AN ANALYSIS OF A SLIDE AT THE COTTONWOOD BRIDGE

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May 14th, 1954.

The Council,
The Association of Professional Engineers
of the Province of British Columbia,
1166 West Pender Street,
Vancouver 1, B.C.

Gentlemen,

I herewith submit an engineering report entitled "An Analysis of a Slide at the Cottonwood Bridge" as partial fulfillment of the requirements for registration as a Professional Engineer in Geological Engineering in the Province of British Columbia.

The work on which this report is based was done for the B.C. Department of Mines in co-operation with the engineering staff of the Pacific Great Eastern Railway. I am normally engaged in studies for the B.C. Department of Mines of unconsolidated materials and ground-water. In this instance I assisted in the interpretation of the results of test drilling done for the Pacific Great Eastern Railway to determine the cause and extent of a slide which threatened the railway bridge over the Cottonwood River.

The report is my own work, but I wish to express my appreciation for the ideas and information gained in the course of the study from discussions with engineers of the Department of Mines, of the Pacific Great Eastern Railway, and of Engineering Drillers Ltd. of Vancouver. The manuscript was read by H.Sargent, S.S.Holland, and M.S. Hedley of the Department of Mines, by Mr. D.A. MacLean of the Provincial Water Rights Branch, and by Mr. A.L. Carruthers, consulting engineer for the Pacific Great Eastern Railway, and they made helpful suggestions.

Yours truly,

Hugh Nasmith

AN ANALYSIS OF A SLIDE
AT THE COTTONWOOD BRIDGE
ON THE PACIFIC GREAT EASTERN RAILWAY

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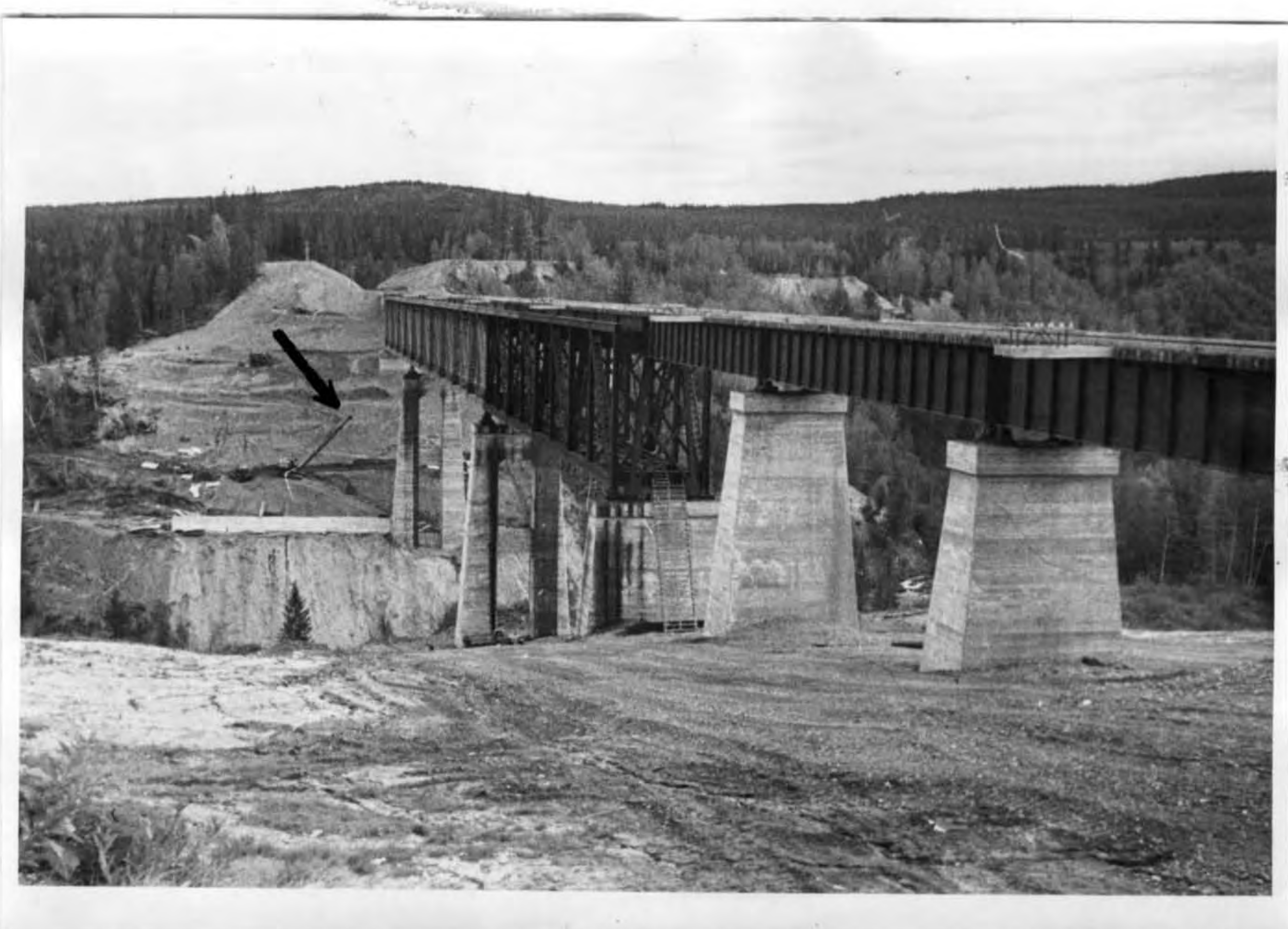
Hugh Nasmith

B. C. Department of Mines

May 1954



Cottonwood Bridge of the P.G.E. Railway looking north across the canyon of the Cottonwood River. Arrow indicates the general area of the slide on the north bank. (Photo by M.S. Hedley)

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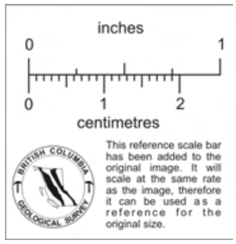
ILLUSTRATIONS

INTRODUCTION

In 1949 construction was resumed after a lapse of more than thirty years on the extension of the Pacific Great Eastern Railway from Quesnel to Prince George. One of the major problems to be overcome was the crossing of the Cottonwood River, a problem which had troubled the engineers thirty years earlier, for this river flows in a broad deep valley several hundred feet below the level of the surrounding terrain, and the banks of the valley are notoriously subject to sliding. The site for the bridge to cross the Cottonwood River that was finally chosen is about ten miles northeast of Quesnel as shown on the sketch map Figure 4, where the river flows through a narrow rock walled canyon. The span to bridge the gap was not excessively long, and the foundation conditions appeared to be more favourable than at any other place along the river.

The bridge was completed late in the fall of 1951, and as no unusual conditions had been observed during the construction no particular concern was felt for the stability of the foundations. In the spring of 1952 however, a small slide took place between the two piers of the bridge on the north bank of the river. Although the quantity of material involved in the slide was small when compared to some recent slides along the Fraser River in this vicinity, it indicated possible instability of the whole bank. As can be seen from Figure 1 which is a copy of the Pacific Great Eastern Railway location



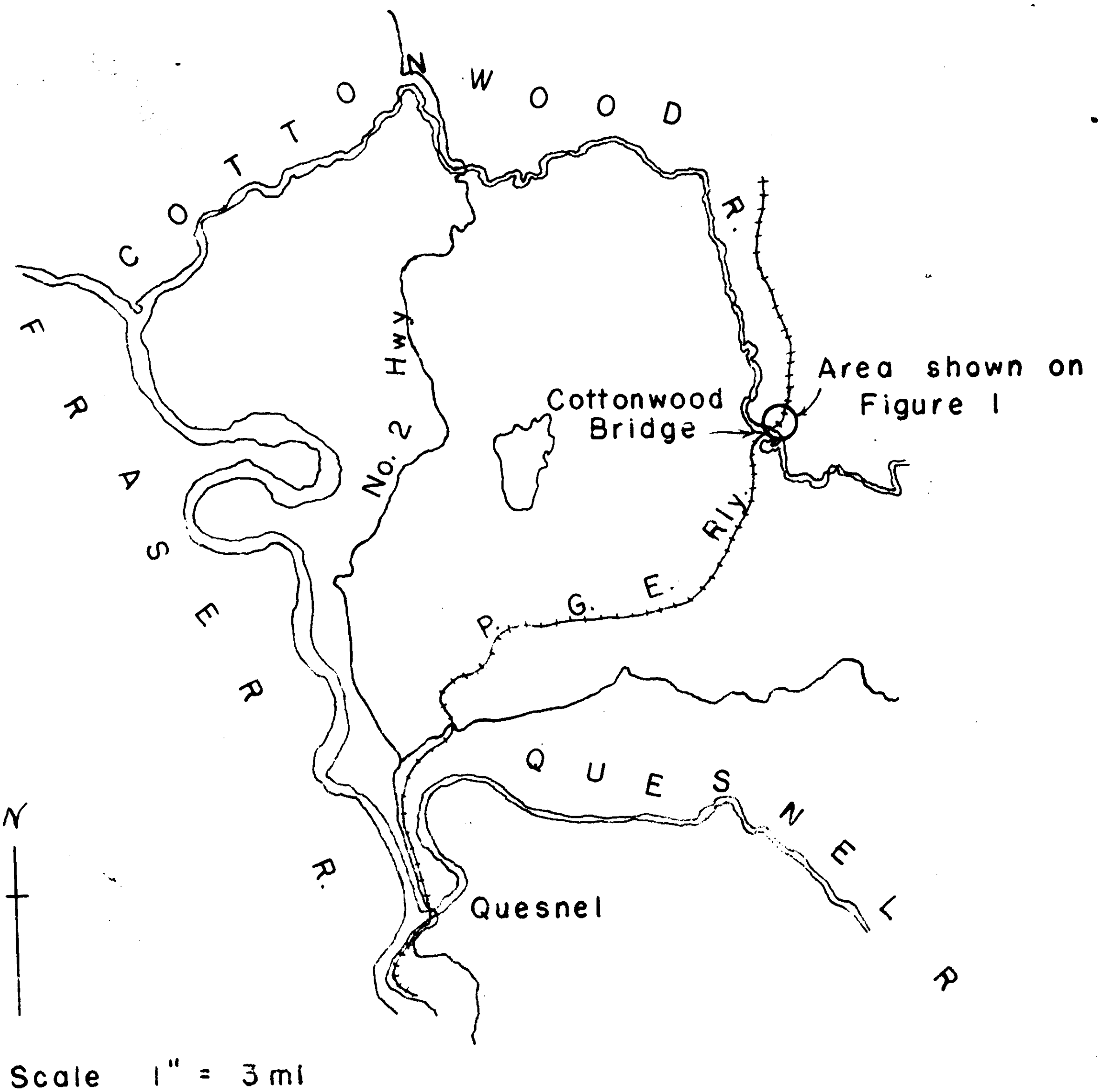


Figure 4 Sketch map showing location of Cottonwood Bridge

survey along this section of the railway, if further sliding took place involving material higher up the bank one of the bridge piers might be moved. Accordingly preventative measures were undertaken in an effort to stabilize the slope and a program of test drilling was begun in order to determine the extent and cause of the slide.

GEOLOGY OF THE SLIDE AREA

Figure 2 is a section through the area of the slide drawn along the center line of the bridge as shown on Figure 1, and showing significant information revealed by the test drilling together with graphic logs of two typical test borings. At the left of the section the Cottonwood River flows in a narrow postglacial canyon excavated in bedrock. The bedrock wall on the north side of the river rises sharply to an elevation of 2370' and then apparently slopes down to the north. The configuration of bedrock in the river channel upstream and downstream from the canyon, combined with the evidence from test drilling, clearly indicates that an interglacial or pre-glacial channel of the Cottonwood River lies immediately north of the present channel. The approximate location of this buried channel is shown dotted in red on Figure 1.

The ancient river channel is filled with more than 200 feet of unconsolidated sediments deposited during the glacial period, and the rock ridge that separates the ancient river channel from the present one acts as an immense natural retaining wall to prevent the mass of unconsolidated material from slumping into the river. Upstream and downstream from the rock walled canyon the Cottonwood River apparently flows in its ancient channel, and much of the glacial debris has slumped into the river and been washed away.

The character of the unconsolidated materials which make up the bank in the vicinity of the slide was not investigated by drilling below about elevation 2370', the top of the natural rock buttress. From this elevation up to about 2390' the unconsolidated material consists of an unsorted mixture of sand, silt, clay, and gravel up to cobble size. This material is moderately tough and cohesive, and may be a basal till deposited by a glacier that invaded this area during the ice age; or it may be an ancient mudflow which was overridden and compacted by a glacier.

At about elevation 2390' this unsorted mixture of sand, silt, clay, and gravel rapidly changes to a silty deposit still containing scattered pebbles, and from about 2400' to 2410' the material is a silty clay in which are layers a fraction of an inch thick of very plastic sticky clay and several layers an inch or two thick of clean-washed medium to coarse sand. This silty clay layer with interbedded sand strata is crumpled and contorted. A very striking feature of this layer are glassy slickensided surfaces where the clay has been sheared. At first it was thought that this shearing and crumpling were related to the movement of the slide. However, similar features were found when drill holes outside the area of the slide reached this silty clay horizon which is essentially horizontal throughout the area tested by drilling. It is believed that these silty clays with interbedded sand layers were deposited in a lake adjacent to the ice sheet, and that as the ice advanced into the lake these lake sediments were crumpled and sheared.

Above these silty clays from 2410' to 2500' is the basal till deposited from the bottom of the ice as it advanced into the lake. This basal till is a dense compact mixture of sand, gravel, silt and clay. Although very similar to the material underlying the silty clay layer, the basal till overlying the silty clay contains a consistently lower proportion of clay and seemed to be somewhat tougher and harder to drill. Within this basal till there are apparently discontinuous lenses of washed gravel and a few thin water bearing strata a fraction of an inch thick, probably consisting of sand. At the top of the slope above elevation 2500' about 30 feet of varved silts overlie the basal till. These varved silts were probably deposited in an extensive lake which formed immediately after the retreat of the last glaciers in this region.

MODIFICATION OF THE NATURAL SLOPE

On Figure 3, which is the same section as Figure 2, the original undisturbed profile of the slope is shown as a dashed line. When the excavation for the foundation of the pier which stands on the rock ridge was made, some of the material upslope sloughed into the excavation, and the slope was accordingly graded to a uniform 2 to 1 slope as shown by the dotted line.

The material which was removed th the course of grading the slope was probably similar to the material composing the rest of the slope except that it may have been loosened by weathering, frost action and the influence of roots and burrowing animals. There is a possibility, however, that the bulging slope from about elevation 2370' to 2410' was made up of sands and gravels that were deposited when the Cottonwood River flowed at a higher elevation as it was cutting its channel down to the present grade following drainage of the postglacial lake.

In the course of construction of the piers and erection of the bridge the slope was notched in several places for access roads and two small pits were dug to facilitate erection of the steelwork of the bridge.

OUTLINE OF THE BLOCK OF GROUND THAT MOVED

In order to asses the seriousness of the slide and decide upon the proper corrective measures it was desirable to define the ~~outlines~~ outline of the block of ground involved in the slide and to determine the nature of the movement. Fortunately it was possible to do this fairly accurately with the information from test drilling and direct observation.

The headwall of the slumped block was clearly visible at the surface. Thus in Figure 3 one point on the plane of failure of the slide is defined. In the course of early efforts to check the slide a trench had been excavated and a drain put in along the headwall of the slide. The position and elevation of this drain was recorded so that a second point was known. Drilling in the slide area indicated fairly clearly that the ground below elevation 2390 was undisturbed by the slide, and the exposures on the slope suggested that the plane of failure had intersected the ground surface not lower that 2400'. From this information it is possible to draw the probable outline of the slide on Figure 2 with some assurance.

Within the slide area the drill holes intersected the upper surface of the silty clay layer at a lower elevation than it appeared at the surface. This indicated, as shown in Figure 2, that the silty clay layer within the slide was tipped back into the slope. In the undisturbed ground outside the slide the silty clay horizon is essentially horizontal. Therefore it is clear that the downward and outward movement of the slide was accompanied by a backward rotation of the slumped block.

COMPUTATIONS BASED ON SHEAR ALONG A CIRCULAR ARC

The curved form of the inferred plane of failure and the indication of a backward rotation of the slumped block suggest that computations based on a circular arc of failure such as proposed by various soil mechanics authorities* would be justified. Figure 3 shows the slope and the inferred outline of the slide in section. The circular arc A B C has

*see for example Terzaghi, K., The Mechanism of Landslides, in Application of Geology to Engineering Practice, Sidney Page, Ed. Geological Society of America, New York, 1950.

its centre at D and is as good a fit to the inferred plane of failure as can be obtained. Several other circular arcs which nearly fitted the plane of failure were tried and computed, and they gave essentially similar results.

The segment bounded by the arc A B C and the 2 to 1 slope of the ground surface at the time of failure has an area of approximately 1990 square feet. Considering a section one foot thick with an average density of 120 pounds per cubic foot this segment has a weight of 238,800 pounds which may be considered to act through the centre of gravity of the segment at E. Thus the weight of the segment acts along a vertical line 14.5 feet to the right of the centre D and exerts a turning moment of 3,460,000 ft. lbs (238,800 x 14.5) tending to rotate the segment in a clockwise direction. This turning moment is opposed by a counterclockwise moment made up of the total shearing resistance developed along the arc A B C acting at a distance D A (the radius of the arc) from the centre of the arc D. The length of the arc A B C is 115' and the radius D A is 51'. In order therefore, for the segment to be stable as to sliding along the arc A B C the moment developed by the shearing resistance must be equal to or greater than the moment developed by the weight of the segment. Therefore, average shearing resistance per square foot along A B C x 115 x 51 $\geqq$ 3,460,000; and on the average each square foot along the arc must be capable of developing a shearing resistance equal to or greater than 590 pounds.

This theoretical value of shearing resistance which would be overcome when failure took place along the arc A B C probably bears no close resemblance to any comparable value which could be determined from tests on samples of the soil making up the slope. Among the reasons for this are; the true length of the plane of failure is uncertain; the soil mechanics properties probably vary within the recognized divisions of material making up the slope; the actual mechanism of such a slide even in uniform material is not clearly understood. Although the figure 590 pounds per square foot probably has little absolute significance and should be interpreted very cautiously, it serves to make useful comparisons as is shown in the following paragraphs.

Similar computations were made for a segment of the same circle centred at D bounded by the original ground surface instead of the surface graded to a 2 to 1 slope. This represents the conditions which prevailed along the theoretical arc of failure before construction was begun, conditions under which the undisturbed slope had evidently remained stable for many years prior to construction. These computations indicate that prior to construction an average shearing force of 380 pounds per square foot had to be developed along the theoretical arc of failure in order to prevent sliding. Thus, as a result of grading the slope, the shearing force along the plane which actually failed was increased by about 55%. It is thought that this large percentagewise increase in shearing stress was an important factor in bringing about the failure at this point on the slope.

Eight other potential planes of failure along circular arcs were also computed. These circular arcs were drawn on the assumption that they would come to the surface on the upslope side in a vertical direction, and that they would be tangent to the tough material underlying the silty clays at elevation 2390'. Although computations based on these potential arcs of failure are not strictly comparable they seem to have some significance.

It is interesting to note that the percentage increase in shear stress on potential arcs of failure when comparing conditions before and after grading the slope was greatest (as much as 140%) for arcs downslope from the actual failure, and rapidly decreased (to less than 25%) for arcs which emerged upslope from the actual failure.

If the undisturbed slope was stable because the steepness of the slope was adjusted to the shearing strength along potential arcs of failure within the bank, it is to be expected that failure would take place where the disturbance was greatest; that is, along the arc where the percentage increase of shear stress was greatest. This seems to have been the case as photographs taken shortly after the slide suggest that the ground moved in three or more slices. Possibly the initial failure took place along the arc of maximum percentage increase of shear stress. This initial failure would then increase the shear stress along an arc farther up which would fail next. This sequence would continue up to the arc along which the increase was insufficient to cause failure, or at any rate insufficient to cause immediate failure.

GROUND-WATER

Although the changes in shear stress conditions on the plane of failure which were brought about by grading the slope were probably an important factor in producing the slide, they were not the sole cause. After completion of construction, the graded slope was stable for nearly six months. The immediate cause of failure, the factor which actually triggered the slide, was probably the unusual local ground-water conditions which existed in the spring of 1952.

A program of test drilling and observation of ground-water conditions conducted over a short part of a year will give a very incomplete picture of the ground-water conditions as they may have existed at a different season that year. The sequence of sediments revealed by drilling was the same as at the time the slide took place; the physical properties of the sediments probably did not change greatly between the time the slide took place and the time when they were examined; but the ground-water conditions as they were observed in mid-summer were undoubtedly vastly different from what they were when the slide occurred in the spring. Therefore the ground-water conditions as they were at the time of the slide can only be inferred from a few clues, and the significance of the ground-water conditions in relation to the slide is difficult to evaluate.

During the summer of 1951, when construction was in progress, it is reported that no springs or seepages were observed on the slope. The material which was excavated was damp but not abnormally so. However, the slide in the spring of 1952 was accompanied by an outflow of saturated mud.

In the seven months preceding the slide precipitation was abnormally high, amounting to 12.9 inches at Quesnel, in contrast to a long term average of only 8.3 inches for the same months. During the corresponding months in the year preceding construction precipitation amounted to 11.0 inches. Therefore during the months preceding the slide the precipitation was substantially greater than usual and might account for exceptional ground-water conditions in comparison to the conditions that had existed at the time construction was in progress. In addition to precipitation, factors which cannot be readily evaluated such as the form in which the precipitation occurred, whether rain or snow, and the state of the ground when moisture was available, whether frozen or unfrozen, may have combined to produce almost unique ground-water conditions in the spring of 1952. Possibly if the slope had not failed in 1952 it would be many years before circumstances combined again to produce conditions which could bring about failure of the slope.

In the course of test drilling no great quantity of water was discovered, but though the quantity of water was small, in several instances it was found under sufficient pressure to raise it above the level at which it was encountered. The highest level to which the water rose in any of the drill holes was about 2430'. The water evidently occurred in thin sand layers, not more than an inch thick, contained in the glacial till, and the impervious character of the till suggests that the water in these sand layers had not percolated vertially downward to them, but had migrated laterally from outside the immediate area of the slide.

As can be seen from Figure 1, the obvious place where precipitation might percolate into the ground and then move laterally to the area of the slide is the little valley containing an intermittent stream about 1000 feet north of the slide. No test drilling was done in this area so that the character of the material beneath the surface is unknown, but coarse bouldery material is present in the bed of the little stream. Possibly this little valley is underlain by deposit of gravel which can collect and store infiltration from the bed of the stream, and from which the water can flow laterally into any permeable horizons with which it is in contact. The elevation of the bottom of the creek is adequate to account for the greatest hydrostatic head found in any of the drill holes.

Although no water under pressure was found in the sand layers which are interbedded with the silty clay, a substantial quantity of water was collected from these sandy layers by the drain at the headwall of the slide. The silty clay band is horizontal and apparently unbroken outside the slide area and it is quite possible that these sand layers are able to transmit water which percolates into the ground beneath the intermittent stream to the north. The hydraulic gradient within these sand layers could be as high as 50 feet in 1000. Possibly the flow of water from these sand layers immediately prior to the slide was sufficient to wash some of the sand out, leaving the overlying material unsupported and increasing the shearing force along the plane of failure. This would be similar to piping which has often initiated the failure of an earth fill dam. As noted earlier the material which made up the bulge in the natural slope between elevations 2360' and 2410' and which was removed when the slope was graded, was probably similar to the material making up the rest of the bank. If however, this bulge consisted of material reworked by the Cottonwood River, it would be much more permeable than the rest of the material composing the slope and by allowing the sand layers to drain freely without developing high seepage velocities would act as a natural toe drain to prevent piping whenever seepage through the sand layers was exceptionally great.

Thus the grading of the bulging natural slope to an artificial 2 to 1 slope may not only have directly increased the shear stress along the plane of failure, but may have increased the effectiveness of the ground-water in making the slope unstable.

In addition to producing piping in the sand layers, the hydrostatic pressure acting through all the minute interconnected pore spaces in the material making up the bank would exert a pressure up and out to reduce the stability of the bank.

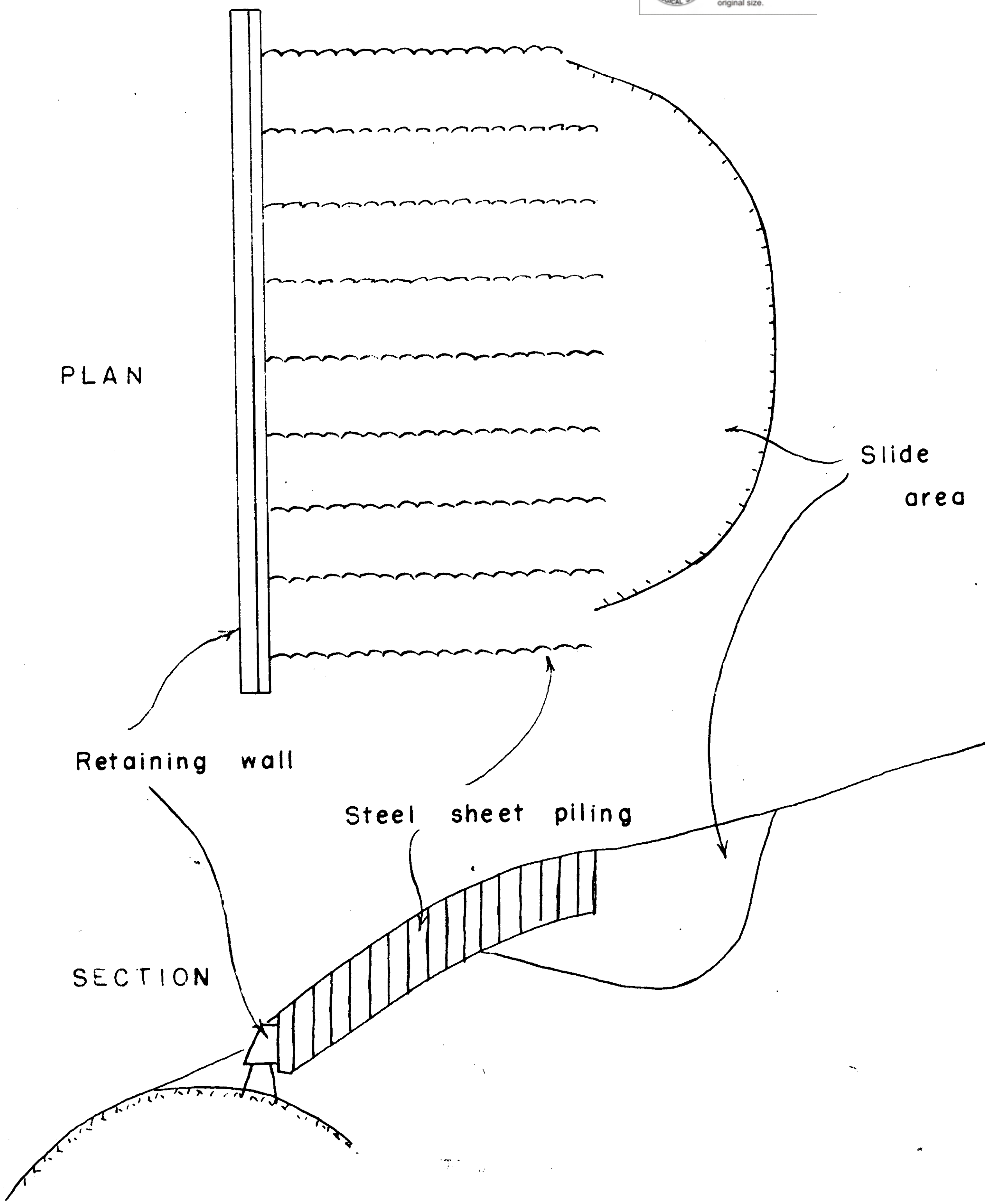
SUMMARY OF THE CAUSES OF THE SLIDE

The increase in shearing stress on the plane of failure brought about as a result of grading the natural slope was an important cause of the slide. The position of the plane of failure was partly governed by the presence of a silty clay band which had been sheared by glacial action and had much lower shear strength than the remaining material of the slope. The position of the headwall of the slide may have been governed by the location of a notch cut in the slope for an access road. Seepage through sand layers within the silty clay may have been sufficient to induce piping and consequent settlement of overlying material. Hydrostatic pressure within the slope produced by apparently exceptional ground-water conditions no doubt contributed to the failure.

MEASURES TO STABILIZE THE SLOPE

Because of the urgency of the situation steps were taken before the test drilling and analysis of the slide were complete in an effort to stabilize the bank. The corrective measures ultimately adopted however, were based on an interpretation of the slide essentially as outlined in the preceding section.

The slope was rebuilt to approximately the profile it had prior to construction so that the influence of the bulge between elevation 2360 and 2410 as a load on the toe of the



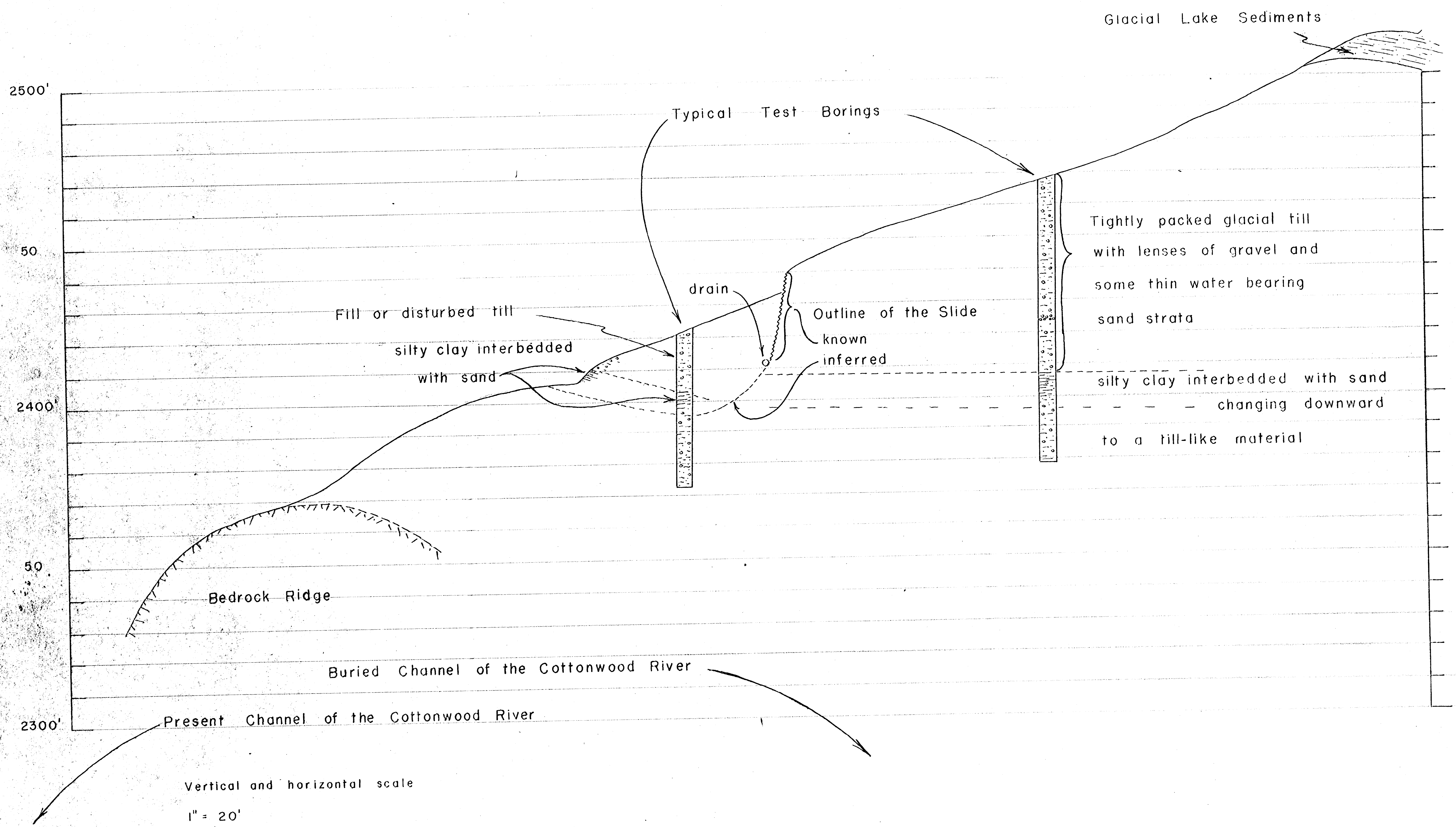
Sketch showing general arrangement of counterforts

Figure 5

block of ground that moved was restored. In addition the stability of the slope was increased by a series of steel sheet piling counterforts driven up the slope at right angles to the headwall of the slide. Figure 5 illustrates diagramatically the arrangement of these sheet piling counterforts which are anchored outside the area of the slide to a heavy concrete retaining wall set on steel piling driven to bedrock. The top of the retaining wall is at elevation 2380 so that it is below the lowest point on the plane of failure and its resistance to movement is carried into the area of the slide by the counterforts.

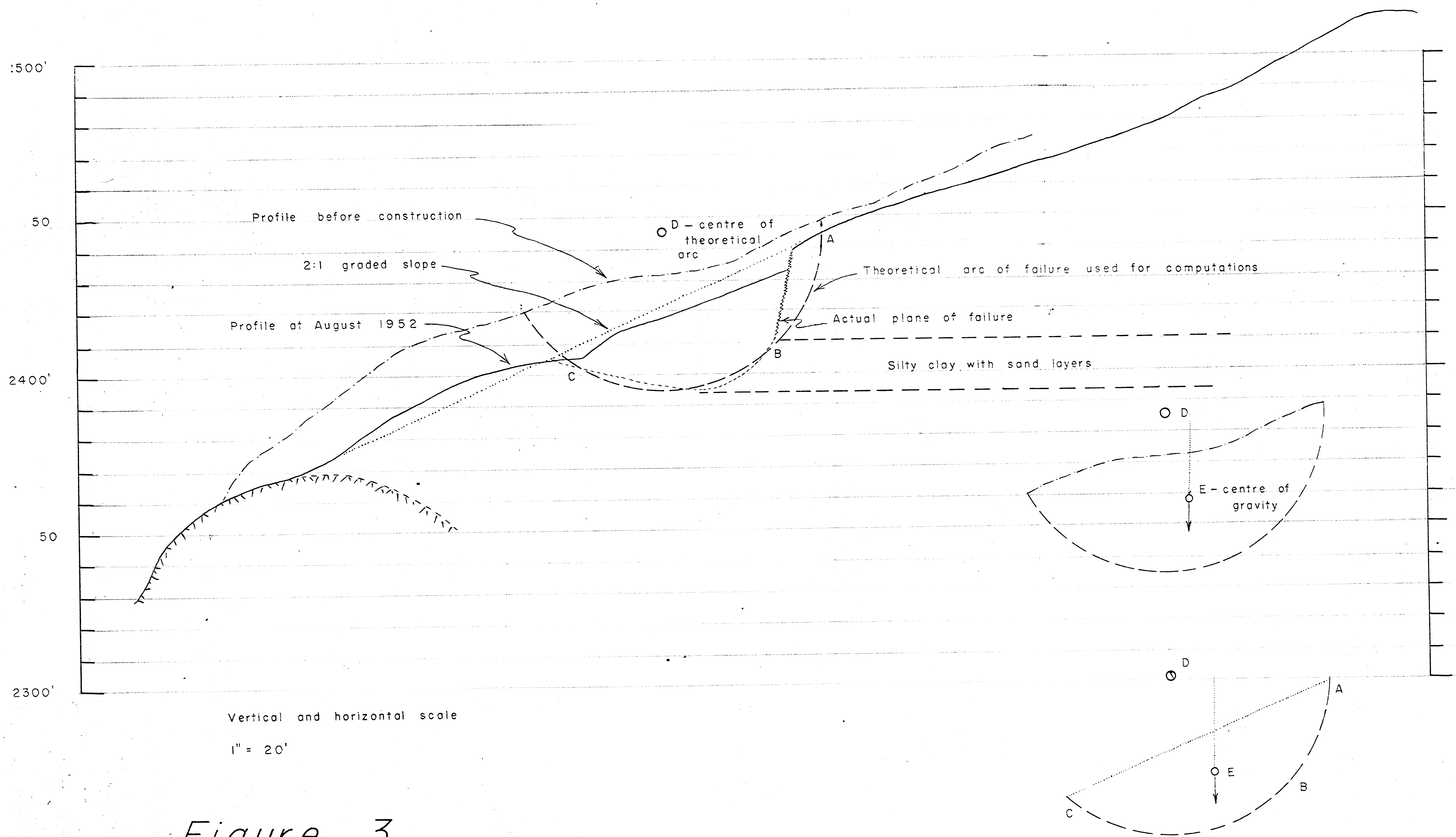
Measures to correct the unfavourable ground-water conditions consist of a drainage tunnel in the silty clay layer. It is driven in a northeast direction as shown on Figure 1. A series of vertical holes were drilled from the surface to the tunnel and packed with gravel. These vertical drains are expected to collect seepage from any water bearing strata they pass through, and to carry it to the tunnel which is also packed with free draining material. It is believed that this tunnel and the vertical drains will effectively cut off seepage into the area of the slide from the north as well as reduce any hydrostatic pressure which might develop.

Steps have therefore been taken to correct what are regarded as the chief causes of this slope failure. If the interpretation of the causes is correct and the corrective measures effective, the slope has been restored to a condition possibly more stable than it was prior to construction. However, because of uncertainties inherent in this type of problem the slope will have to be regularly observed for any indications that it may not yet be stabilized.



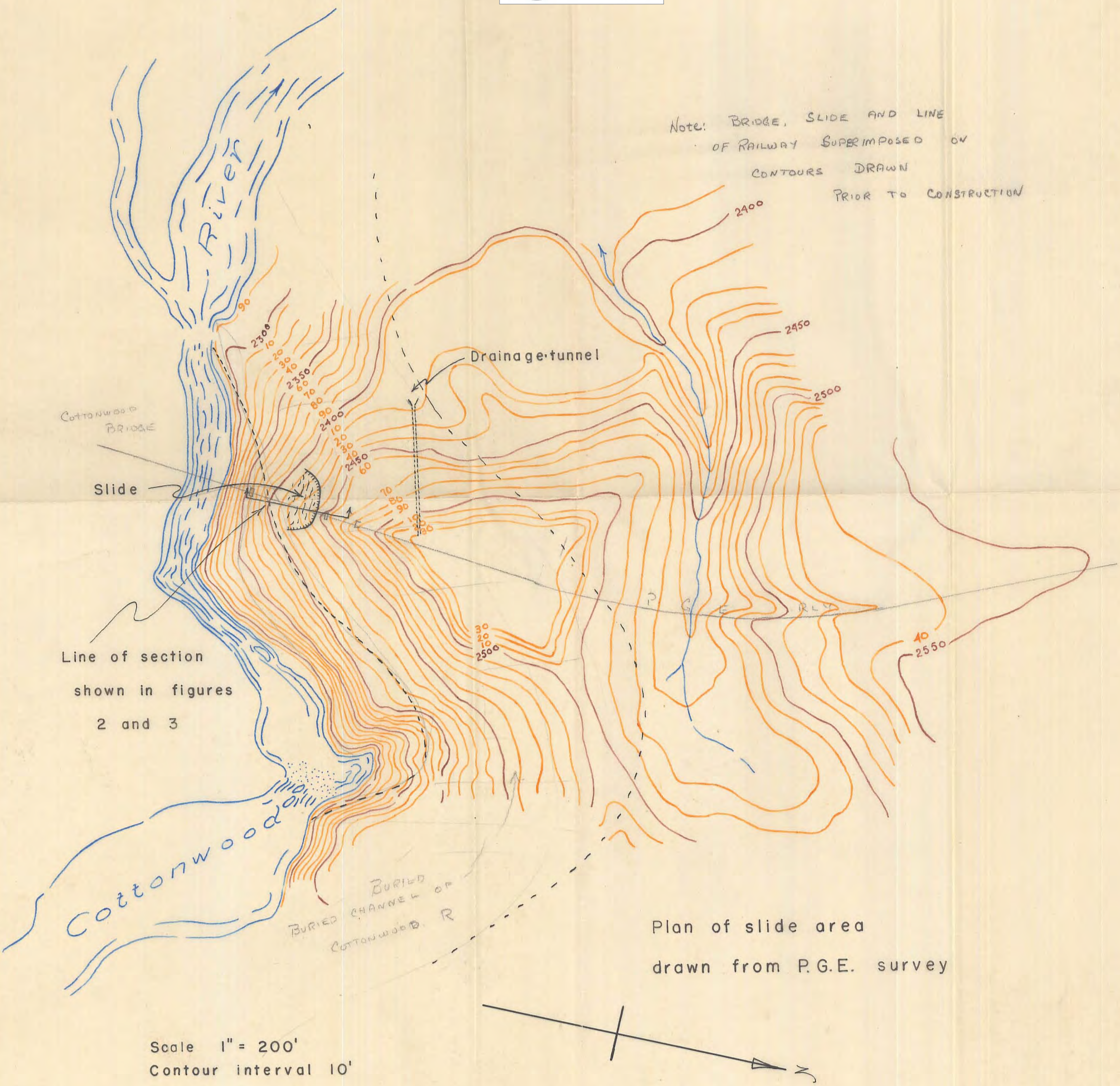
Glacial Lake Sediments
2500'
Typical Test Borings
50
Tightly packed glacial till
with lenses of gravel and
some thin water bearing
sand strata
drain
Fill or disturbed till
Outline of the Slide
known
inferred
silty clay interbedded
with sand
silty clay interbedded with sand
changing downward
to a till-like material
2400'
50
Bedrock Ridge
Buried Channel of the Cottonwood River
2300'
Present Channel of the Cottonwood River
Vertical and horizontal scale
1" = 20'

Figure 2



2500'
50
2400'
50
2300'
Profile before construction
2:1 graded slope
Profile at August 1952
D - centre of theoretical arc
A
B
C
Theoretical arc of failure used for computations
Actual plane of failure
Silty clay with sand layers
D
E - centre of gravity
D
E
A
B
C
Vertical and horizontal scale
1" = 20'

Figure 3



Plan of slide area drawn from P.G.E. survey

Scale 1" = 200'
Contour interval 10'

Figure 1