132	-10.401	6.212
	ERROR	±0.002	±0.003	±0.002m
VD2	'83	245.196	-9.168	27.785
	'84	**242.462**	**-10.321**	**165.662**
	'85	242.458	-10.315	8.392
	ERROR	±0.003	±0.004	±0.002m
SM26	'83	336.319	-9.975	31.569
	'84	336.309	-10.025	165.858
	'85	336.301	-10.063	8.585
	ERROR	±0.004	±0.004	±0.003m
OW9	'83	"184.992"	"18.197"	0.969
	'84	183.756	18.049	135.253
	'85	183.766	18.055	337.971
	ERROR	±0.002	±0.003	±0.002m
OW2	'83	214.574	16.592	7.964
	'84	214.543	16.567	142.246
	'85	214.550	16.623	344.972
	ERROR	±0.003	±0.003	±0.002m
OW4	'83	260.267	15.652	14.116
	'84	260.232	15.626	148.400
	'85	"260.240"	"16.214"	351.125
	ERROR	±0.003	±0.004	±0.002m
SM16	'83	347.226	15.516	19.572
	'84	347.216	15.571	153.845
	'85	347.206	15.494	356.578
	ERROR	±0.004	±0.004	±0.003m
SM23	'83	272.274	45.277	347.731
	'84	272.238	45.017	122.008
	'85	OBSTRUCTED		
	ERROR	±0.003	±0.004	±0.004m
OW17	'83	293.995	45.833	352.285
	'84	294.015	45.618	126.565
	'85	293.998	45.795	329.295
	ERROR	±0.003	±0.004	±0.003m
OW18	'83	339.749	44.842	359.517
	'84	339.745	44.609	133.800
	'85	339.754	44.800	336.529
	ERROR	±0.004	±0.004	±0.003m

REDUCED SURVEY DATA FROM BS1

STATION		DISTANCE	ELEVATION	HORIZONTAL ANGLE
BS2	'83	664.262m	-51.194m	153.153
	'85	664.239	-51.186	5.251
	ERROR	±0.006	±0.007	±0.006m
SM36	'83	621.569	-62.963	152.969
	'85	621.531	-63.034	4.555
	ERROR	±0.006	±0.006	±0.006m
OW9	'83	500.954	-33.144	161.546
	'85	500.930	-33.154	13.128
	ERROR	±0.005	±0.006	±0.005m
OW2	'83	462.424	-34.647	160.698
	'85	462.425	-34.611	12.280
	ERROR	±0.005	±0.005	±0.004m
OW4	'83	410.800	-35.570	159.643
	'85	410.715	-35.577	11.227
	ERROR	±0.004	±0.005	±0.004m
SM16	'83	319.589	-35.704	158.386
	'85	319.568	-35.691	9.968
	ERROR	±0.003	±0.004	±0.003m

APPENDIX D.

Computer program to reduce raw survey data

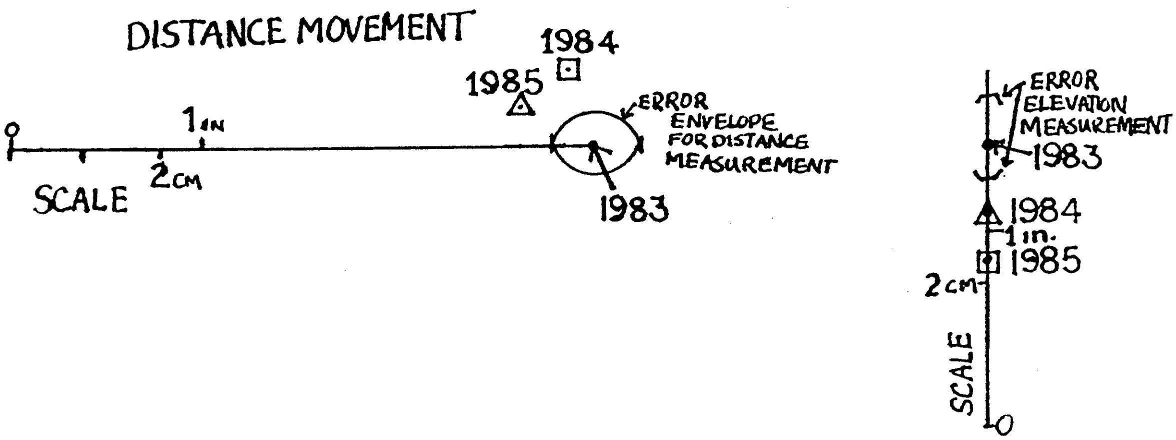
```
100 'SURVEY DATA REDUCTION
110  INPUT "HOW MANY STATIONS:"; N
120  DIM STAT$(N), HORZ#(N,3), HORZ2#(N,3), VERT2#(N,3), VERT#(N,3), SDIST#(N),
INSHT#(N), PRMHT#(N), PRMHT2#(N), HANG#(N), HANG2#(N), VANG2#(N), VANG#(N), SDIS
T2#(N), PVANG2#(N), ALPHA#(N), ALPHA2#(N), DIFANG#(N), ANG#(N), DIST#(N), DIST2#
(N), ELEV#(N)
125  DIM ELEV2#(N)
130  GOSUB 1000  'ENTER DATA
140  GOSUB 2000  'PRINT DATA
150  GOSUB 3000  ' DATA REDUCTION
160  END
170 '
1000 'SUBROUTINE TO ENTER DATA
1010 'ENTERING DATA
1020   OPEN "DATA2" FOR OUTPUT AS #1
1030    FOR I = 1 TO N
1040        INPUT "STATION"; STAT$(I)
1050        INPUT "1ST H: DEG, MIN, SEC "; HORZ#(I,1), HORZ#(I,2),HORZ#(I,3)
1060         LET HANG#(I)=(((HORZ#(I,3)/60) + HORZ#(I,2))/60) + HORZ#(I,1)
1070        INPUT "2ND H: DEG, MIN, SEC ";HORZ2#(I,1), HORZ2#(I,2), HORZ2#(I,3)
1080         LET HANG2#(I)=(((HORZ2#(I,3)/60) + HORZ2#(I,2))/60) + HORZ2#(I,1)
1090        INPUT "1ST V: DEG, MIN, SEC "; VERT#(I,1),VERT#(I,2),VERT#(I,3)
1100         LET VANG#(I)=(((VERT#(I,3)/60) + VERT#(I,2))/60) + VERT#(I,1)
1110        INPUT "2ND V: DEG, MIN, SEC "; VERT2#(I,1), VERT2#(I,2),VERT2#(I,3)
1120         LET PVANG2#(I)= (((VERT2#(I,3)/60) + VERT2#(I,2))/60) + VERT2#(I,1)

1130         LET VANG2#(I) = 360 - PVANG2#(I)
1140        INPUT "SLOPE DISTANCE 1 & 2"; SDIST#(I), SDIST2#(I)
1150        INPUT "INSTRUMENT HEIGHT AND PRISM HEIGHT 1 & 2"; INSHT#(I), PRMHT#(
I), PRMHT2#(I)
1160 '
1170       WRITE #1, STAT$(I), HANG#(I), HANG2#(I), VANG#(I), VANG2#(I), SDIST#(
I), SDIST2#(I), INSHT#(I), PRMHT#(I), PRMHT2#(I)
1180 '
1190    NEXT I
1200  RETURN
1210 '
1220 '
2000 'PRINTING SUBROUTINE
2020   LET HEAD$=                     "STAT  HORIZ1  HORIZ2   VERT1   VERT2
SDIST   SDIST    INSTHGT PRMHGT PRMHT2"
2030   LET ROW$=                      "\    \ ###.### ###.### ###.### ###.###
####.### ####.###  ##.###  ##.### ##.###"
2040    LPRINT HEAD$
2050     FOR K = 1 TO N
2060      LPRINT USING ROW$ ; STAT$(K); HANG#(K); HANG2#(K); VANG#(K);VANG2#(K),
 SDIST#(K); SDIST2#(K); INSHT#(K); PRMHT#(K); PRMHT2#(K)
2070     NEXT K
2080   RETURN
2090 '
2100 '
2500 ' INPUTING AND PRINTING DATA FROM AND EXISTING FILE
2510     OPEN "DATA1" FOR INPUT AS #2
2520     LET HEAD$= "STAT HORZ1   HORZ2    VERT1  VERT2    SDIST    SDIST     IN
STHT  PRMHT   PRMHT2"
2530     LET ROW$=  "\   \ ###.### ###.###  ###.## ###.###  ####.### ####.###  ##
.###  ##.###  ##.###"
```

```
2540        LPRINT HEAD$
2550         FOR K = 1 TO N
2560         INPUT #2, STAT$(K), HANG#(K), HANG2#(K), VANG#(K), VANG2#(K), SDIST#
(K), SDIST2#(K), INSHT#(K), PRMHT#(K), PRMHT2#(K)
2570          LPRINT USING ROW$ ; STAT$(K);HANG#(K); HANG2#(K); VANG#(K); VANG2#(
K); SDIST#(K); SDIST2#(K); INSHT#(K); PRMHT#(K); PRMHT2#(K)
2580        NEXT K
2590   RETURN
2600 '
2610 '
3000 'DATA REDUCTION TO ELEVATION AND DISTANCE IN FEET
3010   OPEN "RES2" FOR OUTPUT AS #3
3020    LET RES$ = " STAT  DISTANCE  DISTANCE2  ELEVATION  ELEVATION2  HORIZ "
3030    LPRINT RES$
3040    LET RESR$ = "\     \ ####.###   ####.###   ####.###   ####.###   ###.###"

3050    FOR L = 1 TO N
3055     IF SDIST2#(L) = O# THEN 3080
3060     LET DIFANG#(L) = (HANG#(L) + 180#)  - HANG2#(L)
3070     IF DIFANG#(L) >= O# THEN 3080 ELSE 3100
3080     LET ANG#(L) = HANG#(L) - (DIFANG#(L) / 2#)
3090     GOTO 3110
3100     LET ANG#(L) = HANG#(L) + (ABS(DIFANG#(L)) / 2#)
3110     LET ALPHA#(L) = (90# - VANG#(L)) * .017453
3120     LET DIST#(L) = SDIST#(L) * COS(ALPHA#(L))
3130     LET ELEV#(L) = (SDIST#(L) * SIN(ALPHA#(L))) + INSHT#(L) - PRMHT#(L)
3140     IF SDIST2#(L) = O# THEN 3180
3150     LET ALPHA2#(L) = (90# - VANG2#(L)) * .017453
3160     LET DIST2#(L) = SDIST2#(L) * COS(ALPHA2#(L))
3170     LET ELEV2#(L) = SDIST2#(L) * (SIN(ALPHA2#(L))) + INSHT#(L) - PRMHT2#(L)

3180     WRITE #3, STAT$(L), DIST#(L), DIST2#(L), ELEV#(L), ELEV2#(L), ANG#(L)
3190     LPRINT USING RESR$; STAT$(L); DIST#(L); DIST2#(L); ELEV#(L); ELEV2#(L);
ANG#(L)
3200   NEXT L
3210  RETURN
3220 '
```

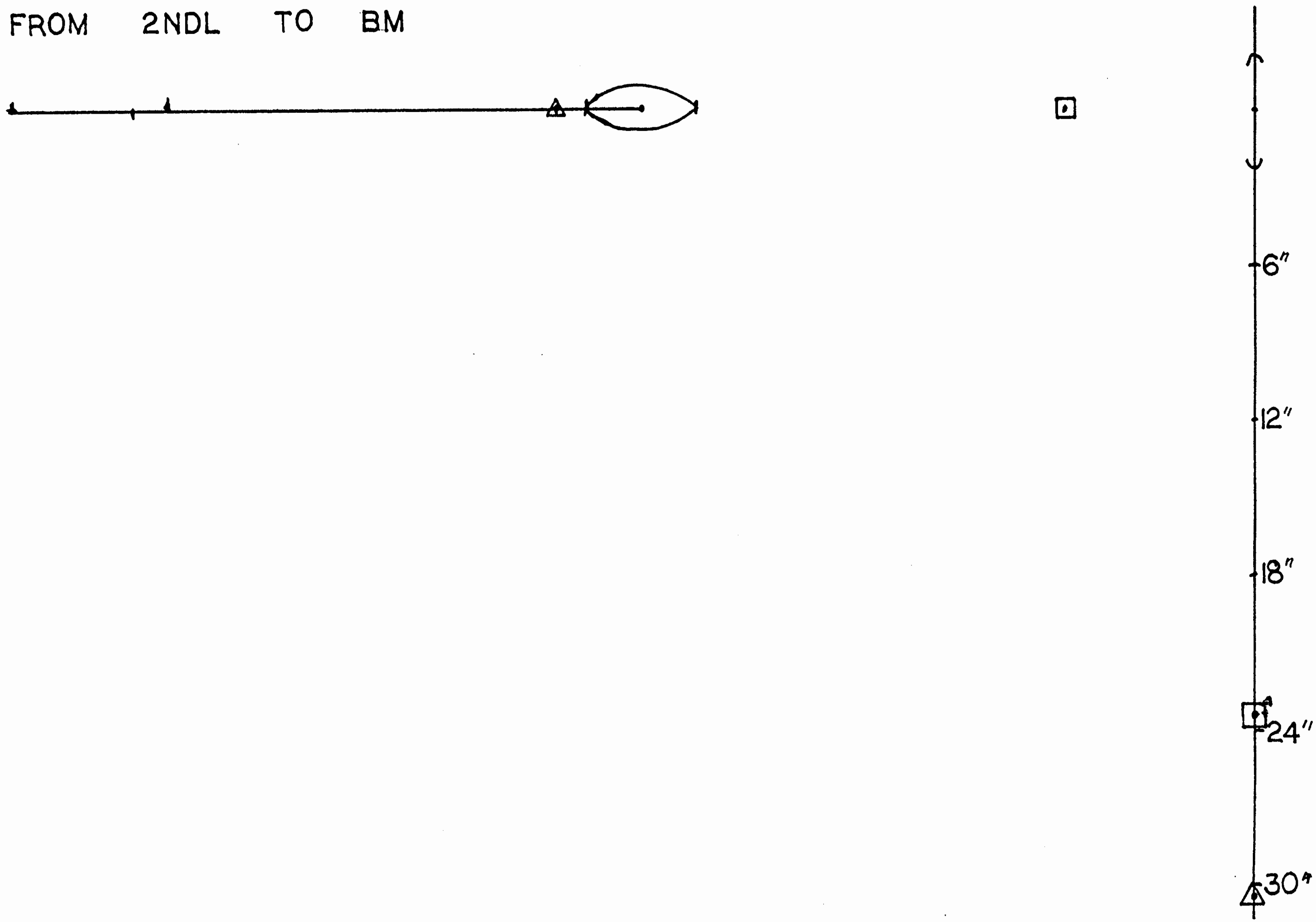
APPENDIX E.

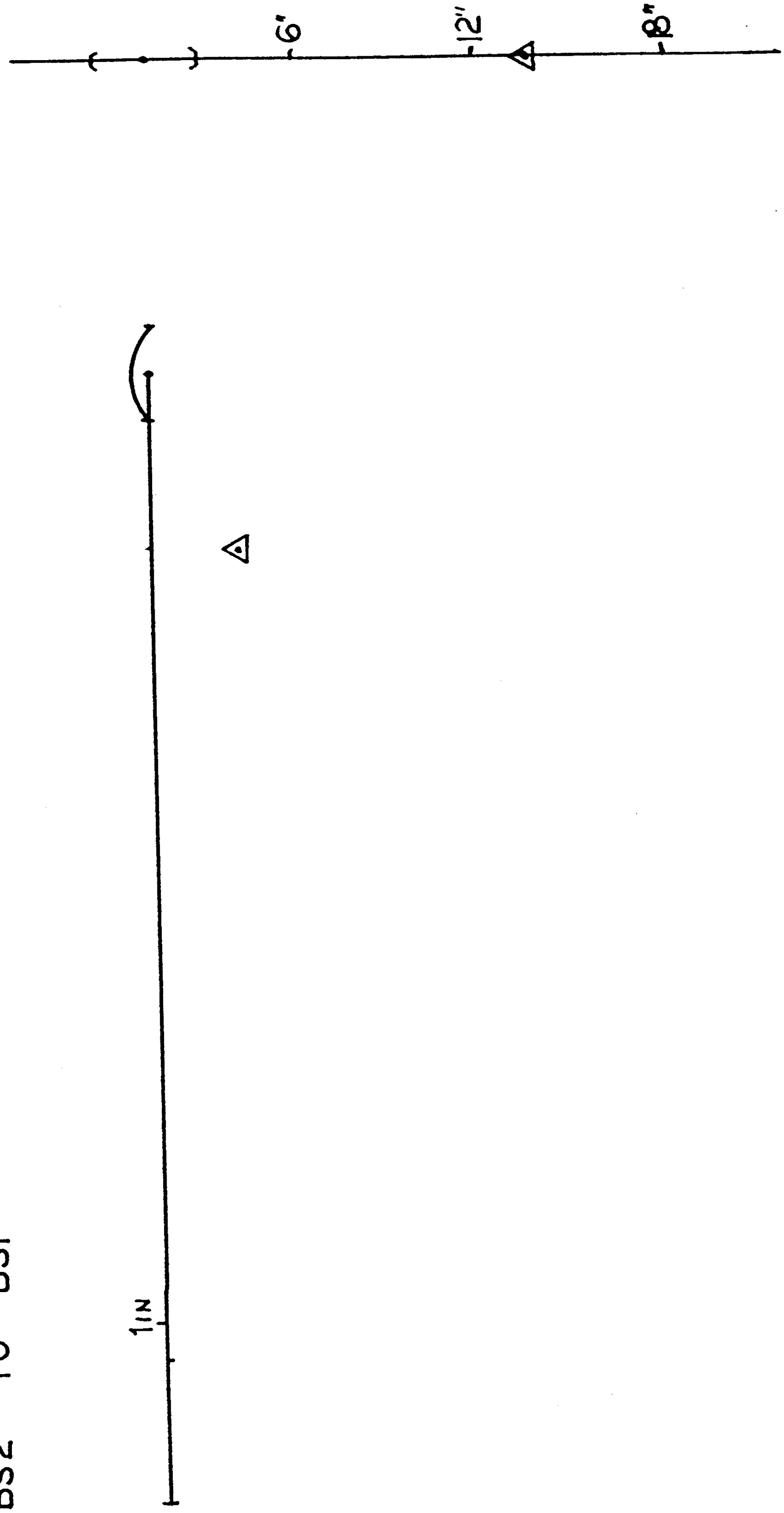
Surveyed movement and elevation changes for each station. Initial reading March 26, 1983.

EXAMPLE:

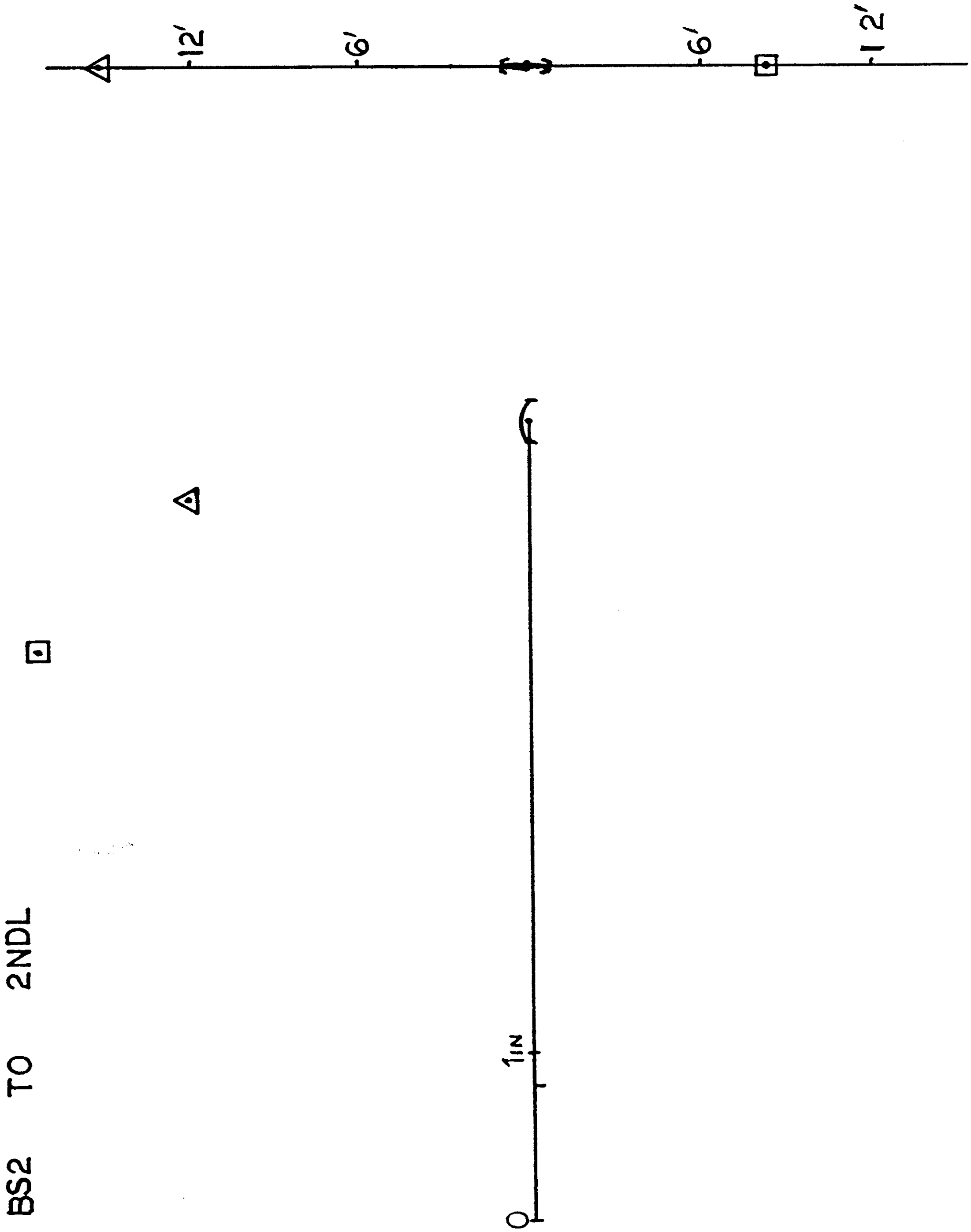


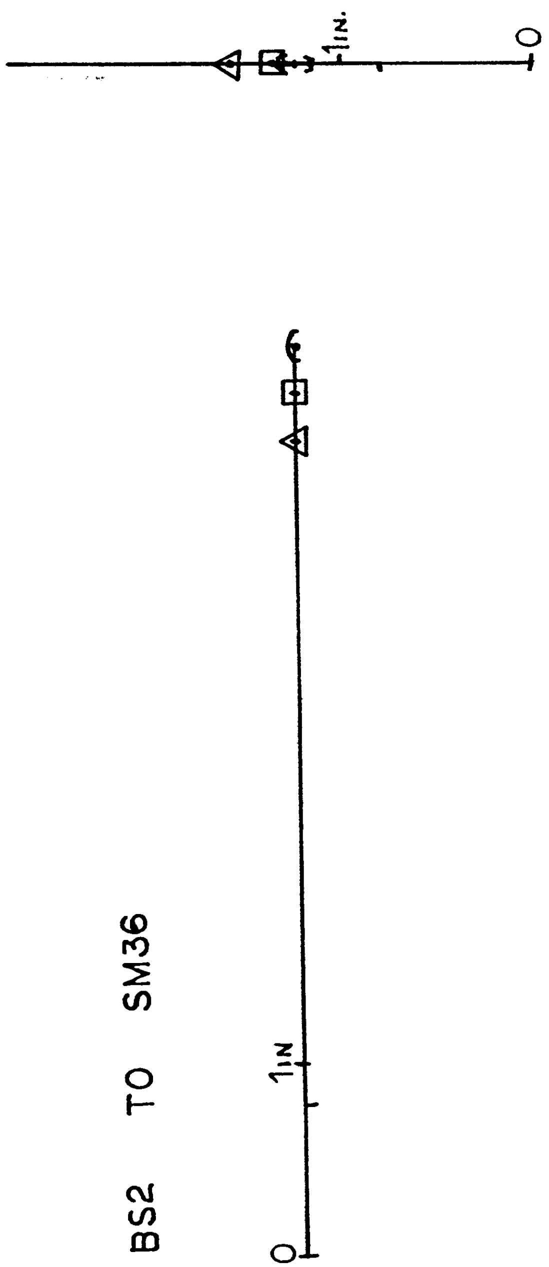
FROM 2NDL TO BM

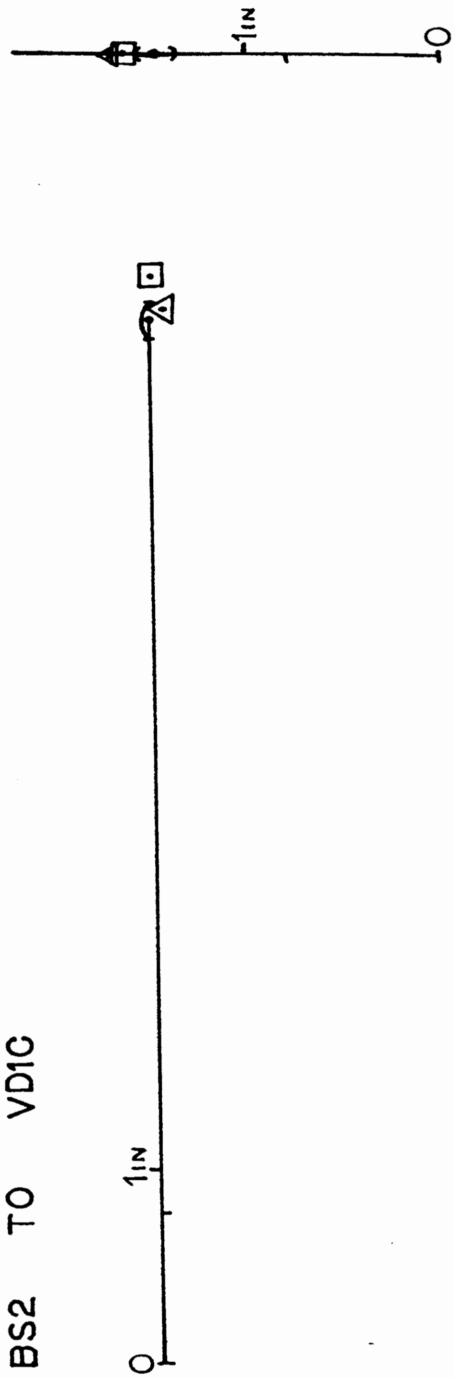




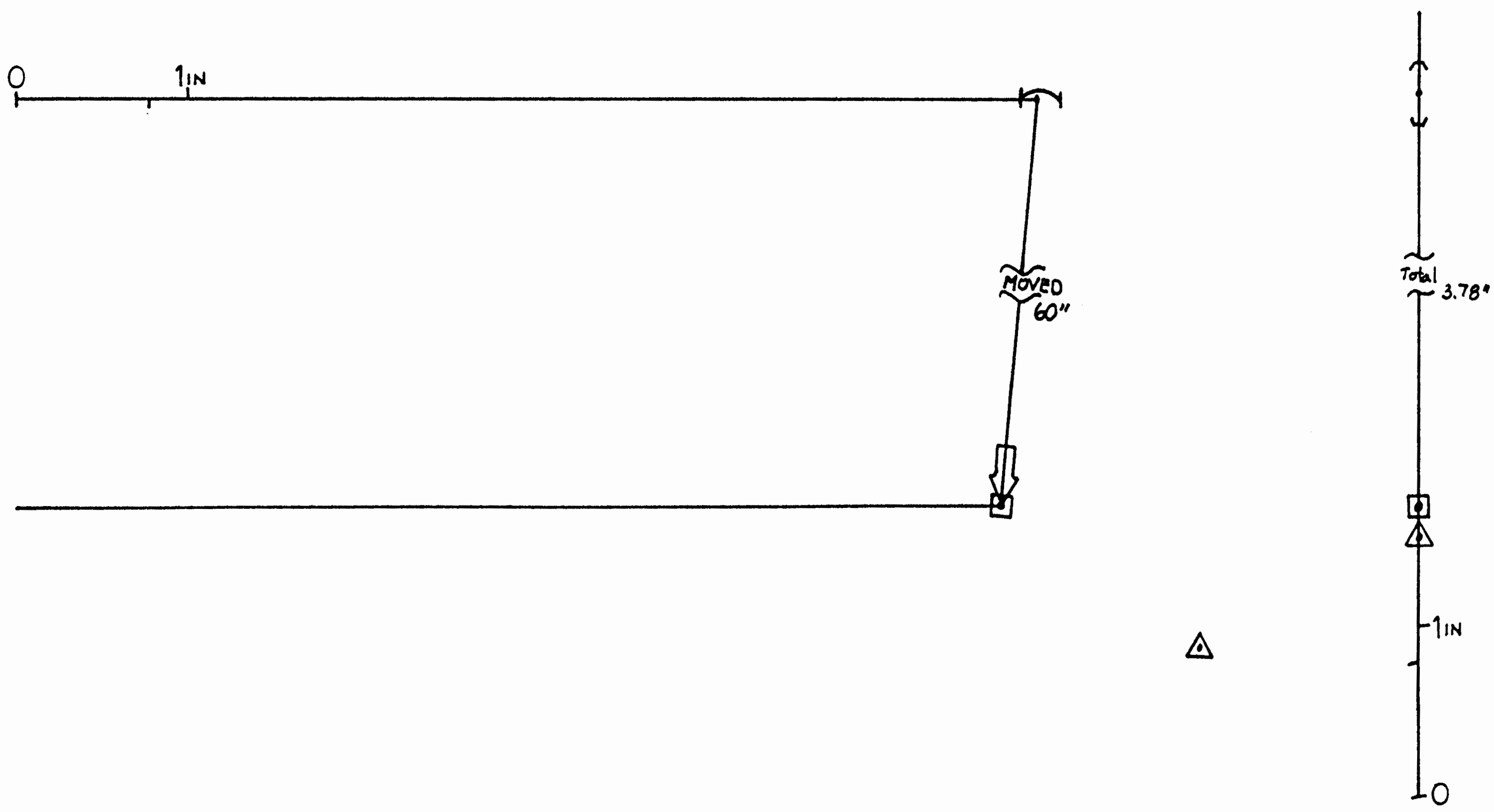
BS2 TO BS1


BS2 TO 2NDL
12'
6'
6'
1 2'
0
1IN

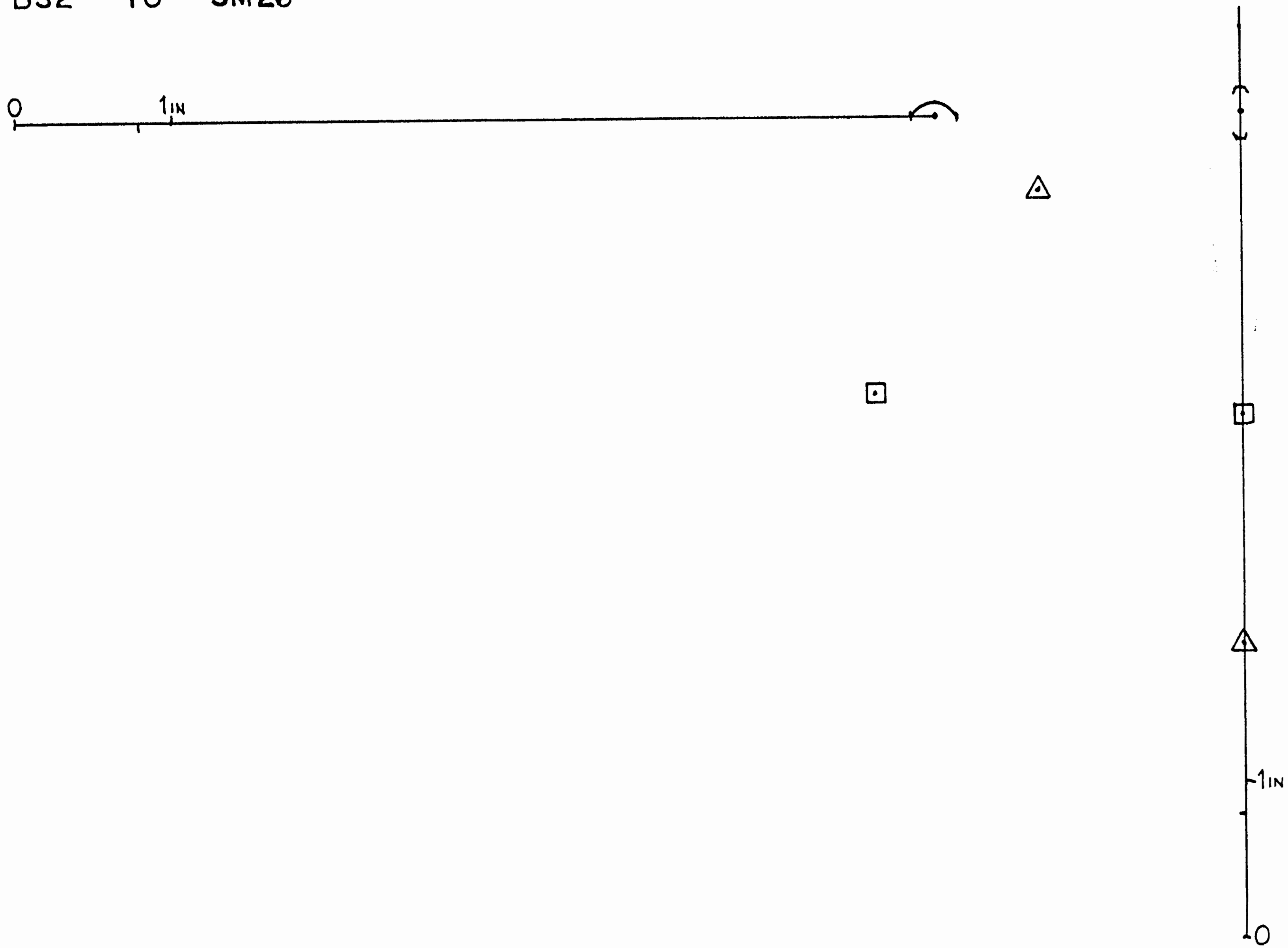

1 IN.
0
BS2 TO SM36
1 IN
0

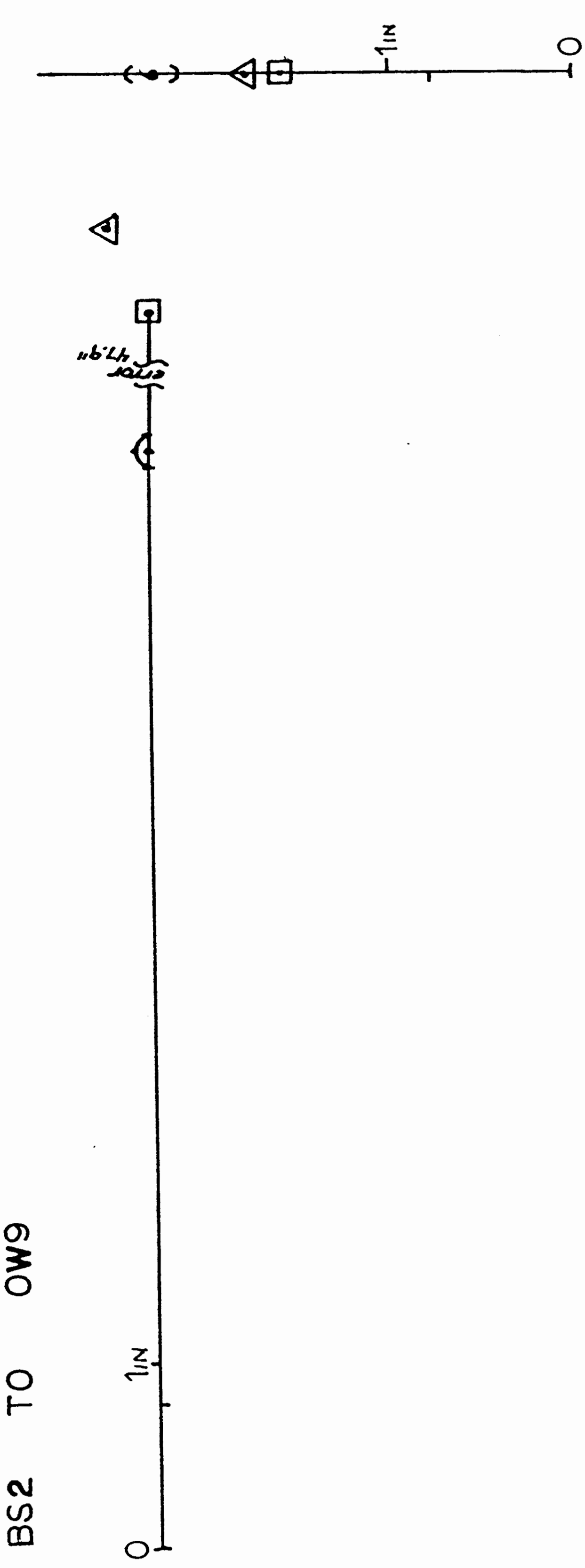

BS2 TO VD1C
0
1IN
1IN
0

BS2 TO VD2

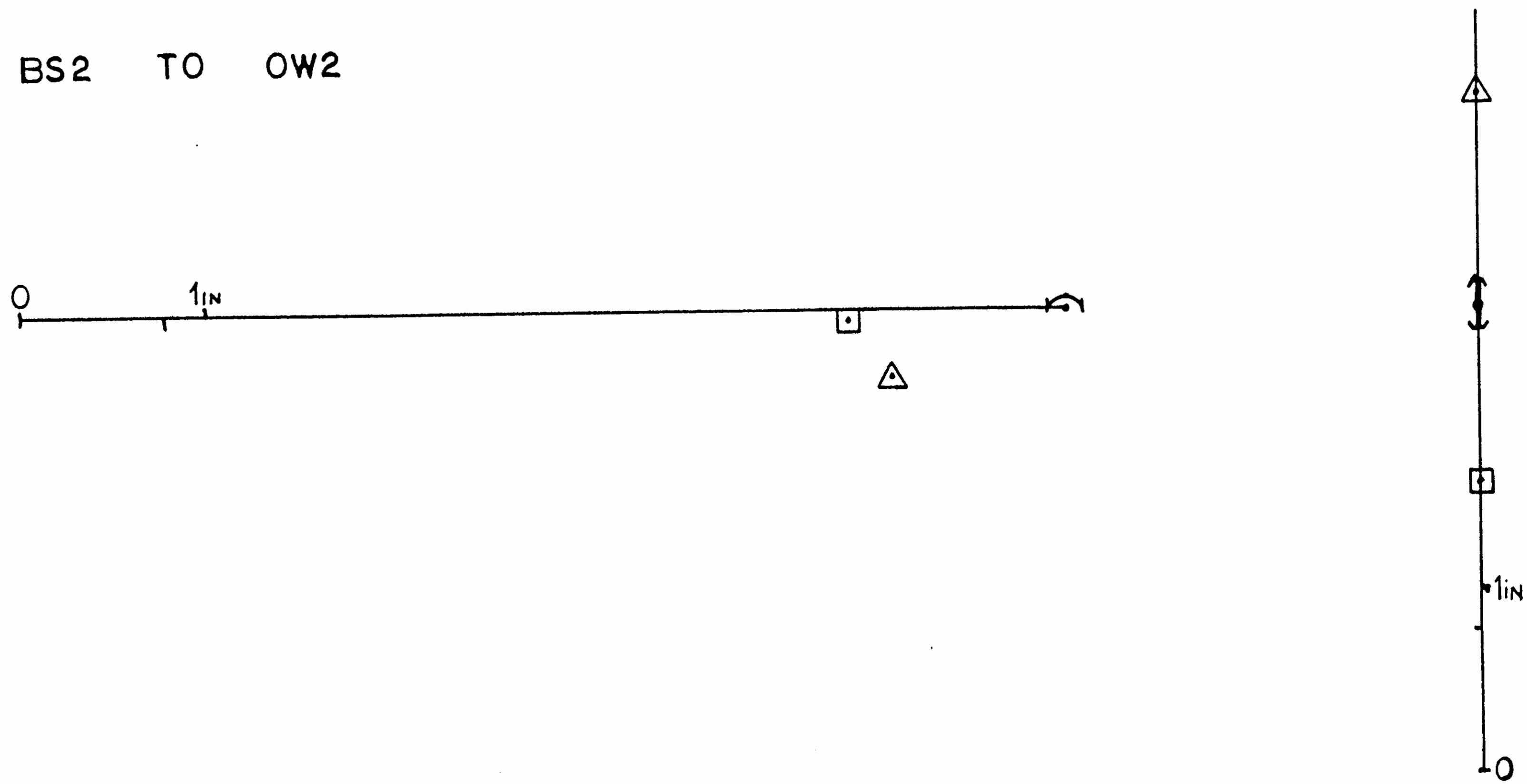


BS2 TO SM26

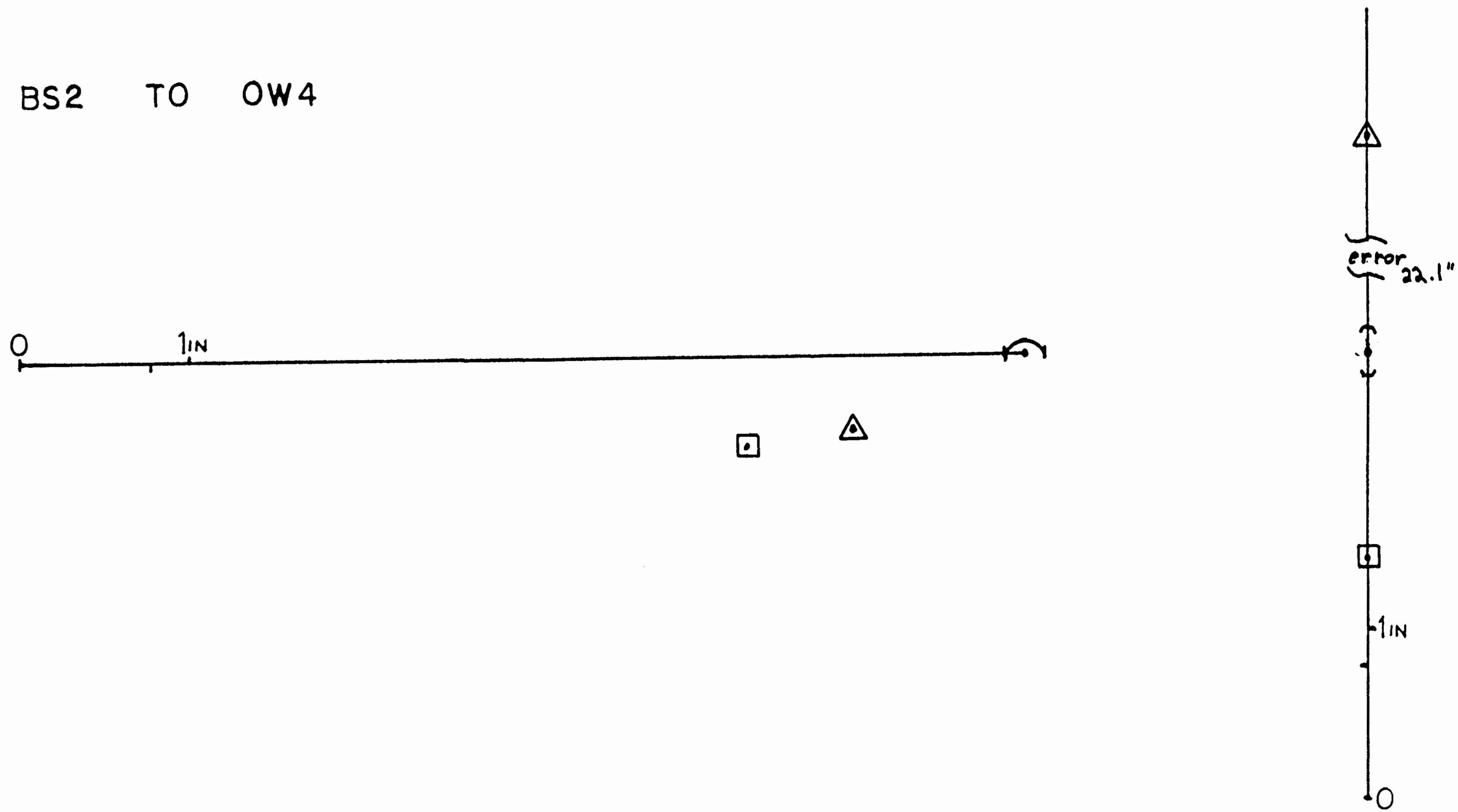


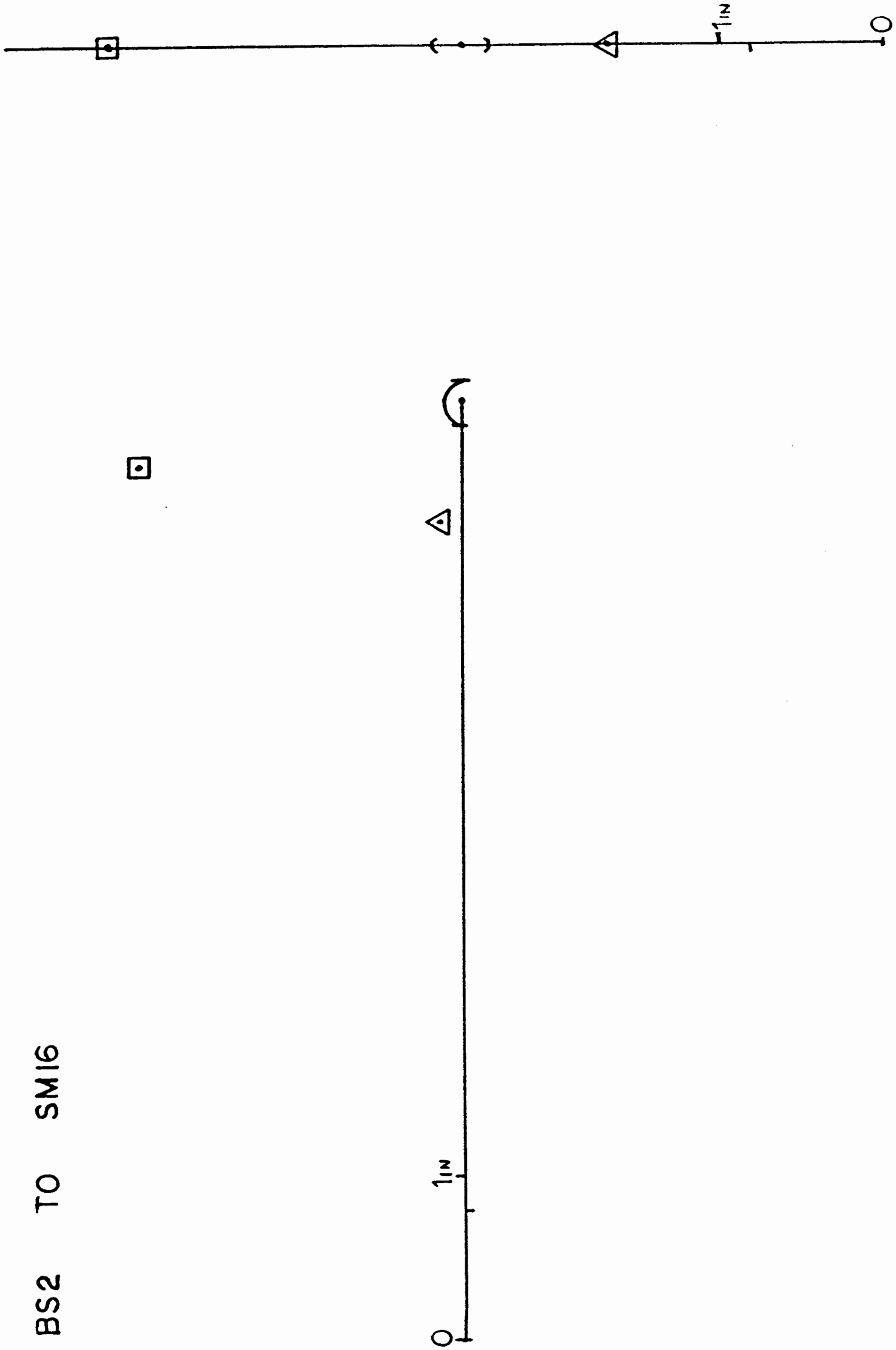

BS2 TO OW9
0
1IN
error 47.9"
1IN
0

BS2 TO OW2



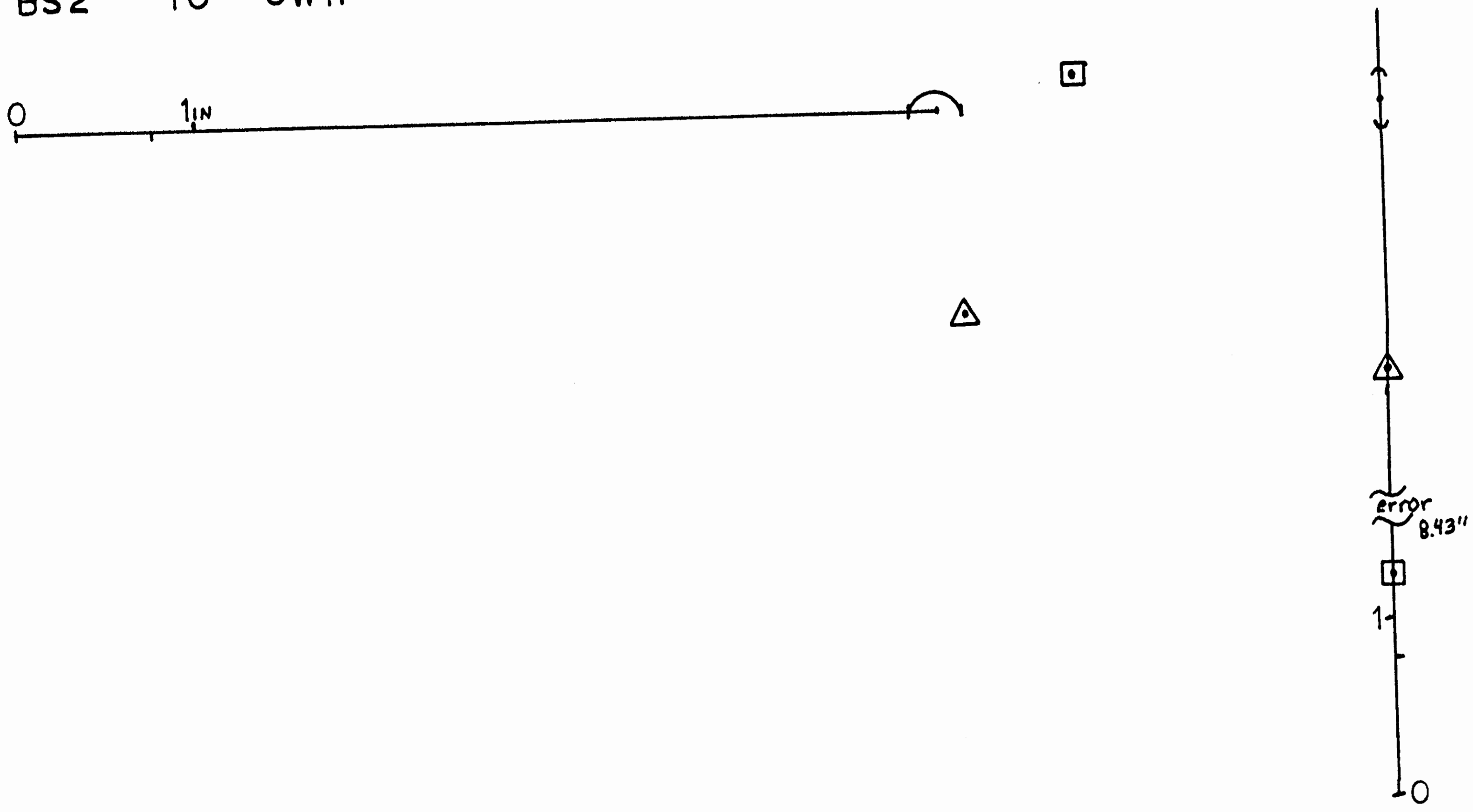
BS2 TO OW4



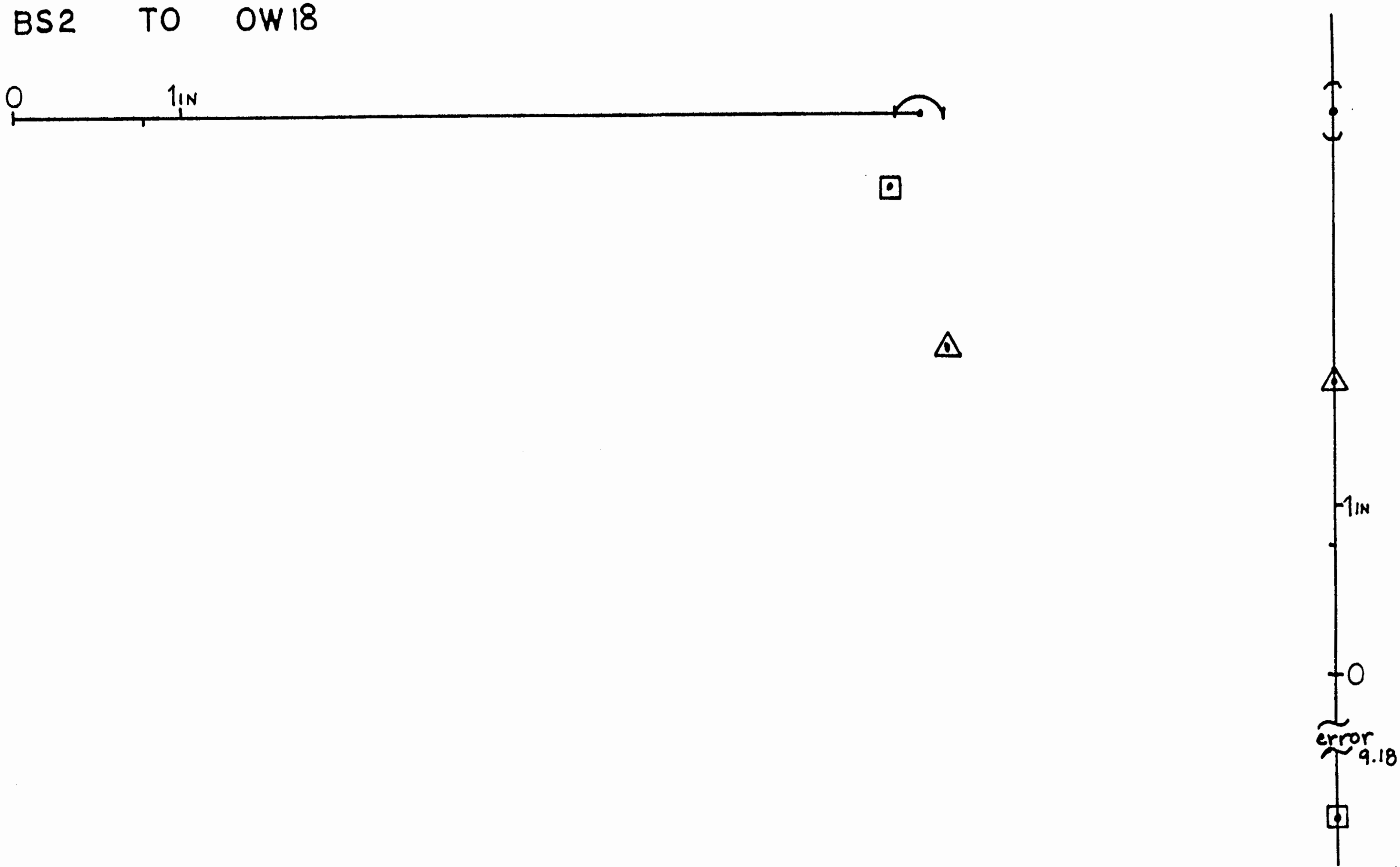


BS2 TO SM16

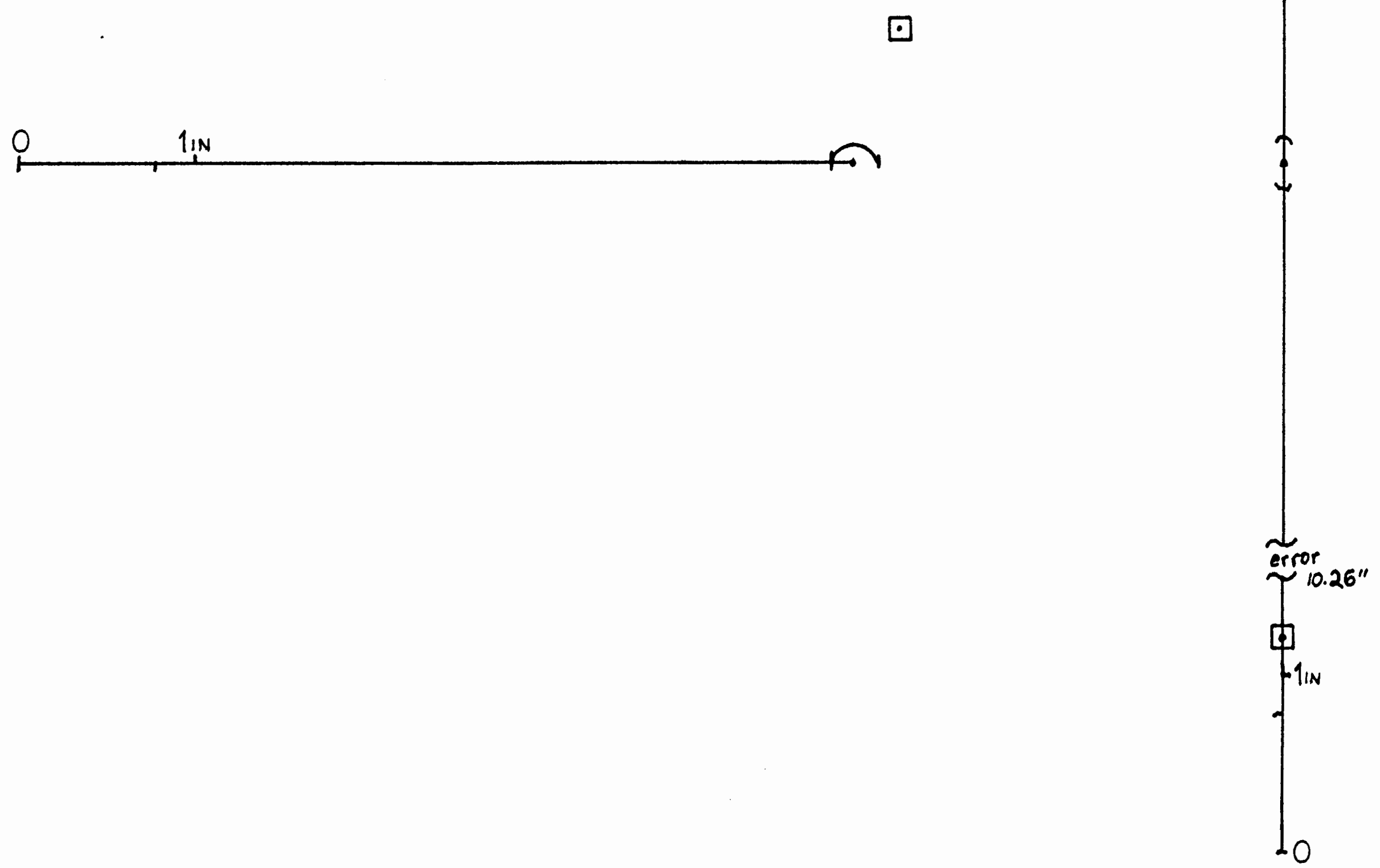
BS2 TO OW17



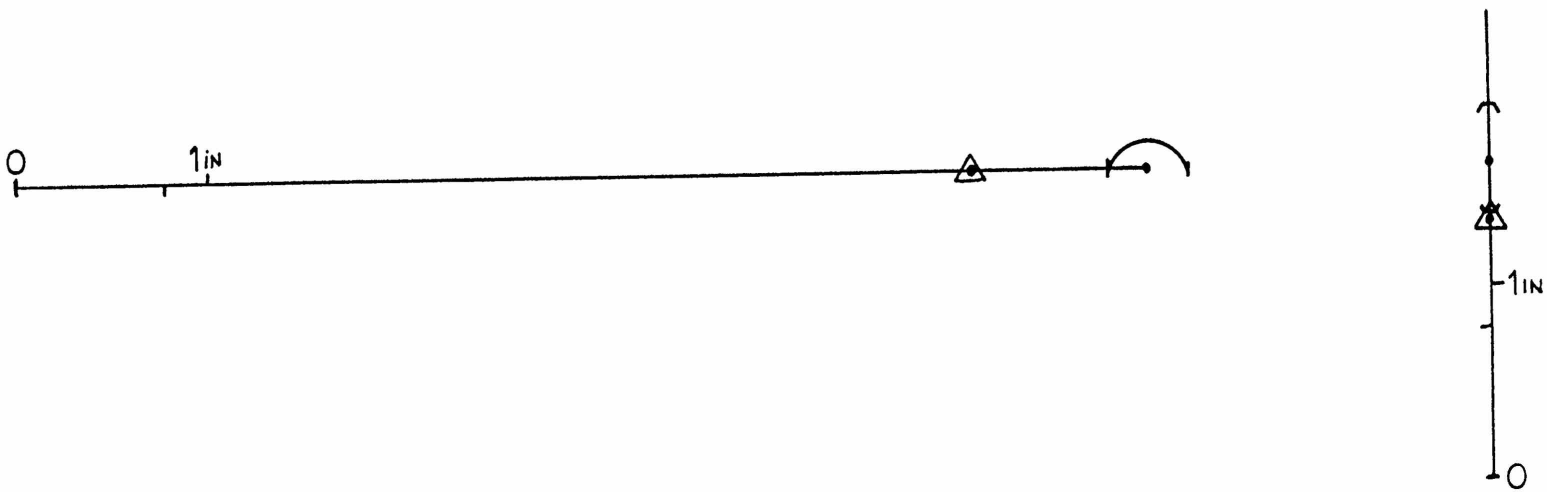
BS2 TO OW18



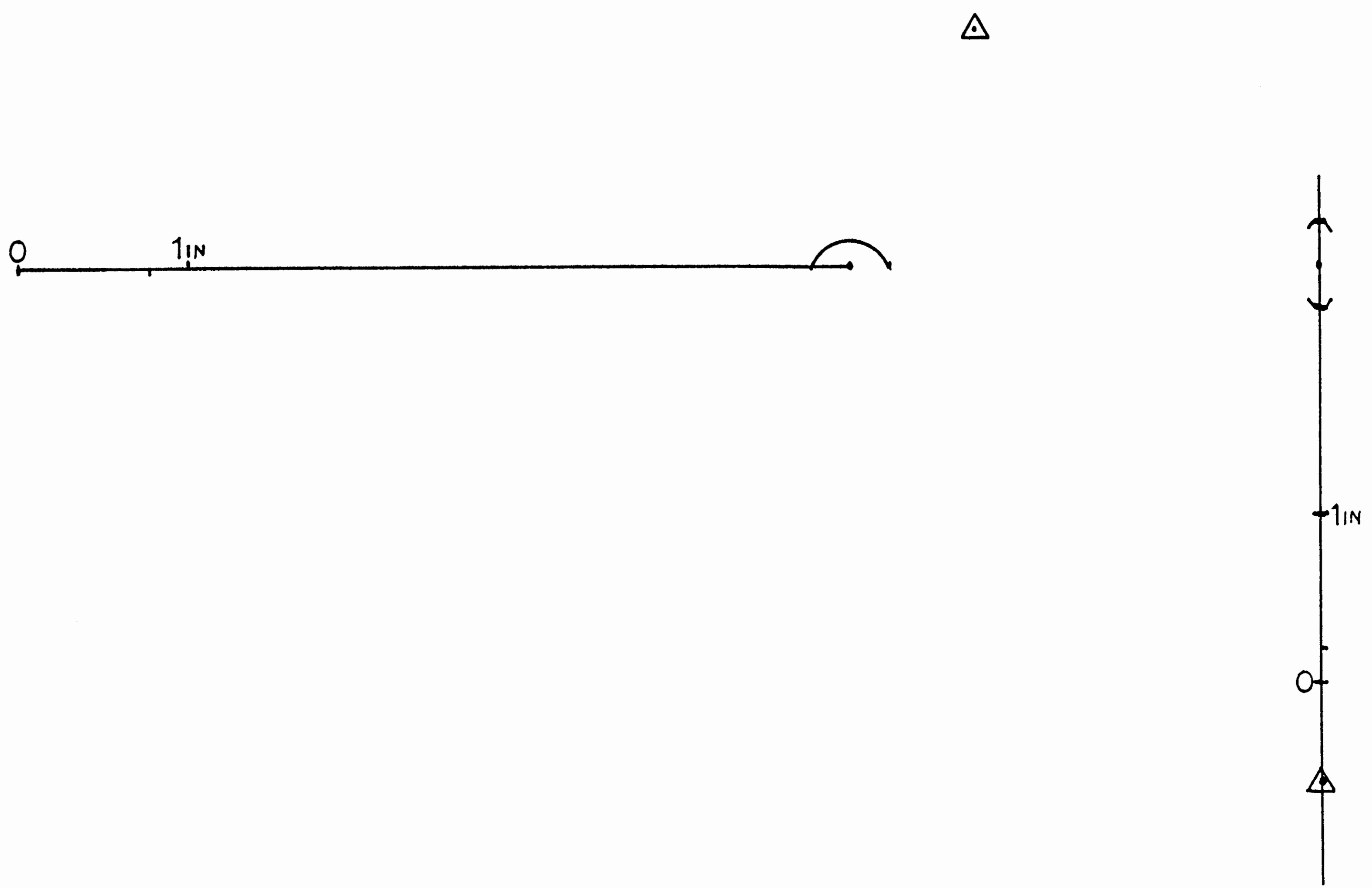
BS2 TO SM23



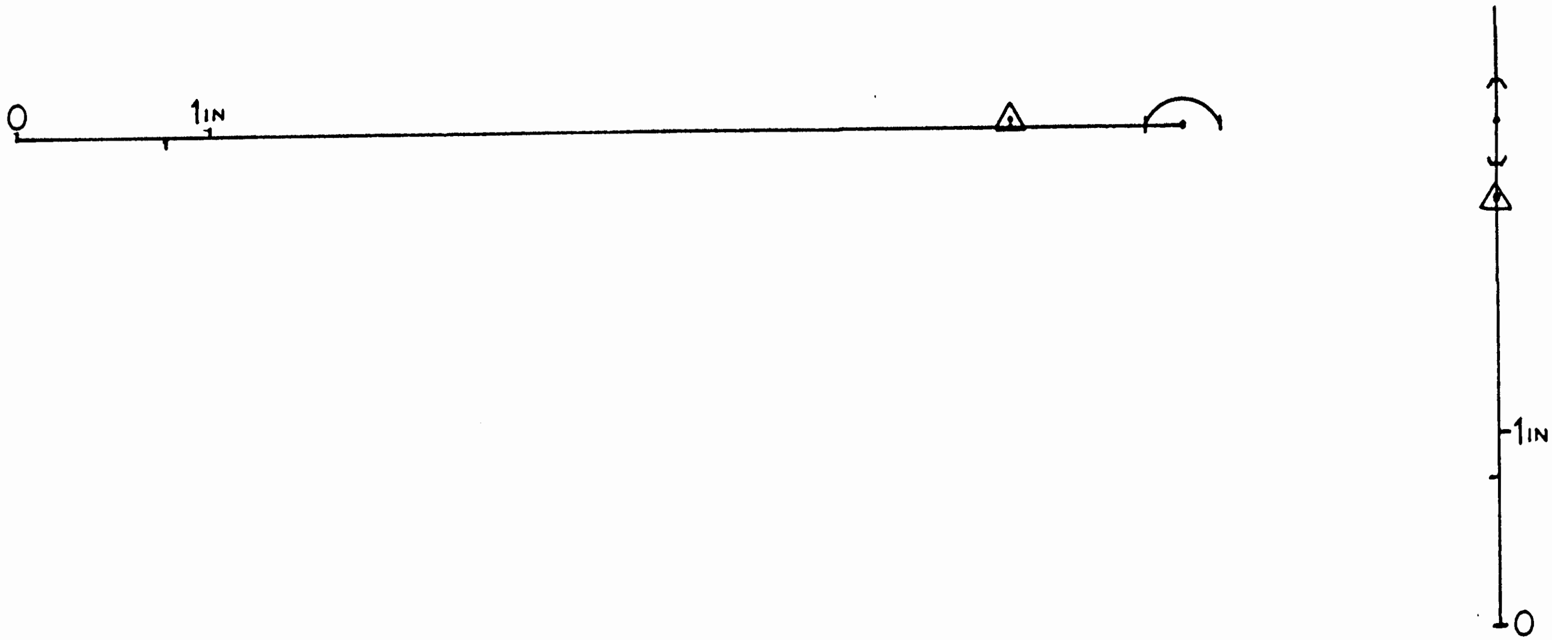
BS1 TO BS2



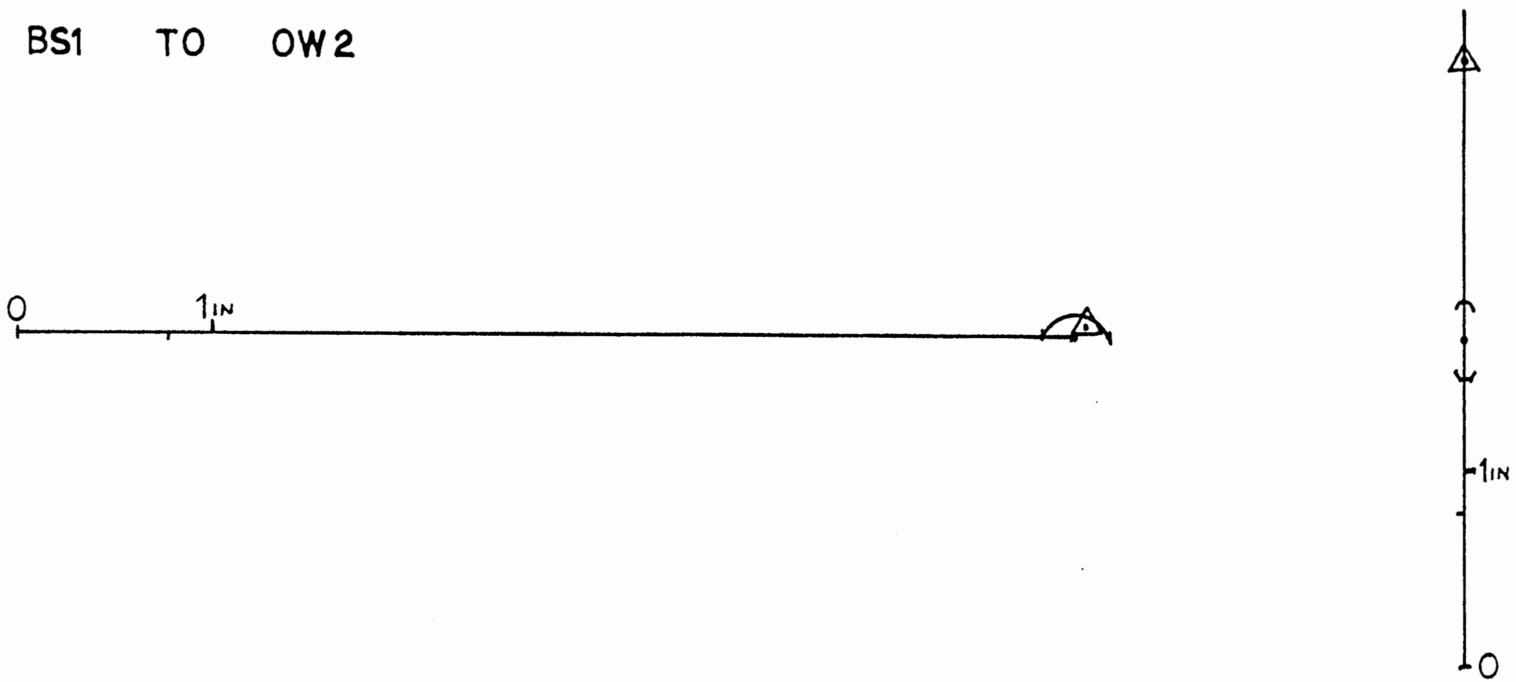
BS1 TO SM36



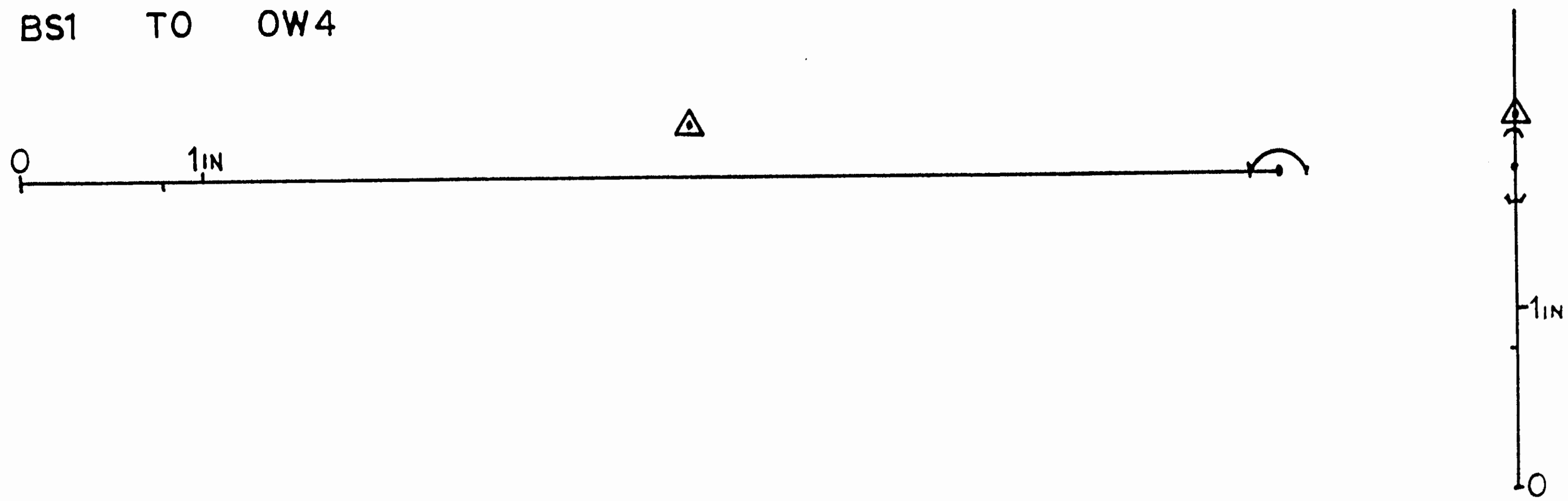
BS1 TO OW9



BS1 TO OW2



BS1 TO OW4



BS1 TO SM16