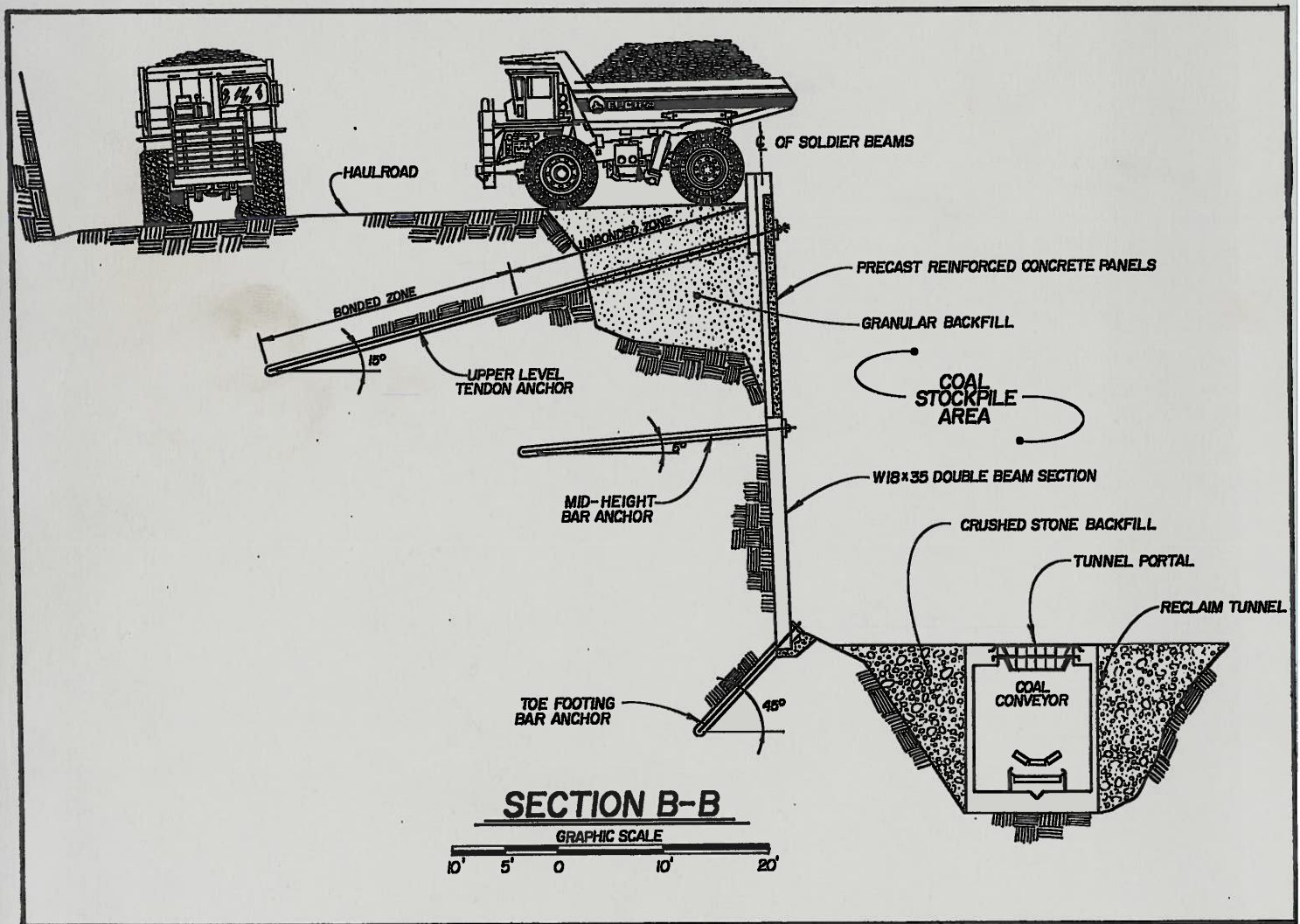


Ohio River Valley Soils Seminar XX

Construction in and on Rock



October 27, 1989
Louisville, Kentucky

ORVSS XX
CONSTRUCTION IN AND ON ROCK
October 27, 1989

8:00 Registration

Morning Session

- 8:45 Welcome
- 9:00 "The Cumberland Gap Pilot Bore," by Robert M. Leary and W. Randall Sullivan
- 9:30 "Pre-Grouting of the North Shore Tunnel, Milwaukee, Wisconsin," by James P. Rose and Roger C. Ilsley
- 10:00 "Mechanical Pre-Cutting as a Tunneling Technique," by Robert Longelin, Yves Le Goer, and E. van Walsum
- 10:30 Break
- 11:00 "Computer Analysis of Long-Term Stability of a Salt Dome in Relation to CAES Cavern Development," by M. Reza Salami and Sameer A. Hamoush
- 11:30 "Exploration and Testing for the Harlan Diversion Tunnels," by John L. Stanton
- 12:00 "Rock Testing for the Gallipolis Replacement," by Stephen T. Hornbeck
- 12:30 Luncheon

Afternoon Session

- 1:30 "Evaluation of In Situ Rock Mass Modulus by Use of Borehole Pressure Cell at Two Dam Sites," by Gordon M. Matheson and Jack E. Mason
- 2:00 "An Overview of Current U.S. Practice in Dam Stabilization Using Prestressed Rock Anchors," by Donald A. Bruce
- 2:30 "Rockfill Dams on Rock Foundation - Case Histories," by David C. Cowherd and Vlad G. Perlea
- 3:00 Break
- 3:30 "Subsurface Conditions in and Foundation Construction on Pinnacled Carbonate Bedrock," by James J. Belgeri and C. J. Shin
- 4:00 "Testing Rock-Socketed Drilled Piers Using the Osterberg Load Cell," by James D. Smith and Timothy Crowl
- 4:30 "Stabilization of a Coal Dumping Highwall Using a Tied-Back Structure," by Gary T. Brill and Bill Wells
- 5:30 Social Hour and Pizza Party - Hosted by Student Chapter, ASCE
Univ. of Louisville

PROCEEDINGS OF THE TWENTIETH
OHIO RIVER VALLEY SOILS SEMINAR

CONSTRUCTION IN AND ON ROCK

October 27, 1989
Radisson of Louisville Hotel
Louisville, Kentucky

Sponsored by

KENTUCKY GEOTECHNICAL GROUP, ASCE

UNIVERSITY OF LOUISVILLE
DEPARTMENT OF CIVIL ENGINEERING
AND CENTER FOR CONTINUING STUDIES

UNIVERSITY OF KENTUCKY
DEPARTMENT OF CIVIL ENGINEERING

and

CINCINNATI GEOTECHNICAL GROUP, ASCE

UNIVERSITY OF CINCINNATI
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

ORVSS XX ORGANIZING COMMITTEE

Stephen H. Bickel, Chairman
Fuller, Mossbarger, Scott & May
Louisville, Kentucky

Keith D. Coombs
Fuller, Mossbarger, Scott & May
Louisville, Kentucky

John W. Storm
Ground Engineering & Testing Service
Louisville, Kentucky

William J. Pfalzer
Division of Materials
Kentucky Department of Transportation
Frankfort, Kentucky

C. Robert Ullrich
Department of Civil Engineering
University of Louisville
Louisville, Kentucky

D. Joseph Hagerty
Department of Civil Engineering
University of Louisville
Louisville, Kentucky

ORVSS XX EXHIBITORS

BRAINARD-KILMAN DRILL COMPANY
Stone Mountain, Georgia

A. B. CHANCE COMPANY
Centralia, Missouri

DRAVO LIME COMPANY
Butler, Kentucky

RICHARD GOETTLE, INC.
Cincinnati, Ohio

MACCAFERRI GABIONS
Williamsport, Maryland

MOBILE DRILLING COMPANY, INC.
Indianapolis, Indiana

VIBRA-TECH ENGINEERS, INC.
Hazelton, Pennsylvania

TABLE OF CONTENTS

THE CUMBERLAND GAP PILOT BORE, by Robert M. Leary and W. Randall Sullivan

PRE-GROUTING OF THE NORTH SHORE TUNNEL, MILWAUKEE, WISCONSIN, by James P. Rose and Roger C. Ilesley

MECHANICAL PRE-CUTTING AS A TUNNELING TECHNIQUE, by Robert Longelin, Yves Le Goer, and E. van Walsum

COMPUTER ANALYSIS OF LONG-TERM STABILITY OF A SALT DOME IN RELATION TO CAES CAVERN DEVELOPMENT, by M. Reza Salami and Sameer A. Hamoush

EXPLORATION AND TESTING FOR THE HARLAN DIVERSION TUNNELS, by John L. Stanton

ROCK TESTING FOR THE GALLIPOLIS REPLACEMENT, by Stephen T. Hornbeck

EVALUATION OF IN SITU ROCK MASS MODULUS BY USE OF BOREHOLE PRESSURE CELL AT TWO DAM SITES, by Gordon M. Matheson and Jack E. Mason

AN OVERVIEW OF CURRENT U.S. PRACTICE IN DAM STABILIZATION USING PRESTRESSED ROCK ANCHORS, by Donald A. Bruce

ROCKFILL DAMS ON ROCK FOUNDATION - CASE HISTORIES, by David C. Cowherd and Vlad G. Perlea

SUBSURFACE CONDITIONS IN AND FOUNDATION CONSTRUCTION ON PINNACLED CARBONATE BEDROCK, by James J. Belgeri and C. J. Shin

TESTING ROCK-SOCKETED DRILLED PIERS USING THE OSTERBERG LOAD CELL, by James D. Smith and Timothy Crowl

STABILIZATION OF A COAL DUMPING HIGHWALL USING A TIED-BACK STRUCTURE, by Gary T. Brill and Wells

THE CUMBERLAND GAP PILOT BORE

Robert M. Leary
Geotechnical Engineer
Federal Highway Administration

W. Randall Sullivan
Principal
Golder Associates

Abstract. Twin highway tunnels, 4100 feet long, will be excavated through Cumberland Mountain in the early 1990's. The tunnels will be driven through sandstones, shales, and limestones dipping at 35 to 50 degrees and striking nearly perpendicular to the tunnel alignment.

One of the early phases of this \$237 million project was the construction of a 10-foot by 10-foot pilot bore at the crown of the proposed southbound tunnel. The primary purpose of this pilot tunnel excavation was to expose the geology along the main tunnel alignment. It was believed that a more precise assessment of the actual geology and rock mass characteristics would not only allow a more efficient and less costly design, but would allow physical access to this geology for potential contractors for the main bore project and thereby allow more accurate bidding. Significant instrumentation was also planned for the pilot bore to aid in the prediction of ground response during main tunnel construction.

This paper describes the construction and instrumentation of the Cumberland Gap pilot bore project and briefly summarizes the geologic and geotechnical information gathered. A qualitative assessment of the costs and benefits for this project indicates potential savings well in excess of the actual cost of the pilot bore design, construction, mapping, and instrumentation.

Introduction

Cumberland Gap has been a major transportation route through the Appalachians since it was formed by preferential erosion along a geologic fault. It was used as a migration route by buffalo and other animals and by Native Americans long before Thomas Walker documented the Gap's existence in 1750. After portions of the Wilderness Road were widened and cleared by Daniel Boone and some thirty axemen in 1775, over 300,000 settlers came through the Gap on their way to Kentucky's Bluegrass region. Although both sides expected

the Gap to be of great strategic value during the Civil War, most of the military action was garrison duty and no serious fighting took place here, although the Gap changed hands several times. In 1908, one of the first object lesson roads constructed with a macadam surface was completed over the Gap by the Bureau of Road Inquiry.

In an effort to preserve this historical area, local residents donated land to what would eventually become the 20,000 acre Cumberland Gap National Historical Park. The saddle of the Gap

and some 400 surrounding acres are now listed on the National Register of Historic Places. However, the Gap also carries over 18,000 vehicles per day on a steep and winding alignment of U.S. 25E, a major transportation corridor which connects two Interstate highways. The relocation of this roadway with its steadily increasing traffic is necessary if restoration and historic interpretation of this area is to continue. A 1978 environmental impact statement, completed in conjunction with a master plan for the park, formally recommended relocating U.S. 25E through twin tunnels south of the Gap. A similar recommendation was made to Congress over twenty years earlier by the acting director of the National Park Service.

Money for design and construction became available in 1979. The initial construction project was awarded in 1984 and included a 10-foot by 10-foot pilot bore to investigate geologic conditions along the length of the tunnels. The overall project as now designed includes twin 4100-foot long tunnels, nearly five miles of four lane roadway, two interchanges, and eight bridges. The total project cost is now estimated at about \$237 million and the tunnels are scheduled to be opened to traffic in 1995. An overall project layout is shown in Figure 5.

The Federal Highway Administration (FHWA) is serving as the design and construction manager for the project, on behalf of the U.S. Department of Interior, National Park Service. Parsons, Brinkerhoff, Quade, and Douglas is designing the main tunnel project. Golder Associates is serving as geotechnical consultants to FHWA for both design and construction. Vaughn and Melton is providing construction engineering services for the project.

Preliminary Investigations

Geotechnical investigations for the project have been extensive and have proceeded in stages since 1981. They have included a geologic literature review, inquiries into the construction and performance of the nearby railroad tunnel through Cumberland Mountain, seismic refraction surveys and core borings in the vicinity of the portals, aerial photo analysis, and an extensive field reconnaissance to collect geologic and geotechnical information from existing rock outcrops. The investigation defined the overall geology at the project site and provided general geotechnical information on each of the rock units.¹

The next phase of investigations involved a series of horizontal core borings along the tunnel alignment that penetrated up to 2000 feet into the Mountain from the Kentucky portal.² As a part of the horizontal coring program,

the first field trial of a newly developed combined geophysical probe system was conducted to produce resistivity, seismic, and electromagnetic logs in the first 1000 feet of one borehole.³

These investigations provided much additional information about the geology of the area and served as the basis for the design of the pilot tunnel. Cumberland Mountain is an overthrust block near the junction of the Valley and Ridge and Appalachian Plateau provinces. The rocks within the mountain range from Silurian to Pennsylvanian age formations. Rock conditions are extremely variable and range from uniform shales and limestones to interbedded sandstones, shales, and coals. Bedding dips at 35 to 50 degrees toward Kentucky and strikes nearly perpendicular to the tunnel alignment. Figure 1 shows a generalized geologic profile along the tunnel alignment.

Design and Construction

The pilot tunnel was designed as a crown drift along the proposed southbound tunnel. Plans called for five instrumented cross-sections within the tunnel to monitor ground deformations and groundwater pressures, as shown in Figure 2. Monitoring of ground deformations was also planned at both portals. It was intended that the data collected from these instrumented sections would enable designers to more accurately predict ground responses during main bore construction.

The strength, jointing, and engineering characteristics of the various geologic formations along the tunnel were known to vary widely. The pilot tunnel design, therefore, was developed with four different categories of support as shown in Figures 3 and 4. Category 1, crown bolts, was used in rock with rough, widely spaced, non-continuous joints. Category 2, which used a closer bolt spacing and two inches of shotcrete, was used in rock with more closely spaced and irregular planar joints. Category 3 added reinforcing to the shotcrete and was used where the rock consisted of small and poorly interlocked blocks and wedges. Category 4, consisting of structural steel support and timber blocking, was used in both portal areas and in areas of inadequate bolt anchorage and soft ground. In all cases, an attempt was made to leave as much of the rock exposed as was possible and prudent.

The pilot tunnel excavation was carried out by Crosstown Contractors, a joint venture based in Washington, D. C., between December 9, 1985, and December 5, 1986. Excavation was by drill and blast with all drilling done by hand-held jackleg drills. Small rail equipment was used for muck handling. Resin grouted rockbolts were also installed using jackleg drills. Dry-mix shotcrete, with powdered

Information Obtained

accelerator added just prior to application, was used for crown support where required. A thirty-minute window was specified between the excavation and the support cycles to allow geologic mapping of each round.

The specifications called for the contractor to propose the support category based on his observation of the excavated round. This recommendation was assessed using the results of the geologic mapping and the experience of the on-site inspector. Usually, agreement was reached immediately and support installed. In those rounds where differences did occur, the contractor usually installed the more conservative support for safety.

Pilot Tunnel Costs

Ultimately the cost of the pilot tunnel must be weighed against the likely savings in main tunnel construction costs. Estimated pilot tunnel costs are summarized as follows:

Design	350,000
Construction contract	2,800,000
Management and inspection of construction	350,000
Mapping and instrumentation	700,000
Credit for ease of ventilation and reduced initial water inflows	(700,000)
Total	\$3,500,000

To be beneficial, pilot tunnels must provide information that results in the total project cost being lower than it would have been without the pilot tunnel. In tunneling, experience has shown that costs are strongly affected by uncertainties relating to geologic conditions and ground behavior.^{4,5} Uncertainties often lead to an overly conservative design and bids with high contingencies. Tunneling construction costs often increase rapidly when unexpected ground conditions lead to major delays in excavation or to other substantial changes in the contractor's operations. Thus, a major benefit of the pilot tunnel comes from reducing uncertainties relating to ground conditions.

The actual construction costs for the main tunnel excavation and lining project will not be known for some time. At the present stage of design, however, the total pilot tunnel expenditures are estimated to be less than seven percent of these costs. This is significantly less than the anticipated geotechnically-related claims, if only a minimal investigation had been performed.⁶

The excavation of the pilot bore has allowed direct physical access to the geology along the alignment of the south-bound tunnel. This access has provided verification of the overall geologic profile and direct observation of the stratigraphy; the major structural features such as faults and shears; the weathering and alteration; the orientation, roughness, persistence, spacing, and infilling of typical bedding and other joint sets; and the intact strength of the various units. The pilot tunnel offers substantial advantages over coring and outcrop mapping for collecting this type of information.

Significant detail has also been observed and documented at specific locations along the tunnel such as zones of relatively thin but soft and soil-like material within the sandstone formations and individual joints with very high initial groundwater inflows. Actual measurements of groundwater pressures around the tunnel and inflows to the tunnel have been made. This information has been used to predict inflows to the ultimate twin tunnel configuration. Observation and measurement of ground deformation and ground response to tunneling activities have allowed predictions of ground behavior during main tunnel construction to be made with some confidence.

Very specific commitments were made in the environmental impact statement concerning the quality of water which would flow from the tunnel both during and after construction. Some of the shales in Cumberland Mountain are acid producers and the groundwater in some of the sandstones is quite high in iron and manganese. Water chemistry was investigated in and immediately around the pilot tunnel shortly after its completion. A main tunnel waterproofing and drainage system was devised which will minimize the tunnels' impact on surrounding groundwater and surface drainage water quality.

Detailed records of advance rates, powder consumption, overbreak, support requirements and other construction activities were maintained throughout the pilot bore project. This information was kept on a round by round basis and has been compiled with the geologic mapping. It will be made available to potential bidders for the main tunnels to assist them in the preparation of bids in the hope that more accurate bidding with less contingencies will result.⁷

Because of the potential for encountering methane when tunneling through coal

seams below the water table and through the Chattanooga shale, the entire pilot bore was excavated using permissible mining techniques. No measurable methane was encountered during the project despite significant monitoring efforts. Although monitoring will still be necessary, the permissible mining requirements will not have to be used for the main tunnel project. The contractor for the pilot bore has estimated that the additional cost of the permissible mining requirements represented at least fifteen percent of the cost for the pilot bore. A similar savings by removing these requirements from the main tunnel project would be nearly three times the total cost of the pilot bore project.

Solution features varying from small conduits to large caves were encountered in the limestone units. Some of these caves have openings and breakdown rooms with lateral and vertical dimensions which exceed sixty feet. These caves extend to hundreds of feet either side of the tunnel alignment and in at least one case, to hundreds of feet above the tunnel grade. The pilot bore excavation has provided physical access to many of these solution features, and optical surveys have been conducted to determine precisely how the twin tunnels will intersect these caves. Exploration has also disclosed that while some of the caves are open, some are mud-filled and at least one carries large quantities of water across the tunnel alignment. Detailed designs for tunnel support and water handling have been developed for the known caves which will intersect the tunnels.

Differences between the geologic profile predicted from surface mapping and that within the pilot bore were apparent, especially in the vicinity of the Pennsylvanian/Mississippian contact. Resolution of these differences was considered an academic issue since the rock mass characteristics could be observed and their precise geologic history would not affect the tunnel design. Nevertheless, the opportunity to observe a cross-section through Cumberland Mountain was considered too valuable to pass up and the completed pilot tunnel was made available to Eastern Kentucky University for geologic exploration and mapping. At least one Master of Science thesis resulted, which appears to resolve the differences between the surface mapping and the observed stratigraphy.⁸

Conclusions

The Cumberland Gap pilot tunnel has exposed ground conditions along the main tunnel alignment so that geologists, engineers, and contractors are able to observe first hand the conditions important to them in their respective roles on the project. Each is also able to make informed judgments about main tunnel conditions from what he observes in the pilot tunnel.

The pilot tunnel had made it possible to directly observe the ground behavior and support performance in a small tunnel driven through the same geologic conditions that the main tunnels will encounter. If the scale difference of the tunnels is taken into account, the pilot tunnel observations can be used to provide considerable insight into main tunnel ground behavior. As a result, main tunnel support systems and excavation methods can be selected with reasonable confidence, and consequently, cost overruns resulting from major changes in the contractor's operations can be minimized.

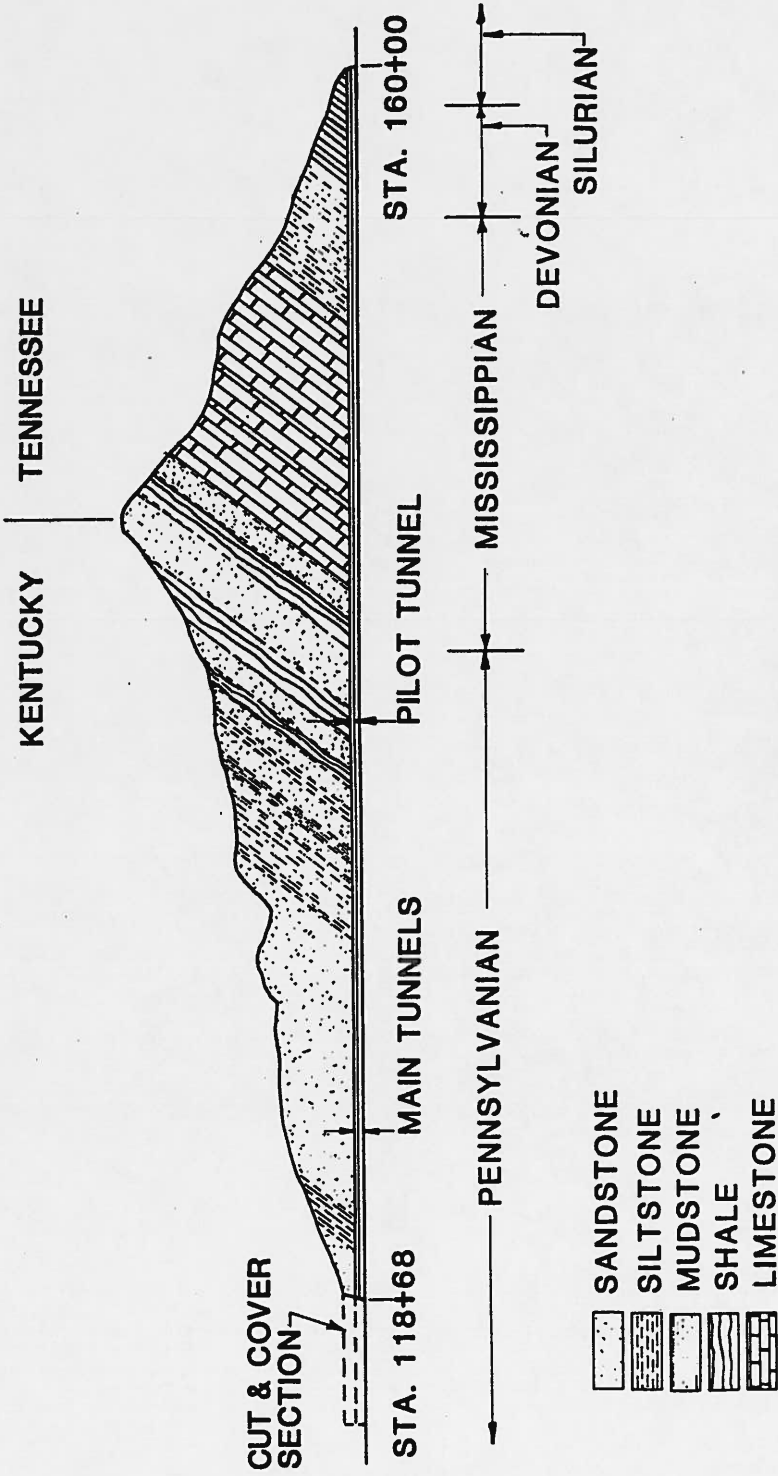
In short, the pilot tunnel has provided information that reduces geologic uncertainties along the main tunnel alignment to a practical minimum. Historically, unexpected geologic conditions are a common cause of major cost overruns on tunneling projects. Therefore, there is a substantial benefit in minimizing these uncertainties on any project of this size.

The total cost of the pilot bore is much less than ten percent of the estimated cost of the main tunnel excavation and lining project. Savings of many times the cost of the pilot bore are anticipated. It has allowed the development of a more efficient design, it will provide more accurate bidding with less contingencies, and it will allow the selection of the most appropriate excavation techniques and equipment. A peripheral benefit, more accurate knowledge of the stratigraphy in this geologic section, has also been realized.

References

1. G.H. Collison, "Geotechnical Data Collection for Design of the Cumberland Gap Pilot Bore," 34th Annual Highway Geology Symposium, Atlanta, Georgia, 1983.
2. W.F. Brumund, P.M. Ingram, and R.W. Humpheries, "Case Histories of the Explorations of Two Tunnel Sites Utilizing Horizontal Coring and Other Techniques," Association of Engineering Geologists, Raleigh, North Carolina, 1985.
3. J.C. Leifer, "Combination Probe - A New Instrument for Subsurface Exploration," Public Roads, Vol. 48, No.1, 1984
4. U.S. National Committee on Tunneling Technology, Recommendations on Better Contracting for Underground Construction, National Academy of Sciences, Washington, D.C., 1976.
5. U.S. National Committee on Tunneling Technology, Geotechnical Site Investigations for Underground Projects, Vols. 1 and 2, National Academy Press, Washington, D.C., 1984.
6. W.R. Sullivan and R. M. Leary, "Benefits of the Pilot Tunnel on the Cumberland Gap Highway Project," Proceedings of the Rapid Excavation and Tunneling Conference, New Orleans, La., 1987.

7. R. W. Humpheries, K.H. Rippere, W. R. Sullivan, and J.J.C. Paul, The Cumberland Gap Pilot Tunnel and Geotechnical Recommendations for the Main Tunnels, Report submitted to the Federal Highway Administration, Vols. 1,2,3,and 4, Golder Associates, Atlanta, Ga.,1987.
8. J.D. Vanover, Stratigraphy of Pilot Tunnel through Cumberland Mountain (Kentucky-Tennessee), Master of Science Thesis, Department of Geology, Eastern Kentucky University, Richmond, Kentucky, 1989.



GEOLOGIC PROFILE
FIGURE 1

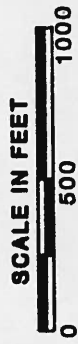
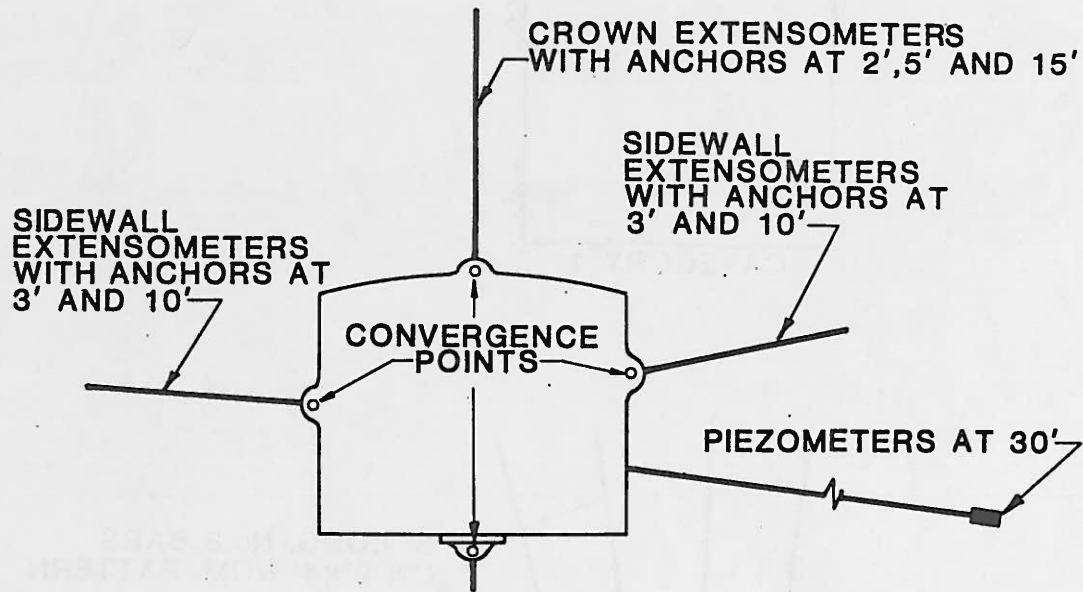


Figure 1. Geologic Profile

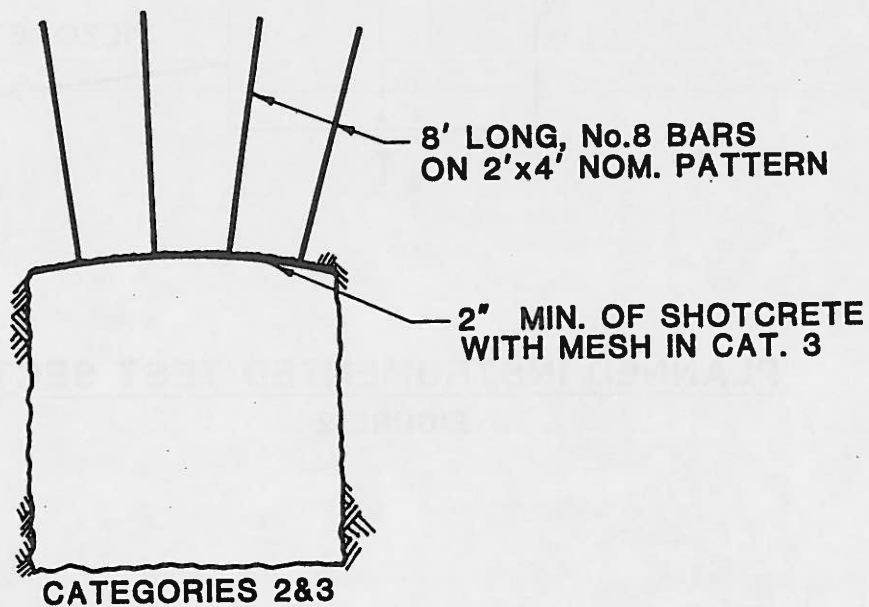
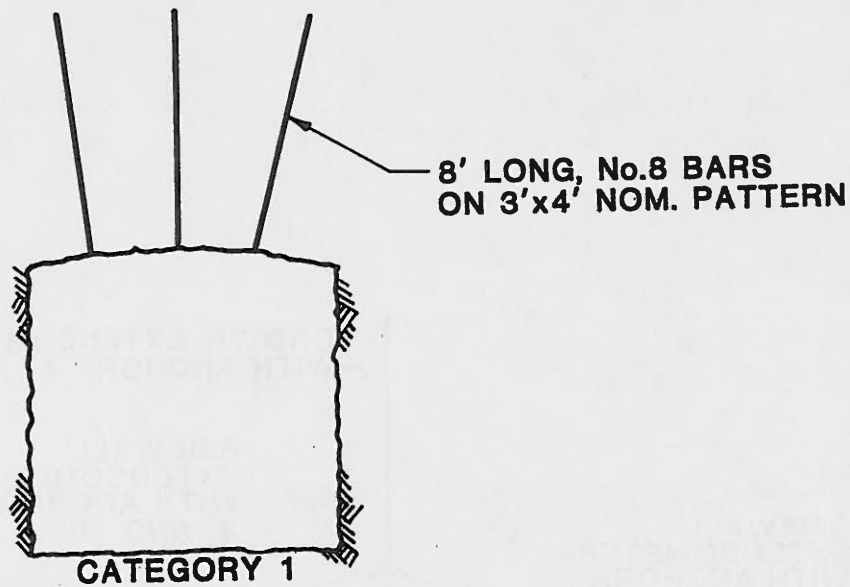


PLANNED INSTRUMENTED TEST SECTION

FIGURE 2



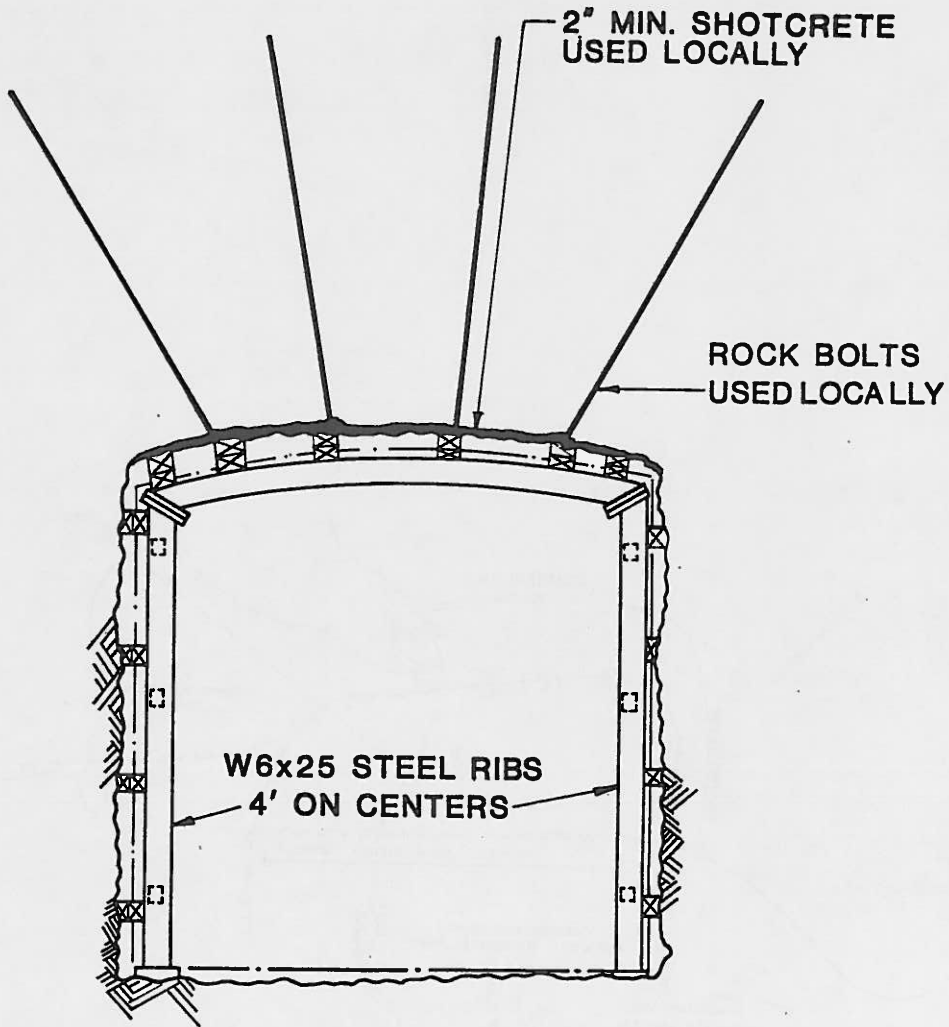
Figure 2. Planned Instrumented Test Section



PILOT TUNNEL SUPPORT-CATEGORIES 1,2 & 3
FIGURE 3



Figure 3. Pilot Tunnel Support - Categories 1,2 & 3



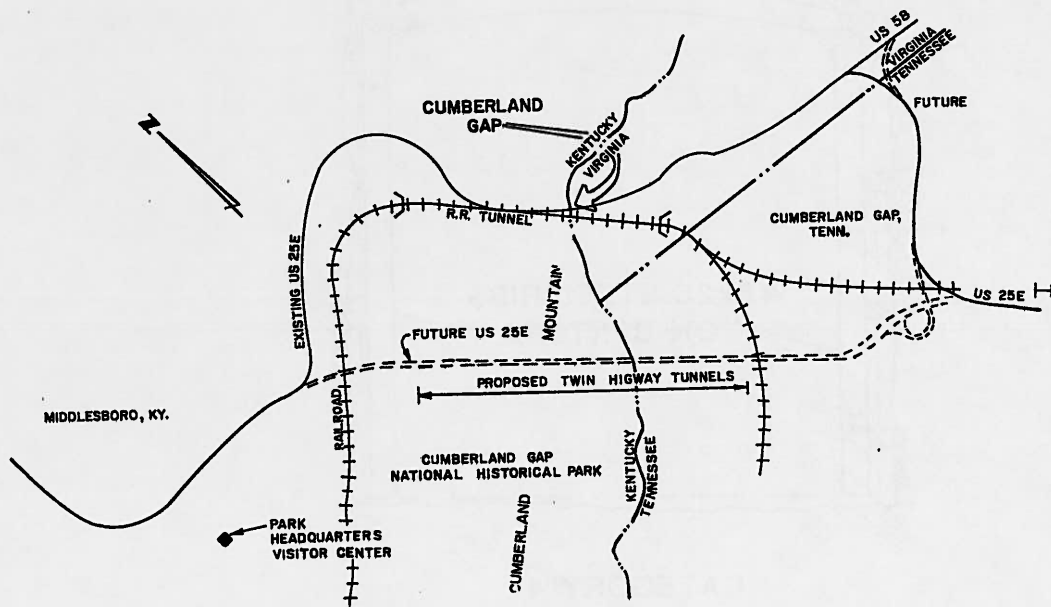
CATEGORY 4

PILOT TUNNEL SUPPORT-CATEGORY 4

FIGURE 4



Figure 4. Pilot Tunnel Support - Category 4



PROJECT LAYOUT



Figure 5. Overall Project Layout

PRE-GROUTING OF THE NORTH SHORE TUNNEL
MILWAUKEE, WISCONSIN

by

James P. Rose, Roger C. Ilsley
Senior Geologist and Principal Engineering Geologist
STS Consultants, Ltd.
Milwaukee, Wisconsin

Abstract. The thirty-two (32) foot diameter North Shore Tunnel, which was being driven through the Upper Silurian Waubakee Dolomite, encountered stability and groundwater problems which stopped the mining of the tunnel. Before mining could be resumed, the support system and water handling system for the tunnel were redesigned and pre-grouting of the tunnel envelope for a distance of one mile was performed. The pre-grouting was accomplished by drilling angle grout holes from the ground surface. Depths of the ground holes ranged from 300 to 450 feet and diameters ranged from BQ to NQ. Type III Portland cement grout mixes were used with water cement ratios ranging from 7:1 to 1:1 (weight basis), to grout rock types ranging in composition from thinly bedded dolomite to highly solutioned limestone, calcarenite, and fault gouge. The grouting program was successful in that relatively small additional groundwater inflows were encountered when mining was resumed.

Introduction

The North Shore Phase IA Tunnel is part of the northern reach of the storage and conveyance tunnels of the Milwaukee Pollution Abatement Program (Fig. 1). The tunnels will be used to store and convey combined storm-water and sewage flows during periods of heavy rainfall, thus preventing their discharge to surface water courses. The tunnel is 28,000 feet long, 32.3 feet in mined diameter and varies between 260 and 310 feet deep in bedrock.

Geologic setting

The geologic investigation for the North Shore Tunnel included a series of exploratory borings. An initial series of borings were sampled at 2-1/2 foot intervals to the top rock and cored continuously to obtain general bed-rock stratigraphic and structural information along the alignment. After the alignment and shaft locations were determined, a series of continuously sampled vertical borings was performed at perspective shaft locations, and alignment borings sampled in the prospective tunnel envelope were performed on or near the tunnel alignment. The alignment borings were vertical borings, drilled on land as close as possible to the alignment where it was beneath the Milwaukee River.

From the top down, rock and soil units in the project area consist of recent alluvial deposits; Pleistocene glacial deposits; the Devonian Milwaukee Formation which consists of dolomite, and mudstone; the Devonian Thiensville Formation, consisting of solutioned limestone, dolomite and calcarenite; Silurian Waubakee Dolomite, and the Silurian Racine Dolomite. Paleozoic carbonate and clastic sedimentary rocks continue below the Racine Formation. This discussion will concentrate on the three formations (Fig. 2) which most directly impact the tunnel construction: the Thiensville Formation, the Waubakee Formation, and the Racine Formation. The engineering properties of these formations are presented in Table 1.

The Thiensville Formation consists of predominantly carbonate rocks of highly variable composition. Rock types include dolomite, limestone, calcarenite and intra-formational breccia. A few thin layers of gray to black shale also occur within the Thiensville Formation. The Thiensville Formation has been highly solutioned with the solutioning concentrated along bedding surfaces and other discontinuities. Water pressure tests on the Thiensville Formation yielded hydraulic conductivities (permeabilities) during the exploratory program ranging from greater than 1×10^{-3} cm/sec to 1×10^{-4} cm/sec. Shaft sinking through the Thiensville Formation is a very difficult undertaking and requires extensive grouting in order to be successful. Vertical and angled drill holes through the Thiensville are unstable and require casing or some other stabilization.

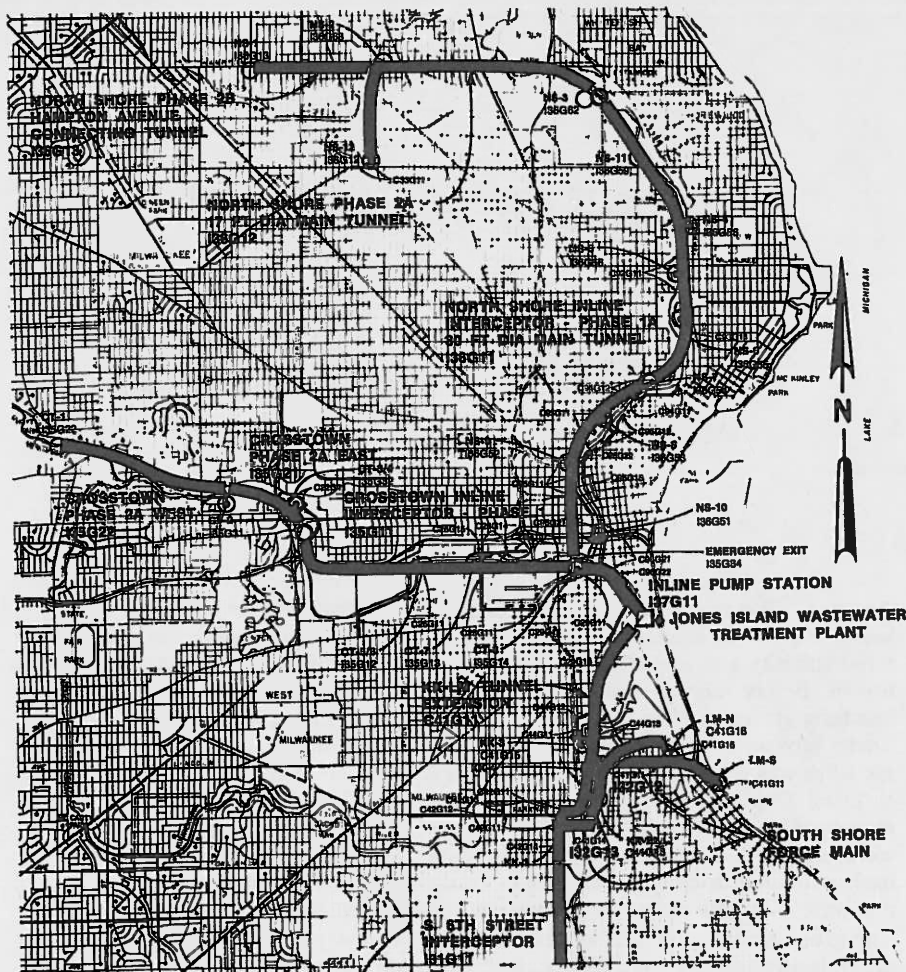


FIGURE 1 MILWAUKEE DEEP TUNNEL SYSTEM

Formation	Generalized Description		Formation Thickness (ft)	RQD (%)	Bulk Density (pcf)	Unconfined Compressive Strength (psi)	Tangent Modulus (psi x 10 ⁶)	Indirect Tensile Strength (psi)
Thiensville	Dolomite & Limestone	Max	85.7	100.0	164.6	19,500	8.00	2,235
		Min	53.8	0.0	132.7	205	0.02	191
		Avg	64.4	37.2	154.5	8,042	3.92	953
		N	17	202	12	13	13	8
Waubakee	Dolomite	Max	99.3	100.0	173.7	25,150	11.63	3,009
		Min	30.0	0.0	148.6	5,840	7.18	573
		Avg	78.1	55.5	163.3	16,992	8.78	1,449
		N	18	273	10	9	10	9
Racine	Dolomite	Max	162.5	100.0	173.7	42,800	10.10	2,870
		Min	123.0	11.7	150.4	13,700	3.79	748
		Avg	145.9	93.4	164.5	26,401	6.54	1,841
		N	10	1,039	50	42	42	40

- 1) Data given only for occurrences of full formation penetration in exploration borings.
- 2) Calculated from consecutive 5-foot increments of borehole beginning at starting depth of coring.
- 3) Tangent elastic modulus at 50% of peak stress.
- 4) Standard test; failure was through the material.
- 5) Failure was along a bedding plane or parting.

TABLE 1 ENGINEERING PROPERTIES OF ROCK UNITS

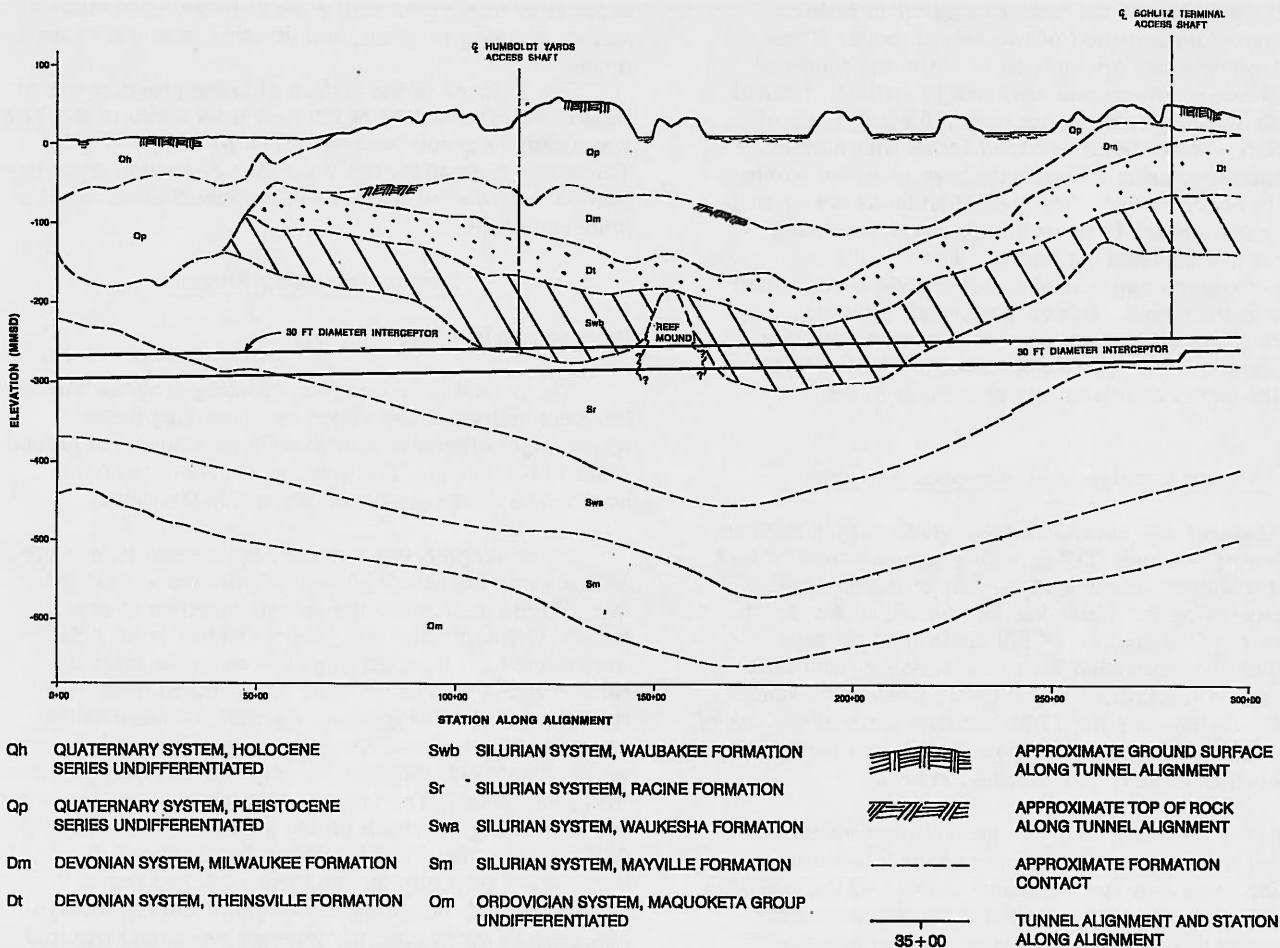


FIGURE 2 NORTH SHORE PHASE IA GEOLOGIC PROFILE

The Waubakee Formation consists of fine grained white to gray dolomite with bed thickness varying from less than 2 inches to 1.5 feet. The Waubakee Formation tends to separate readily along bedding planes. Unweathered Waubakee Formation is generally non-vuggy, but weathered exposures have numerous small angular vugs 0.05 to 0.10 inches in size. The Waubakee Formation is a basin margin deposit deposited in a highly saline sea which occupied the Michigan Basin during late Silurian time. Based on information from exploration borings, it was believed that the Waubakee Formation was horizontally bedded except on reef flanks where dips up to 30 degrees were measured. However, based on exposures of the Waubakee Formation in the North Shore Tunnel, the Waubakee Formation has numerous small scale folds. These folds appear to be related to slumping immediately after deposition and before lithification, and have approximate amplitudes of 6 inches to tens of feet and approximate spacing of 1 to tens of feet. Water pressure testing during the exploratory program indicated that the Waubakee Formation had permeabilities ranging from 5×10^{-4} to 1×10^{-7} cm/sec. Experiences with the mining through the Waubakee Formation are detailed in the next section.

The Racine Formation consists of fine grained gray dolomite, the predominantly lithologic type being inter-reef dolomite consisting of medium to thick bedded dolomite with thin shale partings between beds. The Racine Formation is generally impermeable except along solutioned joints and bedding planes. Water pressure testing of the

exploratory core holes indicated permeability ranging from 5×10^{-4} to 1×10^{-7} cm/sec. The Racine Formation generally requires little support during tunneling. In the Milwaukee area tunnels up to 32 feet in diameter have been mined with only crown rock dowels for support. The Racine Formation was deposited at the edge of the Michigan Basin and contains numerous accumulations of biologically deposited dolomite called reefs. Reef deposits in the Racine Formation are generally more porous than the inter-reefal beds. Before mining reached the reef, there was some concern that the reef talus deposits on the flanks of the reef may be poorly cemented and unstable, and that the reef deposits themselves may have higher permeabilities than the inter-reefal beds. However, when the tunnel reached the reef, the talus deposits were of limited extent and were generally well cemented, and there was little additional permeability in the reef itself.

Bedrock in the project area is folded to form a gentle syncline with its axis approximately perpendicular to the alignment of the North Shore Tunnel. Thus, as the North Shore Tunnel was mined southward from the access shaft, it climbed progressively upwards through the stratigraphic section into the Waubakee Formation. Based on exploratory borings, the Waubakee cover between the tunnel crown and the base of the Thiensville Formation was estimated to be a minimum of 15 to 20 feet. However, based upon information obtained during the surface grouting program, this distance was found to be as little as 8 feet at the syncline axis.

Discontinuities in the bedrock other than bedding plane separations consisted of two sets of joints. These sets had approximate orientations of northwest/southeast, and northeast/southwest and were nearly vertical. Normal and strike slip displacement has occurred along some of these joints. Minor faults and fault forms with normal displacements of up to 20 feet have been observed within the North Shore Tunnel. The individual faults are up to 1 foot in width and are filled with fault gouge consisting of breccia in a clay and silt matrix. These faults are capable of transmitting several hundred gallons per minute of water to the tunnel. During the surface grouting program, it was discovered that one of these faults with an approximately vertical displacement of 10 feet, intercepted the tunnel approximately at the axis of the syncline.

Mining Problems and Remedial Measures

The tunnel was excavated down-gradient by a Robbins tunnel boring machine (TBM), with a crown pattern of rock dowels for support, starting at the Schlitz Access Shaft. Upon penetrating the Waubakee Formation, roof support problems required erection of full circle steel rib supports. Further penetration led to increasing groundwater inflows which reached 3,500 gpm at approximately station 212+00. At this time the TBM was shut down as the risk of flooding the heading, due to a power outage or further inrush of groundwater, had become too high.

The unexpected quantity of groundwater inflow was postulated to be due to the thinning of the Waubakee Formation cover between the tunnel crown and the overlying Thiensville Formation (Fig. 2) and the hydraulic connection between this formation (a major aquifer) and the tunnel being greater than anticipated. Remedial measures included upgrading of the tunnel pumping system to a

capacity of 6,000 gpm with a diesel backup generator, a surface grouting program, and grouting from within the tunnel.

The purpose of the surface grouting program was to reduce the permeability of the rock mass ahead of the TBM, specifically to grout the lower 10 to 20 feet of Thiensville Formation, the Waubakee Formation cover between the formation contact and the tunnel crown and the tunnel envelope.

Surface Grouting Program

Purpose and Planning

The overall purpose of the grouting program was to intercept and grout any major water carrying features which might otherwise contribute large amounts of groundwater to the tunnel. The grouting program was not intended to create dry conditions within the tunnel.

To accomplish this purpose, angle grout holes were drilled along the tunnel alignment from the ground surface. These intercepted the tunnel centerline at approximately 37.5 foot spacings. The individual grout holes were fanned out from drilling stations on the ground surface with 4 to 8 grout holes being drilled from each station as shown in Figure 3. Because the tunnel alignment was mostly under the Milwaukee River, or inaccessible wooded parkland, this was the only way to reach the tunnel with grout holes. The procedure also minimized the impact of the grouting operation on the ground surface, reduced drill rig setup time, and increased the likelihood of intercepting vertical joints and faults. It had the disadvantage of increasing the length of drilling footage necessary to accomplish the grouting work, and required the use of more expensive drilling equipment and more costly drilling techniques.

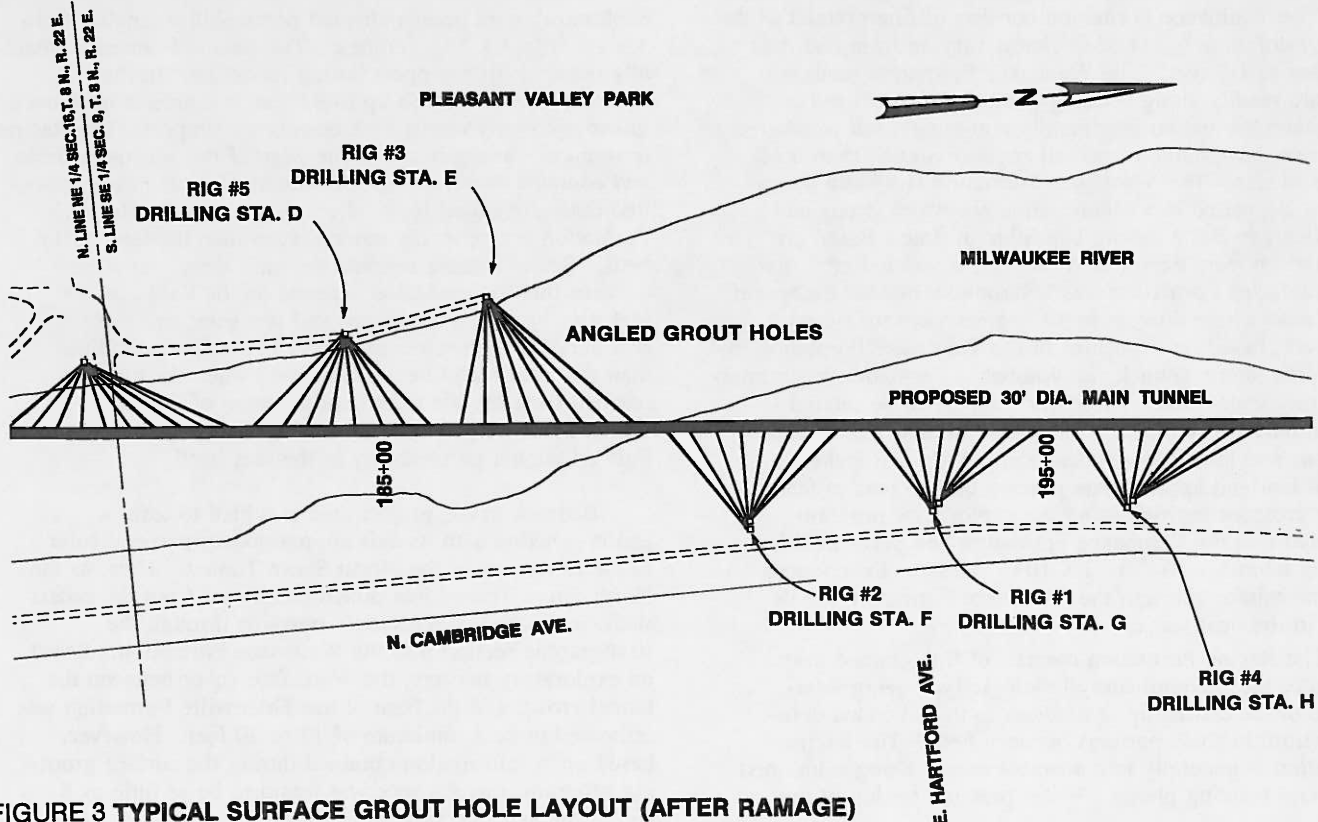


FIGURE 3 TYPICAL SURFACE GROUT HOLE LAYOUT (AFTER RAMAGE)

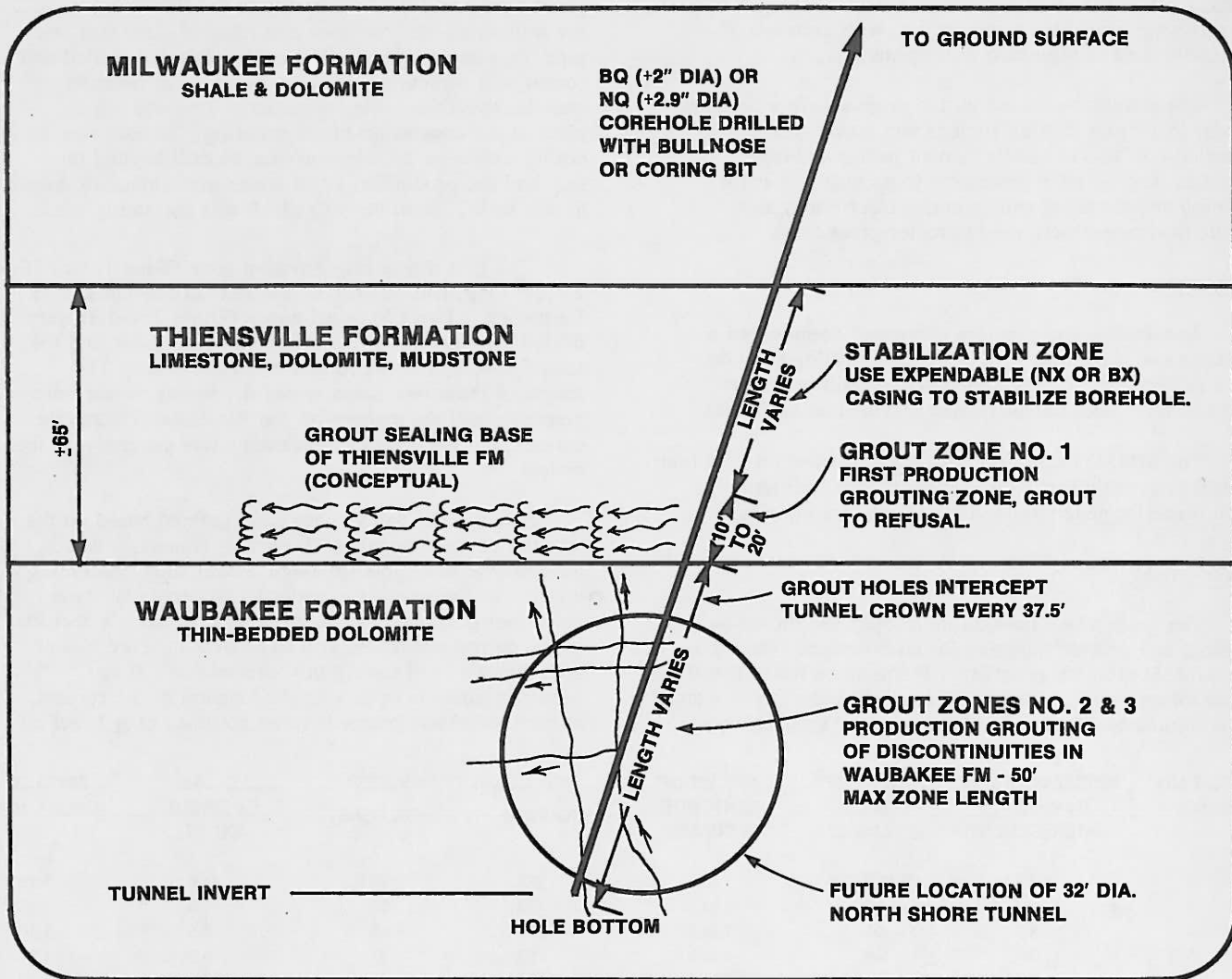


FIGURE 4 CONCEPTUAL ILLUSTRATION OF GROUTING ZONES

The grouting program was designed (Fig. 4) to seal the base of the Thiensville Formation 10 to 20 feet (Zone 1) above the contact, and to grout any major joints or faults in the Waubakee Formation (Zones 2 and 3) which formed a direct connection between the tunnel and the Thiensville Formation. Because of the expected potential problem with the reef and the Racine Formation, the grouting program was extended southward to include the reef.

Grout Hole Layout

Initial grout hole orientations and drilling stations were laid out based on accessibility considerations and on negotiations between the Milwaukee Metropolitan Sewerage District and the Milwaukee County Park Commission which owned most of the land where the drilling was performed. The locations of individual drilling stations were adjusted in the field by the drilling contractor and by the MMSD inspection staff and the stake was set at each location. The stake location and elevation was then surveyed by MMSD surveyors who then computed horizontal bearings and vertical angles to each target tunnel intercept station. After performing these computations, the surveyors returned to the field, provided a table of vertical drilling angles and horizontal bearings to the field inspection staff, and established an array of stakes at each

drilling station in order to allow the drill rig to be aligned. Two stakes were provided for each hole and a transit was used by the field inspection staff to align the drill rig.

Equipment Types

Six drilling rigs were used on the surface grouting program. These consisted of four skid mounted Boyles 37 coring rigs, and a Longyear 38 and a Longyear 44, both truck mounted rigs. The skid rigs were used in restricted areas.

Wire line coring equipment was used to advance the grout holes. The grout holes were drilled to top of rock with an HQ casing advancer, which is a removable tri-cone bit. The grout hole was then continued through the Milwaukee Formation and the upper part of the Thiensville Formation with NQ size coring equipment using either a coring bit and a double tube core barrel, or a bull-nose (non-coring) bit. NQ or BQ size coring equipment was used to drill the grout zones.

Grout mixers consisted of Chemgrout trailer mounted mixers. These were capable of mixing 40 to 60 gallons batches of grout, used Vortex mixers to mix the grout, and Moyno pumps to inject the grout. Bagged Portland Type III

cement rather than bulk cement was used. Four grout mixers were available on the project, with each mixer generally used to serve two drilling stations.

Other equipment used on the project were a backhoe loader to prepare drilling stations and access routes, two wheeled fork lifts to handle cement pallets and other supplies, and portable generators to provide power for lighting and electrical equipment. Fisher Porter electronic flowmeters were used to meter grout flows.

Personnel

The drilling and grouting contractor operated on a 24-hour two shift basis with a 3 person drilling crew on each drilling rig. A shift supervisor, a drilling foreman and full time mechanic were also provided on each shift.

The MMSD inspection personnel operated on a 24 hour, 3 shift basis with 3 to 4 inspectors on each shift and each inspector generally monitoring two drilling rigs.

Grout Mixes

The grout mixes used on the project are shown on Table 2 and provide the basis for all mixes used throughout MMSD grouting programs. It should be noted that the grout mixes were established on a weight basis rather than on a volume basis, as is more common for grout mixes.

GROUT MIX ID NO.	GROUT MIX RATIO BY WEIGHT WATER: CEMENT	APP. WT OF CEMENT (POUNDS)	APP. WT OF BENTONITE (POUNDS)	APP. QUANTITY OF WATER (POUNDS)	APP. QUANTITY OF WATER (GALLONS)	VOLUME OF GROUT (CU FT)	SP GR OF GROUT MIX
2	7:1	94 (1 BAG)	1 to 2	658	79	11.0	1.10
3	5:1	94	1 to 2	470	56	8.0	1.13
4	4:1	94	1 to 2	375	45	6.5	1.16
5	3:1	94	1 to 2	282	34	5.0	1.21
6	2:1	94	1 to 2	188	23	3.5	1.30
7	1.5:1	94	1 to 2	142	17	2.7	1.41
8	1:1	94	1 to 2	94	11	2.0	1.52
9	0.75:1	94	1 to 2	71	8.5	1.6	1.67
10	0.60:1	94	1 to 2	54	6.5	1.4	1.72

TABLE 2 GROUT MIXES

Procedures

Before grouting the tunnel envelope, it was necessary to advance the grout hole through the overburden, Milwaukee Formation, and the upper part of the Thiensville Formation. The grout holes were advanced through soil using NQ casing and casing advancers. The casing advancer bit was then removed, leaving the casing in place. Then the end of the casing was sealed with a bentonite slurry. Smaller sized coring equipment (NQ in size) was then used to drill through the Milwaukee Formation and the Upper Thiensville Formation. Extremely difficult drilling conditions were encountered in the Upper Thiensville Formation, with high drilling fluid losses and caving conditions. The Upper Thiensville Formation therefore required sealing and stabilization before the grout hole could be advanced. This was initially accomplished by using thick grout mixes to seal the formation. However in some holes, hundred of bags of grout were required for this purpose, and excessive amounts of time were required to allow the grout to set before the borehole could be advanced. Procedures were therefore changed. After the

grout hole was drilled to the top of the first grout zone, the drill string was removed and relatively low cost steel pipe the same size as the NQ drill rods was installed and sealed with cement grout in the borehole in order to stabilize the Thiensville Formation. This was left in place at the conclusion of the grouting. Smaller size coring rods were therefore needed to drill beyond this step and the production grout zones were therefore reduced in size to BQ, from the NQ which was previously used.

The first production grouting zone (Zone 1) was 10 to 20 feet long, and constituted the base of the Thiensville Formation. Two additional zones (Zones 2 and 3) were drilled through the Upper Waubakee Formation and the tunnel envelope to the tunnel invert elevation. The length of these two zones varied depending on the hole geometry and the location of the Waubakee-Thiensville contact, but the length of each zone was generally 50 feet or less.

Because the grout zones were defined based on the location of the Waubakee-Thiensville contact, it was necessary to have good geologic information concerning the location of this contact in order to properly plan each individual grout hole. The grouting contractor's decision to use coring equipment and to recover the core was of great benefit in obtaining this information. It was generally possible to predict the location of the contact in each individual borehole to an accuracy of ± 2 feet by

using information obtained from previous grout holes. When less accurate information was available, such as in the situation when the first hole from a drilling station was drilled, a conservatively high estimate of the location of the contact was used in order to be sure to properly grout the base of the Thiensville Formation.

Because of the varying rock types in each zone, it was necessary to grout each of the zones in a different fashion. Generally thicker grout mixes were used in the Thiensville Formation, and when highly permeable fracture zones were encountered. Grout mixes were selected based on the concept that there was an optimal grout mix for each zone at which a maximum volume of grout could be injected in order to permit maximum grout penetration into the surrounding rock while minimizing shrinkage when the grout set. During the first 7 grout holes, it was determined that the 2:1 (weight basis) water to cement ratio grout was the optimum mix for this purpose. However, in some holes, based upon pressure build up considerations,

it was determined that thinner grout mixes should be used. Therefore a grouting procedure was developed whereby grouting commenced with 5:1 grout mix and proceeded in progressively thicker increments of a hundred cubic feet each until the 2:1 grout mix was reached. If resistance to grout injection developed on any particular mix, as evidenced by reductions in grout take rates and increases in grouting back pressure, grouting was continued with that mix until refusal was reached. Thus the response of each grout zone to progressively thicker grout mixes was the primary method used to select a grout mix for any particular zone.

It should be noted that after a grout zone had taken extremely high volumes of 2:1 grout, arbitrarily defined as 200 bags, the grout mix began to be again progressively thickened in 200 bag increments until the 1:1 grout mix was reached. At that point if the grout hole continued to take large quantities of 1:1 grout, a decision was made in the case of each individual grout zone on how to proceed with thickening the grout mix further. If the grout zone taking the large quantity of grout was within or very close to the tunnel envelope, grouting continued using the 1:1 mix without adding additional thickeners in order to inject as much grout was possible into the tunnel envelope. In other zones especially zones within the Thiensville Formation where the Thiensville Formation was 40 feet above the crown of the tunnel or more, thickeners such as sawdust, cellulose, and bentonite were added to eliminate grout take in that zone.

Water pressure testing did not play a major role in selecting mix thicknesses for grouting. A 10 minute water pressure test was performed on each zone before grouting primarily to determine whether or not to grout the zone. If the zone took 2 gallons per minute or more of water at maximum pressure during the Packer test, the zone was grouted. Later in the grouting process, this test also played a limited role in selecting the initial grout mix in Zone 1 at the base of the Thiensville Formation. If there was no pressure built up during water test, grouting commenced with a 3:1 mix, but if pressure built up to above 65 psi with a take of less than 15 gallons a minute, the 4:1 mix was used. The maximum pressure used in the

water test was also the maximum pressure used during the grouting. This pressure was determined in a relatively conservative fashion in order to prevent hydrofracturing of the rock formations using a maximum pressure equal to the sum of 0.5 psi/vertical foot of soil, and 1.25 psi/vertical foot of rock. In order to prevent a potential safety hazard due to bursting of grout lines, a maximum limit on the grout pressure of 300 psi at the ground surface was used.

This technique resulted in relatively thin grout mixes being used in the Waubakee Formation, and relatively thick mixes being used in the Thiensville Formation. Grouting in the Waubakee Formation typically went to refusal with the 3:1 or 4:1 mix. Grout holes in the Thiensville Formation typically went to refusal using the 2:1 or thicker mix. When the grouting program reached the reef area, it was found that even the thinnest mix typically used the 5:1 mix, immediately choked off the grout zone, so within the reef, grouting started using the 7:1 grout mix. The amount that each grout zone took was extremely variable, depending on the formation penetrated, the presence of faults and other high permeability zones, and the mix design used. In the following discussion, the grout take will be discussed on the basis of the number of bags of cement used. Table 3 shows grout takes from several typical holes along the alignment. Except for Hole No. 141, Zone 1 is in the Thiensville Formation and Zones 2 and 3 are in the Waubakee Formation. In Hole No. 141, the entire grouting zone was in the Racine Formation. As indicated in the grout quantities, Zone 1 in the Thiensville Formation generally took several hundred bags or more of grout. The zones of the Waubakee Formation took less but still significant amounts of grout. The maximum grout take in the project was Hole No. 40 located within a fault at the location of the lowest thickness of Waubakee cover between the tunnel crown and the Thiensville Formation and this zone took over 6000 bags of cement. The two upper grout zones which each took over 3,000 bags of cement, took approximately 72 hours each to grout to refusal.

HOLE NO.	TUNNEL STATION	GROUT ZONE	CEMENT (BAGS)	GROUT (m ³)	GROUT (FT ³)	HOLE NO.	TUNNEL STATION	GROUT ZONE	CEMENT (BAGS)	GROUT (m ³)	GROUT (FT ³)
88	202+50	1	408	43.7	1,560	40	185+00	1	3,174	195.2	6,970
	Waubakee	2	304	55.9	1,998		Waubakee	2	3,069	222.4	7,943
		3	587	51.9	1,853		(low cover)	3	399	47.9	1,710
TOTALS	HOLE: 88		1,299	151.5	5,411	TOTALS	HOLE: 40		6,642	465.5	16,623
74	197+75	1	810	73.6	2,627	111	165+50	1	485	48.1	1,719
	Waubakee	2	552	65.0	2,323		Waubakee	2	0	0	0
		3	700	105.0	3,752			3	6	0.3	12
TOTALS	HOLE: 74		2,062	243.6	8,702	TOTALS	HOLE: 111		491	48.4	1,731
48	188+00	1	743	66.8	2,385	141	154+50	1	1	0.2	8
	Waubakee	2	225	51.7	1,845		Reef Area	2	4	1.2	44
	(low cover)	3	345	67.0	2,394			3	7	1.1	41
TOTALS	HOLE: 48		1,343	185.5	6,624	TOTALS	HOLE: 141		12	2.5	93

NOTE: SEE FIGURE 4 FOR DESCRIPTION OF GROUT ZONES.

TABLE 3 TYPICAL GROUT HOLE DATA (AFTER RAMAGE)

Upon completion of grouting in each zone, the grout was allowed to set for 2 to 6 hours depending on the take amount. A sample of the last mix injected was taken and used by the inspector to determine when the set grout in each zone be drilled out. At the completion of grouting in each hole, a tremie pipe or a plastic hose was placed in the hole and the hole was backfilled from bottom to top with 1:1 grout in order to prevent potentially dangerous water inflows within the tunnel when it intercepted the grout hole.

The most severe rock and groundwater conditions were encountered at tunnel station 185+00 corresponding to grout hole No. 40. At that location, Waubakee cover over the tunnel crown was only approximately 8 feet, and zone of highly fractured and faulted rock was present along the syncline axis. In that zone for approximately a 500 foot section, additional split space grout holes were performed. As described in Ramage et al., horizontal small diameter grout holes were also extended from the tunnel face when the tunnel reached the area so that the entire zone was covered by three complete phases of grouting. These were the initial surface grout holes on 37-1/2 foot spacings, the split spaced surface grout holes, and the horizontal grout hole arrays from the tunnel heading. The total quantities of surface grouting activities are presented in Table 4.

The results of the grouting program are also described in the Ramage et al. and summarized below. Groundwater inflows at the time mining was halted was approximately 3,500 gallons per minute. At the time mining resumed, this had been reduced to approximately 2,000 gallons per minute. This was primarily the result of reductions in head within the bedrock aquifers during the shutdown period, and grouting from within the tunnel. However, grout from the two surface grout holes closest to the tunnel was observed to be flowing into the tunnel face, and resulted in approximately 200 gallons per minute reduction in groundwater inflows at the tunnel face.

When mining was resumed, there was no net increase in groundwater inflows to the tunnel until the low cover area was reached approximate Station 190+00. At that station groundwater inflows increased to approximately 2,800 gallons per minute, and increased further to approximately 3,400 gallons per minute through the low cover zone (Fig. 5). The surface grouting thus appeared to be nearly totally effective except in the worst area and partially effective through the low cover area. Ramage et al. contains an evaluation of the effectiveness of the water inflow measures in which he uses the average increase in inflow per hundred feet of tunnel. Before the mining was stopped, the water inflows were approximately 200 gallons per minute per hundred feet of tunnel. In the area of the surface grouting, which initially were expected to have much worse groundwater conditions, after grouting inflows were 40 gallons per minute per hundred feet of tunnel.

DRILLING FOOTAGE Angle from vertical	QUANTITIES		QUANTITIES	
	(m)	(ft)	(m)	(ft)
0-10 degrees	778	2,554	Redrilling	3,120 10,238
10-20 degrees	2,173	7,128	Total Holes	150
20-30 degrees	7,086	23,247	Hole Connections	532
30-40 degrees	4,997	16,394	Cement (bags)	95,631
40-50 degrees	1,593	5,228	Bentonite (bags)	1,477
TOTAL DRILLING	16,715	54,841	Grout Volume	9,489 (m ³)/335,180 (ft ³)

TABLE 4 TOTAL QUANTITIES - SURFACE GROUTING PROGRAM (AFTER RAMAGE)

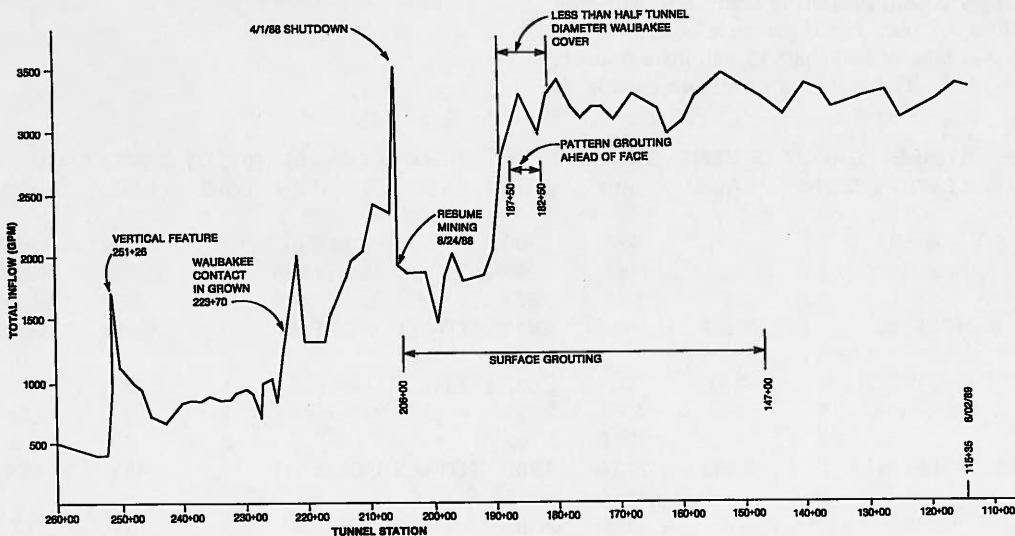


FIGURE 5 WATER INFLOW PHASE IA TUNNEL (AFTER RAMAGE)

Acknowledgments

John Ramage, CH2M Hill, Inc. was responsible for conceptual design and implementation of the surface grouting program. Other key individuals who made important contributions in developing and refining the grouting

concepts in the field included: Mitchell Daniels, Joe Halaburda, and Gerald Martin of the Lane Northwest Company, and Steve Fradkin, Robert Cannestra, Paul Botcher, and Charlie Manderfield, and Tom O'Brien, of the MMSD Program Management Office staff.

MECHANICAL PRE-CUTTING AS A TUNNELING TECHNIQUE

by

Robert Longelin, SIPREMEC, France
Yves Le Goer, SIPREMEC, France, and
E. van Walsum, Consulting Civil Engineer,
Montreal, Quebec, Canada

Abstract. The cutting of rocks by mechanical means is extensively used, world wide, in the mining of minerals such as coal and salt. Trenches for cables have been cut mechanically through limestone and shale.

Mechanical cutting is now also used, with excellent results, for the excavation of tunnels and caverns to specific cross sections and with minimal environmental impact. The technique is referred to as "Mechanical pre-cutting" and can be applied in cohesive soils, in non-cohesive soils and in medium-hard to soft rocks.

The paper describes the technique, its application in the past and possible applications in the future.

Introduction

The excavation of large underground openings in cohesive and non-cohesive soils and in soft to medium hard rocks is often complicated by excessive surface settlements, by overbreak and by environmental concerns such as objectionable noise and vibrations. Mechanical pre-cutting, used as a tunneling technique, can overcome all of these concerns economically and under a wide variety of conditions.

Mechanical cutting is used extensively in the mining of the softer rocks such as salt and coal and in the cutting of trenches through limestone, shale and similar ground for cable laying and such purposes. When applied to tunneling, the technique is referred to as "Mechanical Pre-cutting" for reasons which will become clear. A typical pre-cutting machine for a double track railway tunnel is shown in Figure 1. The machine consists of:

- a cutter, which resembles an over-sized chain saw,
- a rack and pinion track in the shape of the tunnel profile to be cut,
- a portal frame which gives the track its structural stability,
- a mechanism to advance or retract the machine along the length of the tunnel.

While the design and manufacture of the cutting teeth changes with the type of ground to be cut, the same pre-cutting machine can serve in cohesive and non-cohesive soils as well as in rock. The tunnel construction method for each of the three major soils types mentioned is however quite distinct.

Mechanical pre-cutting under varying ground conditions

In cohesive soils: Mechanical Pre-cutting with Pre-lining.

When ground can be excavated by mechanical shovels and without blasting, stability of the underground opening becomes often a problem. Mechanical pre-cutting provides here a unique solution for the stabilization of the tunnel opening. As the peripheral slot along the tunnel profile is being cut ahead of the face, see Figure 2, the slot is filled with sprayed concrete after the cutter has passed. In this way, a concrete lining is in place before the ground under the lining is excavated. This pre-lining assures the safety of the tunneling operation and it almost eliminates surface settlements. With the pre-lining in place, tunnel excavation can proceed full-face with economically sized equipment. The width of the slot may

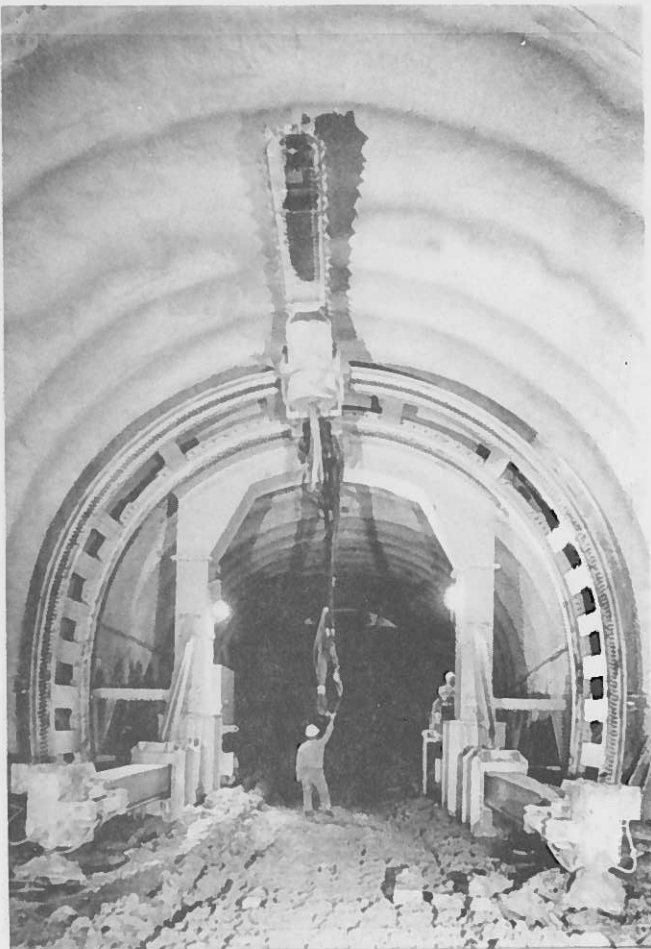


Figure 1. Typical pre-cutting machine, retracted from the face, in a double track railway tunnel.

vary from 7.5 to 25 cm (3 to 10 inches), depending on the required thickness of the concrete lining. The depth of cut, which is equal to the depth of penetration of the cutter, is a maximum of 4.5 m (14 ft 9 in). In difficult ground, the depth of penetration would be reduced, possibly to 1.5 m (5 ft). When cutting a slot for a pre-lining, the cutter is angled slightly outwards so that it describes a conical shape. Such successive conical shapes overlap with each other over a distance of at least 30 cm (12 in), thus assuring continuity of the lining, see Figures 3, 4 and 5. A larger overlap, sometimes up to 2.0 m (6 ft 7 in), would reduce the rate of advance per round but would increase the strength of the continuous lining. Additional support for the lining can be provided by steel sets, see Figure 5.

The construction of a pre-lining in cohesive soils generally proceeds in stages, see Figure 6.

1. Advance the pre-cutting machine up to the face.
2. Adjust the position and orientation of the machine, guided by a laser beam.
3. Penetrate cutter into the soil.

4. Cut the first stage of the slot and stop the machine. Cutter remains in place while the slot is cleaned out. Shoot concrete into the slot to fill the space between the bottom of the slot and the cutter.
5. Cut the second stage, clean out and shoot concrete. Repeat for successive stages.
6. Cut the last stage. Withdraw the cutter from the slot. Clean the bottom of the last cut and fill with concrete to complete the pre-lining section.
7. Back up the machine, away from the face, to make room for other activities such as the installation of additional support, if required, and mucking under the protection of the just completed pre-lining.

The rate of advance of the cutter through cohesive soils is in the order of 1 m per minute. The number of stages per pre-lining depends on ground conditions. The amount of time required per stage may vary from 3 to 15 minutes. The total time required per pre-lining may vary from 3 to 5 hours. The advance of the tunnel face per pre-lining is equal to the length of penetration of the cutter minus the length of overlap.

The minimum radius for pre-cutting is 2 m (6 ft 7 in).

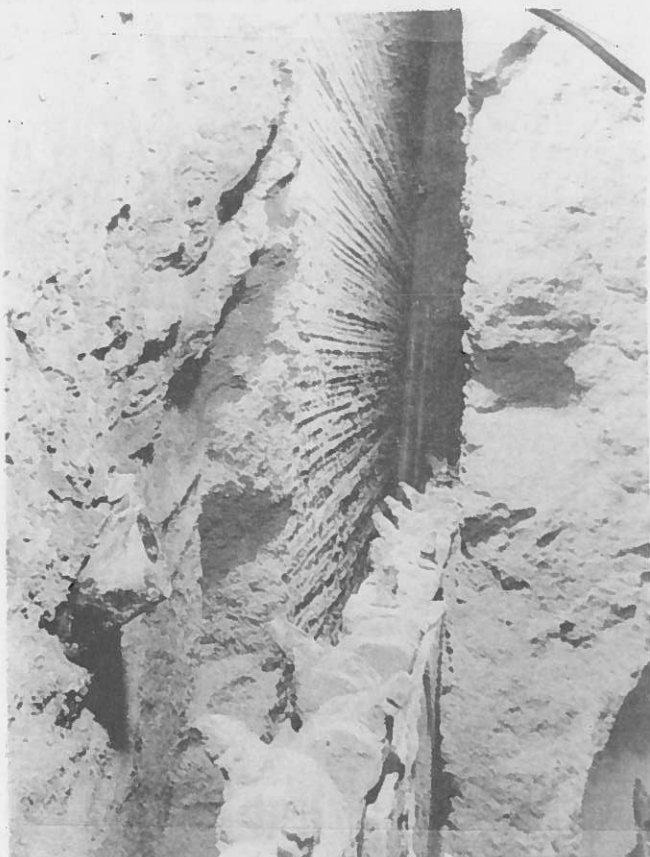
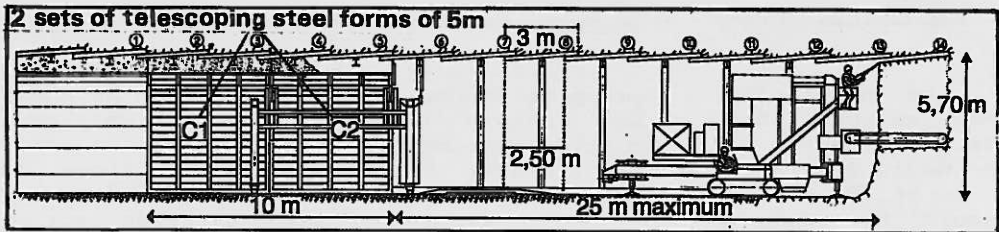


Figure 2. Pre-cutting of a slot along the tunnel periphery in progress.



CONSTRUCTION OF PRE-LINING SECTION nr. 14

TOP: CUT SLOT nr. 14 AND FILL WITH SHOTCRETE
 PENETRATION OF CUTTER 3.00 m
 OVERLAP BETWEEN CUTS 0.50 m
 ADVANCE PER CUT 2.50 m

FOR PERMANENT LINING, POUR CONCRETE UNDER SECTIONS nr. 4 AND nr.5

BOTTOM: PLACE STEEL SETS, IF REQUIRED UNDER SECTION nr. 13
 MUCK UNDER PRE-LINING SECTION nr. 14

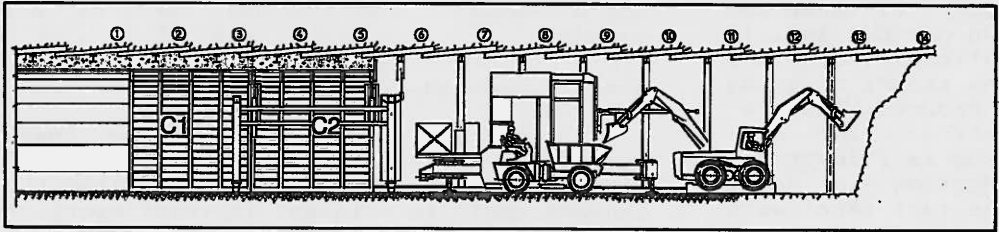


Figure 3. Pre-cutting with pre-lining in soft ground.

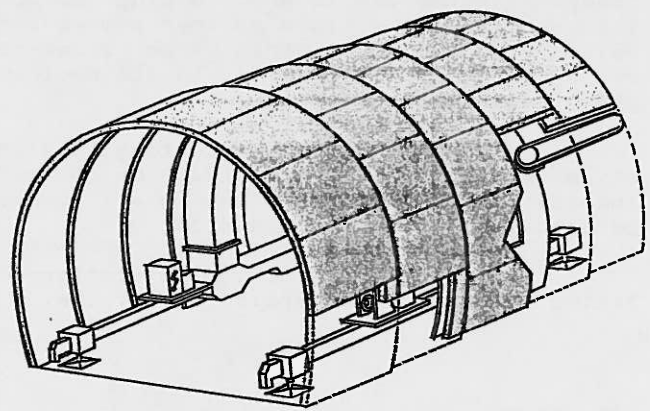


Figure 4. Construction of the pre-lining. The cutter is angled slightly outwards so that the pre-cut slots have a conical shape, each one overlapping with the previous one.



Figure 5. Concrete, shot into the pre-cut slots, creates overlapping concrete shells of conical shape. When required, steel ribs can be added for extra strength as shown.

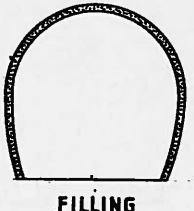
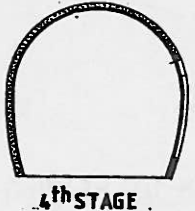
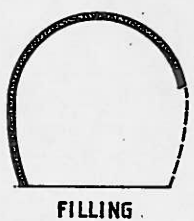
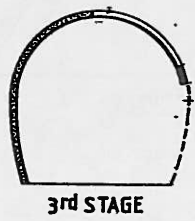
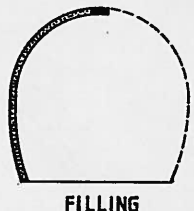
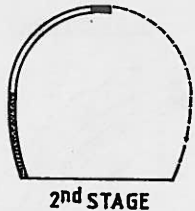
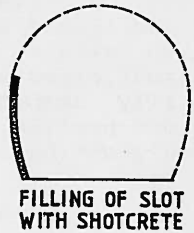
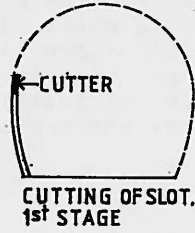
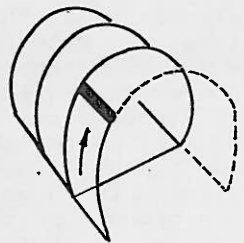


Figure 6. Construction of a pre-lining segment in stages.

(The length of a slot, cut per stage, will depend on the nature of the ground.)

In non-cohesive soils, Mechanical Pre-cutting with Pre-lining.

The construction of the pre-lining proceeds in general terms as described above for cohesive soils. The depth of penetration of the cutter would generally be reduced from 4.5 m to 3 or 2 m. The length of the stages would be reduced, possibly to as short as .40 m (16 in).

In certain cases, grouting of sands and gravels may have to precede pre-cutting. In comparison with conventional tunneling methods in such soils, the amount of grout required can be greatly reduced. The pre-cutting tunneling technique in non-cohesive soils with grouting is illustrated in Figure 7. The reduction in grout quantities stems from the fact that with the pre-cutting technique, the grout can be concentrated like a "halo" around the tunnel periphery. With this technique, grout can be injected where it is needed. As the pre-cutter slices through the grouted soil, grout pipes are inserted in the pre-cut slot after the cutter has passed. These grout pipes are extended horizontally into the ground ahead of the slot and this ground is then subsequently grouted. Concrete is shot into the pre-cut slot and around the grouting pipes in the slot to make a new section of pre-lining. The grout pipes stay in place and the next conically shaped slot is cut just inside the cone profile, formed by the previously placed grouting pipes.

In rock, Mechanical Pre-cutting.

In rocks with compressive strengths of up to 70 MPa (10 000 psi), such as shale and certain limestones, mechanical pre-cutting combines the advantages of drill and blast techniques with those of tunneling machines:

- almost any shape of cross section is feasible,
- overbreak has been eliminated and as a result, the tunnel surfaces are smooth,
- noise and vibrations have been greatly reduced.

Mechanical pre-cutting in these types of rock is a drill and blast technique which however differs from the conventional techniques in some important aspects. With conventional, full face drill and blast tunneling, the first blasting delays serve to break into the face. The holes for the first rounds are drilled and loaded with the aim to shoot a plug out of the centre of the face so that subsequent delays slash towards that free space to bring the tunnel opening to its desired shape, see Figure 8.

With mechanical pre-cutting, a slot of a width of 7.5 to 12.5 cm (3 to 5 inches) is cut all along the exact tunnel periphery, see Figures 9 and 10.

For cutting rock, the cutter is fitted with tungsten carbide teeth. The

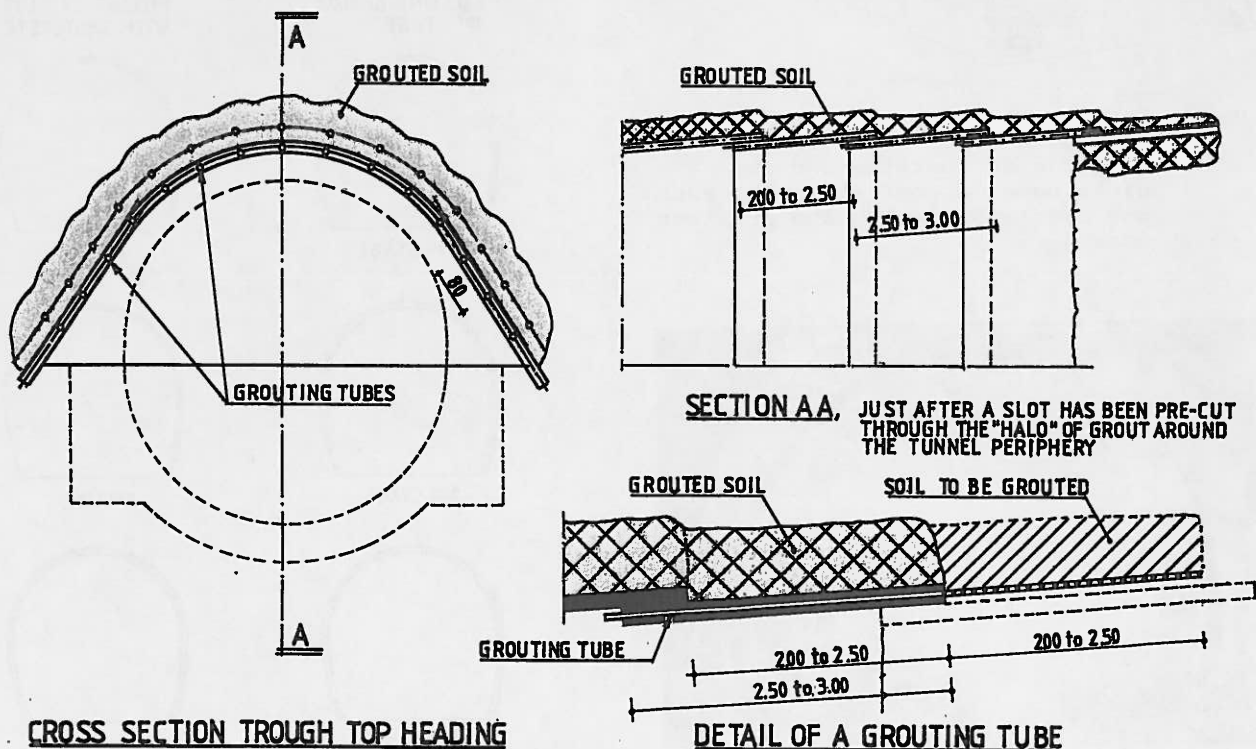


Figure 7. The pre - cutting tunneling technique as applied to non-cohesive soils, requiring grouting.

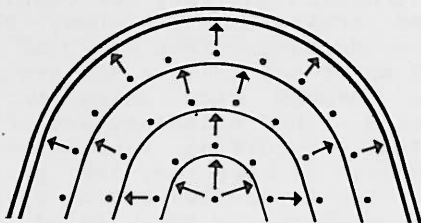


Figure 8. Conventional drilling and blasting pattern for a full face advance.

slot along the tunnel periphery now serves as a substantial free surface towards which the rock can be broken. The first blasting delay therefore is situated on the outside and successive delays are placed more and more towards the centre. In this way, the amount of drilling is greatly reduced, by as much as 80 %, and the amount of explosives by as much as 50 %. And with it all, we deal with a much safer blasting operation, giving a smooth tunnel surface without overbreak. The greatly reduced intensity of explosions per round and the shielding of these explosions from the surrounding rock mass by the pre-cut slot has resulted in the elimination of all objectionable vibrations. It is not surprising that the first applications of mechanical pre-cutting in rock were prompted by environmental requirements, imposed by an urban environment. It was to serve Paris, France, that the first double-track tunnels, which had to pass under old existing buildings, were excavated using mechanical pre-cutting as a tunneling technique.

In rock, the slot cut along the tunnel periphery, serves to:

- replace the "plug" which normally would be shot out of the face by the first blasting delay;
- suppress the travel of shock and sound waves;
- eliminate over break;
- greatly reduce the amount of drilling;
- simplify drilling operations (all blast holes are parallel);
- reduce the amount of explosives;
- simplify blasting procedures (all blasting is essentially "bench" blasting);
- preserve the natural state of the surrounding ground mass;
- reduce the amount of temporary and permanent support;
- reduce grout consumption in cases where grouting is called for;
- improve safety.

For applications in rock, the maximum depth of pre-cut slot made so far has been 3 m (10 ft); the width of the slot has varied from 7.5 cm to 12.5 cm (3 to 5 in) and the minimum radius of curvature of any part of the cross section has been 2 m (6 ft 7 in).

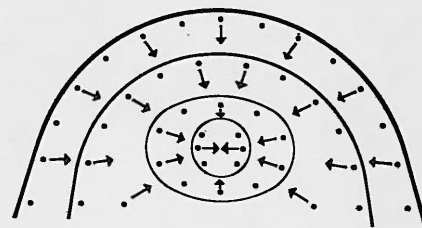


Figure 9. Drilling and blasting pattern for a full face advance with pre-cutting. Advantage is taken of the free surface provided by the pre-cut slot along the tunnel periphery.

Some typical projects executed over the last 15 years

Tunnel of the regional subway (RER) at Fontenay-sous-Bois near Paris in cohesive soils.

The cross section of this tunnel and its location within clay strata is shown in Figure 11. The geology shown is typical for the region near Paris, France. During the construction of this tunnel from 1974 to 1976, measurements of surface settlements were made.

At the same time, measurements of surface settlements were made for a similar tunnel under construction in similar ground at Grigny for the French National Railways (SNCF), using the New Austrian Tunneling Method. Both tunnels were driven with a double heading. The principal difference between these two construction methods is that with Mechanical Pre-cutting, the supporting concrete shell is in place and has hardened before the material under the shell is excavated. With the New Austrian Tunneling Method, shotcrete for the supporting shell is applied after the ground has been excavated. Measurements of surface settlements for these two tunnels were made. In order to compare the results, the surface settlements for the two projects were plotted on one graph as a function of time, see Figure 11. The zero point on the time scale corresponds to the time at which the tunneling front of the upper heading passed the surface point under observation. The tunnel, driven with Mechanical Pre-cutting, was 20 % larger in diameter than the tunnel driven with the New Austrian Tunneling Method. Yet surface settlements above the tunnel with Pre-cutting was 70 % less than that for the tunnel driven with the New Austrian Tunneling Method.

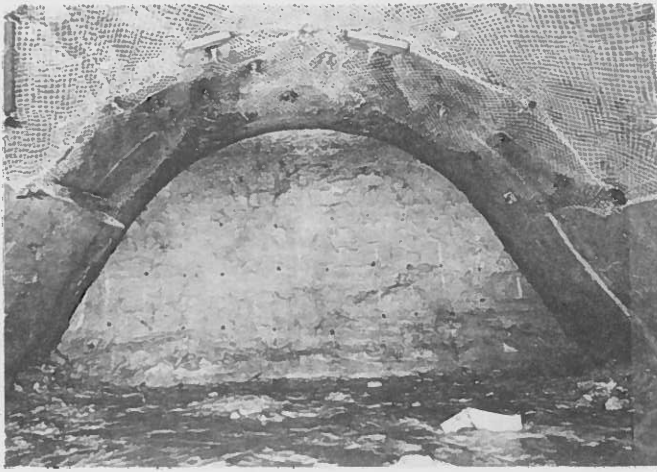


Figure 10. Pre-cut slot along the periphery of a tunnel in rock.

The large tunnels, presently under construction in Italy for double track high speed trains and highway traffic with driven cross sections of 70 m² and 100 m² (753 sq ft and 1076 sq ft) are driven full face through clay using Mechanical Pre-cutting with Pre-lining, see Figure 1. The Italian pre-cutting machines were designed so that, if need be, the machines can be converted to double-heading operation. Such conversion has so far, as of August 1989, not been necessary. The two pre-cutting machines, presently in operation on the construction of the new subway system of Toulouse, France, were designed in similar fashion.

A single heading advance has obvious advantages from a construction point of view but it is also the most effective way of reducing surface

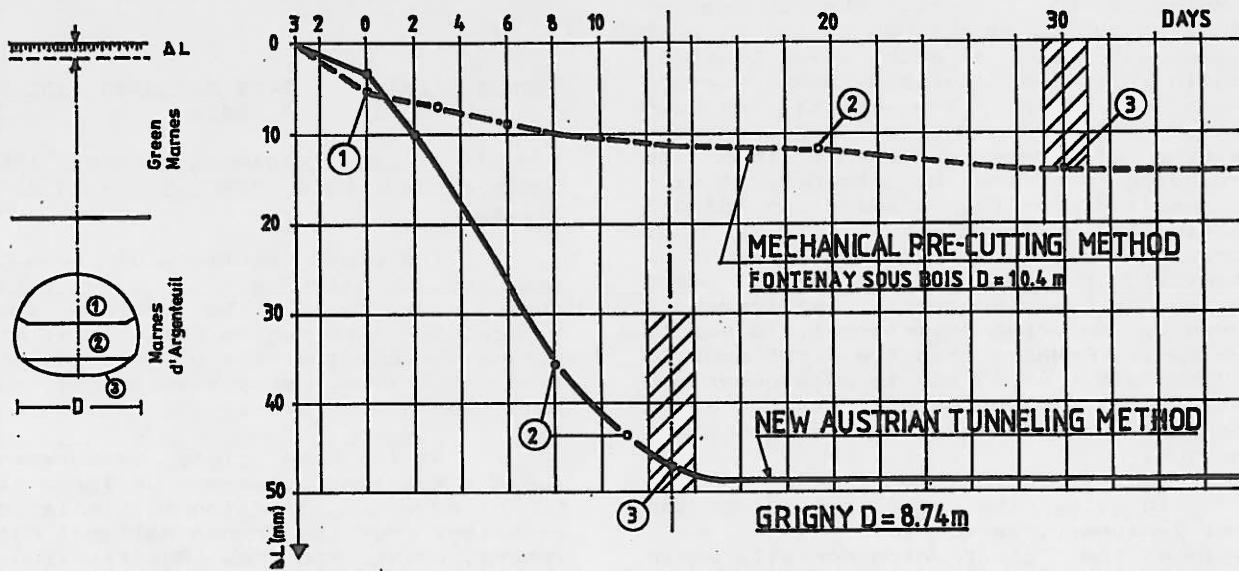


Figure 11. Comparison of surface settlements, RER tunnel at Fontenay-sous-Bois (Mechanical pre-cutting) and SNCF tunnel at Grigny (New Austrian Tunneling Method).

settlements and convergence of the tunnel opening due to the fact that the complete tunnel lining can be in place at the earliest possible time. With mechanical pre-cutting, the decompression of the ground mass around a tunnel can be avoided. The tunnel lining serves to provide lateral support for the surrounding ground.

Some recent subway, railway and highway tunnels:

Full-face advance or double heading

In the early stages of development of the Mechanical Pre-cutting tunneling method, double headings were used as mentioned above for the tunnel at Fontenay-sous-Bois. For the subway tunnels at Lille, in the North of France, the pre-cutting machines were designed so that they would start with the upper heading and, if all went well, could be converted to full face operation. As it turned out, the first 33 % of the Lille tunnels were driven with twin headings while the remaining 67 % were driven full face. The excavated face of the tunnels in Lille was 55 m² (592 sq ft). This project was completed in 1982.

The pre-cutting machine

A pre-cutting machine consists of a structural portal which gives the machine the necessary rigidity. This portal rests on two mobile units which provide not only locomotion but also means of orienting the portal so that the cutter will run its proper theoretical course. The portal with its mobile units provides adequate space for drilling jumbos and mucking machinery to pass under it. Cantilevered from the structural portal is a steel arch on the inside of which is mounted a double steel rack. The pinions, driving the cutter carriage, engage with the teeth of the arch-mounted racks which thus guide the

cutter along the arch. The arch is custom built to meet the specified shape of the tunnel cross section. The mounting of the cutter carriage is adjustable so that the cutter can be made to cut a conical shape as required for the formation of any desired pre-lining. The cutter carriage mounting also allows an adjustment of the cutter in radial direction. This means that the radius of the curve, described by the cutter, can be adjusted within the limits required for mechanical pre-cutting with pre-lining. The cutter head proper consists of a motor with drive pinion and cutter chain, mounted on its support and guide.

Depending on ground conditions, the cutters are equipped with chains which can cut to a depth of up to 4.5 m (14 ft 9 in) with the width of the cut varying from 7.5 to 25 cm (3 in to 10 in); the installed power of the cutter motors may vary from 90 to 375 kVA. The machines are totally hydraulically operated.

While state-of-the-art technologies are used for the various components of the machines such as hydraulics, cutting tools, remote controls, robotics, lasers, shotcrete, concrete aggregates and additives, the machines are simple in concept and easily understood by newly trained operators.

Table 1. Summary of projects on which Mechanical Pre-cutting has been applied.

Project and Client	years of construction	tunnel lengths	finished tunnel width	geology	Mechanical Pre-cutting application
Start up adits for Robbins TEM, link Chatelet - Gare de Lyon, RER Line A, R.A.T.P.	1973	4 x 15 m	7.00 m	coarse Lutetien limestone	top heading
Link Luxembourg-Chatelet, RER Line B, R.A.T.P.	1974-75-76	700 m and 220 m	5.70 m and 12.00 m	coarse Lutetien limestone	top heading
Link Fontenay Sous Bois - Marne la Vallée, RER Line A, R.A.T.P.	1974-75-76	558 m	8.70 m	blue Argenteuil clay and gypsum	top heading with pre-lining
Link Luxembourg-Chatelet, RER Line B, R.A.T.P.	1976-77	80 m	5.70-6.00 m	clay and callovian	top heading with pre-lining
Extension of line 10 to Boulogne R.A.T.P.	1977-78-79	1 000 m	7.10 m	chalk with bands of Silex	top heading with pre-lining
Link Chatelet-Gare du Nord, RER Line B, R.A.T.P.	1978-79-80	various adits, 450 m, and tunnel, 1 100 m	5.70 m 8.00 m 9.10 m	clay, sand with gypsum, limestone	top heading with pre-lining
Lille subway, contracts 6 & 7 Lille urban community	1980-81-82	1 700 m	6.40 m 6.70 m 6.90 m	fissured and metamorph chalk	full face
High speed train (TGV), Fontenay	1985-86	474 m	10.00 m	clays	full face with pre-lining
High speed train (TGV), Soeaux	1985-86	827 m	10.00 m	clays	top heading with pre-lining
Euro tunnel under the Channel from France to Great Britain	1987	various start up adits for TEMs, 260 m	11.00 m	clays	top heading with pre-lining
High speed train (TGV), Sibari to Cosenza, Italian National Railways	1986-87	4 000 m	8.00 m	clays	full face with pre-lining
High speed train (TGV), Arrezzo Italian National Railways	1988	3 200 m	7.00 m	clays and sands	full face with pre-lining
idem	1988-91	1 800 m	10.40 m	clays and sands	top heading with pre-lining
High speed train (TGV), Bari Italian National Railways	1988-91	5 000 m	10.40 m	clays and sands	idem
Subway of Toulouse, contracts 3, 4 & 5 City of Toulouse	1989-92	2 000 m	6.80 m 10.00 m	mixed alluvium	full face with pre-lining

Possible future applications

The avoidance of damage to the surrounding rock mass makes Mechanical pre-cutting in rock often suitable in situations where the available rock cover is inadequate for conventional, full face tunneling. With Mechanical pre-cutting, explosives can be used in situations where they would otherwise be prohibited.

Apart from tunnel construction, Mechanical pre-cutting has wider applications, for instance in the construction of underground storage caverns for liquid gas. The freedom of choice of the shape of the caverns, the lack of overbreak and the avoidance of damage to the surrounding rock mass are important considerations which suggest the use of Mechanical pre-cutting for this type of installation.

For major tunneling projects through long stretches of variable geology, a pilot tunnel is at times called for. Traditionally, pilot tunnels have been expensive luxuries. They may have yielded important information, but at a price. In terrain of medium hard to soft rock, pilot tunnels can be used in combination with blasting, to reduce tunneling costs and to improve the quality of the end product as well as the safety and speed of construction. These aims can be achieved by integrating the pilot tunnel into the overall tunneling process. Only after the permanent tunnel profile has been pre-reinforced and pre-cut would blasting to the full tunnel section proceed. The proposed method is fully within presently available tunneling technology and has been described in some detail in Reference 3.

List of Figures

1. Typical pre-cutting machine, retracted from the face, in a double track railway tunnel.
2. Pre-cutting of a slot along the tunnel periphery in progress.
3. Pre-cutting with pre-lining in soft ground.

4. Construction of the pre-lining. The cutter is angled slightly outwards so that the pre-cut slots have a conical shape, each one overlapping with the previous one.

5. Concrete, shot into the pre-cut slots, creates overlapping concrete shells of conical shape. When required, steel ribs can be added for extra strength as shown.

6. Construction of a pre-lining segment in stages.

7. The pre-cutting tunneling technique as applied to non-cohesive soils, requiring grouting.

8. Conventional drilling and blasting pattern for a full face advance.

9. Drilling and blasting pattern for a full face advance with pre-cutting. Advantage is taken of the free surface provided by the pre-cut slot along the tunnel periphery.

10. Pre-cut slot along the periphery of a tunnel in rock.

11. Comparison of surface settlements, RER tunnel at Fontenay-sous-Bois (mechanical pre-cutting) and SNCF tunnel at Grigny (New Austrian Tunneling Method).

References

1. Y. Le Goer, "Le creusement des tunnels en site urbain par predecoupage mecanique," Travaux, December 1982.
2. J.F. Bougard, "The Mechanical Pre-Cutting Method," Tunneling and Underground Space Technology, Vol.3 No. 2, pp. 163-167, 1988, Pergamon Press plc, Great Britain.
3. E. van Walsum, "Pilot tunnels that pay off," unpublished manuscript, copies of which may be obtained by sending \$8.00 to: E. van Walsum, P.Eng., M.Eng. 26 Claremont, Pointe Claire, Qc Canada, H9S 5C5

B:07078902

COMPUTER ANALYSIS OF LONG-TERM STABILITY OF A SALT DOME IN
RELATION TO COMPRESSED AIR ENERGY STORAGE (CAES) CAVERN DEVELOPMENT

by

M. Reza Salami¹ and Sameer A. Hamoush²

¹Assistant and ²Adjunct Professors, Department of Civil Engineering,
North Carolina Agricultural and Technical State University, Greensboro,
North Carolina, 27411

Abstract. This paper contains the preliminary results of stability assessment of CAES caverns to be excavated in the McIntosh salt dome, Alabama. The study consists of two major parts: (1) the stability assessment of the whole dome under the operation of Compressed Air Energy Storage (CAES) caverns and the operation of Olin chemical corporation, and (2) the determination of safe boundary distance for CAES caverns using a single cavern. In this paper the whole dome was analyzed. Two Finite Element Models (FEM), one using a vertical cross-section of the dome and the other using a horizontal cross-section of the dome, are constructed to analyze the whole dome. These two models include the two CAES caverns belonging to the Alabama Electric Co. The other caverns in the dome belong to Olin Chemical Corp. The general geology of the area was obtained from the information provided by Olin Chemicals Corp. Based on the model assumptions regarding the initial stress state and the properties of material involved, the results obtained from the analysis lead to the following conclusions: (1) The present condition of the entire salt dome is found to be stable, except for some high concentration of stresses and strains around

Introduction

For many years, Compressed Air Energy Storage (CAES) has been the subject of research among electric utility companies, in order to determine the feasibility of storing off-peak excess electricity in the form of compressed air. Conceptually, the compressed air would be stored in underground geological formations which have pressure integrity and are stable under CAES operating pressure and temperature ranges. This compressed air would later be converted back into electricity to meet peak hour demand by running the turbans. It has been realized for many years that underground salt domes are very suitable for this purpose, since the first CAES cavern, excavated in a salt dome in Huntorf, West Germany, has been operating successfully since 1977.^{1,2}

caverns to be excavated in McIntosh salt dome, Alabama. EPRI (2) was considering the possibility of CAES operation in McIntosh salt dome in Alabama. In this regard, SGI has undertaken the task of analyzing the stability of the same under CAES cavern operation.

The stability of McIntosh salt dome under the present operations of Olin Chemical Corporation was analyzed in relation to the proposed CAES caverns operation. In order to fully comprehend the nature and magnitude of the effects of the CAES cavern operation on long-term stability of the entire salt dome structure, including all the caverns, a sensitivity analysis will be conducted by varying combinations of parameters.

This paper was accomplished under Electric Power Research Institute (EPRI), on geomechanical investigation of Compressed Air Energy Storage (CAES)

Scientific Basic of Study

The development of high speed digital computers for the use of engineering analysis has made the finite element method (FEM) a powerful tool for obtaining solutions to complex geomechanical boundary value/initial value problems. The early developments of FEM were incorporated with linear elastic theory for constitutive behavior of materials, which were at later stages extended to include the theory of plasticity. These theories based on classical plasticity were inadequate in representing the long-term behavior of caverns surrounded by complex brittle-ductile media. For this reason, Serata Geomechanics, Inc. (SGI)^{1,3,4} has developed a proprietary finite element program (FEM) based on the rheological characterization of material behavior. The FEM program was developed during past 20 years of field experience and it has been successfully verified for its capability of simulating field conditions. FEM has been extensively used in analyzing the behavior of oil, natural gas, chemical and compressed air. The present REM program is a finite element program specially designed and field-validated to interrelate real earth material properties with the corresponding behavior of the ground opening.

Within the scope of this study, the effects of the initial stress state and the initial material properties on the stability of the cavern field were investigated.

Because of some uncertainties in estimating the above data, a wide range of possible conditions were examined. Our general approach was to make conservative assumptions regarding of the major unknown quantities.

This study is considered to be a preliminary study aimed to a general assessment of the overall safety condition of CAES and the cavern field. The study was, therefore, limited to a 2-dimensional analysis, although the nature of the problem is 3-dimensional.

Preliminary Analysis

Due to the uncertainty and unavailability of the full data required for analysis, numerical finite element analysis of solution mining and cavern storage networks has been difficult. The data needed for computer simulation include the geology of the area, the geometry of the cavern network, the properties of the materials involved, in situ stress level, and cavern operating conditions. The existing data in these categories should be carefully evaluated to obtain the necessary information related to the particular analysis. The following subsections describe the data which were used to

analyze of the McIntosh salt dome cavern field.

Literature Review

Literature relevant to salt domes and other types of salt deposits in the U. S. was gathered and reviewed. Literature on salt domes was obtained from sources including the Alabama Electric Cooperative, Battelle Pacific Northwest Laboratories, Louisiana State University (LSU), Electric Power Research Institute (EPRI), Olin Chemical Corp., Sandia National Laboratories, and the Solution Mining Research Institute (SMRI).

The items of literature obtained were categorized and indexed based on their contents in relating to the following topics:

1. Geological formations: Salt domes and related deposits, Refs.^{5,3}.

2. Materials properties: Salt dome halite and related geological materials, Refs.^{7,8}.

3. Present operations: McIntosh dome solution-mined caverns, Refs.^{1,9,10}.

Geology

The geology of the salt dome and caprock was obtained from the general well information record maintained by Olin Chemical Corp. These records provided the thickness of caprock at well locations and the approximate thicknesses of different layers above the caprock. The geology and geometry of the layers can be established from the data obtained from well logs. A typical geological section for the McIntosh salt dome is shown in Figure 1. The boundary (vertical flanks) of the dome is close to the cavern field and hence particular attention was paid to establishing the dome boundary. Figure 2 depicts a plan view of the area, indicating the locations of wells and the salt dome boundary of the McIntosh salt dome.

CAES Cavern Field

The geometry of individual CAES caverns plays a significant role in the stability analysis of the dome. The amount of excavation represented by the excavation geometry controls the area of stress relief around the CAES caverns, and the shape of the CAES cavern dictates the stress concentration. Good quality sonar survey data provided by the Olin Chemical Corp. are utilized in determining geometry. As seen from these data, the geometry of some wells is rather arbitrary, with many sharp edges along the cavern boundaries. In preparing site-specific finite element meshes for FEM analysis, the irregularity of cavern shape was taken into account as much as feasible, for more realistic representation of cavern geometry.

Material Properties

The results of the finite element analysis depend greatly on the material property parameters which are used to characterize the ground material. The reliable methods of obtaining such parameters are (1) laboratory testing on cores obtained from the particular sites; and (2) in situ measurements. Unfortunately, in this case, no test data were available, therefore, the property parameters were estimated based on the experience and engineering judgement. Most of the available data show that the mechanical properties of salt do not vary greatly regardless of the source of the salt, as compared to other geological materials. The coefficient values of the material properties always fall within a certain predetermined range. Based on this observation, the parameters were categorized as weak, normal and strong. The coefficient values corresponding to these categories within the context of FEM are given in Ref.(1), respectively. For the analysis of the McIntosh cavern field, it was decided to use norm material parameters (the best average parameters). This uncertainty was compensated for, during the parametric analysis, by using the weak value for the most critical material property parameters.

In-Situ Stress

In situ stress is the primary cause of cavern deformation and perhaps the most important variable in the analysis of deep geological structures. The in situ state of stress can be fully represented by three components of principal stress: the vertical component, the maximum and minimum horizontal components, which are perpendicular to each other.

The vertical stress at a point in the ground can be estimated accurately from the weight of the overburden materials. However, the lateral stresses or the horizontal components of the stresses can be less than, nearly equal to, or greater than the vertical stress. The amount by which the lateral stress differs from the vertical stress is identified as the excess lateral stress. There are no data available on the excess lateral stress of the McIntosh salt dome.

Cavern Pressure

All the caverns in the McIntosh dome, except the CASE caverns, were assumed to be filled with brine, which has a density of 1.2 g/cm^3 . The pressure on the cavern wall at a given depth was computed as the pressure, that exists in a continuous brine column, extending from that depth to the surface. EPRI personnel expect to maintain the pressure inside the CASE caverns below 875 psi. Hence, in all our analysis, a

pressure of 875 psi was used for CAES caverns.

Method of Study

The method of study involves construction of finite element models (FEM) of the McIntosh salt dome, representing the actual excavation geometries of existing brine wells and the geometries of the proposed CAES caverns. The information gathered was combined to develop a preliminary finite element model of the McIntosh salt dome.

Site-Specific Meshes

In order to simulate the 3-dimensional nature of the deformation, two 2-dimensional plane strain meshes of the whole dome were constructed:

1. A horizontal section through the entire salt dome, including all brine wells and CAES caverns.
2. A vertical section passing through the maximum possible number of brine wells and CAES caverns.

Figure 3 shows the finite element mesh of the vertical section of the dome. Portions of the mesh are presented in Figs. 4 and 5. Figure 4 shows the configuration inside the salt dome, whereas Figure 5 shows the configuration around the CAES caverns. The finite element mesh, constructed using the horizontal section of the dome, is shown in Figure 6. Figs. 7 and 8 illustrate the configurations inside the dome and in the immediate vicinity of the CAES caverns.

The exact dimensions of Olin's cavern were used to construct the meshes. For the CAES caverns, the following "norm" dimensions were used, allowing the possibility of changing them at later stages:

Diameter of CAES cavern, D	: 150 ft
Height of CAES cavern, L	: 622 ft
Depth from the top of the CAES cavern to ground surface, H	: 1500 ft
Distance from center of CAES cavern to dome boundary	: 1000 ft
Center-to center distance between CAES caverns, S	: 625 ft

The vertical section of the finite element mesh consists of 2205 4-node elements and 2337 nodes. The horizontal section mesh consists of 2751 4-node elements and 2782 nodes.

Parametric Study

There are a number of factors that must be considered in choosing parameters for sensitivity analyses. The sensitivity analysis is performed by (1) in order to evaluate the nature and significance of the uncertainty due to limited knowledge of certain parameters.

Excavation Sequence

The following sequences of cavern excavation were adopted for the FEM analyses: Using the norm parameters for vertical analysis, Caverns 1 through 10 had been excavated at zero time (1956) and the analysis was carried out for a period of 30 years, at which time CAES Caverns 1 and 2 and Olin Caverns 10, 11, 12 and 13 were excavated. Subsequent analyses were then carried out for another 30 years. A similar procedure was adopted for norm parameters in the horizontal analysis, in which Caverns 1-10 were excavated at zero time. The other caverns, C1, C2, and 10-13, were excavated after 30 years of the above excavation

Results of Vertical Plain Strain Analysis

Results obtained for the vertical section using plane strain analysis are discussed here.

Global Deformation

Figure 9 illustrates the global displacement patterns after 60 years of excavation for the analyses. Figure 10 shows the enlarged view of the deformation pattern around the CAES caverns at 30 years.

Stress and Strain Distributions

The global pattern of principal stress distributions at the 60th year is shown in Figure 11 for the norm run. It is clearly seen that a strong secondary stress envelope was formed around the caverns. In this run, along the vertical cavern walls, the stress remains nearly hydrostatic indicating that there is no immediate danger of wall failure. A very high concentration of stresses is seen at the sharp edges. Figure 12 shows an enlarged view of the stress pattern around the CAES caverns at 30 years. Figure 13 shows the strain distribution around the cavern after 60 years of excavation for the norm analyses. Very high strains are observed at the cavern edge; otherwise, the strain distribution is uniform. Figure 14 shows an enlarged view of the strain pattern around the CAES caverns at 30 years.

Variation of Horizontal and Vertical Stress/and Strain into Pillar and Abutment, with Time

The stress and strain variations in the pillar between CAES Caverns 1 and 2 are shown in Figure 15. The increase of the vertical stress at the middle may be due to the overlapping of two primary stress envelopes from the two CAES caverns. Strain is maximum at a distance of about 275 ft from CAES Cavern 1. The maximum strains are 1.4% in the vertical direction and -1.65% in the horizontal direction.

Figure 16 shows stress and strain variations into the abutment of CAES Cavern 1. The maximum vertical and horizontal strains at a distance of about 150 ft from CAES Cavern 1 are 0.4% and -0.8%, respectively.

Vertical and Horizontal Closure of CAES Caverns

Figure 17 provides a comparison of the vertical closure of CAES Caverns 1 and 2 over a 60-year period. The maximum vertical closures for CAES Caverns 1 and 2 at the end of 60 years are 0.58 ft and 0.76 ft, respectively. Figure 18 shows a comparison of horizontal closure of CAES Caverns 1 and 2 over a 60-year period. The maximum horizontal closures for CAES Cavern 1 and 2 at the end of 60 years are 0.62 ft and 0.64 ft., respectively.

Surface Subsidence

Surface subsidence gives a good indication of the deformation consequences of cavern excavation. Figure 19 illustrates the comparison of time variation of surface subsidence over CAES Caverns 1 and 2. The surface subsidences of CAES Caverns 1 and 2 at the end of 60 years are 0.15 ft and 0.475 ft, respectively.

Results of Horizontal Plain Strain Analysis

Results obtained for the horizontal section using plane strain analysis are discussed here.

Global Deformation

Figure 20 shows the deformation patterns of enlarged partial meshes for the norm Run. Figure 21 shows close-ups of the displacement pattern around CAES Caverns 1 and 2. In general, the surrounding area of CAES caverns is experiencing very small deformations.

Principal stress and strain distribution

The global pattern of principal stress distributions at the 60th year is shown in Figure 22 for the norm run. It is clearly seen that a strong secondary stress envelope has been formed around the dome

boundary. Figure 23 shows the close-up of principal stress patterns around CAES Caverns 1 and 2 after 30 years of CAES cavern excavation for the norm Run.

Figure 24 shows the strain distribution around the caverns after 60 years of excavation for norm analyses. Figure 25 shows an enlarged view of the strain pattern around the CAES caverns at 30 years of CAES cavern excavation. Very high strains are observed around the boundary of the caverns, otherwise the strain distribution is uniform.

Variation of horizontal and vertical stress/strain into pillar and abutment with time

The stress and strain variations in the pillar between CAES Caverns 1 and 2 are shown in Figure 26. Strain is maximum at a distance of about 500 ft from CAES Cavern 2. The maximum strains are 0.18% in the vertical direction and -0.26% in the horizontal direction.

Figure 27 shows stress and strain variations into the abutment of CAES Cavern 1. The maximum strains are 0.2% in the vertical direction and - 0.2% in the horizontal direction.

Maximum and minimum closure of CAES caverns

Figure 28 shows the maximum and minimum closure of CAES Cavern 1 over a 30-year period, as predicted by the norm run. The maximum closures for CAES Caverns 1 and 2 at the end of 30 years are 0.33 ft and 0.345 ft, whereas the minimum closures of the above caverns are 0.24 ft. and 0.25 ft, respectively.

Deformation of the Salt Dome Boundary

The magnitudes of deformation of the salt dome boundary are presented in Figure 29, for the norm run. The deformation magnitude of two diametrically opposite points on the boundary (e.g., Points A and B) are plotted against the angle shown in the figure. It can be seen from the figure that the dome is squeezing in all directions. The norm run shows that most of the squeezing takes place during 30 years of the initial excavation.

Conclusions

The following conclusions are drawn from this study:

1. Stability of Cavern Field

Finite element analysis shows a stable condition around the cavern field.

2. Stability of CAES Caverns

The whole dome analysis shows that the site selected for the study is stable,

where the sensitivity analysis strengthens this conclusion. The cavern location at more than 200 ft. from the dome boundary is satisfactory.

3 Stress Distributions

A large plastic region is developing at the bottom of the solution caverns, but not around the CAES caverns.

4. Material Property Parameters

Correct estimation of the material property parameters plays a major role in this analysis.

5. Method of Study

The method employed in this study must be improved to realistically simulate field behavior. Three-dimensional modelling of the cavern field should be conducted in the future; however, 2-dimensional analysis provides an economic and time-saving alternative to the 3-dimensional approach, without much loss of accuracy.

It is worth while to mention here that the above analysis is important for the future use in analyzing and designing all the compressed under-ground storage used for storing oil, chemical or air.

Acknowledgment

The authors acknowledge the support that was provided by the Serata Geomechanics, Inc. and Electrical Power and Research Institute (EPRI). Also the authors acknowledge the support provided by Department of Civil Engineering of North Carolina A and T State University,

References

1. M. Reza Salami, Serata Geomechanics Inc., "Preliminary Computer Analysis on Long-Term Stability of McIntosh Salt Dome in Relation to Compressed Air Energy Storage (CAES) Cavern Development," Preliminary report for EPRI, Presented at Solution Mining Research Institute Spring Meeting, Tulsa, Oklahoma, 1987.
2. Electric Power Research Inst., "General Electric Company, Conceptual Design for a Pilot/Demonstration Compressed Air Storage Facility Employing a Solution-Mined Salt Cavern," section 7 1977.
3. Serata Geomechanics, Inc., "Theoretical Definition and Determination Procedures A Property Coefficient Values of FEM. Constitutive Model, Laboratory Method of Material Testing," Rock Mechanics School - Solution Mining, Houston Texas, 1987.
4. S. Serata, "Prerequisites for Application of Finite Element Method

to Solution Cavities and Conventional Mines", Proc. 3rd Symposium on Salt, The Northern Ohio Geological Society, Inc., Cleveland, Ohio, 1973.

5. G. W. C. Kaye and T. H. Laby, "Tables of Physical and Chemical Constants" 14th Ed., Longmans, London 1973.
6. Law Engineering Testing Company, "Gulf Coast Salt Domes Geologic Area Characterization," Report, Vol. I, Introduction, ONWI-117, Prepared for Office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH, 1982.
7. Pfeifle, Tom W., Kirby D. Mellegard, and Paul E. Senseny, ".Constitutive Properties of Salt From Four Sites," ONWI-314, Prepared by RE/SEC Inc. for office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH, 1983.
8. Senseny, Paul E., " Review of Constitutive Laws Used to Describe the Creep of Salt," Prepared for office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbia, OH.,1983.
9. Law Engineering Testing Company, "Gulf Coast Salt Domes Geologic Area Characterization Report, North Louisiana Study Area," Vols..IV and V, ONWI-119, Prepared for Office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH., 1982.
10. Bechtel Group, Inc. and Low Engineering Testing Company, "Site Characterization Plan - Gulf Cost Salt Domes," Prepared for office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH., 1983.

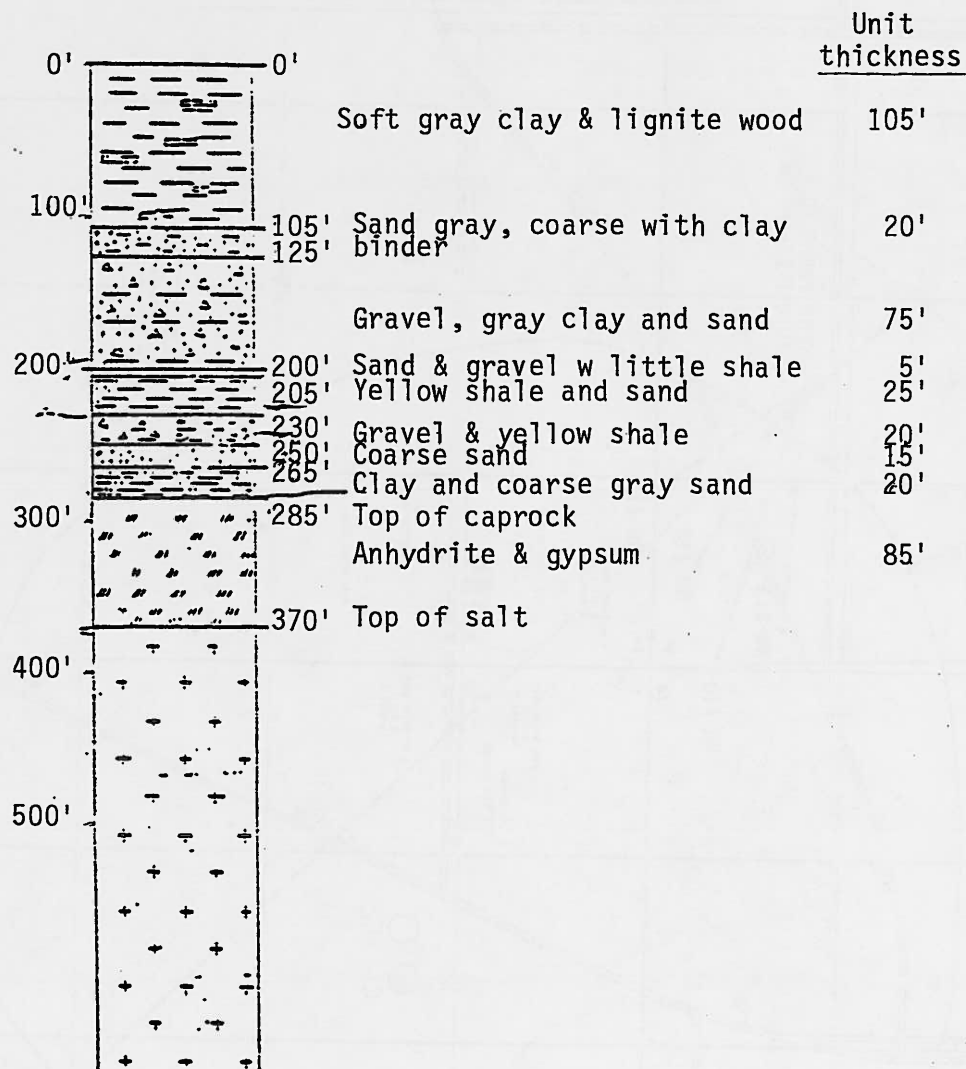


Figure 1. The typical geological section for the McIntosh salt dome.

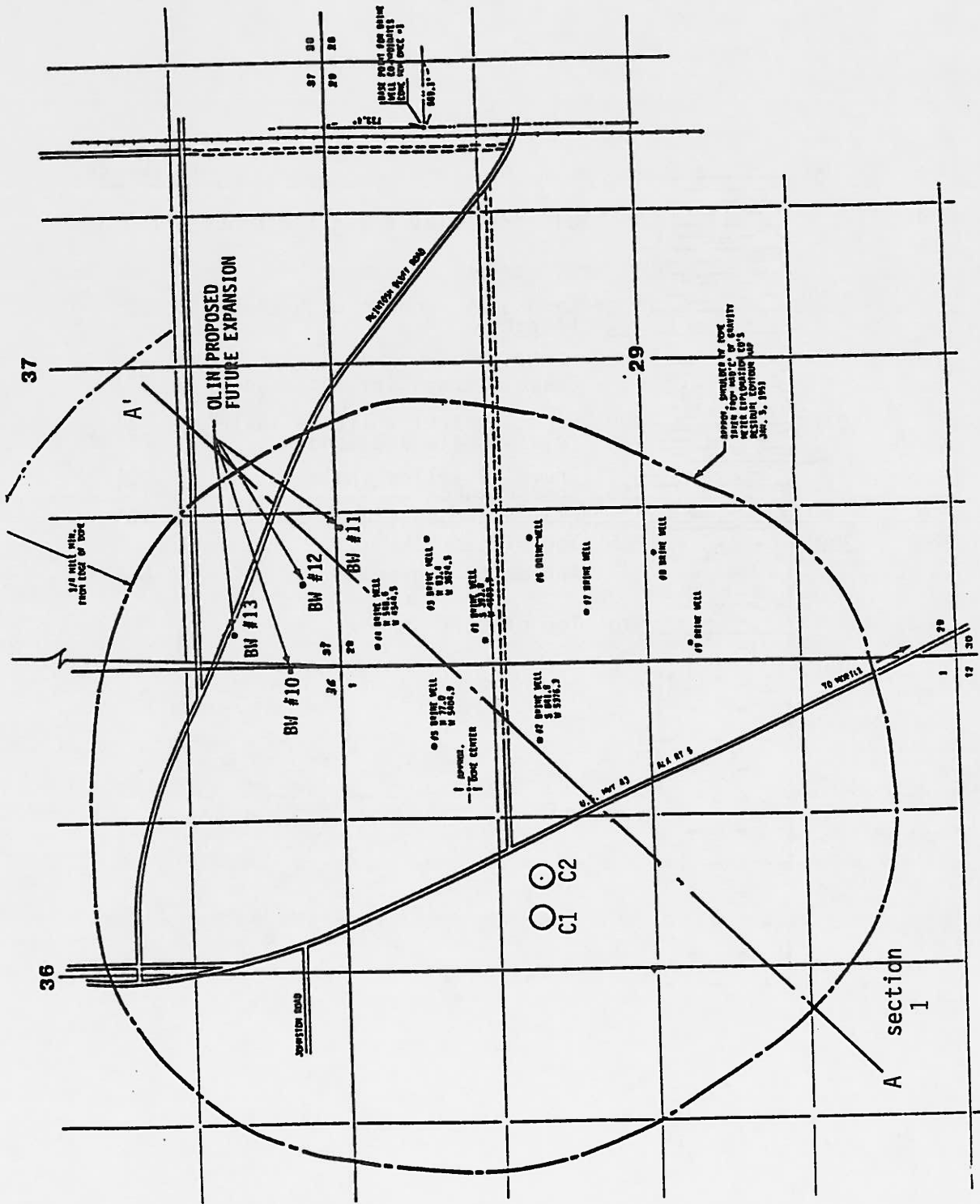
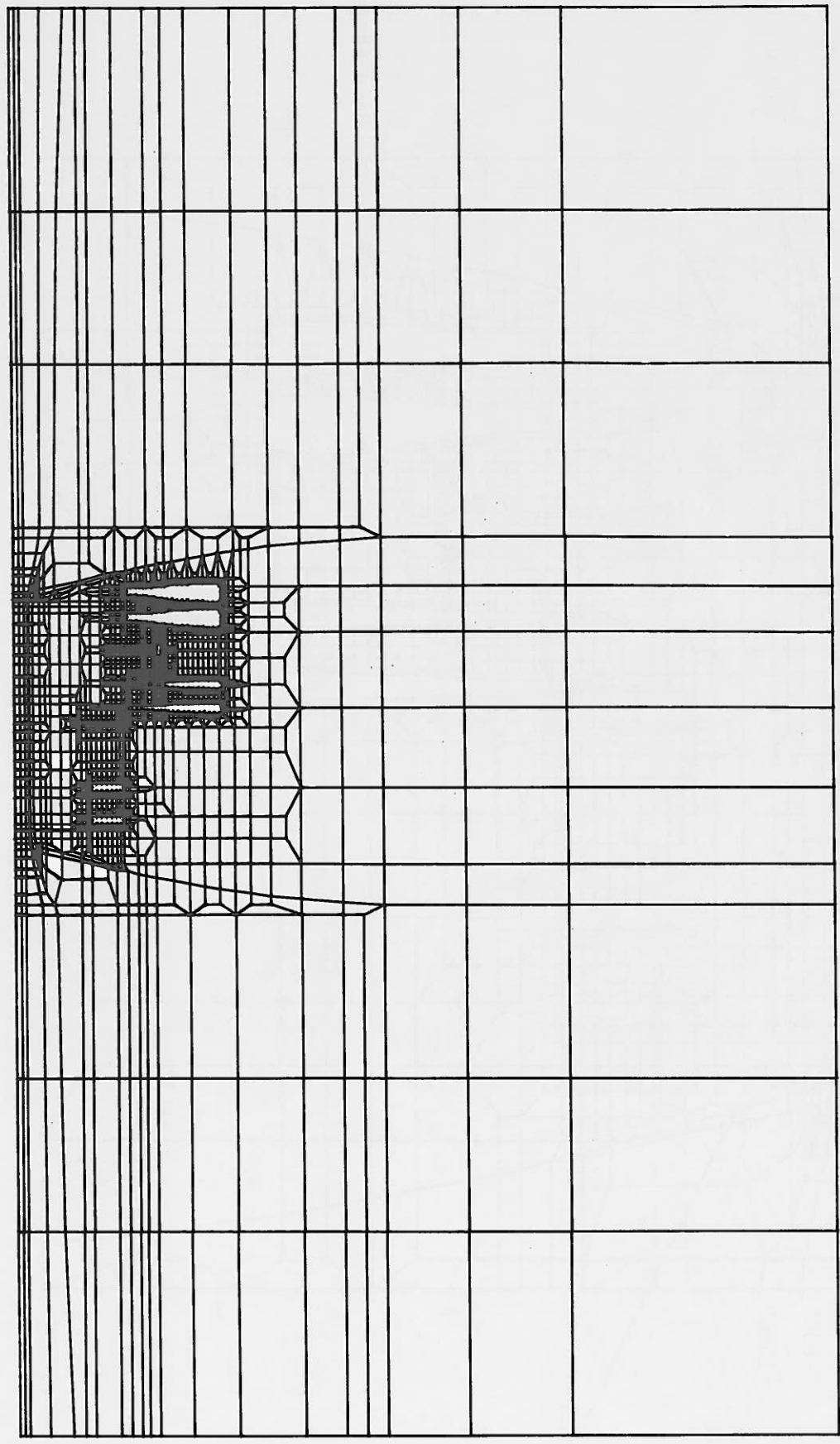


Figure 2. The plan view of the McIntosh cavern field and the boundary of the salt dome.

28000'

0



-16000'

Figure 3. The complete finite element network of 8 - cavern system for the vertical section.

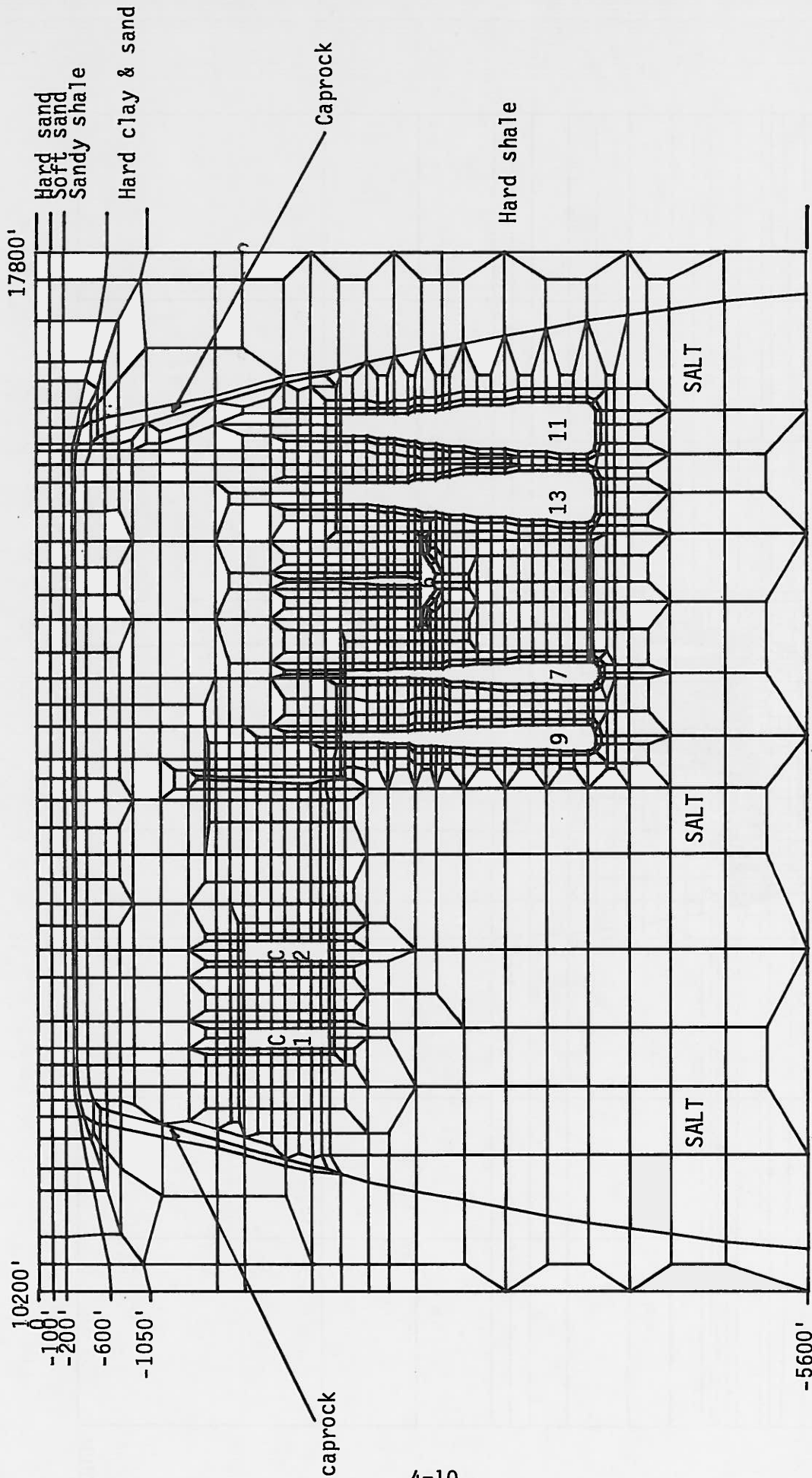


Figure 4. Close-up of the finite element network of 8 - cavern system illustrating the cavern geometry and the geological formations for the vertical section.

FG, KL : Roofs
 HI, MN : Floors
 CD, OE : Pillars
 AB : Abutment

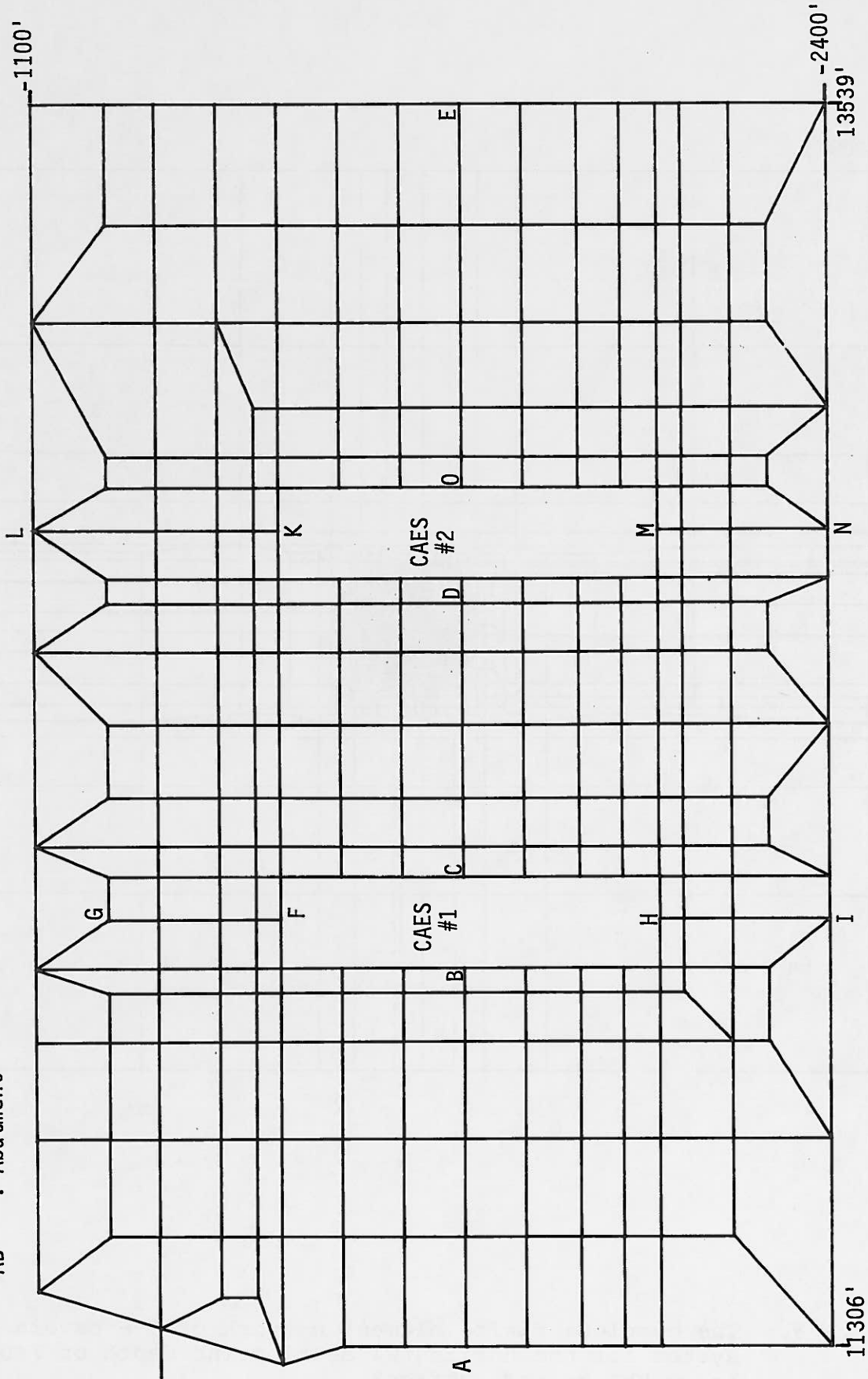


Figure 5. Close-up of the finite element mesh of CAES 2-cavern system for the vertical section.

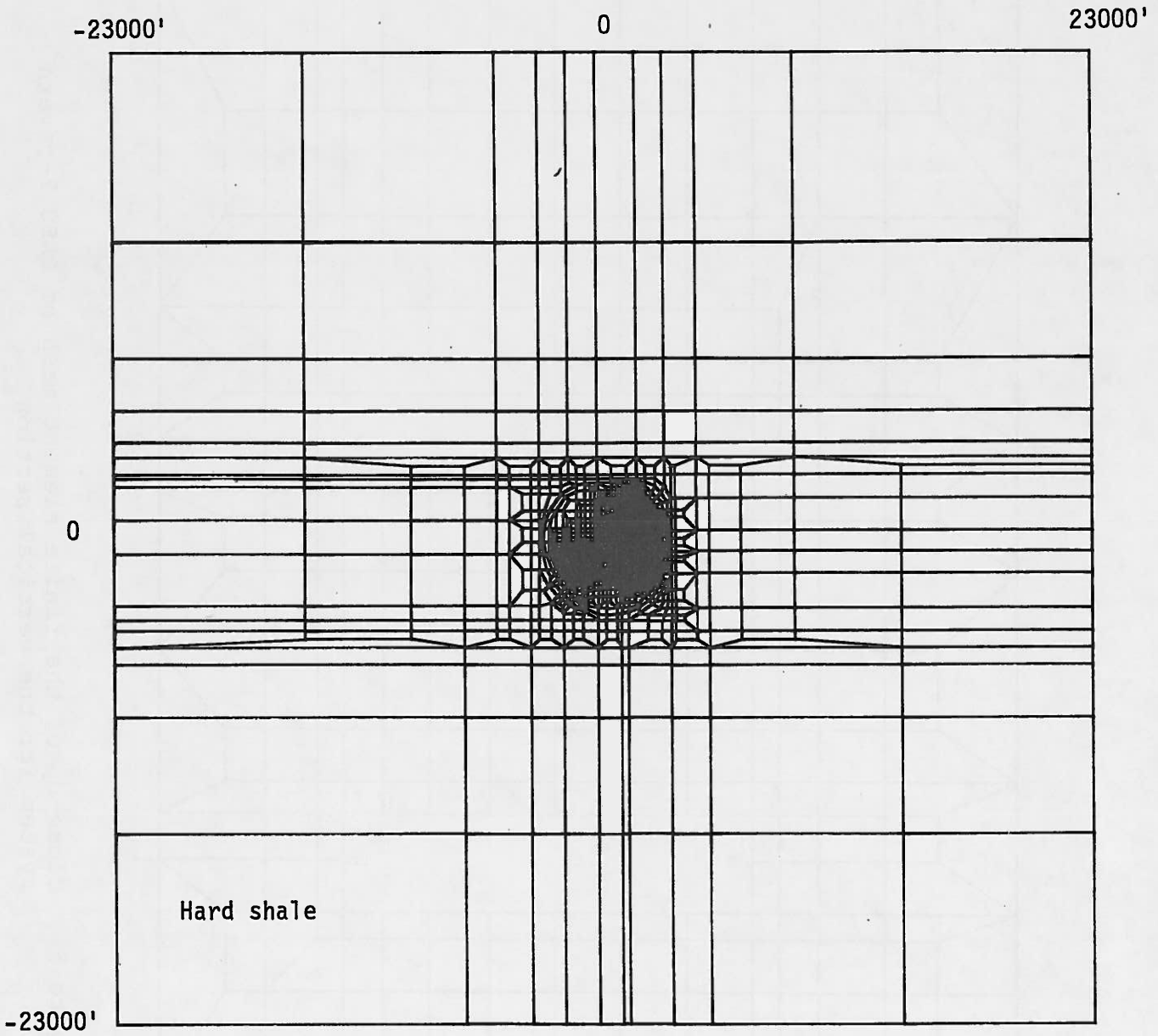


Figure 6. The complete finite element network of 8 - cavern system for the horizontal section (at depth of 2400 ft below the ground surface).

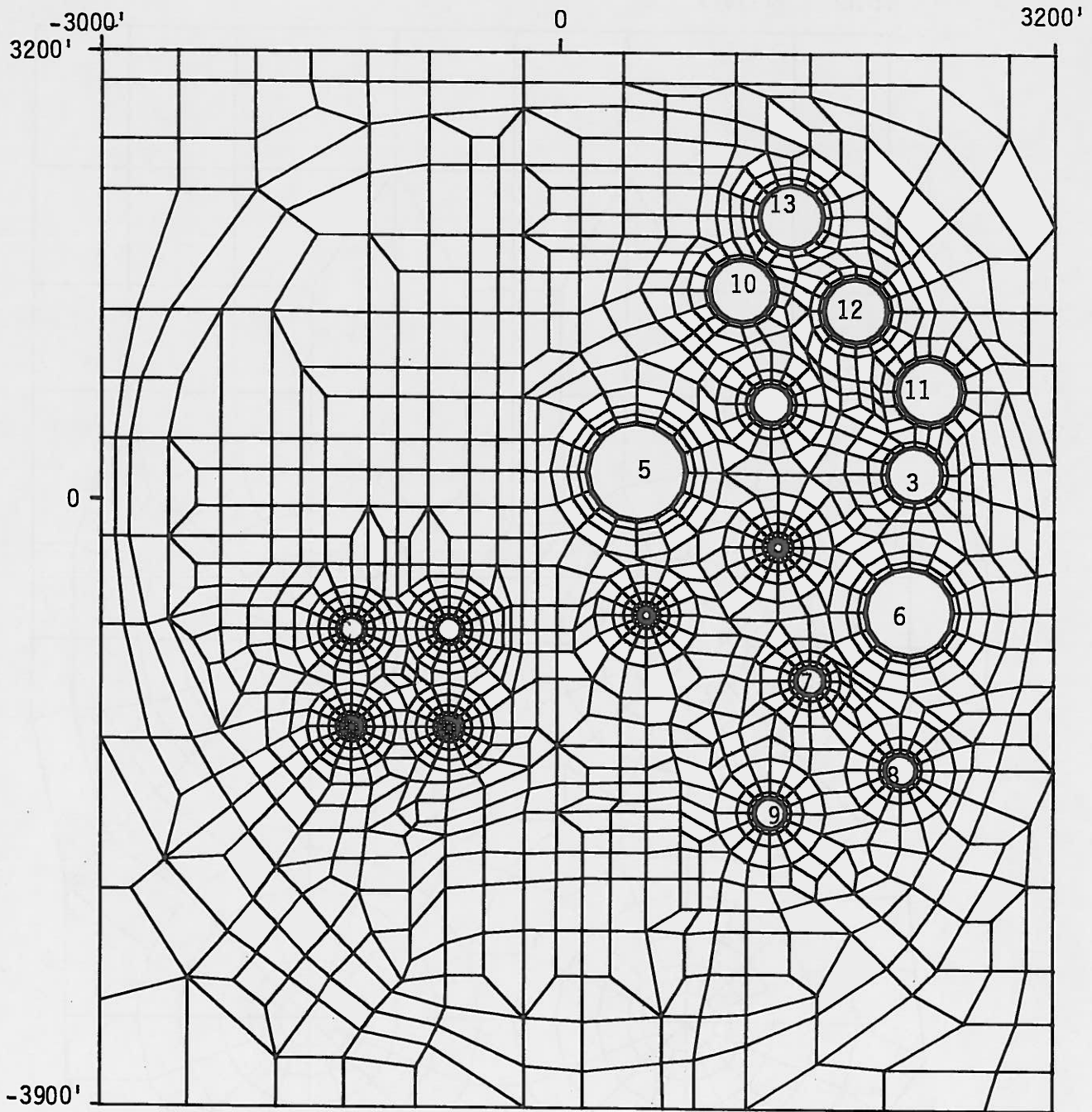


Figure 7. Close-up of the finite element network of 15 - cavern system illustrating the cavern geometry and the geological formation for the horizontal section.

AB : Abutment
CD,EF : Pillars

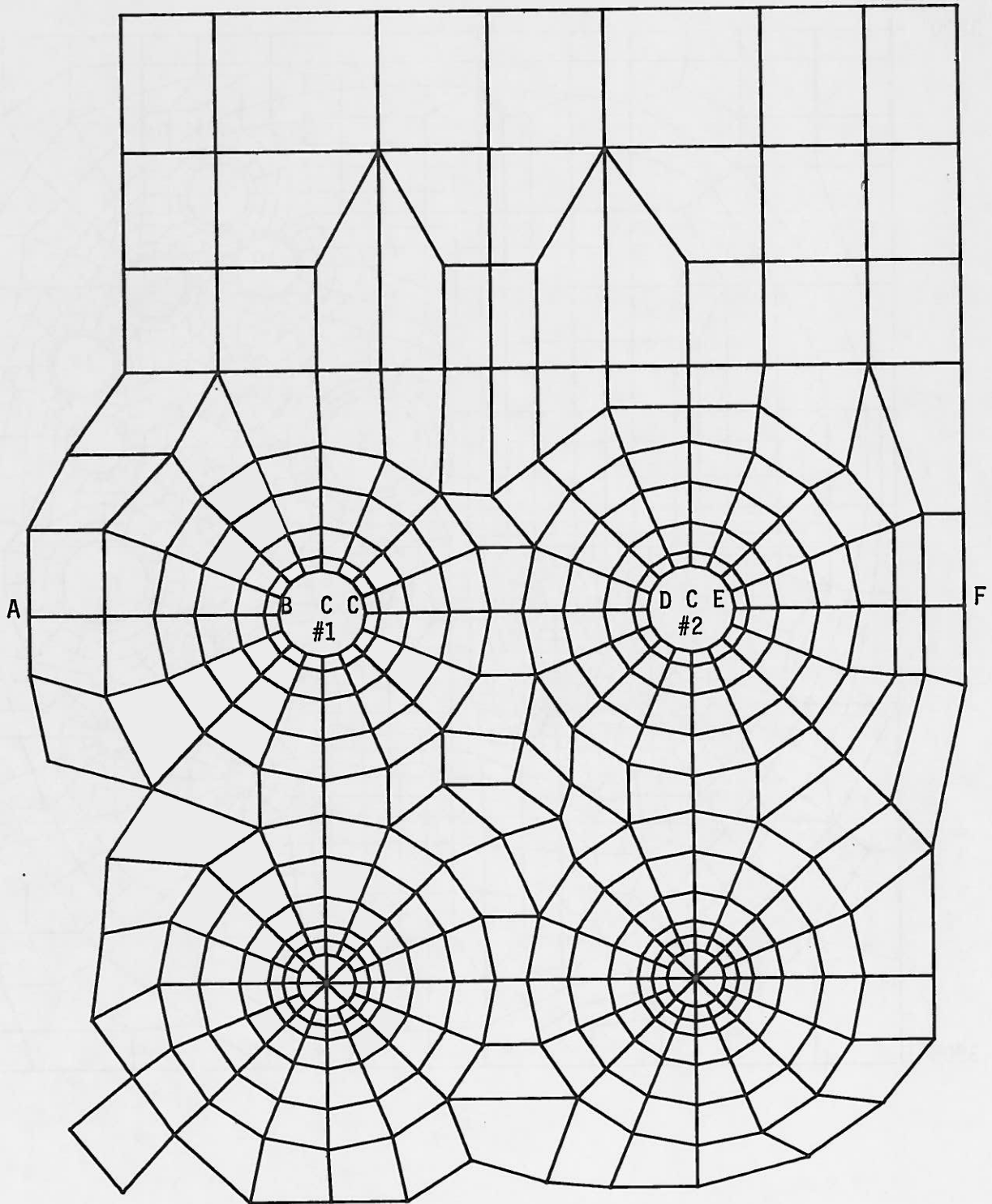


Figure 8. Close-up of the finite element mesh of CAES 2 - cavern system for the horizontal section.

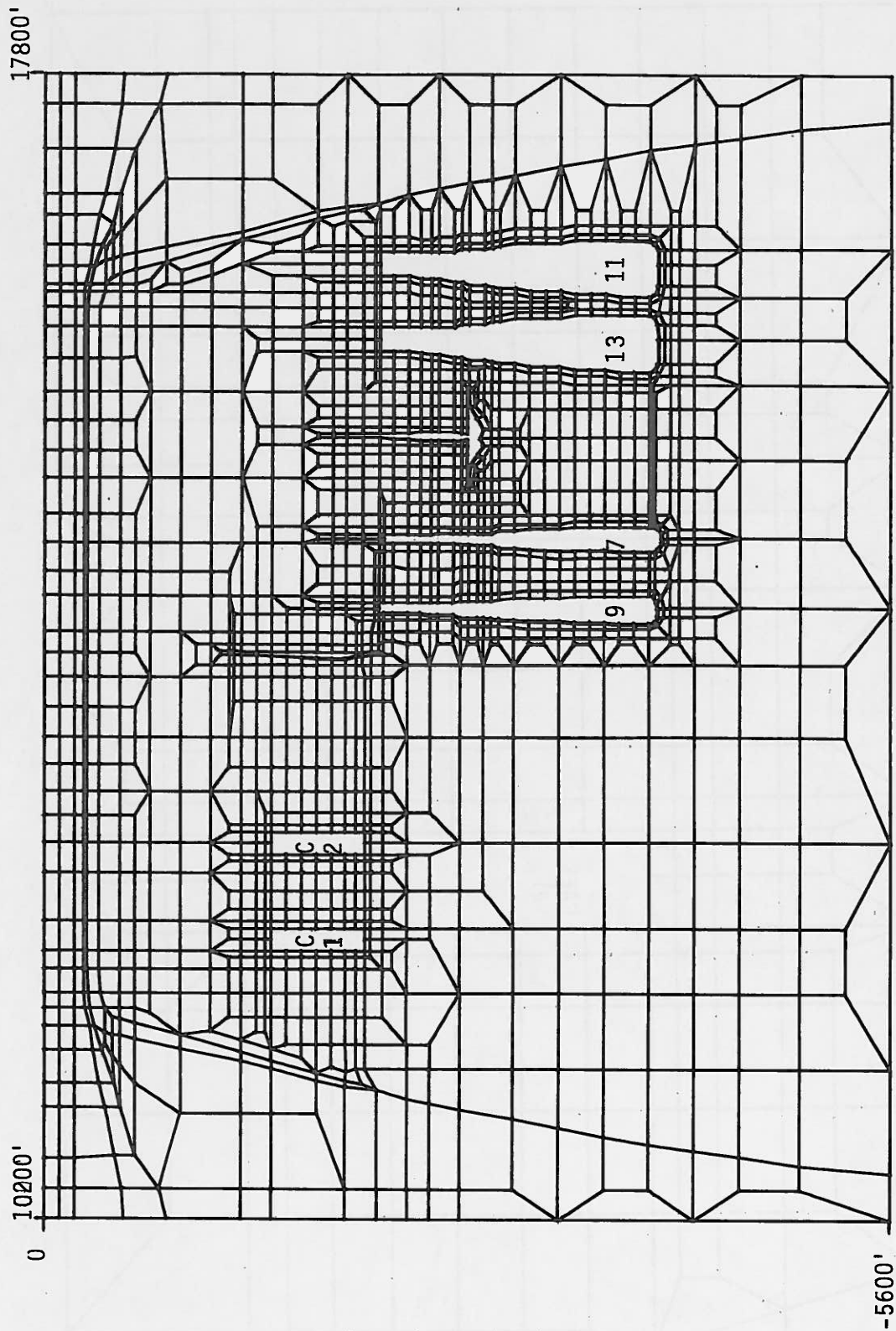


Figure 9. Displacement pattern around caverns C1, C2, 2, 6, 7, 9, 11 and 13 after 60 years of excavation -- the vertical norm run (magnification x 2).

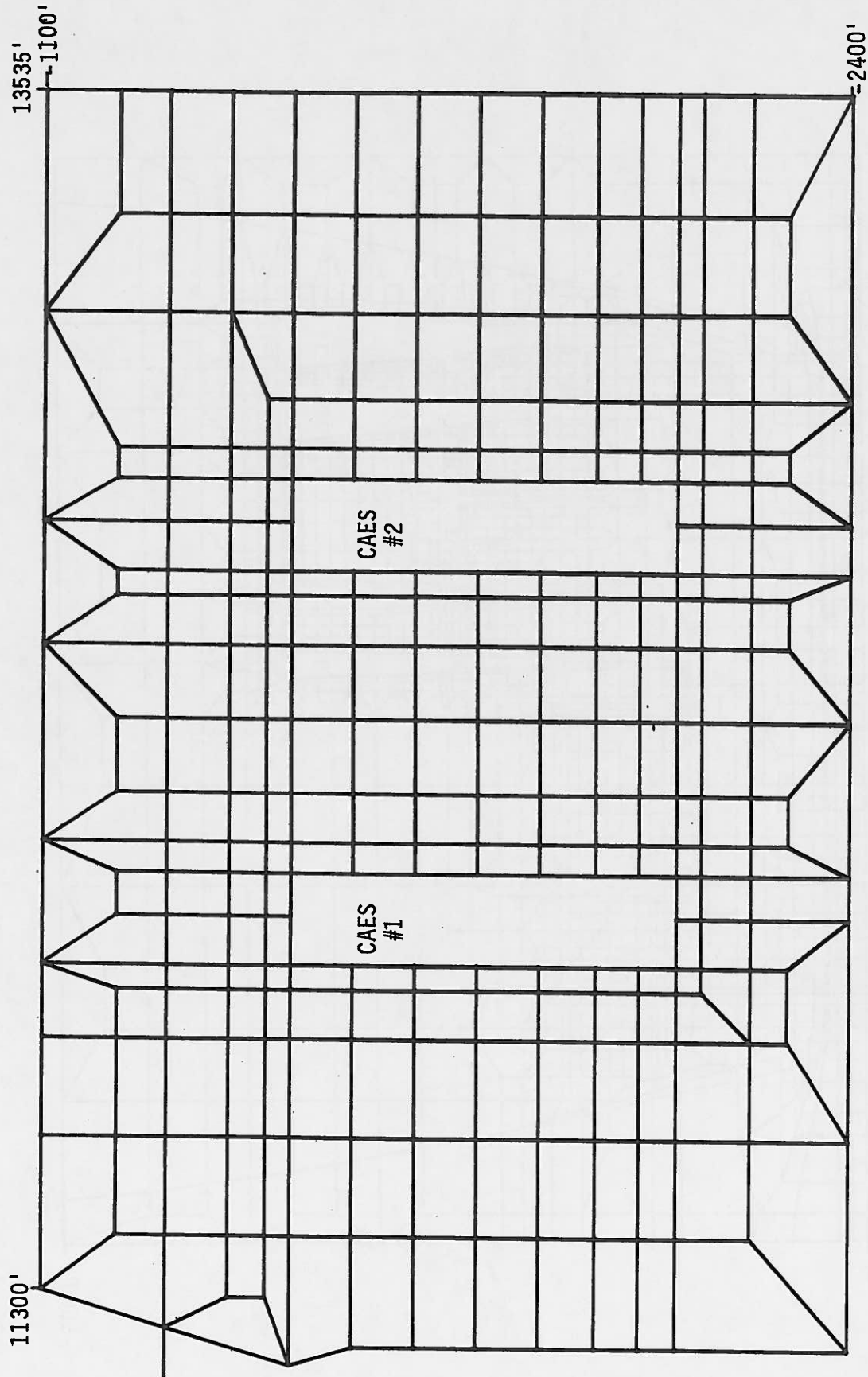


Figure 10. Close-up of the displacement pattern around CAES caverns 1 and 2 after 30 years of excavation -- the vertical norm run (magnification x 2).

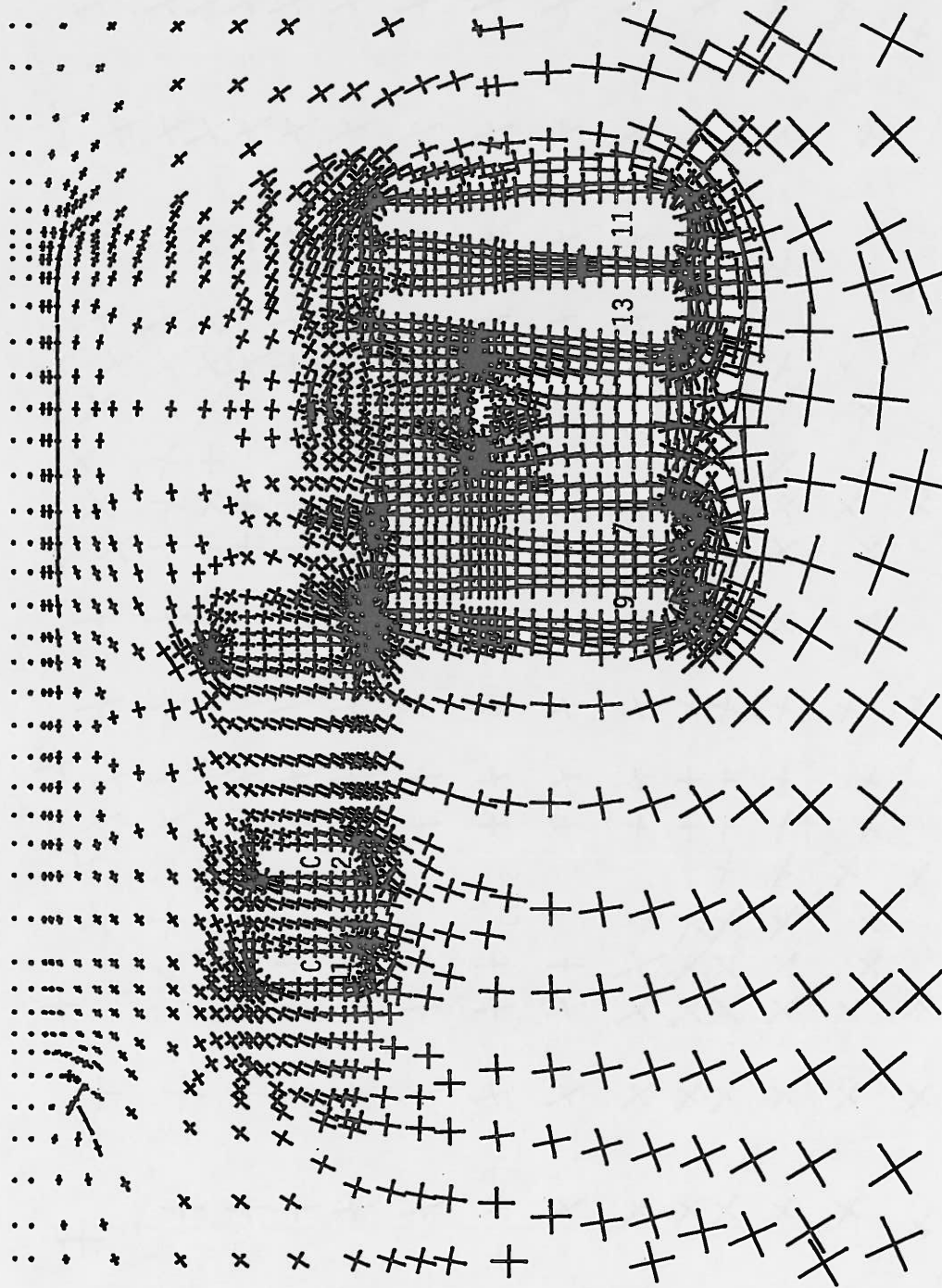


Figure 11. The principal stress distribution pattern around caverns C1, C2, 2, 6, 7, 9, 11 and 13 after 60 years of excavation -- the vertical norm run (scale: 1"=15,000 psi).

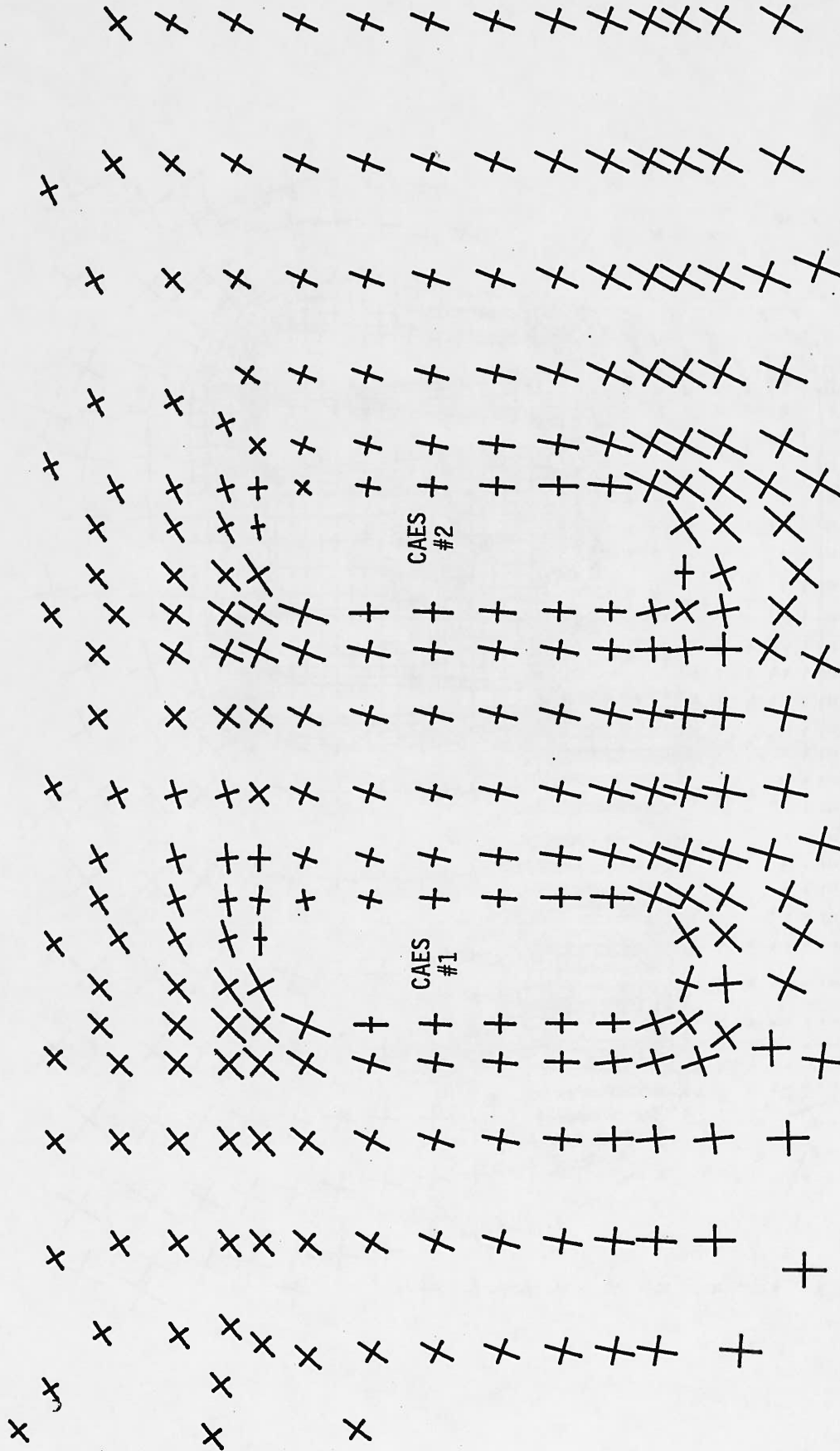


Figure 12. Close-up of the principal stress distribution pattern around CAES caverns 1 and 2 after 30 years of excavation -- the vertical norm run (scale: 1"=8000 psi).

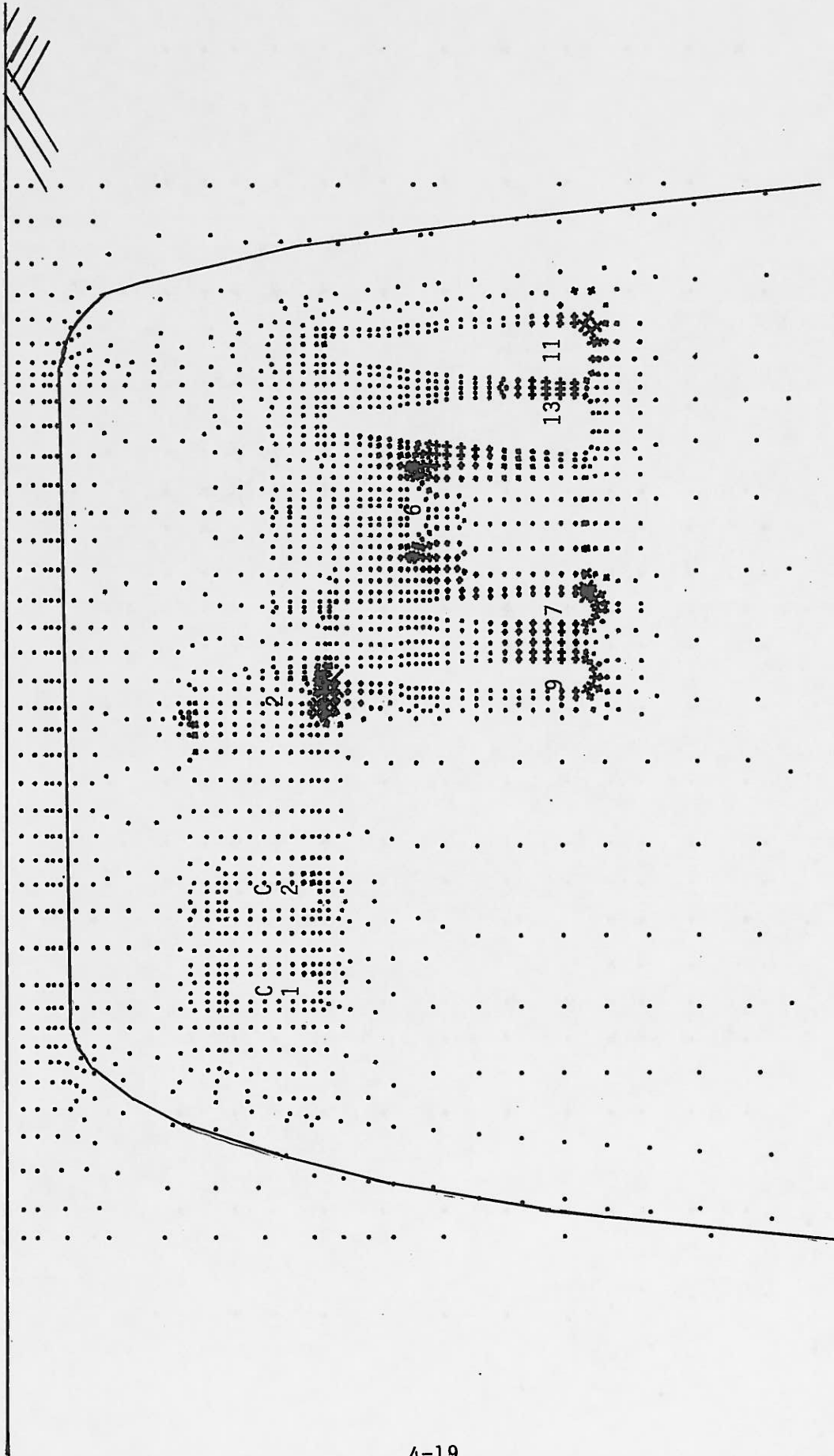


Figure 13. The principal strain distribution pattern around caverns C1, C2, 2, 6, 7, 9, 11 and 13 after 60 years of excavation -- the vertical norm run (scale: 1"==25%).

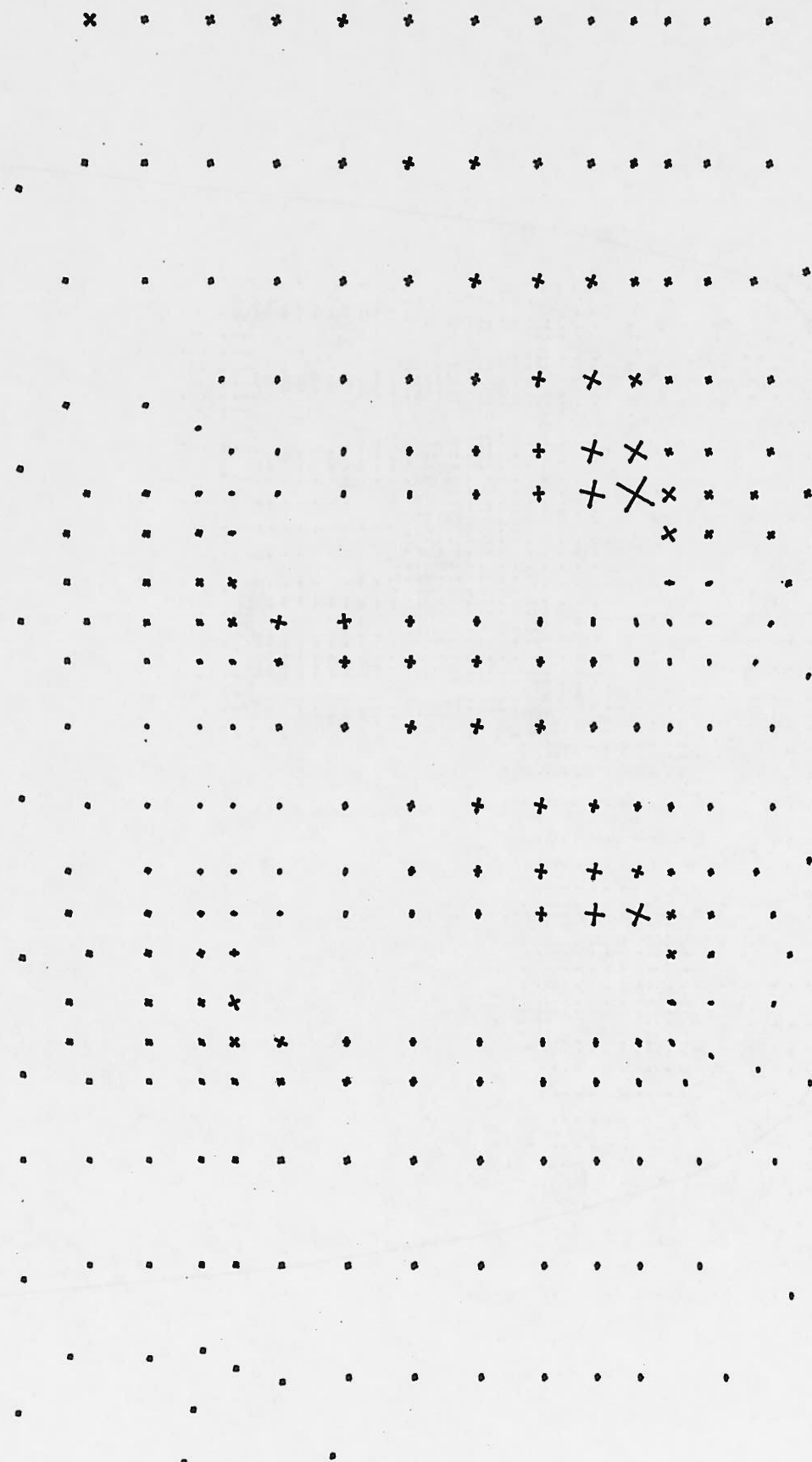


Figure 14. Close-up of the principal strain distribution pattern around CAES caverns 1 and 2 after 30 years of excavation -- the vertical norm run (scale: 1"=25%).

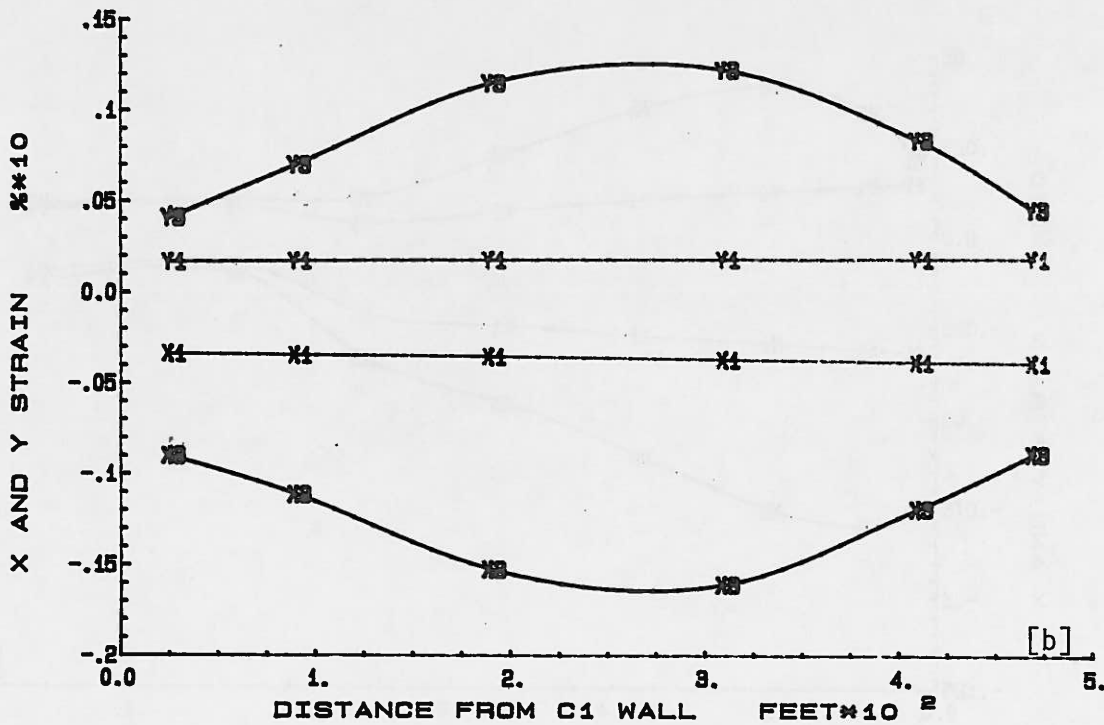
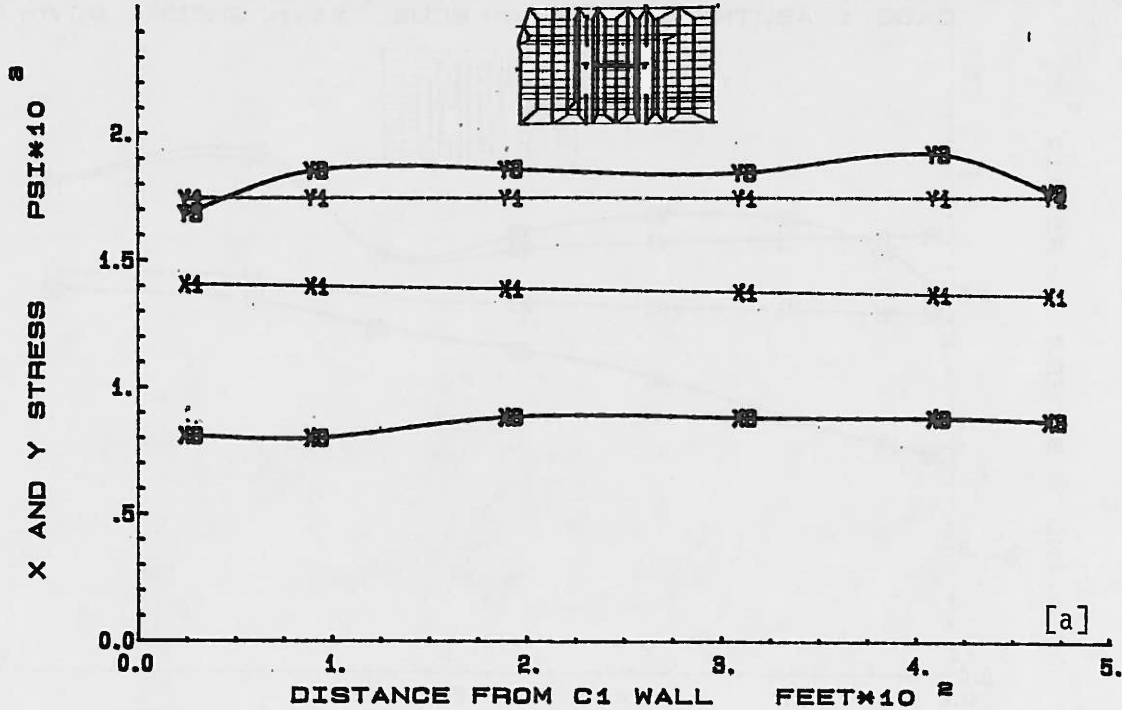


Figure 15. Variation of the horizontal and the vertical stress and strain into the pillar between CAES caverns 1 and 2 -- the vertical norm run.

EPRI VERTICAL PLANE STRAIN RUN 1 MAR 8, 1987
CASE 1 ABUTMENT 30yr: BLUE 31yr: GREEN 80yr: RED

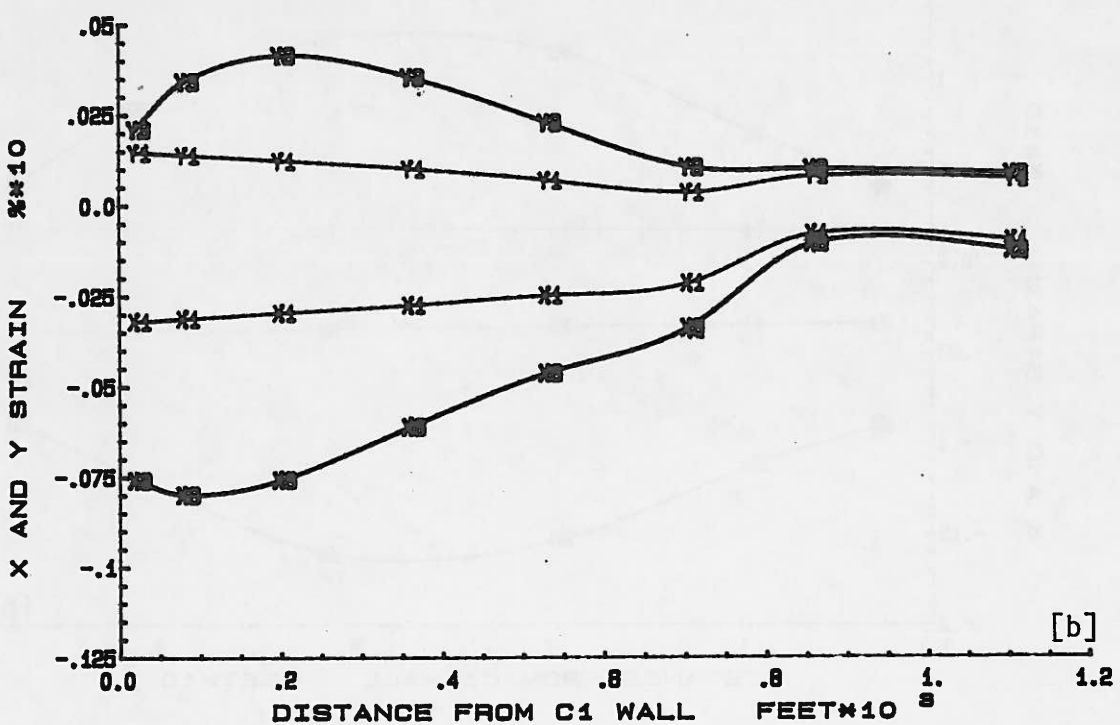
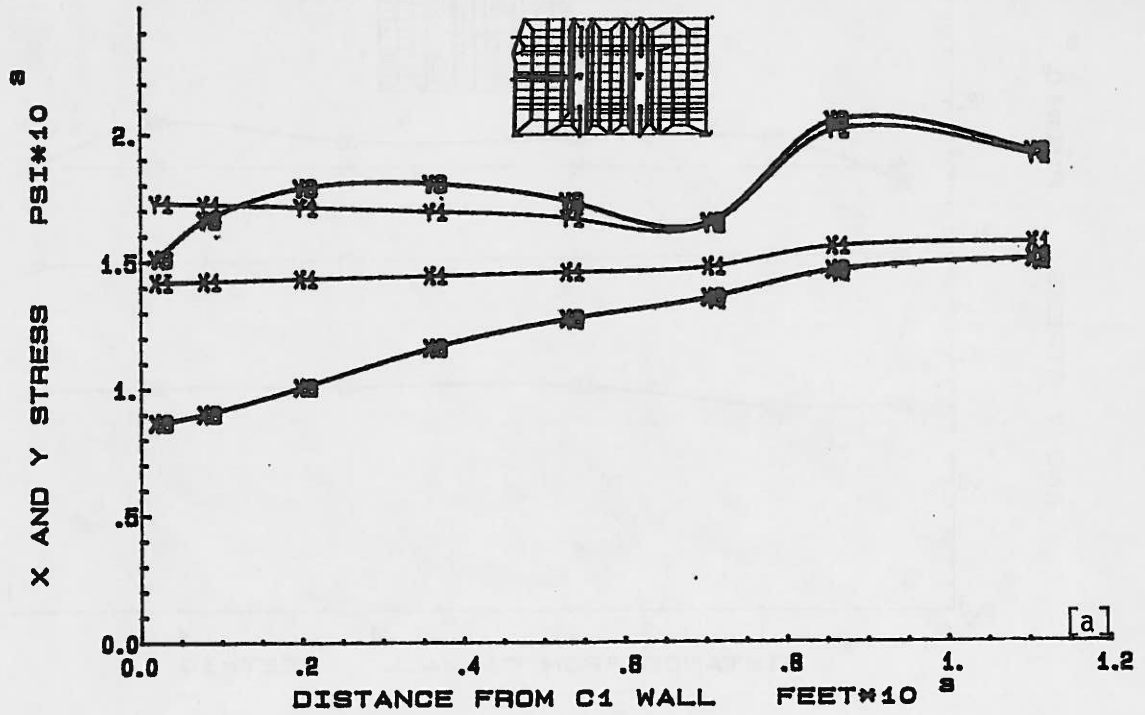


Figure 16. Variation of the horizontal and the vertical stress and strain with time into the abutment of CAES cavern 1 -- the vertical norm run.

EPRI VERTICAL PLANE-STRAIN RUN 1 MAR 10, 1987
 VERTICAL CLOSURE OF CASE 1 (41) & CASE 2 (42)

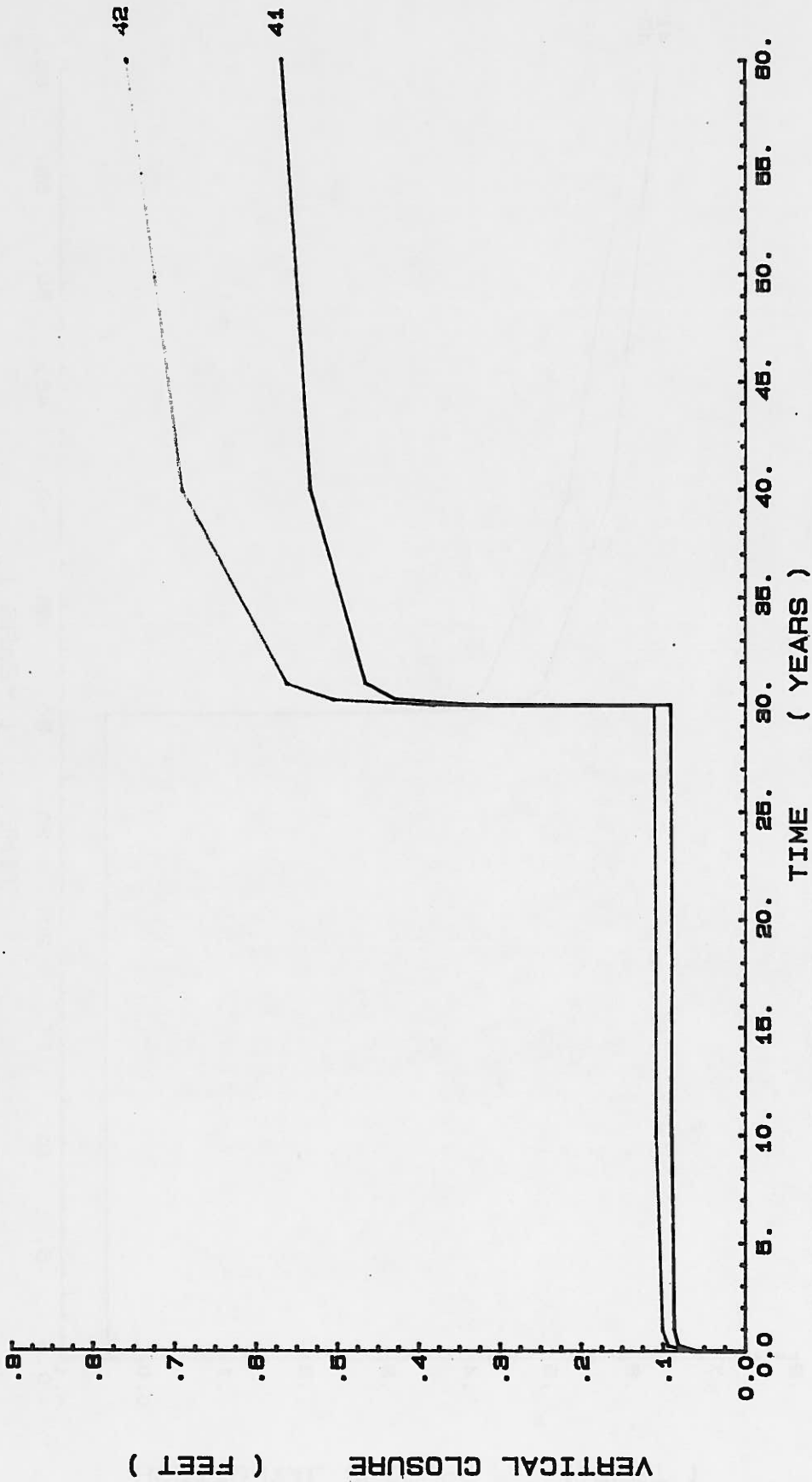


Figure 17 Three vertical closure of CAES caverns 1 and 2 -- the vertical norm run

EPRI VERTICAL PLANE-STRAIN RUN 1 MAR 10, 1987
 HORIZONTAL CLOSURE OF CASE 1 (41) & CASE 2 (42)

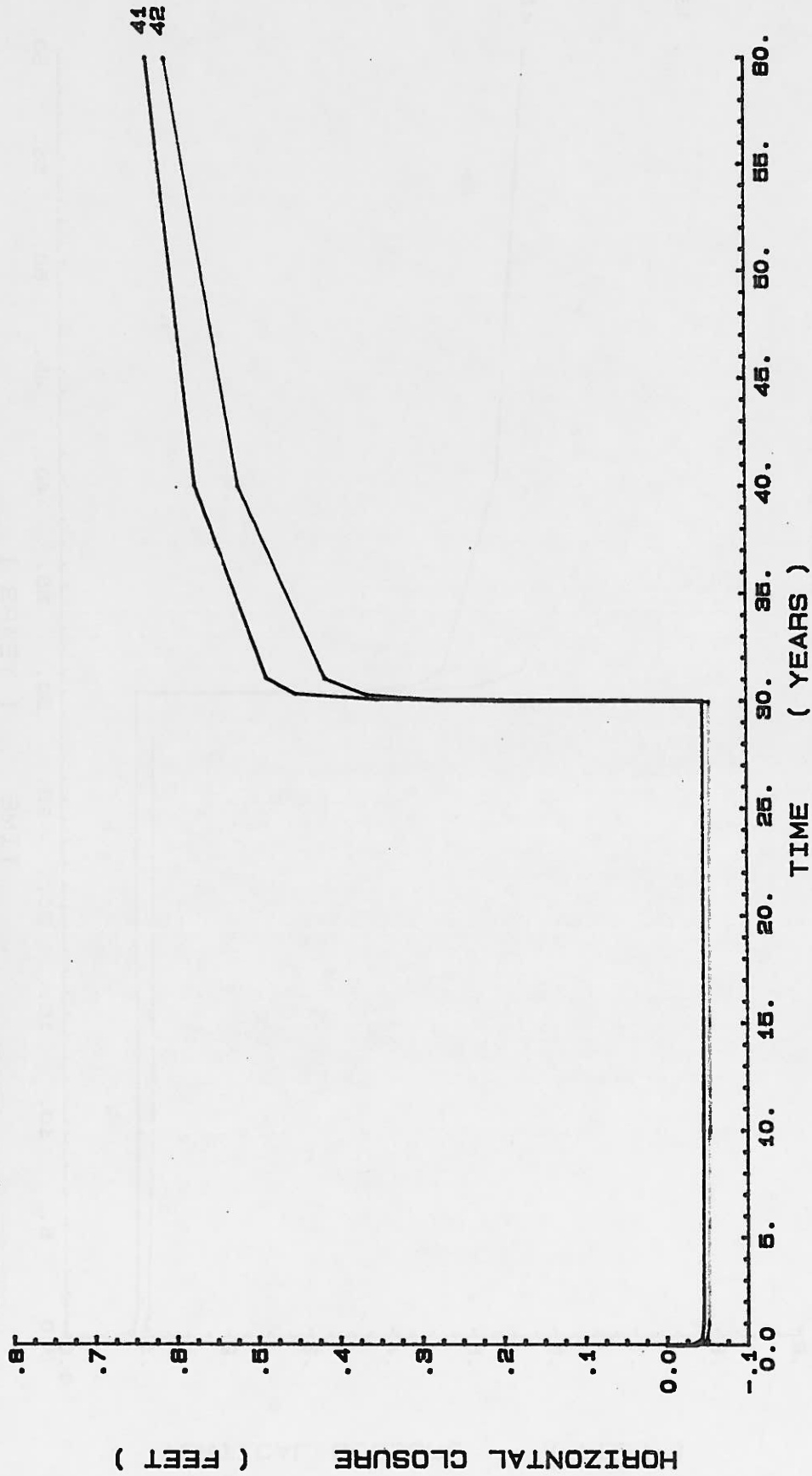


Figure 18. The horizontal closure of CAES caverns 1 and 2 -- the vertical norm run

EPRI VERTICAL PLANE-STRAIN RUN 1 MAR 10, 1987
 SURFACE SUBSIDENCE OVER CASE 1 (41) & CASE 2 (42)

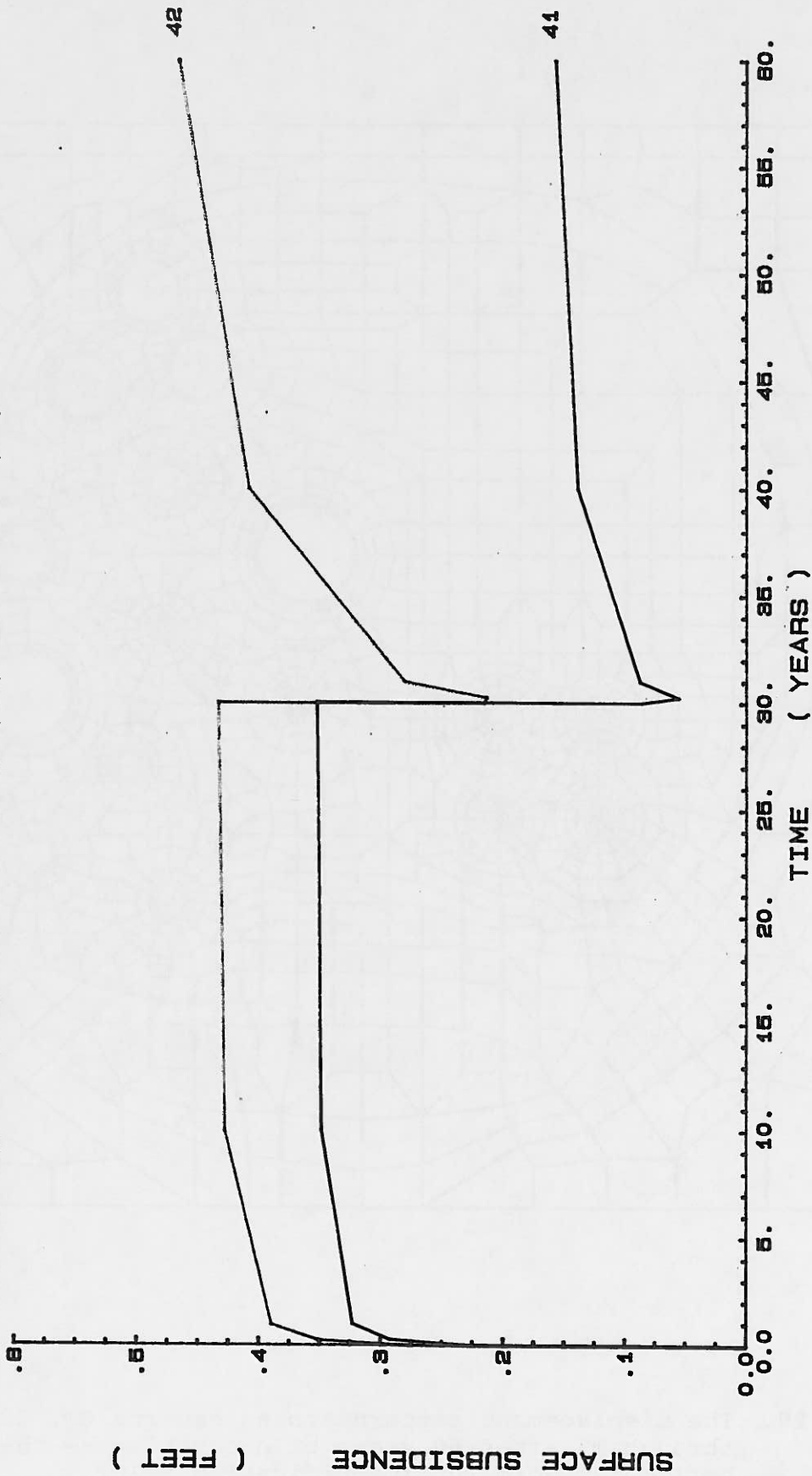


Figure 19. The surface subsidence over CAES caverns 1 and 2 --- the vertical norm run

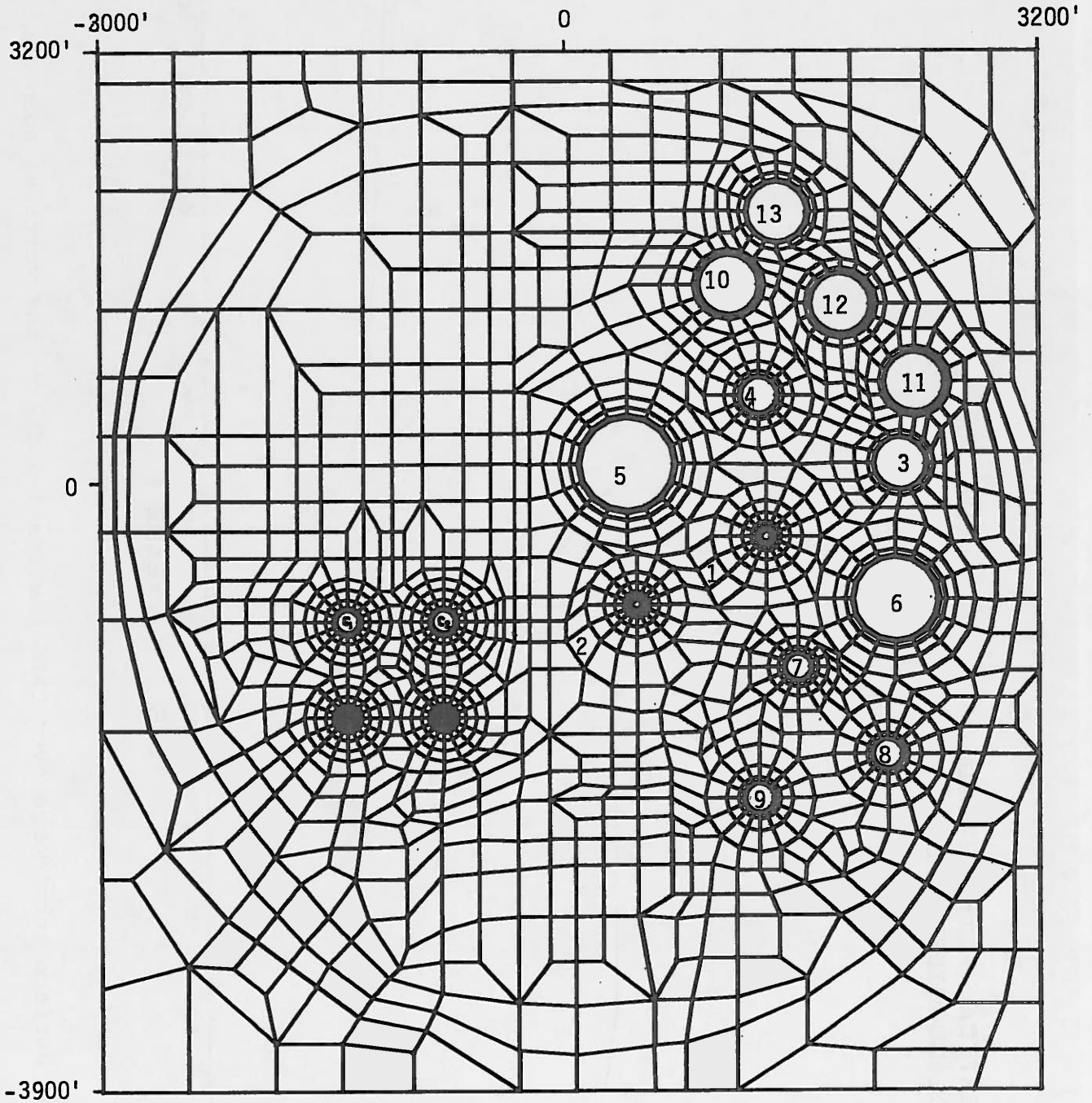


Figure 20. The displacement pattern around caverns C1, C2, and 1 through 13 after 60 years of excavation -- the horizontal norm run (magnification x 10).

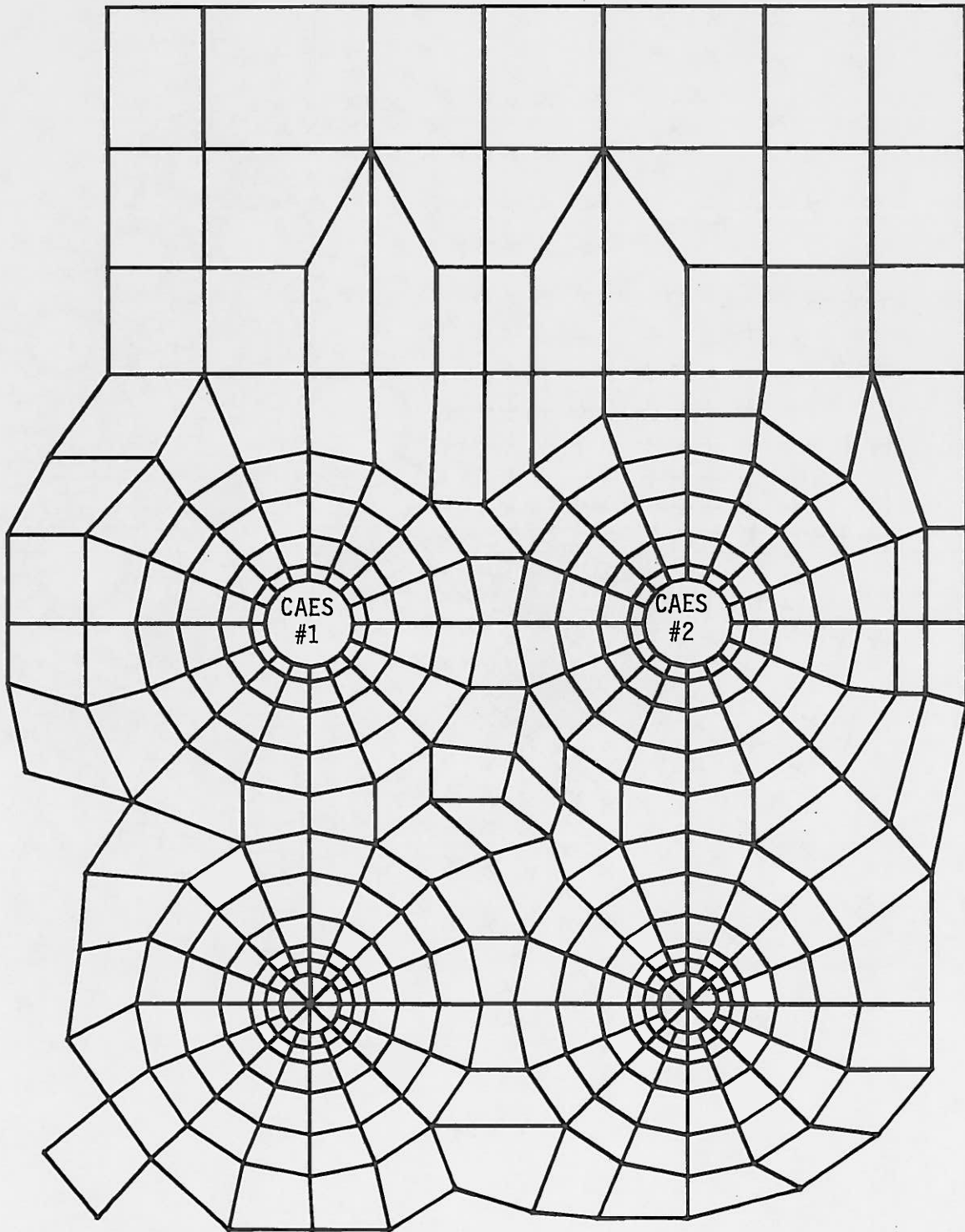


Figure 21. Close-up of the displacement pattern around CAES caverns 1 and 2 after 30 years of excavation -- the horizontal norm run (magnification x 10).

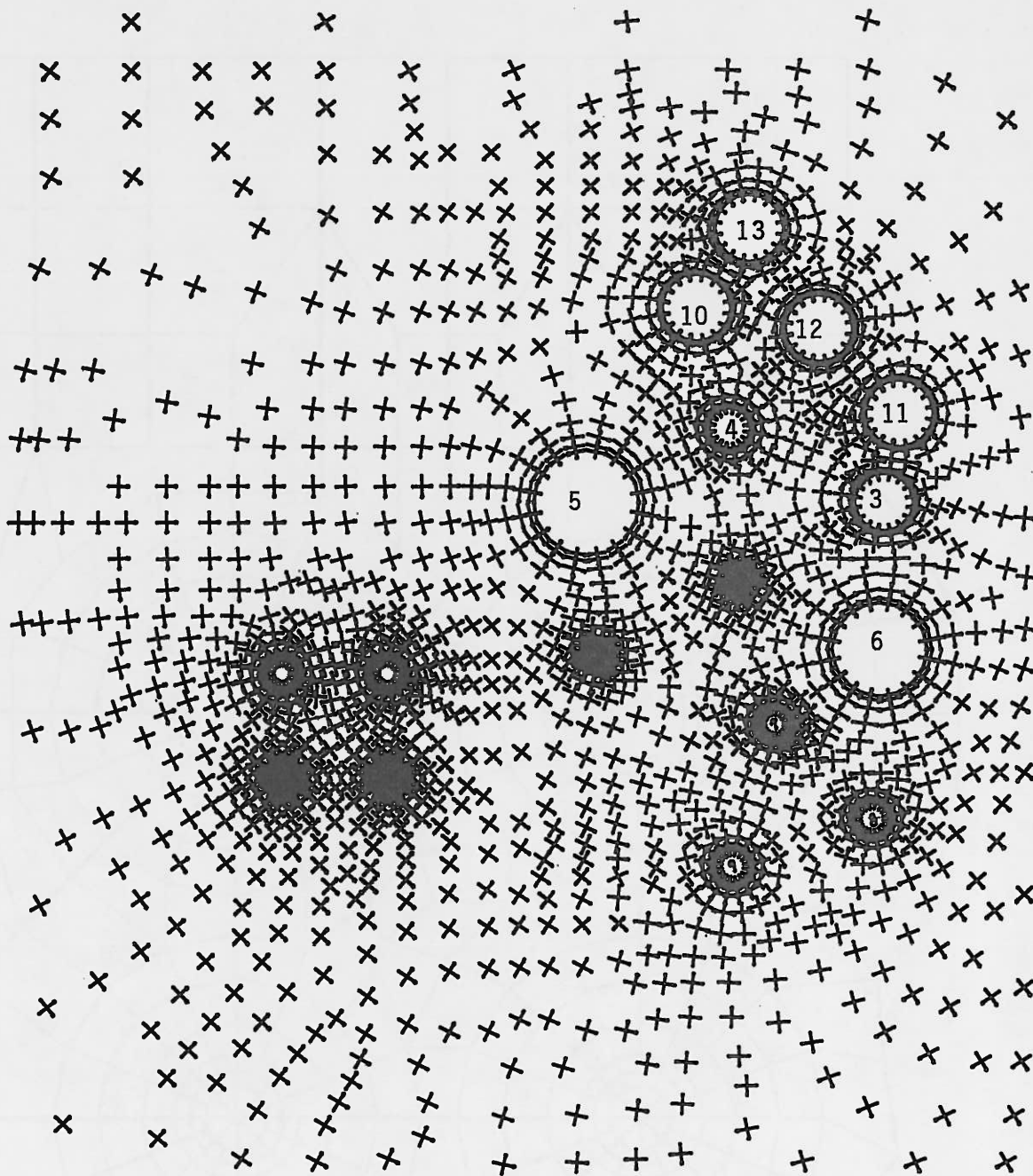


Figure 22. The principal stress distribution pattern around caverns C1, C2 and 1 through 13 after 60 years of excavation (at the depth of 2400 ft below ground surface) -- the horizontal norm run (scale: 1"=20,000 psi).

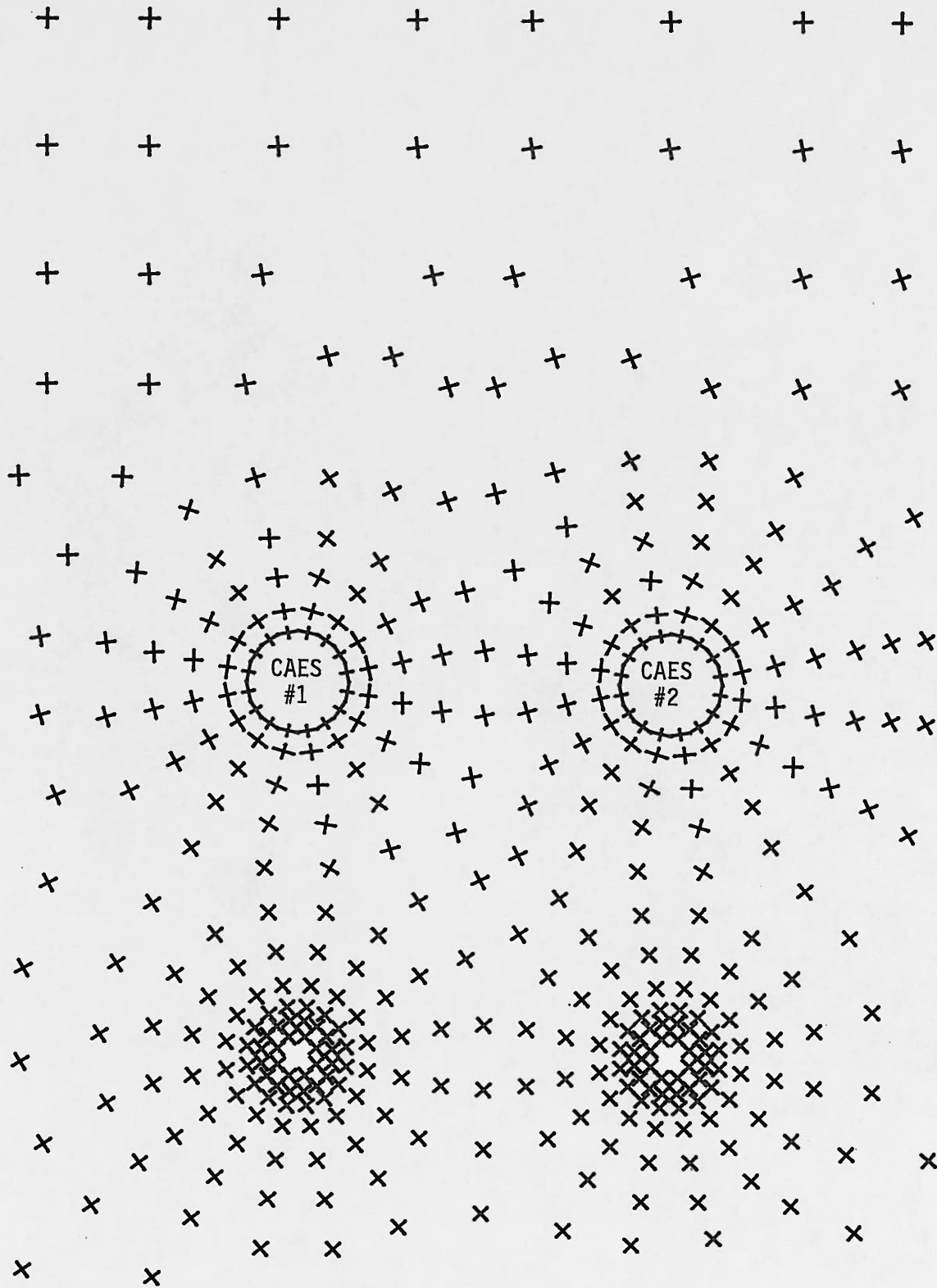


Figure 23. Close-up of the principal stress distribution pattern around CAES caverns 1 and 2 after 30 years of excavation -- the horizontal norm run (scale: 1"=20,000 psi).

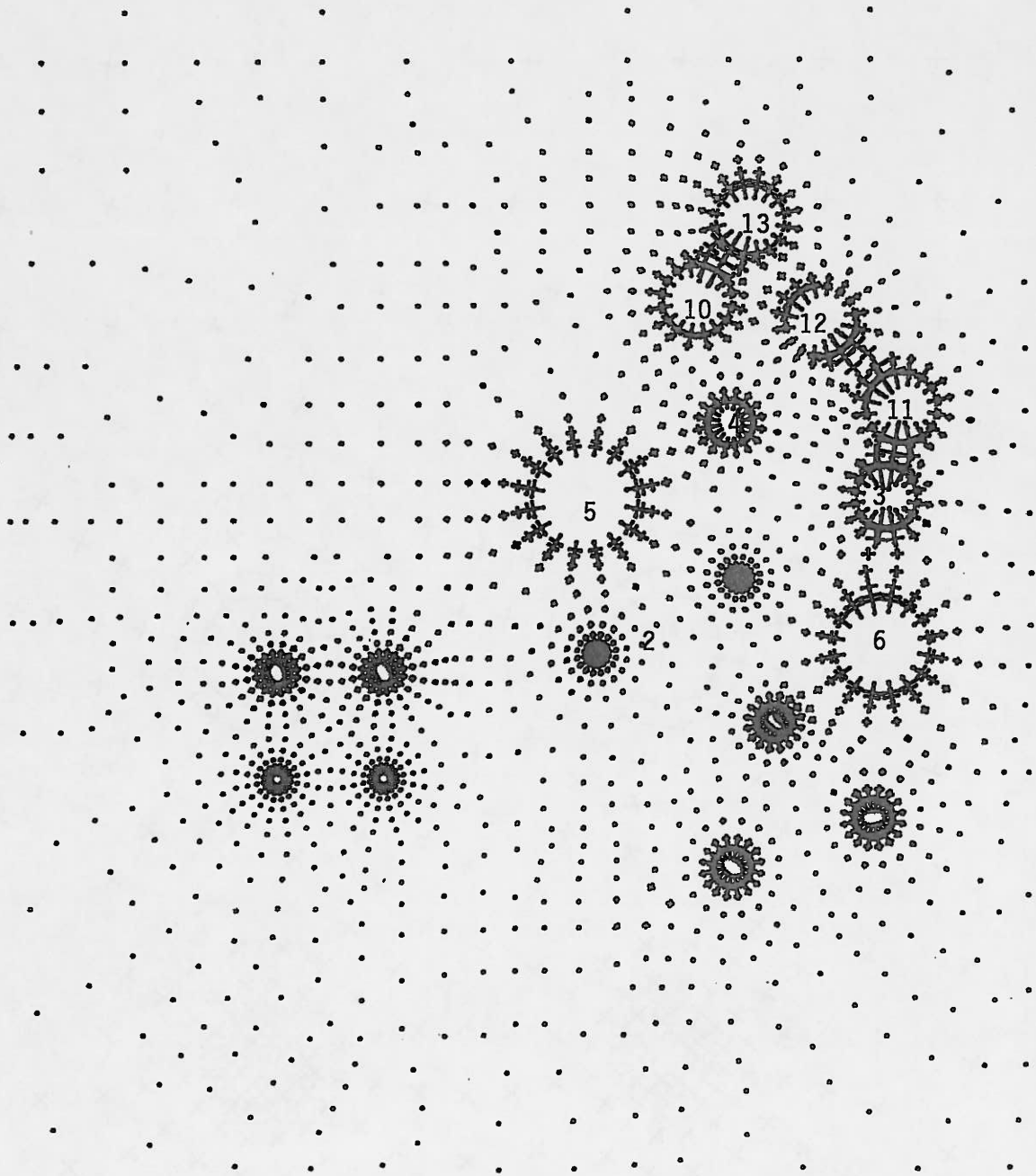


Figure 24. The principal strain distribution pattern around caverns C1, C2 and 1 through 13 after 60 years of excavation -- the horizontal norm run (scale: 1"=2.5%).

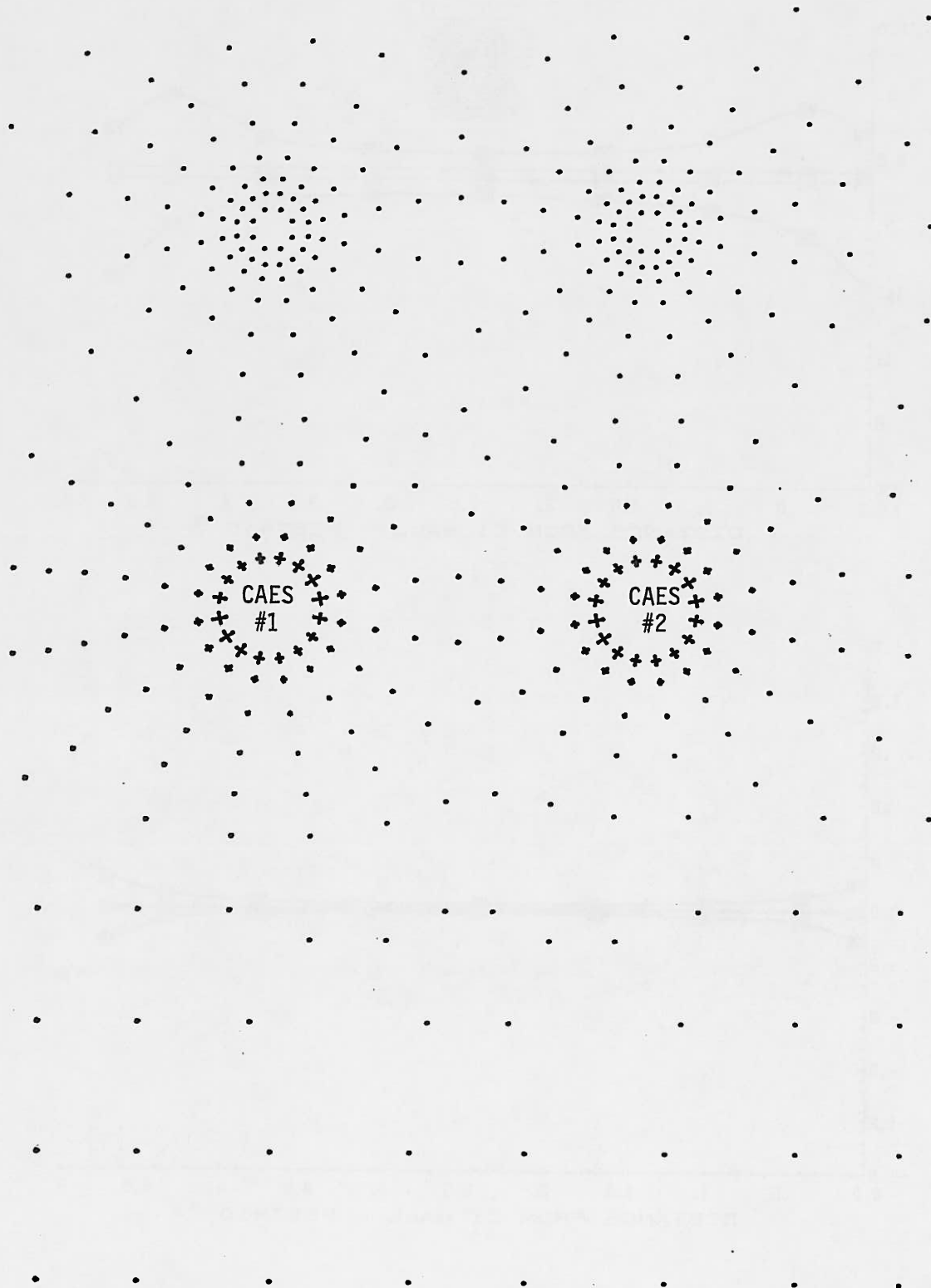


Figure 25. Close-up of the principal strain distribution pattern around CAES caverns 1 and 2 after 30 years of excavation -- the horizontal norm run (scale: 1"=2.5%).

EPRI HORIZONTAL RUN 1
 PILLA BTWN C1&C2 30yr: BLUE 31yr: GREEN 30yr: RED

MAR 8, 1987

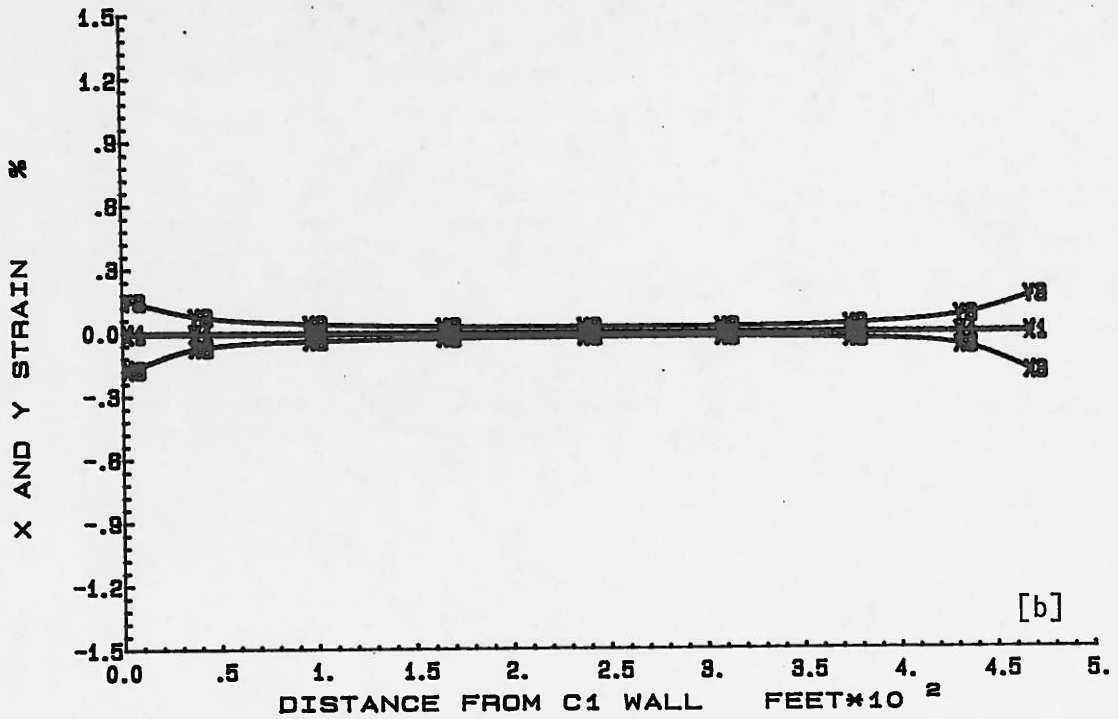
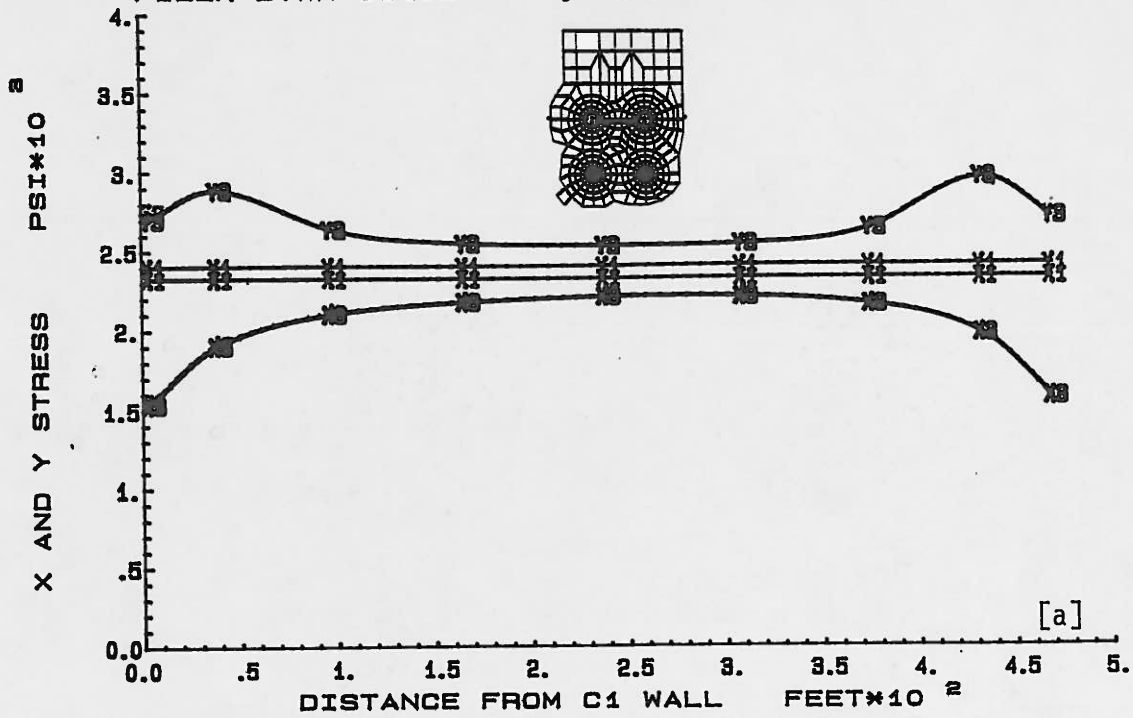


Figure 26. The vertical and the horizontal stress and strain into the pillar between CAES caverns 1 and 2 with time -- the horizontal norm run.

EPRI HORIZONTAL RUN 1
CASE 1 ABUTMENT 30yr: BLUE 31yr: GREEN 60yr: RED
MAR 8, 1987

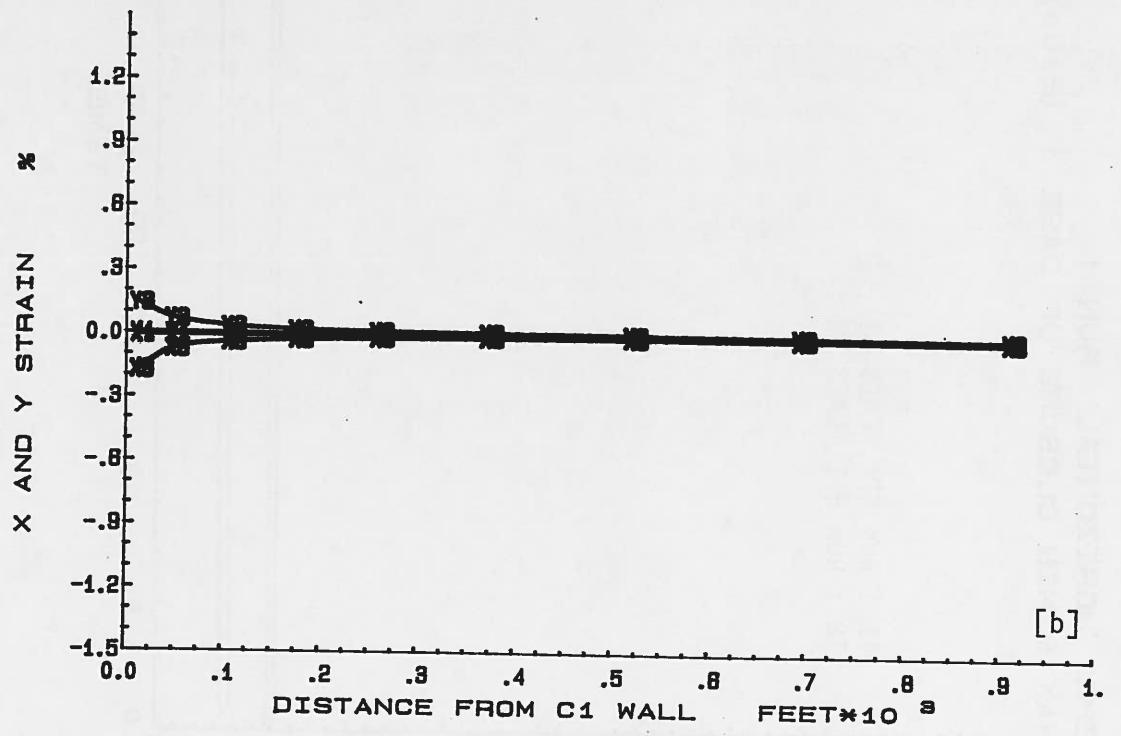
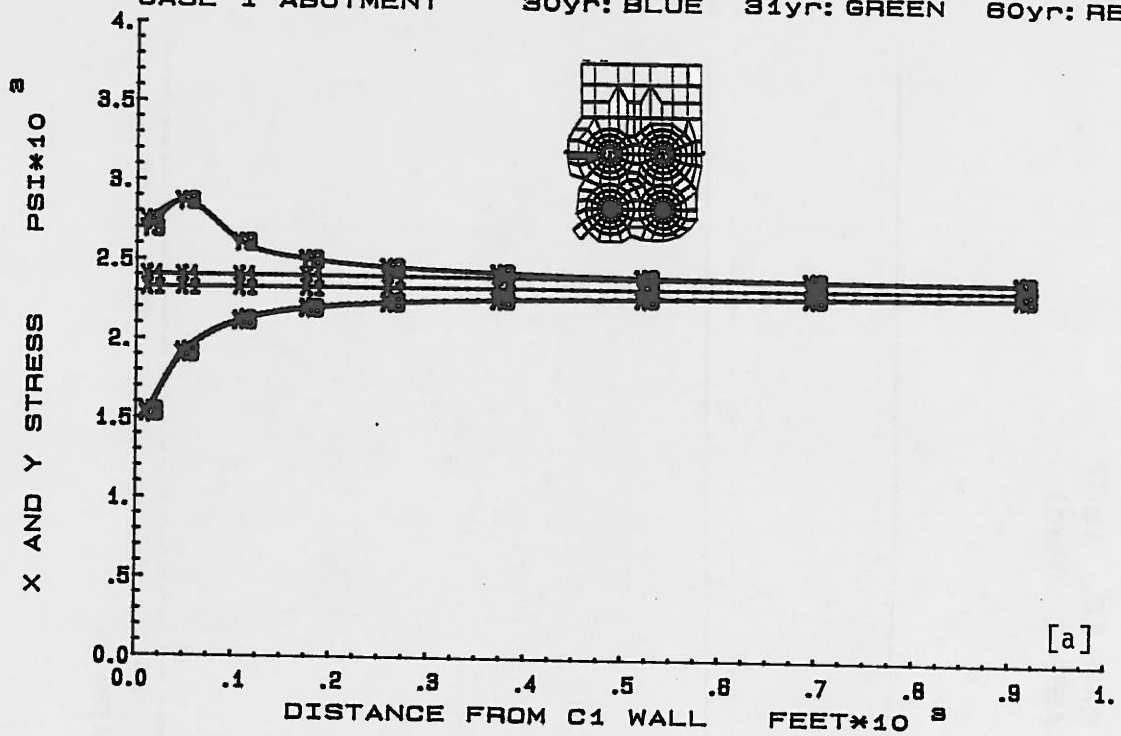


Figure 27. Variation of the horizontal and the vertical stress and strain with time into the abutment of CAES cavern 1 -- the horizontal norm run.

EPRI HORIZONTAL RUN 1
 MAR 10, 1987
 MAX & MIN CLOSURE OF CASE 1 (Blue) & CASE 2 (Red)

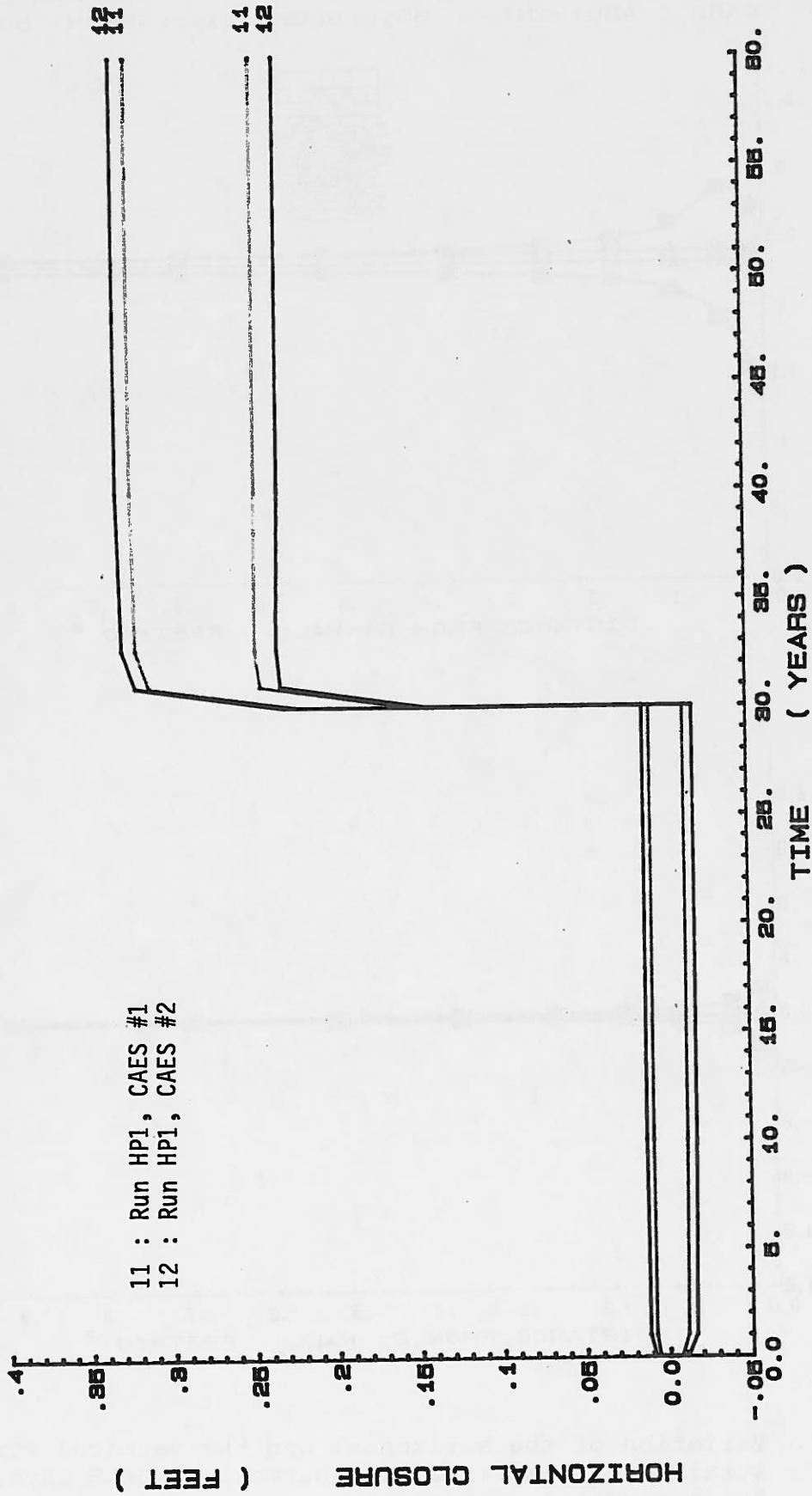


Figure 28. Variation of the maximum and the minimum closures of CAES caverns 1 and 2 with time -- the horizontal norm run.

EPRI HORIZONTAL RUN 1
 CLOSURE OF DOME 30yr: BLUE 31yr: GREEN 30yr: RED

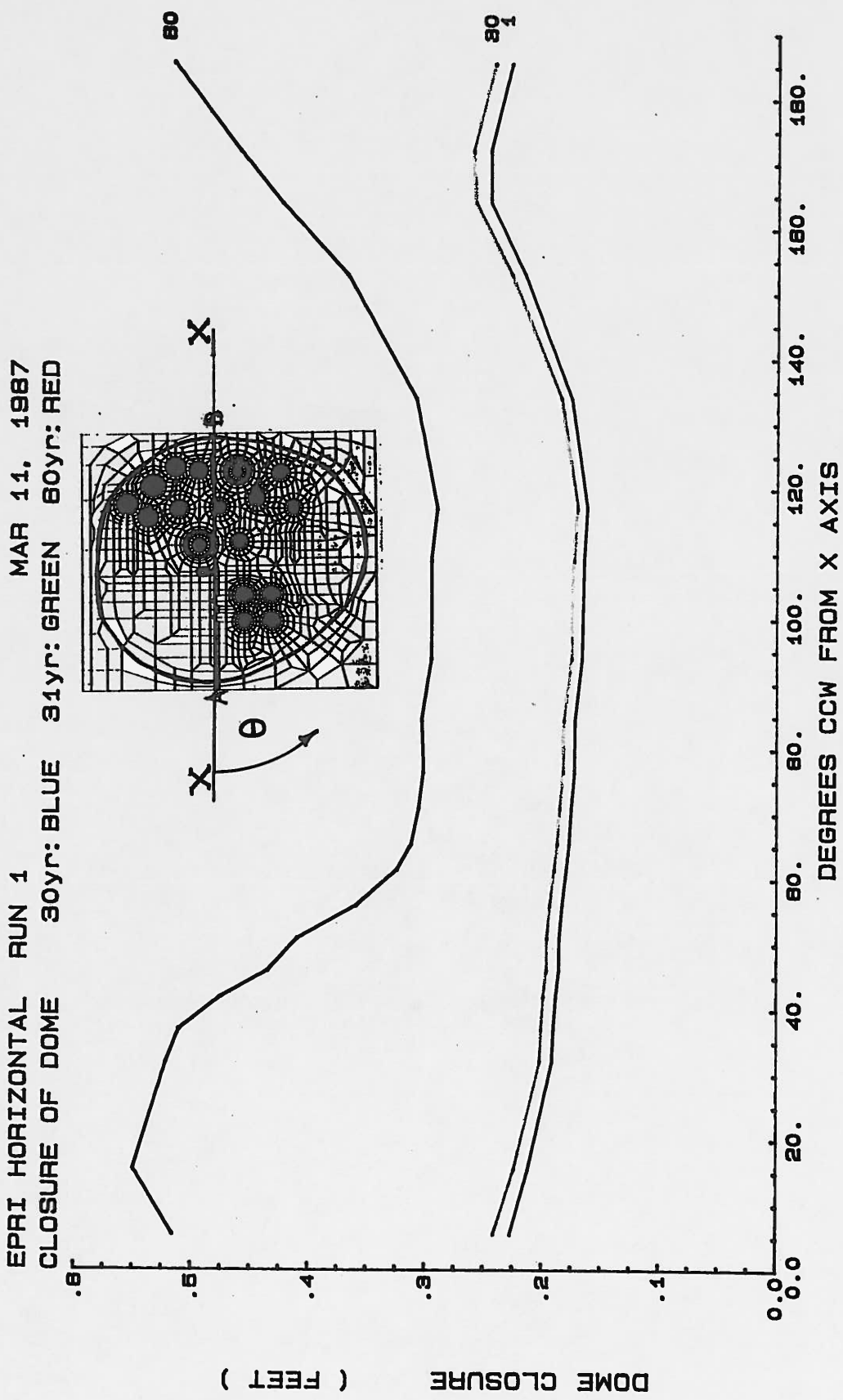


Figure 29. Time variation of the deformation of the salt dome boundary at the depth of 2400 ft below the ground surface -- the horizontal norm run.

EXPLORATION AND TESTING
FOR THE HARLAN DIVERSION TUNNELS

by

John L. Stanton,
Staff Geologist,
Marvin D. Simmons,
Chief, Geology Section,
Nashville District, U.S. Corps of Engineers,

Abstract. Harlan, in eastern Kentucky has a long history of destructive flooding, most recently in 1946, 1957, 1963, and 1984. The record flood occurred in April 1977. The flood protection scheme, designed by the Corps of Engineers, includes a permanent river diversion around the Harlan central business district via four parallel tunnels. Each tunnel is an inverted "U" shape, 34-feet wide by 32-feet high. The tunnels average 1,935-feet long and are to be made in flat lying Pennsylvanian Age siltstone using a roadheader for part of the excavation. The rock in the tunnel roof and the portal slopes is to be supported with resin anchored rock bolts and shotcrete. This paper describes the exploration, laboratory and field investigations made for design for the tunnels. Several techniques were used in evaluating the project including: hydraulic modeling of the intake portal; horizontal holes, providing continuous coverage along the alignment of the diversion; cross-hole seismic velocity profiling; borehole camera surveys; point load testing; computer stress analyses; and rock mechanics testing.

Introduction

Project Location

The project is located in the City of Harlan, in southeast Kentucky, at roughly 36°52' North latitude, 83°18' West

longitude. The Cumberland River originates near Harlan from three headwater streams: Poor Fork, Clover Fork, and Martins Fork. The entrance and exit for the diversion are located at approximately Clover Fork Miles 2.60 and 0.55. A general project location is shown on Figure 1.

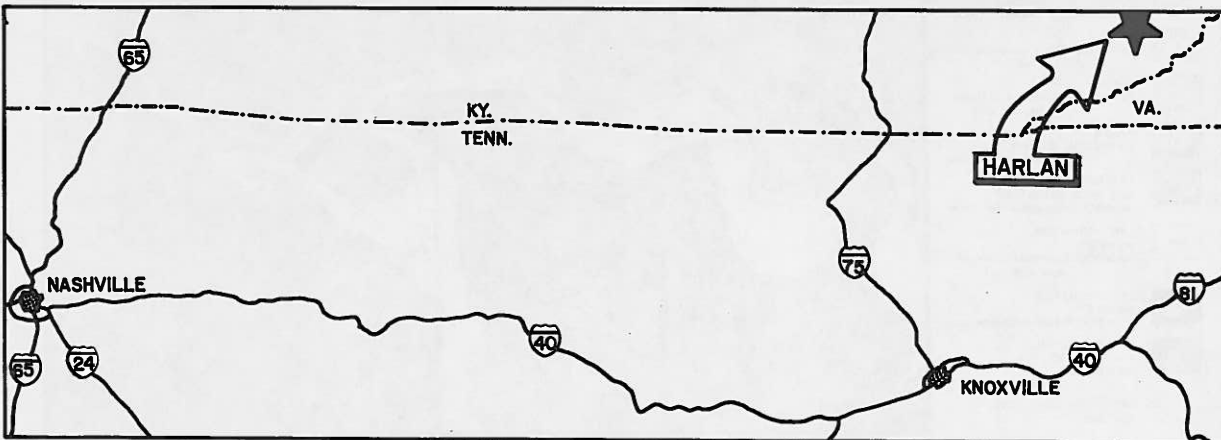


Figure 1. General Project Location.

Project Description

The Harlan, Kentucky Diversion project involves tunneling through Ivy Hill. For complete diversion of the river four tunnels are needed. Each will have an inverted "U" shaped cross section, 34-feet wide by 32-feet high and be separated by 34 feet. The average length of the tunnels will be 1,935 feet. At each end, open excavations roughly 100-feet deep and 290-feet wide will be required to develop the portals.

Road relocations and associated bridges will be required to cross each portal.

A diversion structure will be built to direct the river into the tunnels. The structure will be a membrane covered rock fill embankment with a slurry trench cut off, tied into the left abutment with a concrete wall. A flood gate across a single track railroad line will be built into the wall section.

Approximately one mile of the channel downstream of the diversion dike will be abandoned due to the diversion. The downstream portion of the abandoned channel will be used as a ponding area to collect internal drainage.

Regional Geology

Physiography and Topography

The Cumberland River basin above Cumberland Falls is a triangular area of 1,997 square miles, 1,677 in southeastern Kentucky, and 320 in Tennessee. Upstream of Pineville, Kentucky, the basin is conterminous with the Cumberland Mountains region (Fenneman, 1938), a subdivision of the Cumberland Plateau physiographic province. The region is roughly 25 miles wide by 125 miles long and lies entirely within the Pine Mountain Overthrust, as discussed in

the next paragraph. It is bounded on the northwest by Pine Mountain and on the southeast by Cumberland Mountain, two asymmetric ridges with crest elevations of about 2,700 feet. The basin is mountainous with irregular ridges radiating from the peak elevations at around 3,000 feet. The highest point in Kentucky, the Double on Black Mountain (El. 4,150), is located in the headwaters of the basin. The valleys are narrow and V-shaped. Most development is concentrated in the narrow floodplains.

Geology

Harlan is located in the Eastern Kentucky Geosyncline, a broad depositional basin which formed on the western border of the much larger and older Appalachian Geosyncline. It filled with sediment during the Pennsylvanian age (290 million years BP) and is the source of the extensive coal deposits in the area. Near the end of the Appalachian Orogeny (190-200 million years BP) the Pine Mountain Overthrust was formed, as a 25-mile wide by 125-mile long slab of this rock was thrust approximately seven miles to the northwest along a low-angle fault formed in shale of Devonian Age. The rocks along the leading edge were tilted by about 30° to the southeast as they rode up on the adjacent ground. The asymmetrical profile of Pine Mountain results from this dip. On the northwest slope of Pine Mountain, outcrops are of Devonian and Mississippian Age. Southeast of Pine Mountain the dips diminish quickly and, in the project area, the beds are horizontal. These Pennsylvanian Age rocks belong to the Lee formation and the Breathitt Group which is subdivided into the Hance, Mingo, Catron, Hignite, and Bryson formations (Ashley and Glenn, 1906). Cumberland Mountain, a ridge parallel and similar to Pine Mountain, with the beds dipping to the northwest, forms the trailing edge of the overthrust. A geologic map of the area is shown on Figure 2.



Figure 2. Geologic Map of the Project.
Adapted from USGS Geologic Quadrangle of Harlan

Broad, gentle warping of the beds occurs over much of the area within the overthrust with joints and shear planes occurring in various orientations. The dominant control near the larger rivers is the process of valley stress relief, resulting in high angle jointing parallel to the valley and low angle shear zones in the weaker rocks underlying the valley bottom.

A rose diagram based on a lineament study is shown on Figure 3.

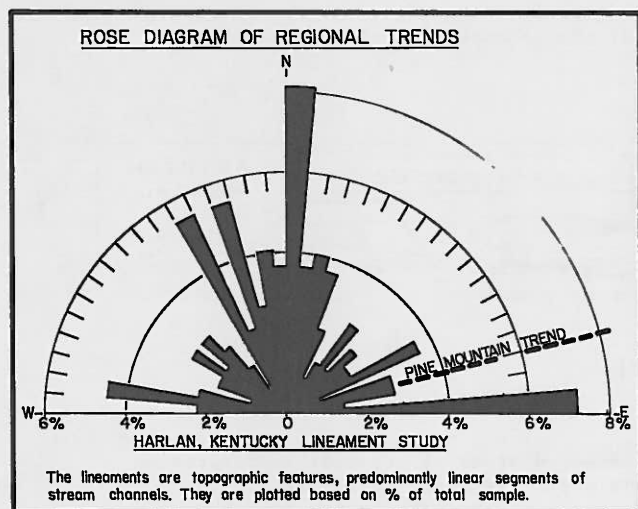


Figure 3. Lineament Study, Rose Diagram.

Site Geology

Site Topography

The tunnels will be made through Ivy Hill, a southwesterly trending spur which extends from the southwest end of Black Mountain. Generally, elevations vary from between 1,152 feet at the downstream portal to over 3,300 feet at the closest peak on Black Mountain. The maximum elevation above the tunnels is 1,524. The average cover over the crown of the tunnels is about 217.6 feet. The hillside slopes are generally steep, varying between 1 vertical on 1 to 3 horizontal and averaging roughly 1 vertical on 2 horizontal. The Cawood Sandstone member, described later, forms prominent cliffs above elevation 1,320 in the area. The floodplain of the Cumberland River and its major tributaries varies in width due to channel migration. The banks of the river are quite steep sloping on about 1 vertical on 3 to 3.5 horizontal.

Soils

Soils in the Harlan area are generally one of four types: residuum, colluvium, man-placed fill, or alluvium. Each of these soils contain a variety of particle types and sizes, and strengths.

Residuum. The upper portion of the rock at this project has weathered to a soil-like consistency. This residual layer, which preserves much of the fabric of the parent rock, is variable in thickness and composed of silt and clay sized particles.

Colluvium. Mass wasting from the slopes has resulted in variable colluvial deposits up to 50-feet thick. They are composed of sandstone boulders, 15 feet or more in maximum dimension, cobbles, gravel, sand, silt, and clay. The boulders are derived from the Cawood Sandstone member.

Man Placed Fill. Fill material, used for roadway and residential construction is frequently found in the area. It is mainly derived from the alluvial materials and is similar to them.

Alluvium. Alluvium occurs in the valley bottom, but near the tunnel portals it is often covered or mixed with colluvium from the upper slopes. It averages about 13.4-feet thick and is composed of silty sands, sandy silts, and sandy gravel with some occurrences of clayey fines.

Bedrock Stratigraphy

The Harlan Diversion project is located entirely within the outcrop area of the Hance formation which is between 1,050- and 1,350-feet thick. The basal contact of the Hance formation with the Bee Rock Sandstone Member of the Lee formation is roughly 500-feet below the tunnel invert elevation. This basal 500 feet is only exposed on the southeast flank of Pine Mountain. The material immediately below the tunnel is part of the upper Hance coal zone. It is represented at elevation 1140+ by a bed of hard black carbonaceous shale. Above this is 160 to 180 feet of slightly micaceous, moderately hard gray siltstone in which the tunnels will be driven. This material is variable across the site being more shaly in some areas, such as the upstream portal area and more massive in others. The x-ray analysis and the appearance of most of the core supports a classification as siltstone however the bedding in some locations suggest a shale. In most of the cores the bedding is indistinct except where marked by changes in lithology such as thin sandy layers and nodular zones or where the beds of the more shaly facies are present. Several thin beds of up to two feet in thickness are used as "markers" within the tunnel profile. These discontinuous, harder beds are mainly in the middle one-third of the tunnel and vary from sandy siltstone to calcareous sandstone to silty limestone. X-ray diffraction detected a significant percentage of dolomite and calcite in a sample from one of these beds. At most they constitute roughly 15% of the tunneling. Correlation of individual beds is difficult because of their discontinuous nature, and variations in the core hole alignment. Above these beds there is an extensive, thin zone of brown, phosphatic nodules. Several of the core holes also penetrated stratified nodules of oxidized iron sulphide.

The upper contact of the siltstone facies is transitional with the Cawood Sandstone member becoming interbedded with sandstone. The Cawood Sandstone member of the Hance

As is common in many argillaceous deposits very curved joint sets are present near the surface. These features are made of closely spaced parallel, tight, smooth joints which end suddenly. They resemble exfoliation and appear to be related to variations in hardness, and weathering along joint planes.

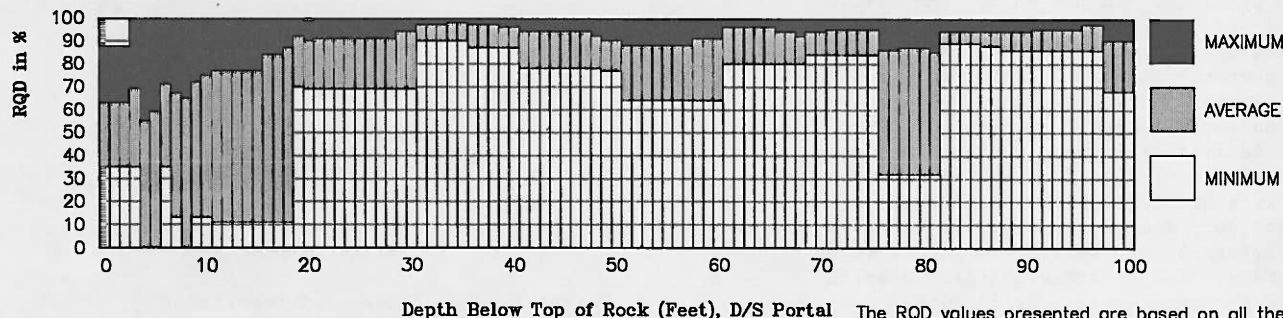
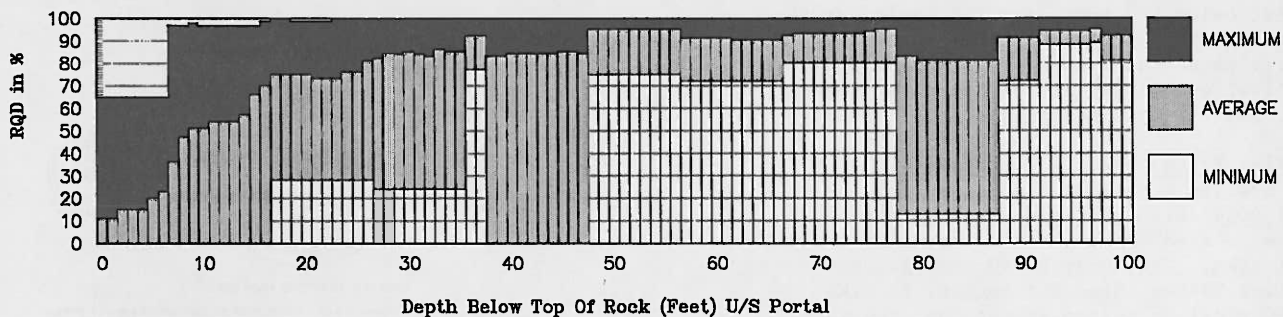
Low angle and sub-horizontal slickensided surfaces are found in many cores. These surfaces are tight, highly polished, and often have pronounced grooves. Most of the slicks were found in holes near the portals along contacts with beds of different hardness. Their orientations and locations suggest that they are the product of valley stress relief.

Displacement of beds in the downstream portal area along with core loss zones in the horizontal boring suggest that there is a fault or large shear zone present. The plane, which strikes roughly N 9° E and dips 68° NW, is within the downstream

portal area excavation. Judging from the material recovered from this zone and the amount of unaccountable loss encountered, the zone is comprised of a stiff clay gouge matrix with embedded siltstone fragments. It is up to 2.7 feet in true thickness but appears to pinch out at depth. The nature of this plane suggests that a product of valley stress relief. Movement of the beds on this zone is around 5 to 10 feet.

Weathering of the Bedrock

To define the depth of weathering below the top of rock the RQD values for all of the exploration was compiled for each portal and the average RQD plotted versus depth below top of rock. In most areas the contact between weathered rock and overburden is not clearly defined. The degree of weathering in some cases has reduced the rock to a soil like consistency with only relict bedding being present to identify the material as in-situ. These plots are included as Figure 7.



The RQD values presented are based on all the holes drilled in the portal areas using top of rock as a zero datum.

Figure 7. RQD Summary Plots

Exploration

General

General information on the geology of the area was obtained from publications of the U.S. Geological Survey¹, The University of Kentucky, The Kentucky Geological Survey and The Tennessee Geological Survey.

Aerial photography of the area was flown in 1981. Interpretation of these photos provided information on the geology.

Surface mapping, limited by the scarcity of outcroppings, was conducted in

the portals, on road cuts along Highway 421 and along the project alignment This work focussed on the orientation of jointing.

Subsurface Exploration

General. Core drilling was conducted for the period of 1984 to early 1989. A total of 74 borings were made, consisting mainly of NQ wireline, with PQ wireline drilled chiefly for direct shear testing. Early borings were made to investigate the open cut scheme. NQ angle borings were made to define the jointing characteristics of the rock mass. A listing of the number and type of borings made is given in Table 1.

Table 1. Summary of Subsurface Exploration Program

PROJECT FEATURE	# of HOLES	DRILL FOOTAGE					
		NQ	PQ	SPT	NQ angle	NQ horiz.	UNSAMPLE
Tunnels	19	2,139.1	383.7	0	996.7	2,337.9	567.6
Upstream Portal	30	1,483.6	99.5	88.7	678.8	0	201.7
Downstream Portal	12	404.5	0	97.5	83.7	0	55.8
Diversion Struc.	10	39.9	0	128.4	0	0	0

Horizontal Holes. The drilling of near horizontal holes through the mountain along the tunnel alignment by Corps crews was a key part of the exploration program. The downstream hole was 1,002.7-feet deep and the upstream hole reached 1,085.2 feet. Before drilling, stable rock faces and work platforms were developed. The upstream hole was started roughly 45-feet above the tunnel crown and angled downward at 2°. It bottomed out at elevation 1,144.6 roughly 18-feet below the tunnels. The boring at the downstream portal was started horizontally, 25-feet above the tunnel crown and dropped to elevation 1,153.4 at the bottom of the hole, about 8-feet below the tunnel invert.

Drilling Procedures. The horizontal holes were drilled with a skid-mounted Longyear Model 0034 core drill rig. New NQ drill rods were used to minimize movement of the drill stem. The Longyear NQ-Underground Wireline System, used for coring, is like the NQ wireline system except that the core barrel and retrieval tools are fitted with rubber seals and pumped into the hole using water pressure. On the whole, the system worked very well except that engaging the retrieval tools onto the core barrel took high pumping capacity. Additional problems were experienced if the core springs were not changed at regular intervals. After about 60 feet of coring, the abrasive rock polished the teeth off of the springs and they were unable to grip the core. A surface set, diamond, coring bit was used, and performed very well. The stones were G-1 grade, "Congo" stones, 3-2/3 to 4-1/2. stones per carat size, set in 30-45 Rockwell hardness matrix. Production rates, including time for core retrieval and camera shots, varied from 60 feet ± per 10 hr. shift for the first several hundred feet to less than 30 feet per shift near the bottom of the hole. The decreased productivity resulted from increased time required for core retrieval and surveys. The driller used high RPM's (around 240 to 300), low water pressure (60 to 70 psi) and moderate to low down pressure. This combination maintained good hole alignment.

Hole Surveys. The trajectory of the hole was monitored by taking readings with a NL Sperry Sun, type B single shot camera fitted with a 30-minute timer and a 120° compass head. Every 50 to 80 feet the

drill stem was withdrawn 25 feet and the instrument, mounted on 35 feet of non-magnetic spacer bar, was pumped to the bottom of the hole, using the NQ-U overshot assembly. With the camera projecting 25 feet beyond the drill string, the magnetic influence of the drill steel was minimal. This same camera system using a cable hoist was used to conduct surveys on the angle holes as well. A diagram of the camera equipment is shown on Figure 8.

NL SPERRY SUN TYPE B SINGLE SHOT CAMERA WITH 120° COMPASS HEAD

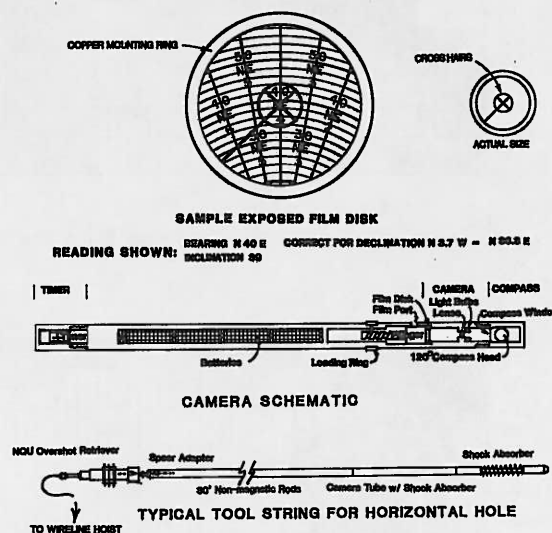


Figure 8. Downhole Camera

Results. One of the most remarkable observations to come from the horizontal hole drilling is the exceptional quality of the rock which was encountered. The core recovery in both holes exceeded 98.6% and the RQD averaged 96%. If the first 35 feet of the upstream boring were disregarded, the RQD would exceed 99% for both borings. That the rock amazingly free of breaks is attested to by the fact that 75% of the core pulls were intact 10-foot lengths. The longest unbroken length was 224 feet.

Electric Logging

Geophysical logs were run on 8 holes. Natural gamma, spontaneous potential, and single point resistance logs were run on each hole.

Cross-Hole Seismic Velocity Profiles

The Waterways Experiment Station conducted cross-hole seismic velocity profiles on three sets of holes. Each set consisted of two vertical holes separated by roughly 50 feet. Each set had a PQ hole for the hydrophone and an NQ hole for the blasting caps. A deviation survey was run on each hole to determine the precise hole alignment so that accurate distances between holes could be computed. Testing consisted of accurately measuring the time required for the P-wave to travel from the source to the receiver. This process was repeated at 5-foot intervals up the hole to develop a seismic velocity profile within the tunnel portion of the holes. The data were reduced using the WES computer program "CROSSHOLE2". Based on this, the average p-wave velocity of the material in the tunnel profile is 17,400 feet per second which is notably higher than the laboratory data for the siltstone, discussed later. It was felt that the thin sandstone layers, which have laboratory velocities in the range of 14,000 to 19,000 feet per second, were refracting the p-wave and yielding higher apparent velocities. WES personnel did not feel that this was likely.

Pressure Testing

Single packer pressure testing was conducted on 10 of the holes. The zones tested included the entire hole and the zone from roughly 50-feet above the tunnel crown to the bottom of the hole. The whole hole tests indicated takes in the Cawood member, however, the tunnel zone tests had little or no take, varying from 0.04 to 0.00 gals. per minute per foot of hole.

Combustible Gas Monitoring

The risk of encountering combustible gas in the tunnels was discussed with the Kentucky Bureau of Mines and Minerals. "Gassy" mines in their jurisdiction all

fell in two main groups: (1) "Wet" mines operating 100 feet, more or less, below river level. (2) Deep mines in the range of 2 to 5 miles. In both cases, fracturing is needed to have transmittal of the gas. Consequently, they felt that it was unlikely that tunnels would encounter any gas.

In order to test the site for gas, a horizontal hole was checked with a MSA model 361 Portable Alarm equipped to detect hydrogen sulphide, combustible gas, and oxygen. The reading was taken at a depth of 810 feet at the start of the shift to allow any gas which might be present to collect overnight. No H₂S or combustible gas was detected and it is not anticipated in the excavation of the tunnels.

Borehole Television Camera Surveys

To obtain additional details on the joint condition and orientation, a borehole TV camera, operated by the Corps' Southwest Division Lab, was used in 10 holes in the upstream portal. The camera was fitted with interchangeable lenses. A full view lens, pointing down the hole axis, is used for general examination. A radial view lens, equipped with a conical mirror with a compass in the viewing area, is used to inspect the sides of the hole and determine the orientations of joints. The videotape is examined in the lab where measurements on the geologic features are taken.

Rock Mechanics Testing

General

The rock mechanics testing for Harlan has involved a wide range of both field and laboratory techniques. Both NQ and PQ core have been used in these investigations. In general, for tunneling, it is suggested that 8 characteristics of a rock mass be investigated.² The characteristics and some recommended tests are as follows:

Table 2. Suggested Testing Methods for Tunnel Design

Characteristic	Suggested Tests
a. Hardness	Schor Scleroscope, Schmidt Hammer
b. Strength	Uc Strength, Brazilian Test, Punch Shear, Point Load Test
c. Texture	Quartz Content, Texture Coefficient, Grain size and Shape
d. Drillability (Translational) (Penetrative) (Percussive)	Goodrich Drillability, Seviere's "J" number, Taber Abradability NCB Cone Indenter Index, Morris' Drillability, Handwith Test Rock Impact Hardness Number, Protodyakonov Test
e. Abrasiveness	Goodrich Abrasivity, Cerchar Abrasivity, LCPC Abrasivity, Taber Abrasivity
f. Geologic Structure	CSIR or NGI Rock Mass Classification Systems
g. Seismic Properties	P&S Laboratory Velocities, Field Seismic Velocities
h. Swelling Properties	Slake Durability, Free Swell Test, Swell Pressure Test

Point Load Testing

Test Equipment and Methods. The point load test is an index test which is easy to run in the field and which, through a simple conversion, can be related to the unconfined compressive (Uc) strength. Use of the point load test allows a broader coverage of the rock strength than is possible with lab Uc tests and allows testing of cores directly after they are recovered. The apparatus consists of a load frame, hydraulic jack, hand pump, and oil pressure gage which reads directly in pounds exerted at the platens. The platens are hardened steel, 60° cones, with a 5mm radius tip. Tests are run both across the diameter and along the axis of the core. The sample is loaded along the central axis of the assumed failure plane, and the failure load measured by the gage which is fitted with a deflection needle.

Records and Data Reduction. The location, and orientation of the sample along with the test results were entered on the field logs. Data on each of the tests were entered in a database and analyzed using a micro-computer. Numerous authors discuss the data reduction techniques for point load test data.^{3,4} It is very prone to scale and shape effects with larger samples yielding lower values. To compensate for

these effects it is recommended that the index be computed based on standardized break area and shape. The failure load is plotted versus the break area on a log-log plot and a straight line, based on linear regression analysis, is interpolated to the standardized size. This point is used to compute the representative point load index by dividing the load by the break area. For convenience the standard size is equivalent to an NQ core. For this size the point load index is equal to 1/24 of the unconfined compressive strength of the material.

Test Results. A planar failure surface is often not possible with this test which results in a lot of scatter in the data. Scatter also resulted from lateral changes in the strength, especially for the axial tests. This variation in bedding plane strengths is typical, resulting from differences in cementation, planarity, cohesion, texture, etc. On the whole the tests indicate that the rock at the tunnel level is quite homogeneous for a bedded material with the average strength across the beds being 170 to 250% higher than the tests with the beds. Plots of the Uc strength, computed from the point load index, Is, versus depth for the horizontal holes is shown on Figure 9.

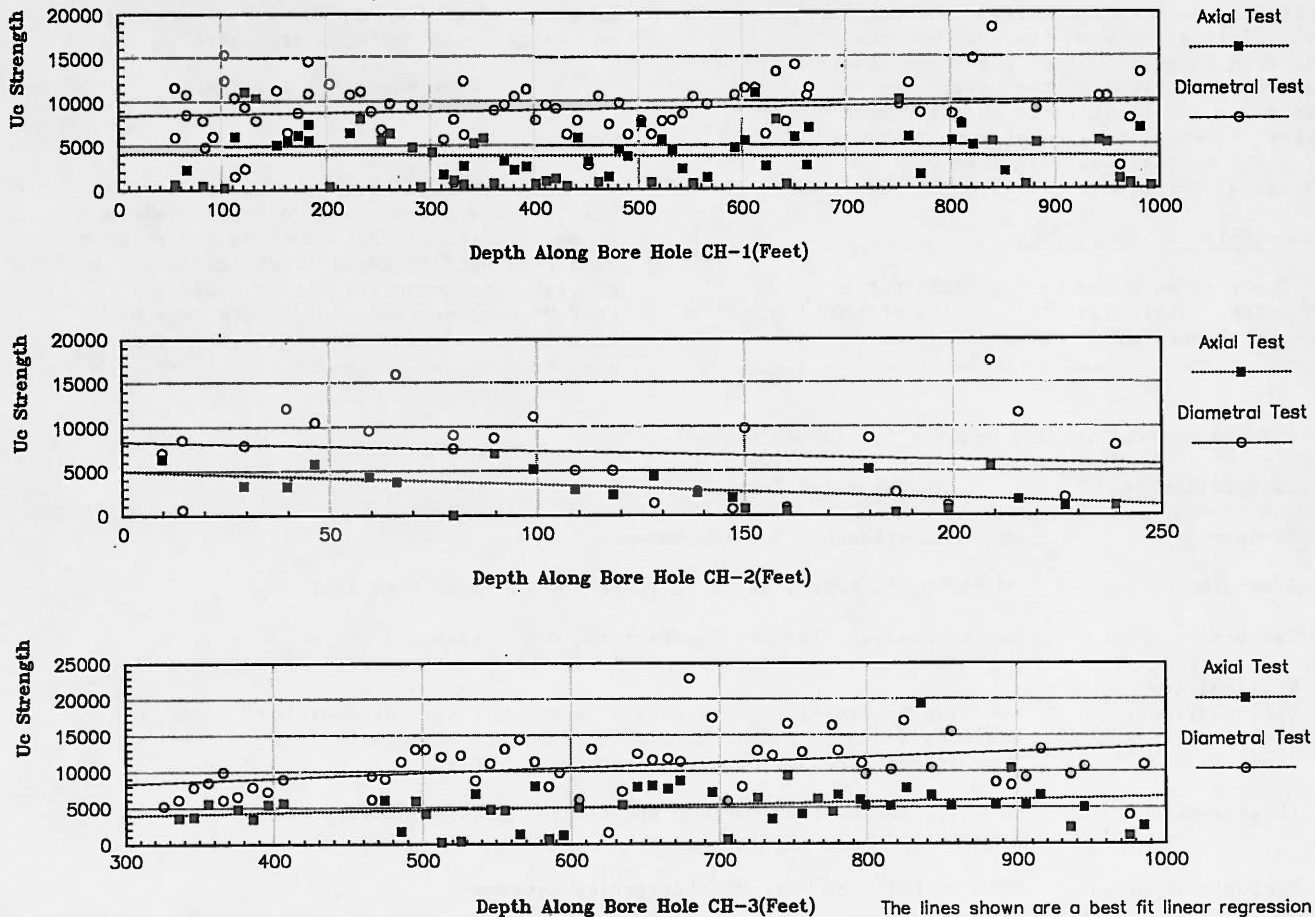


Figure 9. Point Load Test Data.

The lines shown are a best fit linear regression analysis. Unconfined strength is based on the Point Load Index multiplied by 24.

Pull Break Test

As an approximation to the direct tensile test, the pressure required to break the core at the end of selected runs was measured. A new gage was used on the drill rig to reduce inaccuracies which

might result from worn equipment. Figure 10 shows the results of this study. There is good correlation between the tensile strengths derived by this method and the laboratory test results and a rough correlation with fluctuations in the point load strength data versus depth.

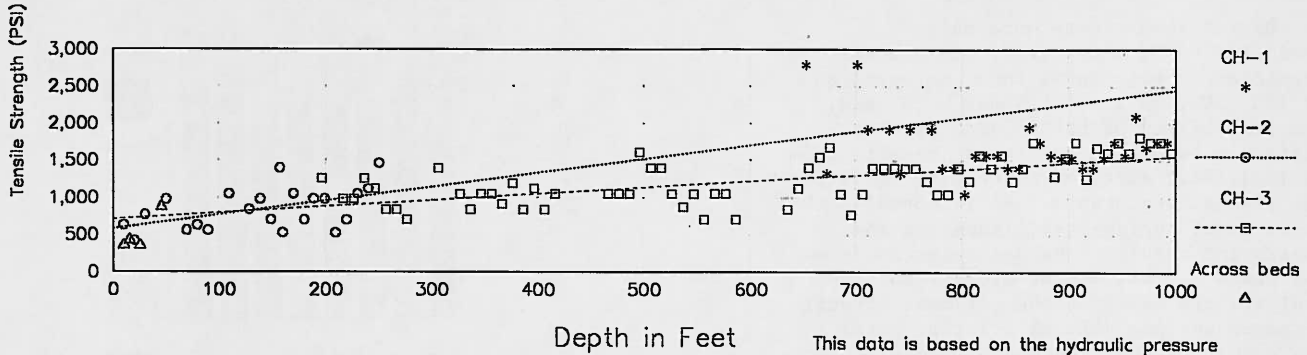


Figure 10. Pull Break Tensile Strength Test

This data is based on the hydraulic pressure required to break the core after each run. It approximates the tensile strength of the rock.

Unconfined Compressive Strength

Unconfined compressive strengths, with deformation readings and peak strength only, were determined. On the whole the Uc strength data is quite consistent. Both PQ and NQ cores from vertical borings were tested with good correlation between these data. In addition, three samples from the horizontal hole, CH-1 were used to test the strength parallel to the bedding and three cores were tested at various angles to the bedding. It is interesting to note that

the data parallel to the beds is comparable to that which is perpendicular to the beds which indicates that the material is quite homogeneous. The Uc strength data, summarized on Figure 11, averaged 7,485 psi for the siltstone based on the vertical data with a standard deviation of 2,766 psi. The values ranged between 3,750 psi and 14,250 psi. The average strength for the siltstone, horizontal data, is 6,650 psi with a standard deviation of 1,173 psi. These values ranged between 5,260 psi and 8,130 psi.

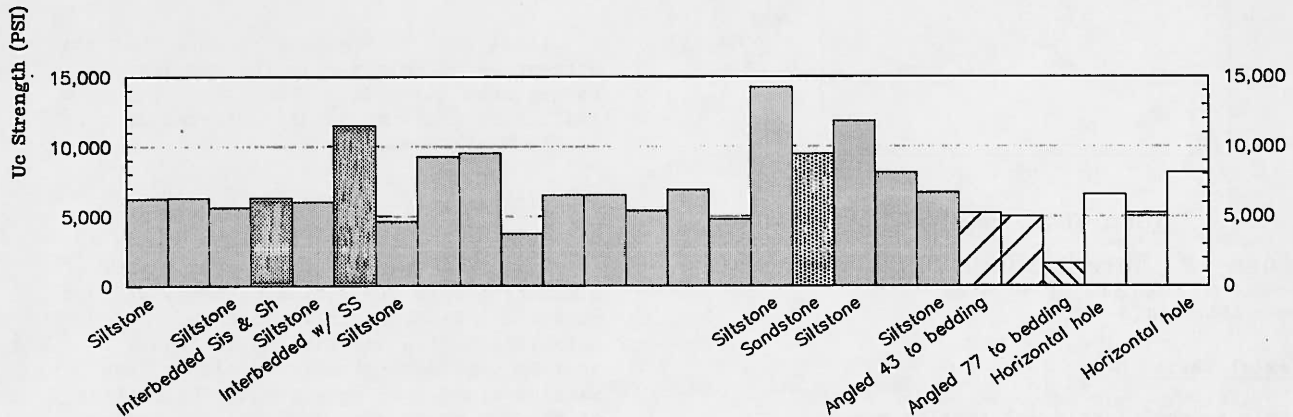


Figure 11. Unconfined Compressive Strength

Modulus values from these test were somewhat variable ranging from 0.677×10^6 psi to 6.529×10^6 psi for Young's Modulus and from 0.186 to 0.259 for Poisson's ratio. Some inconsistencies between these data and the dynamic modulus indicated by the seismic velocity studies exist, however, these are only in the higher range of values. The dynamic (Young's) modulus calculated from the seismic velocity data ranged from 0.444×10^6 psi to 2.203×10^6 psi.

The unconfined data suggests that there is some variation in the properties of the rock within the tunnel profile

beside that which is indicated by visual examination of the core. Some horizons which are visually similar to the shaly siltstone assumed to be typical for the whole tunnel are, apparently stronger. This phenomenon may affect the efficiency, but should not preclude the use of machine excavation techniques.

Brazilian Tensile (Splitting) Test

A total of 5 Brazilian tensile tests were run, 3 perpendicular to the bedding and 2 parallel to the bedding. As expected, the data perpendicular to the bedding were

higher than the data parallel to the bedding. The former tests averaged 1,220 psi. The latter averaged 555 psi. This compares well with the information shown on Figure 10.

Direct Shear Testing

Direct shear tests were only conducted on PQ size cores. To bracket the predicted in-situ conditions, normal loads of 100, 200, 300, and 400 psi were used. The variability of the bedding made it very difficult to obtain consistent results from an individual suite of tests especially in the peak strength data. As is normally the case, sawed surface tests gave the most consistent results. Values selected from the tests for use in the slope stability analysis are based on the ultimate strength or sawed surface sliding friction data. The resulting shear strength parameters are $\phi = 31^\circ$ and cohesion = 46 psi for the ultimate strength and $\phi = 21.5^\circ$ and cohesion = 30 psi for the sawed surface tests. A plot of the average data and the failure envelopes is shown on Figure 12.

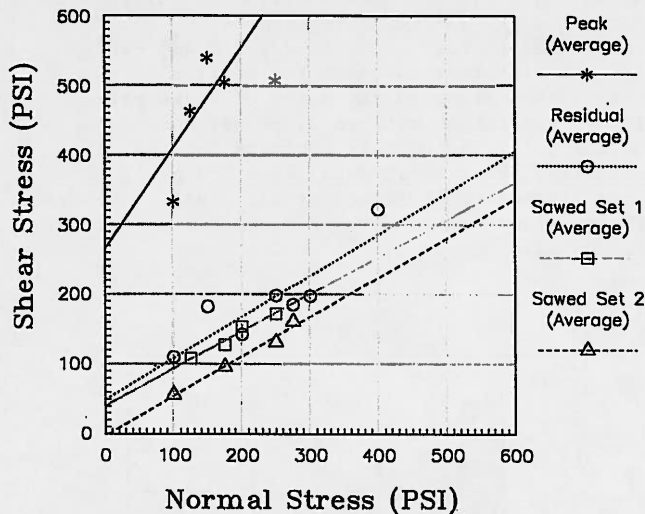


Figure 12. Direct Shear Data, Averages
Light lines are based on linear regression analysis of the data.

Triaxial Tests

Six suites of triaxial testing were run to provide additional stress information for the boundary element stress analysis which is discussed later. Most of the failure surfaces were formed, at least in part, on bedding which was angled to the loading direction.

X-Ray Diffraction

To adequately define the mineralogy of fine grained rock it was necessary to use x-ray diffraction. Both visual examination and the x-ray studies indicate that the material is variable laterally as well as vertically. Samples for the x-ray analysis were selected from the tunnel prism. On average the samples contained 57.5% quartz. These results are applicable in evaluating

machine performance and pick wear.⁵ Figure 13 graphically represents the x-ray diffraction data.

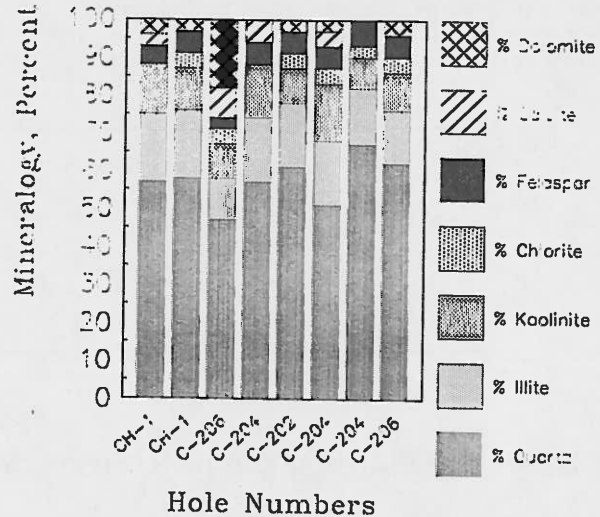


Figure 13. X-Ray Diffraction Data

The clay mineralogy of the samples does not indicate a swelling tendency being mostly illite and kaolinite with a trace of chlorite. The swelling characteristics of the clay fraction were determined by the ethylene glycolation test. This conflicts with the findings of the swell testing, discussed later, but suggests that the swelling observed is a surface slaking phenomenon.

It is also interesting to note that the percentage of feldspar in the samples ranges from 3% to 7%. This is an indication that the material tested is not highly weathered.

Laboratory Seismic Velocity

Seismic velocity is one of the main criteria for determining the rippability of a material when field ripping tests are not possible. The seismic velocity of the two main lithologies present in the tunnel profile were determined. Initially long samples of NQ core were used. The platens of the equipment were designed for larger core and significant signal loss was noted on the smaller core. Subsequent to this short samples of PQ core were used which yielded much higher values.

Independent seismic velocity measurements on the sandy "marker" beds in the tunnel profile were performed by Caterpillar Inc.'s rock mechanics testing facility. These tests indicated an average seismic velocity of 19,293 feet per second. This material is not considered rippable without pre-blasting. Based on visual examination by Caterpillar, the siltstone is considered rippable, however this is not confirmed by testing and absence of natural breaks in the rock mass suggests otherwise. Figure 14. shows the results of the seismic

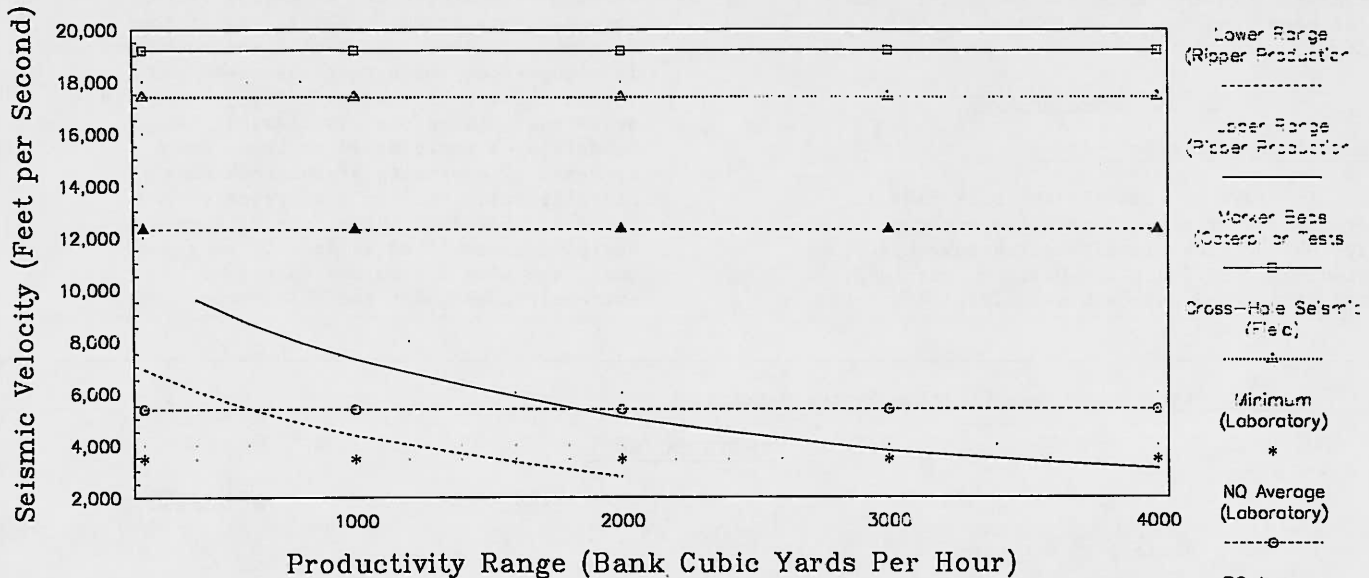


Figure 14. Rippability Estimate

This graph is adapted from the "Handbook of Ripping", Seventh Edition, January, 1983
Published by the Caterpillar Tractor Co.

Swell Testing

Experience in tunneling in shale at the Orange - Fish tunnel in South Africa determined that most of the failures which occurred were the result of the swelling properties of the wall rock.⁷ The swell tests which were run on the Harlan core were aimed at defining this potential problem and were of two main types: (A) Free swell tests where the samples were allowed to swell in water without confinement and the axial deformation measured at intervals, and, (B) Swell pressure tests where the samples were placed in water and confined and the swell pressures at zero strain measured. Diametral deformation was not monitored in this test.

The values of free swell and swell pressure were higher than anticipated based on the clay mineralogy and slake durability testing. The exact nature of the swelling in these samples is not fully understood but may in part be related to negative pore pressure in the fine grained material.

Slake Durability Testing

Slake durability testing was performed by the Corps and by the Kentucky Department of Transportation for design of a nearby road cut. All of the test data is similar and shows the siltstone to be of medium high to very high durability in the range of 90 to 95% SDI.

Unit Weight Testing

Unit weight determinations were performed on the various lithologies. The data on the siltstone is between 166.5 and 168.7 pounds per cubic foot. The samples were tested with as received moisture

content which may have affected the results slightly but not appreciably.

Schmidt Hammer Testing

The Schmidt Hammer test is a commonly used index test for the hardness of a material. The magnitude of the recoil of the hammer weight is dependent on the elastic properties of the material being tested, provided failure of the material does not occur. The elastic properties, in turn, relate to the strength of the material. This test has been applied to evaluation of tunnel boring machine advance rates by some authors.⁸ Total hardness, which equals the Schmidt Hardness times the square root of the Abrasion Hardness is used in these studies. Empirically, siltstones and shales normally fall in the range of 20 to 60, total hardness. The siltstone from Harlan falls in the upper range of these values.

Specialized Testing

Because of the individual performance characteristics of various types of tunnel excavation machinery, different manufacturers have devised different test methods to evaluate what is commonly referred to as a "Drillability Index". These tests are specialized and not usually run by most laboratories. Samples of core were sent to various manufactures for testing. Results of this testing were interpreted to predict machine productivity, pick wear, and other design considerations. The wear coefficient data for the siltstone ranged from 0.010 to 0.067 N/mm². The sandstone, as expected, had a higher wear coefficient, averaging 0.165 N/mm². The Cerchar Abrasivity Index for the two rock types were 0.5 and 1.3

respectively. This data indicates that the siltstone is medium to low abrasivity and the sandstone medium to high abrasivity.

Other Investigations

Rock Classification Systems

In order to classify the rock mass for evaluation of support requirements two systems of rock classification have been used; The NGI Tunneling Quality Index and the CSIR Geomechanics Classification

System.^{9,10} For many tunneling jobs it is necessary to divide the tunneling reach into sections based on variations in the rock mass quality. At Harlan the rock mass is homogeneous throughout the reach and it is unnecessary to subdivide the job. It is quite easy, therefore, to classify the tunnels as a whole based on these two systems. The results of the rock mass classification studies are presented in Table 3. Based on these, the rock mass at Harlan is classified as good to very good under the CSIR system and very good to extremely good under the NGI system.

Table 3. Rock Mass Classification System Results.

<u>Q-SYSTEM (NGI)</u>			
		<u>HIGH VALUE</u>	<u>LOW VALUE</u>
1.	RQD Range from Borings	100	95
2.	J _n Joint # (Massive to One Joint Set)	2.0	0.5
3.	J _r Joint Roughness (Smooth to Undulating)	4.0	3.0
4.	J _a Joint Alteration (Unaltered)	1.0	1.0
5.	J _w Joint Water (Dry to Minor Inflows)	1.0	1.0
6.	SRF Stress Reduction Factor (Low to Medium)	2.5	1.0
Q = RGD/J _n * J _r /J _a * J _w /SRF			
Maximum Q = 100/0.5 * 4/1 * 1/1 = 800 Extremely Good Quality Rock			
Minimum Q = 95/2 * 3/1 * 1/2.5 = 57 Very Good Quality Rock			
<u>RMR SYSTEM (CSIR)</u>			
		<u>HIGH VALUE</u>	<u>LOW VALUE</u>
1.	Rock Uc Strength (10,557 psi - 5,412 psi)	6	3
2.	RQD Rating (95 - 100)	20	20
3.	Joint Spacing (1 ft. - 10 ft.)	30	20
4.	Joint Roughness (Very Rough - Slightly Rough)	25	20
5.	Groundwater (Dry - Moist)	10	7
6.	Adjustment for Joint Orientation (Bedding)	<u>-10</u>	<u>-10</u>
Rock Mass Rating		82	61
Rock Classification		Very Good	Good

Computer Stress Analysis of the Tunnels

The computer analysis of the tunnels was performed by Mr. Richard Humphries of Golder Associates, under contract to the Corps.

Boundary Element Stress Analysis. A modified form of boundary stress analysis presented in Underground Excavations in Rock (Hoek et al, 1980) was used initially to calculate the stresses around the tunnels.¹¹ It was assumed the rock mass behaves as an isotropic, homogeneous, linearly elastic material under plane

strain conditions. The cases analyzed were for a single top heading, a single tunnel, and four tunnels. Each of these cases was analyzed using 0.5, 1.0, and 2.0 vertical to horizontal stress ratios with the vertical stress being equal to the maximum overburden pressure. Stress distributions were calculated using a finite element grid resulting in a tensor plot for the maximum and minimum principal stresses.

A rock mass containing bedding planes, such as is present at Harlan, may fail along these weaker planes or through intact rock. The direction of the major principal

stress with relation to the weaker planes, to a great extent, determines the failure mode. This anisotropic behavior of the rock mass is described by an anisotropic strength model (Ward, 1978).¹²

The relative strengths of the intact rock and bedding planes, as determined by the rock mechanics testing program, are used in the anisotropic strength model. An overstressed zone of rock may exist around a tunnel where the computed stresses exceed the strength of the rock. By dividing the anisotropic strength available, by the elastic stress vector at each nodal point on the finite element grid, the zones of overstress can be defined and contoured. The area between the boundary of the tunnel opening and the 1.0 contour line is the zone of overstressed rock. Figure 16. shows the results of one of these analyses with a horizontal to vertical stress ratio of 2.

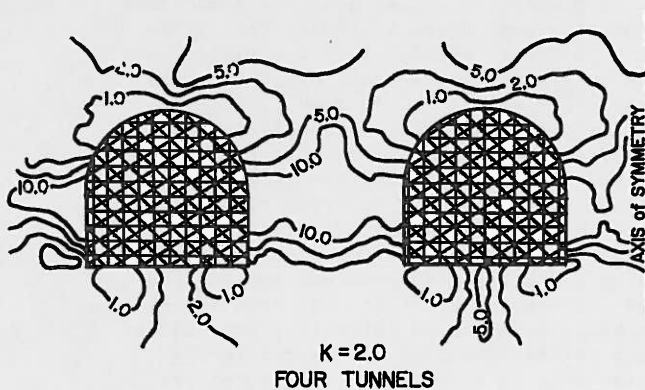


Figure 15. Boundary Element Analysis

FLAC Analysis. Since the boundary element analysis, discussed above, is limited by the assumed elastic, linear behavior, interaction between the rock and the support system are not taken into account. The computer program Fast Lagrangian Analysis of Continua (FLAC), developed and marketed by Itasca Consulting Group, Inc., was used to compensate for this short coming. This program is an explicit finite difference code which simulates the behavior of structures built on soil, rock, or other materials which may undergo plastic flow when their yield limit is reached.

Results of this analysis revealed that the most probable failure mechanism in the rock mass is slippage along bedding planes. It further indicated that displacement inside the tunnels would be very small, on the order of tenths of an inch. The loads calculated in the rock bolts are well within the capacity of a number 8 bolt and the axial stresses in the shotcrete in the crown of the tunnels, although close to the 28-day strength, will not be excessive. Most of the displacement will have taken place prior to the actual placement of the shotcrete. Figure 17. shows the results of

a FLAC run which included the 12-foot bolts, shotcrete and a bedding plane.

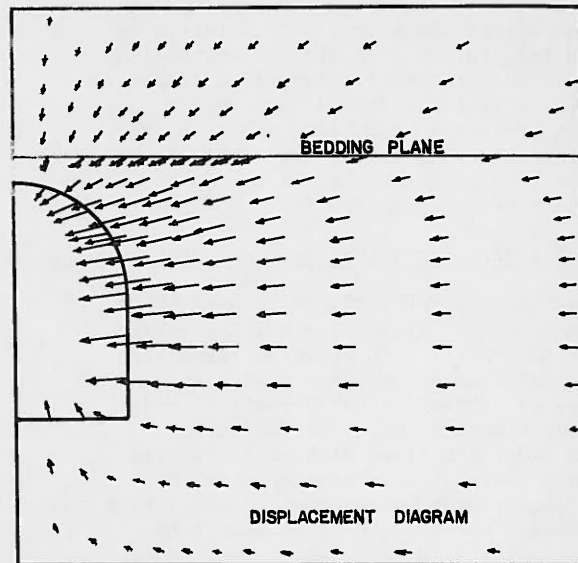


Figure 16. FLAC Analysis

Rock Slope Stability

The stability of the rock slopes was analyzed using "Rock Pack", a software package developed by C. F. Watts & Associates, (c) 1986. These programs analyze the stability based on the limit equilibrium methods presented in "Rock Slope Engineering" by E. Hoek and J. W. Bray.¹³ Rock strengths were developed in the lab testing. Stability for both plane and wedge failure was checked. For the plane failure analysis four sets of slope data were analyzed under four load cases. Fixed variables for all sets were: a 90-foot bench height, an upper slope angle of 31°, and a rock unit weight of 167.6 pounds per cubic foot. The four sets of variables used were as follows:

Cut Slope	ϕ	Cohesion (psi)
4V on 1H	31°	46
3V on 1H	31°	46
4V on 1H	24°	30
3V on 1H	24°	30

The four cases used for each set were:

Case #	Description
1	Wet slope with no tension crack
2	Dry slope with no tension crack
3	Wet slope with tension crack
4	Dry slope with tension crack

The wedge failure analysis was conducted using a bench height of 90 feet, a ϕ of 31°, a cohesion of 46 psi, a unit weight of rock of 167.6 pounds per cubic foot, a 4V on 1H cut slope, a vertical tension crack, and an upper slope angle of 31°. Composite joint combinations which met the criteria for potential instability (i.e. the slope of their line of intersection is greater than the ϕ angle of the planes and, it pierces the cut face) were then analyzed for dry slope and for wet slope for various locations of the tension crack. The wet

slope analysis used 100% saturation.

For both of the failure mechanisms it was found that the slopes were universally stable for drained conditions. Instability was mainly a problem for saturated slopes, especially in the wedge failure when a tension crack was located to minimize the weight of the wedge. For saturated slopes with no tension crack most of the joint combinations were stable.

Hydraulic Modeling of the Upstream Portal

The tunnels are required to pass the standard project flood plus a 30% blockage factor for debris. To optimize the design of the upstream portal with regard to passage of debris a 1 to 30 physical model was constructed at WES. The model was tested under SPF flows with scaled debris loading. Several runs were made on the model with various geometries on the intake and debris "noses". It was found that incorporating a 10-foot radius "bell mouth" at the intake eliminated vortices and greatly improved the hydraulic efficiency. The "noses" were modified several times to improve their ability to shed debris. It was found that any sharp corners below the water surface tended to capture material so the outline of the "noses" was smoothed out. The final design is shown on Figure 15.



Figure 17. Hydraulic Model of the Upstream Portal

Construction

Rock Excavation

General. Factors which affect the choice of excavation method are: control of water, blasting effects, working area, labor and maintenance requirements, preservation of the rock mass, scheduling, availability of equipment, and production rates all of which affect costs.

Surface Excavations. Surface water will be controlled by ditching, sumps, and by leaving plug sections in place at both ends of the project until the tunnel excavation

is complete. Overburden and weathered rock will be excavated with conventional earth moving equipment. Rock will probably be excavated by drill and blast techniques. The proximity of residences will require pre-blast surveys of buildings and careful vibration control measures. The use of an impact ripper for rock removal offers advantages but is not considered feasible. Presplitting will be required to develop the perimeter of the portals and preserve the rock mass. Test blasting will be used to verify the contractor's blast design.

Tunnel Excavation. Ground water seepage can increase the cost of underground work by up to 40% but is not anticipated for the tunnels. Excavation of the top headings is to be performed by machine. The bottom heading may be efficiently excavated as a bench using drilling and blasting. Two types of machines were considered for the top headings: (1) Full face tunnel boring machines, which bore a circular tunnel using a rotating cutter head. The machines generally have three sections; the cutter head in front, a gripper in the middle, and the muck handling or trailing gear at the rear. The tunnel is advanced by pushing the grippers against the tunnel sides, rotating and advancing the cutter head, retracting and advancing the grippers and repeating the cycle. For long tunnels these machines are very efficient. However, at Harlan, this type of machine would have to be partially disassembled, rotated 180°, and reassembled after driving each tunnel or backed out, moved laterally, and started into the next heading. This requires a great deal of effort and greatly reduces the efficiency of the operation. Five circular tunnels would be needed to provide the necessary open area, increasing the portal excavation and bridge requirements. Availability and cost are also a problems with very large full face TBM's. (2) Road headers utilize a rotating cutting head on an articulated boom. Replaceable carbide picks perform the excavation of the rock. Automatic mucking and track drives are available on most machines. Two types of road header are made. They differ based on the axis of rotation of the cutter head; the milling type rotates perpendicular to the boom axis and the ripper type rotates parallel to the boom axis. Based on prior experience, the availability equipment, the versatility and mobility of this type of machine, and the minimization of damage to the rock mass, it is felt that a heavy duty road header, is the best method of excavating the top headings at Harlan.

Ground Support

Two types of ground support will be required; rock bolts and shotcrete. Drain holes for the surface excavation slopes will also be required.

Rock Bolts. Support of the rock mass in the portal excavations and in the tunnels will be provided, in part by resin encapsulated rock bolts. Based on experience on

Corps of Engineers projects and empirical data developed by the Post Tensioning Institute, the working bond stress for the rock at Harlan should be on the order of 80 psi. Confirmation of this will require rock bolt pull-out tests. Bolts for both applications will be epoxy coated to resist corrosion. Portal bolts will be stressed using the dual resin cartridge anchorage system. Tunnel bolts will not be stressed provided they are installed within a specified time and distance from the working face.

Shotcrete

General. Two types of shotcrete are to be used at Harlan; mesh reinforced for the portals and fiber reinforced for the tunnels. To ensure uniformity and durability, wet mix, air entrained shotcrete will be specified for both applications. Four-inch minimum thickness will also be required.

Mesh Reinforced Shotcrete. With mesh reinforced shotcrete welded wire fabric is anchored close to the rock face prior to application of the shotcrete. This method of reinforcement is more labor intensive than fibre reinforced but offers greater continuity of reinforcement.

Fiber Reinforced Shotcrete. With fiber reinforced shotcrete, steel or polyester fibers are mixed in the shotcrete before it is applied. Steel fibers are to be used at Harlan. These fibers are "hairpin" shaped and roughly 1 to 2" long. Ideally they are dispersed throughout the mix and give a uniform reinforcement.

References:

1. Geologic Map of the Harlan Quadrangle, U.S. Geological Survey.
2. P. J. Tarkoy, Poster Session for the 6th International Congress on Rock Mechanics, Motreal, Quebec, September, 1987 and Ian McFeat-Smith, "Considerations for Mechanized Excavation of Rock Tunnels", 5th Australian Tunneling Conference, Melbourne, Australia, March, 1987
3. E. Broch and J. A. Franklin, "Point-Load Strength Test", International Journal of Rock Mechanics Min. Sci., Vol. 9, pp. 669-697, Pergamon Press, (Nov. 1972)
4. Z. T. Bieniawski, "Estimating the Strength of Rock Materials", Journal, South African I. Min. Met., pp. 312-320, (March 1974)
5. D. L. Leibee, "State of the Art of Cutting Tools for Continuous Miners, Road Headers and Tunneling Machines", Proceedings, German-American Symp. on Mining Tech., Vol. 2, Paper 4, (April 1981)
6. Caterpillar Tractor Co., "Handbook of Ripping", 7th Ed., (Jan. 1983)
7. H. J. Oliver, "A New Engineering-Geological Rock Durability Classification", Engin. Geol., 14, pp. 255-279, (Oct. 1978)
8. Tarkoy, P. J. [1975], "A Study of Rock Properties and Tunnel Boring Machine Advance Rates in Two Mica Schist Formations," Applications of Rock Mechanics, Proc. 15th Symp. on Rock Mechs., ASCE, New York, pp. 415-447, (Sept. 1973)
9. N. Barton, R. Lien and J. Lunde, "Engineering Classification of Rock Masses for the Design of Tunnel Support", Rock Mechanics, Vol. 6, No. 4, pp. 189-236, (1974)
10. Z. T. Bieniawski, "Rock Mass Classification in Rock Engineering", Symp. on Exploration for Rock Eng., Vol 1., pp. 97-106, (1976)
11. E. Hoek and E. T. Brown, "Underground Excavations in Rock", Institution of Mining and Metallurgy, London, England, (1980)
12. W. H. Ward, "Ground Supports for Tunnels in Weak Rocks", 8th Rankine Lecture, Geotechnique, Vol. 23, No. 3, pp. 187-223, (1978)
13. E. Hoek and J. Bray, "Rock Slope Engineering", Institution of Mining and Metallurgy, London, England, (1974)

ROCK TESTING FOR THE GALLIPOLIS REPLACEMENT

by

Stephen T. Hornbeck
US Army Engineer District, Huntington
Huntington, West Virginia 25701

Abstract. The Gallipolis Locks Replacement Project involves the construction of a new and larger set of locks in a canal adjacent to the existing locks and dam. The foundation for the new locks consists of sandstones, siltstones, and claystones. Extensive testing was performed on the foundation rock to determine its strength properties as well as the strength of the future concrete to rock interface. For some of this testing a statistical lower bound interpretation was used to ensure that the strength parameters were both conservative and appropriate. Confirmatory testing was performed on one of the rock types for which a lower bound interpretation was used and it substantiated the derived strength parameters. Inclined shear tests were run in an unsuccessful attempt to define the cross bed shear values for a passive wedge. Further testing was performed during construction to evaluate the influence of Aerospray Binder 70 on the strength of the bond between the concrete and rock.

The Gallipolis project is now in construction and it can be seen that the testing program was an accurate, cost effective tool leading to a proper foundation design for the project.

Introduction

The Gallipolis Locks and Dam are located on the Ohio River at river mile 279.2. Considerable delays were experienced by the towing industry at this project due to the heavy volume of traffic and the fact that the main lock is presently 600 feet in length. To remedy this situation the District is currently constructing a new set of locks with a 1200-foot main lock in a canal which is adjacent to the old locks. This will allow the old dam to remain in place and continue to function after rehabilitation.

Site Conditions

The concrete monoliths which form the lock walls are generally founded in rock at a depth of about 75 feet below top of ground. The overburden consists of approximately 25 feet of cohesive materials above 35 feet of granular soils. The rock at the site consists of sandstones, siltstones, and claystones of the coal-bearing Pennsylvanian Age rocks. Top of rock is at a depth of 63 feet. The lock walls are generally founded about 15 feet into the rock.

Loading Conditions

The lock walls are subjected to a variety of loads which include differential water levels, ice, barge impacts, soil backfill, and earthquakes. Current Corps criteria require a factor of safety of 2 for most loading conditions.

Exploratory Drilling

Although a small number of exploratory holes were drilled in 1964 and 1972 to examine the feasibility of the canal scheme, the majority of core holes were drilled in 1982 and 1983. Approximately 65 holes were drilled as the first stage of a two stage drilling program. After these cores were evaluated, approximately 20 holes were drilled in stage 2. Thus, 85 holes were drilled and with 257 monoliths each hole represented, on the average, 3 monoliths. Considering that certain areas were drilled in detail each hole actually represented about 5 monoliths, or 200 linear feet of wall.

Each core hole was drilled twenty feet below the proposed founding elevation at the hole location. Preliminary estimates of the loading indicated that at this depth the loads were reduced to about 20% of the compressive strength of the weakest rocks at the site. In addition, the core from several holes was wrapped in plastic to preserve the natural moisture for testing.

The evaluation of the core was ongoing throughout the exploratory drilling. The evaluation began with the field log prepared by the drilling contractor. The cores were then examined and logged at a warehouse facility. During the final logging potential samples were identified for later reference. When several final logs were completed geologic cross section were started and correlation of the stratigraphy began. When the correlation had progressed to the point where the site could be characterized properly samples for testing were selected and sent to Ohio River Division Laboratories in Cincinnati, Ohio.

Laboratory Testing

At the laboratory the samples were prepared for direct shear and unconfined compression testing. The four rock types, sandstone, siltstone, claystone, and indurated clay, were tested for sliding and bearing capacity with direct shear and unconfined compression tests. For direct shear each rock type was tested for both peak shear and sliding friction and both vertical and horizontal deformation readings were taken. To develop the sliding envelopes six points

were run for each rock type with both peak and sliding friction tests performed on each sample. For bearing capacity unconfined compression tests were run with deformation readings taken for Modulus of Elasticity and Poisson's Ratio. Generally six compression tests were run for each rock type. In addition, several samples of each rock type were tested for unit weight, specific gravity, and moisture content.

Data Interpretation

As is common with coal-measure rocks the test data had a considerable amount of scatter for some of the rock types. To aide in the evaluation of the data a lower bound statistical analysis was performed on the direct shear and sliding friction tests for each rock type. The development of the lower bound analysis was a joint effort with Carol Spease who is now with the Bureau of Labor Statistics in Washington, DC. The analysis provides a 95% lower bound for the strength envelope. In theory if 100 sampling and testing programs were run on the same material and linear regression analyses performed, 95 of these linear regressions would fall above the lower bound.

The lower bound envelopes are not always reasonable and in certain circumstances they provide illogical results such as a negative cohesion. The analysis is a useful tool in the hands of an experienced designer, but it is not a panacea for poor data or inexperience.

Three envelopes can be seen on Figure 1. The lower envelope is the 95% lower bound, the middle envelope is the design envelope, and the upper envelope resulted from routine linear regression analysis of the data. From the lowest to the highest the change in the tangent of the angle of friction is slightly larger than .08, while the cohesion intercept changes by a factor of 2. For the loading conditions examined at Gallipolis the influence of the cohesion intercept is greater than the influence of the tangent of the friction angle. In a sliding block analysis the restraining force of the cohesion is based on the area over which the cohesion acts. For Gallipolis many of the monoliths are 40 feet by 40 feet so that 1600 square feet are available which translates to over 200,000 square inches and at 8 psi this will furnish somewhat less than two million pounds of resisting force.

Figure 2 portrays what may happen when the lower bound analysis encounters an extremely variable material with low strength such as the indurated clays. After consideration there is good reason

for the appearance of this lower bound. The indurated clays are essentially over-consolidated claystones with fractures and planes of previous failures present throughout the material. Naturally the strength of a small sample is influenced considerably by the orientation of the loading with respect to these planes of weakness. Since the plane which will control the strength cannot be identified prior to testing, the loading is applied at random with respect to the planes. The resulting scatter in the data is obvious.

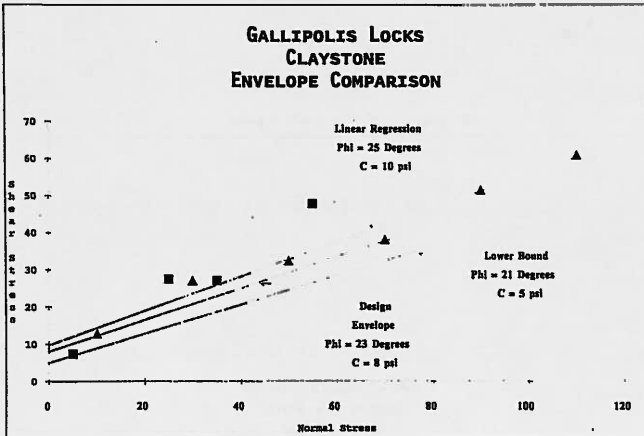


Figure 1

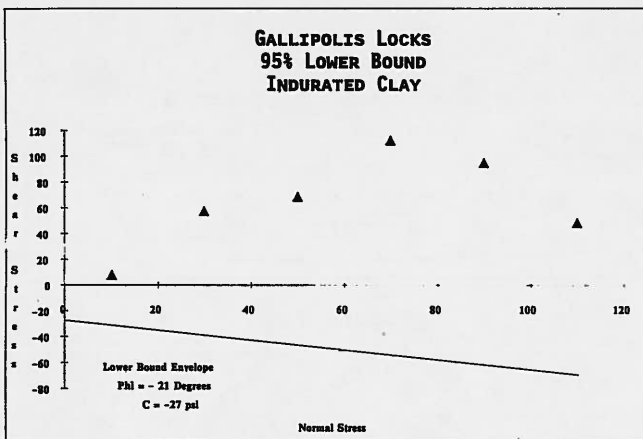


Figure 2

Using these types of interpretation, linear regression, lower bound, and reasonable fit the design envelopes for the various rock types were determined. Figures 3 through 6 show all test data and the design envelopes assigned to their respective rock types. The characteristics of the sandstones and siltstones are normal for these types of rock and are familiar to most people. In terms of strength they are adequate for most engineering purposes particularly when intact, and they are usually consistent in their characteristics. Claystone is somewhat of a misnomer for the Gallipolis site since the unit is actually an interbedded material with approximately 60% siltstone and 40% claystone. The

claystones are of low enough strength that they control the strength of the unit. The claystones are often bounded above and below by clay seams which appear to have resulted from the greater consolidation of the claystones during lithification. With greater consolidation greater deformation is required in order to readjust to the loading. These deformations appear to have generated the clay seams. The claystones appear as discontinuous lenses within the siltstone and do not correlate from hole to hole. The strength parameters for this unit had to be conservative enough to account for the variation from a relatively strong siltstone to a relatively weak clay seam. For this reason the sliding friction tests for the claystone were used to model this material. Because of the number of structures founded in this material confirmatory tests were run in order to ensure that the strength parameters were appropriate. Figure 5 shows the strength envelope for the claystones with the initial and confirmatory tests delineated by different symbols. By referring back to Figure 1 it can be seen that the linear regression is too optimistic and the lower bound is too conservative. While selecting a strength without the regression or lower bound analyses would have resulted in approximately the same envelope, these type of analyses provide a rational method for the selection of the envelope rather than a subjective visual fit to the data.

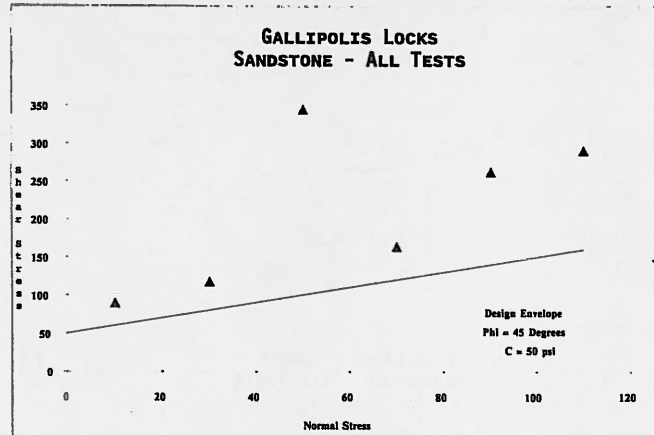


Figure 3

Figure 6 shows the indurated clays which have already been discussed in some detail. Some of the variability of the material can be attributed to the orientation of the planes. Further variation results from the amount of silt sized particles in the material. Those indurated clays with very low silt contents are very weak indeed, while those with relatively high silt contents (although still considerably less than 50%) begin to resemble siltstones. This percentage of silt has a dramatic influence on the recovery of the materials during coring. Unfortunately one cannot

test those materials which are not recovered so the strength parameters for for the indurated clays are necessarily more conservative in appearance. None of the major structures at Gallipolis are founded in the indurated clays. Some of the smaller structures were founded so as to be a minimum of 5 feet above the indurated clays. In this way the imposed loads were kept to only a fraction of the compressive and shear strengths of the material. Previous experience in the Huntington District has shown that the indurated clays are dependable if the material is constrained and not subject to wetting and drying. Confirmatory tests were performed on the indurated clays and are shown on Figure 6.

To determine the strength of the concrete to rock interface direct shear tests were performed on cores with grout molded to the specimen. The result of these test are shown on Figures 7, 8, and 9. In general the strengths were lower than expected but still within reasonable limits.

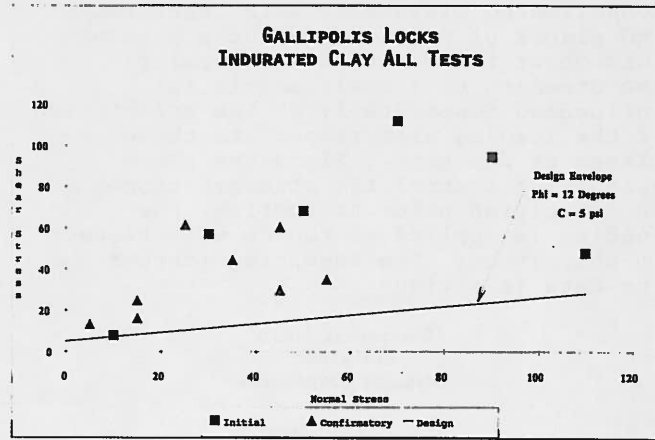


Figure 6

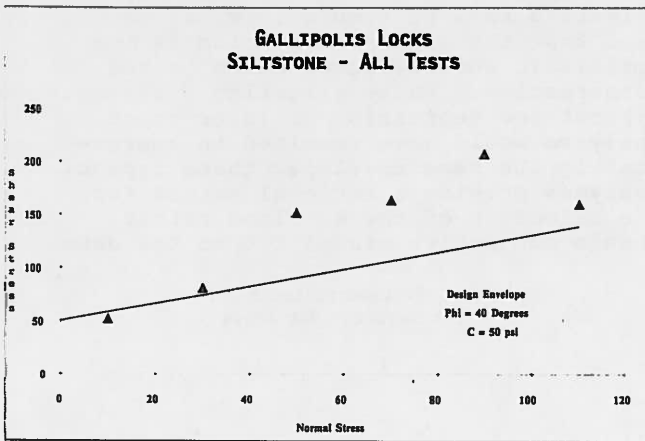


Figure 4

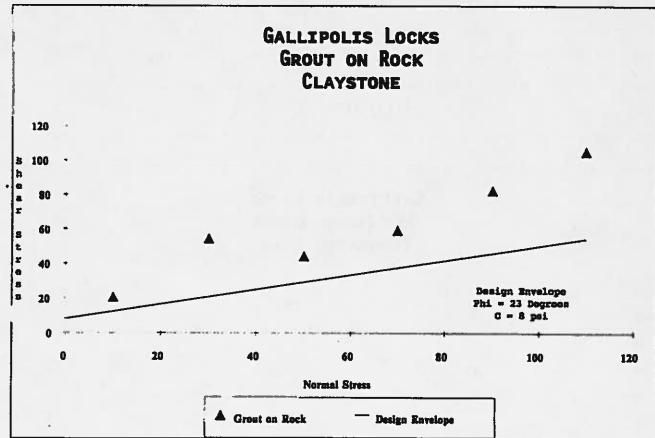


Figure 7

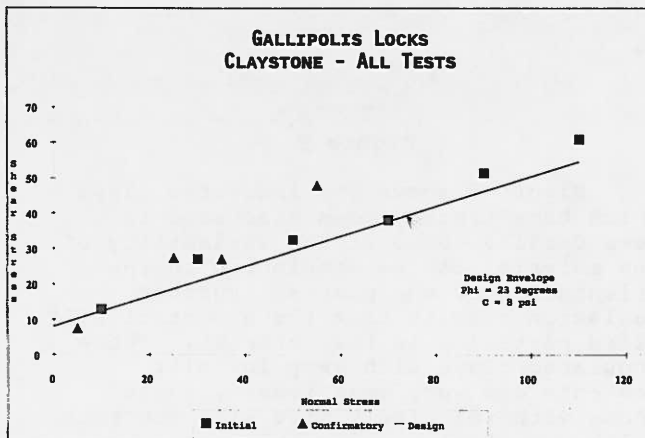


Figure 5

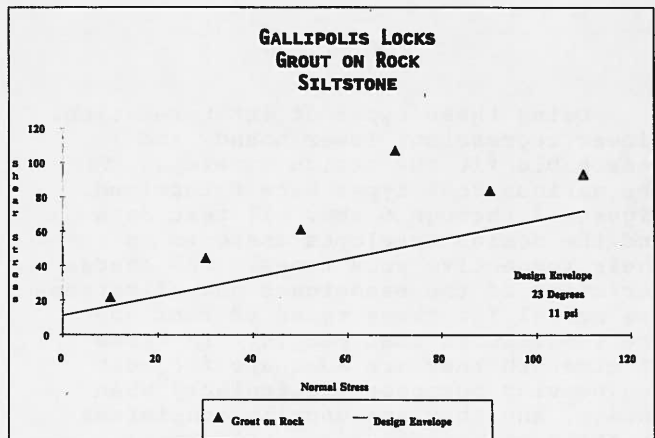


Figure 8

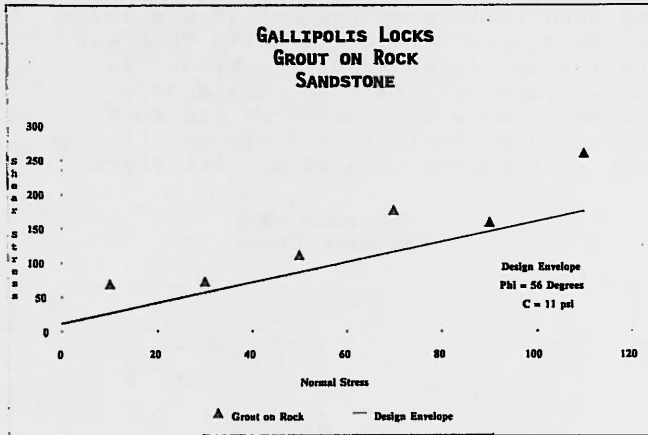


Figure 9

Cross-Bed Shear Tests

As mentioned earlier current Corps of Engineers criteria require a factor of safety of 2 for most loading conditions. With the strength parameters discussed so far several of the monoliths require additional restraining force to reach a factor of safety of 2. Figure 10 shows the factors of safety for the various monoliths of the river wall without consideration of any passive wedge. For those monoliths with factors below 2 the restraining force of a passive wedge was required to meet criteria. The strength of the plane of failure for the passive wedge is difficult to estimate since there are no small scale tests which model the condition accurately. Failure must occur across the bedding and possibly through several units. Higher authority requested that Huntington District try to conduct testing in an attempt to confirm the cross bed shear strength for the claystone (interbedded) and siltstone units at Gallipolis. It was felt that there were only three possibilities for this conformation: 1-Large scale testing, 2-Triaxial Testing, and 3-Inclined direct shear. Large scale testing was impractical since the rocks were covered by 60 feet of overburden and dewatering would be required to reach top of rock. For triaxial testing a literature search was conducted on the previous testing in the Huntington District where both triaxial and direct shear testing had been performed on the same rock units. Based on three projects it was found that the tangent of the phi angle from a triaxial test averaged less than half of that from a direct shear test. Furthermore, the cohesion from a triaxial test averaged over 5 times that determined from a direct shear test, shown on Figure 11. The standard deviation for the tan phi average was .05 and for the cohesion was 1.79 indicating that the averages were reasonable. Since these averages refer to peak values from the direct shear test it appeared that triaxial test would yield an unreasonably large value at Gallipolis. Thus inclined

direct shear tests were attempted to better define the cross bed shear value.

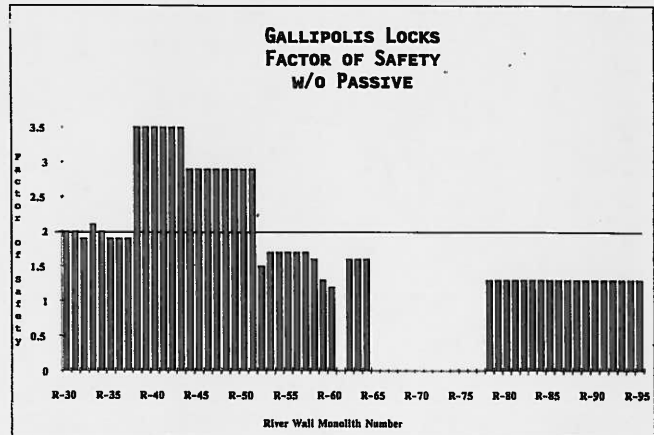


Figure 10

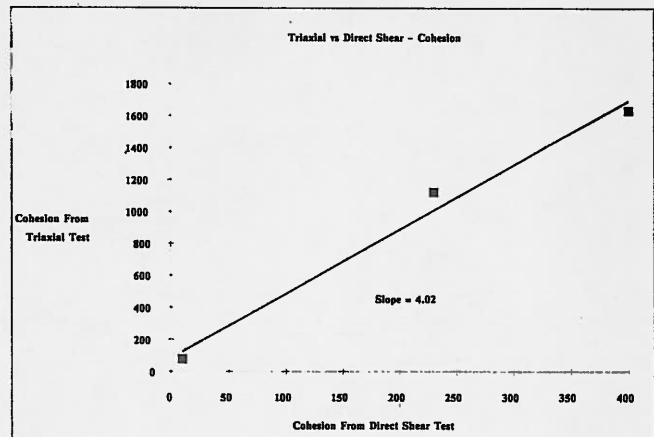


Figure 11

For the inclined shear tests the core was molded into the shear blocks at a 30 degree angle to model the failure through the material across the bedding. During testing it was noted that the failure appeared to occur along the bedding planes even though the sample was oriented so that uphill sliding was necessary to achieve this. During the evaluation this was confirmed since the strength obtained was approximately equal to the peak strength with 30 degrees added to the friction angle. This data is shown on Figures 12 and 13. In retrospect it might have been more realistic to test the core edgewise. That option was discarded because of the unknowns associated with stress distribution between the core and the material used to mold the samples into the shear box.

Aerospray Binder Tests

The claystones and indurated clays at Gallipolis are expected to degrade with exposure to wetting and drying. To combat this deterioration a binder material which provides a protective membrane to retard water loss and gain was specified. There was concern for its

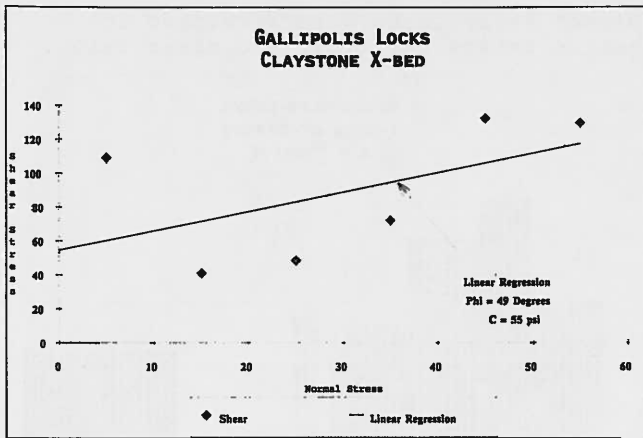


Figure 12

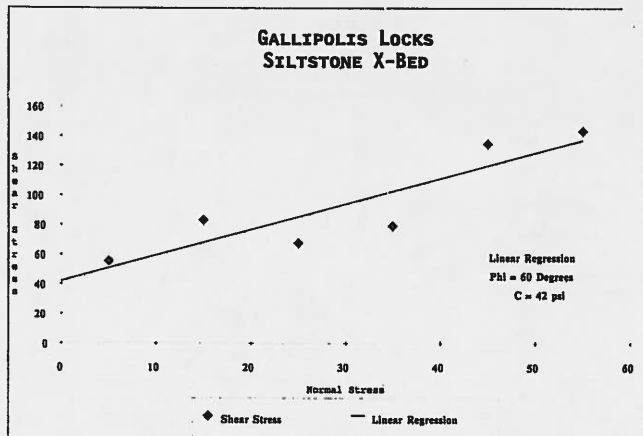


Figure 13

influence on the concrete to rock bond strength where the bond was on the horizontal founding surface. Tests were

performed only on the claystones since no structures were founded in the indurated clays. Figure 14 shows that the influence of the aerospray binder is negligible. In fact the tests were probably more indicative of the rock strength since failure often occurred in the rock rather than at the interface.

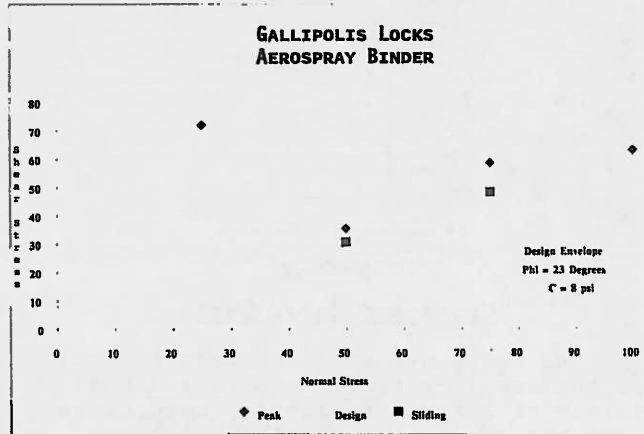


Figure 14

Conclusion

The current cost for the construction of the locks is \$217,000,000.00. The laboratory testing program considerably improved the efficiency of the design leading to savings well in excess of the \$15,000.00 cost for the testing. The project is now under construction with approximately 70% of the foundation excavated, and concrete placement has begun. Inspections at the site by a number of people have shown that the site was characterized properly and that the design strengths were appropriate.

EVALUATION OF IN SITU ROCK MASS MODULUS BY USE OF
BOREHOLE PRESSURE CELL AT TWO DAM SITES

By

Gordon M. Matheson, Ph.D., P.E.
Jack E. Mason, M.S., CPG
Schnabel Engineering Associates
Richmond, Virginia

Abstract. A borehole pressure cell was used to measure the in situ rock mass moduli at two sites being considered for Roller Compacted Concrete dams. One site was located in northern Maryland in a phyllite and schist terrain and is planned to be a 110-foot high structure. The second was located in southwest Virginia in limestones, siltstones and shales, and is planned to be a 242-foot high structure. The rock at both sites was classified by the Geomechanics Classification System using core drilling and testing data. Rock mass quality ratings obtained through this system were correlated with the in situ rock mass modulus using the methods developed by Bieniawski¹ (1978). The field measured moduli were found to be over estimated by the Bienawski correlation at the Virginia site, and to be beyond the limits of applicability of the method at the Maryland site. At the Maryland site, the in situ testing delineated the magnitude of the rock mass modulus that could not be predicted by the Geomechanics Classification of the rockmass and therefore had important implications in the suitability of the site for an RCC dam. Care should be taken in the interpretation of the borehole pressure cell data from both sites, however, since the volume of rock being measured is relatively small compared to the "critical volume" of rock that would represent the average rock mass deformation characteristics. The results of the testing provided data that will be used to assess the potential for settlement induced dam cracking and foundation stripping depth requirements.

Introduction

The relatively recent interest in construction of Roller Compacted Concrete (RCC) dams as an alternate to earth or rockfill dams has required better definition of rock foundation conditions for proposed dam sites. As with conventional concrete dams, the rock foundation for RCC dams must be adequate to maintain structural integrity from both a stability and deformability view point. Differential properties of foundation materials can create complicated rock-structure interaction problems. In addition, the designer must assess the required depths of excavation for

foundation preparation. Any over excavation required due to perceived inadequate foundation conditions can significantly impact construction economics. Although many factors must be considered in this decision, the stiffness of the foundation is often cited as one of the major factors in stripping depth specification. As a result, direct measurement of foundation stiffness by in situ testing methods has been completed during geotechnical investigations of many dam sites.

This paper presents the results of in situ modulus testing completed at two RCC dam sites in widely different terrains.

The case histories show the value of performing relatively inexpensive in situ tests as opposed to relying entirely on published correlations between rock mass classification and estimated in situ modulus.

In Situ Modulus Testing Considerations

A number of methods are available to assess the in situ deformation properties of a rock mass. These include: borehole tests, plate load tests, flat jack testing, and several other more exotic testing techniques. All testing techniques suffer from various disadvantages. From a broad perspective, these disadvantages may be separated into the two categories of 1) volume of sample tested, and 2) ability to easily test at various depths and locations. Borehole techniques suffer most from the first category since they only test a relatively small volume immediately surrounding the borehole (e.g., for one-foot long cell in an NX hole the tested volume is less than a cubic-foot). These methods do, however, allow testing at many locations at a site in a relatively quick and straight forward manner. The plate load tests, flat jack tests, and other larger scale tests while testing a larger volume of rock, require extensive site preparation and can usually only be performed at surface exposures or in underground adits. As a result, not only do they provide limited information on the variety of site materials, but they are very costly and time consuming.

Due to the nature of dam projects, it was felt that the use of borehole tests would provide the most feasible method to collect in situ modulus data. For these programs, a borehole pressure cell based on the design developed by Hustralid and Hustralid² (1972) at Colorado School of Mines was used. This cell, commonly referred to as the CSM cell, was chosen over the well known Goodman Jack due to its low cost, ease of operation and ease of data interpretation.

The CSM cell uses a one-foot long neoprene membrane that expands against the perimeter of a borehole. Once seated against the borehole, the dilation of the borehole caused by increasing pressure, can be measured in terms of volume change in the system. These measurements are then compared to calibration measurements completed on a material of known property to calculate the measured in situ rock mass shear modulus. The elastic modulus can then be estimated from the shear modulus by assuming Poisson's ratio and using classical elastic theory.

The CSM cell allows testing of the rock with pressures up to 5,000 psi. Again, it should be noted that the rock volume tested by the cell is relatively small as compared to the "critical volume" (i.e., the volume above which the measured

rock mass properties become relatively constant) which is normally assumed adequate to represent rock mass response. As suggested earlier, the volume tested by the cell is less than one cubic foot of rock while typically somewhere in the range of 20 to 200 cubic feet of rock is considered necessary to include sufficient rock discontinuities to be considered the "critical volume." This will have the effect of producing a relatively large variation in the borehole pressure cell data with some data indicating a high modulus in unfractured to slightly fractured zones and other data indicating a low modulus in very fractured zones. If the data is considered as a whole, however, the relative magnitude of the modulus of various rock units should be well approximated by the data.

Correlations between in situ modulus and various rock mass classification parameters have been suggested. Bieniawski³ (1978) reviewed these methods and suggested a correlation based on the Geomechanics Classification System's Rock Mass Rating. The Rock Mass Rating (RMR) uses various measurable rock properties in conjunction with rating criteria for the properties to calculate a number between 0 to 100 that provides an indication of the rock mass quality. The rock mass parameters considered in the RMR include compressive strength, fracture frequency, Rock Quality Designation (RQD), water pressure, joint wall strength, and relative fracture orientation. For this classification, the higher the RMR, the better the rock mass quality. Table 1 presents Bieniawski's qualitative RMR classification. According to Bieniawski, the following relation approximately represents the correlation between RMR and in situ modulus:

$$E(x10^4\text{psi}) = 0.29 \text{ RMR} - 14.5$$

where:

E = in situ modulus

RMR = Rock Mass Rating

Note that the above relation has been transformed into english units from Bieniawski's original metric unit equation.

TABLE 1 GEOMECHANICS CLASSIFICATION OF ROCK MASSES

Class	DESCRIPTION OF ROCK MASS	RMR RANGE
I	Very good rock	81-100
II	Good rock	61-80
III	Fair rock	41-60
IV	Poor rock	21-40
V	Very poor rock	0-20

For further discussion of the Geomechanics Classification System see Goodman⁴ (1980) or Bieniawski (1976). Bieniawski's modulus correlation with the RMR is based on rock testing at various sites and includes both large scale and borehole jacking test data. Bieniawski indicates that his correlation with the estimated moduli should only be used for preliminary design and site specific testing should be completed if the in situ moduli is a critical design parameter.

General Site Conditions

The sites considered are for the proposed Gillis Falls Dam in Northern Maryland and the proposed Spring Hollow Dam in Southwest Virginia. The general location of these sites are shown on Figure 1. As currently envisioned the Gillis Falls Dam would be approximately 110 feet in height while the Spring Hollow Dam will be about 243 feet in height. Both dams are envisioned to be a typical RCC construction with a near vertical upstream face and a 0.8 to 1.0 downstream face and a 16 to 20-foot crest width.

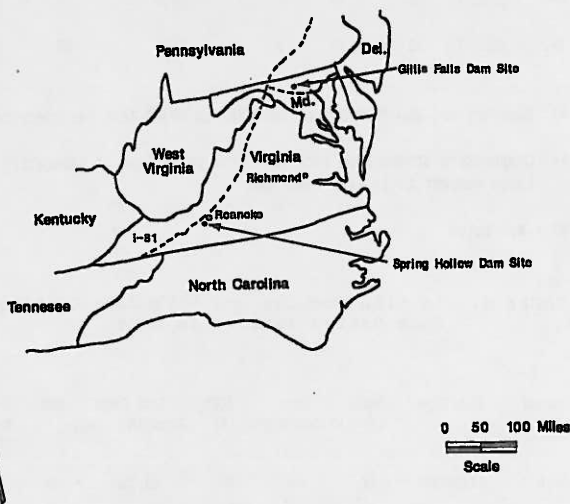


Figure 1: Relative Location of Gillis Falls and Spring Hollow Dam Sites

Gillis Falls Dam Site

The Gillis Falls Dam site lies within the Piedmont Physiographic Province of the eastern coast of the United States. The Piedmont is a northeast trending band of old metamorphic rocks, separating the Coastal Plain Province to the east from the Ridge and Valley Province to the west. The dominant rock types in the site area are phyllites and schists of the Ijamsville formation which also contains beds of quartzite, quartz veins, and quartz porphroblasts. These foliated phyllites and schists have been severely faulted and folded.

The proposed 110-foot high impoundment structure is located in a narrow valley about 500 feet wide with relatively steep side slopes estimated at about 2.0 to 3.0 horizontal to 1.0 vertical. Topographically the site ranges from about elevation 540 feet at the valley bottom to elevation 700 feet on a hill top near the left abutment.

A truck mounted drill rig was used to obtain NX rock core samples at the Gillis Falls dam site. The core recovery from the drilling ranged from 10% to 90% with an average of about 44%. The RQD was generally low, ranging from 0% to 50%, with an average of about 14%. This low RQD was due to the heterogeneity of the rockmass. Relatively hard quartz layers lenses and porphroblasts present in the predominately phyllite rockmass tended to break and detach from the softer mica matrix and plug the core barrel. Advancement of the plugged core barrel continued through the rock without significant core recovery until the next quartz body was encountered. At that point, it would become apparent that the core barrel was plugged. This process and rock characteristic tended to yield artificially low core recovery at the site.

Point Load Tests were performed on the rock core recovered. Due to the anisotropic nature of the rock, they were tested parallel and perpendicular to the foliation planes. Variations and ranges in uniaxial compressive strengths were similar for both orientations with a combined average compressive strength of about 7800 psi and a range from less than 1000 psi up to about 17,200 psi.

Spring Hollow Dam Site

The Spring Hollow Dam Site is located in the upper Roanoke River Valley within the Ridge and Valley Physiographic Province of Virginia. The sedimentary paleozoic rocks at the site have been folded and faulted. The bedding of the rock strikes perpendicular to the reservoir valley and dips 60 to 70 degrees downstream.

The reservoir site is underlain by rocks of the Rome Formation of Lower Cambrian geologic age. The Rome formation consists of siltstones, shales, calcareous shales, dolomites, and limestones. Karst features are present in the carbonates as sinkholes, subsurface voids of varying sizes and solution channels.

The abutment slopes at the dam site are relatively steep and range in grade from 1.6 to 2.0 vertical to 1.0 horizontal. Maximum topographic relief at the site is approximately 530 feet, ranging from about elevation 1185 feet along the streambed at the dam axis to elevation 1714 feet at the ridgecrest on the left abutment.

A truck mounted drill rig was used to obtain NX rock core samples at the Spring Hollow dam site. The core recovery at this site was generally excellent and ranged from 32% to 100% with an average of 99%. The measured RQD ranged from about 40% to 100%. The average RQD was about 85%. The variations in the RQD result from zones of folding, faulting, and solutioning. Calcite fills many fractures and vugs, recementing much of the rock into a relatively solid mass in many zones. The rehealing of the carbonates tended to increase the RQD and therefore, the RMR of the rock mass at the site.

Point load testing indicated an average compressive strength of about 13,000 psi for fresh to slightly weathered siltstones calcareous shales, shaley limestones and limestones. Non-calcareous shales tended to have a lower compressive strength, that averaged about 9,000 psi. Lower rock strengths were measured for the weathered rocks. In general, however, rock considered suitable for the foundation support had a compressive strength of at least 5000 psi.

In Situ Testing Results

Gillis Falls Dam Site

In situ modulus testing was completed in three boreholes at the Gillis Falls Dam site. A total of 12 tests were completed. Table 2 presents a summary of the test results along with selected parameters from the test zones.

Review of Table 2 suggests that modulus values for Borings E-1 and S-2 are consistently low and are generally less than 0.5×10^6 psi. Modulus values for Boring E-3 are higher and are generally over 1.0×10^6 psi. As discussed previously, core recovery from these borings appeared to be hampered by the presence of quartz veins and porphyroblasts in the rock that plugged the core barrel. As a result, the quality of the rock mass as indicated by the core recovered and the in situ modulus measurements vary substantially. The generally poor core recovery and low RMR for borings E-1 and S-2 appear intuitively consistent, however, with the low moduli values measured.

Spring Hollow Dam Site

In situ modulus testing was completed in two boreholes at the Spring Hollow Dam site. A total of 15 successful tests were completed. Table 3 presents a summary of the tests completed with selected parameters from the test zones.

Review of Table 3 indicates that core recovery was excellent in all testing zones. Measured in situ moduli values were generally greater than 1.0×10^6 psi. A general correspondence can be seen between high and low RMR and moduli values.

Table 2. In Situ Modulus and Rock Core Quality Data From Gillis Falls Dam Site

Bore	Rock Type	Depth Feet	Core Recovery* (%)	RQD* (%)	Rock Comp. Strength** (psi)	RMR	In Situ Modulus (10^6 psi)
E-1	SCHIST	25	26	6	2800	44	0.30
E-1	SCHIST	30	15	4	4800	44	0.11
E-1	SCHIST	35	15	4	12,000	44	0.38
E-1	SCHIST	40	10	6	4200	36	0.02
E-1	SCHIST	45	10	6	4,500 TO 8,000	41	0.54
E-3	SCHIST	15	40	0	NOT TESTED	NR	0.05
E-3	SCHIST	20	18	0	NOT TESTED	NR	1.02
E-3	SCHIST & QUARTZ	25	44	0	NOT TESTED	NR	1.31
E-3	SCHIST	30	17	0	NOT TESTED	NR	1.17
E-3	SCHIST	33	17	0	NOT TESTED	NR	1.05
S-2	SCHIST	25	46	2	1,900	39	0.05
S-2	SCHIST	30	48	2	3400	37	0.10

* Recovery and RQD from 5-foot interval in which test was conducted

** Compressive strength on rock core from point load or laboratory tests nearest to in situ test zone

NR = Not Rated

TABLE 3. In Situ Modulus and Rock Core Quality Data from Spring Hollow Dam Site

Boring	Rock Type	Depth (feet)	Core Recovery*	RQD* (%)	Rock Comp. Strength** (psi)	RMR	In Situ Modulus (10^6 psi)
B-16	LIMESTONE	15	100	100	23,700	79	2.96
B-16	LIMESTONE	20	100	54	12,100	57	1.78
B-16	LIMESTONE	25	100	54	11,600	57	1.69
B-16	LIMESTONE	35	100	100	16,000	79	4.42
B-16	LIMESTONE	40	100	80	16,700	76	2.58
B-16	LIMESTONE	45	100	80	16,700	76	2.64
B-16	LIMESTONE	50	100	68	10,500	57	1.07
B-16	LIMESTONE	55	100	94	27,000	79	2.80
B-16	SHALEY LIMESTONE	60	100	94	25,000	79	3.95
B-19	CALCARIOUS SHALE	10	100	44	7,500	38	0.21
B-19	SHALEY LIMESTONE	15	100	86	7,500	53	0.84
B-19	SHALEY LIMESTONE	20	100	100	10,500	79	3.90
B-19	LIMESTONE	25	100	94	13,800	74	4.70
B-19	LIMESTONE	30	100	76	27,000	62	3.51
B-19	LIMESTONE	53	100	94	19,000	71	4.70

* Recovery and RQD from 5-foot interval in which test was conducted

** Compressive strength based on Point Load Test Data

Comparison of In Situ Test Results

Using the data generated from the two dam sites, a comparison can be made between the measured field values and the approximate relation developed by Bieniawski. For the Gillis Falls Dam site, the data from Boring E-3 was not considered suitable for direct comparison to the published data. This was because it was apparent from the modulus measurements that the rock core data did not adequately reflect actual rock mass conditions due to the drilling difficulties. Although the data from borings E-1 and S-2 from the Gillis Falls site were also somewhat affected by the same drilling difficulties, it is the authors' opinion that an estimated RMR could be developed for each of the test zones.

Figure 2 presents a plot of the data available from the dam sites. In addition, the line that represents Bieniawski's relation is shown on the plot. It should be noted that according to Bieniawski, the lower limit of applicability of his relation is a RMR of 55.

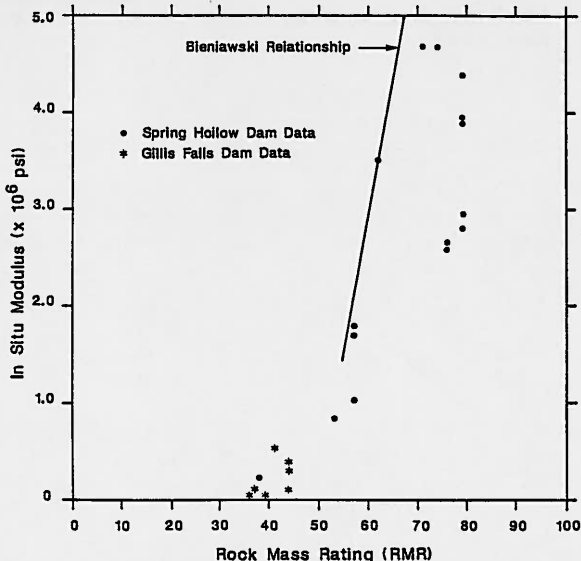


Figure 2: Plot of RMR versus In Situ Moduli Data

Review of Figure 2 indicates that, in general, for the range of applicability of the Bieniawski's relation, the relation consistently overestimates the in situ modulus of the rock zones by a factor of up to 3. Table 4 presents a direct comparison of the predicted versus measured in situ modulus values. This result was somewhat surprising since one would expect a general over estimation of the modulus if compared to Bieniawski's relation due to the relatively small volume of material tested. This could be due to variations in selection of RMR classification parameter values. However, the authors' review of the sensitivity of

the parameter selection does not indicate that the variation should be significant enough to result in the differences.

TABLE 4 COMPARISON OF MEASURED MODULI TO PREDICTED MODULI BY BIENIAWSKI RELATION

BORING	DEPTH	MEASURED MODULI (X10 ⁶ PSI)	RMR	PRED. MODULI (X10 ⁶ PSI)	RATIO OF PRED./MEASURED MODULI
B-16	15	2.96	79	8.41	2.84
B-16	20	1.78	51	2.03	1.14
B-16	25	1.69	51	2.03	1.20
B-16	35	4.42	79	8.41	1.90
B-16	40	2.58	76	7.54	2.92
B-16	45	2.64	76	7.54	2.86
B-16	50	1.07	57	2.03	1.90
B-16	55	2.80	79	8.41	3.00
B-16	60	3.95	79	8.41	2.13
B-19	20	3.90	79	8.41	2.16
B-19	25	4.70	74	6.96	1.48
B-19	30	3.51	62	3.48	0.99
B-19	53	4.70	71	6.09	1.30

Examination of the measurement data as a whole, suggests several ranges of modulus verses RMR. In general, it appears that for an RMR of 60 to 80, the in situ modulus can be expected to range between approximately 2 to 5 million psi. This probably approaches the intact rock modulus for RMR values above 75. For a RMR of 50 to 60, the in situ modulus appears to fall within the range of about 0.75 to 2.0 million psi. For an RMR of 40 to 50 the moduli appear to fall between 100,000 to 750,000 psi. If the RMR falls below 40 the in situ moduli value could drop to the range of 10,000 to 100,000 psi. RMR values below 35 probably represent more of a very highly weathered saprolitic rock material and would better be evaluated by soil mechanic methods (i.e., pressuremeters). These ranges are summarized in Table 5.

TABLE 5 Estimated Range of In Situ Rock Modulus Related to RMR

RMR	In Situ Moduli (psi)	Comments
Greater than 60	2 to 5 million	RMR greater than 75 probably approaches intact moduli of rock
50 to 60	0.75 to 2 million	
40 to 50	0.10 to 0.75 million	
Below 40	less than 0.10 mil.	RMR less than 35 represents probably saprolite

Although the ranges presented in Table 5 appear relatively wide, they should be adequate for most design studies for structures in rock. It should be noted that the ranges of data presented are based on a limited amount of data and may require modification as the database enlarges.

Conclusions

Based on the in situ moduli data presented, the following general conclusions have been drawn:

- 1) The measurement of in situ modulus can assist in evaluation of rock foundation conditions in areas where core recovery may be influenced by local adverse conditions.
- 2) The Rock Mass Rating (RMR) does appear to be correlated to various ranges of in situ moduli.
- 3) The in situ modulus correlation suggested by Bieniawski appears to over estimate the in situ moduli values for the data collected.
- 4) A range of in situ moduli values for ranges of RMR have been presented in Table 5. These ranges can be used to estimate in situ rock moduli for use in rock structure design.

Acknowledgements

The authors gratefully acknowledge the support of Schnabel Engineering Associates in the preparation of this manuscript.

References

1. Bieniawski, Z.T. (1978) Determining Rock Mass Deformability-Experience from Case Histories, Int. J. Rock Mechanics, Min. Sci., Vol. 15, No. 5, pp. 237-248.
2. Hustralid, William and Hustralid, Andrew (1972). Development of a Borehole Device to Determine the Modulus of Rigidity of Coal Measure Rocks: for U.S. Dept. of Interior, Bureau of Mines, Final Report, Part I and II, 55 p.
3. Bieniawski, Z.T. (1976) Rock Mass Classification in Rock Engineering, Proceedings of Symposium on Exploration for Rock Engineering (Balkema, Rotterdam) Vol. 1 pp. 97-106.
4. Goodman, Richard E. (1980) Introduction to Rock Mechanics, John Wiley and Sons, New York.

AN OVERVIEW OF CURRENT U.S. PRACTICE IN DAM STABILIZATION
USING PRESTRESSED ROCK ANCHORS

Dr. Donald A. Bruce
Technical Director, Nicholson Construction Company
Bridgeville, Pennsylvania 15017

Abstract. Rock anchors have been exploited in many applications around the world since their inception at Cheurfas Dam, Algeria, in 1934. Their arrival in the United States is a more recent phenomenon, but during the last 20 years or so, there has been a growing application in the field of dam repair, in line with the general theme of infrastructure rehabilitation and upgrading. The paper highlights aspects of anchor design which are of most common concern to U.S. engineers, and reviews typical construction methods. Performance assessment and monitoring are also discussed in the light of the absence of a true native standard. Details from some recent major contracts are discussed throughout by way of illustration.

Introduction

No concrete dam has ever failed in the United States. However, largely as a result of the Federal Energy Regulatory Commission's revision of its flood load criteria, and the fact that many concrete hydroelectric dams are up for renewal of their 50 year licenses, there is rapidly increasing activity in fields related to dam safety enforcement.¹

To indicate the scale of the problem, it may be noted that State governments regulate more than 80,000 dams, and the Federal government owns 3,000 more. According to a Corps of Engineers' inventory, nearly one quarter of all U.S. dams are now at least 50 years old, compared to 5% in 1980, and a projected 33% in 2000.² In 1981 the Corps completed inspecting about 8800 non-federal dams classified as high hazard, based on threats to habitations downstream. It found one third unsafe by current criteria.

In this highly charged atmosphere, major steps are being made in the dam safety research effort. For example, the Corps launched a six-year, \$35 million research program in 1984, while the Bureau of Reclamation's \$650 million evaluation and repair program covering its 350 dams is expected to peak in 1992.¹ Universities, and bodies such as the Association of State Dam Safety Officials (ASDSO) and the U.S. Commission on Large Dams (USCOLD) are also contributing increasingly useful pools of information.

The burden of actually repairing the threatened structures has largely passed to outside contractors. Specialist geotechnical engineering and structural repair companies have responded vigorously to the challenge. For example, ingenious methods have been evolved for installing deep cut-offs through and under existing embankment dams,³ for bonding fissured concrete structures under full hydraulic head,⁴ and for combatting foundation liquefaction potential^{5, 6} and seepage⁷ by grouting methods. At the same time, levels of activity and expertise in the longer established technique of rock anchoring have reached new heights.

The use of prestressed rock anchors in dam engineering is as old as the technique itself: the first recorded use of anchors was to stabilize Cheurfas Dam, Algeria, in 1934. Since then, anchoring has gained worldwide recognition and application not only in connection with dams but for a multitude of other purposes.⁸

The classic applications for dams are to provide resistance to overturning (Figure 1) and restraint to sliding (Figure 2). However, there are countless examples of anchors being used as tie-downs for spillway construction (Figure 3) and for straightforward rock slope stability around tunnel portals or abutment excavations. Again, another famous example was in the Service Spillway Plunge Pool of Tarbela Dam, Pakistan (Figure 4) where almost

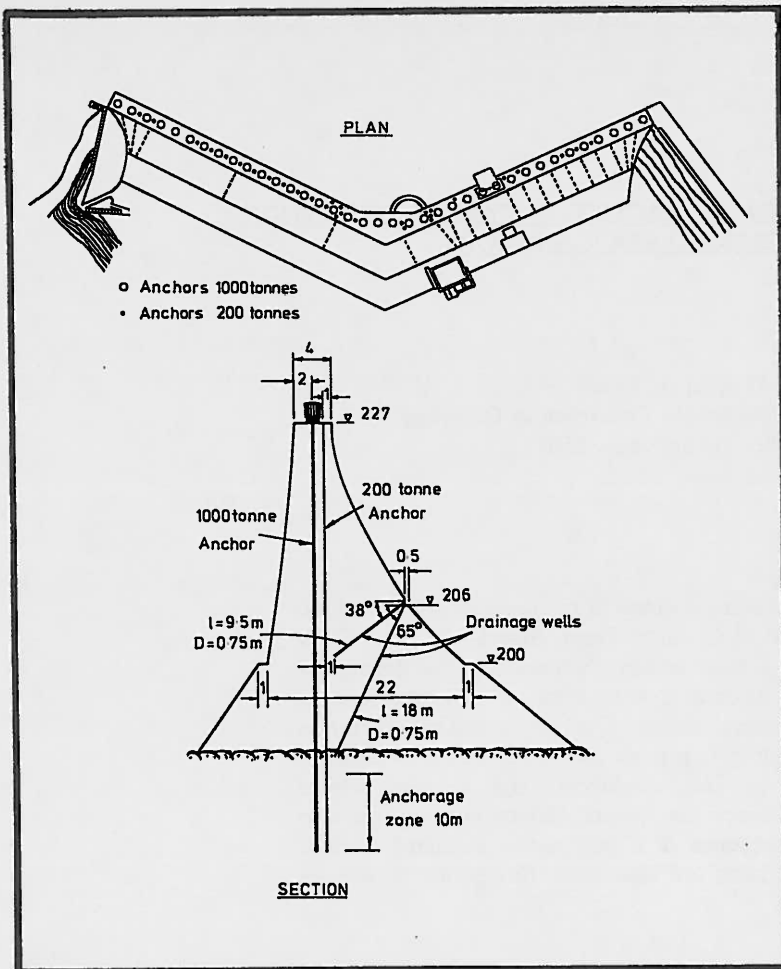


Figure 1. Layout of anchors to resist overturning, Cheurfas Dam, Algeria⁹.

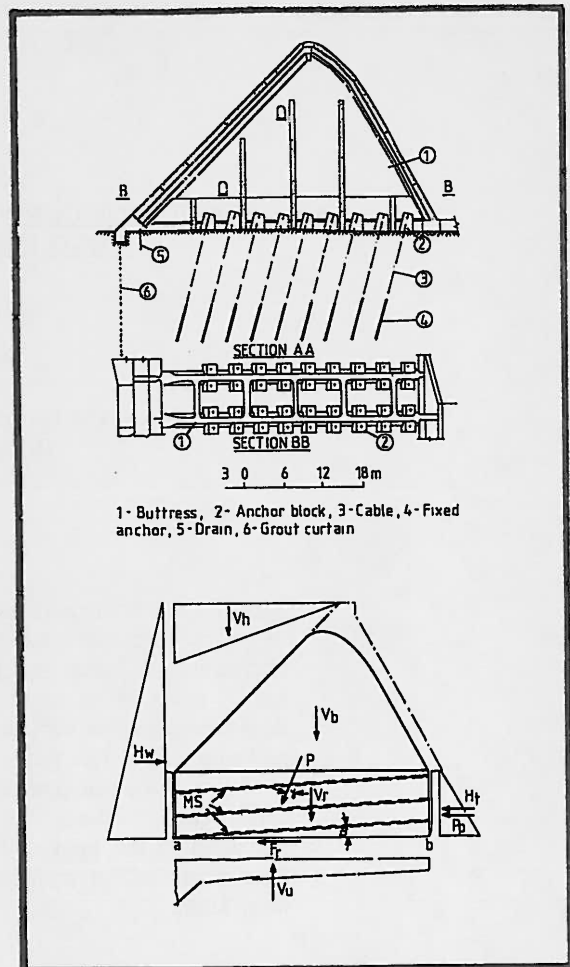


Figure 2. Anchors to resist sliding, Muda Dam, Malaysia¹⁰.

2000 anchors of 430 ton ultimate tendon capacity were installed to provide, in effect, a zone of "compressed rock" to further resist the tremendous dynamic forces exerted during operation of the Spillway/Flipbucket structure. A newer concept is foreseen for the repair of Stewart Mountain Dam, AZ: not only will the anchors provide against a general overturning failure of the concrete structure as a unit, they will also act to compress contiguous blocks (separated by horizontal construction joints) to prevent their relative movements within the structure in the event of high seismic activity.¹²

As noted below, there is a wealth of literature available to the engineer who wishes to design anchor systems. Equally there is generally a high degree of experience and expertise to exploit from within the ranks of the specialist contractors. This paper does not attempt to summarize these data, and is not a fundamental guide to the various steps of design, construction and stressing. Rather, it highlights major points in modern rock anchor technology which have been found to be controversial, little appreciated or novel. It therefore demands of the reader a certain knowledge of the basic theory and practice. Its major purpose is to help reduce conflict and debate between Owner and Contractor so that the technical and

economic advantages of the technique can be exploited to mutual benefit.

Sources of Data

The stabilization of dams by anchors is always a process involving a high engineering content, and frequently provides unique or exceptional facets such as those arising from difficult access restrictions, extreme construction tolerances or tendons of great weight or length. For example, anchors of 1400 ton ultimate tendon capacity up to 380' long were installed from the 12' wide crest of Lalla Takerkoust Dam, Morocco,¹³ while in the U.S., tendons weighing over 5 tons have been installed under equally restrictive conditions at Shepaug Dam, CT. For these same reasons, good case histories abound in the engineering press, usually authored by the design engineer or specialist contractor involved.^{14, 15} Frequently, however, such papers can have a strongly commercial motive, and although they contribute to the general pool of knowledge, often do not approach the subject in a generic manner. In contrast, major literature survey reviews have proved very useful to the anchor community in that they provide a perspective on all aspects of the technology from design

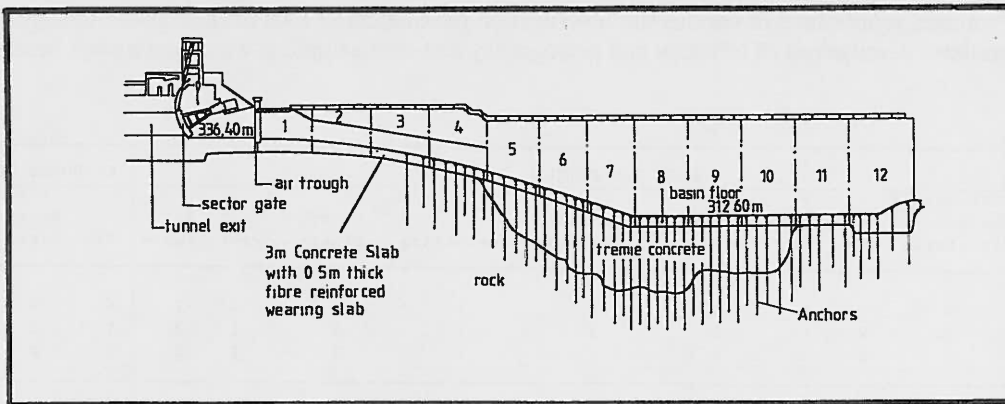


Figure 3. Anchors to repair stilling basin, Tarbela Dam, Pakistan¹¹.

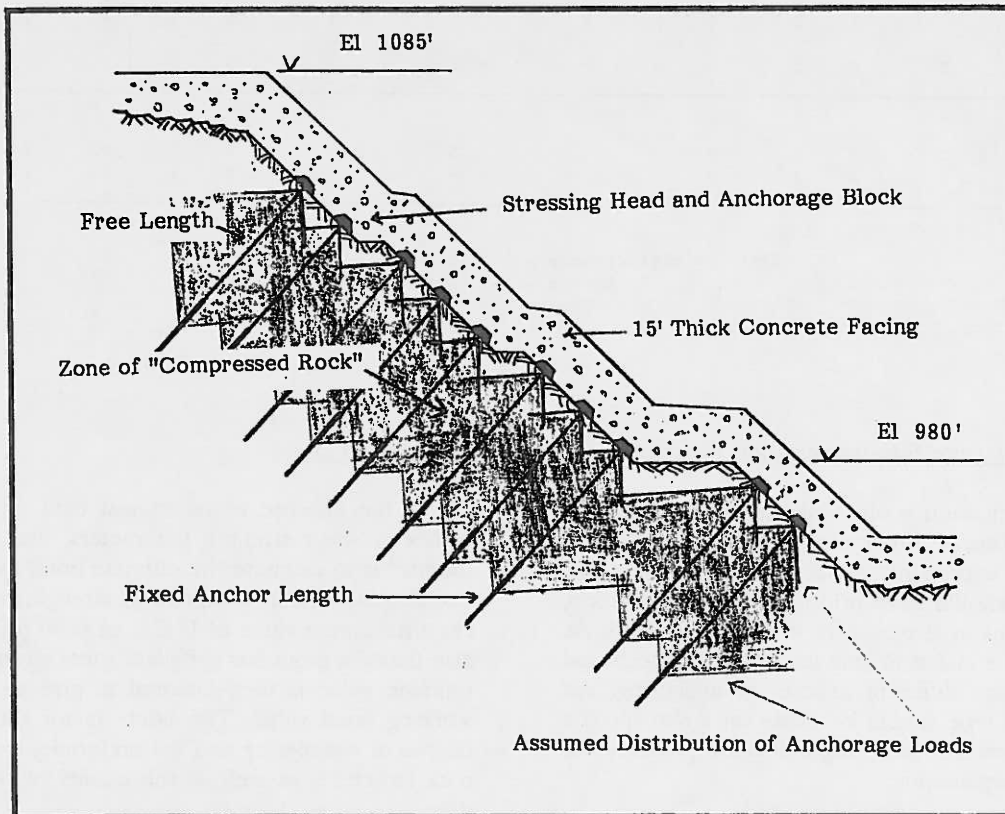


Figure 4. Anchors for slope stability and rock reinforcement, Service Spillway Plunge Pool, Tarbela Dam, Pakistan (idealized section).

and construction, through corrosion protection to stressing, testing and long-term performance.^{16, 17, 18}

With respect to national construction standards, there is, for example, currently no ASTM Code for rock anchors, earlier attempts having been disrupted by conflicting proprietary viewpoints. However, the Post Tensioning Institute Recommendations do constitute an acceptable and well regarded substitute, having been prepared by a representative cross section of anchor specialists.¹⁹

Probably the most comprehensive and up-to-date work on the world scene is the new British Code of Practice BS 8081²⁰ which supersedes the Draft for Development DD81²¹ for six years tried, tested, and modified by the industry. These documents complement the excellent FIP* documents on Design and Construction²² and Corrosion and Corrosion Protection.²³

*FIP are the familiar initials of the Fédération Internationale de la Précontrainte.

Table 1. Proposed significance of various site investigation parameters in rock anchorage technology. (Note: excludes descriptions of lithology and petrography and core sampling, which are always necessary.)

	ROCK PARAMETERS										HYDROGEOLOGIC PARAMETERS		
	Structure/ Bedding/ Fractures	UCS	Shear Str.	Abrasive- ness	Tensile Strength	Weathering	Permea- bility	Unit Wgt.	E Value		Flow/ Movement/ Pressure	Agress- ivity	
DESIGN													
Overall Stability	4	3	3	0	3	3	2	4	2		4	4	0
Rock Grout Bond	2	4	4	0	3	4	3	1	3		1	1	1
Grout-Steel Bond	1	3	1	0	1	1	2	1	3		0	0	0
CORROSION PROTECTION													
Design of Tendon	2	0	0	0	1	2	4	0	1		4	4	4
CONSTRUCTION													
Drilling	3	3	2	4	2	3	1	2	3		3	1	1
Installation	3	0	0	0	0	2	0	0	0		1	4	0
Water Test/Pre-grout/Redrilling	4	1	1	0	0	2	4	0	0		4	4	2
Grouting	3	0	0	0	0	2	4	0	0		3	3	4
PERFORMANCE													
Short Term	1	2	2	0	1	3	1	0	3		0	0	0
Long Term	3	2	2	0	1	3	3	0	1		3	3	4

Key: No significance - 0
V. Little - 1
Some - 2

A Lot - 3
Essential - 4

Requirements from Site Investigation Programs

The site investigation is obviously the starting point in the serious evaluation of a rock anchor design. Frequently, however, there is uncertainty about what parameters are most relevant, or should have priority, in an investigation, leading to omissions or irrelevances in the final report. As a guide, Table 1 is a matrix linking the various geotechnical parameters with the different aspects of anchoring. An assessment of this type should be made on a site specific basis by the planner. The following points are provided for clarification and explanation.

Overall Stability

Conventionally the overall stability (or ultimate uplift capacity) is calculated on the assumption that a single anchor engages a cone of rock, of a certain geometry and location (Figure 5). The uplift capacity is then estimated as the weight of rock in that cone (based on submerged weight, where appropriate). Investigations show clearly that this is normally an extremely conservative approach as it does not take into consideration the contribution of "rock strength" acting over the surface of the theoretical failure volume.^{16, 24} An appreciation of the rock mass structure will give a more accurate indication of a likely "failure volume," and also of the main rock strength parameters likely to contribute most strongly within and around it. Analyses of this type will therefore permit anchor embedment lengths to be minimized rationally, and so reduce overall costs.

Rock Grout Bond

In the absence of direct test data on ultimate bond values or shear strength parameters, one reliable rule of thumb²⁵ is to estimate the ultimate bond to be 10% of the U.C.S. (unconfined compressive strength) of massive rock (to a maximum value of U.C.S. of 6000 psi). This assumes also that the grout has sufficient shear strength in situ. The ultimate value is then factored to give an estimated safe working bond value. The safety factor should reflect the degree of weathering and the uniformity and nature of the rock structure, as well as the quality of the investigation data.

Significance of Knowing E Value of Rock Mass

Theoretical analyses²⁶ and field data²⁷ confirm that the distribution of bond stress in and along the fixed anchor length is dependent on the ratio of the elastic moduli of the anchor material (E_a) and the rock mass (E_r). The smaller this ratio (i.e., the higher the rock modulus) the greater is the bond stress concentration at the top of the fixed anchor (Figure 6). Only in very soft rocks is it reasonable to assume, therefore, that the bond is evenly distributed and that the design may be based accurately and directly on the shear strength of the weaker medium. The probable stress concentration may also be used as a guide to the selection of the position of the apex of the "cone of rock" approach, used in overall stability design (above). This also affects the assessment of anchor

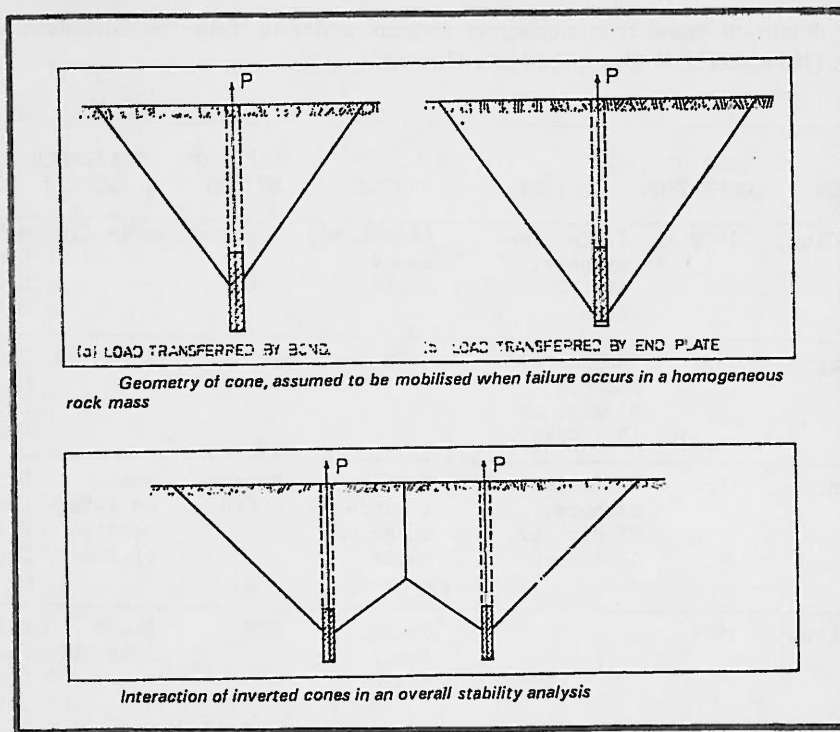


Figure 5. Conventional basis of designing for ultimate resistance to pull out in homogeneous rock masses¹⁶.

stressing information, especially during Performance Testing (see below) where careful analysis will indicate the amount of apparent debonding in the fixed anchor length and so influence judgements on anchor acceptability.

Corrosion Protection

Although this is discussed below it is worth reiterating that this must be regarded as an integral and vital part of anchor design. The degree of corrosion protection to be provided must be based not only on the chemical and dynamic properties of the groundwater, but also on the permeability of the rock mass—before and after any phase of pregrouting and redrilling. It should be noted that rock mass permeability testing, if conducted according to Hously's method, will provide information on fissure geometry as well as water acceptance characteristics.²⁸

Construction

Throughout every phase, it is clear that the more detailed the site investigation data provided, the less uncertainty there is about the suitability of the techniques foreseen. This in turn will lead to optimized technical and program performance and reduced costs.

Construction

Drilling

There is no reasonable argument against the normal use of a down-the-hole hammer to drill unreinforced concrete and competent rock. It is a rotary percussive method routinely employing delivered compressed air pressures of up to 300 psi (actual air pressures acting on

the borehole wall are, of course, much less). It provides holes of exceptional straightness as the percussion is applied at the bottom of the hole, just above the bit: the drill string is thus pulled as opposed to pushed. Typical tests show deviations of less than 1 in 100 to be routinely attainable when proper drilling practices are observed. For the same reason, rates of penetration are more or less constant with depth, as no percussive energy is lost between drill head and drill bit. Penetration rates are high (up to 60 ft./hr. is not exceptional). Presently, mechanical practicalities restrict conventional hole sizes to 4-12" in

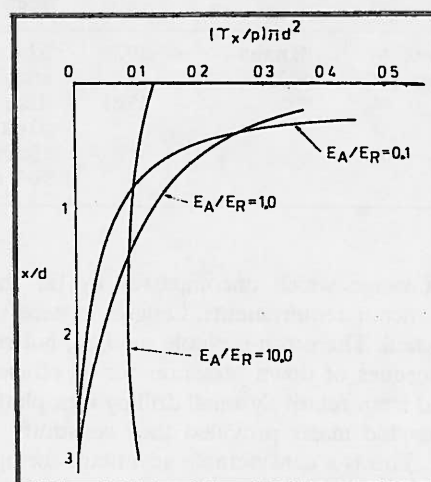


Figure 6. Variation of shear stress with depth along the rock/grout interface of the bond length. E_R and E_A are the moduli of elasticity of the rock and anchor grout respectively²⁶.

Table 2. Summary details of some dam anchoring projects undertaken by the Nicholson Construction Company. (Note: DTH = Down-the-Hole Hammer).

CONTRACT	LOCATION	YEAR OF COMPLETION	CONTENT	BEDROCK	DRILLING METHOD	TOLERANCE (WORST)	OWNER
Cannelton Lock & Dam	Tell City, ID	1972	27 inclined anchors, 9" dia. av. 165' deep	Sandstone, sandy shale	Rotary	1 in 120	U.S. Army Corps, Louisville District
Newburg Dam	Newburg, ID	1973	18 inclined anchors, 9" dia. av. 135' deep	Soft shale	Rotary	1 in 120	U.S. Army Corps, Louisville District
Hildebrand Lock & Dam	Morgantown, WV	1974	70 inclined anchors, 6" dia. av. 110' deep	Siltstone & sandstone w/ shale seams	Rotary & DTH	None: no inter-section of holes	U.S. Army Corps, Pittsburgh District
Lake Lynn Dam	Lake Lynn, PA	1974		Shale, sandy shale	DTH	About 1 in 100	Allegheny Power Services Corp.
Lock & Dam 3	Monongahela River, PA	1977		Med. hard silty shale	DTH	None	U.S. Army Corps, Pittsburgh District
Pickwick Lock & Dam	Savannah, TN	1978 & 1981	129' vertical & inclined anchors, 9" dia. to 145' deep	Shale, limestone	DTH	Holes within 2' of 80' high conc. face	TVA Chattanooga
Edgar M. Hoopes	Wilmington, DE	1979	10 vertical anchors, 9" dia. 140 - 220' deep	Mica schist	DTH	1 in 120	City of Wilmington
Wallenpaupack Dam	Wallenpaupack, PA	1979	12 inclined anchors, 6" dia. 100' deep	Sandstone and quartzite	Rotary (DTH not allowed)	18" radius at 100' i.e. 1 in. 67	PA Power & Light Allentown
Lock & Dam 1	Minneapolis, MN	1979 & 1981	57 inclined anchors, 6" dia., 70' long plus 57 anchors 5" dia. 50' deep	Limestone, shale	DTH in conc. Rotary in rock	1 in 60	U.S. Army Corps, St. Paul District

diameter—a range which encompasses by far the larger portion of anchor requirements. Lengths in excess of 300' can be reached. The down-the-hole method, not relying on very high torques or down pressures for its efficiency, can be operated from relatively small drilling rigs, platforms, or crane suspended masts provided they constitute a stable, rigid frame. This is a considerable advantage for operations on existing dams in that contractors can routinely install anchors in places and with orientations judged technically optimal by the designer. In addition, the fact that the percussion is activated in the hole renders the method relatively quiet, and provided the rock debris blown out of the hole during drilling are properly dealt with, the whole

system is very sympathetic to the environment, especially as there are no drilling fluids to dispose of. There have been no published instances of the method causing damage to the existing concrete structure, even when holes have been drilled as close as 2' to free faces, e.g. at Pickwick Lock and Dam, Tennessee. In short, diamond drilling may be necessary for only the reinforced portion if substantial steel reinforcement of the concrete is foreseen, and rotary (tricone) drilling in the softer sedimentary rocks may be apposite, but overall, down-the-hole drilling is the most logical and economic choice in most instances. Examples of projects conducted by the author's company alone are summarized in Table 2.

CONTINUED

Bagnell Dam	Lake of Ozarks, MO	1982	277 vertical anchors, 9" dia., max. 300' deep	Limestone	DTH	1 in 200, on each	Union Electric Co., St. Louis
Bath Co. P.S.	Warm Springs, VA	1982	24 inclined anchors, 8" dia. max. 75' deep	Vertically bedded shale (Drilling strike)	DTH	1 in 576	Virginia E & P Co. Warm Springs, Virginia
Wilson Shoals,	Muscle AL	1982	8 vertical anchors, 9" dia. max. 188' deep	Limestone	DTH	1 in 120	TVA Chattanooga
Barker Dam	Denver, CO	1984	59 vertical plus 35 anchors, 9" dia. 70 - 225' deep	Grantic Gneiss	DTH	1 in 100	Public Service Co. Colorado
Montgomery Lock & Dam	Beaver, PA	1985 to 1986	379 anchors, 4-1/2" - 6" dia. 110' deep	Shale	DTH in conc. & rock	1 in 120	U.S. Army Corps Huntingdon District
Elkhart Dam	Elkhart, IN	1986	31 inclined anchors av. 210' long (9 - 6")	Overburden & shale	DTH in conc. rotary in rock	1 in 60 overall, 1 in 240 for straightness	Indiana & Michigan Electric Co
Wallen-paupack Dam 2	Wallen-paupack, PA	1986	33 vertical anchors 8" dia. av. depth 100'	Sandstone and Quartzite	Rotary (DTH not allowed)	1 in 120 to 1 in 240 in places	PA Power Light Co. Allentown
Lock & Dam 8	Point Marion, PA	1988	63 inclined anchors av. 80', 6" dia.	Sandy shale	DTH	1 in 100	U.S. Army Corps, Pittsburgh District
Shepaug Dam	Near Danbury, CT	1988	97 inclined anchors from 75-210' long 10" dia.	Schist	DTH	Max. deviation of 1/2°	N.E. Utilities Connecticut
Great Falls Dam	Near McMinnville, TN	1989	114 inclined anchors from 127-146' long, 8 5/8"	Limestone	DTH	Could not be measured	TVA Chattanooga

Regarding borehole deviation, specific project requirements should always be addressed. A good drilling contractor should routinely provide holes with an overall deviation not greater than 1 in 50, although deviations of 1 in 100 or less can be provided, if really required, by the careful use of certain drilling accessories, e.g., drill string centralizers, and using the best drilling practice. To specify tolerances finer than this is practically unreasonable, and in any event may not be possible to verify given the in-hole instrumentation currently available to measure them. As a general guide, PTI recommends that "drill holes normally can be started within an angle tolerance of 1 to 3 degrees from their planned orientation. A deviation of 1-2" in 10' can be maintained with normal drilling methods." Bearing

in mind the length and proximity of most anchors on dams, these recommendations should be regarded as a minimum standard. It is interesting that recent international standards allow deviations of up to 1 in 30 for routine anchor installations. Drill deviations are typically greater in inclined holes than in vertical holes.

As a final word on drilling, it is worth reiterating that it remains good practice to over drill every hole by 2-4', thus providing a sump which will accommodate any drilling and other debris and so permit the subsequent introduction of the tendon to the full, designed depth. This extra length should be measured and paid.

Permeability Testing

Excluding the case of exceptionally porous sedimentary deposits, the permeability of rock masses is a reflection of the geometry and characteristics of the discontinuities, principally their size and frequency. Therefore, it is fundamentally incorrect to express rock mass permeabilities in terms of soil units such as cm/sec. When water-testing rock anchor boreholes, we are more interested in what can escape from the hole, as opposed to what the rock mass can accept (which is the grouter's viewpoint).

A fluid cement grout is particulate: what geologically defines its penetrability (and so potential escape from an anchor borehole) is the number and aperture of the fissures in the surrounding rock mass that the hole intersects. It has intrinsically nothing to do with the diameter of the hole. Therefore, permeability test analyses—which dictate whether pregrouting is necessary, and also impact the choice of corrosion protection—should be conducted with understanding and care.

Experimental studies suggest that a fissure tighter than 160μ will not accept Type I cement grout particles. A fissure of this width will permit a flow of about 0.4 gal/min at an excess head of 1 atmosphere (15 psi). Therefore, if the total water loss from a rock stage is less than this value, the possibility of appreciable anchor grout loss may be discounted, and no phase of pregrouting and redrilling will be required. It is noteworthy that with the smaller cement particles common in finely ground cements the limiting fissure width reduces to 100μ , through which water flow would be 0.15 gal./min./atmosphere.

It is clear that this loss of grout potential should not be related to borehole length: in a worst case, a single fissure over 160μ wide may exist in a 10' borehole or in a 100' borehole. Therefore, it is strongly suggested that the limiting value for determining the need for pregrouting should be specified simply in terms of gal./min./atmosphere without reference to hole diameter or length. This contradicts the current PTI Recommendations¹⁹ but is in line with FIP²² and BSI²⁰, which set a realistic criterion of about 1 gal./min. at an excess pressure of 1 atmosphere. The position of the local water table must of course be measured in order to calculate the excess pump pressure. In addition, FIP confirms that any hole showing artesian water gain should be pregrouted irrespective of the magnitude of inflow.²²

Tendon Handling and Installation

Reviews of good practice¹⁶, and recent codes and recommendations, all confirm that the presence of rust on strands per se is no detriment to bond development with grout, but rather quite the contrary. Only if the rust is loose or flaky should remediation be demanded. All practical steps should, of course, be taken to protect tendons during transportation, storage, and handling.

There is a trend towards longer and heavier anchor tendons as projects become more ambitious, and the 220' long Shepaug Dam tendons weighing up to 5 tons have

already been noted. To avoid damage to the tendons and their corrosion protection—as well as for safety and practicality—it is essential to specify (1) the necessity for some form of mechanical installation device (for example, cranes and helicopters have been used) and (2) proper, strong and frequent spacer/centralizer units at regular intervals along both free and fixed lengths. Bar tendons are typically limited to anchors of short length (less than 50') or low capacity (50 tons), and where access to the drill hole entry point is easy. They, therefore, have restricted application in major dam repairs. As a final word, research³⁰ has shown that within the fixed anchor length, not more than about 15% of the borehole volume should be occupied by steel (as opposed to grout). This limit should ensure that there will be sufficient interstitial space to permit the grout to penetrate uniformly between the strands thus allowing proper load transfer and efficient corrosion protection.

Grouting - Mixes and Mixing

Poor or inefficient grouting procedure represents the most common cause of anchor failure when construction—as opposed to design or corrosion protection—is to blame, and within this field improper mixing and grout mix design are the major culprits.

High speed grout mixers³¹ must always be used to ensure uniform and intimate mixing of the cement particles and the water. This high wetting efficiency permits the preferred lower water cement ratios (say w/c = 0.4-0.45 by weight) to be used, leading directly, therefore, to higher and earlier strengths, and greatly reduced bleed (Figure 7) without the need for additives. Bleed potential is essential to monitor, as bleed water in long ducts such as in the case of anchors may not necessarily migrate to the top of

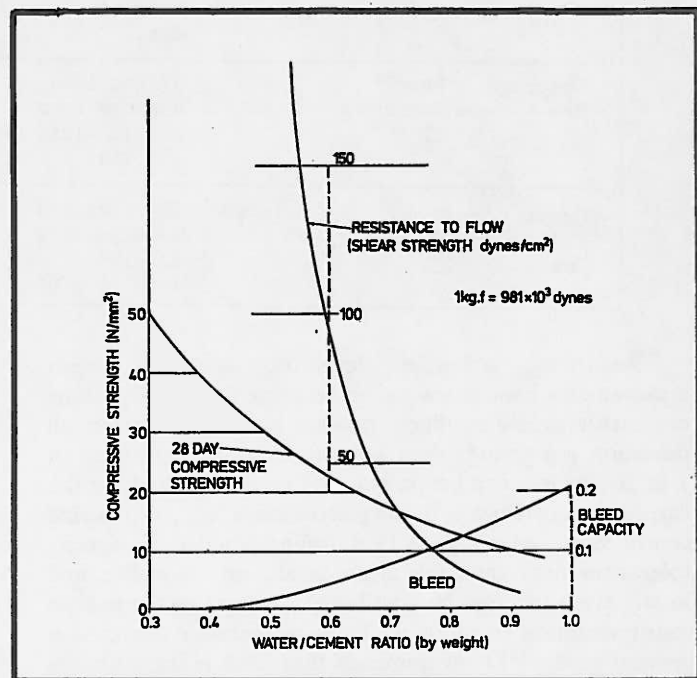


Figure 7. Influence of water cement ratio on grout strength, bleed and fluidity¹⁶.

the grout column. It typically forms long "ribbons" or lenses which may be in contact with the strands. Upon setting of the grout, therefore, the absence of uniform grout/steel contact will reduce the efficiency of bond development and compromise the security of the corrosion protection. BS 8081²⁰ states that for relatively impermeable ground conditions, anchor borehole grout bleed should not exceed 2% after 3 hours, or 3% ultimately (at 68°C). Where a w/c ratio lower than 0.40 is truly required, for example to reach very high early strength, then the use of a plasticising additive will be necessary. Generally this is not common in America.

It was recently confirmed that selected plasticiser additives were necessary to mix Type I grouts of $w \leq 0.4$.³² It was also confirmed that gas producing expanding additives (e.g., aluminum powder) should never be used, for many reasons but especially the tendency for large gas bubbles to agglomerate locally. Equally, gelling or thixotropic additives for primary anchor grout should be strictly avoided due partly to the extreme sensitivity of the grout properties to the amount of agents added, and partly as their presence severely compromises bond development in situ.

It is strongly recommended, therefore, that, provided the grout is to be mixed in a high-speed mixer and the w/c ratio is 0.4-0.45, there is no need for the use of additives under normal conditions. Indeed, it has been demonstrated that certain additives, even when used consistently in the correct dosages, can have a markedly deleterious effect on set grout properties. However, in very hot conditions, and/or for long pumping distances, a preblended cement, incorporating a minor amount of pozzolan, may be used at equivalent water contents to improve pumpability. Such proprietary preblends should offer set grout properties at least as good as comparative pure cement grouts with respect to stability, durability, and ultimate strength. Alternatively, under such extreme conditions, the use of a carefully selected retarding additive may be considered, but with strict batching control.

Generally cement grout type should be chosen to reflect the aggressiveness of the ground. FIP lists²² the following criteria indicative of water aggressive to concrete or hardened cement paste:

- pH values < 6.5 (later²³ revised to 5.5)
- hardness < 3°d
- CO₂ content > 15 ppm
- NH₄₊ content > 15 ppm
- Mg₂₊ content > 100 ppm
- SO₄²⁻ content > 200 ppm

Grouting - Quality Control and Assurance

Conventionally, grout quality control has revolved around using bagged cement and a water measuring tank or meter to guarantee the target water-cement ratio. Quality assurance has been provided retrospectively by cube crushing results, the conventional 28-day target being a minimum of 4000 psi. On larger projects there is a trend towards using bulk handling and mixing facilities. These

offer a number of practical and economic advantages, principally, electronic precision weigh batching, reduction in waste, less labor involvement, and lower material costs. However, it must be borne in mind that such systems are notoriously sensitive to atmospheric moisture and dams are usually built in areas of appreciable rainfall. It is good practice, therefore, to conduct periodic checks on the resultant fluid grout prior to pumping. For example, the Baroid mud balance gives a direct reading of specific gravity (and hence the w/c ratio - Table 3), while a flow cone or Colcrete Flowmeter quickly demonstrates pumpability and fluidity. These very simple instruments can be calibrated to provide absolute readings of w/c ratio, or can just as easily be used to confirm repeatability and consistency of batching.

Grouting Operations

Most rock anchors do not require pressure grouting to develop adequate rock-grout bond resistances, and so grout pumps need not have very high pressure capacities. Nevertheless, pumping distances in dam anchor work are frequently long, and holes may be deep. High line pressures may, therefore, be developed, especially during the final stages of tremie grouting if the injection line has not been extracted during grouting. Pumps—either progressive cavity or piston—must therefore provide a certain guaranteed minimum rating—and 150 psi may be regarded as that minimum.

Regarding grouting in cold weather, BS 8081²⁰ states that special precautions be taken to prevent the temperature of grout falling to 32°F during the early stages of hardening. Grouting in air temperatures below 36°F should only be undertaken if the following precautions are taken:

- a) grout temperature during injection is at least 41°F,
- b) mix constituents are free from snow, ice or frost,
- c) tendon and any surface which will be in contact with the fresh grout must be free of snow, ice, or frost, and preferably should be at a temperature which will not chill the grout.

Pregrouting (or consolidation grouting) of the anchor borehole is conducted when the hole fails the water test, or when collapsing conditions prevent the hole being drilled stably to full depth. Typically, any major instabilities and water losses (or gains, if the hole connects with fissures passing upstream) occur at and just below the base of the

Table 3. Calculated Specific Gravity vs. Water-Cement Ratio¹⁶.

Specific gravity	Water/cement ratio
2.10	0.3
1.95	0.4
1.84	0.5
1.74	0.6
1.67	0.7
1.61	0.8
1.56	0.9

dam. Under these conditions, it is simplest and safest to carry the hole down to the top of the intended bond zone, and pregrout this free length via a packer placed at the top of the hole in the dam. Afterwards the hole can be continued to full depth and the critical bond length tested via a down-the-hole packer, with the security that the water is not simply escaping above the packer into very open zones, and that the bond length itself is being properly investigated. This procedure is operationally, technically, and economically preferable to trying to insert a steel liner through the suspect zone, to the top of the bond length to prevent collapse. If the bond length fails the subsequent water test then it can be regrouted, as before, and redrilled.

Corrosion protection of the fixed anchor length by applying an outer corrugated plastic sheath is becoming increasingly more common. Except in the case of very short monobar tendons, it is impractical to specify pregrouting of the tendon inside the corrugation, and letting it set, prior to later installation. Such a system vastly complicates handling and insertion—especially in dam environments—and in any case there is a high probability that the grouted encapsulation will be internally cracked and damaged during installation. It is much more practical, and safer, to grout the inside and the outside of the protected tendon after installation, even if this may dictate the use of two grouting tubes.

Assuming the free length is greased and sheathed, anchor hole grouting is most easily accomplished in one operation. Of course, where the newer "Stewart Mountain" type approach is to be used, then a Primary/Secondary sequence is necessary—the latter being done only after stressing has been completed. In general, however, two-stage grouting (1) does create a "construction joint" (above the Primary grout) which may be a weak point in a corrosion protection system, and (2) does introduce another separate operation into the overall construction program.

As a general rule, tendon installation and grouting should be conducted as soon as is practical after final preparation of the borehole. The actual amount of time a borehole may remain open prior to tendon installation will vary from site to site and will be dictated by the composition and structure of the bedrock. Under U.S. conditions, there is typically no real prospect of borehole wall deterioration within the normal contract timeframe.

Equally, any restrictions on work sequencing should be site-specific. For example, in especially laminated rock, it may be necessary to stipulate that no drilling will be conducted, say, within 30' of any anchor grouted for less than 24 hours. In very massive, tight rock, it may be that no interconnections between even adjacent hole positions occur, and so the spacial separation can be relaxed.

Stressing and Testing

The PTI Recommendations¹⁹ form a good basis for conducting and analyzing stressing programs. Guidance is provided on both short term testing and long term

performance. The following points are made, with special reference to dam anchoring.

Setting AL

Tendons are usually long and may comprise many tens of strands. Stressing is therefore conducted with a multijack which loads the strands simultaneously. However, individual strand breakages, and anomalous extensions have frequently been recorded at stress levels from 70 to 80% of the overall tendon guaranteed ultimate tensile stress. In the great majority of cases, this has resulted from unequal setting of AL, the Alignment Load, typically 2-10% (average 5%) of the design load. This can be done accurately and uniformly if the time and care are taken to apply AL to individual strands with a monojack, prior to routine stressing with the multijack. As an example, for tendons over 200' long and comprising 54 strands 0.6" in diameter installed at Shepaug Dam, CT., a monojack was used to apply AL, and the apparent extensions of individual strands recorded to reach this load varied by about 6". This merely reflects the fact that different strands lie in different paths in the hole and need, therefore, to be straightened out by different amounts. Multijack stressing was conducted thereafter to 80% theoretical tendon GUTS with no records of individual strand rupture.

Phenomenon of Wedge Pull-In

During stressing, the gripping wedges, holding each individual strand in the head plate, are progressively "sucked in" as the load is increased. For smooth profile 0.6" diameter (Dyform) strand (GUTS 33 tons), the measured amount of pull-in is shown in Figure 8. The magnitudes shown are in close comparison with data recorded for normal 0.6" strands at other sites. As a preliminary step, therefore, in analyzing stressing data—especially for the important Performance Tests—this amount of relative movement must be subtracted from gross anchor head (or jack ram) extension. In addition, any change of elevation of the stressing head during stressing—i.e., through bedding in or by structural movement—must also be measured and subtracted from gross ram extensions prior to detailed analysis. Care should be taken during the initial seating of the gripping wedges. A "power seater" provides consistently uniform setting of individual wedges and their component parts, and allows this to be done without danger to hands and fingers.

Performance Tests

Such tests (also known as "on-site suitability tests" by FIP²² are conducted on the first group of anchors installed. As defined by PTI¹⁹ they determine whether the anchor has sufficient load carrying capacity, that the free length has actually been established, and indicate the magnitude of the residual (or permanent) anchor movement.

Typically, testing is conducted to 1.33 or, better, 1.5 times Design Working Load, while ensuring the tendon stress at Test Load does not closely approach 80% GUTS. The cyclic nature of the stressing allows the elastic and permanent components of total extension to be separated

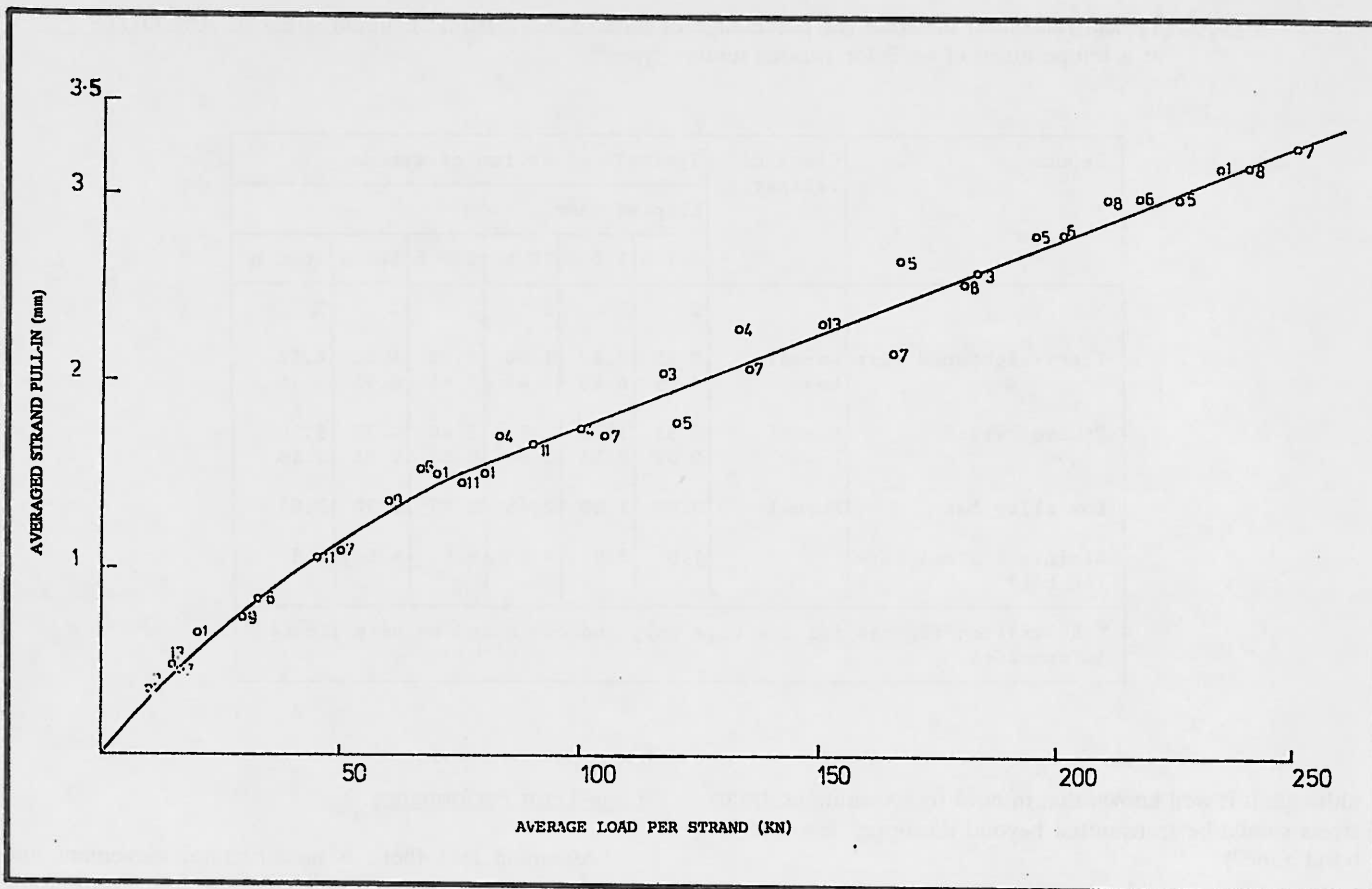


Figure 8. Amount of strand wedge pull-in measured on individual strands during testing of 23 full scale 10 strand anchors. Numbers refer to number of readings at each point ($8.9\text{KN}=1 \text{ ton}$)³⁰.

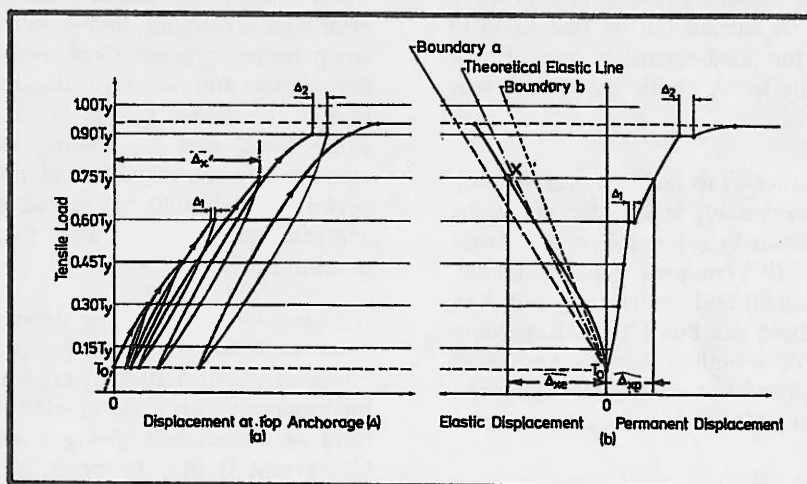


Figure 9. Resolution of total tendon extension (a) into Elastic and Permanent (Residual) Components, (b) for analysis of Performance Tests³³.

out for each successive cycle maximum (Figure 9). Examination of the former permits conclusions to be drawn about the efficiency of load transfer down the free length, and the apparent extent of effective debonding in the fixed anchor. Reflecting the principles of world practice, PTI recommends acceptance if the elastic movement at Test Load exceeds that movement corresponding to 80% of the designed free length (say, Line A) but is less than that movement corresponding to 100% design free length plus

50% of designed fixed length (say, Line B). These criteria may be regarded as being generous in dam anchoring, especially if the data corrections listed above are applied in advance. For example, an anchor performing near the 80% line implies that load is being dissipated above the fixed anchor length. On long tendons, this could possibly be within the dam, affecting therefore overall stability concepts. Equally, a tendon apparently debonding 24.9' in a 50' fixed anchor zone would theoretically be acceptable,

Table 4. Typical relaxation of stress (as percentage of initial stress), from an initial stress of 70% GUTS at a temperature of 68°F for various tendon types²⁰.

Tendon	Class of relaxation	Typical relaxation of stress					
		Elapsed time					
		0.1 h	1.0 h	10 h	100 h	500 h	1000 h
Prestraightened wire	Normal	0.25	0.80	1.90	2.90	4.00	4.50
	Low	0.09	0.20	0.40	0.65	0.95	1.10
7-wire strand	Normal	0.35	1.15	2.10	3.40	4.70	5.50
	Low	0.07	0.33	0.60	0.84	0.98	1.10
Low alloy bar	Normal	0.90	1.80	2.45	2.75	2.90	3.05
Stainless steel wire and bar*		1.0	3.0	4.5	5.1	5.9	6.3

* Relaxation figures for are wire only and are based on very limited information.

although it is well known that in hard rock conditions, little stress should be transmitted beyond the upper few feet of bond zone.¹⁶

BS 8081²⁰ sets the boundary lines as being 90% of a free length for Line A, and either free plus 50% fixed or 110% free, for Line B. Where the inferred free tendon length falls outside these boundary Lines A and B, a further two cycles should be carried out to Test Load to gauge reproducibility of the load-extension data. If the anchor behaves consistently in an elastic manner, it may be accepted.

PTI¹⁹ provides no guidance as to limits of acceptability for residual movements, presumably leaving the choice to engineering judgement, backed by reference to the elastic and creep performances. FIP²² confirms that the matter should be "agreed by consultant and contractor jointly," as a basis for analyzing the subsequent Proof Tests. Regarding creep performance, PTI¹⁹ does outline the procedure and the analysis but notes that "in all but the most decomposed rock formations (it) seldom yields any useful information."

Proof Tests

This is a fast economical test, (known by FIP as "Routine Acceptance Test") which, used in conjunction with Performance Test data, verifies the acceptability of the installation. It does not feature cyclic loading, and so examination of performance in terms of elastic and residual contributions is not directly possible. However, if the Performance Tests can provide a reliable and consistent data base of the scale and range of residual movements to be expected at different tendon stress levels, then, with care and understanding, a more detailed analysis of the Proof Test data can be undertaken for investigatory reasons, in appropriate conditions.

Long-Term Performance

Assuming that there is no structural movement and that the corrosion protection is adequate to prevent attack to the tendon, the significant sources of long term load loss after lock off for anchors in existing dams* are creep in the rock or grout or relaxation of the strand. The former is not considered a major potential problem in anchors in most rock conditions, and should be quantifiable from the creep testing process. Conversely, all steel tendons relax, the amount and rate reflecting many factors including the type of the strand, (Table 4) the temperature, its lock off stress level, and its loading history¹⁶. However, these amounts are quantifiable and predictable, and long term performance should be judged extraordinary only if the characteristics conflict with these tendon performance parameters.

Long-term monitoring demands access to the anchor head and this may not be practical in certain cases. However about 5-10% of anchors are typically equipped for monitoring on a world wide basis. This may be in the form of a load cell, giving a local or remote read out. Conversely, it may be practical to perform a "Lift off" check, wherein the load needed to pick up the head plate by a certain distance—usually the thickness of a dollar bill—is recorded and equated to residual tendon load. When conducting a lift off check, it is preferable to lift the head plate directly, and not the strands: seperately lifting the strands out of their wedges may lead to later damage to that top anchorage system and subsequent loss of load as a result of strand/wedge slippage.

*Eberhardt and Veltrop³⁴ concluded that for new dams concrete shrinkage and creep were significant sources of possible load loss, contributing up to a 6% reduction in lock-off load.

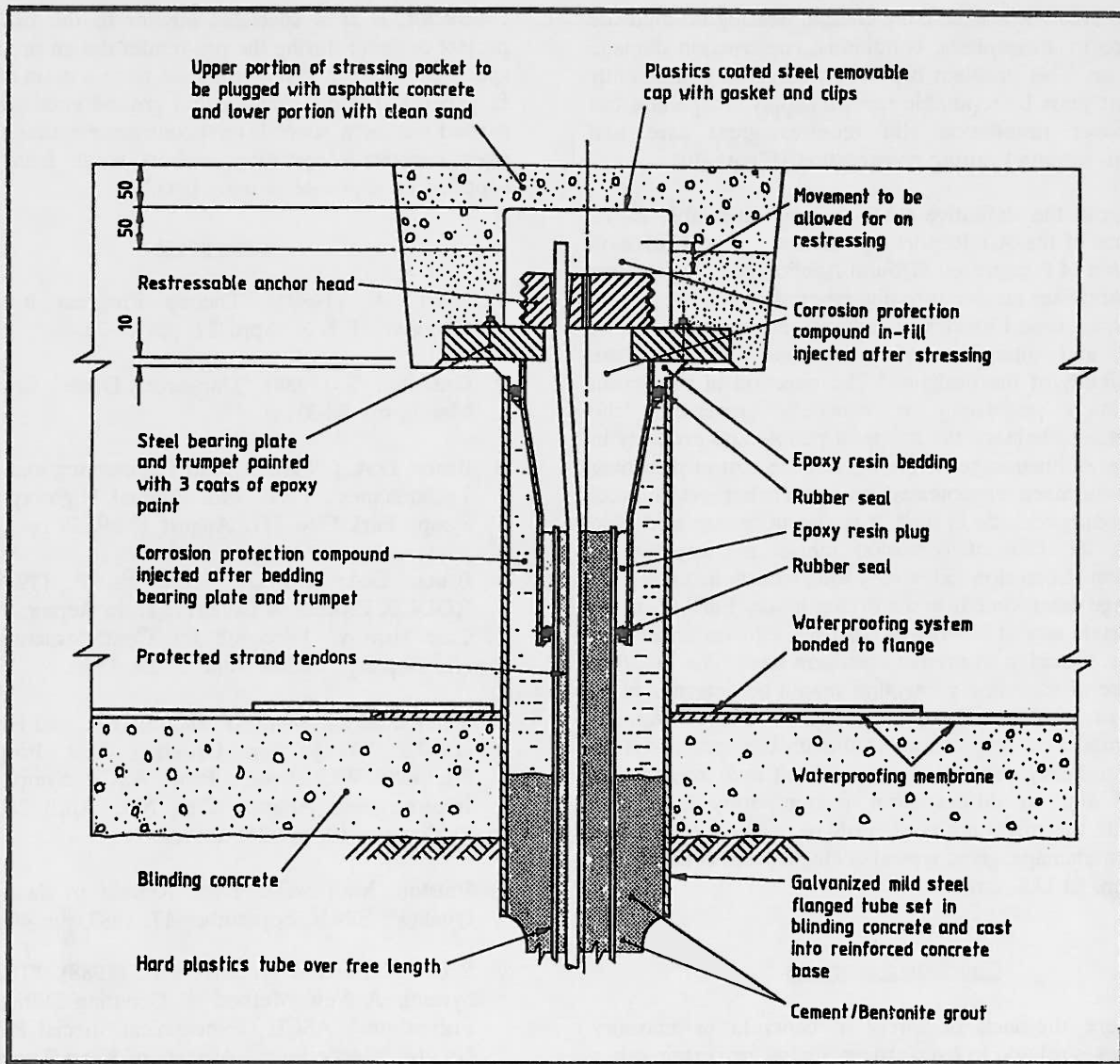


Figure 10. Typical anchor head detail for double corrosion protection of a restressable strand tendon in a water retaining structure²⁰.

Corrosion Protection

Anchors for dams are nearly always permanent installations. All international standards argue that permanent anchors must be properly protected against corrosion. As FIP²³ states, "the protection system against corrosion should be designed in such a way that the presence of aggressiveness is always assumed." Whereas uncracked cement grout generally provides an effective protection, cracks are almost inevitable, especially in the fixed anchor length. For example, the exhumation of a test anchor at the Tarbela project showed radial and longitudinal fissures up to 0.1" wide as far as halfway down the fixed anchor length. In addition, there was crushed grout at the top of that length.

However, it is relevant to consider the ground anchor market in those European countries where the standards were born. Most anchors are installed in older industrial, urban or seafront applications where at least the free length is usually in materials other than rock. The

environment is usually aggressive, whether from natural or artificial causes, and there are great pressures on speed of execution. It is therefore wholly understandable that an extremely high level of protection is demanded under such conditions. In contrast, anchors for U.S. dams are commonly installed into the very hard, impermeable bedrocks so assiduously sought by our original dam builders. Furthermore, dam anchors are typically and deliberately conservative in design, and are installed with an unusually high level of expertise and supervision. As a consequence, the traditional reliance on "grout only" protection for the tendon in the bond length should perhaps not automatically be dismissed as being old fashioned, or even necessarily dangerous.

Regarding the free length, plastic sheathing (polyethylene or polypropylene) has proven effectiveness and suppliers can tailor properties to ambient conditions and temperatures. Special attention must be paid to the "transition points"—especially between stressing head and free length. The proper protection of the stressing head

and just below it is absolutely critical, bearing in mind the exposure to atmospheric conditions, construction damage and so on. This problem has been addressed competently in recent years by reputable tendon supply companies, but the proper installation still requires great care and attention to detail during construction (Figure 10).

Overall the definitive work currently available is the FIP State of the Art Report on "Corrosion and Corrosion Protection of Prestressed Ground Anchorages".²³ It reviews and summarizes mechanisms and types of corrosion, ground aggressivity, case histories of failures and various types of systems and intensities of protection—which are "the responsibility of the designer." The question of protection specification philosophy is concisely expressed: "the problem is to balance the safety of people and property in the event of anchorage failure against the cost of providing protection. Since unprotected ground anchorages of steel will probably corrode in time, it is also necessary to decide whether the rate of corrosion merits the expense of protection. Corrosion rates vary enormously according to anchorage environment and working mode. Further, there is no certain way of identifying corrosive circumstances with sufficient precision to predict corrosion rates. The question of degree of corrosion protection should be determined by the Owner and the Designer jointly, as this will have a major impact on project cost. Although U.S. practice does not favor heavy protection of the fixed and free tendon lengths, and so differs from contemporary European standards, this might not necessarily be a source of possible long-term damage, given typical geological and construction conditions in U.S. dams.

Concluding Remarks

Where the lack of safety in concrete or masonry structures is related to overturning, sliding, or certain other forms of structural distress, then anchoring is a tried, proven and reliable technique. Anchors are also used in new dam construction, but clearly this market is more limited today. Rock anchor practice in the U.S. in general compares favorably with practice anywhere in the world, and indeed has some unique aspects stemming from the scale and complexity of the individual projects. Only in the controversial subject of tendon corrosion protection may it be inferred that a reappraisal of our approach may be constructive. However, even in that case, it can be argued strongly that our approach is acceptable, given the local conditions.

Anchoring is an extremely valuable technique to the dam engineering community and the potential for its continuing development is tremendous. It is also, however, a complex technique, and the advice given by Rutledge³⁶ of FIP should be carefully heeded: "The work of designing, fabricating, installing, grouting, stressing and monitoring ground anchors is of a highly specialist nature in which standards and methods are improving world wide at a rapid rate. Technical specifications and directions cannot replace professional experience and conscientiousness of a contractor's staff at all levels. A valuable role of a specialist subcontractor, as compared with a main

contractor, is as a specialist adviser to the main overall project designer during the pre-tender design process. Such specialist advice is rarely available from a main contractor. In general, it is my opinion that ground anchoring is best carried out by a specialist subcontractor rather than by a main contractor installing anchors made from material supplied by a post-tensioning firm."

References

1. Soast, A. (1989). "There's Progress but Threat Remains". E.N.R. April 27, pp. 24-35.
2. Greenhut, S. (1988). "Dangerous Dams". Constructor, March, pp. 28-31.
3. Bruce, D.A. (1988). "Urban Engineering and the New Technologies". Proc. 39th Annual Highway Geology Symp., Park City, UT, August 17-19, 29 pp.
4. Bruce, D.A. and De Porcellinis, P. (1989). "The RODUR Process of Concrete Dam Repair: A Recent Case History". Proc. 4th Int. Conf. Structural Faults and Repair, London, June 27-29, 15pp.
5. Salley, J.R., Foreman, B., Baker, W.H. and Henry, J.F. (1987). "Compaction Grouting Test Program at Pinopolis West Dam". Proc. ASCE Symp. on Soil Improvement, Atlantic City, N.J., April 28, Special Publication 12, pp. 245-269.
6. Ralston, M. (1987). "Dam Rebuilt to Save it from Quakes". E.N.R. September 17, 1987, pp. 46-47.
7. Bruce, D.A. and Gallavresi F. (1988). "The MPSP System: A New Method of Grouting Difficult Rock Formations". ASCE Geotechnical Special Publication No. 14, "Geotechnical Aspects of Karst Terrains," pp. 97-114. Presented at ASCE National Convention, Nashville, Tennessee, May 10-11.
8. Littlejohn, G.S. (1982). "The Practical Applications of Ground Anchorages". Proc. 9th FIP Congress, Stockholm, Sweden, June 6-10, 6 pp.
9. Mohamed, K., Montel, B., Civard, A., and Luga, R. (1969). "Cheurfas Dam Anchorages: 30 years of control and recent reinforcement". Proc. 7 Intl. Conf. Soil Mech. Found. Eng., Mexico City, Specialty Session 15, pp. 167-171.
10. Gosschalk, E.M. and Taylor, R.W. (1970). "Strengthening of Muda Dam foundations using cable anchors". Proc. 2 Cong. Intl. Soc. Rock Mech., Belgrade, 3, pp. 205-210.
11. Water Power (1978). "Use of rock anchors at Tarbela." February, pp. 44-47.
12. Nuss, L.K. (1988). "Strengthening of a Thin Arch Dam with Post Tensioned Anchors: Stewart Mountain Dam, Arizona". Proc. 8th Annual USCOLD Lecture Series. Paper 8, 28 pp.

13. Thurnherr, F. (1982). "Permanent Ground Anchors - Recent Developments". Proc. Int. Symp. Soil and Rock Improvement Techniques, AIT, Bangkok, Thailand, November - December, 17 pp.
14. Standig, K.F. (1984). "Rehabilitation of the Delta Dam". Water Power and Dam Construction, December, pp.21-24.
15. Henn, R.W. and Yow, M. G. (1985). "Field installation: large post-tensioned anchors". Hydro Review, Winter, pp. 81-85.
16. Littlejohn, G.S. and Bruce, D.A. (1977). "Rock Anchors: State-of-the-Art". Foundation Publications Ltd., Brentwood, Essex, England. 50 pp.
17. Hanna, T.H. (1982). "Foundations in Tension- Ground Anchors". Trans. Tech. Publications. McGraw Hill Book Co., New York, 573 pp.
18. Hobst, L. and Zajic, J. (1983). "Anchoring in Rock and Soil". Elsevier Scientific Publishing Co. Amsterdam, Holland.
19. PTI (1986). "Recommendations for Prestressed Rock and Soil Anchors". Post Tensioning Manual, Fourth Edition, pp. 236-276. Published by PTI, 301 W. Osborn, Suite 3500, Phoenix, AZ 85013.
20. British Standards Institution (1988). "Draft BS 8081: Ground Anchorages". Prepared for Issue in 1989 by Technical Committee CSB/22. BSI, London.
21. British Standards Institution (1982). "Recommendations for Ground Anchorages". Draft for Development DD81, BSI, London, 123 pp.
22. FIP (1982). "Recommendations for the Design and Construction of Prestressed Concrete Ground Anchors". FIP, Wexham Springs, Slough, England, 31 pp.
23. FIP (1986). "Corrosion and Corrosion Protection of Prestressed Ground Anchorages". Thomas Telford, Ltd., London, 28 pp.
24. Littlejohn, G.S., Bruce, D.A. and Deppner, W. (1977). "Anchor field tests in Carboniferous Strata". Revue Francaise de Geotechnique, 3 pp. 82-86.
25. Littlejohn, G.S. (1972). "Some empirical design methods employed in Britain". In: Questionnaire on Rock Anchor Design, Geotechnics Research Group, Dept. of Engineering, Aberdeen University, Scotland (Unpublished).
26. Coates, D. F. and Yu, Y.S. (1970). "Three dimensional stress distributions around a cylindrical hole and anchor". Proc. 2nd Int. Conf. on Rock Mechanics, Belgrade, pp. 175-182.
27. Berardi, G. (1967). "Sul Comportamento Degli Ancoraggi Immersi in Terreni Diversi". Univ. of Genoa, Inst. Constr. Science, Series 111, No. 60, 18 pp.
28. Housby, A.C. (1976). "Routine Interpretation of the Lugeon Water Test". Quart. Jour. Eng. Geol., 9(3), pp. 303-313.
29. Deere, D.U. (1980). "Some Recent Experiences in curtain grouting for high dams". Joint Meeting ASCE and AEG, Denver Section, Sempthember 8, 29 pp.
30. Bruce, D.A. (1976). "The Design and Performance of Prestressed Rock Anchors with Particular Reference to Load Transfer Mechanics". PhD Thesis, Dept. of Engineering, Aberdeen Univ., Scotland. 2 Volumes. 670 pp.
31. Gourlay, A.W. and Carson, C.S. (1982). "Grouting Plant and Equipment". Proc. Conf. Grouting in Geotechnical Engineering, New Orleans, February 10-12, pp. 121-135.
32. Housby A.C. (1988). "Improvements in Grouting of Large Ground Anchors". ASCE Jour. Geot. Eng. Divn., 114, (4), April, pp. 448-468.
33. DIN (1972). "Verpressanker fur vorubergehende Zwecke in Lockergestein, Bemessung, Ausfuhrung und Prufung". Vol I, June.
34. Eberhardt, A. and Veltrop, J.A. (1965). "Prestressed anchorages for large trainter gate". ASCE Jour. Struct. Div. 90 (6), June, pp. 123-148.
35. Graber, F. (1981). "Excavation of a VSL rock anchor at Tarbela". VSL Silver Jubilee Symposium, Losinger Ltd., Berne, Switzerland.
36. Rutledge, J.C., Chief Geotechnical Control Office, Hong Kong, and Member of Ground Anchor Working Committee, FIP (1982).

ROCKFILL DAMS ON ROCK FOUNDATION - CASE HISTORIES

David C. Cowherd, M.S., P.E.
President and Chief Geotechnical Engineer, Bowser-Morner Associates, Inc.
Dayton, Ohio 45401-0051

And

Vlad G. Perlea, Ph.D., P.E.
Senior Engineer, Bowser-Morner Associates, Inc.
Dayton, Ohio 45401-0051

ABSTRACT. Case histories of three large earth-rockfill dams bearing on rock are presented. Records of settlement and other pertinent features of these large embankments are presented. The embankments range in height from 240 to 330 feet and were constructed over old strip mined areas with spoil put back on the bench, new strip mined areas, or over deep mined areas. The embankment constructed over strip mined areas contain upwards of 100 feet of fill over a highwall area where the fill in the embankment is some 60 to 80 feet thicker on the strip bench than above the highwall. Settlement monitoring devices were placed to measure the settlement on either side of the highwall over a period of time. One of the embankments was also placed over deep mines which were relatively shallow and pneumatic settlement devices were placed to monitor the settlement over the deep mined area. Records were kept during construction as well as after construction.

Estimations of the expected settlement are presented and comparisons are made between the expected and actual settlements recorded. The embankments were basically rockfill dams with a thin upstream cohesive soil membrane. Records were maintained of whether or not the cohesive soil membrane cracked due to any differential settlement over the highwalls. The techniques of monitoring settlement as well as the values of settlement are described. The performance of high rockfill dams with thin cohesive soil facings with abrupt changes in fill height over rock foundations is detailed. Seepage through fractured rock abutments was also measured and is detailed.

Introduction

Coal companies must dispose of millions of tons of coal refuse and excess spoil from strip mining, and provide sediment control for thousands of areas of strip mine land each year. In addition, large quantities of make-up water for coal washing plants is needed. A design of multi-purpose dams to provide for all of these needs has been devised.

Case histories of three (3) of these large earth-rockfill dams are presented. The embankments described range in height from 240 to 330 feet and contain from 2.5 to 9.5 million cubic yards of fill. The basic design of all the embankments is essentially the same consisting of a large rockfill zone of the embankment with an appropriate filter and a relatively thin upstream cohesive facing. The embankments were generally constructed with the abutments bearing on rock and the center portion of the dam bearing on either original soil or fill which had been previously placed into the valley. All dams were instrumented with settlement monitoring devices and pneumatic piezometers for monitoring water levels. The purpose of the dams was to provide a multi-purpose structure to provide for sediment control during mining, long-term refuse disposal for both coarse and fine refuse, disposal of excess spoil from a strip mining operation, and a source of plant make-up water. All structures are detailed as to post-construction settlement and performance.

Design Concept

The embankments were designed to be primarily constructed of rock to utilize strip spoil as it came from the strip operation without further processing. The top size of the rock was set at 4 feet. In each case the filter was designed to be coarse refuse from the plant site, and on-site cohesive soils were used. Figure 1 shows the embankment concept. The design concept calls for maximum utilization of on-site available materials without the need for processing. Figure 2 shows the gradation of the rock and the filter zones used within the embankment. The rock material to be placed in the embankments was specified to have a slake durability index of 90%. The filter material consisted of coarse refuse and the cohesive material consisted of on-site cohesive material.

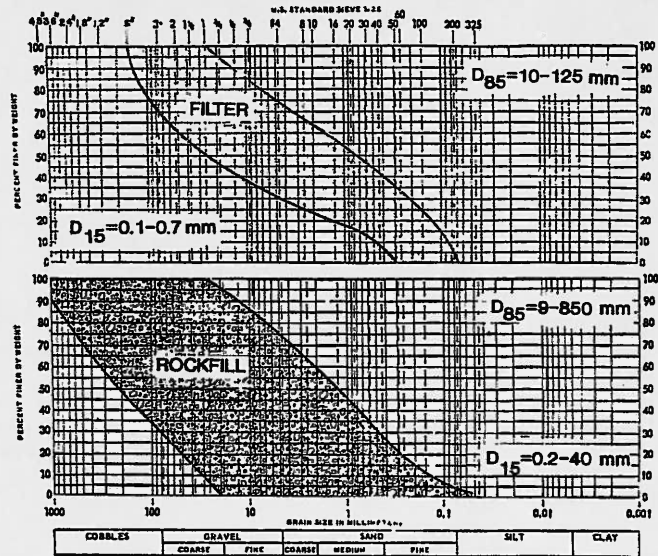


Figure 2. Gradations of filter and rockfill used in embankments.

A sound sandstone rock drain was placed beneath the embankment in order to facilitate drainage, and free-draining rock was wrapped up the sides of the abutments to drain any seepage from the abutments into the underdrain (Figure 3).

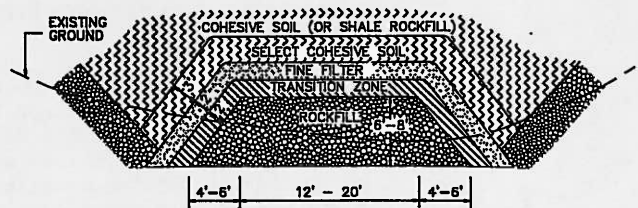


Figure 3. Internal drainage.

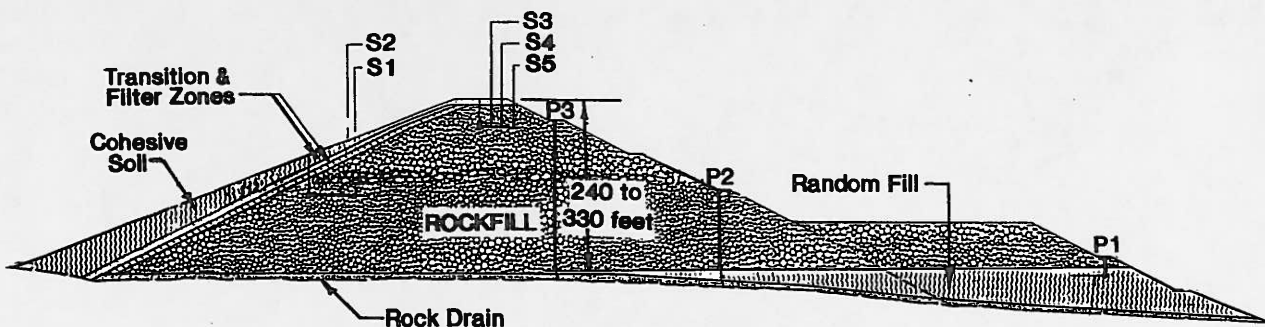


Figure 1. Typical embankment cross-section.

Gradation tests were performed on the filters and on the cohesive zone during construction. In the one instance where there was a coarse filter the coarse filter gradation was checked by excavating a hole in the filter material and running the gradation using large sieves and a calibrated barrel for weighing the material. Figure 4 shows the apparatus used for performing the gradation tests.



Figure 4. Field gradation test apparatus.

The instrumentation installed in the embankments generally consisted of three (3) pneumatic piezometers and four (4) pneumatic settlement monitoring devices. The piezometers were Sinco Model 514178. The settlement monitoring consisted of liquid settlement gauges, Sinco Standard Model 514163, and, in the more recent work, Sinco Amplified Model 514083.

Seepage Through Fractured Abutments

One consideration in the design of these embankments is the potential for seepage through rock abutments. Basically in most of these embankments coal is either stripped within the embankment level or there was an old strip embankment, or the abutments themselves are relatively fractured. The potential obviously exists for seepage around the cohesive zone. In order to handle this problem an underdrain extending all the way from the filter zone to the toe of the embankment was constructed. The underdrain essentially consists of rock up to 4 feet in diameter, sound sandstone excavated from the strip spoil operation. Figure 3 shows a typical underdrain. Coarse sound rock was wrapped up the embankments to allow any water draining through the abutments to drain down into the underdrain. The underdrains have functioned very well. Measurements were made on one of the underdrains as will be described in the case histories below to determine the amount of seepage to date. There has been no phreatic surface build-up within the embankments and the underdrains have functioned well in terms of picking up seepage from abutments and any seepage that may come through the embankment itself.

Case History No. 1

This embankment consists of an earth-rockfill embankment some 260 feet high containing some 4.4 million cubic yards of material. Figure 5 shows the design of this embankment. The embankment was constructed over 35 feet of soil in the base of the valley. A portion of the embankment was stripped to mine coal. The remainder of the abutments were stripped to rock. Figure 6 shows the approximate configuration of the cross-valley profile and the various materials upon which the embankment is constructed.

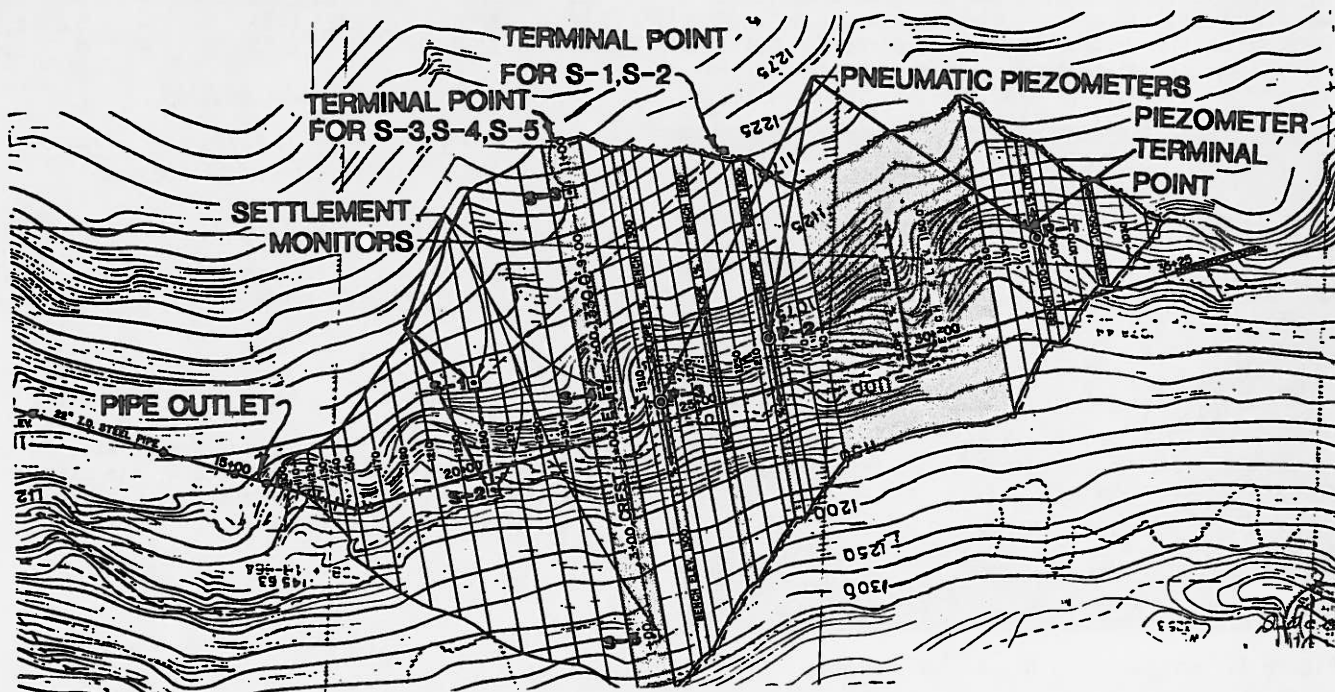


Figure 5. Case history No. 1 - Design concept.

was completed in March, 1988. The monitors have been read on a weekly basis since installation.

Refuse is currently being disposed upstream of the embankment (Figure 7). This obviously adds an additional load on the embankment and could contribute to post-construction settlement. The coarse refuse is deposited adjacent to the embankment while the fine refuse is pumped in upstream of the embankment.

The piezometers have shown no water level. Their tips are a few feet above the bottom of the rockfill. The piezometers are at the elevation shown in Table 1.

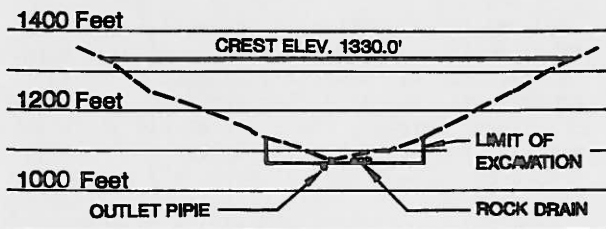


Figure 6. Cross valley section through crest.

Instrumentation

The location of the settlement monitoring and piezometers are shown on Figure 5. The purpose of the spacing of the settlement monitoring was to monitor the settlement over the abutment or a very little portion of the fill and to monitor the settlement over a large amount of fill within the valley proper. Limits of settlement at which point settlement may become a problem were set between the monitors. This was done on the following basis. A literature search showed five of 18 membrane-faced dams constructed of dumped rockfill, all of which showed a good behavior, had crest settlement in excess of 1.25% per unit height¹. The settlement corresponds approximately to a 1% horizontal deflection, defined by the ratio of the differential settlement divided by the distance between the corresponding two points. It was decided that the cohesive upstream zone could safely tolerate a horizontal deflection of approximately 1.0%. Based on this the difference between settlement readings in adjacent monitors approximately 200 feet apart was set at 2.0 feet. Conservatively, it was specified that the safe deflection will be 0.5% horizontal; i.e., half the above-referenced differential settlement. The settlement monitors were installed relatively high within the embankment to monitor basically post-construction settlement. The site development for the embankment construction was begun in May, 1987 and the embankment



Figure 7. Case history No. 1 - Aerial View.

Table 1. Piezometer Elevations

Piezometer	Tip Elevation (ft)
P-1	1073.9
P-2	1083.9

The water level within the reservoir currently is at elevation 1200 feet. The readings within the piezometers have consistently shown there is no water at the level of the piezometers. Any water is exiting from the drain and is not building up within the embankment.

The settlement monitors have been monitored weekly. Tables 2 and 3 give a summary of the settlement values to date.

Table 2. Summary of Settlement Movements

Gauge	Date of Installation	Height of Fill (ft) Below & (Above) the Gauge (Final)	Settlement (ft)		
			12/87	3/88	3/89
S1	12/87	126 (87)	0	1.73	2.80
S2	12/87	128 (85)	0	1.61	2.00*
S3	3/88	28 (37)	-	0	1.72
S4	3/88	222 (41)	-	0	2.35
S5	3/88	48 (42)	-	0	1.50

*Settlement on 1/89

Table 3. Horizontal Deflection After Completion of Settlement

Monitors	Distance Between Monitors (ft)	Differential Settlement (ft)	% Horizontal Deflection
S1 - S2	220	0.80	0.36
S3 - S4	400	0.63	0.16
S4 - S5	500	0.85	0.17

Although the total settlement was moderate, up to 1.1% of the total height of the dam, the differential settlement and the horizontal deflection remained within safe limits.

Conclusions

The embankment is functioning as designed and there is no build-up of phreatic surface within the embankment. The drain is handling any seepage water, both from abutments and from the surrounding material and from water within the reservoir. The settlement monitors indicate that settlement has been within the values predicted and well within tolerable values. As an example, Figure 8 shows the settlement time history and the increase in overburden loading over monitor S1 during construction.

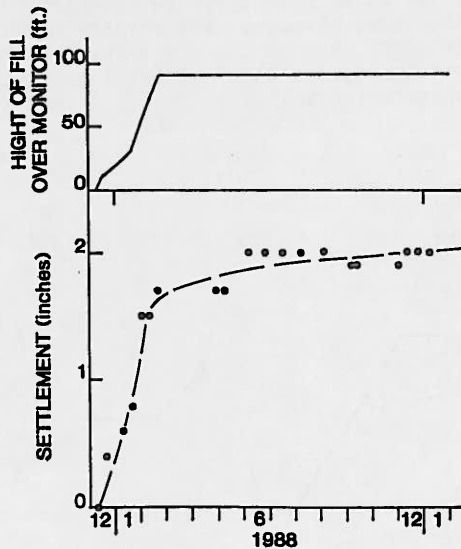


Figure 8. Time history of monitor S-1 settlement.

Case History No. 2

Case History No. 2 consists of a 330-foot high embankment with 9.5 million cubic yards of material built in the configuration shown in Figure 9. Basically the structure consists of an earth-rockfill embankment as previously described with a large excess spoil disposal area which acts as a storage yard for the plant site at the project. The construction of this embankment is essentially as described for all embankments. In this particular embankment there is a chimney drain consisting of sound sandstone, a filter zone which was constructed of both reddog which was present on the site and coarse refuse, and an upstream cohesive zone. The underdrain system is essentially as described for all embankments with the above-noted modification. One unique feature of this embankment was the construction of an underdrain which led to a collector pipe. The underdrain does not extend all the way out through the valley fill, but is collected in a pipe which then exits below the toe of the fill (Figure 9). This system allows collection of all seepage at one collection point and makes it very easy to monitor the seepage. In order to monitor the seepage a weir was constructed and installed on the downstream end of the exit pipe (Figure 10).

Instrumentation

The embankment is instrumented with four (4) piezometers installed essentially just above the underdrain. These piezometers are in the locations shown in Figure 9. The elevations are summarized in Table 4.

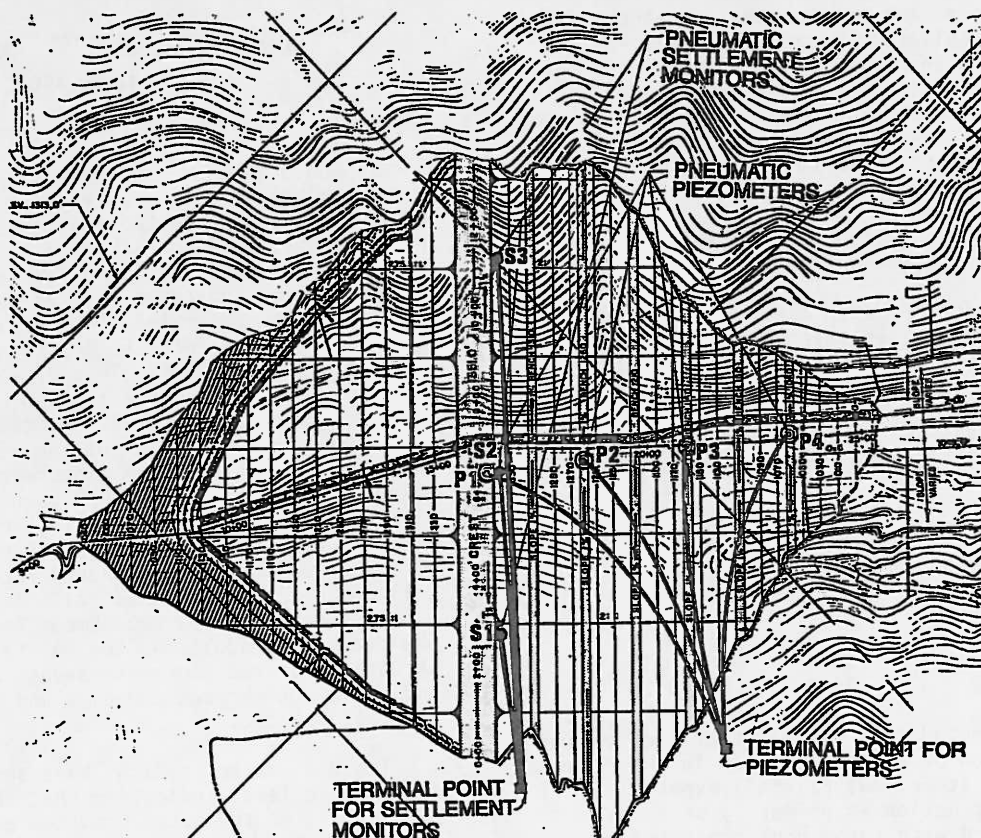


Figure 9. Case history No. 2 - Design concept.



Figure 10. Weir for seepage control.

Table 4. Summary of Piezometer Elevations

Piezometer No.	Elevation (ft)
P-1	1023.8
P-2	1022.7
P-3	1019.4
P-4	1019.0

The pool elevation currently is 1095 feet.

This embankment is not yet completed and the settlement monitors have not yet been installed as the purpose of these monitors is to monitor post-construction settlement. The other instrumentation consists of the weir on the downstream end of the seepage collector pipe. This weir was installed before there was any impoundment in this reservoir and almost immediately water began to exit from the pipe. Measurements have been made of the flow through the pipe versus the upstream reservoir elevations. The following is a summary of this information.

Table 5. Summary of Information on Seepage Flow and Rainfall

1989 Date	Elev. Top of Embankment (ft)	Water Elev. In Reservoir (ft)	Flow From Seepage Pipe (cfs)
5/16	1110		0.21
22	1114		0.17
6/ 7	1125		0.096
13	1128		0.096
20	1132		0.18
26	1143	1093	0.18
7/ 5	1160		0.16
10	1164		0.16
17	1164		0.16
24	1167		0.16
27	1167	1094	0.14
8/ 7	1172	1095	0.16

It will be noted from Table 5 that the seepage is not a function of water elevation in the reservoir. In fact, it follows rainfall events. The embankment construction is primarily of large diameter rock and with sound rock connected to

the drain wrapping up the embankments of the walls. The rises in seepage fairly well monitor rainfall, lagging a few days behind the rainfall. The amount of seepage monitored by the weir versus rainfall events is shown in Figure 11. It will be noted that the seepage is reasonably well independent of the water level within the embankment. For the first about half month of reading there was no water level upstream of the embankment, but the current water elevation is approximately 1095 feet in the reservoir; however, there has been no increase in seepage due to this water head. This is as would be expected as it will take many years before the water saturates the upstream cohesive zone and seeps through. The primary amount of seepage comes through the abutments seeping into the embankment, and rainfall falling directly on the embankment itself.

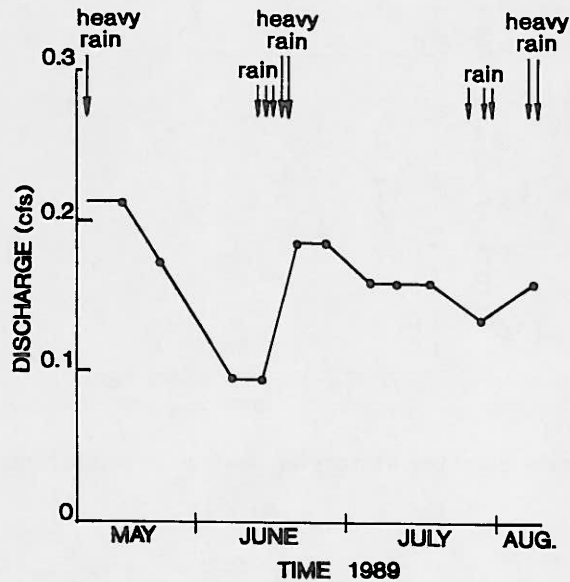


Figure 11. Seepage and rainfall versus time.

It would be concluded from this information that fair amounts of seepage occur simply through natural drainage from the abutments particularly when the abutments are stripped to rock as in most locations this has been done for this embankment. The monitoring indicates that as rain falls onto the embankment the rainfall seeps into the embankment itself, down into the drain and then exits out through the embankment. This of course will be reduced when construction is completed as the area will be covered, vegetated, and the benches on the structure will carry most of the water off. It is anticipated that once the embankment is completed there will be less seepage through the embankment simply because there will be less rock exposed for water to fall into. Currently there are some 35 acres of reasonably open graded rock open to rainfall and all the rain that falls onto the structure simply seeps into the embankment and out through the drain. In addition, the rainfall that seeps through fractured abutments seeps into the drain and exits out through the pipe and this also is rainfall related.

The piezometer readings have shown no build-up in phreatic level indicating that the underdrain is exiting all of the water from the embankment without any seepage problems.

Case History No. 3

This embankment consists of a 240-foot high embankment consisting of 2.5 million cubic yards. The construction is essentially the same as described for the embankments with an upstream cohesive zone, a filter zone consisting of coarse refuse, and a rock zone. The foundation for this embankment consists of some 35 feet of fill previously placed in the valley and there are deep mines and an old strip mine under a portion of the embankment. The piezometers have been installed at the elevations shown in Table 6.

Table 6. Piezometer Elevations

Piezometer	Tip Elevation (ft)
P-1	1178.0
P-2	1179.9

The settlement instrumentation was set at locations over deep mines and adjacent to deep mines, and in the center of the valley to monitor settlement that occurred both in the relatively thick fill underneath the embankment and to monitor any settlement over deep mine locations. Figure 12 shows the location of the monitoring in reference to the centerline of the valley and in reference to the deep mines beneath the embankment. The deep mines under the embankment are located between 65 feet and 110 feet below the surface of the ground and are 4 feet in height.

This structure also contained a strip mine along the right abutment looking downstream. This area had been backfilled with strip spoil material as a part of the strip mining reclamation. This material was not removed from the major portion of the embankment but only from beneath the cohesive zone in order to prevent any excess seepage. The settlement monitors indicate that this is a satisfactory procedure and no detrimental settlement has occurred.

Table 7. Settlement Monitoring Requirements

Monitors	Distance Between Monitors (ft)	Differential Settlement (ft)	
		Safe Limit (1% Deflection)	Limit for Engineer Notification (0.5% Deflection)
S1-S2	400	4.0	2.0
S2-S3	180	1.8	0.9
S3-S4	215	2.2	1.1

The settlement monitors show the following amounts of settlement.

Table 8. Settlement Monitor Readings

Gauge	Date of Installation	Height of Fill Under The Gauge (ft)	Settlement (ft)	
			Through 3/89	3/89-5/89
S-1	11/88	17.0	0.26	0.86
S-2	11/88	27.0	1.06	2.16
S-3	1/89	28.0	0.08	0.72

By comparing settlements available during the same period of time; i.e., March 89 through May 89, the following horizontal deflections are calculated.

Table 9. Actual Measured Deflections

Monitors	Differential Settlement Between 3/89 and 5/89 (ft)	Percent Horizontal Deflection
S1-S2	0.52	0.13
S2-S3	0.48	0.27

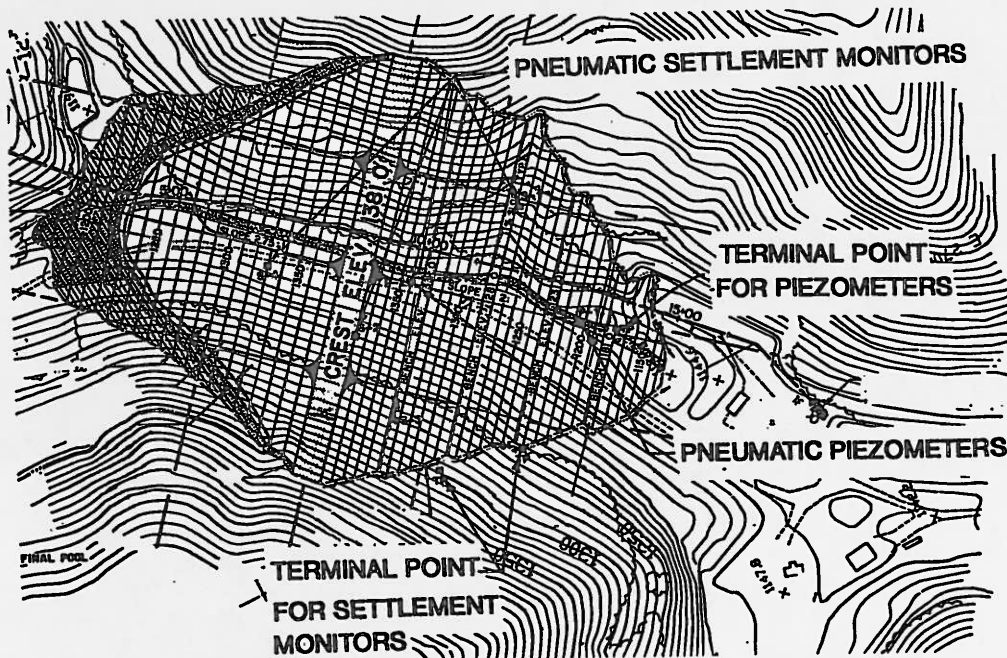


Figure 12. Case history No. 3 - Design concept.

It will be noted from Table 9 that the settlement values are well within the values predicted and well within tolerable values. The cohesive zone shows no detrimental effects from settlement and is intact.

Conclusions

Large earth-rockfill dams can be economically constructed utilizing run of mine strip spoil, coarse refuse as a filter, and on-site cohesive material to provide relatively economical disposal of coarse refuse and sediment control. Very little processing of material, if any, is required. These structures function well even when placed over relatively large amounts of in-place material over strip mined areas and over deep mines. The coarse rock provides a material which is capable of tolerating relatively large amounts of settlement and no detrimental effects to the cohesive liners have been found.

References

1. R.P. Clements, "Post-Construction Deformation of Rockfill Dams", Journal of Geotechnical Engineering, ASCE, Vol. 110, No. 7, pp. 821-840, July, 1984.

SUBSURFACE CONDITIONS IN AND
FOUNDATION CONSTRUCTION ON PINNACLED
CARBONATE BEDROCK

by

James J. Belgeri, P.E.
Manager - Tennessee Operations
Westinghouse Environmental and Geotechnical Services, Inc.
Blountville, Tennessee 37617

C. J. Shin, Ph.D.
Engineering Manager
Westinghouse Environmental and Geotechnical Services, Inc.
Nashville, Tennessee 37211

Abstract. Often the key to successful design or construction of foundations in carbonate bedrock is the successful extrapolation of borehole data to an interpretive "model" or profile of the subsurface. The Paper first reviews the physical-chemical weathering process in carbonate bedrock terrain, reviews subsurface investigation methods and techniques and discusses some common keys to interpretation based on widely-used drilling methods. Also, the Paper proposes a classification scheme for the residual weathered profile exhibited by most all lithified carbonate-rock sites located in humid regions of the southeast U.S. Finally, the Paper provides two case histories in which all the information required for interpretation was collected, but lack of interpretation or misinterpretation resulted in serious technical and economic hardship on the project

Introduction

The success or failure of many foundation projects is often a function of the quality of the interpretation made during the foundation investigation phase of a project. As anyone who has ever drilled a boring, driven a pile or drilled a caisson pier can attest, erratic conditions laden with surprises are to be expected. Nevertheless, there are some recurring features that should enable the practitioner to remove some of the guess work and guide the project owners, designers, and decision-makers. These are described herein.

More importantly, the writers believe that the geotechnical engineers who practice in these regions should be willing to commit to making such interpretations and let the project benefit from their experiences as opposed to simply

letting the boring logs "do the talking." Granted some surprises are still inevitable and disclaimers to protect the professional from same are appropriate. However, too often interpretations which are safe and straightforward are left unmade or are made by individuals less well equipped to make the interpretation.

As will be illustrated in Part II of this paper, interpretation is not only necessary during the investigation phase but also a key element of the construction process. During footing inspections an interpretation of what position is occupied within the weathered profile is key to adequate foundation support.

Part I. Interpretation

Areas of Occurrence

Limestone and dolomite are very common carbonate sedimentary rocks which occur in wide spread areas of the U.S. and the world. General areas of occurrence for the eastern U. S., excepting Coastal Plain areas, are shown on Figure 1. Carbonate rocks do occur in the west; however, because of the arid environment west of the Great Plains carbonate rocks generally are ridge formers and not subject to the weathering in humid zones as described herein. Carbonate sedimentary rocks of young geologic age as found in the Coastal Plains of the east are a slightly different case due to the slightly differing mechanisms and processes and absence of residual mantle.

Our study of interpretation is confined to the so-called "hard-rock" limestones and dolomites of the Paleozoic era.

Objectives of Interpretation

The primary objective of interpretation is to arrive at some rational model of the subsurface to enable evaluation of one or more characteristics of a site. For example, interpretation may be required to enable an estimate of excavation quantities of soil and rock for general grading. In the regions shown on Figure 1 these interpretations are made every day, with varying success, by various individuals ranging from the farmer to the developers of multi-million dollar commercial and industrial facilities. Obviously, a good interpretation is needed to avoid, if possible, the added cost of rock excavation which can multiply the grading cost by 5 to 10 times over the cost of soil grading.

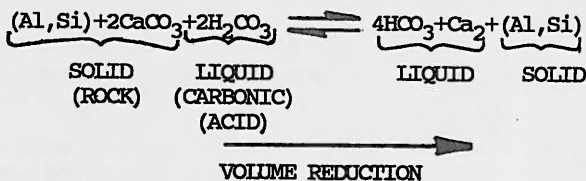
Similarly, estimating the cost of foundations requires an interpretation of rock quantities as rock in foundations is usually 10 to 30 times the cost of soil excavation. Many otherwise successful developments have been shackled with unexpected construction cost due to inaccurate, or no, interpretation of the subsurface.

In addition to the economic risks, there are technical risks that may also have economic impact and may limit ultimate utilization of a site. In this category are the subsidence risk assessment and environmental/contamination risk assessments.

Finally, as project engineers practicing in the region, we are called upon to make daily decisions regarding development and modification of boring programs. Proper decisions in this area will definitely impact our professional standing and career progression.

General Subsurface Configuration

The sometimes wildly erratic subsurface conditions found underlying carbonate rock sites are no accident. The basic mechanism which causes such development is the physical/chemical weathering process wherein the calcium carbonate radical is taken into solution in slightly acidic water. The reversible equation for this weathering is simplified as follows:



The real key is the volume reduction. In this equation the (Al, Si) is used to represent insoluble material (alumino silicates). Initially, these minerals are part of the rock structure. After solutioning, they are what is left, or the residual soil. The amount of volume reduction that occurs is a function of the percentage of insoluble material. In simple terms, a cubic foot of limestone weathers to something less than a cubic foot of residual material.

The residual profile develops as new "layers" of residual soil are "inserted" between the older (upper) residuum and the weathering surface of the rock. The older residuum tends to become stiffer due to overconsolidation by desiccation while the newer residuum tends to become softer since the surface on which it rests is being continually reduced by the weathering process.

The weathering process is concentrated where there are pre-existing discontinuities in the rock such as joints. Accelerated weathering or reduction in this zone assures drainage to this low spot and further concentration of weathering. This is the process which yields the familiar "slots" or solution-enlarged joints common in carbonate rock.

On further maturing, the typical weathering profile begins to take shape. As shown on Figure 2, four zones are usually present as follows:

- 1.) Upper Crust
- 2.) Soil/Rock Interface
- 3.) Weathered Rock Zone
- 4.) Unweathered Rock Zone

The writers' interpretation and description of each zone follows.

Upper Crust. As previously described, the upper crust represents the older residuum. This zone is commonly of stiff to hard consistency due to its overconsolidation, primarily by desiccation. The thickness of this zone will vary appreciably; however, all things being equal, its thickness will be a reflection of the percentage of insoluble residue in the parent rock. The upper crust will tend to rest upon and, in fact, "drape" over the pinnacles of the soil rock interface zone. The amount of overconsolidation and, as a result, the standard penetration resistance (N) will tend to decrease with depth within this zone.

Soil/Rock Interface. Below the Upper Crust is the Soil/Rock Interface. This zone is commonly a wide interval loosely defined as existing from the tops of the pinnacles to the base of the "slots" weathered in to the surface of the rock (see Figure 2). In the writers' estimation the Soil/Rock Interface is the most crucial zone in the entire profile. It is within this zone that the initial soil cavities which eventually may yield surface subsidence begin. Also it is within this zone where large masses of rock may protrude above a site excavation level. Finally, it is within this zone that raveling failures may occur if the zone is exposed to precipitation or surface drainage by site grading or foundation excavation.

Weathered Rock Zone. Beginning at the base of the Soil/Rock Interface is the Weathered Rock Zone. This zone is defined as extending to the base of serious weathering and cavitation. By this definition, the zone can be considered at some locations to extend several hundred feet in depth.

This zone is considered to be of little importance to soil-bearing foundations and to be of great significance to rock-bearing foundations, such as drilled caissons (piers). The writers consider this zone to be of little or very minor importance from the standpoint of ground subsidence. The collapse of roofs of caves has been documented as a cause of ground subsidence, but in the writers' experiences no more than about 1 to 3% of all occurrences are at all related to the Weathered Rock Zone.

Unweathered Rock Zone. The final zone in the profile extends from the base of the Weathered Rock Zone to indeterminate depths. This zone is insignificant to all but the most massive and sensitive structures - nuclear or waste containment, for instance - or to hydraulic structures such as high-head dams where grout curtains may be employed. In most other engineering studies, identification of this zone is largely academic.

Keys to Interpretation

Virtually every subsurface investigation which employs conventional investigative methods yields data which can be interpreted well beyond an analysis of strength and compressibility. The keys to interpreting these data are described in the following paragraphs.

Penetration Resistance. The penetration resistance or N-value (ASTM D1586) is an extremely diagnostic tool in interpretation. The N-value, as previously discussed, will generally be in the 8 to 30 bpf range when in the Upper Crust and decrease with depth. If the top of a pinnacle is encountered at the top of the Soil/Rock Interface a significant decrease in N-value will probably not be observed. On the other hand, if the bore hole enters a "slot" in the bedrock, very low N-values and perhaps even voids can be expected. This condition is an expected consequence of the volume reduction phenomenon and the fact that the Upper Crust will "arch" between pinnacles. Thus, the full effective overburden pressure will not be applied to the soils in a "slot". These soils are commonly normally-consolidated or underconsolidated. An abrupt increase in N-value can also indicate the presence of gravel and infilling which is also common in a "slot".

Penetration resistance is also a good indicator when attempting to determine if a boulder is "floating" in the overburden or part of the bedrock mass. If "floating", the N-value in the soil below the boulder should be similar to other Upper Crust values. If the boulder is part of the bedrock mass, the significant N-value decrease described in the previous paragraph should be observable.

Drilling Reactions. Drilling tools, particularly augers which bind or lead-off at an angle without attaining auger refusal, can usually be attributed to sloping rock surfaces and adjacent "slots". If an auger encounters refusal when surrounded by stiff soils, there will be very little tendency to lead-off.

When drilling with water or mud, drill fluid losses are very common near the upper contact between soil and rock. It is this upper elevation, which should be the top of the Soil/Rock Interface, that is important and not the elevation where rock is finally encountered.

Soils which are cohesive but ravel or cave badly into the bore hole are normally indicative of "slot"-filling soils.

Ground Water. Erratic and varied ground water measurements in bore holes are the rule rather than the exception. In general, this is the result of water being perched on layers which interrupt the vertical seepage downward to the saturated zone. High or uniform ground water levels on an otherwise well-drained site can be indicative of a relatively sound and continuous rock absent of joints or voids and a relatively shallow Unweathered Rock Zone. Conversely, ground water levels uniformly deep below the rock level can be indicative of highly fractured and solutioned rock and a deep Soil/Rock Interface.

Case Histories

Ramada Inn - Birmingham, Alabama. A traditional subsurface investigation was conducted for the owner by a testing laboratory. The conditions encountered were essentially as shown on Figure 3. The testing laboratory provided boring

results and no interpretations or recommendations. The owner's structural engineer, from a western state, designed a caisson foundation system to support grade beams under continuous masonry walls. Because the structure was load-bearing masonry, the designer could have used a few, well-placed, high-capacity caissons under the grade beams or many, low-capacity caissons under small grade beams to support the structure. Sadly, he chose many, low-capacity caissons. During installation, the caisson auger, in virtually all instances, refused at the top of the Soil/Rock Interface and excavation was required into the Weathered Rock Zone to achieve sound bearing. Once achieved, sound bearing was several times higher than the required capacity. Nevertheless, virtually all the caissons had to extend through the Soil/Rock Interface using rock excavation techniques. A foundation cost overrun of about 300% was the result of this lack of interpretation.

In retrospect, interpretation would have yielded the following:

- 1.) A conclusion that rock had to be penetrated to attain virtually any sound bearing.
- 2.) Sound bearing capacity, when achieved, would be much more than the design required.
- 3.) A few caissons would be preferable to many caissons since each, whether heavily-loaded or lightly-loaded, would have to penetrate the same amount of rock.

Manufacturing Plant - Maryville, TN. A traditional and very thorough subsurface investigation was conducted for the designer by a geotechnical engineering firm. The firm provided recommendations and did an excellent job of selecting boring locations to yield maximum information. In fact, the borings penetrated virtually the entire range of conditions. There should have been no surprises. Conditions encountered are represented by Figure 4.

Whether by policy or practice, the firm did not provide an interpretation of subsurface conditions with respect to site grading. It relied on the fact that irregular rock is common and so stated in its report. Unfortunately, the designer and estimators for the contractor, for the lack of direction to the contrary, took the auger refusal elevation as the elevation of rock requiring blasting. In most cases the elevation used was the bottom of the Soil/Rock Interface. When grading at the site commenced, it was apparent that rock excavation methods would be required to excavate materials beginning at the top of the Soil/Rock Interface. As a result and because of the expanse of the facility, rock excavation quantities at the site were underestimated by some 110,000 cubic yards. Additionally, as described in Part II, a high percentage of the footings required field redesign upon inspection.

The real tragedy in this case was that experienced professionals who skillfully extracted the maximum data from a site were constrained from full utilization of the data by either policy, expectation of the profession, or concern over liability.

Part II. Foundation Construction on Pinnacled Bedrock

As described herein, the subsurface condition in limestone terrain is complex. The typical Soil/Rock Interface is a zone rather than a line. A complete investigation of the zone would be costly and impractical. Hence, the success of foundation construction in this zone is greatly dependent upon the confirmation of design parameters during the actual construction. Foundation construction on pinnacled carbonate bedrock involves problems like the stability of bedrock, stability of foundation, and problems associated with the excavation of bedrock. Guidelines to solve the problems are available for a field geotechnical engineer. However, the erratic subsurface conditions would generally require the engineer to treat the foundation on a case-by-case basis. The engineer would conduct some in-situ testing to evaluate the subsurface condition at the location of the foundation. If the finding is not favorable to the design, a treatment to the subsurface or bearing condition would be needed.

This section of the paper presents a case history on the foundation construction of a manufacturing plant in East Tennessee. The site is located in a limestone terrain. Part of the foundation bearing level is at the Soil/Rock Interface. Shallow footing foundations were designed to support the structure. The subsurface and bearing conditions of the foundation were evaluated and problems presented were addressed by the authors during the construction.

Background Information

The referenced project is a single-story manufacturing plant. The structure is steel-framed and about 27 feet in height. Column loads range from 30 to 460 kips. The majority of the building areas are on cut. The building is supported by 391 spread footings. The original design had 69 rock-bearing footings, and 322 soil-bearing footings. At the end of foundation construction, the records show that there are actually 114 rock-bearing footings and 277 soil-bearing footings. Also, among the 69 designed rock-bearing footings 19 are actually bearing on the soil. The 114 rock bearing footings are almost all constructed in the Soil/Rock Interface.

Published geologic information indicates that the project site is underlain by Maryville Limestone. The project site is located in the

Valley and Ridge Physiographic Province. Locally, the rock was compressed into long narrow folds whose axes lie a northeast-southwest alignment. The rock beds were tilted downward toward the southeast at angles ranging from 20 to 80 degrees. The unaltered Maryville Limestone has a dark blue-gray color and some beds with high calcium carbonate content. The subsurface investigation conducted for the project indicates that:

"The bedrock of the Maryville Limestone formation is pinnacled, containing slots and weathered seams. This is substantiated by the rock coring data and observation of limestone outcrops at this site. Rock coring performed indicated boring refusal materials and rock exposures observed at this site generally represent bedrock rather than isolated boulders within the residuum."

Encountered Problems

The erratic subsurface conditions at the Soil/Rock Interface made the foundation construction difficult. The problem usually involved the stability of the foundation, the stability of the bedrock, and the disturbance caused by rock excavation. In general, the first two problems cannot be avoided; however, the third can be eliminated by careful planning and control.

Stability of Footing. This problem usually occurs when the bearing stratum is not uniform. For instance, the partial-soil-partial-rock situation indicated in Figure 5.a would cause the footing to rotate and result in a bending moment at the junction of the column and the footing. Figure 5.b indicates a situation where the subgrade reaction would concentrate at the rigid supports, i.e., the rock ledges. These concentrated reactions could cause a high shear stress in the footing and could result in a structural failure of the footing.

Stability of Bedrock. Weathered seams, solution pockets and solution channels are typical features in the Maryville Limestone. These features could create weak planes and reduce the overall stability of the rock. Figure 6 depicts a condition in which the rock would act like a cantilever beam. In this condition an evaluation of the ability of the rock to support the total column load is required rather than the bearing capacity of the rock.

Disturbance Caused by Rock Excavation. Maryville Limestone is a hard, fine-grained rock. Blasting is generally employed for rock removal to achieve design grade. Due to the weak planes of the rock and the soil fill between the rock, blasting could create a zone of soil-rock mixture and make the soil difficult to evaluate for foundation support. The disturbance could loosen the in-situ soil and create voids between rock fragments. Figure 7.a indicates a typical example of soil-rock mixture generated by blasting. Sometimes the drilled blast holes were terminated just before penetrating through the rock pinnacle as indicated in Figure 7.b. Since the blast hole is usually stemmed with backfill, the gas pressure goes in the direction with the

least resistance, in this case, downward. The blast energy is then absorbed by the soil between the rock pinnacles leaving a highly-disturbed soil.

Vibration of Fresh Concrete Due to Blasting. At this site, foundation construction was conducted the same time as site grading. Blasting was a regular activity on site. In fact, the erratic rock pinnacles frequently required blasting in the vicinity of fresh foundation concrete. Also, the construction of utility lines would run into rock pinnacles after hundreds of feet of construction in soil. Due to the special subsurface conditions, the development of a blast criterion for the project was needed.

In-Situ Testing

In-situ testing was conducted for two reasons. For soil bearing footings, it provided information like the shear strength and, thus, the allowable bearing capacity of the soil. For rock bearing footings, the testing was used to determine the integrity and continuity rather than the bearing capacity of the rock. As mentioned before, the Maryville Limestone is a hard rock with a very high compressive strength. Problems with rock-bearing footings are usually caused by the presence of weathered seams, solution channels and solution pockets. In-situ testing on the rock was used to determine the overall stability of the rock.

The testing on soil included penetration tests using a dynamic cone penetrometer and pocket penetrometer. Power and manual-driven augers were used to drill test holes in the footing excavation. Auger cuttings were examined by the field engineer and penetration tests were conducted at various depths.

The testing on rock involved drilling test holes with air-track or hand-operated rock drill. The engineer would record the speed of drilling, the amount of return (thick, medium, thin or no dust), and the color of the dust. The speed of drilling could be an index of the hardness of the rock, although an experienced operator who wished to do so could control penetration rate. The return from the drilling could provide the engineer with some ideas as to the continuity of the rock. If a thick dust with rock chips is blown out of the hole continuously, the test hole is passing through a continuous rock. On the other hand if no dust comes out of the test hole, there is probably a void in the rock. A change in the color of the dust could indicate a change of material. Sometimes several test holes would be drilled in one rock to determine the stability of the rock under loading. If a series of test holes were drilled in the rock of Figure 6, for example, the dust from the drilling in one hole may return out of the other drill holes.

Solutions to Problems

Undercutting/Backfilling. This measure was usually applied to soil-bearing footings with noncompetent bearing materials. Such conditions involved low-strength natural soil or disturbed soil/rock. In the former case, the depth and width of undercutting was selected based on the

strength of the soil and the stress distribution characteristics of the backfill material. In the latter case, the disturbed material was generally completely removed.

For a partial-soil-partial-rock situation (see Figures 5.a and 5.b) the main concern is the non-uniform contact pressure which would cause undesirable or higher-than-design stresses in the footing. By cutting the rock pinnacles and backfilling with a soil, a cushion is built between the footing and top of rock. As a result, the column load can be distributed more evenly. The depth of undercut would depend on the relative area and the relative rigidity of the soil and the rock. Generally, 12 to 24 inches undercut was employed for footings with less than 30% rock. The stability of the rock would become important if the rock covered an area greater than 30% of the footing area. Sometimes rock pinnacles protruded from the side of the footings. In this case the rock was trimmed and then a bond breaker would be placed to separate the footing and the rock.

Plugging Solution Channels And Weathered Seams.

Generally, the soil in a solution channel is moist and soft. This is probably due to the lack of drainage paths and the shielding effect of the surrounding rock. When solution channels are found in the bearing stratum of a footing, it is desirable to stabilize the solution channel. The process usually involves removing the soil and backfilling with full strength or lean concrete. Figures 8.a and 8.b show two examples.

Bridging. Sometimes a solution channel may have a shape and size that a concrete backfill would not be stable without any attachment to the adjacent rock. In this case it is more effective to build a rigid mat that bridges across the solution channel. The following paragraphs indicate two typical examples.

Figure 9 shows the profile of a continuous footing. A solution pocket was found at section 1 and a solution channel was found at section 2. The solution pocket was cleaned and lean concrete was backfilled. The solution channel had a width of 7 feet and a depth greater than 6 feet. It was clear that the solution was in between two parallel bedding planes. A bridge with full strength concrete and reinforcements was built over the entire section 2 and the footing was constructed on top of the bridge beam.

Figure 10 shows the plan view of a spread footing. A solution pocket was found at the center of the footing. The bottom of the excavation was about 5 feet below the surface. Cleaning by a backhoe would be difficult. It was decided to build a rigid 12 x 12-foot reinforced concrete mat on the top of the rock. A regular 8 x 8-foot footing was then built on top of the mat.

Stepping Bedrock Or Utilizing Dowels. Stepping and dowelling were used to avoid shear failure along an inclined rock surface. Figures 11 and 12 show a schematic view of the methods. The use of dowels as shown in Figure 12 should be selected with care as the capacity of these dowels in shear is very limited and prestressing may be required.

Vibration Control. A blasting criterion was developed for the protection of fresh foundation concrete from vibration. The criterion was developed at the early stage of construction and used throughout the construction. The development involved recording 23 shots and plotting the relationship between the measure peak particle velocity and scaled distance (see Figure 13). The latter is defined as the distance (between the blast hole and geophone) divided by the square root of the weight of the charge. By using the upper bound of the response and the allowable peak particle velocities, a criteria as indicated in Figure 14 is obtained. Note that a more conservative criterion would be developed with the upper bound than lower bound response.

Studies in the past have shown that the allowable peak particle velocity is strongly related to the compressive strength of concrete.^{1, 2} However, the allowable peak particle velocity for concrete fresher than one day is a controversial issue. A summary on the research effort in this area was provided by Akins and Dixon.³ For this project the authors employed a criteria of 0.25, 0.36, 0.5, 1.2, 1.5 and 2.0 in./sec. as the allowable peak particle velocity for concrete age of 1, 2, 3, 7, 14 and 28 days, respectively.

Acknowledgements. The authors wish to express their appreciation to Albert Kahn Associates, Inc., the designer of the project. The help from Project Manager, Mr. Robert P. Walsh, P.E.; Structure Designer, Mr. Gar M. Hoplamazian, P.E.; and Field Representatives, Mr. William K. Kinner and Mr. Scott Z. Hazen, P.E., is gratefully acknowledged.

References

1. John F. Wiss, "Construction Vibrations: State-Of-The-Art", Journal of Geotechnical Engineering, GT2, pp. 167-181, 1981.
2. E. V. Howes, "Effects of Blasting Vibration on Curing Concrete", Proceeding, Twentieth U.S. Symposium of Rock Mechanics, pp. 455-458, 1979.
3. Kenneth P. Akins, Jr. and Donald E. Dixon, "Concrete Structures and Construction Vibrations", American Concrete Institute, SP 60-10, pp. 213-247, 1979.

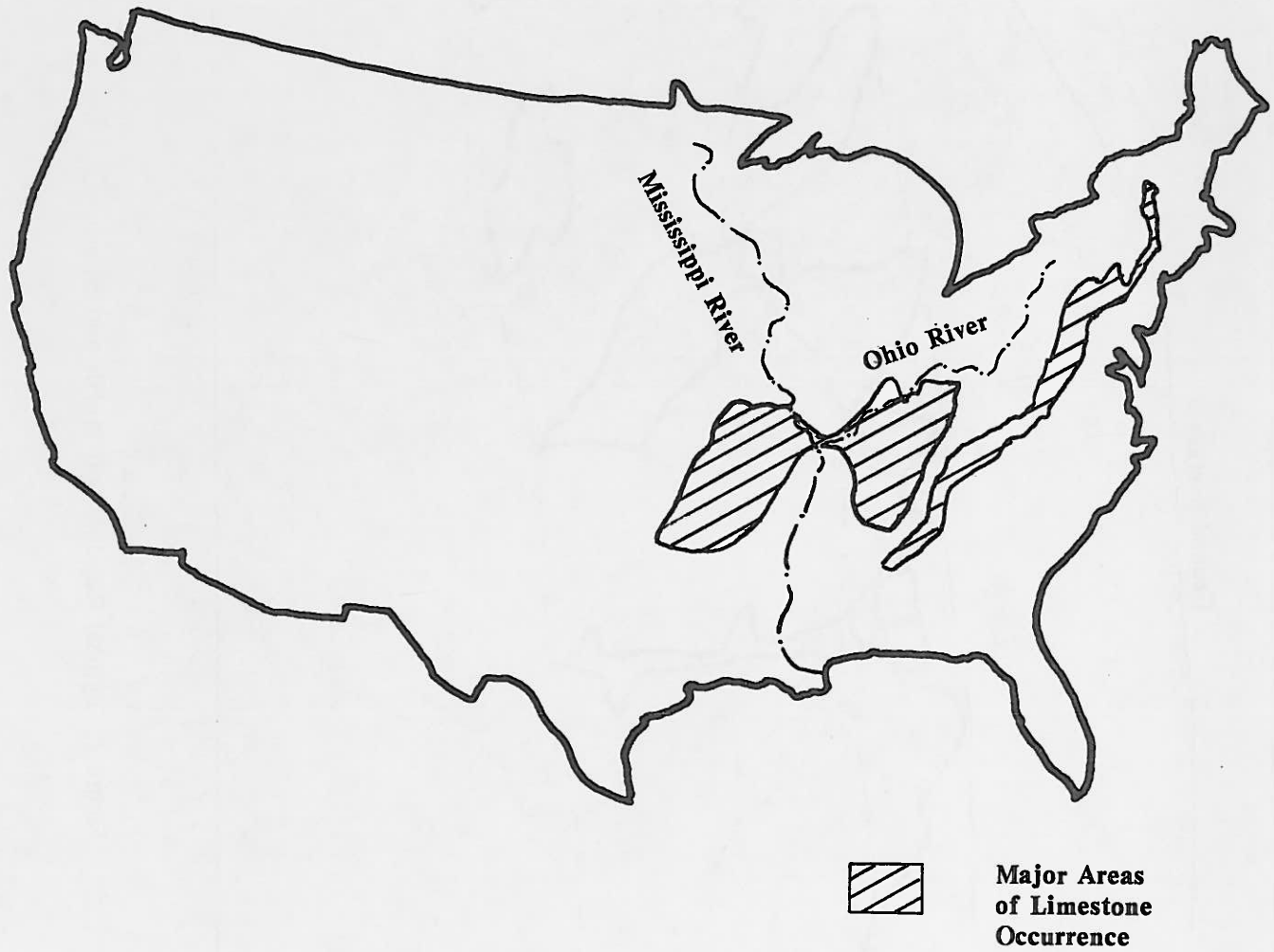


Figure 1. Areas of Limestone with Residual Profile.

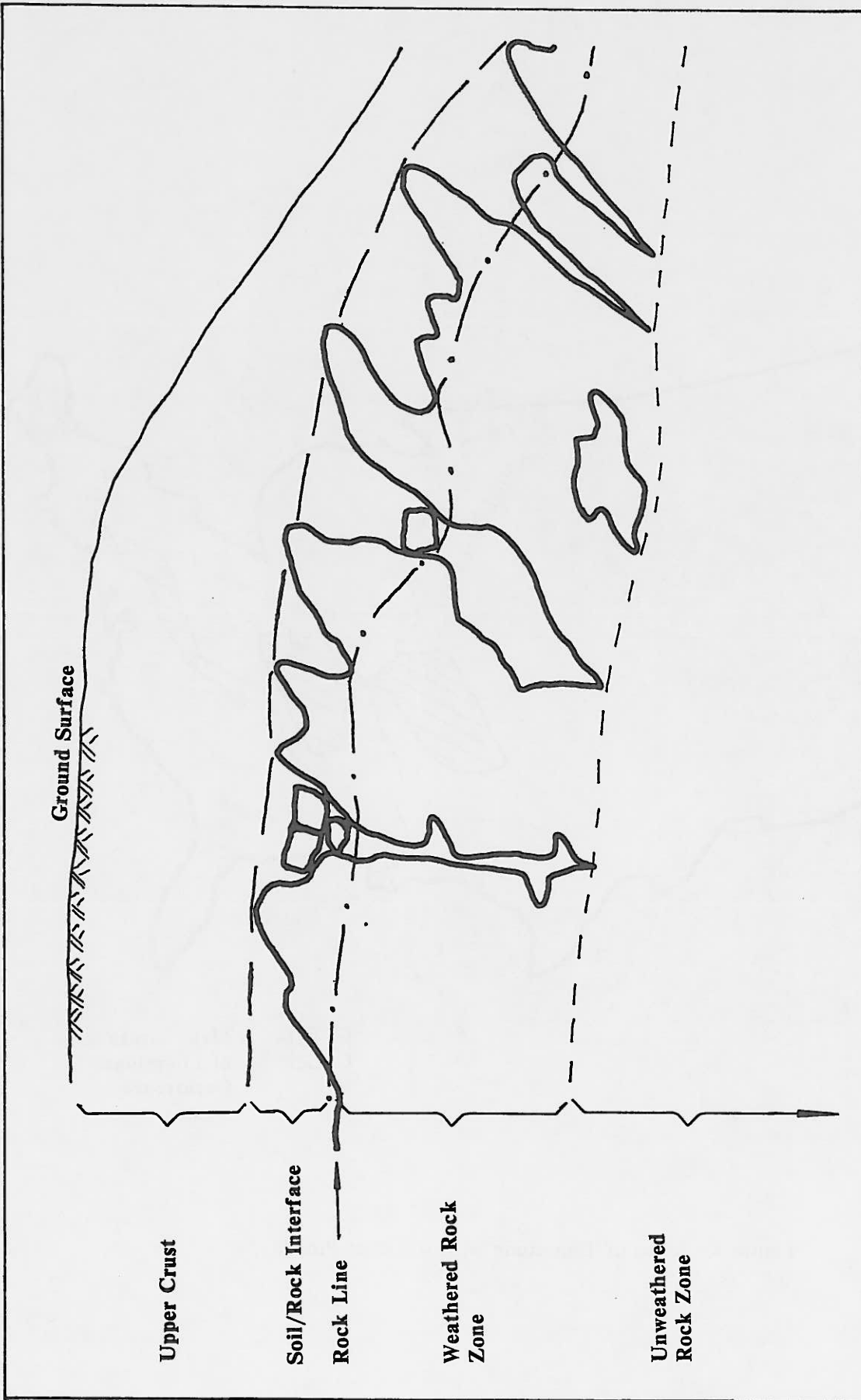


Figure 2. Typical Residual Soil Profile and Divisions.

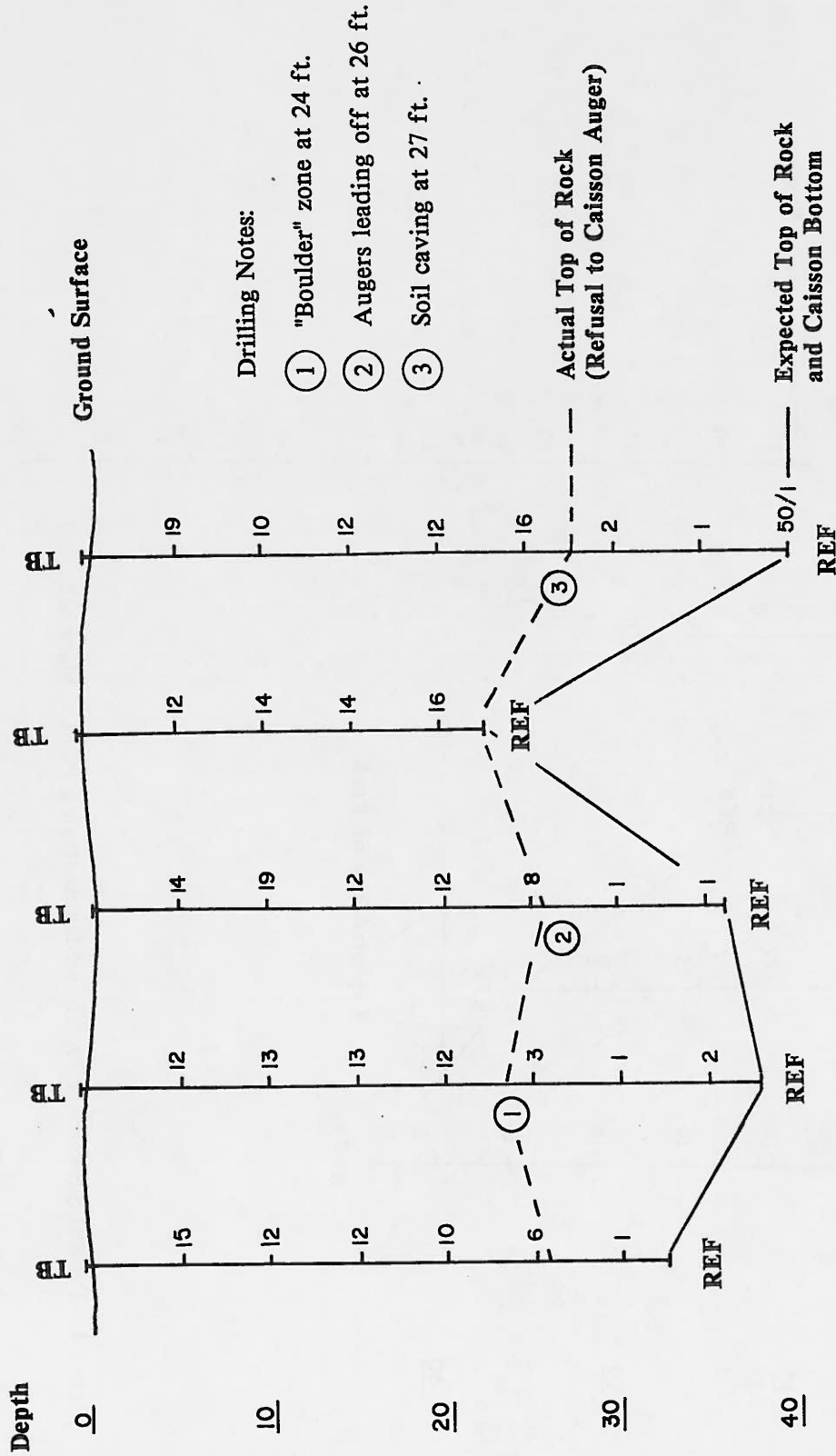
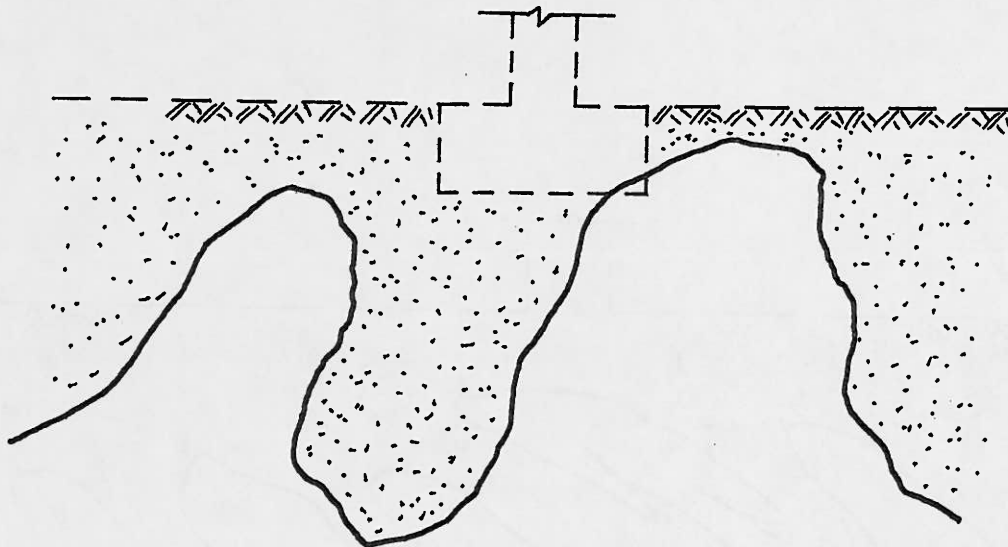
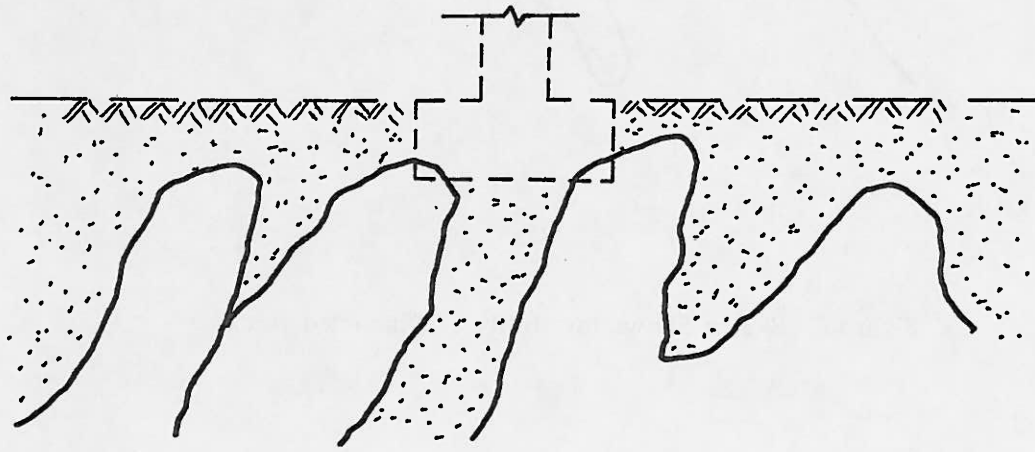


Figure 3. Typical Subsurface Profile - Ramada Inn - Birmingham, AL.



(a)



(b)

Figure 5. Foundation with Partial-Soil-Partial-Rock Situations.

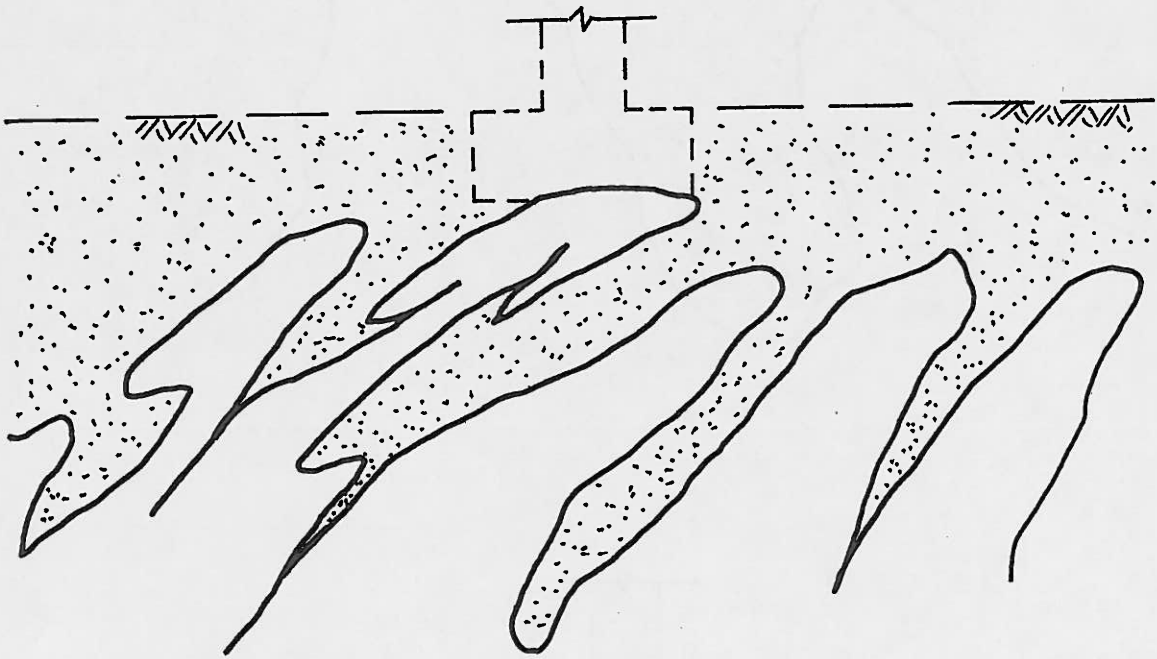
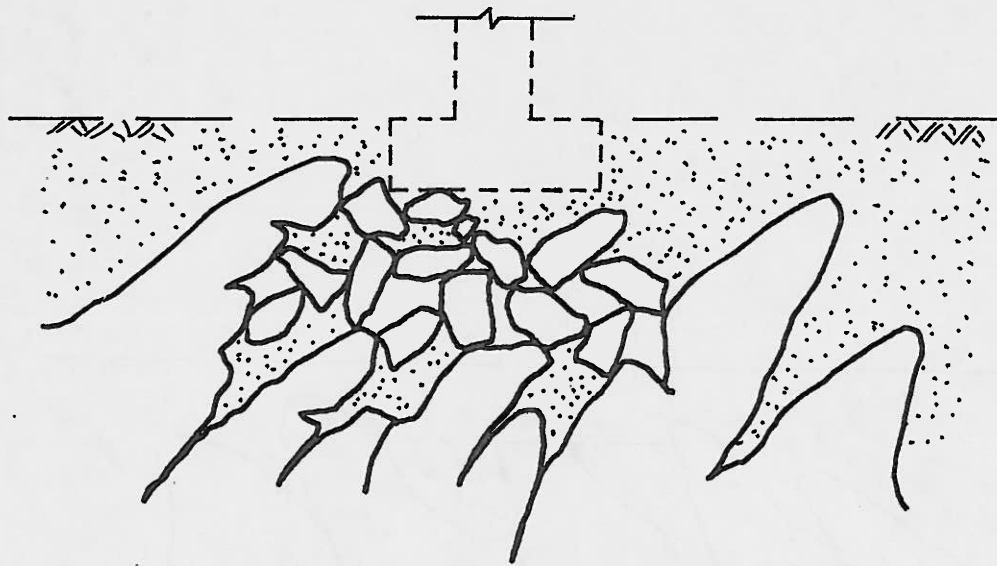
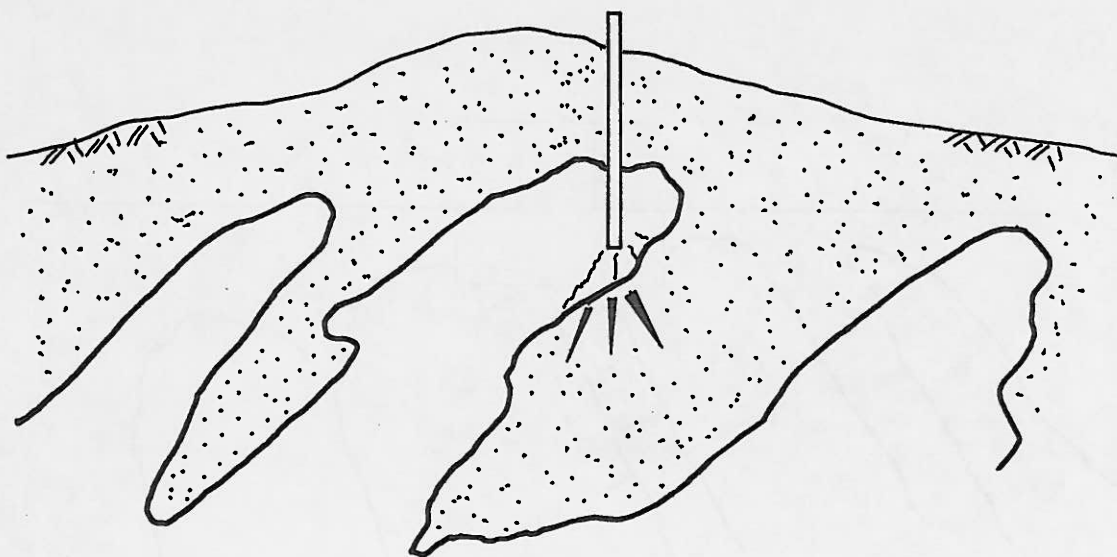


Figure 6. Sketch Shows Instability of Pinnacled Rocks.



(a)



(b)

Figure 7. Sketch Shows Disturbances by Blasting.

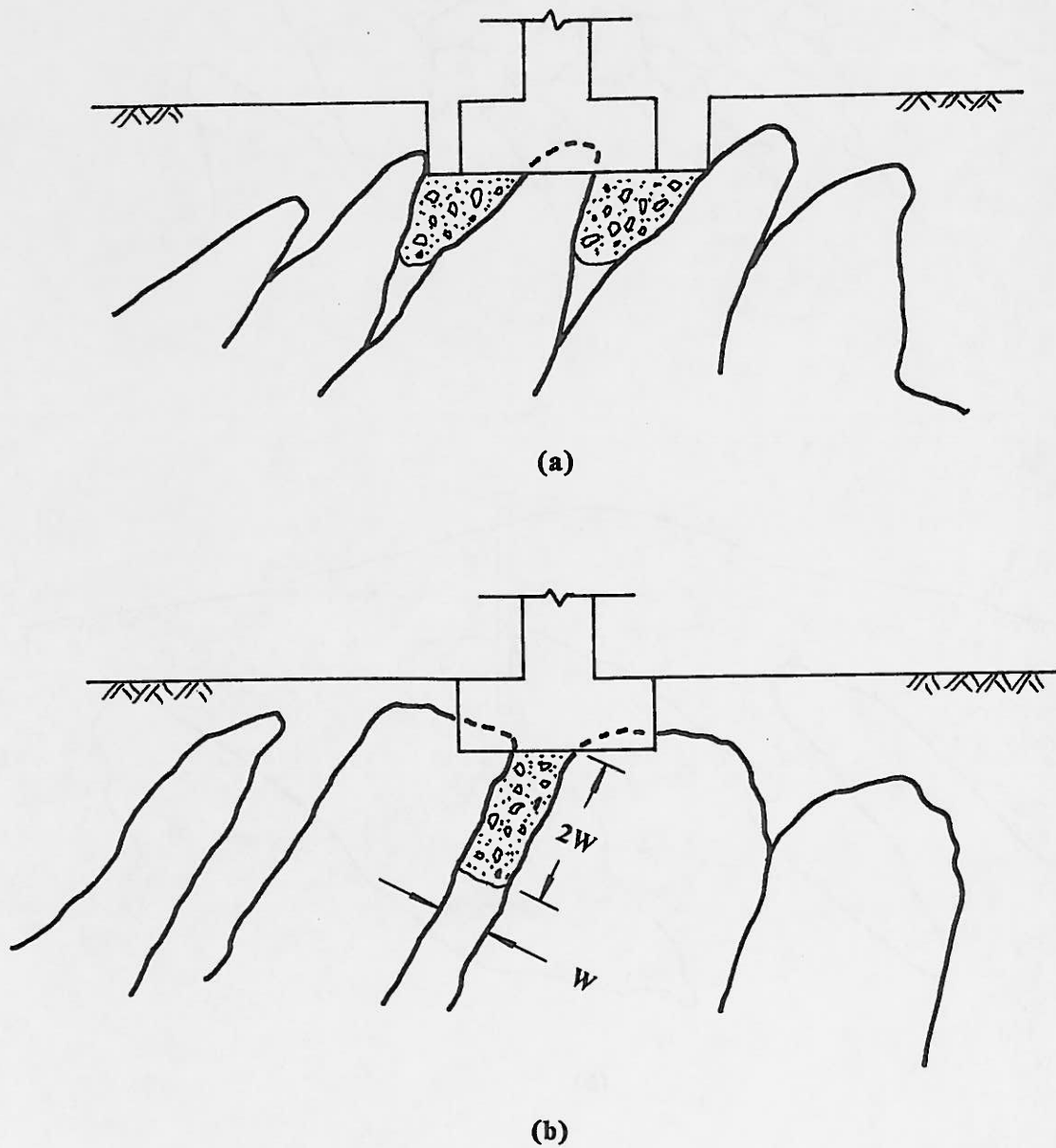


Figure 8. Plugging Solutions with Concrete:
(a) Solutions of Wedge or Conical Shape.
(b) Solution between Two Parallel Bedding Planes.

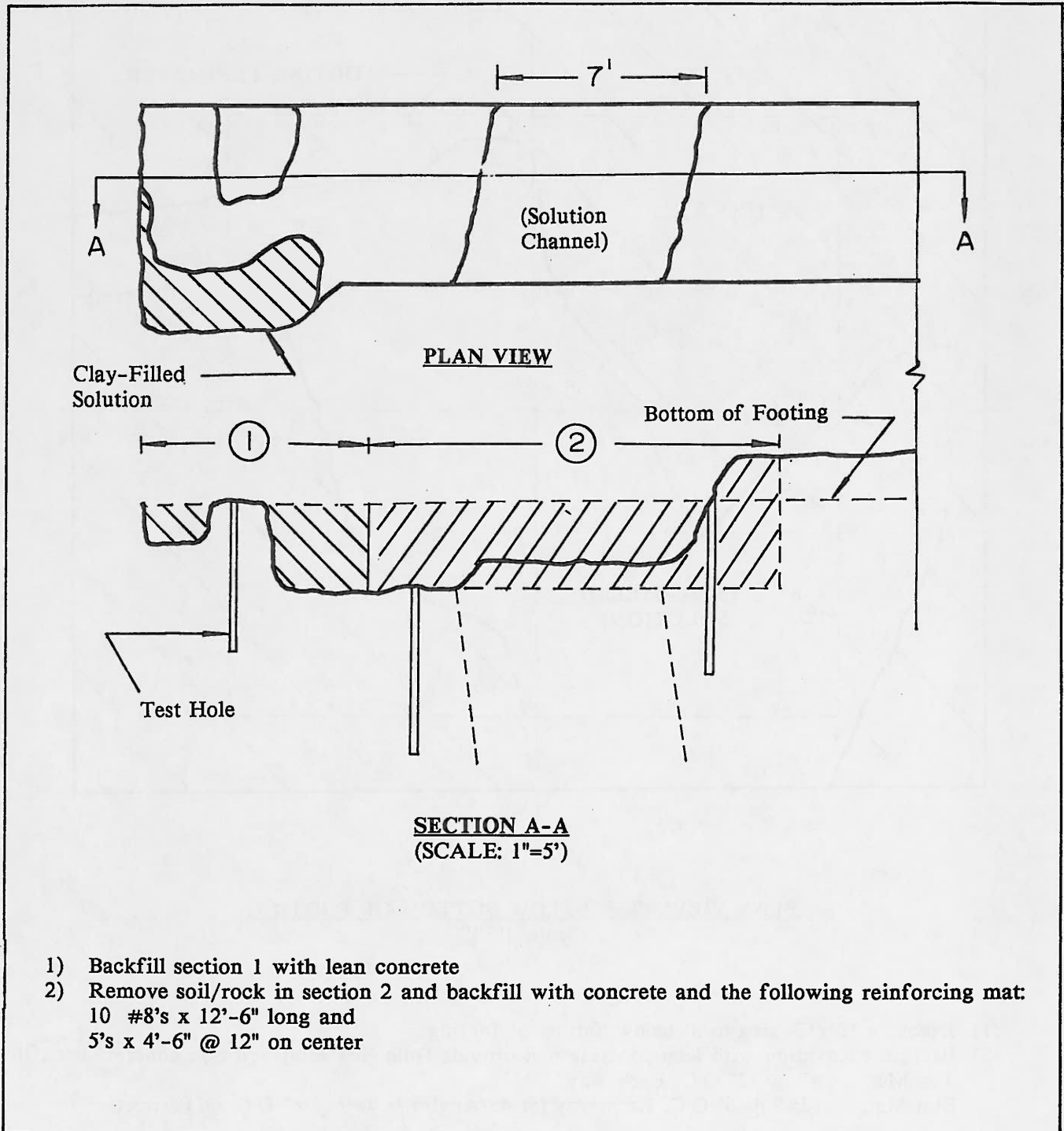
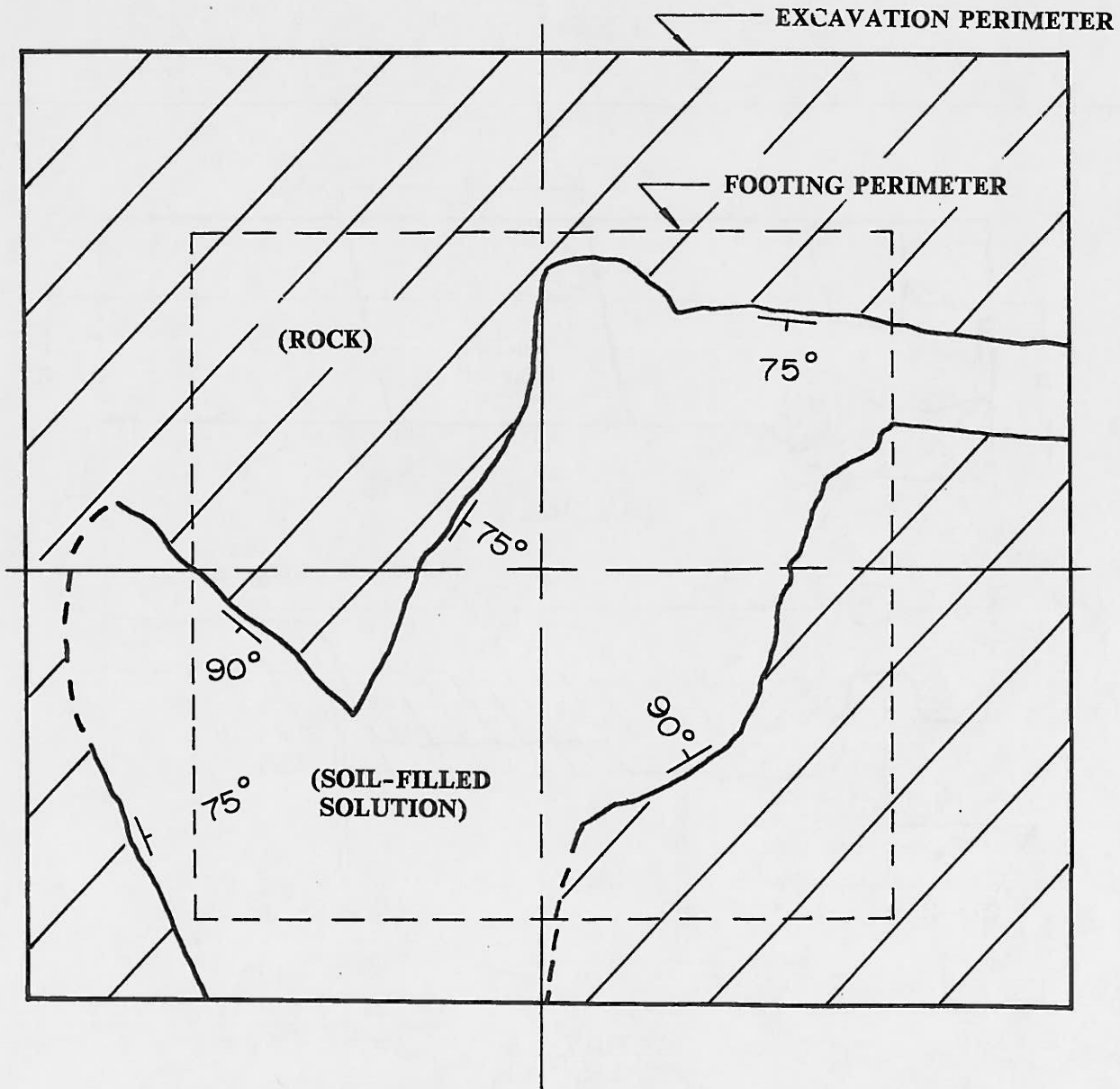


Figure 9. Sketch Shows A Continuous Footing with Solution Pocket and Channel.



PLAN VIEW AT 3' BELOW BOTTOM OF FOOTING

(Scale: 1"=2')

- (1) Excavate 12'x12' area to 3' below bottom of footing.
- (2) Backfill excavation with lean concrete and provide following reinforcing in concrete backfill:
 Top Mat: #7 @ 12" O.C. Each Way
 Btm Mat: 12#9 @ 6" O.C. Each Way (at 45°Angle) & 4#9 @ 6" O.C. @ Perimeter

Figure 10. Sketch Shows A Square Footing with A Deep Solution.

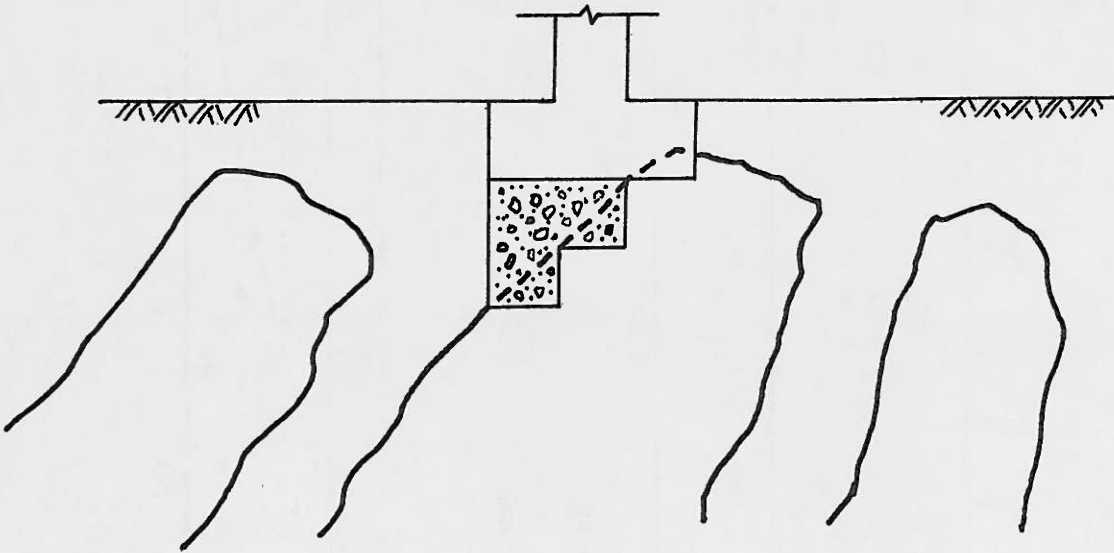


Figure 11. Foundation with Stepped Rock.

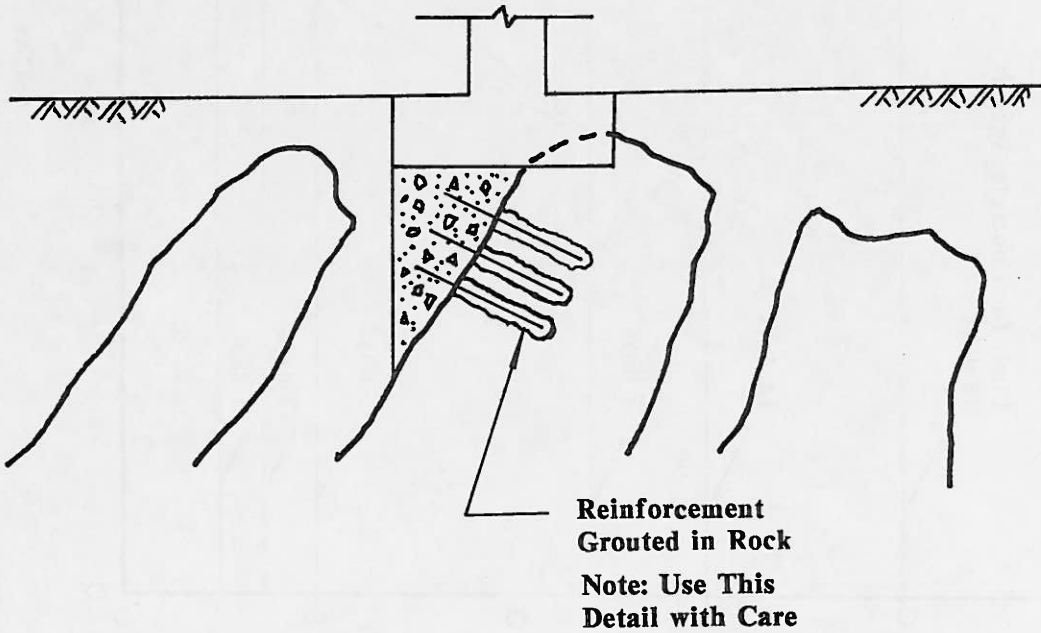


Figure 12. Foundation with Dowels.

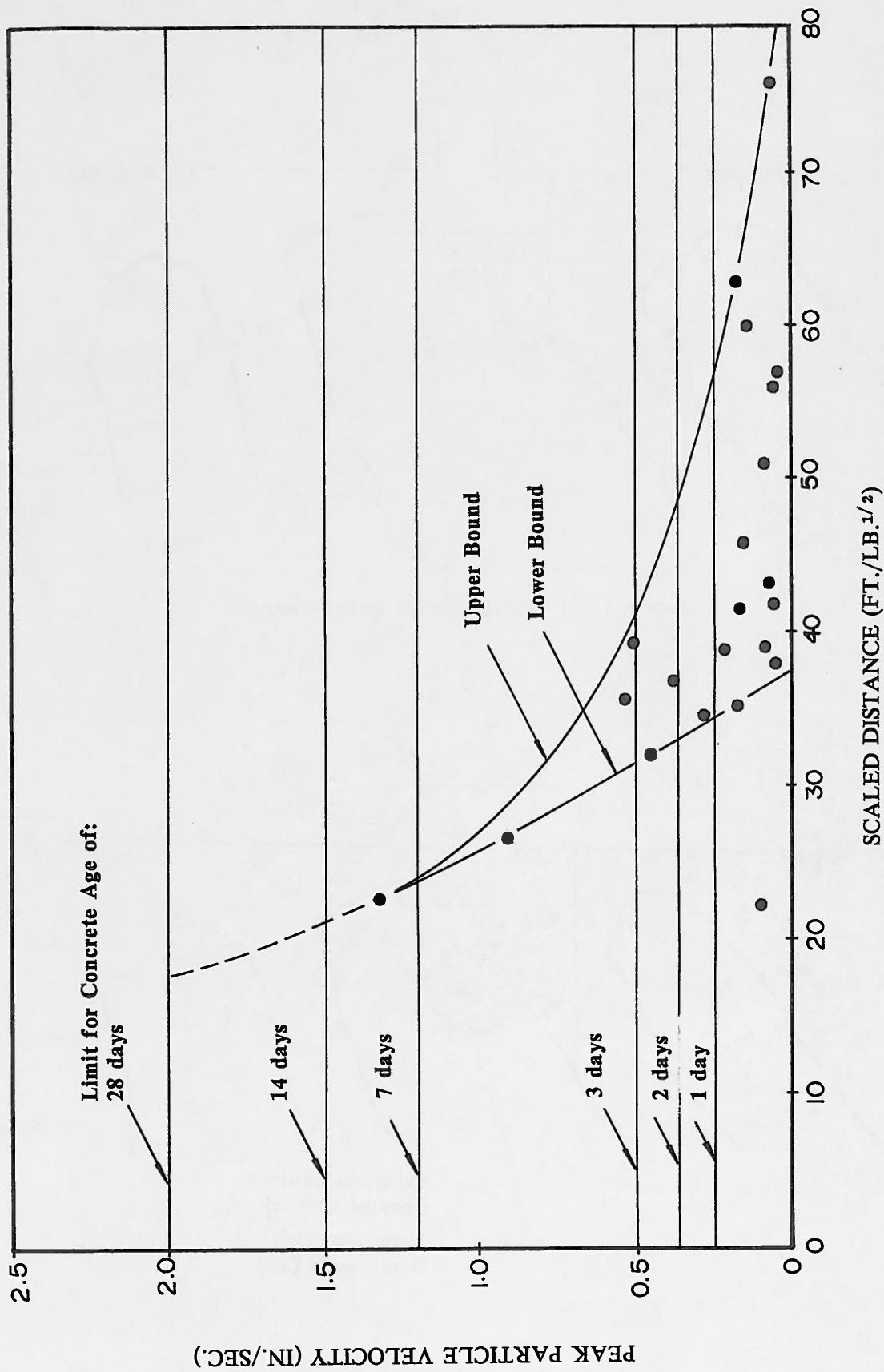


Figure 13. Relationship of Peak Particle Velocity and Scaled Distance.

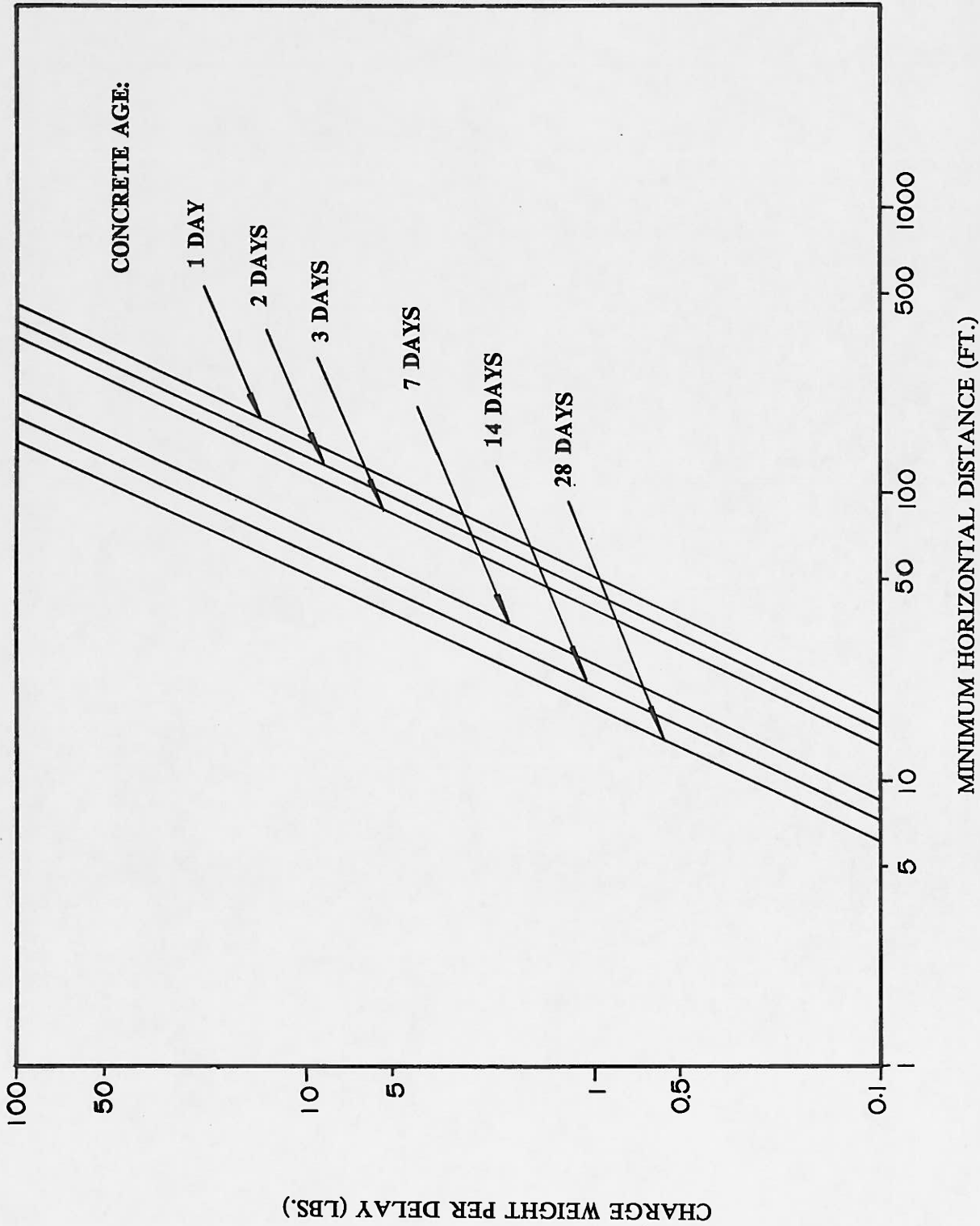


Figure 14. Blast Criteria: Relationship of Charge Weight and Minimum Horizontal Distance.

TESTING ROCK-SOCKETED DRILLED PIERS
USING THE OSTERBERG LOAD CELL

by

James D. Smith, P.E.
Senior Engineer, H&A of New York*
and
Timothy Crowl
Assistant Engineer, H&A of New York*
Rochester, New York

Abstract. This paper describes the results of a full-scale load test conducted on a drilled pier socketed in bedrock. The test was performed using the recently-developed Osterberg Load Cell to produce and measure the test load on the pier. The pier, consisting of a 6-ft. long, 24-in. diameter reinforced concrete element, was socketed in competent sandstone, within a borehole advanced a total of 30.5 ft. below ground surface. The load cell was positioned at the bottom of the borehole prior to placing concrete for the pier element. Instrumentation was installed to monitor pier movements at the top and bottom of the test section as well as at the reaction point beneath the load cell. Vibrating wire strain gages were installed within the concrete section to determine stress distribution resulting from the loading. An analysis of the data is discussed which includes an assessment of the ultimate bearing capacity and side resistance capacity of the test pier socket, along with in situ rock modulus implied by the test results. Comparisons are made to original design assumptions related to the strength of the rock mass as well as the relative stiffness of both the rock and concrete pier element.

Introduction

Engineers typically have had difficulty in evaluating load transfer mechanisms for drilled pier foundations, particularly when piers are socketed into bedrock. It is known that some load transfer occurs through shaft side resistance and that the remaining load is resisted by end bearing at the bottom of the pier. However, evaluating the distribution of load between these two support mechanisms is seldom accurately accomplished during design.

Typically engineers assign a nominal value to the allowable side resistance based on local practice or experience. Occasionally, a pull-out

test may be performed to establish the ultimate resistance between a test element and the foundation medium. The engineer usually then selects the allowable resistance as some fraction of the ultimate, which is hopefully compatible with the anticipated pier settlements. Both approaches are less than satisfactory when trying to develop an optimum pier design taking advantage of both side resistance and end bearing.

This paper describes a load test performed on a drilled pier, socketed in bedrock, using the recently developed Osterberg Load Cell to both produce and measure the test load on the pier. The unique aspect of the test, and the Osterberg Load Cell itself, is that the cell is positioned at the bottom of the pier and produces an upward force that can be used to measure the ultimate side resistance along the pier shaft. An

* Affiliate of Haley & Aldrich, Inc.,
Cambridge, Massachusetts.

equal and opposite force is generated downward which is resisted in end bearing. The test set-up provides for direct measurement of deflections at the base of the cell and at various points within the pier element, which allows the engineer to evaluate stress-deformation behavior for both end bearing and side resistance.

Project Description

The project for which the load test was performed involved construction of a large manufacturing facility in Rochester, New York. Design column loads for the proposed structure were estimated to range from approximately 4000 to 5500 kips.

Subsurface explorations revealed that soil conditions at the site generally consisted of approximately 20 ft. of soil overburden underlain by bedrock. The soils were comprised of miscellaneous fills and lacustrine sands and gravels overlying a 4 to 6-ft. stratum of dense glacial till. The underlying bedrock consisted of a competent sandstone known locally as the Grimsby Sandstone. The formation is a Silurian age, fine to medium-grained, thin to medium-bedded sandstone with an average unconfined compressive strength of approximately 17,400 psi.

Test borings indicated that the upper 1 to 2 ft. of bedrock was highly jointed and blocky. In addition, weathered zones and argillaceous partings were prevalent to a depth of approximately 6 ft. below top of rock. Below that depth the rock was generally of fair to good quality with Rock Quality Designation values, as described by Deere, et al (1), ranging from 56 to 100 percent.

Based on the required foundation capacities and configuration, it was recommended that the building columns be supported on drilled piers socketed a minimum of 6 ft. into the underlying sandstone to penetrate the more weathered zones. Structural design requirements dictated that the piers be a minimum 6-ft. diameter. Within these constraints, it was ultimately recommended that the piers be designed using a maximum allowable end bearing stress of 55 tons per square foot (tsf) and a maximum allowable side resistance of 6.5 tsf. Using design solutions developed by Kulhawy and Goodman (2) and others, and estimates of rock and concrete modulus, it was anticipated that approximately 25 to 40 percent of the axial pier loads would be carried in side friction for pier sockets ranging from 6 to 10.5 ft. in depth.

The allowable design pressure of 55 tsf was slightly higher than had been used previously in the Grimsby formation, and with approximately one third of the pier loads expected to be taken in side resistance, it was recommended that a load test be performed to verify the design assumptions.

After coordinating with the drilled pier contractor for the project, Northeast Caissons, Inc. of Akron, New York, it was arranged that the test would be performed using the Osterberg Load Cell.

Osterberg Load Cell

Background

The Osterberg Load Cell is a device, originally conceived by Dr. Jorg Osterberg about five years ago, designed to simplify testing for drilled piers and to allow assessments of both shaft resistance and end bearing. A prototype device was fabricated and tested in collaboration with Case International in 1984 (3). The load cell was a bellows type, approximately 4 ft. in diameter, and was used to load a 10-ft. long test shaft to approximately 90 tons. A patent for the device was applied for and received by Dr. Osterberg in September 1986.

Subsequently, Osterberg teamed with C.L. Guild at American Equipment and Fabricating Corp., East Providence, RI, to develop a similar device suitable for use with a driven pile. Several cells were produced for proposed use with 14-in. square prestressed concrete piles, but were never tested (4). Ultimately, in October 1987, an 18-in. diameter model was fabricated for use on a Haley & Aldrich projection on the Saugus River, in Lynn, MA, where the cell was fitted to the bottom of an 18-in. O.D. steel pipe pile which was driven to end bearing at a depth of approximately 127 ft. A load test was subsequently performed failing the pile in side friction at a load of approximately 142 tons.

The load cell used for the test described in this paper was also fabricated by American Equipment and Fabricating Corp. and was essentially identical to the one used on the Saugus River project. The cell was selected based on its size, capacity, and immediate availability.

Design

The load cell was designed and fabricated as a two piece hydraulic jack as shown in the schematic in Figure 1. The jack was designed to withstand the rigors of pile driving and was more robust than the bellows type prototype originally developed by Osterberg.

The jack consisted of the piston and cylinder as shown, coupled to a 2-in. diameter double strength steel pipe connected to the hydraulic pressure source at the ground surface. O-rings sealed both the hydraulic line and the piston/cylinder assembly. A 1.25-in. diameter steel rod was fitted within the hydraulic line and threaded into the piston as shown. The rod extended to the ground surface, passing through a packer at the top of the hydraulic line allowing independent movement between the inner rod and the outer pipe. Together, the pipe and inner rod functioned as telltales providing direct measurements of movement at the top and bottom of the load cell.

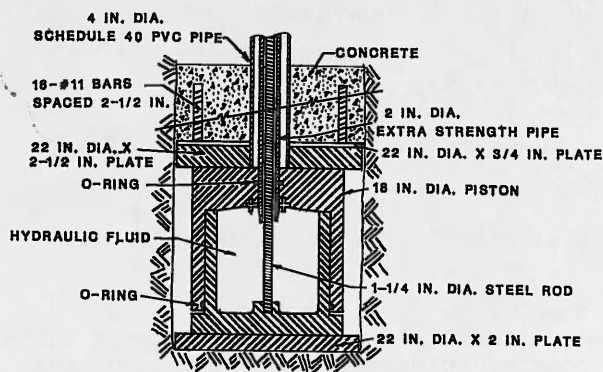


Figure 1. Schematic diagram of Osterberg Load Cell assembly.

As indicated above, the load cell was fabricated with intention for use on an 18-in. diameter pipe pile. To meet the needs of this load test, 22-in. diameter steel plates were fitted to the top and bottom of the cell, providing the necessary bearing area. A 4-in. diameter PVC pipe sleeve was also fitted to isolate the hydraulic line from the concrete pier element above.

Hydraulic pressure was supplied to the load cell by a nitrogen actuated pump located at the ground surface. Pressure measurement was made using a

6-in. diameter, 10,000-psi gage located at the connection to the steel hydraulic line. Prior to shipment to the site, the load cell and pressure gage were calibrated by Richard Dudgeon, Inc. of Stamford, CT, to a maximum load of 500 tons at an operating pressure of approximately 6000 psi.

Test Pier and Load Cell Installation

A pier element approximately 24 inches in diameter and 6 feet in length was chosen for the test. The dimensions were selected to be compatible with the available load cell and with the proposed foundation design. The 24-in. diameter was judged to be large enough to provide a reasonable correlation between the test results and performance of the actual pier foundations. The 6-ft. length was chosen to correspond to the recommended minimum pier socket depth and to ensure a reasonable probability of failing the pier in side resistance with the available load cell. The actual pier diameter, as installed, was approximately 22.75 in. as shown in Figure 2 and discussed in the following section.

Pier Installation

The test pier was installed by Northeast Caissons, Inc., on 21 January 1988, using a Hughes Model LDH truck-mounted drill rig. Auger bits were used to advance the shaft through soil overburden and into bedrock. The shaft was initiated with a 42-in. diameter bit which was advanced to a depth of 8 ft. Drilling was continued with a 36-in. diameter bit until weathered rock was encountered at 19 ft. The open borehole was advanced to a total depth of 21 ft. using the auger bit and a clean-out bucket. A 20-ft. length of 36-in. diameter temporary casing was subsequently installed and seated at the top of rock.

The rock socket was advanced initially to a depth of 28 ft. using a 22.75-in. diameter rock auger bit. Visual inspection of the socket revealed severely jointed and blocky rock near the top of the rock surface, and the socket was subsequently advanced to a depth of 30.5 ft. to permit installation of the test pier beneath the weathered material within competent rock. A generalized description of soil materials and rock conditions encountered during drilling is summarized in Figure 2.

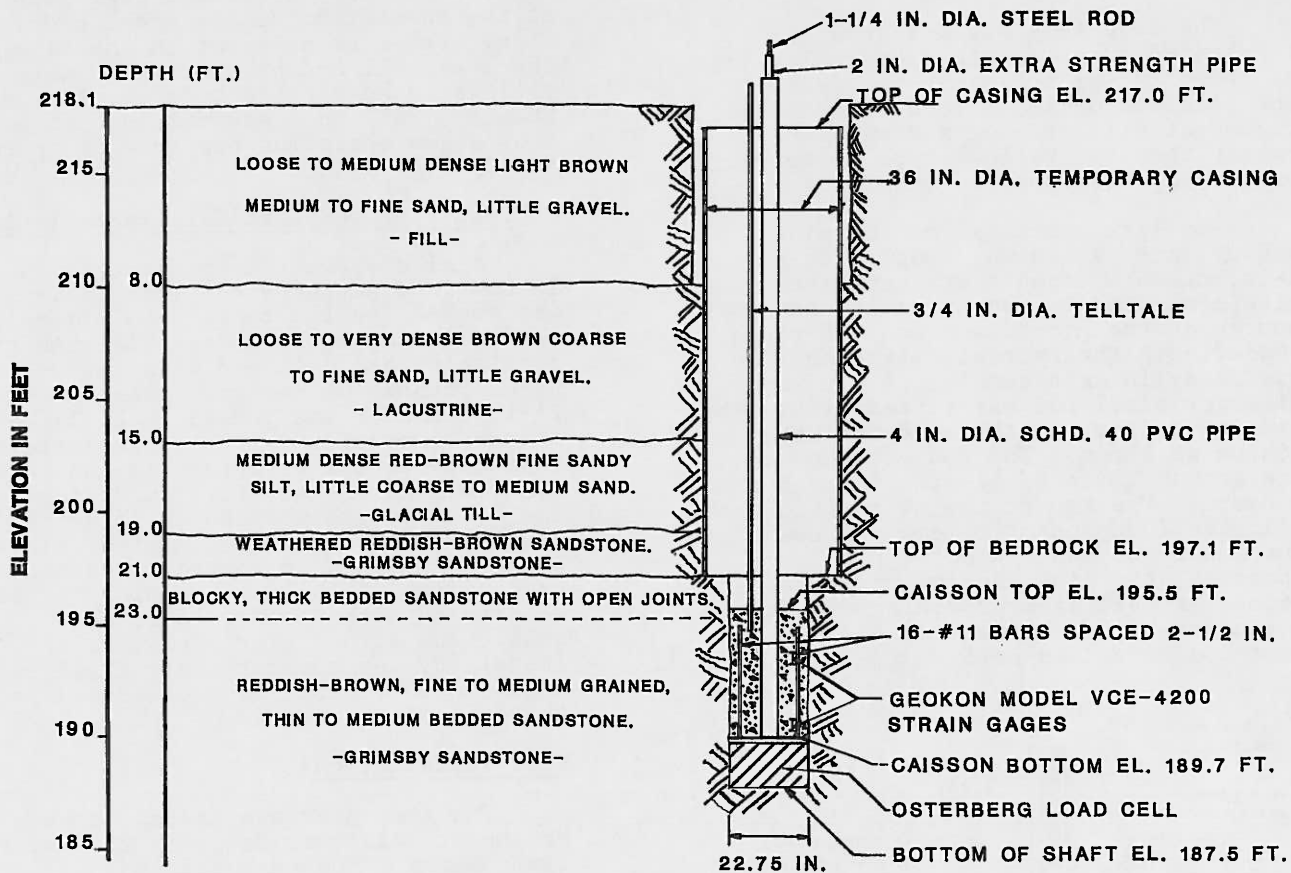


Figure 2. Generalized subsurface conditions and test pier installation details.

The Osterberg Load Cell, bearing plates, hydraulic lines, and telltales were assembled at the ground surface after drilling was complete. A 4-in. diameter PVC sleeve was placed over the hydraulic line to isolate it from the concrete forming the test pier. A reinforcing cage consisting of 16 No. 11 deformed bars was also assembled with the load cell equipment.

Two Geokon Model VCE-4200 vibrating wire strain gages were secured to the reinforcing steel to provide strain measurements within the test pier section during loading. The strain gages were located approximately 8 in. and 45 in. above the top of the load cell at the positions shown in Figure 2.

The drilled shaft was dewatered using a submersible pump, and the test assembly was lowered to the bottom of the borehole. High-early strength concrete was then placed above the load cell to a depth of 22.5 ft. below ground surface, providing an approximate 5.8-ft. column of concrete.

Test cylinders were prepared during concrete placement for subsequent compression testing. Compressive strengths of approximately 4500 psi were reported 12 days after the concrete had been placed.

Testing

The pier load test was conducted on 2 February 1988 by H&A of New York personnel, with assistance provided by representatives from Northeast Caissons and American Equipment and Fabricating Corp.

Prior to the start of testing, a reference beam consisting of a C4x5.4 channel section was installed to support the monitoring instruments. The beam was supported at two points sufficiently away from the temporary casing by 2-in. square steel tubing, driven approximately 5 ft. into the ground. One end of the beam was welded to the support, while the other end remained free, allowing the beam to expand or contract without buckling as the temperature varied.

An array of three Ames dial gages, capable of measuring to the nearest 0.001 in., were installed to monitor movements of the test pier and load cell. The dial gages were positioned as shown in the schematic diagram in Figure 3.

Dial Gage A was placed over the 2-in. diameter steel hydraulic line which served as a telltale to indicate upward movement at the top of the load cell and bottom of the pier.

Dial Gage B measured movements of the 1-1/4-in. diameter steel rod extending through the hydraulic line and attached to the base of the load cell.

Dial Gage C recorded movements of a 3/4-in. diameter telltale anchored 19 in. below the top of concrete, which reflected movement experienced near the top of the pier.

Strain and temperature data from the two strain gages were obtained using a Geokon Model GK401 vibrating wire gage readout and a Micronta digital volt/ohm meter. A third, null strain gage had been installed in a concrete cylinder cast at the same time the test pier was installed. The cylinder was lowered down the temporary casing and allowed to cure under the same conditions as the test pier. The null gage was used to verify that the monitoring equipment was functioning properly and to correct for any possible temperature fluctuations during the test.

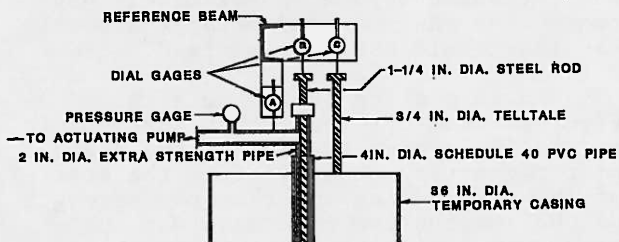


Figure 3. Schematic diagram of load test instrumentation.

An engineer's level was employed throughout the loading, to monitor movements of the reference beam and the temporary casing. No movements in the reference beam were detected.

The test was conducted by applying discrete load increments and monitoring movements until failure occurred. In general, loads were applied in 25-ton increments and maintained constant for approximately 20 to 30 minutes as deflections were monitored. The 225-ton load, which corresponded to the recommended allowable side resistance of 6.5 tsf, was maintained for one hour. After failure, loads were removed from the pier in 100-ton decrements with each load held constant for a 15-min. period.

Prior to actually conducting the test, two initial loading cycles were performed, first to approximately 50 tons and then to 100 tons in 25-ton increments, each time returning to a seating load of approximately 8 tons. These load cycles were performed to identify any seating error above or below the load cell due to incomplete cleaning or disturbance during installation.

All pertinent pressure gage, dial gage, and vibrating wire strain gage data were recorded during and after each load increment. Elevations of the reference beam and temporary casing were also checked after each load increment.

Load Test Results

Load test results are graphically summarized in Figures 4, 5, and 6. Figure 4 shows a plot of recorded pier and casing movements for each load increment. An enlargement of the data plotted in Figure 4 for the first 100 tons of loading is presented in Figure 5 showing the results of the two initial loading cycles. Strain data collected from the vibrating wire strain gages are plotted for each load increment in Figure 6.

As shown in Figures 4 and 5, a small amount of initial seating took place during the first load increment. The top of the load cell moved upward approximately 0.06 in. and the bottom moved downward approximately 0.07 in. before meeting with any substantial resistance. These initial movements were most likely due to imperfect cleaning of the borehole below the jack and slack space between the bearing plates and bottom of the test pier. Once the initial seating was established, however, movements of both the top and bottom of the pier continued at smooth, but steadily increasing rates until failure was reached.

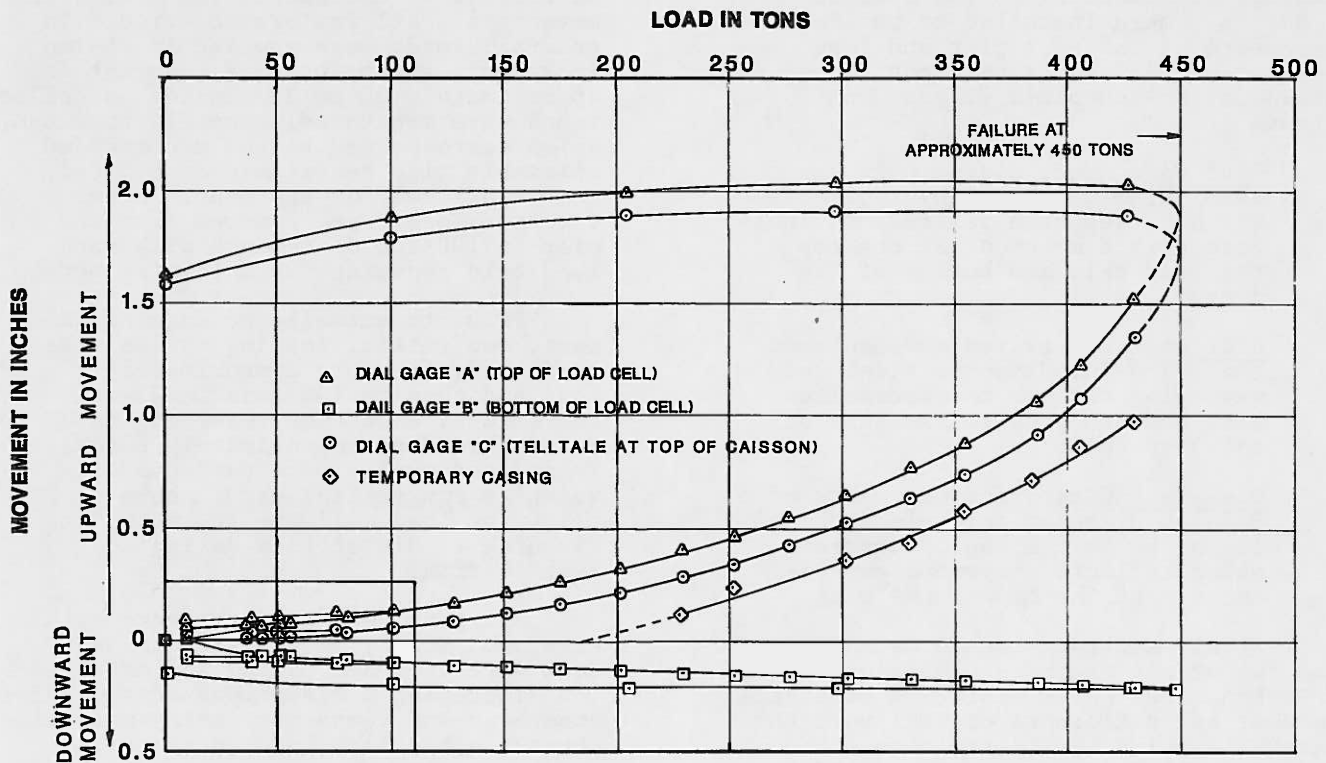


Figure 4. Telltale and casing deflections.

Downward movements at the base of load cell took place at a generally constant rate until the test was completed. A total of approximately 0.15 in. of movement was recorded after the initial seating load, of which approximately one half was recovered during unloading.

Upward movements of the test pier continued at generally increasing rates from the initial seating until the test neared failure. The rate of movement increased dramatically as the test load was increased beyond approximately 430 tons. The maximum load of approximately 450 tons could not be sustained.

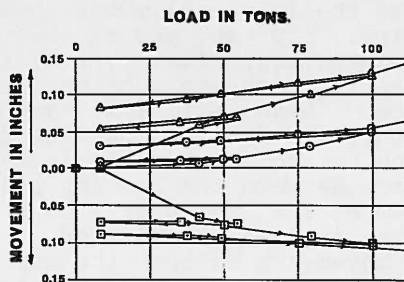


Figure 5. Telltale deflections during initial load cycles.

An inspection of the data in Figures 4 and 5 reveals that movement in the telltale near the top of the test pier began very shortly after the start of the test indicating that the strength of the surrounding rock mass was mobilized very quickly. This is further evidenced by the fact that the temporary casing began moving upward at some point between the time the 150 and 200-ton loads were applied. The precision of the measurements on the casing was approximately one to two orders of magnitude less than that of the telltales. Consequently the exact time at which the casing began moving could not be determined.

An initial divergence in the deflection curves for the upper and lower telltales can be detected in Figure 5,

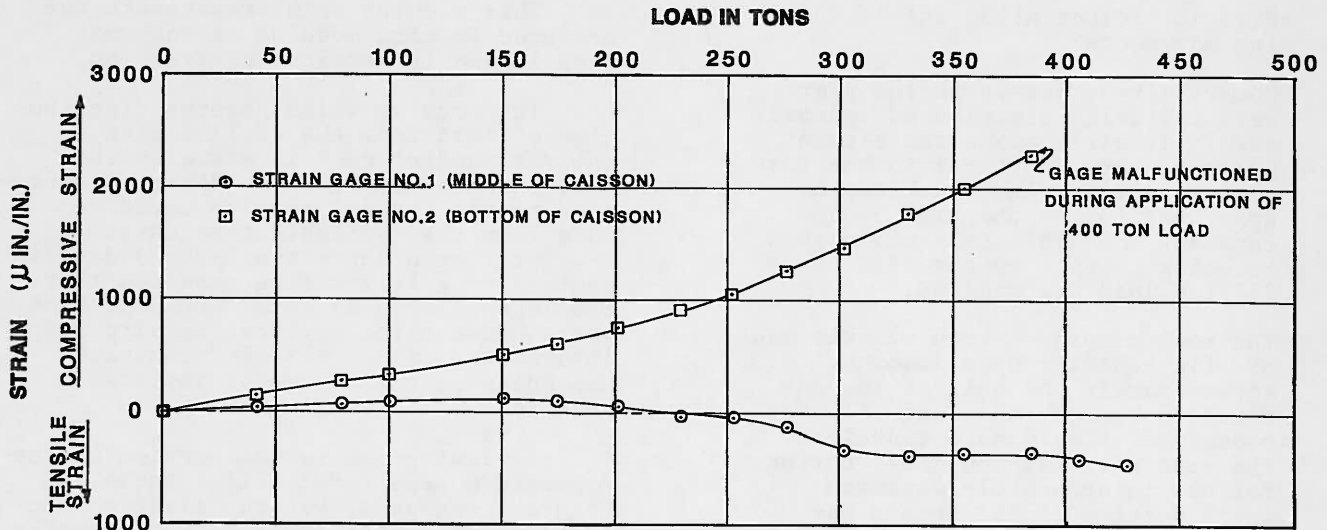


Figure 6. Vibrating wire strain gage measurements.

indicating an initial compression of the concrete pier element. This trend was observed as the test proceeded, as can be seen in Figure 4, up until approximately the point at which the 200-ton load was applied. From that time until the end of the test, the upper and lower telltales essentially moved in unison.

After the 200-ton load was applied, and movement in the casing was noted, the deflection curves for the pier telltales and the casing in Figure 4 are essentially parallel, suggesting that little or no movement between the concrete and surrounding rock was taking place. At about the point the test load reached 350 tons, movement of the test pier began to exceed that of the surrounding rock, indicating that the concrete to rock bond strength was being approached. Failure occurred when the 450-ton load was applied.

The strain gage data in Figure 6 generally support the observations made in the deflection data. Steadily increasing compressive strain was measured by the lower strain gage, located approximately 8 in. from the load cell. Compressive strains were initially detected in the upper strain gage, located approximately 45 in. above the cell. However, as the test proceeded, compressive strains in the upper portion of the pier began to decrease, and by the time the test load reached 225 tons, no compressive stress from the test load was being detected. As the test continued, the gage began to register tensile loads.

It can be noted that the reduction in compressive strains at the upper gage correlates closely with the start of movement of the temporary casing. The exact cause of the tensile strains is unclear. However, they may have resulted from the substantial movements of the upper rock mass in the late stages of the test.

In general, observations and conclusions made during the load test can be summarized as follows:

1. Some seating error was evident due to incomplete cleaning or slack in the loading system. A total of approximately 0.13 in. (0.06 top and 0.07 bottom) took place before the test pier was engaged.
2. The strength of the surrounding rock mass was mobilized very quickly, as evidenced by the movements of the temporary casing seated on rock.
3. Failure of the concrete rock bond occurred at a test load of approximately 450 tons which corresponds to an average side resistance of approximately 13 tsf over the entire test pier. However, if it is assumed that no stress was transmitted beyond the upper strain gage, the average effective side resistance was approximately 20 tsf.
4. The maximum bearing pressure exerted at the bottom of the load cell was approximately 161 tsf with a maximum deflection of approximately

0.15 in. (discounting initial seating movements).

5. Compressive stresses in the pier were initially distributed approximately linearly along the element. However, very shortly into the test compressive strains reaching the upper portion of the pier became constant and ultimately diminished, becoming tensile by the time the 250-ton load was applied.
6. The rock displayed both elastic and plastic behavior upon loading. Approximately one half of the deflection beneath the load cell was recovered. The upward travel of the test pier was too great during failure to accurately estimate which portion of the strain was elastic and which was plastic.

Design Implications

Based on the test results described above, it was apparent that both the allowable side resistance and end bearing pressures recommended during design (6.5 tsf and 55 tsf, respectively) were well below those implied by the load test, assuming appropriate factors of safety. Therefore, the pier design would appear to be adequate, although apparently not taking full advantage of the end bearing capacity that was available.

However, the test results also suggested that the modulus values of both the rock and concrete pier were lower than had been assumed for design. The average rock modulus was estimated to be approximately 350 ksi using the data from the end bearing portion of the test and the following equation suggested by Kulhawy et al (5):

$$E = \frac{Q (1 - \nu^2)}{s \beta \sqrt{A}}$$

Where: E = Elastic modulus

Q = Applied load

ν = Poisson's ratio

s = Deflection

β = Shape and rigidity factor (assumed 1.13)

A = Loaded area

This modulus value represents the measured in situ modulus of the rock when loaded in vertical compression.

The modulus which governs distribution of load from the drilled pier to the surrounding rock is actually the lateral rock modulus. No direct measurement of the lateral modulus could be made from the available test data. However, for a horizontally bedded sedimentary rock it would be expected that the lateral modulus value would be somewhere between the vertical in situ modulus and the intact elastic modulus, depending on the degree of vertical jointing.

Reviewing the reload portion of the downward movement deflection curve in Figure 5 suggests that the elastic modulus upon recompression was approximately 550 ksi. Therefore, the modulus value controlling distribution of load from the pier to the rock would be between 350 and 550 ksi. A value of 700 ksi had been assumed during design.

Data from the lower strain gage plotted in Figure 6 suggests that the concrete modulus for the test pier was approximately 1200 ksi. This value was determined assuming a load reduction of approximately 5 to 10 percent at the monitoring point, based on the results of studies by Kulhawy and Goodman (3) and Osterberg and Gill (5). This implied modulus was substantially below the 3500 ksi value assumed for design.

The reason for the anomalously low concrete modulus was not clear, although it may have been related to the placement technique or the relatively slow loading during the test. Unfortunately, no strain measurements were made when the test cylinders were broken to provide comparison of laboratory values to the field measurements.

The implication of these field modulus determinations on design were that 1) the modulus ratio between the concrete and rock was much lower than had been expected, assuming the measured concrete modulus was accurate, or more likely; 2) the concrete/rock modulus ratio was somewhat higher than anticipated and, therefore, the distribution of load to side resistance would be lower than assumed. Instead of 25 to 40 percent of the pier loads being taken in side resistance as previously assumed, it now appeared that only approximately 15 to 25 percent may be shed off, resulting in higher end bearing pressures.

After reevaluating stresses for the planned pier configurations, it was estimated that bearing pressures would range from approximately 60 to 72 tsf for the anticipated column loads with required side resistance ranging from approximately 2.7 to 3.5 tsf for piers socketed from 6 to 9.5 ft. below top of rock.

Although the end bearing pressures were higher than had been recommended for design, they were below the allowable value estimated from the load test, which was judged to be in excess of 80 tsf.

Summary

The load test conducted with the Osterberg Load Cell was highly successful. The test produced direct measurements of the maximum concrete/rock bond strength and load/deformation behavior for the rock mass. In addition, modulus values were obtained for both the concrete pier and rock mass which enabled the authors to more fully evaluate the performance of the proposed pier foundations.

The Osterberg Load Cell provided several distinct advantages over a conventional load test.

1. The device was relatively easy to install, instrument, and operate. Installation was much less involved than a conventional test setup, resulting in a lower cost to the client. The cost of the test was approximately one half the price bid by the same contractor for a more conventional pull-out test.
2. No reaction piers or load frame were required, which makes the device suited for difficult access sites such as over water.
3. Direct and distinct measurements were obtained for both the end bearing and side resistance portions of the test load. Deformation measurements were also made that allowed determination of the in situ rock modulus.
4. The loading mechanism resulted in more accurate determination of side resistance than a pull-out because Poisson's effects under this loading configuration were consistent with those for the actual foundation elements.

This test represented the second time an Osterberg Load Cell had been used to test a foundation element for an

actual project. The first time was the Saugus River project mentioned earlier. At the completion of this load test, Northeast Caissons successfully retrieved the load cell by coring and drilling out the concrete pier element. After retrieval, the cell was shipped back to American Fabricating, where it was cleaned up and fitted with a new seal in preparation for use on a second pile load test for a bridge replacement on the Pines River, also near Lynn, MA.

The load cell was fitted to an 18-in diameter pipe pile that was driven to end bearing at a total depth of approximately 140 ft. The cell was subsequently used to load test the pile to failure in side friction at a load of approximately 215 tons.

At both of the pile load test locations, the cells were grouted in place and the test piles were used for production piles.

Since the time this test was completed, American Fabricating has assembled two bellows type cells which were used to test 36-in. and 54-in. diameter drilled shafts on a Florida Department of Transportation and Federal Highway Administration project at Port Orange, FL. (6).

With increased experience using the device and the many advantages offered by it, the Osterberg Load Cell should prove to be a valuable tool for drilled pier designers.

Acknowledgments

The load test described was performed while under contract to CRS Serrine, Inc., who were the project designers. The Eastman Kodak Company is the owner of the facility.

Appreciation is expressed to Mr. Charles Guild of American Fabricating and Equipment Corp. and Dr. Jorg Osterberg who contributed during the planning and conduct of the test and preparation of this manuscript.

References

1. Deere, D.U., Merritt, A.H. and Coon, R.F., "Engineering Classification of In Situ Rock," Technical Report No. AFWL-TR-67-144, Air Force Weapons Laboratory, Kirtland AFB, New Mexico, 279p., 1969.
2. Kulhawy, F.H. and Goodman, R.E., "Design of Foundations on Discontinuous Rock, Vol. 1, Sydney, May 1980, pp. 209-220.

3. Osterberg, J.O., "A New Simplified Method for Load Testing Drilled Shafts," Foundation Drilling, Vol.23, No.6, August 1984, pp.9-11.
4. Presentation by G.L. Guild at a Transportation Research Board Meeting in Washington, D.C., January 1988.
5. Osterberg, J.O. and Gill, S.A., "Load Transfer Mechanism for Piers Socketted in Hard Soils or Rock," Proceedings, 9th Canadian Symposium on Rock Mechanics, Montreal, Quebec, November 1973, pp.235-262.
6. Osterberg, J.O., "New Device for Load Testing Driven Piles and Drilled Shafts Separates Friction and End Bearing," paper presented at the International Conference on Piling and Deep Foundations, Deep Foundation Institute, London, May 1989.

STABILIZATION OF A COAL DUMPING HIGHWALL
USING A TIED-BACK STRUCTURE

by:

Gary T. Brill
ERC/EDGE, Inc.
Knoxville, Tennessee

Bill Wells
Mingo Logan Coal Company
Wharnccliffe, West Virginia

Abstract. At a large coal preparation complex being operated by Mingo Logan Coal Company in Mingo County, West Virginia, raw coal will be stockpiled by dumping from trucks at the top of a 45 feet highwall. The coal will be removed as needed from the stockpile through a reclaim tunnel at the base of the highwall. During site grading, the upper 20 to 25 feet of the highwall was badly broken by blasting and a bench was excavated in the highwall in an attempt to remove the majority of the broken rock. However, the remaining bench was not suitable for founding the necessary retaining structure to rebuild the highwall. In order to achieve the raw coal stockpiling capacity required at the complex, the highwall was rebuilt to the required configuration using a tied-back structure. The tied-back structure consisted of steel soldier beams with concrete panel lagging, and was supported by a series of bar and tendon anchors. Presented herein is a case history of the tied-back structure. Details of the design and construction, including bar and tendon anchor design and load testing results are included.

Introduction

The efficient operation of a coal processing plant requires the use of a raw coal stockpile to enable coal mining and processing to operate independently of each other. Because a constant raw coal feed is required for the plant to operate, a reliable, low maintenance method of coal removal from the stockpile is also essential. In the mountainous terrain of the Appalachian coal fields, a common method of raw coal stockpiling/handling is to dump raw coal from the top of a vertical excavation (highwall) and to remove coal as needed from the base of the highwall by means of a conveyor installed within a reclaim tunnel.

At the mining complex of Mingo Logan Coal Company, located in Mingo County, West Virginia, coal will be mined at various locations and trucked to a central handling facility. During the initial phase of the project, raw coal from strip mining operations will be crushed and screened before being loaded onto rail cars. During a later phase, a preparation plant will be built to process coal from deep mining operations.

During both phases of development, raw coal will be stockpiled adjacent to a highwall and it will be removed through a reclaim tunnel at the base of the highwall. The highwall ranges in height from about 40 to 45 feet and is approximately 370 feet long. A plan view of the limits of the highwall and reclaim tunnel are shown on Figure 1.

Blasting damage of the upper portion of the weathered bedrock of the highwall made it unusable as a truck dump facility. In order to allow efficient use of the originally proposed stockpile area, the highwall needed to be rebuilt to its original location. This paper presents a case history of the design and construction of a permanent tied-back wall structure which was used to stabilize the highwall.

Subsurface Conditions

Geologic Setting

The area of the highwall is located within the Allegheny Plateau Physiographic Province. The site is underlain by Pennsylvanian Age

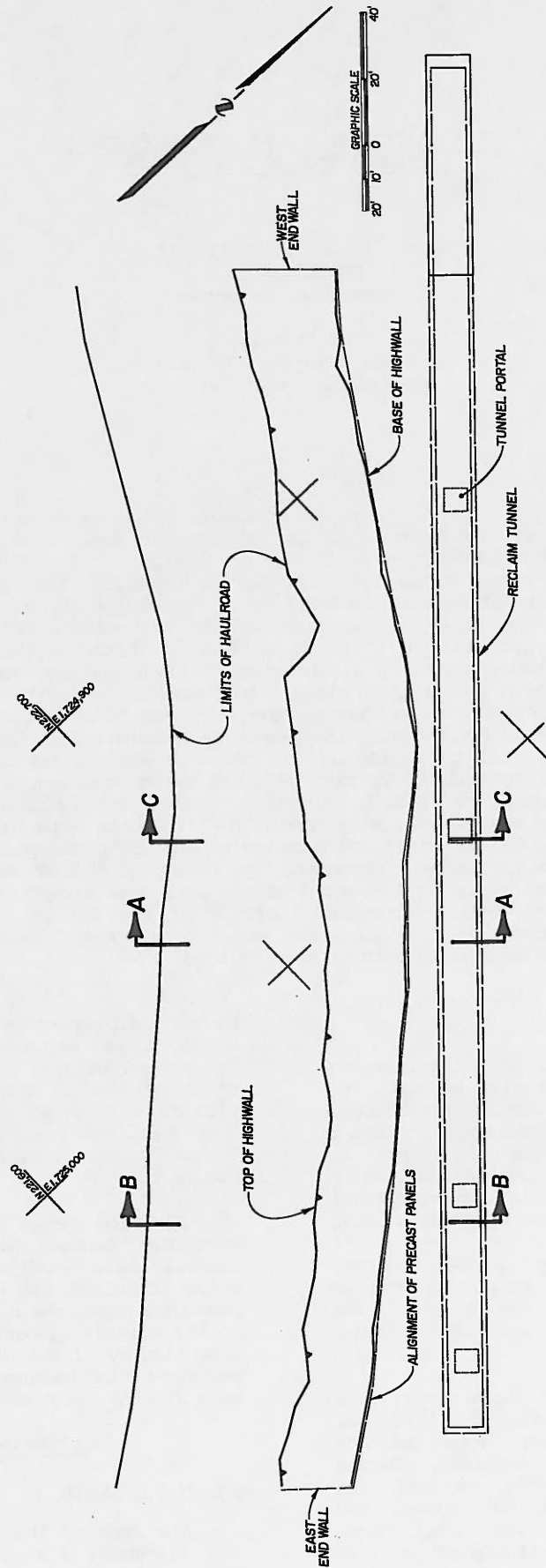


Figure 1. Plan view of the limits of the highwall and reclaim tunnel.

bedrock of the Kanawha Formation and consists of a thick sequence of interbedded sandstone, shale and siltstone, with several significant beds of coal. The bedrock strata are essentially flat-lying, having an average regional dip toward the northwest of about 1.5°, with no evidence of faulting shown on the published geologic map of the area.

Bedrock/Highwall

The bedrock in the area of the highwall consists of interbedded sandy shales and shaley sandstones along with two thin coal seams. Several high angle joints are located along the highwall and generally have a dip of about 86°. During site grading, the upper 20 to 25 feet of the highwall was badly broken by blasting and a portion of the material had to be removed for safety reasons resulting in a 15 to 20 feet wide bench in the highwall as shown on Figure 2. The material remaining on this bench consists of severely fractured shale debris ranging from large wedges of intact rock to highly weathered soil-like material. The majority of the lower 20 to 25 feet of the highwall consists of moderate strength, massive sandy shale on a near vertical slope as shown on Figures 3 and 4. Portions of the highwall are not only severely broken in its upper portion, but have up to a 20 degree sloped rock face where a wedge of rock has fallen at the surface contact of a high angle joint.

The shale exposed in the highwall is underlain by medium hard to hard sandstone. The reclaim tunnel is situated within an excavation in the sandstone as shown on the generalized geologic cross-section (Figure 5) of the highwall area.

Ground Water

The ground water table in the area of the truck dump wall is generally below the base of the highwall. However, some ground water flow was observed along some of the joints in the highwall and within the coal seams. The flow appeared to be perched and highly dependent upon rainfall amounts.

Design Considerations

As stated previously, raw coal will be stockpiled by dumping from trucks at the top of the 45 feet highwall. The coal will be removed as needed from the stockpile through a reclaim tunnel at the base of the highwall. In order to achieve the raw coal stockpiling capacity required at the complex, the highwall had to be re-built to its original location.

Design and construction of the dumping wall was complicated by other considerations which include:

- (1) Storage capacity was desired along the full length of the highwall;
- (2) The highwall was continuously turning along its full length;
- (3) The remaining highwall was heavily jointed and the bench at the upper portion of the highwall was not suitable for founding a structure;
- (4) The highwall extension would have to support the weight of fully loaded 90 ton gross

weight coal trucks in addition to any structural weight;

- (5) The reclaim tunnel at the base of the highwall was not designed to withstand increased vertical or lateral loading from any new structure; and,
- (6) The effects of corrosion would have to be considered in any new construction because the area would be periodically covered in coal.

Under these conditions, in conjunction with economic considerations, a continuous tied-back retaining wall was selected as the most attractive alternative. Details of the design and construction of the selected alternate are included in the following sections.

Tied-Back Retaining Wall Design

As shown on Figures 6 and 7, steel I-beams supported by rock anchors will brace the lower portion of the existing exposed highwall. The I-beams will extend to the desired dumping level and will be supported near the top of the wall using tied-back tendon anchors. Pre-cast concrete panels installed at the upper portion of the wall will enable the dumping location to be re-built by backfilling. The wall will derive its support laterally through the anchors, utilizing the strength of the existing intact portion of the rock highwall such that no significant load from the structure will be transferred to the walls of the tunnel.

In order to estimate the loads which would be imparted to the various anchors and beam system, earth pressure diagrams were developed for two cases:

- (1) A partially backfilled wall as shown on Figure 6; and,
- (2) A fully backfilled wall as shown on Figure 7.

To estimate the lateral loading effects of the backfill on the wall, a rectangular apparent earth-pressure diagram was used based on the following formula for open cuts in sand:¹

$$P = 0.65K_A \gamma H \quad (1)$$

where:

P = Lateral Earth Pressure

K_A = The Rankine Active Earth Pressure Coefficient

γ = Moist Unit Weight of Backfill Material

H = Height of Wall

Values of K_A for the backfill and the bedrock were calculated using estimated angles of internal friction of 36° for the backfill and 60° for the jointed and fractured rock.

To estimate the lateral loading effects of the 90 ton coal trucks on the wall, equations developed by Boussinesq and modified by experiment for estimation of horizontal pressures from a point load were used.² Impact loads were estimated to be as much as 25% of the weight of a truck. The cumulative loading effects of the backfill and coal trucks, in conjunction with a selected beam section spacing of 10 feet, were used to develop total pressure diagrams for both cases.

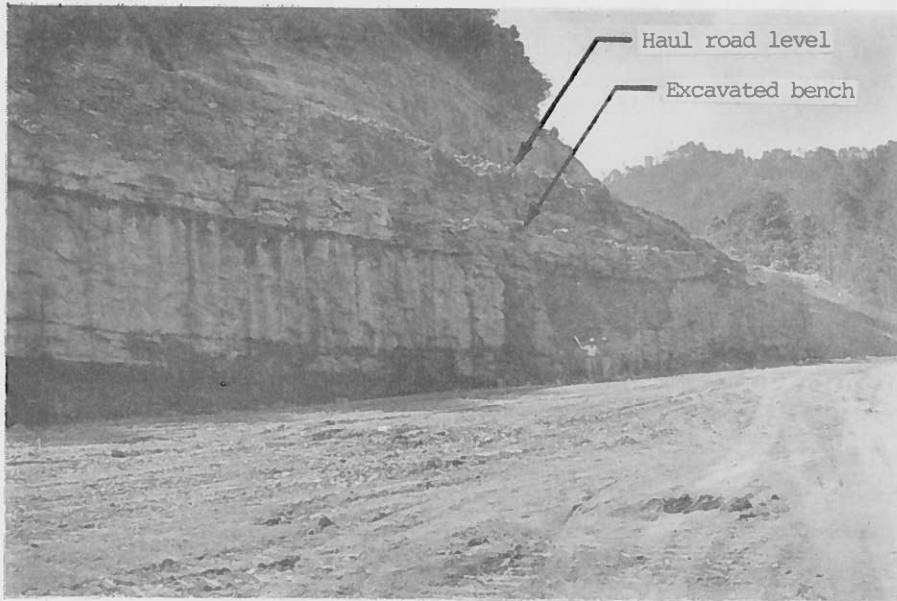


Figure 2. Photograph of highwall showing haul road and excavated bench.

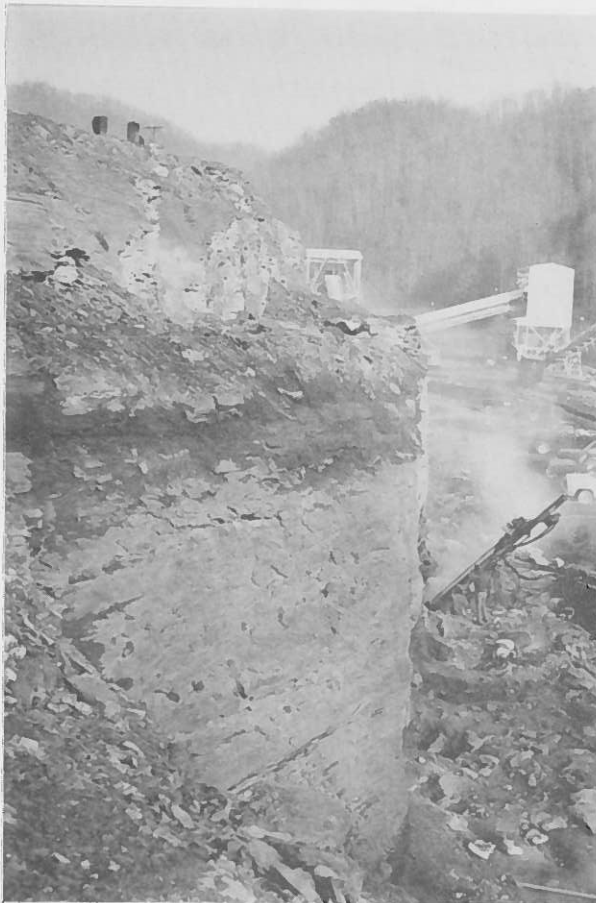


Figure 3. Photograph of excavated bench above intact portion of highwall.

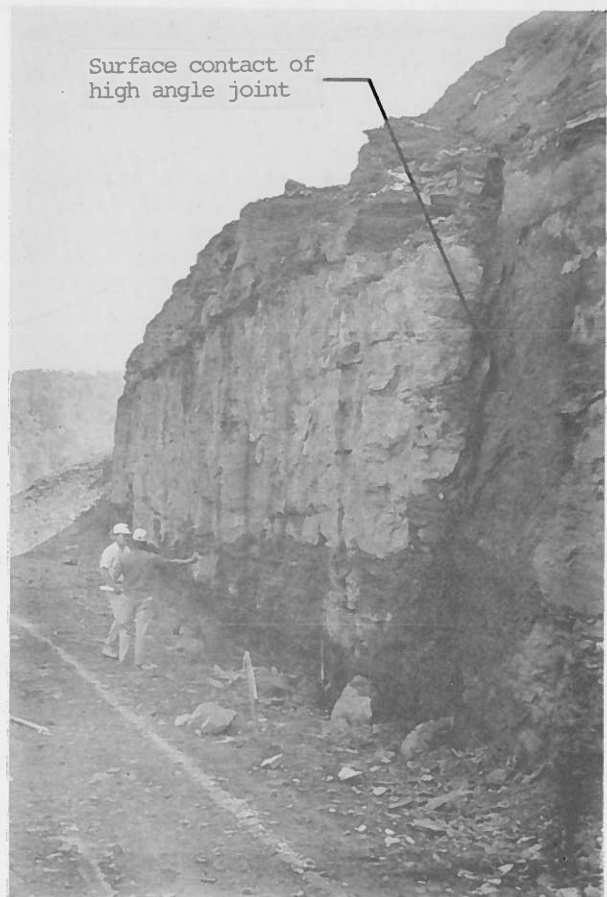


Figure 4. Photograph of intact portion of highwall.

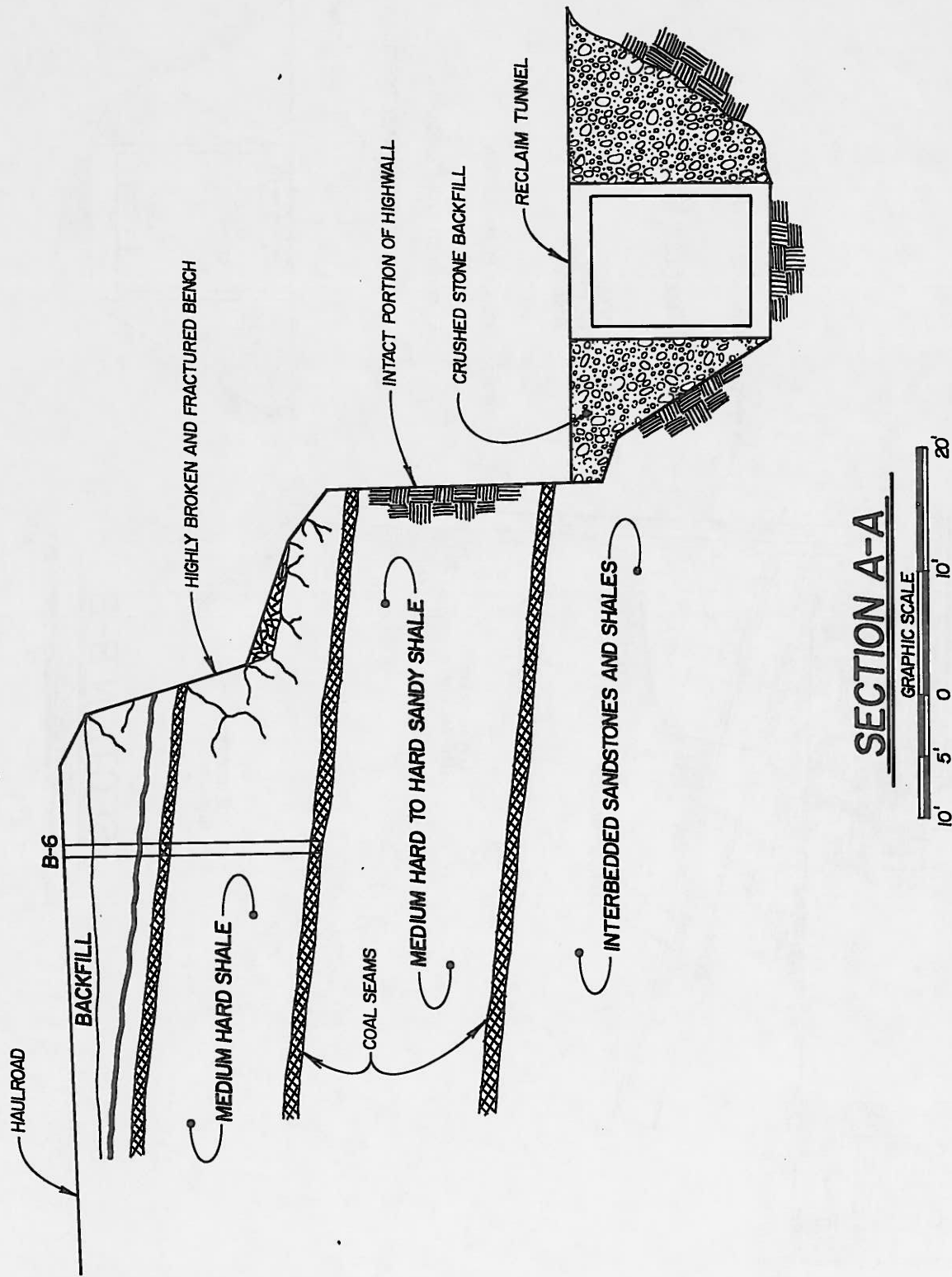


Figure 5. Generalized geologic Section A-A through highwall.

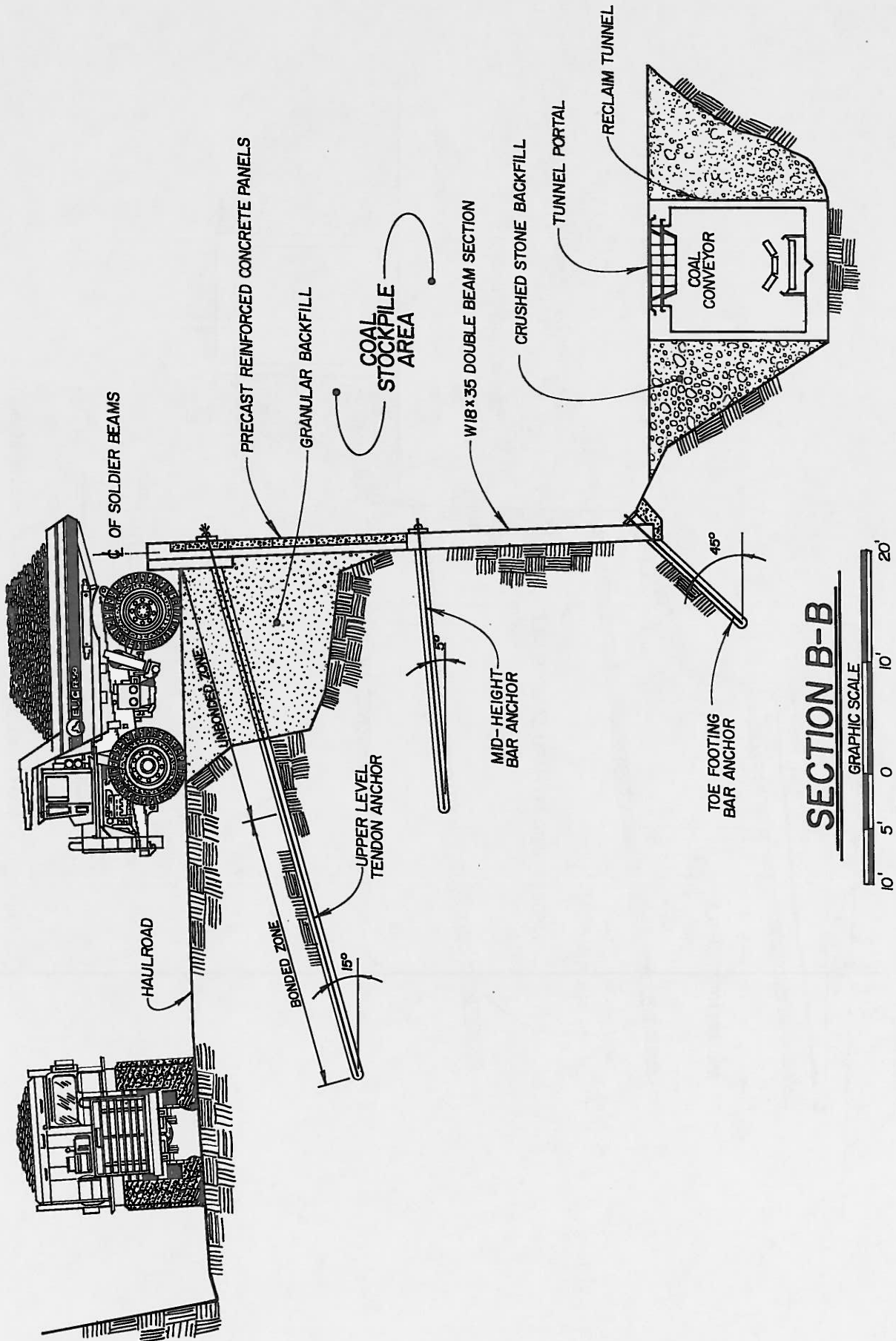


Figure 6. Typical partially backfilled tied-back wall section.

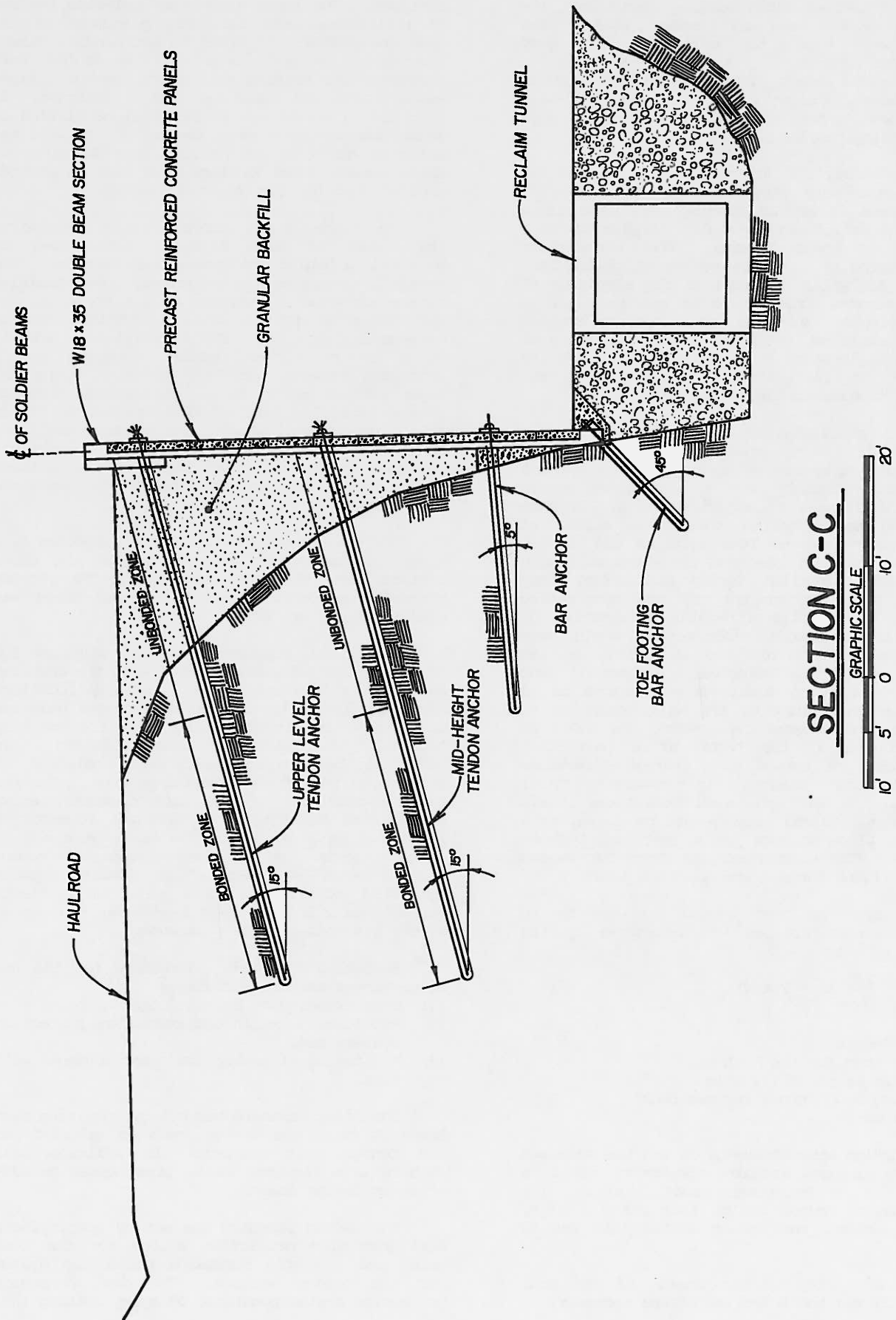


Figure 7. Typical fully backfilled tied-back wall section.

The two wall cases were then analyzed using a computer program CBEAMC, developed by the U.S. Army Corps of Engineers Waterways Experiment Station.³ Based on input loading conditions, the program provides beam and anchor loads. The program was used to estimate the most advantageous spacing of anchors, not only in terms of anchor loads, but also bending moments in the beam sections. In addition, end-wall sections at either end of the highwall were designed using similar methods.

In general, the selected anchor spacing for the two cases are shown on Figures 6 and 7. Minor changes in anchor spacing were made along the dumping wall due to variable field conditions not shown on these figures. These conditions were estimated by taking a series of photographs along the highwall, and scaling the elevation of various features from a stadia rod included in the photographs. Surveyed cross-sections coupled with the detailed geologic assessment at each soldier beam location were provided to allow the contractor to fabricate the majority of the structural components in the shop.

The traditional method of building a tied-back wall using steel beams and concrete panel lagging consists of spacing single H-beams in a nearly straight row and dropping panels between the flanges of adjacent beams. Anchors would then be installed through a series of walers spanning between beam sections and through the concrete panels. Because tied-back walls are traditionally installed during excavation, they are built beginning at the top and progressing downward. Due to the irregular orientation of the existing highwall, this method would have been extremely difficult and expensive, and an alternate method was selected. The method used consisted of placing double beam sections on 10 foot centers and building the wall beginning at the bottom and progressing upward. As shown on Figures 8 and 9, the beams at a particular section could be turned to different directions to accommodate changes in concrete panel directions. In addition, a minimum space of six inches was maintained between the two beams at a section to allow anchors to be installed between the beams. This eliminated the need for walers and for drilling through the concrete panels.

The required bond length for anchors to develop their capacities was estimated by the following equation:

$$BL = P/\pi(d)\tau_w \quad (2)$$

where:

BL = Bond Length

P = Design Load for the Anchor

d = Diameter of the Drill Hole

τ_w = Working Bond Stress Between Rock and Grout

A working bond stress of 50 psi was selected for design purposes assuming the anchor would be installed in a relatively soft shale.⁴ The maximum tendon anchor design load was 138 kips, while the maximum bar anchor design load was 69 kips.

The next step in the design of the coal dumping wall was the selection of the necessary

structural components. Based upon the maximum bending moments from the computer analysis, W18X35 beams were selected for the double beam sections. The beams were also selected because of their deep webs for easy placement of the concrete panels. In order to accommodate direct bearing of anchor loads on the double beam sections, web bearing stiffeners, anchor plates, wedge plates and field welds were designed. In addition, a continuous bumper rail consisting of W12X26 beams connected at the top of the wall was selected not only to provide some transfer of loads between beam sections, but also to provide a safety barrier for the coal trucks.

The highwall was designed to be rebuilt to the level of the adjacent haul road by backfilling behind the double beam sections. The backfill consisted of relatively free draining bottom ash which classified as a silty sand, and was compacted to 95% of its standard Proctor maximum dry density. The backfill was held in place by reinforced precast concrete panels installed between the beam sections along the upper portion of the wall. To increase drainage in the backfill, panel drains wrapped in filter fabric were placed between the precast panels and the backfill. In addition, a french drain was designed to be placed at the interface between the backfill and the rear end of the excavated highwall bench.

The coal dumping wall will be subjected to a highly corrosive environment due to the close contact with the coal stockpile. To provide corrosion protection, all structural steel was coal tar epoxy coated.

The final components to be selected for design of the coal dumping wall were the anchors. As shown on Figures 6 and 7, at the locations along the length of the beams where the beam was in or near to direct contact with the existing highwall, bar anchors were selected. In addition, bar anchors were also selected to anchor the toe of the beams into rock. The bar anchors consisted of one inch diameter, epoxy coated rock bolts with an ultimate strength of 150 ksi. In areas where the beams were not in contact with the existing highwall, tendon anchors were selected. The tendon anchors consisted of four tendons with an ultimate strength of 270 ksi. Each tendon was made up of seven, 0.6 inch diameter, strands.

Recommended industry standards for the use of permanent anchors includes:⁵

- (1) Overstressing of selected test anchors;
- (2) Providing a stringent corrosion protection system; and,
- (3) Monitoring of anchor loads and movement with time.

The first standard was met by requiring test loads of twice the design loads on selected bar and tendon test anchors. In addition, all anchors were required to be prestressed to 133% of their design loads.

The second standard was met by specifying a dual corrosion protection system for the rock bolts and a triple corrosion protection system for the tendon anchors. The dual corrosion protection system consisted of epoxy coating the



Figure 8. Photograph of erected double beam sections and initial precast concrete panel placement.



Figure 9. Photograph of backfilling operations behind wall and coal pile access in front of wall.

rock bolts and surrounding their entire lengths with corrosion resistant cement grout. The triple corrosion protection system consisted of surrounding the strand tendons with corrosion resistant cement grout contained in a corrugated polyethylene sheath which was also surrounded by cement grout. The corrosion protection systems provided adequate protection in the bonded length of the anchors. However, the greatest potential hazard of anchor corrosion is in the unbounded length of the tendon, and at the interface between the anchor head and the start of the anchor drillhole.⁶ Therefore, the tendon anchors were protected in their unbounded length with an extruded polypropylene sheath filled with corrosion inhibiting grease and surrounded by corrosion resistant grout up to the anchor heads. Grease-filled anchor caps were then placed on the exposed anchor ends after stressing operations were completed.

The third standard was met by requiring lift-off tests on all anchors after the prestressing operations were completed to determine the amount of stress relaxation that was occurring on individual anchors.

Construction

The initial phase of construction consisted of excavating a slot into rock at the base of the highwall at each beam location and pouring a footing and leveling pad for each beam. Boreholes for installation of the toe anchors were then drilled to a depth of 10 feet using a rotary-percussion drill rig. The rock bolts were then inserted in the boreholes and grouted in place using a cement grout with a minimum compressive strength of 3500 psi.

Due to the highly variable geometry of the highwall, the most difficult phase of construction was the alignment and erection of the double beam sections. In order to maintain the necessary minimum spacing between sections and also allow enough clearance to place the precast concrete panels, the beams were connected with guide struts of the same width as the panels, and erected as units. The result was increased preparation on the ground, but reduced adjustments by crane and assurance that panel placement would be successful.

After the beams were erected, the toe anchors were brought into bearing against the beams and stressed to their design loads. Concrete was then formed behind the beams in near vertical areas of the highwall where the flanges of the beams were not in complete contact with the rock. Precast concrete panels were then placed at the base of the fully backfilled portions of the wall up to the level of the second row of anchors as shown on Figure 8, and backfilling operations were initiated.

At this point in construction, access for drilling and anchor testing equipment would have been very difficult if not for the dual purpose of the wall area. Since the area in front of the wall would eventually be used as a coal stockpile area, coal was brought in during wall construction and used as access for installation of the rows 2, 3 and 4 anchors. The remaining construction consisted of raising the coal pile

to the various anchor levels and installing anchors while backfilling behind the wall as shown on Figure 9.

The most crucial portion of the work for construction of the coal dumping wall was the installation and stressing of the anchors. Two test anchors were installed and stressed to 200% of their design load in order to verify the estimated grout-rock bond stress in the drill hole. The anchors were subjected to repeated increased on-off loading such as would be applied by the raising and lowering of the coal pile and by the loaded coal trucks. The anchor and wall movements which occurred during these loadings were monitored over a period of time as shown on Figure 10 to determine a range of anticipated anchor behavior. The remaining production anchors were then stressed to 133% of their design load. The load-displacement results for the production tendon anchors are shown in Figure 11. All of the anchors were locked-off to 70% of their estimated dead loads after prestressing operations were completed. In addition, lift-off tests were performed on the anchors to determine the amount of stress relaxation that was occurring on individual anchors. If the loads measured during the lift-off tests varied by more than 10% from the 70% lock-off load, then the anchor load was readjusted. The major advantages of prestressing the production anchors were to keep future in-service displacements to a minimum and enable unacceptable anchors to be detected by comparison with the test anchor behavior. A total of 54 tendon anchors and 79 bar anchors were installed for the project. In addition, since the production anchors and test anchors were required to be stressed well above the dead loads exerted by the backfill, substantial temporary bracing of the wall was necessary to keep wall movements to a minimum. None of the anchors failed nor exhibited excessive deflection during stressing. Figure 12 shows the completed wall after several cycles of stockpiling and removal of coal.

Conclusions

The construction of a tied-back structure can be a cost effective, low maintenance alternative for rebuilding a highwall used as a coal dumping wall. The use of permanent anchors for a tied-back structure requires stringent testing and corrosion protection requirements. Furthermore, providing adequate temporary bracing of tied-back walls with anchors that are stressed above in-service loading is required in evaluating load test results and preventing damage to the wall during stressing. Finally, when building a tied-back retaining wall, work that can be accomplished in the shop or prior to erection of structural components can provide an increased time and cost savings during construction.

Acknowledgements

The general contractor for the structure was the G.E. Ray Division of Danis Industries Corp., Charleston, West Virginia.

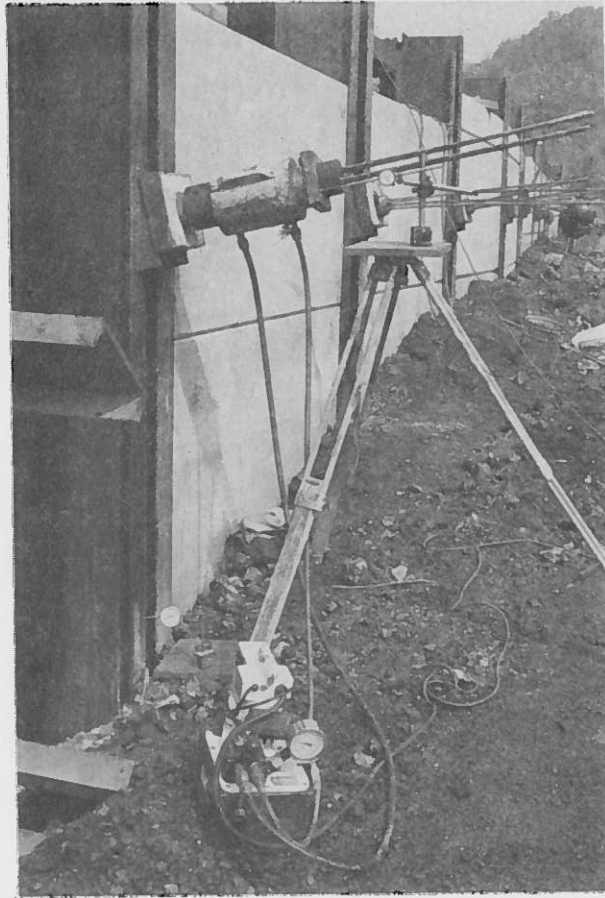


Figure 10. Photograph of stressing of test tendon anchor to 200% of its design load.

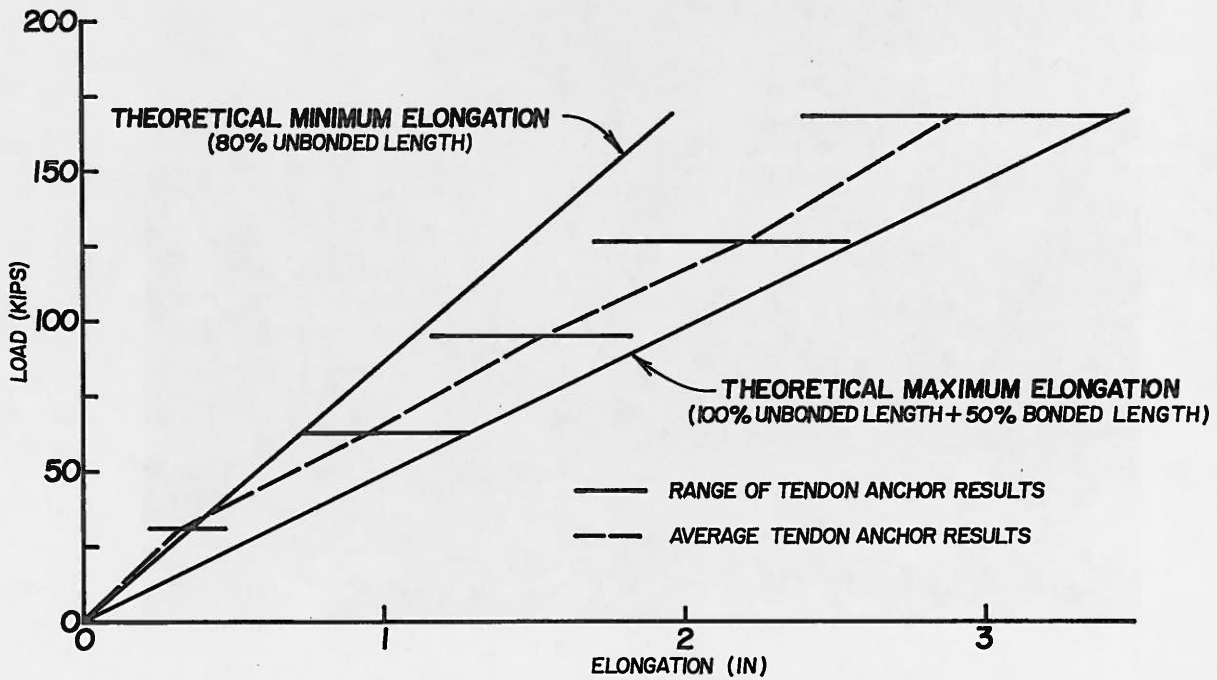


Figure 11. Load versus displacement results for production tendon anchor.

References

1. K. Terzaghi and R.B. Peck, "Soil Mechanics in Engineering Practice", John Wiley and Sons, Inc., New York, NY, 1967.
2. NAVFAC DM-7, "Design Manual-Soil Mechanics, Foundations, and Earth Structures", Department of the Navy, Naval Facilities Engineering Command, Alexandria, Virginia, 1971.
3. William P. Dawkins, "Computer Program for Analysis of Beam-Column Structures with Nonlinear Supports (CBEAMC)", U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1982.
4. PTI Committee on Rock and Soil Anchors, "Recommendations for Prestressed Rock and Soil Anchors", Post-Tensioning Institute, USA, 1986.
5. T.H. Hanna, "Foundations in Tension - Ground Anchors", McGraw-Hill Book Company, USA, 1982.
6. D.E. Weatherby, "Tiedbacks", U.S. Department of Transportation, Federal Highway Administration Report No. FHWA/RD-82/047, Washington, DC, 1982.



Figure 12. Photograph of completed tied-back coal dumping wall.

APPENDIX

Past Proceedings of
Ohio River Valley Soils Seminars

- ORVSS I: BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, Kentucky
- ORVSS II: EARTHWORK ENGINEERING, START TO FINISH October 15, 1971, Louisville, Kentucky
- ORVSS III: LATERAL EARTH PRESSURES, October 27, 1972, Fort Mitchell, Kentucky
- ORVSS IV: GEOTECHNICS IN TRANSPORTATION ENGINEERING, October 5, 1973, Lexington, Kentucky
- ORVSS V: ROCK ENGINEERING, October 18, 1974, Clarksville, Indiana
- ORVSS VI: SLOPE STABILITY AND LANDSLIDES, October 17, 1975, Fort Mitchell, Kentucky
- ORVSS VII: SHALES AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN AND CONSTRUCTION, October 8, 1978, Lexington, Kentucky
- ORVSS VIII: EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION, AND PERFORMANCE, October 14, 1977, Louisville, Kentucky
- ORVSS IX: DEEP FOUNDATIONS, October 27, 1978, Fort Mitchell, Kentucky
- ORVSS X: GEOTECHNICS OF MINING, October 5, 1979, Lexington, Kentucky
- ORVSS XI: EARTH PRESSURES AND RETAINING STRUCTURES, October 10, 1980, Clarksville, Indiana
- ORVSS XII: GROUNDWATER: MONITORING, EVALUATION, AND CONTROL, October 9, 1981, Fort Mitchell, Kentucky
- ORVSS XIII: RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, Kentucky
- ORVSS XIV: FOUNDATION INSTRUMENTATION AND GEOPHYSICAL EXPLORATION, October 14, 1983, Clarksville, Indiana

APPENDIX (cont'd)

- ORVSS XV: PRACTICAL APPLICATION OF DRAINAGE IN
GEOTECHNICAL ENGINEERING, November 2, 1984,
Fort Mitchell, Kentucky
- ORVSS XVI: APPLIED SOIL DYNAMICS, October 11, 1985,
Lexington, Kentucky
- ORVSS XVII: NATURAL SLOPE STABILITY AND INSTRUMENTATION,
October 17, 1986, Clarksville, Indiana
- ORVSS XVIII: LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING
AND CONSTRUCTION, November 6, 1987, Fort
Mitchell, Kentucky
- ORVSS XIX: CHEMICAL AND MECHANICAL STABILIZATION OF SOIL
SUBGRADES, October 21, 1988, Lexington,
Kentucky