HIGHWAY RESEARCH RECORD

Number 57

Construction on Marsh Deposits

1 Report

Tunnel Site Geologic Research

1 Report

Ground Water Control

1 Report

Presplit Blasting

1 Report

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Contents

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.

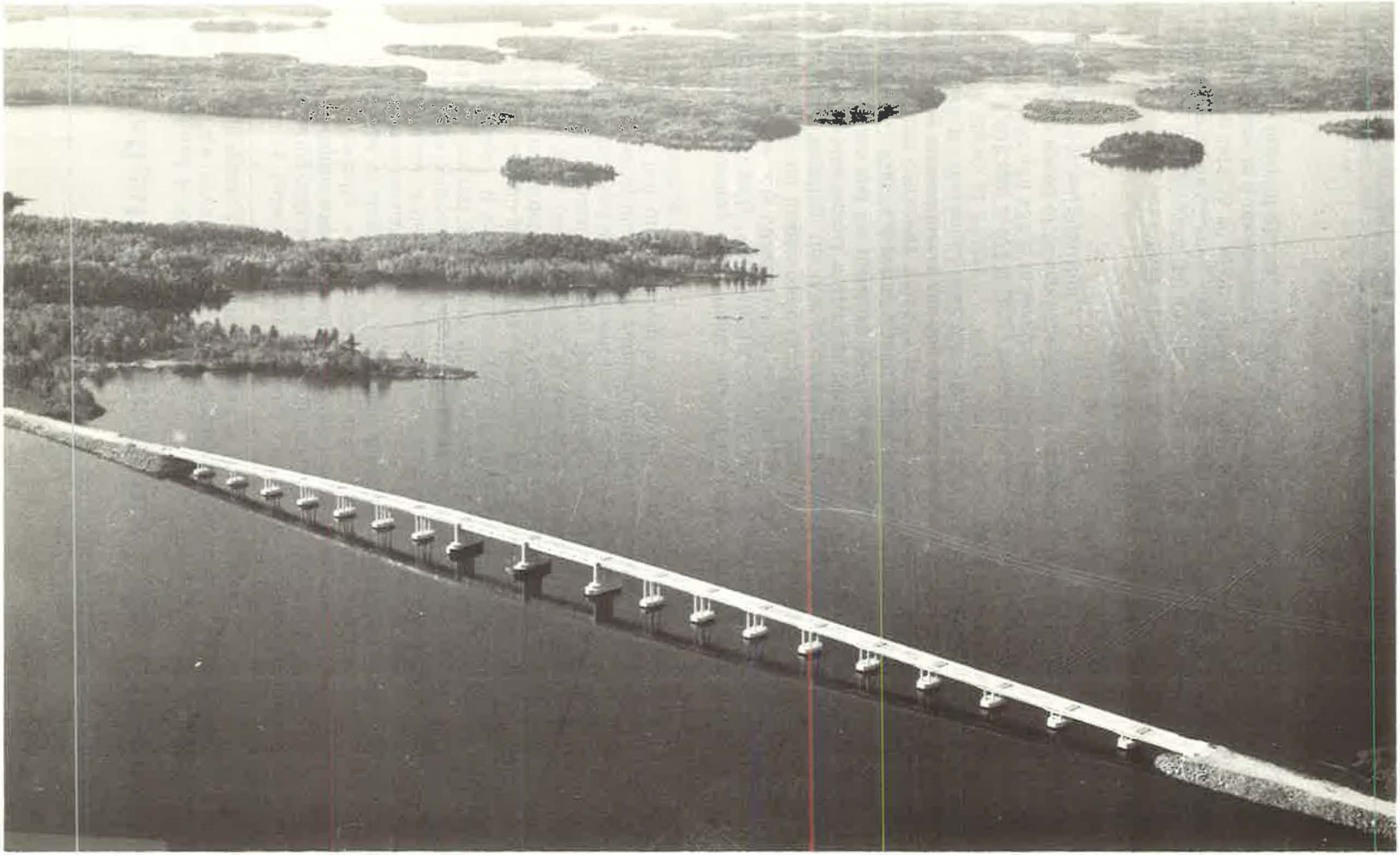

Figure 1. Navigation span.

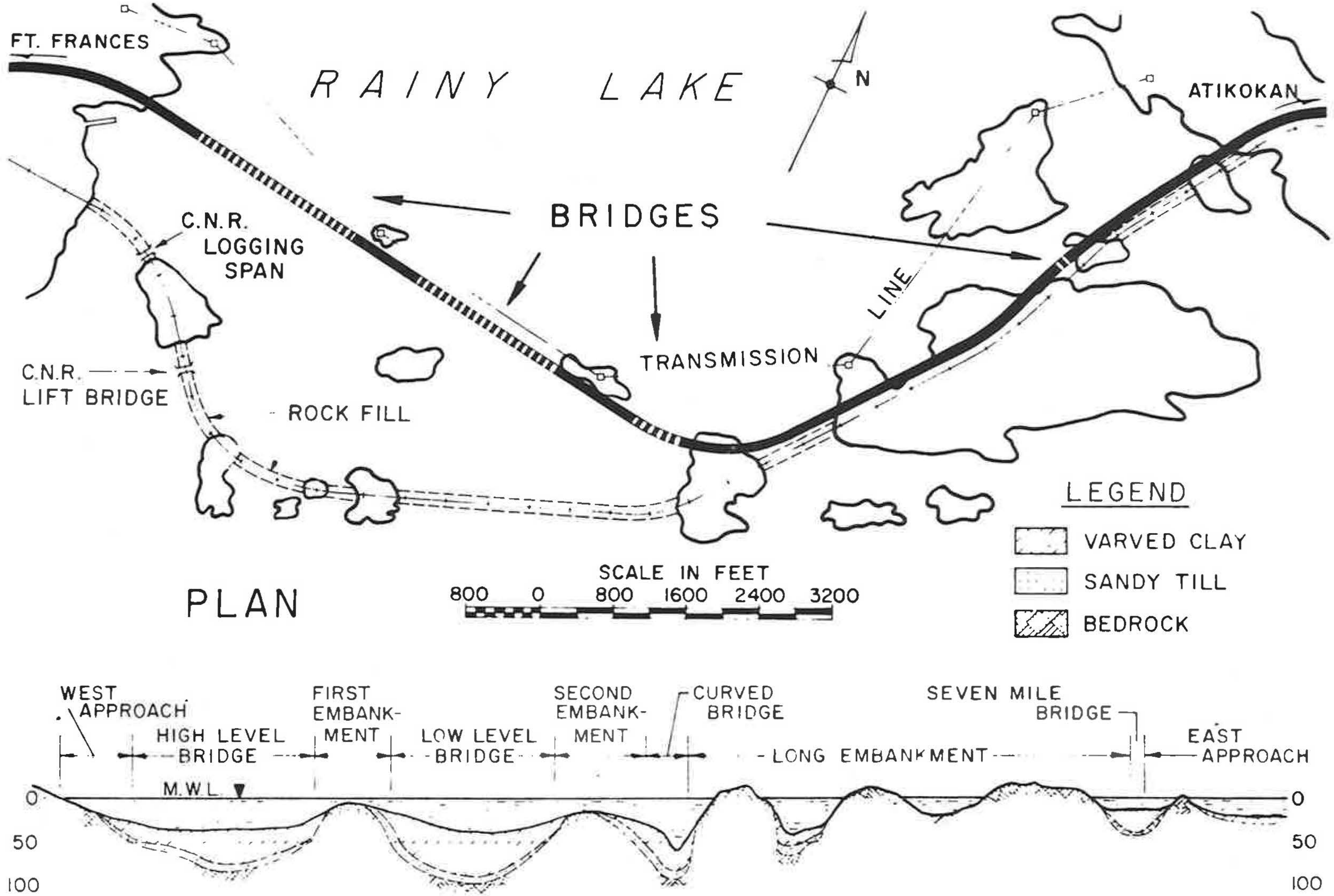

Figure 2. Plan and soil stratigraphy.

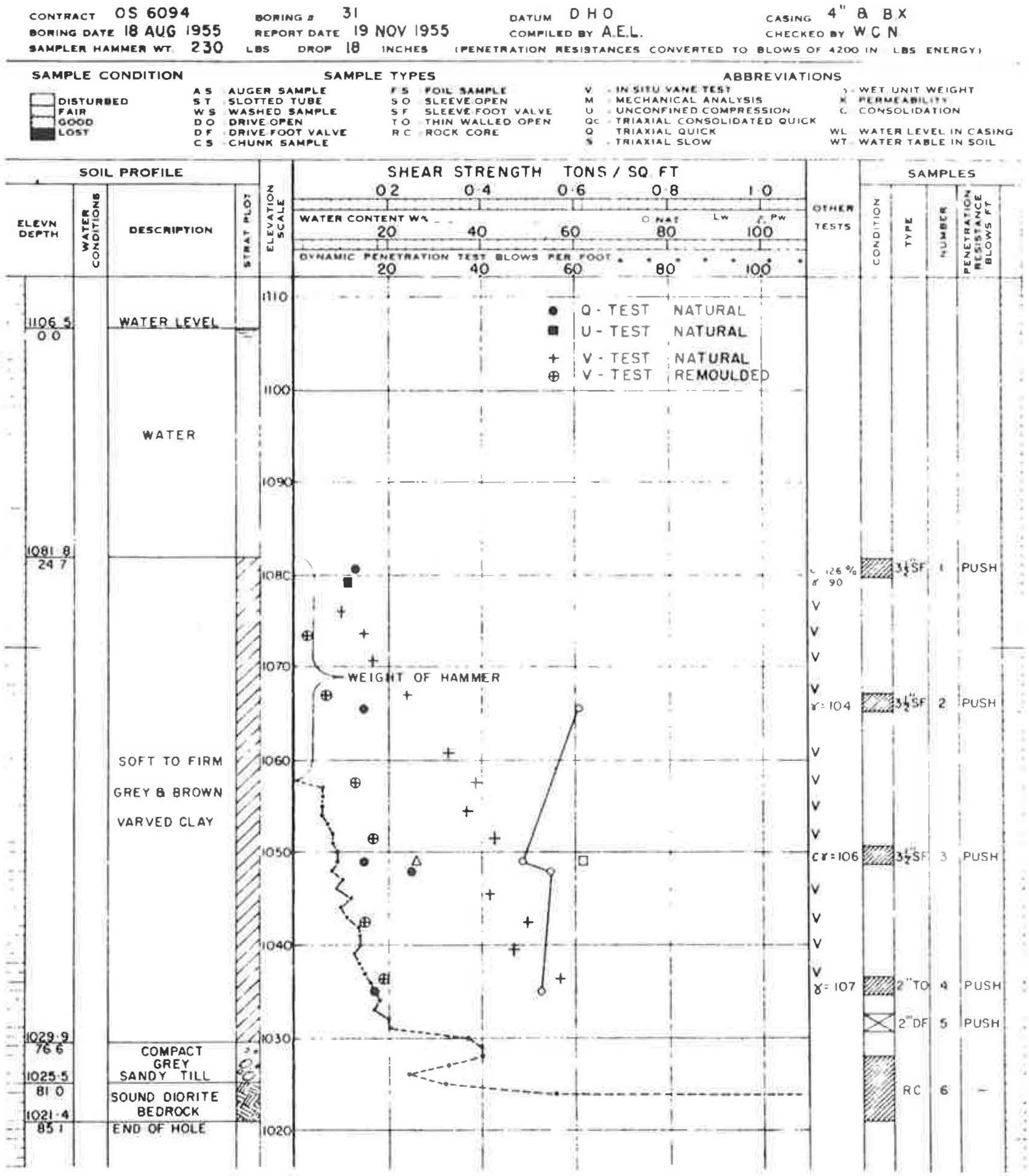

Figure 3. Typical borehole log.

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 $\frac{1}{32}$ in. to 1 in. with an average of about $\frac{1}{4}$ 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.

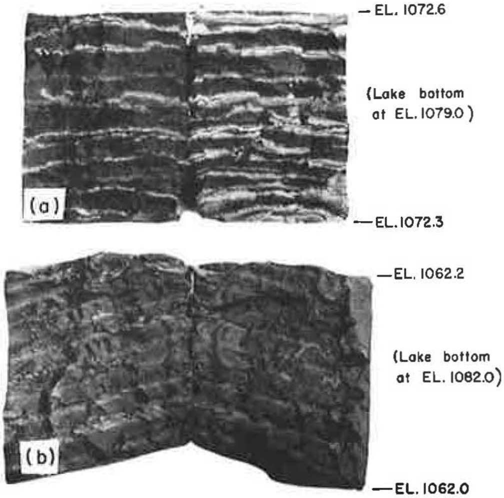

Figure 4. Typical sample of (a) grey varved clay, and (b) reddish-brown varved clay.

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

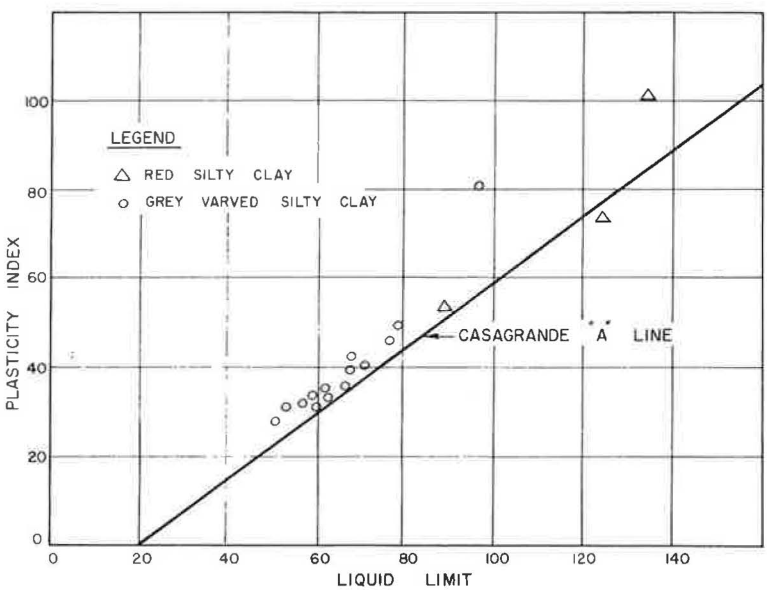

Figure 5. Plasticity chart.

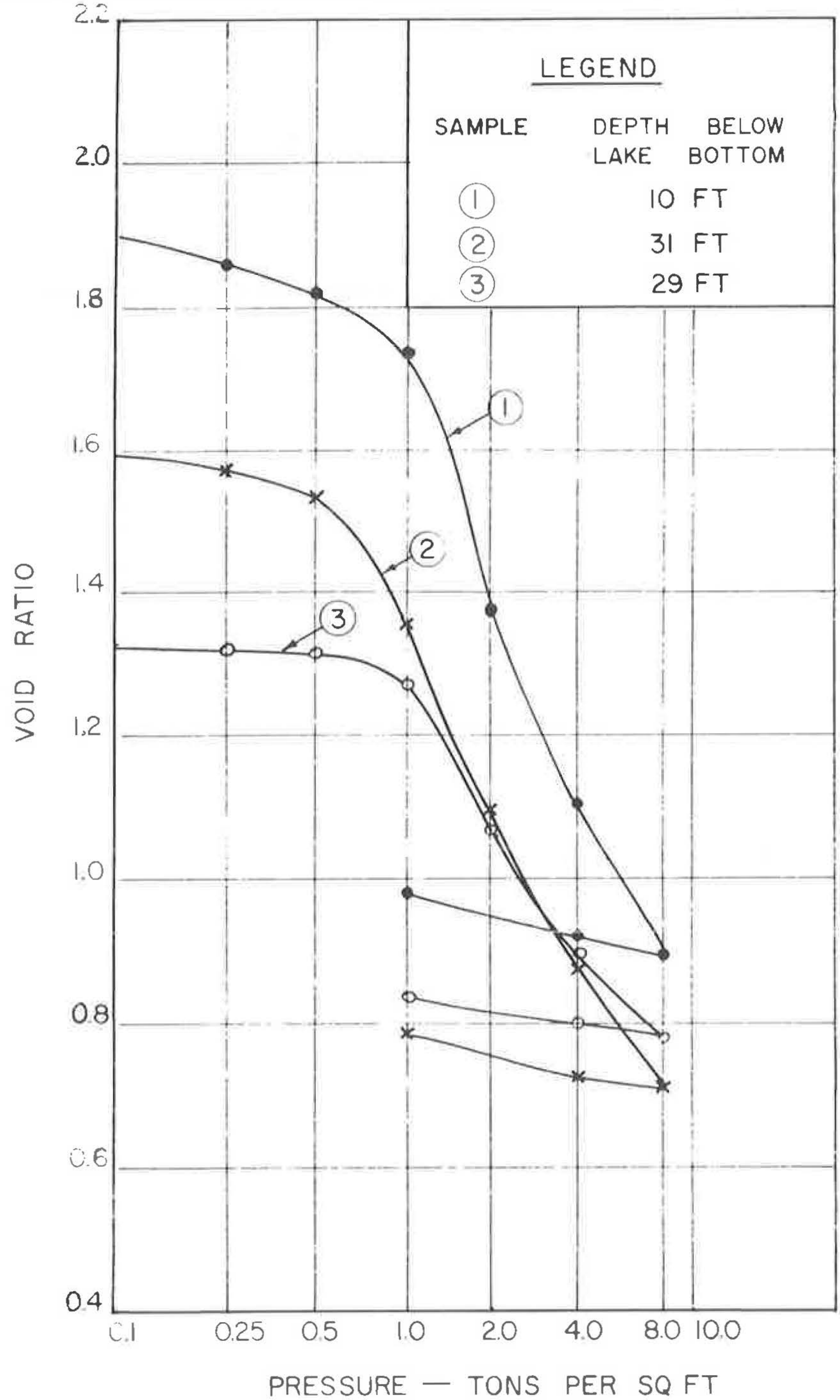

Figure 6. Pressure-void ratio curves.

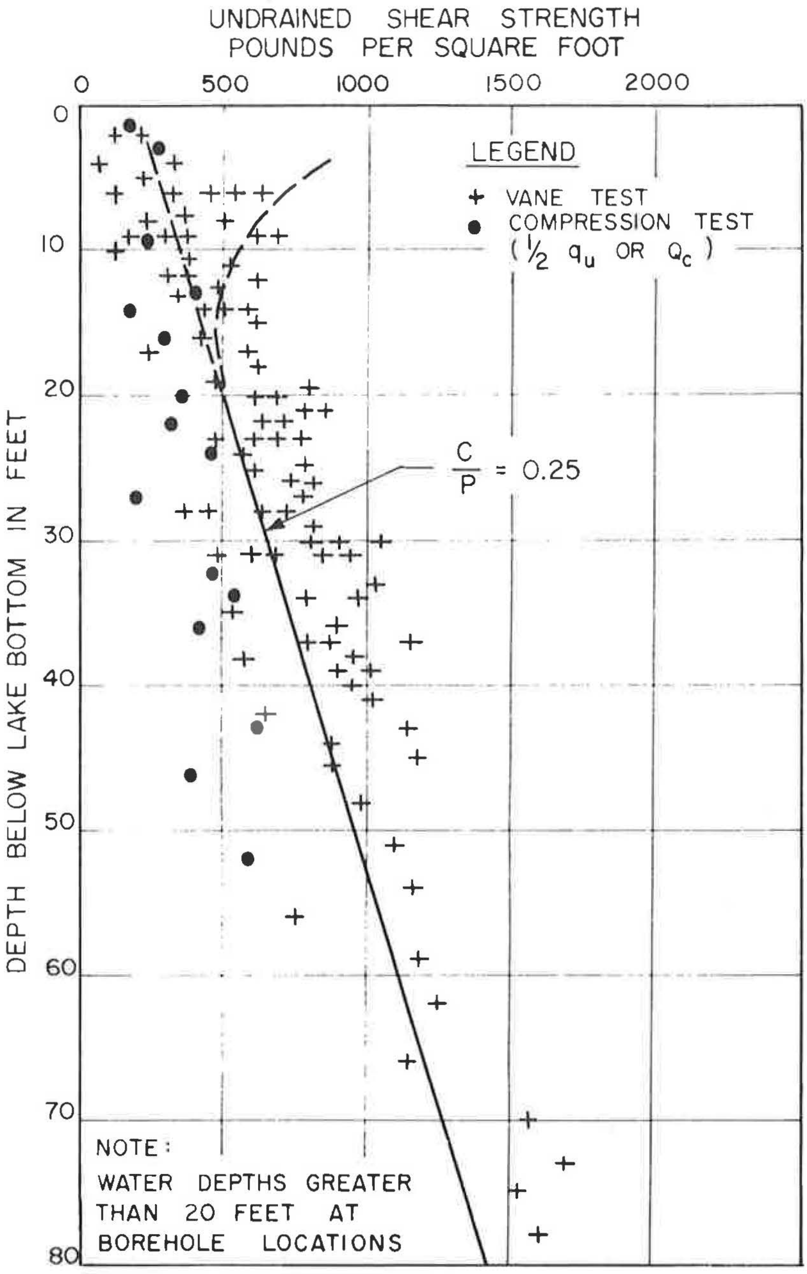

Figure 7. Shear strength plot.

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

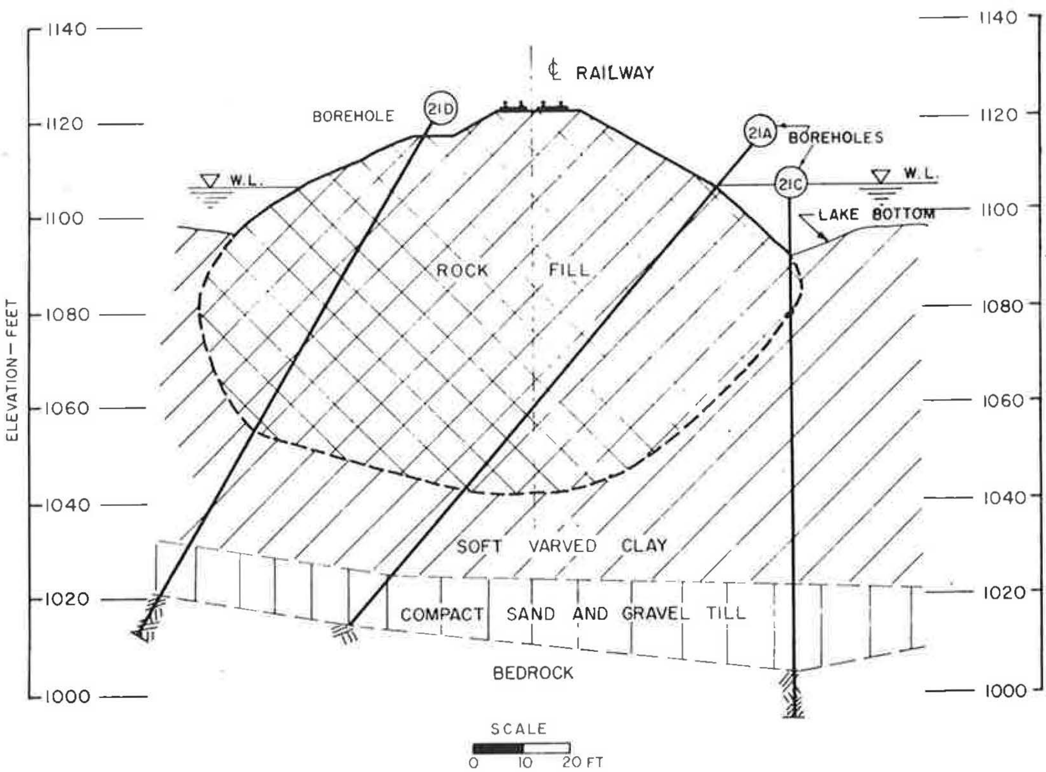

Figure 8. Cross-section existing railway fill.

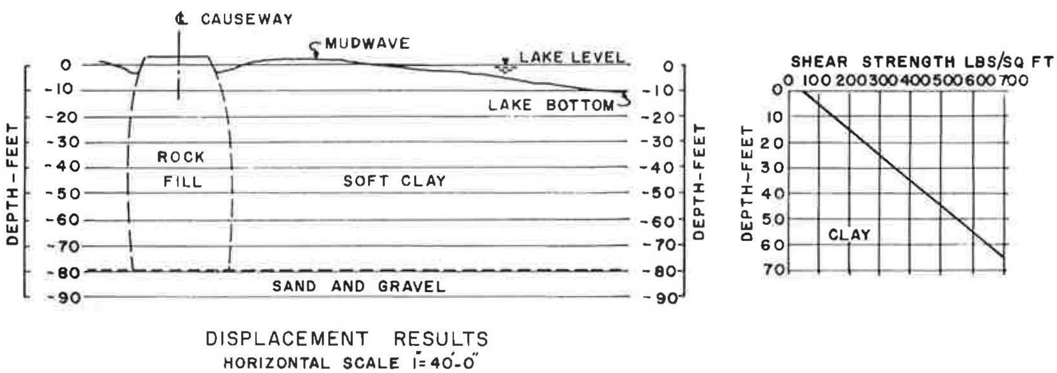

Figure 9. Cross-section Quemont Mines Causeway.

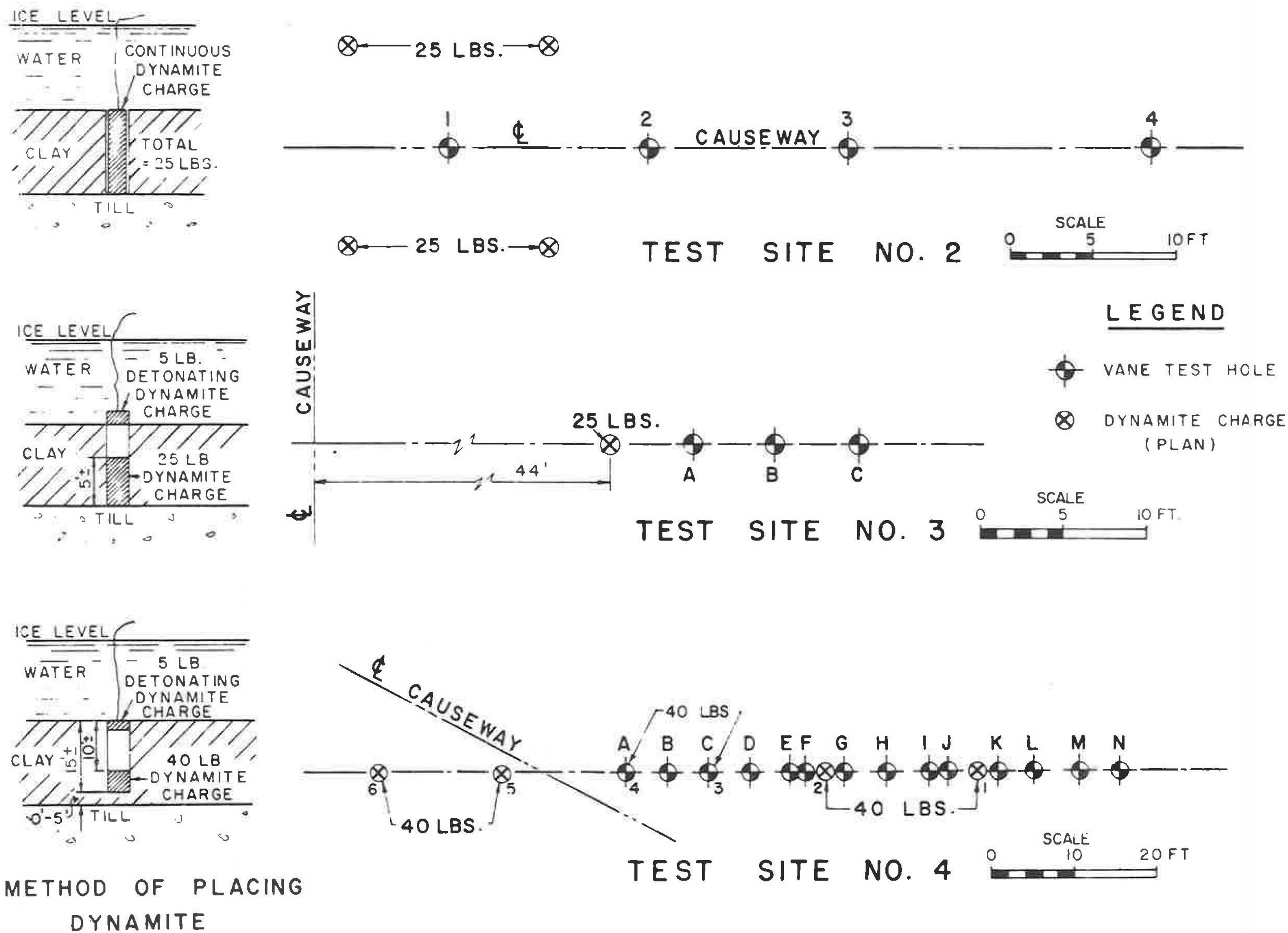

Figure 10. Layout of dynamite charges and vane tests.

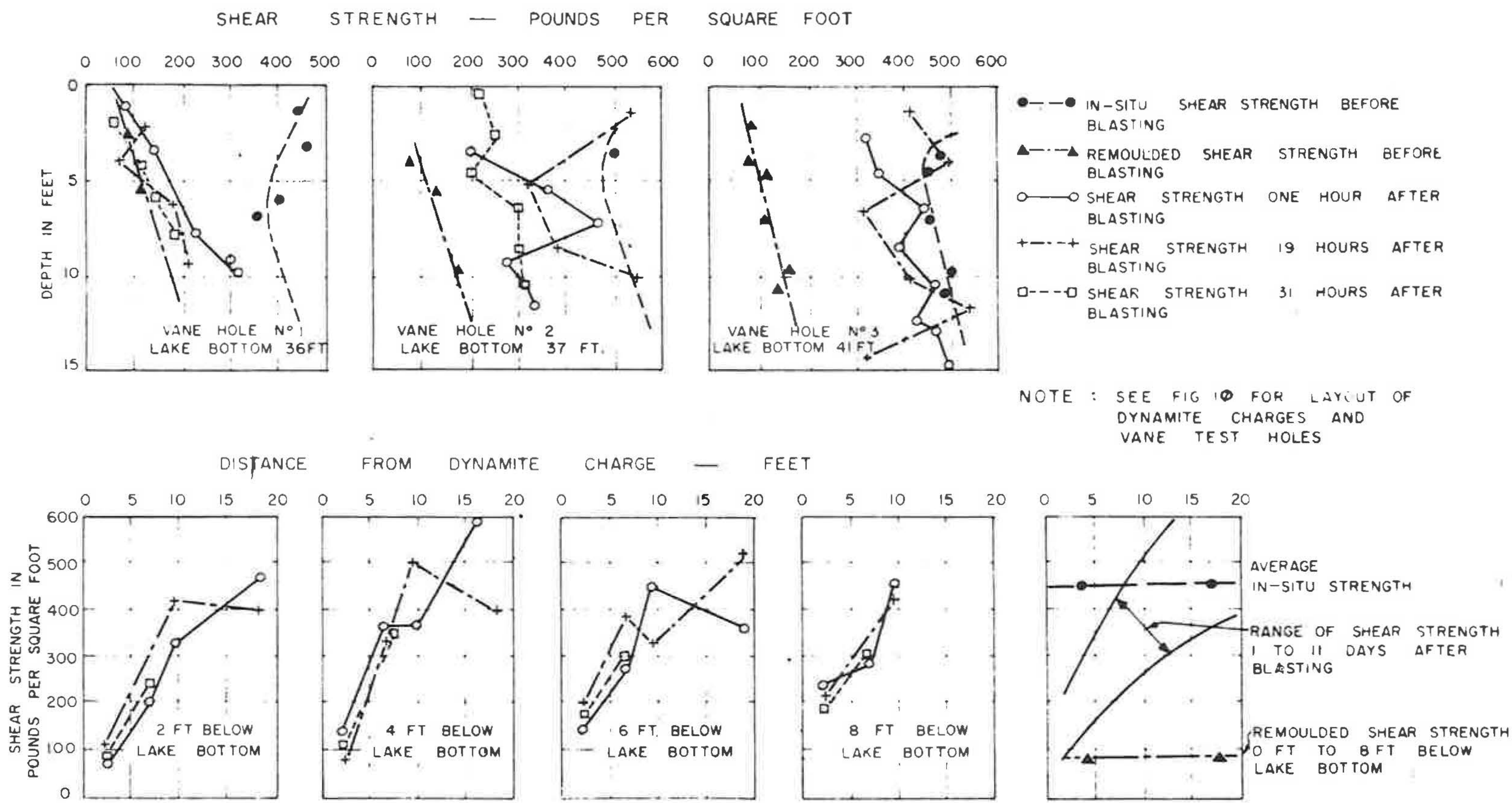

Figure 11. Blasting test results—Test Site 2.

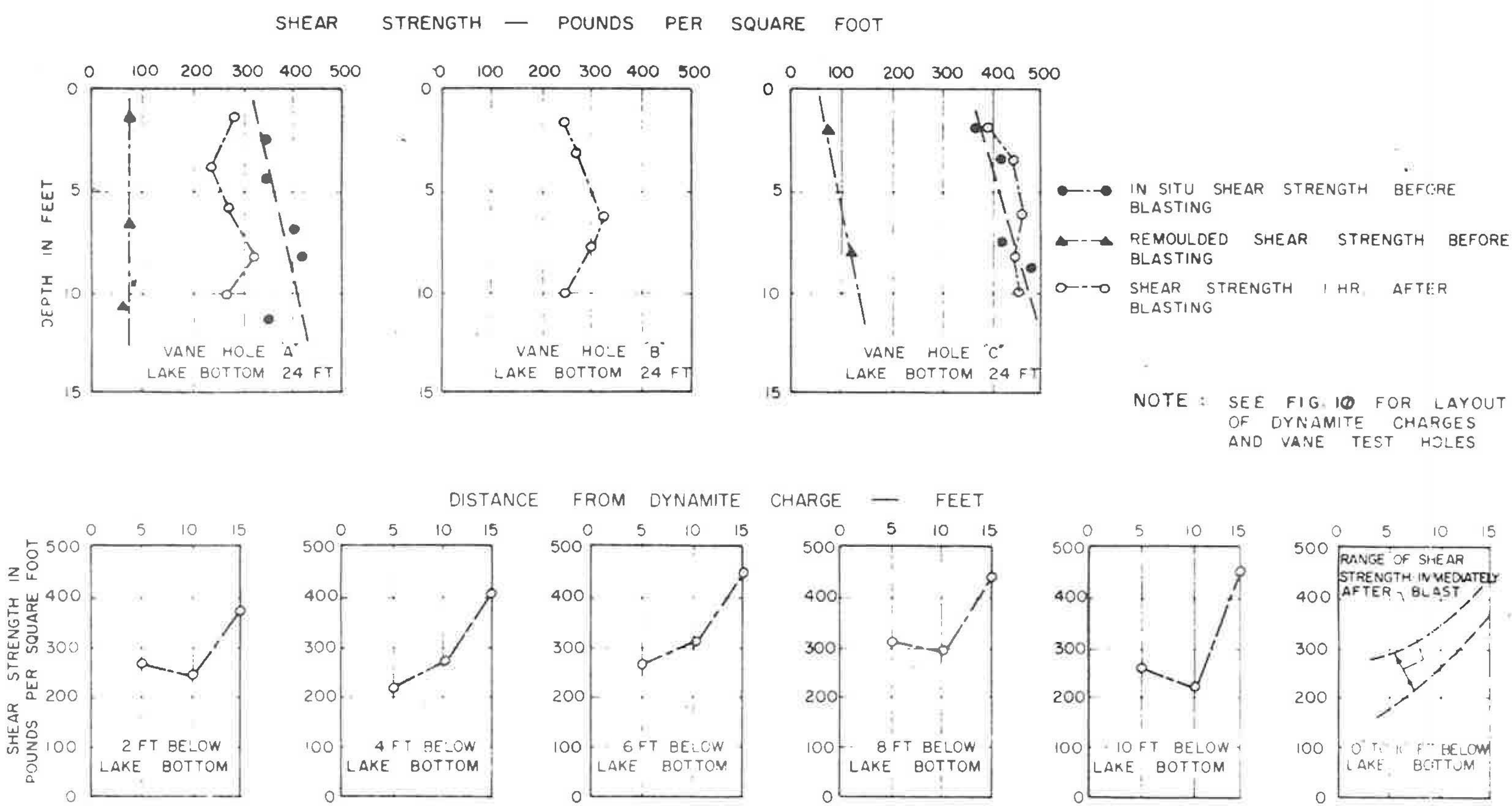

Figure 12. Blasting test results—Test Site 3.

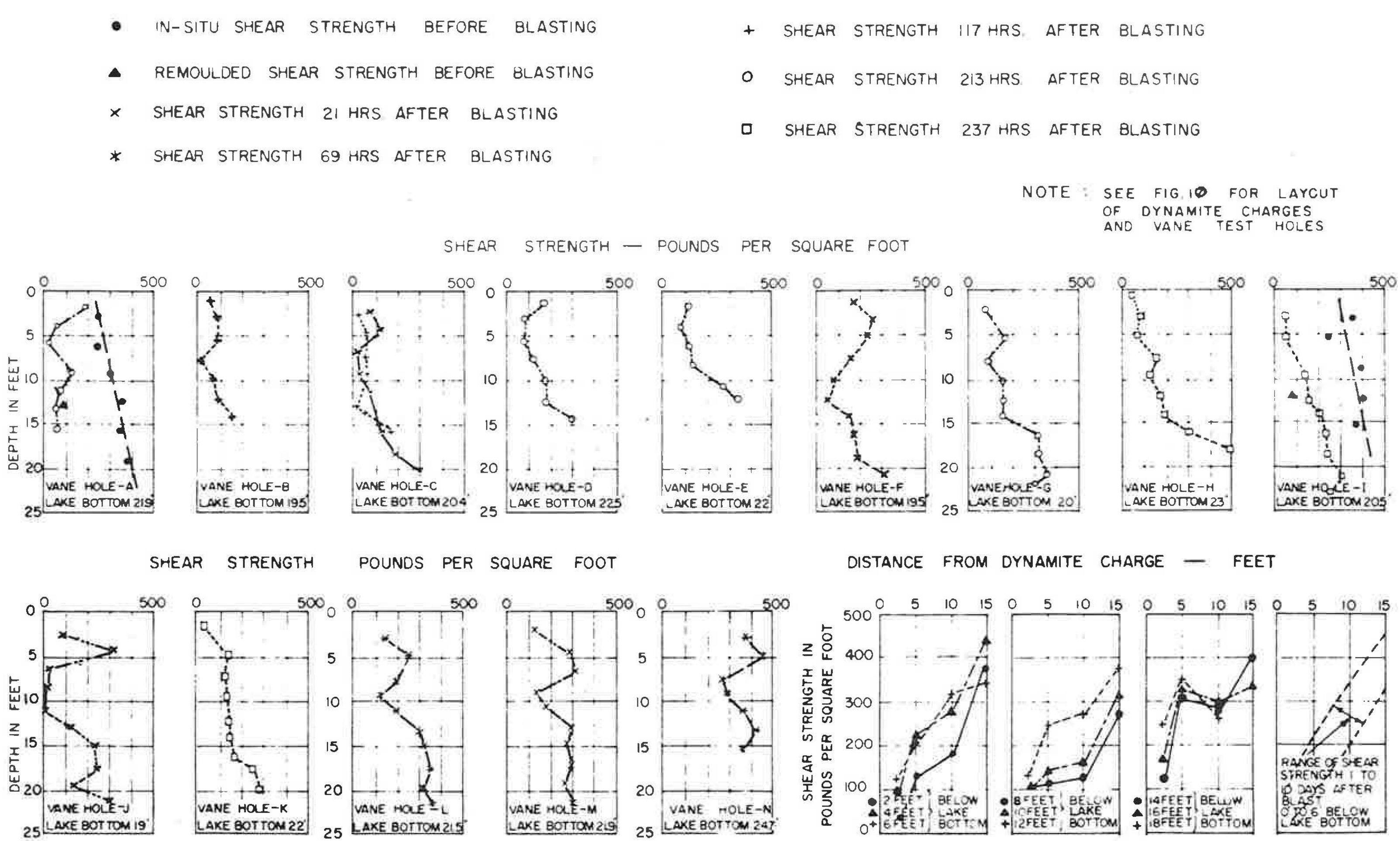

Figure 13. Blasting test results—Test Site 4.

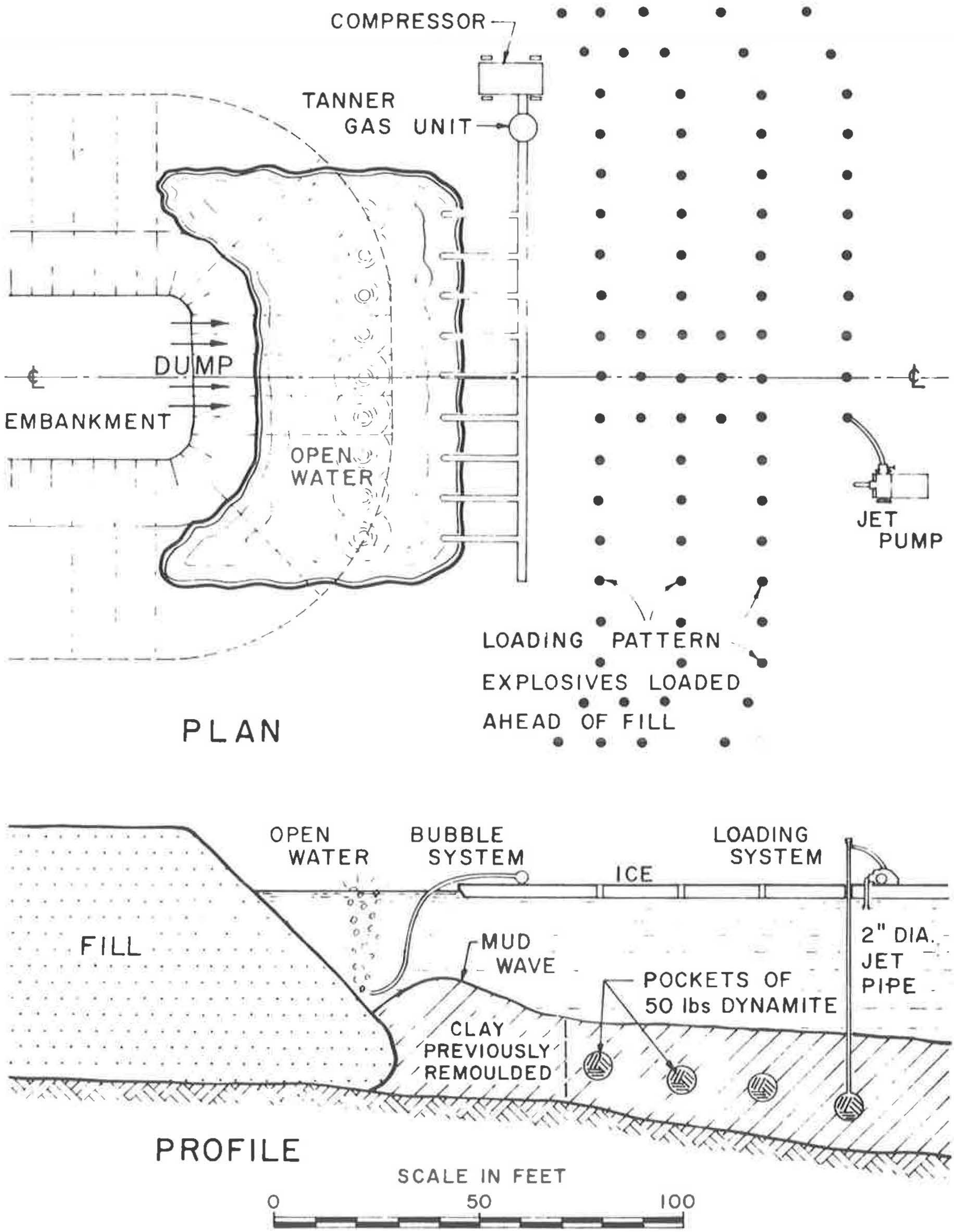

Figure 14. Embankment construction.

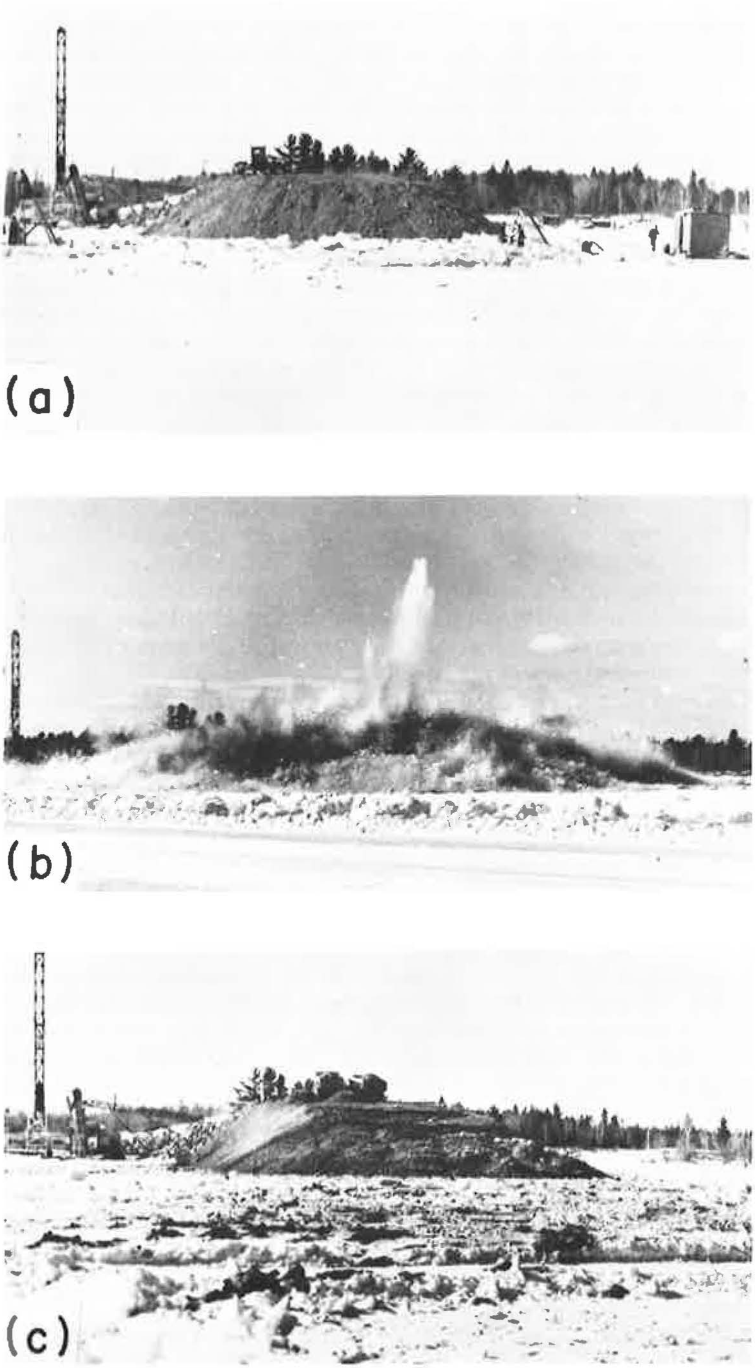

Figure 15. Dynamite blasting ahead of fill: (a) before blast, (b) during blast, and (c) after blast.

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

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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.

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Geologic Research at the Straight Creek Tunnel Site, Colorado

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The cost of constructing tunnels in the Rocky Mountains of Colorado, and in many other places in the world, usually has exceeded the estimates of the cost, sometimes by a wide margin. In most instances the reason for the increased cost has been geologic. The U. S. Geological Survey, in cooperation with the Colorado Department of Highways, is conducting a research project at the Straight Creek tunnel site, on I-70, in order to apply and evaluate new ideas in the use of geology and geophysics for prediction of geologic conditions at depth, in the belief that more accurate predictions of geologic conditions can reduce the cost of construction.

The surface outcrops in the vicinity of the tunnel were mapped in detail; particular attention was given to the percentage of different rock types, the attitude of the foliation of the rock, and the attitude and spacing of faults and joints. The surface observations were supplemented by the drilling and logging of two core holes; records were kept as to the percentage of rock types, the attitude of the foliation, and the attitude and spacing of faults and joints.

The geologic investigations were supported by geophysical logging of the drill holes—including resistivity, gamma-ray, and gamma-ray density—by seismic profiles at the surface run parallel and at right angles to the tunnel line, and by laboratory measurement of physical and engineering properties of samples from the surface and the drill holes.

From all these data a statistical model of the geology at tunnel level was constructed. Then from this model it was possible to calculate the probable rock loads, spacing of sets, spacing of lagging, probable sections of the tunnel that should be tested by feeler holes, probable amount of grout that would be required to seal badly broken ground highly saturated with water, probable amount of water that would flow from the portal of the tunnel as the face was advanced, and probable initial amount of water that might be expected to flow from a fault zone within any interval of the tunnel. Construction of a pilot bore for the Straight Creek tunnel has recently been started. Geologic and geophysical investigations are being made in this pilot bore to determine the accuracy of the predictions that were made, and in the hope that this information will lead to new ideas that will give even better predictions of the geology for tunnels to be constructed in the future.

•THE COST of constructing tunnels in the Rocky Mountains of Colorado usually has exceeded, sometimes by a wide margin, estimates of their cost. This also has been true in other areas of the United States, and in other parts of the world. The reason for the increased cost, in nearly every instance, can be attributed to the presence of geologic conditions that, from an engineering point of view, either were not known or were not fully understood before the construction of the tunnel.

Paper sponsored by Committee on Landslide Investigations.

Examples in Colorado of excessive cost for tunnel construction, as a result of adverse geologic conditions, are the Moffat and Leadville tunnels. The Moffat tunnel, which was constructed in 1923-27 to take water and the Denver and Rio Grande Western Railroad through the Continental Divide, cost about 4 times the estimated cost (Lovering, 4, p. 339). According to Lovering (4, p. 338), the original estimate was based on the assumption that the tunnel would be driven in solid rock, but actually about 2 mi of the tunnel was in weak material that required much support. The worst zone encountered, the Ranch Creek fault, was about 1,000 ft wide and required 2 tons of steel per running foot of tunnel to reinforce a concrete lining that averaged about 30 in thick. The Leadville tunnel, which was constructed in 1943-45 and 1950-52 to drain the Leadville mining district in Colorado, cost almost twice the estimated amount (1, 6). The principal reason for the increased cost in the Leadville tunnel was a water-saturated, gravel-filled glacial stream channel 4 to 12 ft above the roof of the tunnel. The tunnel roof collapsed, filling part of the tunnel with unconsolidated material and necessitating the construction of a bypass tunnel. Construction costs for many other tunnels in Colorado have been substantially higher than the original estimates, owing primarily to some unpredicted geologic feature or features.

The most recent tunnel completed under the Continental Divide in Colorado is the Harold D. Roberts tunnel. This tunnel, which was constructed by the Denver Board of Water Commissioners to bring water from the western to the eastern side of the Divide, is about 23 mi long. The route for the tunnel was selected after several years of geologic study of possible routes by T. S. Lovering, E. E. Wahlstrom, and others. The cost savings of these studies can only be estimated, but comparison with problems encountered in other tunnels suggests that the construction cost was reduced by at least 25 percent.

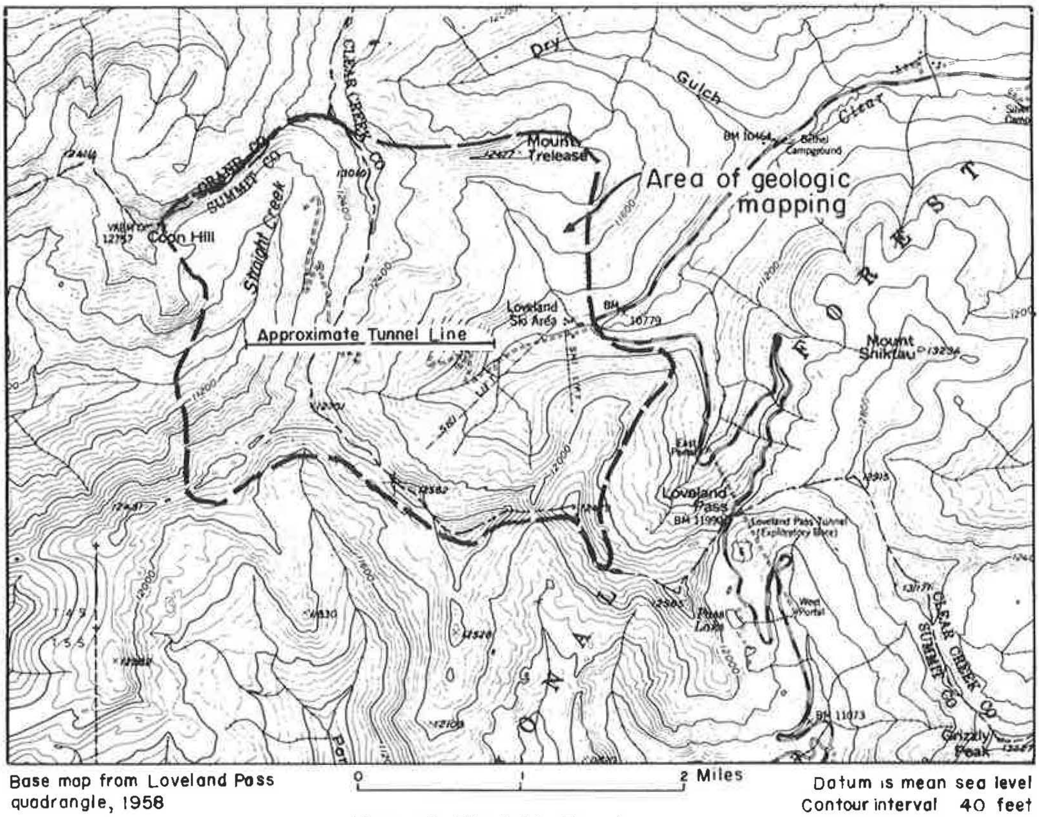
The present research investigation of the Straight Creek tunnel site is the direct result of a post-construction study of the engineering geology of the Roberts tunnel by the U. S. Geological Survey in conjunction with E. E. Wahlstrom and L. A. Warner. Some of the preliminary results of the Roberts tunnel study have been published by Wahlstrom (8); Wahlstrom, Warner, and Robinson (1961); Wahlstrom and Hornback (1962); and Wahlstrom, Robinson, and Nichols (in preparation).

The studies of the Roberts tunnel showed that different types of geologic features—faults, veins, joints, and contacts between rock types—could be projected to tunnel level with different degrees of accuracy, and that the accuracy depended on a thorough knowledge of the regional geology—and therefore the origin of these features—and on a detailed examination of the surface over the tunnel route. It was also determined that the rock type was not as important in the construction of a tunnel as were the number of fractures per foot of tunnel, attitude of bedding, foliation, and fractures in relation to the trend of the tunnel, presence or absence of hydrothermal alteration, the type of clay minerals in fault gouge, etc. It seemed possible that a statistical compilation of such geologic features as the attitude of joints and faults could be projected to a tunnel level more accurately than could the individual features, and would give a better basis for engineering design and construction. It also seemed that known geophysical techniques could be used to supplement the data obtained by surface examination and core drilling—particularly in areas of poor outcrop and poor core recovery.

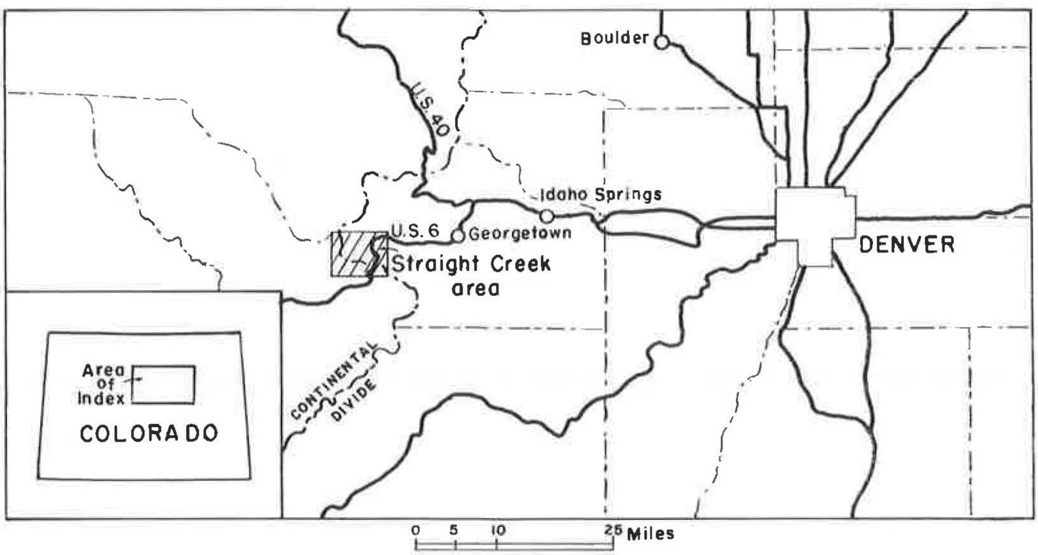
The U. S. Geological Survey, in cooperation with the Colorado Department of Highways, is conducting a program of geologic and geophysical research for the pilot bore for the Straight Creek tunnel. The purpose is to apply the ideas developed from the study of the Roberts tunnel to see if better predictions and engineering interpretation of geologic conditions can be made.

Although most of the methods of investigation of the Straight Creek tunnel site were based on studies of the engineering geology of the Roberts tunnel--which in part was constructed in a similar geologic environment—it is believed that with minor modifications based on the regional geology, most of the methods of investigation are applicable to determining the engineering geology of any proposed tunnel site. This paper describes the preconstruction geologic, geophysical, and laboratory investigations that have been made, and the engineering interpretation and prediction of the results.

The Straight Creek tunnel site is about 50 miles west of Denver (Fig. 1). The



Map of Straight Creek area



Location of Straight Creek area

Figure 1. Index maps of Straight Creek area, Colorado.

Straight Creek tunnel will carry I-70 through the Continental Divide and so eliminate the use of Loveland Pass on present US 6. The pilot bore, on which construction was started in November 1963, will be about 10 × 10 ft and approximately 8,000 ft long. The final tunnel will consist of twin bores, each about 32 ft in diameter and 8,000 ft long.

GEOLOGIC INVESTIGATIONS

The geologic investigations were started in 1962, and consisted of the geologic mapping of approximately 6 sq mi in the vicinity of the tunnel site and the geologic logging of 2 core holes, the drilling of which was supervised by the Colorado Department of Highways.

Mapping

The surface mapping was done at a scale of 1:12,000; particular attention was given to the percentage of different rock types, the attitude of the foliation of the rock, and the attitude and spacing of faults and joints. Figure 2 is a generalized geologic map of the area showing the distribution of the different rock types.

Geologic Formations.—The area consists predominantly of Precambrian granite with inclusions of metamorphic rocks. The granite is medium to fine grained and consists of approximately equal amounts of quartz, potash feldspar, and plagioclase feldspar, and from 5 to 15 percent biotite. A distinct foliation is recognizable in most outcrops. The foliation is the result of a subparallel orientation of the potash feldspar grains. In some outcrops, particularly in those containing a higher percentage of biotite, the biotite is oriented parallel to the potash feldspar grains. The rock appears fresh, but petrographic examination shows that most of the biotite has been altered to chlorite and that the plagioclase feldspar has been slightly altered to sericite. The granite is extensively altered only in, or adjacent to, some of the shear zones and faults. Mapped with the granite were small pods and dikes of pegmatite, that consist predominantly of quartz and potash feldspar.

The metamorphic rocks consist of a variety of biotite-rich gneiss. Common types are biotite-quartz-microcline gneiss, biotite-quartz-plagioclase gneiss, hornblende-biotite-plagioclase gneiss, and sillimanitic biotite-plagioclase gneiss. These rocks generally are fine grained. Their foliation is the result of the concentration and orientation of the constituent minerals into bands that range from less than 1 to about 10 mm in width. In nearly all outcrops the biotite, and some of the hornblende, is altered to chlorite. In and adjacent to the faults and shear zones the metamorphic rocks are commonly altered to a green plastic clay, in which the foliation, although considerably contorted by folding, is still recognizable.

Diorite dikes, probably of Tertiary age, crop out north of the tunnel line; and might be encountered during construction of the tunnel. The dikes range from a few feet to about 1,000 ft in maximum dimension. They consist of fine-grained to aphanitic augite and plagioclase with varying, and smaller, amounts of biotite and hornblende.

The bedrock throughout much of the area is mantled by Quaternary surficial deposits. These include swamp and morainal deposits at lower elevations in the valleys and colluvial deposits of soil, talus, and landslides on the upper slopes.

Structure.—Geologic structures probably are the most important factors in increasing the cost of tunnel construction above the estimates. Accordingly, they were given, particular attention during mapping. The structural features mapped were the attitude of the foliation and the attitude and spacing of faults, shear zones, and joints.

Foliation is the result of the orientation and layering of mineral grains. In the granite, the foliation is not a principal direction of weakness and should not affect the engineering properties of the rock. In the metamorphic rocks, however, the foliation is a major direction of weakness; those layers that are composed predominantly of biotite are relatively much weaker than those composed predominantly of quartz and feldspar.

Faults and shear zones are numerous in the area mapped. This area lies within a wide zone of regional faulting and shearing that is probably related to the Loveland Pass fault (Lovering and Goddard, 1950, pl. 2). Figure 3 is a generalized map of the

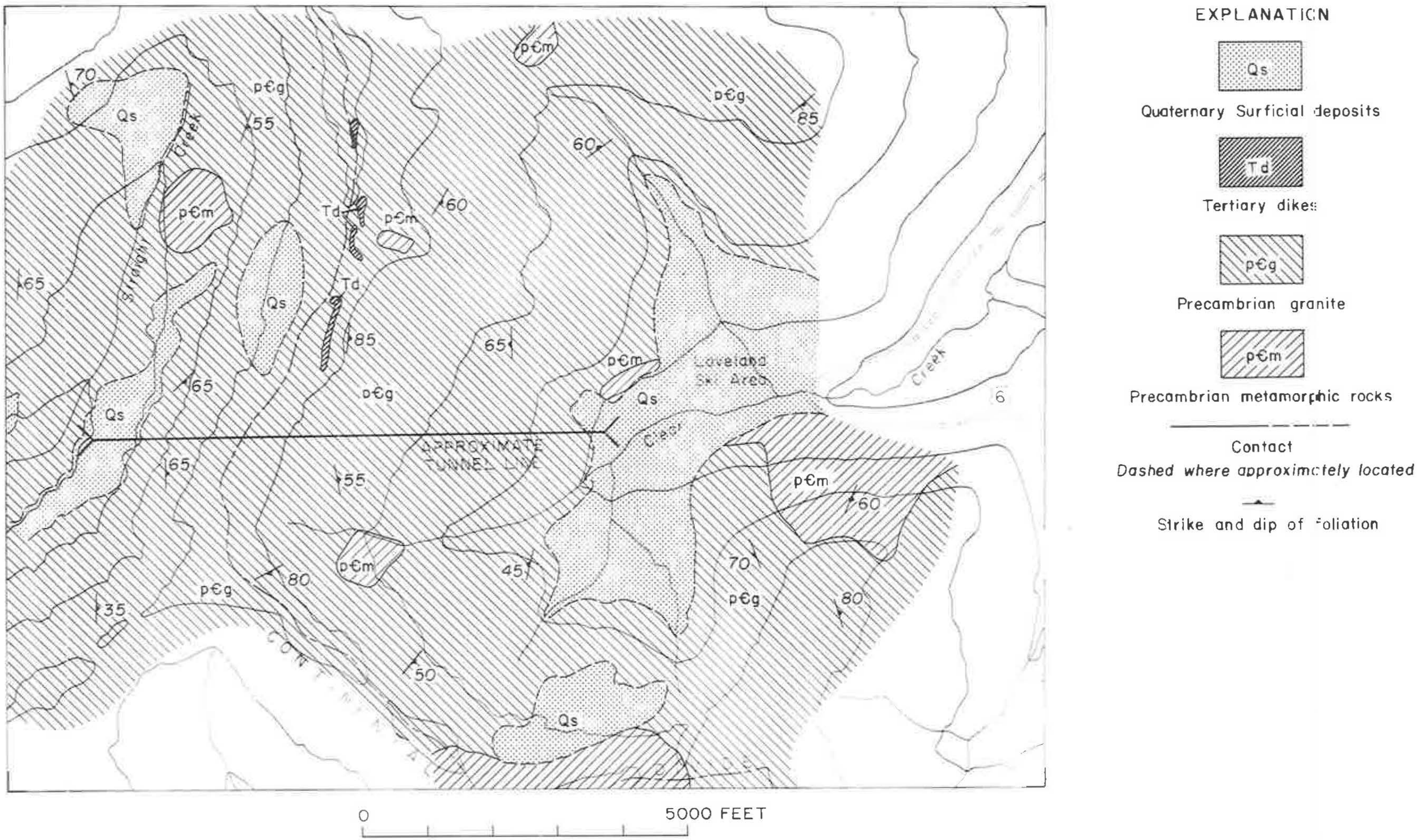


Figure 2. Generalized geologic map of Straight Creek area showing distribution of rock types.

faults and shear zones of the area. The distinction between faults and shear zones is one only of magnitude; faults are elongate areas of crushed rock from 5 to 50 ft wide and shear zones are elongate areas more than 50 ft wide. Faults less than 5 ft wide are too numerous to be shown on this generalized map.

The faults and shear zones consist of rock showing varying degrees of crushing or shearing. The borders of these zones are gradational—the intensity of shearing decreasing outward from the center of the zone. Near the margins, the rock consists of slivers from less than 0.1 to 1 in. wide bounded by slickensided shear planes. Where the shearing was more intense, the rock has been crushed to a coarse to fine sand and the shear planes are less than 0.01 to 0.5 in. apart and lie in all directions. Where the most intense shearing occurred--probably accompanied by some alteration—the material consists of clay (fault gouge) with variable amounts of quartz and/or feldspar grains. The gouge usually does not occur near the center of the sheared zone but is nearer one margin than the other—most commonly adjacent to the footwall of the zone. The gouge occurs as disconnected streaks elongated parallel to the trend of the shear zone.

The faults and shear zones vary in width within short distances. The individual faults or shear zones pinch and swell and may end abruptly against relatively unbroken rock. Some shear zones contain blocks of rock, up to about 100 ft in maximum dimension, that are relatively unsheared although surrounded by intensely sheared rock. The shear zones and faults, because of their general discontinuity, were not projected through covered areas. For this reason, the abrupt ending of most of the shear zones, as shown in Figure 3, is the result of cover and not the result of an observed abrupt ending of the zone. Most of the fractures recorded as joints are microfaults and shears. In mapping, the attitude and the maximum, minimum, and average distance between joints in each set were recorded. An effort was made initially to distinguish between tension and shear joints, but nearly all joints showed some evidence of shearing.

Logging

Two core holes were drilled, approximately equally spaced between the portals of the proposed tunnel, to determine the geology in the vertical dimension. Four holes had previously been drilled during preliminary investigations in 1955. At the start of the present investigation, the drilling of more than two holes was proposed. Detailed surface mapping in this type of geologic environment, however, was considered to be of more value—and much less expensive—than additional drill holes. A core represents only about a 2-in. diameter sample (for an NX hole) for a part of the length of the hole. The significance of this sample can be evaluated, and the information obtained by drilling extrapolated, only on the basis of detailed geologic mapping and a thorough knowledge of the regional geology.

Figure 4 is a section of a geologic log of one of the drill holes. This type of log gives a maximum amount of geologic information for engineering interpretation. It differs from standard logs primarily in that the sizes—range and average length—of the pieces of core have been recorded. The size should be indicative of the competency of the rock and the way the rock will break during mining operations. In addition, because the rock has a distinct foliation, it was possible to measure the attitude of the joints in relation to the strike of the foliation.

GEOPHYSICAL INVESTIGATIONS

The geophysical investigations consisted of resistivity, radioactivity, and gamma-ray density logging of the drill holes, and seismic profiles at the surface along and at right angles to the proposed tunnel line. The geophysical investigations were conducted to experiment with new instruments and techniques for determining the engineering properties of rocks in situ, and to obtain supplemental data for the geologic interpretations.

Figure 5 is part of a geophysical log of one of the drill holes. Shown on the geophysical logs are the geologic column, a description of the geology and the casing size, date of installation of casing, and cemented intervals. For the example given (Fig. 5), the casing was installed after the geophysical logging.

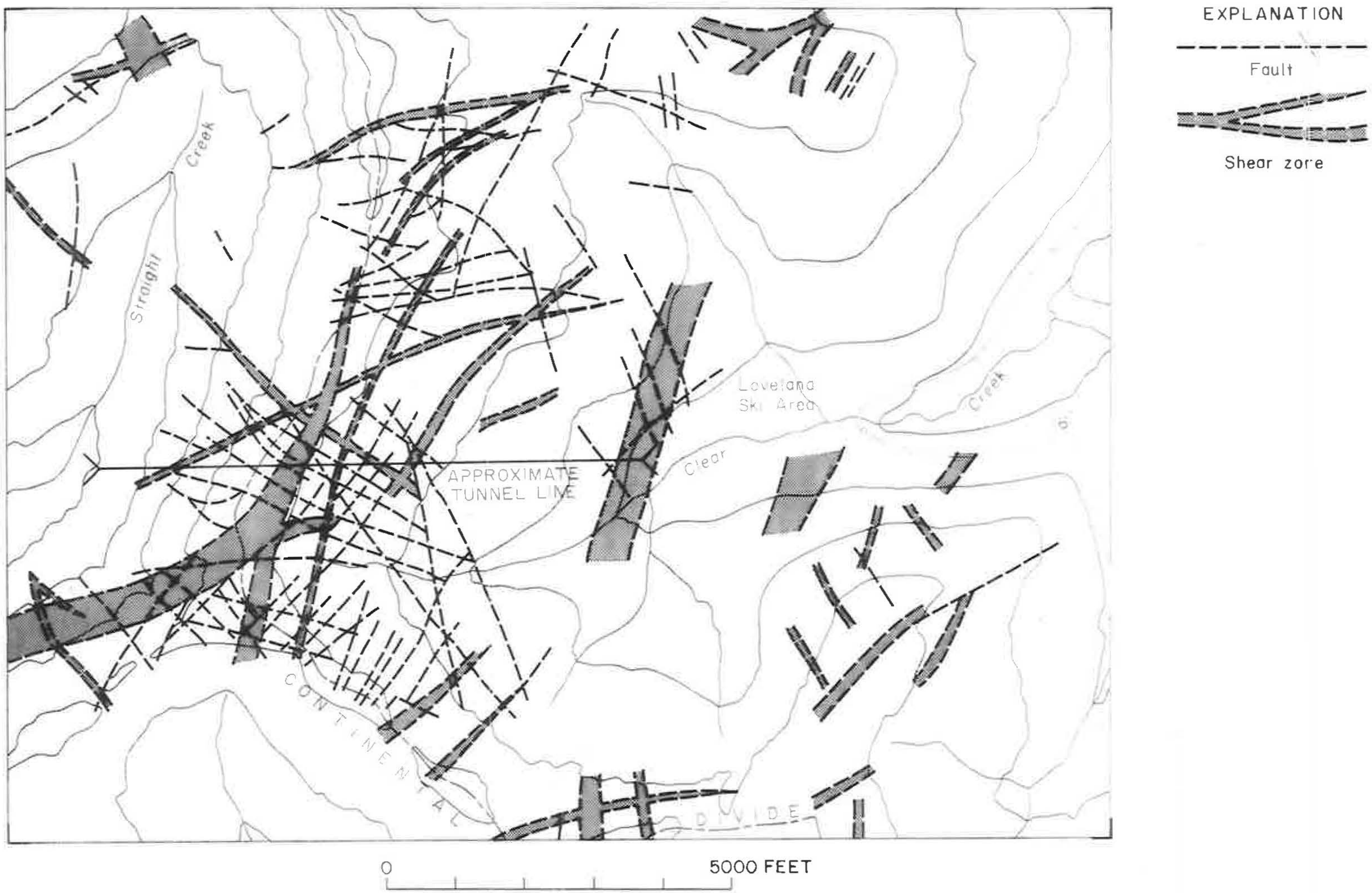


Figure 3. Generalized map of faults and shear zones of Straight Creek area.

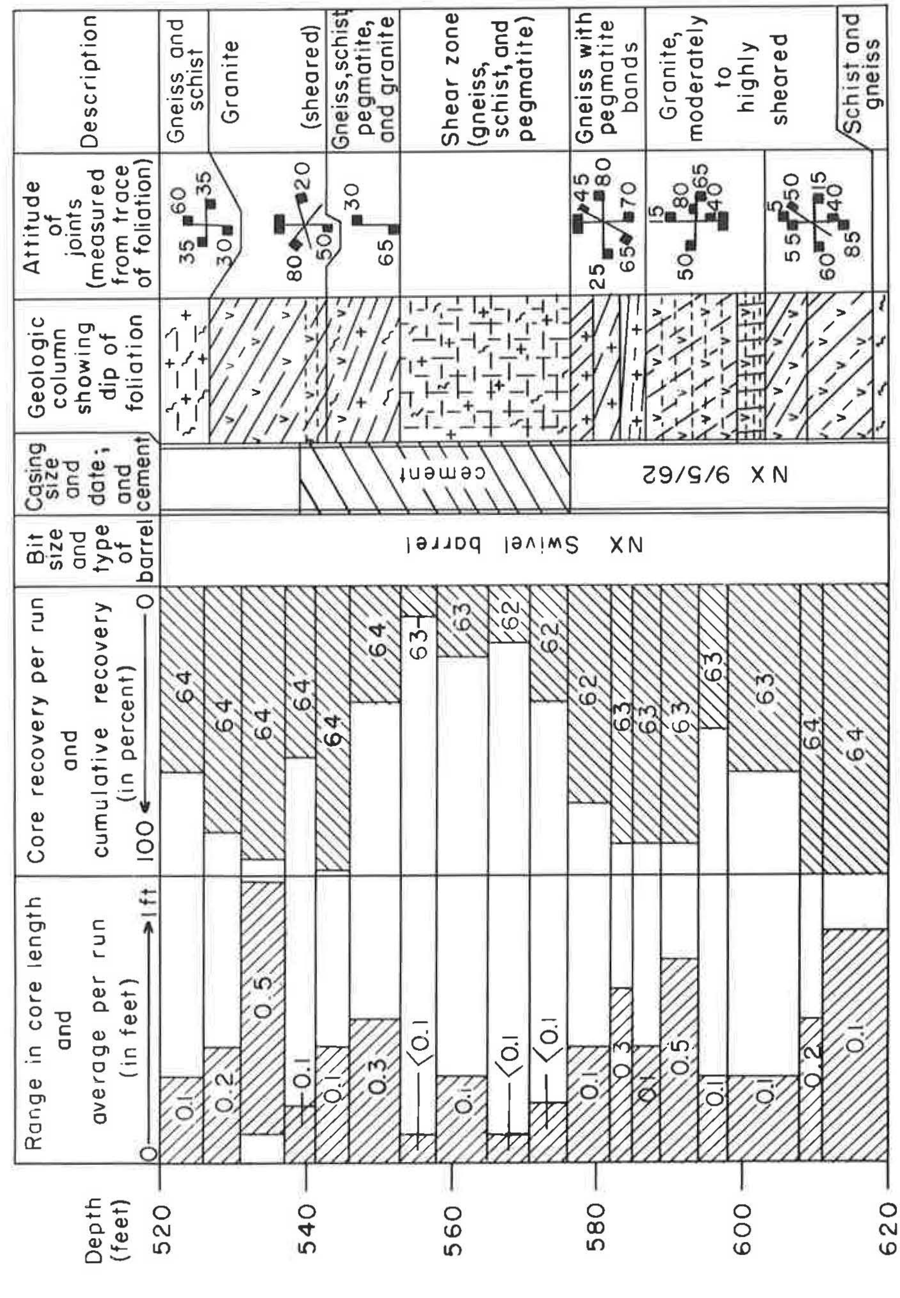


Figure 4. Example of geologic drill hole log, Straight Creek project.

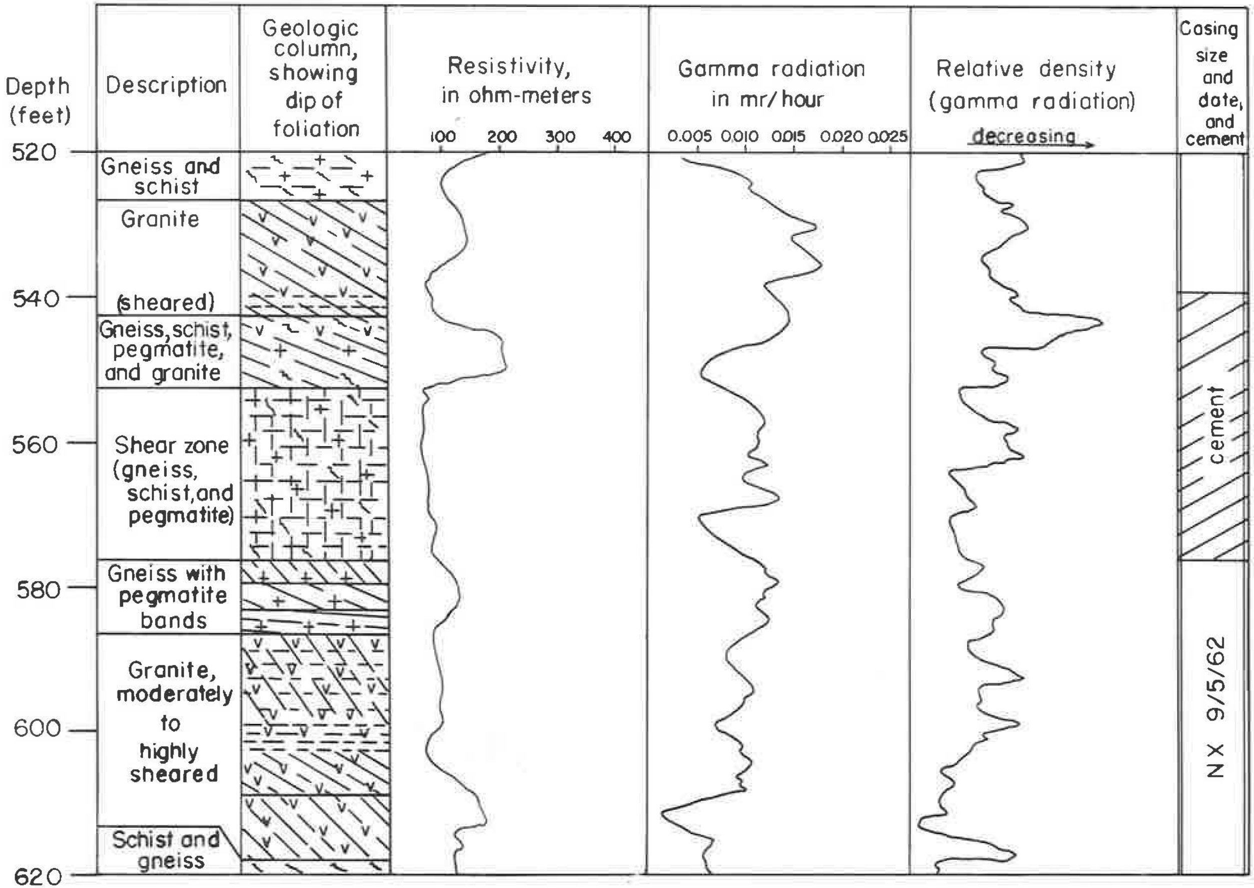


Figure 5. Example of geophysical drill hole log, Straight Creek project.

Resistivity values are largely dependent on the amount and nature of water contained within the rock mass, which in turn is dependent on the amount of fracturing. The resistivity logging supplemented the geologic logging and was used primarily to determine or confirm zones of shearing, particularly where core recovery had been poor. The resistivity logging was done by C. J. Zablocki.

The natural gamma-radiation logging was also primarily used to interpret rock conditions, particularly rock type in those intervals with poor core recovery. The Precambrian igneous and metamorphic rocks of this area, as with most rocks, have a characteristic natural gamma radiation. The natural gamma radiation of the different rock types is affected by the amount of shearing and alteration and the logging, therefore, where the rock type was known, gave a relative value for the amount of shearing and alteration.

The relative density of the rock in the walls of the drill holes can be determined by lowering a source of gamma radiation into the drill hole and measuring the absorption of the gamma rays. Such relative density logging gave a relative value for the amount of shearing and alteration. The gamma-radiation and density logging was done by W. A. Bradley.

All the logging encountered a common difficulty: those sections of the holes in badly broken ground caved (often within a few hours of drilling) so that the probes could not reach the bottom, or required cement or casing, which prevented the effective operation of one method or another.

Seismic surveys were conducted along the tunnel line. The purpose was to determine the seismic velocities, and the changes therein, across the surface and with depth; it was hoped that these velocities could be correlated with the geologic mapping and its resulting definition of rock conditions. An excellent discussion of the application of seismic techniques to the determination of rock properties in situ has recently been presented by D. Wantland (1963) of the U. S. Bureau of Reclamation. The seismic surveys were under the direction of R. A. Black and B. L. Tibbetts of the U. S. Geological Survey.

Geophones were set both along and at right angles to the proposed tunnel line at the surface, and seismometers (specifically designed for the purpose) were placed in the two core holes. Records were obtained from charges detonated in drill holes in bedrock near the proposed tunnel portals and from airblasts aboveground. The results have not been as satisfactory as hoped for. The investigation was hampered by a lack of precedent on which to design the equipment and of tested procedures for such a survey. Additional seismic surveys with modified equipment and procedures are planned which hopefully will provide better records.

Another seismic survey was conducted by R. M. Hazlewood and C. H. Miller of the U. S. Geological Survey, in the vicinity of the east portal to determine the thickness of surficial material. On the basis of this survey the Colorado Department of Highways was furnished a map showing the surface and bedrock topography that allowed the calculation of the amount of surficial material that had to be excavated at the east portal.

LABORATORY INVESTIGATIONS

A continuing program of laboratory investigations is being conducted in coordination with the geologic and geophysical field investigations. The purpose is not only to furnish the necessary data for the interpretation of field geologic and geophysical information for engineering purposes, but also to attempt to correlate physical and engineering properties as determined in the laboratory with those determined in situ, and to conduct research on new techniques and instruments for determining physical and engineering properties in the laboratory and field. The laboratory investigations have been supervised by T. C. Nichols of the U. S. Geological Survey.

The mineralogy, porosity, grain density, dry bulk density, and saturated bulk density were determined for surface and drill-core samples. The elastic properties of selected samples have been measured by dynamic and static methods to determine if, from a geologic analysis (composition and structure), these two methods of measuring elastic properties can be correlated. Compressive and shear strengths have been determined

under various confining pressures and are being correlated with a geologic analysis of the samples. A good correlation can be established between compressive strength and the mineralogy of the samples. Many of these laboratory investigations do not have a direct quantitative bearing on the construction of the Straight Creek tunnel because the samples tested were the most homogeneous geologically. It is hoped that correlation of geologic analyses with physical and engineering properties will in the future allow translation of laboratory data to field conditions and correlation of laboratory results with in-situ measurements.

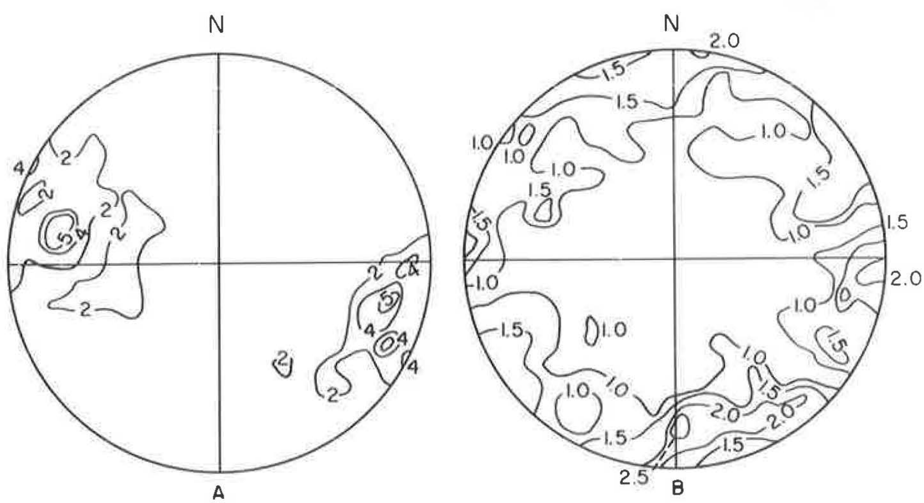
Of direct application in the construction of the Straight Creek tunnel has been the measurement of the swelling properties of the clay in fault gouge that results from the physical and chemical alteration of rock. As discussed by Wahlstrom, Robinson, and Nichols (in preparation), the relative swelling properties of clay can be determined by a PVC (potential volume change) meter (3). A knowledge of the swelling properties, including the final swell pressure, will allow the design of adequate support to contain this pressure.

COMPILATION OF RESULTS OF INVESTIGATIONS

In areas as complex as the Straight Creek site, the projection of surface geologic features to tunnel level has been unsuccessful (Wahlstrom, in press). A statistical compilation of the results of the geologic, geophysical, and laboratory investigations, however, can give estimates of the type and magnitude of the geologic features to be encountered in the driving of a tunnel.

A compilation of the data from the surface mapping and drilling shows that 75 percent of the Straight Creek tunnel will be in granite and 25 percent in metamorphic rock. The metamorphic rock occurs as inclusions in the granite ranging in average maximum dimension from less than 1 to 200 ft, and having an average maximum dimension of about 20 ft.

The attitudes of foliation of the granite and metamorphic rock along a 1-mi strip with the tunnel line at the center are shown in the equal area plot in Figure 6a. The diagram, compiled from 189 measurements, shows that the foliation in general strikes from N to N 30° E and dips from 60 to 90° NW or SE. The use of petrofabric studies and the methods of compilation, have recently been discussed by Friedman (2).



Foliation in granite and metamorphic rocks (189 poles)

Joints in granite (838 poles)

Figure 6. Lower hemisphere equal area plots of attitudes of foliation and joints: (a) foliation in granite and metamorphic rocks, and (b) joints in granite.

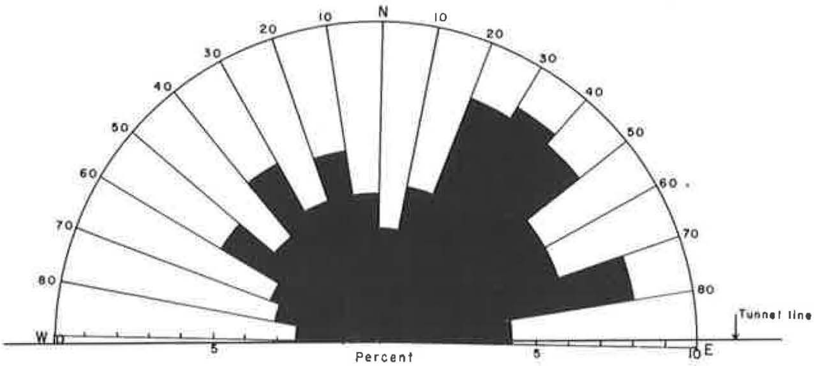


Figure 7. Strike frequency diagram of 284 faults and shear zones.

A contour diagram of the attitude of 838 joints measured in granite along the same 1-mi wide strip is shown in Figure 6b. The diagram shows that the joints strike in any direction and dip from 45 to 90° and average about 70°.

Similar diagrams for the attitude of the foliation and the joints were prepared from the logs of the drill holes. The strike of the foliation and joints could not, of course, be determined from the core. The average strike of the foliation in the core was assumed to be the same as that compiled from the surface mapping (Fig. 6a); it was then possible to orient the joint diagrams compiled from the drill core, as the strike of the joints in the core was measured in reference to the strike of the foliation. The diagrams prepared from the surface data differed from those prepared from the core data only in that the average dip for the foliation and joints in the core was about 50° as compared to 70° at the surface.

In most areas one or more principal directions to the strike of the joints are to be expected. By the use of statistical compilations it might then be possible to orient a tunnel to intersect the principal foliation and joint directions at a maximum angle. In the case of the Straight Creek tunnel, where there are no principal directions of strike, a change in the orientation of the tunnel would not improve construction conditions.

The number and magnitude of the faults and shear zones, and their attitudes in relation to the orientation of the tunnel, are the principal structural features that will affect the construction of the tunnel. The strike frequency diagram (Fig. 7) illustrates the relation of the trend of 284 observed faults and shear zones to the trend of the proposed tunnel. The diagram shows that most of the faults and shear zones intersect the proposed tunnel line at angles of greater than 30°, and that only about 4 percent of the faults and shear zones trend parallel to the tunnel. No fault or shear zone wider than 5 ft was noted at the surface that would be expected to follow the proposed tunnel line at depth. The dips measured on 74 of the 284 faults and shear zones ranged from 35 to 90°, and averaged 75°.

The average spacing between all types of fractures (faults, shear zones, and joints) was plotted on a geologic map and contoured. The contour intervals used represented an average distance between fractures of from less than 0.1 to 0.5 ft, 0.5 to 1 ft, and from 1 to 3 ft. The map could then be divided into a series of zones: (a) the zones of greatest fracturing, where the average distance between fractures was less than 0.1 to 0.5 ft (these were essentially the major shear zones); (b) the zones of intermediate fracturing, where the average distance between fractures was 0.5 to 1 ft, and about 20 percent of the area was represented by faults and shear zones; (c) the zones of least fracturing, where the average distance between fractures was 1 to 3 ft, and about 10 percent of the area was represented by faults and shear zones.

GEOLOGIC PREDICTIONS

The data compiled from the geologic, geophysical, and laboratory investigations were used to construct a predicted geologic section along the proposed tunnel line. Figure 8 is a generalization of this geologic section.

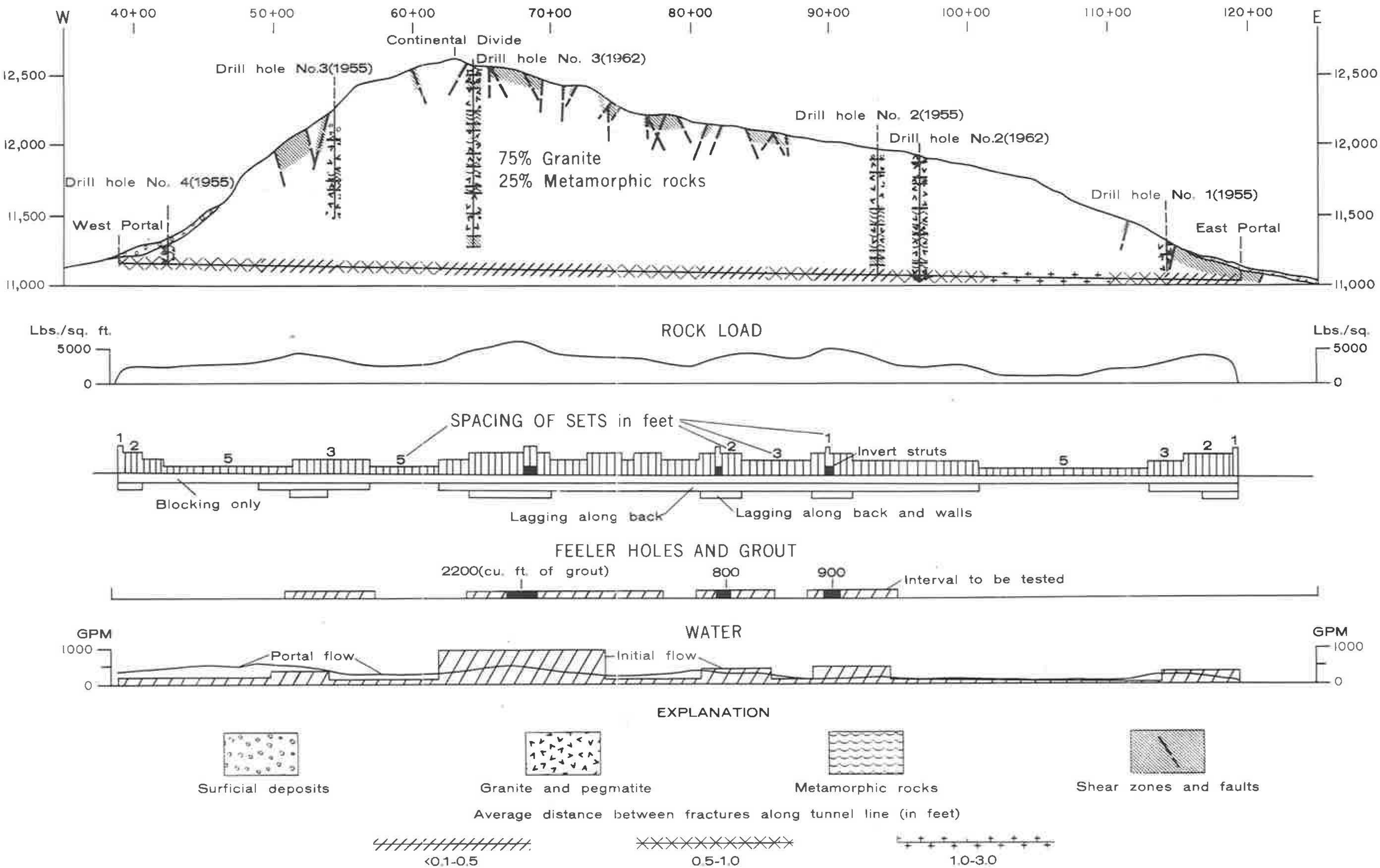


Figure 8. Predicted geologic section and engineering data along proposed Straight Creek tunnel pilot bore.

In constructing the geologic section, no effort was made to project individual rock units, faults, or shear zones to tunnel level. As previously discussed, it is impossible to predict the position of the metamorphic rock inclusions, because their maximum dimension averages only about 20 ft. It can be stated, however, that, as shown in Figure 8, 75 percent of the tunnel will be in granite and 25 percent in metamorphic rock. Rather than project the individual faults and shear zones from surface to tunnel level, the different zones of the average spacing between fractures as determined at the surface were projected to tunnel level, on the basis of the average dip of faults and shear zones as measured from the surface and the drilling data.

It should be understood that the boundaries between the different zones for each section of the tunnel are gradational and indefinite—this is an interpretation—and that the location of each zone is based on the projection of statistical values. The length and position of each of these zones may be in error by as much as 50 percent, but the percentage of rock in the different zones in relation to the total length of the tunnel is probably correct within less than 25 percent. This then gives a basis for the calculation of engineering data necessary for estimating the cost of construction of the pilot bore tunnel.

CALCULATION OF ENGINEERING DATA

The engineering data calculated for estimating the cost of construction of the tunnel were the rock load, spacing of support sets and lagging, the need for feeler holes and grouting, and the amount of ground water. The calculations were based on the predicted geologic section and the laboratory data, and the results are shown in Figure 8 as a series of graphs below the geologic section.

Rock Loads

The calculations for rock load were based on the assumptions that the pilot bore, outside the timbers, would be about 10.5 ft wide and 11.5 ft high; and that the rates of driving the pilot bore and of installing supports—both factors will affect the rock load—would be as efficient as possible. The rock load, then, would primarily be the result of geologic conditions.

The calculations were made using the following formula, which is modified from Terzaghi (7, p. 61):

$$P = C\,(b + h)\,X\,W \qquad (1)$$

in which

P = rock load in psf,
C = a constant depending on rock conditions,
b = width of tunnel,
h = height of tunnel, and
W = weight per cu ft of rock.

The value for W was determined to be 166.88 based on the measurement of the saturated bulk density of samples. The values for C ranged from 0.35 to 1.60 depending on the interpretation of the rock conditions expected and the possible presence of fault gouge. A maximum load of about 5,900 psf was calculated.

Support

Rock pressure, fracture density, and the amount of fault gouge or clay alteration products vary from place to place, although they are interrelated to a considerable degree. The total effect of these geologic factors on tunnel construction can be expressed, more or less quantitatively, by the amount of support that may be required. The estimates given in this report concerning support requirements should be regarded as only a relative measure of the severity of geologic conditions, and not as an attempt to formulate definitive engineering design requirements.

For purposes of calculation, it was assumed that the sets would be so designed as

to support a maximum load of 10,000 psf and that the tunnel would be driven on a 3-shift-a-day basis. The principal factor then that will influence the amount of support is the fracture density. The results of the calculations are shown on a graph below the section in Figure 8. Note that invert struts may be required for some sections. These are the sections where wide zones of fault gouge may be encountered.

The prediction of the spacing of sets and amount of lagging is empirical. The spacing of sets and the amount of lagging that will actually be required can be determined only at the time of excavation. The purpose of the graph is only for estimating the probable total amount of timber that will be required and the approximate location where timbering problems may be encountered.

Feeler Holes and Grouting

The geologic conditions ahead of some sections of the tunnel should be determined by drill holes in advance of the face. The probable location of these sections is shown in Figure 8. Such feeler holes will test for badly broken or crushed rock that may be filled with water. Many such zones noted on the surface and in the drill holes might be expected to contain large volumes of water at the depth of the tunnel. If these zones are located in advance of the face, it may then be possible to seal off the water and consolidate the broken rock in advance of the tunnel.

Variations in the geologic conditions encountered will also determine the amount of grout. The sections in which the geologic conditions indicate grouting may be required are shown in Figure 8. The amount of grout given is based on the amount used in the Roberts tunnel, which averaged about 10 cu ft (sacks) per foot of grouted section.

It should be emphasized that the sections indicated to be tested by feeler holes and the sections to be grouted are predictions only. In practice, rock conditions at the face should be continually and carefully observed, and if there is any indication of a possible water-bearing section ahead of the face, the section should be tested by feeler holes. The cost of feeler holes is cheap in comparison to the cost of recovering a caved face.

Ground Water

The amount of ground water that will be encountered depends on the porosity and permeability of the rock and the height of the water table above the tunnel level. The porosity of the rock depends primarily on the average distance between fractures as calculated for each interval of the tunnel, and the number of faults and shear zones. The permeability of the rock depends primarily on the size and interconnection of the fractures. These factors were determined from field and laboratory investigations and equated with records of water flows in other tunnels and with test data from wells in similar rocks. In making these calculations the authors were assisted by George H. Chase of the U. S. Geological Survey.

The average amount of ground water that is expected to flow from the tunnel portal is shown by a graph in Figure 8. This graph is based on an estimated rate of advance of the heading of 1,000 ft per month. An increase in the rate of advance will increase the amount of flow, and conversely, a slower rate will decrease the amount of flow. If this rate is projected beyond the total time of driving the tunnel, the average flow at the portal should decrease to about 300 gal per min within 2 weeks and to about 100 gal per min within a year.

The maximum amount of flow that might be expected from the various fracture intervals of the tunnel is shown by a histogram in Figure 8. This amount is the maximum initial flow that might be expected from a fault or shear zone within the interval. This flow should decrease rapidly within 5 to 10 days. According to these calculations, the maximum average flow from the portal is estimated at about 500 gal per min; the maximum initial flow from any section of the tunnel is estimated at about 1,000 gal per min.

It should be understood that these figures are based on statistical averages that were obtained by comparing the predicted geologic conditions with well data for similar rocks and with records of flows from other tunnels. Within any section of the tunnel, the water will not issue at a uniform rate but will flow primarily from the faults and shear zones and the more closely jointed rock within that section.

FUTURE RESEARCH

Geologic, geophysical, and laboratory studies are being continued currently with the pilot bore construction, which was started about November 1, 1963. The primary purpose is to evaluate the geologic and engineering predictions so that predictions for future tunnels can be more accurate. The Colorado Department of Highways has included, as part of the contract for construction of the pilot bore, certain items of work that will allow the U. S. Geological Survey as well as the Department's consultants to conduct fundamental research on the measurement of physical and engineering properties of rock in situ.

Detailed geologic maps of the tunnel are being made as the face is advanced, and the related engineering data recorded. A recording flume has been installed at the portal of the tunnel to register the water flow from the portal; and the amount of water issuing from the face is recorded after each round. The amount of initial water flowing from the larger water channels is recorded, and the measurements repeated until the flow becomes constant. The amount of timber and the spacing of sets are recorded, as are the number, direction, and length of feeler holes and the amount of grout used.

The Colorado Department of Highways has retained Patrick Harrison, Inc., to instrument the tunnel to determine quantitatively the rock loads. This is to be done at 200-ft intervals—and as dictated by the geologic conditions—by the use of load cells in the sets and single- and multiple-position borehole extensometers. On the basis of the geologic conditions, these data can be extrapolated to the probable support requirements in the final twin bores.

The current geophysical investigations include resistivity and sonic velocity measurements along the walls and in drill holes. The purpose is to devise instruments and procedures, and to evaluate the use of such instruments and procedures based on these principles, for measuring engineering properties of rocks in situ. Also, these data will aid in the prediction of geologic conditions to be encountered in driving the final twin bores, and possibly in selection of the position for these final bores in relation to the pilot bore.

ACKNOWLEDGMENTS

The investigations have been conducted with the full cooperation of the Colorado Department of Highways. The authors specifically thank Mark U. Watrous, former Chief Engineer, and Charles E. Shumate, Chief Engineer; Adolph Zulian, Engineer of Plans and Surveys; George N. Miles, District Engineer; Fred A. Mattei, Project Engineer; Winfred D. Fitzpatrick, Engineer; and Stanley N. Mitchell, Geologist, for their help during the investigations. Members of the U. S. Geological Survey who have assisted the authors are acknowledged at the appropriate places in the text.

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Ground Water Control for Highways

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California Division of Highways, Sacramento

Two recently completed highway contracts are used as a background for the discussion of subsurface drainage procedures in use in the design and construction of highways in California.

Investigations to determine the needs for subsurface drainage are described and can be grouped under field reviews, geologic studies, borings, tests, and analyses.

Methods of subsurface water controls most commonly used are stripping and blanketing with permeable material, stabilization trenches, horizontal drains, and other somewhat specialized measures. The application, construction, and effectiveness of these methods are discussed.

Particular consideration is given to the characteristics of the permeable material that is used as a part of most of the subsurface water control measures.

•CONSTRUCTION OF FREEWAYS covered by two contracts will be used as a background for discussing subsurface drainage practices in the construction of highways in California. Under these two contracts, approximately 7.8 mi of freeway were constructed along I-5 in the northern part of California. The construction started in the latter part of 1958 and was completed early in 1961.

The magnitude of the subsurface drainage that is involved on major highway construction in the fairly mountainous terrain (Fig. 1) can best be illustrated by comparing the costs of construction of these two projects and the costs of the subsurface drainage facilities.

The first contract awarded in 1958 was for $4,140,000 of which $434,000 or approximately 10 percent was for subsurface drainage facilities. Contract Change Orders in the amount of $860,000 were approved, and $86,000 or 10 percent of this amount was for subsurface drainage facilities. Thus, the total contract for $5,000,000 included $520,000 or approximately 10 percent for subsurface drainage facilities.

The other contract was started in 1959 and was for approximately $5,095,000 and the subsurface drainage facilities represented $965,000 or approximately 19 percent of the total cost. Contract Change Orders in the amount of $944,000 were approved and of this amount $82,000 or 9 percent was for subsurface drainage facilities. Of the original drainage facilities, $200,000 was deleted. Thus, on the total contract for $6,038,000 the subsurface drainage facilities represented $847,000 or approximately 14 percent. Certainly these expenditures totaling some $1,367,000 out of a total of a little more than $11,000,000 indicate that a substantial portion of the expenditures for highway construction in this type of terrain is for subsurface drainage. The actual breakdown of items for ground water control and costs is given in Table 1.

Ground water is a major factor in instability of embankment foundations and cut slopes. By far the majority of landslides on California's highway system are the results of instability caused by subsurface water. The most common means of improving stability is removal of the ground water. If the ground water is not removed or adequately controlled, landslides often result and these may be very serious in regard to disruption of traffic as well as cost. Figure 2 shows destruction of a portion of highway caused by

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Figure 1. Typical mountainous terrain.

Figure 2. Landslide—fill foundation failure.

TABLE 1

COMBINED COSTS OF CONTRACT ITEMS FOR GROUND WATER CONTROL[1]

Item	Cost ($) Unit	Total
Stabilization trench excavation	1.30 cu yd	512,962
Permeable material:		
Stabilization trenches and blankets	1.65 ton	286,844
Underdrains	7.00 cu yd	93,792
Drainwells	5.75 lin ft	98,270
PMP underdrains	2.00 lin ft	139,008
Horizontal drains	2.75 lin ft	224,443
Collector pipes	2.00 lin ft	11,300
Ground water control items		1,366,619
Other contract items		9,672,658
Total		11,039,277

[1] Contract Nos. 59-2TC18 & 60-2TC2.

failure of a fill foundation. Traffic was detoured for several weeks over a rather inadequate detour and the cost of correction and reconstruction was in excess of $100,000.

IMPORTANCE OF SUBDRAINAGE

Subdrainage has become increasingly important as a means of stabilization of cut slopes and foundations for embankments in highway construction in California in recent years. The mountainous terrain and the high annual rainfall of northern California have combined to create a situation conducive to foundation problems (1).

Three factors have contributed to the increased importance of subdrainage. First, the necessity for more favorable alignment and grade in highway construction has resulted in much larger cuts and fills in the mountainous terrain that is prevalent throughout much of California. Second, traffic volume has, on much of the highway system, made the older two-lane road obsolete, and it has been necessary to replace it with modern freeways with four or more lanes. This has also resulted in cuts and fills of far greater magnitude than were necessary a few years ago. Third, much of the mountainous terrain, particularly in northern California, is located in areas of moderately high rainfall and somewhat poor foundation conditions.

If subsurface water exists and stability of fill foundation or cut is somewhat doubtful, removal or alleviation of the adverse subsurface water conditions is usually a must. To ignore the subsurface water in the construction of cuts or fills in these areas, in the hopes that construction or natural conditions will improve the subsurface drainage, can be a disastrous and costly process. Almost without exception it is more economical and more practical to correct adverse subsurface drainage conditions before construction rather than to attempt to handle this situation as a maintenance operation.

METHODS OF ANALYSIS AND DESIGN

Exploration

In the design and construction of the subsurface drainage facilities for highways, exploration plays an important part. The various stages of the exploration usually

consist of one or more of the following parts: field review, geologic studies, borings, tests, and analysis of data.

Field Review

In the early stages of any project, usually as early as the planning stage and certainly during early phases of design, a field review is made to determine to some degree the foundation problems that may be encountered and to obtain some impression of the magnitude of corrective measures that may be necessary to construct a stable road. This field review will usually include representatives of several departments such as planning, materials, design, and construction and will serve as a basis for making the further studies that will be necessary for design of foundation treatments.

As the design nears completion, a more detailed field review should be made to study the relationship of the planned subsurface drainage features to the project as a whole to make certain all anticipated problems have been provided with a satisfactory solution. This review will usually include representatives from some or all of the units involved on the earlier field review.

The following is a quotation from a letter prepared as a result of a field review made in connection with one of these projects:

> At all locations where embankments are to be constructed the borings show foundations of questionable stability. The soil is predominantly wet soft clay containing numerous cobbles and boulders. Free water was encountered in most of the borings, apparently in large quantities. The depth of the wet material was indeterminate; in several of the borings no firm material was found at depth of about 70 feet, the maximum depth penetrated.
>
> Although the transverse slope of the natural ground is relatively flat, it is our opinion that the risk of embankment slipouts will be excessive unless extensive sub-drainage treatment is provided. Fill failures would not only cause interruption to movement of traffic on the freeway, but would also jeopardize or destroy streets and buildings adjacent to the right-of-way.

Geologic Studies

Some of the districts in the Division of Highways have engineering geologists on the staff of their Materials Departments. The Materials and Research Department, a headquarters unit of the Division of Highways, also has several engineering geologists on the staff. Typically, Materials Reports include a geologic description of the area involved in construction. The nature and magnitude of the geologic mapping and study that is done will depend to a large degree on the nature of the topography, type of terrain and magnitude of the problems encountered. The following is a quotation from the geologic phase of the Materials Report on one of the projects:

> The project is located on the west side of the Sacramento River Canyon and traverses rugged mountainous terrain. Rock types found within the limits of the project consist of ultramafic rocks of various types, some with a schistose structure. The rocks have a general northeast-southwest strike and dip steeply to the southeast. Recent flows of pyroxene andesite are found along both sides of the canyon and occupy earlier channels of the river. The present channel of the river follows the trend of the weaker rock structure in the ultramafic rocks and is in part responsible for the many slides that occur in the canyon. The soil mantle in general is silty clay, sandy gravelly clay, tuffaceous clay, silty sandy gravel and residual boulders of various types.

Borings

The drilling that is done in exploring projects of this nature is determined largely from field reviews, available geologic information and information obtained as drilling progresses. On the two projects under consideration, a total of 150 borings were

made. These borings varied in depth from a few feet to in excess of 100 ft. These borings included considerable exploration in connection with cut slope design, as well as the borings necessary for fill foundation stabilization which consists primarily of subsurface drainage. Most of the borings were made with power equipment and continuous flight augers. A limited amount of exploration was done with power equipment and 2-in. diameter samplers that are pushed or driven to obtain undisturbed soil samples for visual inspection and testing. Some sampling was done with 1-in. diameter hand-driven samplers to secure samples primarily for visual inspection. The exploration was concentrated in the areas where field review and geologic mapping indicated that questionable foundation conditions might exist. Most of the exploration was done during design stage after the actual line and grade had been adopted. In areas where this exploration revealed extremely unfavorable foundation conditions, line or grade changes were frequently made to improve stability. An appreciable portion of the exploration was done in the early stages of construction after clearing and pioneering had been completed. This exploration was aimed primarily at more completely delineating the extent of foundation treatment required and exploring areas where foundation problems were encountered during construction that had not been evident during design.

Testing

The testing on these two projects was rather limited, due primarily to the nature of the problems encountered and to the type of soils and rock that were present. Much of the foundation soils for the embankments consisted of layered or heterogeneous combinations of fairly firm soil mixed with rock; and soft saturated clays, silty clays or clayey silts intermixed with soft to hard rock. Most of these softer formations or zones were water bearing. Figure 3 shows a typical boring profile. Securing representative samples of these softer materials was difficult although some testing was done to get a comprehensive picture of the strength characteristics of this material. The testing consisted primarily of unconfined compression tests and a limited quantity of consolidation and triaxial compression tests. To supplement these tests, numerous unit weight, moisture content, grading, and Atterberg limits tests were made.

Ground Water Observations

A careful survey was made of evidences of ground water and seepage in the area where the two projects being described were to be constructed. As the borings were made, signs of excess moisture or seepage were carefully noted. Soil samples were examined to determine moisture conditions, and the borings were sounded frequently during drilling operations to determine if there was free water in the borings. Measurements were made of the rise of water in the borings subsequent to completion of the borings.

During construction, observations were made to determine the effectiveness of the ground water control measures that had been or were being incorporated in the construction. Ground water conditions were observed in the various excavations as they were made. Observations were also made of flows of water from the various ground water installations that were a part of the construction.

During construction of the projects, pumping tests and drawdown observations were made in vertical relief wells that are described later. These tests were made to determine the effectiveness of the combined installation of vertical relief wells and horizontal drains. Figure 4 shows typical pumping test data depicting the drawdown.

Analysis of Data

Information available from the field review, geologic studies, borings, tests and analyses was used in an effort to determine the nature and magnitude of any corrective measures that were necessary to construct a stable road. Subsurface drainage facilities were incorporated in the design when it was believed that data from the aforementioned sources of information indicated that cuts or fill foundations would be unstable or borderline for stability. The most common situation that indicated the necessity

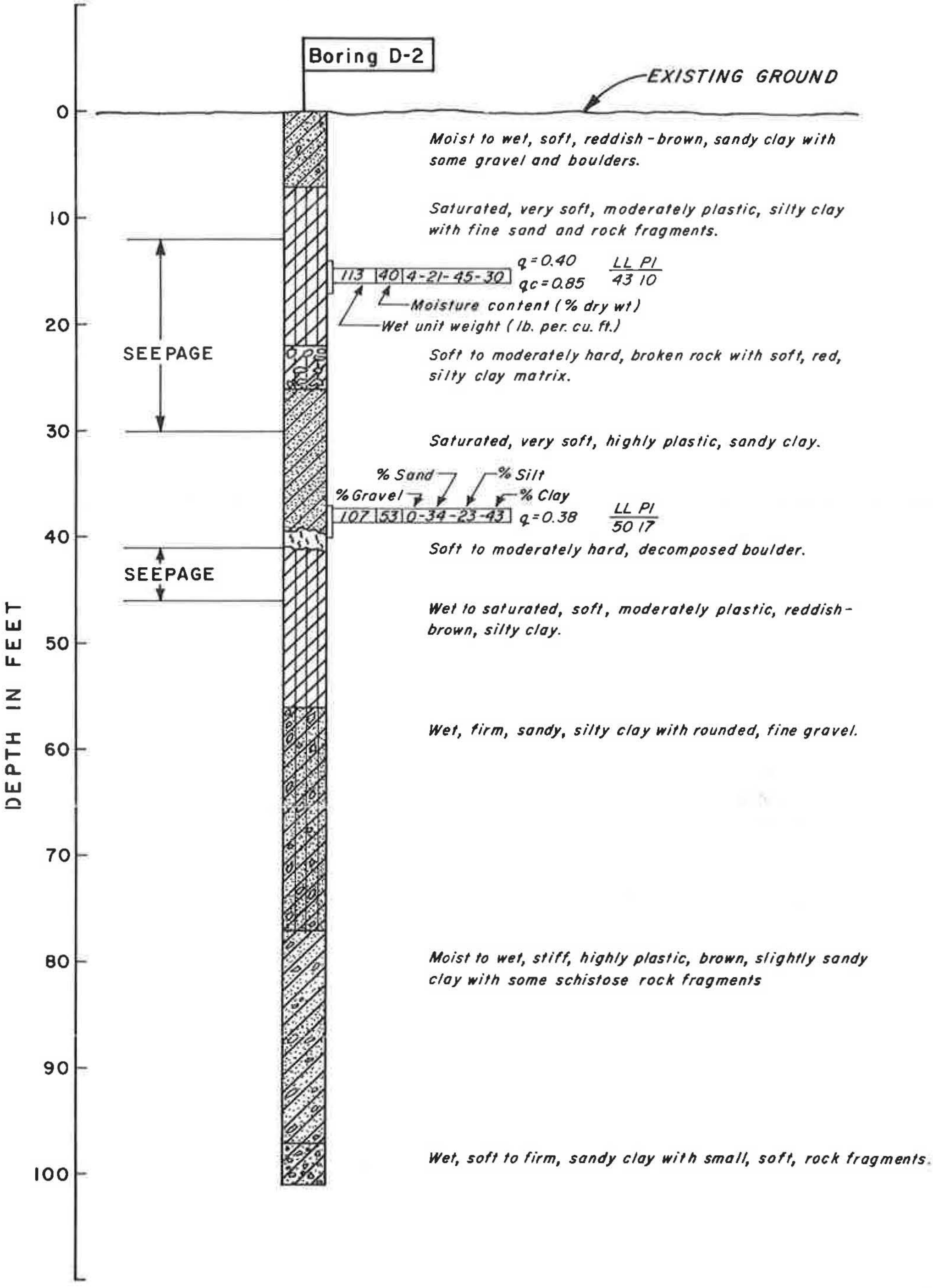


Figure 3. Typical boring profile.

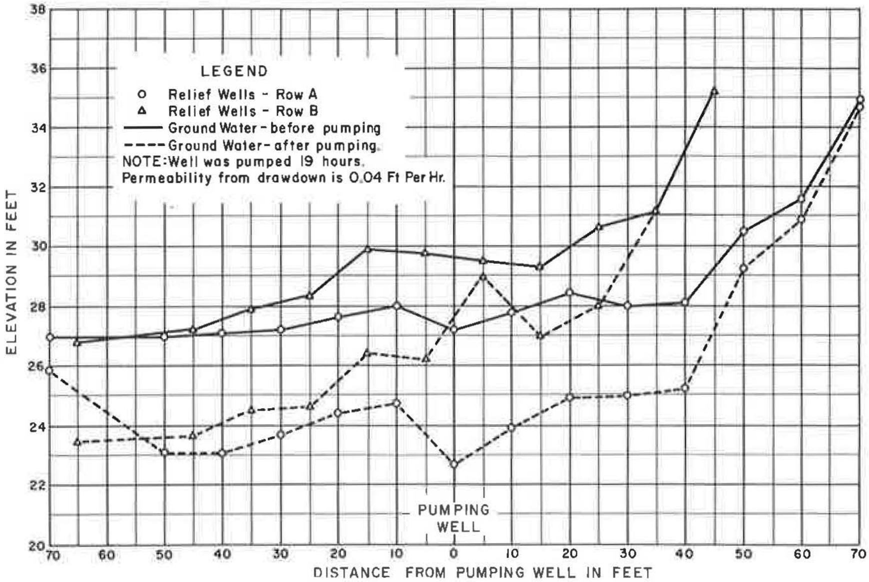


Figure 4. Typical graph of drawdown in vertical relief wells.

for subsurface drainage on the fill foundations was layers of water-bearing soft material interspersed with drier, firmer layers of soil containing more rock. Strength testing was concentrated in the soft zones. If embankments are placed over these areas, there is usually a tendency to compress the water-bearing strata and to reduce the ability of these strata to provide drainage. Thus, hydrostatic pressures are increased, especially during wet seasons and the stability of the foundations is endangered. Experience in the California Division of Highways indicates that most of the problems in subsurface drainage occur in soils of grain sizes that normally would be classified as relatively impervious. Most of the water appears to be moving in strata that are relatively fine grained and that may contain relatively high percentages of clay. The water is moving along fissures, fractures, joints, or bedding planes that are somewhat more capable of carrying water than the main soil mass.

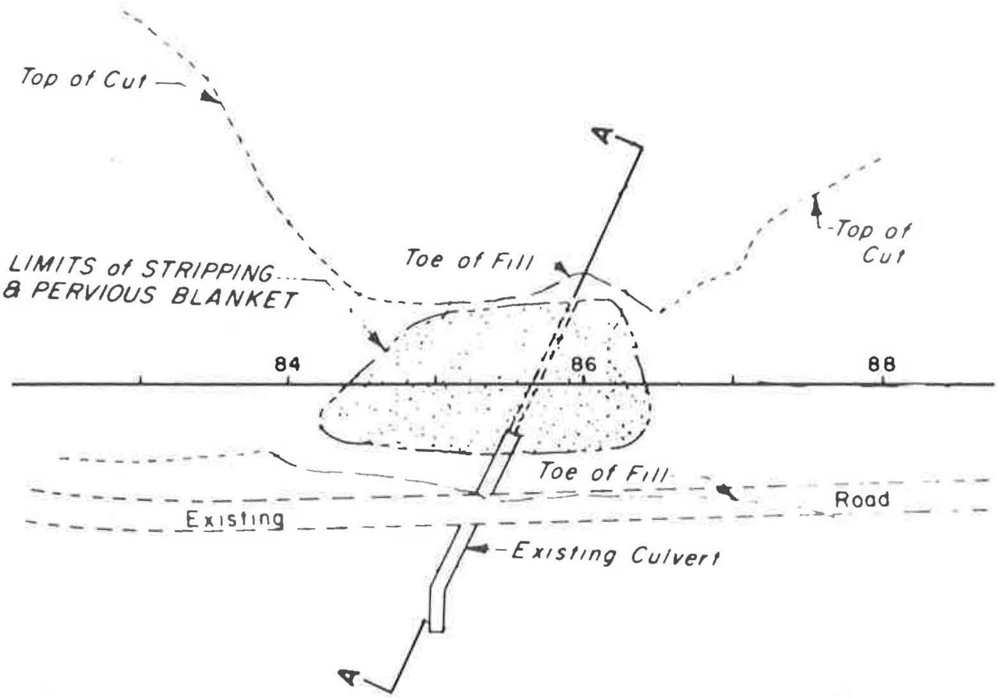
It is apparent that the soil conditions and types of formations described previously do not lend themselves readily to rigorous solutions by the usually accepted soil mechanics procedures. Rather, the design of subsurface drainage facilities under these conditions is largely a matter of using the information available from all sources such as field review, geology, borings, tests, and analyses, as an aid to experience and judgment in the design of these facilities. More rigorous soil mechanics procedures are used assuming certain parameters in checking the designs that are indicated by the rational methods previously described.

METHOD OF DRAINAGE

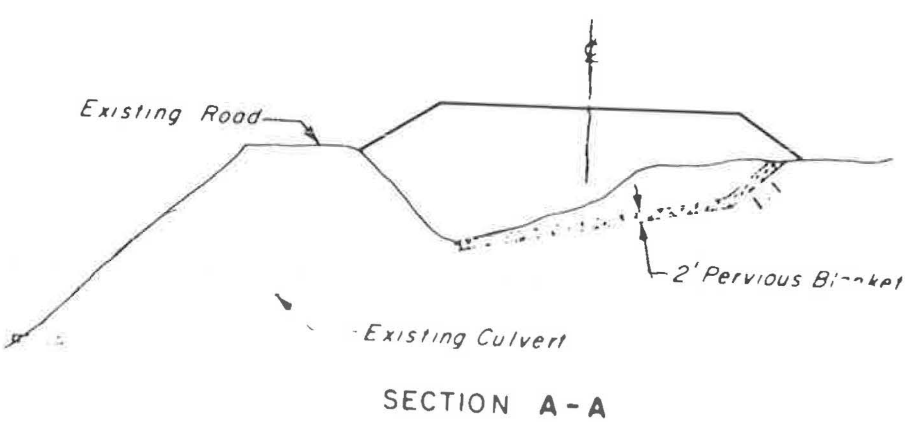
Several methods of drainage or treatment are used depending on soil conditions and ground water conditions that prevail. The subsurface drainage methods that are most commonly used to stabilize fill foundations are (a) strip and provide a drainage blanket, (b) construct stabilization trenches, (c) install horizontal drains, and (d) occasionally use relief wells. The most commonly used methods of subsurface drainage in cuts are underdrains and drainage blankets or horizontal drains. The methods used are discussed separately, although in actual practice subsurface drainage at any single location frequently entails a combination of two or more of these procedures.

Stripping and Blankets

If the zone of water-bearing material is fairly shallow, less than 10 to 20 ft, and is



PLAN



SECTION A-A

Figure 5. Stripping and blanketing.

underlain by firmer material, a common method of treatment is to strip the soft material and to provide a pervious blanket to remove the ground water. This procedure is illustrated in Figure 5 which shows a plan and cross-section through an area where stripping and blanketing were accomplished. This procedure serves a dual purpose of removing the wet, weak material and replacing it with material that is compacted and of appreciably higher strength. It also provides a layer of permeable material which serves as a means of egress for the ground water, the primary cause of the trouble. Limiting conditions for the use of this type of treatment would be the depths of the soft, water bearing material, and slopes of the surrounding area which would determine the feasibility of providing outlets for the drainage layer. If this procedure is adaptable to the conditions that exist, it is a relatively positive method of correction. One precaution in its use is to determine by exploration that all of the water-bearing material is actually being removed, and that the stripping does not merely extend to a zone of stronger material that in turn is underlain by weaker, water-bearing material that is the basic source of the ground water.

The nature of the permeable material that is used for the blankets is discussed later. Experience has indicated that it is inadvisable to provide a pervious blanket without also providing a perforated pipe to remove the water from the pervious blanket.

Stabilization Trenches

Stabilization trenches to remove the subsurface water were used extensively on the two contracts under consideration. The method of exploration where stabilization trenches are used would be identical with the exploration that would be necessary if stripping and pervious blankets are used. In fact, stabilization trenches might be considered a special type of stripping operation. Much of the credit for the development and early use of stabilization trenches should be given to A. W. Root who was for many years head of the Foundation Section of the Materials and Research Department (2). He retired from service with the California Division of Highways in April 1962. A plan of a longitudinal stabilization trench is shown in Figure 6 and a cross-section of a similar trench is shown in Figure 7. Trenches may be either longitudinal or transverse, depending on the terrain and the relationship of the topography to the roadway. Stabilization trenches have been used most extensively where subsurface water is encountered in the exploration at depths between 10 and 30 to 40 ft below the existing ground. Trenches have been generally constructed with a bottom width of 12 ft or more and with side slopes as steep as they will stand during construction. The bottom width of 12 ft or more is largely predicated on the use of usual dirt-moving equipment for excavation. The side slope can generally be constructed somewhat steeper than it is anticipated would be possible for permanent construction, because in the normal construction operations trenches will be constructed and backfilled within a few days to a few weeks. Typically, on the two contracts under discussion, side slopes on the trenches were in the order of 1:1 to 1½:1. Figure 8 shows the completed excavation of a longitudinal stabilization trench. This same stabilization trench is shown in Figure 9 during placement and compaction of backfill material. Some slides within the trenches occurred when side slopes of 1:1 were used especially in cuts of 20 ft or more in height. Almost no difficulty with slides was encountered where 1½:1 side slopes were used even though the slopes were high (50 to 100 ft or more). Maximum depth at centerline of the trenches on these two contracts was in the order of 25 to 35 ft. Thus, slopes on the low side of the trench were in the order of 10 to 25 ft, and on the high side, slopes in the order of 50 ft or more were not uncommon. Generally the bottom, high side and ends of the trenches were blanketed with a layer of 3-ft thick permeable material. One or more perforated pipes were placed in the bottom of all of the trenches to remove the water from the stabilization trench. An outlet was provided to remove the water from the lower end of the trench. These outlets in reality usually constitute a short transverse stabilization trench. A diagrammatic sketch of a stabilization trench is shown in Figure 10.

Trenches will effectively remove the subsurface water from an area if they can be constructed deep enough to intercept the water-bearing strata. They do not have the

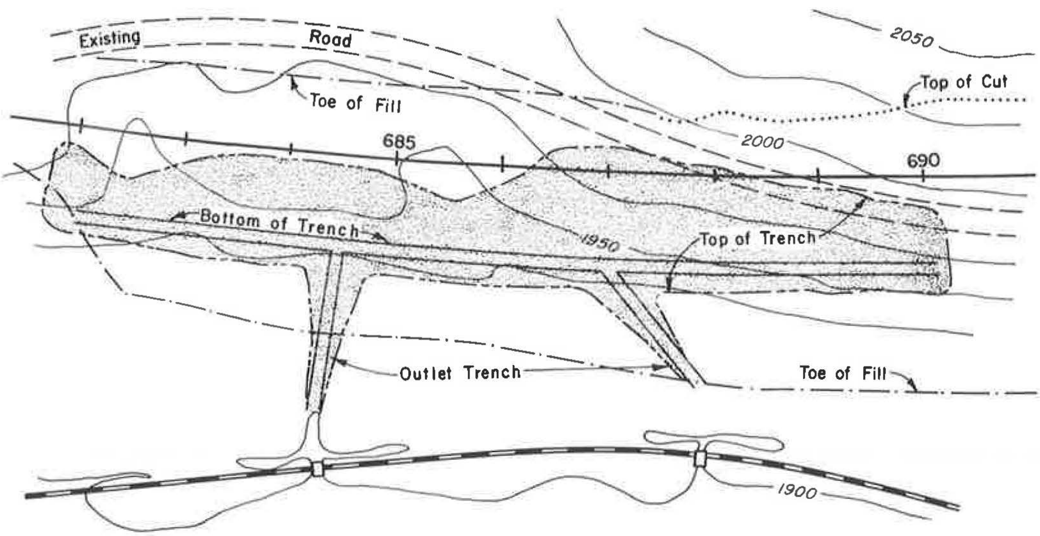


Figure 6. Plan of longitudinal stabilization trench.

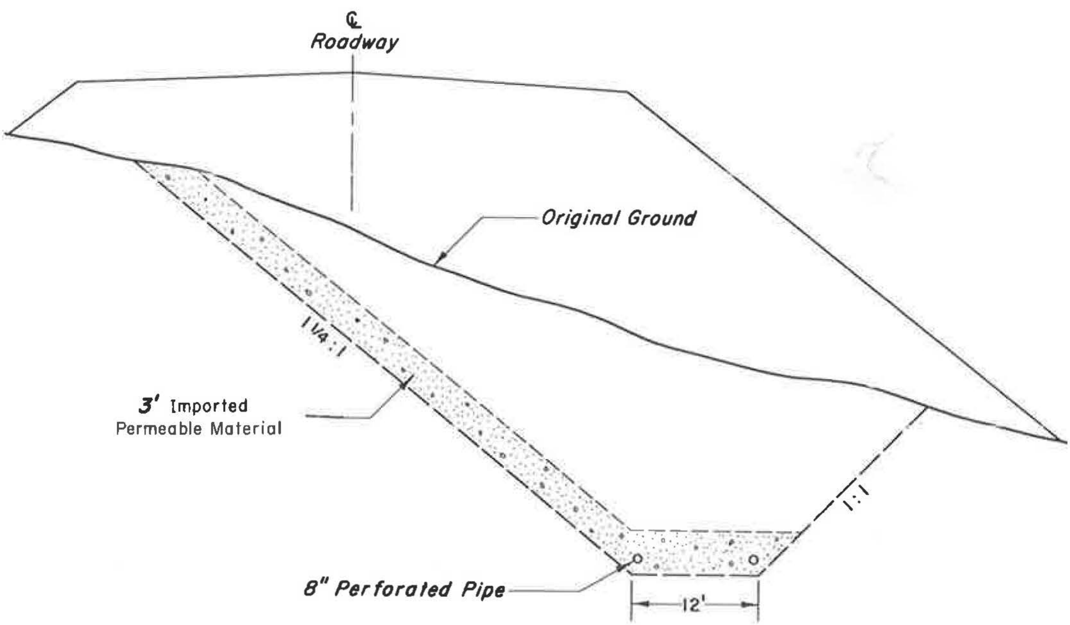


Figure 7. Cross-section of longitudinal stabilization trench.

Figure 8. Excavated longitudinal stabilization trench.

Figure 9. Partially filled longitudinal stabilization trench.

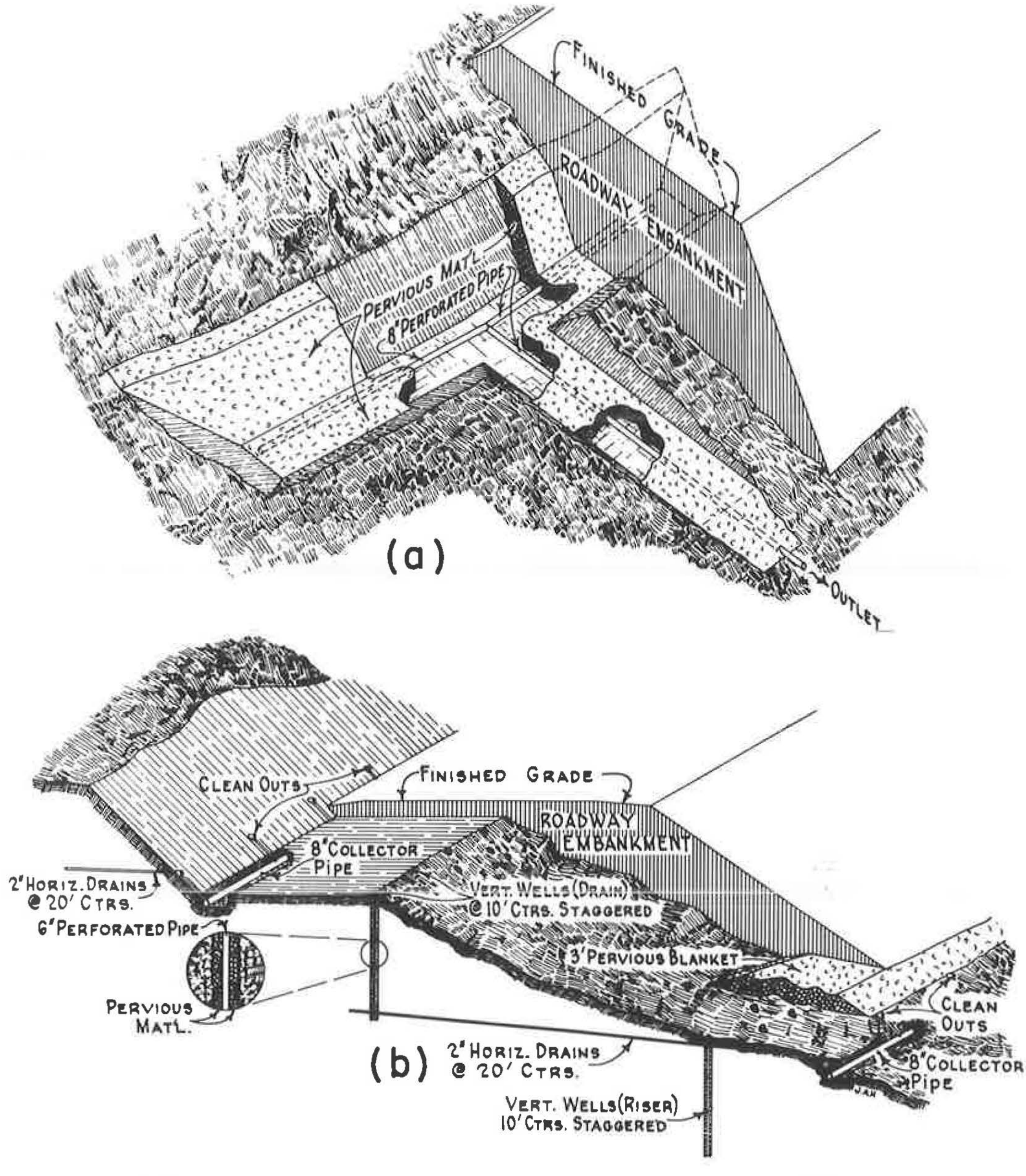


Figure 10. (a) Diagrammetric sketch of stabilization trench, and (b) diagrammetric sketch of embankment stabilization with horizontal drains and vertical relief wells.

feature of removing as great a percentage of the underlying weak material as is usually possible if stripping is used. Reference to the cross-section shown in Figure 7 will indicate that, due to the narrow bottom width of the stabilization trench, considerable weak material is left in place under the roadway prism. These stabilization trenches do have the feature of being able to intercept ground water at a greater depth than is often economically practical with total stripping. They provide a wedge of stable compacted material that is keyed into the stable underlying foundation soil.

Horizontal Drains

Although horizontal drains are most frequently used in connection with stabilization

of cut slopes or landslide correction they are occasionally used as a preventative measure on fill foundations where subsurface water is a problem. They are usually installed from the toe of the proposed fill slope or some convenient position at a lower elevation. This will afford access for maintenance. It is sometimes possible to remove subsurface water by this method to depths greater than is economically practical by the use of stripping or trenches. Drains are frequently installed to depths varying from 150 to 300 ft. Thus, is is often possible to reach well beyond the toe of slope on the upper side of the road with drains that have been installed from the lower toe of slope. Drains are sometimes installed from the upper toe of slope to attempt to remove subsurface water before it has an opportunity to reach the foundation of the embankment. Drains for cut stabilization are usually installed from the toe of cut or from benches on the cut slope. Grades for the drains vary from 2 or 3 percent to as steep as 15 to 20 percent. The installation of horizontal drains was pioneered by the California Division of Highways in the later 1930's, and is described in considerable detail in earlier publications (2, 3, 4, 5, 6).

The holes are drilled with 3- to 4-in. diameter roller rock or drag bits with water used as a circulating medium. Casing consists of 2-in. perforated steel pipe that has been asphalt dipped. Casing is butt welded on installation. Collecting systems to remove the water from the outlet of the horizontal drains and to prevent its infiltration into the surrounding soil are generally necessary. Horizontal drains are an effective means of removing subsurface water with proper soil conditions. They increase the strength of the material by removal of the water and reducing hydrostatic pressure.

Relief Wells

Something of an innovation was used for subsurface drainage purposes on the two contracts covered by this paper. Ground water was at such a depth in many of the foundation areas that it was not practical to construct trenches deep enough to intercept the water. Vertical relief wells were installed and horizontal drains were drilled from the lower toe of slope to intercept these relief wells at some elevation between the ground surface and the bottom of the relief wells. These relief wells were in the order of 40 ft deep and were approximately 24 in. in diameter. They were drilled with a bucket-type auger and were not cased. Six-inch diameter perforated transite pipe was placed in the center of these wells and the concentric area between the pipe and the wall of the well was backfilled with permeable material. These relief wells were installed in two rows on 10-ft centers (Fig. 11). The line of wells underneath the main part of the fill did not have a layer of permeable material placed at the ground surface,

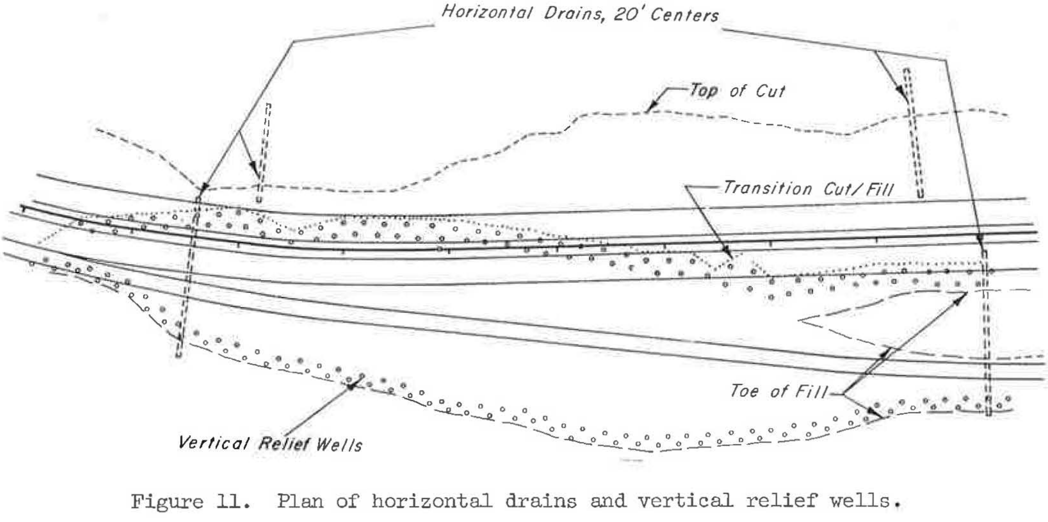


Figure 11. Plan of horizontal drains and vertical relief wells.

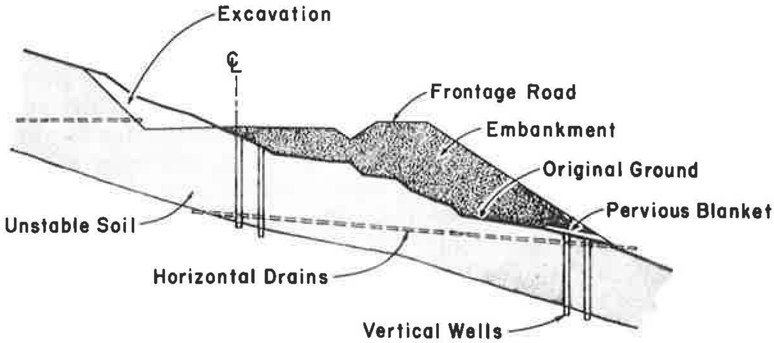


Figure 12. Typical section of horizontal drains, vertical relief wells, and pervious blanket.

but the lower part of the wells was drained by horizontal drains. However, the line of wells near the lower toe of slope had a pervious blanket of permeable material 2-ft thick placed over the original ground, but had no outlets from the bottom of the wells (Fig. 12). The spacing and diameter of the relief wells were such that slightly more than 50 percent of the horizontal drains would intercept relief wells. Apparently it was not necessary to intercept the wells because the subsurface water would flow through the native soil and relieve the pressure within the wells. In some areas these vertical relief wells were installed in the bottom of stabilization trenches. A diagrammatic sketch of vertical relief wells and horizontal drains is shown in Figure 10.

These vertical relief wells are primarily a means of relieving hydrostatic pressure. They do not effectively drain all of the water from the soil mass.

Underdrains and Pervious Blankets

A method commonly used for removal of subsurface water in highway cuts is by installation of underdrains or underdrains in combination with pervious blankets. A typical section of an underdrain is shown in Figure 13. The underdrain consists essentially of a narrow trench, 20 in. wide and 2 to 8 ft deep. A 6-in. layer of permeable material is placed in the bottom of the trench and perforated pipe 8 in. in diameter is then laid. The remainder of the trench is then filled with permeable material. These underdrains are installed primarily to intercept water that is flowing laterally into the roadway through rather well-defined zones. Underdrains are generally installed under one shoulder in cuts. They are occasionally installed at both shoulders and in the median if conditions warrant. Underdrains have been beneficial in areas where the subsurface water is flowing through the entire mass of soil and may be moving into the cut from underneath. However, they are not necessarily completely effective under these conditions and their effectiveness would depend largely on the quantity and source of water involved and the uniformity of the soil mass. Generally, under these conditions a pervious blanket is used in combination with a system of underdrains. In highway practice in California, the minimum thickness of blanket that is used is 1 ft, and this blanket is used in combination with one or more underdrains. Usually no pipe is installed in the blanket proper but the pipe in the underdrains serves as an outlet for the water that is picked up by the blanket as well as the water picked up by the underdrain. Transverse or diagonal underdrains are frequently installed at the transition from cut to fill. This tends to prevent the migration of the subsurface water from the cut areas into the adjacent fill areas. The primary purpose of underdrains as well as the pervious blankets in cuts is to prevent distress of the pavement rather than to improve stability in the cuts.

Permeable Material

No discussion of methods of subsurface drainage would be complete without some

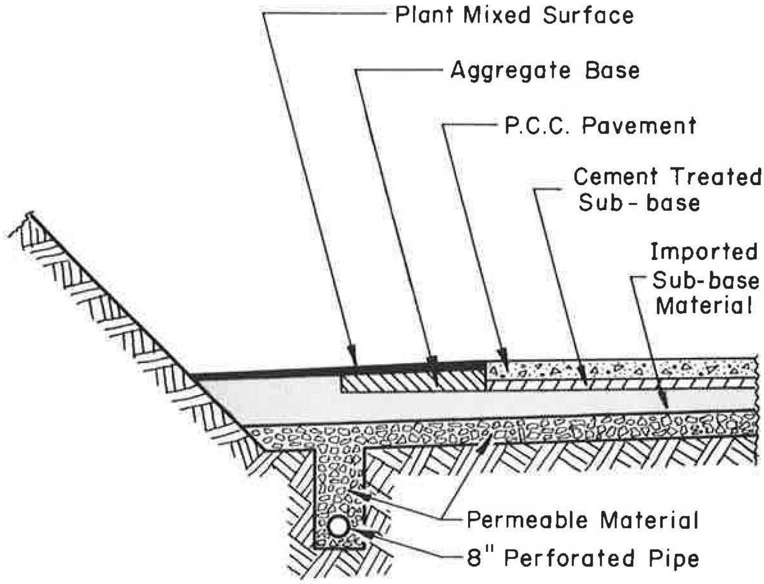


Figure 13. Typical section of underdrain and pervious blanket.

consideration of the characteristics of the permeable material that will be used to drain water from the unstable areas. In the process of stripping and blankets, stabilization trenches, relief wells, and underdrains, permeable material is provided to serve as a means of removing the subsurface water. Perforated pipe of some sort is used along with the permeable material as outlets in these various installations.

Permeable materials for drainage purposes should have two characteristics that are somewhat contradictory. First, the material must be many times as permeable as the surrounding soil from which water is to be drained; hence, the desirability for the material to be very porous and capable of carrying large quantities of water. Second, the permeable material should not contain voids sufficiently large to permit the migration of the soil into the permeable material or to permit the migration of the fine portion of the permeable material through the coarse phases of the permeable material.

The history of the grading of permeable material that has been used for subsurface drainage purposes in California highways shows trends that appear to be characteristic of the thinking both in the Division of Highways and in other agencies that have made similar installations. Two or three decades ago permeable material usually consisted of rock or cobbles with very large voids. Each successive step in the process shows grading specifications that used smaller and smaller sizes. With the California Division of Highways this trend was reversed in 1960 when coarser material or at least cleaner material was required than had been used in 1954 specifications. Table 2 gives grading specifications at various times from 1927 to the present time. Typical specifications are shown on the grading curves in Figure 14.

TABLE 2

GRADING SPECIFICATIONS OF PERMEABLE MATERIAL

Year	Sieve Size (% pass.)														
	6 In.	2½ In.	2 In.	1½ In.	1¼ In.	1 In.	¾ In.	⅜ In.	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
1927-40	100					0									
1940-45	100						0								
1945-54		100					40-100		15-50	5-30		0-5			0-2
1954-60		100		80-100			60-95		35-65	25-50		5-25			0-3
1960:															
Coarse portion			100	90-100			60-80		40-60						
Fine portion									100	65-90	45-70	20-40	8-16	0-4	0-2
Proposed						100	90-100	40-100	25-40	18-33		5-15	0-7		0-3

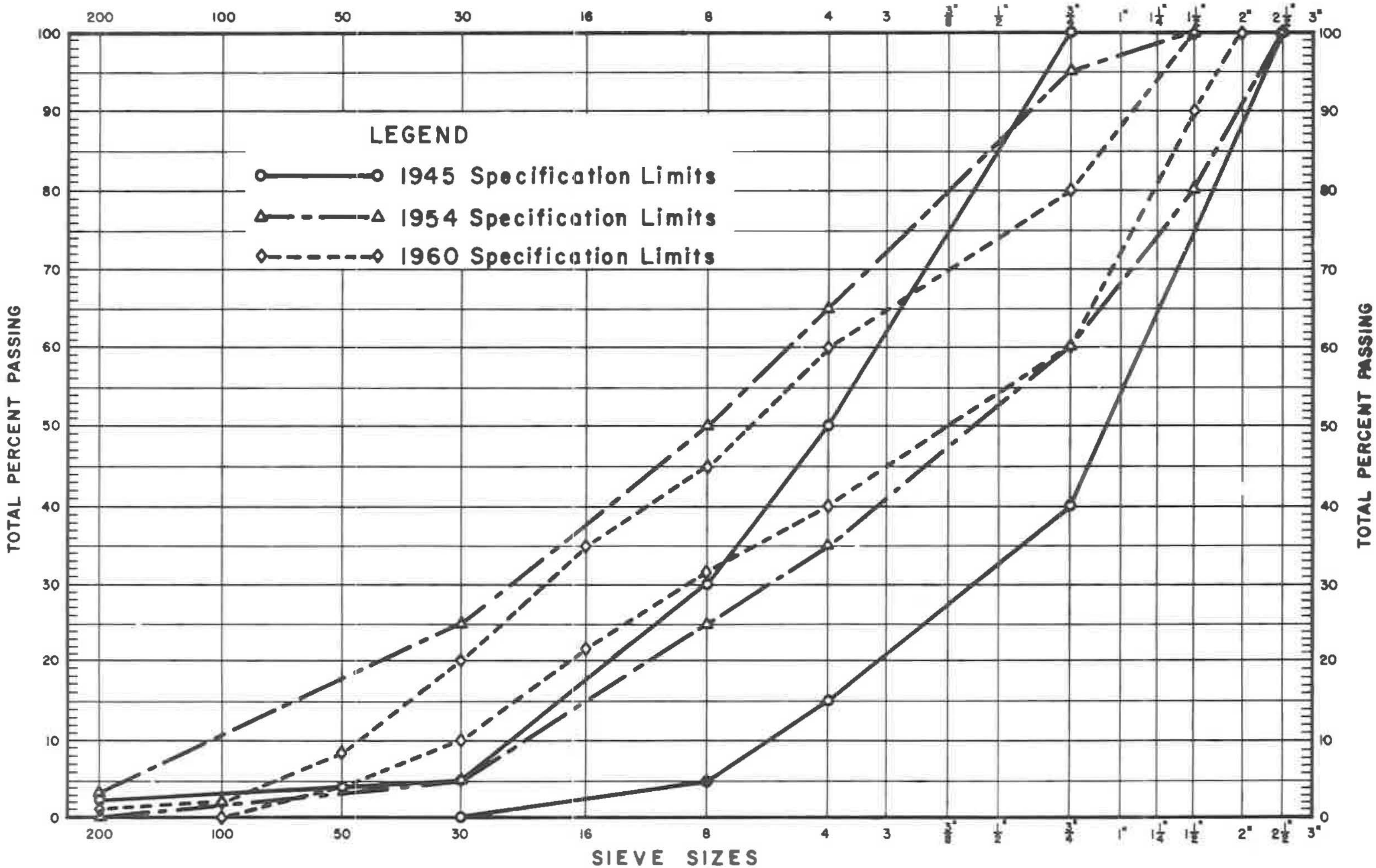


Figure 14. Grading specification limits for permeable material.

It is recognized that permeability rather than grading should probably be the basic criterion for specifying permeable material. Permeability tests are difficult tests to use as a construction control test. Although permeability tests have to a large extent become standardized, they are not highly reproducible except by skilled technicians with considerable experience. Therefore, the procedure has been to use grading and quality tests on the aggregate in the preparation of specifications for permeable material rather than using permeability. It is also recognized that the permeable material should be tailored to fit the soil encountered in the various cuts and stabilization work (7). In highway construction this is usually difficult due to the extreme variation of material encountered in the various cuts and subsurface drainage features. Since 1960, an effort has been made to base the specifications for the permeable material on the character of soil to be drained somewhat in conformity with the general requirements advocated by Barron (8) and Barber (9).

Most of the permeable material used in the construction of California highways has been secured from commercial sources; however, every effort is made to use local deposits where suitable material is available and soil conditions are such that it can be used.

SUMMARY AND CONCLUSIONS

Provision for adequate control of ground water in highway construction is imperative. Experience on California highways in general, and particularly on the two projects described herein, indicates that extensive subsurface drainage was necessary to prevent the expenditure of far larger sums of money for corrective measures.

The need for thorough investigation in connection with subsurface drainage cannot be overemphasized. If thorough investigation is not made, the need for subsurface drainage cannot be ascertained. Similarly, the investigation will point to the type and degree of subsurface drainage that is necessary. The investigation for subsurface drainage may take many forms, and the nature of the problem encountered should dictate the type and nature of the investigation that is warranted.

The two contracts described illustrate the types of subsurface drainage that are commonly or on occasion used in California highway practice. There is every evidence that these subsurface drainage methods are reasonably successful. The fact that considerable additional subsurface drainage was determined necessary, as a result of observations during construction, is evidence that the original subsurface drainage was not overdesigned. On these two contracts no embankment slipouts of major proportions have occurred since construction. Some slides have occurred, but these have for the most part been rather minor slides that have obstructed not more than one or two lanes of the freeway.

If proper subsurface drainage is not provided in areas where it is needed, the results may vary from disastrous to minor inconvenience. There have been cases where major highways have been closed as a result of slides or slipouts that could have been prevented by more extensive subsurface drainage. In other cases minor inconvenience has occurred such as the necessity of the removal of small quantities of slide debris from the traveled way thus reducing highway capacity to some degree. Although it is recognized that the cost of subsurface drainage is an item of major proportions on much of the California highway system, it is strongly believed that the expenditure of these sums of money actually results in savings far beyond their cost and produces highways that are far more serviceable to the traveling public.

It should be emphasized that the subsurface drainage facility is no better than the construction and material that is incorporated in the facility. Proper investigation and adequate design will not insure a workable, satisfactory facility unless the features are incorporated in the finished product. One feature that should be particularly emphasized is the necessity of good-quality permeable material. The permeable material is the device that must ultimately remove the subsurface water from the natural soil; hence, the need for material that is sufficiently permeable to remove the ground water without developing excess hydrostatic pressure and at the same time will not become clogged with fines and cease to function.

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Presplit Blasting Procedures, Bull Run Steam Plant, Clinton, Tennessee

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Geologic conditions and design requirements at the Bull Run Steam Plant presently under construction by the TVA 13 mi west of Knoxville, Tenn., dictated close control of blasting procedures to obtain safe and economical excavations. Various techniques of presplit blasting were used in the initial stage of all major excavations and other specialized loading and detonation patterns were used in later stages.

Hole diameters for presplitting varied from 1½ to 9 in., and hole spacing ranged from 12 to 33 in. on centers. Charges varied from ½ lb to 175 gr per lin ft of hole. For both 9-in. diameter holes and the 2¾-in. diameter holes, the cost of materials was less than $0.10 psf of face developed; however, drilling and labor costs were considerably higher for drilling and loading the larger diameter holes.

Using the presplit method, clean vertical walls up to 70 ft in height were obtained in moderately dipping, though complexly sheared and jointed argillaceous limestone. As the work progressed, the presplit program was modified to meet the varying geologic conditions and job requirements.

This work was done on a production basis. Therefore, refinements and experimentation normally carried on in a research program could not be fully utilized. It is felt, however, that the methods developed on this project could have useful applications on other major projects and in highway construction.

•SINCE ITS DEVELOPMENT in 1958, presplit blasting has become an accepted method of obtaining essentially smooth, vertical walls in rock excavation. The usual technique has been to drill a line of vertical blast holes spaced from 1 to 2 ft on centers, generally to the depth of the required cut. These holes are loaded with charges made up of 1¼- by 8-in. cartridges of 40 percent strength explosive spaced at intervals of 12 to 24 in. along a length of 50-gr detonating fuse. A slightly heavier charge is customarily placed at the bottom of the hole, whereas the charge is somewhat reduced near the ground surface. Explosives usually are not attached to the detonating fuse closer than 2½ ft from the top. After filling with sand or stone chips, the holes are detonated simultaneously, thereby splitting the rock in the plane of the drill holes. Often a crack so developed will be several inches wide at the ground surface, evidence of the slight displacement of the rock near the surface. Drilling and blasting to fragment the rock to be removed then follows.

The Tennessee Valley Authority is presently constructing the Bull Run Steam Plant, 13 mi west of Knoxville, Tenn. (Fig. 1). Geologic conditions at the site, together with design requirements, dictated close control of blasting to obtain safe and economical

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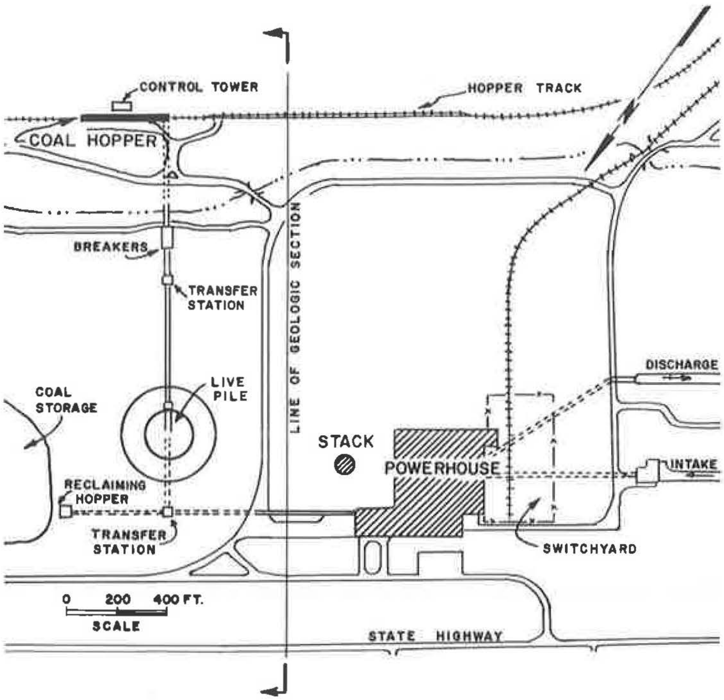

Figure 1. Generalized plan of plant structures.

excavations. Presplitting was stipulated for this project to help provide smooth rock walls to serve as forms in concreting, as well as to reduce overbreak.

The Bull Run site occupies a northeastward trending valley approximately 2,000 ft wide, flanked by ridges which rise 300 to 400 ft above the valley floor. This valley is underlain by a stratigraphic thickness of 1,800 ft of thin-bedded Middle Ordovician Chickamauga limestone and siltstone which dip to the southeast at an average of 30 deg (Fig. 2). Along the ridge to the southeast, the top of the Chickamauga is truncated by the Copper Creek fault, a major Valley and Ridge structure, which in this locality has thrust the Lower Cambrian Rome sandstone and shale over the Middle Ordovician limestones. Minor shears and thrusts associated with the Copper Creek fault are common in the underlying Chickamauga formation, and these usually dip southeastward at a steeper angle than the bedding.

Before start of construction, 363 core holes totaling 24,600 lin ft were drilled in two exploration programs. The first program developed the suitability of the site and the second delineated the foundation conditions at the locations of major structures. Analysis of the drilling data indicated that special methods would have to be used and certain precautions taken in excavation to maintain stable cuts in the moderately dipping and complexly sheared argillaceous limestones and siltstones. The preliminary orientation of the powerhouse was turned 90 deg so that a dip slope would occur only across the northwest end of the first unit and would not be a factor in the construction of any succeeding units; even so, excavations for the intake and discharge conduits and for the coal-handling facilities could not avoid dip slope faces.

New procedures for presplitting were developed to utilize holes of various diameters, as the availability of drilling equipment largely determined the size of the hole used. These varied from 1½ up to 9 in. in diameter.

The two main rock excavations were for the powerhouse and for the coal-unloading facilities. A low ridge at the powerhouse site was excavated to the general plant grade.

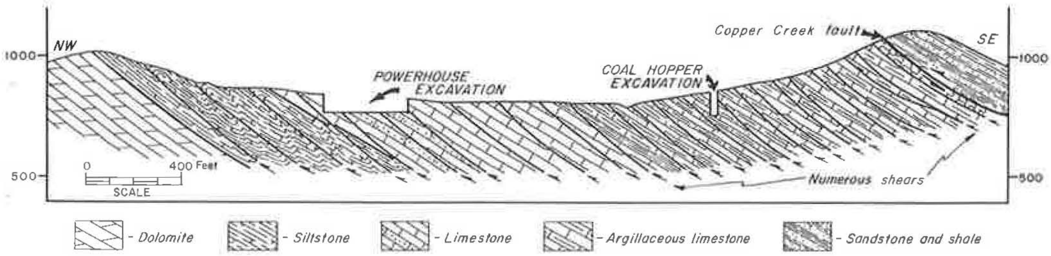

Figure 2. Generalized geologic section, Bull Run steam plant.

The average depth of the excavation from this level was 40 ft. A width of 175 ft and a length of 375 ft defined the rock excavation of 100,000 cu yd. Vertical rock walls designed for the powerhouse excavation made an access ramp necessary, also having presplit walls (Fig. 3).

The general practice in the powerhouse area was to drill the presplit holes to the full depth of the desired wall. Holes were centered on various spacings in a line 6 in. behind the desired neat line. The procedure first used was to drill $2\frac{3}{4}$-in. holes spaced on 12-in. centers with only every other hole charged with explosives. Following the customary practice, the presplit charges were made up in the field (Fig. 4). A length of detonating fuse about 18 in. longer than the length of the hole to be loaded was strung out on planks at waist level for convenience. One-half pound of explosive in the form of a $1\frac{1}{4}$- by 8-in. cartridge of 40 percent gelatin was secured to the end of the detonating fuse which would be at the bottom of the hole. Half cartridges were secured at 12-in. intervals along the fuse. Charges near the top were reduced to one-third of a cartridge and stopped 4 ft from the end. Charges thus assembled were lowered into the hole followed by stemming of $\frac{3}{8}$-in. stone chips or sand. The detonating fuses from the loaded holes were attached to a trunkline of fuse, and the shot fired by electric blasting caps secured to the trunkline. Even these relatively light charges occasionally produced slight fractures extending into the wall rock behind the charged sections of the drill holes. Presplit results were satisfactory along the butt slope and end walls. However, rock on the dip slope was often displaced along one of numerous moderately dipping shear planes, either by presplitting or by the excavation blasting that followed.

Because of the availability on the job of a large rotary drill, 9-in. diameter

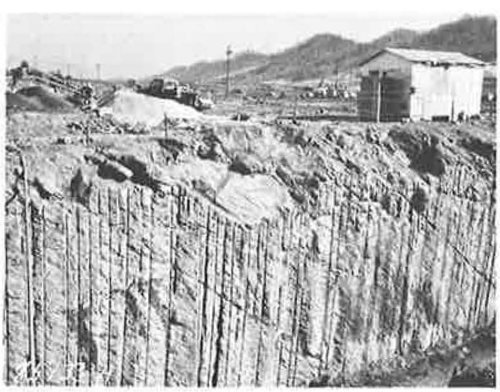

Figure 3. Presplit face parallel to dip of rock strata in powerhouse access ramp.

Figure 4. Assembly of presplit charges.

holes were introduced into the presplit program. At the powerhouse site, these large diameter holes were centered 6 in. behind the neat line and spaced 33 in. center-to-center. A $2\frac{3}{4}$-in diameter hole was drilled halfway between adjacent 9-in. holes. Only the $2\frac{3}{4}$-in. holes were loaded; the 9-in. diameter holes were left open. Results from this method were comparable to those obtained when all holes were $2\frac{3}{4}$ in. in diameter.

As the excavation program uncovered sections of the rock walls that had been presplit by procedures so far described, observations of the drill hole scars indicated that the practice of loading only every other hole in line should be eliminated. The scars showed that the plane of presplitting was determined by the axes of the holes that had been charged. Often, if the unloaded hole between two charged holes was out of the plane established by these two holes, the resulting presplit surface did not deviate from this plane and bypassed the off-line hole. If an unloaded hole drifted a distance equal to or greater than its diameter, only that section of the hole circumference within the established plane was split. Thereafter, every presplit hole was loaded; charges or spacing were altered to obtain the maximum results.

In the interest of better control and safety, 1- by 4-in. cartridges of explosives were purchased, replacing the $1\frac{1}{4}$- by 8-in. sticks originally used. When cutting and using portions of the 8-in. cartridges, small particles fell from the open end. The 1- by 4-in. cartridges eliminated the hazard associated with the accumulation of loose explosive on the ground in the work area. It also made the use of reduced charges more practical, as the 1- by 4-in. cartridge is about equal in volume to one-third of a $1\frac{1}{4}$- by 8-in. cartridge. The increase in the cost per hundredweight of the smaller cartridge can be balanced by using less expensive semi-gelatin explosives as the cartridges are left intact.

In using $2\frac{3}{4}$-in. diameter holes and loading every hole, the spacing between hole centers varied from 1 to 2 ft, depending on the depth, location, and rock conditions. Good results were obtained when charges were assembled with 1- by 4-in. cartridges secured to 50-gr detonating fuse at intervals of 18 in. for the holes spaced on 1-ft centers, and at 12-in. intervals for holes spaced at 2 ft.

Experience, at this project and others, has shown the tendency for the development of fractures radiating from the presplit blast hole into the wall rock. These radial fractures are induced by the detonation of the charges on the presplit "string." Furthermore, under favorable geologic conditions, rock movement or destruction of the bond along joint or bedding planes can take place with the normal loading used in the conventional presplitting method.

To prevent such effects under the adverse rock conditions known to exist at the site of excavation of the coal-unloading hopper, a method was required that would minimize movement along shear, joint, and bedding planes, as well as the fracturing of the rock behind the presplit line. Before start of excavation in the hopper area, a detonating fuse of 175 gr of explosive per lin ft was successfully tested as the only explosive in presplitting the shallow excavations for the cable tunnel and for the circular chimney foundation. Manufactured for use as a high-strength detonator of relatively insensitive explosive mixtures, this fuse, 0.43 in. in diameter, contains 3 to 4 times more explosive than does conventional fuse.

This detonating fuse used without attaching other explosives has the advantage of an even, continuous charge the full length of the blast hole. The energy is not concentrated at the intervals of attached explosives. In practice, this eliminates the radial fractures occasionally induced in the back of the hole at the charge intervals. Because of the reduced charge—437.5 gr = 1 oz—the chance of rock movement is greatly lessened.

The rock excavation for the coal-unloading hopper was the most difficult encountered at Bull Run. It required a trench 360 ft long, 27 ft wide, and up to 70 ft deep (Fig. 5). This vertical-walled excavation was nearly parallel with the strike of the rock strata in the dipping and sheared argillaceous limestone, 400 ft below a major overthrust fault (Fig. 2).

Two 25-ft wide ramps for the conveyor tunnels enter the hopper at right angles from up dip on a 22 percent descending grade. Both conveyor ramps are designed with vertical walls. One ramp enters the hopper at the extreme southwest end, and the second joins the excavation nearly in the center of the 360-ft length. These parallel ramps,

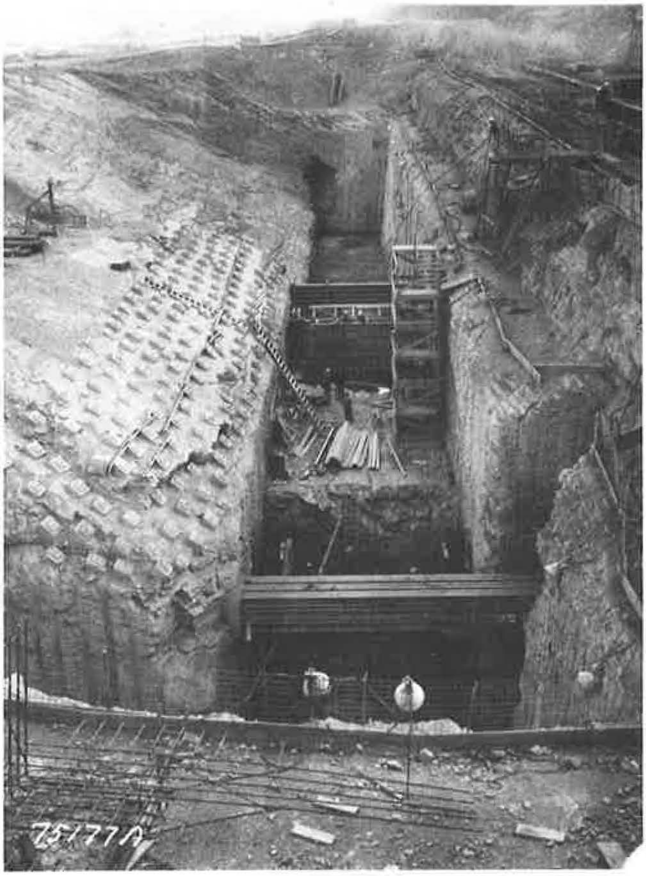

Figure 5. Presplit faces parallel to strike of rock strata in coal hopper excavation.

each approximately 240 ft long, isolate a mass of rock and overburden on the northwest slope of the hopper, 134 ft wide and 70 ft high. The slope is underlain by numerous shear planes at angles of 30 to 45 deg dipping into the hopper excavation. Positive, protective measures were taken to insure stability of the dip slope but these are not within the scope of this paper.

In presplitting the excavations for the unloading hopper and conveyor tunnels, both $2\frac{3}{4}$-in. and 9-in. holes were used. All holes were started 6 in. behind the desired line and drilled the full depth of the rock wall. The deepest $2\frac{3}{4}$-in. hole presplit was 70 ft, whereas 50 ft was the greatest depth for a 9-in. hole.

The necessary split without displacement or radial fracturing was obtained along the northwest and southeast face of the hopper, and in the upper end of the conveyor excavations using $2\frac{3}{4}$-in. holes. Although this size hole was drilled initially up to the full 70-ft depth, presplit loading did not exceed 35 ft. Many of the holes were blocked by loose rock and mud which could not be cleared with compressed air for the total depth. When the upper 35 ft of rock was removed, the presplit holes were reopened and loaded the remaining depth.

Along the northwest and southeast walls of the hopper, a network of intersecting, high-angle, recemented shear planes was potentially hazardous if disturbed. Here, holes were spaced on 12-in. centers, and charged by lowering a strand of 175-gr detonating fuse, doubled for 5 ft at the bottom end.

Equally good results were obtained along the conveyor walls with $2\frac{3}{4}$-in. diameter holes spaced up to 2 ft on centers and charged the full depth with two strands of 175-gr fuse. The strands were kept from spreading by taping at 6-ft intervals (Fig. 6).

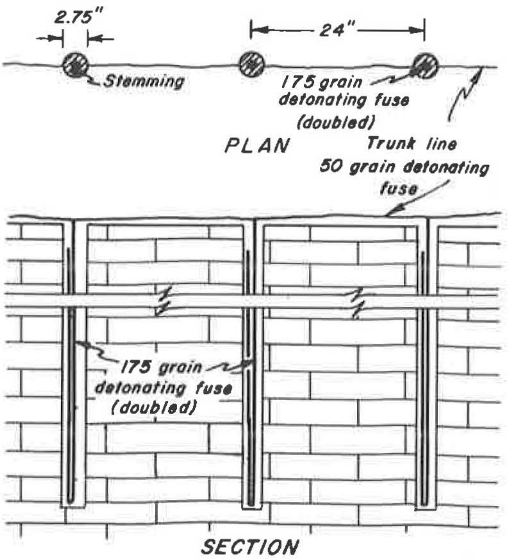

Figure 6. Presplitting with high-strength detonating fuse.

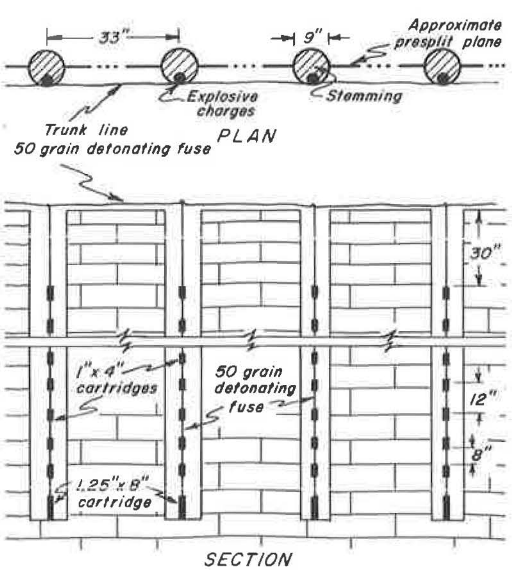

Figure 7. Presplitting with "off-center" loading in large diameter holes.

Holes of 9-in. diameter had been used in presplitting at the powerhouse site where alternated with $2\frac{3}{4}$-in. diameter holes. In the excavation for the southwest end of the hopper and sections of the conveyor conduit walls a different presplitting technique using only 9-in. holes was developed. These holes were drilled in line on centers up to 33 in. The presplit charges were 1- by 4-in. cartridges of 40 percent strength taped to 50-gr detonating fuse at 12-in. intervals with a $1\frac{1}{4}$- by 8-in. cartridge at the bottom. The explosive string was placed along the side of the 9-in. hole toward the excavation side of the presplit line, and the hole was then filled with stemming (Fig. 7). As the 9-in. rotary-drilled holes were plumb, no difficulty was experienced in being able to load in this manner. This technique has the advantage of giving direct contact of the explosive with the rock to be removed. At the same time, the side of the hole that adjoins the rock surface to be preserved, is protected by a thickness of stemming material nearly equal to the diameter of the hole. Good presplit surfaces resulted and the plane of rupture still extended approximately through the center of the holes.

To retain sound rock walls in the final excavation, care must not be limited only to presplit blasting. At Bull Run, emphasis was placed in planning the drilling and millisecond delay excavation blasting, not only to fragment the rock for economical handling, but also to reduce the amount of energy transmitted across the presplit plane to a point where damage would not result to the wall rock. The blast patterns and criteria for fragmenting the rock to be removed cannot be covered in this paper, but it should be emphasized that without such care the presplit surfaces at Bull Run could not have been preserved.

The best cost figure for materials per square foot of presplit surface blasted, using $2\frac{3}{4}$-in. holes, was \$0.06 compared with \$0.07 for the 9-in. holes. These figures are based on spacing the smaller holes 2 ft on centers and include the cost of a double line of 175-gr detonating fuse and stemming material. The figure for the larger diameter hole is based on 33-in. spacing, center-to-center, and includes the cost of 1- by 4-in. cartridges, 50-gr detonating fuse and stemming material.

Although the cost of materials for both size holes are similar, it can be readily understood that the costs of drilling and assembling charges for the large diameter holes are considerably higher than for the $2\frac{3}{4}$-in. holes. These material costs reflect the light explosive loads used at Bull Run. Naturally, variable conditions of type of rock and the results desired would alter these costs.

It is believed that the presplitting procedures developed at Bull Run could be successfully adapted to rock excavation in highway construction.

At Bull Run 175-gr detonating fuse was used as the only explosive in small-diameter jackhammer holes to presplit footing excavations and sumps; also, in trimming unsound rock from slopes. Similar uses would seem to be applicable in highway bridge foundation construction. This same technique could also be applied to trimming certain areas of unsound rock along existing roadways.

The off-center loading of large-diameter presplit holes would seem to lend itself to use on deep highway cuts. Under conditions less critical than those described, the use of heavier charges might well allow a substantial increase in the 33-in. spacing reported for the 9-in. diameter holes. Greater spacings would reduce the cost of presplitting with these holes and perhaps make fuller use of available drilling equipment at some operations.