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Office of Reseach and Development

Industrial Environmental Research Laboratory
Cincinnati, Ohio 45268

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November 1977

CATAWISSA CREEK MINE DRAINAGE ABATEMENT PROJECT

Interagency
Energy-Environment
Research and Development
Program Report

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6. Scientific and Technical Assessment Reports (STAR)
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CATAWISSA CREEK MINE DRAINAGE ABATEMENT PROJECT

by

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Grant No. 14010 DSD

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OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

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FOREWORD

When energy and material resources are extracted, processed, converted, and used, the related pollutional impacts on our environment and even on our health often require that new and increasingly more efficient pollution control methods be used. The Industrial Environmental Research Laboratory-Cincinnati (IERL-Ci) assists in developing and demonstrating new and improved methodologies that will meet these needs both efficiently and economically.

Reported here are the results of a study to develop methods to control acid mine drainage from underground mines. The reconstruction of a stream bed to divert water away from underground mine working has shown effect in reducing acid discharges. The construction of seals in tunnels that drain underground anthracite coal mines was found technically feasible, but were not constructed as part of this study because of high construction costs. Seals of this type might also be feasible for tunnels draining hard rock mines. This research will be of interest to state and federal agencies developing control strategy for abandoned underground mines. In addition design details presented in the report will be useful to design engineers. For further information contact the Resource Extraction and Handling Division.

David G. Stephan
Director
Industrial Environmental Research Laboratory
Cincinnati

ABSTRACT

The objective of this study was to determine the feasibility of flooding underground coal mine workings in an isolated basin of coal, thereby restoring or partially restoring the groundwater table in the basin and reducing the production of acid mine drainage. Flooding the mined seams would prevent atmospheric oxygen contact with the acid-forming materials, thus breaking the chain of chemical reactions in the formation of acid mine drainage. To enable this determination, a relatively small discrete basin of coal in east-central Pennsylvania at Sheppton was selected.

This basin, extensively deep mined during the last 85 years and intensively strip mined for the last 50 years, is drained by three water-level tunnels driven through the rock to intercept the deep mine workings at their deepest points. In addition, during the period of strip mining, the watershed's streamflow was diverted into the basin's deep mine workings.

Preliminary investigations conducted during 1966-1968 under an earlier contract had indicated that this project appeared viable. To determine project feasibility with a higher degree of certainty, detailed investigations were undertaken, including studies of the regional and areal geology, the extent of strip and deep mining, and the water-level tunnel flows and water quality. The nature and condition of the rock were studied throughout the basin by internal tunnel investigations, core borings above potential seal sites, and core borings at anticipated future overflow points. It was concluded that approximately 80 percent of the basin could be inundated by sealing the water-level tunnels, resulting in a reduction of approximately 1,100 kilograms per day of acid being discharged into Catawissa Creek.

As the first step, the watershed's streambed was relocated to prevent streamflow from passing into, and emitting from, the mined basin. Approximately 518 meters of streambed was reconstructed at a cost of $58.94 per meter, eliminating 0.253 m^3/s of water from entering the underground mine workings. Even though the mine sealing was deemed to have much merit, it was cancelled because of its high costs after plans and specifications for sealing the three tunnels were prepared and bids were taken for sealing one water-level tunnel. Bid cost for constructing the one seal was in excess of $600,000.

This report was submitted in fulfillment of Project Number 14010 DSD by Gannett Fleming Corddry and Carpenter, Inc., under the joint sponsorship of the United States Environmental Protection Agency and the Commonwealth of Pennsylvania. The report covers the period January 1969 to August 1975, and work was completed as of July 1976.

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ACKNOWLEDGMENT

Sincere appreciation is expressed to Ronald D. Hill, Henry R. Thacker, and Donald O'Bryan of the Environmental Protection Agency for their support, guidance, and assistance on this project.

The following personnel of the Pennsylvania Department of Environmental Resources contributed much effort in the technical assistance and administration of this project: Messrs. E. Bates, A. Beacher, R. Buhrman, J. Demchalk, D. Fowler, A. Friedrich, F. Oldham, D. Perrego, and A. Ranieri.

Special thanks are given to Messrs. P. Hino, A. Joyce, and G. Sterling of the Pennsylvania Department of Environmental Resources for their cooperation and assistance in the underground investigations of the water-level tunnels.

Gratitude is expressed to Mr. William Calovine and the Blue Knob Rod and Gun Club for their cooperation during this project.

Finally, a special note of appreciation is expressed to Drs. H. B. Charmbury and D. R. Maneval and Mr. John J. Buscavage who were instrumental in initiating this project under the jurisdiction of the former Pennsylvania Department of Mines and Mineral Industries.

I

INTRODUCTION

BACKGROUND

Congress, through the Federal Water Pollution Control Act, authorized comprehensive watershed studies within major river basins throughout the Nation in October 1961. One of the principal objectives of these studies is the development of a water quality management program for each major basin. To effectively develop such programs, a determination of the extent of water pollution, as well as the methods for and costs of eliminating or reducing such pollution, must be made.

The Susquehanna River - Chesapeake Bay is one basin in which the U.S. Environmental Protection Agency has undertaken such studies. Because mine drainage is a major source of pollution in this basin, investigations were authorized in five specific coal mining areas of the basin, all within Pennsylvania. These investigations determined for each area: (1) the causes and extent of mine drainage pollution; (2) alternative mine drainage abatement plans that could be used to achieve the Pennsylvania Department of Environmental Resources' mine drainage discharge limitations, and associated costs; and (3) the abatement plan considered most desirable. The report [1] that summarized the results of these investigations was submitted to the Federal Water Pollution Control Administration (subsequently called U.S. Environmental Protection Agency) in December 1968.

One of the five areas covered by this report lies in the vicinity of the village of Sheppton and consists of a separate small basin of anthracite coal known as the South Green Mountain Basin. Acid mine drainage flows from the basin to Catawissa Creek through three water-level tunnels driven into the underground mine workings. The recommended plan for this area included the following abatement measures: (1) reconstruction of the Catawissa Creek stream channel to divert flow away from the deep mine workings; (2) construction of water-tight seals in the three tunnels, causing partial inundation of the deep mine workings and creating two new mine drainage discharges of better quality at higher elevations; and (3) after the quality of these new

(1) Gannett Fleming Corddry and Carpenter, Inc. Acid Mine Drainage Abatement Measures for Selected Areas Within the Susquehanna River Basin. Engineering Report, Contract No. WA 66-21, U.S. Department of Interior, Federal Water Pollution Control Administration, 1968.

discharges has been established, construction of treatment plants as necessary to meet mine drainage discharge limitations. In this report, it was further recommended that a geologic investigation be undertaken of the tunnels, the basin, and the surrounding geologic formations to determine with greater certainty the feasibility of constructing the seals and creating the new mine drainage discharges.

The findings, conclusions, and recommendations set forth in the above report were made available to EPA and the Pennsylvania Department of Environmental Resources several months before its formal submittal. To expedite implementation of the recommended abatement plan, the Department in the spring of 1968 requested federal funds to partially support the recommended geologic investigations and implement the first two steps of the recommended plan. The Environmental Protection Agency, on June 18, 1968, awarded the Department a Research and Development Grant, which provides 70 percent of the estimated amount required for implementation of this project and its subsequent evaluation. The Department subsequently entered into an agreement for consulting engineering services to implement this project.

REGIONAL GEOLOGIC SETTING

The project area, as shown in Figure 1, covers approximately 20 square miles of the Eastern Middle Anthracite Field in northern Schuylkill and southern Luzerne Counties, Pennsylvania, and centers around the villages of Sheppton and Oneida. Green Mountain, the major topographic feature of the project area, is a westward projection of the Eastern Middle Field, one of four synclinoria, or broad downwarps, that comprise eastcentral Pennsylvania's four anthracite fields.

Resistant sandstone and conglomerate beds of the Pottsville formation, which underlie the coal measures, form topographically elevated ridges around the outer rim of the synclines. The underlying Mauch Chunk formation, which is predominantly composed of shales, has been eroded to form the adjacent lower valleys. The shales and coals overlying the Pottsville formation, and forming the core of the folded unit, are also less resistant. In the broad fold that forms the Northern Anthracite Field, erosion of these units has formed a deep valley enclosed by the high, more resistant ridge. Conversely, the fold in the project area is much more narrow from rim to rim, and erosion has lowered the enclosed units only slightly below the surrounding ridge to form the high, plateau-like topography of Green Mountain.

Exposed rocks of the anthracite region were originally deposited as soft sediment about 250 to 350 million years ago during the Pennsylvanian and Mississippian geologic periods. Coal was formed from the compaction and chemical alteration of peat accumulated in large swamps, which flourished at that time.

Geologic structural features of the region are the result of the Appalachian Orogeny beginning approximately 230 million years ago, the last great pulse of mountain building in eastern North America. Compressional forces applied from the southeast caused the earth's crust to fold, wrinkle, and fracture. Subsequent selective erosion has resulted in the present

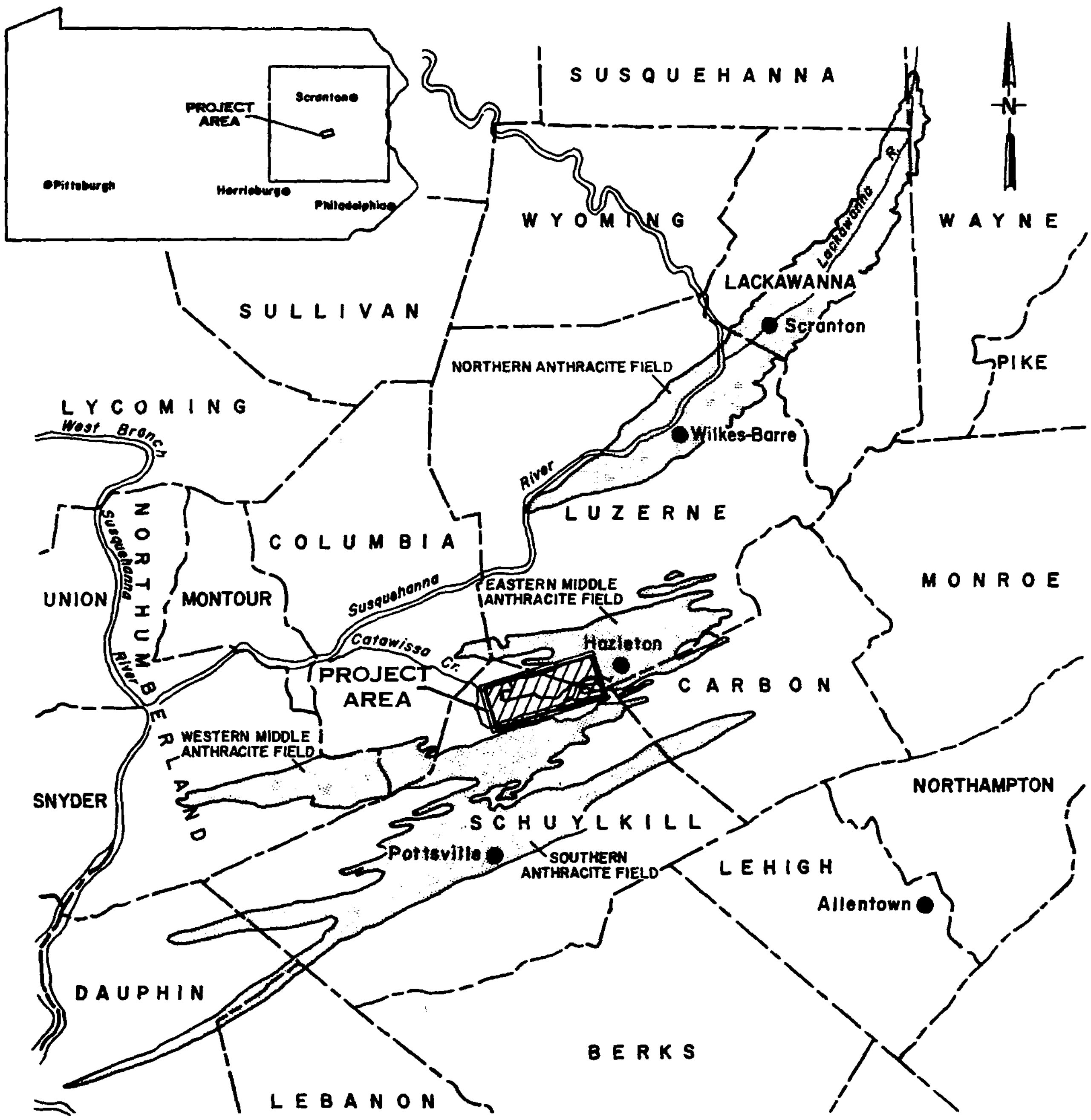
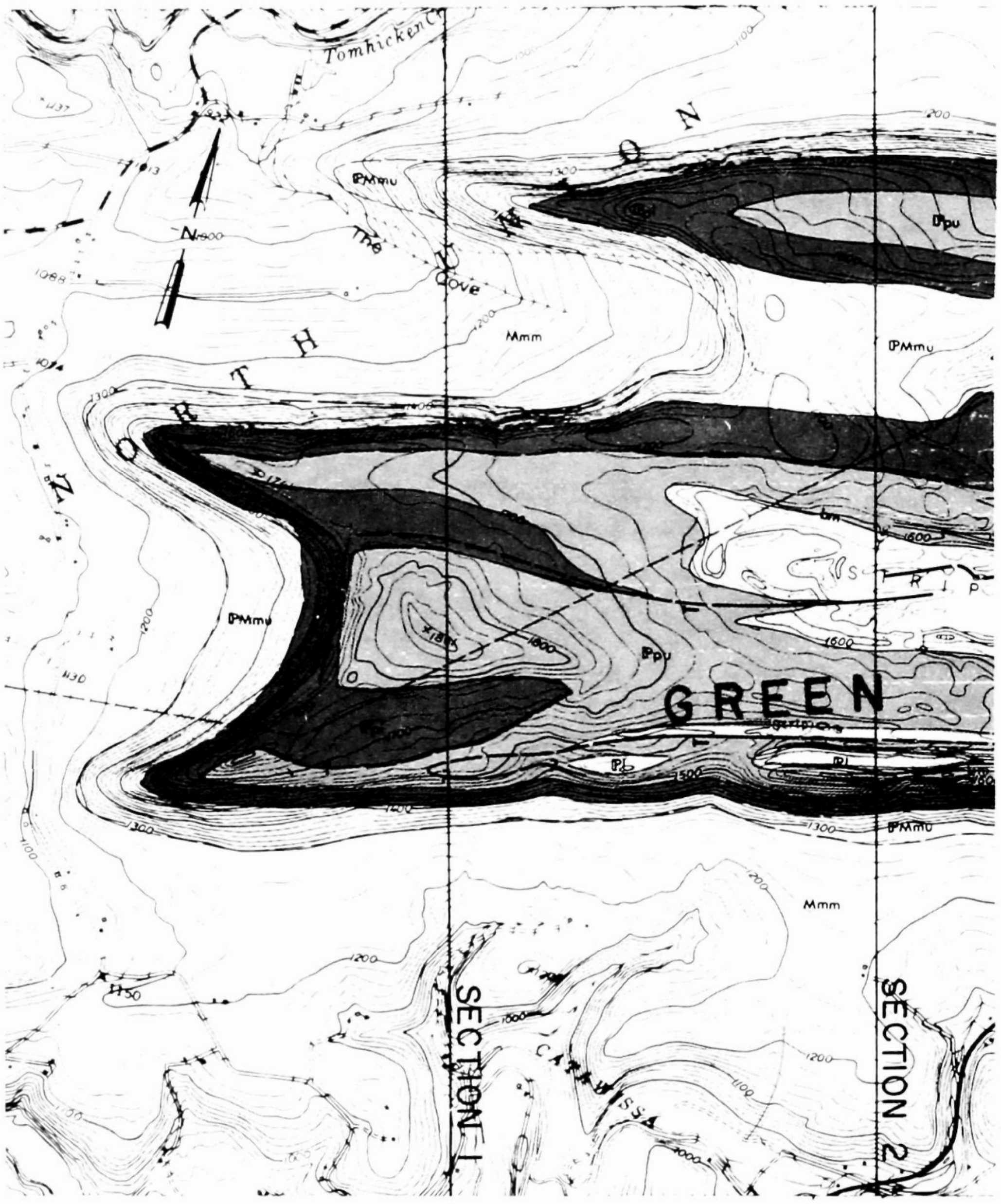


Figure 1. Location map of project area.



Note Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

PENNSYLVANIAN

FORMATIONS

Pl — Llewellyn Formation

Ppu — Pottsville Formation
Ppu, upper member
Ppl, lower member

PENNSYLVANIAN AND MISSISSIPPIAN — PMmu

MISSISSIPPIAN — Mmm

Mauch Chunk Formation
PMmu, upper member
Mmm, middle member

Drainage Tunnel

Present MD Discharge Point

Future MD Discharge Point (Overflow)

CONTACTS

Definitely Located

Approximately Located

? Precise Nature or Existence Uncertain

bm Buck Mountain Coal Vein

THRUST FAULTS

T Definitely Located, T on Upthrown Side

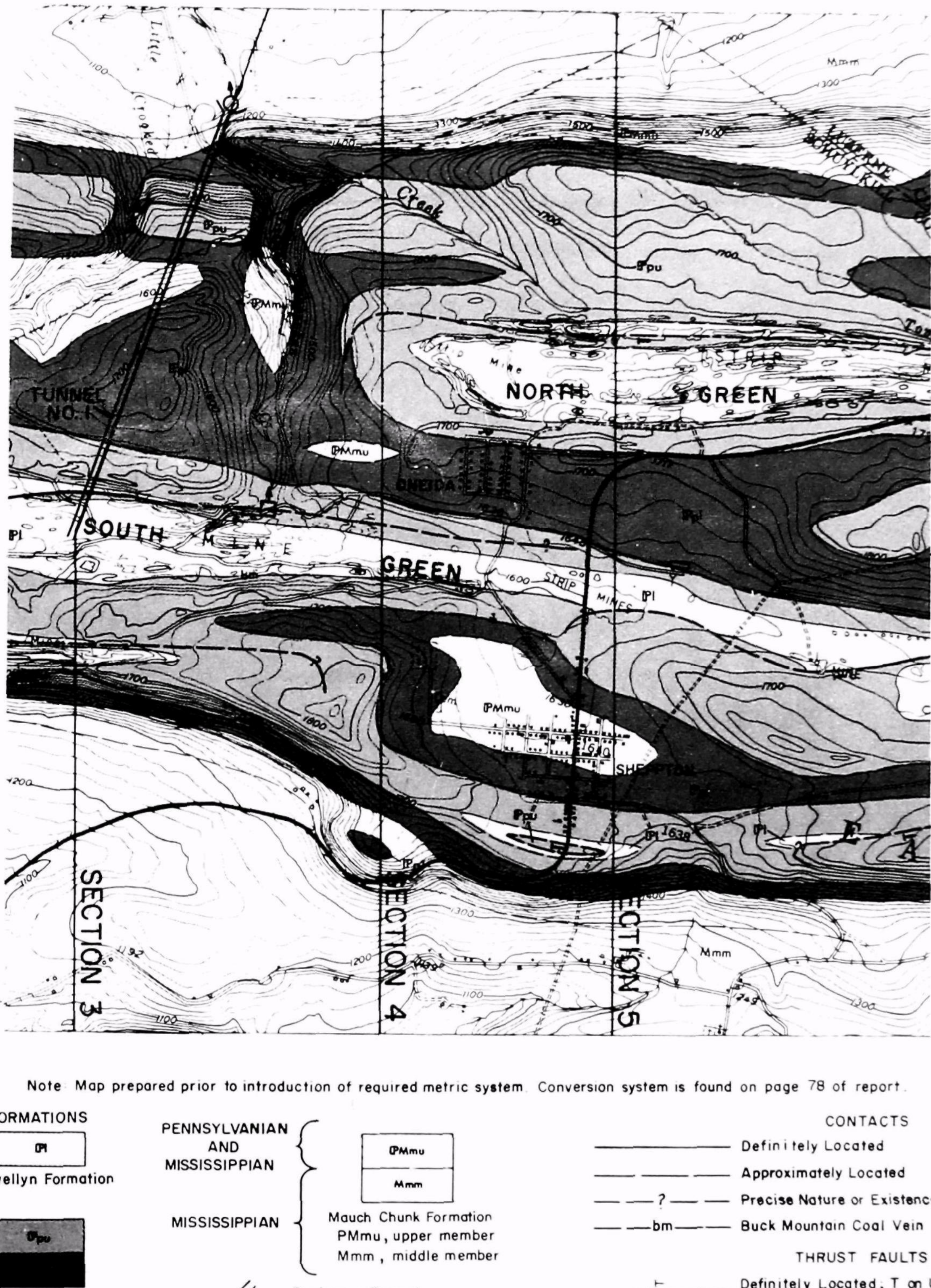
Approximately Located

? Precise Nature or Existence Uncertain

2000 1000 0 2000

SCALE IN FEET

Figure 2a. Geologic map of South Green Mountain Basin and vicinity.



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T — Definitely Located; T on Upthrown Side

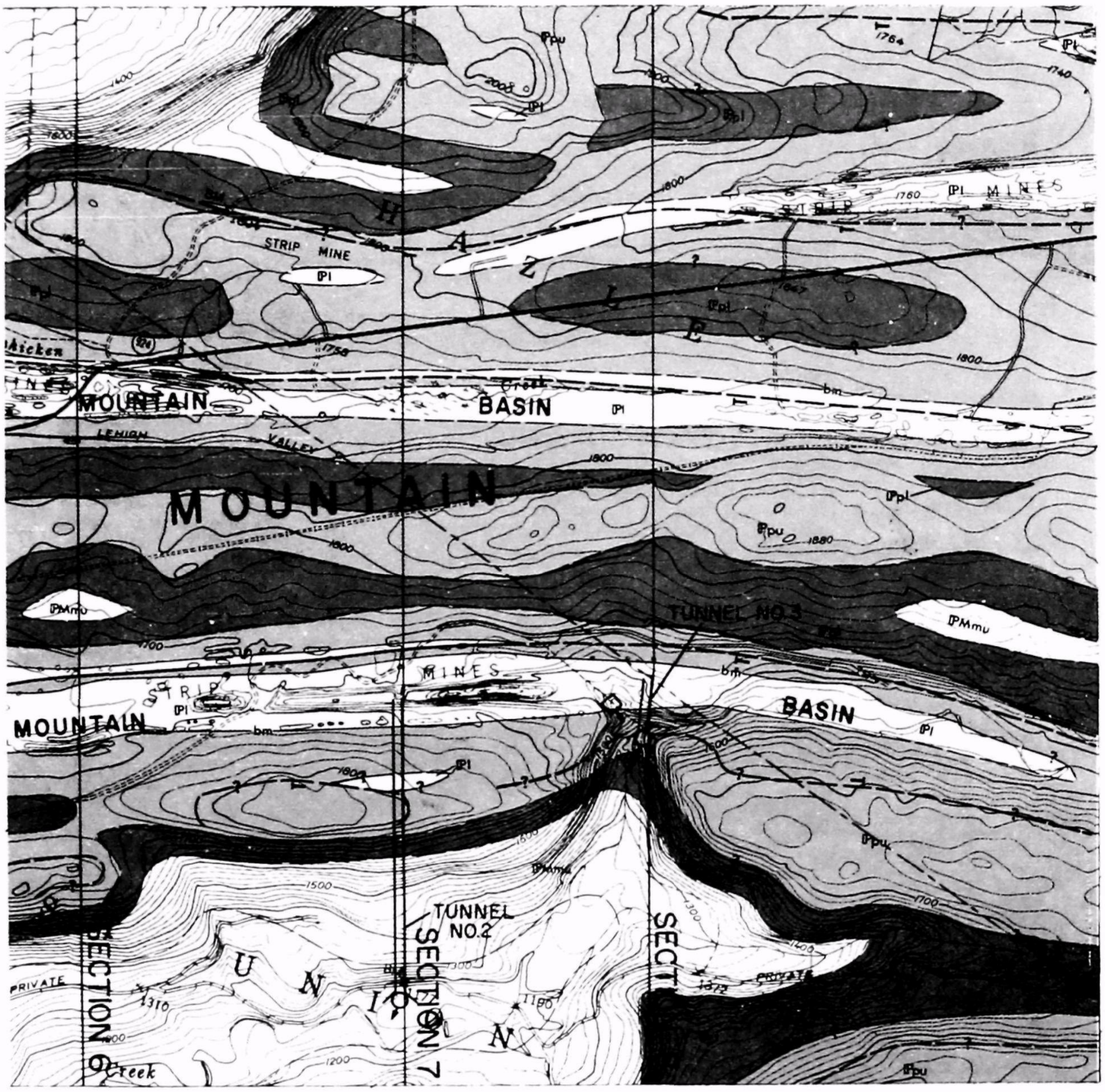
Approximately Located

? — Precise Nature or Existence Uncertain

2000 1000 0 2000

SCALE IN FEET

Figure 2b. Geologic map of South Green Mountain Basin and vicinity.



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PENNSYLVANIAN

FORMATIONS

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Ppu / Ppl — Pottsville Formation
Ppu, upper member
Ppl, lower member

PENNSYLVANIAN AND MISSISSIPPIAN — PMmu

MISSISSIPPIAN — Mmm

Mauch Chunk Formation
PMmu, upper member
Mmm, middle member

Drainage Tunnel

Present MD Discharge Point

Future MD Discharge Point (Overflow)

CONTACTS

Definitely Located

Approximately Located

Precise Nature or Existence Uncertain

bm — Buck Mountain Coal Vein

THRUST FAULTS

Definitely Located; T on Upthrown Side

Approximately Located

Precise Nature or Existence Uncertain

2000 1000 0 2000

SCALE IN FEET

Figure 2c. Geologic map of South Green Mountain Basin and vicinity.

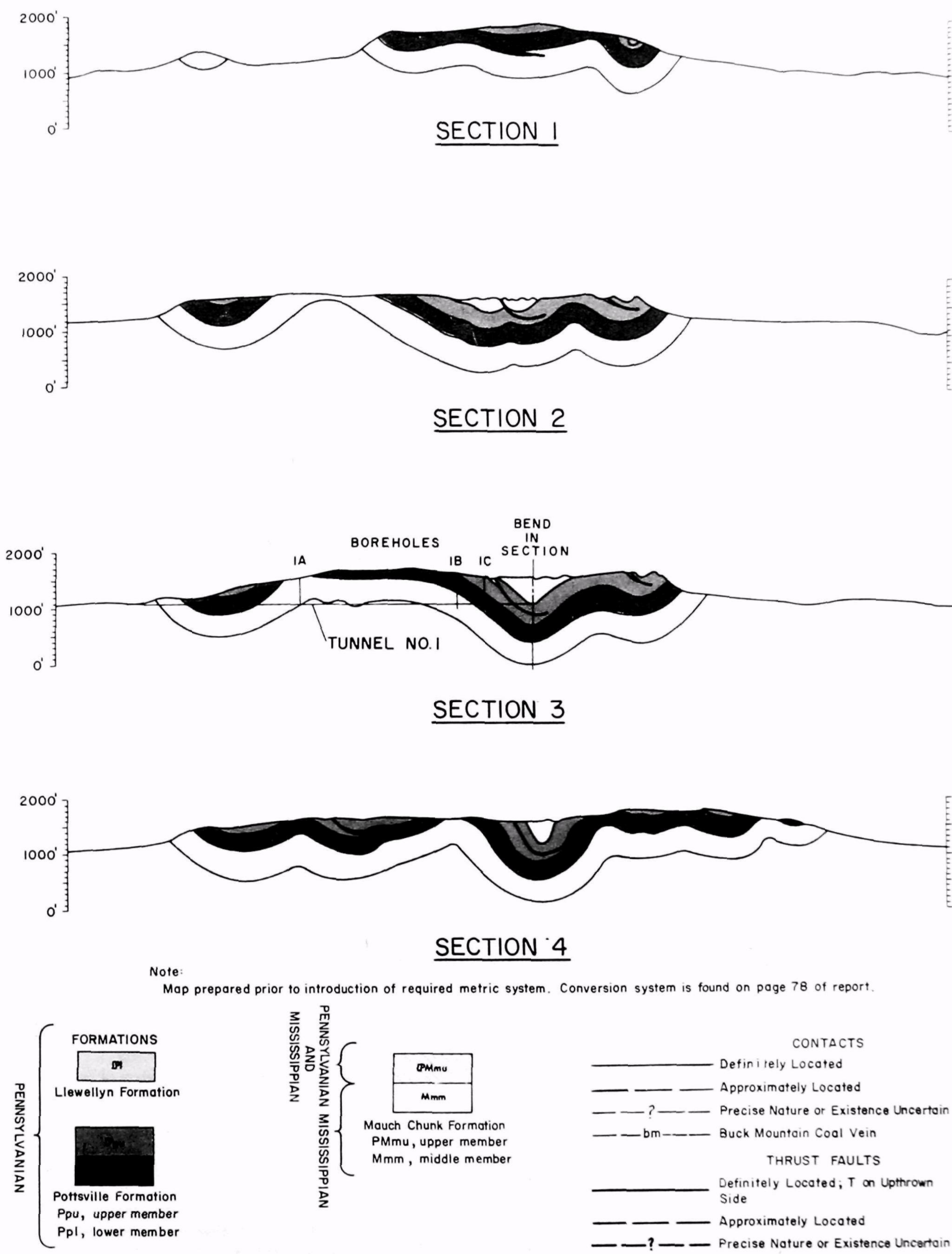
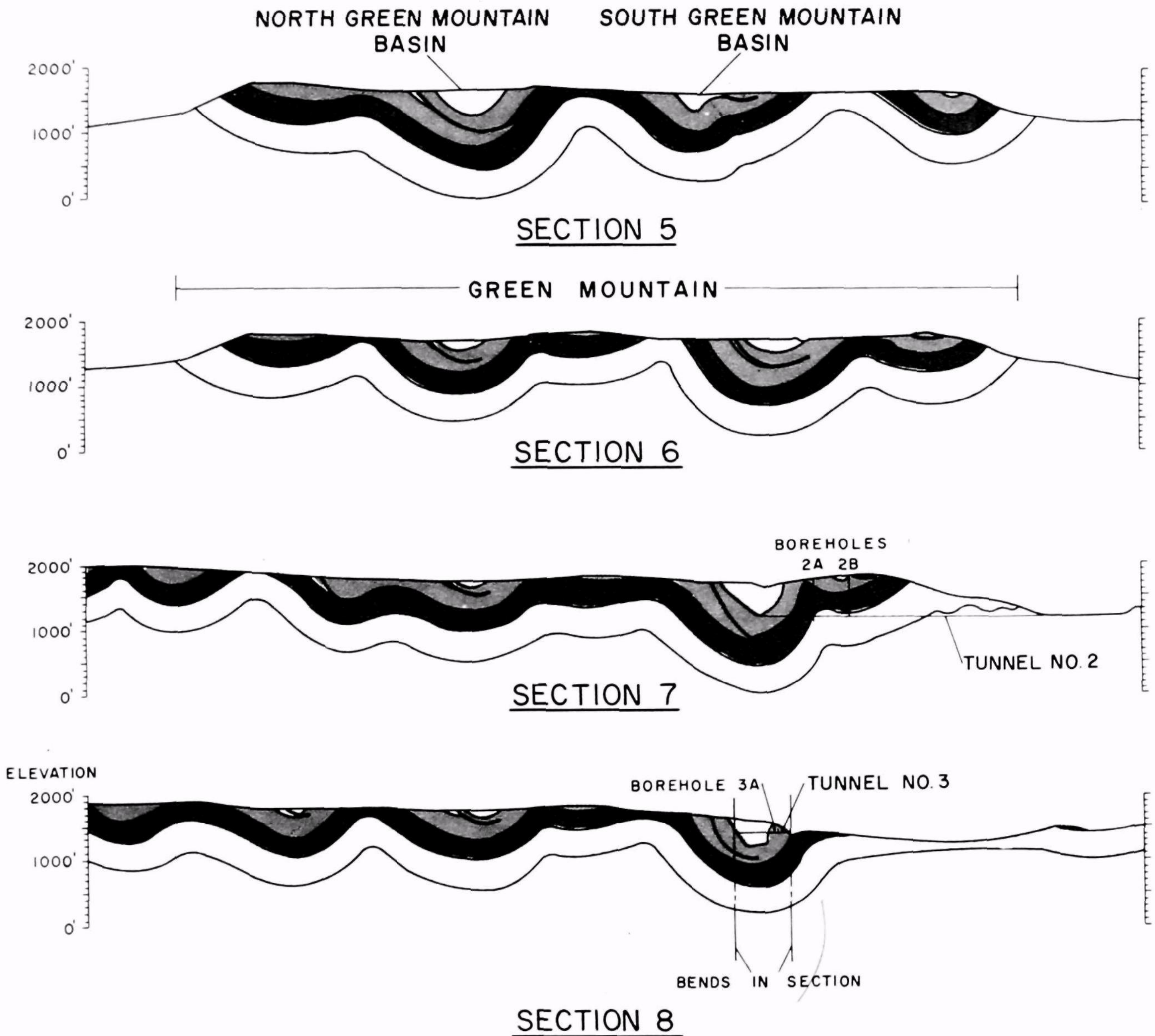


Figure 3a. Geologic cross sections of South Green Mountain Basin and vicinity.



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PENNSYLVANIAN

FORMATIONS

Pl

Llewellyn Formation

Ppu

Pottsville Formation
Ppu, upper member
Ppl, lower member

PENNSYLVANIAN AND MISSISSIPPIAN

PENNSYLVANIAN MISSISSIPPIAN

PMmu

Mmm

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Mmm, middle member

CONTACTS

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Approximately Located
? Precise Nature or Existence Uncertain
bm Buck Mountain Coal Vein

THRUST FAULTS

Definitely Located; T on Upthrown Side
Approximately Located
? Precise Nature or Existence Uncertain

Figure 3b. Geologic cross sections of South Green Mountain Basin and vicinity.

complex mountain structure. The coal-bearing strata of eastcentral Pennsylvania were folded and deformed to their configuration during this Orogeny. The map and cross sections on Figures 2 and 3 show the deformed shape of the South Green Mountain Basin.

HISTORY OF MINING

Mining in the South Green Mountain Basin began about 1890. Coxe Brothers and Company, Inc., developed the western portion of the basin at the Oneida No. 3 and No. 4 Mines. The Honey Brook Division of the Lehigh and Wilkes-Barre Coal Company and its successor, the Glen Alden Coal Company, developed the eastern portion at the Green Mountain Mine. The major companies continued operations into the late 1940's. "Bootleg" operations continued to engage in deep mining until the mid 1960's. No deep mining is presently conducted in the basin.

Nine coal seams have been mined within the basin. Review of mine maps and cross sections has revealed that the mining companies gave various names to these coal seams. To maintain consistency of terminology in the figures, only Coxe Brothers and Company coal seam names have been used. They are:

Primrose
Top Mammoth
Middle Mammoth
Bottom Mammoth
Wharton
Top Gamma
Bottom Gamma
Buck Mountain
Little Buck Mountain

An estimated 9 million cubic meters of coal were removed from the basin during deep mining, with 3.5 million cubic meters having been left behind as shaft and slope reserves, barrier pillar, and where mining was difficult or dangerous.

Resumption of underground operations is not currently anticipated in the basin primarily because of safety considerations. Where accessible, mine workings can be observed in various stages of total collapse. It is believed that this condition prevails throughout the basin, with complete collapse where supporting pillars were removed, and few areas still intact. A 30.5-meter-thick barrier pillar of unmined coal was left in place between the east and west properties as a physical boundary. This pillar, if unbreached, would isolate the two ends of the basin from each other and prevent the flow of mine waters from one to the other. However, it is suspected that, while the pillar may be partly intact, considerable interflow will occur. The probability of interconnection increases near the surface, where strip mine activity most surely has cut the barrier.

Strip mining started in the basin at about the time of World War I but was not conducted extensively until World War II. Stripping has continued to the present time. Virtually all of the coal close to the ground surface not removed by deep mining has been recovered by stripping, except for the Little Buck Mountain seam. No reasonable estimate of coal removed by stripping can be made because of irregular conditions and erosion.

Original efforts to keep the basin deep mine workings dry primarily consisted of a system of surface diversion ditches to prevent the entry of most surface water. However, when the workings advanced below the permanent groundwater table, pumping was required. About 1900, in order to reduce costs associated with pumping, three tunnels were driven between the deep mine workings and the adjacent valleys. Workings were advanced to the deepest coal in the basin, where Tunnel Nos. 1 and 2 could be constructed at an elevation higher than the deepest workings but at the lowest available point of gravity drainage to the adjacent valley. Detailed information concerning these tunnels is provided in Figures 4, 5, and 6.

Tunnel No. 1 (the Oneida No. 3 Drainage Tunnel) was driven 2,139 meters to the south from Tomhicken Creek, a tributary of Catawissa Creek, to intercept the workings at an elevation of 331 meters. Tunnel No. 2 (the Green Mountain Drainage Tunnel) was driven 1,253 meters to the north from Catawissa Creek to intercept a second low point (elevation 358 meters) within the workings. Tunnel No. 3 (the Green Mountain Water-Level Tunnel) was driven 256 meters to the north from Catawissa Creek to a third point (elevation 427 meters) within the workings. Figures 2b and 2c show the locations and positions of the tunnels.

MINE DRAINAGE POLLUTION

The origin of the mine drainage pollution problem within the South Green Mountain Basin lies in the nature of mining, the material mined, and the mine water removal method employed. Anthracite seams within the South Green Mountain Basin are relatively thin sedimentary strata, separated by thick beds of sandstone and shale. Mining of most of the coal in each seam reduced roof support to a point where overlying strata collapsed into the workings. Coupled with strip mining, this completely destroyed the natural drainage patterns of the surface streams. Within the South Green Mountain Basin, surface waters now infiltrate downward through the broken strata, pick up acid and iron from oxidizing sulfide minerals in the coal and closely associated strata, and discharge to the surface through the drainage tunnels.

THEORY OF MINE DRAINAGE ABATEMENT

As has been previously well documented, acid mine drainage results from the oxidation of exposed acid-forming material closely associated with mined coal measures. The ferrous sulfate thus formed is readily dissolved by water contacting it. When the water containing these dissolved oxidation products (acid mine drainage) flows from the mine workings to surface streams, water quality in those streams becomes degraded. When sufficient acid mine drainage is discharged into a stream to overbalance its available alkalinity, the stream becomes acid.

Acid mine drainage formation can be abated if one or more of the links in the reaction -- the acid-forming material, the oxygen (air), or the water

TUNNEL NO. 1

(1) RED SANDSTONE INTERBEDDED WITH RED SHALE AND SILTSTONE

(2A) GREENISH-GRAY ROCK FRAGMENT PEBBLE CONGLOMERATE AND SANDSTONE INTERBEDDED WITH RED SANDSTONES AND SILTSTONE.

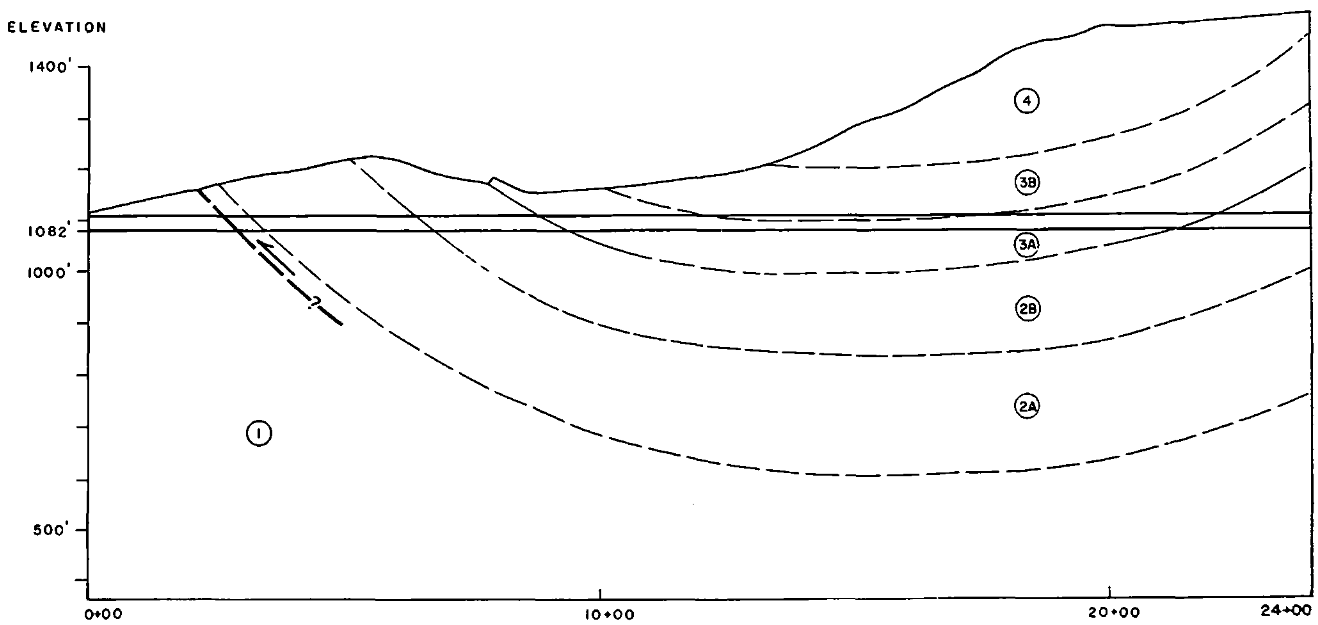
(2B) RED SHALES, SILTSTONES, AND SANDSTONES INTERBEDDED WITH GRAY SANDSTONES

(3A) GRAY TO GREENISH-GRAY SANDSTONE AND ROCK FRAGMENT PEBBLE CONGLOMERATE WITH THIN BEDS OF RED AND GRAY SHALE AND SILTSTONE.

(3B) GRAY INTERBEDDED SANDSTONE AND SILTSTONE WITH A THIN RED SHALE BED NEAR MIDDLE

(4) LIGHT TO DARK GRAY QUARTZ PEBBLE CONGLOMERATE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO DARK GRAY SANDSTONE; SEVERAL THIN BEDS OF BLACK SHALE WITH CARBONACEOUS PARTINGS.

(5) GRAY TO BUFF SANDSTONE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO BLACK SHALE; SEVERAL THIN TO THICK COAL BEDS AND UNDERCLAYS.

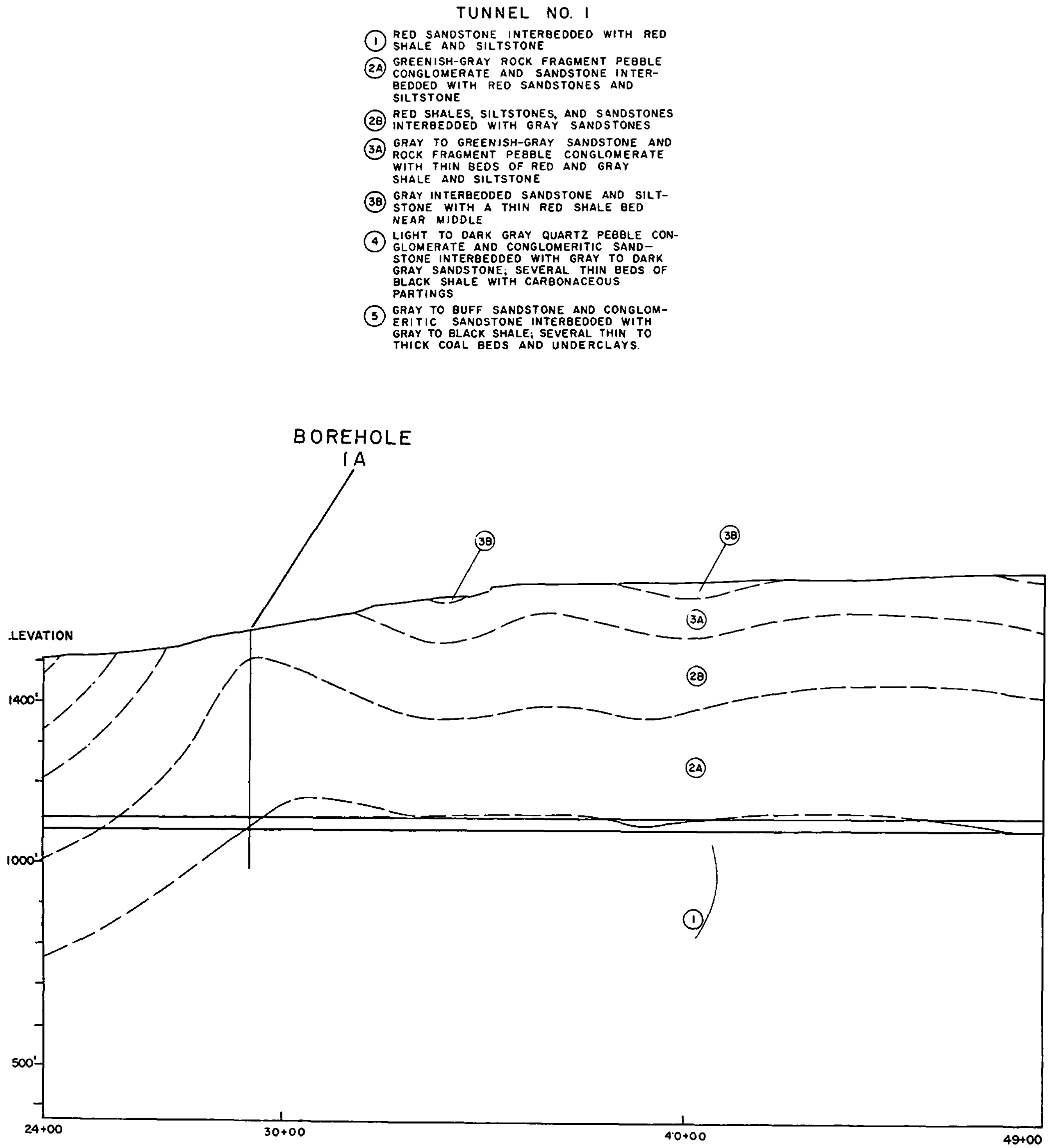


Note: Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

400 200 0 400 800

SCALE IN FEET

Figure 4a. Cross section through Tunnel No. 1.



Note: Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

400 200 0 400 800

SCALE IN FEET

Figure 4b. Cross section through Tunnel No. 1.

TUNNEL NO. 1

(1) RED SANDSTONE INTERBEDDED WITH RED SHALE AND SILTSTONE

(2A) GREENISH-GRAY ROCK FRAGMENT PEBBLE CONGLOMERATE AND SANDSTONE INTERBEDDED WITH RED SANDSTONES AND SILTSTONE

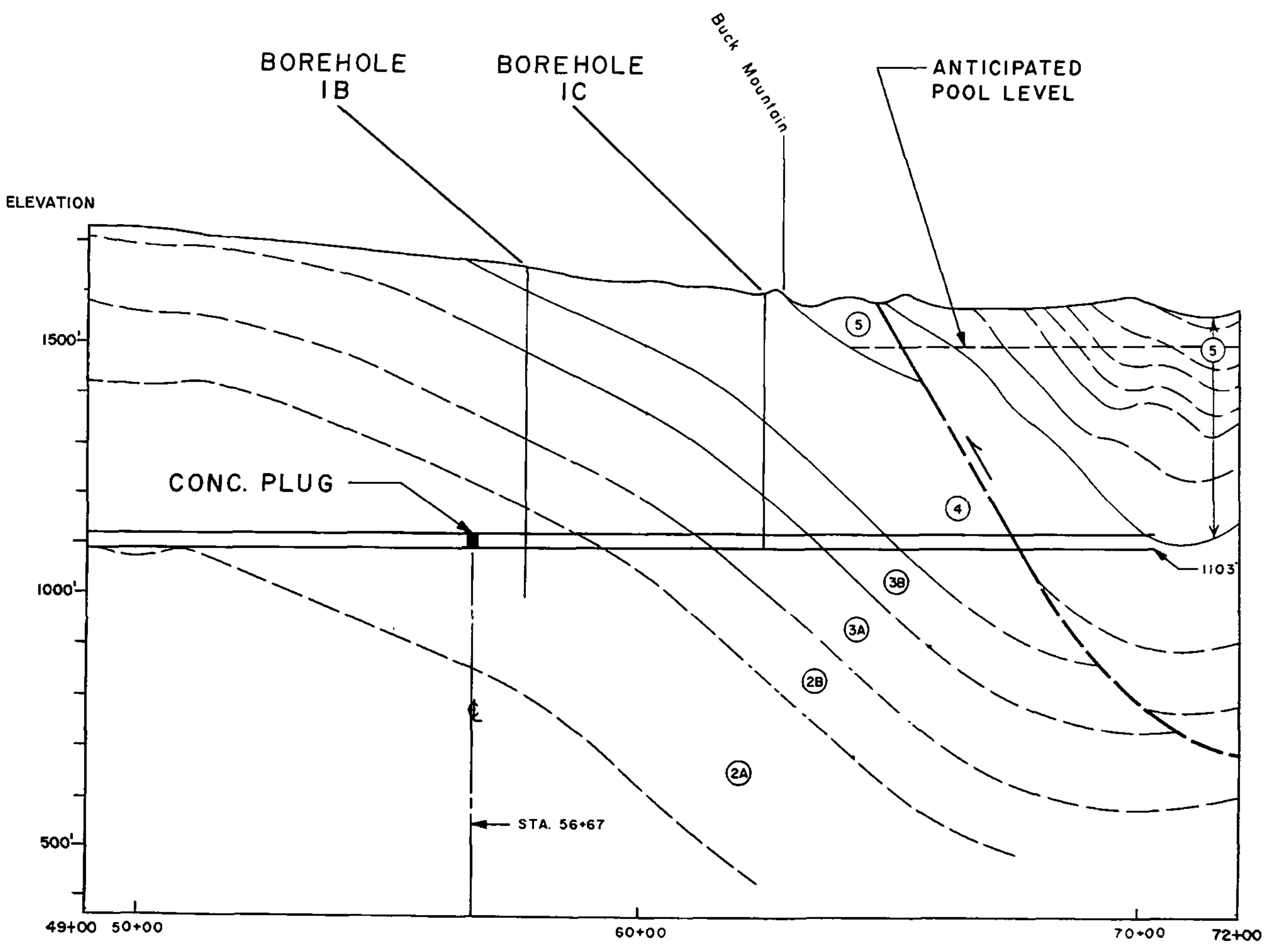
(2B) RED SHALES, SILTSTONES, AND SANDSTONES INTERBEDDED WITH GRAY SANDSTONES

(3A) GRAY TO GREENISH-GRAY SANDSTONE AND ROCK FRAGMENT PEBBLE CONGLOMERATE WITH THIN BEDS OF RED AND GRAY SHALE AND SILTSTONE.

(3B) GRAY INTERBEDDED SANDSTONE AND SILTSTONE WITH A THIN RED SHALE BED NEAR MIDDLE.

(4) LIGHT TO DARK GRAY QUARTZ PEBBLE CONGLOMERATE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO DARK GRAY SANDSTONE; SEVERAL THIN BEDS OF BLACK SHALE WITH CARBONACEOUS PARTINGS.

(5) GRAY TO BUFF SANDSTONE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO BLACK SHALE; SEVERAL THIN TO THICK COAL BEDS AND UNDERCLAYS.

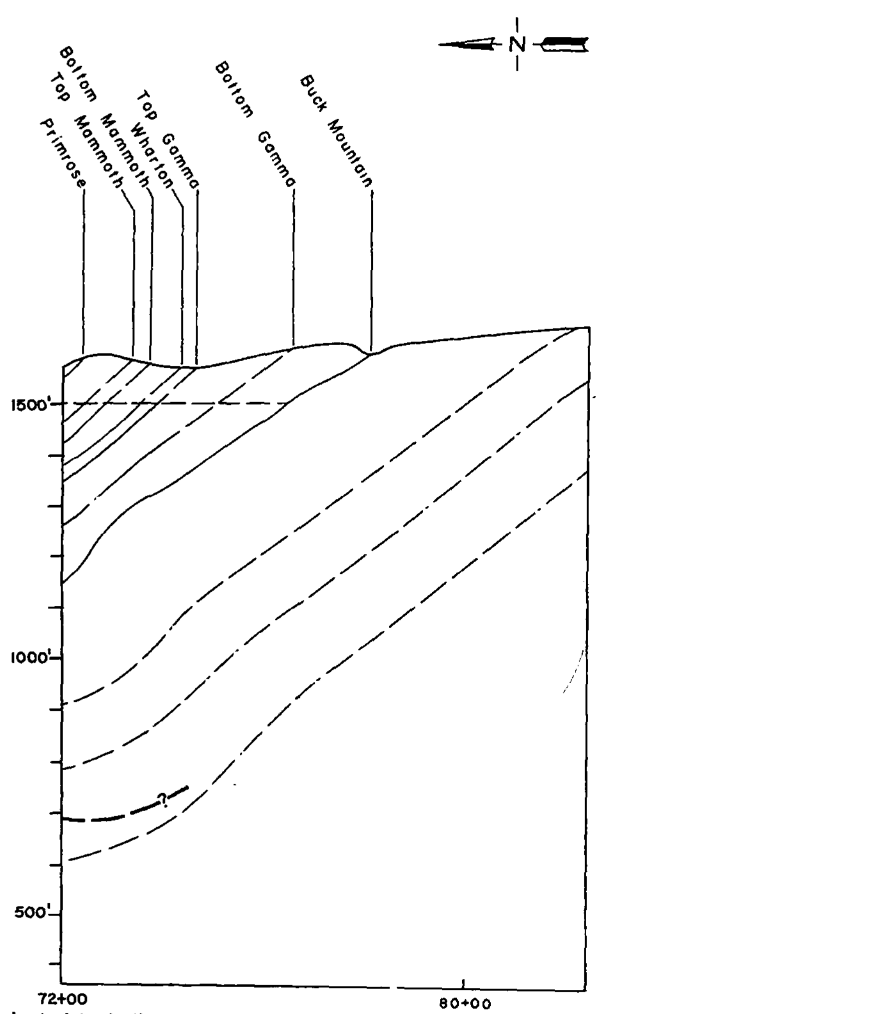


Note: Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

Figure 4c. Cross section through Tunnel No. 1.

TUNNEL NO. 1

1. RED SANDSTONE INTERBEDDED WITH RED SHALE AND SILTSTONE.
2A. GREENISH-GRAY ROCK FRAGMENT PEBBLE CONGLOMERATE AND SANDSTONE INTERBEDDED WITH RED SANDSTONES AND SILTSTONE
2B. RED SHALES, SILTSTONES, AND SANDSTONES INTERBEDDED WITH GRAY SANDSTONES
3A. GRAY TO GREENISH-GRAY SANDSTONE AND ROCK FRAGMENT PEBBLE CONGLOMERATE WITH THIN BEDS OF RED AND GRAY SHALE AND SILTSTONE
3B. GRAY INTERBEDDED SANDSTONE AND SILTSTONE WITH A THIN RED SHALE BED NEAR MIDDLE
4. LIGHT TO DARK GRAY QUARTZ PEBBLE CONGLOMERATE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO DARK GRAY SANDSTONE, SEVERAL THIN BEDS OF BLACK SHALE WITH CARBONACEOUS PARTINGS.
5. GRAY TO BUFF SANDSTONE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO BLACK SHALE; SEVERAL THIN TO THICK COAL BEDS AND UNDERCLAYS.



Note: Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

Figure 4d. Cross section through Tunnel No. 1.

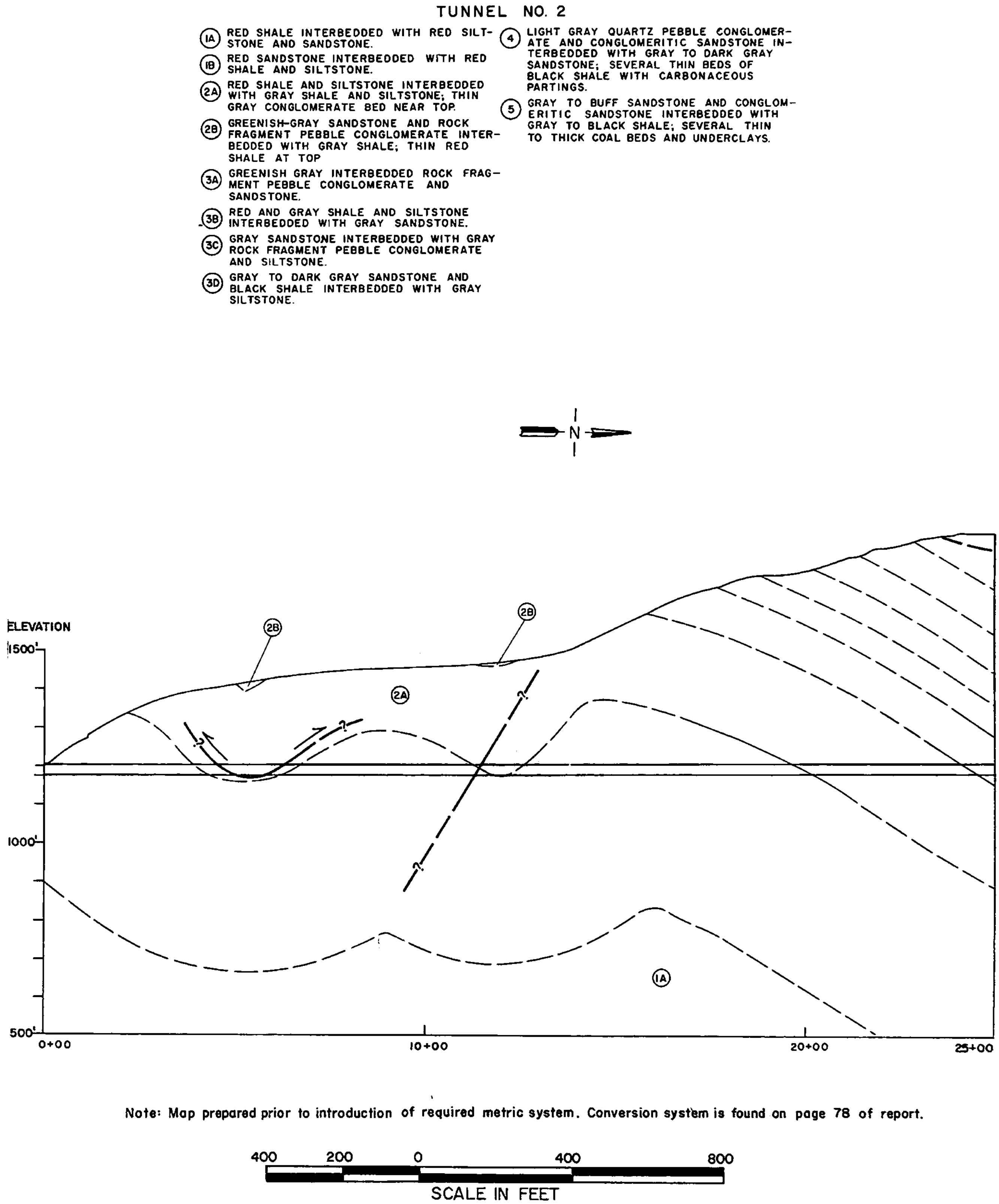


Figure 5a. Cross section through Tunnel No. 2.

TUNNEL NO. 2

(1A) RED SHALE INTERBEDDED WITH RED SILTSTONE AND SANDSTONE

(1B) RED SANDSTONE INTERBEDDED WITH RED SHALE AND SILTSTONE.

(2A) RED SHALE AND SILTSTONE INTERBEDDED WITH GRAY SHALE AND SILTSTONE; THIN GRAY CONGLOMERATE BED NEAR TOP

(2B) GREENISH-GRAY SANDSTONE AND ROCK FRAGMENT PEBBLE CONGLOMERATE INTERBEDDED WITH GRAY SHALE; THIN RED SHALE AT TOP

(3A) GREENISH GRAY INTERBEDDED ROCK FRAGMENT PEBBLE CONGLOMERATE AND SANDSTONE.

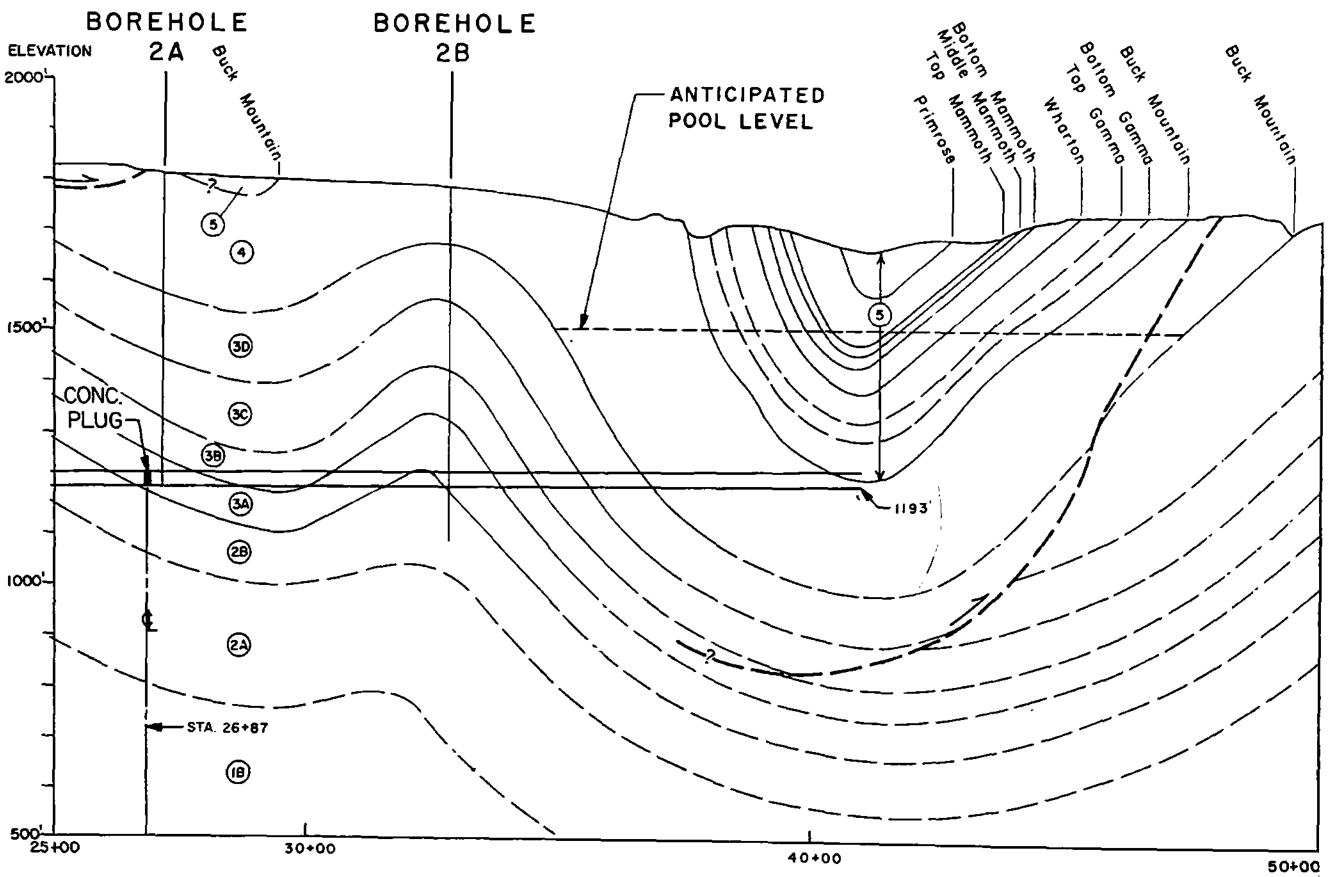
(3B) RED AND GRAY SHALE AND SILTSTONE INTERBEDDED WITH GRAY SANDSTONE.

(3C) GRAY SANDSTONE INTERBEDDED WITH GRAY ROCK FRAGMENT PEBBLE CONGLOMERATE AND SILTSTONE.

(3D) GRAY TO DARK GRAY SANDSTONE AND BLACK SHALE INTERBEDDED WITH GRAY SILTSTONE

(4) LIGHT GRAY QUARTZ PEBBLE CONGLOMERATE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO DARK GRAY SANDSTONE; SEVERAL THIN BEDS OF BLACK SHALE WITH CARBONACEOUS PARTINGS.

(5) GRAY TO BUFF SANDSTONE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO BLACK SHALE, SEVERAL THIN TO THICK COAL BEDS AND UNDERCLAYS



Note: Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

400 200 0 400 800

SCALE IN FEET

Figure 5b. Cross section through Tunnel No. 2.

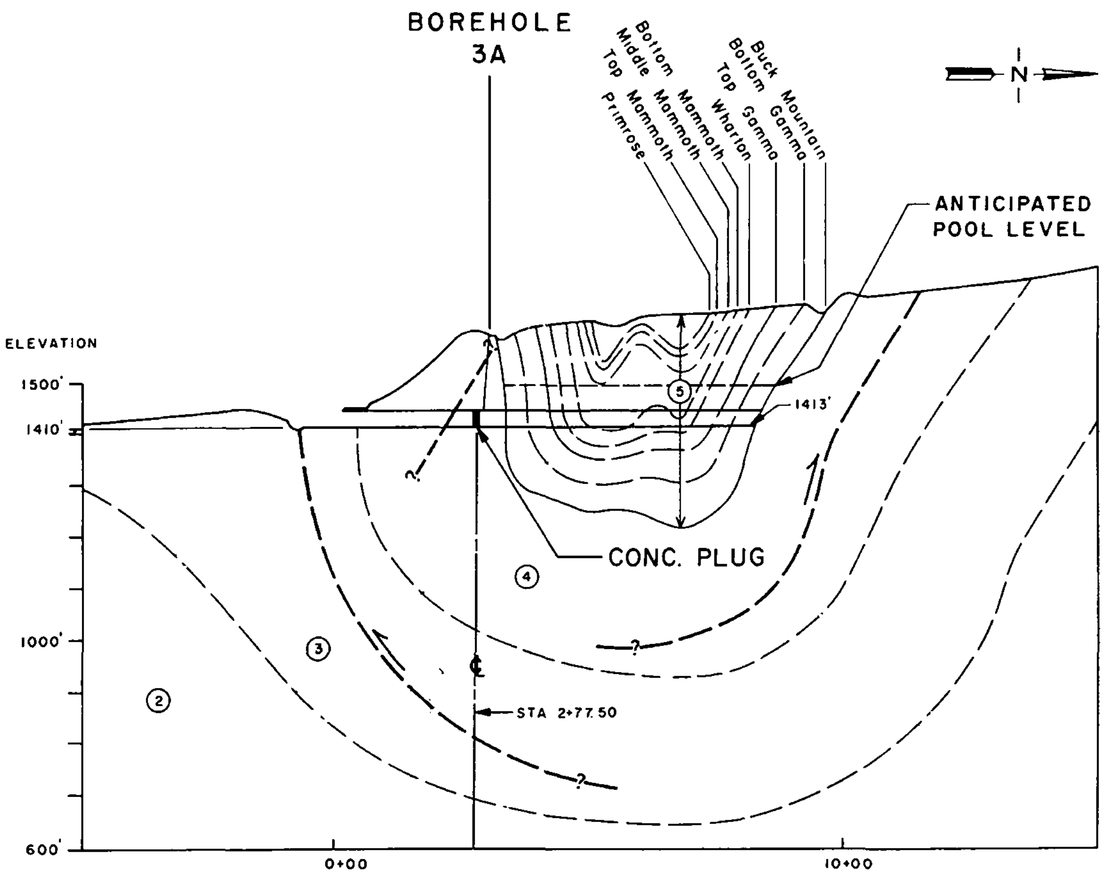
TUNNEL NO. 3

(2) RED SHALE, SILTSTONE AND SANDSTONE INTERBEDDED WITH GRAY TO GREENISH-GRAY SHALE, SILTSTONE, SANDSTONE AND ROCK FRAGMENT PEBBLE CONGLOMERATE

(3) GRAY TO GREENISH-GRAY INTERBEDDED SANDSTONE AND ROCK FRAGMENT PEBBLE CONGLOMERATE; SOME THIN BEDS OF GREENISH-GRAY SHALE AND SILTSTONE

(4) LIGHT TO DARK GRAY QUARTZ PEBBLE CONGLOMERATE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO DARK GRAY SANDSTONE; SEVERAL THIN BEDS OF BLACK SHALE WITH CARBONACEOUS PARTINGS

(5) GRAY TO BUFF SANDSTONE AND CONGLOMERITIC SANDSTONE INTERBEDDED WITH GRAY TO BLACK SHALE; SEVERAL THIN TO THICK COAL BEDS AND UNDERCLAYS



Note: Map prepared prior to introduction of required metric system. Conversion system is found on page 78 of report.

400 200 0 400 800

SCALE IN FEET

Figure 6. Cross section through Tunnel No. 3.

--can be removed or made unavailable. As an example, the Works Progress Administration's Air Sealing Program during the 1930's was aimed at preventing atmospheric oxygen from entering abandoned underground mines. Mine entries were sealed to exclude air but to allow water to leave the mines, and all surface breaks overlying the mine workings were repaired to prevent air from entering the workings. Similarly, if all access to the mine workings at lower elevations is sealed to prevent water from leaving the mine, the water level will rise until it finds relief. As the pool forms, the water displaces the air and inundates the mine workings, preventing further oxidation of the acid-forming material. If a pool level can be achieved whereby virtually all of the mine workings are inundated, the formation of acid mine drainage will cease, and the water quality of the pool overflow will ultimately improve. In this project, the latter technique appeared worthy of investigation.

Another link in the reaction that causes the formation of acid mine drainage can be broken by preventing or limiting the volume of water that can come in contact with oxidized acid-forming material, thereby eliminating or reducing the amount of acid mine drainage being discharged. This technique was also believed to be applicable to this project.

SCOPE AND APPLICABILITY OF PROJECT

In addition to the three discharges from this basin, two other acid discharges enter Catawissa Creek via water-level tunnels. One, from the Jeansville Basin, enters Catawissa Creek upstream from the basin. The other, from the North Green Mountain Basin, flows into Tomhicken Creek, which enters Catawissa Creek several miles downstream. These five discharges have caused Catawissa Creek to be acid from the point where the Jeansville Basin discharge enters throughout its remaining length until it flows into the North Branch of the Susquehanna River near Catawissa, Pennsylvania. If water quality in Catawissa Creek is to be improved, some means of abating these water-level tunnel discharges must be developed.

In addition to these three basins, water-level tunnels were driven into, and between, other basins in Pennsylvania's anthracite field to reduce water handling costs during mining. Therefore, if the technique of sealing these water-level tunnels can be perfected, this technique would have wide applicability throughout this field. Consequently, a research and development project was recommended for the South Green Mountain Basin. This project was to be comprised of the following measures:

1. Reconstruction of a portion of Catawissa Creek through and along the edge of a strip mined area where the entire streamflow is intercepted by the strip mine and is directed into the underlying deep mine workings in the basin; and

2. Construction of watertight seals in the three water-level tunnels presently draining the basin.

The purpose of the first preventive measure was simply to keep a large volume of water out of the mined basin, thus preventing that water from contacting acid-forming material. The purpose of the second preventive measure was to inundate about 80 percent of the mine workings by creating overflows from the basin at much higher elevations. This inundation would take advantage of the phenomenon that has been observed many times where mine drainage discharge quality has improved significantly after inundation has naturally occurred. Two factors are involved: (1) further oxidation of the inundated acid-forming material is prevented; and (2) stratification of water occurs in a quiescent pool between the acid mine water, which is heavier, and the groundwater, which is lighter.

WORK OBJECTIVES

The immediate objectives of the project were to:

1. Determine the effect on water quality of the drainage from the South Green Mountain Basin by reconstructing a portion of Catawissa Creek and sealing the three water-level tunnels draining the basin;

2. Maintain complete records, including cost, relative to construction, as well as operation and maintenance, of the proposed preventive measures; and

3. Verify the rational method that was used to estimate total and individual mine drainage volumes, constituents, and characteristics, and the percent reductions attributable to the separate preventive measures.

II

CONCLUSIONS

Based upon the effort expended during this project, it can be concluded that:

1. Reconstruction of the Catawissa Creek streambed has proved to be effective in reducing the volume of mine drainage being discharged, and consequently the acid loading, from the South Green Mountain Basin. The streambed reconstruction has caused a reduction of 0.253 m^3/s in the flows from Tunnel Nos. 2 and 3, based upon average monthly streamflow data collected during a year of normal precipitation. Although the flow reduction from Tunnel No. 3 has been accompanied by an increase in acidity of its remaining flow, a decrease of 830 kilograms per day in the acid load from Tunnel No. 3 has occurred.

2. Approximately 518 meters of streambed was reconstructed to handle a maximum design flow of 20.2 m^3/s. This reconstruction was accomplished at a total cost, based on 1969 price levels, of $30,529.68 or $58.94 per meter of streambed reconstructed. Over a five-year interval since completion of construction, no maintenance has been required; consequently, no operating or maintenance costs have been incurred.

3. The geologic investigations of the South Green Mountain Basin confirm that:

(a) It is feasible to construct effective seals in the three tunnels and to contain the anticipated impoundment within the basin; and

(b) Although minor leakage is probable, the basin will contain water at a sufficient elevation to improve the quality of the mine drainage discharges.

4. The Department wished to pursue the objective of constructing watertight seals, thereby improving the quality of the mine drainage discharges, in an orderly fashion by sealing one tunnel and evaluating the effectiveness of that seal before sealing the other tunnels. Several Department-requested design changes added to the project's complexity during inflationary cost spirals, resulting in the bids that were deemed excessive. Consequently, the seals were not constructed. Since construction did not occur, no conclusions regarding this concept can be formulated.

5. Although there is surely a correlation between precipitation and

flows in Catawissa Creek and the basin's water-level tunnels, the weekly (and often less frequent) flow and quality data collected at these locations were obviously insufficient to verify the rational method of estimating mine drainage flows and flow reductions resulting from Catawissa Creek streambed reconstruction. In addition, several upstream complicating factors in estimating Catawissa Creek streamflow, such as public water supply reservoirs, strip mine impoundments with intermittent withdrawals and ultimate discharge outside the Study Area, varying wastewater flows from a municipal wastewater collection system, and changes in runoff characteristics caused by major highway construction, must be considered. Continuous flow and precipitation records for the Study Area extending for one hydrologic year before, and one hydrologic year after, construction are felt necessary as a minimum to enable such determination.

III

RECOMMENDATIONS

Based on the conclusions drawn during this project, the following recommendations are made:

1. For sound design of watertight seals, detailed geologic investigations comprising a study of the areal geology, surface investigations, internal investigations of the water-level tunnels, core borings over the tunnels and the potential overflow points, and physical and chemical rock tests should be performed.

2. Technical feasibility of watertight seals was established for this site. Even though the initial cost of sealing the water-level tunnels was high, this technique was deemed a viable one for this basin and for similar areas, and future demonstration projects should be considered.

3. An intensive pre- and postconstruction monitoring program should be implemented to verify the rational method of estimating mine drainage flows. This intensive monitoring program comprising continuous flow and precipitation records for a project should extend as a minimum for one hydrologic year before, and one hydrologic year after, construction.

4. Periodic inspection should be performed on the streambed reconstruction so that any needed maintenance can be accomplished to maintain its effectiveness.

IV

WORK PROCEDURES

ESTABLISHMENT OF PROJECT SCHEDULE

In order to achieve the project objectives, an orderly progression of events was planned over a total project time of 4.5 years.

During the first year, it was proposed to shore the water-level tunnels so that internal mapping of the rock encountered in the tunnels could be accomplished. In addition, geologic investigations of the area were planned. Rock cores over the tunnels were also proposed to be taken to complete the collection of geologic information deemed necessary to determine, with a high degree of assurance, the feasibility of sealing the three water-level tunnels. During this same time, the proposed gauging, sampling, and analyzing program was planned to be initiated so that data could be collected on a weekly basis for one full year before construction was accomplished. Concurrently, the preparation of construction plans and technical specifications for reconstructing the Catawissa Creek stream channel and for sealing the three water-level tunnels was planned.

During the second year, reconstruction of the stream channel as well as sealing of three water-level tunnels was contemplated. General and resident supervision of this construction was also planned. In addition, observation holes were to be drilled into the tunnels so that the pool level behind the seals could be monitored as desired. Following reconstruction of the stream channel and before sealing of the water-level tunnels, gauging, sampling, and analyzing of the tunnel discharges on a weekly basis was planned.

After the tunnels were sealed near the end of the second year, the pool level behind the seals would be monitored and water quality determined on a weekly basis. It was believed that perhaps 6 to 12 months would elapse before the pool would overflow at the anticipated overflow points. This portion of the program was scheduled for one year.

Once the pool began to overflow and during the next year, the overflows were to be gauged, sampled, and analyzed to determine improvement in discharge quality that was expected to occur. No change in the discharge flow rate was expected from the basin as a result of sealing the water-level tunnels. Although one year was not felt to be sufficient time for overflow quality to stabilize, it was believed that a trend toward stabilization could be observed during this time.

Finally, six months were allocated to write a draft report, have it reviewed, and complete the final report on the project.

For a number of reasons, which will be discussed in subsequent chapters of this report, sealing of the water-level tunnels was not accomplished. Consequently, no conclusions concerning the effectiveness of this technique could be drawn, nor could any verification of the rational method of determining mine drainage flows be made.

V

RECONSTRUCTION OF CATAWISSA CREEK STREAMBED

PREDESIGN CONDITIONS

Many years ago, the deep mine operator working in the eastern portion of the basin had constructed a new streambed for Catawissa Creek, approximately 1,433 meters long, so that most of its flow would bypass his deep mine workings. This new streambed appears as the relatively straight alternate channel adjacent to the eastern end of the basin on Figure 7. Subsequently, however, an unidentified strip mine operator excavated a ditch leading from this new channel into an inactive strip mine, which had cut into the underlying deep mine workings, in the eastern end of the basin. All of the Catawissa Creek streamflow, therefore, then entered the deep mine workings via the inactive strip mine and eventually returned to Catawissa Creek through Tunnel No. 3.

After flow through Tunnel No. 3 became restricted by falling debris at its mouth, part of this flow passed over the hump in the bottom of the basin to the west and discharged via Tunnel No. 2 into Catawissa Creek. This latter bypassing of streamflow to Tunnel No. 2 only occurred during those times when Tunnel No. 3 could not completely accommodate large flows associated with high runoff from precipitation or melting snow. Because there was no way to determine exactly what portions of the streamflow were bypassed to Tunnel No. 2, it has been assumed that reductions in flow achieved by streambed reconstruction would occur at Tunnel No. 3. Accordingly, any Tunnel No. 3 flow reduction that was confirmed by flow measurements taken at Tunnel No. 3 before and after streambed construction (before clearing of Tunnel No. 3) would be less than actual reductions at Tunnel Nos. 2 and 3.

DESIGN PHASE

Before streambed reconstruction (and other project work) could proceed, it was necessary to secure easements from the affected property owners. Information concerning work areas, as well as the manner and extent to which the various properties would be affected by the project, was provided to the Commonwealth of Pennsylvania. Commonwealth representatives then secured the cooperation of the property owners via signed agreements, which would allow the project to be accomplished on their properties. An example of this form is included in Appendix A.

ENGINEERING CALCULATIONS

In order to establish a sufficient channel for the reconstructed stream-

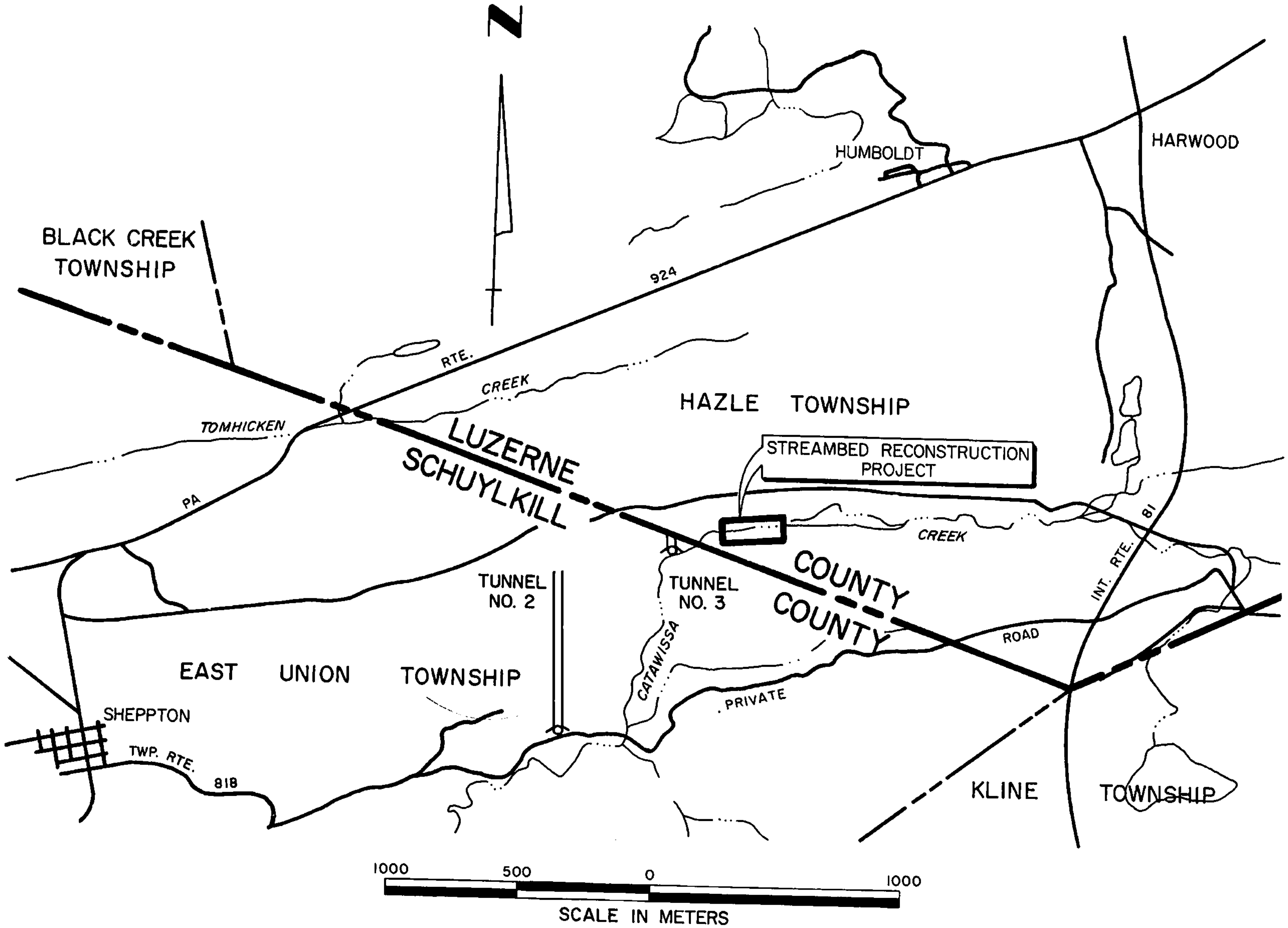


Figure 7. Plan view of proposed Catawissa Creek streambed reconstruction.

bed, the streamflow records on Wapwallopen Creek near Berwick, Pennsylvania, the closest USGS gaging station, were examined. The 48-year record revealed a peak flow of 88.9 m^3/s from the 114 square-kilometer drainage area tributary to the gaging station. This August 18, 1955 storm also provided maximum flow readings for many gaging stations in the general area. Catawissa Creek has terrain similar to that of Wapwallopen Creek. However, there are numerous reservoirs and strip pits that would retain runoff on the upstream 25.9 square-kilometer drainage area of Catawissa Creek. Consequently, streamflow in Catawissa Creek would probably have been less than that experienced in Wapwallopen Creek. On a proportionate basis, the peak flow in Catawissa Creek would have been 20.2 m^3/s. Considering a 3.05-meter base, 1 1/2 horizontal to 1 vertical side slopes, a slope of .0035, and a roughness coefficient of .03, a water depth of about 1.8 meters would accommodate a flow of 20.2 m^3/s. The streambed was so designed.

TECHNICAL PLANS AND SPECIFICATIONS

It was planned to complete the Catawissa Creek streambed construction at about one year after the gauging, sampling, and analyzing program was initiated. Consequently, construction plans and technical specifications were prepared and submitted to the Commonwealth during May 1969. Following review by the Commonwealth and minor revisions, the plans and specifications were resubmitted for a July 24, 1969 bid opening. The construction cost estimate of $33,222 for reconstruction of approximately 518 meters of streambed was as follows:

Category	Volume (cu yd*)	Unit Price	Estimated Cost
Channel Excavation	15,600	$ 1.30	$ 20,280.
Rock Excavation	1,700	4.00	6,800.
Rolled Embankment	720	1.10	792.
Stripping	100	1.50	150.
Clearing and Grubbing	Lump Sum		5,200.
		Total	$ 33,222.

*The English system of measurement was required in the technical specifications and bidding documents and, therefore, is used in this discussion. A table for conversion to the metric system is included on page 78.

The streambed reconstruction was advertised, and bids were opened on July 24, 1969. One bid in the amount of $95,600 was received. Because the bid, nearly 200 percent over the estimate, was considered unreasonable, a decision was made to readvertise. Bids were opened again on August 28, 1969. Six bids ranging from a low of $38,959 to a high of $61,445.60 were received.

CONSTRUCTION PHASE

Following a review of the contractors' bids, financial information, as well as equipment and personnel availability, the construction contract was awarded to Wyoming Sand and Stone Company in the amount of $38,959. The contractor was authorized to proceed with the work on December 1, 1969;

streamflow was turned into the reconstructed streambed on February 25, 1970; and construction was completed on the following day, February 26, 1970. The total construction cost of $30,529.68 was derived as follows:

Category	Volume (cu yd)	Unit Price	Cost
Channel Excavation	15,222.26	$ 1.37	$20,854.50
Rock Excavation	835.74	8.98	7,504.95
Rolled Embankment	588.00	0.95	558.60
Stripping	81.48	1.37	111.63
Clearing and Grubbing	Lump Sum		1,500.00
		Total	$30,529.68

The streambed reconstruction was completed for 8.1 percent less than the estimated construction cost of $33,222.

POSTCONSTRUCTION PHASE

The contractor turned streamflow out of the basin and into the reconstructed streambed on February 25, 1970. He then completed his work the following day. A view of the reconstructed streambed, looking downstream from its upstream end, is shown in Figure 8. This reconstructed streambed has easily accommodated the runoff from two significant rainfalls. During June 21 thru 23, 1972, Hurricane Agnes dropped 20.6 centimeters and 20.1 centimeters of rain at the Zion Grove and the Tamaqua 4N Dam reporting stations, respectively. This storm exceeded the return frequency of 1 in 100 years for these stations. In addition, 15.1 centimeters and 14.7 centimeters of rain were recorded at the Mahanoy City 2N station (this station replaced the Zion Grove station in May 1975) and the Tamaqua 4N Dam station, respectively, during Hurricane Eloise, September 23 thru 27, 1975. This storm exceeded the return frequency of 1 in 50 years for these stations. Consequently, the reconstructed streambed is considered entirely adequate for the foreseeable future. The Zion Grove reporting station was located about 8 kilometers west of the basin. The Tamaqua 4N Dam and the Mahanoy City 2N stations are approximately 13 kilometers east and 6.4 kilometers south, respectively, of the basin.

EFFECTIVENESS OF STREAMBED RECONSTRUCTION

To determine the effectiveness of construction of the project, a flow and water quality monitoring program was initiated on March 4, 1969. This program was conducted weekly through December 15, 1969 at Tunnel Nos. 1, 2, and 3 as well as at Catawissa Creek immediately upstream from the streambed construction area. However, from the middle of December 1969 until April 2, 1970, flow and quality data were only occasionally obtained at these four locations because of significant accumulations of snow in the project area. Flow and water quality data collection was discontinued at the Catawissa Creek upstream site immediately after completion of the streambed construction.

Weekly flow and water quality data collection was resumed at the three

Figure 8. Catawissa Creek reconstructed streambed, March 9, 1970.

tunnels on April 2 and continued through June 11, 1970. Subsequently, data collection was performed monthly through January 15, 1971 with some additional data being obtained during April, May, and July 1971. These data are presented for Tunnel Nos. 1, 2, and 3 and for Catawissa Creek upstream in Tables 1, 2, 3, and 4, respectively. As can be seen, the flow and quality data are arrayed by month for the periods before streambed reconstruction (March 1969 through February 1970) and after streambed construction (March 1970 and after).

At Tunnel No. 1, no significant changes in water quality were noted during these two periods although variations occurred. Its pH ranged between 3.7 and 4.3 during the first year, and from 3.7 to 4.2 thereafter. Similarly, the acidity of this discharge ranged from 50 to 83 mg/l during the first year, and between 40 and 92 mg/l thereafter. Its other constituents - iron, sulfate, total solids, aluminum, and manganese - - exhibited similar variations but no marked changes.

Similarly, no significant differences were evident in Tunnel No. 2 water quality during these two periods although variations were noted. Its pH ranged between 3.4 and 3.8 during the first year, and from 3.5 to 3.8 thereafter. The acidity of this discharge ranged from 67 to 110 mg/l during the first year, and between 64 and 104 mg/l thereafter. Its other constituents - - iron, sulfate, total solids, aluminum, and manganese - - showed similar variations but no significant changes.

On the other hand, some deterioration in water quality at Tunnel No. 3 appeared to occur after streambed reconstruction due to the elimination of the dilutional effect from Catawissa Creek flows. Its pH ranged between 3.9 and 6.5 before streambed reconstruction, and from 3.2 to 4.0 thereafter. Similarly, the acidity of this discharge ranged from 30 to 103 mg/l during the first year, and between 44 and 136 mg/l thereafter. Its iron concentration also increased from a range of 0.2 to 1.3 mg/l before streambed reconstruction to between 0.4 and 10.9 mg/l thereafter. Its other constituents - sulfate, total solids, aluminum, and manganese - - did not appear to change drastically although variations were noted.

Flow and water quality were only monitored in Catawissa Creek upstream from the streambed reconstruction during the year immediately preceding construction, and for two weeks thereafter. Catawissa Creek water was slightly acid during this time as indicated by its pH ranging between 4.5 and 7.0 and its acidity varying from 22 to 71 mg/l. The iron concentration in Catawissa Creek water ranged from 0.2 to 2.5 mg/l. The acid and iron concentrations noted in Catawissa Creek water are the result of the stream flowing across the extensively mined Jeansville Basin, which underlies its headwaters. Its other constituents - sulfate, total solids, aluminum, and manganese - - also indicate some quality degradation.

An evaluation of flow data could not be undertaken without a concurrent review of rainfall records. Therefore, precipitation data were tabulated for the Zion Grove and Tamaqua 4N Dam stations, located 8 kilometers west and 13 kilometers east of the basin, respectively. These data are summarized by month in Table 5 for March 1969 through February 1970, prior to streambed

TABLE 1. AVERAGE FLOW AND WATER QUALITY DATA - TUNNEL NO. 1

Month	Flow (m^3/s)	pH Range	Acidity (mg/l)	Acidity (kg/day)	Total Iron (mg/l)	Total Iron (kg/day)	Sulfate (mg/l)	Total Solids* (mg/l)	Aluminum* (mg/l)	Manganese* (mg/l)
Before Streambed Reconstruction										
March 1969	0.238	3.7-4.0	68	1,400	0.5	10.4	87	158	5.7	1.1
April	0.363	3.8-3.9	52	1,630	0.3	9.5	65	152	3.9	0.6
May	0.347	3.9	50	1,590	0.4	11.8	62	165	2.5	0.7
June	0.319	3.9	52	1,430	0.5	13.6	73	139	2.5	0.8
July	0.387	3.7-3.9	66	2,210	0.4	13.2	96	165	6.0	1.2
August	0.495	3.8-4.3	68	2,900	0.3	12.7	89	231	5.5	1.1
September	0.273	3.8-3.9	65	1,530	0.3	7.3	91	154	3.9	0.8
October	0.104	3.8	80	720	0.3	2.7	101	182	3.5	1.0
November	0.163	3.8	83	1,170	0.5	7.3	108	237	3.1	0.9
December	0.238	3.7-3.9	74	1,520	0.4	8.2	91	167	2.5	1.1
January 1970**	-	3.9	72	-	0.5	-	75	-	3.4	1.1
February	0.530	3.7-3.9	68	3,110	0.6	27.7	83	147	2.6	0.8
After Streambed Reconstruction										
March 1970	0.403	3.9-4.0	72	2,490	0.3	10.4	79	-	-	-
April	1.007	3.7-3.9	50	4,350	0.2	17.2	76	165	0.4	0.5
May	0.548	3.9	54	2,550	0.2	9.5	65	161	2.2	0.6
June	0.192	3.9	62	1,030	0.8	13.2	80	-	-	-
July**	0.184	3.9	64	1,020	0.5	8.2	85	-	-	-
August**	0.134	3.9	90	1,040	0.4	4.5	98	-	-	-
September#	-	-	-	-	-	-	-	-	-	-
October#	-	-	-	-	-	-	-	-	-	-
November#	-	-	-	-	-	-	-	-	-	-
December**	0.363	4.2	68	2,130	0.1	3.2	71	-	-	-
January 1971**	0.293	3.8	52	1,320	0.5	12.7	71	-	-	-
April**	0.447	3.8	40	1,540	0.7	27.2	74	136	3.3	0.5
May**	0.429	3.8	44	1,630	0.2	7.3	48	-	-	-
July**	0.265	4.0	92	2,100	0.2	4.5	74	-	-	-

* One analysis monthly.
** One sample only.
\# No data available.

TABLE 2. AVERAGE FLOW AND WATER QUALITY DATA - TUNNEL NO. 2

Month	Flow	pH	Acidity		Total Iron		Sulfate	Total Solids*	Aluminum*	Manganese*
	(m^3/s)	Range	(mg/l)	(kg/day)	(mg/l)	(kg/day)	(mg/l)	(mg/l)	(mg/l)	(mg/l)
Before Streambed Reconstruction										
March 1969	0.153	3.7-3.8	67	890	0.5	6.8	116	-	8.3	1.7
April	0.223	3.7-3.8	76	1,470	0.4	7.7	84	201	5.3	1.1
May	0.138	3.6-3.7	72	860	0.8	9.5	87	196	6.6	1.1
June	0.102	3.6-3.7	79	700	1.0	8.6	115	202	8.0	1.5
July	0.197	3.4-3.6	110	1,870	0.8	13.6	159	246	7.9	2.1
August	0.196	3.7-3.8	90	1,520	0.5	8.6	124	-	-	-
September	0.124	3.6	92	990	0.7	7.7	142	235	5.0	1.8
October	0.078	3.6-3.7	87	580	0.4	2.7	145	-	-	-
November	0.052	3.6-3.8	92	410	0.5	2.3	137	250	5.0	1.0
December	0.076	3.7	81	530	0.7	4.5	104	196	5.6	1.5
January 1970#	-	-	-	-	-	-	-	-	-	-
February	0.158	3.3-3.7	108	1,470	1.7	23.1	113	200	4.4	1.2
After Streambed Reconstruction										
March 1970	0.129	3.7-3.8	92	1,030	0.6	6.8	105	-	-	-
April	0.237	3.6-3.7	92	1,890	0.6	12.2	101	316	0.4	0.5
May	0.128	3.7	83	920	0.4	4.5	94	250	5.8	0.7
June	0.103	3.6	88	780	1.8	15.9	106	-	-	-
July**	0.085	3.6	100	730	1.4	10.4	125	-	-	-
August**	0.109	3.6	92	860	0.7	6.4	120	-	-	-
September**	0.108	3.5	104	970	2.9	26.8	134	-	-	-
October	0.082	3.5-3.6	92	650	1.7	12.2	119	-	-	-
November#	-	-	-	-	-	-	-	-	-	-
December**	0.082	3.6	80	570	0.4	2.7	103	-	-	-
January 1971**	0.089	3.6	76	590	1.1	8.6	97	-	-	-
April**	0.111	3.6	84	800	0.9	8.6	201	214	5.4	0.7
May**	0.104	3.7	64	580	0.2	1.8	65	-	-	-
July**	0.056	3.7	92	440	0.7	3.2	130	-	-	-

* One analysis monthly.
** One sample only.
\# No data available.

TABLE 3. AVERAGE FLOW AND WATER QUALITY DATA - TUNNEL NO. 3

Month	Flow (m^3/s)	pH Range	Acidity (mg/l)	Acidity (kg/day)	Total Iron (mg/l)	Total Iron (kg/day)	Sulfate (mg/l)	Total Solids* (mg/l)	Aluminum* (mg/l)	Manganese* (mg/l)
Before Streambed Reconstruction										
March 1969	0.410	4.7-6.3	30	1,060	1.5	53	61	-	-	1.6
April	0.604	4.5-4.9	33	1,720	0.6	31	45	151	1.2	1.6
May	0.561	4.3-4.6	39	1,890	0.9	44	49	133	1.6	1.4
June	0.249	3.9-4.1	51	1,100	1.3	28	85	177	2.3	2.3
July	0.298	3.9-4.2	50	1,290	1.3	34	123	237	1.6	2.9
August	0.618	3.9-4.3	56	2,990	0.5	27	117	246	1.2	2.5
September	0.310	4.5-5.1	103	2,750	0.9	24	130	387	2.2	2.9
October	0.052	5.3-6.5	33	150	0.4	2	117	241	0.1	2.4
November	0.134	4.5-6.2	41	480	0.2	2	95	172	0.1	1.1
December	0.180	4.6-4.8	57	880	0.6	10	89	200	1.2	2.5
January 1970#	-	-	-	-	-	-	-	-	-	-
February	0.412	4.6	48	1,700	0.7	25	54	59	1.1	1.0
After Streambed Reconstruction										
March 1970	0.058	3.9	68	340	0.7	4	75	-	-	-
April	0.206	3.8-4.0	64	1,140	0.4	7	55	118	0.3	0.2
May	0.074	3.7-.39	57	360	1.1	7	54	137	3.0	0.4
June	0.018	3.5	82	130	7.1	11	82	-	-	-
July**	0.019	3.4	96	150	10.8	18	115	-	-	-
August**	0.064	3.9	90	490	0.4	2	98	-	-	-
September	0.003	3.2	136	40	4.8	1	148	-	-	-
October	0.045	3.3-3.7	100	390	10.9	43	116	-	-	-
November#	-	-	-	-	-	-	-	-	-	-
December**	0.013	3.7	48	50	1.6	2	71	-	-	-
January 1971**	0.047	3.6	52	210	3.3	13	65	-	-	-
April**	0.057	3.7	52	260	1.7	9	75	142	2.1	0.5
May**	0.085	3.8	44	320	1.8	13	37	-	-	-
July**	0.015	3.6	88	120	5.5	7	97	-	-	-

* One analysis monthly.
** One sample only.
\# No data available.

TABLE 4. AVERAGE FLOW AND WATER QUALITY DATA - CATAWISSA CREEK UPSTREAM

Month	Flow (m^3/s)	pH Range	Acidity		Total Iron		Sulfate (mg/l)	Total Solids* (mg/l)	Aluminum* (mg/l)	Manganese* (mg/l)
			(mg/l)	(kg/day)	(mg/l)	(kg/day)				
Before Streambed Reconstruction										
March 1969	0.258	5.0-6.9	22	490	1.6	36	55	218	0.05	1.6
April	0.486	4.8-5.3	32	1,340	0.3	13	39	191	0.4	1.8
May	0.447	4.8-5.7	46	1,770	0.4	15	51	139	1.3	1.6
June	0.140	4.5-5.6	57	690	1.0	12	114	268	<0.05	3.3
July	0.179	4.5-6.1	51	790	0.5	8	157	365	<0.05	3.5
August	0.460	4.5-4.6	65	2,580	0.2	8	111	186	4.2	1.8
September	0.177	6.1-6.7	40	610	0.5	8	136	260	0.05	3.0
October	0.036	6.7-7.0	15	50	0.5	1	122	313	0.4	2.8
November	0.078	5.8-6.5	43	290	0.6	4	104	200	0.2	1.7
December	0.122	4.8-6.4	71	750	0.5	5	87	319	2.6	3.0
January 1970**	-	6.4	24	-	2.5	-	96	261	5.3	2.3
February	0.402	4.8-5.1	46	1,600	0.3	10	48	98	0.3	1.0
After Streambed Reconstruction										
March 1970	0.310	5.6-6.0	44	1,170	0.2	5	44	-	-	-

* One analysis monthly.
** One sample only.

TABLE 5. PRECIPITATION RECORDED AT AREA REPORTING STATIONS

Month	Zion Grove Normal(*) (cm)	Zion Grove Actual (cm)	Zion Grove Departure (cm)	Tamaqua 4N Dam Normal(*) (cm)	Tamaqua 4N Dam Actual (cm)	Tamaqua 4N Dam Departure (cm)
Before Streambed Reconstruction						
March 1969	7.09	4.24	-2.85	9.50	6.48	-3.02
April	9.78	11.33	+1.55	11.05	10.67	-0.38
May	9.25	9.09	-0.16	10.19	8.61	-1.58
June	7.29	8.20	+0.91	8.94	7.57	-1.37
July	9.45	18.49	+9.04	11.05	22.28	+11.23
August	10.85	7.92	-2.93	11.48	12.14	+0.66
September	8.89	3.23	-5.66	9.93	5.89	-4.04
October	7.70	4.19	-3.51	8.56	5.69	-2.87
November	9.02	11.96	+2.94	10.74	12.75	+2.01
December	7.24	11.71	+4.47	8.61	12.07	+3.46
January 1970	5.59	1.04	-4.55	7.62	1.14	-6.48
February	5.31	7.54	+2.23	7.70	9.78	+2.08
After Streambed Reconstruction						
March 1970	7.09	5.11	-1.98	9.50	8.08	-1.42
April	9.78	10.41	+0.63	11.05	11.58	+0.53
May	9.25	7.82	-1.43	10.19	8.86	-1.33
June	7.29	9.68	+2.39	8.94	8.61	-0.33
July	9.45	14.30	+4.85	11.05	21.77	+10.72
August	10.85	8.08	-2.77	11.48	5.28	-6.20
September	8.89	5.77	-3.12	9.93	6.93	-3.00
October	7.70	12.47	+4.77	8.56	14.20	+5.64
November	9.02	10.41	+1.39	10.74	18.14	+7.40
December	7.24	7.11	-0.13	8.61	3.15	-5.46
January 1971	5.59	4.32	-1.27	7.62	5.21	-2.41

* Based upon 19 years of record (1952-1970).

reconstruction, and for 11 months thereafter. On an annual basis, there appeared to be little difference in precipitation over the basin during these two periods.

As indicated on Table 1, flows from Tunnel No. 1 were considerably higher during March, April, and May 1970 than for the comparable period in 1969. The higher 1970 flows resulted from significant snow melts during those months. Little such additional contributions to flow occurred during March, April, and May 1969. During other times, the flow from Tunnel No. 1 varies with precipitation and runoff that have occurred within 12 to 72 hours prior to the flow measurement.

Slightly higher flows appeared to have occurred from Tunnel No. 2 before streambed reconstruction when compared to flows afterward, based on the flow data summarized in Table 2. However, prior to streambed reconstruction, some of Catawissa Creek streamflow entering the basin when flow was high passed over the saddle immediately west of Tunnel No. 3 and actually contributed to Tunnel No. 2 flow. A rockfall that occurred some years ago at the mouth of Tunnel No. 3 had severely restricted Tunnel No. 3 flow, thereby causing some diversion of flow to Tunnel No. 2 as described above. This flow diversion was apparent from the definite odor of sanitary wastewater noted in Tunnel No. 2 flows on several occasions. The untreated wastewater in Catawissa Creek streamflow originates in McAdoo Borough, situated near the creek's headwaters.

As was expected, Tunnel No. 3 flow was dramatically reduced after streambed reconstruction when compared to its flow before construction was completed. The flow that should have emitted from the mined area contributing water to Tunnel No. 3 before streambed reconstruction should have been the measured Tunnel No. 3 flow less the measured Catawissa Creek upstream flow. However, part of this Catawissa Creek flow that entered the basin was diverted to Tunnel No. 2 during this time as previously described. On the other hand, because of the restriction in Tunnel No. 3 flow, higher flow measurements may have resulted from water being impounded in the mined areas in this part of the basin. Consequently, the best way of describing the effectiveness of the streambed reconstruction is to conclude that, if this streambed reconstruction had been completed one year earlier, some 0.253 cubic meters per second (the average monthly streamflow during this year of normal precipitation) would not have passed through the basin to emerge from Tunnel Nos. 2 and 3.

It is recognized that measuring flows once a week, or less frequently on occasion, at these three tunnels and in Catawissa Creek will not provide the best flow data. The flows at these four locations are subject to wide fluctuations connected with runoff resulting from precipitation or snow melt. However, general orders of magnitude can at least be developed from the flow measurements.

As displayed in Table 6, there has been an average acid load reduction from Tunnel No. 3 of 830 kilograms/day, while the iron load has increased slightly. These figures are based upon average monthly water quality data obtained over comparable periods at Tunnel No. 3 before and after Catawissa Creek streambed reconstruction, and assuming that an average flow of 0.253 m^3/s was diverted from the basin.

Since no maintenance on the channel is required, and no operating costs are involved, the average annual cost of abatement decreases yearly. Based upon an average acid load reduction of 830 kg/day, a construction cost of $30,529.68, and no operation and maintenance costs, the first year cost would be about $101/tonne of acid abated. If the construction cost was spread over 25 years, the average acid load reduction remained the same, and no operation and maintenance costs were incurred, the average cost would be about $4.03/tonne of acid abated. Similarly, if these same conditions held over 50 years, the average cost would be about $2.02/tonne of acid abated.

TABLE 6. ACID AND IRON LOAD REDUCTIONS AT TUNNEL NO. 3 AFTER CATAWISSA CREEK STREAMBED RECONSTRUCTION

	Avg. Flow	Avg. Acid		Avg. Iron	
	(m^3/s)	mg/l	kg/day	mg/l	kg/day
Before Streambed Reconstruction (March 1969 - February 1970)	0.348	49	1,474	0.8	24
After Streambed Reconstruction (March 1970 - January 1971)	0.095*	79	644	4.1	34
Load Reduction			830		-10**

* Based upon average flow reduction of 0.253 m^3/day.
** Load increase, rather than reduction.

VI

CONSTRUCT WATERTIGHT SEALS

GEOLOGIC MAPPING

Feasibility of sealing the water-level tunnels and inundating the mine workings is dependent upon the physical properties and the structural competence of rock formations involving the basin. Consequently, a detailed knowledge of basin geology was considered a prerequisite to making an engineering decision to proceed with the project. Initial efforts in developing this geologic information were directed toward locating existing geologic data. The most detailed geologic map of the South Green Mountain Basin area was published in 1889 by the Second Pennsylvania Geological Survey. In light of reconnaissance field observations, review of mine maps, and assessment of more recent information gathered by geologists working in the Region, it became apparent that a geologic mapping program would need to be undertaken for a complete presentation of the geology for the project area.

Although this report is basically concerned with the South Green Mountain Basin, the complex geology of the area requires that the entire Green Mountain Region be considered as a single geologic entity. Therefore, the geologic investigations covered the entire Green Mountain Region. The geologic investigations included (1) aerial mapping, (2) geologic mapping of the tunnels, (3) core borings over tunnels, and (4) core borings at future overflow points.

All surface features noted in stereoscopic photo interpretation, including outcrops, faults, and fracture traces, were plotted on the aerial photographs. Field examinations of these and other features were conducted. A detailed structure contour map of the South Green Mountain Basin was prepared from mine maps, aerial photographs, and available geologic information. Rock types and geologic structure were logged and used in constructing geologic maps of each tunnel. Diamond drill borings from subsurface investigations at the tunnels and anticipated overflow points provided rock cores of the geologic section and data concerning general rock condition and permeability. Data compiled from all these sources were used in constructing Figures 2 through 6 and serve as the basis of discussion in the following.

AREAL GEOLOGY

Green Mountain is a western extremity finger of the Eastern Middle Anthracite Field synclinorium. The summit area of the mountain is a broad, rolling plateau with an average 61 meters of relief, ranging from approximately 488 to 549 meters above sea level. This plateau rises about 214 meters above the surrounding valleys, where elevations range from approximately 274 to 334 meters above sea level.

Five mappable rock units of Upper Mississippian age through Upper Pennsylvanian age are exposed within the project area. The oldest rocks, exposed in the low-lying anticlinal valleys, are the red beds of the Middle Member of the Mauch Chuck formation. Lying on the lower slopes of Green Mountain is the Upper Member of the Mauch Chunk formation, an intertonguing sequence of red beds (characteristic of underlying rocks), as well as gray sandstones and conglomerates (characteristic of the overlying Pottsville formation).

The Pottsville formation has been subdivided into two easily identifiable members. The Lower Member regionally is an interbedded sequence of gray shales, siltstones, sandstones, and conglomerates. Pebbles in the conglomerates consist of a wide variety of rock types, including white vein quartz, white to dark gray quartzite, many colors of chert, sandstone, shale, gneiss, phyllite, and others. In contrast, the Upper Member consists almost entirely of sandstone and conglomerate beds. Pebbles consist predominantly of white vein quartz and white to gray quartzite, making the distinction between the two members quite obvious.

Both members of the Pottsville formation crop out on the upper slopes and plateau area of Green Mountain. Most natural outcrops in the area consist of one or the other of these hard conglomeritic strata. The youngest Pennsylvanian rocks exposed within the area are those of the Llewellyn formation. These rocks, exposed on the plateau area of Green Mountain, are a sequence of interbedded coals, clays, shales, and sandstones. All mineable coal in the project area, except the Little Buck Mountain seam, lies within the Llewellyn formation.

The structural grain within the study area is approximately N 77° E from a series of synclines and anticlines forming an irregular en echelon pattern. Two major synclines, the North Green Mountain syncline and the South Green Mountain syncline, traverse the area. The two principal Green Mountain coal basins lie within these two synclines. The South Green Mountain Basin stretches 11.3 kilometers across the southern portion of the Green Mountain plateau. Two other major coal basins extend westward into the study area from the vicinity of Hazleton. Three minor synclines preserve coal in small basins lying wholly within the Study Area. The geologic map on Figures 2a, 2b, and 2c shows the extent and locations of these synclines and coal basins.

Faults noted during the geologic investigations include bedding-plane slippage, low-angle thrust, and low-angle reverse faults. Although bedding-plane slippage faults have been positively identified only in a few strip pits and in the drainage tunnels, they are believed to be significant contributing features to the Regional deformation. Major displacements from low-angle thrusts have been identified at four points on the north limbs of coal basins: one on the North Green Mountain Basin, two on the South Green Mountain Basin, and one on a small unnamed basin west of Sheppton. Other thrusts are suspected but have not been verified on the north limbs of several additional synclines. A single low-angle reverse fault exists on the south limb of the South Green Mountain Basin in the vicinity of Tunnel No. 3. Figures 2a, 2b, 2c, 3a, and 3b show the nature of thrust and reverse faulting in the area.

Drainage for the plateau area of Green Mountain outside the major coal basins is provided by Tomhicken, Little Tomhicken, Catawissa, and Stony Creeks, as well as Little Crooked Run. The headwaters of Tomhicken, Little Tomhicken, and Stony Creeks, and Little Crooked Run are within the area. Catawissa Creek flows into the area from the east, crosses its southeastern corner, and continues westward to the creek's confluence with the North Branch of the Susquehanna River. Before mining disrupted the natural drainage patterns within the South Green Mountain Basin, Little Tomhicken Creek drained its western portion and Catawissa Creek drained its eastern portion. Extensive surface mining, interconnected with the past underground mining, has intercepted virtually all surface drainage tributary to the basin, causing this drainage to flow into and through the deep mine workings to eventually discharge via the 3 water-level tunnels.

Lineations formed by straight segments of stream valleys, and other topographic features, were mapped on aerial photographs as possible fracture traces for the purpose of locating potential leakage points from the flooded basin. Only two of these traces approach the basin close enough to be interpreted as potential leakage points. These points are located in the two gaps where overflows from the mine water pool are expected to form. These gaps are located at the points where Little Tomhicken Creek and an unnamed tributary of Catawissa Creek formerly carried surface drainage from the South Green Mountain Basin.

ENGINEERING GEOLOGIC INVESTIGATIONS

Certain engineering geologic aspects of the area were investigated to determine the locations of seal sites within the tunnels, as well as the locations and elevations of the anticipated overflows. Several approaches were taken to acquire the necessary engineering information (including internal tunnel investigations, core borings above potential seal sites, and core borings at anticipated future overflow points) to present the following findings and conclusions:

Tunnel Seal Sites

Internal Investigations Of Drainage Tunnels

Internal investigations were conducted within each of the three drainage tunnels for the purposes of delineating geologic formations and structure, physical condition of the rock, and potential for water leakage.

Dimensions of the tunnels range from approximately 1.5 meters high by 3 meters wide to 3.7 meters high by 4.6 meters wide, and average 2.1 meters high by 3.7 meters wide. Smaller dimensions exist where the tunnels penetrate hard rock, with larger dimensions in softer rock. The larger tunnel dimensions in softer rock commonly are the result of roof falls that have occurred since construction.

The danger of further roof falls, and mine water pools behind these falls, made clearing and shoring work in certain areas of the tunnels necessary before internal investigations could begin.

Geologic data gathered for each tunnel during the initial phase of the internal investigations were plotted on cross sections with a ground surface profile taken from USGS topographic maps. The cross sections were extended along the center line of each tunnel to show the coal veins present in the basin and their relationship to the tunnels.

These cross sections were used to interpret the geologic structure and stratigraphy along the line of each tunnel and to select the best potential seal sites for further intensive study: three sites in Tunnel No. 1 (Sites 1-A, 1-B, and 1-C); two sites in Tunnel No. 2 (Sites 2-A and 2-B); and one site in Tunnel No. 3 (Site 3-A). Figures 4, 5, and 6 show the locations of all potential seal sites explored.

Tunnel No. 1 is 2,139 meters long and was driven through several broad to moderately tight synclines and anticlines cutting through (1) red shales, siltstones, and sandstones; (2) gray shales, siltstones, and sandstones; and (3) conglomerates, black shales, and coals. Red shales, hard gray sandstones, and conglomerates predominate. Red shale is the rock most prone to roof falls. The clearing work done within this tunnel was largely concerned with removing falls of red shale to release impounded pools of water. Other work involved the placing of timbers to support occasional loose roof rock. The potential seal sites studied were located in the hard gray sandstone and conglomerate.

Only two significant open fractures were geologically mapped: one at 79 meters from the portal and the other near the mine workings at 2,112 meters inside the portal. The one nearest the portal will not compromise seal effectiveness. The other, an obvious fault, is oriented in such a way that seepage along it presents no apparent leakage problem. The fault does not crop out on the surface below 457 meters elevation, the anticipated pool level (Figure 4).

Tunnel No. 2 is 1,253 meters long and was driven through moderately to tightly folded synclines and anticlines. Rock types similar to those of Tunnel No. 1 were encountered. Rock falls have occurred in the red shale, although not to an extent requiring clearing and shoring. Two significant open fractures were encountered: at 345 meters and 1,207 meters from the portal. Two potential seal sites were located in the hard gray sandstone and conglomerate, not adjacent to or affected by the open fractures (Figure 5).

Tunnel No. 3 is 256 meters long, but 155 meters are within the mine workings and not of concern in seal site selection. The 101 meters outside the workings penetrate hard gray sandstone and conglomerate. Clearing and shoring were limited to the mouth of the tunnel, where many loose boulders had collapsed over the original opening. A significant open fracture encountered 61 meters from the portal cuts across the tunnel at an angle that indicates it may intercept the mine workings and, thereby, provide a possible leakage route (Figure 6).

Core Borings Over Tunnels

Investigations of the rock in the vicinity of potential seal sites required the drilling of boreholes, starting from points on the ground surface directly overlying the sites and continuing down to or below the invert elevations of the tunnels. One borehole was drilled over each potential seal site.

Data in this study of the recovered core rock included rock type, core recovery rates, length of core pieces, dip of bedding surfaces and fractures, hardness, jointing, faulting, degree of weathering, and staining of fractures.

Pressure testing the boreholes provided additional information on the extent and permeability of fracturing in the rock. In general, each 15 meter section of hole was isolated with expanding rubber packers and tested by pumping water under pressure into that section. Tests were also conducted where core recovery, fracturing, loss of recirculating drill water, or other features indicated the possibility of permeable zones. Pressures used in testing were determined by the general formula of 0.07 kilogram per square centimeter (kg/cm^2) increment for each 0.3 meters of depth to the packer, to a maximum of 21.1 kg/cm^2. Pressures thus applied represent conditions about one and one-half times as severe as the flooded mine workings will present.

Every effort was made to maintain the return flow of drill water to the surface. Its loss during drilling operations indicates the interception of pervious rock. Every fracture or zone that caused the loss of return drill water was pressure tested. After testing, these openings were grouted as necessary to regain the flow of drill water to the surface.

Drillers' logs, grouting data and pressure testing information for the tunnel drilling program are presented in Appendix B, pages 83 through 125.

Tunnel No. 1--Borehole 1-A was drilled to a depth of 175 meters, beginning at a ground elevation of 477 meters, and penetrated alternating beds of red and gray shale, and gray sandstone with very minor amounts of conglomerate. It passed near the tunnel at elevation 332 meters where it penetrated 8.2 meters of very hard, fine-grained gray sandstone, dipping at approximately 50 degrees. The borehole passed through a number of fractures and faults, many of which are slickensided and partially or completely sealed with quartz fillings.

Pressures maintained during the five tests above elevation 398 meters ranged from 4.2 to 18.3 kg/cm^2, with water loss ranging from 0.00 to 1.6 l/s. Most of this loss probably resulted from fractures opened by weathering of the rock. During pressure testing of each 15-meter interval below elevation 398 meters, a pressure of 21.1 kg/cm^2 was maintained for five minutes, with a cumulative water loss rate of 1.4 l/s for the entire depth of the hole below elevation 398 meters. This rate, a summation of seven tests ranging between 0.00 to 0.8 l/s, is indicative of highly impermeable rock. Another indication of this impermeability is that no grouting was required below elevation 414 meters to maintain the return flow of drill water to the surface.

Borehole 1-B, beginning at a ground elevation of 502 meters, penetrated alternating beds of gray shale, sandstone, and conglomerate in the upper 76 meters, while red and gray shale, and gray sandstone with some conglomerate composed the remainder to a maximum penetration of 198 meters. The borehole passed the vicinity of the tunnel at elevation 335 meters, penetrating 10 meters of very hard gray conglomeritic sandstone. This borehole passed through solid rock within 0.3 meter of the west wall of the tunnel. Subsequent internal investigations revealed that a large block of rock had broken away from the tunnel wall, exposing the borehole. This must have occurred after completion of the pressure testing, because a pressure of 21.1 kg/cm^2 was maintained for five minutes in that section of the borehole.

Five pressure tests were performed above elevation 429 meters at pressures ranging from 4.1 to 17.6 kg/cm^2, for a cumulative water loss rate of 7.5 l/s, thus indicating the presence of some weathered rock open joints. Although water losses did occur above elevation 427 meters, those below elevation 484 meters were slight, and only minor grouting was necessary to regain full return flow at the surface. Eight pressure tests conducted below elevation 427 meters were accomplished at 21.1 kg/cm^2, held for five minutes, with a cumulative water loss rate of 1.3 l/s, ranging from 0.00 to 0.8 l/s for the different intervals. This suggested that highly impermeable rock occurs below elevation 427 meters.

Borehole 1-C, beginning at a ground elevation of 487 meters, penetrated alternating conglomerate and thin sandstone beds with some carbonaceous partings to an elevation of 410 meters. Below elevation 410 meters there occurred interbedded gray shale, sandstone, and conglomerate. At elevation 338 meters, the borehole passed through the tunnel roof, following penetrations of 13 meters of hard gray fine-grained sandstone; 20 meters of very hard gray conglomeritic sandstone; and 1.2 meters of hard gray shale, all dipping at about 60 degrees. Because this borehole penetrated the tunnel, it was preserved and capped for future use as an observation well.

Six tests were conducted above elevation 405 meters at pressures ranging from 3.3 to 19.0 kg/cm^2, with a cumulative water loss rate of 3.3 l/s, an indication of some weathered rock open joints. Even above elevation 405 meters, the rock was sound enough that no grouting was necessary below 451 meters to maintain the return flow of drill water to the surface. The four pressure tests conducted below elevation 405 meters were accomplished at 21.1 kg/cm^2, held for five minutes, with a cumulative water loss rate of 0.03 l/s ranging from 0.00 to 0.02 l/s for the different intervals. Highly impermeable rock was bored below elevation 405 meters.

Tunnel No. 2--Borehole 2-A, beginning at a ground elevation of 545 meters, penetrated alternating beds of gray conglomerate and sandstone to a depth of 73 meters, below which occurred red shale, gray shale, sandstone, and comglomerate. The borehole penetrated the tunnel roof at elevation 363 meters after cutting 11 meters of very hard gray conglomeritic sandstone and conglomerate. This hole was also prepared and capped for future use as an observation well.

Five pressure tests were conducted above elevation 463 meters at pressures ranging from 5.1 to 18.8 kg/cm^2, with a cumulative water loss rate of 0.004 l/s, indicating highly impermeable rock throughout the upper part of the borehole. Rock in this borehole required no grouting to maintain the return flow of drill water to the surface. Each of the six pressure tests conducted below elevation 463 meters was performed at 21.1 kg/cm^2, held for five minutes, with no loss of water, suggesting the rock is extremely impermeable.

Borehole 2-B, beginning at a ground elevation of 537 meters, penetrated conglomerate to a depth of 34 meters, below which occurred alternating beds of red shale, gray shale, sandstone, and conglomerate to a total depth of 212 meters. The tunnel was passed at elevation 362 meters, above which the hole penetrated 30 meters of conglomerate dipping at 50 degrees.

In nine pressure tests conducted between elevation 537 meters and elevation 392 meters, pressures maintained ranged from 0 to 19.0 kg/cm^2, with a cumulative water loss rate of 8.4 l/s, ranging from 0.00 to 3.2 l/s for the different intervals tested. During drilling of this hole, difficulty was experienced in maintaining return drill water. Copious amounts of Portland Cement, sawdust, and a sealing compound were used to try to seal fractures to elevation 394 meters. A large open fracture, pressure tested at 0 kg/cm^2 and 3.2 l/s water consumption, was encountered at elevation 394 meters. This fracture could not be sealed. Drilling was continued below elevation 394 meters without return drill water, and the pressure testing interval was reduced from 15 meters to 6.1 meters. These high rates of water loss resulted from open fractures caused by intense folding of the rock drilled.

Partial loss of drill water within 61 to 91 meters of the surface was undoubtedly due to open joints in weathered rock. Each of the pressure tests conducted below elevation 392 meters was accomplished at 21.1 kg/cm^2, held for five minutes, with a cumulative water loss rate of 2.4 l/s, ranging from 0.00 to 1.2 l/s for the different intervals tested. The 2.4 l/s water loss occurred between elevation 392 to 368 meters, entirely above the level of the tunnel. The hole from elevation 368 to 325 meters showed no water loss.

Tunnel No. 3--Borehole 3-A, beginning at a ground elevation of 483 meters, penetrated conglomerate to its final depth of 55 meters or elevation 428 meters. A major fracture zone, thought to be a continuation of the one observed during the internal investigation of Tunnel No. 3, was encountered at elevations 428 to 450 meters. Borehole pressure testing was conducted with the packer being set at four different elevations within the borehole, namely 449, 456, 467, and 476 meters, and tests were made from each of these points to elevation 445 meters.

The three pressure tests of the hole below the elevation of 469 meters were performed at 9.8 kg/cm^2, with a cumulative water loss rate of 0.6 l/s. Water loss in that section of the hole containing the fracture zone was 0.4 l/s at 9.8 kg/cm^2. Hydraulic pressure tests and examination of the fractured zone in the tunnel and the borehole indicated no major threat of leakage from the impoundment. However, observation of this area during filling is suggested. Drilling from elevation 445 meters to the bottom of the hole showed the rock to be extremely hard and tight.

Future Overflow Points

Inundation of the mine workings will begin after construction of the proposed water seals. The pool levels within the inundated mine workings will be determined by leakage from the basin. If significant water loss occurs through fractures and faults at low elevations, the pool level will be low. If leakage is insignificant at low elevations, as expected, the level will ultimately reach the elevation of two topographic gaps on the rim of the basin. It is anticipated that overflow will form in these two gaps: one 4.5 kilometers east of Sheppton; and the other 1.1 kilometers west of Oneida, where the originally draining streams cut through the sides of Green Mountain (Figures 2b and 2c). The permeability of strata at these gaps is expected to finally determine the levels maintained. Tight conditions would cause the overflow levels to be at or near the lowest surface elevations in the gaps. Open, pervious, and weathered rock conditions at depths below the surface would probably cause leakage and a lowering of the pool levels. Drilling to investigate the subsurface conditions in these two areas was conducted.

Boreholes were oriented at 40 degrees below the horizontal, to provide the greatest amount of subsurface information immediately beneath the gaps by intercepting steeply dipping fractures. Drilling procedures were similar to those previously described.

Gap East of Sheppton

A test hole, located at a surface elevation of 459 meters, was drilled on the angle at 40 degrees from the horizontal to a total length of 30 meters, giving a vertical component of penetration of approximately 20 meters to elevation 439 meters. The borehole penetrated hard gray sandstone, conglomeritic sandstone, and conglomerate. Pressure tests were conducted by seating the packer at elevations of 443, 448, and 454 meters. Test results showed water loss rates of 0.04 l/s at 7.0 kg/cm^2, 2.01 l/s at 7.0 kg/cm^2, and 0.9 l/s at 4.2 kg/cm^2, respectively. The upper 9 meters of the borehole were cased through overburden and were not pressure tested.

Gap West of Oneida

A test hole at this site was located at a surface elevation of 463 meters. It was drilled at 40 degrees from the horizontal to a total length of 29 meters, penetrating to an approximate elevation of 444 meters. The borehole penetrated hard gray sandstone, conglomeritic sandstone, and conglomerate. Pressure tests were conducted by seating the packer at elevations 452 and 457 meters. Test results showed water loss rates of 0.08 l/s at 7.0 kg/cm^2, and 0.9 l/s at 5.6 kg/cm^2, respectively. The upper 7.6 meters of the borehole were cased through overburden and were not pressure tested.

Physical and Chemical Rock Tests

In addition to the previously described field investigations, physical and chemical tests were performed on rock samples obtained at several potential seal sites. These tests were conducted to determine the soundness of the rock and its resistance to solution by mine drainage. Conglomerate and siltstone samples for physical tests were collected by hand within Tunnel No. 3 and selected from the cores recovered from Boreholes 1-C and 2-A. Conglomerate and shale samples for chemical tests were collected by hand within Tunnel Nos. 2 and 3 and selected from the cores recovered from Borehole 2-A. The work performed, as well as the findings and conclusions drawn from it, is presented in the following:

The laboratory report summarizing the physical tests is set forth in Appendix C. The results of chemical testing of the rocks are presented in Appendix D.

Physical Tests

Specific Gravity--Specific gravities of these rock samples were obtained to determine the weight of overburden above the tunnels. This information was then used to determine the resistance of rock and the proposed concrete plugs to shear failure and the resistance of the proposed concrete plugs to deformation.

Specific gravities of oven-dried specimens ranged from an average of 2.65 for two conglomerate samples to 2.72 for a single siltstone sample. Such values are typical for these sedimentary rock types. Differences in specific gravity between oven-dried and water-saturated specimens were negligible, indicating that the rocks have only small volumes of pore space in which water may be absorbed. This substantiates the results of pressure tests on drill holes and indicates low porosity and permeability.

Compressive Strength--Triaxial tests were performed to evaluate the compressive strength of rock samples at confining pressures ranging from 1.4 to 28 kg/cm^2. Maximum compressive strengths ranged from 291 to 425 kg/cm^2 in siltstone samples and from 1,490 to 1,750 kg/cm^2 in conglomerate samples. Maximum compressive strengths of conglomerate specimens are significantly greater than for siltstone specimens, and are also higher than values expected for most common seal materials. However, these high strengths are desirable because rocks are nonhomogeneous and, consequently, do not have definite consistent physical properties.

Shear Strength--Two types of direct shear tests were performed: rock-on-rock; and rock-on-concrete. The results of the former indicate the resistance of rock to shear failure at the seal sites, and results of the latter indicate the resistance of rock-concrete bonds to shear failure. Two rock-on-rock shear tests performed on siltstone, with axial loadings of 18 and 21 kg/cm^2 applied on the ends of the specimens, gave peak shear stress of 125 and 109 kg/cm^2, respectively. Only siltstone samples were tested because, as the triaxial tests show, conglomerate specimens are far stronger.

Results of the rock-on-concrete shear tests showed that, with axial loadings ranging from 5.6 to 73 kg/cm^2, peak shear stresses along conglomerate-concrete bond surfaces ranged from 30 to 96 kg/cm^2. During preparation of the test specimens, the concrete was cast against the smooth, sawed ends of the rock samples. Because concrete would be poured against rough rock surfaces in the actual seal construction, rock-concrete bond strengths would be somewhat greater than indicated by the testing.

Deformation Moduli--Deformation moduli were determined for purposes of calculating possible changes in tunnel diameter and deformation (compression) of the rock surrounding the seals under anticipated hydrostatic heads. The moduli were computed from the results of unconfined compressive strength tests. Maximum compressive strengths of two conglomerate specimens were found to be 1,170 and 1,790 kg/cm^2.

Unconfined compressive strength tests were also performed on concrete specimens that were cast from the same mix as was used in the rock-on-concrete direct shear tests. Maximum compressive strengths of these concrete specimens ranged from 136 to 237 kg/cm^2. The compressive strengths of the conglomerate rock greatly exceed those for the concrete specimens.

Chemical Tests

In an attempt to determine the long-term effect of mine drainage contact with tunnel rock, chemical tests were conducted under conditions much more severe than actually expected. Conglomerate and shale samples used in the chemical tests were approximately 1.3 cm diameter fragments, each weighing about 100 grams. The fragments provided a large surface area to total weight ratio. Test solutions consisted of mine drainage (pH 3.5), 1 percent sulfuric acid (pH - 1.2), and 5 percent sulfuric acid (pH - 0.5) maintained at temperatures significantly warmer than would be encountered in the tunnels. Tests lasted approximately four and one-half months.

Three portions of the rock types selected (one shale and two conglomerates) were each immersed in the different test solutions. At certain intervals, aliquots of the test solutions and the rock samples were subjected to chemical and physical tests in an attempt to determine the solubility of the rock samples. The chemical solutions were analyzed for pH and aluminum concentration, and the rock samples were dried and weighed.

Test results showed that the samples underwent weight losses in all three solutions. As expected, weight losses experienced in the 5 percent sulfuric acid solution were significantly greater than those experienced in the 1 percent sulfuric acid and mine drainage samples. The weight losses for the samples in all three solutions showed a general decreasing trend with time.

The weight losses examined were not considered significant. At the end of the test period, the total weight loss in mine drainage solutions noted for the conglomerate samples varied from 0.00037 to 0.00088 grams per day, and was 0.00029 grams per day for the shale sample.

The use of rock chips from the tunnel contributed to the severity of the chemical tests as conducted, and acted to magnify the solution losses. The use of chips in the tests provided a large surface area containing a relative abundance of soluble mineral constituents. In addition to the predominant silica minerals in the conglomerate and clay minerals in the shale, tunnel rock contains small quantities of carbonaceous fragments, mica, feldspar, and ferro-magnesium minerals, which are significantly more soluble. These minerals are not a continuously connected labyrinth of particles. Rather, they are isolated individual grains or groups of grains. Their removal by solution would not, therefore, create permeable paths through the rock or otherwise endanger its integrity. Weight losses in the tests represent largely the effects of solution on these constituents. When exposed soluble particles are once removed, the rate of reaction would be expected to decrease markedly, and the rock to be relatively unaffected through time.

Observation at the watercourses within the tunnels substantiates this explanation. No significant erosion by mine drainage flows through the tunnels can be seen. Rock surfaces are considered to be essentially as they were following completion of tunnel construction in about 1900. Seventy years' exposure to mine drainage has produced no observable solution effects.

Summary

The South Green Mountain Anthracite Basin is totally contained within a tightly folded, "canoe-shaped" syncline. Resistant, competent sandstone and conglomerate beds of the Pottsville formation, which underlie the coal measures, form the enclosing basin and the elevated plateau-like mountain. The softer underlying shales of the Mauch Chunk formation crop out in the adjacent valleys and have been worn down by erosion to lower elevations.

Deep and surface mining operations are confined to the coal measures and are entirely contained by the folded competent members. The containing formations were breached in three places with small diameter tunnels driven through the perimeter as drains to dispose of the water accumulating in the workings. The tunnels, traversing from the lowest fold of the coal measures to the adjacent valleys, effectively disposed of the mine water. Before tunnel construction, pumping of water from the mines was required. This leads to the conclusion that effective sealing of the three tunnels would impound water in the abandoned mines. The elevation to which this pool would rise without leakage, however, is not part of this conclusion.

The purpose of the geologic study, therefore, was to determine:

1. the geologic feasibility of constructing effective seals in the tunnels; and

2. the level to which impounded water would rise, without major loss by leakage.

Normally, a tightly folded syncline is accompanied by fractures and faults. If this fracturing and faulting were extensive, and if the resultant cracks were open, considerable water leakage would result. It would be difficult to construct effective seals, and the resultant leakage from the pool would not permit effective impoundment of water. Considerable effort was expended to define and describe fractures and faults in the basin. The permeability of the enclosing rock units, because of the rock composition, faults and fractures, is of prime concern to the total project.

Overthrusts and bedding-plane slips are not readily observed on the surface or in the mine map data. Data obtained from drilling and mapping of the tunnels show the presence of low-angle thrusts and slips. These faults and slips, being nearly parallel to the bedding, do not traverse the rock from the workings outward and do not provide leakage paths. Surface mapping and structure contouring of coal measures from mine map data did not reveal faults presenting serious leakage potential.

The quality and condition of rock observed in the tunnels and in drill cores are remarkably good. Little fracturing can be observed, and staining or other evidence of water travel is minimal. The areas near the portals of tunnels and disturbed areas adjacent to the mine workings do exhibit some of these features. They are minor in extent, however, and are virtually confined to these zones. Middle portions of the tunnels consist of thoroughly sound, competent, and relatively unbroken and unweathered rock.

Pressure tests of the drill holes indicate very tight, impervious conditions in most places. A zone of somewhat open fractures near the surface was observed, but it does not materially affect the tightness of the basin. Zones of open rock observed in cored rock are believed to represent low-angle faults. These faults probably have a low potential for water leakage from the South Green Mountain Structural Basin.

Although possible minor leakage is anticipated, no paths of serious water loss were noted from interpretation of data collected during this study. With the placement of competent watertight seals in the three drainage tunnels, it is believed that the basin is sufficiently watertight to contain a 5.3 billion liter pool in the underground mine voids to an elevation of 458 meters. Anticipated hydrostatic heads against the seals under these conditions are 122, 100, and 27 meters for Tunnel Nos. 1, 2, and 3, respectively.

Potential seal locations were selected to satisfy several criteria. Seals must be in locations relative to fault zones that minimize the possibility of leakage. Rock at the seal sites must be able to support the substantial loads imposed by the hydrostatic head. The rock unit in which the seal is placed must be impervious and unfractured, and it must be unaffected by prolonged exposure to acid water.

The portions of all tunnels near the portals, where considerable fracturing was observed, were not considered for seal locations. Rock tests and physical characteristics of the available rock types show an obvious preference for the conglomeritic sandstone over the shale-siltstone units. Laboratory compressive strengths obtained for siltstone samples collected from the project area indicate that the rock qualities are approximately the same as concrete, whereas conglomeritic samples have strengths several times that of concrete. Rock, a nonhomogeneous material, should attain sufficient strengths to develop adequate factors of safety. The conglomeritic sandstone is, therefore, recommended. Pressure tests show it to be sufficiently impervious, and solubility tests indicate that it is sufficiently resistant. Seal sites were selected in the conglomerate for these reasons. Although some sites near the portal may have been adequate, the best possible locations were selected to satisfy the above criteria. It is felt that the long-term requirements of the project and the possibility of unknown developments warrant this selection.

Neither the exact volume nor specific sites of possible water leakage can be determined without actual inundation of the mine workings. However, the geologic investigation performed in this study is believed to be sufficiently detailed to indicate that no major leakage is expected. However, water loss is anticipated at several locations. As the pool levels increase, hydraulic head increases, and water begins to flow into the more fractured and weathered rock near the surface areas, the probability of water loss will increase.

The ultimate water levels also cannot be predicted. However, the available information suggests that, because leakage at low elevations will be low, levels will be near the elevations of the two gaps. Leakage may be expected in the vicinity of the gaps as water levels approach an elevation of 442 meters. Springs may develop, and some initial erosion of overburden may occur. Water levels are expected to rise above elevation 442 meters and should approach elevation 457 meters, but the exact level and seasonal fluctuation are indeterminate.

The physical condition of the barrier pillar is unknown since the pillar is inaccessible for observations and recorded information related to the more recent mining operations is of poor quality. There is a definite possibility that the pillar has been breached at some elevation. If so, water levels on either side of the pillar should be the same at the point of the breach. Below that point, however, two water levels may be independent of each other and each will rise in proportion to the inflow of water to that portion of the workings.

It is concluded that:

1. It is geologically feasible to construct effective seals in each of the three tunnels, and to contain the anticipated impoundment within the structural basin.

2. Although minor leakage is probable, the basin will contain water at a sufficient elevation to serve the intended project purpose of controlling the formation of acid mine drainage.

DESIGN PHASE

The preparation of construction plans and technical specifications was started once it was established from the geologic investigations that there was a high degree of certainty that the water-level tunnels could be successfully sealed. Preliminary bid documents were delivered on September 9, 1971 to the Department of Environmental Resources for review, followed by seal design computations on September 21, 1971. The theory of seal design was discussed with Department personnel in considerable detail on October 1, 1971. Subsequently, questions and comments by the Department and EPA were given to the consultant. A response to these was made on January 3, 1972.

After accommodating the requested revisions, a further submission comprising the Official Notice, Invitation for Bids, Bid Forms, and Special Requirements along with construction plans and specifications was made on February 21, 1972. The Construction Cost Estimate for sealing the three tunnels was presented on March 3, 1972.

These documents were then evaluated, and another meeting was held on April 21, 1972 to discuss them. At this meeting, alternative means of placing the seals (remote placement of concrete) and controlling or drawing down the mine water pool (deep well pumps) were broached by the Department. Although these alternatives had already been considered before final design was established, it was agreed they would be reevaluated and cost comparisons made. Consequently, a detailed analysis of these items, supporting the original design, was presented in a May 24, 1972 letter.

The Department during September 1972 requested additional information concerning the cost of installing remote control devices to operate the valves in Tunnel Nos. 1 and 2, the length of time to drain the pool (assuming all three tunnels were sealed and pool water level approximated 457 meters), the percentage of mine drainage pollution in Catawissa Creek originating from the basin, and the need to place valves in all three tunnels. These questions were answered on September 28, 1972.

The Department then requested information concerning water wells in the basin. The logs for 8 wells and their locations were provided on October 2, 1972. The Department was advised on October 5, 1972 that sealing all three tunnels would have no adverse effect on these wells.

The Department then directed on December 13, 1972 that: only Tunnel Nos. 2 and 3 should be sealed; the seal in Tunnel No. 3 should be a standard seal with no valve; and revised plans, specifications, and construction cost estimates should be submitted no later than February 1, 1973. Then on January 4, 1973, the Department requested that further work on the revisions be delayed until the Department received approval from EPA concerning the change in scope of the project.

Answers to a series of questions concerning the effectiveness of sealing only Tunnel Nos. 2 and 3 were then provided on February 5, 1973 to EPA. In addition, a longitudinal section through the basin was given to the Department at its request on February 13, 1973. Then in response to the Department's March 9, 1973 authorization, revised construction plans, contract documents, and construction cost estimates for sealing Tunnel Nos. 2 and 3 were submitted on April 30, 1973.

Subsequently, on October 22, 1973, the Department requested that all work on the project cease until the Department acquired all necessary property easements. Then on January 17, 1974, the Department advised that Tunnel Nos. 2 and 3 would be sealed.

Representatives of the Department, EPA, and the consultant met for a joint plan review meeting on February 5, 1974. Subsequently, on March 1, 1974, the Department requested that instrumentation comprising pore water pressure cells, deflectometers, and extensometers be added to the seal in Tunnel No. 2.

After considerable delay in obtaining needed specific information concerning the proposed instrumentation from the supplier suggested by the Department, final contract documents and construction cost estimates were submitted to the Department on August 6, 1974. These documents were reviewed on August 21, 1974 when it was agreed that, because of a significant increase in the cost of the multiconductor cable for the instrumentation, an alternate method of placing this cable from the seal in Tunnel No. 2 to the ground surface would be explored.

On October 24, 1974, the Department decided to separate Tunnel No. 3 from the contract documents and indicated that work on Tunnel No. 3 would be bid separately if the Department decided to seal this tunnel. However, it was requested that the drawings be maintained in a way that would enable their use if a later decision was made to seal Tunnel No. 3. Final contract documents for sealing Tunnel No. 2 were delivered to the Department on December 6, 1974.

Design Considerations

The physical conditions in the South Green Mountain Basin required that special criteria be used to design the drainage tunnel plugs. The anticipated hydrostatic heads on the plugs and surrounding rock were large. It was anticipated that maximum head would approach 122 meters. Accordingly, the following special factors were considered in the design of the proposed plugs.

Type of Construction

Methods of mine seal construction have been and are being investigated under research and development programs. Known programs have been conducted in mines subject to relatively low heads (2 to 9 meters) when flooded. Problems with leakage through and around seals constructed by the various methods of intruding grout into aggregate-filled sections of mine tunnels and into inflatable bags have been common. To our knowledge such methods have not been used for heads approaching those that would prevail in the plugged drainage tunnels of this project. Forces on the plugs would require dependable structural competence. Grout plugs, intruded masses, and other similar approaches do not appear to provide such strengths. Consequently, it was decided that conservative decision concepts should be applied, and the seals should be constructed of concrete with the surrounding rock and concrete-rock interface grouted to prevent potential leakage.

Plug Shape

Two basic shapes were considered: a thin arch plug with the concave side facing the mine workings; and a gravity plug with sloping sides. These are shown in Figure 9. From a structural viewpoint, with regard to both concrete and rock mechanics, an elliptical shape is preferred to a rectangular shape because it eliminates stress concentrations at corners. However, the cost of excavation to obtain an elliptical shape in the existing rectangular tunnel would be considerably higher. A tapered gravity plug would distribute the load at the lowest possible unit load to the concrete. Consequently, it was decided to use the tapered gravity plug.

Modes of Failure

Plug design was based on the ability of the plug to resist (a) sliding and (b) excessive deformations of concrete and rock under the application of

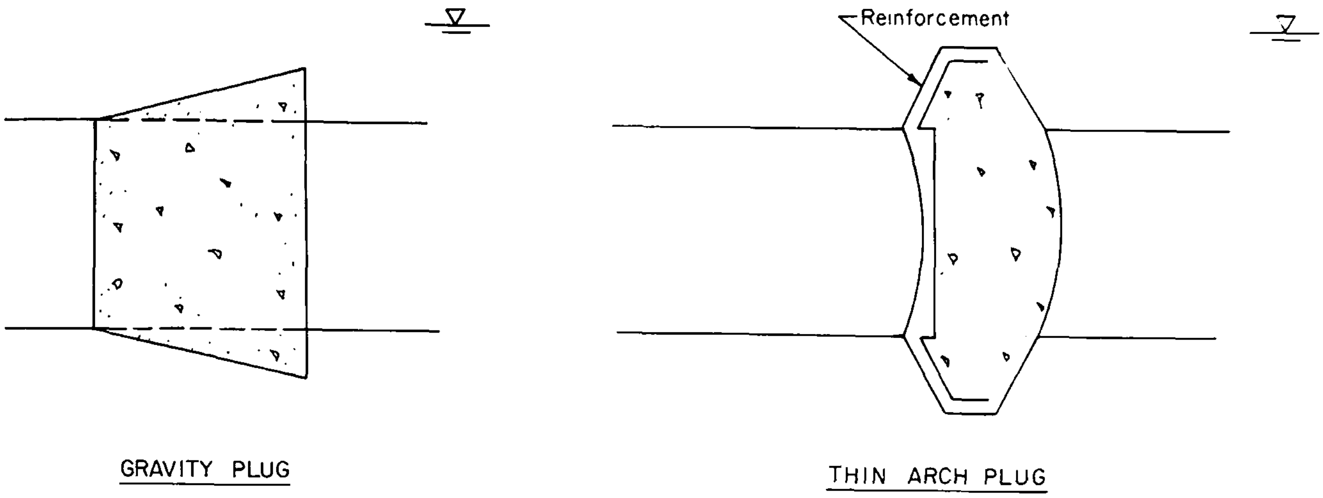


Figure 9. Basic shapes of plugs considered.

the proposed hydrostatic loads and potential earthquake forces. Sliding of the plug could take place by shear failure in the rock surrounding the plug, in the concrete mass forming the plug, or in the contact surface of the rock and the concrete plug. When the plug was found safe against sliding forces, it was then checked for excessive deformation under the proposed maximum loads.

Consideration of Earthquake Forces

Earthquakes of considerable magnitude have occurred along the east coast. Records show values as high as 10 on the Rossi-Forel Scale. Movement resulting from tremors can accelerate the strata in any direction, but, for design purposes, the horizontal and vertical directions are considered to envelop possible reactions to the phenomenon. Accepted structural design criteria used in the design of most civil works facilities in the area of the plug site normally do not include earthquake load criteria. However, it is good practice to apply a horizontal earthquake acceleration of 0.1 times the acceleration of gravity, in an upstream direction, in the design criteria governing dam design because of the safety standards associated with such structures. The noted earthquake design criterion was used in the design of the plug.

Horizontal movement of the strata in a direction other than upstream, or vertical movement in either an up-or-down direction, would not result in plug overstress because of the rock-concrete contact, that is, acceleration of both rock and concrete would be equal and no overstressing, differential load would result.

Anchoring the Plug into Rock with Steel

Methods illustrated in Examples (a) and (b) of Figure 10 require movement of the concrete before stress transfer takes place. Theoretically, this concept implies initial failure of the plug before the load is resisted. The bond between concrete and rock around the plug would be broken, allowing acid

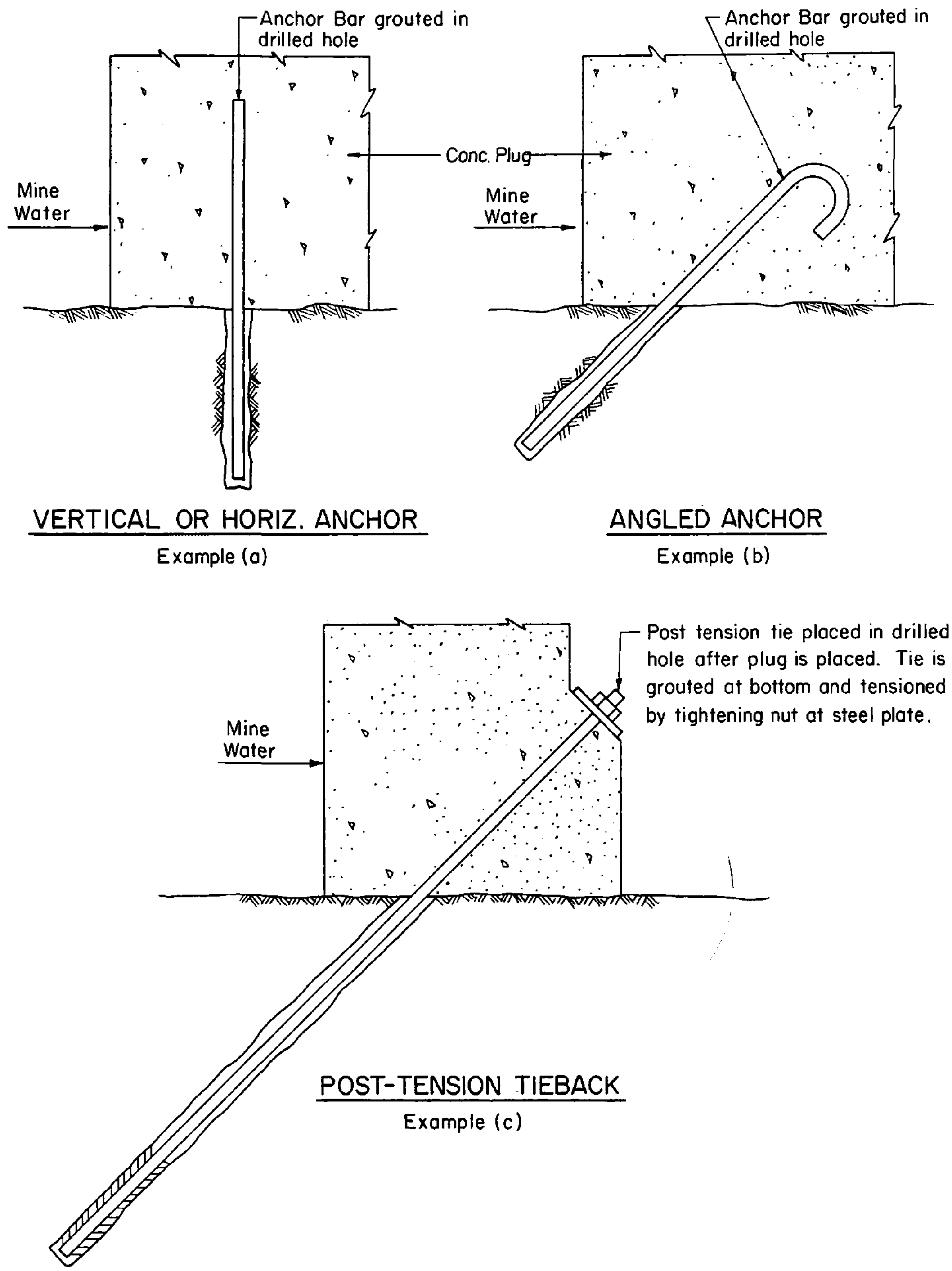


Figure 10. Methods of anchoring concrete plugs.

water to flow between concrete and rock, causing progressive deterioration of the concrete and progressively more leakage around the plug.

If either of these two illustrated methods is used in conjunction with the tapered roof and wall concept, the plug could not move unless some failure occurred in the concrete plug. However, if the plug did move, the load would transfer from the rock to the steel. Depending upon the initial mode of tapered plug failure: (1) failure in shear; or (2) failure of concrete along the tapered surface, as a result of acid deterioration, the plug would act in one of two ways: (a) if the plug failed in shear, the load would be picked up by the steel, which in turn would transfer it to the rock until movement halted because of the restraining capability of the steel; or (b) if the plug failed because of failure of the concrete along the tapered surfaces, the plug would push forward or downstream until the steel came into play to restrain the movement. In either case, the plug could leak and, in either case, initial movement would result in ultimate failure of the plug. The object is to avoid initial movement, and, if movement takes place, to limit the movement to an incremental amount as opposed to a sudden and complete failure. Remedial work would have to be carried out after the initial movement to prevent ultimate failure of the plug.

The plug cannot be keyed into the rock with only steel as the restraining force as illustrated in Examples (a) and (b). The use of steel can only be considered as a secondary restraining system that would become active after partial or total failure of the prime method of restraint.

Post-tension ties illustrated in Example (c) of Figure 10 could hold the concrete plug in place, without movement. Simply, the idea of post-tension tie is that the ties would be stressed, after concrete placement, to the extent that the concrete would be held in place with a force in excess of the design load. The ties would have to fail before the plug could move. If the ties failed, failure would be instantaneous. It is not possible to guarantee that acid water would not reach the ties, even though full precautions were taken to fully grout the system to protect the tieback material. A stainless steel tieback system would be costly, relative to the cost of the tapered plug construction and its use would not eliminate the dependency of plug safety on construction methods and procedures.

The tapered plug concept distributes the load to the concrete at the lowest possible unit load on the concrete. Based upon laboratory tests as shown in Appendix C, the rock is stronger than the concrete. Concrete compressive stresses created in the plug are well within acceptable values, making the capability of the concrete to resist the load transfer in shear the critical consideration. A shear failure could cause a rapid deterioration of the plug to the extent that failure could be considered instantaneous. In considering the shear factor of safety and the desirability of providing a specific contingency against total and sudden failure due to poor construction, it seemed prudent to install some steel anchorage to act as a "fail-safe" feature. Reinforcing steel grouted in drilled holes was, therefore, added.

Shear Factory of Safety

The factory of safety is the ratio of the allowable working load or service load to the load that will be imposed on the facility. Through design calculations, it was determined that the critical design consideration was the capability of the concrete to resist the load transfer in shear. The factor of safety for shear was believed acceptable. However, final consideration for the "safe" and "fail-safe" features of the plug design resulted in the addition of three feet in the length of the plug for Tunnel No. 1, the addition of an earthquake load to the design criteria and the installation of reinforcing steel grouted in drilled holes to prevent a sudden, total movement of the concrete mass.

Calculated factors of safety under various conditions are shown in the following:

Concrete plug, Tunnel No. 1, design head = 122 meters

Factor of safety - shear V

Allowable unit shear: $V_c = 7.73$ kg/cm^2

100 percent peripheral contact, no earthquake forces		F.S. = 4.6
75 percent peripheral contact, no earthquake forces		F.S. = 3.5
100 percent peripheral contact, earthquake force	= 0.1g	
75 percent peripheral contact, earthquake force	= 0.1g	

Computed factors of safety in the tabulation are based on a working stress in shear of 7.73 kg/cm^2, that which is allowed by the current A.C.I. Code. The allowable shearing stress used in ultimate stress design procedures is 13.1 kg/cm^2. In other words, there is an inherent factor of safety in the working stress used in final design computations. The additional factor of safety is equal to 13.1/7.73 = 1.7, which is in addition to the F.S. values in the tabulation.

Original design computations assumed a condition of the finished plug that resulted in only 50 percent of the peripheral area being in contact with the rock of the tunnel. A factor of safety of the design condition was computed for a test of the concrete shearing resistance. The original condition is felt to be too severe to be a practical test.

The assumption of 75 percent contact was made because it is believed that excellent contact can be achieved at the floor and walls. Floor and wall areas not in contact would be small. Load transfer from the plug to the rock would take place in contact areas. The resolution of the internal load from shear to the compressive stress at the floor and wall contact would adjust to those areas of contact. This assumption provides for no contact along the roof at all, where reasonably good contact can be achieved with proper construction methods. The tabulation shows acceptable factors of safety for the assumed conditions.

The reinforcing steel was not considered in computations to determine the factor of safety with regard to the capability of the facility to resist the imposed load in shear.

Grouting

It was anticipated that the rock surrounding the concrete plugs would be fractured somewhat by blasting operations. It was also anticipated that some shrinkage could take place at the concrete-rock interface. In order to prevent leakage through this zone and to consolidate the mass of rock around the plug, it was proposed to drill three radial grout rings before plug placement and to grout after the plug concrete had hydrated and cooled. The upstream and downstream rings would be considered as low pressure contact grout rings, and they would be grouted before the center ring. Grout holes of the center ring would be drilled deeper than those of the outer rings, and they would be grouted at higher pressures.

Concrete Placement

Concrete to be placed in the plugs could be transported through the tunnels to the plug sites or pumped through pipe placed in drill holes from the ground surface. There are practical and costly problems inherent in both methods. The transportation of materials through several hundred meters of tunnel to the plug locations is perhaps the most obvious obstacle to the method using tunnel access. The mobilization and demobilization of equipment to provide transportation through the tunnels is a major item of expense. It could be accomplished, however, and should produce the desired result with the degree of control appropriate for the severity of the design criteria.

The second approach, that of placement through drill holes from the ground surface, also presents serious problems. First, drilling the required holes to intersect the tunnels at the chosen plug sites would be difficult and costly. Drill holes can be well enough aligned to allow placement of pumps, but most holes will deviate from true plumb to some degree. The exploratory holes drilled for the subject project are examples. All were located on the ground surfaces, directly on the projection of the tunnel centerlines. Yet of the six holes so drilled, only two actually intersected the tunnels. Drill hole cost would be significant, and drill holes that missed the tunnels would increase the cost. It would be necessary, in addition, to construct substantial access roads to the surface locations of each drill hole. Further, tunnel access would still be required to place bulkheads and piping, to excavate rock, and to grout.

Aside from financial considerations, the placement of concrete through 152 meters-deep drill holes presents quality control and placement problems of considerable magnitude. Concrete cannot be permitted to "free fall", but must be restrained by devices placed at intervals in the system. Adequate air venting must be provided, with appropriate valves and restraints at the terminal end. It is felt that complete failures in the midst of placement operations are definitely possible. While placement of concrete has been accomplished by these means to depths of 152 meters, it is described by experienced people as "difficult".

The placement of concrete by pumping down to the plug sites through drilled holes provides no savings in costs that are related to other aspects of sealing. Both pregrouting of fractures in peripheral rock and postgrouting of the shrinkage annulus will be required. Despite the ability to surcharge the concrete plug forms, shrinkage after initial set will still occur.

It is our belief that, though the method of construction by tunnel access presents unusual construction problems and costs, it is the best alternative for sealing the South Green Mountain tunnels.

Chemical Attack

The acid water environment had a definite effect on all design considerations. Although the flowing water contains relatively low concentrations of acid and sulfate, it is believed that the stored water could contain concentrations of acid and sulfate to as much as 2,000 mg/l, or more. Both adversely affect concrete and certain metals.

Sulfates combine with cement to form insoluble compounds that disrupt the physical characteristics of the concrete because their volume is greater than the volume of the cement matrix from which they are formed. The result is a cracking and spalling of the concrete surface. If the concrete mass is dense, the action is superficial, such as rust on the surface of metal. If the concrete is porous, the action can be progressive through the mass. The stronger the sulfate concentration, the more active the corrosion. The U.S. Bureau of Reclamation has had considerable experience with sulfate attack on concrete. The following summarizes their experience:

TABLE 7. ATTACK ON CONCRETE BY WATERS CONTAINING SULFATE

Sulfate in Water Samples (as mg/l)	Relative Degree of Sulfate Attack
0 to 150	Negligible
150 to 1,000	Positive
1,000 to 2,000	Considerable
Over 2,000	Severe

Sulfates react chemically with the hydrated lime and hydrated calcium aluminate in cement paste to form calcium sulfate and calcium sulfoaluminate which are expansive. Concrete containing cement that has a low content of the vulnerable calcium aluminate is highly resistant to attack by sulfates.

Acids combine with constituents of concrete to form soluble compounds that can be removed by leaching through cracks, poorly bonded interface areas between metal and concrete or the foundation and concrete, or through voids that interconnect. Progressive failure of the concrete from acid attack occurs with water movement through the concrete. Covering the concrete with an acid-resistant surface is the best protection afforded against acid deterioration. Accordingly, the upstream faces of the plugs were to be lined with Neoprene rubber.

Acid water can only get to the concrete by moving through a failed rubber liner, by moving through the rock and reaching the plug via cracks or fissures in the rock, or by moving along the concrete-rock interface and into cracks in the concrete. Good construction methods and procedures can greatly minimize or eliminate these possibilities.

Accordingly, construction materials that resist attack by acids and sulfates were specified. These included metals used in the structure and chemical grout. Aggregate that would react to acids and sulfates was prohibited. In addition, A.S.T.M. Type II cement, resistant to sulfate attack, was specified.

It is not possible to predict the rate at which the concrete will deteriorate through contact with the acid mine water. Variables include: (1) the ultimate concentrations of acid and sulfate in the stored water and (2) the extent to which the concrete comes into contact with the acid mine water and the form of that contact, that is, whether it is contact with motionless water or water in motion. Specified ingredients of construction will provide a facility that is resistant to attack. Construction methods and procedures will reduce the vulnerability of the facility to attack by the acid mine water. At best, it is believed that the facility will be problem-free for fifty years or more; at worst, it should be trouble-free for at least 10 to 15 years. In any case, the very nature of the project requires properly controlled surveillance. The aggressiveness of the acid mine water on the concrete can be checked periodically by obtaining a small sample core from an area close to the upstream face. One such set of samples obtained each year for three years and two more sets obtained at three-year intervals will establish a degree of concrete reaction to the environment, and, at the same time, provide continued information on the safe condition of the plug. In addition, the acid and sulfate concentrations and other characteristics of the stored water in contact with the seal can be determined periodically.

Water Control

The diversion of waterflow during plug construction is required. A suggested plan for diverting the tunnel flow through the plug site during construction using a concrete block barrier and temporary piping was incorporated into the construction plans and specifications. However, the contractor was responsible for the preparation of a diversion plan to be approved by the engineer before construction began.

To control pool elevations after construction, a stainless steel piping system with a regulating valve and energy dissipating chamber was incorporated into seal design. This system would enable stage filling of pools, and it could also be used for pool dewatering in an emergency.

The use of deep wells for pool water-level control or drawdown was explored in lieu of the piping system. Their use was not considered an acceptable alternate method of providing dewatering of the mine workings. Cost comparisons alone ruled out this approach. Using 1975 prices for equipment, and assuming only sufficient capacity to draw down the expected impoundment in a 1-year period of constant pumping, it was estimated that drilling costs would approach \$100,000 and pump costs would be \$300,000. More rapid drawdown would incrementally increase costs. Pumping capacity that provides such small discharge volumes over inflow would extend the total drawdown time significantly if any power or pump failures occurred. The cost of required power line construction and maintenance could not be computed without additional investigation not believed necessary to reach a conclusion regarding this method. The cost of electricity required for a single drawdown cycle of one year, using a cost of \$0.01per kilowatt hour, would approximate \$150,000. An additional negative aspect to the pumping alternative is that the serviceability of pumps and controls over long periods of time would be questionable. Depreciation and replacement costs would be high in any economic analysis of a pumping scheme.

Because a diversion of waterflow during plug construction would be required, a pumping scheme would not replace the total proposed piping system cost. Either piping to effect diversion through the plugs, or auxiliary plugs coupled with pumping, would be required. In any case, only a portion of the cost estimated for the proposed plan could be replaced.

Plug Instrumentation

Information on rock mass response, especially during the initial period of impoundment, could be critically important in accurately evaluating the long-term integrity of the plugs. Accordingly, the plug in Tunnel No. 2 was designed to be equipped with instrumentation, including pore water pressure cells to monitor water pressure in the rock outward from the plug, extensometers to monitor minute displacement of the plug in the downstream direction, and deflectometers to monitor shearing deformation at the plug-rock interface and outward in the rock.

Finally, the instrumentation and the outlet facilities for the proposed plug in Tunnel No. 2 were designed to be remotely controlled through a multiple conductor cable suspended in a borehole drilled from the surface overlying Tunnel No. 2.

Design Calculations

Tunnel No. 1 represents the highest loading condition among the three drainage tunnels. The plug designed for this tunnel would be adequate, therefore, for the other two tunnels.

Modes of Possible Failure--

The plug must be safe against:

Shear Failures

a. Rock-on-Rock Sliding. In this case, failure will take place as a result of slippage along the joints and fracture surfaces in the rock around the plug.

   Pw = applied force = weight of water in front of the plug

R = resisting force = shear strength of the rock along the joints and fracture surfaces

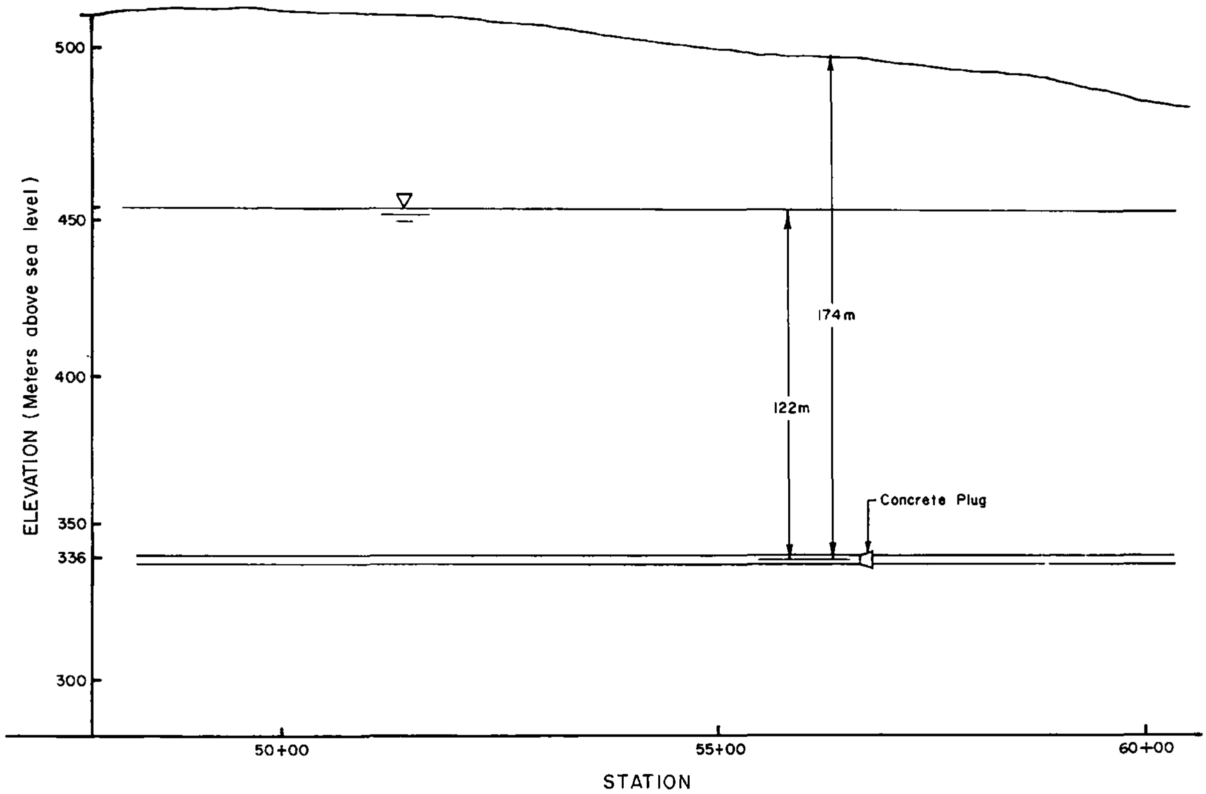


Figure 11. Profile of Tunnel No. 1.

Results of laboratory direct shear tests on rock samples from Tunnel No. 1 are set forth in Appendix C. The tests indicate the following post-peak shear parameters:

Borehole 1-C (Sample No.)	Cohesion (kg/cm^2)	Angle of Internal Friction
1	0	44°
1	3.9	30°
2	0	27°

These laboratory shear values reflect the range of shear parameters that can be used in computing the resisting shear force, R. The computations will be carried out conservatively, using the test results of Sample No. 2.

Applied force = $Pw = 1/2 \times 2.1\ m \times (p_1 + p_2)$

$= \frac{2.1\ m}{2} [d_w\ (122\ m \quad 1.07\ m) + d_w\ (122\ m + 1.07\ m)]$

Where d_w = density of water
$= 2.1\ m \times d_w \times 122\ m = 2.1\ m \times 1{,}000\ kg/cm^2 \times 122\ m$
$= 256{,}200\ kg/m \times 3.1\ m = 794{,}000\ kg$

Resisting force = R = T x p where
p = perimeter of rock slide area x length
$T = C + P_3 \tan a$
P_3 = Vertical pressure on the tunnel
= 174 m x density of rock, submerged ($d_{r,s}$)

Density of rock, dry = apparent specific gravity from lab x $1{,}000\ kg/m^3$
$= 2.76 \times 1{,}000\ kg/m^3 = 2{,}760\ kg/m^3$

Density of rock, submerged ($d_{r,s}$) = density of rock, dry- (1 - porosity) x $1{,}000\ kg/m^3$

Assume a porosity of 4%
$d_{r,s} = 2{,}760\ kg/m^3 - 0.96 \times 1{,}000\ kg/m^3 = 1{,}800\ kg/m^3$
$P_3 = 174\ m \times 1{,}800\ kg/m^3 = 313{,}200\ kg/m^2$
$T = C + 313{,}200\ kg/m^2 \times \tan 27°$
$= 0 + 313{,}200\ kg/m^2 \times 0.509 = 159{,}419\ kg/m^2$
$p = (2 \times 3.7\ m + 2 \times 2.7\ m)\ 3.36\ m = 12.8\ m \times 3.36\ m = 43.0\ m^2$

This is the perimeter, p, along the most critical sliding surface in the rock as shown in Figure 12. In reality, the sliding would have a tendency to take place along existing joint and fracture planes in the rock (dip 30° to 40° as shown in Figure 12), resulting in an increase in p and thus the computed resisting force, R. The sliding condition shown in Figure 13 results in minimum p, and thus minimum R.

R = T x p
$= 159{,}419\ kg/m^2 \times 43.0\ m^2 = 6{,}855{,}017\ kg$

Factor of Safety (F.S.) = $\frac{\text{resisting force}}{\text{applied force}}$ = R/Pw

$= \frac{6{,}855{,}017\ kg}{794{,}000\ kg} = 8.6$

b. Concrete-on-Rock Sliding. This condition would come about if the concrete plug slides along its contact surface with the surrounding rock. The calculations will be carried out assuming the contact surface is not grouted. This assumption is obviously conservative,

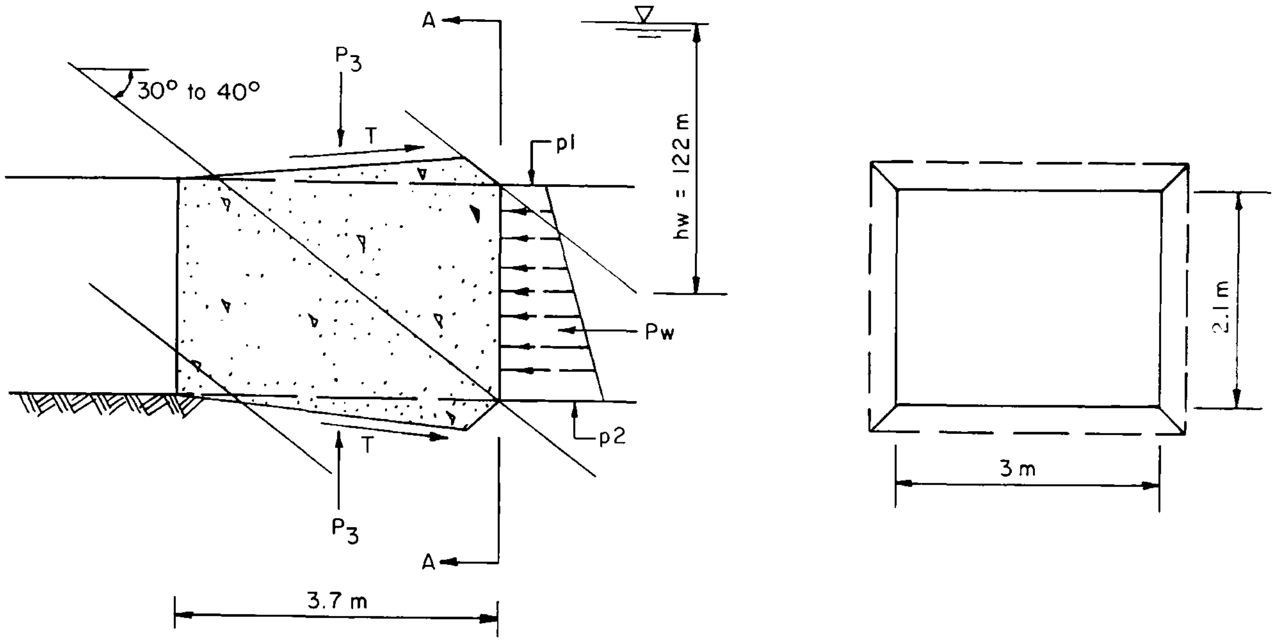


Figure 12. Force diagram of plug.

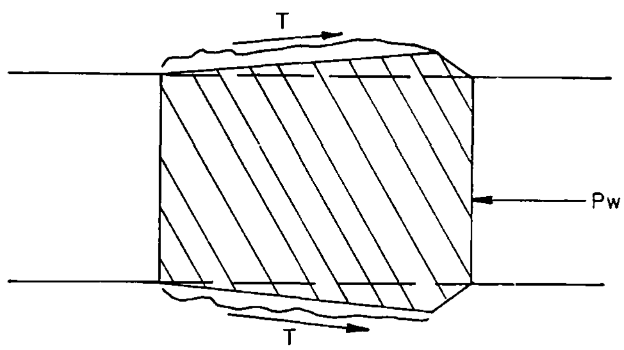


Figure 13. Critical surface sliding area.

as the grouting would have a tendency to keep the concrete plug and the rock together.

In order to calculate the resisting force, R, shear parameters must be known. Laboratory direct shear tests were carried out on rock-concrete samples by forcing the failure plane along the rock-concrete interface. Rock samples used in these tests were obtained from Tunnel Nos. 2 and 3. Samples from both tunnels yielded the same results. The post-peak shear values are C = 0, angle a = 20°. These values are quite conservative since the rock surface against which the concrete was placed in the laboratory was much smoother than the one that would result from blasting in the field. Using these values:

$R = T \times p$ where
$T = C + P_3 \tan a$

$= 0 + 313{,}200 \text{ kg/m}^2/\tan 20°$
$= 313{,}200 \text{ kg/m}^2 \times 0.36 = 122{,}752 \text{ kg/m}^2$
$p = 43.0 \text{ m}^2$
$R = 112{,}752 \text{ kg/m}^2 \times 43.0 \text{ m}^2 = 4{,}848{,}336 \text{ kg}$

$$F.S. = r/P_w = \frac{4{,}848{,}336 \text{ kg}}{794{,}000 \text{ kg}} = 6.$$

c. Concrete Shear Failure. In comparing the strength properties of the rock obtained from the laboratory tests with those of concrete, it becomes evident that the concrete is weaker than the rock. It, therefore, becomes important to evaluate the possibility of a shear failure with the shear planes located completely within the concrete plug. This condition is illustrated in Figure 14.

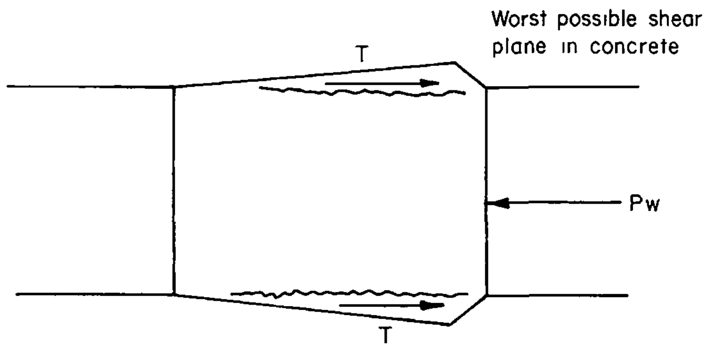


Figure 14. Critical concrete shear failure.

$R = V_{all} \times p$ where
V_{all} = allowable shear = 52,733 kg/m² (for $f_c' = 211 \text{ kg/cm}^2$, ACI Code)
p = perimeter of concrete slide area x length
$= 3.7 \text{ m } (2 \times 2.1 \text{ m} + 2 \times 3.0 \text{ m}) = 38.0 \text{ m}^2$
$R = 52{,}733 \text{ kg/m}^2 \times 38.0 \text{ m}^2 = 2{,}003{,}854 \text{ kg}$

$$F.S. = R/P_w = \frac{2{,}003{,}854 \text{ kg}}{794{,}000 \text{ kg}} = 2.5$$

This factor of safety ignores the influence of reinforcing bars. The addition of reinforcing bars to concrete along the plug faces would increase the calculated factor of safety.

d. Influence of Contact Area. The computations carried out thus far assume the plug and the surrounding rock are in complete contact around the periphery of the plug. In the field, such a contact may or may not be achieved depending on the thoroughness of concrete placement and subsequent contact grouting. It, therefore, becomes important to evaluate the influence of the contact area on the calculated factors of safety.

If only 50% of the plug periphery is in contact with surrounding rock, then the calculated factors of safety would be reduced by 50%. Thus:

F.S. against rock-on-rock sliding	= 4.2
F.S. against concrete-on-rock sliding	= 3.0
F.S. against concrete-shear failure	= 1.25

The assumption of 50% contact is quite conservative, however, and in fact, a much better contact area can be achieved with proper field inspection.

Deformations--From previous calculations, it was concluded that the proposed plug is safe against shear failures. The next step is to evaluate the order of magnitude of the deformations which the plug would undergo under the applied hydrostatic force of the proposed pool.

The maximum possible deformation that the plug could undergo would take place if all the overburden load is transferred to the plug, which in this case:

$E = P_3/E_{concrete}$ = Strain perpendicular to plug axis

$P_3 = 313{,}200\ kg/m^2$

$E_{concrete} = 2.46 \times 10^9\ kg/m^2$ (assumed)

$$E = \frac{313{,}200\ kg/m^2}{2.46 \times 10^9\ kg/m^2} = 1.27 \times 10^{-4}$$

Deformation Δh_p, perpendicular to tunnel axis = E x hp where

hp = change in height of plug = 2.1 m

$\Delta hp = 1.27 \times 10^{-4} \times 2.1\ m = 2.7 \times 10^{-4}$ m or 0.027 cm

Deformation Δlp, along the tunnel axis = E' x lp where

$E' = P_w/E$ = Strain along the plug axis

$P_w = 1{,}000\ kg/m^3 \times 122\ m = 122{,}000\ kg/m^2$

$$E' = \frac{122{,}000\ kg/m^2}{2.46 \times 10^9\ kg/m^2} = 4.96 \times 10^{-5}$$

$\Delta lp = 4.96 \times 10^{-5} \times 3.7\ m = 1.84 \times 10^{-4}$ m or 0.0184 cm

These deformations are small and are not expected to result in serious movements of the plug.

PRECONSTRUCTION PHASE

The proposed sealing of Tunnel No. 2 was advertised, and two bids in the amounts of $600,990 and $688,605 were received on April 22, 1975. Because both bids were significantly higher than the estimated construction cost of $320,000, a decision was made to readvertise.

The contract documents were clarified, and the November 29, 1974 construction cost estimate was adjusted to $398,000 on May 15, 1975 to accommodate increases in costs since November 1974, increased concrete prices based on recent bidding experience, increased electrical work costs to extend a power line to the site, and increased instrumentation prices quoted by the supplier on April 15, 1975. The project was readvertised, and bid opening was held on July 24, 1975. A single bid in the amount of $600,360 was received from the contractor who had submitted the low bid on April 22, 1975.

Although this bid price was about 50 percent higher than the estimate, its acceptance was recommended on the basis that this difference represented the contractor's view of the element of risk associated with the work. However, the Department rejected the bid and terminated the project on August 21, 1975 because of the excessive cost.

Discussion of Bid

On July 25, 1975, the Department requested comments on the abstract of the lone bid. This abstract, together with the engineer's estimate, is presented in Table 8.

As part of the analysis of the bid, prospective suppliers and subcontractors were contacted to determine whether the bidder had obtained quotations for the various specialty items that were part of the project. While the products referred to were not exclusively confined to one source, they were not readily available from more than a few sources. Specifically, this applied to the instrumentation items, the special valves, and the armored borehole cable.

At the direction of the Department, the included instrumentation was defined as similar, or equal, to that available from Terrametrics, Inc., of Golden, Colorado. Terrametrics reported that they had furnished quotes to the bidder.

The 8-inch[1] regulating valve is specified as similar, or equal, to that manufactured by Allis-Chalmers. This valve is a special item, which incorporates energy dissipating features required by the nature of the installation. Allis-Chalmers reportedly furnished a quote to the bidder.

The 16/60 multiple conductor cable has special requirements related to its installation in a 183-meter deep borehole. Such a product, as specified, is manufactured by Okonite Corporation of Cherry Hill, New Jersey, and the Vector Cable Company of Sugar Land, Texas. While other products may meet the specification as well, neither Okonite nor Vector was contacted for a quote by the bidder.

(1) The English system of measurement was required in the technical specifications and bidding documents and, therefore, is used in the discussion of the bid. A table of conversion to the metric system is included on page 78.

TABLE 8. ABSTRACT OF BID

Item No.	Description	Approx. Quantity	Unit	Low Bid Unit Price	Low Bid Total	Engineer's Estimate Unit Price	Engineer's Estimate Total
1	Mobilization and Demobilization						
	(a) Mobilization and demobilization, plant and equipment	Job	Job	L.S.	$18,000.00	L.S.	$78,000.00
	(b) Bonds and required insurance	Job	Job	L.S.	10,000.00	L.S.	5,700.00
2	Diversion and care of water	Job	Job	L.S.	40,000.00	L.S.	2,000.00
3	Excavation, Rock	Job	Job	L.S.	27,000.00	L.S.	5,000.00
4	Drilling and Grouting						
	(a) Mobilization and demobilization	Job	Job	L.S.	36,000.00	L.S.	6,400.00
	(b) Drilling 3-inch (NX) grout holes, Ring Grouting	1,100	L.F.	$35.00	38,500.00	$40.00	44,000.00
	(c) Drilling 1½-inch (EX) grout holes	100	L.F.	30.00	3,000.00	30.00	3,000.00
	(d) Grout pipe	2,600	Lbs.	10.00	26,000.00	3.75	9,750.00
	(e) Connections to grout pipe	100	Ea.	35.00	3,500.00	19.25	1,925.00
	(f) Placing grout	1,000	Gals	20.00	20,000.00	23.00	23,000.00
5	Concrete	40	C.Y.	600.00	24,000.00	400.00	16,000.00
6	Steel Reinforcement	2,250	Lbs.	4.00	9,000.00	2.00	4,500.00
7	Neoprene Rubber Cover	125	S.F.	50.00	6,250.00	37.00	4,625.00
8	Drainage System						
	(a) 10-inch gate valve, complete (stainless steel)	1	Ea.	25,000.00	25,000.00	13,750.00	13,750.00
	(b) 8-inch Regulating Valve complete with motor operator	1	Ea.	45,000.00	45,000.00	40,150.00	40,150.00
	(c) Piping assembly	1	Ea.	25,000.00	25,000.00	12,760.00	12,760.00
	(d) Trashrack, complete	1	Ea.	9,000.00	9,000.00	3,850.00	3,850.00
9	Security Fence	Job	Job	L.S.	2,000.00	L.S.	660.00
10	Electrical Work	Job	Job	L.S.	140,000.00	L.S.	56,400.00
11	Instrumentation						
	(a) Field Representative	Job	Job	L.S.	9,000.00	L.S.	9,000.00
	(b) Pore Water Pressure Cell	8	Ea.	1,200.00	9,600.00	700.00	5,600.00
	(c) Deflectometer	6	Ea.	1,800.00	10,800.00	935.00	5,610.00
	(d) Extensometer	2	Ea.	1,400.00	2,800.00	980.00	1,960.00
	(e) Drilling 3-inch Holes	188	L.F.	25.00	4,700.00	50.00	9,400.00
	(f) Conduit-Junction Box	Job	Job	L.S.	2,000.00	L.S.	660.00
	(g) 16/4 Signal Conductor	Job	Job	L.S.	2,000.00	L.S.	700.00
	(h) 16/60 Multiple Conductor Cable	620	L.F.	38.00	23,560.00	18.00	11,160.00

(Continued)

TABLE 8. (Continued)

Item No.	Description	Approx. Quantity	Unit	Low Bid Unit Price	Low Bid Total	Engineer's Estimate Unit Price	Engineer's Estimate Total
11	(i) Switching Unit Lock Box	Job	Job	L.S.	$ 2,000.00	L.S.	$ 1,320.00
	(j) Conductor Conduit	Job	Job	L.S.	1,500.00	L.S.	880.00
	(k) Readout Instruments	Job	Job	L.S.	3,650.00	L.S.	2,250.00
12	Construct 6-inch Drilled Hole with 3-inch PVC Casing Pipe						
	(a) Mobilization and Demobilization	Job	Job	L.S.	3,000.00	L.S.	4,840.00
	(b) Construct 6-inch Drilled Hole	600	L.F.	$13.00	7,800.00	$ 7.25	4,350.00
	(c) Furnish and Install 3-inch PVC Casing Pipe	600	L.F.	3.00	1,800.00	4.40	2,640.00
	(d) Grout PVC Casing Pipe						
	(1) Grout	80	Bags	10.00	800.00	22.00	1,760.00
	(2) Grout Stop and Start	1	Ea.	100.00	100.00	440.00	440.00
13	Switching Unit Lock Box Housing, Complete	Job	Job	L.S.	8,000.00	L.S.	4,360.00
		Total Amount Bid			$600,360.00		$398,000.00

In addition to these materials, the grouting for this project required special expertise and materials. An extensive search was conducted during design to locate a material that would be resistant to acid conditions, would have adequate strength, would bond to wet rock and concrete, and would have a viscosity at placement temperature that would enable its injection into very small openings. While many chemical grout products are available, most do not satisfy these rather stringent requirements. The Halliburton Services Company of Duncan, Oklahoma, is the only source located by our search for a product. Halliburton's office in Pittsburgh sent a quote to the bidder on April 17, 1975.

Prospective suppliers of the 10-inch stainless steel gate valve included The William Powell Company of Narberth, Pennsylvania, and Stockham Valves and Fittings of Pennsauken, New Jersey. Neither of these prospective suppliers was asked to quote the valve by the bidder.

Analysis of the prices bid for individual items is as follows:

No. 1 The bidder's total price of $28,000 was $55,700 below the engineer's estimate. It is believed that the bidder included the cost of moving materials into and out of the tunnel in Items 3 and 5 instead of including this cost in Item 1. Furthermore, it is believed that the cost of providing temporary power was included in Item 10 rather than Item 1.

No. 2 This price represented the bidder's evaluation of costs and inherent risks related to project water problems. It is difficult to visualize the problems and risks in the order of magnitude represented by the bid.

No. 3 It is believed that the bid price for this item included mobilization costs, which properly should have been included in Item 1.

No. 4 Bid prices could not be justified.

No. 5 It is believed that the bid price included the cost of transporting the concrete through the tunnel to the plug site. This cost should properly be included in Item 1.

No. 6 No comment.

No. 7 No comment.

No. 8 According to the general comments, Allis-Chalmers quoted a price for the 8-inch regulating valve; however, known sources of the 10-inch stainless steel gate valve did not furnish a quote to the bidder. The bid price could not be justified.

No. 9 No comment.

No. 10 It is believed that the bidder included the cost of temporary power in this item rather than Item 1; however, the bid price could not be justified.

No. 11 Terrametrics quoted the lone bidder a price for the instrumentation. Furthermore, Terrametrics quoted a price of $25.87 per lineal foot for supplying the 16/60 multiple conductor cable. Contacts with the Okonite Corporation and the Vector Cable Company revealed that these suppliers could provide the cable for approximately $11.00 per lineal foot. The engineer's estimate for this project was based on that price. Assuming that the bidder used the quote from Terrametrics for the cable, the revised engineer's estimate for Item 11 (h) would be $20,379.40 [($25.87 - $11.00) 620 + $11,160)], and the total engineer's estimate for Item 11 would be $57,359.40. However, the difference between this estimate and the bid price would still be $14,250.60. This difference cannot be justified.

No. 12 No comment.

No. 13 No comment.

Two bidders responded to the first advertisement of the project. Both bids were about double the engineer's estimate. The engineer's recommendation concerning this response was "that bids be rejected and that the project be readvertised." This recommendation was based on the inability to justify the cost to the Commonwealth, which the low bid represented, and also on the element of confusion surrounding two items of concern: (1) the Anthracite Mining Law requirement of two means of ingress to, and egress from, underground workings (the contract documents were not clear on this point); and (2) the installation of permanent power to the site. The specifications could have been interpreted as requiring the construction, by the contractor, of a power line from Township Route 818. Both of these above items were clarified in the documents issued for the second bidding.

In the evaluation of the bid prices received in response to the first advertisement, certain individual bid prices could not be justified. The evaluation of the second bidding, based on the tangible features of the work, resulted in the same conclusion. However, the result of the second bidding modified that conclusion to the extent that justification could only be assigned to the apparent element of risk associated with the work as represented in the three bids received when they are compared to the engineer's estimate. Accordingly, it was recommended that the Commonwealth accept the one bid received and award the contract to that firm.

Project Assessment

The required costly instrumentation, delays in reaching agreement and making decisions during inflationary cost spirals, and the lone bidder's evaluation of the risk connected with the work substantially increased the cost of constructing a watertight seal over that initially visualized.

From experience gained concerning the design and bidding process for construction work of this nature, it is recognized that the construction features, the specified methods and procedures covering construction, and the environment associated with the work are not acceptably applicable to a unit price contract. The inclusion of dollars in a bid to cover a calculated risk is not equitable to the contractor or the owner. It is strongly recommended that alternate methods be considered in the future for obtaining equitable bid prices. A bidding process based on the cost of time, material, and equipment, with a "Not to Exceed" bid price, would be one consideration.

In constructing watertight seals, the basic concept is simply a matter of improving the quality of mine drainage by substantially flooding the mined areas within the coal measures. It was the Department's desire to fully explore this concept in an orderly fashion by sealing one tunnel and evaluating the effectiveness of this seal before sealing the other two tunnels draining the structural basin.

If Tunnel No. 2 were sealed, the water behind the seal would rise and begin to exert hydrostatic pressure against the seal and the surrounding rock mass. This hydrostatic pressure would increase until the pool found relief. The interconnections between the underground mined areas drained by Tunnel Nos. 2 and 3 are substantially lower than the estimated level to which the mine water pool would rise if all three tunnels were sealed. In addition, the barrier pillar that separated the mined areas drained by Tunnel Nos. 1 and 2 is believed to have been significantly breached by recent mining. Consequently, the pool created by sealing only Tunnel No. 2 would likely find relief by overflowing into the mine workings currently drained by both Tunnel Nos. 1 and 3. These overflows would have to drain through substantial underground workings before discharging. Therefore, it is questionable whether there would be any significant improvement in water quality. Furthermore, since the pool level behind the seal in Tunnel No. 2 would be substantially lower than design, an evaluation of the seal would be confined to the effect this reduced hydrostatic pressure would exert on the seal itself. Under these conditions, no conclusions could be realistically drawn concerning the concept of improving water quality by the construction of watertight seals. If this concept is to be developed, the project must include the sealing of Tunnel Nos. 1 and 3 together with an intensive water flow and quality monitoring program.

Following the Department's decision to terminate the project, the concept remains to be proved although it could have widespread use, especially in the anthracite region of Pennsylvania. The only remaining realistic abatement alternative available for the region impacting stream quality is collection and treatment. This alternative remains the most costly, however, due to its high capital, operation, and maintenance costs.

VII

VERIFY RATIONAL METHOD

DESCRIPTION

During preliminary investigations in the basin for the Federal Water Pollution Control Administration, a methodology was developed by which limited gaging and analytical data were translated into volumes and qualities of discharges at various design flow conditions:

Design Average--Average daily volumes, constituents, and characteristics during a year of normal precipitation;

Design Wet Weather--Average daily volumes, constituents, and characteristics during spring high groundwater levels caused by normal precipitation from December through April;

Design Maximum--Maximum daily volumes, constituents, and characteristics resulting from the maximum 24-hour accumulation of rainfall occurring, on the average, no more often than once every ten years.

Design Average as well as Maximum volumes were calculated using precipitation records, estimates of surface-water runoff coefficients, and estimates of evaporation-transpiration losses. Design Wet Weather volumes were calculated by adjusting, on the basis of precipitation over the period of record, flows observed during that portion of the FWPCA gauging, sampling, and analytical program conducted during high groundwater level periods. Constituents and characteristics for Design Average as well as Wet Weather conditions were based upon those found by the FWPCA gauging, sampling, and analytical program during periods of low and high groundwater level periods, respectively. Design Maximum constituents and characteristics were estimated from the results obtained during the FWPCA gauging, sampling, and analytical program.

Basically, the mine drainage discharges from the basin's water-level tunnels were assumed to originate from precipitation falling on two separate areas:

1. That watershed area overlying and tributary to the underground mine workings on which both surface-water runoff and water infiltrating the ground eventually enters the underground mine workings.

2. That watershed area contributing flow to Catawissa Creek upstream to the basin with subsequent entry into the underground workings

via a direct interconnection between Catawissa Creek and these underground workings.

Calculations and assumptions used to determine design mine drainage volumes during those original investigations are presented in the following.

CALCULATIONS AND ASSUMPTIONS FOR DESIGN AMD VOLUMES FOR FWPCA INVESTIGATIONS

Design Average AMD Volume

Estimated total average yearly precipitation in the Study Area and vicinity over the period of record = 127 cm or 0.0145 cm/hr

Study Area = 1,198 hectares

Area outside of the Study Area contributing ground and surface water to the Study Area = 1,611 hectares

Surface runoff coefficients
Study Area = 0.01
Area outside of the Study Area contributing ground and surface water to the Study Area = 0.25

Base flow from area outside of the Study Area contributing ground and surface water to the Study Area = 0.0066 $m^3/s/km^2$

Thirty (30) percent of the total precipitation on the Study Area assumed lost to the atmosphere by evaporation and transpiration

Precipitation on the Study Area infiltrating to underground workings

Total available precipitation

$$127\ \frac{cm}{yr} \times 1{,}198\ ha \times 10{,}000\ \frac{m^2}{ha} \times \frac{1m}{100cm} \times \frac{1\ day}{86{,}400sec} \times \frac{1\ yr}{365\ day} = 0.482\ m^3/s$$

Losses
Surface runoff direct to streams

$$0.01 \times 0.0145\ \frac{cm}{hr} \times 1{,}198\ ha \times 10{,}000\ \frac{m^2}{ha} \times \frac{1m}{100cm} \times \frac{1\ hr}{3{,}600sec} = 0.00482\ m^3/s$$

Evaporation and transpiration

$$\frac{30}{100} \times 0.482\ m^3/sec = 0.145\ m^3/s$$

Infiltration to underground workings = 0.33 m^3/s

Precipitation outside of the Study Area contributing ground and surface water to Study Area underground workings -

Base flow

$$0.0066\ m^3/s/km^2 \times 1{,}611\ ha \times 0.01\ \frac{km^2}{ha} = 0.106\ m^3/s$$

Surface runoff

$$0.25 \times 0.0145\ \frac{cm}{hr} \times 1{,}611\ ha \times 10{,}000\ \frac{m^2}{ha} \times \frac{1m}{100cm} \times \frac{1\ hr}{3{,}600sec} = 0.16\ m^3/s$$

Total contribution to Study Area underground workings = 0.27 m^3/s

Design Average volume (total precipitation discharged from Study Area underground workings as AMD) = 0.60 m^3/s

Design Wet Weather AMD Volume

Estimated total average precipitation in the Study Area and vicinity from December through April over the period of record = 43.2 cm

Conditions during gauging, sampling, and analytical program from December 1966 through April 1967

AMD volume during high groundwater level period = 0.89 m^3/s

Estimated total precipitation in the Study Area and vicinity = 41.4 cm

Precipitation deficiency = 4.1%

Design Wet Weather volume (total precipitation discharged from Study Area underground workings as AMD) = 0.93 m^3/s

Design Maximum AMD Volume

Estimated total 24-hour accumulation of rainfall that will occur no more frequently than once every 10 years = 11.6 cm or 0.486 cm/hr

Study Area = 1,198 hectares

Area outside of the Study Area contributing ground and surface water to the Study Area = 1,611 hectares

Surface runoff coefficients

Study Area = 0.01

Area outside of the Study Area contributing ground and surface water to the Study Area = 0.35

Base flow from area outside of the Study Area contributing ground and surface water to the Study Area = 0.016 $m^3/s/km^2$

Thirty (30) percent of the total rainfall on the Study Area assumed lost to the atmosphere by evaporation and transpiration

Rainfall on the Study Area infiltrating to underground workings

Total available rainfall

$$11.6 \frac{cm}{day} \times 1{,}198 \text{ ha} \times 10{,}000 \frac{m^2}{ha} \times \frac{1m}{100cm} \times \frac{1 \text{ day}}{86{,}400sec} - 16.1 \text{ m}^3/s$$

Losses

Surface runoff direct to streams

$$0.01 \times 0.486 \frac{cm}{hr} \times 1{,}198 \text{ ha} \times 10{,}000 \frac{m^2}{ha} \times \frac{1m}{100cm} \times \frac{1 \text{ hr}}{3{,}600sec} = 0.16 \text{ m}^3/s$$

Evaporation and transpiration

$$\frac{30}{100} \times 16.1 \text{ m}^3/s \quad 4.83 \text{ m}^3/s$$

Infiltration to underground workings = 11.1 m^3/s

Rainfall outside of the Study Area contributing ground and surface water to Study Area underground workings

Base flow

$$0.016 \text{ m}^3/s/km^2 \times 1{,}611 \text{ ha} \times 0.01 \frac{km^2}{ha} = 0.26 \text{ m}^3/s$$

Surface runoff

$$0.35 \times 0.486 \frac{cm}{hr} \times 1{,}611 \text{ ha} \times 10{,}000 \frac{m^2}{ha} \times \frac{1m}{100cm} \times \frac{1 \text{ hr}}{3{,}600sec} = 7.5 \text{ m}^3/s$$

Total contribution to Study Area underground workings = 7.8 m^3/s

Design Maximum volume (total rainfall discharged from Study Area underground workings as AMD) = 18.9 m^3/s

WATER FLOW AND QUALITY DATA

The flow and quality data obtained to determine the effectiveness of the Catawissa streambed reconstruction were also to be used in an attempt to verify the calculations and assumptions, and consequently the methodology, in determining original design flows.

The data, based on periodic grab samples and instantaneous flow measurements, have been previously presented in Tables 1 through 4.

During the one-year period, from March 1969 through February 1970, before completion of streambed reconstruction, the total average flow from the three drainage tunnels was 0.798 m^3/s. During the same period, the average flow of Catawissa Creek entering the underground mine workings was 0.253 m^3/s. Flow measurements taken after streambed reconstruction indicated total average flow of 0.561 m^3/s from the three drainage tunnels during the period of March 1970 through January 1971.

HYDROLOGIC CONSIDERATIONS

As presented in Section V, precipitation data used in this report were obtained from the two closest reporting stations: Zion Grove and Tamaqua 4N Dam. During the gaging and sampling program from March 1969 through February 1970 before Catawissa Creek streambed reconstruction, their average annual rainfall was 107 cm, which closely approximated the average normal annual rainfall of 106 cm for these two stations over an 18-year period of record. During the 11 months from March 1970 through January 1971, precipitation at the two reporting stations averaged 103.7 cm. Average normal precipitation for this time at the two reporting stations over the period of record was 99.9 cm. However, original estimates of mine drainage discharges were made on the presumption that normal precipitation in the Study Area was 127 cm - - the long-term regional normal precipitation.

A surface runoff coefficient for the upstream watershed area of 0.25 was originally used to estimate Catawissa Creek's contribution to the underground mine workings in the basin. A later study of this upstream watershed area revealed that a significant portion of that area overlies the Jeansville Basin of coal. This overlying surface area has been very extensively strip mined and is directly interconnected with the Jeansville Basin's underground mine workings, which discharge to Catawissa Creek via the Audenried Tunnel.

Other complicating factors that exert an influence on the surface runoff coefficient for this upstream watershed area include:

1. Several public water supply reservoirs;

2. Several recently constructed impoundments in strip mines not interconnected with the underground mine workings, used for fishing or as sources of water for coal preparation facilities;

3. Intermittent withdrawal of water from these impoundments for use in these coal preparation facilities with ultimate discharge to the Audenried Tunnel rather than to Catawissa Creek upstream from the basin;

4. Varying wastewater flows to Catawissa Creek upstream from the basin contributed by the McAdoo Borough wastewater collection system from homes and a major industry; and

5. Construction of Interstate Highway 81 across the upstream watershed area after the original investigations were completed.

SUMMARY

As discussed earlier, instantaneous flow measurements and grab samples for analysis were planned to be taken weekly from Catawissa Creek immediately upstream from the basin and from the three water-level tunnels. During most of the year before Catawissa Creek streambed reconstruction (March 1969 through December 15, 1969), this program was followed. However, snowfall accumulations of up to 91.4 cm in the basin during the winter of 1969-1970 prevented this weekly collection of information, which was eventually resumed during April 1970. These data were used to estimate the acid load reduction resulting from the streambed reconstruction.

It is clearly evident, however, that this information is not sufficient to verify the methodology used to determine the original design flows presented in the 1968 FWPCA report. The rates of flow in Catawissa Creek and the water-level tunnels are continually changing. Although there is a correlation between these flows and rainfall, this correlation can only be determined if complete and continuous precipitation and flow records are available. Obviously, one instantaneous flow measurement each week (or at times less frequently) at each of the measuring points will not provide accurate results on which verification of this method could be established. Continuous flow and precipitation records for at least one hydrologic year before and one hydrologic year after construction are felt necessary as a minimum to enable such determination.

CONVERSION TABLE

	Customary Equivalents			Metric Equivalents	
	Multiply		By	To Obtain	
	Unit	Symbol		Unit	Symbol
Length	inch	in	2.54	centimeter	cm
	foot	ft	0.3048	meters	m
	mile	mi	1.61	kilometer	km
Area	square yard	sy	0.836	square meter	m^2
	acre	ac	0.405	hectare	ha
	square mile	-	2.59	square kilometer	km^2
Volume	cubic yard	cy	0.7645	cubic meter	m^3
	gallon	gal	3.785	liter	l
Mass	pound	lb	0.4536	kilogram	kg
	ton	-	0.9074	tonne	t
Flow	gallons per minute	gpm	0.06309	liter per second	l/s
	cubic foot per second	cfs	0.02832	cubic meter per second	m^3/s
	million gallons per day	mgd	0.0438	cubic meter per second	m^3/s
Pressure	pound per square inch	psi	0.07031	kilogram per square centimeter	kg/cm^2
	pound per square foot	lb/sq ft	4.88	kilogram per square meter	kg/m^2
Density	pound per cubic foot	lb/cu ft	16.02	kilogram per cubic centimeter	kg/cm^3

APPENDIX A

TYPICAL AGREEMENT AND GRANT OF EASEMENT
(Reproduced as Written)

THIS AGREEMENT AND GRANT OF EASEMENT made and given this 1st day of August, 1974 by and between Butler Enterprises, Inc., a Pennsylvania corporation and Corson Realty Corporation, a Pennsylvania corporation, both with offices in Hazleton City, Luzerne County, Pennsylvania, hereinafter at times called "Grantors", and the Department of Environmental Resources, acting as agent for the Commonwealth of Pennsylvania, hereinafter at times called "Grantee".

WITNESSETH: That in consideration of the benefits which may accrue to the Grantors and to the General Public from the Catawissa Creek Mine Drainage Pollution Abatement Project, the Grantors do hereby grant and convey unto the Grantee the right and to delegate this right to other agencies or individuals as the work may require, to enter upon and into that certain tract of land as outlined in red on the attached map in East Union Township, Schuylkill County and in Hazle Township, Luzerne County with full rights of ingress, egress, and regress upon and into said land for the purpose of performing such work as may be required for planning and completing said project, and for the consideration aforesaid, the Grantor does hereby grant and convey to said Grantee the following rights, right of way and easements pertaining to the surface of said land:

a. To construct, and operate vehicles and equipment on access roads to sites where work will be performed by man and equipment. After these roads have served their purpose which were constructed in performance of work under this easement which the Grantors do not wish to maintain will be leveled, regraded, and revegetated or made inaccessible by other means mutually acceptable to the Grantors and Grantees.

b. To remove garbage and debris, from areas where work is to be performed to a place and disposal as agreed upon by the parties hereto.

c. To backfill, grade, and ditch in strip mine areas with the understanding that vegetation and trees will be planted on reclaimed areas.

d. To transport men and equipment and to operate vehicles on private roads.

e. To store materials and equipment on the ground surface on the areas outlined on the attached map.

f. To dispose of rock and debris removed from the water level tunnels on the ground surface at the sites indicated on the attached maps.

g. To seed in a cover crop of grass and legumes, and to plant brush and trees to provide soil stabilization, in areas disturbed.

h. To apply soil amendments in areas disturbed to promote growth of vegetation; substances which may be used, include agricultural fertilizers, digested sewage sludge, distillery wastes, saw-dust, wood chips, limestone, and fly ash.

For the consideration aforesaid, the following rights, rights of way and easements are hereby granted with relation to the subsurface of said land:

a. To enter into the drainage tunnels.

b. To construct weirs at the entrance to the drainage tunnels.

c. To drill observation boreholes at approximate locations indicated on the attached maps.

d. To drill test holes into the strata above the proposed sites for the drainage tunnel seals.

e. To construct suitable seals in the drainage tunnels with or without water traps outside of strip mine areas, at the locations indicated on the attached map, and to innundate the coal basin to an elevation of approximately +1510 feet.

f. To construct and/or create an overflow from the South Green Mount Basin at the site indicated on the attached map.

The Grantor agrees that for a period of five (5) years after the date of this Agreement, it will not commit any act to cause the release of water through any mine seals in said mine tunnels. All flow into Catawissa Creek will be from natural stream flow and mine overflow as indicated on the attached map. In the event of an emergency where damage to life and/or property is involved, the Grantee has the right to lower the mine pool at any time.

The Grantee agrees, after a five (5) year period, to release the impounded water, within thirty (30) days after receiving written notice from the Grantor, to provide flow augmentation in the event the Grantor constructs an impoundment below the project area and the natural stream flow is insufficient because of changes in the hydrological characteristics of the water shed due to the abatement project, providing the water is of adequate quality for stream release.

All rights, rights of ways and easements herein granted are for the purpose of permitting the Grantee and its delegates to do the things hereinbefore set forth, all for the purpose of planning, developing, monitoring and completing the project and shall expire five (5) years from the date hereof. It is covenanted by the Grantors that they will not voluntarily do any act or permit any act to be done that will destroy or materially hurt or change the complete project.

It is understood by the grantors that acceptance of this Agreement and Grant of Easement by the grantee does not relieve the grantors of any obligations otherwise due the grantee or by the State of Pennsylvania, and does not exempt grantors from any requirements of the laws of the State of Pennsylvania; in any event the same shall in no way be construed to impose any financial obligation against the undersigned parties.

IN WITNESS WHEREOF, the parties hereto have executed this Agreement and Grant of Easement the date first above written, intending to legally bind themselves, their successors and assigns.

Attest:

(Signed) Philip S. Seltzer
Philip S. Seltzer, Secretary

Corporate Seal:

Butler Enterprises, Inc.

By: (Signed) Nathan R. Seltzer
Nathan R. Seltzer, President

Attest:

(Signed) Theodore R. Laputka
Theodore R. Laputka, Secretary

Corporate Seal:

Corson Realty Corporation

By: (Signed) Anthony Blass
Anthony Blass, President

Approved As to Form And Legality

(Signed) Gary L. Martin
Assistant Attorney General

ACKNOWLEDGMENT

Corporation

COMMONWEALTH OF PENNSYVLANIA
COUNTY OF LUZERNE

On this 1st day of August, 1974, before me, the subscriber, a Notary Public, personally appeared Anthony Blass (known to me or satisfactorily proven to be the person described in the foregoing instrument), who acknowledged himself to be the President of Corson Realty Corporation a Corporation, and that he, as such President, being authorized so to do, executed the foregoing instrument for the purpose therein contained, by signing the name of the corporation by himself as President.

IN WITNESS WHEREOF, I hereunto set my hand and official seal.

(Signed) Virginia Hinkle,
NOTARY PUBLIC
Virginia Hinkle, Notary Public
Hazleton, Luzerne County, Pennsylvania
My Commission expires April 12, 1977

SEAL

APPENDIX B

LOGS OF TEST BORINGS

Project Catawissa Creek Tunnel #1 Hole No. 1 Sheet 1 Of 3

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
		10.0		14.5	4.5	3.6	5.0' Light Brown Sand and Boulders	
	5.0	14.5		25.0	10	9.1		
		25.0		39.0	14.0	13.8	8.0' Red Shale, Soft Broken & Fractured	
	13.0	39.0		51.4	12.4	12.6		
		51.4		59.0	8.6	8.3	11.7' Gray Shaley Sand-stone	Medium Hard
	24.7	59.0		69.0	10.0	10.1		
		69.0		85.0	16.0	16.4	42.1' Red Shale Medium Hard, Broken & Fractured	
	66.8	85.0		95.0	10	11.0		
		95.0		105	10	10.4	38.2' Brown Sandstone, Broken & Fractured	
	105.0	105		109	4.0	4.0		
		109		129	20	20.1	3.8' Gray Sandstone	Fine Grain
	108.8	129		149	20	20.3		
		149		159	10	10	7.3' Gray Shale	Hard
	116.1	159		179	20	20.4		
		179		199	20	20.2	78.9' Red and Gray Shale	Hard
	195.0	199		209	10	10.1		
		209		219	10	10	18.0' Gray Sandstone Fine Grain	Hard
	213.0	219		239	20	20.4		

LOG OF TEST BORING

Project Catawissa Creek Tunnel #1 Hole No. 1 Sheet 2 Of 3

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
		239		259	20	20.2	82.7' Gray and Red Shale	Very Hard
	295.7	259		279	20	20.4		
		279		299	20	20.3	25.8' Gray Shaley Sand-stone, Fine Grain	Hard
	321.5	299		309	10	9.8		
		309		319	10	10.3	14.5' Red and Gray Shale	Very Hard
	336.0	319		339	20	20.3		
		339		352.3	14.3	14.3		
		352.3		359	6.7	6.3	24.5' Gray Shale Broken & Fractured	Very Hard
	360.5	359		379	20	20.4		
		379		396	17	17.2	17.3' Red Shale	Very Hard
	377.8	396		409	13	13		
		409		429	20	20.1	27.2' Gray Shale	Hard
	405.0	429		449	20	20.4		
		449		459	10	10	21.0' Gray Sandstone	Very Hard
	426.0	459		479	20	20		
		479		499	20	20.1	25.5' Red and Gray Hard Shale	Very Hard
	451.5	499		509	10	9.9		

Project Catawissa Creek Tunnel #1 Hole No. 1 Sheet 3 Of 3

Elevation	Depth	Casing Blows From	Sample No.	Sample Blows To	Graphic Log Run	Core Recovery Rec	Description of Material	Remarks
		509		529	20	20.	27.1' Gray Sandstone Fine Grain	Very Hard
	478.6	529		549	20	20.3		
		549		559	10	10	0.9' Gray Shale	Hard
	479.5	559		575	16	16		
							50.5' Red and Gray Shale	Very Hard
	530.0							
							30.1' Gray Sandstone Fine Grain	Very Hard
	560.1							
							4.9' Gray Shale	Very Hard
	565.0							
							10.0' Red Shale	Very Hard
	575.0							

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 1 River ______ Hole No. 1 Rig No. ______

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Pa. Drilling Co. Driller Jack Johnson :Elev. top of hole 1566

Type & No. of Pump John Bean (435) Serial No. 125204 :No. of Meter Rockwell 19915723 :Elev. top of rock 1546

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal. or cu.ft. per min
From	To	From	To								
21.5	59	1544.5	1507	60	1:15	1:20	5	290.1	304.6	14.5	2.9
59	109	1507	1457	110	3:00	3:05	5	305.3	321.1	16.8	3.36
109	159	1457	1407	160	1:17	1:22	5	321.3	321.3	0.0	0.00
159	209	1407	1357	210	3:32	3:37	5	323.3	332.5	9.2	1.84
209	259	1357	1307	260	2:30	2:35	5	339.9	339.9	0.0	0.00
259	309	1307	1257	300	10:30	10:35	5	344.2	348.6	4.4	0.88
309	359	1257	1207	300	11:00	11:05	5	350.0	350.6	0.6	0.12
359	409	1207	1157	300	9:00	9:05	5	351.7	351.7	0.0	0.00
409	459	1157	1107	300	10:00	10:05	5	357.1	365.6	8.5	1.70

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
21.5	59	1544.5	1507	Dropped 60 psi in 15 sec					4-13-71
59	109	1507	1457	Dropped 110 psi in 20 sec					4-14-71
109	159	1457	1407	Pressure held at 75 psi					4-16-71
159	209	1407	1357	Dropped 210 psi in 45 sec					4-19-71
209	259	1357	1307	Pressure held at 70 psi					4-21-71
299	309	1307	1257	Pressure held at 20 psi					4-23-71
309	359	1257	1207	Pressure held at 140 psi					4-26-71
359	409	1207	1157	Pressure held at 160 psi					4-28-71
409	459	1157	1107	Dropped 300 psi in 10 sec					4-29-71

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 1 River ____________ Hole No. 1 Rig No. ______

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Pa. Drilling Co: Driller Jack Johnson :Elev. top of hole 1566

Type & No. of Pump John Bean (435) :No. of Meter Rockwell 19915723 :Elev. top of rock 1546
Serial No. 125204

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal. or cu.ft. per min
From	To	From	To								
459	509	1107	1057	300	10:30	10:35	5	372.1	372.4	0.3	0.05
509	559	1057	1007	300	2:35	2:40	5	375.1	376.0	0.9	0.18
559	575	1007	991	300	8:40	8:45	5	378.4	378.4	0.0	0.00

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									Remarks
Sec. of hole tested				Gage pressure at test intervals from					
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
459	509	1107	1057	Pressure held at 180 psi					5-3-71
509	559	1057	1007	Pressure held at 140 psi					5-4-71
559	575	1007	991	Pressure held at 175 psi					5-5-71

Description of operations and general information:

BY G.D.S. DATE ______ SUBJECT Grouting SHEET NO. 1 OF 1

CHKD.BY ______ DATE 4-13-71 to 5-5-71 JOB. NO. 5081

Tunnel No. 1 - Hole No. 1 135-4 Tunnel No. 1

LOG OF GROUTING

Project SL-135-4 Hole No. 1 Sheet No. 1 Of 1

Date	Depth (Interval Grouted)	Reason for Grouting (Loss or gain of water, caving hole, or other)	Material (Portland Cement or other)	Mix (W/C)	Method (Describe use of packer or other)	Pressure (If any)
4-13-71	21.5'-59'	Loss of water	Allentown (4 bags)	1/1	Pumped to bottom through drill rods	0
4-14-71	59'-109'	Loss of water	Allentown (4 bags)	1/1	Pumped to bottom through drill rods	0
4-19-71	159'-209'	Loss of water	Allentown (3 bags)	1/1	Pumped to bottom through drill rods	0
4-20-71	159'-209'	Insufficient return water	Allentown (6 bags)	1/1	Pumped to bottom through drill rods	0
5-5-71	0'-100'	Hole closure on Borehole No. 1	Allentown (5 bags)	1/1	Plug at 100' grout to G.E.	0
5-5-71	0'-100'	Hole closure on Borehole No. 2	Allentown (5 bags)	1/1	Plug at 100' grout to G.E.	0

LOG OF TEST BORING

Project Catawissa Creek Tunnel #1 Hole No. 2 Sheet 1 Of 3

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
		18.5		27.0	8.5	8.5	14.0' Overburden, Brown Sand & gravel with Conglomerate Boulders	
	14.0	27.0		38.8	10.8	10.0		
		38.8		42.8	4.0	3.6	4.5' Red-Brown Soft Sand Trace of Clay	
	18.5	42.8		54.8	12.0	12.0		
		54.8		59.1	4.3	4.3	18.3' Gray Conglomeratic Sandstone, Fine Grain	Very Hard
	36.8	59.1		79.0	19.9	19.4		
		79.0		96.7	17.7	17.9	6.0' Gray Brown Clay Shale	Medium Hard
	42.8	96.9		99.0	3.3	3.3		
		99.0		119	20	20	6.6' Brown Shale	Medium Hard
	49.4	119		139	20	20.4		
		139		149	10	10	78.8' Gray Shale Few Joints	Hard
	128.2	149		169	20	20.1		
		169		189	20	20.1	70.6' Gray Sandstone Fine Grain	Very Hard
	198.8	189		199	10	10		
		199		209	10	10	62.3' Gray Conglomeratic Sandstone	Very Hard
		209		229	20	20.2		
		229		249	20	19.9		
	261.1	249		269	20	20.1		

LOG OF TEST BORING

Project Catawissa Creek Tunnl #1 Hole No. 2 Sheet 2 Of 3

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
		269		289	20	20.3	3.9' Gray Shale	Hard
	265.0	289		299	10	10.1		
		299		319	20	20.2	14.3' Red Shale	Hard
	279.3	319		339	20	20.3		
		339		349	10	10	47.9' Gray Conglomeratic Sandstone, Fine Grain	Very Hard
	327.2	349		368.8	19.8	19.9		
		368.8		388.8	20	20.1		
		388.8		399	10.2	10.3	13.3' Gray Shale	Hard
	340.5	399		419	20	20		
		419		439	20	19.6	14.0' Red Shale	Hard
	354.5	439		449	10	10.0		
		449		459	10	10.4	10.5' Gray Sandstone	Very Hard
	365.0	459		479	20	20		
		479		499	20	20.1	7.5' Gray Conglomerate	Very Hard
	372.5	499		519	20	20		
		519		539	20	20.2	6.0' Red Shale	Hard
	378.5	539		549	10	10.2		
		549		559	10	9.8	20.0' Gray Conglomerate	Very Hard

LOG OF TEST BORING

Project Catawissa Creek Tunnel #1 Hole No. 2 Sheet 3 Of 3

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
	398.5	559		579	20	19.4		
		579		599	20	20.2	116.0' Red Shale	Very Hard
	514.5	599		619	20	20.3		
		619		639	20	20.2	23.1' Gray Shale	Very Hard
	537.6	639		650	11	11.0		
							33.8' Gray Conglomeratic Sandstone	Very Hard
	571.4							
							7.6' Gray Sandy Shale	Hard
	579							
							71.0' Red Shale	Hard
	650							

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 1 River ____________ Hole No. 2 Rig No. ______

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Pa. Drilling Co:Driller Jack Johnson :Elev. top of hole 1648.00

Type & No. of Pump John Bean (435) Serial No. 125204 :No. of Meter Rockwell 19915723 :Elev. top of rock 1629.50

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time start-ed	Time stop-ped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
19.0	59	1629	1589	59	9:43	9:48	5	168.0	194.2	26.2	5.24
59.0	99	1589	1549	100	3:40	3:45	5	200.8	229.7	28.9	5.78
99.0	149	1549	1499	150	2:44	2:49	5	231.3	255.9	24.6	4.92
149.0	199	1499	1449	200	3:06	3:11	5	256.2	256.2	0.0	0.00
199.0	249	1449	1399	250	11:00	11:05	5	257.1	257.1	0.0	0.00
249.0	299	1399	1349	300	2:26	2:31	5	257.3	257.3	0.0	0.00
299.0	349	1349	1299	300	1:31	1:36	5	257.9	257.9	0.0	0.00
349.0	399	1299	1249	300	10:16	10:21	5	258.6	258.9	0.3	0.06
399.0	449	1249	1199	300	8:45	8:50	5	261.7	263.9	2.2	0.45

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									Remarks
Sec. of hole tested				Gage pressure at test intervals from					
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
19	59	1629	1589	Dropped 59 psi in 4 sec					3-12-71
59	99	1589	1549	Dropped 100 psi in 2 sec					3-15-71
99	149	1549	1499	Dropped 150 psi in 2 sec					3-16-71
149	199	1499	1449	Pressure held at 150 psi					3-17-71
199	249	1449	1399	Pressure held at 190 psi					3-19-71
249	299	1399	1349	Pressure held at 250 psi					3-23-71
299	349	1349	1299	Pressure held at 75 psi					3-24-71
349	399	1299	1249	Pressure held at 50 psi					3-25-71
399	449	1249	1199	Dropped 300 psi in 50 sec					3-26-71

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 1 River ______ Hole No. 2 Rig No. ______

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Pa. Drilling Co: Driller Jack Johnson :Elev. top of hole 1648.00

Type & No. of Pump John Bean (435) Serial No. 125204 :No. of Meter Rockwell 19915723 :Elev. top of rock 1629.50

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
449	499	1199	1149	300	3:03	3:08	5	264.6	264.6	0.0	0.00
499	549	1149	1099	300	11:16	11:21	5	265.4	265.4	0.0	0.00
549	599	1099	1049	300	1:21	1:26	5	271.6	274.8	3.2	0.04
599	650	1049	998	300	8:40	8:45	5	231.0	289.2	8.2	1.64

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
449	499	1199	1149	Pressure held at 50 psi					3-30-71
499	549	1149	1099	Pressure held at 70 psi					3-31-71
549	599	1099	1049	Dropped 300 psi in 50 sec					4-1-71
599	650	1049	998	Dropped 300 psi in 35 sec					4-5-71

Description of operations and general information:

BY G.D.S.	DATE 3-1-71	SUBJECT Grouting	SHEET NO. 1 OF 1
CHKD.BY	DATE		JOB. NO. 5081
	Tunnel No. 1	Hole No. 2	135-4 Tunnel No. 1

LOG OF GROUTING

Project SL-135-4 Hole No. 2 Sheet No. 1 Of 1

Date	Depth (Interval Grouted)	Reason for Grouting (Loss or gain of water, caving hole, or other)	Material (Portland Cement or other)	Mix (W/C)	Method (Describe use of packer or other)	Pressure (If any)
2-12-71	19'-59'	Loss of water	Portland (3 bags)	1/1	Packer seated at 19' - grout pumped into hole through rods resting on bottom of hole rods are then pulled, releasing grout into hole	None
2-15-71	59'-99'	75% Loss of H_2O	Portland (5 bags)	1/1	"	"
2-16-71	99'-149'	75% Loss of H_2O	Portland (3 bags)	1/1	"	"
4-2-71	549'-599'	75% Loss of H_2O	Portland (3 bags)	1/1	"	"

LOG OF TEST BORING

Project Catawissa Creek Tunnel #1 Hole No. 3 Sheet 1 Of 5

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
	16.7	16.7		20.9	4.2	4.2	16.7' Brown Sand and Gravels with very hard Sandstone Boulders	Boulders 0.7 3.0'Thickness
	20.9	20.9		22.7	1.8	1.8	4.2' Red-Brown Conglomerate Sandstone, Broken	Very Hard
	25.0	22.7		28.9	6.2	5.7		
		28.9		38.9	10	9.9		
		38.9		42.9	4.0	3.8		
		42.9		46.9	4.0	4.1	229.1' Gray Congomertic Sandstone, Massive Bedded Few Vertical Seams	Very Hard
		46.9		55.4	8.5	8.6		
		55.4		62.9	7.5	7.0		

LOG OF TEST BORING

Project Catawissa Creek Tunnel #1 Hole No. 3 Sheet 2 Of 5

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
		62.9		66.9	4.0	3.9		
		66.9		76.9	10	9.8		
		76.9		86.9	10	10.2		
		86.9		96.9	10	10.1		
		96.9		104.9	8.0	7.6		
		104.9		108.9	4.0	4.4		
		108.						
		108.9		112.8	3.9	3.9		
		112.8		118.9	6.0	6.0		
		118.9		128.9	10	10		

Project Catawissa Creek Tunnel #1 Hole No. 3 Sheet 3 Of 5

Elevation	Depth	Casing Blows	Sample No.	Sample Blows	Graphic Log	Core Recovery	Description of Material	Remarks
		From		To	Run	Rec		
		128.9		138.9	10.	10.1	(229.1' Gray Conglomertic Sandstone, Massive Bedded Few Vertical Seams)	
		138.9		144.6	5.6	5.6		
		144.6		157.6	13.0	13.2		
		157.6		165	7.4	7.4		
		165		169	4.0	4.1		
		169		180.	11.7	11.7		
		180.7		199	18.3	18.4		
		199		219	20	20		
		219		239	20	19.5	(229.1' Gray Conglomertic Sandstone, Massive Bedded Few Vertical Seams)	

Project Catawissa Creek Tunnel #1 Hole No. 3 Sheet 4 Of 5

Elevation	Depth	Casing Blows From	Sample No.	Sample Blows To	Graphic Log Run	Core Recovery Rec	Description of Material	Remarks
		239		249	10	10.3		
	250	249		264	15	14.3		
							4.0' Gray Very Hard Shale Mixed with Conglomerate	Massive Bedded
	254	264		269	5.0	4.1		
		269		288.8	19.8	20.5		
							115.0' Gray Very Hard Shale	Massive Bedded
		288.8		309	20.2	19.9		
		309		319	10	10		
		319		338.5	19.5	19.3		
		338.5		358.7	20.2	20.4		
	369	358.7		369	10.3	10.6		

LOG OF TEST BORING

Project Catawissa Creek Tunnel #1 Hole No. 3 Sheet 5 Of 5

Elevation	Depth	Casing Blows / From	Sample No.	Sample Blows / To	Graphic Log / Run	Core Recovery / Rec	Description of Material	Remarks
		369		385	16.0	15.9	44.0' Gray Hard Fine Grain Sandstone	Massive Bedded
		385		399	14.0	14.2		
	413.0	399		419	20	20.4		
		419		439	20	20.4		
		439		459	20	20	67.0' Gray Very Hard Conglomertic Sandstone	Massive Bedded
		459		469	10	10		
	480							
	484.2	469		491.6	22.6	15.2	4.2' Gray Very Hard Shale	
	491.6						7.4' Open tunnel	
							Note: 22.7' of Casing and 4" Diamond Reaming Bit Left in Hole Per Instructions from Engineer	

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 1 River ______ Hole No. 3 Rig No. ______

Location of hole ~~East Union Township, Schuylkill County, Pennsylvania~~

Contractor Pa. Drilling Co: Driller Jack Johnson :Elev. top of hole 1596.00

Type & No. of Pump John Bean (435) Serial No. 125204 :No. of Meter Rockwell 19915723 :Elev. top of rock 1572.50

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
24.5	46.9	1571.5	1549.1	47	10:26	10:31	5	41.0	57.6	16.6	3.32
47.5	66.9	1548.5	1529.1	67	10:14	10:19	5	138.5	157.6	19.1	3.82
66.9	119.0	1529.1	1477.0	117	2:15	2:20	5	162.3	165.9	3.6	0.72
119.0	169.0	1477.0	1427.0	170	11:21	11:26	5	161.1	<161.2	<0.1	0.02
169.0	219.0	1427.0	1377.0	220	10:52	10:57	5	162.4	162.4	-	-
219.0	269.0	1377.0	1327.0	270	11:32	11:37	5	161.3	161.5	0.2	0.04
269.0	319.0	1327.0	1277.0	300	1:35	1:40	5	161.9	161.9	-	-
319.0	369.0	1277.0	1227.0	300	11:32	11:37	5	162.1	162.1	-	-
369.0	419.0	1227.0	1177.0	300	2:58	3:03	5	162.3	162.4	0.1	0.02

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									Remarks
Sec. of hole tested				Gage pressure at test intervals from					
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
24.5	46.9	1571.5	1549.1	Dropped 47 psi in 5 sec					2-5-71
47.5	66.9	1548.5	1529.1	Dropped 67 psi in 32 sec					2-9-71
66.9	119.0	1529.1	1477.0	Dropped 117 psi in 1 min					2-11-71
119.0	169.0	1477.0	1427.0	Pressure held at 45 psi					2-21-71
169.0	219.0	1427.0	1377.0	Pressure held at 75 psi					2-23-71
219.0	269.0	1377.0	1327.0	Pressure held at 140 psi					2-24-71
269.0	319.0	1327.0	1277.0	Pressure held at 100 psi					2-25-71
319.0	369.0	1277.0	1227.0	Pressure held at 275 psi					2-26-71
369.0	419.0	1227.0	1177.0	Pressure held at 210 psi					3-1-71

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 1 River ______ Hole No. 3 Rig No. ______

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Pa. Drilling Co: Driller Jack Johnson :Elev. top of hole 1596.00

Type & No. of Pump John Bean (435) :No. of Meter Rockwell Serial No. 19915723 :Elev. top of rock 1572.50
125204

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press.	Time	Time	Time	Water Meter Readings			
Depth		Elevation		Gage							
From	To	From	To	lbs/	start-ed	stop-ped	min.	At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
419	469	1177	1127	300	3:15	3:20	5	163.1	163.1	-	-

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									
Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
419	469	1177	1127	Pressure maintained at 200 psi					3-2-71

Description of operations and general information:

BY G.D.S.	DATE 2-5-71	SUBJECT Grouting	SHEET NO. 1	OF 1	
CHKD. BY	DATE 2-11-71		JOB. NO. 5081		
	Tunnel No. 1 - Hole No. 3		135-4	Tunnel No. 1	

LOG OF GROUTING

Project SL-135-4 Hole No. 3 Sheet No. 1 Of 1

Date	Depth (Interval Grouted)	Reason for Grouting (Loss or gain of water, caving hole, or other)	Material (Portland Cement or other)	Mix (W/C)	Method (Describe use of packer or other)	Pressure (If any)
2-5-71	24.5-46.9	Loss of water	Portland (2 bags)	1/1	Packer seated at 24.5' into hole. Grout was pumped into rods resting on bottom of hole	0
2-9-71	47.5-66.9	Loss of water	Two bags of Portland cement	1/1	Packer seated at 47.5'. Same method as above	0
2-11-71	66.9-116.9	25% reduction of drill water return	Two bags of Portland cement	1/1	Packer seated at 67.5'. Same method previously described	0

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S .AGUE & HENWOOD, Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 1

Sheet #1 of 4 **FOUNDATION TESTING and SOIL SAMPLING RECORD**

Dept. of Mines & Mineral Industries LOCATION: Shepton, Pa.

SURFACE ELEVATION 1761.0 RIG NO. ___ DATE: From 9/25 To 12/12 19 70

BORING LOG		SPOON SAMPLE AND CORE DATA				D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE
DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	CORE RECOV'D — NO. PCS. REMARKS *
0'0"– 13'0"	Brownish gray fine to coarse sand & gravel & boulders		Core runs			
		1	13'0"– 15'0"		2'	Badly broken medium hard
13'0"– 35'0"	Gray sandstone (coarse grain)	2	15'0"– 26'0"		5'6"	Badly broken medium hard
		3	26'0"– 35'0"		6'6"	Badly broken medium hard
35'0"– 43'0"	Brown sandstone (coarse grain)	4	35'0"– 42'0"		3'2"	Broken, very hard
		5	42'0"– 50'0"		6'2"	Broken, very hard
43'0"– 84'0"	Conglomerate	6	50'0"– 58'0"		8'	Broken, very hard
		7	58'0"– 69'0"		10'	Partly broken very hard
~~81'7"~~ 84'0"– 90'0"	Gray sandstone (fine grain) with pea conglomerate	8	69'0"– 70'0"		1'	Partly broken, very hard
		9	70'0"– 80'0"		10'	Partly broken very hard
90'0"– 111'6"	Conglomerate	10	80'0"– 90'0"		10'	Partly broken very hard
		11	90'0"– 100'0"		10'	Partly broken very hard
111'6"– 148'6"	Gray sandstone (fine grain)	12	100'0"– 110'0"		10'	Partly broken very hard
		13	110'0"– 120'0"		10'	Partly broken very hard
148'6"– 150'0"	Sand slate	14	120'0"– 130'0"		10'	Solid, medium hard
		15	130'0"– 140'0"		10'	Solid, Medium hard
150'0"– 180'6"	Gray sandstone (fine grain)	16	140'0"– 150'0"		10'	Solid, medium hard
		17	150'0"– 156'0"		6'	Solid, medium hard
180'6"– 209'6"	Gray sandy shale	18	156'0"– 161'0"		5'	Solid, medium hard
		19	161'0"– 171'0"		10'	Solid, medium hard

BLOWS ON CASING

0-1	4"	51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12	NX	62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

Blows on pipe and casing not required

Samples of soil not required

GROUND WATER

DEPTH	HOUR	DATE
Lost drill water @ 30'		
#2		12/12/70

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON
NX	52'	broke off while bumping it out of hole
also casing shoe bit		
#54147		

DISTANCE HAMMER DROP -- INCH
DRIVE HAMMER -- LBS.
SPOON HAMMER -- LBS.
CASING SIZE 4, NX INCH
SPOON SIZE -- INCH
SIZE OF CORE BIT NX INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Edward Tomko
Helper Michael Cvejkus
Helper ___

S. AGUE & HENWOOD, Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 1

Sheet #2 of 4 FOUNDATION TESTING and SOIL SAMPLING RECORD

Pa. Dept. of Mines & Mineral Industries LOCATION: Shepton, Pa.

SURFACE ELEVATION 1751.0 RIG NO. ____ DATE: From 9/25 To 12/12 19 70

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
209'6"-231'0"	Gray sandstone (fine grain)	20	171'0"-181'0"		10'	Solid, medium hard
		21	181'0"-191'0"		10'	Solid, medium hard
231'0"-251'0"	Gray sandstone (fine grain) with pebbles of conglomerate	22	191'0"-201'0"		10'	Solid, medium hard
		23	201'0"-211'0"		10'	Solid, medium hard
251'0"-262'6"	Conglomerate	24	211'0"-221'0"		10'	Solid, medium hard
		25	221'0"-231'0"		10'	Solid, medium hard
262'6"-272'6"	Gray sandy shale	26	231'0"-241'0"		10'	Solid, hard
		27	241'0"-251'0"		10'	Solid, hard
272'6"-301'0"	Gray sandstone (fine grain) with some pebbles of conglomerate	28	251'0"-261'0"		10'	Solid, hard
		29	261'0"-271'0"		10'	Solid, medium hard
301'0"-327'6"	Conglomerate	30	271'0"-281'0"		10'	Solid, medium hard
		31	281'0"-291'0"		10'	Solid, medium hard
327'6"-336'6"	Gray sandy shale	32	291'0"-301'0"		10'	Solid, very hard
		33	301'0"-306'0"		5'	Solid, very hard
336'6"-354'6"	Conglomerate	34	306'0"-312'0"		6'	Solid, very hard
		35	312'0"-322'0"		10'	Solid, very hard
354'6"-356'0"	Gray sandy shale	36	322'0"-332'0"		10'	Solid, Hard
		37	332'0"-337'0"		5'	Solid, hard
356'0"-363'0"	Red sandy shale with streaks of gray shale	38	337'0"-343'0"		6'	Solid, very hard
		39	343'0"-353'0"		10	Solid, very hard

BLOWS ON CASING

0-1		51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER

DEPTH	HOUR	DATE

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE ______ INCH
SPOON SIZE ______ INCH
SIZE OF CORE BIT ______ INCH

NOTE: "Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

* Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Edward Tomko
Helper Michael Cvejkus
Helper ______

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S .AGUE & HENWOOD, Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 1

Sheet #3 of 4

FOUNDATION TESTING and SOIL SAMPLING RECORD

Pa. Dept. of Mines & Mineral Ind. LOCATION: Shenton, Pa.

SURFACE ELEVATION 1761.0 RIG NO. ____ DATE: From 9/25 To 12/12 19 70

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
363'0"-392'0"	Gray sandstone (fine grain)	40	353'0"-363'0"		10'	Solid, hard
		41	363'0"-380'0"		14'3"	Solid hard
392'0"-410'0"	Red sandy shale	42	380'0"-400'0"		20'	Solid, hard
		43	400'0"-410'0"		48"	Partly broken medium hard
410'0"-419'0"	Grayish red sandy shale	44	410'0"-419'0"		9'	Medium hard, broken
		45	419'0"-429'0"		10'	Medium hard, broken
419'0"-428'6"	Gray sandy shale	46	429'0"-447'0"		18'	Partly broken medium hard
		47	447'0"-466'0"		19'	Partly broken medium hard
~~428'6"-44? ?"~~ 443'0"	Red sandy shale	48	466'0"-475'0"		8'	Seamy broken medium hard
		49	475'0"-495'0"		20'	Very hard, partly broken
443'0"-467'0"	Light gray sandy shale	50	495'0"-515'0"		20'	Solid, very hard
		51	515'0"-535'0"		20'	Solid, very hard
467'0"-472'6"	Dark gray sandy shale	52	535'0"-555'0"		20'	Solid, very hard
		53	555'0"-575'0"		20'	Solid, very hard
472'6"-475'0"	Light gray sandstone coarse grain, with pebbles of conglomerate	54	575'0"-595'0"		20'	Solid, very hard
		55	595'0"-615'0"		20'	Solid, medium hard
475'0"-590'0"	Green conglomerate					
590'0"-610'0"	Red sandy shale					

BLOWS ON CASING

0-1		51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER

DEPTH	HOUR	DATE

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE ______ INCH
SPOON SIZE ______ INCH
SIZE OF CORE BIT ______ INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Edward Tomko

Helper Michael Cvejkus

S. .AGUE & HENWOOD, Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 1

Sheet #4 of 4 **FOUNDATION TESTING and SOIL SAMPLING RECORD**

Pa. Dept. of Mines & Mineral Ind. LOCATION: Shepton, Pa.

SURFACE ELEVATION 1761.0 RIG NO. ____ DATE: From 9/25 To 12/12 19 70

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM–TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM–TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
610.0'– 695'0"	Grey sandstone fine grain	56	615'0"– 635'0"		20'	Solid, medium hard
			635'0"– 655'0"		20'	Solid, medium hard
			655'0"– 675'0"		20'	Solid, medium hard
			675'0"– 695'0"		20'	Solid, medium hard
			Bottom of hole 695'0"			
			44 NX wood core boxes			

BLOWS ON CASING

0-1	51-52
1-2	52-53
2-3	53-54
3-4	54-55
4-5	55-56
5-6	56-57
6-7	57-58
7-8	58-59
8-9	59-60
9-10	60-61
10-11	61-62
11-12	62-63
12-13	63-64
13-14	64-65
14-15	65-66
15-16	66-67
16-17	67-68
17-18	68-69
18-19	69-70
19-20	70-71
20-21	71-72
21-22	72-73
22-23	73-74
23-24	74-75
24-25	75-76
25-26	76-77
26-27	77-78
27-28	78-79
28-29	79-80
29-30	80-81
30-31	81-82
31-32	82-83
32-33	83-84
33-34	84-85
34-35	85-86
35-36	86-87
36-37	87-88
37-38	88-89
38-39	89-90
39-40	90-91
40-41	91-92
41-42	92-93
42-43	93-94
43-44	94-95
44-45	95-96
45-46	96-97
46-47	97-98
47-48	98-99
48-49	99-100
49-50	100-101
50-51	101-102

GROUND WATER

DEPTH	HOUR	DATE

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE ______ INCH
SPOON SIZE ______ INCH
SIZE OF CORE BIT ______ INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

* Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions etc at depth encountered.

Driller Edward Tomko
Helper Michael Cvejkus
Helper ______

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 2 River ______ Hole No. 1 Rig No. 1

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole 1761

Type & No. of Pump S&H6392 My-3-14-S&H #3+4 :No. of Meter Rockwell 19589953 :Elev. top of rock ______

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
73	120			53	11:35	11:40	5	1261.3	1263	1.7cf	0.34cf
117.5	171			105	10:44	10:49	5	1264.6	1264.6	0	0
169.5	221			170	9:48	9:53	5	1268.5	1271.2	2.7cf	0.54cf

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									Remarks
Sec. of hole tested				Gage pressure at test intervals from					
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
73	120			Dropped 53 psi in 15 sec					9-30-70
117.5	171			Dropped 105 psi in 8 sec					10-1-70
169.5	221			Dropped 170 psi in 85 sec					10-2-70

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 2 River ______ Hole No. 1 Rig No. 1

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole 1761

Type & No. of Pump S&H6642 Moyno-4-4 :No. of Meter Rockwell 19589953 :Elev. top of rock ______

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time start-ed	Time stop-ped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
217	271			220	1:42	1:57	5	1278.7	1279.7	1.0cf	.20cf
270	322			270	7:56	8:01	5	1289.7	1292.4	2.7cf	0.54cf
318.5	363			0	10:41	10:46	5	1300.0	1309.5	9.5cf	1.9cf
350	363			300	10:55	11:00	5	1314.0	1318.1	4.1cf	.82
339	363			240	11:10	11:15	5	1327.6	1335.8	8.2cf	1.64cf
330	363			0	11:25	11:30	5	1341.6	1356.1	14.5cf	2.9cf
319	363			0	11:48	11:54	5	1359.7	1373.0	13.3cf	2.66cf
316.5	343			160	11:41	11:46	5	1474.5	1479.2	4.7cf	.94cf
361.5	410			0	8.56	9:01	5	1491.2	1600.0	8.8cf	1.76cf

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
217	271			Dropped 220 psi in 20 sec					10-5-70
270	322			Dropped 270 psi in 2 sec					10-7-70
318.5	363			No pressure					10-8-70
350	363			Dropped 300 psi in 7 sec					10-8-70
339	363			Dropped 240 psi in 6 sec					10-8-70
330	363			No pressure					10-8-70
319	363			No pressure					10-8-70
316.5	343			Dropped 160 psi in 15 sec					10-12-70
361.5	410			Loss of water - no pressure					10-14-70

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 2 River ______ Hole No. 1 Rig No. 1

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole 1761

Type & No. of Pumps S&H6287 Moyno #MN-2-6 :No. of Meter Rockwell 19589953 :Elev. top of rock ______

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal. or cu.ft. per min
From	To	From	To								
405.5	429			200	11:22	11:27	5	1565.0	1589.6	24.6cf	4.92cf
428.5	475			0	11:20	11:25	6	1597.5	1631.5	34.0cf	6.80cf
475	495			300	8:55	9:00	5	1621.0	1634.0	13.0cf	2.60cf
493.5	515			300	8:10	8:15	5	1645.0	1656.6	11.6cf	2.32cf

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									Remarks
Sec. of hole tested				Gage pressure at test intervals from					
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
405.5	429			Dropped	200 psi	in 10	sec		10-19-70
428.5	475			Loss of	water	- no pressure			10-21-70
475	495			Dropped	300 psi	in 15	sec		11-18-70
493.5	515			Dropped	300 psi	in 5	sec		11-19-70

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 2 River ______ Hole No. 1 Rig No. 1

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole 1761

Type & No. of Pump S&H6287 Moyno 1-2-6 :No. of Meter Trident 5106823 :Elev. top of rock ______

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
515	535			300	10:00	10:05	5	0000.0	000.2	.2cf	.04cf
530	555			300	9:10	9:15	5	0000.2	0000.3	.1cf	.02cf
553	575			300	9:13	9:18	5	0000.3	0000.3	0	0
574	595			300	9:12	9:17	5	0000.3	0000.3	0	0
592	615			300	2:30	2:35	6	0000.3	0000.3	0	0
614.5	635			300	1:52	1:57	5	0000.3	0000.3	0	0
634	655			300	2:00	2:05	5	0000.3	0000.3	0	0
655	675			300	2:47	2:52	5	0000.3	0000.3	0	0
673	695			300	2:56	3:01	5	0000.3	0000.3	0	0

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
515	535			Dropped 300 psi in 1 sec					11-20-70
530	555			Dropped 300 psi in 5 sec					11-23-70
553	575			Dropped 300 psi in 3 sec					11-24-70
574	595			Pressure maintained					11-27-70
592	615			Pressure maintained					12-8-70
614.5	635			Pressure maintained					12-9-70
634	655			Pressure maintained					12-10-70
655	675			Pressure maintained					12-11-70
673	695			Pressure maintained					12-12-70

Description of operations and general information:

BY G.D.S. DATE ______ SUBJECT Grouting SHEET NO. 1 OF 3

CHKD.BY ______ DATE ______ ______ JOB. NO. ______

Tunnel No. 2 Hole No. 1

LOG OF GROUTING

Project Catawissa Creek Hole No. 38A Sheet No. 1 Of 3

Date	Depth (Interval Grouted)	Reason for Grouting (Loss or gain of water, caving hole, or other)	Material (Portland Cement or other)	Mix (W/C)	Method (Describe use of packer or other)	Pressure (If any)
10-9-70	322'-363'	Loss of water	Allentown Portland Cement (4 bags)	1/1	Grouting through drill rods from the bottom up	none
10-12-70	DRILLED OUT CEMENT FROM 341' - 363					
10-14-70	363'-410'	Loss of water	Allentown Portland Cement (4 bags)	1/1	Grouted through drill rods - from bottom up	none
10-15-70	DRILLED OUT CEMENT FROM 387' - 410'					
10-19-70	410'-429'	Loss of water	Allentown Portland Cement (4 bags)	1/1	Grouted through drill rods - from bottom up	none
10-20-70	DRILLED OUT CEMENT FROM 400' - 429'					
10-21-70	429'-475'	Loss of water	Allentown Portland Cement (4 bags)	1/1	Grouted through drill rods - from bottom up	none
10-23-70	473' Bottom up hole	Still losing all water being pumped in hole	Allentown Portland Cement (4 bags)	1/1	Grouted through drill rods - bottom up 473' up	none
10-24-70	473' Bottom up hole	Losing all water being pumped in hole	4 bags of Quick Gel, 4 bags of cement	12/1	Grouting through rods, bottom up 473' up	

BY G.D.S. DATE ______ SUBJECT Grouting SHEET NO. 2 OF 3

CHKD.BY ______ DATE ______ JOB. NO. ______

Tunnel No. 2 - Hole No. 1

LOG OF GROUTING

Project Catawissa Creek Hole No. 38A Sheet No. 2 Of 3

Date	Depth (Interval Grouted)	Reason for Grouting (Loss or gain of water, caving hole, or other)	Material (Portland Cement or other)	Mix (W/C)	Method (Describe use of packer or other)	Pressure (If any)
10-26-70	473' up	Loss of water	Allentown Portland Cement (4 bags)	1/1	Grouted through packer set at 154'	none
10-27-70	472' up	Loss of water	Allentown Portland Cement (10 bags)	1/1	Grouted through packer seated at 197'	none
10-29-70	DRILLED OUT CEMENT FROM 343' - 467' Lost the drill water again at 467'					
10-30-70	467' up	Loss of water	Allentown Portland Cement (3 bags)	1/1	Grouted through packer set at 197'	none
10-31-70	467' up	Loss of water	Portland Allentown Cement (5 bags)	1/1	Grouting through packer set at 198'	none
11/2/70	DRILLED OUT CEMENT FROM 72' - 470 - Lost the drill water again at 467.5'					
11/3/70	467.5' up	Loss of water	Allentown Portland, 3 bags of cement, 3 gallon saw-dust	1/1	Grouted through packer set at 462'	none
11/4/70	DRILLED OUT CEMENT MIXED WITH SAWDUST FROM 460' - 467' and lost water at 467'. We pumped in one bushel of sawdust in hole with no success					

BY G.D.S.	DATE	SUBJECT Grouting	SHEET NO. 3	OF 3
CHKD.BY	DATE		JOB. NO.	
	Tunnel No. 2 Hole No. 1			

LOG OF GROUTING

Project Catawissa Creek Hole No. 38A Sheet No. 3 Of 3

Date	Depth (Interval Grouted)	Reason for Grouting (Loss or gain of water, caving hole, or other)	Material (Portland Cement or other)	Mix (W/C)	Method (Describe use of packer or other)	Pressure (If any)
11-5-70	467' up	Loss of water	5 bags cement 5 gallon sawdust	1/1	Grouted through packer set at 465'	none
11-16-70	DRILLED OUT HARD CEMENT FROM 240' to 465'					
11-17-70	DRILLED OUT CEMENT FROM 465' - 475' Lost water again at 471.5'					

S. .AGUE & HENWOOD, Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 2

Sheet #1 of 3

FOUNDATION TESTING and SOIL SAMPLING RECORD

Gannett, Fleming, Corddry & Carpenter LOCATION: Sheppton, Pa.

SURFACE ELEVATION 1786.0 RIG NO. ____ DATE: From 4/2 To 4/29 1971

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM–TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM–TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH P=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
0.0'-20.0'	Boulders, brown clayey sand	1	20.0'-30.0'		10'	Broken, very hard
		2	30.0'-41.0'		11'	Broken, very hard
20.0'-106.5'	Conglomerate	3	41.0'-61.0'		20'	Partly broken very hard
		4	61.0'-72.0'		11'	Solid, very hard
106.5'-108.5'	Dark gray sandstone	5	72.0'-92.0'		20'	Solid, very hard
		6	92.0'-101.0'		9'	Solid, very hard
108.5'-185.0' 185.0'	Sandy conglomerate	7	101.0'-121.0'		20'	Partly broken very hard
		8	121.0'-141.0'		20'	Solid, very hard
185.0'-23?.0' 237.0'	Conglomerate	9	141.0'-151.0'		10'	Solid, very hard
		10	151.0'-161.0'		10'	Solid, very hard
237.0'-255.0'	Dark gray sandstone with pebble conglomerate	11	161.0'-171.0'		10'	Solid, very hard
		12	171.0'-191.0'		20'	Solid, very hard
255.0'-317.0'	Gray sandstone (fine grain)	13	191.0'-201.0'		10'	Solid, very hard
		14	201.0'-217.0'		16'	Partly broken very hard
		15	217.0'-237.0'		20'	Solid, very hard
		16	237.0'-257.0'		20'	Solid, very hard
		17	257.0'-267.0'		10'	Solid, hard
		18	267.0'-287.0'		20'	Solid, hard
		19	287.0'-297.0'		10'	Solid, hard
		20	297.0'-317.0'		20'	Solid, hard

BLOWS ON CASING

0-1	4"	51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6	NX	56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER

DEPTH	HOUR	DATE
tunnel	opening	

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON
NX	22'	by order
4"	5'	of engineer
NX Casing bit #54148		

DISTANCE HAMMER DROP ____ INCH
DRIVE HAMMER ____ LBS.
SPOON HAMMER ____ LBS.
CASING SIZE 4, NX INCH
SPOON SIZE ____ INCH
SIZE OF CORE BIT NX, NS INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

* Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Edward Tomko

Helper Michael Cvejkus

Helper ____

S[illegible] & [illegible]WOOD, Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 2

Sheet #2 of 3 **FOUNDATION TESTING and SOIL SAMPLING RECORD**

Gannett, Fleming, COrddry & Carpenter LOCATION: Sheppton, Pa.

SURFACE ELEVATION 1786.0 RIG NO. ____ DATE: From 4/2 To 4/29 19 71

BORING LOG: DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE; CORE RECOV'D — NO. PCS. REMARKS *
317.0'-377.0'	Gray sandstone (Fine grain)	21	317.0'-337.0'		20'	Solid, hard
		22	337.0'-357.0'		20'	Solid, hard
377.0'-391.0'	Conglomerated sandstone	23	357.0'-377.0'		20'	Solid, hard
		24	377.0'-395.0'		18'	Partly broken very hard
391.0'-394.0'	Dark gray sandy shale	25	395.0'-414.0'		19'	Solid, very hard
		26	414.0'-434.0'		20'	Solid, very hard
394.0'-414.0'	Sandy conglomerate	27	434.0'-454.0'		20'	Solid, very hard
414.0'		28	454.0'-464.0'		10'	Solid, very hard
414.0'-424.0' 424.0'	Conglomerate	29	464.0'-474.0'		10'	Solid, very hard
		30	474.0'-494.0'		20'	Solid, hard
424.0'-476.0'	Light gray conglomerated sandstone (Fine grain)	31	494.0'-514.0'		20'	Solid, hard
476.0'-485.0'	Grayish red sandstone	32	514.0'-534.0'		20'	Solid, hard
485.0'-502.0'	Gray sandstone with pebble conglomerate					
502.0'-511.0'	Red sandy shale					
511.0'-533.0'	Conglomerated sandstone					

BLOWS ON CASING	
0-1	51-52
1-2	52-53
2-3	53-54
3-4	54-55
4-5	55-56
5-6	56-57
6-7	57-58
7-8	58-59
8-9	59-60
9-10	60-61
10-11	61-62
11-12	62-63
12-13	63-64
13-14	64-65
14-15	65-66
15-16	66-67
16-17	67-68
17-18	68-69
18-19	69-70
19-20	70-71
20-21	71-72
21-22	72-73
22-23	73-74
23-24	74-75
24-25	75-76
25-26	76-77
26-27	77-78
27-28	78-79
28-29	79-80
29-30	80-81
30-31	81-82
31-32	82-83
32-33	83-84
33-34	84-85
34-35	85-86
35-36	86-87
36-37	87-88
37-38	88-89
38-39	89-90
39-40	90-91
40-41	91-92
41-42	92-93
42-43	93-94
43-44	94-95
44-45	95-96
45-46	96-97
46-47	97-98
47-48	98-99
48-49	99-100
49-50	100-101
50-51	101-102

GROUND WATER: DEPTH	HOUR	DATE

PIPE AND CASING LEFT IN HOLE: SIZE	AMOUNT	REASON

DISTANCE HAMMER DROP ____ INCH
DRIVE HAMMER ____ LBS.
SPOON HAMMER ____ LBS.
CASING SIZE ____ INCH
SPOON SIZE ____ INCH
SIZE OF CORE BIT ____ INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.
Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Ed Tomko
Helper Michael Cvejkus
Helper ____

S. [illegible] & [illegible], Inc.
SCRANTON, PA.

Tunnel No. 2
Hole No. 2

Sheet #3 of 3 FOUNDATION TESTING and SOIL SAMPLING RECORD

[illegible]nnett Fleming Corddry & Carpenter LOCATION: Sheppton, Pa.

SURFACE ELEVATION 1786.0 RIG NO. ____ DATE: From 4/2 To 4/29 19 71

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
533.0'- 535.0'	Gray sandy shale	33	534.0'- 554.0'		20'	Solid, medium hard
		34	554.0'- 564.0'		10'	Solid, hard
535.0'- 560.0'	Red sandy shale with streaks of gray sandy shale	35	564.0'- 574.0'		10'	Solid, hard
		36	574.0'- 594.0'		20'	Solid, very hard
560.0'- 570.0'	Gray sandstone with pebble conglomerate	37	594.0'- 596.0'		2'	Solid, very hard
		38	596.0'- 602.0'			Tunnel
570.0'- 596.0' 596.0'	Conglomerate		Samples of soil not required			
59[illegible]0'- 60[illegible].0' 602.0'	Opening, Tunnel					
			Gary D. Schaffer - Inspector			
			Hole ended at 596'			

BLOWS ON CASING

0-1		51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER

DEPTH	HOUR	DATE

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE ______ INCH
SPOON SIZE ______ INCH
SIZE OF CORE BIT ______ INCH

NOTE: *Classification of [redacted] made by the driller and has not been checked by a soil[redacted]. Classification of rock has been made by the driller and has [redacted] checked by a geologist.

* Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Edward Tomko
Helper Michael Cvejkus
Helper ______

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 2 River ____ Hole No. 2 Rig No. 1

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole 1786.0

Type & No. of Pump S&H6287 Moynol 1-2-6 No. of Meter Trident 5106823 :Elev. top of rock ____

Elev. W.S. before test ____

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time start-ed	Time stop-ped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
24.5	72			70	10:30	10:35	5	2.5	2.5	0.0	0.00
70	121			121	9:00	9:05	5	2.8	2.8	0.0	0.00
121	171			171	2:45	2:50	5	2.8	2.8	0.0	0.00
171	217			217	3:15	3:20	5	3.0	3.5	0.5	0.01
217	267			267	3:10	3:15	5	3.6	3.6	0.0	0.00
267	317			300	10:00	10:05	5	3.7	3.7	0.0	0.00
317	367			300	11:15	11:20	5	3.7	3.7	0.0	0.00
367	414			300	9:00	9:05	5	3.7	3.7	0.0	0.00
414	464			300	1:15	1:20	5	3.7	3.7	0.0	0.00

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									
Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
24.5	72			Pressure held at 50 psi					4-8-71
70	121			Pressure held at 90 psi					4-10-71
121	171			Pressure held at 142 psi					4-12-71
171	217			Pressure held at 190 psi					4-13-71
217	267			Pressure held at 65 psi					4-14-71
267	317			Pressure held at 125 psi					4-19-71
317	367			Pressure held at 95 psi					4-20-71
367	414			Pressure held at 105 psi					4-22-71
414	464			Pressure held at 75 psi					4-23-71

Description of operations and general information:

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 2 River ______ Hole No. 2 Rig No. 1

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole 1786.0

Type & No. of Pump S&H6287 Moynol S&H2-6 :No. of Meter Trident 5106823 :Elev. top of rock ______

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
464	514			300	12:20	12:25	5	3.7	3.7	0.0	0.00
514	564			300	4:00	4:05	5	3.7	3.7	0.0	0.00

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									
Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
464	514			Pressure held at 125 psi					4-26-71
514	564			Pressure held at 115 psi					4-27-71

Description of operations and general information:

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SP. ‹GUE & HENWOOD, Inc.
SCRANTON, PA.

Tunnel No.3

Sheet #1 of 2

FOUNDATION TESTING and SOIL SAMPLING RECORD

70° hole

Gannett Fleming Corddry & Carpenter LOCATION: Sheppton, Pa.

SURFACE ELEVATION -- RIG NO. ____ DATE: From 7/27 To 8/18 19 71

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM–TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM–TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
0'0"–16'0"	Brown fine to coarse sand & gravel, cobbles & boulders		Core Runs			
		1	16'0"–25'0"		7'0"	Badly broken, medium hard
16'0"–43'0"	Conglomerated sandstone	2	25'0"–31'0"		5'6"	Broken, hard
		3	31'0"–37'0"		6'0"	Broken, hard
43'0"–76'6"	Conglomerate	4	37'0"–44'0"		7'0"	Partly broken, hard
		5	44'0"–53'0"		9'0"	to 73'
76'6"–77'0"	Slate	6	53'0"–62'0"		9'0"	
		7	62'0"–64'0"		2'0"	
77'0"–181'0"	Conglomerate	8	64'0"–73'0"		9'0"	
		9	73'0"–81'0"		8'0"	Broken, very hard
		10	81'0"–90'0"		9'0"	to 132'
		11	90'0"–100'0"		10'0"	
		12	100'0"–109'0"		9'0"	
		13	109'0"–112'0"		3'0"	
		14	112'0"–117'0"		5'0"	
		15	117'0"–119'0"		2'0"	
		16	119'0"–123'0"		4'0"	
		17	123'0"–126'6"		3'6"	
		18	126'6"–132'0"		5'6"	
		19	132'0"–142'0"		10'0"	Solid, very hard

BLOWS ON CASING (NX Casing)

0-1	51-52
1-2	52-53
2-3	53-54
3-4	54-55
4-5	55-56
5-6	56-57
6-7	57-58
7-8	58-59
8-9	59-60
9-10	60-61
10-11	61-62
11-12	62-63
12-13	63-64
13-14	64-65
14-15	65-66
15-16	66-67
16-17	67-68
17-18	68-69
18-19	69-70
19-20	70-71
20-21	71-72
21-22	72-73
22-23	73-74
23-24	74-75
24-25	75-76
25-26	76-77
26-27	77-78
27-28	78-79
28-29	79-80
29-30	80-81
30-31	81-82
31-32	82-83
32-33	83-84
33-34	84-85
34-35	85-86
35-36	86-87
36-37	87-88
37-38	88-89
38-39	89-90
39-40	90-91
40-41	91-92
41-42	92-93
42-43	93-94
43-44	94-95
44-45	95-96
45-46	96-97
46-47	97-98
47-48	98-99
48-49	99-100
49-50	100-101
50-51	101-102

GROUND WATER

DEPTH	HOUR	DATE
88'		8/18/71

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON
	none	

DISTANCE HAMMER DROP --- INCH
DRIVE HAMMER --- LBS.
SPOON HAMMER --- LBS.
CASING SIZE NX INCH.
SPOON SIZE -- INCH
SIZE OF CORE BIT NX INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Ed Tomko

Helper Ray Ford

Helper ____________

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SP GUE & HENWOOD, Inc.

SCRANTON, PA.

Tunnel No. 3

Sheet #2 of 2

FOUNDATION TESTING and SOIL SAMPLING RECORD

Note: 70° hole

Gannett Fleming Corddry & Carpenter LOCATION: Sheppton, Pa.

SURFACE ELEVATION ______ RIG NO. ______ DATE: From 7/27 To 8/18 19 71

BORING LOG: DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SPOON SAMPLE AND CORE DATA: SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE; CORE RECOV'D — NO. PCS. REMARKS *
		20	142'0"-152'0"		10'0"	Solid, very hard to 181'
		21	152'0"-162'0"		10'0"	
		22	162'0"-172'0"		10'0"	
		23	172'0"-181'0"		9'0"	

Bottom of hole 181'

Cemented hole with 9 bags cement

Used 30 gallons of Dramus (cutting oil)

12 NX wood core boxes

BLOWS ON CASING

0-1		51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER

DEPTH	HOUR	DATE

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE ______ INCH
SPOON SIZE ______ INCH
SIZE OF CORE BIT ______ INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

Under Remarks mention kind of Bit, loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground water conditions, etc., at depth encountered.

Driller Edward Tomko

Helper Ray Ford

Helper ______

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Tunnel No. 3 River ______ Hole No. 1 Rig No. 142 S&H

Location of hole East Union Township, Schuylkill County, Pennsylvania

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole --

Type & No. of Pump Myers 3+4 :No. of Meter Trident 17395076 :Elev. top of rock ______

Elev. W.S. before test ______

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time start-ed	Time stop-ped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
110.5	126.5			140	10:45	10:50	5	8055	8059	4	.80
90.5	126.5			140	11:15	11:20	5	8090	8127	37	7.40
55.0	126.5			140	11:40	11:45	5	8133	8182	49	9.90
23.0	126.5			140	12:05	12:10	5	8253	8326	73	14.60

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									Remarks
Sec. of hole tested				Gage pressure at test intervals from					
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
110.5	126.5			Dropped to 45 psi in 20 sec - Held at 45 psi					
90.0	126.5			Dropped 140 psi in 70 sec					
55.0	126.5			Dropped 140 psi in 115 sec					
23.0	126.5			Dropped 30 psi in 2 sec					

Description of operations and general information:

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SPRAGUE & HENWOOD, Inc.

SCRANTON, PA.

Gap east
of Sheppton

FOUNDATION TESTING and SOIL SAMPLING RECORD Site #1 40° hole

Gannett Fleming Corddry & Carpenter **LOCATION:** Sheppton, Pa.

SURFACE ELEVATION ______ **RIG NO.** ______ **DATE:** From 8/23 To 8/31 19 71

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D -- NO. PCS. REMARKS *
0'0"- 29'0"	Brown fine to coarse sand & gravel, cobbles & boulders		Core runs			
		1	29'0"- 38'0"		8'8"	Broken hard
29'0"- 47'0"	Sandy conglomerate	2	38'0"- 47'0"		9'0"	Broken, hard
		3	47'0"- 56'0"		9'0"	Broken, very hard
47'0"- 100'0"	Conglomerate	4	56'0"- 66'0"		10'	Broken, very hard
		5	66'0"- 76'0"		10'	Partly broken, very hard
		6	76'0"- 84'0"		8'0"	Partly broken very hard
		7	84'0"- 94'0"		10'	Solid, very hard
		8	94'0"- 100'0"		6'	Solid, very hard
			Six (6) NX Wood Core Boxes			
			Fifty (50) gallons Dramus (cutting oil)			
			Five (5) bags Cement			

BLOWS ON CASING

0-1		51-52	
1-2	NX to 29'	52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER			PIPE AND CASING LEFT IN HOLE		
DEPTH	HOUR	DATE	SIZE	AMOUNT	REASON
42'		8/30/71		none	

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE NX INCH
SPOON SIZE ~~XX~~ INCH
SIZE OF CORE BIT NX INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

* Under Remarks mention kind of Bit. loss of sample, loss of Drilling water, soft seamy or broken Rock, Caving, Cavities, unusual Ground [illegible]

Driller Edward Tomko

Helper R. Ford

Helper

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Gap east of Sheppton River ____________ Hole No. - Rig No. 1

Location of hole East Union Township, Site #1 - 40° hole - Schuylkill Co.,Pa.

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole -

Type & No. of Pump Myers 3+4 :No. of Meter Trident 17395076 :Elev. top of rock ____________

Elev. W.S. before test ____________

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time started	Time stopped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
75	100			100	9:50	9:55	5	8333	8336	3	0.6
50	100			100	10:10	10:15	5	8342	8358	16	3.2
30	100			60	10:35	10:40	5	8410	8482	72	14.4

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									
Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
75	100			Dropped 100 psi in 30 sec					
50	100			Dropped 100 psi in 15 sec					
30	100			Dropped 60 psi in 20 sec					

Description of operations and general information:

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Sprague & Henwood, Inc.

SCRANTON, PA.

FOUNDATION TESTING and SOIL SAMPLING RECORD

Gap west of Oneida

Gannett Fleming Corddry & Carpenter LOCATION: Sheppton, Pa.

SURFACE ELEVATION ______ RIG NO. ______ DATE: From 9/8 To 9/15 19 71

BORING LOG		SPOON SAMPLE AND CORE DATA				
DEPTH FROM—TO	DESCRIPTION OF MATERIAL Based On Samples Recovered Plus Observation Of Material Returned Between Samples	SAMPLE NUMBER	DEPTH FROM—TO	BLOWS PER FT. ON SAMPLES	ROCK CORE RECOV'D	D=DRY U=UNDISTURBED T=TRAP W=WASH R=ROD C=CORE CORE RECOV'D — NO. PCS. REMARKS *
0'0"-24'0"	Brown sand, gravel & boulders		Core runs			
		1	24'0"-30'0"		3'	Broken seamy, hard
24'0"-37'0"	Light gray sandy conglomerate	2	30'0"-39'0"		9'	Broken, seamy, hard
		3	39'0"-49'0"		10'	Broken, very hard
37'0"-96'0"	Light gray conglomerate	4	49'0"-53'0"		4'0"	Broken, very hard
		5	53'0"-63'0"		10'	Broken, very hard
		6	63'0"-66'0"		3'	Broken, very hard
		7	66'0"-76'0"		9'	Solid, very hard
		8	76'0"-86'0"		10'	Solid, very hard
		9	86'0" 96'0"		10'	Solid, very hard
			Bottom of hole 96'			
			(5) NX Wood Core Boxes			
			(60) gallons Dramus (cutting oil)			
			(5) bags cement			

BLOWS ON CASING

NX Cased to 24'

0-1		51-52	
1-2		52-53	
2-3		53-54	
3-4		54-55	
4-5		55-56	
5-6		56-57	
6-7		57-58	
7-8		58-59	
8-9		59-60	
9-10		60-61	
10-11		61-62	
11-12		62-63	
12-13		63-64	
13-14		64-65	
14-15		65-66	
15-16		66-67	
16-17		67-68	
17-18		68-69	
18-19		69-70	
19-20		70-71	
20-21		71-72	
21-22		72-73	
22-23		73-74	
23-24		74-75	
24-25		75-76	
25-26		76-77	
26-27		77-78	
27-28		78-79	
28-29		79-80	
29-30		80-81	
30-31		81-82	
31-32		82-83	
32-33		83-84	
33-34		84-85	
34-35		85-86	
35-36		86-87	
36-37		87-88	
37-38		88-89	
38-39		89-90	
39-40		90-91	
40-41		91-92	
41-42		92-93	
42-43		93-94	
43-44		94-95	
44-45		95-96	
45-46		96-97	
46-47		97-98	
47-48		98-99	
48-49		99-100	
49-50		100-101	
50-51		101-102	

GROUND WATER

DEPTH	HOUR	DATE
18'7"		9/15/71

PIPE AND CASING LEFT IN HOLE

SIZE	AMOUNT	REASON
	none	

DISTANCE HAMMER DROP ______ INCH
DRIVE HAMMER ______ LBS.
SPOON HAMMER ______ LBS.
CASING SIZE NX INCH
SPOON SIZE ______ INCH
SIZE OF CORE BIT NX INCH

NOTE: *Classification of soil has been made by the driller and has not been checked by a soils engineer. Classification of rock has been made by the driller and has not been checked by a geologist.

* Under Remarks mention kind of Bit, loss of sample, loss of Drilling Water, soft seamy or broken Rock, Caving, Cavities, unusual Ground Water conditions, etc., at depth encountered.

Driller Ed Tomko

Helper H. Jones

Helper ______

REPORT OF WATER PRESSURE TESTING IN CORE DRILL HOLES

Site Gap west of Oneida River ____________ Hole No. ________ Rig No. 1

Location of hole East Union Township, Schuylkill County, Site #2, Pa.

Contractor Sprague & Henwood :Driller Ed Tomko :Elev. top of hole -

Type & No. of Pump Myers 3+4 :No. of Meter Trident 17395076 :Elev. top of rock ________

Elev. W.S. before test ________

DATA ON FLOW TEST

PART I

Sec. of hole tested				Press. Gage lbs/	Time start-ed	Time stop-ped	Time min.	Water Meter Readings			
Depth		Elevation						At start of test	At end of test	Total gals/of water used	Gal.or cu.ft. per min
From	To	From	To								
54	96			100	9:25	9:30	5	8484	8490	6	1.2
25	96			80	10:10	10:15	5	8490	8560	70	14.0

HOLDING TEST - MAXIMUM PRESSURE

PART II

Data on Pressure									
Sec. of hole tested				Gage pressure at test intervals from					Remarks
Depth		Elevation		lb.	lb.	lb.	lb.	lb.	
From	To	From	To						
54	96			Pressure hold at 40 psi					
25	96			Pressure dropped 80 psi in 25 sec					

Description of operations and general information:

APPENDIX C

REPORT ON LABORATORY TESTS ON ROCK SAMPLES FROM CATAWISSA CREEK TUNNELS PROJECT

SUBMITTED
TO

GANNETT FLEMING CORDDRY AND CARPENTER, INC.

BY

GEOTECHNICAL ENGINEERS, INC.
934 MAIN STREET
WINCHESTER, MASSACHUSETTS 01890

AUGUST 31, 1971

INTRODUCTION

This report is a summary of the results of laboratory tests on 5 rock samples taken in connection with the Catawissa Creek Tunnels project. Most of these results were previously reported by telephone to Mr. Karim Habibagahi during the course of the testing.

The tests that were performed are:

- 2 unconfined compression tests, including determination of uniaxial modulus of deformation and Poisson's ratio
- 8 - triaxial tests on intact rock
- 2 direct shear tests on intact rock (In one of these tests, seven determinations of post-peak strength were made at various normal stresses; in the other, four determinations of post-peak strength were made.)
- 9 - direct shear tests on rock/concrete interfaces (In 6 of these tests, two determinations of post-peak strength were made, at different normal stresses.)
- 3 specific gravity determinations

This testing program was authorized verbally by Mr. Karim Habibagahi on July 21, 1971.

SAMPLES TESTED

Sample Descriptions

The samples that were tested were taken in the Pottsville Formation. They comprise:

2 pieces of NX core from Borehole 3, taken between depths 479.0 feet and 483.5 feet and consisting of a gray metaconglomerate in the upper part and gray siltstone in the lower part.

2 pieces of NX core from Borehole 36A, taken between depths 591 feet and 594 feet and consisting of greenish gray metaconglomerate.

1 chunk sample consisting of white to dark gray metaconglomerate. (The sampling location is not known.)

Table C-1 includes a petrologic description of each of the 5 samples. Our geologic classification of the rock types was made on the basis of visual inspection alone and without knowledge of the exact location and areal geology of the site from which the samples were taken. Therefore, the classification may differ slightly from that which has been made by geologists who have made detailed petrologic studies of these rocks or who are familiar with the site.

TABLE C-1. DESCRIPTION OF SAMPLES

Borehole No.	Sample No.	Type of Sample	Depth (feet)	Specimen*	Description
3	2	NX core 14" long	479.0-480.2	A,B	Mottled white, light gray, and dark gray meta-conglomerate. White portion consists of sub-rounded (with some sub-angular) quartz particles up to about 1.5 cm maximum size. Light gray groundmass is fine-grained, has a Moh hardness of about 5, and contains tiny biofite flakes that are visible under a hand lens. Dark gray material occurs in very minor amounts as stringers or irregularly shaped inclusions, with dimensions ranging from a few millimeters to a centimeter or more, in the light gray groundmass, and has a Moh hardness of about 4.
				C	Sharply defined contact plane at about 40° to core axis. Rock above contact sample as that in Specimens A and B; rock below contact same as that in Specimens D and E.
				D,E	Same as Sample 1 from Borehole 3. (See description below).
3	1	NX core 30" long	481.0-483.5	A,B,C,D,E, F,and G	Dark gray, fine grained siltstone, with faint, darker-gray lineations at about 35° to core axis. Ends of sample consist of rough but approximately plane fracture surfaces at about 35° to core axis. Moh hardness = 4-5.

(Continued)

TABLE C-1. (Continued)

Borehole No.	Sample No.	Type of Sample	Depth (feet)	Specimen*	Description
36A	2	NX core 12" long	59-592	A,B,C,D, and E	Mottled white, light greenish gray, and dark greenish gray, fine to medium grained metamorphic rock. White portion consists of irregularly shaped quartz masses ranging in size from a millimeter or less up to a centimeter or more. Light and dark greenish gray portions are fine to medium grained and have a Moh hardness of 5-6.
36A	1	NX core 24" long	592-594	A,B,C,D, E, and F	Same as Sample 2 from Borehole 36A (See description above).
--	--	Chunk	?	A,B,C,D, and E	White to dark gray metaconglomerate, consisting predominantly of angular to sub-angular quartz grains, ranging from a few millimeters to about 1 cm in size, embedded in a black, fine to medium grained groundmass that has a Moh hardness of about 6. Well developed slickensides on the face of chunk marked with yellow paint, with some graphite on slickenside surfaces. Some layering adjacent to the slickenside surface, but not elsewhere. Some cracks approximately perpendicular to the slickensided face.

*See Figure C-1 for locations from which individual test specimens (A,B, etc.) were cut from each sample.

Fig. C-1 shows the location within each sample of the specimens that were prepared for the laboratory tests.

Evidence of Anisotropy

The siltstone from Borehole 3, which comprises the bottom part of Sample 2 and all of Sample 1, appears to be anisotropic, as indicated by the fact that the two fracture surfaces in the siltstone at the ends of Sample 1 are parallel and inclined at about 35° to the core axis, and the one fracture surface in the silstone at the lower end of Sample 2 is inclined at about 35° to the core axis and is roughly parallel to the contact between the siltstone and the metaconglomerate that comprises the top half of Sample 2. The contact is inclined at about 40° to the core axis. In addition to the orientation of the fracture surfaces at the ends of the samples, there are some faint color lineations inclined at about 35° to the core axis in Sample 1.

There is no apparent indication of anisotropy in the metaconglomerate that comprises the top half of Sample 2, Borehole 3, or in the metaconglomerate that comprises both samples from Borehole 36A.

The chunk sample had well developed slickensides and some graphite on the face that was marked with yellow paint, and there was a second well developed lamination less than one inch beneath, and parallel to, the slickensided surface. There were also a number of cracks in the chunk roughly perpendicular to the slickensided surface. The portion of the chunk from which the samples were taken showed slight evidence of layering, as indicated by differences in the sizes of the quartz grains that comprise a significant part of the chunk sample.

Although we did not perform any laboratory tests to measure the effects of anisotropy, it is our opinion that anisotropy was not significant for the specimens we tested. The presence of the slickensided surface and the graphite on one face of the chunk sample indicates that there may be very significant anisotropy of that rock in situ. Also, joints, shear zones, or chemically altered zones may have a significant effect on the properties of the rock mass in situ, and are not taken into account by the results of the laboratory tests on the intact rock specimens.

UNIT WEIGHTS

Table C-2 gives the specific gravities of 3 samples determined in accordance with ASTM Designation C127-68.

The bulk specific gravity of the siltstone from Borehole 3 is 2.72, a typical value for this type of rock.

The bulk specific gravities of the greenish gray metaconglomerate from Borehole 36A and the gray metaconglomerate of the chunk sample (both of which are rich in quartz) are 2.68 and 2.63, respectively, and these values are typical for this type of rock.

The small difference between the bulk specific gravity and the apparent

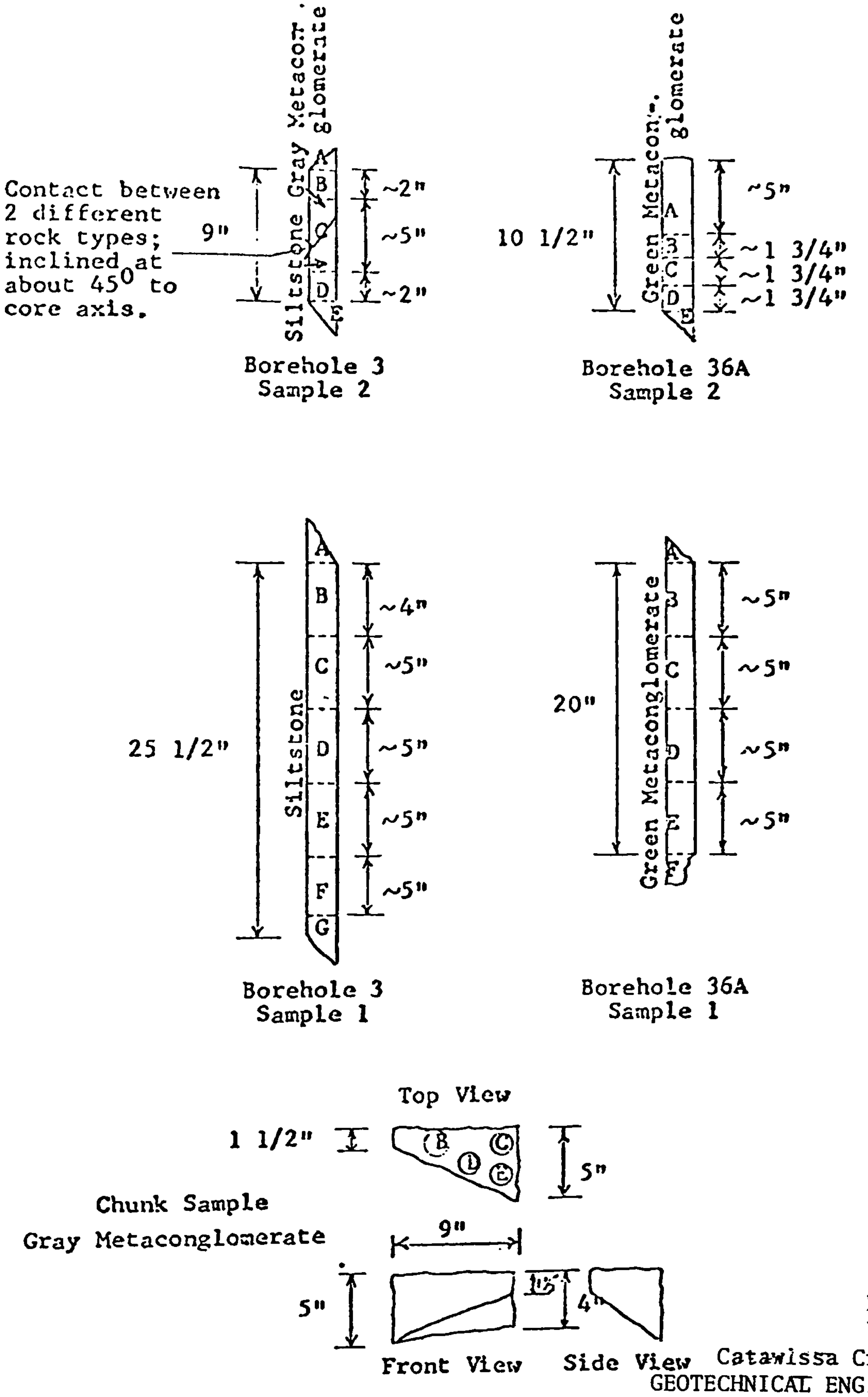

FIG. C-1
ROCK SAMPLES
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971

TABLE C-2. SPECIFIC GRAVITIES

Borehole No.	Sample No.	Specimen	Rock Type	Specific Gravity		
				Bulk*	Bulk* (Saturated Surface-Dry Basis)	Apparent*
3	1	G	Siltstone	2.72	2.73	2.76
36A	1	A	Greenish-gray meta-conglomerate	2.68	2.68	2.69
--	Chunk	--	Gray meta-conglomerate	2.64	2.64	2.64

*Specific gravities as defined in Sections 5, 6, and 7 of ASTM Designation C 127-68.

specific gravity for each of the three samples indicates that these rocks have low porosity. (Low porosity, and an absence of significant microcracks, is also indicated by the shape of the stress- strain curves for the unconfined compression tests on the greenish-gray metaconglomerate from Borehole 36A, as discussed in Section 4 of this report.)

UNCONFINED COMPRESSIVE STRENGTH AND DEFORMATION CONSTANTS

Scope of Testing

One unconfined compression test was performed on the greenish-gray metaconglomerate from Borehole 36A and one on the gray conglomerate that comprised the chunk sample. The results are summarized in Table C-3 and the stress-strain curves are shown in Figs. C-2 and C-3.

Measurements Made for Computing Deformation Constants

For the purpose of computing the modulus of deformation and Poisson's Ratio, axial and circumferential strains were each measured with a set of three SR-4 strain gages bonded to the surface of the specimen. The values of strain measured by the three gages in each set were generally consistent among themselves, except close to failure when cracking and splitting resulted in some inconsistencies. (It is of interest to note that the stratus computed from the displacement of the loading crossarm on the testing machine were roughly twice those measured with the strain gages, probably because of testing errors such as seating deformation, which confirms the importance of using strain gages bonded to the specimen when it is desired to measure the deformation constants accurately.)

Modulus of Deformation and Compressive Strength

The unconfined compressive strengths of the greenish-gray metaconglomerate from Borehole 36A and the gray metaconglomerate of the chunk sample are 25,400 psi and 16,600 psi respectively; the corresponding values of the secant modulus of deformation at 50% compressive strength are 8.91×10^6 and 5.15×10^6 psi, respectively. The ratio of the modulus of deformation to the unconfined strength is 350 for the greenish-gray metaconglomerate and 310 for the gray metaconglomerate. All of these values look reasonable for these types of rock.

Stress-Strain Curve

The absence of any significant reversal of curvature in the stress-strain curve at low axial stress for the greenish-gray metaconglomerate from Borehole 36A (Fig. C-2) indicates that the rock has low porosity and is free of any significant microcracks.

The stress-strain curve for the gray metaconglomerate of the chunk sample does have a significant reversal of curvature at low axial stress, which does indicate significant microcracking. There are three possible causes of the microcracking: (1) The deformation of the rock mass that produced the slickensides on the chunk sample; (2) blasting damage, if the

TABLE C-3. UNCONFINED COMPRESSION TESTS ON INTACT ROCK

	Test No. U1	Test No. U2*
Borehole No.	36A	-
Sample No.	2	Chunk
Specimen	A	B
Rock Type	Greenish-gray metaconglomerate	Gray meta-conglomerate
Unconfined Compressive Strength qu (psi)	25,400	16,600
Strain at Failure	0.0029	0.0028
Secant Modulus of Deformation at 50% qu (psi)	8.91×10^6	5.15×10^6
Tangent Modulus of Deformation at 50% qu (psi)	9.52×10^6	8.33×10^6
Poisson's Ratio		
Secant Value at 50% qu	0.14	0.11
Secant Value at 25% qu	0.12	0.09
Value for Incremental Load at 50% qu	0.20	0.17
Value for Incremental Load at 25% qu	0.18	0.12

*No slickensides were apparent in this triaxial specimen, but there was one crack roughly parallel to the axis of the specimen.

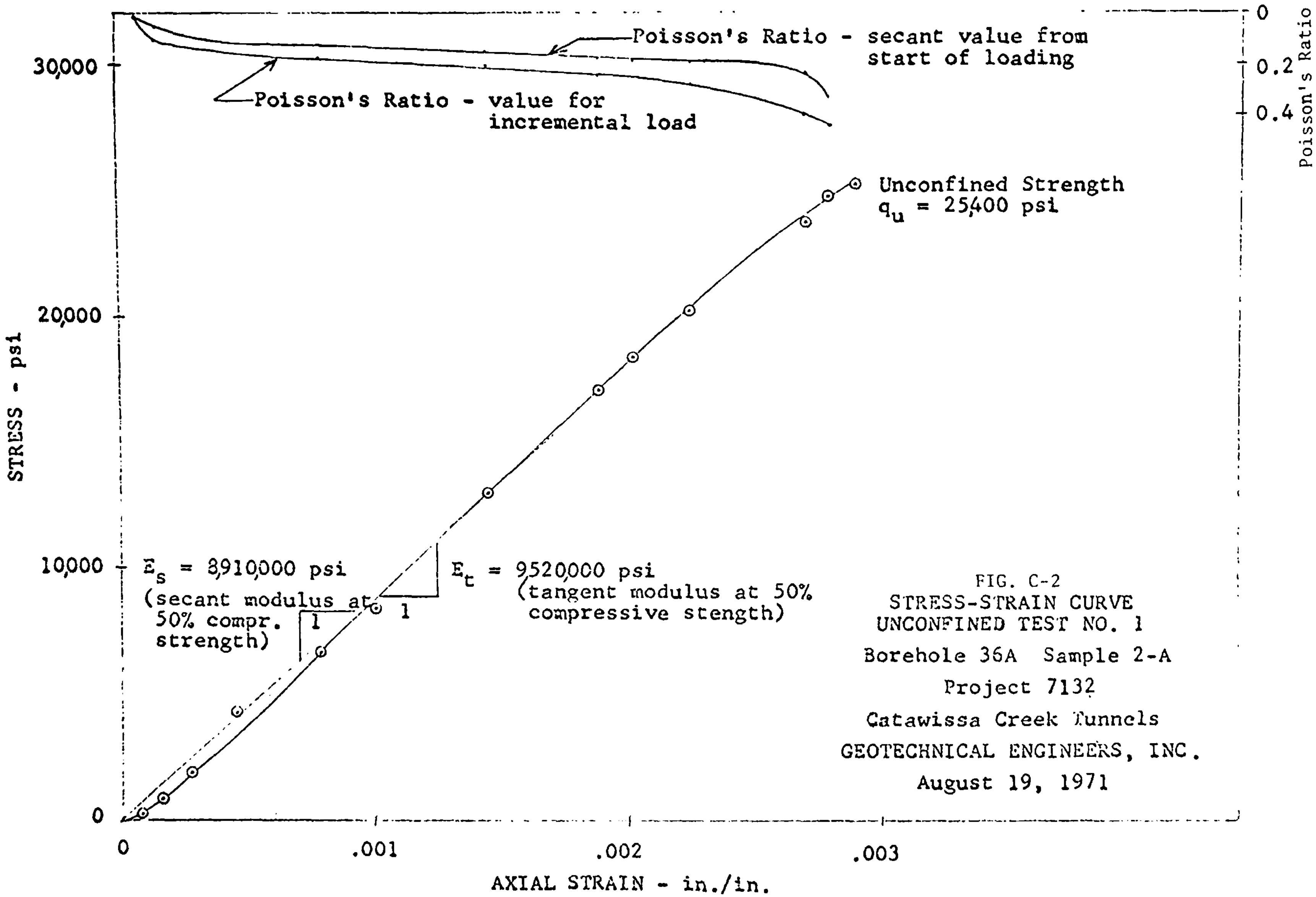

FIG. C-2
STRESS-STRAIN CURVE
UNCONFINED TEST NO. 1
Borehole 36A Sample 2-A
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971

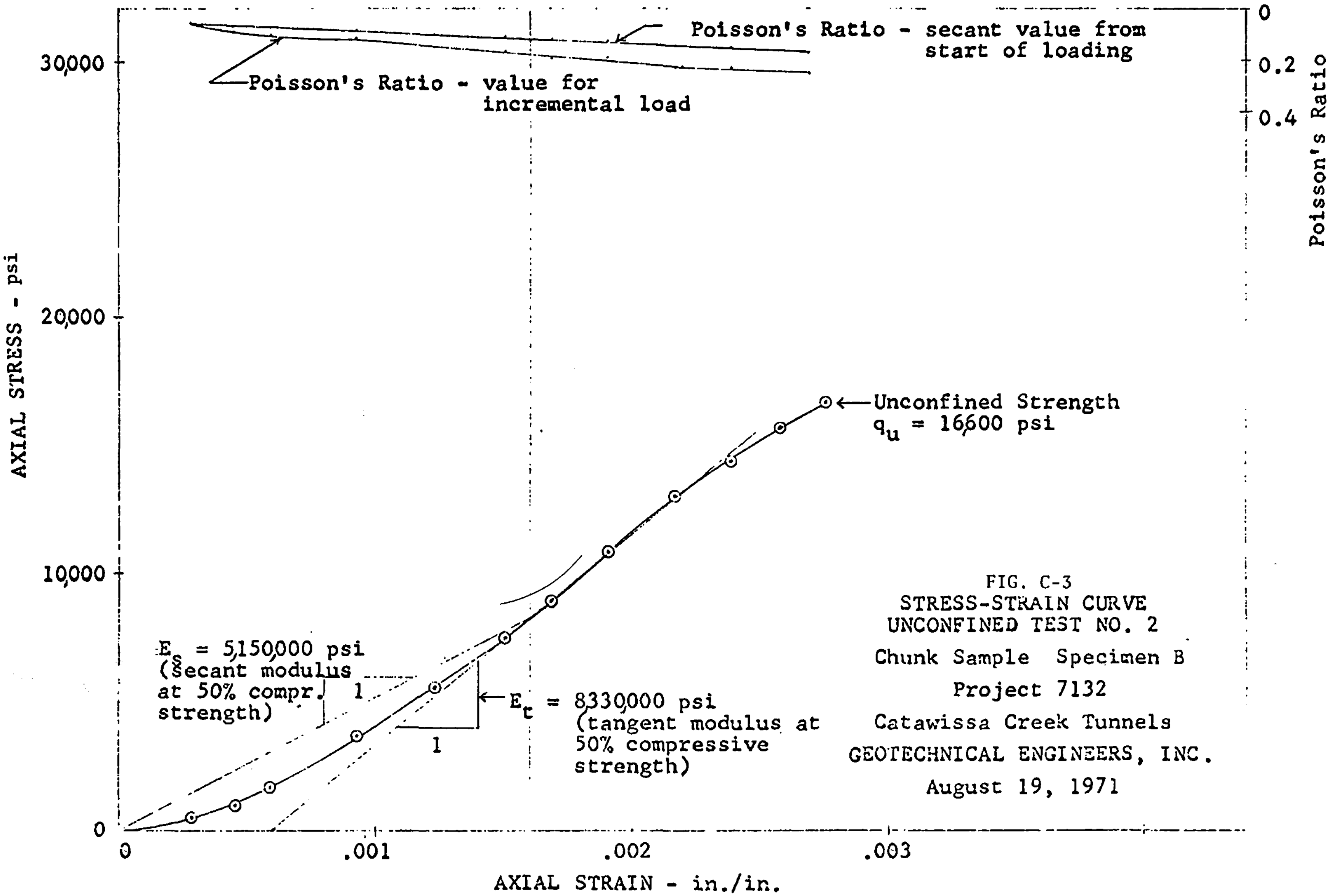

FIG. C-3
STRESS-STRAIN CURVE
UNCONFINED TEST NO. 2
Chunk Sample Specimen B
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971

sample was taken from a tunnel that had been excavated by blasting; (3) stress relief, if the sample was taken from a greater depth.

Both samples failed at less than 0.3% strain, and are thus quite brittle.

Poisson's Ratio

Two values of Poisson's ratio are plotted as a function of axial strain in Figs. C-2 and C-3, a secant value and an incremental value. The secant value is computed by dividing the circumferential strain by the axial strain at any point during the test; the incremental value is computed by dividing the change in circumferential strain by the change in axial strain for a given load increment. The value that should be used in any computations obviously depends on the initial state of stress in the rock and will vary with the magnitude of the stress changes being considered.

The secant value of Poisson's ratio for both of the rocks tested increases with increasing axial stress, from values of 0.05 or less at the start of loading, to about 0.1 at 25% of the compressive strength, to about 0.2 near failure. These results are typical for these types of rock.

TRIAXIAL TESTS ON INTACT ROCK

Scope of Testing

One series of four triaxial tests, at confining pressures of 20, 100, 200, and 400 psi, was performed on the siltstone from Boring 3 and a similar series was performed on the greenish-gray metaconglomerate from Boring 36A. Stress-strain curves have not been plotted, because the axial displacements were measured outside the triaxial chamber and are thus not reliable for computing the strain of specimens that have a high modulus of deformation, as these rocks do. The results are summarized in Tables C-4 and C-5.

Specimen Preparation

The specimens consisted on NX core, ground and lapped so that the ends would be plane and perpendicular to the core axis. The lengths of the specimens ranged from 4.84 to 4.95 inches, except for one specimen which was only 4.10 inches long.

Peak Compressive Strengths

The peak compressive strengths for the triaxial tests are summarized in Table C-4. Mohr strength circles for the tests on Sample 1, Borehole 3, are plotted in Fig. C-4, and for the tests on Sample 1, Borehole 36A, in Fig. C-5.

Because the range of confining pressures that was used (20 to 400 psi) is small compared to the unconfined compressive strength (estimated to be of the order of 4000 psi, for the siltstone in Sample 1, Borehole 3, and 20,000 psi for the greenish-gray metaconglomerate in Sample 1, Borehole 36A) the

TABLE C-4. TRIAXIAL TESTS ON INTACT ROCK

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Confining Pressure (psi)	Peak Strength (psi)	Axial Displacement at Peak Strength* (in.)	Post-Peak Strength** (psi)	Axial Displacement at Post-Peak Strength* (in.)
TI	3	1	F	Siltstone	20	4,141	.017	414	.106
T2	3	1	C	Siltstone	100	6,050	.021	1,220	.150
T3	3	1	D	Siltstone	200	5,820	.024	2,160	.094
T4	3	1	E	Siltstone	400	5,120	.023	2,160	.152
T5	36A	1	B	Greenish-gray meta-conglomerate	20	21,200	.034	-0	.083#
T6	36A	1	C	Greenish-gray meta-conglomerate	100	24,900	.047	55	1.15#
T7	36A	1	D	Greenish-gray meta-conglomerate	200	22,300	.044	690	0.67##

(Continued)

TABLE C-4. (Continued)

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Confining Pressure (psi)	Peak Strength (psi)	Axial Displacement at Peak Strength* (in.)	Post-Peak Strength** (psi)	Axial Displacement at Post-Peak Strength* (in.)
T8	36A	1	E	Greenish-gray meta-conglomerate	400	23,800	.044	480	0.53##

* Strains were not measured in the triaxial tests by attaching SR-4 strain gages to the specimens. The displacements recorded in this Table are the changes in the distance between the Table and loading crossarm of the testing machine. From measurements made during unconfined tests we estimate that these displacements are at least twice as large as the corresponding changes in length of the rock specimens at the peak strength.

** Because only a limited displacement can be practically developed along the shear plane in a triaxial specimen it is not possible to measure the true residual strength (i.e., the strength at which unlimited displacement can occur.) We do not know of any reliable residual strength determinations that have been made on rocks harder than clay-shales, and therefore, we would have no basis for estimating the true residual strength of these specimens, which will probably be lower than the tabulated "post-peak" strengths.

\# Sample shattered badly. Measured post-peak strength is not considered significant.

\## Sample shattered, membrane broke before post-peak point was reached. Measured post-peak strength is not considered significant.

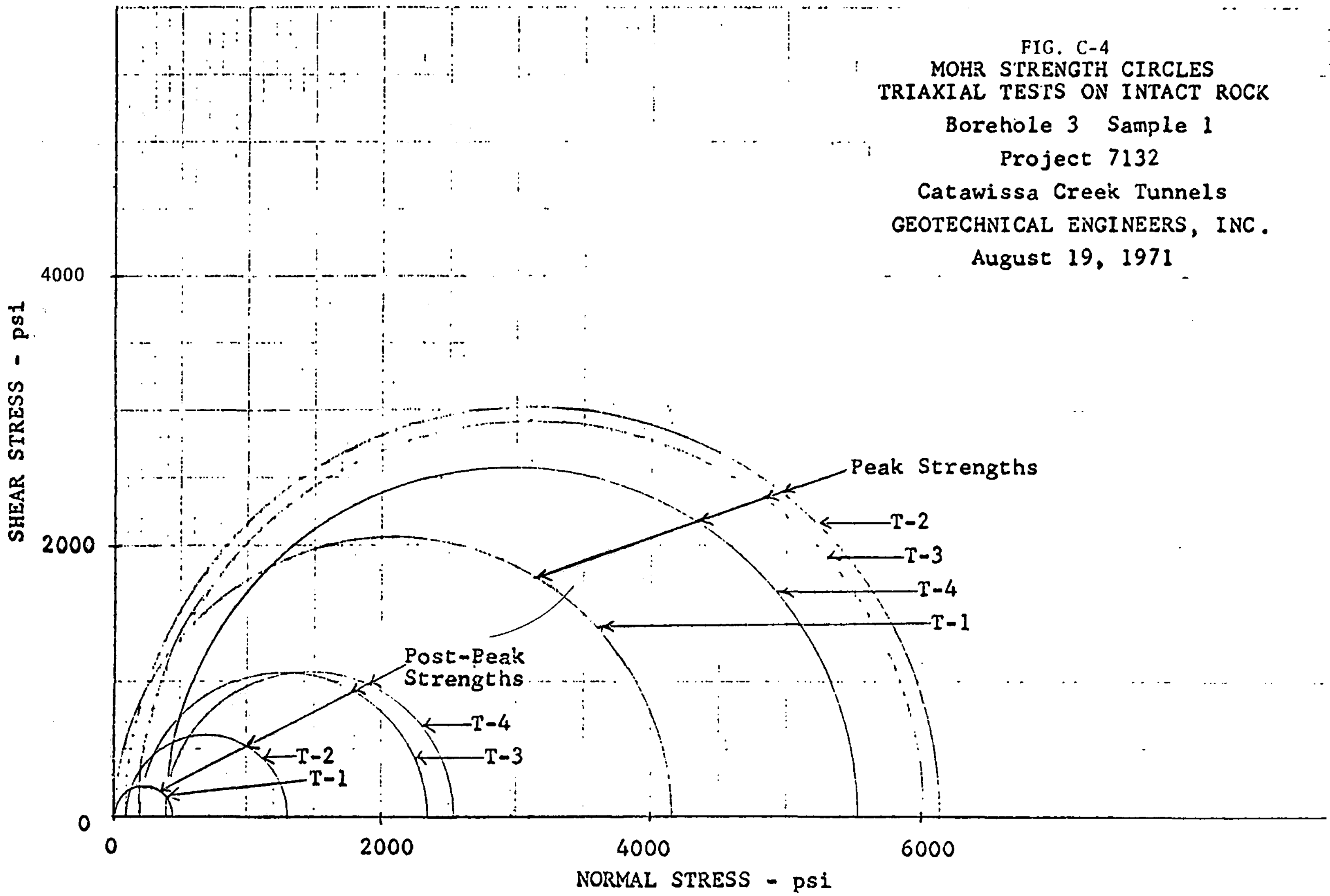
FIG. C-4
MOHR STRENGTH CIRCLES
TRIAXIAL TESTS ON INTACT ROCK
Borehole 3 Sample 1
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971
Peak Strengths
T-2
T-3
T-4
T-1
Post-Peak
Strengths
T-4
T-2
T-3
T-1
SHEAR STRESS - psi
4000
2000
0
0
2000
4000
6000
NORMAL STRESS - psi

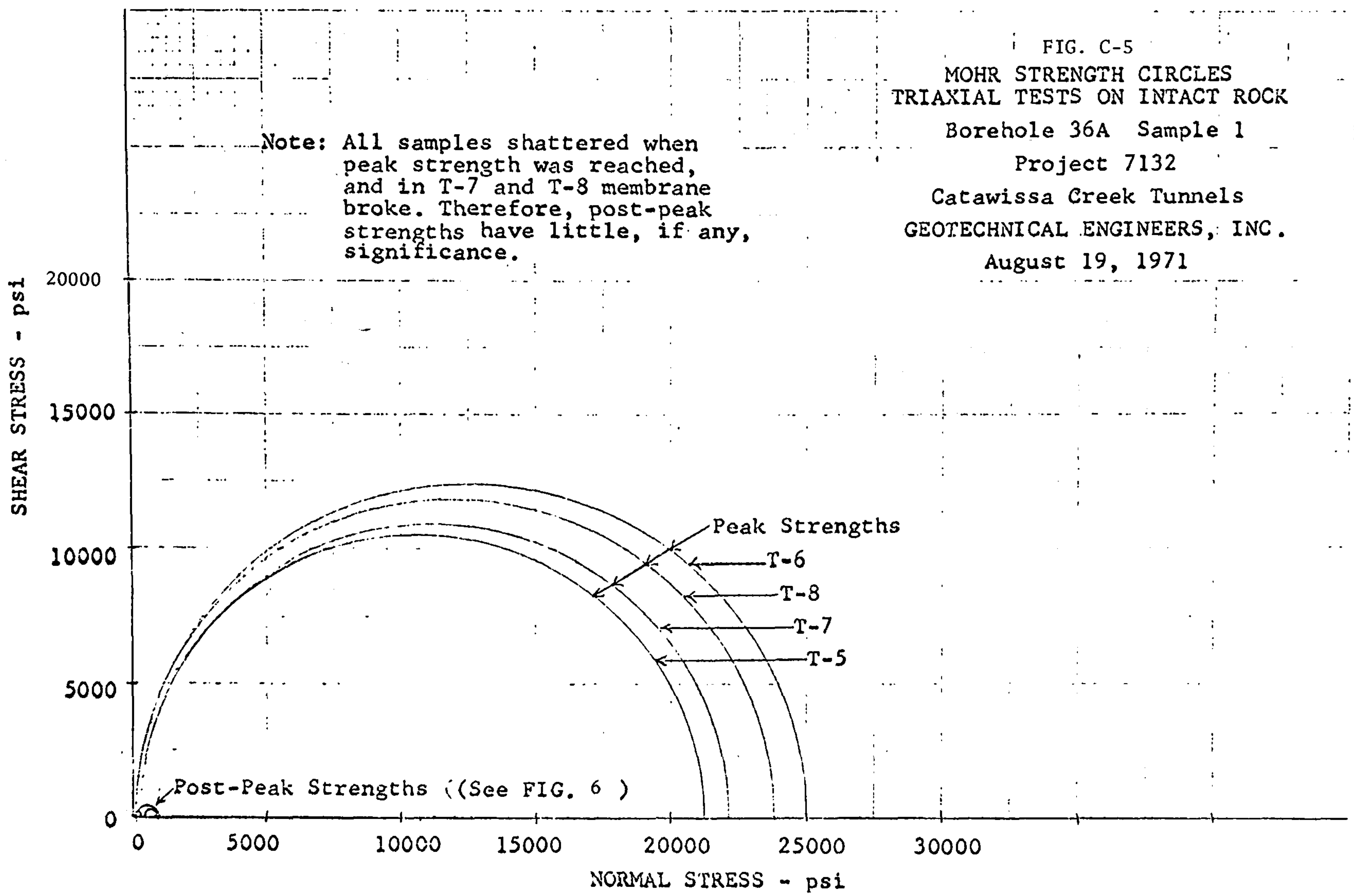

FIG. C-5
MOHR STRENGTH CIRCLES
TRIAXIAL TESTS ON INTACT ROCK
Borehole 36A Sample 1
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971

natural scatter of results due to nonhomogeneity of the rock completely masks the effect of the confining pressure on the strength. Therefore, no attempt has been made to draw Mohr strength envelopes for the peak-strength circles in Figs. C-4 and C-5.

Post-Peak Compressive Strengths

An attempt was made to measure the strengths of the triaxial specimens after failure had occurred. These strengths are referred to as "post-peak" strengths rather than "residual" strengths, because we believe that the strains that can be developed in the triaxial specimens are too small to get down to the true residual strengths. (Tests performed by Dr. LaGatta, of our firm, indicate that the strains required to reach the true residual strength for shale are orders of magnitude larger than those that can be developed in direct shear or triaxial tests. His tests were performed in a ring-shear apparatus. We do not know of any tests that have been performed to measure the true residual strengths of rocks other than shales.)

In Table C-4, the axial displacements measured outside the triaxial chamber are tabulated corresponding to the peak strength and to the recorded value of post-peak strength. The displacements at the post-peak strength are of the order of 5 to 10 times the displacements at the peak strength. If it were practicable to produce still larger displacements, the post-peak strengths might become smaller than the tabulated values. Also, most of the samples of greenish-gray metaconglomerate from Borehole 36A shattered badly when they failed and hence did not produce the more-or-less regular failure plane that would be required to measure the residual strength.

It is our opinion, based on our knowledge of the strength along joint surfaces in similar rocks, that the post-peak strengths for the siltstone are considerably larger than the residual strength that might be developed at larger displacement along a more-or-less planar surface. For the greenish-gray metaconglomerate from Borehole 36A, the post-peak strengths plotted on Fig. C-5, and to an enlarged scale on Fig. C-6, show so much scatter that it is impossible to draw any conclusions about the post-peak strength of that rock.

It is our opinion that the values of post-peak strength measured in the triaxial tests should not be used for design purposes. (The values of post-peak strength measured in the direct-shear apparatus for the siltstone appear to be more consistent with the strengths measured on joint surfaces for similar rocks, although even they are probably not down to the true residual strength.)

DIRECT SHEAR TESTS ON INTACT ROCK

Scope of Testing

One direct-shear test was performed to measure the peak shear strength of the siltstone from Borehole 3, at a normal stress of 300 psi; post-peak strengths were measured at normal stresses of 80, 140, 300, and 500 psi.

One direct shear test was performed on the specimen from Sample 2,

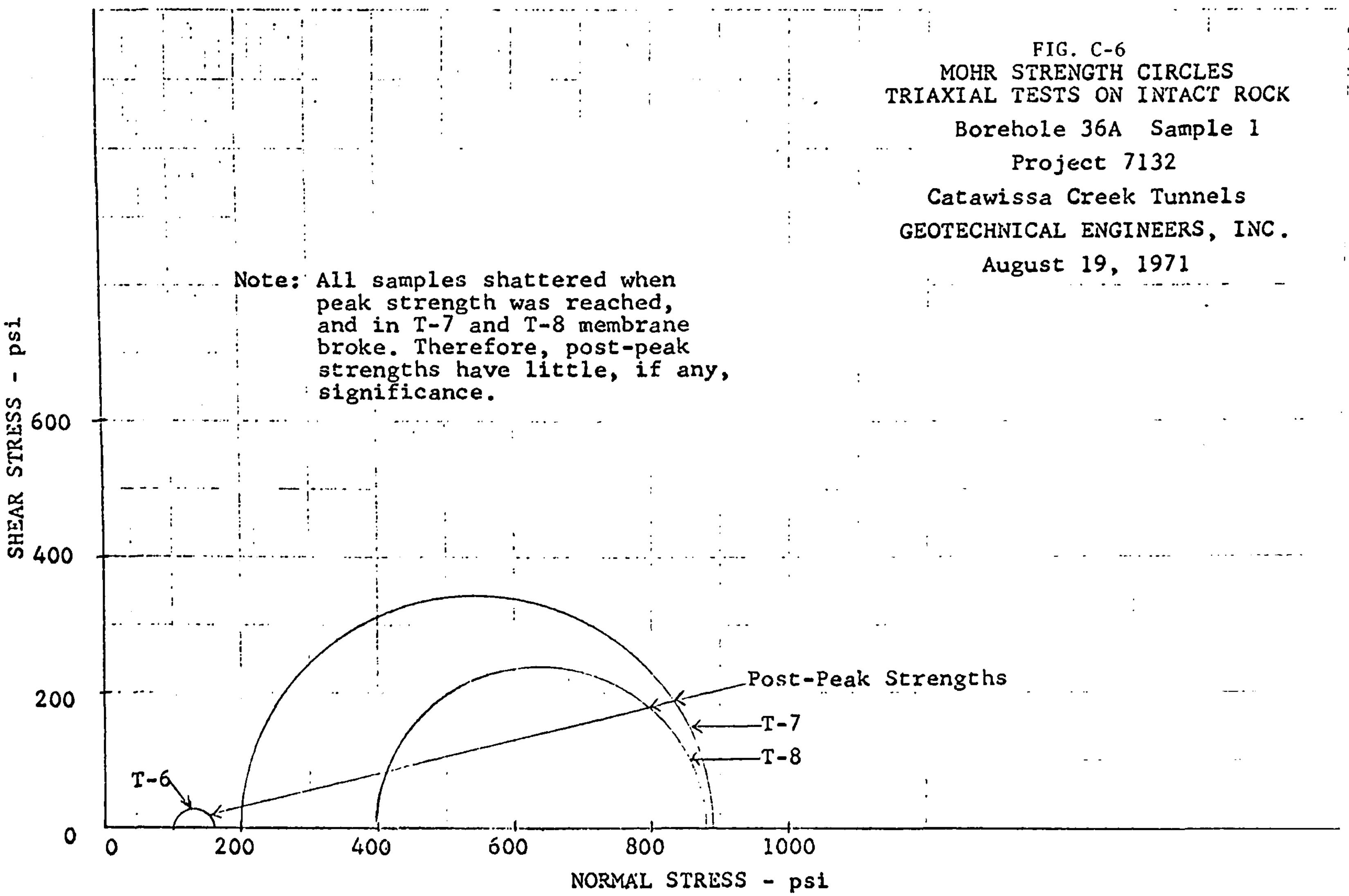
FIG. C-6
MOHR STRENGTH CIRCLES
TRIAXIAL TESTS ON INTACT ROCK
Borehole 36A Sample 1
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971
Note: All samples shattered when peak strength was reached, and in T-7 and T-8 membrane broke. Therefore, post-peak strengths have little, if any, significance.
Post-Peak Strengths
T-7
T-8
T-6
SHEAR STRESS - psi
0
200
400
600
0
200
400
600
800
1000
NORMAL STRESS - psi

Borehole 3, that contained the contact between siltstone and metaconglomerate. The specimen was oriented in the direct shear apparatus so that the plane of the contact coincided with the plane midway between the two halves of the shear box. The peak strength was measured at a normal stress of 251 psi, and post-peak strengths were measured at normal stresses of 67, 117, 251, and 418 psi.

The results are summarized in Table C-5 and Figs. C-7 and C-8.

Peak Strength

The peak compressive strength measured at a single normal stress for each of the two specimens is given in Table C-5.

For Test DS-8 (on the contact between siltstone and metaconglomerate) failure occurred entirely through the siltstone rather than at the contact itself.

The peak strengths from the two tests, both representing failure through the siltstone, lie slightly above the Mohr peak-strength circles for triaxial tests on the siltstone from Borehole 3 (see Fig. C-4).

Post-Peak Strength

Measurements of post-peak strength, as defined in Section 5.4 of this report for the triaxial tests, were also made during these two direct shear tests on intact rock. The results are summarized in Table C-5 and plotted on Figs. C-7 and C-8. The displacements at the peak strength are generally too small to be measured with any confidence that they are representative of the physical behavior of the rock in the shear zone. The displacements at which the post-peak strengths were measured are recorded in Table C-5. These post-peak strengths are probably higher than the true residual strength.

The post-peak strengths of the siltstone, plotted in Fig. C-7, were measured by continuing the displacement and altering the normal stresses. At normal stresses of 80 and 140 psi there were two determinations of post-peak strength, the second one, which was lower than the first, corresponding to a larger displacement and hence probably closer to the true residual strength.

The post-peak strengths for Test DS-8 plotted in Fig. C-8 define a straight line through the origin inclined at 27°.

The reason for the break in the slope of the post-peak strength line in Fig. C-7 is not clear. Since both Fig. C-7 and Fig. C-8 correspond to failure through the siltstone, it would be conservative to use the lower post-peak strengths of Fig. C-8 for analyzing sliding along a plane fracture surface in the siltstone.

DIRECT SHEAR TESTS CONCRETE/ROCK INTERFACE

Preparation of Specimens

TABLE C-5. DIRECT SHEAR TESTS ON INTACT ROCK

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Normal Stress (psi)	Peak Shear Stress (psi)	Post-Peak Shear Stress (psi)	Shear Displacement at Post-Peak Shear Stress (in.)	Remarks
DS-4	3	1	B	Siltstone	300	1,500	332	0.118	All tests were performed on a single specimen. The displacement of one half of the shear box with respect to the other was in one direction only, i.e., the shear box was not moved back to its original position for each successive determination of values of post-peak strength. At the end of the test, an attempt was made to bring the shear box back to its initial position for the purpose of measuring additional values of post-peak stress, but the sliding surface was very irregular and it was covered with crushed material, which made it impractical to perform additional shear cycles.
			B	Siltstone	80	---	152	0.137	
			B	Siltstone	140	---	171	0.169	
			B	Siltstone	500	---	343	0.204	
			B	Siltstone	300	---	221	0.222	
			B	Siltstone	140	---	137	0.245	
			B	Siltstone	80	---	84	0.266	

(Continued)

TABLE C-5. (Continued)

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Normal Stress (psi)	Peak Shear Stress (psi)	Post-Peak Shear Stress (psi)	Shear Displacement at Post-Peak Shear Stress (in.)	Remarks
DS-8	3	2	C	Contact between siltstone and gray conglomerate	251	1,775	140	0.091	This specimen contained a plane contact face (which was inclined at about 40° to the core axis). The specimen was placed in the direct-shear apparatus so that the contact plane coincided with the shear plane between the two halves of the shear box. The intact specimen did not fail along the contact plane; it failed along a surface in the siltstone a few hundreds of an inch away from the contact between the siltstone and the metaconglomerate. Procedure for measuring post-peak strengths was the same as described above for DS-4.
			C		418	---	209	0.189	
			C		117	---	68	0.300	
			C		67	---	41	0.352	

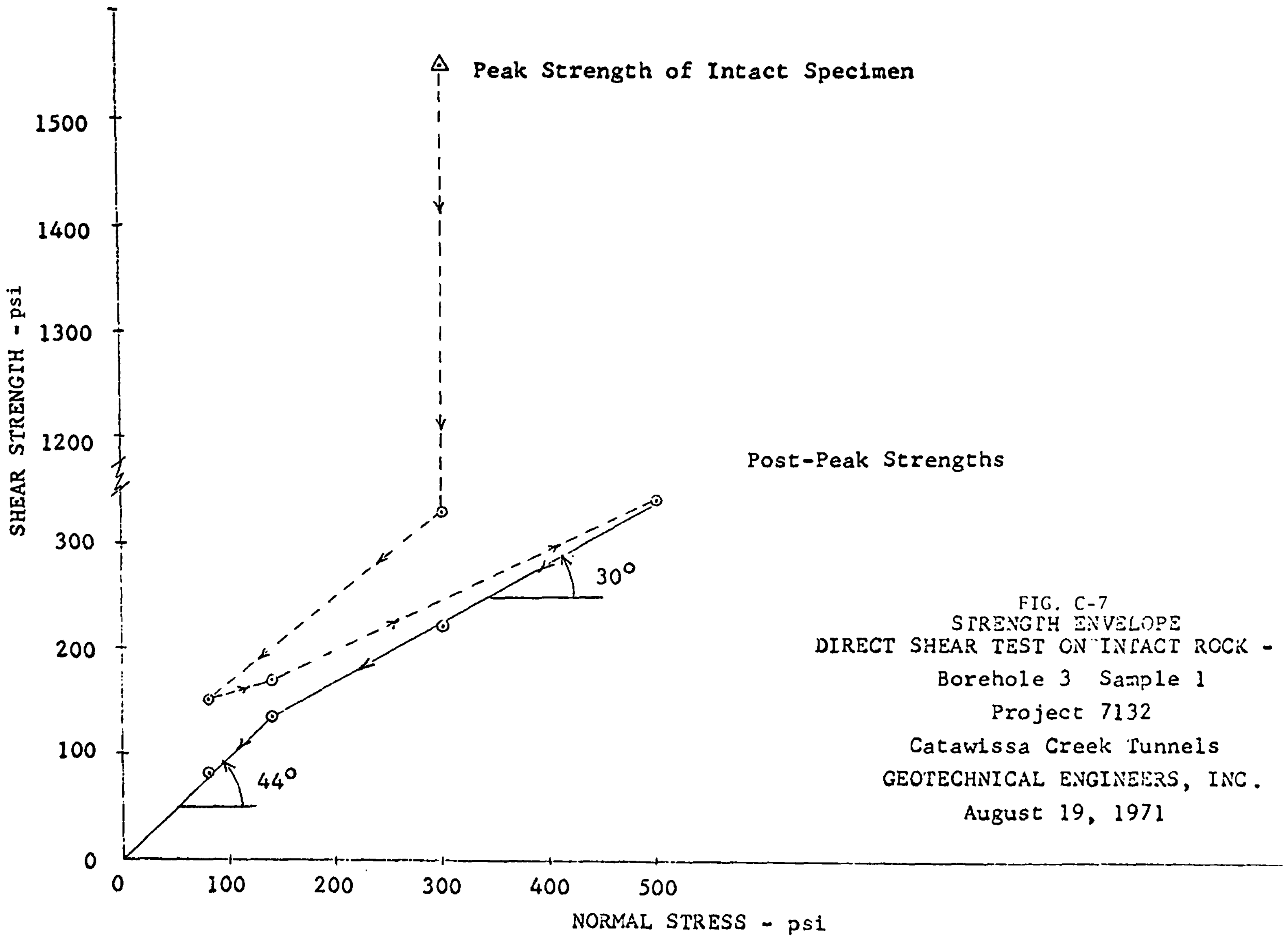

FIG. C-7
STRENGTH ENVELOPE
DIRECT SHEAR TEST ON INTACT ROCK - DS 4
Borehole 3 Sample 1
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
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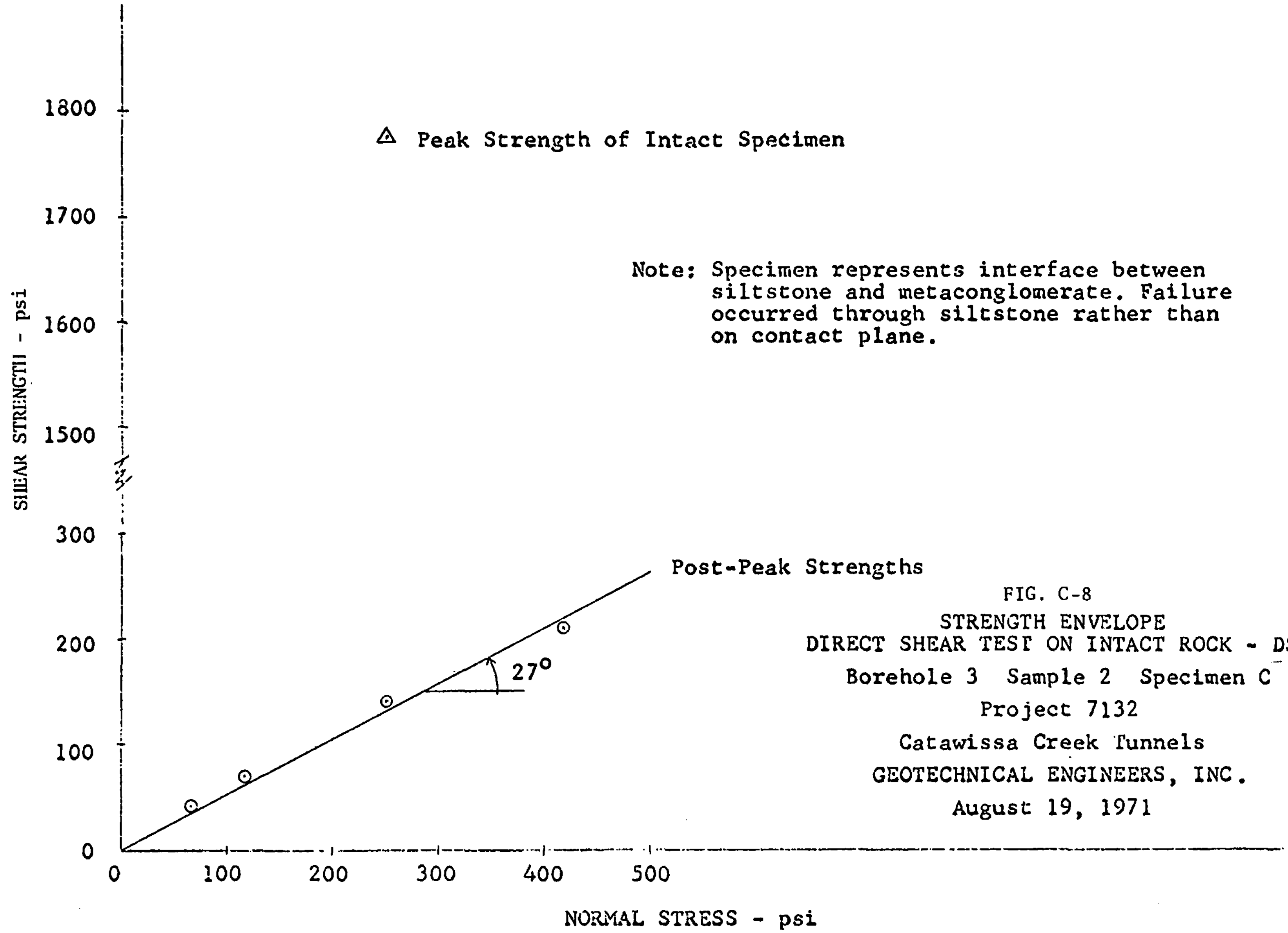

FIG. C-8

STRENGTH ENVELOPE

DIRECT SHEAR TEST ON INTACT ROCK - DS 8

Borehole 3 Sample 2 Specimen C

Project 7132

Catawissa Creek Tunnels

GEOTECHNICAL ENGINEERS, INC.

August 19, 1971

The specimens prepared from Sample 2, Borehole 36A, consisted of pieces of NX core. The end of each specimen was cut with a diamond saw, but was not otherwise ground or polished. The sawed surface was relatively smooth. Concrete was poured directly against the sawed surface in a mold having the same diameter as the NX core, and was allowed to cure for 6 days before the direct shear test was performed. The composite specimen was placed in the shear box such that the concrete/rock interface coincided with the plane midway between the two halves of the shear box.

The test specimens from the chunk sample were prepared in the same way. The cores taken from the chunk sample were of smaller diameter than NX core, however, ranging from 1.49 to 1.68 inches.

The concrete was made of a mixture of approximately 3.4 parts gravel, 4.5 parts sand, 2.5 parts cement, and 1 part water, by weight. Table C-7 gives the compressive strengths of 3 concrete specimens that were made from each of the 4 batches used for making the direct shear specimens. These strengths were measured after a 6-day cure, the same length of cure used for the concrete poured against the rock face for the direct shear specimens. The average strength of 3 of the batches is in the range 2000 to 2500 psi; the fourth batch had a strength of about 3500 psi. The interface strengths for the two tests performed using the 3500-psi concrete (DS9 and DS10) do not appear to be affected significantly by the different concrete strength.

Peak Strengths

The strength data are summarized in Table C-6 and the shear strengths are plotted against normal stress in Figs. C-9 and C-10.

In all cases, the failure took place along the interface, with no damage other than surface scratching on either the rock or the concrete. (Since the rock strength is much higher than the concrete strength, it is quite probable that the concrete would have been more extensively damaged if the rock surface at the interface were rougher than it was for these specimens.)

The peak strengths plotted in Fig. C-9 are consistent with those plotted in Fig. C-10. Two of the peak strengths for Tests DS-6 and DS-7) were not plotted because they were very low -- in fact, close to the post-peak strengths. It is believed that the rock/concrete bond at the interface had been damaged during setup in these two tests.

Post-Peak Strengths

The post-peak strengths plotted in Fig. C-9 are consistent with those plotted in Fig. C-10. A straight line envelope through the origin and inclined at 20° appears to be a good lower bound for all the post-peak values for the rock/concrete interface. This value should be quite conservative since the rock surface against which the concrete was cast is much smoother than the rock surface that would result from blasting in the field.

TABLE C-6. DIRECT SHEAR TESTS ON CONCRETE/ROCK INTERFACE

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Concrete Batch	Normal Stress (psi)	Peak Shear Stress (psi)	Post-Peak Shear Stress (psi)	Shear Displacement at Post-Peak Shear Stress (in.)	Remarks
DS-1	36A	2	B	Greenish-gray meta-conglomerate	1	---	---	---	---	Due to testing error, the specimen was broken in tension at the concrete/rock interface. A new batch of concrete was cast against the rock face, and the test was repeated as Test No. DS-9 (see below).
DS-2	36A	2	C	Greenish-gray meta-conglomerate	1	140	(See remarks)	55	0.060	Measured value of was 290 psi, but this value is wrong because of testing error.
	36A	2	C	Greenish-gray meta-conglomerate	1	300	---	107	0.094	
DS-3	36A	2	D	Greenish-gray meta-conglomerate	1	300	550	137	0.079	
	36A	2	D	Greenish-gray meta-conglomerate	1	500	---	176	0.260	

(See also Tests DS-9 and DS-10 which were performed on Sample 2, Borehole 36A).

(Continued)

TABLE C-6. (Continued)

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Concrete Batch	Normal Stress (psi)	Peak Shear Stress (psi)	Post-Peak Shear Stress (psi)	Shear Displacement at Post-Peak Shear Stress (in.)	Remarks
DS-5	---	Chunk	D	Gray meta-conglomerate	2	1,035	1,371	381	0.218	
			D	Gray meta-conglomerate	2	620	---	300	0.395	
DS-6	---	Chunk	E	Gray meta-conglomerate	2	606	402	276	0.091	Measured value of peak strength too low because of poor contact between loading platen and shear box. Value not plotted. Alternate test performed at normal stress of 300 psi (see DS-12).
DS-6	---	Chunk	E	Gray meta-conglomerate	2	283	---	116	0.410	
DS-7	---	Chunk	C	Gray meta-conglomerate	2	129	124	53	0.141	Measured value of peak strength too low because of poor contact between loading platen and shear box. Value not plotted. Alternate test performed at normal stress of 80 psi (see DS-13).
	---		C	Gray meta-conglomerate	2	246	---	107	0.357	

(See also Tests DS-12 and DS-13 which were performed on the Chunk sample.)

(Continued)

TABLE C-6. (Continued)

Test No.	Borehole No.	Sample No.	Specimen	Rock Type	Concrete Batch	Normal Stress (psi)	Peak Shear Stress (psi)	Post-Peak Shear Stress (psi)	Shear Displacement at Post-Peak Shear Stress (in.)	Remarks
DS-9	36A	2	B	Greenish-gray meta-conglomerate	3	80	425	50		
DS-10	36A	2	C	Greenish-gray meta-conglomerate	3	140	615	74		
DS-11	--	Chunk	D	Gray meta-conglomerate	4	---	---	---	---	Sample was broken apart at the concrete/rock interface during the test setup.
DS-12	--	Chunk	E	Gray meta-conglomerate	4	300	754	197		
DS-13	--	Chunk	C	Gray meta-conglomerate	4	80	615	48		
		Chunk	C	Gray meta-conglomerate	4	140	---	69		

*Because of the limited displacement that can be developed in a single continuous motion in the direct shear device, it is not possible to measure the true residual strength (i.e., the strength at which unlimited displacement can occur.) We do not know of any reliable residual-strength determinations that have been made on rocks harder than clay-shales or on rock/concrete interfaces, and therefore we would have no basis for estimating the true residual strengths of these specimens, which are probably lower than the measured "post-peak" strengths.

TABLE C-7. CONCRETE STRENGTHS

Batch No.	Curing Time (days)	Measured Compressive Strengths (psi)	Tests
1	6	2,250 3,410 2,010	DS-1, DS-2, DS-3
2	6	2,520 2,280 2,500	DS-5, DS-6, DS-7
3	6	3,420 3,730 3,590	DS-9, DS-10
4	6	1,940 2,210 2,100	DS-11, DS-12, DS-13

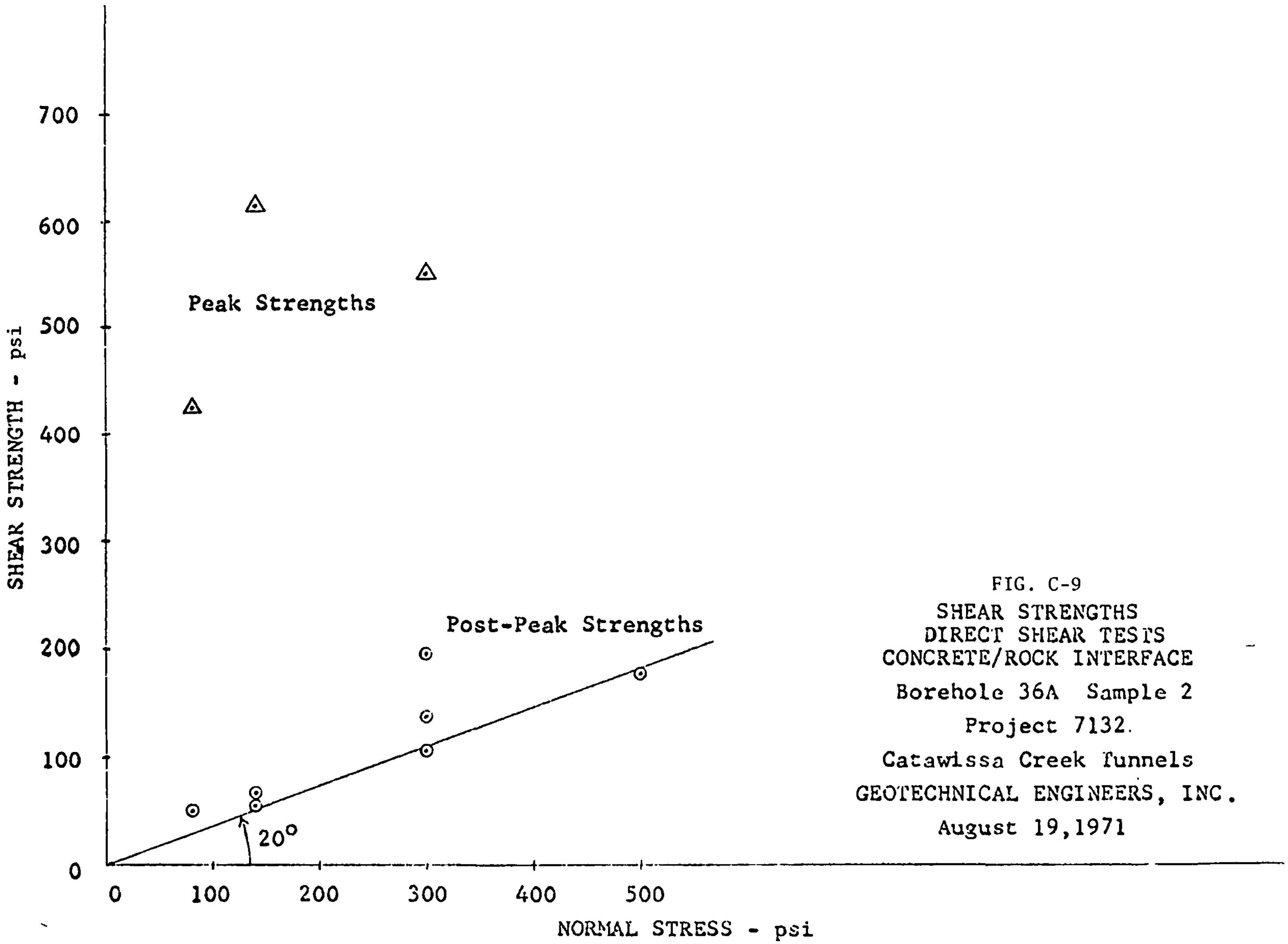

FIG. C-9
SHEAR STRENGTHS
DIRECT SHEAR TESTS
CONCRETE/ROCK INTERFACE
Borehole 36A Sample 2
Project 7132
Catawissa Creek Tunnels
GEOTECHNICAL ENGINEERS, INC.
August 19, 1971

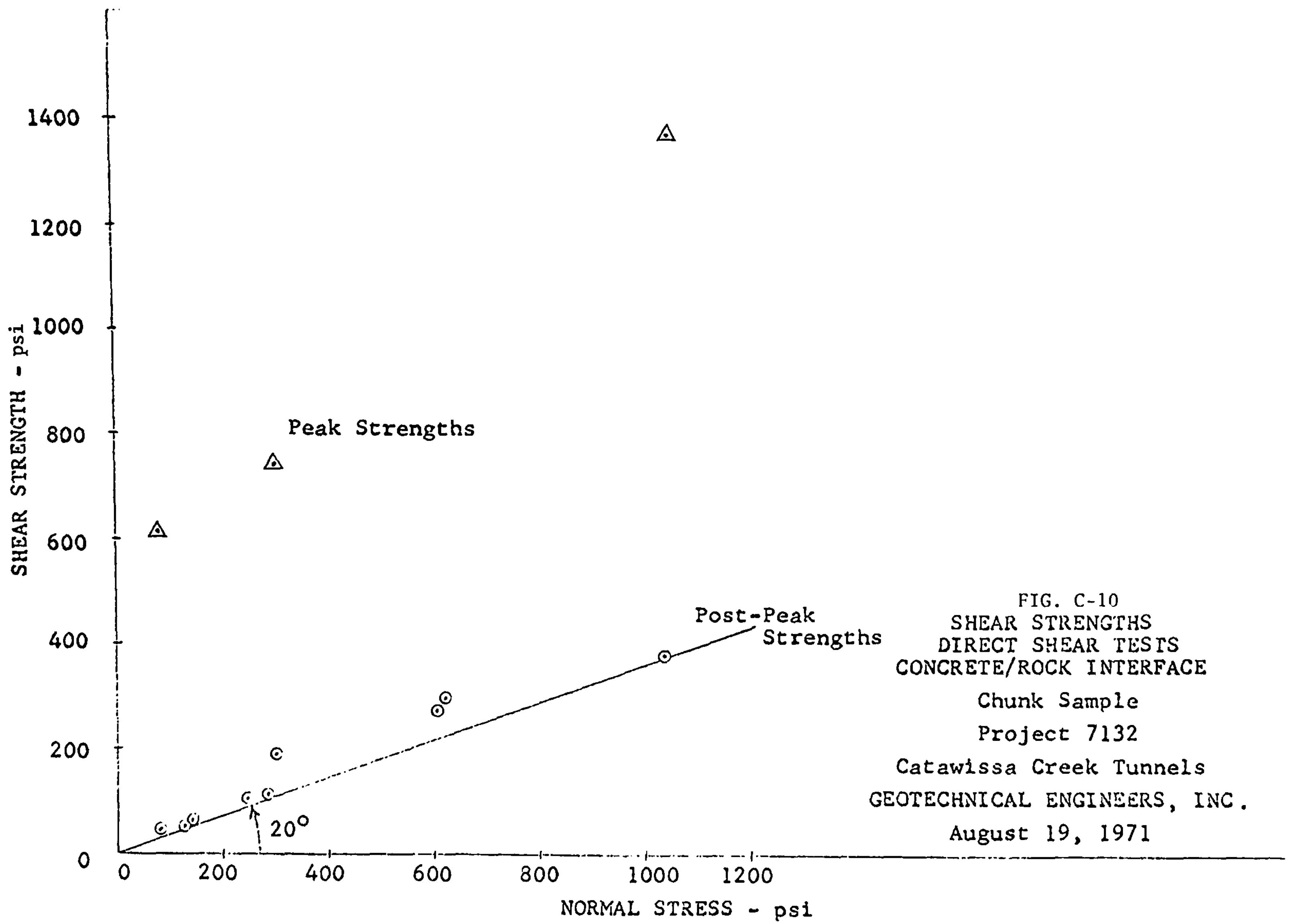

FIG. C-10
SHEAR STRENGTHS
DIRECT SHEAR TESTS
CONCRETE/ROCK INTERFACE

Chunk Sample

Project 7132

Catawissa Creek Tunnels

GEOTECHNICAL ENGINEERS, INC.

August 19, 1971

APPENDIX D

CHEMICAL TESTING OF TUNNEL ROCK

TABLE D-1. CHEMICAL TESTING OF SHALE (TUNNEL NO. 2)

	Date		Elapsed Time-Days	Solution				
				Temp. °C	pH		Al-mg/l (1)	
	Start	Stop			Start	Stop	Start	Stop
5% Sulfuric								
First Cycle	8/13/71	8/16/71	2.63	86-91	0.64	0.4	0.40	29
Second Cycle (5)	8/20/71	8/23/71	2.75	86-90	0.5	0.4	1.9	2,166
Last Cycle (6)	9/10/71	9/13/71	2.67	84	0.4	0.6	2,166	2,500
1% Sulfuric								
First Cycle	8/13/71	9/13/71	30.7	Room	1.2	1.3	0.62	652
Last Cycle (6)	9/27/71	10/11/71	13.7	Room	1.3	1.6	652	780
AMD								
First Cycle	8/13/71	9/13/71	30.7	Room	3.5	4.9	9.2	0.43
Second Cycle (6)	9/27/71	10/11/71	13.7	Room	4.9	4.9	0.43	0.19
Third Cycle (6)	10/15/71	11/11/72	26.7	Room	4.9	5.2	0.19	<0.02
Last Cycle	11/15/71	1/14/72	60.7	Room	5.2	5.5	<0.02	<0.02

(Continued)

TABLE D-1. (Continued)

	Date		Rock Sample Weight		Weight Loss Total		Weight Loss Per Day	
	Start	Stop	Start grams	Stop grams	grams	%	grams	%
5% Sulfuric								
First Cycle	8/13/71	8/16/71	101.807	97.645	4.162	4.09	1.58	1.56
Second Cycle (5)	8/20/71	8/23/71	97.645	94.880	2.765	2.83	1.005	1.03
Last Cycle (6)	9/10/71	9/13/71	94.880	93.106	1.774	1.87	0.664	0.70
1% Sulfuric								
First Cycle	8/13/71	9/13/71	90.208	89.276	0.932	1.03	0.0304	0.0337
Last Cycle (6)	9/27/71	10/11/71	88.276	88.778	0.498	0.565	0.0364	0.0413
AMD								
First Cycle	8/13/71	9/13/71	101.041	101.088	0.048(3)	--	--	--
Second Cycle (6)	9/27/71	10/11/71	101.088	101.009	0.079	0.0781	0.00576	0.0057
Third Cycle (6)	10/15/71	11/11/71	101.009	101.001	0.008	0.00792	0.0003	0.000297
Last Cycle	11/15/71	1/14/72	101.001	100.974	0.027	0.0268	0.00044	0.00044

(Continued)

TABLE D-1. Continued)

	Rock Sample							
	Suspended Weight Loss (2)				Dissolved Weight Loss			
	Total		Per Day		Total		Per Day	
	grams	%	grams	%	grams	%	grams	%
5% Sulfuric								
First Cycle	0.0154	0.015	0.006	0.006	4.147	4.075	1.574	1.554
Second Cycle (5)	0.258	0.26	0.093	0.095	2.507	2.57	0.912	0.035
Last Cycle (6)	0.018	0.02	0.007	0.007	1.756	1.85	0.657	0.693
1% Sulfuric								
First Cycle	0.0	--	--	--	0.932	1.03	0.0304	0.0337
Last Cycle (6)	0.007	0.008	0.0005	0.0006	0.491	0.557	0.359	0.0407
AMD								
First Cycle	0.0(4)	--	--	--	--	--	--	--
Second Cycle (6)	0.0(4)	0.0	0.0	0.0	0.079	0.0781	0.00576	0.0057
Third Cycle (6)	0.0(4)	0.0	0.0	0.0	0.008	0.00792	0.003	0.000297
Last Cycle	0.0093	0.0093	0.00015	0.00015	0.0177	0.0175	0.00029	0.00029

(1) In supernatant; there was some evaporation, particularly of AMD
(2) By filtration
(3) Gain
(4) Visual observation
(5) Fresh H_2SO_4
(6) Same solution as prior cycle

TABLE D-2. CHEMICAL TESTING OF CONGLOMERATE (TUNNEL NO. 2)

	Date		Elapsed Time- Days	Solution				
				Temp. °C	pH		Al-mg/l (1)	
	Start	Stop			Start	Stop	Start	Stop
5% Sulfuric								
First Cycle	8/13/71	8/16/71	2.63	82-91	0.64	0.40	0.40	53.25
Second Cycle (5)	8/20/71	8/23/71	2.75	84-85	0.5	0.4	1.9	604
Last Cycle (6)	9/10/71	9/13/71	2.67	83	0.4	0.7	604	777
1% Sulfuric								
First Cycle	8/13/71	9/13/71	30.7	Room	1.2	1.3	0.62	207.6
Last Cycle (6)	9/27/71	10/11/71	13.7	Room	1.3	1.4	207.6	268.6
AMD								
First Cycle	8/13/71	9/13/71	30.7	Room	3.5	7.4	9.2	0.41
Second Cycle (6)	9/27/71	10/11/71	13.7	Room	7.4	7.4	0.41	0.12
Third Cycle (6)	10/15/71	11/11/71	26.7	Room	7.4	7.5	0.12	<0.02
Last Cycle	11/15/71	1/14/72	60.7	Room	7.5	7.6	<0.02	<0.02

(Continued)

TABLE D-2. (Continued)

	Date		Rock Sample					
			Weight		Weight Loss			
					Total		Per Day	
	Start	Stop	Start grams	Stop grams	grams	%	grams	%
5% Sulfuric								
First Cycle	8/13/71	8/16/71	100.109	97.795	2.314	2.32	0.88	0.88
Second Cycle (5)	8/20/71	8/23/71	97.795	95.205	2.590	2.65	0.94	0.96
Last Cycle (6)	9/10/71	9/13/71	95.205	93.259	1.946	2.04	0.73	0.76
1% Sulfuric								
First Cycle	8/13/71	9/13/71	100.447	98.904	1.543	1.54	0.05	0.05
Last Cycle (6)	9/27/71	10/11/71	98.904	97.223	1.681	1.70	0.12	0.12
AMD								
First Cycle	8/13/71	9/13/71	98.223	98.230	0.007(3)	--	--	--
Second Cycle (6)	9/27/71	10/11/71	98.230	98.171	0.059	0.06	0.0043	0.0044
Third Cycle (6)	10/15/71	11/11/71	98.171	98.148	0.023	0.023	0.00086	0.00088
Last Cycle	11/15/71	1/14/72	98.148	98.127	0.021	0.021	0.00035	0.00035

(Continued)

TABLE D-2. (Continued)

	Rock Sample							
	Suspended Weight Loss (2)				Dissolved Weight Loss			
	Total		Per Day		Total		Per Day	
	grams	%	grams	%	grams	%	grams	%
5% Sulfuric								
First Cycle	0.967	0.97	0.368	0.37	1.347	1.35	0.512	0.51
Second Cycle (5)	1.804	1.84	0.655	0.67	0.786	0.81	0.285	0.29
Last Cycle (6)	1.325	1.39	0.496	0.52	0.621	0.65	0.232	0.24
1% Sulfuric								
First Cycle	1.025	1.02	0.033	0.033	0.518	0.52	0.017	0.017
Last Cycle (6)	1.423	1.44	0.104	0.10	0.258	0.26	0.019	0.02
AMD								
First Cycle	0.0 (4)	--	--	--	--	--	--	--
Second Cycle (6)	0.0 (4)	0.0	0.0	0.0	0.059	0.06	0.0043	0.0044
Third Cycle (6)	0.0 (4)	0.0	0.0	0.0	0.023	0.023	0.00086	0.00088
Last Cycle	0.0143	0.014	0.00024	0.00024	0.0067	0.007	0.00011	0.00011

(1) In supernatant; there was some evaporation, particularly of AMD
(2) By filtration
(3) Gain
(4) Visual observation
(5) Fresh H_2SO_4
(6) Same solution as prior cycle

TABLE D-3. CHEMICAL TESTING OF CONGLOMERATE (TUNNEL NO.3)

	Date		Elapsed Time-Days	Solution				
				Temp. °C	pH		Al-mg/1 (1)	
	Start	Stop			Start	Stop	Start	Stop
5% Sulfuric								
First Cycle	8/13/71	8/16/71	2.63	85-90	0.64	0.42	0.40	42
Second Cycle (5)	8/20/71	8/23/71	2.75	86-88	0.5	0.4	1.9	254
Last Cycle (6)	9/10/71	9/13/71	2.67	83	0.4	0.5	254	255
1% Sulfuric								
First Cycle	8/13/71	9/13/71	30.7	Room	1.2	1.1	0.62	192.6
Last Cycle (6)	9/27/71	10/11/71	13.7	Room	1.1	1.1	192.6	242.6
AMD								
First Cycle	8/13/71	9/13/71	30.7	Room	3.5	3.2	9.2	22
Second Cycle (6)	9/27/71	10/11/71	13.7	Room	3.2	3.3	22	20
Third Cycle (6)	10/15/71	11/11/71	26.7	Room	3.3	3.3	20	40.4
Last Cycle	11/15/71	1/14/72	60.7	Room	3.3	3.3	40.4	45.1

(Continued)

TABLE D-3. (Continued)

	Date		Rock Sample					
			Weight		Weight Loss			
					Total		Per Day	
	Start	Stop	Start grams	Stop grams	grams	%	grams	%
5% Sulfuric								
First Cycle	8/13/71	8/16/71	88.187	87.695	0.492	0.558	0.187	0.212
Second Cycle (5)	8/20/71	8/23/71	87.695	87.496	0.199	0.227	0.0724	0.0825
Last Cycle (6)	9/10/71	9/13/71	87.496	87.333	0.163	0.186	0.061	0.0697
1% Sulfuric								
First Cycle	8/13/71	9/13/71	103.951	103.791	0.160	0.154	0.00521	0.00501
Last Cycle (6)	9/27/71	10/11/71	103.791	103.692	0.099	0.0955	0.00723	0.00697
AMD								
First Cycle	8/13/71	9/13/71	102.287	102.275	0.012	0.0117	0.000391	0.000381
Second Cycle (6)	9/27/71	10/11/71	102.275	102.261	0.014	0.0137	0.00102	0.000997
Third Cycle (6)	10/15/71	11/11/71	102.261	102.251	0.010	0.00977	0.000374	0.000366
Last Cycle	11/15/71	1/14/72	102.251	102.245	0.006	0.00587	0.000099	0.000097

(Continued)

TABLE D-3. (Continued)

	Rock Sample							
	Suspended Weight Loss (2)				Dissolved Weight Loss			
	Total		Per Day		Total		Per Day	
	grams	%	grams	%	grams	%	grams	%
5% Sulfuric								
First Cycle	0.024	0.027	0.009	0.010	0.468	0.531	0.178	0.202
Second Cycle (5)	0.023	0.026	0.0084	0.0095	0.176	0.201	0.064	0.073
Last Cycle (6)	0.055	0.063	0.021	0.0235	0.108	0.123	0.040	0.0462
1% Sulfuric								
First Cycle	0.0	--	--	--	0.160	0.154	0.00521	0.00501
Last Cycle (6)	0.0037	0.0036	0.00027	0.00026	0.0953	0.0919	0.00696	0.00671
AMD								
First Cycle	0.0 (4)	--	--	--	0.012	0.0117	0.000391	0.000381
Second Cycle (6)	0.0 (4)	--	--	--	0.014	0.0137	0.00102	0.000997
Third Cycle (6)	0.0 (4)	--	--	--	0.010	0.00977	0.000374	0.000366
Last Cycle	0.0076	0.00744	0.000125	0.000122				

(1) In supernatant; there was some evaporation, particularly of AMD
(2) By filtration
(3) Gain
(4) Visual observation
(5) Fresh H_2SO_4
(6) Same solution as prior cycle

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-600/7-77-124	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE CATAWISSA CREEK MINE DRAINAGE ABATEMENT PROJECT		5. REPORT DATE November 1977 issuing date
		6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) A. F. Miorin, R. S. Klingensmith, F. J. Knight, R. E. Heizer, J. R. Saliunas		8. PERFORMING ORGANIZATION REPORT NO.
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15. SUPPLEMENTARY NOTES

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16. ABSTRACT

The objective of this study was to determine the feasibility of flooding underground coal mine workings in an isolated basin of coal, thereby restoring or partially restoring the groundwater table in the basin and reducing the production of acid mine drainage. Flooding the mined seams would prevent atmospheric oxygen contact with the acid-forming materials, thus breaking the chain of chemical reactions in the formation of acid mine drainage. To enable this determination, a relatively small discrete basin of coal in east central Pennsylvania at Sheppton was selected.

As the first step, the watershed's streambed was relocated to prevent streamflow from passing into, and emitting from, the mined basin. Approximately 518 meters of streambed was reconstructed at a cost of $58.94 per meter, eliminating 0.253 m^3/s of water from entering the underground mine workings. Even though the mine sealing was deemed to have much merit, it was cancelled because of its high costs after plans and specifications for sealing the three tunnels were prepared and bids were taken for sealing one water-level tunnel. Bid cost for constructing the one seal was in excess of $600,000.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Anthracite Coal Mines Pollution Underground Mining Abatement	Pennsylvania Acid Mine Drainage Catawissa Cost Mine Sealing Streambed Relocation	8G 8I 13B

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