#60-F-291-C

HWY. #17

HAVILLAND BAY

CAUSEWAY

23-58-613.

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January 1960.

Ref: 907

Mr. A. Rutka,
Materials and Research Section,
Department of Highways,
Parliament Buildings,
Toronto 2, Ontario.

Dear Sir:

HAVILLAND BAY CAUSEWAY

I attach my report on the investigation into the failure of the embankment at Havilland Bay on Highway 17, Trans-Canada Highway.

This report should be read in conjunction with my "Technical Note on Construction" dated November 26th, 1959.

Together these two documents give an analysis of the failure and recommendations for the reconstruction.

Yours faithfully,

Hugh Q Golder

HQG:er

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907 REPORT ON HAVILLAND BAY CAUSEWAY - HIGHWAY 17 T.C.H.

Introduction and Terms of Reference

At Havilland Bay, Ontario, Highway 17 is at present being reconstructed on an improved alignment. At the point where the highway crosses Havilland Bay on Lake Superior, it has been necessary to construct an embankment. During construction this embankment failed. This report deals with the investigation of the failure, and the redesign of the embankment across the arm of the lake.

Reference should also be made to a "Technical Note on Construction" dated November 26th which was issued to enable site work to continue as rapidly as possible.

Visit to Site

On November 6th I flew to Sault Sainte Marie with Mr. L. Soderman. On the following day we visited the site of the causeway, and inspected the slip which had occurred, the borings which were in progress, and a possible sand borrow area a few miles from the south end of the causeway.

We collected the drawings which were available of surveys of the bank before and after the slip, and arranged for further necessary measurements to be made. We also talked at length with the engineers on the site who had witnessed the slip.

Description of Failure

The embankment is built of rock which has been taken from an adjacent cut area and tipped over the soft clay bottom of Lake Superior.

In fact two failures have occurred but the first was so gradual that it was considered an unusual settlement rather than a shear failure.

This was between chainages 1388 and 1389 + 50. In this area the clay at the sides of the tipped bank rose considerably, to water level in places. However tipping continued and the movement was slow and finally ceased without any complete failure having occurred. This took place in June-July 1959. The height of the bank was 14 to 17 feet above lake bottom and about 12 feet above water level (601).

Description of Failure (continued)

The second failure was sudden and took place between chainages 1390 and 1397 + 50. The bank in this length was about 27 feet high above lake bottom or 17 feet above water. This collapse took place on October 14th, 1959. Large islands of clay appeared in the water on either side of the bank and the centre section dropped to about water level.

Borings showed that the rock had moved downwards to about level 570 in places as shown in figure 2. The borehole logs made by the Department of Highways, and the results of the in-situ vane tests and laboratory compression tests are given in Appendix I. They indicate a shear strength for the clay of about 600 lb/sq.ft.

Survey of Failure

A very careful survey of the area of the failure was made by the site and District staff of the Department of Highways. This consisted of profiles at 50 foot intervals across the bank for the whole of its length and extending some 200 feet on either side of it. From these profiles the plan and sections shown in figures 1 and 2 have been prepared.

This information indicates quite clearly the cylindrical nature of the second failure and the positions of the ends of the failure surfaces. It also shows that failure took place to both sides at once, the surface of failure extending somewhat further on the lake side, where the water and the clay is deeper, than on the land side.

Analysis of Failure

By analysing the failures it is possible to obtain a value for the shear strength of the underlying clay which can be used in the redesign with more confidence, and therefore with a lower factor of safety, than a value obtained from field or laboratory tests alone.

If both failures are analysed using a factor of safety of unity, which is usual, two different values for the shear strength are obtained, these are 300 - 350 lb/sq.ft. for the slow failure and 400 - 500 lb/sq.ft. for the second and rapid failure. This difference can be explained. If loading is very slow in a clay soil, slow plastic deformations take place under stresses lower than the ultimate shear strength measured by a quick, i.e. normal, test. Thus, in order to compare the values obtained from the two failures, and also to obtain a shear strength which will ensure that slow plastic deformations do not occur in the reconstructed bank, the factor of safety used in the analysis of the second failure should be less than unity. This means that if the bank had been built more slowly it would have failed at a lower height than that it had reached when failure actually occurred. Exactly what value should be used is not known, but if a value of 0.9 is used the shear strength is 360-450 lb/sq.ft.

Analysis of Failure (continued)

In the redesign of the bank a value of 400 lb/sq.ft. has been used together with a factor of safety of 1.2.

The factor of safety of the berm at elevation 606 on the west side between chainages 1395 +50 and 1398 + 30 may be less than 1.2 and it is possible but not likely, that failure of this berm outwards may occur during construction. The cheapest way to deal with this is to rebuild any section which fails to elevation 606, after which it will be stable due to the rise in the lake bottom caused by the failure.

Redesign of the Embankment

The principles of the method of redesign are explained in the "Technical Note on Construction of Causeway", which also shows the final sections adopted.

Very briefly these are:-

a) to lower the embankment as much as possible
b) to balance the weight of the bank by berms
c) to use lighter material for the bank where possible.

Two different grade lines 'A' and 'B' were considered. Grade line 'A' required no lowering of the existing rock floor of the cutting, but incorporated a higher bank section than Grade line 'B'. The choice between these was made on the basis of cost. To do this it was necessary to make some assumptions. These were:-

a) rock from the floor of the cutting would cost twice as much as rock from the wall
b) sand would cost half as much as rock from the wall

Grade A

Total quantity of fill in berms and bank		46,000 cu.yds.
Quantity of above which could be sand		12,000 cu.yds.
Quantity of rock		34,000 cu.yds.

Cost	Rock from wall		34,000 cost units
	Sand 12,000/2	=	6,000 cost units
			40,000 cost units

Grade B

Total quantity of rock		25, 000 cu. yds.
Quantity of above which could be sand		nil
Quantity of rock from floor		4, 000 cu. yds.

Cost	Rock from wall		21, 000 cost units
	Rock from floor 4, 000 x 2	=	8, 000 cost units
			29, 000 cost units

Thus Grade 'A' is about 30% more expensive than Grade 'B'. Grade 'B' was therefore adopted. The sections shown in drawing number F-59-106A in "Technical Note on Construction" are based on the use of Grade 'B'.

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January 1960.

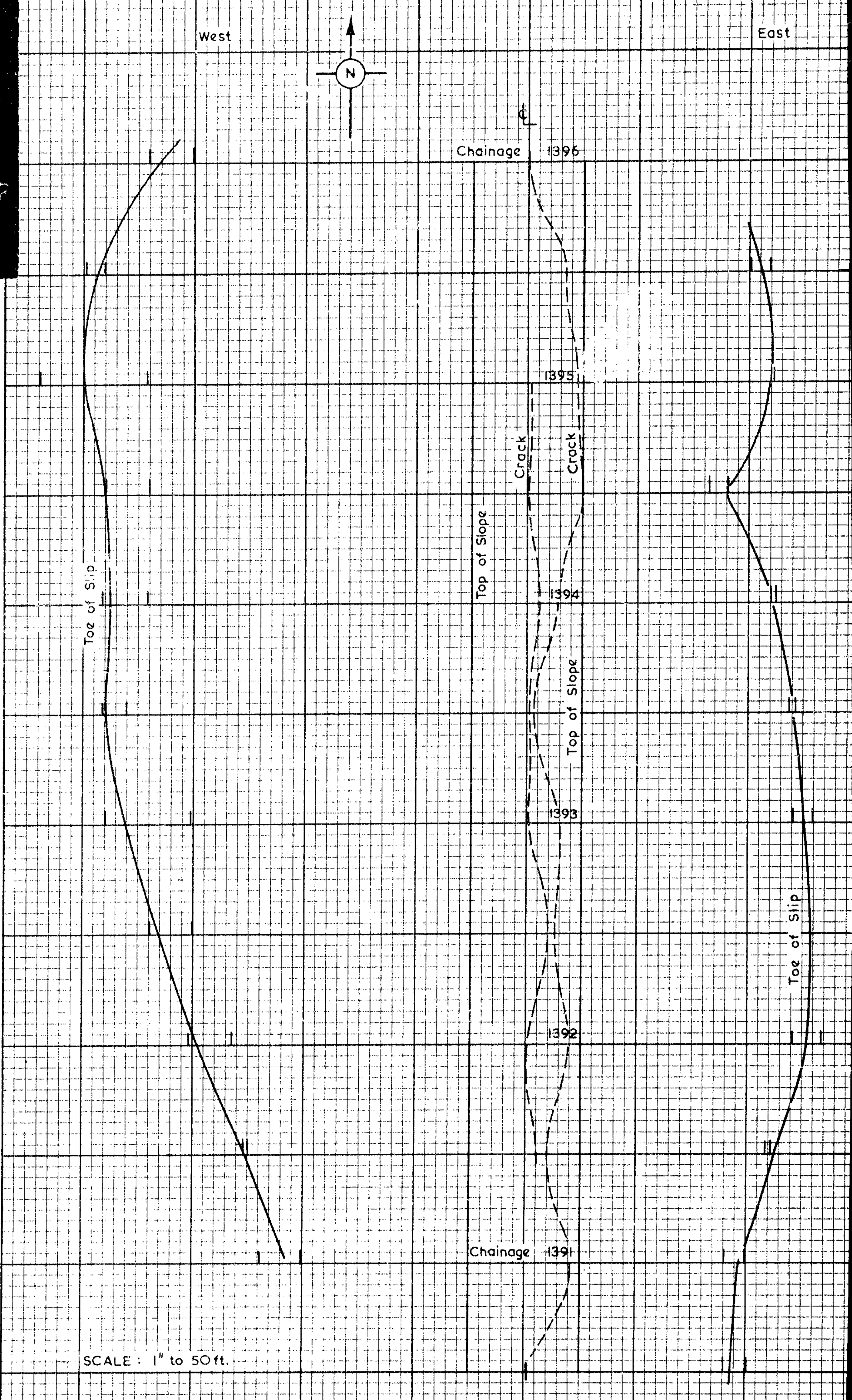

FIG. 1 PLAN OF SLIPPED AREA.

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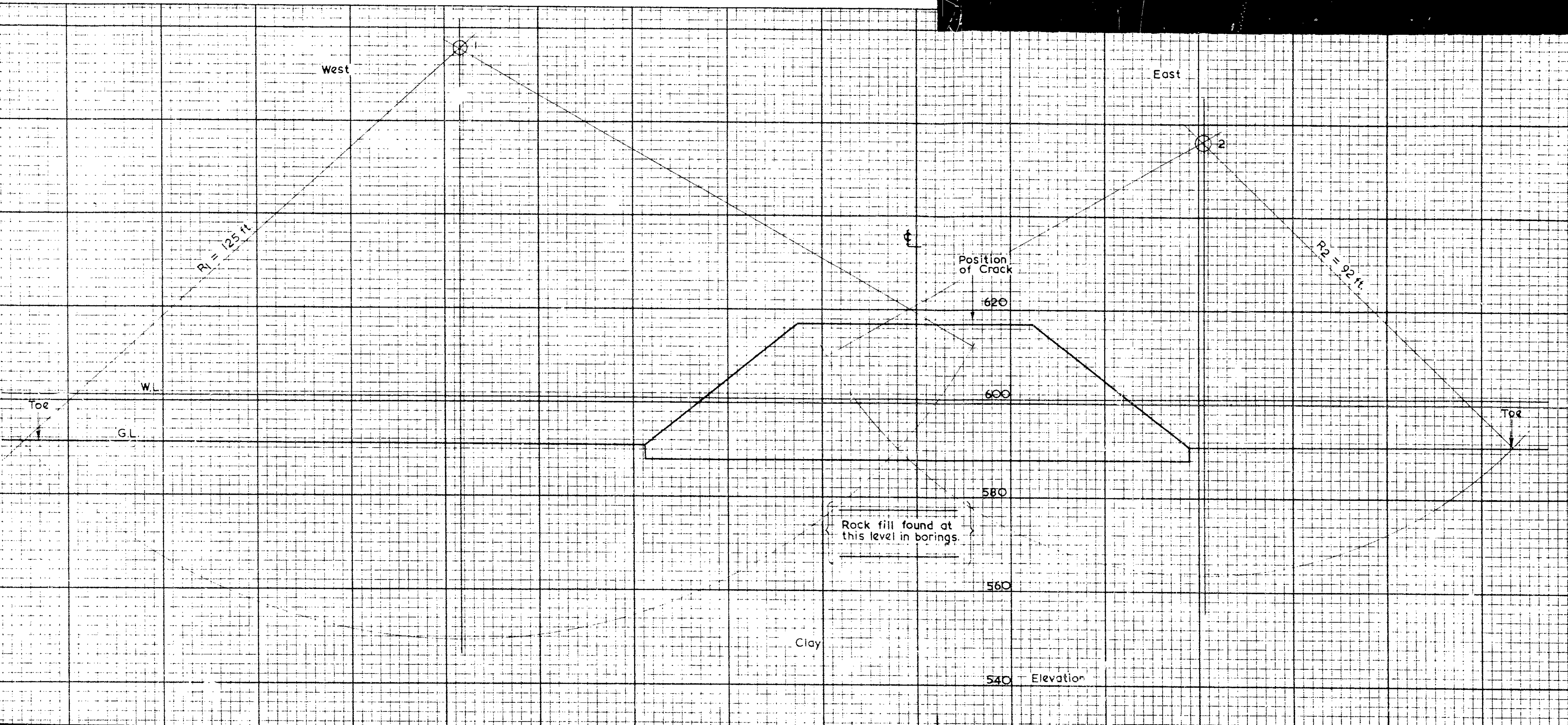

SCALE: 1" to 20ft.

FIG. 2

SECTION THROUGH SLIP