# Soils Investigation for the Rainy Lake Causeway

M. A. J. MATICH and F. C. BROWNRIDGE
Respectively, Vice-President and Chief Engineer, Geocon Ltd., and Special Assignments Engineer, Ontario Department of Highways

The Rainy Lake Causeway is about 2.8 mi long and consists of alternating embankments and bridge structures. At its location, water depths are up to 50 ft and the predominant soil stratum is a generally normally loaded varved clay with a maximum thickness of about 50 ft. The paper describes in detail the soil conditions encountered.

Because of the soft nature of the clay, structure foundations extend to bedrock. Construction of embankments, however, required special treatment for considerations of stability and settlement. It was concluded that total displacement of the clay by the embankments was essential and blasting tests were carried out to determine how this could be effected. In addition, borings were made through an existing railroad fill. The results of the blasting tests, the method of carrying out blasting in practice, and embankment construction are described.

The use of blasting to remold the clay to the extent required to permit full displacement by rock fill, was found to be effective in practice.

•THE Rainy Lake Causeway which forms part of Highway No. 11 in the vicinity of Fort Frances, Ont., has recently been completed. Because the southern end of Rainy Lake lies in Minnesota, the only alternative to an actual all-Canadian crossing of the lake, would have been a 40-mi longer route through rugged terrain around the northern end.

At the causeway location, the lake is crossed by a transmission line and a Canadian National Railways embankment which was built in 1912. The lake is used extensively for logging operations and for pleasure boating for which provision was made in the C. N. R. Causeway. Figure 2 shows a plan of the railway crossing and transmission line, together with the new causeway which is located between them. The new causeway consists of alternating embankments and the bridge structures, and incorporates an elevated bridge section (36-ft clearance) over the channel used for logging and navigation. Part of this section is shown in Figure 1. From east to west (Fig. 2) the embankments are 740, 808, 1,002, 4,892 and 3,402 ft long, respectively, whereas the bridge structures are 2,014, 1,811, 453 and 138 ft in length. The causeway is 44 ft wide providing for two lanes of traffic at a speed of 60 mph and H20-S16 loading. The total length is 2.8 mi and the cost was about $4.5 million.

## GEOLOGY OF SITE

Rainy Lake is located near Fort Frances, Ont., on the Precambrian Shield. The geology of the site is typical of the shield. Igneous bedrock outcrops frequently with smooth, rounded profiles showing evidence of stratification. There is little overburden in land areas. The lake is 30 to 50 ft deep along the line of the causeway, and below the lake bottom there is a stratum of varved clay probably laid down during the last glaciation. A thin layer of glacial till or sand occurs between the varved clay and bedrock in places.

---

Paper sponsored by Department of Soils, Geology and Foundations.

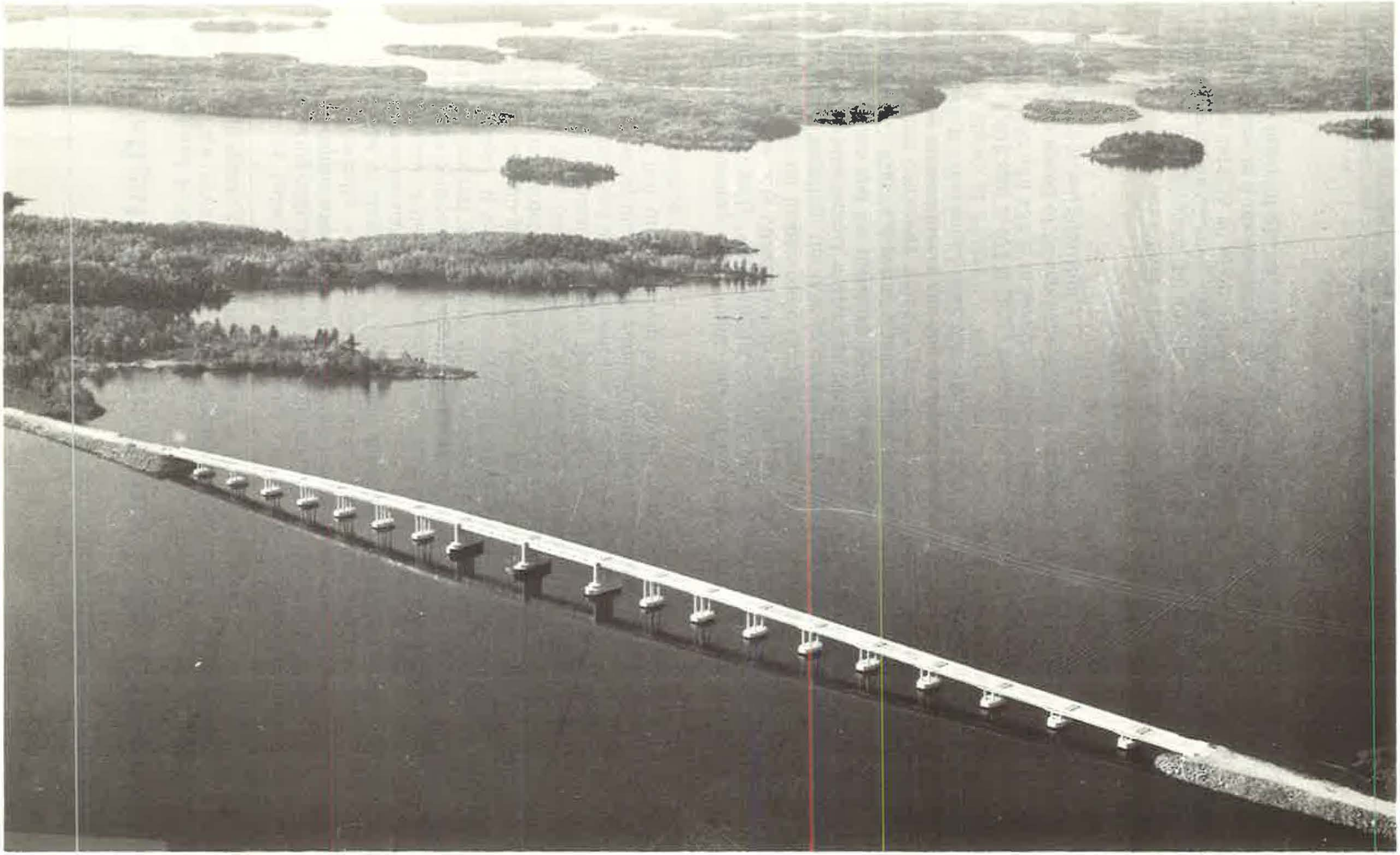

Figure 1. Navigation span.

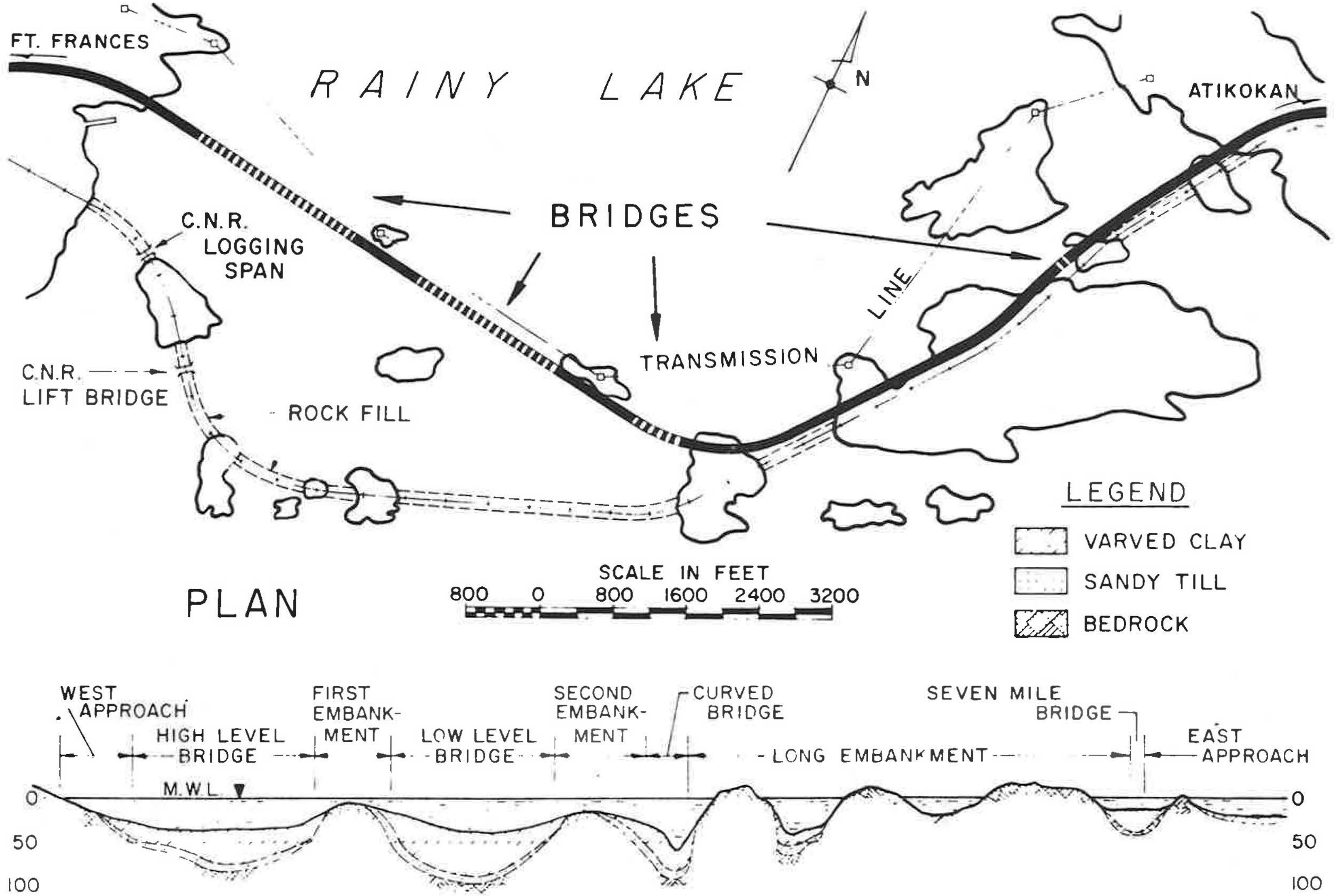


Figure 2. Plan and soil stratigraphy.

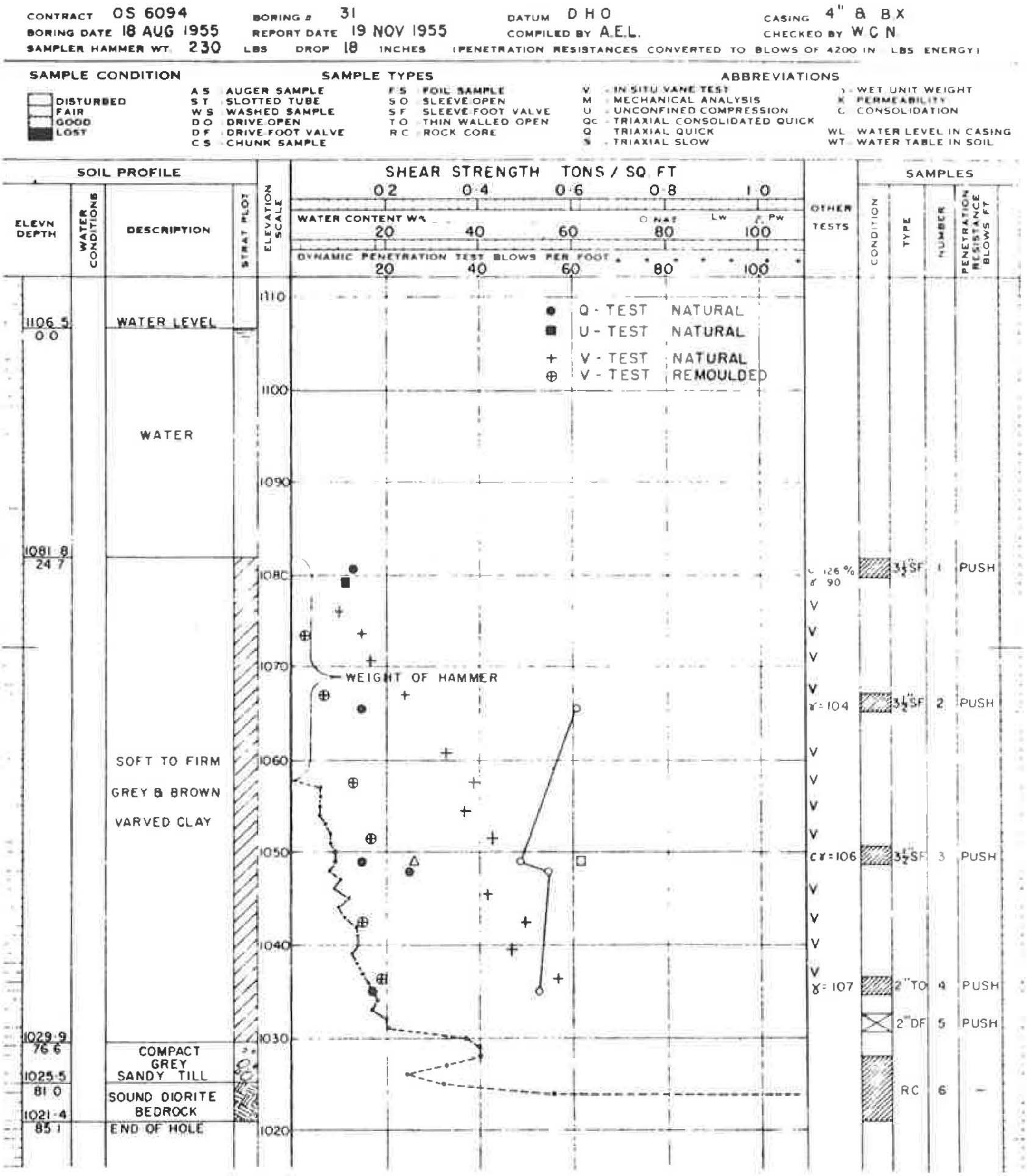


Figure 3. Typical borehole log.

## SOIL CONDITIONS

Investigation of the site began in 1957, and included a comprehensive program of soil sampling and vane testing along several possible alignments. In addition, drilling through an existing railway fill was carried out, and during design, a program of blasting tests to remold the clay was conducted. Topographical surveys, soundings and ice movement measurements were also carried out (1).

Along the selected alignment, the lake bottom is underlain directly by soft to firm varved clay which is up to 50 ft thick as shown on the soil stratigraphy section in Figure 2, but as much as 85 ft thick elsewhere. A typical borehole log is shown in Figure 3.

From the point of view of design and construction, the most significant soil stratum present was the varved clay. Individual laminae in the clay are generally horizontal, although in places they did occur at inclinations of about 10 deg to the horizontal. The laminae are also alternately grey and light grey, although occasionally they are grey and reddish-brown. Their thickness varies from about $^1/_{32}$ in. to 1 in. with an average of about $^1/_4$ in. Typical samples of varved clay (Fig. 4) show the chunky structure of individual clay lamina, particularly in the reddish-brown clay, and the inclination of some of the varves to the horizontal.

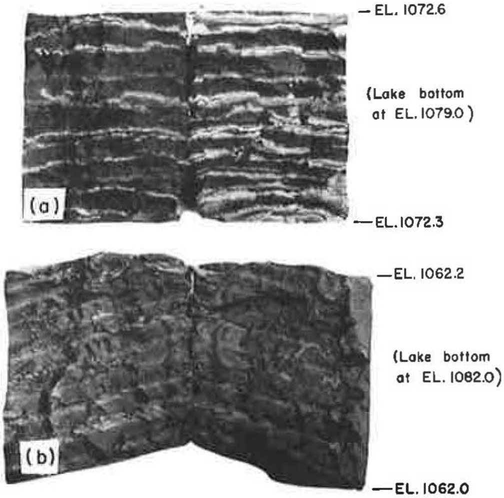


Figure 4. Typical sample of (a) grey varved clay, and (b) reddish-brown varved clay.

The clay generally has a high plasticity. Liquid limits of 50 to 96 were obtained for the grey clay, with corresponding plastic limits of 15 and 31. For the reddish-colored clay the corresponding range in liquid and plastic limits was 88 to 134, and 33 to 49, respectively. Figure 5 is a plot of liquid limit versus plasticity index as obtained for bulk samples of the varved material. The natural moisture content of the stratum was found to vary between 40 and 140 percent with a general value of 60 to 70 percent. It tends to decrease with depth. Pressure-void ratio curves for the clay, of which typical examples are shown in Figure 6, indicate that the clay is generally normally loaded, with some preconsolidation being indicated in the upper part of the stratum only in locations where water depths are less than about 20 ft.

The undrained shear strength of the clay, as measured by vane tests, varied from 100 to about 2,000 psf (lb/sq ft), and there is a trend towards uniform increase with

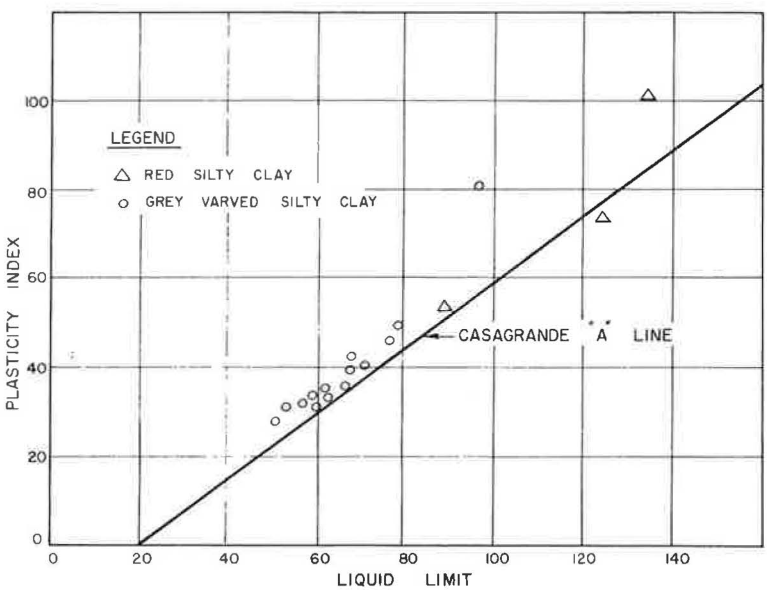


Figure 5. Plasticity chart.

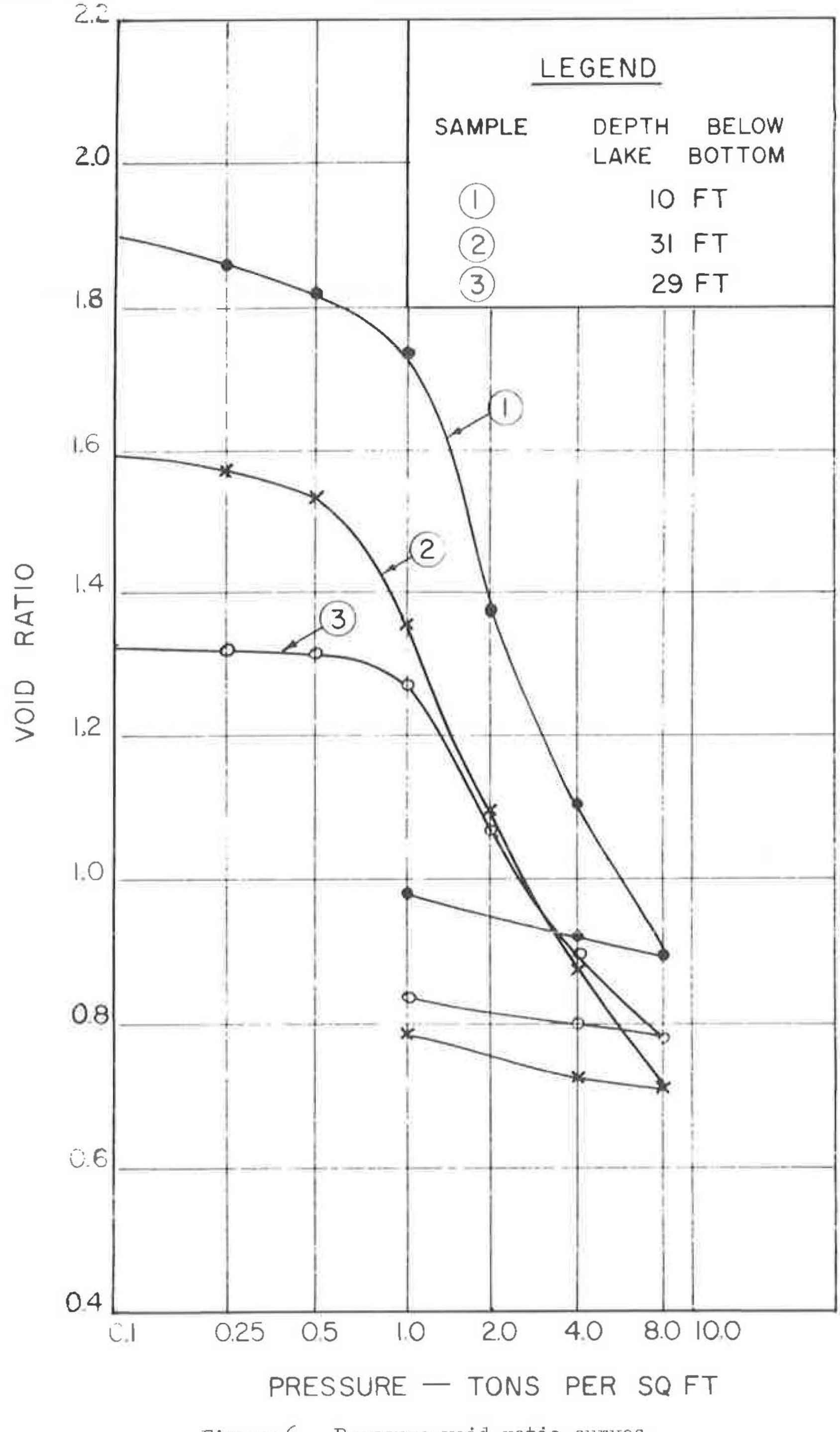


Figure 6. Pressure-void ratio curves.

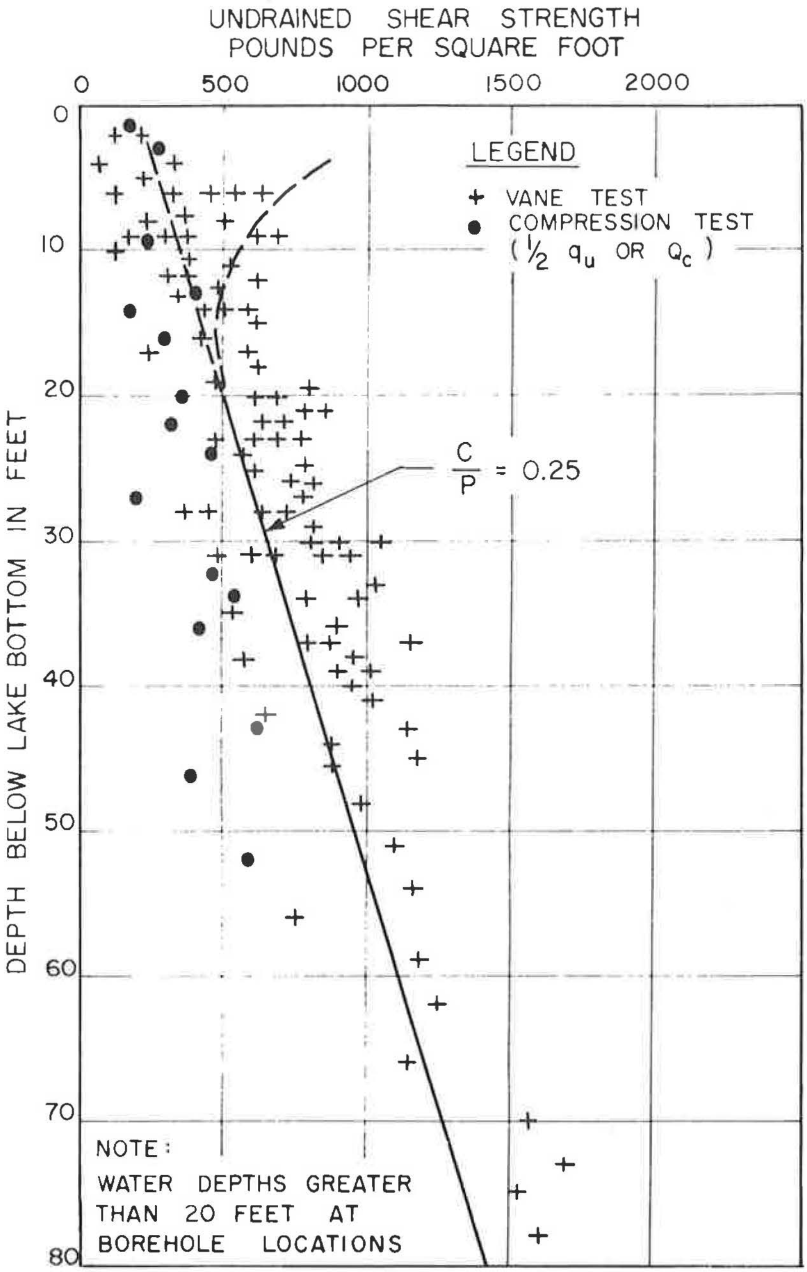


Figure 7. Shear strength plot.

depth below lake bottom, where water depths are greater than about 20 ft. A shear strength versus depth plot is shown in Figure 7. Where water depths are less than 20 ft, the clay strength in the upper 10 ft of the stratum is variable, but the average shear strength is about 400 psf. Undrained triaxial compression tests gave results which are plotted as shear strength in Figure 7. The sensitivity of the clay was generally in the range of 3 to 6.

The varved clay is underlain either directly by bedrock or by a layer of compact sandy till with boulders. This layer is generally thin, although in one instance it was about 32 ft in thickness. Standard penetration resistances of 1 to 44 blows per foot were obtained in the till.

## EMBANKMENT STUDIES

The causeway is 44 ft wide at road level with a free board of 10 ft above high water level. As already mentioned, water depths along the selected alignment were up to 50 ft. For the studies, an end-dumping method of construction was considered which would result in overall side slopes of 1.5 horizontal to 1 vertical.

Because of the sensitive and normally loaded nature of the clay with a consequent low shear strength at the surface, it was evident that it could not generally safely support fills exceeding about 10 ft in height. For end-dumped fills, therefore, of the height required, sinkage into the clay during construction was expected as a general condition. Initial studies were directed towards checking on the amount of sinkage that would occur during normal end-dumping procedures. Preliminary estimates were made, based on the assumption that sinkage would occur until the ultimate bearing capacity of the clay at a given elevation was equal to the applied load of the fill. Methods described by Sinacori, et al. (2), together with circular arc and sliding wedge analyses, were used. In addition, several available examples of sinkage of fills with varved clay were examined, and it was decided to drill through the existing railway embankment close to the causeway alignment.

The railway fill was constructed by end-dumping quarried rock from railway cars. During construction of the western end of the fill, where the clay was up to 60 ft thick, a considerable amount of progressive slippage occurred at the dumping face necessitating special precautions to support the rail track at the face. Since completion of the embankment, subsidences (apparently caused by both consolidation and progressive displacement of underlying clay) have continued and at intervals, most recently in 1949, it was necessary to raise the tracks to grade by adding more ballast. In places, the total settlement since 1912 amounts to 10 ft. Boreholes indicated that the rock fill had displaced the clay to bedrock over part of the length of the embankment, although complete displacement of the clay was not a general condition. In some cases the rock fill was "floating" in the clay (Fig. 8). In some boreholes, individual boulders of rock fill were separated by up to 1 ft of soft clay.

The sinkages of the embankment as computed by the aforementioned methods were generally less than those indicated by the borings through the railway fill.

Based on the estimated sinkages, settlement computations were carried out to check on the amount of consolidation which would take place in the undisplaced clay under the new causeway embankment. These showed that total settlements of the roadway at the centerline of up to about 5 ft could be expected. The analyses also showed that only about one-half of the anticipated settlement would be completed in the first 5 years after construction. These results, combined with the settlement data on the railway fill, showed that with partial displacement only, the performance of the highway embankments would not be satisfactory. Therefore, it was considered essential to displace completely the clay from beneath the new rock fill embankment, particularly in the areas of the structural abutments. Where the clay was less than 8 ft thick, it was estimated that adequate displacement could, in general, be achieved if a continuous rate of end-dumping was maintained. To effect positive displacement where clay depths were greater than 8 ft, however, it was considered essential to lower the shear strength of the clay to about 100 psf, or less, in advance of fill placement.

The most practical method of achieving this appeared to be the use of blasting with dynamite to remold the clay. One example studied, where such a method was used, was a rock fill causeway in northern Quebec. Here, with the use of end-dumping procedures and blasting of the clay, displacement to hard bottom was successfully effected and the resulting cross-section of the fill below clay level was remarkably narrow. The soil conditions and fill cross-section as determined by borings are shown in Figure 9. There are also references in the literature to the successful use of dynamite to effect fill displacement of soft ground. For example, Sinacori, Hofmann and Emery (2), describe some applications in New York where depths of unsuitable material

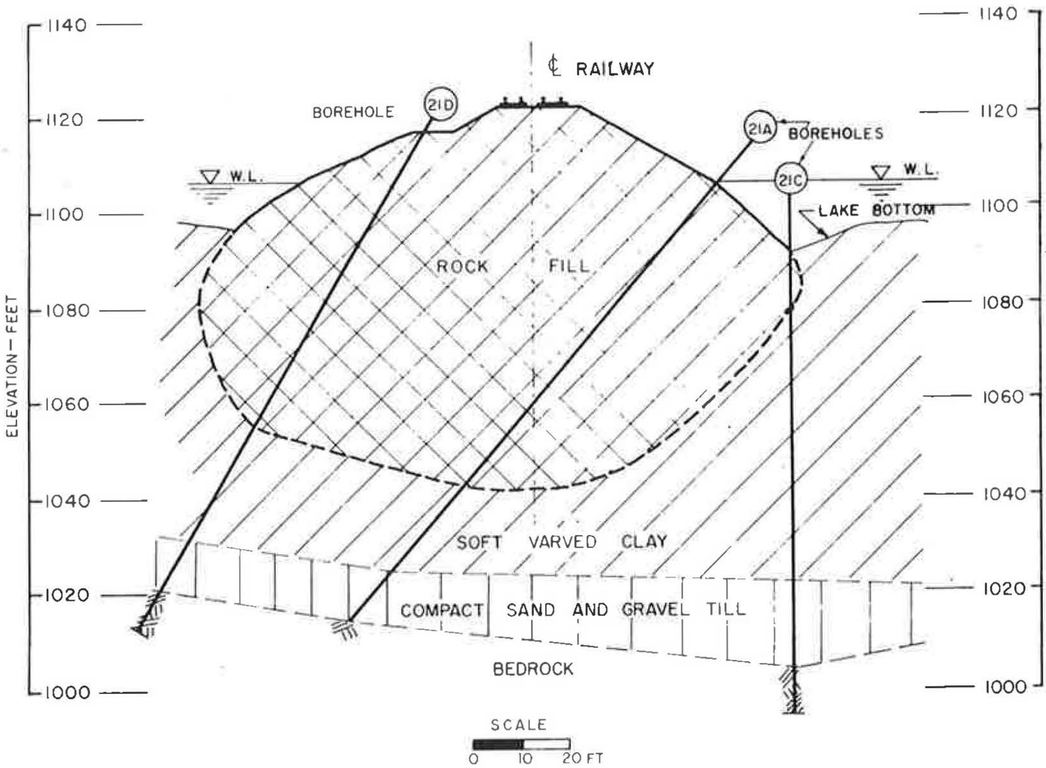


Figure 8. Cross-section existing railway fill.

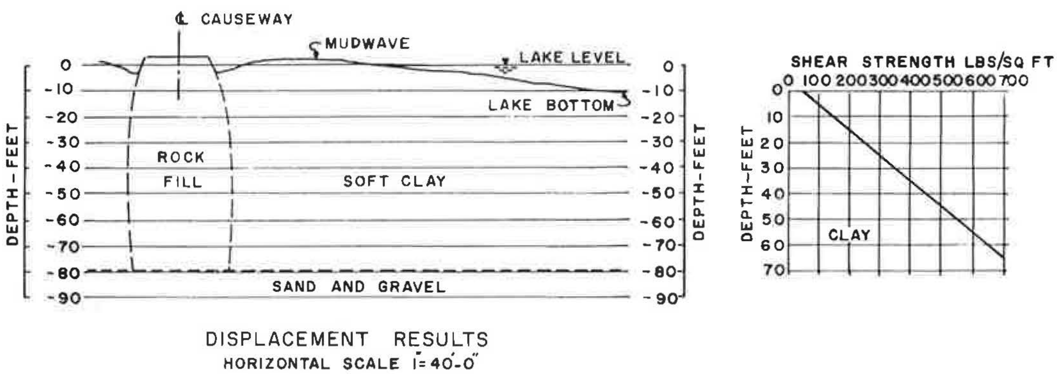


Figure 9. Cross-section Quemont Mines Causeway.

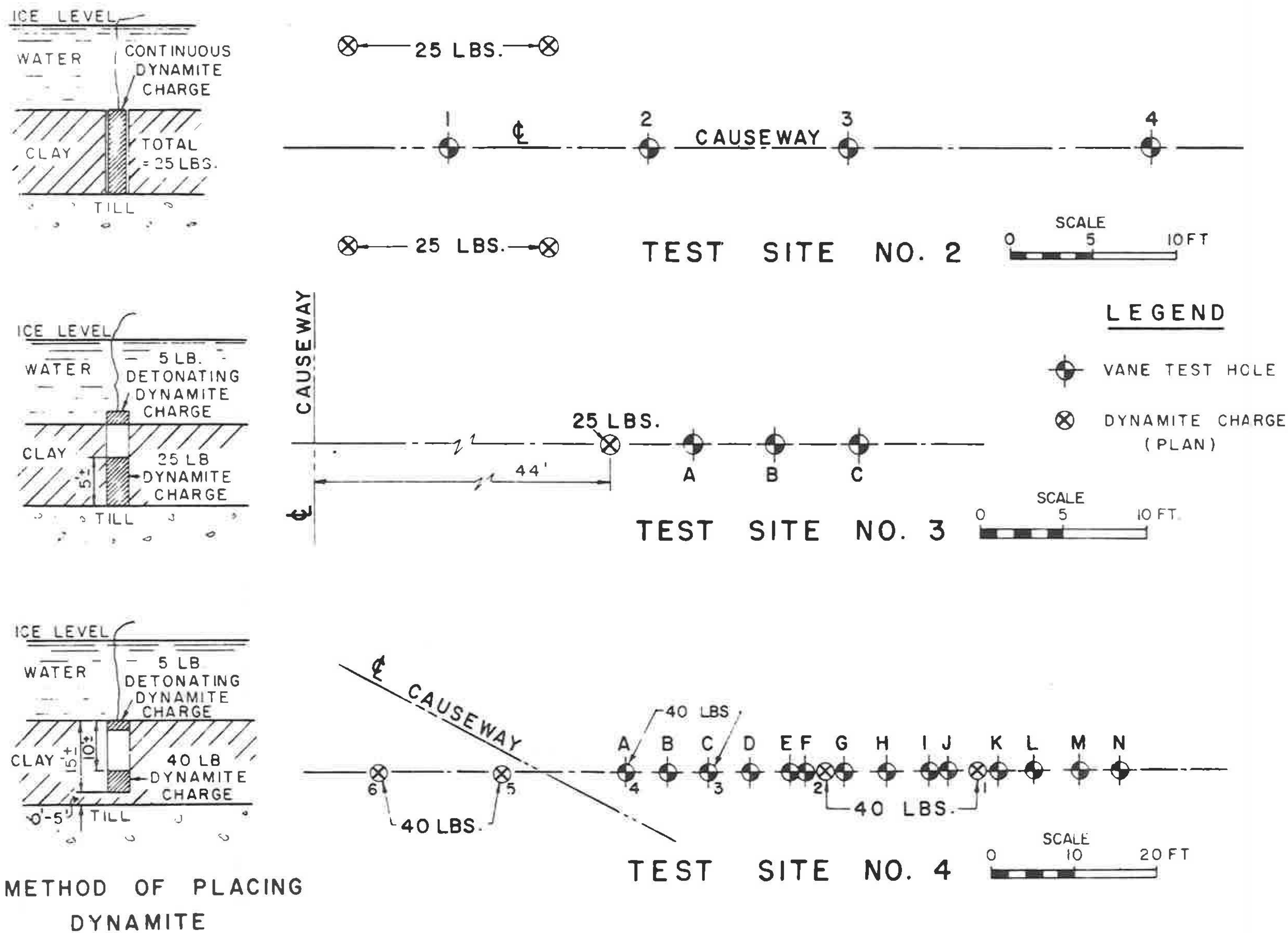


Figure 10. Layout of dynamite charges and vane tests.

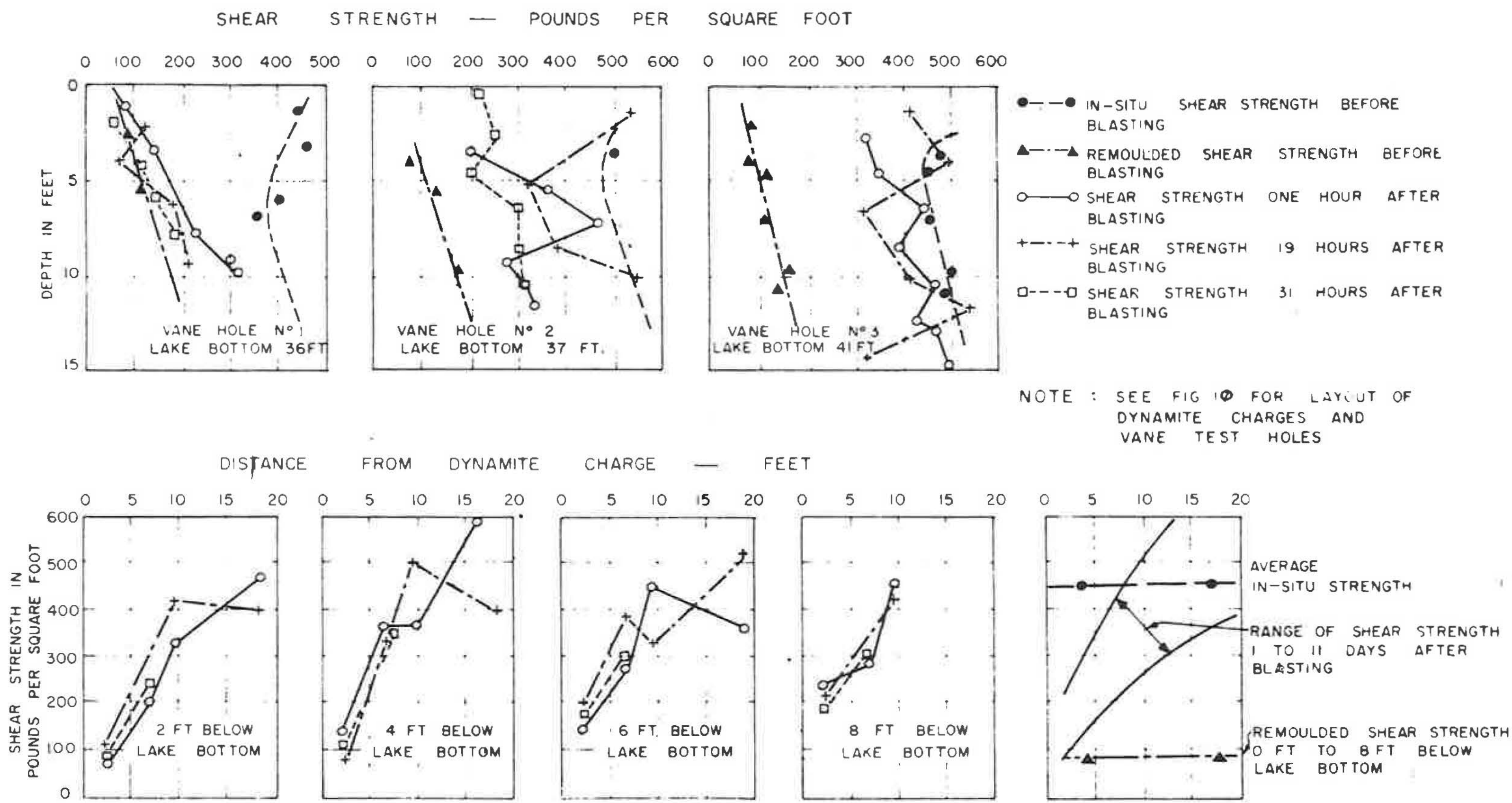


Figure 11. Blasting test results—Test Site 2.

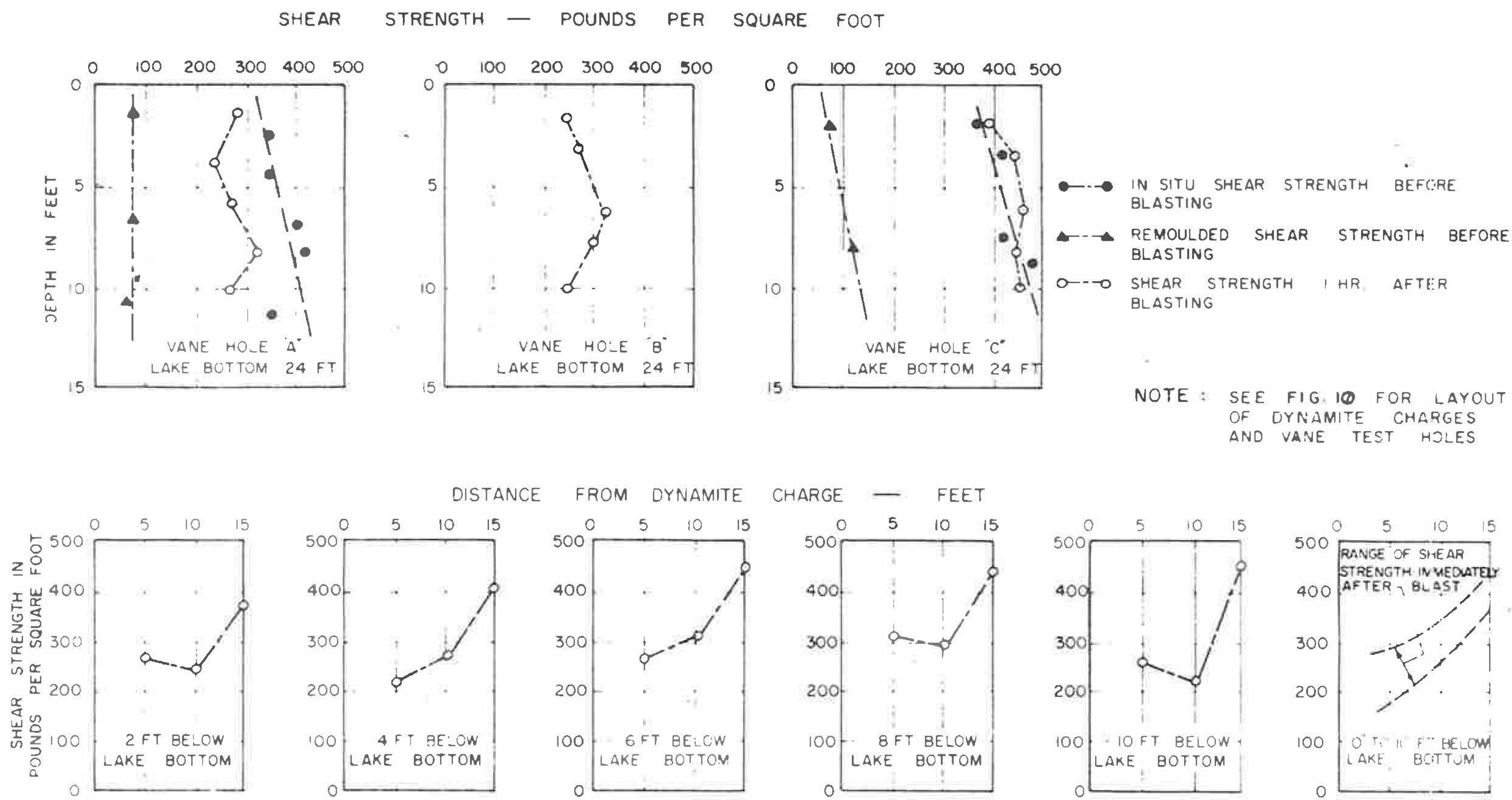


Figure 12. Blasting test results—Test Site 3.

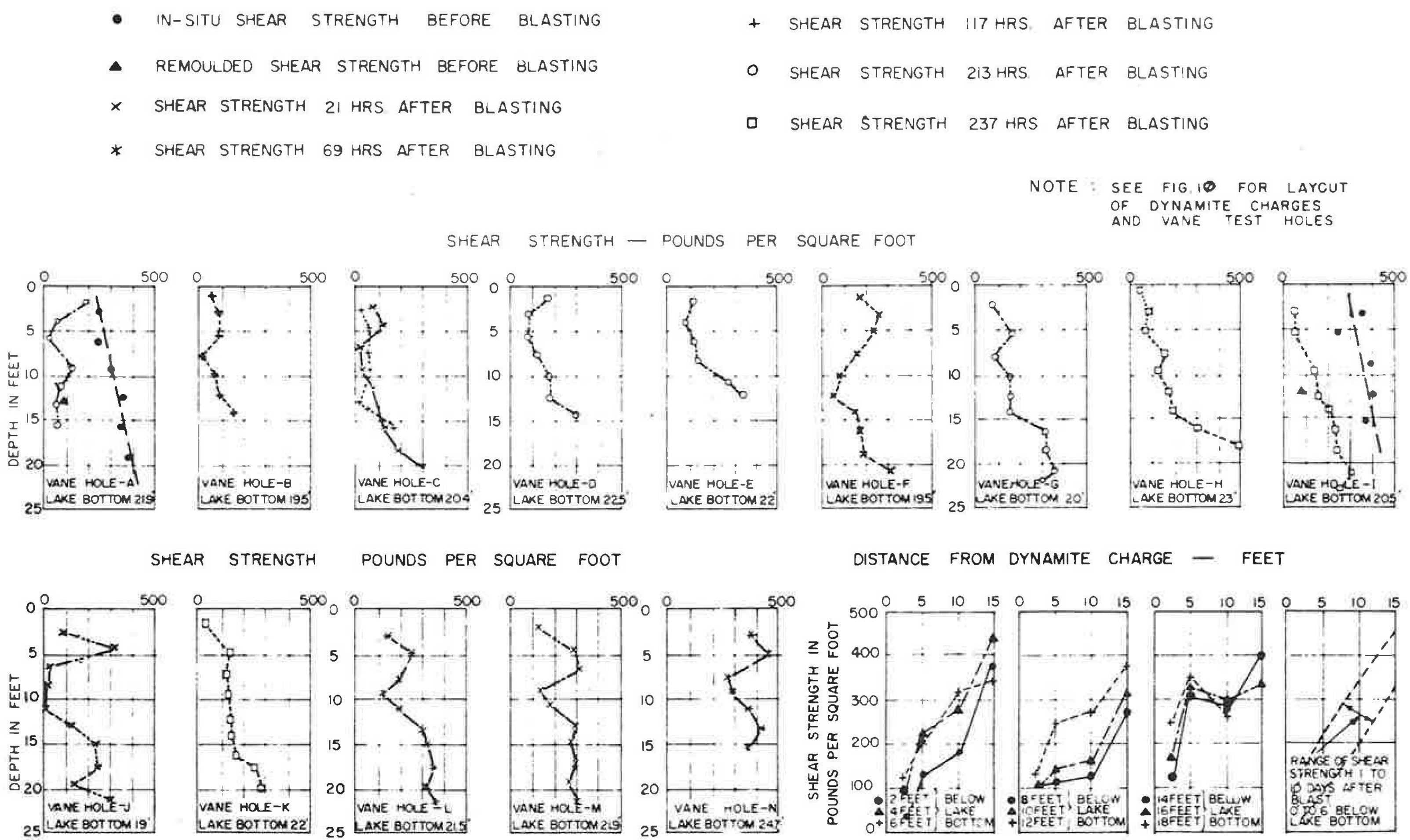


Figure 13. Blasting test results—Test Site 4.

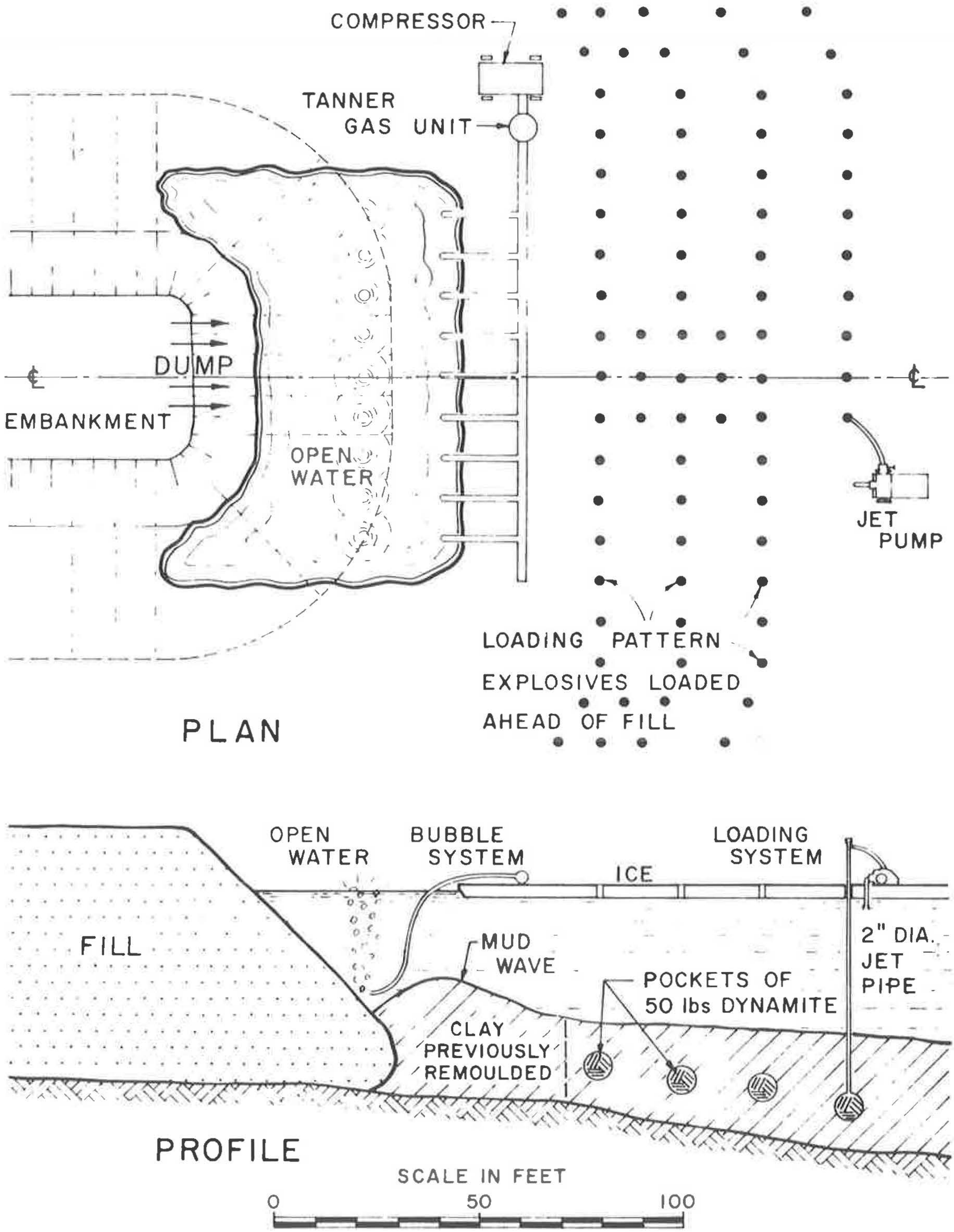


Figure 14. Embankment construction.

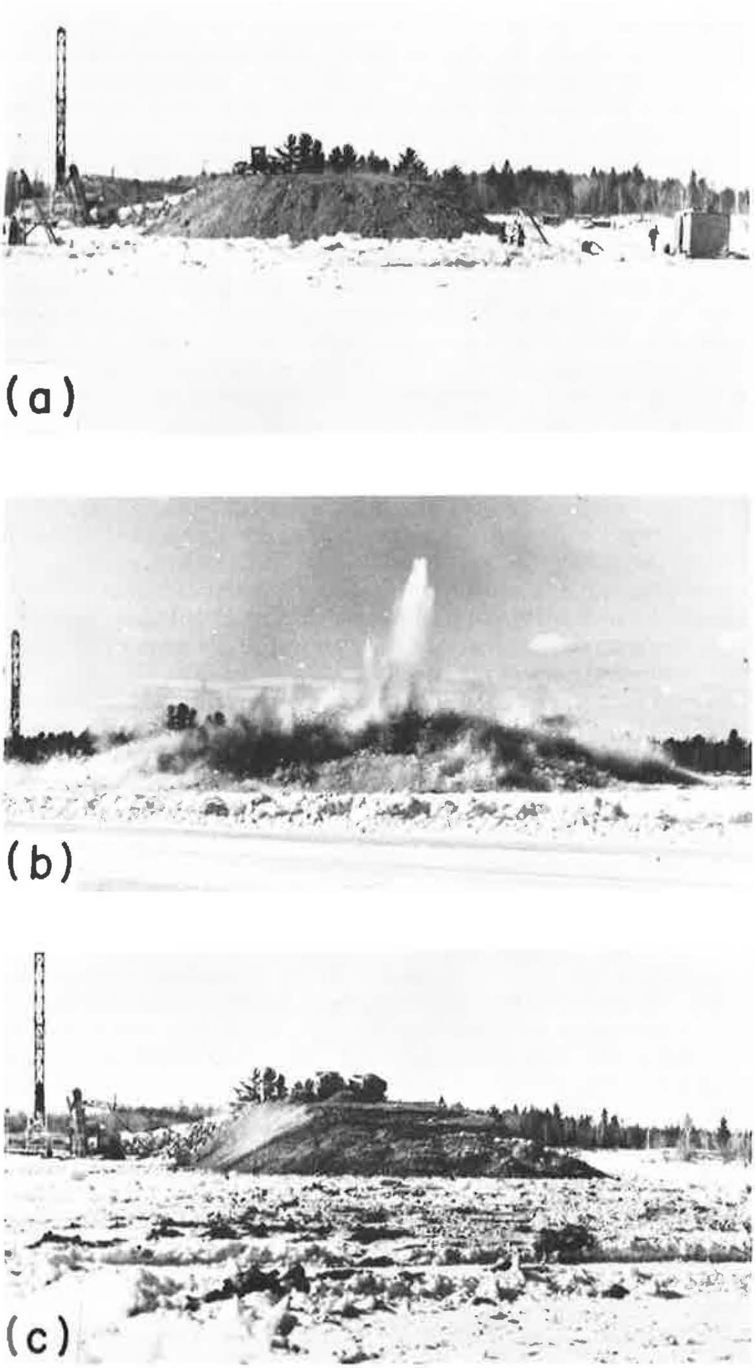


Figure 15. Dynamite blasting ahead of fill: (a) before blast, (b) during blast, and (c) after blast.

displaced by this method have varied from 5 to 24 ft, using ¾ to 1¼ lb of dynamite per cubic yard of material displaced. This reference also states that the subsequent performance of the embankments was satisfactory.

Because there was not a great deal of available precedent applying to remolding of varved clay by blasting, it was decided to carry out blasting tests to determine if the required degree of remolding could be achieved, and to establish the quantity of dynamite needed per cubic yard of clay to be remolded. The tests were carried out from the ice at locations where the in-situ undrained shear strength of the clay varied from 400 to 500 psf. For the blasting, 50 percent Forcite ditching dynamite was used. The strength of the clay, both natural and remolded, was measured using a penetration type vane.

## Blasting Tests and Applications

Test blasting was carried out at five locations. The relative positioning of the dynamite charges and the vane test holes for three representative locations are shown in Figure 10. Dynamite charges with a total weight of 25 lb each and continuous for the full depth of the clay were used at Test Site 2, where the clay stratum was 9 to 15 ft thick. At Test Site 3, single concentrated 25-lb charges of dynamite were placed in the lower 5 ft of the clay stratum, which was about 10 ft thick. At Test Site 4, where the clay was 20 ft thick, single 40-lb dynamite charges were placed about 5 ft from the bottom of the clay stratum.

The results of the tests, expressed as shear strength profiles, are shown in Figures 11, 12 and 13. The results show that where continuous dynamite charges were placed for the full depth of the clay stratum (Test 2), the maximum remolding occurred at the surface of the clay stratum and minimum remolding occurred at the base. A gradual increase in remolded shear strength with distance was also found, as shown. Where dynamite charges were concentrated in a column in the lower part of the clay stratum, the extent of the remolding was more pronounced.

The rate of regain of shear strength of the remolded clay was checked by vane tests taken up to 10 days following blasting. The results are shown in Figures 11, 12 and 13. The measurements indicated that there was generally no appreciable regain in shear strength of the clay for at least several days after remolding by blasting. The test results also showed that maximum remolding was effected by the combined effect of a group of concentrated charges placed near the base of the clay stratum. On the basis of the results obtained, it was concluded that a powder factor of 1 lb of dynamite for each cubic yard of clay was necessary to effect a lowering of the shear strength below 100 psf.

The testing also showed that 50-lb pockets of ditching dynamite would explode by propagation if spaced as much as 18 ft apart. A 10-ft spacing between 50-lb charges was recommended. In practice, the dynamite was loaded within the clay using a jetting procedure which is described by Matich et al. (1), and it was recommended that each blast contain at least 3,000 lb of dynamite. Provision was also made for toe shooting along the sides of the embankments if this were found to be necessary. The details of dynamite loading and blasting, and of embankment construction are described by Matich et al. (1). However, embankment construction is illustrated diagrammatically in Figure 14, and photographs showing the sequence of blasting ahead of the fill are shown in Figure 15.

The cost of remolding of the clay in advance of fill placement amounted to about \$0.50/cu yd of clay remolded. Such treatment enabled the effective use of rock fill approach embankments in this case, between embankment and bridge structure at the transition points.

At the transition points between the embankments and bridge sections, it was not possible in practice to carry out clay remolding ahead of the embankment because this would destroy the lateral support that the clay gives to the bridge bent adjacent to the abutment. A gravel stabilizing berm was therefore used at these locations.

## ACKNOWLEDGMENTS

The authors are indebted to H. W. Adcock, Assistant Deputy Minister, Engineering, Ontario Department of Highways, for permission to publish the data used in this paper. The participation of W. A. Clarke and A. M. Toye, Ontario Department of Highways, at various stages throughout the soil investigation and other studies is gratefully acknowledged.

The authors are also indebted to B. Chappell, Canadian National Railways, for providing information from local experience and construction records, and to Quemont Mining Corporation for the data on their Causeway.

The data given in the paper were also made available through the courtesy of designers of the overall project, Foundation of Canada Engineering Corporation, W. E. Hickey, Chairman of the Board. Special acknowledgment is made to Per F. Anderson, Project Engineer, for his assistance.

A special acknowledgment is made to W. G. Noden, M. P. P. Rainy River for his interest and cooperation.

## REFERENCES

1. Matich, Rutka, and Anderson, "Foundation Aspects of the Rainy Lake Causeway." Engineering Inst. of Canada, Engineering Jour. (Nov. 1963).
2. Sinacori, Hofmann, and Emery, "Treatment of Soft Foundation for Highway Embankments." Proc. HRB, 31 (1952).
3. Terzaghi, K., and Peck, R. B., "Soil Mechanics in Engineering Practice." Wiley (1948).
4. Casagrande, A., "Soil Mechanics in the Design and Construction of Logan Airport." Jour. Boston Soc. of Civil Engineers (April 1949).