Results of Geologic Research at the Straight Creek Tunnel Pilot Bore, Colorado

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Projection of details of surface geology to depth before construction has met with only limited success in many tunneling operations. However, in the research project on the Straight Creek Tunnel pilot bore, good results were obtained by the prediction, based on a statistical study of surface and drill hole features, of the kinds of conditions and their extent, but not their exact locations, that could be expected at tunnel level.

Successful predictions were made regarding percentages of rock types, linear feet of faulted and sheared rock, and attitudes of foliation and fractures, including faults and joints. Predicted rock loads and final swell pressures in gouge and altered rocks agreed well with actual measurements. Groundwater flows occurred in expected amounts, but criteria for estimations proved unsound.

Estimates of the amount of temporary support, footage of feeler holes, and the amount of grouting required provided engineers with a sound basis for estimates of tunnel costs.

•THE U.S. Geological Survey, in cooperation with the Colorado Department of Highways, conducted a research project in engineering geology at the Straight Creek Tunnel site, Colorado, from 1962 to 1966. The purpose of this project was to apply recently developed geologic and geophysical methods and to develop new methods for predicting geologic conditions at the tunnel depth, to present the geologic information in such a manner that it could be used by design and construction engineers, and to evaluate the accuracy of the predictions on construction of a pilot bore.

The preliminary results of the pre-construction investigations, together with the engineering predictions based on that work, were published by the authors (5, 6). Extensive investigations conducted during construction of the pilot bore are summarized here, and geologic conditions and engineering practices predicted are compared with those actually found and used during construction.

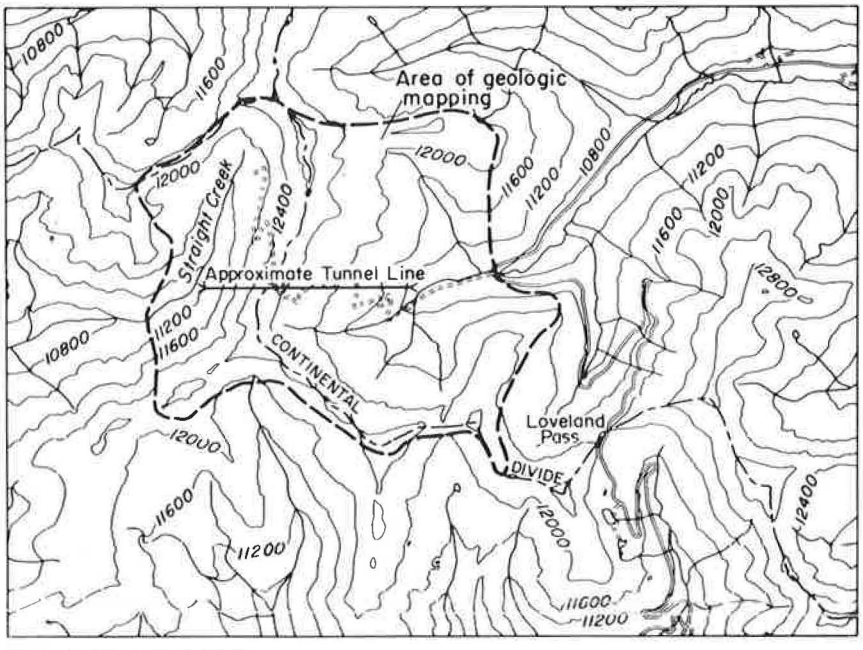
The Straight Creek Tunnel site (Fig. 1) is approximately 55 mi west of Denver. The final tunnel will consist of twin bores, each about 8,300 ft long and 42 ft in diameter. The tunnel, which will be part of I-70, is designed to provide an all-weather route through the Continental Divide and so eliminate the use of Loveland Pass on the present US 6.

SUMMARY OF PRE-CONSTRUCTION INVESTIGATIONS

The pre-construction investigations consisted of detailed geologic mapping of approximately 6 sq mi at the site of the tunnel, geologic and geophysical logging of core holes, geophysical investigations at the surface, and laboratory studies in support of the field investigations (6).

These investigations showed that the bedrock in the area consists predominantly of granite with inclusions of metamorphic rocks—chiefly varieties of biotite gneiss, of

Paper sponsored by Committee on Engineering Geology and presented at the 46th Annual Meeting.

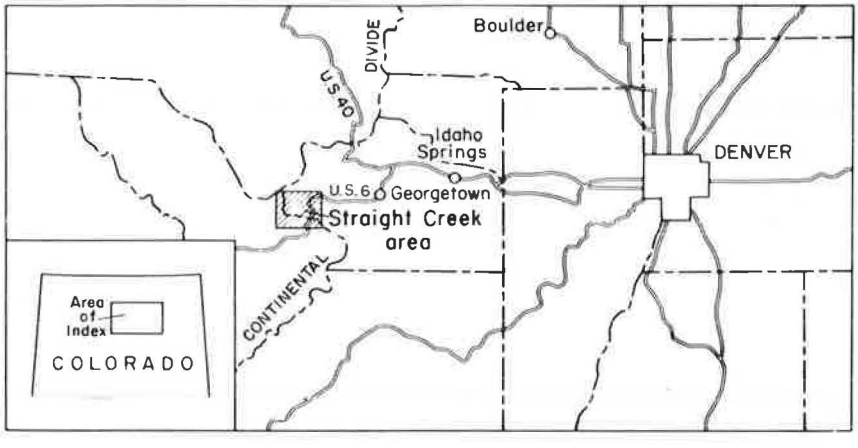


Base map from Loveland Pass quadrangle, 1958

0 1 2 MILES

CONTOUR INTERVAL 400 FEET
DATUM IS MEAN SEA LEVEL

Map of Straight Creek area



0 20 40 MILES

Location of Straight Creek area

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

TABLE 1

PREDICTIONS VS FINDINGS IN STRAIGHT CREEK TUNNEL PILOT BORE, COLORADO

Item	Predictions	Findings
(a) Geologic Measurements[a]		
Rock type (% pilot-bore length)		
Granite	75	75.4
Metamorphic rock	25	23.8
Diorite dikes	[minor]	0.8
Fracture density (% pilot-bore length)		
<0.1 to 0.5 ft	40.1	38.7
0.5 to 1 ft	49.3	42.6
>1 ft	10.6	18.7
Faults and shear zones (% pilot-bore length)	51	49
Faults, principal ranges in trend or attitude (surface, >5 ft wide; pilot bore, >1 ft wide)		
Strike	N. 20°-80° E.	N. 20°-45° E.
Dip	75° NW. or SE.	40°-75° SE.
Joints (principal range and avg. attitude)		
Strike	Any direction	Any direction
Dip	45°-90°	8°-90°
Avg. dip	60°	45°
Foliation (principal range)		
Strike	N.-N. 30° E.	N. 10°-60° E.
Dip	60°-90° SE. or NW.	10°-50° SE.
Statistical maxima		
Strike	N. 15° E.	N. 45° E.
Dip	65° SE. or 70° NW.	30° SE.
Strike	N. 30° E.	
Dip	75° NW.	
Avg. dip	60°	30°
(b) Engineering Measurements		
Rock loads (psf)		
Predicted maximum rock load calculated from Terzaghi (10) on 10.5 × 11.5-ft pilot bore	5900	–
Predicted maximum rock load recalculated from Terzaghi (10) on 13 × 13-ft pilot bore	6970	–
Calculated geometric midpoint for maximum stable geologic rock load from measurements in pilot bore, 13 × 13 ft	–	6600
Avg. final swell pressure (psf) of altered rock and gouge	2233[b]	1727[c]
Groundwater (gal/min)		
Maximum initial flow from any section	1000	750
Maximum flow from portal	500	800
Flow at portal 2 wk after completion of pilot bore	300	130
(c) Construction Practices[d]		
Set spacing (% pilot-bore length)		
1-ft centers	1.6	2
2-ft centers	23	22
3-ft centers	40	30
5-ft centers	35	20.9
Invert struts	1.4	8
Total no. of sets	2691	2172
Total no. of invert struts	113	210
Lagging and blocking (footage in pilot bore)		
100 to 67 percent lagged and blocked	1731	1456
66 to 34 percent lagged and blocked	3659	2447
33 to 0 percent lagged and blocked	2660	4447
Feeler holes (lin ft)	2905	9816
Grout (lin ft of pilot bore)	403	0

[a]Predictions based on 4.0 percent outcrop.
[b]Six samples.
[c]Twenty-nine samples.
[d]Predictions based on pilot-bore length of 8,050 ft; findings based on length of 8,350 ft.

Precambrian age, and a few small dikes of diorite, of probable Tertiary age. Faults, shear zones, and joints are numerous in the tunnel area, which is within a wide zone of regional faulting and shearing that is probably related to the Loveland Pass fault (4, pl. 2).

The pre-construction geologic, geophysical, and laboratory data were compiled and statistically analyzed. These data were the basis for the prediction of geologic conditions

at the depth of the pilot bore and for the calculation of engineering data for estimating the cost of construction (Table 1).

INVESTIGATIONS DURING CONSTRUCTION

During construction of the pilot bore, which began in November 1963 and was completed in December 1964, geological, geophysical, and laboratory investigations were conducted. In addition, rock mechanics investigations were made. The final results of this work are not yet available, but sufficient data were compiled to evaluate the pre-construction geologic projections and predictions. A preliminary report of the results of the investigations is available (7).

Geology

The geology of the pilot bore was mapped at various scales and sampled for various purposes throughout its length. The walls of the bore were mapped at 1:600. On these maps were recorded rock type; attitude of the foliation, faults and shear zones, and joints; percentage of altered minerals in the wall rock; and occurrence of groundwater. Geologic sections of the tunnel face at about 800 stations were made, at a scale of 1:24, by engineers of the Colorado Department of Highways and by the authors. In support of the geophysical investigations and the instrumentation, the geology of one wall or the other was mapped in plan view at 1:60 for 50 ft or more to either side of a geophysical or instrument station.

The wall rock of the tunnel was sampled during the geologic mapping for various laboratory determinations. Systematic samples were collected for petrographic analysis. Samples of altered wall rock and fault gouge were collected for determination of swelling pressures and mineralogy. Linear chip samples from 5 ft on either side of instrument stations were collected for grain size and mineral analyses. Blocks of wall rock about 1 ft in largest dimension were collected and cored in the laboratory for the determination, by dynamic and static tests, of elastic properties.

As the construction of the pilot bore progressed, and as the results of the instrumentation and geophysical investigations became available, a particular effort was made to determine the geologic and engineering conditions influencing these results. It soon became apparent that the categories of fracture density, or the average spacing between fractures (6), as determined from surface mapping, were not definitive enough. At the surface, the fracture density categories mapped were less than 0.1 to 0.5 ft, 0.5 to 1 ft, and 1 to 3 ft. Underground, the fracture density categories mapped were changed to less than 0.1 ft, 0.1 to 0.5 ft, 0.5 to 1 ft, and greater than 1 ft. The instrumentation and geophysical work also dictated the need for mapping the percentage of wall-rock alteration and for sampling for grain size and mineral analyses.

Geophysics

Geophysical investigations were undertaken in the pilot bore to determine whether geophysical instruments and techniques could be used effectively to define the physical conditions around the pilot bore, and whether these conditions could be correlated with the results of the instrumentation and the construction practices used in the pilot bore.

Both resistivity and seismic velocity measurements were made at selected points along the walls of the pilot bore. The apparent resistivity was measured along the wall at 30 stations which corresponded to 30 of the instrument stations. Seismic velocity measurements were made at 5 localities selected to give the best representation of the geologic conditions in the pilot bore. The instruments and procedures used and the results of the geophysical investigations in the pilot bore are described elsewhere (9).

Groundwater

Groundwater investigations were conducted during the construction of the pilot bore and have been continued on a limited scale since its completion. The results of these groundwater investigations were reported by Hurr and Richards (3).

The groundwater investigations included the recording of water flows near the portal and at different intervals within the pilot bore. Where possible, engineers of the Department of Highways recorded initial water flows at the face and from feeler holes, and also the decrease in rates of flow from the face, from the fractures in the walls, and from feeler holes. The water level in a drill hole at the surface above the tunnel was periodically recorded. One water-pressure measurement was made on water flowing from a feeler hole. The specific conductance of the water in the tunnel was measured at many points, and samples for chemical analysis were taken from different points within the tunnel.

Hurr and Richards concluded, from a preliminary analysis of the groundwater data, that the pilot bore could be divided into active and passive groundwater zones. In the active zones, which were near either portal, the groundwater flows were in direct response to the precipitation and runoff at the surface. In the passive zone, which was the central portion of the tunnel, the groundwater flows were initially large, but they decreased rapidly and were not appreciably affected by the annual precipitation and runoff.

Instrumentation

The Colorado Department of Highways retained a contractor to instrument the pilot bore for the purpose of measuring the loads on the sets and for the determination of strain rates and total strain around the pilot bore.

Two types of instruments were installed, in a total of 41 instrument stations, to make the measurements: electronic load cells and bore-hole extensometers. The load cells were placed between the legs of the sets and the foot blocks, and at a few stations they were also placed in horizontal positions in the crown of the sets and between the legs and invert struts to measure horizontal loads. The bore-hole extensometers, which were of single-anchor and multiple-anchor types, were placed in bore holes, generally 25 ft in depth, drilled into the roof and walls of the tunnel. A detailed description of the instrumentation was presented by Abel (1) and Grosvenor and Abel (2).

In support of the instrumentation, the position of the wooden blocking placed between the instrumented sets and the walls and arch of the pilot bore was mapped at 1:24.

From the data furnished it was possible to calculate the total maximum and stable loads on the sets in pounds, the maximum and stable geologic rock loads in pounds per square foot as defined by Terzaghi (10), the wall and arch deflections in inches, and the height of the ground arch (10, p. 60) in feet and to relate these to the geologic conditions and the engineering practices in the pilot bore (7).

Comparison of Predictions and Findings

One of the main purposes of the Straight Creek project was to evaluate a statistical method of compiling geological information and predicting geologic conditions at depth in the pilot bore. Table 1 gives a compilation of the predictions and the findings of this project. In general, there is a relatively close agreement between most of the predictions and the findings, indicating the validity of the method. The table, however, does not tell the complete story, neither where the prediction and findings are in close agreement nor where they are not in agreement; both cases are important and both need some explanation.

The predictions were based on a pilot bore 8,050 ft long, 10.5 ft wide, and 11.5 ft high, supported by square-set timbers. As a result of a landslide at the east portal (8), the east portal was moved about 150 ft south and the portal grade lowered about 16 ft. This lengthened the pilot bore to about 8,350 ft. Also, during construction, steel rather than timber sets were used for the most part, and the diameter of the pilot bore, outside the steel, averaged about 13 ft. Two types of steel sets were used: 4-in. I-beam weighing 7.7 lb/ft and 6-in. H-beam weighing 25 lb/ft.

Geologic Measurements

The geologic features predicted were the percentage of pilot bore length that would be within the different rock types; the percentage that would be within the different

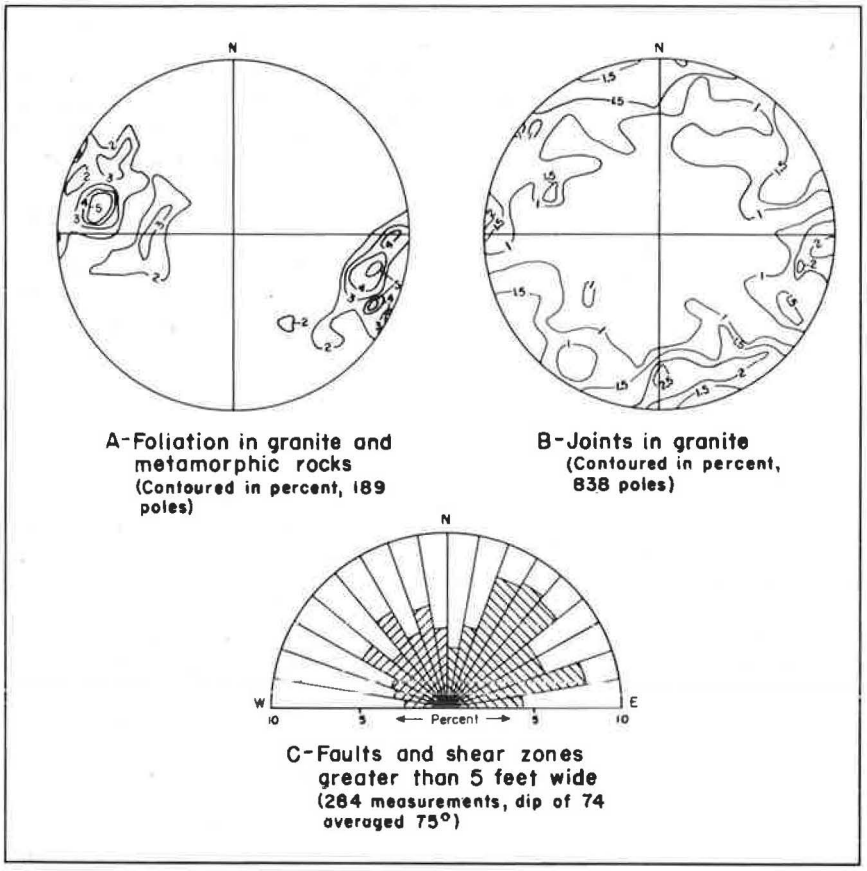


Figure 2. Contour diagrams (lower hemisphere) of the foliation and joints, and a strike-frequency diagram of the trends of faults and shear zones compiled from surface mapping.

categories of fracture density; the percentage that would be in faults or shear zones; and the attitudes of the foliation, joints, faults, and shear zones.

Rock Types—The findings in the pilot bore of 75.4 percent granite, 23.8 percent metasedimentary rocks, and 0.8 percent diorite dikes compare well with the predicted 75 percent granite and 25 percent metasedimentary rock. The results show that the surface and drill-hole data were adequate to define the rock-type percentages.

Fracture Density—The fracture density categories as defined on the surface were modified in the underground mapping. The fracture density categories given in Table 1 are a combination of the surface and the underground systems. This combining was done to make the predictions and findings comparable.

A preliminary analysis of the results of the instrumentation and of the geology of the pilot bore indicates that the fracture density categories used are probably not the most significant ones from an engineering standpoint (7). Apparently, when the average-size block of rock or the average distance between fractures exceeds 0.5 ft, the rock loads that develop are more dependent on the nature of the surface of the fracture than on the size of blocks or the spacing of fractures. Also, the loads increase greatly with an increase in rock alteration. The largest loads developed in the shear zones where the rock had been ground to fine sand or smaller size; most of the minerals were altered to clay minerals, and the zone was damp. A better definition of fracture density, taking into account the amount of alteration, is needed. The attitude and degree of foliation, even in schistose rocks, had only a minor influence on rock loads.

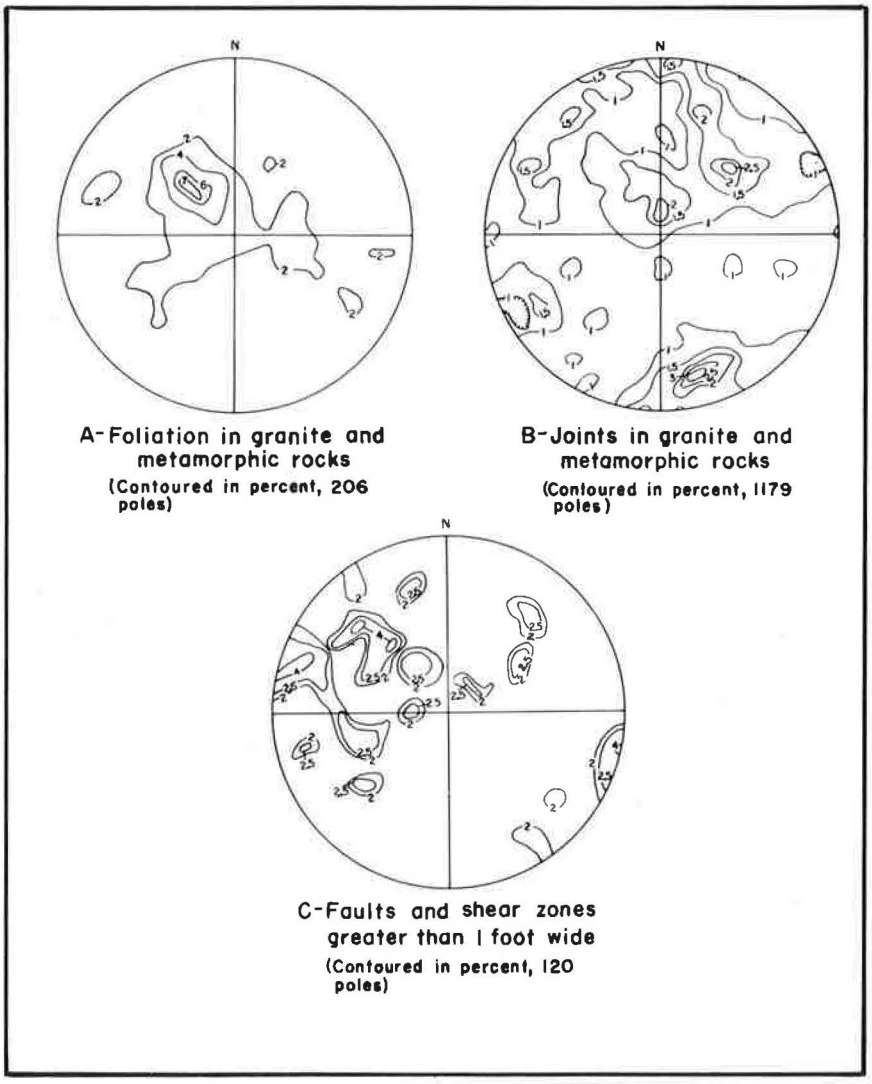


Figure 3. Contour diagrams (lower hemisphere) of the foliation, joints, and faults and shear zone compiled from pilot-bore mapping.

Faults and Shear Zones—Underground, faults or shear zones greater than 1 ft wide were used to calculate the sum of the widths of the faults and shear zones. The prediction, however, was based on surface faults and shear zones greater than 5 ft wide. The change in criterion was made because the pilot bore could be mapped with more precision than was possible on the surface. The prediction that 51 percent of the total length of the pilot bore would be in faults and shear zones is considered well within the limits of mapping accuracy for the measured 49 percent.

In the 6-sq mi area, the strike or trend of 284 faults greater than 5 ft wide was measured, but the dip could be measured on only 74 (Fig. 2c). Of these, 24.7 percent had a strike or trend between N. 20° E. and N. 50° E., and 44.8 percent between N. 20° E. and N. 80° E. The average dip of the 74 faults was 75° either southeast or northwest. Underground, the attitude of 120 faults and shear zones greater than 1 ft wide was measured (Fig. 3c). Two maxima are defined—one representing faults that strike about N. 45° E. and dip 40°-60° SE. and one representing faults that strike about N. 20° E. and dip about 75° SE.

These figures would appear to compare favorably with the predictions when it is considered that there is only 4.0 percent outcrop at the surface in the 6-sq mi area and that the pilot bore is essentially a linear feature across the area but with 100 percent exposure. The prediction that no fault or shear zone greater than 5 ft in width would be expected to follow the tunnel for considerable distance was upheld.

The preliminary analysis of the geology and the results of the instrumentation in the pilot bore (7) indicated that in part the loads are probably related to the apparent angle of dip of the faults and shear zones in relation to the trend of the pilot bore. The maximum loads developed where the apparent dip of faults and shear zones was about 45°. The loads were less where the dips were greater or less than 45°. It was also indicated that the width of a fault or shear zone must be about one-half the diameter of the pilot bore before any effect on the loads could be noticed. Better methods, possibly the geophysical methods, for defining the widths and attitudes of faults and shear zones at the surface are needed.

Attitudes of Joints—The prediction that the joints in the pilot bore would strike in any direction was confirmed by the mapping of the pilot bore (Figs. 2b, 3b). The average dip of 60° determined at the surface was high when compared to the average dip of 45° determined in the pilot bore.

In the mapping of the pilot bore, the attitudes of the joints on the walls and on the heading faces were compiled separately, which resulted in considerably different contour diagrams. On the walls, relatively fewer joints having a N. 45° E. strike and northwest dip were recorded, whereas on the faces fewer joints having a N. 20° E.-N. 20° W. strike and northwest or southwest dip were recorded. This comparison indicates that what is considered a significant joint, and so recorded, depends on the trend of the surface in the tunnel being mapped in relation to the attitude of the joint and the direction of the tunnel heading. Figure 3b was compiled from all the recorded joints on the wall and from about an equal number randomly selected from the face mapping. About four times as many joints were measured on the faces as on the walls because of the scale of mapping and the number of faces mapped.

Attitudes of Foliation—The strike of the foliation at the surface (Fig. 2a) agrees closely with that in the pilot bore (Fig. 3a), but the dip at the surface is considerably higher than that in the pilot bore. A possible explanation of this difference may lie in the number of measurements made in the granite in relation to the number made in the metamorphic rocks; the measurements of both have been combined on the diagrams. Data from surface mapping (Fig. 2a) represent 161 measurements in granite and 28 in the metamorphic rocks; data from pilot-bore mapping (Fig. 3a) represent 93 measurements in granite and 113 in the metamorphic rocks. Also, in the pilot bore, the relation of the surface of measurement to the attitude of the foliation (as with the joints) probably influences the number of observations made.

Engineering Measurements

The items considered under the engineering measurements are rock load, final swell pressure of fault gouge, and groundwater flow.

Rock Load—The maximum rock load of 5,900 psf was based on the preliminary design of a 10.5 × 11.5-ft tunnel, utilizing the theories of Terzaghi (10). The final pilot bore, however, averaged about 13 ft in diameter. The predicted maximum rock load for this size tunnel would be 6,970 psf.

The results of the instrumentation of the pilot bore required a modification of the theories of Terzaghi (10) for stress around a tunnel. As the face advances away from a point, a maximum load develops on the support at that point, which after a period of time usually declines to a stable load. The time required for the development of the maximum load and the magnitude of the maximum load, and the time required for the load to stabilize and the magnitude of the stable load, are dependent on the geologic conditions, the construction practices, and the dimensions of the tunnel. It is possible within certain limits to determine the part of the load that is the result of the construction practices and the part that is the result of geologic conditions (7).

The part of the rock load that is the result of the geologic conditions is termed the geologic rock load, and the part that is the result of the construction practices is termed the engineering rock load. The geologic conditions were divided into three categories, representing the range in rock quality or competency, depending on a range in fracture density and a range in the percentage or alteration. The range in the geologic rock loads for each geologic category was calculated from the results of the instrumentation. From this range in geologic rock loads a geometric midpoint for each geologic category was calculated. The geometric midpoint is that geologic rock load that, when multiplied by or divided by a range factor, gives the range in geologic rock loads for the geologic category. The range factor is simply the number which, when multiplied by the geometric midpoint of the geologic rock load, yields the maximum geologic rock load. The minimum geologic rock load is obtained by dividing the geometric midpoint of the geologic load by the same range factor. For the pilot bore, the geometric midpoint for the worst geologic category, where the maximum geologic rock loads developed, was 6,600 psf with a range factor of 1.5. Thus, the range in geologic rock loads for the worst geologic category was 4,400 to 9,900 psf. As a result of the worst geologic conditions and the construction practices, the maximum rock load that developed in the pilot bore was about 20,000 psf.

The predicted rock load and the measured geologic rock load cannot be directly compared because the existing theories for predicting loads are not entirely adequate. Probably, the calculated geometric midpoint for the worst geologic conditions most closely fits the theory as developed by Terzaghi (10).

Final Swell Pressure of Fault Gouge—Swell pressure predictions were based on the assumption that the clay mineralogy of fault gouge and altered rock would be essentially the same in the pilot bore as at the surface. The average final swell pressure of 29 samples collected from the pilot bore was 1,727 psf, which compares favorably with an average final swell pressure of 2,233 psf from 6 surface samples. Thus, the assumption that the final swell pressures of samples from the surface would be about the same as for samples from the pilot bore appears valid.

Groundwater—The figures for the prediction of the average flow from the portal and the flow actually measured have little significance. The authors failed in their original calculations and predictions to consider the time of year and the influence of the spring runoff. The average groundwater flow from the portal was increased by a factor of 7.5 times as a result of the spring runoff.

The predicted flow of 300 gpm from the portal 2 wk after completion of the pilot bore was based on a constant rate of advance of the pilot bore of 1,000 ft per month. The average rate for the pilot bore, however, was only about 610 ft per month. At this rate of advance, the estimated flow would have been about 183 gpm. These figures, although comparable, are meaningless because the influence of the spring runoff was not considered. If the pilot bore had been completed in the spring, the measured flow would have been much greater than the predicted flow.

All the flow calculations were based on estimates of the porosity and permeability of the faults and shear zones. In the pilot bore, however, the faults and shear zones were essentially dry until they were opened up. The principal water flows came from open joints in relatively competent rock that were beyond the limits of the faults and shear zones. The approximate agreement of the predicted and measured groundwater flows, therefore, can be considered due more to luck than to skill.

Construction Practices

The predictions of the spacing of sets, lagging and blocking, feeler holes, and amount of grout were, of necessity, empirical, because actual requirements can be determined at the time of construction. It was felt that such predictions, however, would be value in estimating the cost of construction. Geologic conditions alone do not determine requirements; other factors, some of which have been discussed in relation to the geologic rock load, also exert an influence.

Set Spacing—In the pilot bore, the sets were not uniformly spaced, particularly where jump sets were added. For the purpose of comparison with the prediction, spacings of

0.5 to 1.5 ft were combined and compared with predicted spacing of 1 ft, 1.5 to 2.5 with 2 ft, 2.5 to 4.5 with 4 ft, and 4.5 and greater with 5 ft.

The predicted and actual spacing of sets agree very well when all the factors that influence support are considered, plus the fact that the length of the pilot bore was increased by approximately 300 ft. The total number of sets calculated from the predicted spacing of sets was 2,691. The actual number used was 2,059, although the calculated number of sets based on our combining of the actual set spacings is 2,274.

It was predicted that 1.4 percent of the pilot-bore length would require invert struts on 1-ft centers, or a total of 113 struts. The contractor used struts for 8.0 percent of the pilot-bore length or 210 struts, but these were on 1 to 3-ft centers.

Lagging and Blocking—The predictions for lagging and blocking specified sections of the pilot bore that would require blocking only, blocking and lagging along the arch, and blocking and lagging along the arch and walls. In practice, it was more convenient to record the percentage of blocking and lagging around the walls and arch. The predicted figures in Table 1 are converted to percentages for comparative purposes.

Feeler Holes—The drilling of feeler holes was recommended in the pre-construction report (5), and the approximate areas in which they might be advisable were indicated. In practice, the Colorado Department of Highways and the contractor considered it advisable to keep at least one feeler hole about 40 ft in advance of the face for most of the length of the pilot bore, a decision in which the authors concurred. For that reason, there is a considerable difference—by a factor of almost 4 times—between the predicted number of linear feet of feeler holes and the footage actually drilled. The larger percentage of the feeler holes did not intersect broken, water-saturated ground, which was their purpose. From economic and safety points of view, however, feeler holes were advisable in that they gave the contractor a better idea of the ground in advance of the face and allowed him to plan more economically for such things as lengths of round and supplies (sets, timber, etc.) needed in the pilot bore.

Grout—It was predicted that it might be economically advantageous to grout, in advance of the face, certain types of ground as determined by feeler holes. The purpose of the grout is to consolidate bad ground and seal off water, and thus reduce the amount of support required and the difficulty of driving through that section. The alternative is closely spaced supports and forepoling. This decision is the prerogative of the contractor and the owner, and it was decided that grout was not needed in the pilot bore.

Cost—The pilot bore was holed through during the first week of December 1964 and cleanup work was completed in January 1965. The total cost of construction of the pilot bore was approximately $1,400,000, which compares favorably with the contractor's bid of $1,300,000.

CONCLUSIONS

The accuracy of a geologic projection depends on the understanding of the geometry of the geology, the amount of time available for surface examination, the amount of time and money for physical exploration involving drill holes and the application of geophysical techniques, and the knowledge and experience of the geologists. The Straight Creek Tunnel project has established that geology can be treated statistically to predict the kinds and percentages of different geologic conditions at depth, and that engineering requirements can be equated with predicted geologic conditions to provide a sound basis for estimating probable cost of construction. The failures of some of the predictions of the project have shown those fields in which there is not adequate geologic and engineering knowledge. Continued research in the prediction of geologic and engineering conditions at the depth of a tunnel should make possible more accurate predictions and so reduce the cost of construction by the amount required for contingencies.

The Straight Creek Tunnel project was conducted in an area with a limited number of geologic variables, which probably in part accounts for its success. It is believed, however, that a similar approach can be successfully applied to the projection of geology to depth in any geologic environment if the geometry of the geology is thoroughly understood and carefully analyzed.

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