REPORT NO. UMTA-CA-06-0122-79-1

FACTORS INFLUENCING THE PERFORMANCE OF FULL FACE HARD ROCK TUNNEL BORING MACHINES

GREGORY E. KORBIN
UNIVERSITY OF CALIFORNIA, BERKELEY
COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

AUGUST 1979

FINAL REPORT

Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161.

PREPARED FOR

U.S. DEPARTMENT OF TRANSPORTATION
OFFICE OF SECRETARY AND
URBAN MASS TRANSPORTATION ADMINISTRATION
WASHINGTON D.C. 20590

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
UMTA-CA-06-0122-79-1		

4. Title and Subtitle	5. Report Date
Factors Influencing the Performance of Full Face Hard Rock Tunnel Boring Machines	March 1979
	6. Performing Organization Code
	8. Performing Organization Report No.
7. Author(s) Gregory E. Korbin	
9. Performing Organization Name and Address University of California 440 Davis Hall Berkeley, CA 94720	10. Work Unit No. (TRAIS)
	11. Contract or Grant No. DOT-OS-70031
12. Sponsoring Agency Name and Address U. S. Department of Transportation Office of the Secretary and Urban Mass Transportation Administration Washington D.C. 20590	13. Type of Report and Period Covered Final Report Sept. 1, 1977-Feb. 30, 1979
	14. Sponsoring Agency Code

15. Supplementary Notes

Technical Monitor: Russell K. McFarland

16. Abstract

This report considers the major factors influencing full face hard rock tunnel boring machine (TBM) performance, both in terms of machine boreability (rock-tool interaction) and machine utilization. It is not intended to encompass all aspects of machine perfomance, but to concentrate on the principle elements affecting penetration rate and tool wear, and their methods of prediction, as well as the geologic factors influencing utilization. The importance of machine design and the interrelation with the tunnel environment is emphasized. Ability to place support and/or reinforcement close to the tunnel face, maintain machine stability, and cope with excessive ground water has a major effect on overall progress. Aside from information derived from field investigations, basic points are illustrated with case histories from the literature. Rock-tool interaction is reviewed to assess the major factors influencing TBM boreability from three separate, although, related methods. First, numerical models are considered with a view to their predictive potential and the foundation for a theoretical understanding of tool interaction. Second, physical models simulating the excavation process and prototype machine behaviour are discussed. These laboratory and field studies represent the major source of information from which a fundamental and practical understanding of rock-tool-machine interaction has evolved. Finally, the indices and index tests widely employed for TBM boreability predictions are evaluated.

17. Key Words	18. Distribution Statement
Tunnel, Boring, Machine, Design, Models, Rock, Excavation	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	263	

Form DOT F 1700.7 (8-72)

Reproduction of completed page authorized

PREFACE

The work reported herein has been sponsored by the US Department of Transportation, Office of the Secretary and Urban Mass Transportation Administration. It was carried out at the Tunnels Division of the Transport and Road Research Laboratory (TRRL), England, between Fall 1977 and Winter 1979 while participating in the US DOT/TRRL tunnelling research and development exchange program. Field investigations were performed at the Kielder water tunnels, England, in addition to numerous site visits in Continental Europe and Scandinavia.

This report considers the major factors influencing full face tunnel boring machine performance, both in terms of machine boreability (rock-tool interaction) and machine utilization. It is not intended to encompass all aspects of machine performance, but to concentrate on the principle elements affecting penetration rate and tool wear, and their methods of prediction, as well as the geologic factors influencing utilization. The importance of machine design and the interrelation with the tunnel environment is emphasized. Aside from information derived from field investigations, major points are illustrated with case histories from the literature. Rock-tool interaction is reviewed to assess the fundamentals of mechanical excavation with a view to predictive capability and potential.

Technical representative for this contract was Mr. R. K. McFarland, formerly of the Office of Systems Engineering, currently Chief of the Division of Program Review. Professor T. L. Brekke, University of California, Berkeley, was the principal investigator. Dr. J. L. Griffin, Head of Structures Division, TRRL, coordinated the exchange in England. Through their efforts, continued support and interest a maximum benefit was derived from the exchange program.

Further acknowledgment gratefully goes to the many others who assisted in carrying out this research effort. In particular the author would like to thank Mr M. O'Reilly, Dr J. Hignett, Dr J. Hudson, Messrs D. Barratt, J. Morgan, M. Ryley and G. West of TRRL. Those who assisted with site visits and contributed much useful information are as follows:

Babtie Shaw and Morton

Mr A. Crozier

Mr H. Eadie

Dr I. McFeat Smith

Be Fo - Swedish Rock Mechanics Foundation

Mr S. Bjurstrom

Mr T. Franzen

Bergbau-Forschung GmbH

Dr J. Henneke

Mr W. Wehrmann

Building Research Establishment

Mr P. Tedd

Cambridge University Engr Dept

Mr R. Mair

Centre d'Etudes des Tunnels (CETU)

Mr J. Pera

Mr J.L. Reith

Mr C. Chapeau

Mr A. Schwenzfeier

Cogefar SpA

Dr P. Lunardi

ETH - Honggerberg

Dr K. Kovari

Mr M. Kaufmann

Electricite de France

Mr G. Musso

Mr J. Margerit

Furuholmen A/S

Mr K. Garshol

Geotechnical Office of Helsinki

Mr U. Anttikoski

Mr A. Saraste

ISMES, Bergamo

Dr R. Riccioni

Dr G. Robutti

Dr P. Rossi

Imperial College, London

Dr S. Priest

Mr M. Barr

Ingenieurgemeinschaft Lasser-Feizlmayr

Dr M. John

Mr J. Wogrin

International Tunnelling Ass, France

Mr C. Berenguier

Institut CERAC

Dr B. Edney

James Williams and Partners

Mr O'Neill

Lemminkainen Oy

Mr L. Hamalainen

Mr V. Vartiainen

Lulea University, Sweden

Dr O. Stephansson

Mr B. Stillborg

Luossavaara-Kiirunavaara AB

Mr H. Blomqvist

MIVA - Bijzondere Studiedienst

Mr De Roy

Mr L. Aerts

NTH - Trondheim

Dr E. Broch

Newcastle University, England

Dr I. Farmer

Norwegian Geotechnical Institute

Dr N. Barton

Dr R. Lien

Ruhr University, Bochum

Dr H. Kutter

SEMALY, Lyon

Mr C. Bouyat

Col G. Degatier

Swedish Fortification Adm

Dr J. Holmgren

Tampella-Tamrock

Dr K. Hakalehto

Mr R. Strom

Technical Research Centre of Finland

Mr K. Saari

Tirolen Wasserkraftwerke AG

Dr R. Widerhofer

Universal BAU

Mr H. Steinacher

TABLE OF CONTENTS

	PAGE
PREFACE	
CHAPTER 1 INTRODUCTION AND BACKGROUND	1
CHAPTER 2 MACHINE BOREABILITY	3
NUMERICAL MODELS	7
Rock-tool Interaction (Single Tool)	8
Groove Interaction (Indexing)	37
Summary	41
PHYSICAL MODELS	45
Full Scale Single Tool Models	45
Single Tool Scaled Models	62
Pilot Scale Models	64
Full Scale Prototypes	67
Summary	91
INDEX TESTS	93
Material Characterization	93
Boreability	96
Tool Wear	101
Summary	108
CHAPTER 3 MACHINE UTILIZATION	110
INFLUENCING FACTORS AND MACHINE DESIGN	111
Cutter Servicing	112
Machine Bracing	113
Ground Support and Control	121
Universal TBM	142
Opening Size	143
SITE INVESTIGATION AND PRELIMINARY CONSIDERATIONS	147
TBM vs Traditional Methods	147
Borehole Investigations	150
Pilot Tunnels	152
Probing Ahead	154
CHAPTER 4 CONCLUSIONS AND RECOMMENDATIONS	157
REFERENCES	161

APPENDIX I INITIAL FAILURE BENEATH A UNIFORMLY LOADED AREA 176
Uniformly Loaded Strip 176
Uniformly Loaded Circular Area of Radius a 177
APPENDIX II PRESSURE BULB MODEL 178
Surface Chip Formation 178
Surface Chip Formation Assuming a Nonlinear Shear Stress Distribution Along the Failure Surface 181
Surface Chip Formation by Pick 183
Side Chip Formation 184
APPENDIX III SPECIFIC ENERGY - MATERIAL YIELD RELATION 185
APPENDIX IV CASE HISTORIES FROM EUROPE 186
Kielder Water Scheme 187
Antwerp Pre Metro 216
Arc Isere Hydroelectric Scheme 221
Arlberg Road Tunnel 224
Bochum Underground 227
Born Railroad Tunnel 232
Frejus Road Tunnel 235
Lyon Metro, Cremaillere Tunnel 240
Oslo Sewer Tunnels 242
Paijanne Water Tunnel 248
Pfander Road Tunnel 250
Sellrain-Silz Hydroelectric Project 254
Underground Design and Construction Practice in Scandinavia 257
Victoria Collieries 262

CHAPTER 1

INTRODUCTION AND BACKGROUND

Mechanical methods of hard rock tunnel construction, and in particular the full face tunnel boring machine (TBM), have become a widely accepted alternative to traditional drill and blast methods. Increased labour costs, declining productivity, and inflation, balanced against improved machine designs and performance have resulted in the steady increase in the proportion of tunnels driven by machine.

The experience gained from the increased use of TBM's has certainly reduced the risks as far as understanding the machine limitations (bibliography of 400 references on TBM's: Wanner, 1975). Application of this information to a new and different tunnelling environment has not been as easy as acquiring the experience. Even today, projects thought to have been given sound consideration have become a good case history of TBM failure. Serious errors of judgment or geological surprises have lead to many spectacular and costly failures (see Plate 3 - 1, page126). Occasionally, the influence of the rock mass on machine performance is overlooked or not fully appreciated. When employing modern relatively impressive machines, it is possible to forget that the ground is master, while the TBM is only a tool with limited applicability. Although it is rare to know the exact geologic conditions to be encountered, the influence of a given condition on machine performance can be assessed and the risks evaluated accordingly.

In general, TBM performance is governed by the interaction of the machine with the rock mass under excavation. For convenience this can be further divided into a consideration of tool-rock mass interaction or machine boreability and the overall influence of the tunnel environment on TBM performance.

Boreability is generally accepted to mean the ability to bore as originally derived from drillability. It is described in terms of machine penetration or cutting rate and the associated tool wear. In order to reasonably evaluate the limitations of mechanical excavation, make predictions of boreability, and determine the acceptability of subsequent cutting performance, it is important to understand the process of mechanical excavation. The first half of this report considers the major factors influencing TBM boreability from three separate, although, related methods. First, numerical models are considered with a view to their predictive potential and the foundation for a theoretical understanding of rock-tool interaction. Second, physical models simulating the excavation process and prototype machine behaviour are discussed. These laboratory and field studies represent the major source of information from which a fundamental and practical understanding of rock-tool-machine interaction has evolved. Finally, the indices and index tests widely employed for TBM boreability predictions are evaluated. Without a reasonable appreciation of the process of excavation the routine application of these indicies can lead to missuse and serious error.

TBM utilization is basically a measure of all the factors affecting the machine availability. Expressed as a ratio of actual to available boring time, the utilization multiplied by the penetration rate is the overall machine performance or progress rate. It considers all aspects of machine operations including breakdowns, debris removal, cutter changes, installation of utilities, and all factors related to the tunnel environment (geological). Due to improved machine reliability, geological factors represent the major uncertainty with respect to the prediction of utilization. The ability of a TBM to maintain line and grade, brace for forward thrust, remove debris, and cope with ground stability problems, strongly depends on the particular machine design and its interaction with the rock mass. The major geological factors affecting TBM utilization and the influence of design on rock mass - machine interaction are considered in the second half of the report. In relation to machine utilization, the important factors crucial to the decision of whether to employ a TBM or traditional methods are also evaluated. Case histories are employed to illustrate the main points, many of which were derived from recent tunnel construction projects in Europe.

CHAPTER 2

MACHINE BOREABILITY

A TBM has two basic functions: rock excavation and debris removal. For rock excavation, the machine is essentially designed to accommodate an arrangement of cutting tools to which are supplied the necessary restraint, energy, and force. The amount of energy required, the forces developed, and the resulting wear in order to achieve a specific advance rate in a given rock mass depends on the rock-tool interaction (boreability). The description of tool interaction will be considered in terms of numerical models, physical models, and index tests. Before proceeding, the major types of cutters employed on hard rock machines and the terminology are introduced.

Drag bits or picks, Plate 2-1, are widely used in the coal mining industry, and more recently, on road headers (point attack machines). They are relatively inexpensive, suitable for weak rock or soil, and require a minimum normal or thrust force for operation (see Figure 2-1 for definition of terms). The low required thrust is a major advantage for TBM's operating in weak rock, as the total forward thrust is limited by the bearing capacity of the bracing system. With increased rock strength tool wear also increases until the point is reached at which the lost time associated with tool replacement makes their use uneconomical. For full face machines this point is near a compressive strength of $10MN/m^2$ (also depends on mineralogical composition). Road headers equipped with picks have been known to operate in rock of $150MN/m^2$ strength, although, with significant wear.

Disc cutters, Plates 2-5 and 2-6 (page 80), are capable of operating over a wide range of rock strength, from less than one up to $300MN/m^2$ unconfined compression. At the lower bound, weak plastic materials will clog the tool. In very strong and abrasive rock excessive tool wear and reduced machine utilization make them uneconomical. The discs operate at a high normal force, typically 5 to 20 tons; rolling or cutting force is approximately one-tenth of normal (see Figure 2-2 for definition of terms). Single, Plate 2-5, double,

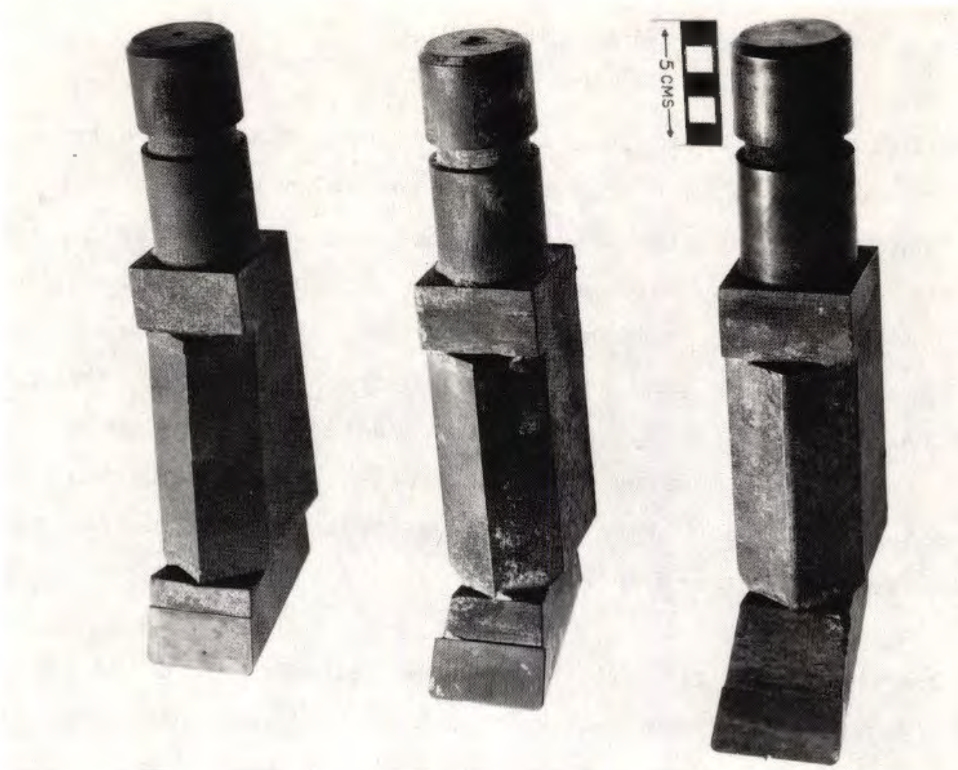


PLATE 2-1. Drag picks - 45°, 30° and 15° rake angles (after Hignett et al, 1977)

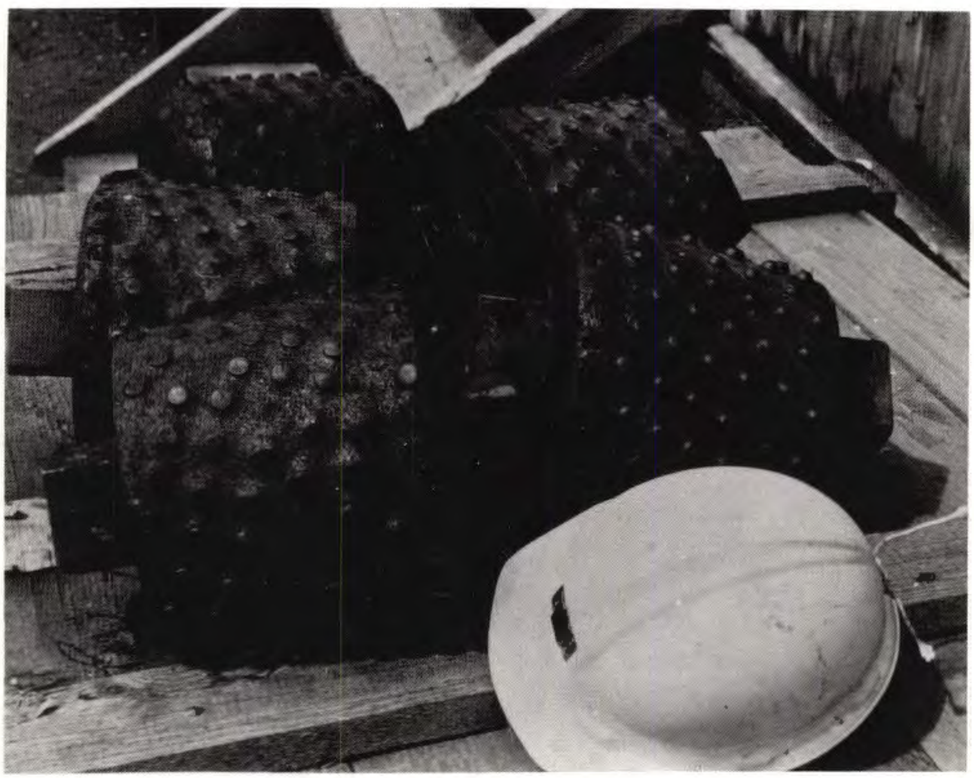


PLATE 2-2. Strawberry button cutters (roller cutters)

F_N = Normal force	S = Spacing
F_C = Cutting force	D = Depth of cut
F_S = Sideways/lateral force	α°= Pick rake angle
W = Pick width	δ°= Break out angle

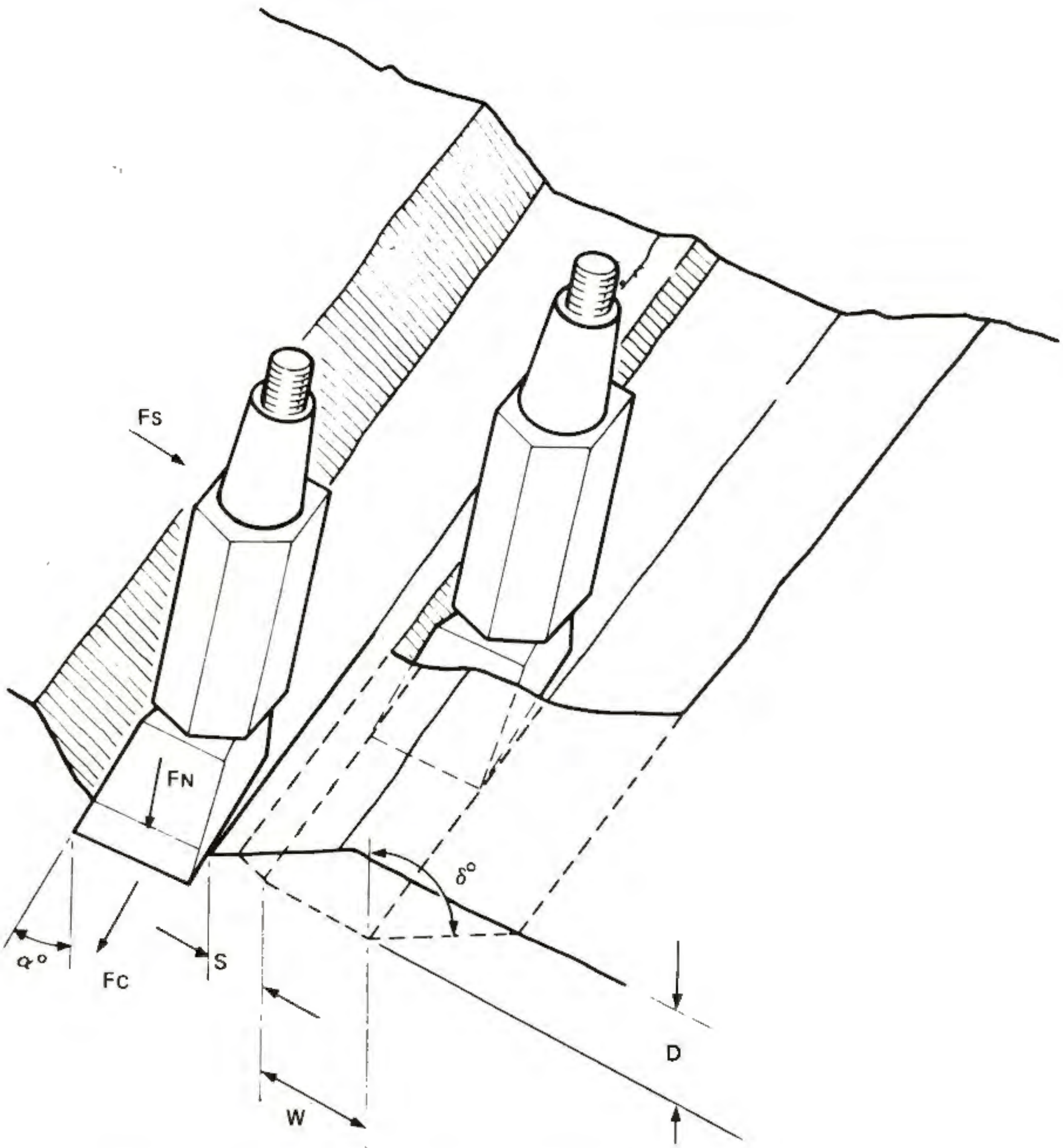


Fig. 2–1 TERMINOLOGY AND GEOMETRY OF DRAG PICK CUTTING
(After Hignett et al, 1977)

F_N = Normal force	S = Spacing
F_R = Rolling force	P = Penetration
F_S = Sideways/lateral force	$2\beta^{o}$ = Disc edge angle
W = Disc width	δ^{o} = Break out angle

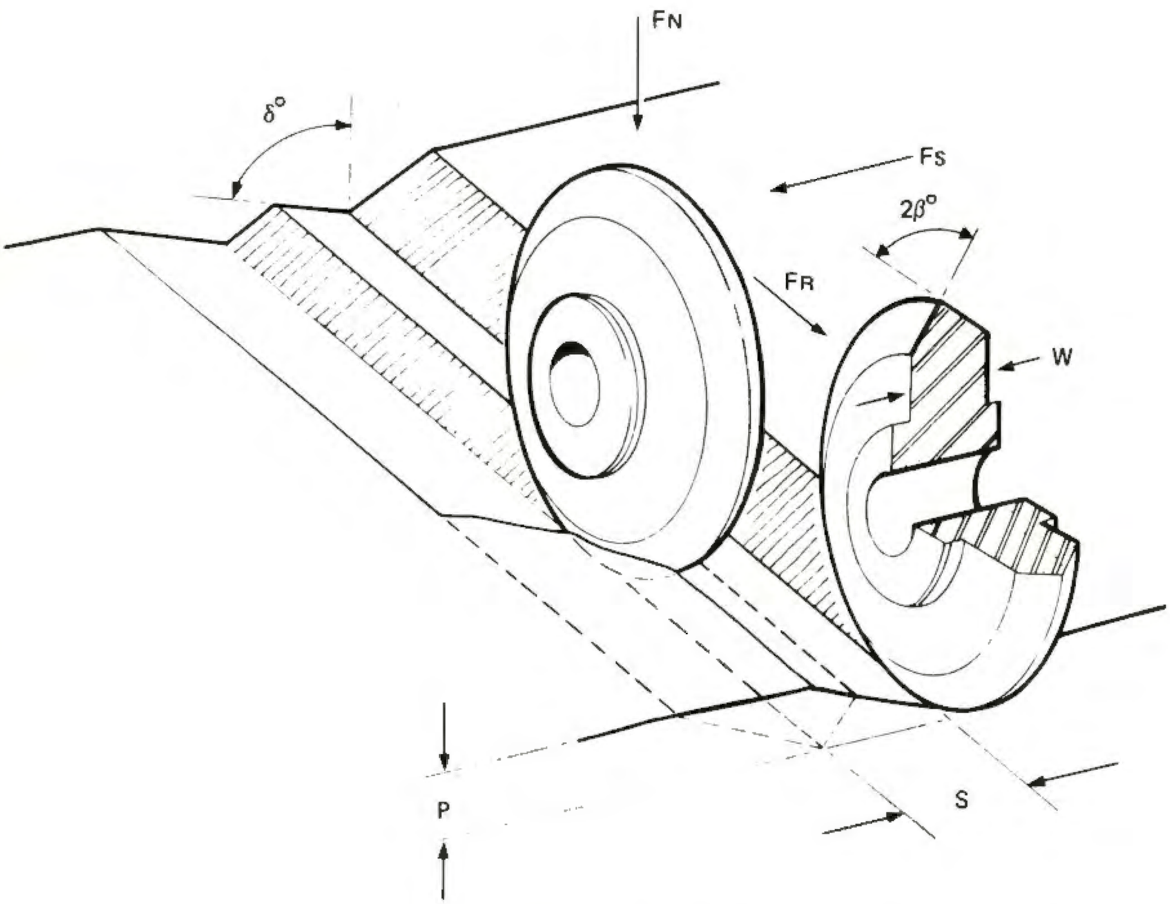


Fig. 2–2 TERMINOLOGY AND GEOMETRY OF DISC CUTTING (After Hignett et al, 1977)

and triple discs, Plate 2-6, are manufactured with various diameters, edge angles, and kerf or edge spacings (multiple disc only). For operation in stronger and abrasive rocks, the discs are often equipped with inserts of tungsten carbide (for example see outside edge of gauge cutters, Plate 2-6). On account of the disc cutters wide range of applicability and use, all subsequent discussion related to rock-tool interaction assumes excavation by disc unless otherwise stated.

For the excavation of very strong and abrasive rock, the TBM is equipped with roller cutters also known as strawberry button cutters, Plate 2-2. The numerous inserts of tungsten carbide indent the rock producing relatively small chips as compared with discs. Very high thrust forces are required to achieve a relatively low penetration rate, on the order of one meter per hour. As a result of the high cutter costs and low penetration, traditional methods of excavation are usually more economical unless there are special considerations (eg driving an incline shaft, ground vibration limitations, etc).

NUMERICAL MODELS

Numerous investigators have proposed models to describe the interaction between rock and tool during the process of indentation. Provided with specific material properties, they can be used to estimate the forces necessary to produce a given penetration and the optimization of tool geometry. All models necessarily idealize a very complicated process by making various simplifications and assumptions. Consequently, it is not surprising that predicted response often deviates from known behaviour or experience. No matter how sophisticated the model, it is imperative that predicted results are compared and calibrated with experimental results.

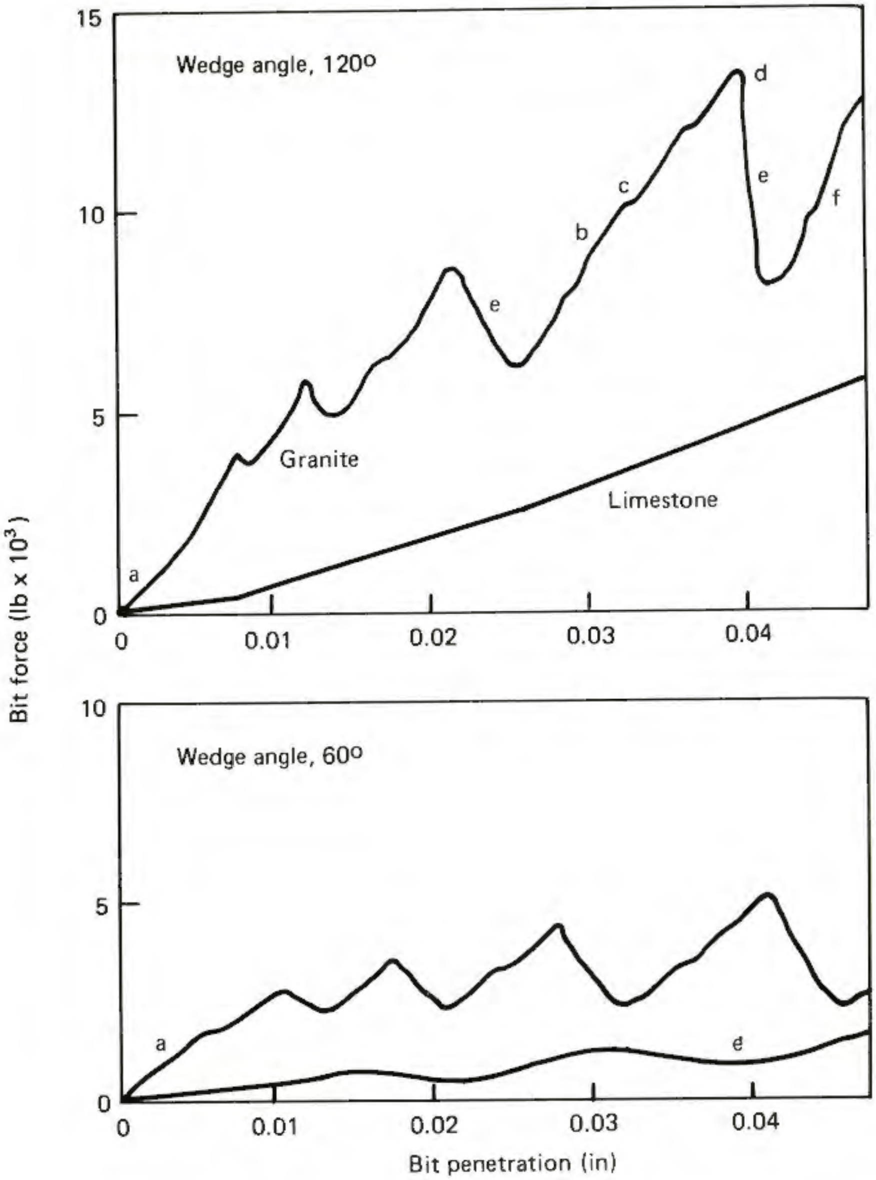
ROCK-TOOL INTERACTION (SINGLE TOOL)

Rock excavation (crater or chip formation) by indentation has been observed experimentally for different rock types and tool geometry (Hartman, 1959; Reichmuth, 1963; Ozdemir et al, 1977). With monotonically increasing penetration the sequence of events is: (a) crushing of surface irregularities, (b) elastic deformation and (c) formation of a crushed zone beneath the tool accompanied with radial fracture of material beyond the zone (inelastic deformation), (d) chip formation along curved trajectories between the crush zone and surface, (e) crumbling away of the crushed zone, (f) repeat of sequence if energy is sufficient. Several cycles of chip formation, (a) through (e), are shown by the labled force-penetration curves, Figure 2-3. For comparison two largely different rock types, a porous limestone and brittle granite, and tool geometry were selected.

In tests on granite, chipping beneath the tool edge proceeds in an explosive manner. Small particles of rock (dust) are liberated from the crushed zone at high velocity. This attests to the extremely high state of triaxial compression within the zone. As compared to granite, the penetration of limestone is almost soundless. The high porosity and ductile response of the limestone lead to a certain degree of compaction beneath the tool bit. Without low porosity and brittleness a high pressure crushed zone can not develop. Instead the bit tends to shear and displace the material as evidenced by distinct slip planes. Despite the ductile behaviour, chips are formed only with minor release of energy.

In brittle materials, chip formation may also result from unloading (decreased penetration) after formation of a crush zone (Swan and Lawn, 1976). Incompatible strains between the crushed region (irreversible deformation zone) and surrounding elastic matrix can lead to a residual stress field and tensile failure.

Force-penetration curves, Figure 2-3, are for essentially static



ROCK PROPERTY	BEDFORD LIMESTONE	CHARCOAL GRAY GRANITE
Compressive strength (MN/m^2)	53	228
Tensile strength (MN/m^2)	4.1	13.1
Static modulus (MN/m^2)	27000	67000
Apparent porosity (%)	15.4	0.77

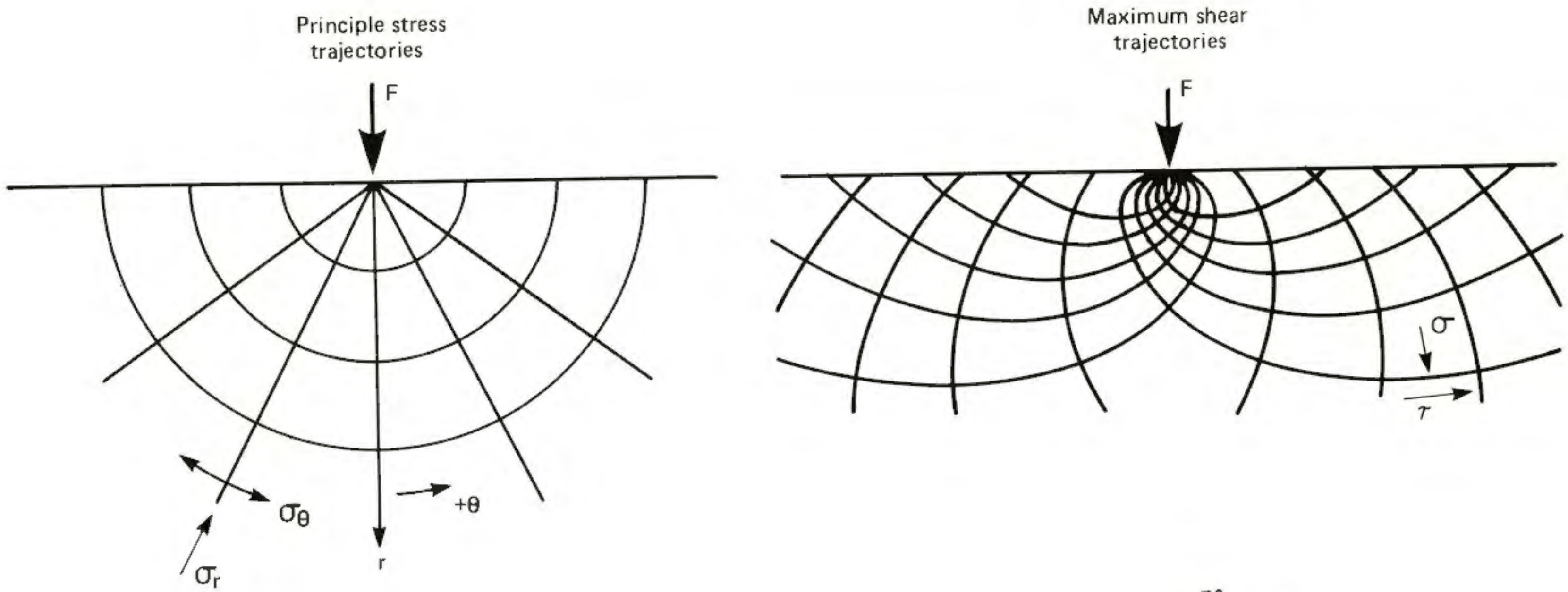
Fig. 2-3 COMPARISON OF LIMESTONE AND GRANITE STATIC FORCE-PENETRATION CURVE (After Reichmuth, 1963)

penetration (low rate). With increased rate of loading the slope of the curves increases proportional to the impact velocity. The peak force levels at which chips form, however, are only moderately increased (Hustrulid and Fairhurst, 1971). Overall curve form also remains largely unchanged which explains the lack of rate effect recorded for discs operated over a range of speeds (Roxborough and Phillips, 1975).

To account for the described behaviour during bit penetration and chip formation, various models have made assumptions concerning the stress field beneath the indentor, geometry of the tool and chip (debris), and the material failure or yield criteria. Since rock excavation by pick, disc, and other indentors are fundamentally the same process the models are described with major consideration for principle rather than geometry. Excavation by pick (asymmetric case) is basically the same as considering the action of one side of a disc (symmetric case).

Several investigators have treated bit penetration as essentially the problem of a line or point load on a semi-infinite plate or half space, Figure 2-4. As shown, the principle radial stress is inversely proportional to distance from the load and tangential stress is zero. Maximum shear and normal stress trajectories follow two sets of logarithmic spirals. To calculate an initial failure load, an appropriate shear or tensile failure criteria is selected and applied.

Based on observations of radal cracking around the crushed zone Reichmuth (1963) employed a tensile strain failure criteria to predict the initial failure load. Penetration was not and can not be considered. One major problem with the model is the inability to account for the crushed zone beneath the tool. As previously described, this region is in a state of triaxial compression rather than uniaxial compression as predicted. Only at some distance beyond the zone should the stress field approximate a point or line load. Another problem is the reasonability of a tensile strain failure criteria without tensile stress (Simon, 1967). Large tensile strains can be obtained without



$\sigma_r = \dfrac{2F \cos\theta}{\pi r}$

$\sigma_\theta = 0$

$\tau_{r\theta} = 0$

$\sigma = \tau = \dfrac{F e^{\mp\theta}}{\pi r_o} \cos \pm\theta$

r_o = constant

Fig. 2-4 STRESS TRAJECTORIES PRODUCED BY A LINE LOAD ON A SEMI-INFINITE HALF SPACE (After Frocht, 1948)

tensile stress and failure simply by heating an unrestrained specimen.

The curved shape of craters formed by impacting spherical projectiles has led Maurer and Rinehart (1960) to propose a shear mechanism for chip formation. Undoubtedly, the high shear stresses within the vicinity of the bit (maximum at θ of $\pm 45°$, Figure 2-4) is extremely important in the formation of the crushed zone. For chip formation to the surface, however, predicted values of ultimate load are often too great.

A natural extension of the above models is to consider a distributed load or contact pressure over the indentation area of the bit. For strength reasons, cutting tools are not made with a sharp point. Usually, the leading edge has a radius and becomes blunt with wear. It is necessary to assume some form of pressure distribution. Since stress concentrations at the rock-tool interface are likely to be reduced by local crushing, a uniform distribution is a good first approximation as illustrated in Figure 2-5. The solution for the stress field beneath a uniformly loaded strip and a circular punch are obtainable in simple closed form (Timoshenko and Goodier, 1951).

On combining the Mohr-Coulomb failure criteria with the solution for a loaded strip, Figure 2-6, the ultimate pressure at the initiation of failure can be calculated (see Appendix I part A for details). Two interesting points are revealed by this formulation. First, initial failure does not occur at the surface, but at roughly one indentor width below the tool. Second, depending on the internal friction of the rock the failure pressure ranges between 1.5 and 4.5 times the unconfined compressive strength, Figure 2-7. Similar results were also found for the case of a uniformly loaded circular punch only the failure pressure is influenced by Poisson's ratio as well as by friction and compressive strength (see Appendix I part B for details).

This analysis also reveals that failure load or contact pressure, obtained on dividing the load by the indented area of the tool, is not a priori equal to the material compressive strength. For porous weak

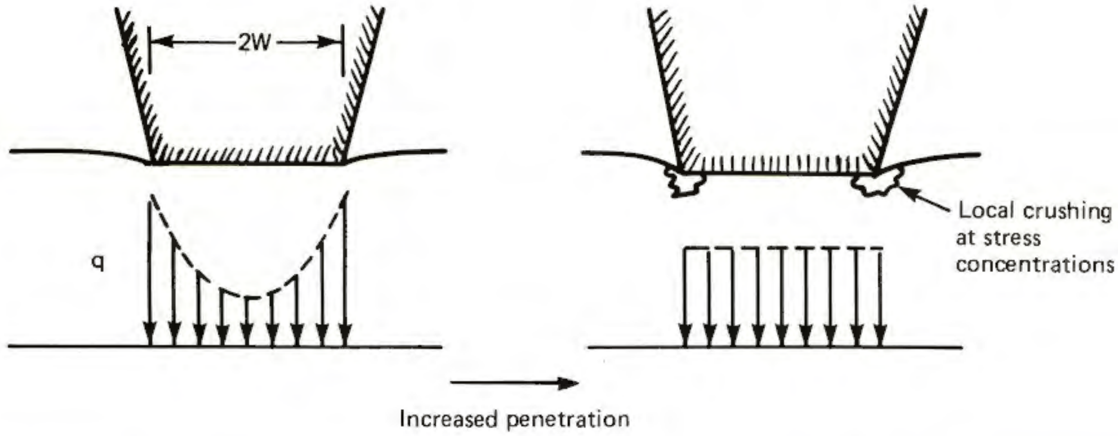
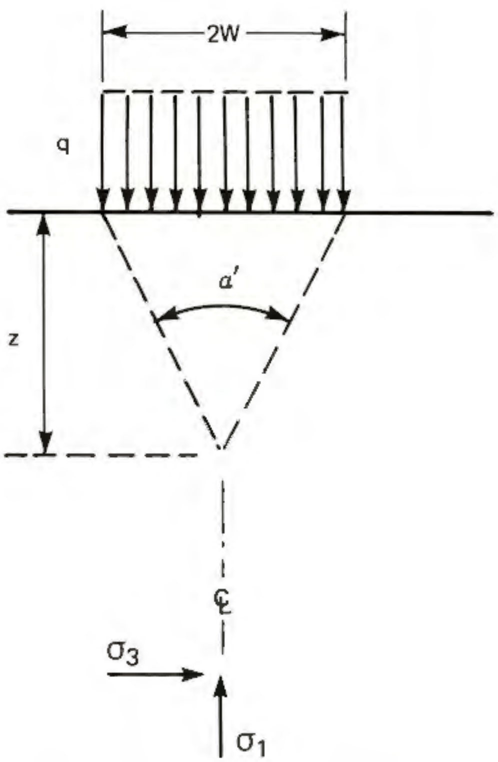


Fig. 2-5 IDEALIZED STRESS DISTRIBUTIONS BENEATH A FLAT INDENTOR WITH INCREASED PENETRATION



$$\sigma_1 = \frac{q}{\pi}\,(\alpha' + \sin\alpha')$$

$$\sigma_3 = \frac{q}{\pi}\,(\alpha' - \sin\alpha')$$

$$\sigma_1 = C_o + \sigma_3 \tan^2(45 + \phi/2)$$

C_o = Unconfined compressive strength

ϕ° = Internal friction angle

Fig. 2-6 LOCATION OF INITIAL FAILURE BENEATH A UNIFORMLY LOADED STRIP (AT DEPTH z, BELOW SURFACE)

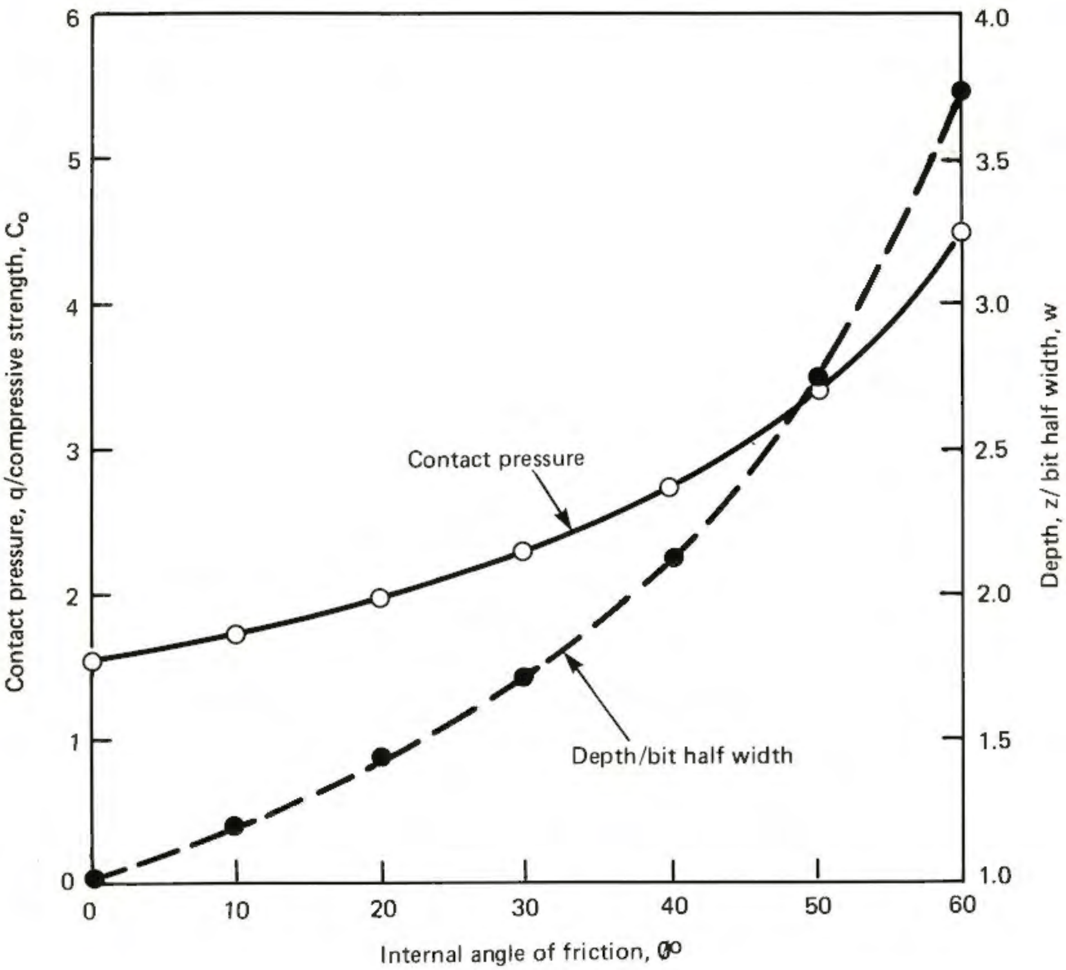
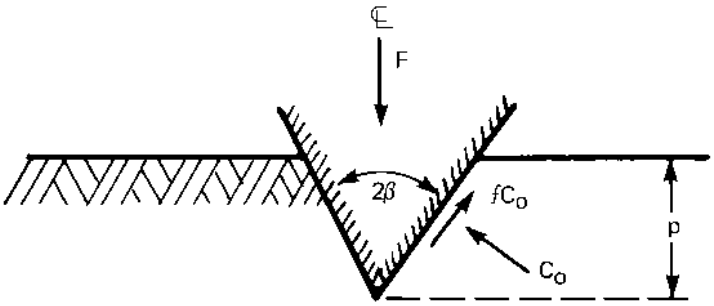


Fig. 2-7 RELATION BETWEEN ROCK STRENGTH, FAILURE PRESSURE, AND FAILURE INITIATION DEPTH BENEATH A UNIFORM STRIP LOAD (Fig. 2-6)

materials with low internal friction, however, this may be a close approximation as shown by Evans and Murrell (1962) for wedge penetration in coal, Figure 2-8. As predicted, this is not valid for more competent rock such as limestone (Gnirk, 1962). Because of chipping which accompanies tool penetration it is difficult to accurately determine the true contact area. Often the area is over estimated resulting in a low or ambiguous contact pressure. As an example, Roxborough and Phillips (1975) report the same penetration force (normal or thrust load) for static indentation of a disc in Bunter Sandstone as for a rolling disc with similar penetration. With complete contact between indented tool and rock, no chips, the static case involves twice the area than that of a rolling disc. Using the smaller area to calculate the contact pressure, both measured and predicted values were at least twice the rock compressive strength rather than equal to it as suggested (internal friction angle of sandstone 22-28°; from Figure 2-7, $q \approx 2.1$ Co).

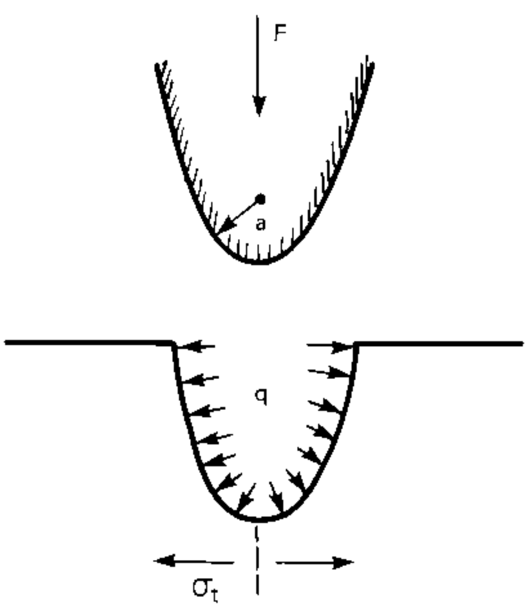
Punch bearing failure tests employing a flat right circular indentor on shale and quartzite revealed contact pressures in excess of five times the unconfined compressive strength (Hodgson and Cook, 1970). The exact multiplier, however, was shown to be strongly dependent on the diameter of the punch. For a constant material strength, with increasing diameter the failure pressure decrease was inversely proportional to the square root of the punch size. At large diameters, in excess of 0.1 m, the failure pressure was approximately the same as the rock strength.

Dalziel and Davies (1964) found the same relation for the wedge identation of coal when using tools with different radius of curvature. They accounted for this phenomenon by modelling the crushed zone as a semi-ellipse with a uniform internal pressure, Figure 2-9. From elastic theory, the tensile stress concentration at the tip of the ellipse is proportional to the square root of the tip radius. Therefore, the experimental and model behaviour were matched and good correlation between tensile strength and failure load was found. Although, the model predicts tensile fracture directly beneath the tool bit, it does not consider the mode of failure responsible for chip formation.



F = 2bpCo (f + tanβ)
f = Coefficient of contact friction
b = Wedge length

Fig. 2-8 WEDGE PENETRATION OF COAL (After Evans and Murrell, 1962)



$$q \propto \frac{\sigma_t}{\sqrt{a}}$$

σ_t = Tensile stress
a = Tool tip radius

Fig. 2-9 INFLUENCE OF SEMI-ELLIPTICAL CRUSHED ZONE WITH UNIFORM INTERNAL PRESSURE (Partly after Dalziel and Davies, 1964)

The apparent effect of decreasing material strength with increasing indentor size or size effect is most likely related to the stress gradients created upon indentation of the rock. It has been suggested that stress gradients strongly influence the pattern of fracture as compared with a uniform stress state in which fracture is controlled by material structure (Hodgson and Cook, 1970). In addition, size effect has been explained in terms of statistical models (Weibull's "Weakest link" concept), volume of material subjected to high stress (Durelli and Parks, 1962), and amount of stored strain energy in the system (Glucklich and Cohen, 1967 and 1968). Linear elastic instability criterion also predict the observed size effect, however, for all stress fields either uniform or nonuniform.

The influence of size effect on the efficiency of rock excavation by mechanical methods has not been adequately considered. Since efficiency is measured in terms of specific energy, the increased work required with a larger contact area is balanced against the volume of material liberated at failure. As illustrated in Figure 2-10, if yield is purely related to tool penetration, as indicated by some investigators, there is no advantage to wider tools (see Appendix III for details). If yield is related to indentor width, there are definite advantages. The overall consideration is complicated by the interaction of the craters created upon excavation and design limitations in terms of cutter bearing loads and machine forces.

Apart from the question of efficiency, size effect may to an unknown extent influence the results of laboratory scale model tests designed to simulate full scale field conditions. Unless the effect is investigated, predictions based on pilot scale models and scaled indentation tests are possibly in error. Obviously, the magnitude of the error is reduced by minimizing the geometric scale factor.

Unfortunately, the initiation of failure beneath a tool, as predicted by the previously described models, does not necessarily imply the start of chip formation. It is possible that additional load will be

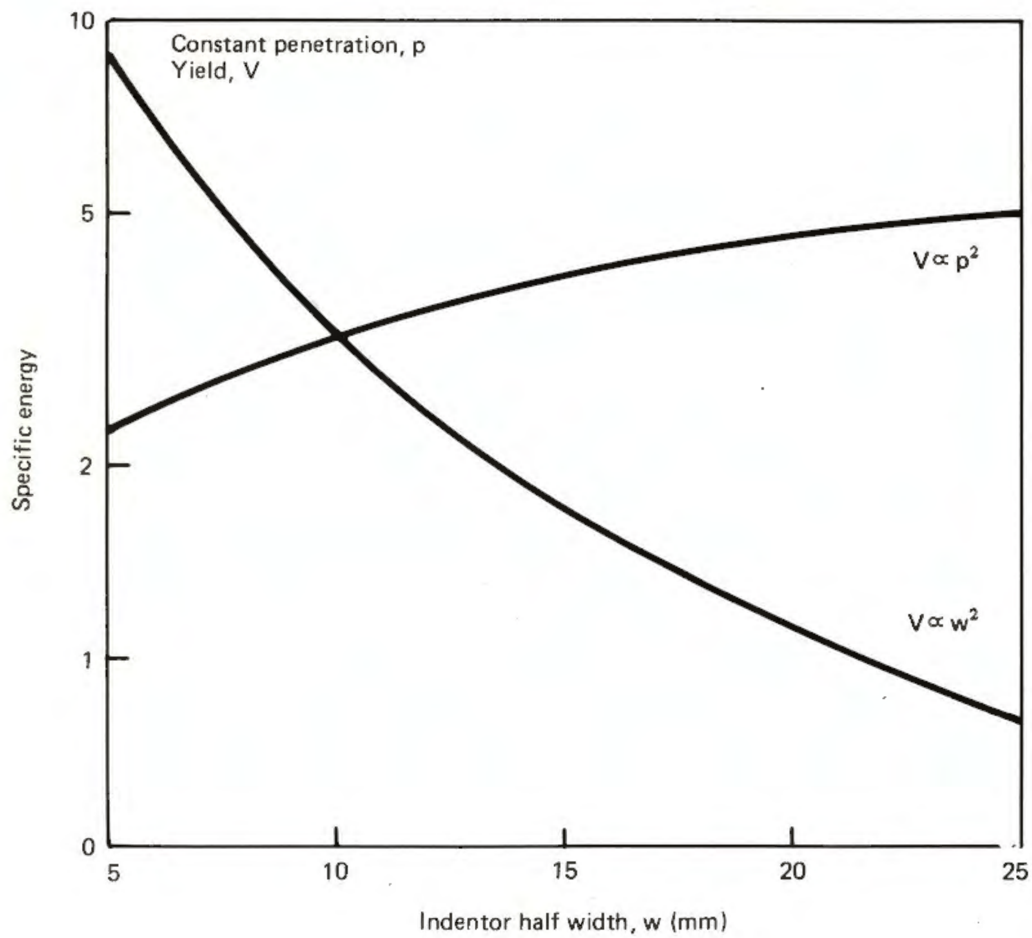


Fig. 2-10 INFLUENCE OF YIELD ON SPECIFIC ENERGY CONSIDERING A MATERIAL SIZE EFFECT

required during which the crushed zone increases in size and internal pressure. At some specific load, the pressure may be sufficient to fail the surrounding material. Not all rocks, however, have the necessary characteristics needed to develope a high internal pressure within the crushed zone before another mode of failure results in chip formation. Material hardness, dilatation, fracture surface energy, brittleness, porosity, and strength are major properties to consider. In addition, tool geometry also influences the size of the crushed zone, the greater the wedge angle the larger the extent of the zone (Fairhurst and Lacabanne, 1957).

Associated with shearing of material within the crushed zone and favouring the development of high pressure is material dilatation. Measurements of volume change during the deformation of various rocks revealed examples of significant dilation at high confining pressures (up to 600 MN/m^2; Edmond and Paterson, 1972). Since the net volume change is a competing process between compaction and dilatation, rock porosity has a major influence. Below the confining pressures listed in Table 2-1, the materials displayed an increased volume change at large shear strains. This volume change was indicative of "cataclastic flow", the movement and fracture of grains and the fracture along grain boundaries. Decreased porosity generally indicated dilatation up to greater confining pressure. The exception is for materials, such as sodium chloride, which deform by intracrystalline plasticity with little or no change in volume.

Very few rock forming minerals deform by intracrystalline plasticity in the absence of confining pressure, although, several rocks simulate what is often called "ductile behaviour" on indentation. These rocks are usually of lower strength and/or high porosity. Characteristic force-penetration curves are nearly linear without radical fluctuations in force with increasing penetration, indicative of brittle behaviour and chip formation. This is clearly illustrated by the comparison between the response of limestone and granite to wedge indentation, Figure 2-3. For the appropriate materials, the classical theories of plasticity provide a good model by which to

TABLE 2-1

Confining pressure for zero volume change
(from Edmond and Paterson, 1972)

Material	Porosity (%)	Confining pressure* (MN/m^2)
Graphite	25	≈ 0
Gosford sandstone	13	100-200
Lithographic limestone	5.9	200-300
Three Springs talc	3.2	500-600
Carrara marble	1.1	400-500
Sodium chloride	0.5	≈ 0

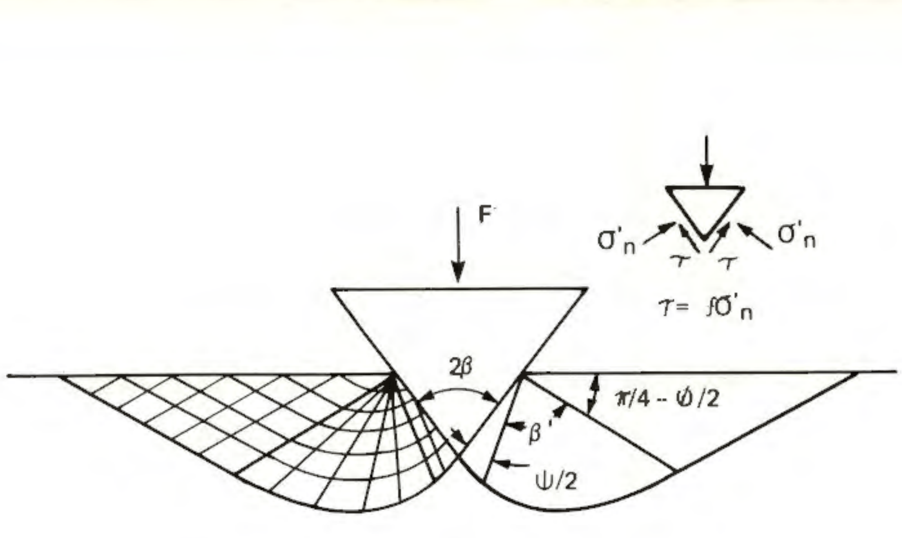
* at large strains, approximately 20 per cent

to approximate the observed response.

Employing the method of characteristics, both Prandtl (1921) and Hill (1947) derived solutions for wedge penetration into a rigid, perfectly plastic material. Cheatham (1958) modified Prandtl's solution to account for contact friction between rock and tool (smooth or rough bit) and used a linear Mohr-Coulomb yield criterion, Figure 2-11. Predicted force-penetration is linear, however, the magnitude of the force for a given penetration is often considerably higher than measured. Agreement naturally improves when reduced values of internal friction are substituted. Employing a parabolic yield envelope, decreased friction with increased confinement, resulted in better agreement (Cheatham, 1964). More recent investigations have considered work hardening or compacting materials to describe volume decreases observed in the drilling of porous rock at depth, under high confining pressure (Miller and Cheatham, 1972). Most of the authors work has been applied to drilling at depth, an area to which the models of perfect plasticity are better suited.

On modification of the plasticity model to include a "false nose", Figure 2-12, agreement of measured and predicted response was further improved (Pariseau and Fairhurst, 1967). Essentially, a "false nose", created by the compaction of sheared material beneath the tool bit, simulates the behaviour of a sharper bit with a decreased included angle. Although the existence of a "false nose" has been observed in some materials, the model has limited application. As for the previously described plasticity models, measured and predicted behaviour were similar only when applied to the appropriate materials.

To this point, models describing the initial material failure beneath the tool, within the crushed zone, and those derived from the classical theories of plasticity have been summarized. To consider the process of chip formation, that being the major volume of material produced from tool indentation, many investigators have derived simple limit equilibrium models. Predicted results are largely dependent on

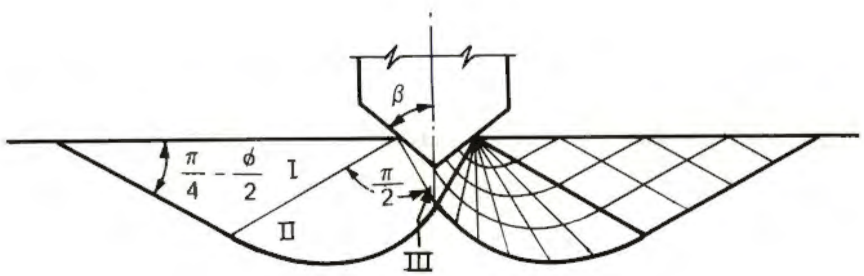


For a smooth bit $(f = 0)$

$$\frac{F}{bpC_o} = \frac{\tan\beta}{\tan\phi\tan\lambda}\left[\exp(2\delta\tan\phi) - \tan^2\lambda\right]$$

where $\lambda = 45 - \phi/_2$, $\delta = \beta$

Fig. 2-11 SLIP LINE FIELD FOR WEDGE INDENTATION OF A RIGID PLASTIC MATERIAL (After Cheatham, 1958)



Region III – "false nose"

When $\lambda \leqslant \beta \leqslant 90^\circ$, $\delta = 90^\circ$ in above equation

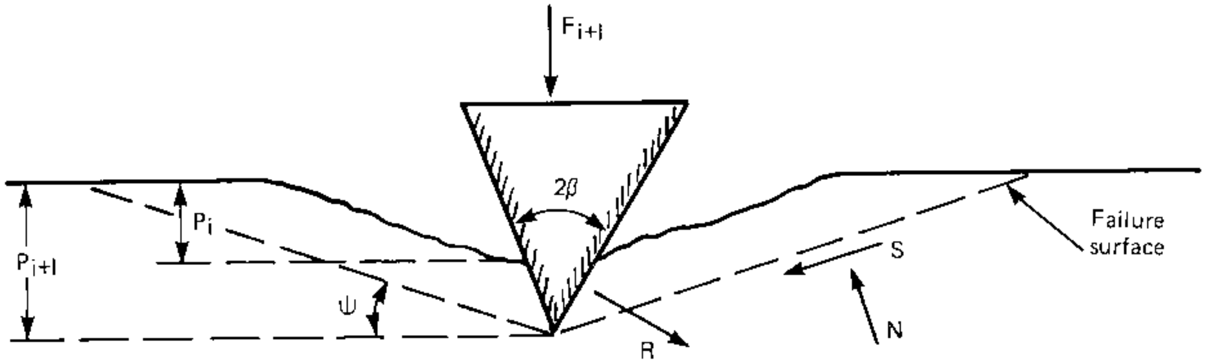
Fig. 2-12 ASSUMED SLIP LINE FIELD FOR A "FALSE NOSE" SITUATION (After Pariseau and Fairhurst 1967)

the simplifications and assumptions related to geometry, mode of failure, yield or failure criterion, and stress distribution. The crushed zone is usually ignored.

Based on the geometric idealization shown in Figure 2-13, Paul and Sikarskie (1965) proposed a model to describe the recurrent nature of brittle chip formation (Figure 2-3, granite). On indentation, rock crushing proceeds along an experimentally determined slope k until the load is sufficient to overcome the force resisting chip formation, given by the failure line of slope K. Resisting forces are derived from the Mohr-Coulomb yield criterion assuming a uniform shear and normal stress distribution along the potential failure line. Although the model adequately describes the process of multiple chip formation, the predicted forces are excessively high unless reduced values of material strength parameters are employed. The authors argue that the main reason for the discrepancy is the assumed uniform stress distribution along the failure surface. Another major factor is the assumed geometry between the resultant tool force, normal to the tool surface, and the failure line. It requires that failure must occur by shear at a relatively high normal stress across the shear plane. The work was extended to consider wedge penetration into a nonisotropic material (Benjumea and Sikarskie, 1969) and conical indentors (Lundberg, 1974).

Dutta (1972) modified the above model to include a "false nose" beneath the tool bit. As previously described, this simulated the behaviour of a sharper wedge. It effectively reduces the angle between the resultant tool force and the failure surface, hence, decreasing the normal stress and resisting shear force. Agreement between predicted and measured behaviour was improved, however, it was necessary to employ reduced material strength values (internal friction angle of less than 20 degrees for rocks with friction in excess of 30 degrees).

Nishimatsu (1972) developed a model employing a nonlinear stress distribution along the failure surface. Even though it was designed to represent excavation by pick, the basic geometry and assumptions were similar to that shown in Figure 2-13. Consequently, the mode of



$$K = 2C_o \frac{\sin\beta\,(1 - \sin\emptyset)}{1 - \sin(\beta + \emptyset)}$$

$$\psi = \frac{\pi}{4} - \frac{\beta + \emptyset}{2}$$, failure plane angle

R = Resultant tool force

N,S = Resisting normal and shear force

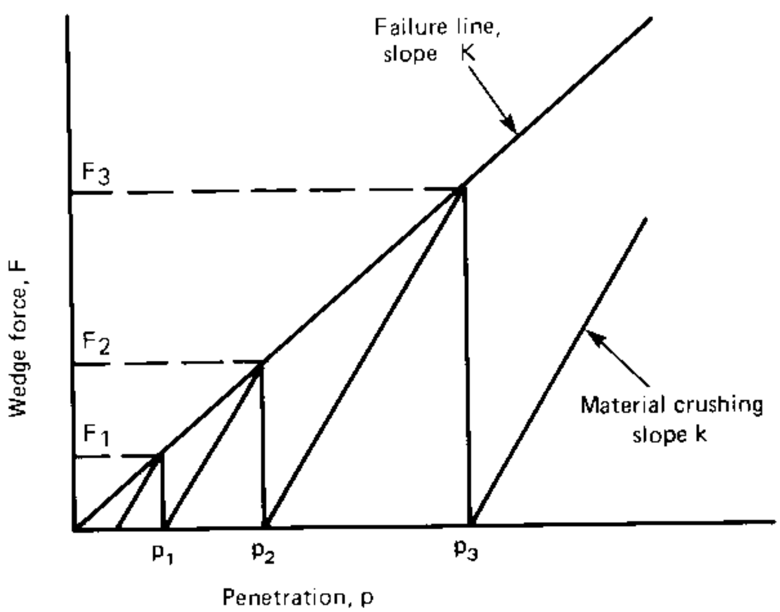
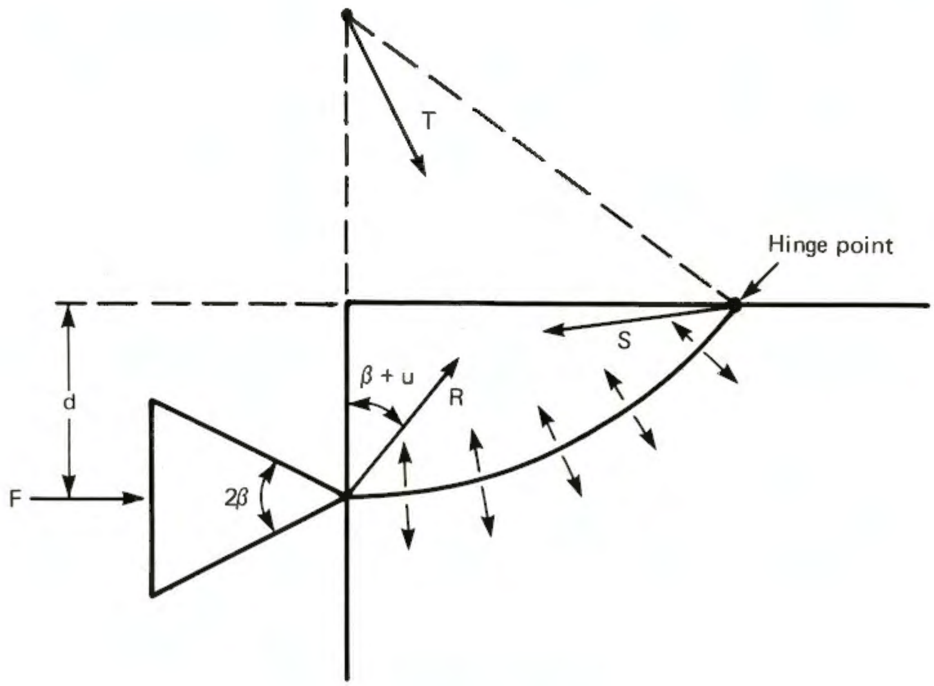


Fig. 2-13 RECURRENT CHIP FORMATION MODEL (After Paul and Sikarskie, 1965)

failure was also by shear at high normal stress. Introducing a nonlinear distribution reduced the force necessary to initiate failure. As the experimental data was curve fit to several of the model variables (rake angle, tool contact friction, and stress distribution factor) agreement between measured and predicted behaviour was good. Overall usefulness of the model is limited by the need to fit the results to the experimental data.

One of the few models based on material failure other than by shear was that proposed by Evans (1962), Figure 2-14. Failure occurred when the assumed uniform tensile stress along the failure surface exceeded the tensile strength. To satisfy equilibrium, an equilibrating force, S, acting through the hinge point was required. As can be seen, a large component of this force could be resolved as a shear stress along the failure surface without altering the calculated results. Therefore, the failure mode could either be pure tension, as described by the author, or tension plus shear. Initially designed for the identation of coal, the model was found not to apply to more competent rocks (Gnirk, 1962).

In summary, many investigators have noted the existence and influence of a crushed zone directly beneath the tool bit, Figure 2-15. It is a region of intense shear under high triaxial compression (positive shear-normal stress space, Figure 2-16). The magnitude of the internal pressure and size of the zone depend on tool geometry and material characteristics. If mobilized pressure is low, the crushed zone will have a minimum effect on chip formation. Failure mechanisms are largely controlled by material shear as described by the theories of plasticity. A high internal pressure is indirectly responsible for radial fracturing of material surrounding the zone and chip formation. The process of brittle chip formation is not adequately described by a compressive shear mode of failure (positive shear-normal stress space). It is most likely that the interval pressure responsible for radial cracking also results in a combined shear-tensile fracture (negative shear-normal stress space, Figure 2-16). The exact failure state depends on the magnitude and distribution of the internal pressure



$$F \doteq \frac{2 T_o \, d \sin(\beta + u)}{1 - \sin(\beta + u)}$$

u = Contact friction angle

T_o = Tensile strength

Fig. 2-14 TENSILE FAILURE MODEL (After Evans, 1962)

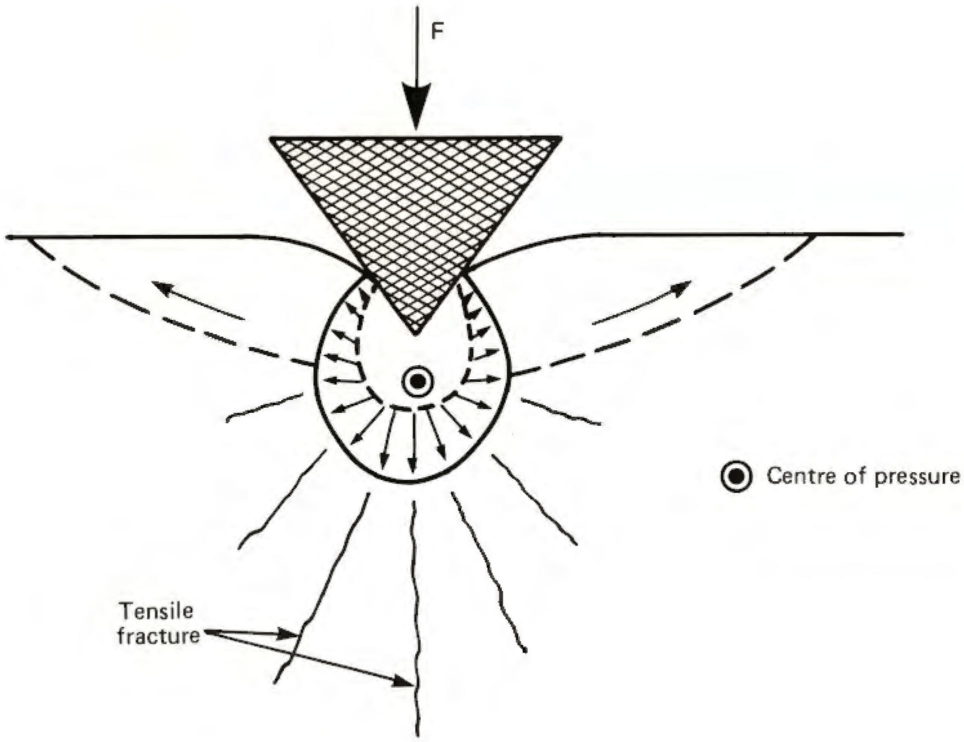


Fig. 2-15 BRITTLE CHIP FORMATION WITH CRUSHED ZONE (SURFACE CHIP)

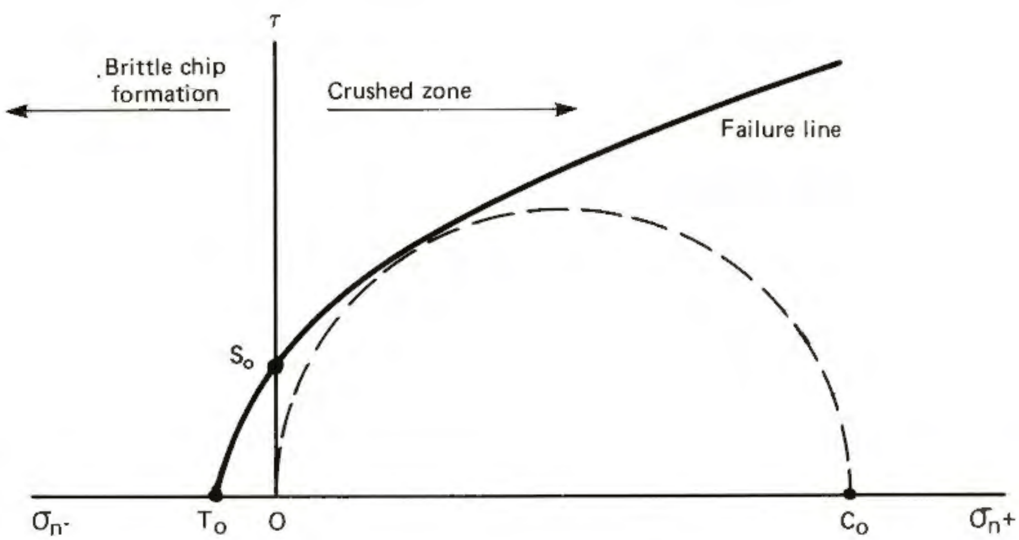


Fig. 2-16 RELATION BETWEEN MODE OF FAILURE AND STRESS STATE

within the crushed zone. Quantitative information on stress, displacement, and material failure within this region and the influence on chip formation has been obtained through computer simulation.

A two dimensional, plane strain finite element model developed to examine the stress and strain field during bit identation essentially substantiates the above hypothesis (Wang and Lehnhoff, 1976). The code considers nonlinear material properties (post failure behaviour or work softening), geometric nonlinearity (large strains) and fracture propagation. Material compaction and/or dilatation were not included, however, it is believed that this would only have modified the pressure within the crushed zone rather than alter the mode of failure. Development of the crushed zone begins at points of stress concentration and directly below the tool as predicted by the distributed load model, Figure 2-6. With increasing load the extent of the crushed zone increases. In response to the developing internal pressure, tensile fractures start from the bottom of the compressive zone and gradually spread around the sides toward the surface. Finally, a chip is formed by a combined tensile-shear mode of failure.

Although the model provides a realistic simulation, it is difficult to use as a practical tool and requires hundreds of iterations for one indentation. It is an excellent demonstration model and indicates the direction for improving simplified models. The essential features to be considered are as follows (see Figure 2-15 for details):

(a) final shape of the crushed zone can be approximated as an elliptical or circular region.

(b) size and position beneath the bit are largely determined by bit geometry and contact area.

(c) geometric center of the crushed zone or center of pressure is predicted by the distributed load model employing a Mohr-Coulomb failure criterion.

(d) induced fractures formed outside the zone tend to radiate from the centre of pressure.

(e) distribution of pressure within the zone is not uniform or hydrostatic, but increases with depth below the surface as indicated or mapped by the length of radial tensile cracks.

Based on the above results, a simplified model has been developed to consider the mechanical excavation of brittle material. Named the pressure bulb model, it is applicable to all types of indentors operating in any geometric configuration (disc, pick, etc), Figure 2-17. Essentially, the model includes a circular or spherical crushed zone, the geometry of which is controlled by centre of pressure (Appendix I) and bit-rock contact area. Pressure within the zone increases linearly from zero at the bit-surface interface to a maximum at the bottom. The pressure magnitude is calculated through the equilibrium of applied load and internal pressure. Potential chip failure surfaces radiate from the centre of pressure and are linear. Failure is governed by a linear relation between tensile and shear strength (negative shear-normal stress space, Figure 2-16). Distribution of the equilibrating stress along the failure surface can be assumed linear or nonlinear. The formulation is considered in Appendix II; because the equations were difficult to simplify, minimization of tool force with respect to chip angle was performed numerically.

There are several realistic, but complicating factors introduced by this model. As shown in Figure 2-17, the tool penetration, p, is not necessarily the same as the crater depth, p'. After the chip is formed, the tool may or may not displace the remaining material within the crushed zone depending on the stiffness of the indentor system and the elastic rebound of the rock. Ignoring rebound, an infinitely stiff system will produce a crater of greater depth than the actual tool penetration. In a soft system (dead weight), both will appear to be approximately the same. As the predicted force is a function of crater penetration, the accuracy of the calculation depends on the value assumed or measured.

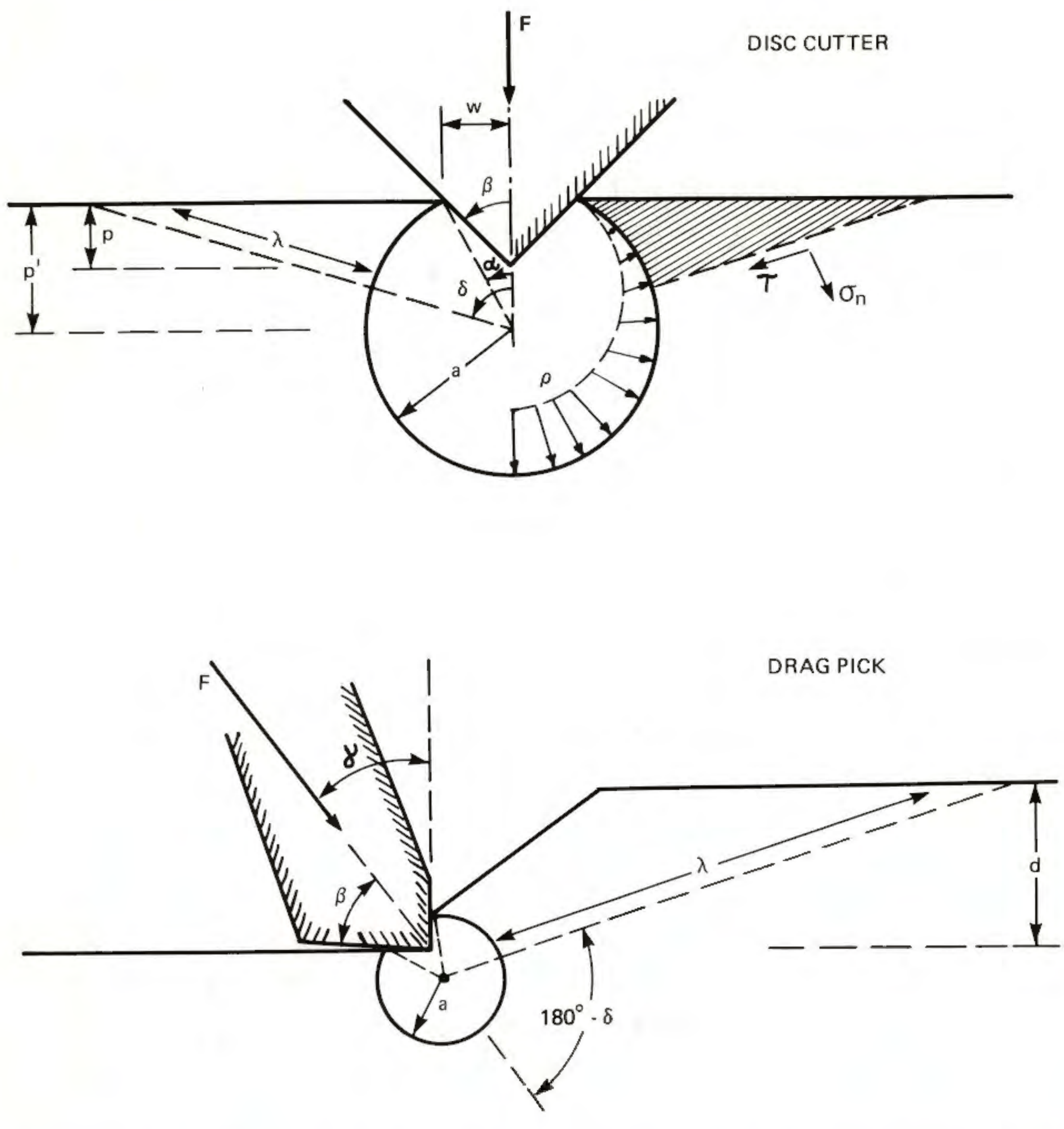


Fig. 2-17 IDEALIZED MODEL OF CRUSHED ZONE AND SURFACE CHIP FORMATION : TOP - SYMMETRIC INDENTATION (DISC), BOTTOM-ASYMMETRIC (PICK)

Most performance tests of picks and discs are carried out with a shaping machine or linear cutter. In those tests performed with soft systems, the force remains nearly constant while penetration, the dependent variable, changes with tool position. The opposite applies for a stiff system, but it should be emphasized that the nearly constant penetration is the crater depth not necessarily the depth of tool penetration. Data from either system are only comparible at similar forces.

For example, it has been assumed that the centre of pressure during wedge indentation is the average of the depth of wedge penetration, p, and the pressure centre, h, derived from a distributed load over the equivalent surface area, Figure 2-18. Given a constant tool force (soft system) over a range of wedge angles the specific energy, based on crater depth (p') and chip angle (δ), increases considerably with wider tool angle as would be expected. For a constant penetration of the tool, an infinitely stiff system will be forced to accept the higher load with increased angle. The corresponding yield, however, will also substantially increase in response to the growing size and pressure of the crushed zone, resulting in an overall slight decrease in specific energy for wider angles. Most test systems and TBM's are between these two extremes as shown by the more typical response for a constant depth of crater, p'. All results converge when compared at similar force irrespective of system stiffness (for wedge half angle of 30 degrees). Additional evidence from laboratory tests to illustrate the importance of stiffness and data interpretation will be presented in the next section.

An added problem when comparing test results is the determination of similar forces. In other than soft systems, the measured forces fluctuate drastically as load is acquired and chips form (Figure 2-3, granite). The difference between the magnitude of peak and mean forces depends on the stiffness of the system; by as much as a factor of two for very rigid systems. Since major chips form in response to peak loads, the average peak values should be the basis for comparison and model prediction (Fairhurst and Lacabanne, 1957). Unfortunately,

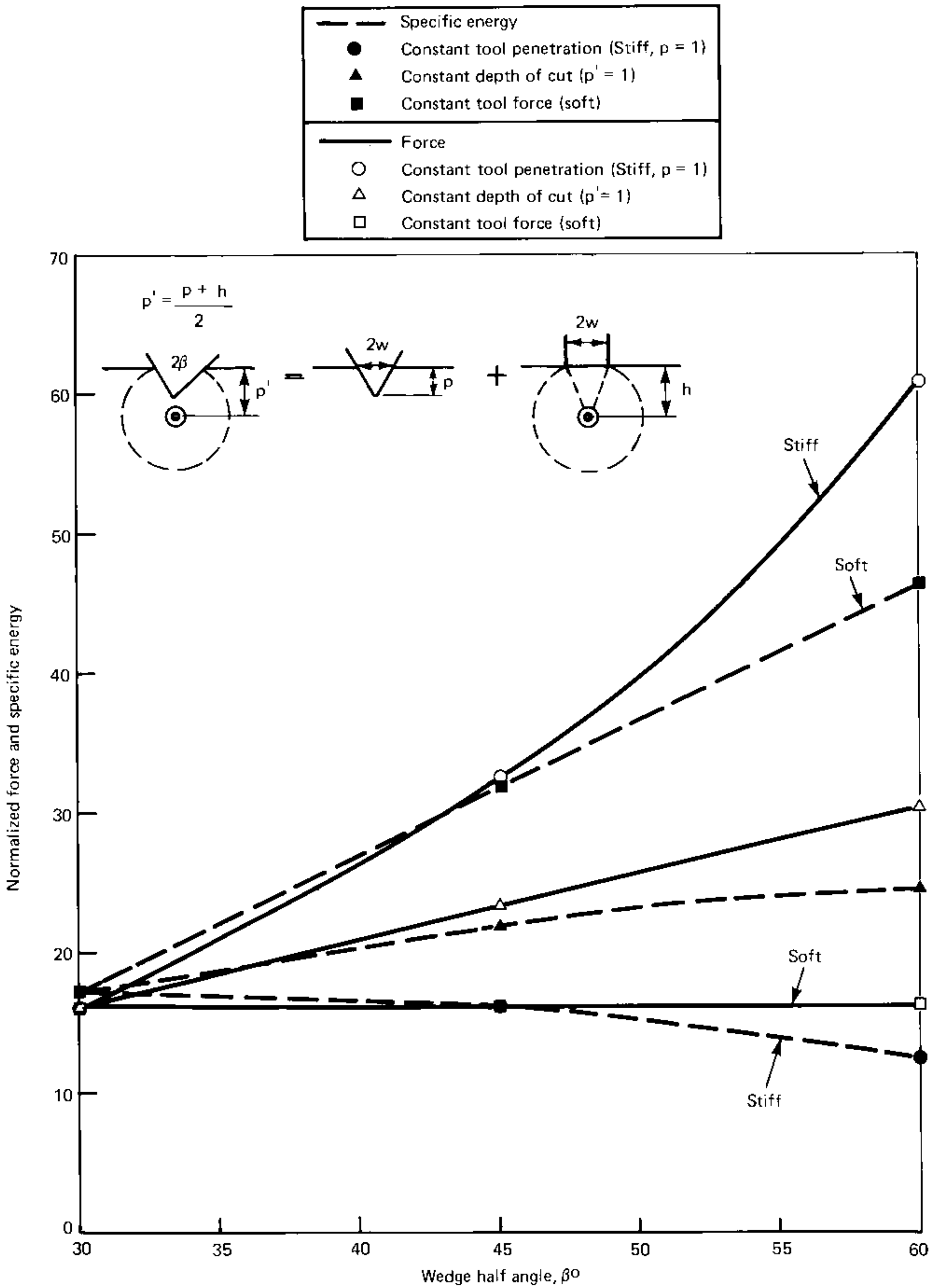


Fig. 2-18 INFLUENCE OF INDENTOR WEDGE ANGLE ON SPECIFIC ENERGY AND ON NORMAL FORCE ASSUMING VARIOUS MODES OF PENETRATION

much of the literature reports and employs mean values of thrust or normal force.

To illustrate the pressure bulb model, a comparison between predicted and measured normal force is shown in Figure 2-19. Measurements were obtained from the TRRL linear cutter employing a single 0.2 m diameter disc with a 45 degree half angle (machine normal stiffness, 500 MN/m). The excavated rock was Plas Gwilyn limestone, a dense medium strength material (unconfined compressive strength, 90 MN/m^2; tensile strength, 7MN/m^2). For reasons previously described, two prediction curves are shown. One assumes that the indicated (pre-set) penetration is a measure of tool indentation, while the other assumes that it is a measure of crater depth. As shown, for penetration in excess of 5 mm the measured peak forces are between the predicted bounds. Below 5 mm the predicted values are low. The high initial force at low penetration, often observed in the indentation of rock, is most likely a result of size effects related to stress gradients combined with the complicated geometry of the disc cutter.

If the disc employed to obtain the force-penetration curve, Figure 2-19, was of an ideally sharp wedge type geometry, the normal force should increase as the penetration to the three-halfs power assuming no size effect (as illustrated by the predicted response). A material displaying a size effect, of the order previously described for coal and quartzite, will result in a linear force-penetration relation. As an approximation, this is commonly found in practice, although, less often at low penetration where initial nonlinearity can be ascribed to geometric complications. Discs are not ideally sharp, but incorporate a radiused tip. For the indentation of this portion of the tool, the bit contact area is not proportional to disc penetration. With increased penetration, beyond the initial geometric nonlinearity caused by the tip, the force-penetration relation approaches a linear function. When it is possible to decouple the tool geometry from the material response, most rocks appear to exhibit a significant size effect. Even though the effect was not considered in the pressure bulb model, it can be included by establishing a relation between material strength and chip fracture

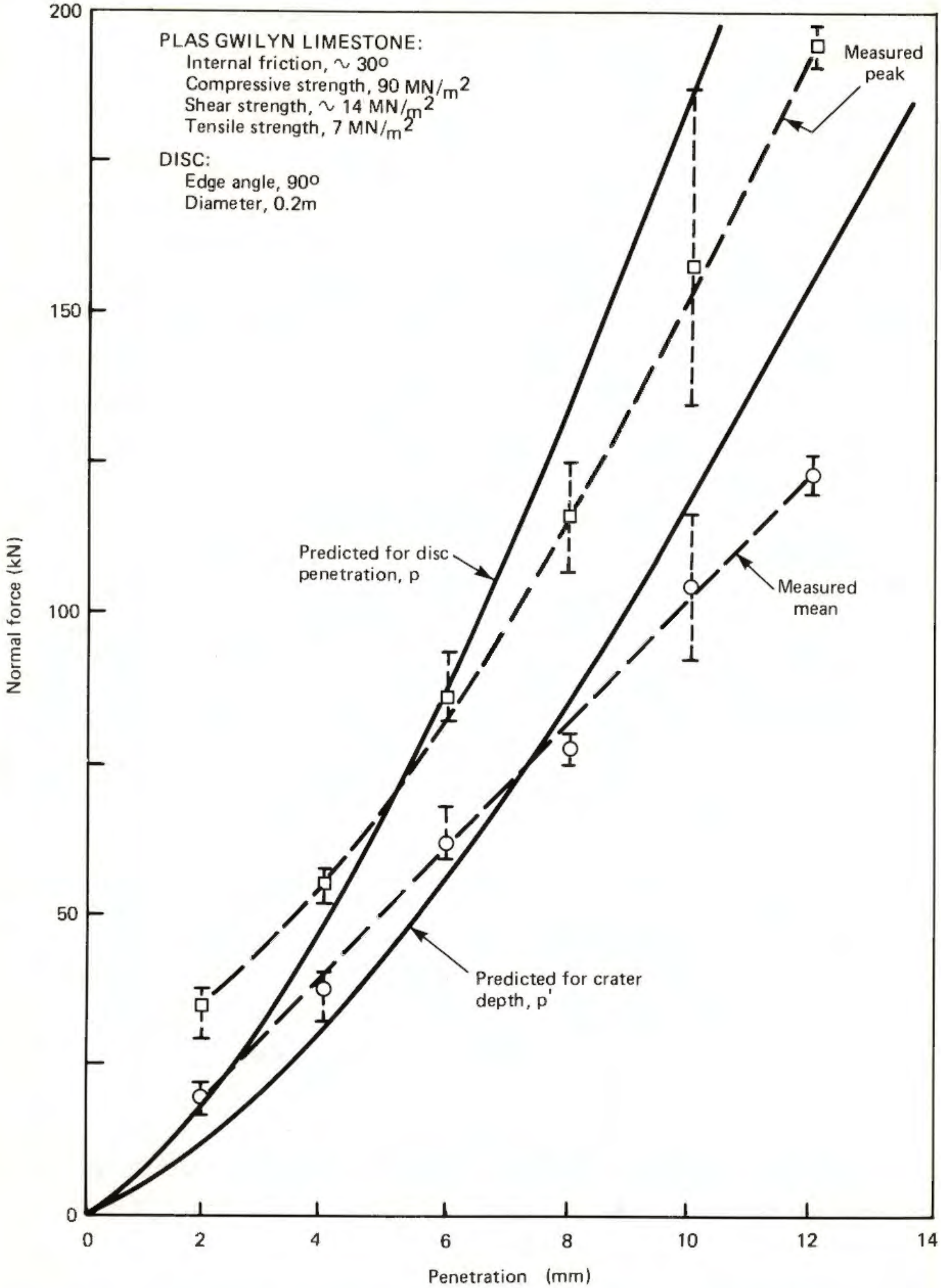


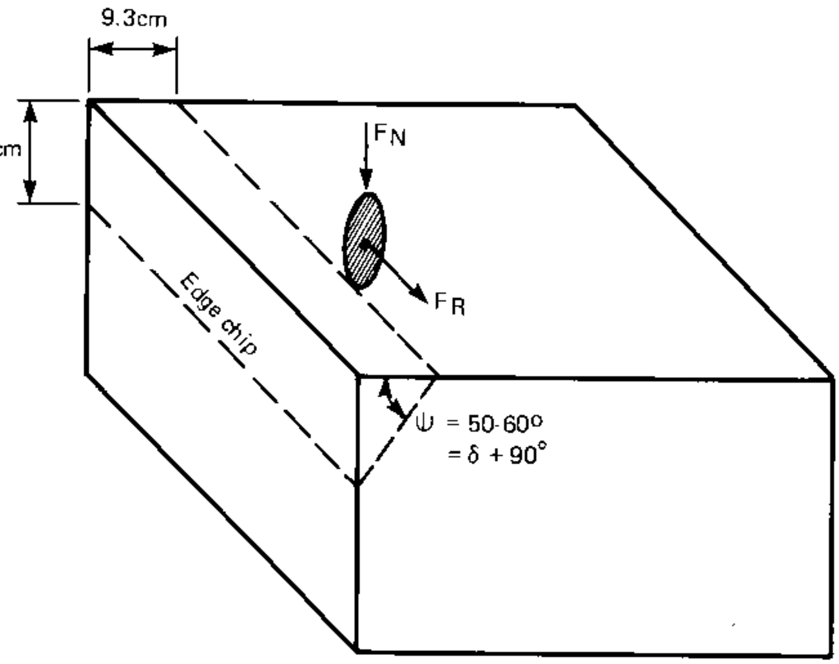
Fig. 2-19 MEASURED AND PREDICTED FORCE-PENETRATION RELATION ASSUMING VARIOUS MODES OF PENETRATION

surface length (also proportional to penetration).

The large force required to obtain the small penetration and yield is indicative of the poor efficiency of TBM's operating at low penetration rates. It is necessary to overcome and operate well past this "threshold" thrust or pressure beyond which the machines performance improves significantly.

When indentations are made too close to the edge of the block of rock being excavated, for a given penetration or load there is a critical distance from the edge at which chips will form to the surface, Figure 2-15, or with equal probability to the side, Figure 2-20. In this situation the indentor or disc operates in the usual mode as well as simulating the operation of a pick. It provided a good comparitive test of the model's general predictive capability. For the cut shown in Figure 2-20, three major chips to the outside edge were formed at an average peak normal load of 101 KN. As can be verified in Figure 2-19, surface chips formed at nearly the same peak force. Given the appropriate geometry and penetration a force of 100 KN, $\pm$20KN depending on assumed material properties, was predicted for both modes of failure, Figure 2-17. In addition, the direction or angle of the failure surface was estimated to within the measured bounds (chip angle, δ, approx 140-150 degrees to side; 60 degrees to surface).

On account of strength and wear considerations, tool bits are not made as sharp as results from some laboratory tests and models would predict is necessary for efficient excavation. Simplified models usually describe the force as proportional to the tangent of the wedge half angle. Percussion drill bits, however, typically employ wedge angles between 100 and 120 degrees. Picks work with negative rake angles and discs are constructed with a leading edge radius. In most cases, wear quickly reduces sharp edges to gently rounded or blunt. Despite the predicted efficiency of sharp wedge indentors, many excavation machines show only a slight loss of performance with dull tools. This largely depends on the mode of chip formation as determined by the material being



Disc diameter, 0.28m
Wedge half angle, 45°
Penetration (p or p'), 7mm
Average peak normal force, 101kN

Fig. 2-20 SITUATION FOR EQUAL PROBABILITY OF SURFACE OR EDGE CHIP FORMATION

excavated. The advantages of a sharp tool in a ductile and porous rock are evident to promote material shear. If, however, a pressure bulb is formed the shape of the indenting tool becomes less important. Formation of an efficient high pressure crushed zone is the required end product.

Tests performed in granite have shown little change in force or efficiency when excavating with blunt discs as compared with sharp (for large tool spacing; Ozdemir et al, 1977). This result considered the interaction of grooves or craters when chips form to the side rather than the surface, Figure 2-21. Chip formation to the surface is adversely influenced by dull tools due to the increased confinement provided, although the effect is not as great as is often suggested. The centre of pressure is pushed further beneath the bit and the internal pressure component acting on a potential surface chip is reduced. Results from both the pressure bulb model and a model describing chip formation in an elastic-brittle solid are not strongly dependent on the details of the surface tractions and are relatively weak functions of wedge geometry (Sikarskie and Altiero, 1973).

GROOVE INTERACTION (INDEXING)

Formation of surface chips is the fundamental mode of excavation by indentation, but not the most efficient. Interaction with previously cut grooves created by a multiple array of tools provides a larger yield of material for a specific amount of energy.

Given an adjacent groove or crater with which to interact during tool penetration, the resultant chip may form to the surface or side as shown in Figure 2-21. The particular mode of failure depends on the equilibrium of forces along the two potential fracture surfaces. As the side chip is subjected to a greater component of the internal pressure within the crushed zone and has a favourable geometry (side relieved by the groove),the volume of material liberated is considerably greater than a surface chip for similar applied load. In both practice and as predicted by most simplified models, the failure mode basically depends

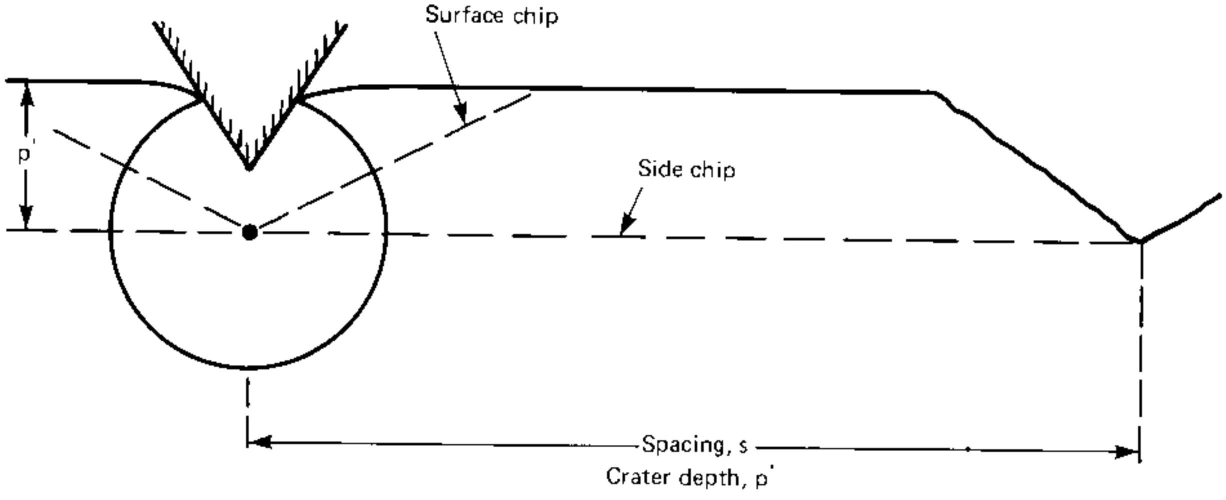


Fig. 2-21 MODE OF EXCAVATION : SIDE CHIP FORMATION WITH GROOVE INTERACTION, NON-INTERACTING SURFACE CHIP

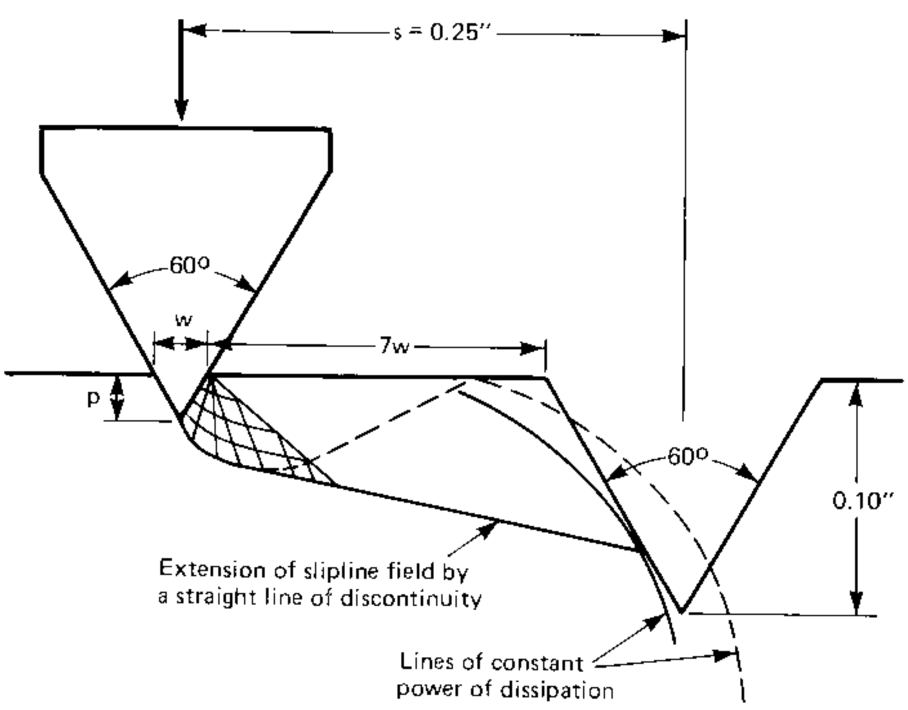
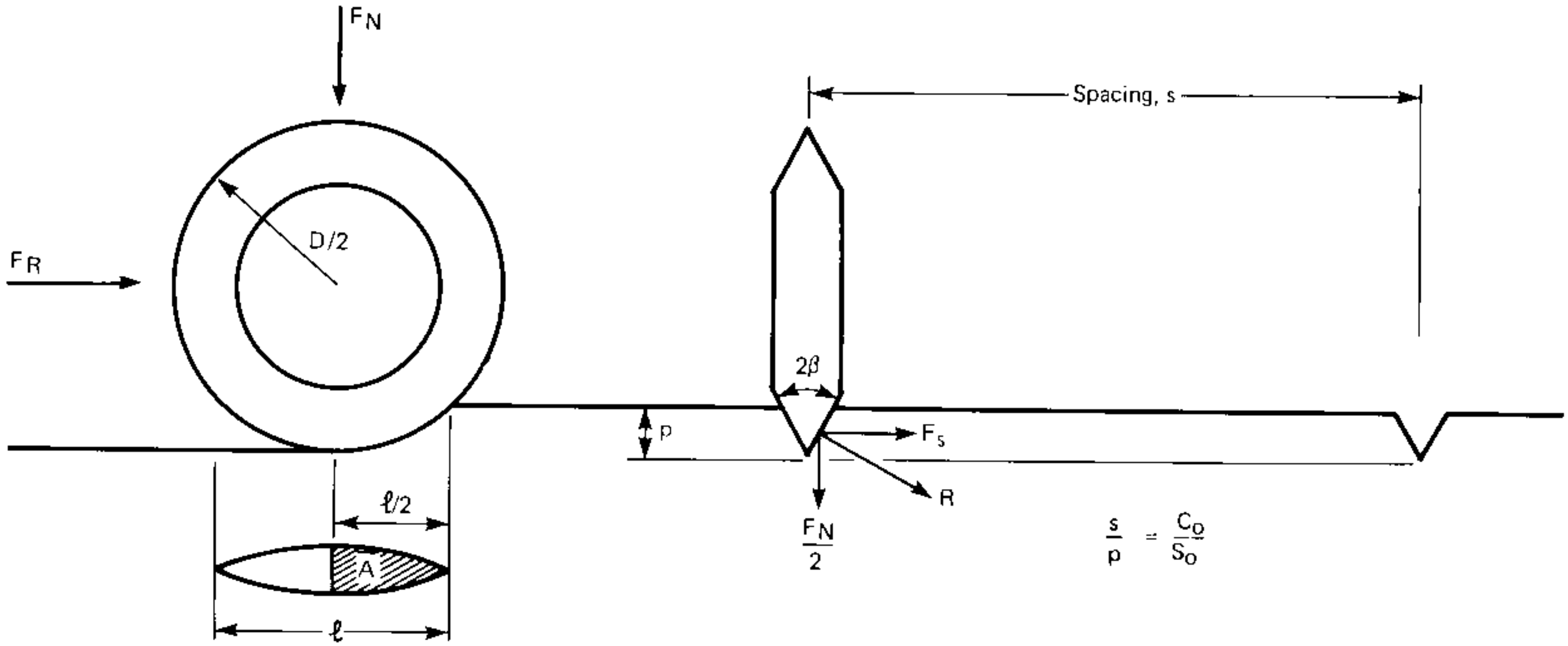


Fig. 2-22 MINIMUM POWER OISSIPATION CONCEPT APPLIED TO INDEXED (SPACED) WEDGE PENETRATIONS (After Cheatham and Gnirk, 1967)

on the ratio of spacing to penetration (crater). Even for very large spacings a side chip can eventually form after a sufficiently deep penetration. This penetration, as is often the case on TBM's employing discs, is cumulative. In other words, for widely spaced tools several passes will be made within the same groove before the critical spacing to penetration ratio is reached for side chip formation. Until that point all chips form to the surface.

Compared to the number of models describing rock-tool interaction and surface chip formation, there are relatively few considering groove interaction or indexing of tools. One of the earliest models was derived from the classical theories of plasticity (Cheatham and Pittman, 1966). By equating the power dissipation required to form a surface chip or bearing type failure and that to interact with a previous identation, the critical spacing to penetration could be obtained for a specific geometry, Figure 2-22. Employing sharp indentors the predicted ratio, between 9 and 10, agrees reasonably with practice when applied to the appropriate materials under the proper conditions (simulating ductile behaviour).

For rock excavation by disc, several investigators have proposed the basic model shown in Figure 2-23 (Roxborough and Phillips, 1975; Ozdemir et al, 1977). Applied loads are resolved into a shear and normal component depending on the bit angle and assumed contact friction. The component of normal force was equated to the compressive strength times the tool contact area and that of shear force to the shear strength over the surface area of the resultant side chip. Based on these assumptions, the spacing to penetration ratio was found to equal the ratio of compressive to shear strength. Although this was in good agreement with experimental results, it depended on the relation between normal force and compressive strength. If the force was a multiple of the strength times the contact area, as is often the case, the calculated spacing to penetration ratio is also increased by the same multiple. In essence the predicted ratio is a lower bound. The predicted forces, either rolling or normal, are directly proportional



$\ell = 2\sqrt{Dp - p^2}$

$F_N = A\,C_o$

$F_S = s\frac{\ell}{2}S_o$

$A = p\ell \tan\beta$

C_o = Unconfined compressive strength

S_o = Shear strength

Fig. 2–23 SIMPLIFIED MODEL OF DISC PENETRATION AND GROOVE INTERACTION (After Roxborough and Phillips, 1975)

to groove spacing and to shear strength while that with penetration depends on the geometry of the indentor.

As applied to the geometry of side chip formation, the pressure bulb model also reveals a similar linear relation between tool force, groove spacing and shear strength (details in Appendix II Part D). For a constant penetration with tools of increasing wedge half angle, however, the critical spacing to penetration ratio was found to increase, Table 2-2. This is attributed to the increased efficiency of the side chip mode of failure as compared to surface chip formation when employing blunt tools. It is further illustrated by the decreased ratio of specific energy for side to surface chip formation despite the larger normal force required to fail the material with wide angled tools. Both predicted trends in spacing to penetration ratio and specific energy have been observed experimentally for changes in tool geometry, Figure 2-24. Although the magnitude of the variables listed in Table 2-2 were related to the Bunter Sandstone used in the laboratory tests, no calibration of the data was attempted. As previously stated it is necessary to differentiate between tool and crater penetration for accurate prediction of force and specific energy. The ratio of spacing to penetration and of specific energy, however, does not depend on this information. As shown in Figure 2-24 and Table 2-2, the measured and predicted values of spacing to penetration ratio largely agree. The range of values listed in the Table account for an assumed damage zone surrounding the groove with which the fracture surface interacts (lower bound, no damage zone). Predicted differences in specific energy for side and surface chip formation were too large. This error was basically related to an under estimation of the surface chip volume by assuming a linear rather than curved failure surface.

SUMMARY

Despite the geometric difference between those models describing single tool-rock interaction and multiple tool groove interaction, the predicted force-penetration relations do not differ by a large amount when tools are spaced at or beyond the critical spacing to penetration

TABLE 2-2

Influence of wedge angle on spacing to penetration ratio and on specific energy

Tool wedge half-angle β^{o}	Normal force (kN)	Spacing-penetration ratio *	Specific energy side chip (MJ/m^3)	Specific energy surface chip (MJ/m^3)	Specific energy ratio: side-surface
30	16.0	6- 8	3.0	17.0	0.18
45	23.5	8-10	2.8	22.0	0.13
60	30.5	10-12	2.6	24.5	0.10

* constant penetration; lower bound does not include damage zone in adjacent groove

Bunter Sandstone: (after Roxborough ahd Phillips, 1975)

Compressive strength	49 MN/m^2
Tensile strength	2.6 MN/m^2
Shear strength	7.3 MN/m^2
Internal friction angle	22-28^{o}

ratio. Consequently, the simpler single tool interaction model can provide a lower bound for the penetration given a specific load.

For most of the models considered, the relation between force and penetration was a function of tool geometry and one or more material strength parameters (ie compressive, shear and tensile strength, and internal angle of friction). Material compaction and dilatation, size effect, anisotropy, and other factors related to rock structure are not considered. To circumvent the inadequate description of material character, assumptions and idealizations are made with respect to the geometry of rock-tool interaction, mechanisms of chip formation, and/or mode of failure. As a result, most models are applicable for a specific rock type, tool geometry, and range of penetration. Ductile materials may be adequately described by plasticity models (eg Figures 2-11 and 2-12), while dense brittle rocks by high pressure crush zones (eg Figure 2-15). Compacting transitional, ductile-brittle, materials are better suited to those assuming chip formation upon full indentation of the bit (Figure 2-13). Within the range of applicability, the models need to be calibrated against experimental results if other than rough estimates are desired.

Aside from the bit-rock contact friction, which reduces penetration for a given force, the influence of abrasion or tool wear is obtained indirectly by modifying tool shape. As contact friction is largely independent of rock type, environment, and bit angle, the effect of rock abrasivity on penetration is a relatively constant factor (Pariseau and Fairhurst, 1967). The effect of tool wear depends on the material under excavation, shape or form of the bit wear, spacing of tools and available thrust (normal force). Although wear may reduce tool penetration, the resulting groove depth can remain unchanged.

Although models are important for understanding the mechanisms of excavation and represent the future in terms of a comprehensive predictive capability, they basically over simplify the process of excavation. At present, numerical models are largely superseded by experience and laboratory tests.

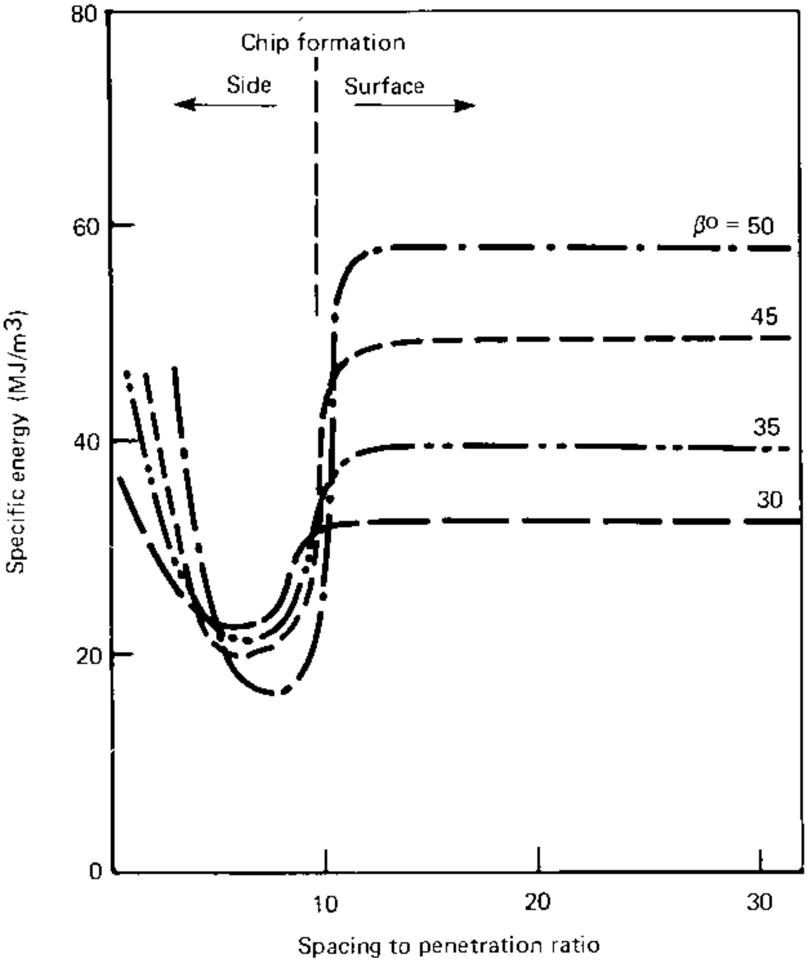


Fig. 2-24 INFLUENCE OF DISC WEDGE ANGLE ON SPACING TO PENETRATION – SPECIFIC ENERGY RELATION FOR EXCAVATION OF BUNTER SANDSTONE
(After Roxborough and Phillips, 1975)

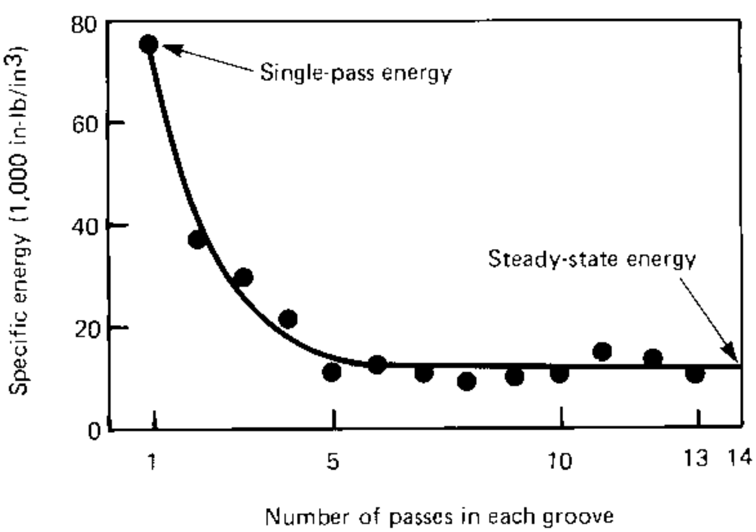


Fig. 2-25 EFFECT OF SURFACE CONDITIONING ON SPECIFIC ENERGY
(After Rad and McGarry, 1971)

PHYSICAL MODELS

Physical model tests are most often performed directly on the material of interest. This is a distinct advantage as it eliminates the need to characterize the rock, the most difficult aspect of numerical modelling. Unfortunately, working with "real" materials does not eliminate the problems associated with scale or size effect, test procedure, machine-tool-rock interaction or system stiffness, and date interpretation. Therefore, it is not surprising to find considerable disagreement in the literature with respect to optimum tool shape and arrangement of tools. Laboratory tests designed for optimization of tool parameters and prediction of penetration must simulate field conditions as accurately as possible. This includes the appropriate rock type, proper range of normal force, tool geometry, sequence of excavation, surface conditioning, and stiffness.

FULL SCALE SINGLE TOOL MODELS

Full scale single tool tests performed on linear cutters, Plate 2-3, are much too complicated and expensive for the estimation of TBM parameters on routine projects. Again, experience has to a large degree superseded large scale laboratory testing. It is, however, invaluable for research and machine design from which practical design charts and advance rate predictions may be made. Important information derived from laboratory results for various rocks and test conditions are the range of optimum spacing to penetration ratios, tool forces and hence, power and thrust requirements, influence of tool shape and type, and calibration data for numerical and scale physical models.

One of the major factors influencing laboratory test results is the conditioning of the rock surface. In the field, the cutter tools pass within the same groove time and time again as the machine advances forward. The interaction of grooves on a smooth surface (unconditioned)

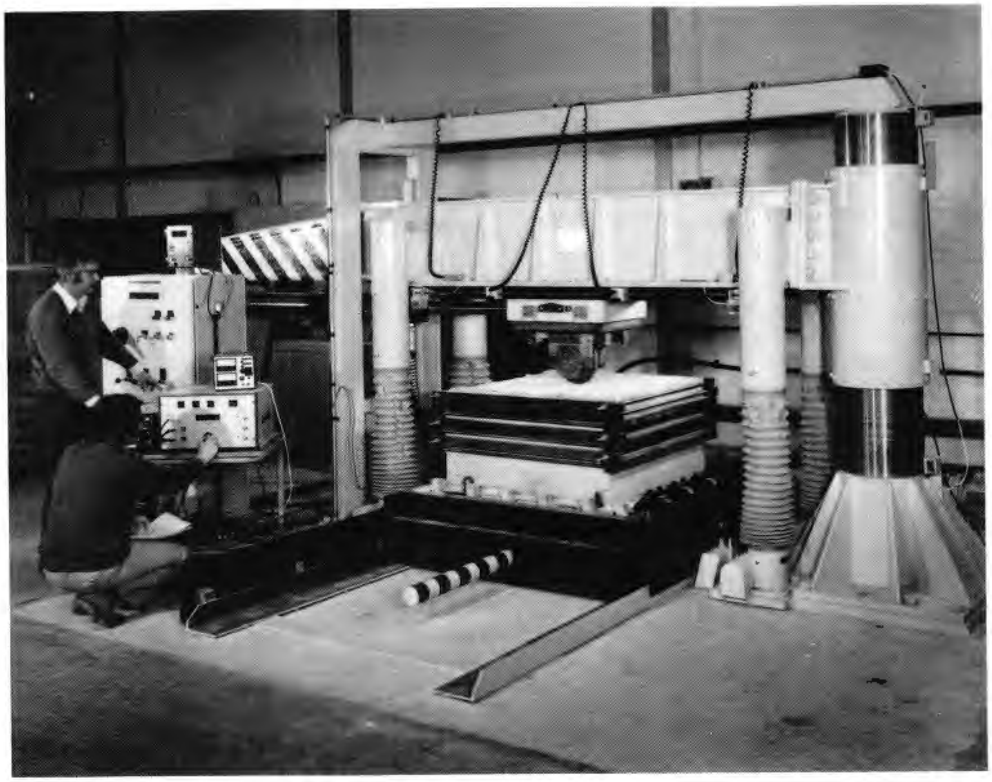


PLATE 2-3. TRRL full-scale linear cutter

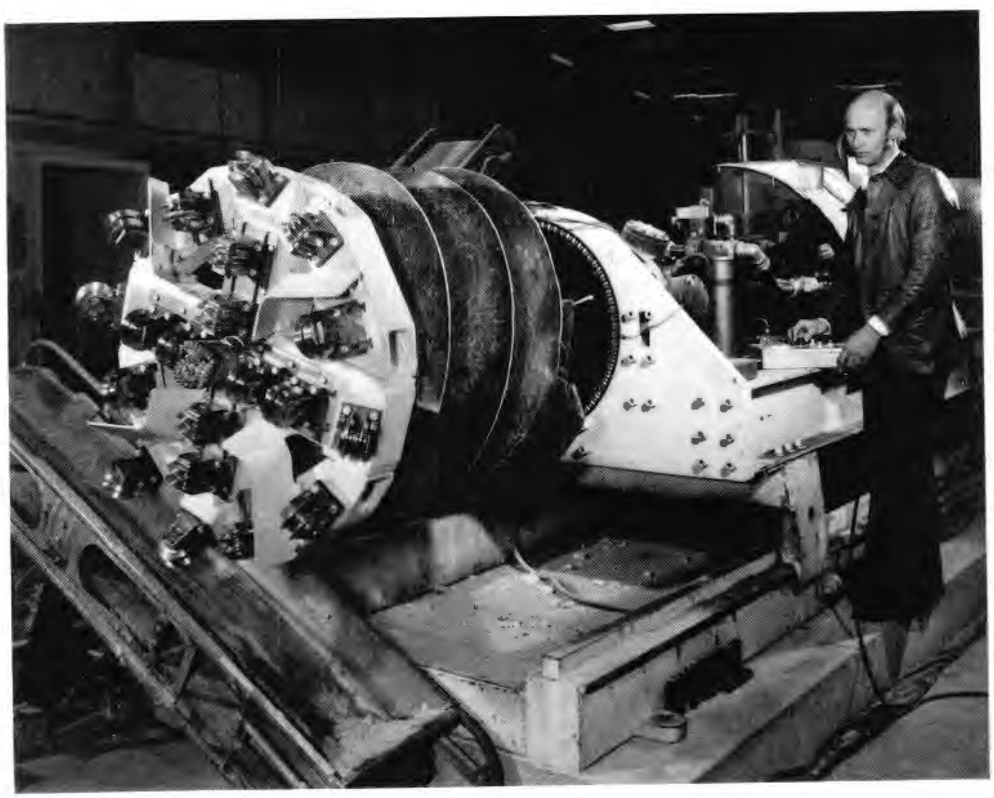


PLATE 2-4. TRRL pilot-scale machine (one meter diameter cutter head)

only represents the first revolution of a machine cutter head on the first day of operation. For the remainder of the project the machine operates in a steady-state mode. The importance of conditioning the rock surface in the laboratory depends on the type of tool and the material under excavation.

For efficient excavation with drag picks, they must fully interact with an adjacent groove at every level of penetration as shown in Figure 2-1. The same has been found for discs operating in weak and porous materials such as chalk. When the side chip does not form on the first penetration the disc tends to bury itself, using extra energy to overcome friction. If groove interaction must occur at each level of penetration, as in the examples above, surface conditioning is of reduced importance. For most rocks with compressive strengths in excess of 10 MN/m^2, however, several passes can be made within the same groove before realizing the optimum spacing to penetration ratio for side chip formation. Even if the optimum ratio occurs on the first pass, a conditioned rock surface does not have the same characteristics as a smooth or unconditioned surface. Damage caused by the previous level in terms of material crushing and fracturing has a significant influence on the response of subsequent cuts.

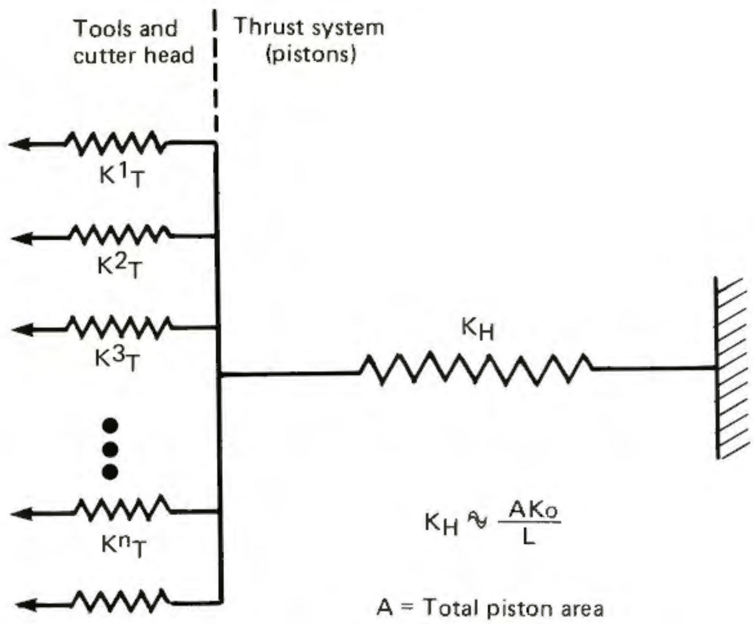
Rad and McGarry (1971) have investigated the effect of multiple passes to achieve steady state excavation. Employing a disc at near constant normal force, a series of 15 equally spaced parallel grooves were made in granite. This constitutes one pass as shown in Figure 2-25. After five passes the material yield and specific energy reached steady-state. For the optimum spacing at steady-state the specific energy was reduced by 50 percent of that found for optimum conditions when working on a smooth surface. More important, the optimum spacing to penetration ratio was also found to increase by 50 percent.

Along with surface conditioning, another major factor influencing results is the overall stiffness of the test system. Most laboratory rigs (linear or circular cutters and shaping machines) operate in either a constant penetration or constant force mode. A TBM, however, operates between these two extremes. The actual stiffness of the machine depends on the many elements in series and parallel as schematically illustrated in Figure 2-26. Of all the elements in the system the hydraulic rams used to develop the forward thrust have the lowest stiffness by a large margin. Since all elements including the tools are in series with the rams, their stiffness basically controls the overall machine behaviour. Most machines have an estimated stiffness on the order of 25 MN/m or higher.

Results derived from soft laboratory machines (normal force developed by a hydraulic ram) show a well defined optimum spacing or spacing to penetration ratio at which groove interaction is maximized, Figure 2-27. At spacings greater than critical the grooves become independent (surface chip formation). Because the system provides a constant force, repeated passes in independent grooves results in successively small amounts of debris (Rad and McGarry, 1971). Unless the normal force is increased or tool spacing decreased productive excavation will eventually cease. Although specific energy is shown to be constant for the independent excavation mode, in practice the repeated passes with little or no yield results in increased specific energy with greater tool spacing ("U" shaped curve).

Tests on a variety of rocks were carried out at a normal force of 32 kN (Rad, 1975) Despite the large difference in material strengths, all optimum spacing to penetration ratios were between 5 and 6 with recorded penetrations around 2 mm, Table 2-3. Since the rock surfaces were not conditioned for reasons of expedience, the optimum ratios for prepared surfaces were most likely between 8 and 10. The basic response as illustrated in Figure 2-27 would be largely unchanged whether the surface was conditioned or not on account of the test mode employed (constant force).

Employing a relatively stiff machine (96 MN/m), a similar series of tests were performed on various rocks covering the same range of



$$K_H \approx \frac{AK_o}{L}$$

A = Total piston area
L = Piston stroke
Ko = Bulk modulus of oil

$$K_M = \frac{nK_T K_H}{nK_T + K_H} \quad \text{since } K_T \gg K_H$$

K_M(machine stiffness) $\approx$ K_H(thrust system stiffness)

K_T = Tool and cutter head stiffness

n = Number of tools

Fig. 2-26 IDEALIZED SCHEMATIC OF TBM SYSTEM STIFFNESS

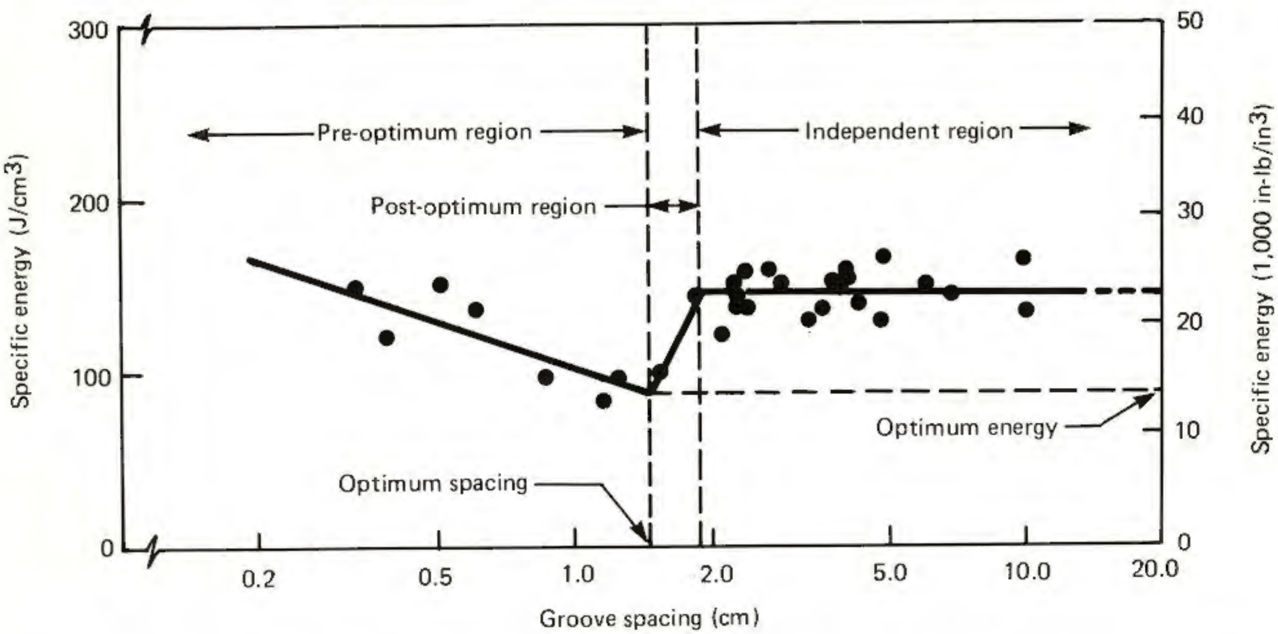


Fig. 2-27 DISC GROOVE SPACING – SPECIFIC ENERGY RELATION FOR EXCAVATION OF GRANITE AT CONSTANT NORMAL LOAD (No surface conditioning; after Rad, 1975)

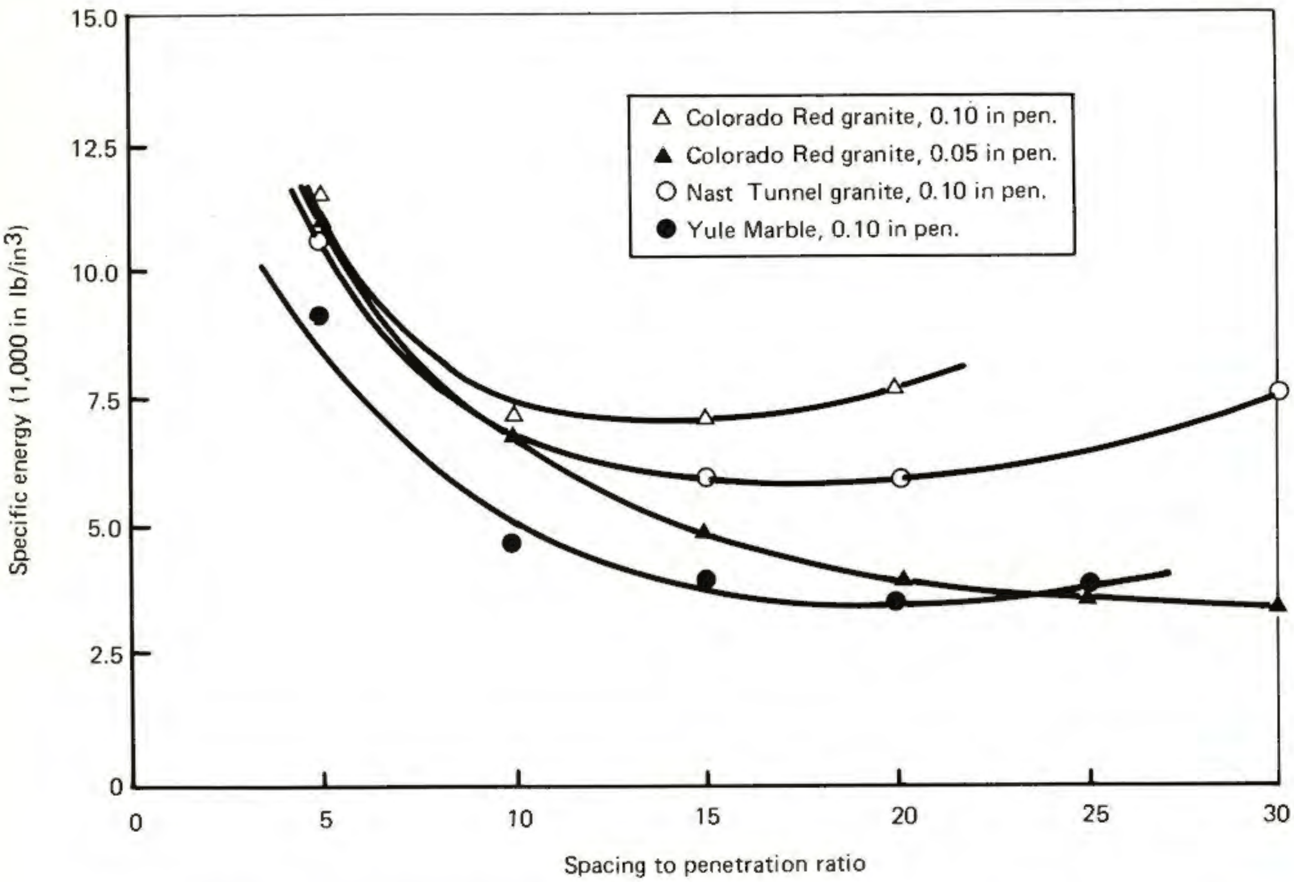


Fig. 2–28 SPACING TO PENETRATION RATIO – SPECIFIC ENERGY RELATION FOR FULL SCALE DISC EXCAVATION OF VARIOUS ROCK TYPES (After Wang et al, 1974; partly after Ozdemir et al, 1977)

TABLE 2-3

Compilation of full-scale linear cutter test results for disc excavation

Rock type	Compressive strength (MN/m^2)	Penetration (mm)	Optimum spacing-penetration ratio (1.5X optimum)	Optimum specific energy (MJ/m^3)	Specific energy-compressive strength ratio	Test conditions	Reference
Tennessee marble	71	2.2	6.3 (9.5)	60	.83	Low normal stiffness (hydraulic system), constant force $\simeq$32 kN, unconditioned surface	Rad, 1975
Limestone	108	2.7	6.5 (9.8)	43	.40		
Charcoal gray granite	183	2.0	5.3 (8.0)	88	.48		
Jasper quartzite	559	1.5	5.9 (8.9)	103	.18		
Colorado red granite	144	2.6	12	48	.34	High normal stiffness (K $\approx$96 MN/m^2), constant penetration, conditioned surface	Wang et al, 1974
Nast granite	137	2.6	15	40	.29		
Yule marble	55	2.6	20	23	.43		
Colorado red granite	138	1.3	>25	23	.17		Ozdemir et al, 1977
Bunter sandstone	49	6	7 (10.5)	20	.41	High normal stiffness (K unknown), constant penetration, unconditioned surface	Roxborough and Phillips, 1975
Lower chalk	1-3	30	3	1.2	.40	Machine trial	Hignett et al, 1977

strengths considered above (Wang et al, 1974; Ozdemier et al, 1977). With the exception of one test, the optimum spacing to penetration ratios derived from the conditioned surfaces were in excess of 15, Figure 2-28 and Table 2-3. Two tests carried out on the same rock, Colorado Red Granite, using the same machine revealed considerably different results. Although the penetrations were not the same, this was not a likely explanation of the difference as the curve with the lowest specific energy was obtained from tests at smaller penetrations (specific energy normally increases with decreasing penetration). If an insufficient number of passes are made for tests performed at large spacing to penetration ratios, it is possible that excessively high forces and consequently, specific energy will be recorded as averages for the steady-state condition. This results from the large number of passes required to reach the critical ratio for sidechip formation when grooves are widely spaced (for example 3 to 5 passes at a spacing to penetration of 30). The investigator who carried out the tests revealing an optimum ratio in excess of 30 and a specific energy of one-half that previously obtained, claimed to perform a large number of passes at large groove spacings to reduce this error. With this evidence, it is also possible that the optimums shown for the Yule Marble and Nast Granite are related to test procedure rather than true optimum groove interaction. In any case, there is relatively little change in curve slope for ratios in excess of 15.

The difference between the results obtained from the relatively soft and stiff testing machines can be interpreted with the aid of the idealized relations shown in Figures 2-29 and 2-30. Consider the first pass on an unconditioned surface for a constant penetration mode of excavation. As shown, groove interaction ceases at a spacing slightly greater than that determined from the optimum spacing to penetration ratio. This behaviour is identical to a constant force mode of excavation at applied load F_1. At the optimum spacing the minimum specific energy, Figure 2-30, is relatively high for the applied load on account of the large amount of energy employed to develop a crushed zone relative to the reduced material yield at low tool spacing. This is represented by the intercept, F', at zero

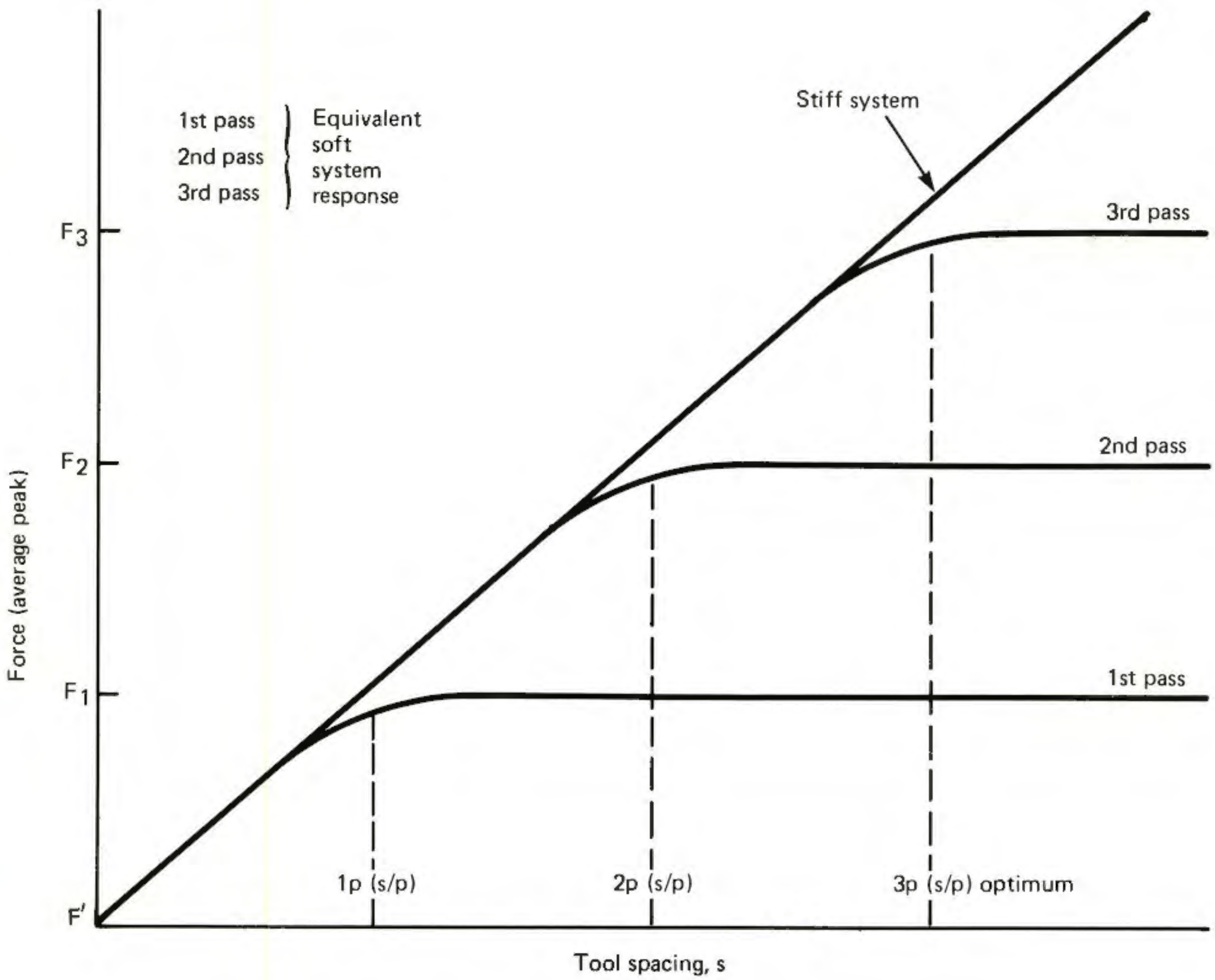


Fig. 2-29 IDEALIZED FORCE-SPACING RELATION (p = penetration)

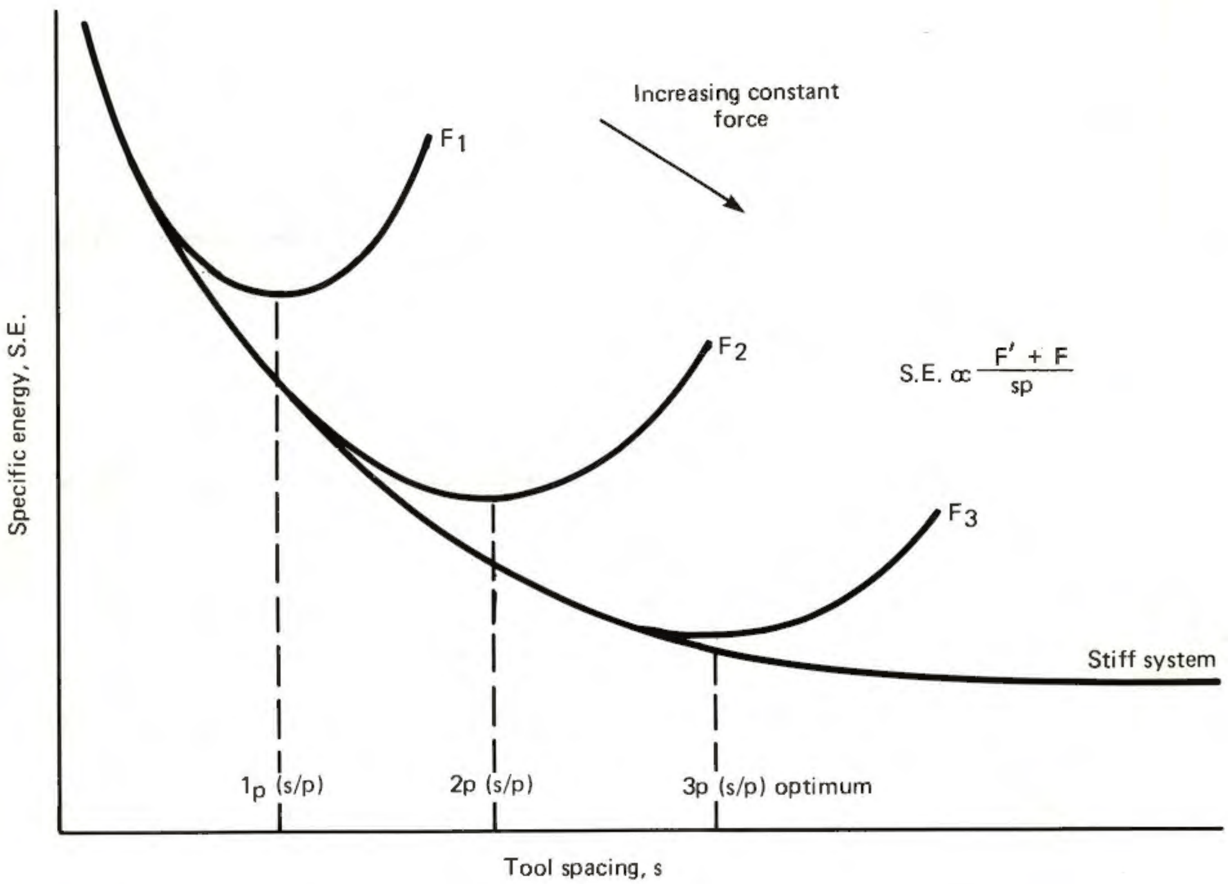


Fig. 2-30 SPECIFIC ENERGY – SPACING RELATION DERIVED FROM IDEALIZED RESPONSE DISPLAYED IN Fig. 2-29

spacing.

With subsequent passes, the optimum tool spacings are related to multiples of the characteristic optimum spacing to penetration ratio of the material. Again, the response is equivalent to tests conducted at the appropriate constant force (F_1, F_2, F_3, ...). As shown in Figure 2-30, the set of minimum specific energies derived from the constant force tests essentially map the results obtained from multiple passes at constant penetration. For a large tool spacing, as the number of passes increases to a steady state condition, the specific energy asymptotically decreases to a constant value (related to the material fracture surface energy).

Results derived from die bit (blunt tool) penetrations of Indiana Limestone at various impact energies (similar to a constant force) and groove spacings (indexing distance), Figure 2-31, essentially confirms the basic form shown in Figure 2-30. Based on this and similar data, a linear relation between spacing and applied energy has been proposed (Hustrulid and Fairhurst, 1971). This is the same relation displayed in Figure 2-30, where force is proportional to applied energy.

The linear force-spacing relation and the monotonically decreasing specific energy relation represent the idealized material response (derived from a stiff test system). It suggests that no optimum tool spacing exists provided the TBM is stiff, has unlimited thrust capacity and the tools have unlimited ability to make use of the applied force. As most efficient discs are designed to operate at near 10 tons normal force (20 ton maximum) and machines have limited stiffness, actual operating characteristics are between the two extremes considered.

For example if the force, F_2, represents the average thrust per cutter the optimum tool spacing may occur at the position shown in Figure 2-32. The degree of increased efficiency displayed by the TBM over that revealed by the constant force test is largely attributed to machine stiffness. With greater stiffness the larger peak forces, responsible for major chip formation, are mobilized. If the system is

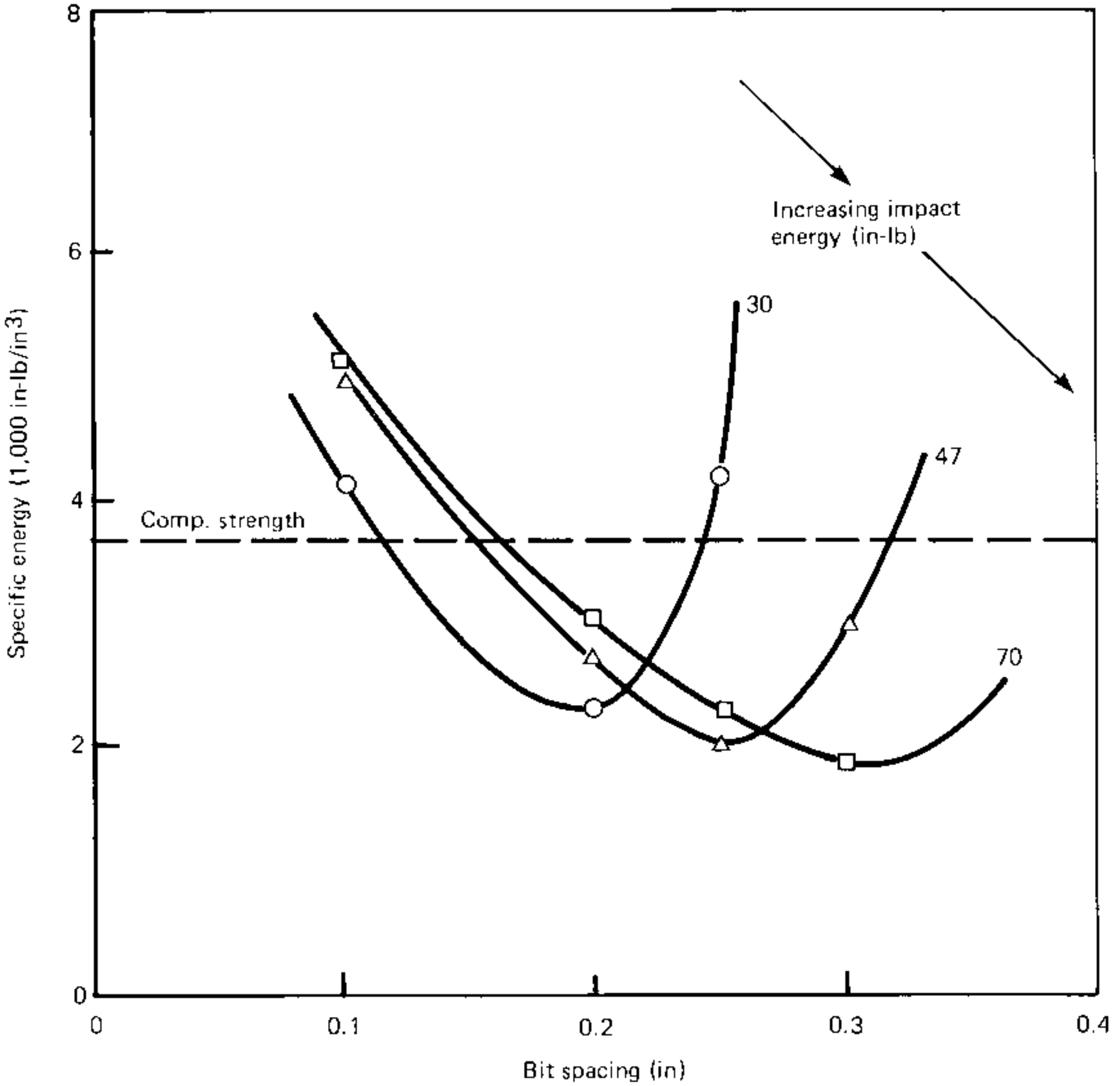


Fig. 2–31 RELATION BETWEEN SPECIFIC ENERGY AND BIT SPACING AS A FUNCTION OF IMPACT ENERGY (0.3 in die bit impacting Indiana limestone; after Hustrulid and Fairhurst, 1971 and Simon, 1963)

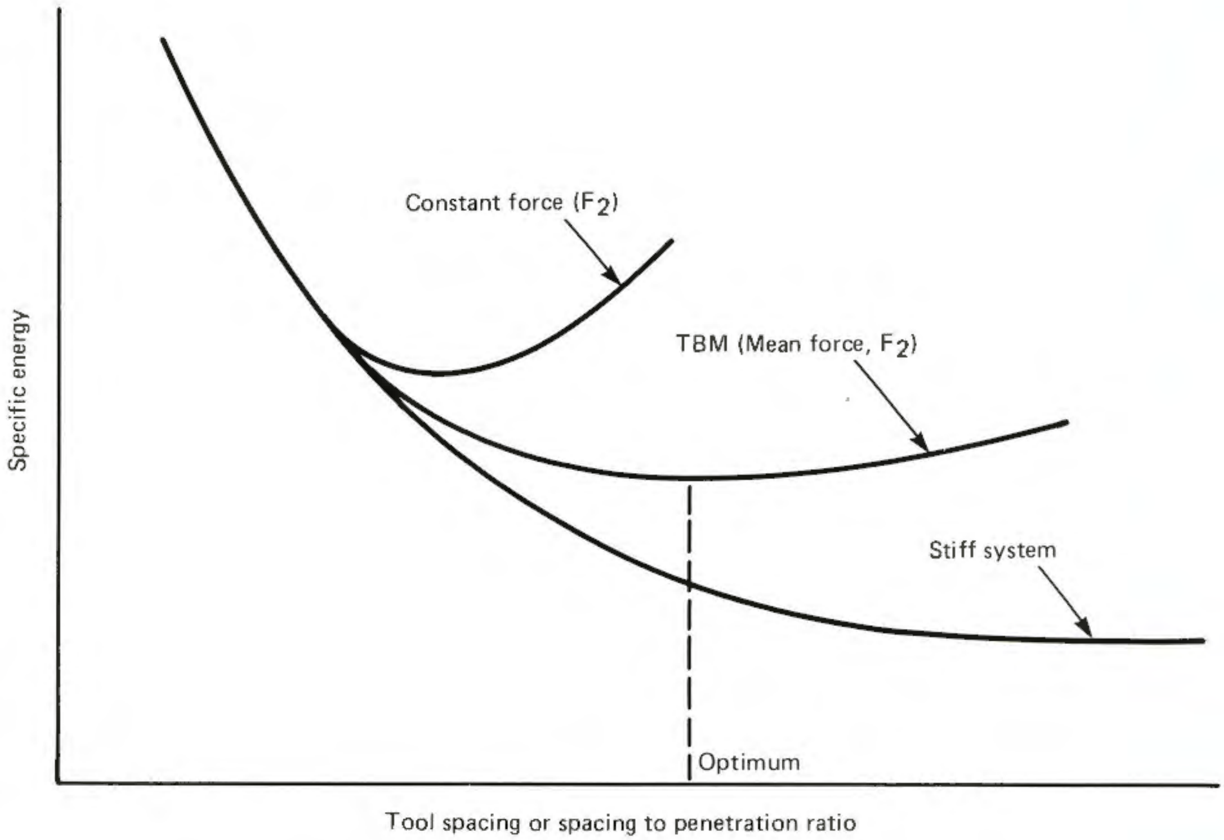
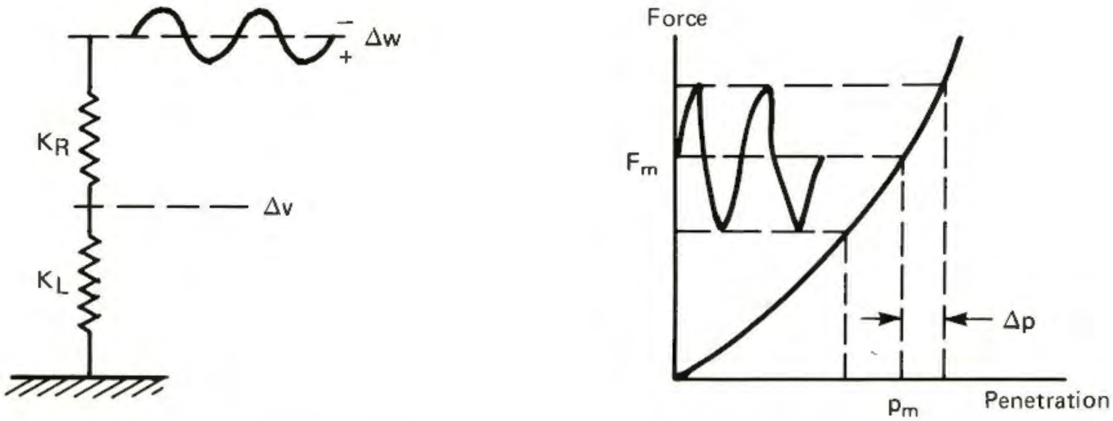


Fig. 2-32 INFLUENCE OF STIFFNESS ON OPTIMUM TOOL SPACING AND MINIMUM SPECIFIC ENERGY

too soft for a particular tool spacing and applied thrust, the tool will rebound from the face rather than remove the chip. To achieve a reasonable advance rate the thrust has to be increased or tool spacing decreased.

The influence of laboratory machine normal stiffness on penetration can be quantified with the simple model shown in Figure 2-33. Given a groove depth variation, Δw, formed as a result of the discontinuous nature of chip formation, the subsequent effect on tool displacement, Δv, depends on the relative stiffness of the rock and machine. As machine stiffness decreases relative to that of the rock, absolute change in penetration ($\Delta w - \Delta v$) also decreases as does the ratio of peak to mean force. Laboratory tests with stiff systems reveal peak forces of 50 to 100 percent greater than mean values, Figure 2-19. This leads to a substantial increase in overall penetration as compared with a soft system employing the same mean normal force. The analogous model of TBM response is complicated by the multiple interaction of the tools, however, the stiffness of the thrust system is a major controlling factor, Figure 2-26. A good indicator of overall system stiffness is the measured peak to mean normal force ratio. Rock strength variations (variable K_R) due to inhomogeneities or a mixed face result in a similar effect as that displayed by groove depth fluctuations.

Laboratory investigations into the relative effect of changes in tool geometry provide information for the selection and arrangement of cutters and the influence of their wear on machine performance. Variation of disc diameter and speed has a relatively minor effect on tool forces and performance in terms of specific energy (Roxborough and Phillips, 1975; Ozdemir et al, 1976). Large diameter discs are required for operation at high normal loads on account of the increased space devoted to bearings. Increased disc wedge angle results in greater normal load for similar penetrations, although, the specific energy remains largely unchanged for increased spacing to penetration ratios. In other words, if blunt tools are to work effectively,



Δp = Tool penetration
Δw = Groove depth
Δv = Tool displacement
(variation about mean)

F_m = Mean normal force
p_m = Mean penetration
K_R = Indentation stiffness of rock (force-penetration curve)
K_L = Laboratory test rig normal stiffness

$$\Delta p = \Delta w - \Delta v$$

$$\Delta v = \Delta w \left(\frac{K_R}{K_R + K_L} \right)$$

Cases: (i) $K_L \gg K_R,\ \Delta p \rightarrow \Delta w$

(ii) $K_L = K_R,\ \Delta p = \frac{1}{2}\Delta w$

(iii) $K_L \ll K_R,\ \Delta p \rightarrow 0$

Fig. 2–33 EFFECT OF LINEAR CUTTER STIFFNESS ON INDENTOR FORCE – PENETRATION RESPONSE

both spacing and thrust per disc must be increased. It should be emphasized that the overall influence of tool shape on machine performance will depend on the available thrust per tool and the system stiffness.

Type of tool employed has a large effect on the machine penetration per revolution. Relative efficiency of a particular tool or arrangement of tools is basically related to the size distribution of the fragments produced. The greater the number of fragments per unit volume, the larger the amount of energy required to produce them (Rittinger, 1867). As shown in Figure 2-34, the material yield obtained from efficient excavation by disc reveals an energy intensive proportion of small sized particles derived from the crushed zone and the more efficient yield produced by chip formation. Strawberry button cutters, often employed for the excavation of very strong materials, use a disproportionate amount of energy on rock crushing as compared with chip formation. At normal operating thrust, penetration is only on the order of 1 mm. Specific energy was shown to be nearly three times that required by a disc at optimum conditions (Wang et al, 1974).

Despite the significant difference in specific energy or penetration displayed by various tools, no conclusion with respect to their use can be made without a consideration of tool wear. Even though wear affects tool penetration as the shape is altered, wear is primarily a question of economics. Tool cost, machine advance rate and utilization are all included in the final calculation which is very dependent on the particular machine, project, and rock type (to be considered in later sections).

Laboratory tests with double and triple discs show significant interaction at spacing to penetration ratios of less than 20 (Ozdemir et al, 1976). Results were obtained in granite and marble and compared with single disc behaviour under similar conditions. Interaction is caused by the simultaneous formation of side chips into the adjacent crushed zones, thus promoting less efficient surface chip formation instead. This problem can be reduced by wide spacing of the

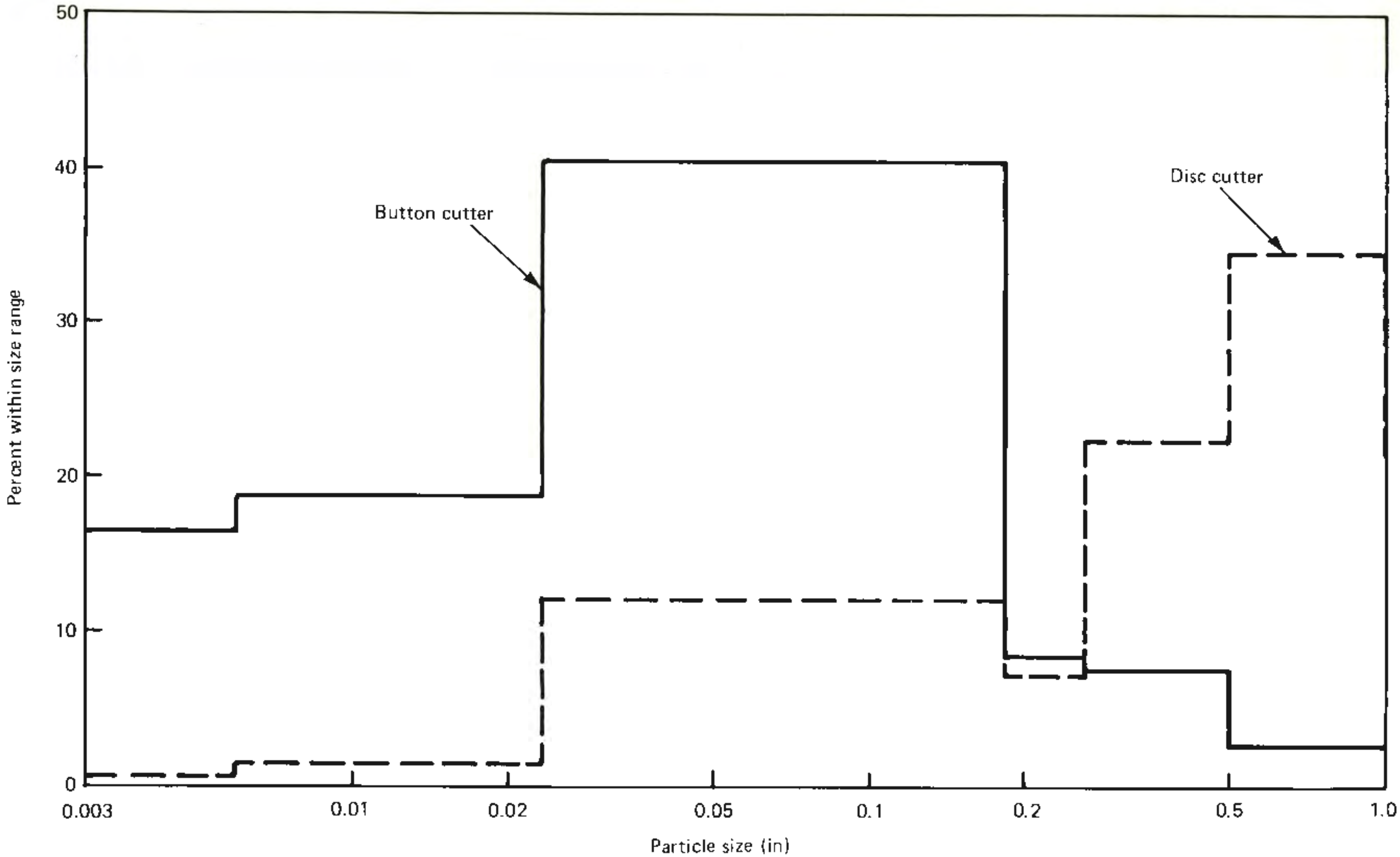


Fig. 2-34 PARTICLE SIZE DISTRIBUTION FROM STRAWBERRY BUTTON CUTTER AND DISC CUTTER DEBRIS (Partly after Norman and Dye, 1978)

tool edges or kerfs and selection of an appropriate sequence of excavation. An appropriate sequence will allow for side chip formation into a previously created groove spaced midway between the multiple disc edges. Scuffing caused by the unequal distance travelled by each edge of a multiple disc (locked design) has a large effect on tool wear, but most likely a minor influence on penetration.

SINGLE TOOL SCALED MODELS

Model tests performed at reduced depths of tool penetration and force are considerably more convenient and less expensive than full scale investigations. As for all physical model tests, it is important to simulate field conditions as accurately as possible. The scale effects associated with a reduced depth of penetration are dependent on many factors including machine stiffness. Differences in dimensionless ratios, such as optimum spacing to penetration, Figure 2-35, are influenced by the stiffness of the test rig and to an unknown extent, material size effect possibly related to stress gradients. Size effect has a definite influence on specific energy as illustrated by the 50 percent drop in energy for a three fold increase in penetration, Figure 2-35.

Prediction of prototype behaviour from scale model results requires the use of a numerical model (or scaling law). Consequently, the accuracy of the prediction is based not only on the quality of the laboratory work, but on the particular numerical model employed. The difficulties and limitations associated with the use of these models has been discussed. They can be used to provide a rough estimate of force and penetration over a limited range as long as the geometric scale factor between model and prototype is not too great. Comparison of full scale results with predicted forces for a range of groove spacings created by a disc in granite has essentially confirmed the limited applicability of the method (Hustrulid, 1972; Ozdemir et al, 1977). The predicted slope of the force-spacing relation was less than one half that of the full scale laboratory results. As used to predict prototype

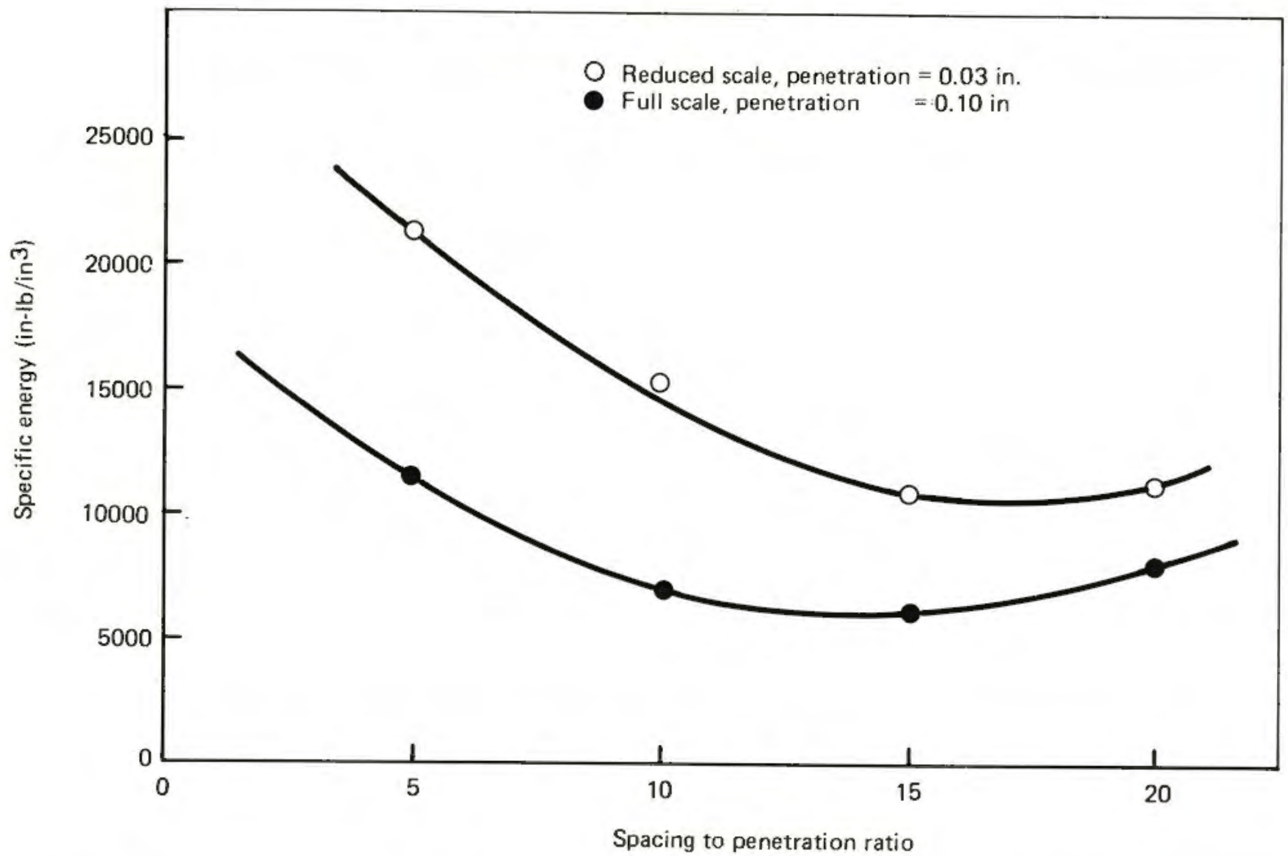


Fig. 2-35 COMPARISON OF FULL AND REDUCED SCALE LINEAR CUTTER RESULTS PERFORMED ON GRANITE (After Wang et al, 1974)

behaviour, the future of scaled physical models depends on improved numerical models based on a fundamental understanding of the mechanisms involved in the process of mechanical excavation.

PILOT SCALE MODELS

Laboratory tests with scale models of TBM cutter heads have been used for optimization of tool arrangement, Plate 2-4 (Robbins, 1956; Hignett and Howard, 1974). The prediction of penetration rate or specific energy from these results has the same limitations previously considered for single tool models. In addition, pilot scale machines often use a disproportionate amount of energy for the excavation of the gauge region (intersection between face and tunnel wall). Consequently, the smaller the model the greater the specific energy of excavation, exclusive of any size effect.

Despite the limitations, pilot scale models are useful to illustrate relative behaviour when compared with a particular standard. One aspect of tool arrangement, the effect of cutter head cone angle on specific energy, has been investigated in this manner, Figure 2-36. To maximize the benefit derived from groove interaction, the relative depth of the groove formed by the indenting tool should be the same as that of the previously created groove to which interaction occurs. This simply results from the fact that side chip formation is a more efficient mode of excavation than chip formation toward the surface. To achieve the proper geometry for optimizing interaction, it is necessary to angle the cutter head. The particular angle depends on the penetration or depth of cut, tool spacing, and relative circumferential position of each tool. For a one-quarter revolution, rearward, pick offset, the geometry for the calculation of cone angle is shown in Figure 2-37. On account of the relatively high tool spacing to penetration ratio employed by most machines operating in hard rock, the calculated angle is less than a few degrees. More important than the magnitude of the cutter head cone angle is the alignment of each tool with respect to its neighbour. Tools placed

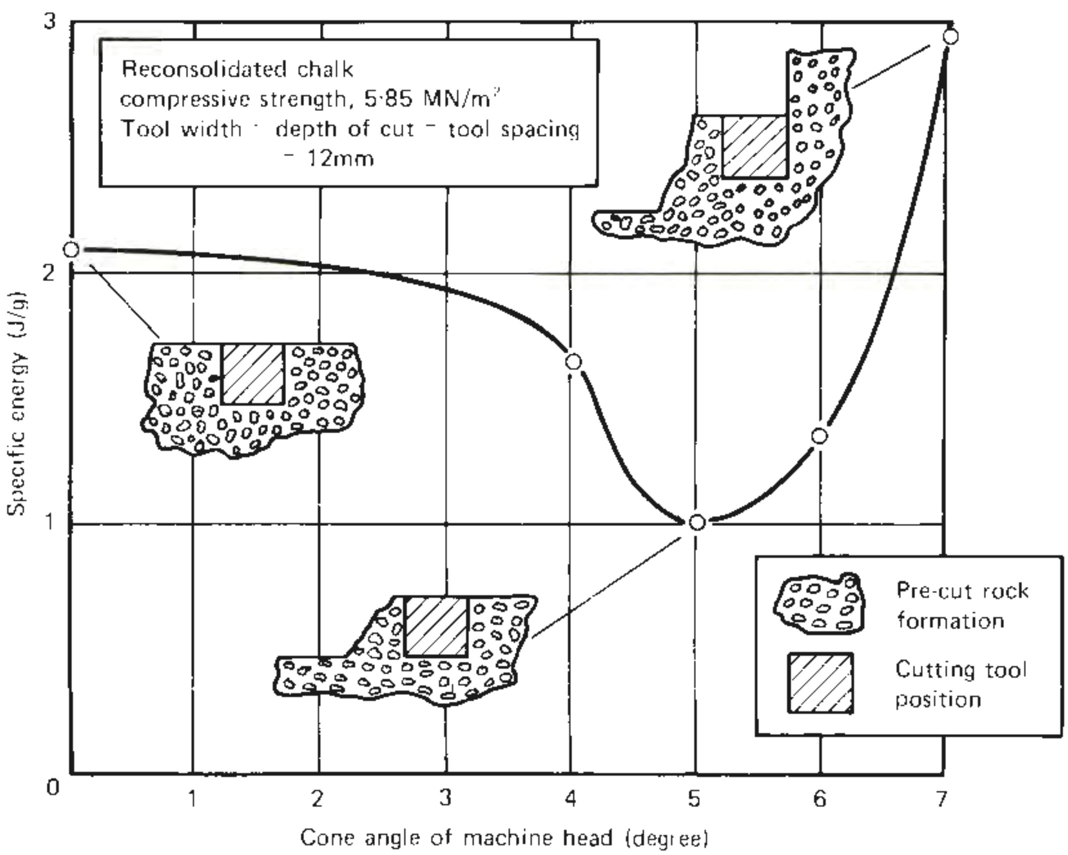
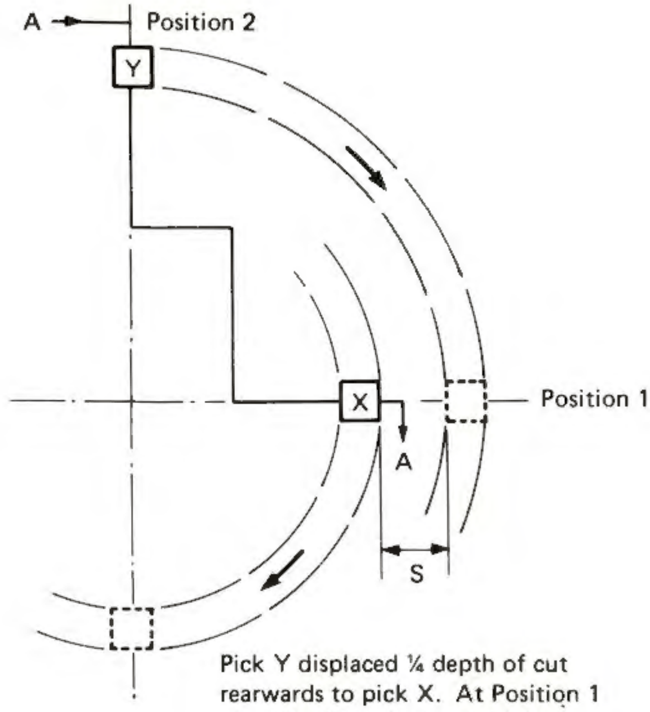
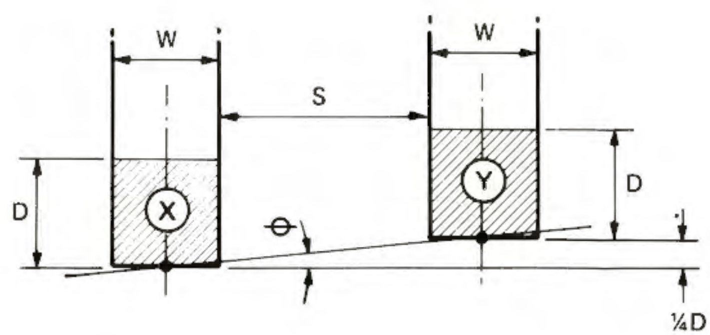


Fig. 2–36 EFFECT OF TOOL ARRAY ON SPECIFIC ENERGY, PILOT SCALE RESULTS (After Hignett and Howard, 1974)



Pick Y displaced ¼ depth of cut rearwards to pick X. At Position 1 pick Y will cut into relief left by pick X and will continue to do so



Section AA

Displacement of pick X relative to pick Y

$$\text{Tan } \Theta = \frac{D}{4(S + W)}$$

Θ = cone angle

D = depth of cut

S = clear space between picks

W = width of pick

Fig. 2–37 EFFECT OF CONING CUTTING HEAD (After Hignett et al, 1977)

too far forward or back from the face result in the same effect as that produced by a large over coning of the head with increased tool wear and reduced penetration. Examples of this will be given in the next section.

FULL SCALE PROTOTYPES

An operating TBM is the ultimate source of information for the calibration of various predictive models and for determining those factors which influence penetration rate and performance. Specific factors, such as rock mass character or discontinuities, are meaningful only when evaluated under field conditions. Unfortunately, the number of variables involved and their interaction makes it difficult to quantify the observed response. As an aid, several investigators have tried to instrument machines with limited success.

One of the more successful investigations has been carried out by the Transport and Road Research Laboratory (TRRL), England (Hignett et al, 1977). The shield machine was designed to operate in weak rock (Lower Chalk, average compressive strength of 1 to 3 MN/m^2), employing either drag picks or discs, Figure 2-38. A large number of tests were performed revealing optimum conditions dependent on tool geometry and arrangement of cutters. When employing discs the optimum spacing to penetration ratio was approximately three at a penetration of 30 mm, Figure 2-39. Unfortunately, these results are only applicable to weak, porous materials (semi-ductile behaviour, no formation of a pressure bulb). For this material, it is of interest to note the reasonable agreement of laboratory and field results, particularly the influence of head cone angle (optimum at 7 degrees for field, 5 degrees for pilot scale model, Figure 2-36). Also, measurement of tool forces at various positions on the cutter head indicated that drag picks located at gauge operated with forces of four to six times those near the centre. This substantiates the need for additional cutters at gauge positions and the observed increased tool wear.

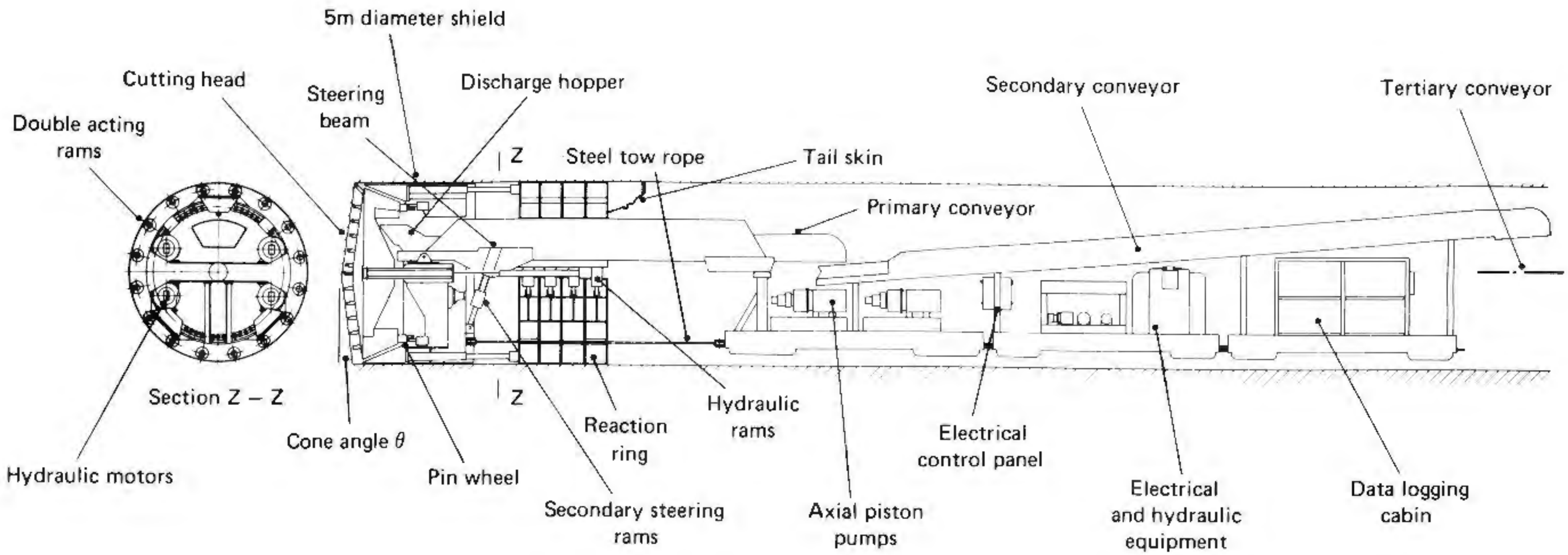


Fig. 2–38 GENERAL LAYOUT OF SHIELD TBM EMPLOYED IN WEAK ROCK (After Hignett et al, 1977)

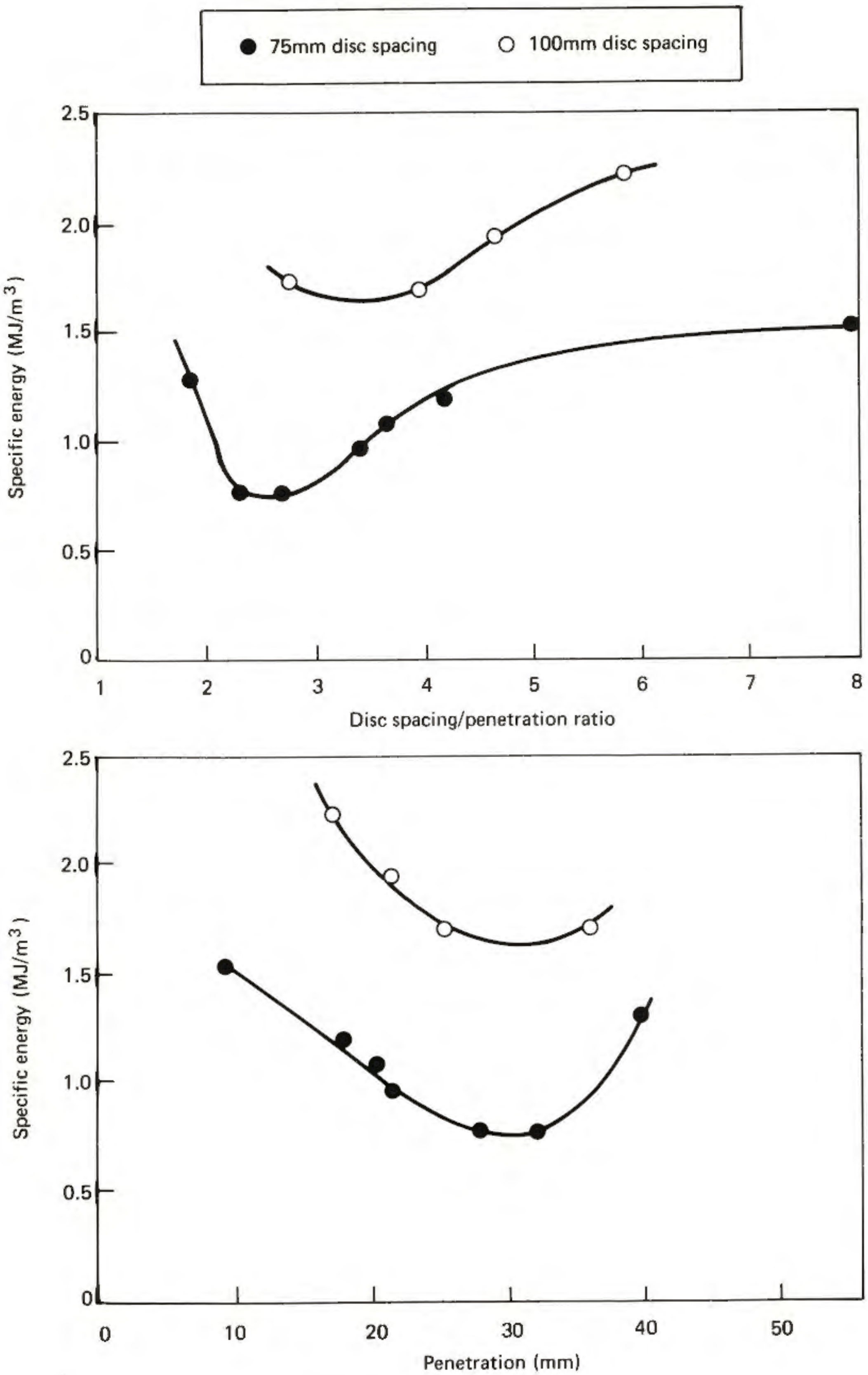


Fig. 2–39 EFFECT OF DISC SPACING AND PENETRATION ON SPECIFIC ENERGY, EXCAVATION OF CHALK (After Hignett et al, 1977)

Additional work in medium strength coal measures rock was attempted by TRRL at Dawdon Colliery, England (Snowdon, 1979). Difficult ground and corrosive ground water forced the abandonment of the trials. In Germany, a fully instrumented 6 m diameter Demag TBM is also operating in coal measures rock. At present, the results are not available (Henneke, 1978).

Considerable practical information can be gained without resorting to extensive instrumentation. Most machines have simple indicators of average thrust and power which can be used to monitor relative changes in operating conditions. The indicators are relative as they include all thrust losses associated with friction (if required to pull backup equipment) and power losses to rotate the cutter head and for debris removal. When absolute values are required, the "no load" thrust and power can be subtracted from the recorded values. Since thrust or normal force is largely proportional to rolling or cutting force (cutting coeficient, rolling to thrust force ratio is approximately 0.1), the power to excavate is also proportional to thrust. It follows that specific energy is directly related to thrust divided by machine advance rate. Simply by plotting these parameters (specific energy or thrust against advance rate) the relative performance of the machine can be evaluated for a specific set of operating conditions.

Every TBM in field operation has a unique set of inherent characteristics and operational conditions which are combined to influence the overall machine response or performance. Inherent characteristics are largely concerned with machine design and stiffness; number, type, spacing and arrangement of tools; available thrust and type of thrust system; and available power to the cutter head. Operating conditions depend on the rock mass characteristics, thrust employed, and condition of the tools. Since it is not possible to consider the full range of variables in the laboratory, simple experiments with the TBM can provide the necessary information to assess performance.

For a specific set of machine characteristics and operational conditions, Figure 2-32 reveals an optimum specific energy at a particular tool spacing or spacing to penetration ratio. Once in the field, however, most machines employ a fixed tool spacing unless there is reason to believe advance rate can be greatly improved. Within the range of operational conditions, thrust can be readily varied to further optimize performance for a given tool spacing. The TBM response, Figure 2-32, is only one of a family of curves at different values of constant thrust. On considering the family of curves, Figure 2-40, it is possible to map the relation between penetration (advance rate), mean normal force (average thrust), and specific energy (proportional to thrust divided by advance rate) for a particular fixed tool spacing. As shown an optimum thrust, F_3, is predicted. Any change in the machine characteristics or operating conditions will shift the optimum operation thrust and the specific energy.

As a practical example, consider the penetration-thrust relation, Figure 2-41, derived from a 6.4 m diameter TBM operating in sound granite (compressive strength of 150 MN/m^2) and equipped with discs (Innaurato et al, 1976). Not until the average thrust reaches a "critical" value, in excess of 350 tons, does the advance per revolution become reasonable. Below this thrust the pressure developed beneath the tool is inadequate to form a pressure bulb. The majority of the energy is expended on local crushing at the bit-rock interface and friction. With the developing pressure bulb, penetration increases rapidly until a minimum specific energy is attained (at 1.5 cm per revolution). This optimum point is controlled by machine characteristics and operational conditions as previously described. A further increase in thrust results in additional unnecessary crushing and friction, and accordingly increased tool wear, for a minor improvement in advance rate. If a greater penetration rate than indicated by the optimum specific energy is desired, machine characteristics should be altered such as increased tool spacing. Obviously, the machine must have the necessary power, thrust, and stiffness for the change to be effective. It may also be of interest to note that the optimum point can be surmised from the inflection point of the thrust-penetration

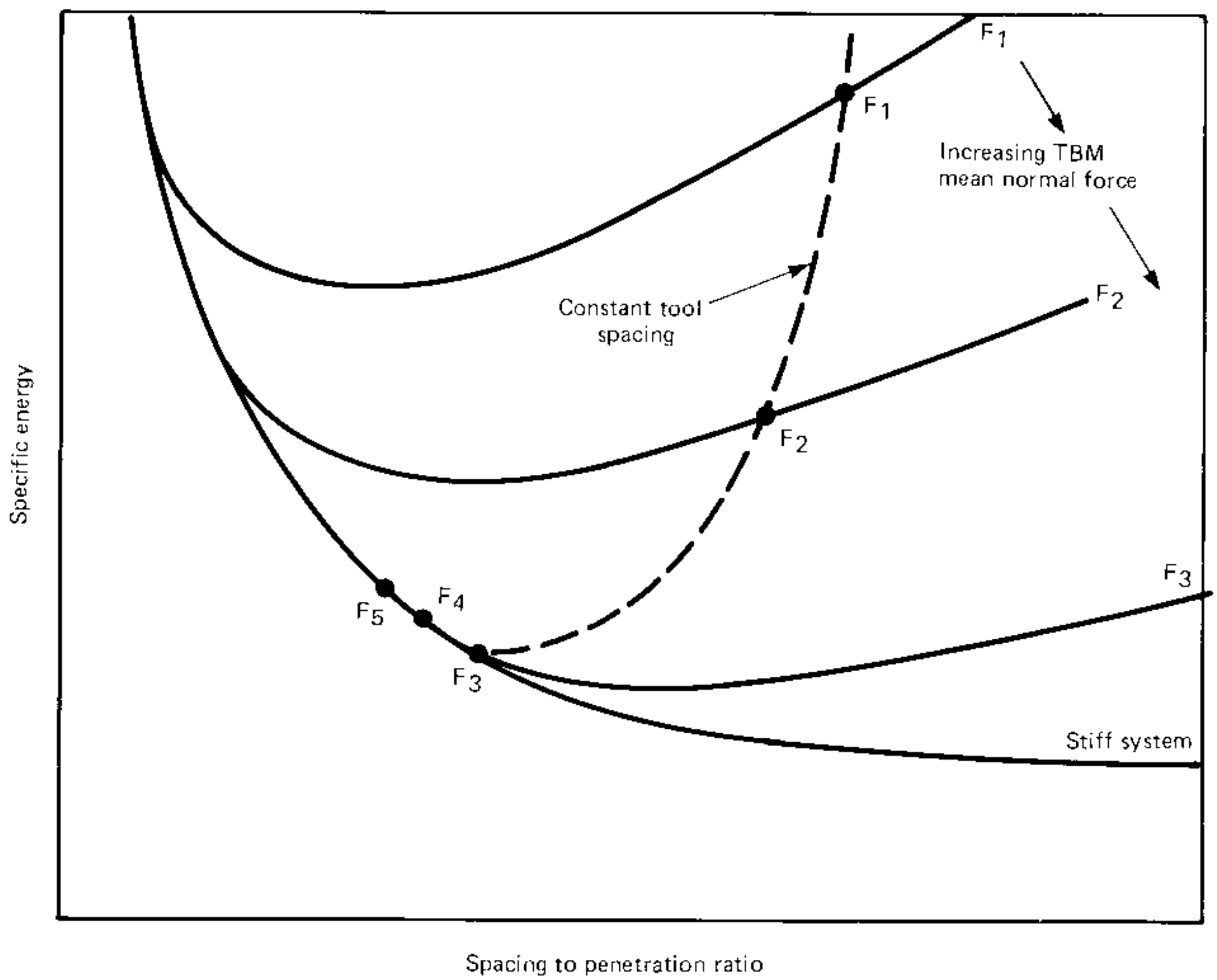


Fig. 2-40 MAP OF TBM RESPONSE FOR INCREASING THRUST AT CONSTANT TOOL SPACING

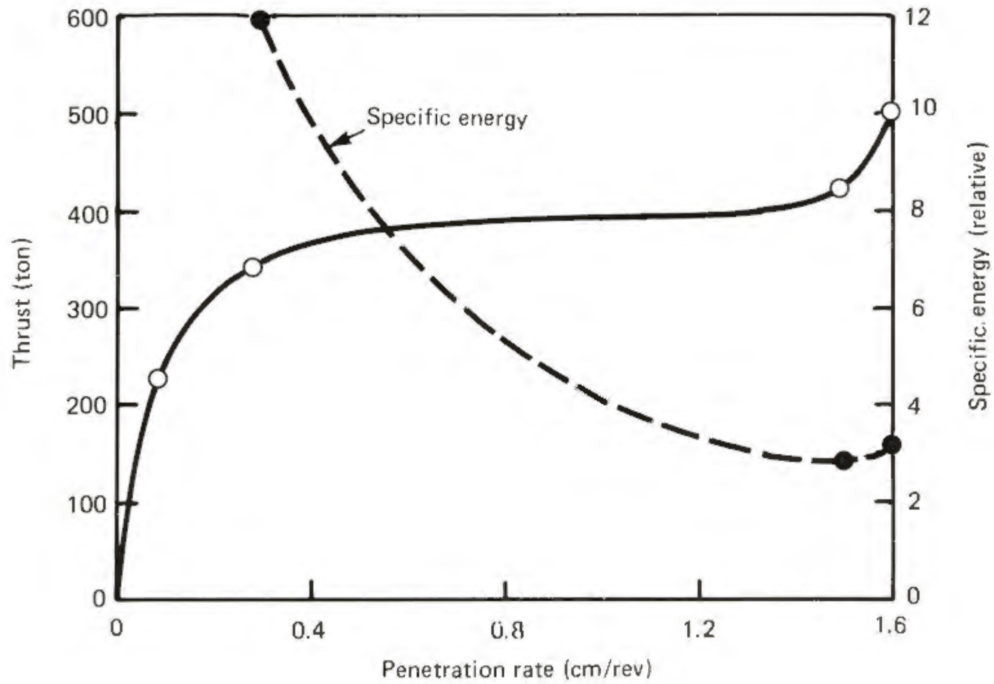


Fig. 2-41 TBM RESPONSE, EQUIPPED WITH DISCS OPERATING IN MEDIUM STRENGTH GRANITE (Partly after Innaurato et al, 1976)

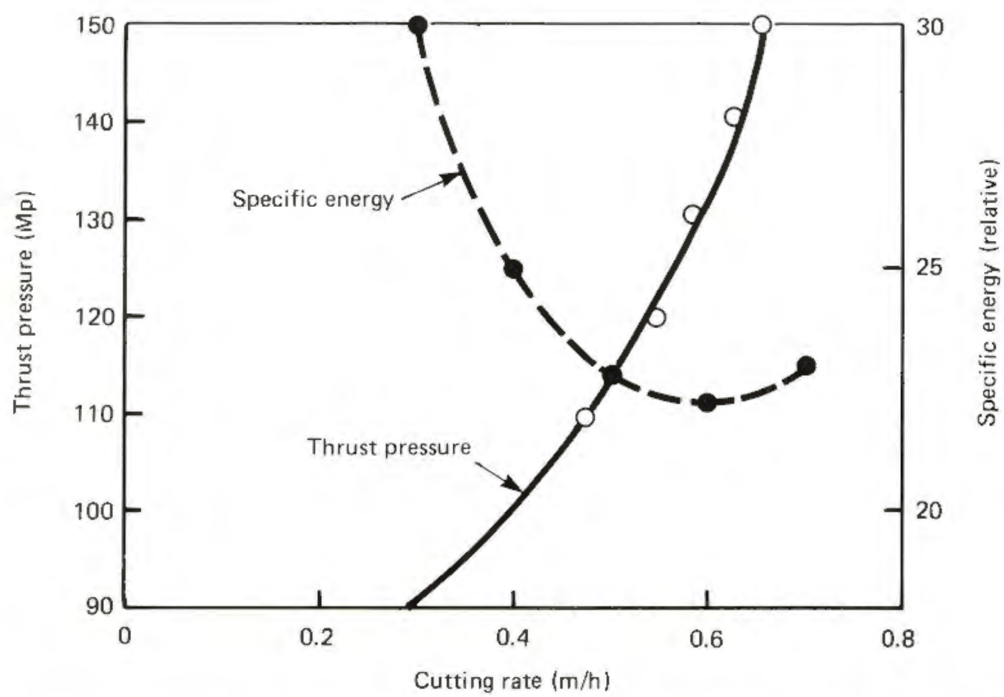


Fig. 2–42 TBM RESPONSE, EQUIPPED WITH STRAWBERRY BUTTON CUTTERS OPERATING IN STRONG GRANITE (Partly after Erkelenz, 1968)

curve without plotting specific energy.

A 2.1 m diameter Wirth TBM operating in very strong and abrasive granite-gneiss (compressive strength of 280 MN/m^2; 72 percent SiO_2) was equipped with 14 strawberry button cutters with tungsten carbide inserts (Erkelenz, 1968). The thrust-penetration relation, Figure 2-42, is considerably different than that displayed in Figure 2-41, although, the essential features are the same. Optimum advance rate was 0.6 m per hour as compared to 0.9 m per hour, the result of the different machine characteristics and operational conditions. At the indicated optimum specific energy, the thrust (130 Mp contact pressure) was also found to be the operating condition at which minimum wear occurred to tool bearings and to the cutting compacts. Operating at minimum energy naturally reduces the proportion of energy available for heat and friction, the agents of tool wear.

In general, the point of optimum specific energy may not correspond to the optimum in terms of economics and/or tool wear. It is reasonable that unless a high premium is placed on advance, operation thrust should not exceed the indicated optimum. Excessive thrust cannot help but lead to unnecessary distruction of tools and machine damage. It is possible, however, that tool design limitations (eg bearing load) available machine power, or debris removal may become ineffective or fail before an optimum in terms of mechanical excavation is achieved.

This was displayed by a 5.5m diameter TBM fitted with relatively widely spaced discs, 127 mm, operating in Dragonby iron ore (average compressive strength of 33 MN/m^2; Gaye, 1972). Machine advance was reduced at high thrust on account of the immense amount of debris produced, causing regrinding of debris or secondary crushing. Provided with an improved debris removal system, the optimum specific energy would most likely have occurred at an advance rate in excess of the recorded 3.9 m per hour. Inadequate debris removal or power to the cutter head at thrust below optimum results in incomplete development of the characteristic "S" shaped curve, Figure 2-43. Beyond the initial nonlinearity, the curve largely remains linear with increasing thrust until the problem causes rapid departure, possibly followed by jamming of the cutter head.

Altering machine stiffness can have a favourable influence on overall performance. For example, the particular design of the Robbins T B M thrust and steering system makes it a relatively soft hard rock machine, as compared to Demag or Wirth. Because of this, when operating in very strong rock, the machine is prone to excessive vibration and shock loading. Consequently, penetration decreases (as shown in Figure 2-33), tool wear increases, and the risk of machine damage, such as a main bearing failure, increases. Bearing life is greatly influenced by high shock load. In particular, lateral forces are a major controlling factor in tool bearing design. By stiffening the machine with the aid of the front pads (banana jack) much of the damaging vibration was eliminated, Fig AIV-3(p 190). Employing a stiff pancake jack, the front pads are forced against the tunnel wall which tends to lock the cutter head in place. As excavation occurs the pads slip along the wall. Should the head try to rebound from the face, it is restrained by friction between the pad and wall rock (shear reaction of approximately 4-11 tons). The increased stiffness imparted to the machine at the normal operating thrust is much greater than the machine stiffness alone, Figure 2-44. Although, the range of thrust over which the increased stiffness acts is limited before slip occurs, the mobilized force is sufficient to dampen a large proportion of the vibration.

This system was employed on a 3.5 m diameter Robbins TBM equipped with single discs for the excavation of a water tunnel in West Aurland, Norway (Garshol, 1978). Operating in strong and abrasive quartzite-gneiss (compressive strength of 150-200 MN/m^2; quartz content 30-40 percent) the machine was able to utilize the high thrust (14-15 tons per disc) necessary to achieve a 1 to 2m per hour advance rate while maintaining reasonable tool wear. Average cutter costs were on the order of 40 to 50 N. Kr. ($10) per cubic meter excavated.

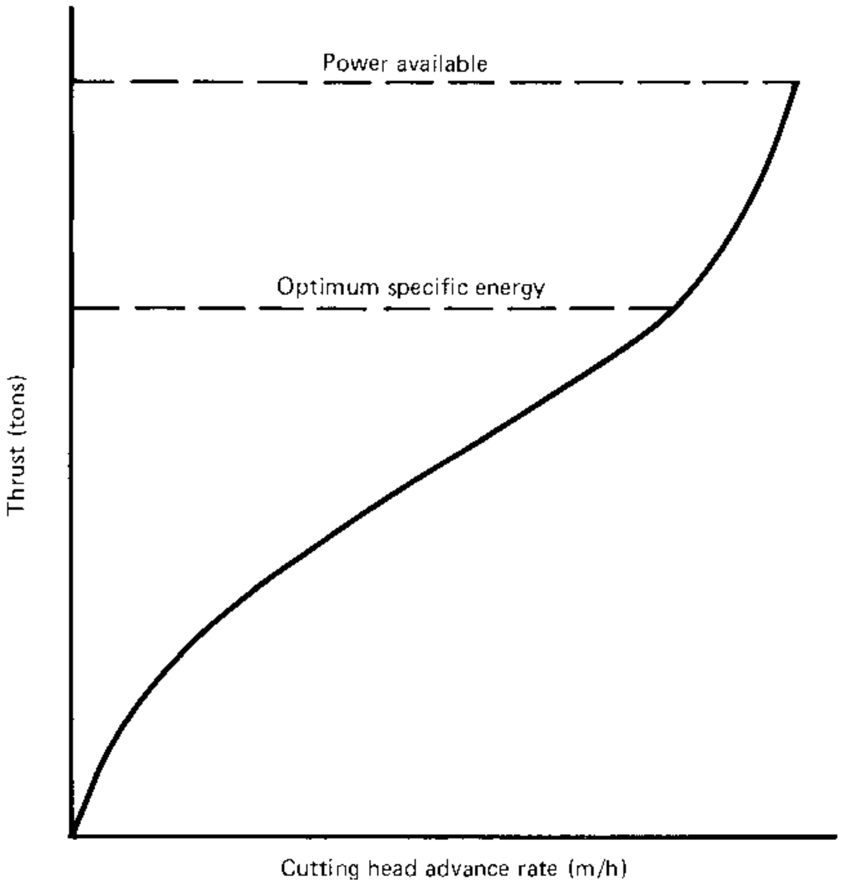


Fig. 2-43 GENERAL TBM RESPONSE, THRUST (OR POWER) VERSUS CUTTING RATE

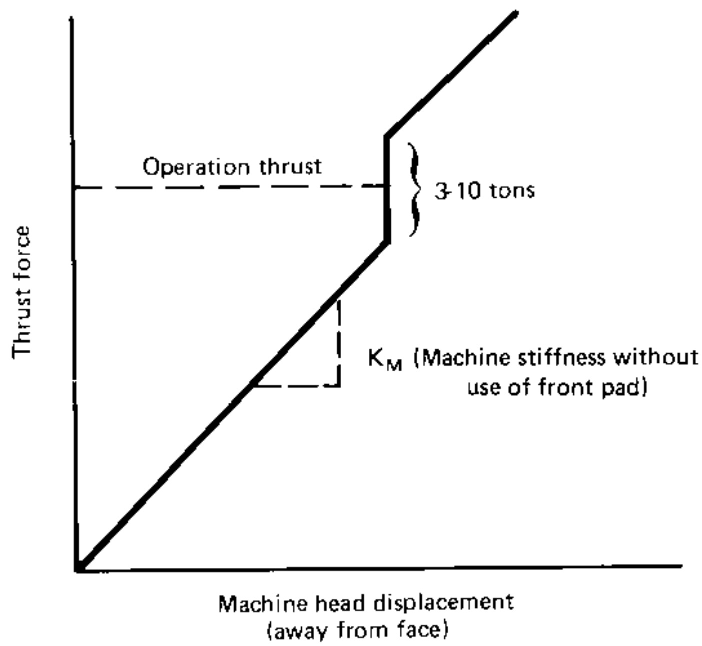


Fig. 2-44 INCREASED NORMAL STIFFNESS OF ROBBINS TBM ON EMPLOYING FRONT STABILIZING PADS

Machine advance rate is often accepted unless it is obviously poor or there is a direct method of comparison, such as another TBM operating under nearly identical conditions. When the opportunity for comparison is presented, as was the case at the Kielder water tunnels in England, it is possible to observe the influence of machine design on performance. To construct the 28 km of 3.5 m diameter water tunnel, two Demag and one Robbins TBM were employed. Although the machines operated in different sections of the tunnel line, they all encountered similar rock types within the gently dipping sedimentary series of limestone, mudstone, and sandstone (additional details in Appendix IV).

Basic machine design, arrangement of tools on the cutter head, and selection of tools were markedly different for the two machines. As indicated in Table 2-4, the Robbins was equipped with 26 single discs, spaced at 80 mm, on a gently curved cutter head for an average maximum thrust of 10 tons per cutter, Plate 2-5. Initially, the Demag was fitted wtih 17 triple discs plus a pilot for a spacing of 40 mm, Plate 2-6 (maximum 6 tons per cutter edge). On account of the low advance rate as compared with the Robbins machine equipped with single discs, the number of triple discs was reduced to 13 (80 mm spacing) and finally replaced by double and single discs (102 mm spacing). After modification of tool spacing, covering the range employed by the Robbins machine, the penetration rate was only marginally improved by 20 percent (Hayward, 1977). As shown in Table 2-5 and Figure 2-45, the Robbins was nearly twice as fast in similar rock types of lower strength and 50 percent faster in rock of higher strength. Differences in penetration per revolution, Figure 2-46, were even greater on account of the higher rotation speed of the Demag head.

There are several possible reasons for the large difference in penetration rates. Available thrust per disc edge was a contributing factor, Table 2-4. Depending on the proportion of friction loss acquired from pulling the machine and back-up equipment, the maximum thrust capacity of the Demag was 20 to 30 percent less than the Robbins at similar spacing of tools. However, considering the large increase in

TABLE 2-4

Specification of TBM's
employed at Kielder water tunnels

Specification	Units	Tunnel boring machine	
		Demag TVM 34-38H	Robbins 123-133
Weight	tons	130	88
Diameter	m	3.5	3.55
Cutter head rotation	rpm	9.65	6.5
Forward stroke	m	0.8	1.05
Drive motor power	kW HP	360 430	440 600
Forward thrust	tons	320	312
Cutters	no.	(a) 13-17 triple disc plus pilot (2 tri cone) (b) 12 double, 5 single plus pilot	26 single disc plus pilot (1 double, 1 triple)
Cutter spacing	mm	(a) 80-40 (b) 102	80
Maximum thrust per disc	tons	(a) 7.8-6.0 (b) 10.3	10.1
Cutter layout		sequential single spiral	sequential single spiral
Nearest support point from face	m	12	2.5-3 and 15
Date built		1975	1970

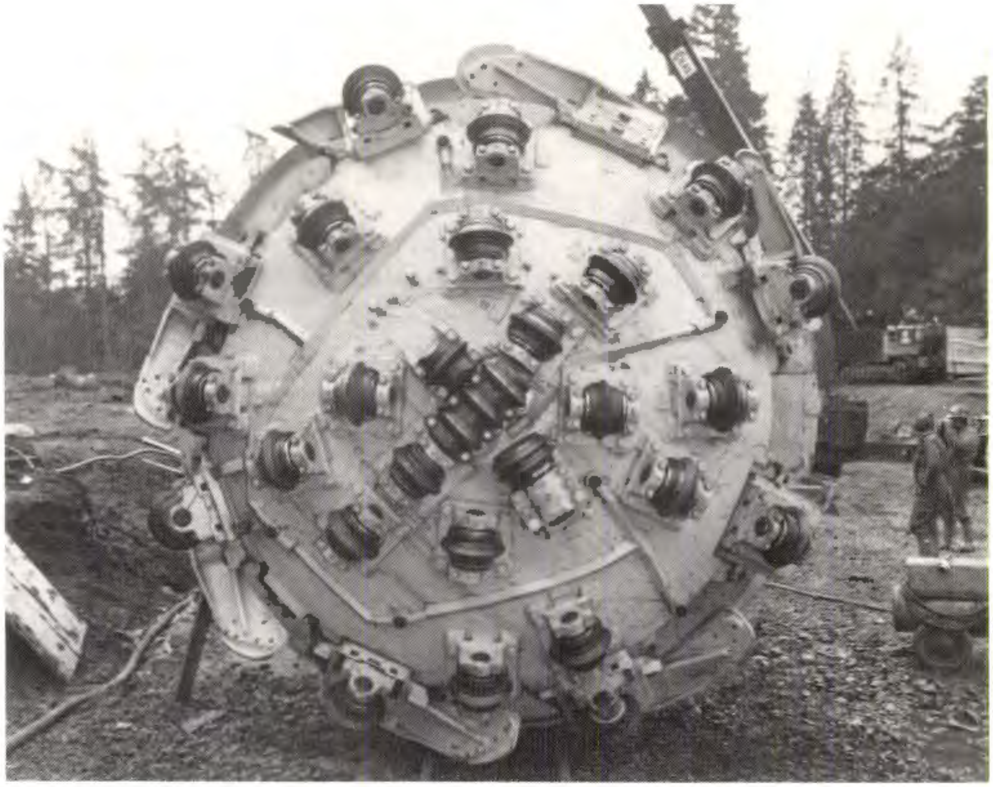


PLATE 2-5. Robbins TBM equipped with single discs

PLATE 2-6. Demag TBM equipped with triple discs

TABLE 2-5

TBM cutting rates for different rock types
encountered at Kielder

Rock type	Mean cutting rate (m/hr)				Compressive*** strength (MN/m^2)	Quartz*** content (%)
	*Demag 2		Robbins			
	Rate	Station (m)	Rate	Station (m)		
Mudstone	2.0	1270–1400	3.1 4.0	859– 941 2600–2700	10– 50	0–18
Mixed beds (mudstone and sandstone)	1.8	120– 310	4.4 4.4	400– 500 5100–5300	20–100	19–60
Sandstone	1.5	310– 600	3.6 2.9 3.8	1500–2200 4100–4300 4900–5100	50–150	60–95
Limestone	1.3	900–1130	1.3 1.9	1052–1065 1325–1346	100–200	0– 5
Dolerite	0.4**	360 m SW	1.3	2716–2743	200–500	0– 2

* 40–80 mm tool spacing

** Demag 1 (S Wear)

*** from Brown and Milow, 1979

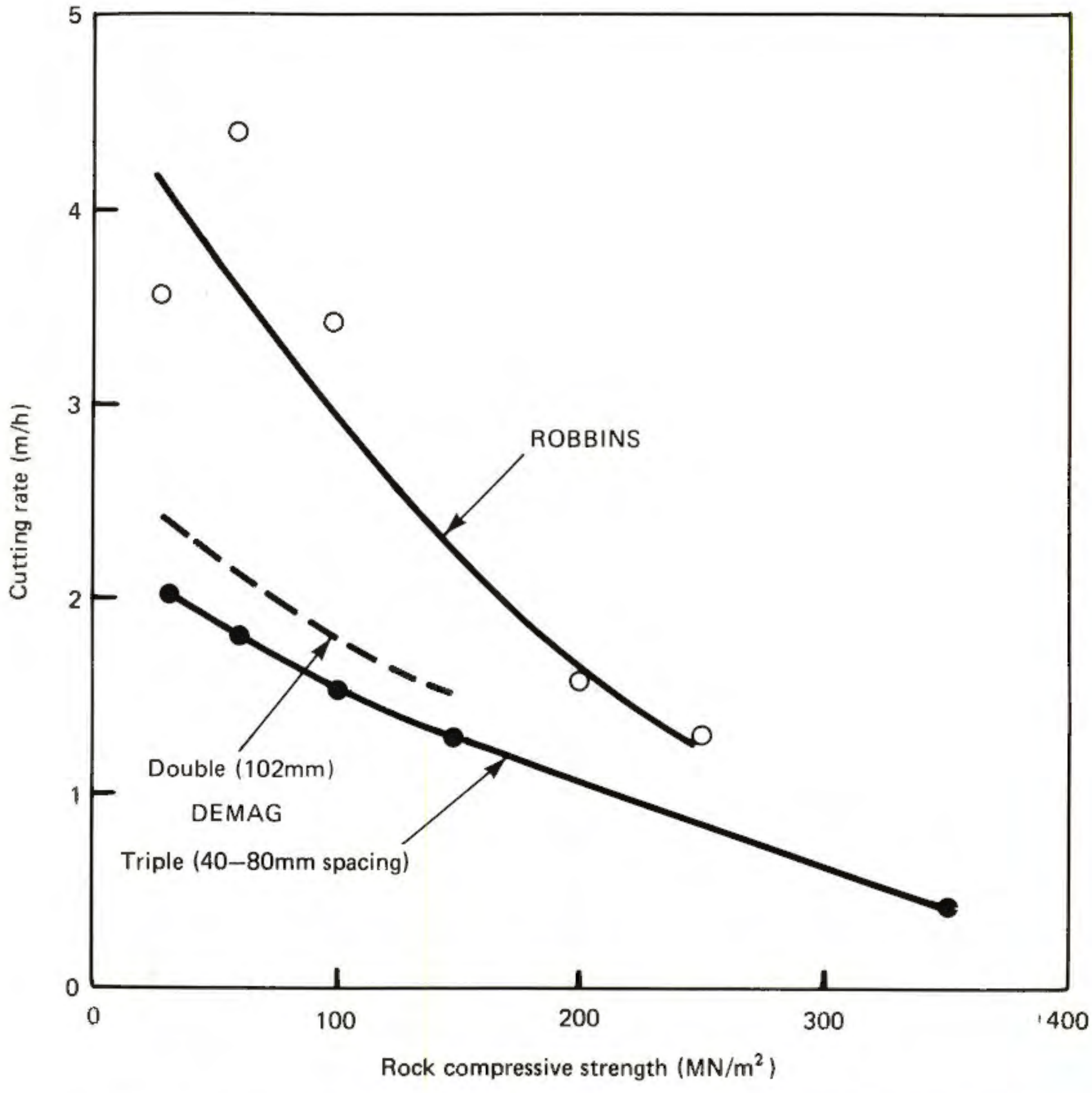


Fig. 2–45 TBM CUTTING RATES VERSUS ROCK UNCONFINED COMPRESSIVE STRENGTH, KIELDER

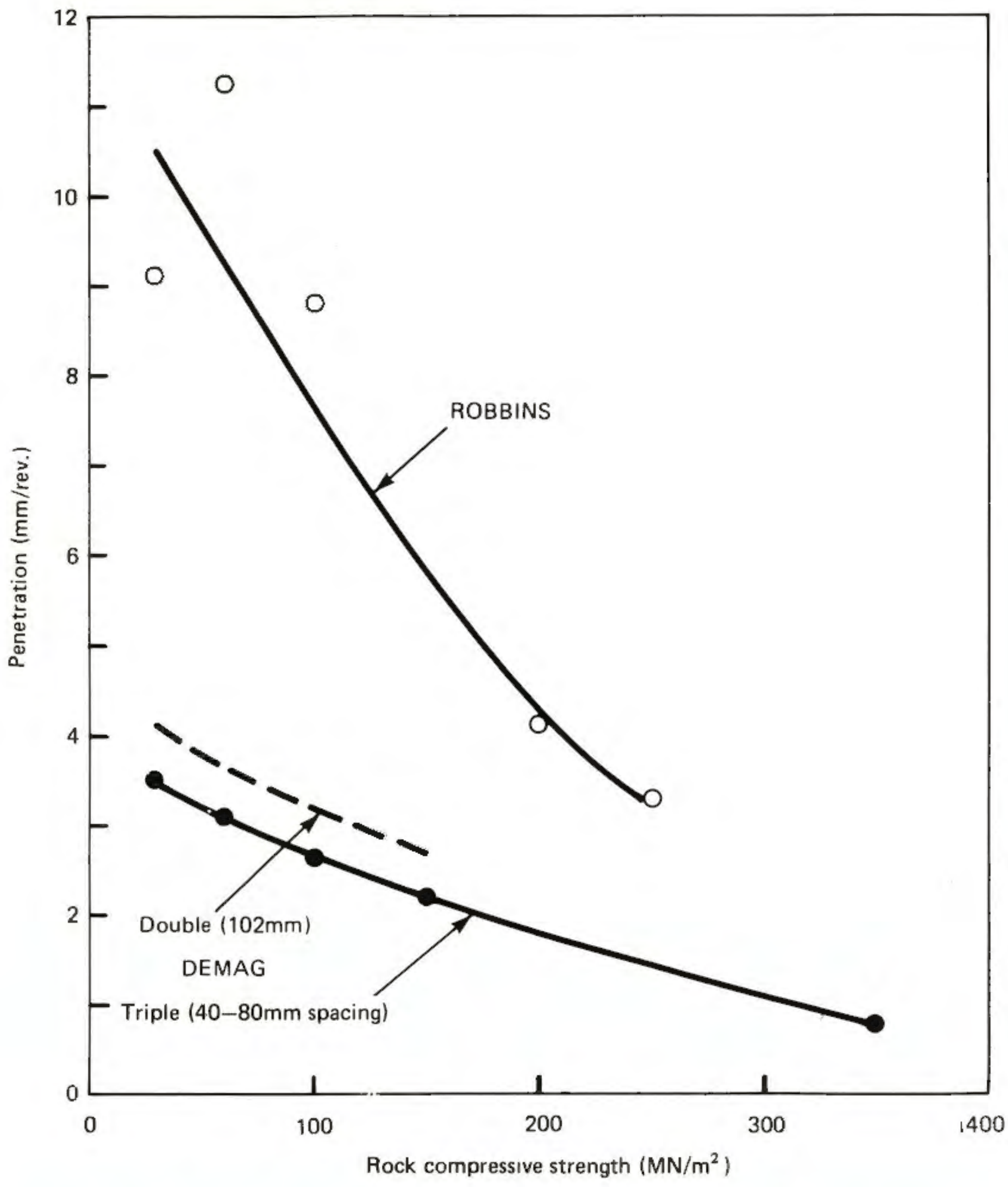


Fig. 2–46 TBM PENETRATION RATES VERSUS ROCK UNCONFINED COMPRESSIVE STRENGTH, KIELDER

available thrust, 70 percent, on reducing the number of cutters and the relatively minor improvement in penetration, it is not likely that increased thrust alone would account for the large difference in performance.

Cutter selection also influenced performance: multiple discs were used exclusively on the Demag, single discs on the Robbins. With multiple discs interaction between kerfs can occur at spacing to penetration ratios of less than 20, thereby, reducing penetration. In addition, restricted rotation leads to scuffing and increased tool wear. But, it is not likely that the type of disc had a large influence for two reasons. First, the tool penetration was rarely great enough, 4 to 5 mm, to cause interaction problems. Second, several machines equipped with triple discs have achieved penetrations similar to that displayed by the Robbins in similar strength ground (for example 10 mm per revolution at 6 tons per cutter edge maximum operating in medium strength shale; Victoria Collieries, Appendix IV).

As previously discussed, the arrangement of tools on the cutter head is known to affect performance. On close inspection, it was evident that the spacing, and more important, the relative penetration of each tool with respect to its neighbour was not given sufficient consideration. Stereo photographs of the tunnel face plotted photogrammetrically revealed the excavation pattern of the Demag cutters, Figure 2-47. As shown, the disc spacing was not consistent and depth variations between adjacent tool edges were as much as 20 mm. Large depth variations, as between the outside edge of tool 4 and the inside edge of tool 5, result in an equivalent over coning of up to 20 degrees (see Figure 2-36). This leads to nonuniform loading, reduced overall penetration, and greater tool wear. On comparing the life of the discs located at position 10 and 12 over a 1.6 km length of tunnel, tool 10 was replaced twice as often as 12. The large depth variations were related to those tools bordering an abrupt head cone angle change (located between tools 4 and 5, 10 and 12, etc). Rather than employ a gently curved head to achieve the angle required for the gauge cutters, the Demag cone angle is altered in a discontinuous fashion,

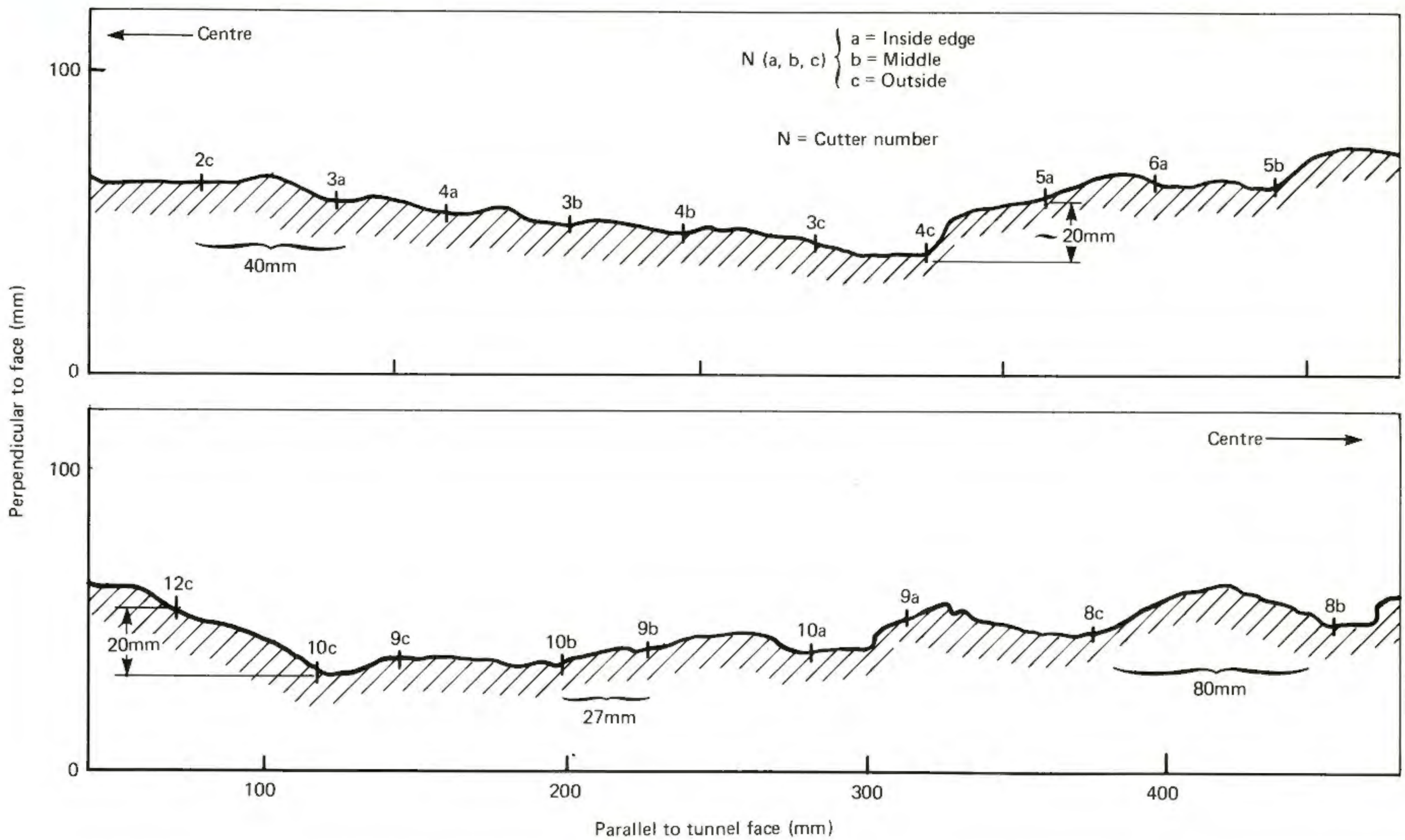


Fig. 2-47 PHOTOGRAMETRIC PLOTS OF MECHANICALLY EXCAVATED TUNNEL FACE, DEMAG TBM EQUIPPED WITH TRIPLE DISCS (After West and Barratt, 1976 and 1977)

Plate 2-6. It is most likely that the poor arrangement of tools on the Demag was a major factor influencing the relatively low penetration rate.

Beside the coal measures strata, both machines encountered sections of high strength dolerite intrusives (average compressive strength, 250-350 MN/m^2). It was difficult to compare their performance on account of the relatively short length excavated along the Robbins drive, 27 m, and the possible lower strength, up to 100 MN/m^2; however, several points were clear. The Robbins machine completed the 27 m with an average cutting rate of 1.3 m/hr and only eight of the outer discs required replacement. When the Demag encountered the dolerite all discs were destroyed in a few meters. On changing to button discs the tool life was increased to roughly 14 m of tunnel per full set with a mean cutting rate of 0.4m/hr. Aside from being slow (even by hard rock standards) the cost was extremely high, up to 60 pounds ($120) per cubic meter excavated (Milow, 1978). Tool costs in sandstone with quartz content over 90 per cent were also high, over 25 pounds ($50), while in less abrasive rocks, costs were under 10 pounds ($20) per cubic meter. In general, tool replacement and cost were at least double that of the Robbins for similar ground (McFeat Smith and Tarkoy, 1979). Again, this difference was most likely related to tool arrangement and the inevitable scuffing which occurs in locked multiple discs. From this comparison, it is evident that the question of tool wear can not be addressed without considering the specific TBM on which the cutters are fitted. Tool wear like advance rate is largely dependent on machine characteristics as well as operational conditions.

Cutting rates for both machines were also shown to be largely dependent on rock type or strength, and rock mass character. Average rock strengths ranged between 30 and 350 MN/m^2 over which the advance changed by a factor of more than five, Figure 2-45. Peak cutting rates for any rock type were approximately 50 percent higher than the mean values listed in Table 2-5. It is of interest to note that the rate of change of penetration with rock strength was roughly 50 percent greater for the Robbins. Also, the relation between cutting rate and compressive strength was a relatively accurate indicator of the Demag performance while only revealing the trend of the Robbins advance.

This may be attributed to the mode of excavation, with the additional crushing (shearing) of material excavated by the Demag.

The influence of joint spacing on excavation rate and specific energy were evident, particularly as joint spacing approached the tool spacing. Table 2-6 presents an example of excavation in relatively massive and highly fractured dolerite. Intact rock strength of the fragments and the massive unit were found to be similar as indicated by Schmidt hammer tests. Cores obtained from larger pieces of dolerite, tested in unconfined compression recorded strengths of 250 MN/m^2. It was most likely that this value was a lower bound since the faulting process which caused the intense fracturing most certainly produced microfractures within apparently intact pieces. Many of the joints were headed with a thin layer of calcite (geologic profile, Fig AIV - 8). Consequently, they were not cohesionless, but possessed a shear strength of approximately 15 per cent of the intact dolerite. Although it was difficult to quantify all the parameters, the three fold increase in cutting rate and factor of four decrease in specific energy of excavation clearly illustrate the influence of jointing. Stability problems within the fractured region were relatively minor as a result of the excellent character of the joints: rough, closed, and partly healed.

As shown above and in many other examples in the literature, joints undoubtly increase the machine penetration rate, although the extent is not clear. To predict their influence, a simple method based on Rittinger's relation for the comminution of rock is proposed. This relation provides a reasonable estimate of the amount of energy required for the mechanical excavation of rock by various methods of comminution (Cook and Joughin, 1970).

According to the hypothesis, the energy required to comminute a unit volume of rock is proportional to the increase in surface area of the fragments (Rittinger, 1867);

TABLE 2-6

Influence of joint spacing
on excavation of dolerite, Robbins drive

Station (m)	Joint spacing (average)	Cutting rate (m/hr)	Specific energy* (MJ/m^3)
2716–2743	>1 m	1.3	46
4030–4044	1–10 cm	3.9	11

* calculated from recorded current to drive less current to rotate cutter head under zero thrust

$$E = K\left(\frac{1}{P} - \frac{1}{F}\right)$$

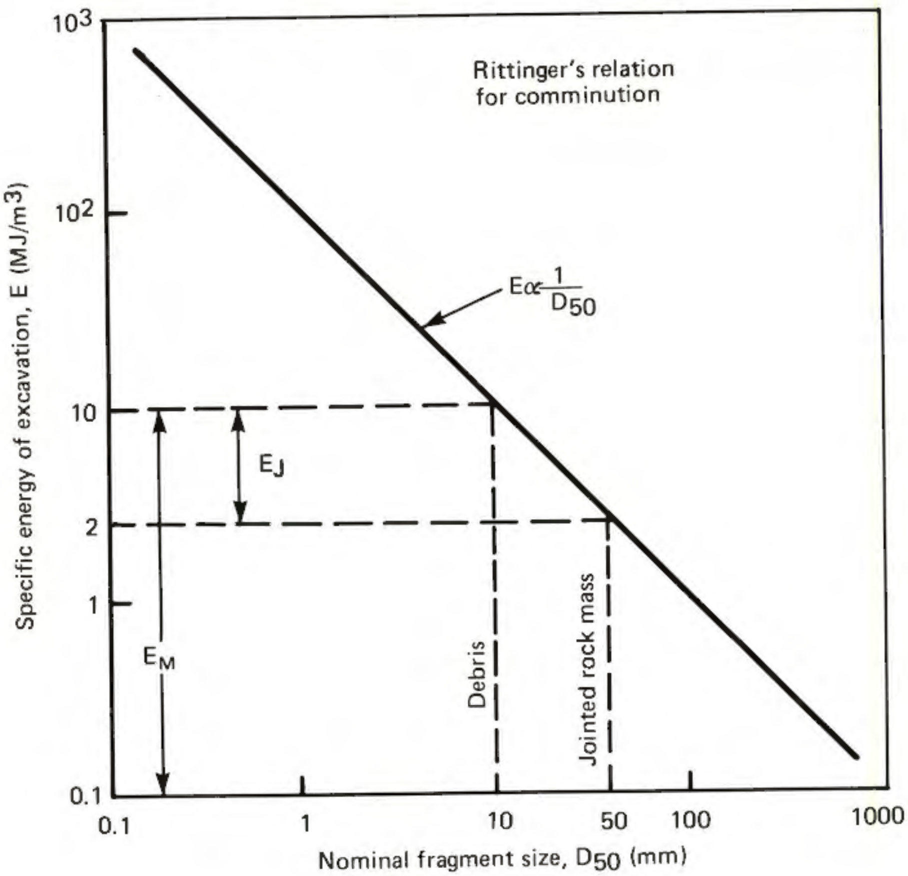
where E = the energy required to comminute a unit volume of solid rock from a nominal feed size F to a nominal products size P,

K = a constant for a given rock type and strength (effective surface energy).

On substitution of average joint spacing for nominal feed size and debris size for products, the relation for comminution is adapted to that of rock mass excavation. By this relation, to effect a 20 per cent decrease in mechanical cutting energy the average joint spacing of the rock mass must be on the order of five times the nominal fragment size of the debris, Figure 2-48. As a first order approximation, the decrease in specific energy is proportional to the increase in penetration rate.

Given the typical debris size produced by TBM's, 1 to 5 cm (Figure 2-34), average joint spacings of less than 5 to 25 cm are required before cutting rates increase substantially (more than 20 percent). Not until the joint spacing is on the order of the tool spacing does the penetration markedly increase, as revealed by the performance of the Robbins TBM in fractured dolerite, Table 2-6. There are, however, cases in the literature where relatively large joint spacings were reported to have a significant influence on machine performance. For example, an average joint spacing of 1.5 m (frequency of 2 per 10 ft) in a gneiss-granite was shown to change specific energy by 50 percent depending on the dip of the joint with respect to the tunnel bearing (Wang et al, 1974). As the machine was equipped with strawberry button cutters (debris size about 1 cm), it is unlikely that the major discontinuities affected average penetration. Most likely features related to rock micro-structure oriented parallel to the principle joint direction (ie material anisotropy) were responsible for the observed improvement.

In other cases, increased joint frequency can lead to decreased intact rock strength as a result of the greater amount of weathering and alteration via the discontinuities. Consequently, reported



E_M = Massive rock specific energy of excavation
E_J = Jointed rock mass specific energy of excavation

Fig. 2-48 APPLICATION OF RITTINGER'S RELATION TO ESTIMATE INFLUENCE OF DISCONTINUITIES ON SPECIFIC ENERGY

improvements in machine penetration may actually be attributed to reduced rock strength rather than purely joint spacing.

Variation of rock strength with sample orientation or anisotropy resulting from material schistosity, bedding planes, residual stresses, etc, has a large effect on the machine penetration rate. Unlike widely spaced joints, the structural features are on a small enough scale to be easily exploited in the excavation process. To take advantage of the weakness planes, their orientation must be close to that of the fracture plane produced during chip formation (parallel to the tunnel face). As shown in Figure 2-49, the best cutting rates were obtained when the tunnel axis formed an angle of incidence of more than 60 degrees with the schistosity (Wanner, 1975). Boring in the direction of schistosity proved very unsatisfactory.

Despite the fact that joints and other discontinuities technically increase penetration rates, the practical consequence may be of little use. As the necessary joint spacing is on the order of tool spacing for significantly improved cutting rates, the character of the rock mass under consideration would in most instances require immediate stabilization measures. Under these conditions it is rarely possible to take advantage of the increased penetration rate. Any increase in progress due to higher penetration is often completely negated by delays associated with ground stability problems.

SUMMARY

In general, attempts to predict prototype behaviour from scaled models have not been successful, considering the effort required and the accuracy of the results obtained. This is largely on account of material size effect, although, test procedure and test system stiffness may be factors. Improved predictions based on scale models depend on a better understanding of the excavation process and improved numerical models.

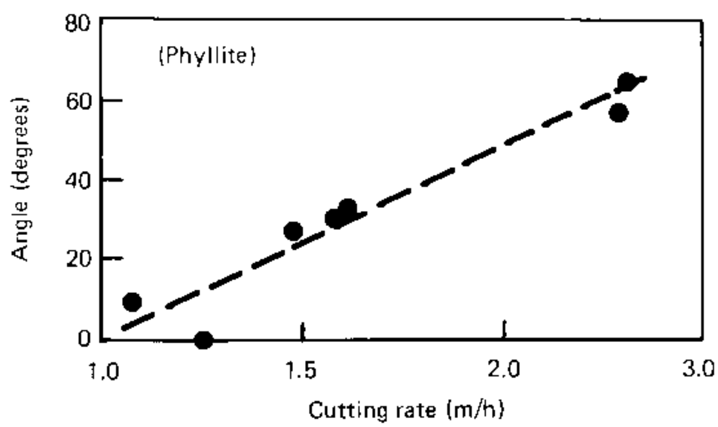


Fig. 2-49 VARIATION OF TBM CUTTING RATE WITH ANGLE BETWEEN ROCK SCHISTOSITY AND TUNNEL AXIS (After Wanner, 1975)

Full scale laboratory tests can provide a wealth of information for either machine design or prediction of penetration. Test procedure, surface conditioning, system stiffness, and interpretation of the data are major factors to consider. On account of the complexity and expense of these tests, the results are of value when applied in a general context rather than performed for a specific tunnel project or rock type. Application of laboratory results to a specific project depends on the accuracy by which the rock tested in the laboratory is equated to that in the field. This is usually accomplished with the aid of several index tests performed on rock core samples.

INDEX TESTS

Material index tests are required for strength parameters employed in numerical models, to relate laboratory physical model results to the appropriate rock types in the field, and as a method by which known machine performance can be applied to predict boreability in similar or different rock masses. The tests can be divided into two broad categories depending on whether material characteristics are determined or it is intended as a specific indicator of machine boreability, either in terms of penetration rate or tool wear. Specialized index tests employed for boreability predictions essentially "model" a particular aspect of the excavation process.

MATERIAL CHARACTERIZATION

Material properties related to strength, deformability, and hardness are readily determined by various index tests, Table 2-7. Although the properties were divided into three convenient categories, they are all fundamentally dependent on mineralogical composition and rock structure (grain size, texture, cementation, etc). On account of this dependence, it is not surprising that relations between these properties exist.

Stimpson (1965) notes that hardness tests have been employed as

TABLE 2-7

Material characterization

Property	Test	References
1. Strength		
(a) Compressive, shear	Triaxial compression	Jaeger and Cook, 1969
(b) Tensile	Point load, direct tension, Brazilian	Broch and Franklin, 1972; Bieniawski, 1975
(c) Specific fracture energy	Bending	Friedman et al, 1972
2. Deformability	(same as for strength)	
(a) Ductility		Edmond and Paterson, 1972
(b) Brittleness	Protodyakonov impact,	Protodyakonov, 1963; Hucka and Das, 1974
	Swedish	Selmer-Olsen and Blindheim, 1970
3. Hardness (micro)		
(a) Indentation	Brinell, Vickers, Rockwell, NCB cone indenter	Das, 1974; Roberts, 1977; Gaye, 1972
(b) Rebound	Shore scleroscope, Schmidt hammer	Paone and Bruce, 1963; Stimpson, 1965; Kolek, 1958
(c) Scratch	Mohs	Shepherd, 1950; Innaurato et al, 1976

Strength)
Deformability) dependent on mineralogical composition and structure
Hardness)

indicators of resistance to scratching, resistance to abrasives, cutting resistance, resistance to plastic deformation, modulus of elasticity, strength, yield point, brittleness, lack of ductility and malleability, etc. Specifically, and more important, he has shown that hardness displays a good correlation with unconfined compressive strength. Likewise, a reasonable correlation was found between Schmidt hammer, point load, and compressive strength for a wide range of Carboniferous rocks (strength of 10-200 MN/m^2; Carter and Sneddon, 1977). Correlations among rock strength parameters (eg compressive to tensile strength ratio of roughly 12) and hardness tests (Brinell and Schmidt hammer; Kolek, 1958 and Vickers and Mohs; Innaurato et al, 1976) have also been identified.

Most of the hardness tests were originally designed for the testing of metals. For use on rock, both the Shore scleroscope and the Schmidt hammer provide a rapid and reasonable method by which to estimate material strength. The point load test, designed for use on rock, provides the same amenities. As with all index tests, the accuracy of the results depend on the test procedure and the statistical significance of the data. Rebound tests are particularly subject to error if the rock surface is not smooth and uniform (Paone and Bruce, 1963). It has been suggested that a rough surface can be made to produce a consistent rebound number if multiple impacts are performed at the same location. This is not recommended as the rock will sustain considerable damage resulting in arbitrarily low or high hardness values depending on the effect of the local crushing and material compaction.

As estimated from Schmidt hammer tests on concrete, rock structure within a 5 cm radius of the indentor has a major effect on the hardness number (Kolek, 1958). The actual range of influence is obviously dependent on the energy of the hammer, however, it is of interest to note that this is approximately the same size as a standard compression or point load test specimen.

The ratio of specific energy and unconfined compressive strength is often employed as a dimensionless index of excavation efficiency. Theoretical justification stems from the relation between specific energy and material strength considering the work required to fail a sample in unconfined compression (Mellor, 1972; Gaye, 1972). As an index, it characterizes the efficiency of the excavation process and is not a rock property. Although the dimensionless quantity is near unity for many drilling operations (Teale, 1965), this does not apply to TBM's (machines equipped with strawberry button cutters excepted). As shown in Table 2-3, the ratio is considerably less than one when discs are employed. A comparison of field and laboratory indices provides a simple method by which the similarity of the excavation processes can be assessed.

BOREABILITY

Aside from a source of data for numerical models and for correlation of physical model results, the various material properties have been employed as boreability indicators. Used alone or in combination with other boreability index tests, Table 2-8, the indicies are used by machine manufacturers and contractors to predict penetration rate and tool wear.

Needless to say, there are major advantages and disadvantages to the use of indicies for boreability predictions. As the indic es do not consider the machine characteristics (stiffness; number, type, spacing and arrangement of tools; available thrust and power) and only certain aspects of the operational conditions (rock mass characteristics, thrust employed, condition of tools), there is considerable margin for error. The <u>only</u> valid use of these indic es is as calibrated by and applied to a specific machine operating under specific conditions. Also, when performing index tests the anisotropic nature of many rocks should not be overlooked. The indice is not necessarily a scalar value, consequently, the orientation of the test sample is to be related to the direction of machine advance.

TABLE 2-8

Boreability index tests
(simulated rock-tool interaction)

	Principle	Test	References
1.	Hardness (macro)	Static indentation with tool bit	Morris, 1969; Handewith, 1970; Ozdemir et al, 1977
2.	Drilling	Microbit drillability	Ross and Hustrulid, 1972
		Sievers miniature	Selmer-Olsen and Blindheim, 1970
3.	Abrasion	Microbit drillability	Ross and Hustrulid, 1972
		Taber abraser	Tarkoy, 1973
		Abrasion value	Selmer-Olsen and Blindheim, 1970
		Dorry	Obert et al, 1946
		F coefficient	Schimazek and Knatz, 1970; Brown and Phillips, 1977

There are many examples of the misuse of boreability indices in the literature. In one case, used to illustrate the inapplicability of unconfined compressive strength to predict cutting rate, a comparison was made between machine performance in granite and limestone rock masses having similar strengths (130 MN/m^2). Average penetration rate in the Nast tunnel (granite-gneiss) was only 0.9 m/hr as compared to 1.8 m/hr in the Lawrence Avenue tunnel (dolomitic limestone). In making the comparison no consideration was given to the fact that two different machines excavated the openings, one of which was equipped with strawberry button cutters (Nast) and the other with discs (Lawrence).

Strength parameters or those properties related to strength, from deformability and hardness, are widely used in boreability predictions with reasonable accuracy and theoretical justification. Numerical models which can provide a rough estimate of penetration, depend solely on strength parameters. Naturally, specific strength, deformability, or hardness properties are more appropriate indicators of boreability and there is considerable debate on this point. Both shear and tensile strength more precisely describe the process of chip formation than docs compressive strength. In addition, as suggested by practical experience and the pressure bulb model, the ratio of shear or compressive strength to tensile strength further influences penetration rate. For a decreasing ratio penetration is reduced, assuming a constant thrust force.

In general, compressive strength is a widely used index property, largely on account of its simplicity. Despite the appropriateness or inappropriateness of the test, reasonable correlations with penetration rates have been found in many investigations (Hustrulid, 1970; Haswell, 1973; Tarkoy, 1974; Barendsen and Cadden, 1976). There are, however, nearly as many examples describing poor correlation, particularly when tool wear is taken into account (Roxborough, 1969). This problem can be reduced by considering tool wear separate from that of penetration rate.

As previously described and shown in Figure 2-45, compressive strength was a good indicator of the Demag cutting rate, but less promising for the Robbins. When the process of excavation involves considerable material crushing (shear under a compressive normal stress), as in the case of the Demag at Kielder and with machines equipped with strawberry button cutters, compressive strength is as good an indicator of penetration rate as most other index tests. Schmidt hammer, point load and NCB cone indentor results, averaged over 100 m lengths of tunnel, also revealed a good correlation with the Demag cutting rate over a 1.5 km test section of the Kielder water tunnel (Morgan et al, 1979). Undoubtedly, the relation between strength, deformability, and hardness is in part responsible for the similar correlations.

To minimize the risk of error, rather than employ one index test alone, several material properties and/or boreability index tests can be considered together. For example, the Robbins Co employs compressive strength in conjunction with an impact toughness test (similar to a dynamic hardness test; Hansen, 1975). Other manufacturers often employ full or reduced scale tool indentation tests along with various material characteristics (Morris, 1969; Handewith, 1970).

Due to size effects, the results from reduced scale indentation tests can be regarded as little more than a hardness indice. Full scale tests, however, provide a good estimation of the force-penetration relation for tools arranged at the optimum spacing to penetration ratio (Ozdemir et al, 1977). A full scale indentation test is the only index test which approaches a form of "model" to the actual excavation process.

Other indices, such as total hardness, combine rebound hardness (Schmidt hammer or Shore scleroscope) and abrasion (Taber abraser) to predict penetration rate (Tarkoy, 1975). On examination of the results from 14 field sites, it is clear that total hardness has the same degree of correlation with penetration rate as does the Schmidt hammer rebound number or abrasion index used alone. At one particular site, when both compressive strength and total hardness were correlated with penetration the degrees of confidence (coefficient of correlation) were revealed to be the same. This further emphasizes the interrelation between material characteristics, Table 2-7, and boreability index tests, Table 2-8. Combining similar index test results will not lead to improved prediction (an abrasion test is basically a specific type of hardness test).

Using a drill rate index (DRI), an empirically combined result derived from a Swedish brittleness, Sievers drill and abrasion tests (Selmer-Olsen and Blindheim, 1970), a reasonable correlation was found with machine penetration rate (Blindheim, 1976). Considering the scatter of data, however, the complexity and number of the tests may not be justified. Again, the various index tests appear to be interrelated. The strong influence of joint spacing, between 1 and 20 cm, was clearly shown in the correlation. If cutting rate predictions are to be improved the effect of discontinuities must be included in the analysis.

To develop the most appropriate indices, a multiple regression analysis of data derived from full scale linear cutter experiments and the corresponding rock properties (compressive and tensile strength, density, static and dynamic elastic modulus, Poisson's ratio, P wave velocity, and Shore scleroscope hardness) was performed for a range of rock types (Morrell et al, 1970; Morrell and Larson, 1974). For the prediction of penetration, their results revealed a good correlation with density and hardness; cutter force was related to density, tensile strength, and hardness. As expected, the important material properties, strength and/or hardness, have the same significance in either laboratory or field predictions.

Minature scale model drilling apparatus, such as the Siever (1950) or Microbit (Ross and Hustrulid, 1972), have been widely used in drillability studies. Employing a standard set of operating conditions, the advance rate and the weight loss of the tool bit are correlated with known performance. As the drilling is performed by abrasion, the index is basically a combined measure of material properties. It does not, however, appear to have any advantages over other simpler index tests when employed as an indicator of boreability (Tarkoy, 1975).

TOOL WEAR

Beside the prediction of penetration rate, many of the index tests listed in Table 2-7 and 2-8 have been employed to estimate tool wear. Again, it should be emphasized that the calibration and use of these indices, to predict either penetration or wear is restricted to a specific machine operating under specific conditions. General data of tool wear or cost related to a set of indices may be grossly in error if the results are applied to a TBM with a different machine characteristic. An example from the Kielder tunnels illustrating the effect of cutter type, spacing, and arrangement of tools on TBM performance was presented in the last section (Physical Models; Full Scale Prototypes). In general, over a similar range of Carboniferous rocks, one single disc was replaced for every 10 m of tunnel excavated by the Robbins machine whereas the Demag required one triple disc (three cutting edges) for every 10 to 15 m (Brown and Milow, 1979; Morgan et al, 1979).

To normalize tool consumption, the number of cutters used or the cost is expressed with respect to the volume of material excavated or the distance travelled. On normalizing with distance travelled, it is apparent that the pilot (or centre) tools and the gauge (or perimeter) tools are replaced more frequently than those in between. Tools positioned near the centre suffer increased scuffing as a result of the small diameter of the circular path followed by the cutter. To excavate the tunnel perimeter the gauge cutters are angled to the tunnel axis (direction of machine advance) resulting in a component of sliding or scuffing. This coupled with the increased load received by gauge cutters accounts for the increased wear of tools located near the perimeter.

One simple method by which to improve the overall tool performance is through the use of several different diameter discs on the cutter head. Small diameter discs should be employed in the pilot region to reduce scuffing. The use of small discs has the added advantage of increasing the space between tool mountings, allowing room for more cutters and increased penetration per revolution of the machine head (ie double or triple start spiral; two or three disc edges running in the same groove per revolution). As the forces are relatively low near the pilot position, the tools should not have problems coping with the load. Near gauge position, large diameter discs are best designed for the higher loads and velocity. Furthermore, they do not require a large angle with respect to the tunnel axis in order to cut the opening perimeter, thereby, reducing scuffing. In between, one intermediate or possibly a range of diameters could be employed.

The life of disc cutters subjected to scuffing or abrasion tends to display a marked dependence on rock type. Whereas, centrally located cutters, between pilot and gauge, are not as significantly influenced by geology (as found at Kielder, Robins drive; Brown and Milow, 1979). This suggests that for tools operating under optimum conditions the process of excavation involves a relatively minor amount of abrasion. If the disc edge formed chips by wedging deeply into the rock, sliding past abrasive minerals, wear should be strongly related to rock type and mineral composition irrespective of tool position. If, however, chip formation was via a crushed zone, the role of tool indentation would be limited to the development of the pressure bulb beneath the bit rather than the displacement of the rock with its edge. The major proportion of tool penetration would consequently occur after the process of chip formation, thus, keeping abrasion to a minimum.

As a result of the disproportionate amount of wear associated with the pilot and gauge tool regions, smaller diameter openings tend to display higher cutter cost per unit volume excavated. In raise boring operations, where opening diameter is on the order of several meters, cutter costs are 10 to 65% of the total cost depending on rock type (Norman and Dye, 1978). This high cost relative to TBM operations is also attributed to the higher than normal thrust employed. Under certain conditions a 50 percent reduction in thrust will result in a 10 fold increase in tool bearing life (Dixon and Worden, 1971).

In general, disc wear is a function of both bearing and cutter edge life. With the advent of replaceable disc edges, tool bearing failure often limits the practical life of the tool. Bearing life is largely determined by a load-velocity-temperature relation, increases of which result in reduced life. To accommodate high tool thrust or cutter head velocity the diameter of the bearing, and hence disc, is increased. The limit of disc size is related to the ability to supply the required thrust, to handle the disc for replacement, and the general disadvantage of displacing the cutter head farther from the face. With increased distance between the head and face, the greater the possibility of large blocks of rock falling into the void resulting in cutter damage and possible jamming of the head. Also, the increased distance reduces the ability to provide immediate support at the face.

Cutter edges are constructed of hard steels, steel alloys, carbide based materials, or tungsten carbide inserts depending on the resistance to abrasion or hardness required. Although, the range of hardness for these materials, Mohs hardness of 5 to 9, is greater than that of many rock minerals, abrasion can occur as a result of temperature effects. With rising temperature the hardness of the material decreases. This is also displayed by tungsten carbide, however, the degree of softening is subject to some controversy (Osburn, 1969).

There is little doubt that high temperatures are generated during excavation. Temperatures as high as 1250 degrees Centigrade have been recorded on the surface of carbide rubbing on quartz (Rae, 1964). Within the body of a carbide insert located in a rotary drill bit, temperatures in excess of 450 degrees Centigrade were measured while drilling in sandstone (Whitbread, 1960). The high temperature generated is equally as damaging to the cutter bearings. Strawberry button cutter bearings can reach 300 to 400 degrees Centigrade, requiring the use of cooling water to increase their life (Erkelenz, 1968).

If the process of mechanical abrasion is uncomplicated by temperature effects, edge wear is less dependent on the thrust applied to the tool. This was revealed in tests with a rotary drag bit operating in granodiorite (compressive strength of 150 MN/m^2, quartz content 23 percent; Nevill and Crone, 1962). Generally, disc cutter wear increases with penetration, but at a reduced rate for deeper indentations than shallow (Sasaki et al, 1972). Increased cutter velocity was also found to increase wear.

When disc cutter consumption was normalized with respect to the energy used, Innaurato et al (1976) found a good agreement between cutter life and average Mohs hardness for a TBM operating in granite and in marls and sandstone. Since energy employed is roughly proportional to material strength given similar machine characteristics, this illustrates the dependence of tool wear on rock strength and on mineralogical composition or hardness. Machine and tool manufacturers usually consider both abrasiveness and strength in the calculation of cutter cost as revealed by the formula employed by Robbins (1976):

Cutter cost per meter of tunnel =

$$f_1 \frac{(\text{Diameter})^2}{\text{Thrust per cutter}} + f_2 \frac{(\text{Rock strength})(\text{Abrasiveness})}{\text{Joint and fracture intensity}}$$

Note the decrease in overall cost with increased thrust per cutter. In essence, this substantiates the reduced rate of wear for greater tool penetration as described above.

Unless discontinuities result in damage to cutters from falling or wedging blocks, their effect will be to reduce overall wear, as suggested by the Robbins formula. By normalizing cutter consumption relative to the energy consumed, however, their influence can be largely eliminated as a variable when operating in a changeable rock mass consisting of the same unaltered rock type (ie constant mineral composition; Innaurato et al, 1976).

Abrasiveness can either be measured through various index tests or by petrographic analysis. Most of the selected abrasion tests listed in Table 2-8 are similar in that the weight loss of a standard bit (Sievers, Abrasion value, Microbit) or of the material sample (Dorry, Taber abraser) is recorded after a specific amount of grinding. Other tests employ the weight loss of a model tool after a specific period or amount of material excavation (eg a model disc; Weber, 1972).

As for the indices used to predict penetration rate, the abrasion index can be correlated directly or in combination with other indices Two examples of combined indices are the Bit Wear Index (BWI: Selmer-Olsen and Blindheim, 1970) and total hardness. Because the abrasion test is basically a specific type hardness test, it is unlikely that combining several hardness or abrasion indices will improve the predictive capability by a significant amount.

Petrographic analysis is often used to determine quartz content and occasionally, mineralogical composition. The adverse effect of quartz, or minerals of equal or greater hardness, on tool wear is well documented. Quantitatively, the effect of quartz content is complicated by the rock structure (eg quartz grain size, texture, cementation, etc). One indice which considers these points is Schimazek's F coefficient of abrasivity (Schimazek and Knatz, 1970; Brown and Phillips, 1977). Formed as the product of abrasive mineral content relative to quartz, average grain size of quartz, and rock tensile strength, the index has been used for the prediction of tool consumption in Carboniferous sediments.

Unfortunately, the F coefficient, or any index considering quartz content alone, does not apply to strong rocks low in hard minerals. The classic example is the comparison of tool wear in a strong limestone with little or no quartz and a moderately strong sandstone high in quartz content. Again, using the Kielder site for illustration, the outside cutter on the Robbins TBM was changed more frequently when operating in limestone than sandstone, Figure 2-50. As shown, for

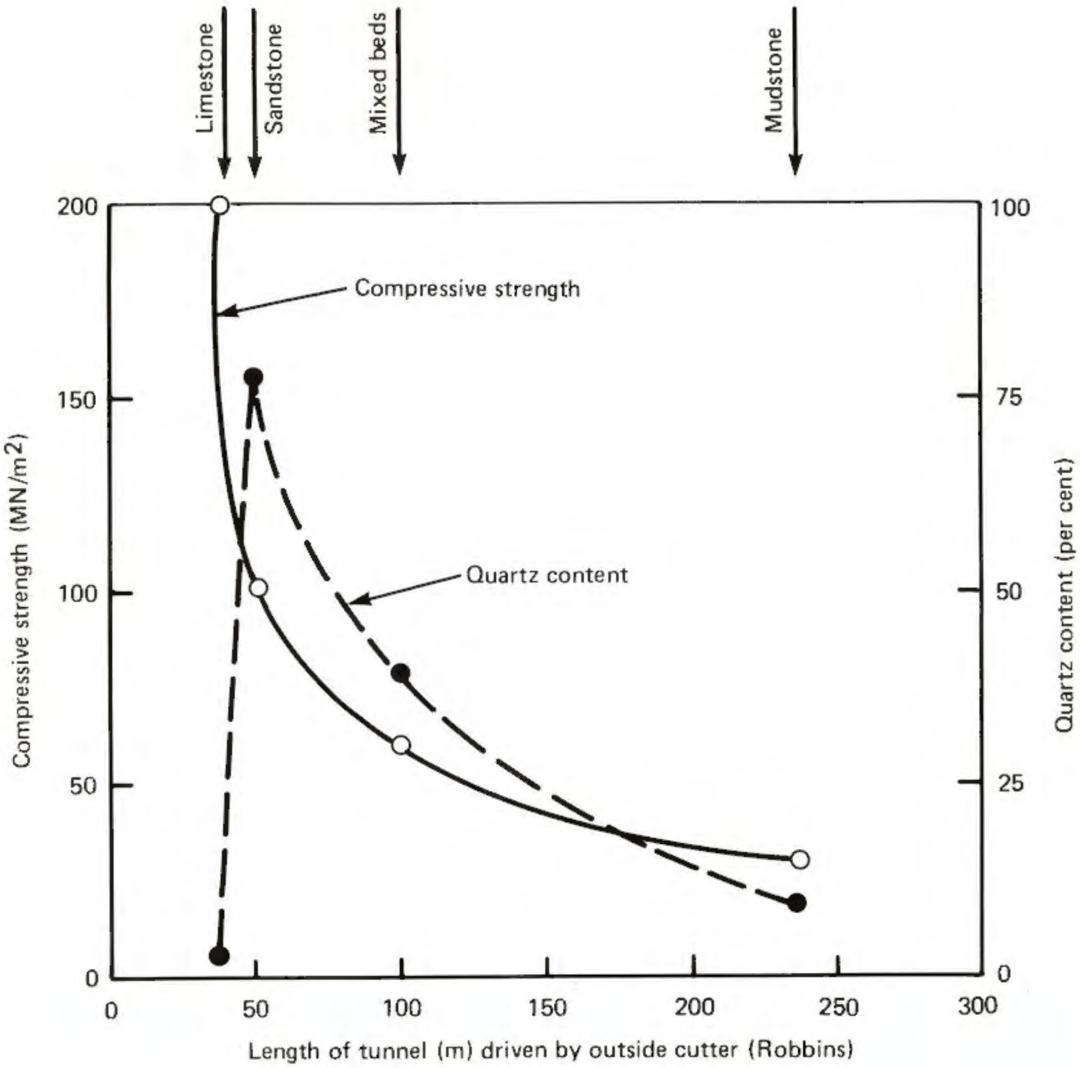


Fig. 2-50 INFLUENCE OF ROCK COMPRESSIVE STRENGTH AND QUARTZ CONTENT ON DISC WEAR (Partly after Brown and Milow, 1979)

the range of rocks between sandstone and mudstone the correlation between quartz content and disc wear was good.

The results presented in Figure 2-50 reveal the strong influence of rock strength on cutter life. Abrasion is undoubtedly a contributing factor, however, rather as a modifying or secondary influence. In the formulation of an abrasivity index based on mineralogical composition, strength and abrasion should not be considered as a product, but as a sum of the two indices. This form of abrasivity index has often been described in principle, although, not quantified (eg Pirrie, 1973).

Mixed face conditions tend to decrease cutter life as a result of the shock loading on travelling from weaker to stronger rock and the uneven distribution of thrust per cutter. The problem is more pronounced on machines equipped with strawberry button cutters, where the excessive load at transition areas is liable to fail the brittle carbide inserts (compacts). To minimize this, it is important to prevent the cutter head from tilting (Erkelenz, 1968). When driving in mixed face conditions at Kielder, the frequency of disc changes was slightly increased and isolated cases of triple disc bearing failures were recorded (Brown and Milow, 1979).

Another major factor affecting tool wear is the efficiency of the debris removal system. Without proper cleaning of the invest a large percentage of the cutting capacity is wasted on regrinding excavated material. This is a very destructive process as it results in a greater amount of tool abrasion than does the original process of excavation. Efficient debris removal systems operating in a dry environment often became ineffective in the presence of ponding or flowing ground water. Further discussion and examples of the influence of ground water are to be presented in the next Chapter.

With the advent of improved cutter materials and bearings, the trend over the past five years has been to higher thrust for improved tool penetration, particularly for operation in harder rocks (single disc mean normal load to 20 tons; Robbins, 1976). This has had the advantage of greater cutting rates and reduced tool wear, however, the disadvantage of a heavier machine and an increased required forward thrust. Theoretically, a higher thrust should result in greater tool spacing and fewer discs on the cutter head, such that the net change in machine forward thrust is unchanged or even reduced (size effect). In practice, this has not been the case. A major factor limiting increased tool spacing is believed to be machine stiffness. To take full advantage of high thrust tools, machine cutter head and forward thrust systems must be altered accordingly.

As an alternative to heavier machines and discs there is improved arrangement of tools employing double and possibly triple start spirals. Operating at moderate penetration and thrust per cutter edge, the multiple cuts per revolution are balanced against one large penetration. Physical limitations due to the space required to mount tools, especially near the pilot, ultimately controls the layout and number of starts. In the final analysis, the choice of fewer tools operating at high penetration and thrust or more cutters at moderate conditions depends on overall tool cost and wear as related to the particular machine under consideration.

SUMMARY

Indices employed for boreability predictions should be readily obtainable from simple tests and provide reasonable correlation with known machine performance. As a first approximation, compressive strength or a relatable index test is a good indicator of penetration rate. Furthermore, it is usually performed as a standard test for site investigations. For increased accuracy it is necessary to consider specific indices related to compressive or shear strength (eg Schmidt hammer and Shore scleroscope) and to tensile strength (eg Brazilian and point load) as well as the influence of discontinuities. Both material strength and an indice related to abrasion hardness or mineralogical composition are important in the estimation of tool wear. Those additional factors which influence the accuracy of penetration rate predictions are equally important to that of tool wear. Finally, and most important, all boreability indices are to be calibrated against a specific type of TBM with specific machine characteristics, the results of which are only applicable to that particular machine.

CHAPTER 3

MACHINE UTILIZATION

Over the last ten years steady improvements in machine and cutter design have led to significantly improved penetration rates, tool life, and machine reliability. Despite these advancements, the overall effect on increased progress rate or machine performance has frequently been limited by low machine utilization. In difficult tunnelling environments utilization of 20 to 30 percent is common and in general, a utilization of over 50 percent is considered good. In rare instances, utilization in excess of 90 percent has been recorded, particularly when penetration rate is low, allowing sufficient time for back up operations, and when the TBM performance is independent of the efficiency of the debris removal system (eg 1 m/hr cutting rate, operating on an inclined shaft; for example case history see Appendix IV, Sellrain-Silz Hydroelectric Project). For a significant proportion of TBM applications, however, a dramatic improvement in machine performance can be realized simply through increased utilization.

Machine utilization, like machine boreability, is strongly dependent on the design and operation of the TBM, its interaction with the tunnelling environment, and the adequacy of the back up systems designed to serve the TBM. On account of the interrelation between machine and tunnel environment it is not possible to estimate utilization without specific information on the design of the particular TBM under consideration. Just as one machine may achieve twice the penetration rate as another under similar conditions, utilization can likewise be markedly different. In general, however, it is possible to evaluate the major factors which influence performance such that the practicability of employing a full face TBM as well as the effect of alternative machine designs can be assessed.

This chapter considers the major factors which influence TBM utilization and the effect of design on cutter servicing, machine bracing, and ground support and control. The possibility of a universal type TBM capable of operating in variable ground conditions as well as the influence of opening size are also discussed. Finally, preliminary considerations such as, site investigation, method of excavation, use of pilot tunnels, and advantages of probing ahead of the tunnel face are briefly reviewed.

INFLUENCING FACTORS AND MACHINE DESIGN

The major factors which influence machine utilization:

(a) machine servicing
(b) cutter servicing
(c) machine bracing and resetting
(d) ground support operations
(e) water control
(f) debris removal
(g) utility supply and installation
(h) alignment survey
(i) operating crew efficiency

can be divided into those related to the tunnelling environment (geological factors) and those related to the machine and its back up systems. Of the two groups geologic factors undoubtedly pose the greatest uncertainty in the evaluation of utilization. In most cases, the uncertainty is a result of incomplete information on the tunnelling environment rather than the limits of machine applicability. At the tender stage it is relatively rare to have more than a rough estimate of the geology along the tunnel alignment. When difficult conditions are envisioned the influence on machine performance, expected delays, and methods by which to minimize the impact are evaluated as reasonably as possible. If these conditions are expected to persist for more than a few percent of the tunnel alignment, it is essential to consider the suitability of a particular machine design. Occasionally, the choice of a specific machine or design including several relatively minor modifications will make the difference between success and failure. Although the geologic factors pose the greatest uncertainty in the evaluation of utilization, major break downs (eg main bearing failure) and inadequate debris removal (eg waiting for muck cars, train derailments) are the main non-geologic factors causing delay.

In the consideration of machine utilization and design, the discussion in this section will concentrate on the influence of the following geologic factors:

(a) rock abrasivity
(b) ground bearing capacity
(c) rock mass stability and deformation
(d) water regime

CUTTER SERVICING

Rock abrasivity is only one of many factors affecting cutter life, and hence, delays associated with tool replacement and inspection. As previously described in the last Chapter, tool wear also depends on the type and arrangements of tools, machine stiffness, operating thrust, homogeneity of the tunnel face, and efficiency of the debris removal system. Regrinding of debris caused by inadequate removal in wet conditions not only increases tool wear, but also reduces penetration. The water source is not always from the ground as in extremely dusty conditions it is often necessary to drench the face for dust suppresion. In addition, sprayed water is occasionally used for the cooling of tools and bearings when operating in rock of high strength.

If the tunnel grade is sufficient, greater than one percent, water will not pond at the face alleviating some of the problems. If there is excessive water at the heading it should be pumped away. It may not only affect cutter performance, but depending on the rock mass character it can lead to accelerated ground softening with loss of steering capability (particularly when employing a Robbins TBM). When the water contains a large quantity of fines a slurry pump should be employed. As it is more difficult to remove slurry than water, the production of fines should be kept to a minimum when possible. Cutter type and arrangement producing large debris are favoured. Under the proper operating conditions an increased forward thrust will increase chip size. It is also important that the debris removal system has the ability to transport the slurry once removed from the invert (eg water tight muck cars).

Most TBM muck removal systems will not effectively dispose of water combined with debris. Even if the cutter head buckets manage to get the slurry on the conveyor system, it often spills off into the invert where it must be cleared resulting in further delays. If there is any possibility of developing excessive water at the face it is important to have the capability to remove it quickly and effectively.

Under normal conditions, cutter servicing should not occupy more than a few percent of the machine utilization. For example, four percent of the shift time was employed on the Robbins drive at the Kielder tunnels. If delays exceed 10 percent consideration should be given to improved performance and possibly a change of cutter type. When operating in hard and abrasive rock, changing from discs to strawberry button cutters will improve utilization, however, at the cost of reduced penetration (button cutters rarely exceed an average penetration of 1 m/hr). Consequently, the selection of cutter type requires a consideration of tool cost, utilization and penetration rate.

MACHINE BRACING

The stability of a TBM or its ability to maintain a specified alignment and develop forward thrust for excavation depends on the design of the machine bracing system, bearing capacity of the ground, and stability of the rock mass. Hard rock machines have traditionally relied on several sets of gripper pads reacting against the tunnel wall for steering and thrust. In terms of machine design, the TBM weight, bearing area of the pads, and required forward thrust determine the limiting conditions for instability. Most hard rock machines will experience considerable difficulties if the rock mass unconfined compressive strength falls below 0.1 to 1 MN/m^2 depending on the specific bracing system under consideration.

Figure 3-1 illustrates the three most common types of bracing systems in use. Both Wirth and Jarva typically employ a double cruciform pattern for a total of eight bearing pads, four front and four rear. This system is very rigid, well suited for operation in high strength rocks, and relatively easy to steer. However, it occupies considerable space, especially in openings of less than 4 m diameter, necessitates increased machine weight, and is vulnerable to common ground stability problems. The position of the pads between spring line and crown is a likely region for minor fallouts in blocky ground. Should overbreak occur, packing between the pad and tunnel wall may be required to obtain sufficient bearing area.

Demag also employs a double braced system, front and rear gripper pads, only with two diametrically opposed sets located in a horizontal plane. The system affords considerable rigidity and positive steering while allowing for increased access at the crown. Rigid axis or Kelly bar type TBM's (ie Wirth, Jarva, Demag) are steered during regrip.

For operation in smaller diameter openings, within the 3 to 4 m range, the Robbins TBM is considerably more versatile than rigid axis type machines. Employing only one set of diametrically opposed gripper pads in conjunction with a sliding pad fixed to the cutter head, this design minimizes weight and maximizes open space for access immediately behind the cutter head (see Plate A IV - 2, page 192). As the engaged pads are designed to float with respect to the machine frame during excavation, the Robbins TBM is steered during the forward stroke.

Compared to rigid axis type machines the Robbins design is not as stiff and in general, more difficult to steer. As previously discussed, the low stiffness can result in reduced penetration rates

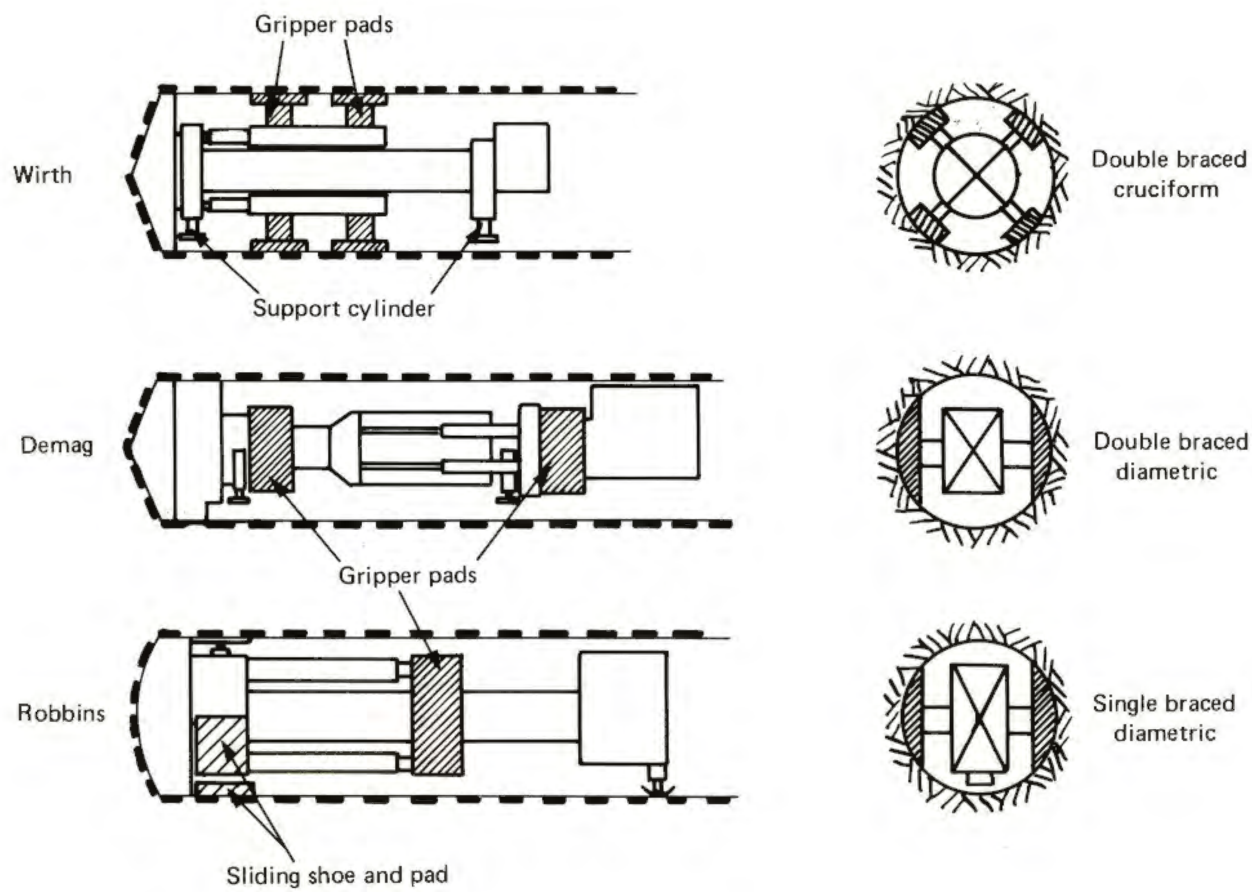


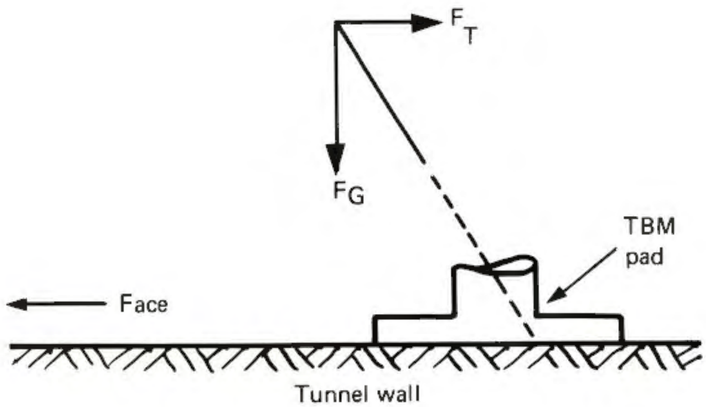
Fig. 3-1 DIFFERENT TYPES OF TBM BRACING SYSTEM (After Henneke, 1978)

and increased tool wear when operating in high strength rock. This condition has been improved on incorporating the front pads into the bracing system with various modifications. In weak ground the front pads can also present special problems. During the forward stroke they have been observed to plough the tunnel side walls leading to increased ground disturbance (Neyland and Murrell, 1970). With a large proportion of the machine weight concentrated at the cutter head, the machine also has a tendency to assume a "nose down" attitude when operating in weak ground.

To estimate the limit of machine stability, simple bearing capacity formulations based on plasticity theory can be employed, Figure 3-2. As shown it is necessary to estimate the applied gripper force as well as the minimum thrust required for excavation. The greater the forward thrust the lower the bearing pressure needed to cause failure of the ground assuming a constant gripper force.

For the evaluation of ground bearing capacity, one of the most important factors is the effect of water. Certain materials which are stable when dry result in a completely changed tunnelling environment when access to ground water exists. Swelling of specific clay minerals and water softening of poorly indurated rocks can reduce the bearing capacity to the point where the TBM sinks under its own weight. In many cases, problems start with slipping gripper pads. Progress is reduced, thereby, allowing for increased ground deterioration and further problems. If conditions do not improve the machine may fail to maintain line and grade as illustrated by several examples.

The Robbins TBM operating at the Kielder water tunnels struggled for 9 m through a fault zone composed of shattered mudstone and blocky sandstone until the water softened mudstone in the invert failed to support the machine. It started to sink and roll eventually dropping 0.3 m below grade. A total of 300 hrs were required to place the TBM back on line.



Bearing capacity factor

$N_c = 5\ (1 - 1.3\ F_T/F_G)$

Bearing capacity

$q = C_u\ N_c$

where C_u = Undrained shear strength
F_T = Forward thrust per pad
F_G = Gripper Force

Fig. 3-2 PAD BEARING CAPACITY

A machine of similar design to the Robbins, employed at Navajo Tunnel No 3, dropped nearly one meter in water softened shale (Sperry and Heuer, 1972). Slow progress was sited as the main reason leading to the difficulties. It was estimated that excess ground water, even in small quantities, reduced average progress by half, largely as a result of slipping pads and delayed installation of support. In another case, water softening of a mudstone encountered during the excavation of a sewage tunnel in Waiblingen, Germany forced the removal of the TBM after it became impossible to brace the sinking machine (Krause, 1976). The opening was stable in dry conditions.

If a TBM drops below grade and there is little likelihood of manoeuvring back on line, it is advantageous to stop the drive for remedial work. After backing the machine off the face a steel and/or concrete ramp can be constructed in the invert. When practical, steel ramps are preferred as they eliminate the time needed for cure of the concrete. After installation the machine cutter head is simply skidded up the ramp to the proper elevation. Naturally, it is first necessary to over excavate the crown in order to make room for the machine. If this is carried out by blasting there is always the possibility of damage. Attempts to jack up the Robbins TBM after it went off line at Kielder did not prove successful; the contractor resorted to a concrete and steel rail ramp.

Temporary problems with gripper pad slip or bearing failure are commonly remedied by increasing the bearing area with timber or concrete packing. In extreme conditions a concrete buttress or wall plate can be formed along the tunnel side wall. Needless to say, this operation would be extremely time consuming. Under certain conditions it is also possible to thrust off of ground supports. Other solutions include reducing the forward thrust required to excavate by constructing a pilot hole ahead of the tunnel face. A one meter square by 2 m deep pilot opening excavated by drill and blast was successfully employed for this purpose when failing side walls prevented normal operation (Embery, 1976).

If the bracing system can not maintain the stability of the TBM over a significant proportion of the drive a soft ground or shield system may be employed. Forward thrust and steering are obtained through a set of hydraulic rams reacting against the tunnel support or less commonly against a reaction ring. The reaction ring system, Figure 2-38, eliminates the necessity to fully line the opening, however, the massive and bulky ring structure is a permanent component of the machine. TBM's have been designed to obtain thrust both through bracing systems and to push off tunnel supports (Norman, 1972). The major problem with adapting these systems to hard rock partially or non-shielded machines is the large delays associated with changing from one method of tunnel driving to another.

In the majority of situations in which hard rock TBM's are employed, soft ground conditions are encountered for only a small percentage of the drive. They are usually in the form of localized faults, shears, weathered, and/or altered zones on the order of meters to tens of meters wide. Their exact location is rarely known and the contractor must be prepared to deal with the problems quickly and effectively.

To illustrate the problems typically encountered when operating in difficult ground, seven examples of machine performance while operating in fault zones are presented in Appendix VI (Kielder Water Scheme, "Machine performance in difficult ground"). The major findings are summarized below.

Small amounts of ground water inflow invariably led to bearing failure due to softening of the shattered mudstone. In dry conditions neither machine bracing nor cutting rates were adversely affected. On several occasions penetration rates actually increased upon excavation within the fault zone, although, progress was reduced by the need to support the ground. Mixed face conditions, shattered mudstone and intact blocky sandstone, required relatively high forward thrust for penetration. However, the bearing capacity of the tunnel side walls was rarely sufficient for normal operation.

Major problems developed on attempting to excavate from the fault zone back into massive rock. Again, it was not possible to obtain the necessary forward thrust. Figure AIV - 6 (page 203) illustrates the high cutting rate on passing through the fault followed by a sharp drop on encountering the massive sandstone. After several days of slow progress (.13 m/hr), caused by slipping pads, the TBM passed out of the fault to a position at which the pads could brace against the sandstone. As shown in the Figure, once the bearing problem started it usually persisted until the distance between the front of the cutter head and the gripper pads in full retracted position was covered.

An ideal bracing system for coping with the difficulties on passing from weak to strong ground was employed on the Lawrence, Alkirk model TBM (Ingersoll Rand Co). The machine can not only obtain thrust through a traditional bracing system, but also employs a hydraulic packer within a pilot hole for pulling. Excavated by a tricone bit, the 0.6 m diameter pilot hole is driven through the centre of the cutter head for up to several meters ahead of the face. Its operation is independent of the machine cutter head. The system has the disadvantage of complicating steering and when used improperly, it can lead to face stability problems. With some modification this system could be used for probing ahead of the tunnel face.

Not all bracing problems result from bearing failure of the ground. Another major source of difficulties is caused by ground instability and overbreak. Given the limited stroke of gripper pad rams, excessive overbreak requires packing between the wall rock and pad. These operations although less troublesome than ground bearing failure, nevertheless cause considerable delay. Solutioned caverns in karstic limestone can have the same effect as excessive overbreak. Control of overbreak to minimize bracing problems is closely related to the rock mass stability and deformation, and the ground control measures employed.

GROUND SUPPORT AND CONTROL

Undoubtedly, one of the major factors influencing overall TBM performance is the rock mass character. Very few openings constructed in "hard rock" require no support. Although the majority of tunnels may need no more than relatively minor stabilization measures, a small percentage of the alignment may require extensive measures. The TBM and its back up systems must be designed not only for the average conditions, but also for the most difficult condition.

The limit of machine applicability is controlled by the rock mass stability and ground deformation. Stability can be assessed in terms of standup time, the time elapsed after excavation that an unsupported tunnel face will remain stable (ie no major fallouts or excessive ground loosening). The most important factors to affect standup time are the following:

(a) time dependent strength - deformation characteristics of the rock mass
(b) insitu state of stress
(c) water regime
(d) size and shape of opening
(e) method and rate of excavation
(f) method and rate of support and/or reinforcement installation.

For the evaluation of opening stability, it is important to note that those factors concerned with the tunnel environment are interrelated with those of the construction method. In other words, interaction of the TBM with the tunnel environment strongly influences the heading standup time.

Ordinary hard rock TBM's can not operate in conditions where the standup time is very short, on the order of minutes or seconds. A collapsed heading only jams the machine cutter head making it impossible to operate. Ground conditions with intermediate standup time, minutes to hours, can be effectively controlled provided the machine has provisions for the rapid installation of support and/or reinforcement near the face. This obviously requires a machine design which maximizes access to the region immediately behind the tunnel face. Further improvements in opening stability are obtainable through methods of ground prereinforcement (described at end of section).

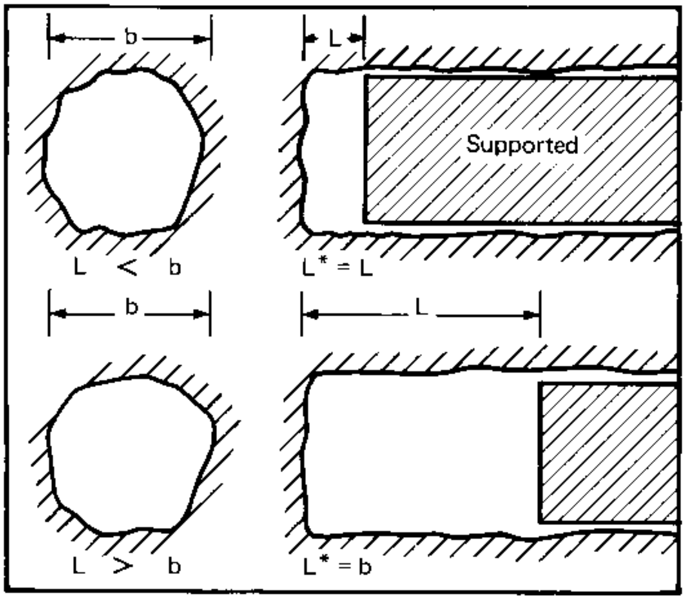
In a qualitative manner, it is possible to describe the influence of various ground conditions on opening standup time and deformation or overbreak, Table 3-1. Geologic factors considered are: intact rock strength, discontinuities, water regime, stability number, and mineralogical composition. Stability number is the ratio of insitu major principal stress to rock mass strength (in soil mechanics the undrained shear strength). At a critical ratio ductile materials undergo plastic flow or squeeze (for clay ratio of roughly 5; Attewell and Boden, 1971). Brittle materials can fail in a violent fashion with considerable release of energy, a rock burst, or less dramatically as rock slabbing. It is fully appreciated that the consideration of discontinuities is over simplified, however, the Table is only for purposes of illustration. A more detailed classification including scale (aperture, persistence and spacing), character (roughness, properties of filling materials or coatings), strength and deformability is necessary for completeness.

Several classification systems exist for estimating tunnel support requirements (Terzaghi, 1946; Bieniawski, 1974; Barton et al, 1974), however, prediction of opening stability essentially remains an art. Based on field observations, Lauffer (1958) developed a semi-quantitative evaluation of standup time, Figure 3-3. It clearly illustrates the dramatic effect of opening size and/or active span as well as rock mass character on stability.

Soft or weak ground conditions are normally excavated with the aid of a tunnel shield. If, however, the ground possesses a small amount of cohesion and water inflow is negligible, a hard rock machine can usually excavate the material as long as the standup time is not

TABLE 3-1. Tunnel environment characterization

GROUND DESCRIPTION	TUNNEL ENVIRONMENT															GROUND RESPONSE			
	Intact rock strength			Discontinuities				Water inflow			Stability number			Mineral comp		Stand-up time		Overbreak or deformation	
	Low	Medium	High	Crushed	Well developed, tight	Open, weak infilling	Massive	None	Minor	Major	Low	Moderate	High	Dependent	Independent	Short	Long	Minor	Major
soft or weak	x						x	x			x				x		x	x	
	x						x		x		x				x	x			x
squeezing	x	x						x				x →	x		x	x ←	x		x
blocky and seamy		x				x		x			x	x		x			x	x	
		x				x			x		x	x		x		x			x
blocky		x	x			x		x			x				x	x			x
		x	x		x			x				x			x		x	x	
		x	x		x				x	x		x			x	x			x
bursting or slabbing			x				x					x →	x		x	x ←	x	x	
ravelling						x		x			x			x		x			x
				x				x							x	x			x
flowing						x			x	x					x	x			x
				x					x	x					x	x			x
sloughing or slaking							x	x	x					x			x	x	
swelling	x	x					x	x						x			x	x	
	x	x					x		x					x			x		x



DEFINITION OF 'ACTIVE SPAN', L*

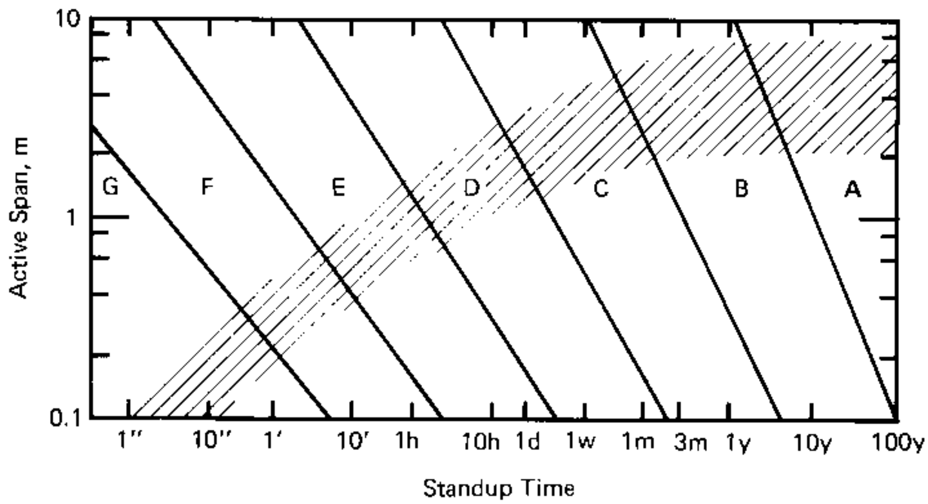


Fig.3–3 ACTIVE SPAN VERSUS STANDUP TIME; A→G, BEST TO WORST GROUND CONDITION; SHADED AREA INDICATES PRACTICAL RANGE OF APPLICATION (After Lauffer,1958)

exceeded, the machine remains stable, cutters function without clogging, and support is placed immediately behind the cutter head as close to the face as possible.

Several examples of successful operation within dry fault zones largely composed of weak shattered mudstone are presented in Appendix IV, Kielder Water Scheme. For the most part, the mudstone was of a similar character to a stiff fissured clay (see Plates AIV - 4, 5 and 7 on pages 201 and 204). Despite the low strength, it possessed sufficient cohesion to remain stable long enough to place steel sets and lagging behind the cutter head (active span of 2-3 m, standup time greater than 2 hrs). Overall progress during operation within the fault zones averaged between 0.3 and 1 m/hr depending on the quantity of support installed. A major factor contributing to the success of the operations was the relatively steady continuous advance. As previously described, a small amount of ground water inflow could completely change the conditions for the worst.

Excessive inflows of water through weak or cohesionless soil type materials with a high permeability can lead to piping of the material, or what is often called flowing ground. As it is not possible to control the heading by conventional tunnelling methods, the ground requires special treatment before excavation can proceed (eg freezing, grouting, dewatering). Special purpose bentonite or slurry shield TBM's have been designed for operation in silt and sand below the ground water table, however, practical application is limited to relatively homogeneous materials (for a case history in which a bentonite shield was employed see Appendix IV, Antwerp Pre Metro).

If there is sufficient reason to suspect a flowing ground condition, such as a shear zone filled with granular material, intersected well below the water table, it is imperative to probe ahead of the tunnel face. An accidental encounter can be catastrophic as clearly illustrated by the photograph, Plate 3-1. This 3 m diameter Robbins TBM was operating in limestone when after 6.1 km the drive was brought to a sudden halt.

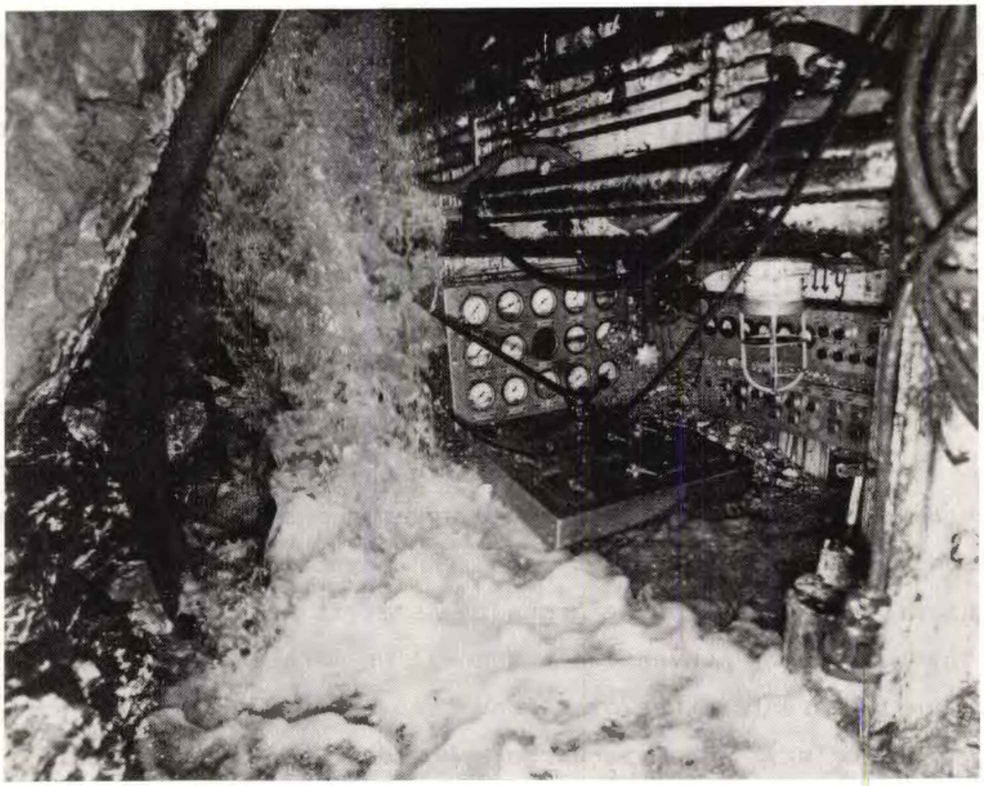


PLATE 3-1. Inundated Robbins TBM after encountering flowing ground condition, Galerie de la Coche, Arc Isere hydroelectric scheme (courtesy of Electricité de France)

On intersecting a crushed zone of several meters in width, the entire front half of the machine was inundated with water and debris. After excavating around the TBM to free the cutter head and assess damage, it was decided to remove the machine and complete the work by traditional methods. Up until the accident the progress rate was better than 200 m per month (for additional details see Appendix IV, Arc Isere Hydroelectric Scheme).

As opposed to flowing ground, ravelling ground is weak soil or rock with negligible cohesion, however, uncomplicated by inflowing water. This includes sheared or crushed sand like materials and heavily jointed rock masses in which the partings are filled with weak materials such as chlorite, talc, or graphite. The ground has little or no standup time and most often requires hand excavation. Forepoles and steel sets are commonly employed to maintain the heading stability. On account of the close spacing of the forepoles, it may be more practical to work ahead of the TBM. Under certain conditions it is possible and/or preferable to grout the material before excavation, although, this may cause as much delay as hand mining. Furthermore, it is necessary to know the location of the problem zone, once intersected by the machine cutter head major delays are already incurred.

A 5.8 m diameter Wirth TBM excavating a 10 km water tunnel in France encountered 13 major shear zones of from 10 to 50 m wide. Filled with a shattered mica schist with little cohesion, eight major delays occurred when large fallouts of the ravelling ground completely blocked the heading. On two occasions the machine head was so severely blocked that conventional tunnelling methods were employed to pass through the zones and free the machine. Although the 300 m of shear zone only totalled three percent of the drive length, over 25 percent of the total construction shift time was required to traverse the difficult ground for an average progress rate of 0.85 m per day. Normal progress in ground without significant stability problems was 16.3 m per day (Appendix IV, Arc Isere Hydroelectric Scheme).

Environmentally sensitive materials which result in sloughing or slaking on exposure to the atmosphere and in swelling given access to water, require a finite period to react. In many cases, if the TBM can make steady progress it will not be affected and remedial work can proceed a short distance behind the tunnel face. Should delays stop progress, ground deterioration and softening will occur. If the TBM has a shield, even a partial shield, stoppages in swelling ground are critical as the free swell can develop into a high pressure immobilizing the machine. The developed pressure depends on among other factors the amount of free swell allowed and the mineralogical composition of the clay component (additional details given by Brekke and Howard, 1973).

Both squeezing and swelling ground result in significant ground deformation, however, the squeeze or plastic flow results from high insitu stress as related to material strength rather than the reaction of a specific mineral component to water. Squeezing grounds often contain a major component of clay minerals or other weak and ductile materials. As with swelling, the squeeze is a time dependent phenomenon.

Efficient methods of dealing with squeezing ground employ flexible stabilization measures, thereby, allowing the rock mass to mobilize its own shear strength. Immediate support and/or reinforcement are essential to minimize the deterioration of the material nearest the opening. The New Austrian Tunnelling Method (NATM), employing rock bolts and shotcrete in a controlled manner, is a proven method of construction (for description of method see case histories in Appendix IV, Arlberg and Pfander Road Tunnels). Rock bolts and mesh without shotcrete have also been successfully used (Appendix IV, Frejus Road Tunnel). If it is possible to work without shotcrete, the tunnel working environment is considerably improved. This is particularly applicable to TBM operations where the shotcrete causes problems of clean up and possible machine damage. As an alternative method of stabilization, yieldable steel sets may be employed. Although this system is not as effective or versatile as reinforcement and shotcrete, it is more adaptable to TBM operations especially in smaller diameter openings.

For successful TBM operations in squeezing ground, it is essentially necessary that the design of the machine allow for the implementation of the proper stabilization measures. This means rapid installation of support and/or reinforcement at or near the face. Aside from a minimum partial shield to protect the cutter head no shield should be used. A shield only delays the immediate stabilization measures the opening requires at the expense of increased difficulties to TBM operations.

Although full face TBM's have not been employed for major drives in squeezing ground, many machines have encountered localized squeezing zones with varying degrees of success. A 4 m diameter Robbins TBM operating in sheared pyroclastics was trapped while excavating an extension for the Mount Lyell Mine (Embery, 1976). To free the machine, it was necessary to blast a cavity over the shield. Delays leading to the problem were largely on account of slipping gripper pads. Subsequent to the excavation, opening closure reached 0.6 m after three months (15 percent closure). In addition, squeezing pressure on the asymmetric 270 degree shield possibly contributed to the loss of grade while operating in a difficult section of tunnel (0.25 m drop over 12 m).

Zones of weak material encountered at depth with limited extent, less than 10 m width, may not exhibit significant squeezing behaviour if bounded by relatively rigid rock. As a result of shear at the zone boundary, the influence of the overburden pressure and ground displacements will be restricted.

Where overstressed ductile materials tend to squeeze or yield in a relatively controlled manner, brittle rocks can fail in a dramatic unpredictable fashion with considerable release of energy. The intensity of the failure depends on the amount of stored energy, as related to surface topography and residual or locked in stresses, and the brittleness of the rock. In severe conditions, detached slabs of rock literally shoot across the opening. Although the volume of fallout is often minor, the danger and damage potential from the flying debris is extreme. In addition, the failures are progressive, eventually resulting in significant overbreak.

Scandinavian hard rock tunnellers have had considerable experience with the control of rock bursts. An economic and effective stabilization measure involves the use of short rock bolts and wire mesh installed close to the tunnel heading. Known as skin or surface bolting, the procedure is basically designed to contain the failed rock rather than prevent failure (for additional details and a discussion of traditional tunnel practice see Appendix IV, Underground Design and Construction Practice in Scandinavia).

In less extreme conditions, the rock tends to slab off the tunnel walls without the violent bursting. Even though the safety of the working environment is improved, immediate stabilization measures are often necessary as illustrated by several case histories.

Operating at a depth of more than 2.1 km, a 3.4 m diameter Robbins TBM encountered considerable difficulties on account of stress induced slabbing of brittle quartzites (Graham, 1976). Resulting overbreak in the sidewalls often extended to the face, causing loss of grip and necessitating the use of packing. This was largely corrected by installing rock bolts and steel sets immediately behind the cutter head, ahead of the rear gripper pads. Side wall fallouts, although troublesome, were not as damaging or difficult to contend with as those occurring at the face. Loosened blocks of rock jammed the cutter head and buckets, and damaged cutters and the conveyor belt. To limit the size of debris entering the buckets and prevent jamming of the head, a system of grid bars and cover plates was installed on the machine cutter head, Figure 3-4. Short cutter life, particularly of the gauge discs, also had a detrimental effect on the drive. This was not only related to the hard and abrasive nature of the quartzite (unconfined compressive strength around 200 MN/m^2), but the relatively low stiffness of the machine. As previously described, the design of the bracing system

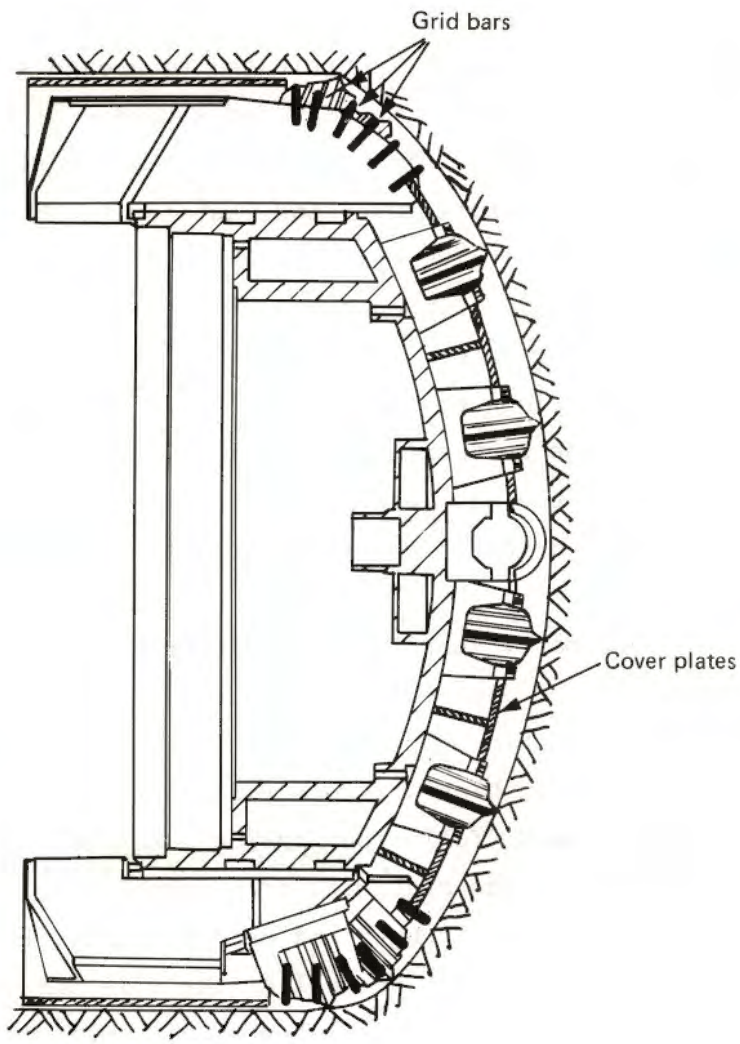


Fig.3–4 ROBBINS CUTTER HEAD SHOWING FALSE FACE CONSTRUCTED WITH COVER PLATES AND GRID BARS (After Graham, 1976)

employed on the Robbins TBM is not well suited for operation in high strength rocks unless modified to increase cutter head stiffness.

Similar problems developed during excavation of the 6.2 m diameter Navajo Tunnel No 3 (Sperry and Heuer, 1972). Stress induced slabbing of the low strength sandstone contributed to ground support problems which limited advance to 3 m per day for a period of over three months (approximately 0.13 m/hr). Ironically, this same drive produced several record advance rates over the last few weeks of operation (33 m in one 8 hr shift, 79 m in one 24 hr day, 325 m in one 5 day week; Etheridge, 1973). Although the rock bolt and sheet lagging reinforcement system performed effectively, it was not possible to install the system close enough to the face. Considerable fallouts occurred between the dust shield, immediately back of the cutter head, and face, a distance of at least one meter. Overall, 37 percent of the drive time was devoted to ground support.

For the most part, "hard rock" TBM's are designed to operate in relatively massive, medium strength ground, at moderate to shallow depths of cover. Within this environment the geologic anomalies most frequently plaguing machine drives are discontinuities. They include joints, bedding planes, foliation partings, seams and/or shears (minor faults). In general, the stability of a tunnel heading depends on their character, scale, strength and deformability, the additional factors listed in Table 3-1, as well as the size and orientation of the opening with respect to the system of discontinuities. As there exists a large volume of published material on this topic (eg Terzaghi, 1946; Brekke and Howard, 1973; Barton et al, 1974), the discussion herein will be restricted to the effect of instability on TBM performance and preventative measures.

To simplify matters, only the general categories of blocky and blocky and seamy type ground are considered (Terzaghi, 1946). Blocky rock masses are characterized by well developed, relatively tight joint systems which upon disaggregation yield intact, though separate blocks of rock. In blocky and seamy ground, the intact rock fragments do not physically interlock, but are surrounded by a matrix of weak alteration and/or weathering products.

The problems related to blocky and blocky and seamy ground are in many aspects similar to those of slabbing rock. Fallouts from the tunnel side walls result in loss of bearing surface for the gripper pads, and consequently, delays associated with installation of packing and removal of loosened debris. Removing debris from around the TBM is a difficult and time consuming task. Often, the larger pieces of rock must be broken into a manageable size before they are carried out. An example of a badly overbroken side wall of blocky mudstone in a machined opening is shown in Plate 3-2. The simplest method by which to control this form of instability is rapid installation of support and/or reinforcement as near to the face as possible.

It is well known that the delayed installation of support only results in increased deterioration of blocky rock masses. Since most hard rock TBM's which employ shields use a relatively flexible partial cover that is not in intimate contact with the tunnel wall, the shield does little to prevent ground deterioration. If the shield is relatively long, the accumulated weight of loosened debris can eventually reach the point at which it is not possible to steer or operate the machine. Increasing the shield cover, stiffness, and intimacy of contact also results in a machine with steering problems. In addition, a complete shield, as shown in Plate 3-3, makes it impossible to perform remedial work near the face or when necessary, attend to bearing problems in the side walls. Shortly after the pictured TBM was placed in operation the rear covers were removed (Snowden, 1979).

Shields installed behind the cutter head are often employed simply because there is no space for support operations near the face. As a consequence, it is necessary to protect the inaccessible portion of the machine from fallouts and possible damage. The long rail shield

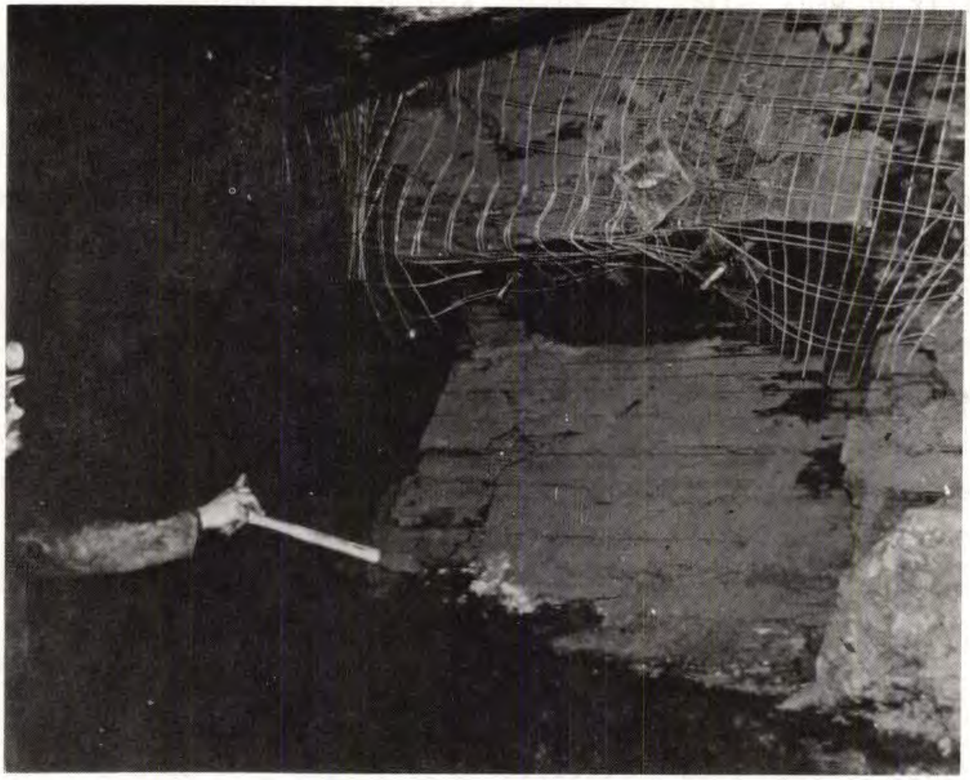


PLATE 3-2. Overbroken tunnel side wall in blocky mudstone

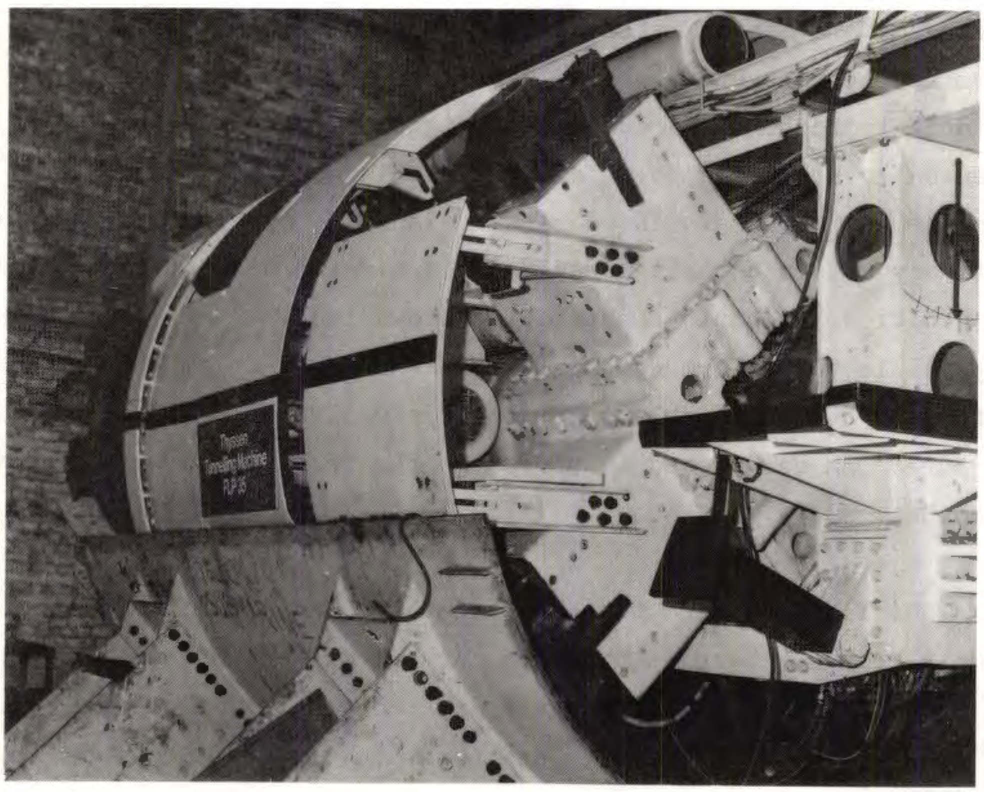


PLATE 3-3. TBM employing a full shield behind cutter head

installed on the 3.5 m diameter Demag TBM's employed for the N and S Wear drives of the Kielder water tunnels is a prime example, Plate AIV-1 (page 192). As shown in Figure AIV-2 (page 189) the nearest support location was 11.7 m from the tunnel face. When operating in blocky mudstone, the volume of loosened debris resting on the shield often totalled several cubic meters. It was necessary to remove this material by hand before the installation of rock bolts and wire mesh. In very blocky and/or shattered mudstone the height of loosened ground occasionally exceeded the opening diameter. This weight of rock deformed the shield and made it impractical to operate the TBM. Before operation could proceed, the ground was again removed by hand and support was placed from on top of the shield, Plate 3-4.

Despite the ability to place support within 3 m of the face, the Robbins TBM was only about 60 percent faster than the Demag when operating under similar difficult ground conditions. Placing steel sets and lagging behind the Robbins cutter head was not an easy operation, requiring at least one hour per set. The main supporting station was located 15 m from the face as shown in Figure AIV-3 (page 190). In blocky ground conditions, with only moderate stability and overbreak problems, support behind the cutter head was necessary for safety, Plate AIV-3 (page 193). Under these less extreme conditions, the Demag TBM actually displayed a higher progress rate than the Robbins, however, most or all of the economic advantage was lost on account of the excessive overbreak. If the rock mass is expected to lead to even minor stability problems, the primary support station must be located near the face, not 11.7 or 15 m behind.

Within the 3 to 4 m diameter range of openings, the Robbins design TBM provides the greatest amount of usable space directly behind the cutter head for support operations. For larger diameter tunnels, the rigid axis type machines offer several advantages. Increasing the size of the Robbins cutter head necessarily results in an increase in width and weight, placing the nearest support location at least 3 m from the face. Although the rigid axis

PLATE 3-4. Overbreak in very blocky mudstone

PLATE 3-5. Articulated support arm employed for drilling ahead of tunnel face

machines are heavier, they are better balanced and can provide for improved access immediately behind the face. A 6.1 m diameter Demag TBM operating in a German coal mine has its primary support station at 1.7 m behind the tunnel face. Employing an erector system to place yieldable steel sets on from 0.75 to 1.5 m centres, the installation time is roughly 20 minutes per set (for additional details see Appendix IV, Victoria Collieries). Other large diameter Demag TBM's are being designed for support installation within 1.7 m of the heading.

Stability problems within blocky rock masses are not always restricted to the tunnel walls behind the cutter head. In many instances, loosened blocks of rock drop out of the face damaging cutters, buckets, and machine head as well as blocking the buckets and jamming the cutter head. Aside from the machine damage, the loose blocks of rock are often dragged around leading to further ground disturbance. A common solution for protection of the cutter head and tools is to employ a false face as previously described, Figure 3-4. This system has not always been successful, particularly when the blocks are strong and abrasive.

To combat overbreak at the face and its adverse effect on the cutter head, a 25 mm thick false face was installed on the 6.4 m diameter Jarva TBM operating in fractured andesite (Bennion, 1976). This did not prevent continued cutter damage and the false face itself suffered considerable deterioration. Subsequently, the thickness was increased to 50 mm and the cover plates were installed so as to further reduce the protrusion of the disc cutters. The second modification also failed to reduce the damage. Eventually, the TBM was moved to another heading with improved ground conditions.

In another example, loosened blocks of granite jammed the head, blocked buckets, and broke cutter mountings on the Wirth TBM employed at Nast Tunnel (Geary, 1972). A false face was used to protect the cutters and a short shield was installed on the head to prevent blocks of rock from becoming lodged in the buckets. Both the rotating shield and the false face eventually wore out. A new cutter head was fitted, incorporating a static circular shield around the bucket race.

As shown in Plates 2-5 and 6 (page 80) and Plate AIV-2 (page 192), the debris removal buckets located on the perimeter of the cutter head are most vulnerable to loosened material. Occasionally, their motion against the tunnel wall is responsible for additional ground loosening. In weak ground the protruding buckets can stall the cutter head given excessive penetration. A static shield of minimum proportions installed over the bucket race will eliminate some of the problems. However, it is necessary to over cut this shield as the leading edge can not penetrate hard rock.

More efficient mucking systems are of an "open head" design, allowing for debris removal through the centre of the head as well as at the perimeter. The Robbins Melbourne type TBM essentially provides a flexible shield cover for the rotating head and mucking through the face with a blade system, Figure 3-5. In addition, the head is flat rather than domed for increased face stability. Openings in the head are large enough for access to the face via the conveyor system. This is most convenient for ground control and cutter servicing. Disc cutters can be replaced with picks to reduce bracing forces if the required forward thrust becomes excessive for ground conditions. The cutter head also employs a reduced head speed of 3.5 RPM which was claimed to minimize ground damage (Neyland and Murrell, 1970).

In general, it is not practical to use the machine cutter head for stabilization of the heading. When large fallouts occur at and ahead of the tunnel face, normal operation can proceed only after the heading stability has been regained. The installation of forepoles in conjunction with steel sets, Figure 3-5, is a common method by which to control overbreak. Widely used as a technique in traditional methods of tunnel construction, it is finding increasing application in machine drives. One of the major limiting factors is the large amount of working space required behind the cutter head and access

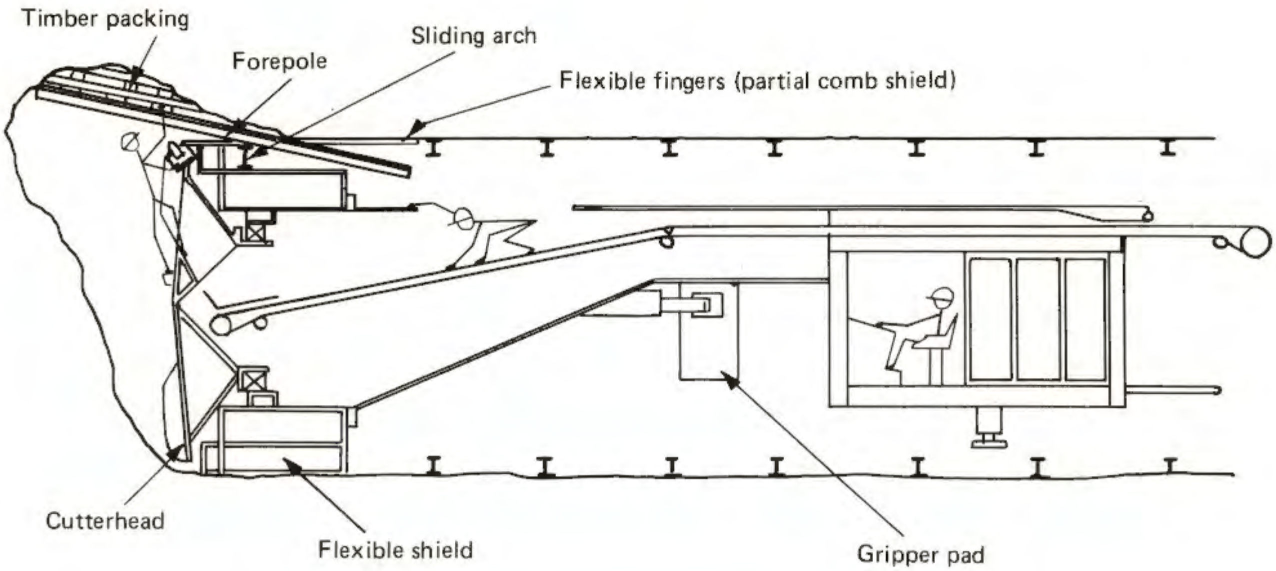


Fig. 3-5 MELBOURNE TYPE ROBBINS TBM (After Sugden, 1975)

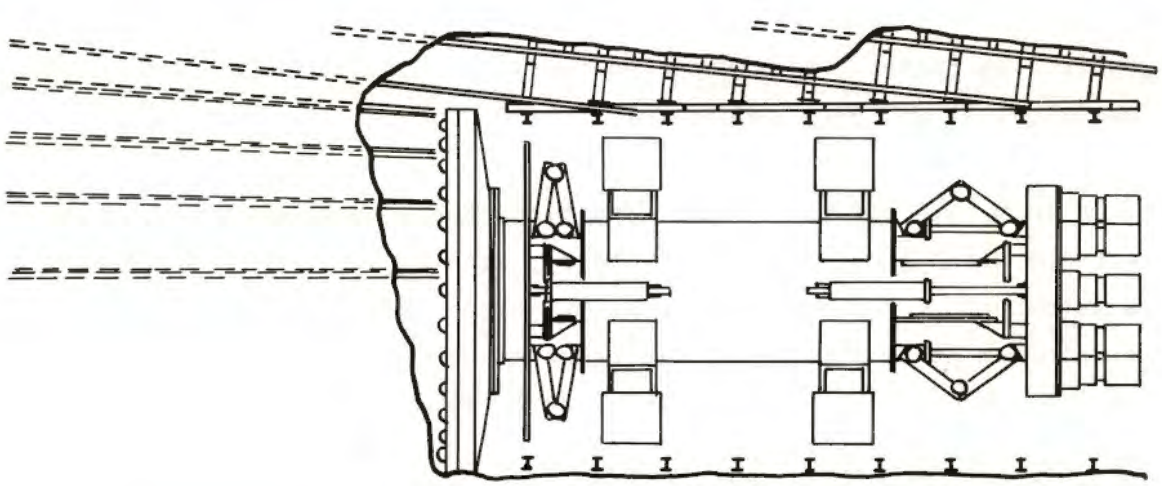


Fig. 3-6 FOREPOLING OVER CUTTER HEAD OF A 6.4m diam. JARVA TBM FOR HEADING STABILIZATION (After Bennion, 1976)

to the face. Unfortunately, the method is slow and labour intensive, however, it is often the only practical solution for the control of ground with little or no standup time.

To accommodate forepoling, the Melbourne type Robbins TBM employs a flexible shield with slots or fingers such that they can be installed close to the face, under the protective cover of the partial shield. This system was effectively employed to stabilize a blocky mudstone with open joints (Neyland and Murrell, 1970). Although considerable hand mining was required to complete the drive, the Melbourne type machine was a significant improvement over the conventional hard rock Robbins TBM originally installed.

A system of forepoling was also used to maintain heading stability during the excavation of a weak volcanic tuff, Figure 3-6 (Bennion, 1976). The 6.4 m diameter Jarva TBM had provisions for the installation of steel sets under a short roof shield immediately behind the cutter head. From this position, heavy square section, 10 m long steel forepoles were installed within predrilled 150 mm diameter holes; the entire fan pattern being repeated every 1.2 m. Although the procedure was slow, it was very successful.

In many difficult ground conditions the technique of forepoling, a temporary support system, can be replaced by spiling reinforcement, a permanent prereinforcement system. Like forepoling, spiling reinforcement is designed to increase standup time or heading stability and reduce overbreak (Korbin and Brekke, 1976 and 1978). It involves reinforcing the surrounding rock mass ahead of an advancing face as shown in Figure 7-7. This is generally accomplished by placing in predrilled holes fully grouted untensioned steel members such as reinforcement bars, on a regular pattern. The advantage of this system over forepoling is the reduced time for installation and the permanent contribution to the ground stabilization system.

To employ this method on a mass production basis would require

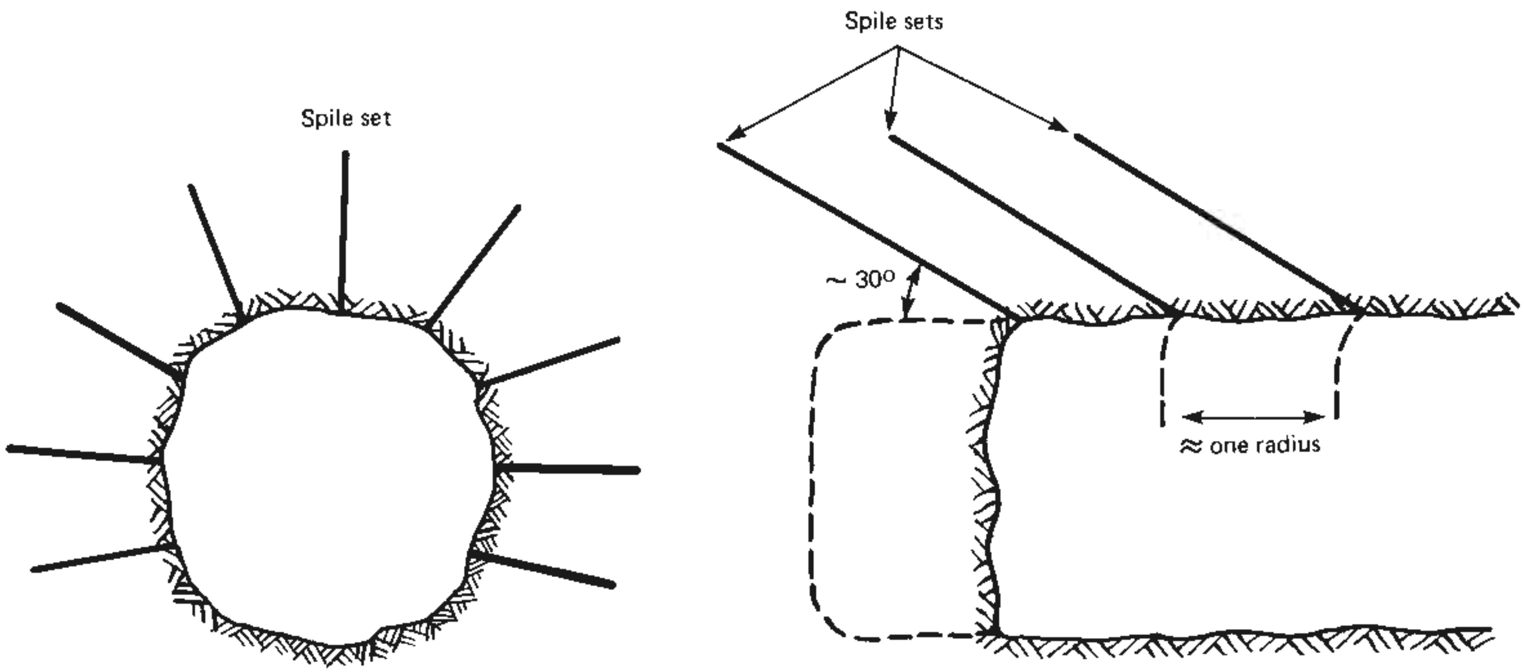


Fig. 3–7 SPILING REINFORCEMENT AHEAD OF TUNNEL FACE
(After Korbin and Brekke, 1976 and 1978)

the installation of one or more drill rigs behind the machine cutter head. Despite space limitations within smaller openings, this has been successfully carried out on two 3.5 m diameter Robbins TBM's and one Wirth machine in connection with tunnelling for the Oslo sewer system (for additional details see Appendix IV, Oslo Sewer Tunnels). The Robbins TBM employs two hydraulic powered drills each positioned by an articulated arm. Both arms are mounted on the underside of the main support frame between the operator's cabin and gripper pads, Plate 3-5. With the present design, holes can be collared within 2.5 m of the face at an angle of six degrees from the tunnel axis, however, a redesign can increase the angle to 30 degrees. Although the entire system is to be used for grouting to control ground water inflow, it is equally applicable to the installation of spiling or grout injection for rock mass stabilization.

In the future, the installation procedure of spiling reinforcement may be simplified by substituting a pumpable bolt for the spile. Similar to a grout, once the fibreglass reinforced polymetric fluid is injected into the hole it rapidly hardens into a usable rock bolt or spile. Lengths of up to 15 m are being considered (Habberstad et al, 1974).

UNIVERSAL TBM

Despite the desirability of a universal TBM, a system which can operate in any ground condition, none exist and the likelihood of constructing such a machine in the near future is remote. Some ground conditions will always require special techniques or machines. In particular, tunnelling in weak or cohesionless permeable materials below the ground water table, normally a flowing ground, is restricted to a special class of slurry or bentonite shield TBM's. Likewise, operation in very strong massive rock requires a heavy machine with high forward thrust, a stiff bracing system and cutter head, and high power output. Between these two extremes, in the range of ground conditions from a ravelling soil type material to a massive medium strength rock, considerable effort has been made to find an appropriate design. This is classified as one which enables the TBM to operate at a good penetration rate in the average ground, yet maintain reasonable steady progress in the more difficult ground.

As related to the design of a universal TBM for operation within a practical range of variable ground conditions, the following factors have been shown to be important:

(a) primary support installation point within a few meters of face (less than 1 m preferred)
(b) sufficient usable space behind cutter head for ground control operations
(c) sufficient gripper pad bearing area for machine stability (double braced system preferred)
(d) short full shield to isolate cutter head and/or bucket race from tunnel side wall
(e) partial flexible shield back of cutter head to provide cover for support operations (max 2-3 m long)
(f) easy access to face
(g) open head type debris removal system
(h) balanced TBM weight distribution
(i) minimum machine weight
(j) interchangeable cutters (disc and pick)

It is apparent that several of the itemized factors are incompatible. For example, a conservative bracing system will result in a heavier machine. As a guide to resolve this conflict, the list of factors is ordered with what is believed to be the more important nearest the top. Obviously, this is open to different interpretations depending on specific qualifications.

OPENING SIZE

As previously indicated, standup time is not solely dependent on the tunnel environment, but also on several construction variables such as rate of excavation and size of opening. The influence of rate and size on opening stability has been demonstrated in physical model studies (Myer et al, 1977) and is fully appreciated in practice through years of experience. When a large opening, under construction by traditional methods, encounters a section of difficult ground with short standup time, the construction method is altered to effectively decrease opening size and increase rate of advance. A full face operation may be changed to top heading and bench, dividing the opening into two parts. In extreme conditions, the tunnel section may be constructed through a series of small pilot tunnels or multiple drifts excavated side by side (eg Eisenhower Tunnel; Hopper et al, 1972). Generally, on decreasing opening size the heading is more manageable and excavation rate is increased.

Unfortunately, tunnel construction with a full face TBM does not offer the versatility of traditional methods. Once the diameter of the machine is selected, for all practical purposes, there is no changing. For the construction of large openings, however, there is an alternative to full face headings. Rather than employ one large diameter machine, a smaller diameter pilot tunnel can be excavated and from this opening the heading can be enlarged in one or more stages with a reaming TBM, Figure 3-8. Manufactured by Wirth, the reaming machines have been employed to enlarge a 3 m diameter pilot up to 10.5 m in two stages of enlargement (Sonnenberg Tunnel in Lucerne, Switzerland).

Depending on whether the pilot tunnel is constructed independently or in series with the reamer, the advantages and disadvantages of this system vary. Constructed independently, a pilot bore has the following important advantages:

(a) complete information on geology and support requirements
(b) access for ground treatment prior to enlargement (eg prereinforcement, dewatering, grouting).

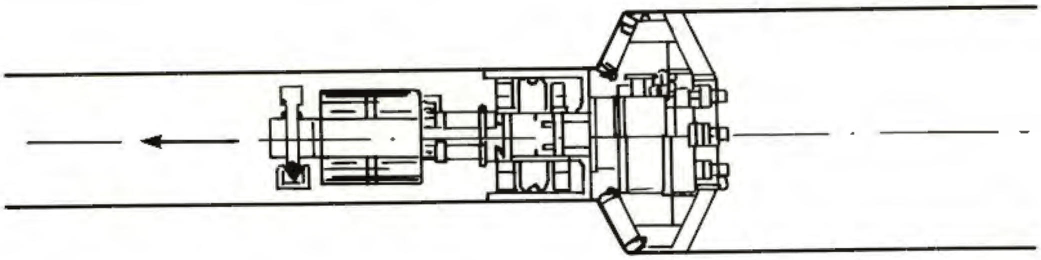


Fig. 3-8 REAMING TBM (After Hambach, 1974)

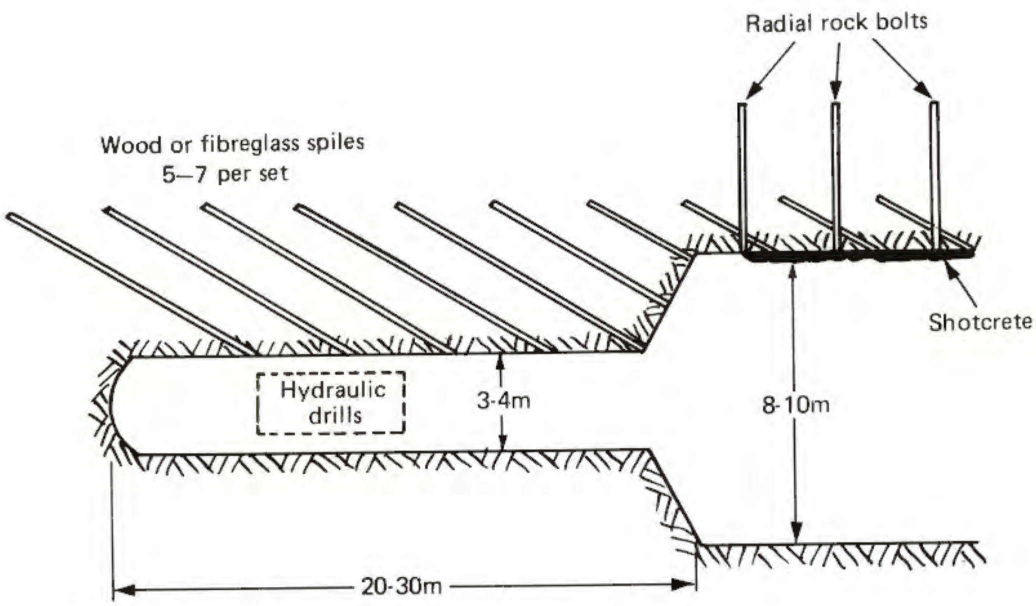


Fig. 3-9 SUPPORT – REINFORCEMENT SYSTEM USED IN CONJUNCTION WITH REAMING TBM OPERATING IN DIFFICULT GROUND

(c) access for additional construction (shafts, special enlargements, etc)

(d) improved tunnel ventilation.

If the pilot is excavated in series with the reamer several advantages are eliminated, nevertheless, it provides valuable information on geology and support requirements, as well as access for ground treatment before enlargement. An example ground support system incorporating spiling reinforcement from within the pilot tunnel is shown in Figure 3-9. Spiles made of wood or fibreglass dowels will not damage the TBM cutters, however, allow for prereinforcement of the ground prior to enlargement. Naturally, extensive ground control operations will delay the progress of the reamer.

In addition, the reaming operation has the following advantages over full face excavation:

(a) increased energy efficiency
(b) relative ease of diameter change
(c) increased penetration rate
(d) good access immediately behind the cutter head for ground support operations

The last point is clearly illustrated by the two stage excavation of a 6.3 m diameter inclined penstock tunnel for the Hornbergstufe at Wehr, Germany (Hambach, 1974). Delays associated with ground control during construction of the 3 m diameter pilot bore amounted to 24 percent of the shift time as compared with 9 percent for the subsequent reaming operation.

Despite all the advantages, there are several major disadvantages to stage excavation by TBM. One of the major factors controling the success of the reaming operation is the condition of the pilot tunnel. If there is excessive overbreak, ground deterioration, or poor alignment, the TBM will have considerable difficulty obtaining forward thrust and maintaining line and grade. Another factor is the increased complexity of the system, especially when employed as a series operation. Mechanical breakdown of one machine will bring the others to a halt. These delays increase as the product of the reliability of each TBM.

SITE INVESTIGATION AND PRELIMINARY CONSIDERATIONS

The importance of a thorough site investigation can not be overstated. A majority of TBM failures are on account of inadequate knowledge of the tunnel environment rather than inappropriate application of a specific machine. As it is rarely possible to investigate every meter of the tunnel alignment, TBM's should always be equipped to deal with adverse conditions. However, the degree of acceptable risks, and hence the economics of the drive, is largely a function of the quality of the site investigation.

TBM vs TRADITIONAL METHODS

In most situations, less than a few percent of the project cost is allocated for site investigation. This general rule was basically derived from experience with traditional methods of tunnel construction which are more versatile than excavation with full face TBM's. A difficult ground condition simply requires a temporary charge in construction method and a delay equal to the time needed to make the change.

Employing a full face TBM is a commitment to a method. It is assured by the physical size of the machine and the amount invested. Although variations are possible, such as forepoling in difficult ground, operating under other than normal or average conditions usually results in a disproportionate reduction in advance rate. In general, conventional or even "universal" type TBM's can not progress at the same rate as traditional methods in difficult ground. This difference is primarily a result of the lack of complete access to the face. When ground conditions get extremely difficult, more effort is often devoted to rescuing the machine than driving the tunnel.

The sensitivity of TBM performance to adverse conditions relative to traditional methods of construction is illustrated by the following example. In a previously described case history, a 5.8 m diameter Wirth TBM was frequently plagued with shear zones composed of a ravelling ground (Appendix IV, Arc Isere Hydroelectric Scheme). The 300 m of problematic ground, representing three percent of the tunnel length, required over 25 percent of the construction effort for a progress rate of 0.85 m per day. A second heading constructed by drill and blast also encountered similar sections of difficult ground. Of the 9 km drive, the 12 percent excavated under difficult conditions required less than 25 percent of the construction effort for a progress rate of 2.7 m per day, at least three times better than that by the TBM. With respect to overall progress, the machine drive was 30 percent faster. If, however, the amount of difficult ground had been over seven percent rather than three, the drill and blast operation would have produced the higher average progress rate.

Although progress rate by itself does not determine the economics of a specific drive, only in rare instances or with special considerations will a TBM operation be more economical than traditional methods if the machine drive is slower. Special considerations are the need to minimize ground disturbance or vibration, and increased safety such as that derived from the excavation of inclined tunnels. Another major factor, depending on the purpose of the tunnel, is minimization of overbreak and reduced volume of concrete if required to line the opening. This is a potentially significant area for large savings; overbreak on drill and blast projects typically results in the use of double the theoretical volume specified to line an opening.

Usually, a higher rate of TBM advance is required to balance the economy between machine and traditional methods of construction. Higher costs for supplies and equipment, cutters and TBM, are balanced against reduced labour by completing the construction faster (less man-days). This relation is further moderated by the length of drive, the longer it is the lower the machine amortization cost per meter.

As an example, haulage road excavation in a German coal mine is being carried out by traditional methods as well as employing a 6.1 m diameter Demag TBM (Appendix IV, Victoria Collieries). Tunnel cost per meter of machine drive was approximately $2,900 for the 5.9 km length, based on an average progress of 13 m per day, employing a work force of 11 to 12 per shift. The drill and blast operation with similar ground conditions, work force, depth and size of opening, was averaging 5 m per day at a cost of around $3,500 per meter. When the machine rate is closer to double the drill and blast rate the costs are about the same. This analysis, however, does not consider the economic advantage of earlier coal production through the machine driven opening. Similarly, early power production in hydroelectric schemes is a major incentive for rapid excavation.

New technology over the last 10 years has not only improved TBM's, making them more competitive, but also added new efficient tools for use in more traditional methods of construction. Road headers or point attack machines have increasingly replaced drill and blast operations in weak or low strength ground, Plate AIV- 17 (page 233). Their use largely eliminates excessive overbreak, damage from blast vibrations, and the discontinuous excavation - mucking cycle, several of the unfavourable aspects of drill and blast, yet maintains the versatility of the construction method (for example case histories in which road headers were employed see Appendix IV, Bochum Underground and Born Railroad Tunnel). Compared to full face TBM operations, road header cutting rates are relatively low, on the order of one meter per hour for a typical section. Their operation is also limited to the lower strength range of rocks, an upper bound unconfined compressive strength of between 70 and 100 MN/m^2 (strongly dependent on machine type and stiffness as well as rock mass abrasivity and structure).

Hydraulic powered drills have substantially improved the working environment as well as doubled penetration rates. There use has contributed to a reduced drill and blast operation cycle time. In addition, smooth wall and pre-splitting techniques for an improved tunnel line and reduced overbreak are more readily employed. As applied to tunnelling in hard and abrasive rock, these techniques and equipment have helped to maintain a competitive edge over machine operations. Particularly efficient drill and blast operations, such as those in Scandinavia, frequently achieve 400 m per month progress rates in the construction of unsupported openings with sections in the 4 m diameter range. The work force usually consists of three to five tunnellers per shift (for example case history see Appendix IV, Paijanne Water Tunnel; for further details on drill and blast operations in Scandinavia see, Underground Design and Construction Practice in Scandinavia).

Only a few of the numerous factors which influence the choice of construction method have been mentioned. Many others are specific to the particular tunnel project. In the final analysis, however, the decision based on economics is derived from the consideration of major factors such as:

(a) boreability of the rock mass
(b) tunnel environment
(c) size of opening and length of drive
(d) tunnel use or purpose
(e) considerations of ground disturbance and/or vibration
(f) project duration and the economics associated with excavation rate.

BOREHOLE INVESTIGATIONS

Downhole methods by which to ascertain the rock mass structure, material properties, and ground water regime are commonly employed for site investigations without regard for the potential method of construction. Methods of investigation include core recovery and orientation techniques, mechanical instruments to test the borehole wall, and geophysical techniques (summary of downhole instruments given by Barr, 1977). As long as cores are obtained and kept in good condition, specific tests for the determination of machine boreability can be performed as required. Often strength and hardness tests such as unconfined compression, point load and/or Schmidt hammer are routinely performed on cores for rock mass characterization. Naturally, these results are equally applicable to stability considerations and machine boreability.

Although the methods available are sufficient for assessing the rock mass character, the statistical significance of the data obtained is always small relative to the total alignment of the tunnel. Determining the average and extreme conditions of a particular tunnel environment depends on the number of relevent samples or spacing of boreholes combined with an insight for the location of potential sections of difficult ground (eg faults, shears, surface weathering, etc).

All tunnel projects benefit from a thorough site investigation. If, however, at the planning stage a TBM operation is considered viable, additional resouces should be allocated for a more detailed study. On account of the increased sensitivity of TBM drives to the tunnel environment as compared with traditional methods, a moderate reduction in risk can markedly improve the economics of a machine operation.

Index tests performed on rock cores and surveys to determine the structural features of the rock mass are commonly employed to assess machine penetration rate and ground stability as separate entities. Subsequently, the two factors are considered together for the evaluation of overall machine performance. It is of interest to note that Morgan et al (1979) found a direct correlation between TBM progress rate and the idices of Schmidt hammer rebound and average joint spacing. This was obtained for the first 1.5 km of the N Wear drive at Kielder, traversing a series of blocky Carboniferous rocks. As a guide this type of correlation may be useful, however, it is susceptible to considerable misuse. In general, average joint spacing and Schmidt hammer rebound are not sufficient indicators of rock mass stability.

PILOT TUNNELS

Use of the pilot tunnel for detailed site investigations has recently found increased application, especially for projects in which a difficult environment is anticipated. Even though the expense is relatively high, on the order of 10 percent of the project civil construction cost, in many cases several times the initial expenditure will be saved as a direct result of the benefits. A list of the major potential benefits or advantages is given below:

(a) complete information on geology and support requirements
(b) access for ground treatment (dewatering, grouting, freezing, prereinforcing)
(c) improved ventilation
(d) access for additional site investigation
(e) access for ancillary works (shafts, enlargements)
(f) reduced specific explosives consumption and ground vibrations if enlarged by traditional methods
(g) possible enlargement by reaming TBM

Along with the increased use of pilot tunnels there has been an increase in the application of TBM's for their excavation. Potentially, a TBM operation will take less time to complete and create less disturbance to the surrounding rock mass than traditional methods of construction. These are only potential realizations, as the TBM could become a victim of the adverse tunnel environment. In either case, a pilot opening which excessively damages the surrounding rock mass can complicate excavation of the main bore to the point where the opening may have resulted in more harm than good.

Several case histories of TBM driven pilot tunnels are presented to illustrate their use. In one unusual application, a 3.1 m diameter Wirth TBM was employed for a short 233 m long drive to investigate the depth and condition of the shallow rock cover and to reduce vibrations upon excavation of the main bore. The opening, a part of the Lyon Metro, passes directly beneath a densely populated region of the city containing old structures. Although progress was slow, 3.2 m per day, a TBM was the only relatively vibration free method by which to construct a pilot tunnel. The high strength of the granite-gneiss precluded the use of a road header, an otherwise suitable alternative. Major delays were largely related to severe space limitations near the face, as it was necessary to provide immediate support through several sections of highly fractured and weathered granite. One unexpected use of the pilot tunnel was for stabilization of a major fallout during excavation of the top heading. Blasting apparently loosened the shallow rock cover leading to ravelling of the ground at and ahead of the face. Before proceeding, the rock was prereinforced with over 70 spiles installed from within the pilot bore (for additional details see Appendix IV, Lyon Metro, Cremaillere Tunnel).

The Born railroad tunnel is another example in which a full face TBM was employed for a short pilot drive, only 810 m long. Minimized ground disturbance was a major factor influencing the selected method of construction. Back up system problems kept the 3.5 m diameter Robbins TBM progress rate down to 11 m per day despite favourable ground conditions. From within the opening boreholes were drilled to delineate the profile of shallow rock cover. Subsequently, the pilot tunnel was used for extracting the large amount of dust produced by the road header upon excavation of the top heading, Plates AIV-17 and-18(page 233). Without improved ventilation health and safety regulations would have made it impractical to employ a road header (further details in Appendix IV, Born Railroad Tunnel).

Two TBM's, a Wirth and Robbins, were use to excavate a 3.6 m diameter, 6.7 km pilot bore for the Pfander road tunnel in Austria.

Aside from the investigation of ground conditions and support requirements, the opening was used for ventilation of the main bore and access for ventilation shaft construction. Both machines made reasonable progress, although, moderate delays resulted from unstable sections of mudstone and marl. Alignment of the pilot tunnel was particularly critical as it was originally planned to allow for the use of a reaming TBM to enlarge the opening. However, enlargement was less expensive by traditional methods (NATM) despite allowances made for reduced ground disturbance, and hence support, on excavation by machine. Total cost of the pilot tunnel was 11 percent of the main bore construction cost (for additional details see Appendix IV, Pfander Road Tunnel).

PROBING AHEAD

Probing ahead of the tunnel face has long been recognized as a desirable method by which information on the tunnel environment could be obtained prior to actual excavation. The advantages of this information, even for as short a distance as several meters ahead of the face, are considerable. Knowledge about a major change in rock mass character, water regime, or presence of dangerous gas would allow the contractor to assess the situation and take appropriate action. Only by removing the element of surprise can tunnel excavation progress in a more orderly continuous manner.

It is always advantageous to prevent loss of the heading, if at all possible, rather than blindly tunnel into a difficult condition followed by days or weeks of remedial work. In many cases, it is too late to apply efficient stabilization measures such as spiling reinforcement once excessive fallouts or loosening of ground has occurred at and ahead of the face. At that point it may be necessary to resort to forepoling. In other cases, probing ahead is largely a question of safety. Excessive water pressure and a flowing ground condition are a hazard to life as well as detrimental to progress. These situations must be dealt with well before they are encountered by the opening.

Of the various techniques employed to probe ahead of the tunnel face, only the downhole methods provide readily usable information (summary of methods given by West, 1975). Geophysical methods employed from within the tunnel opening are subject to differing interpretations, and at present, are primarily areas of research. Downhole methods can be divided into simple core recovery, instrumented probes, and an instrumented drill. Although a core sample provides a relatively complete picture of the rock mass character, it is a time consuming operation. Hydraulic powered high thrust drill rigs, however, are being developed to increase rotary penetration rates (Bergbau-Forschung GmbH, Essen, Germany). Downhole probes, unless a part of the drill string, are also relatively slow operations. Furthermore, they often require an uncased hole of good quality, may be subject to various interpretations and are difficult to employ. One of the more promising techniques for rapid probing ahead is the instrumented drill. By monitoring parameters such as rotary speed, penetration rate, thrust, torque, and water flow rates it is possible to determine not only the strength of the rock mass, but also certain information on discontinuities or rock structure (Brown and Barr, 1978). Currently, this is an area of active research.

A major problem with probing ahead by drilling is the interference caused by the operation regardless of the method of construction employed. This interference is more pronounced in TBM driven tunnels on account of space limitations. Unless the borehole is collared through the centre of the machine cutter head, probe operations at the face can only proceed when the head is at rest. The Alkirk model Lawrence TBM was designed to drill a pilot bore in this manner, however, it is used for forward thrust, not for probing ahead. Driving a probe hole through the centre of the cutter head can lead to specific problems such as: interference with machine steering, delay and machine damage on loss of drill rod, breaking of drill rod if the machine unexpectedly changes line and/or grade, and control of water made by the probe hole. Even though considerable effort would be required to over come some of these problems, the advantages of probing ahead are obvious. Information on the length of a difficult zone, ground bearing capacity, water inflow, and discontinuities would have a marked effect on improved overall progress.

Unless probing ahead is performed as a continuous operation much of the value may be lost. For example, after the 5.8 m diameter Wirth TBM employed at the Arc Isere hydroelectric scheme became trapped on encountering the first of several major shears, a hydraulic drill for probing ahead was installed behind the cutter head. It could only be operated when the machine head was at rest, primarily during cutter servicing. Although it was possible to drill 30 to 50 m ahead in two hours, continuous probing was not practical on account of interference with production. As a result, additional shear zones with ravelling ground were encountered without warning.

CHAPTER 4
CONCLUSIONS AND RECOMMENDATIONS

Machine boreability, that is tool-rock mass interaction, has been considered in terms of simplified numerical models, physical models, and various indices. The predictive capability of all models depend on the pertinent variables selected to describe the process of excavation, or indirectly on the assumed mechanisms of chip formation.

Potentially, the numerical models offer the largest scope in terms of the number of variables which can reasonably be employed to describe the process of rock-tool interaction. Tool geometry, force, penetration, spacing, and mode of excavation are readily included in an analysis. Unfortunately, the present generation of models are basically over simplified. They primarily concentrate on the geometric aspects of tool indentation and chip formation, yet largely disregard material behaviour. Consequently, most models require calibration or are fit to known experimental data. Use is limited to a relatively narrow range about the calibration points for purposes of interpolation rather than extrapolation of known response.

If numerical models are to expand their scope of applicability, the next generation of models must consider material behaviour in greater detail. Frequently, the description is restricted to a simplified Mohr-Coulomb failure criterion employed in a limit equilibrium type analysis. Aspects related to material compaction, dilatation, work hardening and softening, size effect, brittleness, stiffness, and fracture propagation should be given the same consideration as that of tool geometry. It is fully realized that a complete description of material behaviour is not practical or even possible. If, however, the mechanisms involved are understood, the process is often adequately described by an idealized model.

Despite the relatively minor role of numerical models as predictive tools, they should not be ignored. Along with a fundamental consideration of rock behaviour, they provide a theoretical foundation from which the process of mechanical excavation may be evaluated. On this basis, the design of laboratory tests can be improved and appropriate indices selected. In general, the models illustrate the influence of basic rock strength properties, tool geometry, and groove interaction. The pressure bulb model also introduces the concept of tool penetration different from groove depth.

Generally, attempts to predict prototype behaviour from scaled laboratory tests or physical models have not been successful, considering the effort required and the accuracy of the results obtained. This is largely on account of material size effect, although, test procedure and test system stiffness may be contributing factors. Improved boreability predictions based on scale models depend on a better understanding of the excavation process and improved numerical models. Simple geometric scaling of the results is not appropriate.

Full scale laboratory tests can provide a wealth of information for either machine design or prediction of penetration. Test procedure, surface conditioning, system stiffness, and interpretation of the data are major factors to consider. In particular, the influence of system stiffness on optimum cutter spacing to penetration has not been adequately assessed.

On account of the complexity and expense of full scale tests, the results are of value when applied in a general context rather than performed for a specific tunnel project. Successful application of these results to a specific project depends on the accuracy by which the rock tested in the laboratory is equated to that in the field. Again, this requires an understanding of the mechanisms involved in the process of excavation and a selection of pertinent variables by which to relate the rock in the field with that tested in the laboratory. Usually, this is accomplished with the aid of several strength and/or hardness index tests.

Various material index tests or indices are widely used by machine manufacturers and contractors for boreability predictions. Most tests are simple to employ, however, as the indices do not consider the machine characteristics (stiffness; number, type, spacing and arrangement of tools; available thrust and power) and only certain aspects of the operational conditions (rock mass characteristics, thrust employed, condition of tools) there is considerable margin for error. The only valid use of these indices is as calibrated by and applied to a specific machine operating under specific conditions.

The indices employed should be readily obtainable from simple tests and provide reasonable correlation with known machine performance. As a first approximation, compressive strength or a relatable index test is a good indicator of penetration rate. For increased accuracy it is necessary to consider specific indices related to compressive or shear strength (eg Schmidt hammer, Shore scleroscope) and to tensile strength (eg point load, Brazilian) as well as the influence of discontinuities. Both material strength and an indice related to abrasion hardness or mineralogical composition are important in the estimation of tool wear. Those additional factors which influence the accuracy of penetration rate predictions are equally important to that of tool wear.

Discontinuities and material anisotropy can have a major effect on penetration rate; however, with respect to joints the average spacing must approach tool spacing before this effect becomes significant. The reduced energy of excavation and increased penetration can be roughly estimated by application of Rittinger's relation for comminution. In practice, the adverse effect of joints on rock mass stability, and hence progress, usually negates any improvement in machine cutting rate.

As for boreability predictions, it is difficult to make an accurate estimation of machine utilization without specific information on the particular TBM to be employed. The influence of major geological factors (rock abrasivity, ground bearing capacity, rock mass stability and deformation, and water regime) on machine performance is strongly related to the design of the TBM. Cutter servicing depends on rock mass character and presence of water as well as debris removal system efficiency and tool parameters. Machine stability is related to bracing system design, bearing capacity of the ground, and rock mass stability. Both ground bearing capacity and stability are significantly influenced by the presence of ground water. Undoubtedly, one of the major factors affecting overall performance is the stability of the tunnel heading or standup time and the particular ground control measures employed.

In difficult ground conditions, the ability to place support and/or reinforcement immediately behind the cutter head, as near to the face as possible, has a major effect on machine utilization. Generally, the use of a shield is not an adequate substitute for this advantage. Excessive shielding can lead to increased rock mass deterioration with delayed installation of support, reduced access for remedial ground treatment, steering problems, and possible immobilization from squeezing ground or excessive loosening pressure. Minor or partial flexible shields are suitable for protection of the cutter head and as cover for ground control operations.

Under specific conditions, standup time can be markedly improved through stabilization measures employed at and ahead of the tunnel face (eg spiling reinforcement, grouting). Techniques to increase the productivity of these measures, compatible with TBM operations are required. Stage excavation or enlargement by reaming TBM should also be considered as opposed to a large diameter full face excavation.

Due to the increased sensitivity of TBM operation to the tunnel environment as compared to traditional methods of construction, a greater emphasis should be placed on a thorough site investigation. Sufficient application of downhole methods from which to ascertain the average and extreme tunnelling conditions are needed. The possible advantages of a pilot tunnel or probing ahead of the tunnel face by horizontal drilling should not be overlooked. Although continuous probing ahead operations pose several major problems, the potential advantages in terms of improved progress are considerable.

REFERENCES

Anttikoski, U V and A E Saraste, 1977, "The effect of horizontal stress in the rock on the construction of the Salmisaari oil caverns, "First Int Symp on Storage in Excavated Rock Caverns, Rockstore 77, Vol 2, pp 57-62, Stockholm, Sept 5-8.

Attewell, P B and J B Boden, 1971, "Development of stability ratios for tunnels driven in clay", Tunnels and Tunnelling, May, pp 195-198.

Barendsen, P and R G Cadden, 1976, "Machine-bored small-size tunnels in rock, with some case studies", Tunnelling 1976, The Institution of Mining and Metallurgy, London, March.

Barr, M V, 1977, "Downhole instrumentation - a review for tunnelling ground investigations", Technical Note No 90, Construction Industry Research and Information Ass, London.

Barton, N, R Lien and J Lunde, 1974, "Engineering classification of rock masses for the design of tunnel supports", Rock Mechanics, Vol 6, No 4, pp 189-236.

Benjumea, R and D L Sikarskie, 1969, "A note on the penetration of a rigid wedge into a nonisotropic brittle material", Int J Rock Mech Min Sci, Vol 6, pp 343-352.

Bennion, J D, 1976, "The Kaimai tunnel New Zealand - a case history," Tunnelling 1976, Inst Min and Met, London, pp 371-380.

Berry, N S M and J G W Brown, 1977, "Performance of full facers on Kielder Tunnels", Tunnels and Tunnelling, July, pp 35-39.

Bieniawski, ZT, 1974, "Geomechanics classification of rock masses and its applications in tunnelling", Tunnelling in Rock, Bieniawski ed, S African Inst of Civil Engr, Pretoria, pp 89-103.

Bieniawski, Z T, 1975, "The point load test in geotechnical practice", Engr Geology, Vol 9, No 1,pp 1-11.

Blindheim, O T, 1976, "Preinvestigations, resistance to blasting and drillability predictions in hard rock tunnelling", First US - Swedish Underground Workshop, Stockholm, Dec 5-10.

Brekke, T L and T R Howard, 1973, "Functional classification of gouge materials from seams and faults in relation to stability problems in underground openings, US Bureau of Mines (ARPA) Contract No H0220022.

Broch, E and J A Franklin, 1972, "The point load strength test", Int J Rock Mech Min Sci, Vol 9, No 6, pp 669-697.

Brown, J G W and K Milow, 1979, "Driving with full facers - the Kielder Tunnels - N E England", Tunnelling 1979, The Institution of Mining and Metallurgy, London.

Brown, E T and M V Barr, 1978, "Instrumented drilling as an aid to site investigations", 3rd Int Cong of Int Ass of Engr Geologist, Madrid, pp 21 - 28.

Brown, E T and H R Phillips, 1977, "Recording drilling performance for tunnelling site investigation", Construction Industry Research and Information Ass, Technical Note 81, London.

Carter, F G and D A C Mills, 1976, "Engineering geological investigations of the Kielder Tunnels", Q Journal of Engr Geology, Vol 9, pp 125-141.

Carter, P G and M Sneddon, 1977, "Comparison of Schmidt hammer, point load and unconfined compression tests in Carboniferous strata", Proc of Conf on Rock Engr, CORE-UK, April 4-7.

Cheatham, J B, 1958, "An analytical study of rock penetration by a single bit tooth", Proc of Eighth Annual Drilling and Blasting Symp, Univ of Minnesota, Minnesota.

Cheatham, J B, 1964, "Indentation analysis for rock having a parabolic yield envelope", Int J Rock Mech Min Sci, Vol 1, pp 431-440.

Cheatham, J B and P F Gnirk, 1967, "The mechanics of rock failure associated with drilling at depth", Proc of Eighth Symp on Rock Mech, Univ of Minnesota, Sept 1966, pp 410-439.

Cheatham, J B and R W Pittman, 1966, "Plastic limit analysis applied to a simplified drilling problem", Proc of First Cong Int Soc Rock Mech, Lisbon, Vol 2, pp 93-97.

Cook, N G W and N C Joughin, 1970, "Rock fragmentation by mechanical, chemical and thermal methods", Paper 1-C.6, Vi Int Min Cong, Madrid.

Dalziel, J A and E Davies, 1964, "Initiation of cracks in coal specimens by blunted wedges", The Engineer, Vol 217, Jan 31, pp 217-220, London.

Das, B, 1974, "Vicker's hardness concept in the light of Vicker's impression", Int J Rock Mech Min Sci and Geomech Abstr, Vol 11, pp 85-59.

Dixon, R L and E P Worden, 1971, "Roller cutters for raise and tunnel boring", Min Cong J, April, pp 41-48.

Durelli, A J and V Parks, 1962, "Relationship of size and stress gradient to brittle failure stress", Proc 4th US Nat Cong on Applied Mech, Vol 2, pp 931-938.

Dutta, P K, 1972, "A theory of percussive drill bit penetration", Int J Rock Mech Min Sci, Vol 9, pp 543-567.

Edmond, J M and M S Paterson, 1972, "Volume changes during the deformation of rock at high pressures", Int J Rock Mech Min Sci, Vol 9, pp 161-182.

Embery, BD, 1976, "Tunnel boring in the Mount Lyell Mine", 2nd Australian Tunnelling Conf, Melbourne, Australia, Aug, pp 193-202.

Erkelenz, W H, 1968, "Drilling through granite-gneiss", World Mining, April.

Etheridge, D C, 1973, "Rock mole finishes bore in a flourish of records", Construction Methods and Equipment, March, pp 106-110.

Evans, I, 1962, Mining Research, Vol 2, p 761, Pergamon Press, London.

Evans, I and S A F Murrell, 1962, "Wedge penetration into coal", Colliery Engineer, Vol 39, No 455, p 11, England.

Fairhurst, C and W D Lacabanne, 1957, "Some principles and developments in hard rock drilling techniques", Proc of 6th Drilling and Blasting Symp, 1956, University of Minnesota, p 15.

Ferrand, J , C Cottard and A Schwenzfeier, 1977, "Utilisation des explosifs pour l'execution de la galerie Cremaillere du Metro de Lyon", Journees d'etudes Paris, les travaux souterrains en site urbain, Mai 9-11, pp 100-104.

Friedman, M, J Handin and G Alani, 1972, "Fracture surface energy of rocks", Int J Rock Mech Min Sci, Vol 9, pp 757-766.

Frocht, M M, 1948, Photoelasticity, John Wiley and Sons, Inc, New York, Vol 2, Ch 2.

Garshol, K, 1978, personal communication.

Gaye, F, 1972, "Efficient excavation", Tunnels and Tunnelling; Jan, pp 39-48; March, pp 135-143; May, pp 249-259.

Geary, D W, 1972, "Past tunnel excavation history", 1st N American Rapid Excavation and Tunnelling Conf, Proceedings, June, pp 1137-1147.

Glucklich, J and L J Cohen, 1967, "Size as a factor in the brittle-ductile transition and the strength of some materials", Int J Fracture Mech, Vol 3,No 4, Dec,pp 278-289.

Glucklich, J and L J Cohen, 1968, "Strain energy and size effects in a brittle material", Materials Research and Standards, Vol 8, No 10, pp 17-22.

Gnirk, P F, 1962, "An experimental investigation of the indexing phenomenon for static single-tooth penetration in Indiana limestone at atmospheric conditions", School of Mines and Metalurgy, Univ Minn, Aug.

Graham, P C, 1976, "Some problems associated with the use of a tunnel machine in deep level gold mines", Tunnelling 1976, Inst Min and Met, March, pp 397-403.

Gronhaug, A, 1976, "Road tunnels in Norway", Tunnels and Tunnelling, Nov.

Habberstad, J L, R R Thompson, R C Bates and G B Wallace, 1974, "Fiberglass reinforced polymeric rock bolt", 1974 Rapid Excavation and Tunnelling Conf, Proceedings, San Francisco, June, pp 977-993.

Hambach, P, 1974, "Excavation of inclined shaft by reaming in two stages", 1974 Rapid Excavation and Tunnelling Conf, Proceedings, San Francisco, June, pp 1651-1663.

Handewith, H J, 1970, "Machine boring and the mechanical properties of hard rock", Proc, Symp on Raise and Tunnel Boring, Australian Geomech Soc.

Hansen, A M, 1975, "Fullprofilboring av tunnel Frantzebraten - Majorstua", Fjellsprengningskonferansen, Oslo.

Hartman, H L, 1959, "Basic studies of percussion drilling", Mining Engineering, Jan, pp 68-75.

Haswell, C K, 1973, "Tunnel under the Severn and Wye estuaries", Proc, Institute of Civil Engr, England, pp 451-486.

Hayward, D, 1977, "Four month extension granted to TTT at Kielder", New Civil Engr, July 28, pp 14-15, England.

Henneke, J, 1978, personal communication

Henneke, J, 1978, "Entwicklung und stand des vollmechanischen gesteinsstreckenvortriebs im steinkohlenbergbau der bundesrepublik", Bergbau-Forschung GmbH, Essen, Germany.

Hignett, H J and T R Howard, 1974, "A pilot scale machine for tunnel boring research", Transport and Road Research Laboratory, Report 632, Crowthorne, England.

Hignett, H J, R A Snowdon and Temporal J, 1977, "Tunnelling trials in chalk: rock cutting experiments", Transport and Road Research Laboratory, Report 796, Crowthorne, England.

Hill, R, E H Lee and S J Tupper, 1947, "The theory of wedge indentation of ductile materials", Proc Royal Soc London, Vol A188, p 273.

Hodgson, K and N G W Cook, 1970, "The effects of size and stress gradients on the strength of rock", Proc 2nd Cong Int Soc Rock Mech, Belgrade, Vol 2, pp 31-34.

Hopper, R C, T A Lang and A A Mathews, 1972, "Construction of Straight Creek Tunnel, Colorado", 1st N American Rapid Excavation and Tunnelling Conf, Proceedings, June, Chicago, pp 501-538.

Hucka, V and B Das, 1974, "Brittleness determination of rocks by different methods", Int J Rock Mech Min Sci and Geomech Abstr, Vol 11, pp 389-392.

Hustrulid, W A, 1970, "Development of a tunnel boreability index", USBM Contract 14-06-D-6849, Nov.

Hustrulid, W A, 1972, "A comparison of laboratory cutting results and actual tunnel boring performance", 1st N American Rapid Excavation and Tunnelling Conf, Proceedings, June, pp 1299-1323.

Hustrulid, W A and C Fairhurst, 1971, "A theoretical and experimental study of the percussive drilling of rock", Int J Rock Mech Min Sci, Vol 8, pp 335-356.

Innaurato, N, R Mancini and S Pelizza, 1976, "Consideration of rock boring machines: analysis of Italian operations", Tunnelling 1976, The Institution of Mining and Metallurgy, March, London, pp 219-223.

Jacob, E, 1976, "The bentonite shield: technology and initial application in Germany, "Tunnelling 1976, The Institution of Mining and Metallurgy, March, London, pp 201-207.

Jaeger, J C and N G W Cook, 1969, <u>Fundamentals of Rock Mechanics</u>, Methuen and Co, London.

John M, 1976, "Die geotechnischen messungen im Arlbergtunnel und deren auswirkungen auf das baugeschehen", Rock Mechanics, Suppl 5, pp 157-177.

John M, 1977, "Adjustment of programs of measurements based on the results of current evaluation", and "Design of the Arlberg expressway tunnel and the Pfander tunnel", Ingenieurgemeinshaft Lasser-Feizlmayr, Innsbruck, Austria.

Kolek, J, 1958, "An appreciation of the Schmidt rebound hammer", Magazine of Concrete Research, Vol 10, No 28, March, pp 27-36

Korbin, G E and T L Brekke, 1976, "Model study of tunnel prereinforcement", ASCE J Geotechnical Engr Div, Vol 102, No GT9, Sept.

Korbin, G E and T L Brekke, 1978, "Field study of tunnel prereinforcement", ASCE J Geotechnical Engr Div, Vol 104, No GT8, Aug.

Krause, H, 1976, "Geological experiences in the use of tunnelling machines in Baden-Wurttemberg", (in German), Rock Mechanics, Suppl 5, pp 49-60.

Lauffer, H, 1958, "Gebirgsklassifizierung fur den stollenbau", Geologie und Bauwesen, Vol 24, No 1, pp 46-50.

Leeney, J, 1976, "The European tour", Tunnels and Tunnelling, May, pp 64-65.

Leeney, J, 1976, "OETM versus NATM", Tunnels and Tunnelling, Nov, pp 21.

Lundberg B, 1974, "Penetration of rock by conical indenters", Int J Rock Mech Min Sci and Geomech Abstr, Vol 11, pp 209-214.

Maurer, W C and J S Rinehart, 1960, "Impact crater formation in rock", J Applied Physics, Vol 31, No 7, July, pp 1247-1252.

McFeat Smith, I and P J Tarkoy, 1979, "The use of geotechnical data in the assessment of the potential of hard rock tunnelling machines", Tunnelling 1979, The Institution of Mining and Metallurgy, March, London.

Mellor, M, 1972, "Note - Normalization of specific energy values", Int J Rock Mech Min Sci, Vol 9, pp 661-663.

Miller, T W and J B Cheatham, 1972, "Analysis of the indentation of a compacting material by a perfectly rough wedge", Int J Rock Mech Min Sci, Vol 9, pp 475-492.

Milow, K, 1978, "Rock strength and tool costs", Tunnels and Tunnelling, May, p 17.

Morgan, J, D Barratt and J Hudson, 1979, "Tunnel boring machine performance and ground properties on the initial 1.5 km of the N Wear drive, Kielder aqueduct, UK", Transport and Road Research Laboratory, Crowthorne, England (inprint).

Morrell, R J, W E Bruce and D A Larson, 1970, "Tunnel boring technology - Disc cutter experiments in sedimentary and metamorphic rocks", USBM Report of Investigations 7410.

Morrell, R J and D A Larson, 1974, "Tunnel broing technology - Disc cutter experiments in metamorphic and igneous rocks", USBM Report of Investigations 7961.

Morris, R I, 1969, "Rock drillability related to a roller cone bit", 4th Conf Drilling and Rock Mech, Univ of Texas, Austin, pp 79-86.

Muller, L and A Hereth, 1976, "NATM at Bochum", Tunnels and Tunnelling, Sept, p 76.

Myer, L R, T L Brekke and G E Korbin, 1977, "Stand-up time of tunnels in squeezing ground", Report No DOT-TST-77-59, US Dept of Transportation.

Nevill, H F C and J G D Crone, 1962, Australian Inst Min and Met, Feb p 249.

Neyland, A J and R F Murrell, 1970, "Tunnel boring in fractured and weathered sedimentary rock", Proc Australian Geomechanics Soc Symp on Raise and Tunnel Boring.

Nishimatsu, Y, 1972, "The mechanics of rock cutting, "Int J Rock Mech Min Sci, Vol 9, pp 261-270.

Norman, N E, 1972, "Mechanical boring of a mixed face tunnel", 1st N American Rapid Excavation and Tunnelling Conf, Proceedings, Chicago, June, pp 1125-1136.

Norman, N E and J Dye, 1978, "Economic factors of mechanical raise boring", Underground Space, Vol 2, pp 249-260.

Obert, L, S L Windes and W I Duvall, 1946, "Standardized tests for determining the physical properties of mine rock", USBM Report of Investigations 3891.

O'Reilly, M P, 1976, "NATM: settlement and economics", Tunnels and Tunnelling, Jan, p 21.

Osburn, H J, 1969, "Wear of rock-cutting tools", Powder Metallurgy, Vol 12, No 24, pp 471-502.

Ozdemir, L, R Miller and F Wang, 1977, "Mechanical tunnel boring prediction and machine design", NSF Report No RA-770199.

Paone, J and W E Bruce, 1963, "Drillability studies: diamond drilling", USBM Report of Investigations 6324.

Pariseau, W G and C Fairhurst, 1967, "The force - penetration characteristic for wedge penetration into rock", Int J Rock Mech Min Sci, Vol 4, pp 165-180.

Paul, B and Sikarskie, 1965, "A preliminary theory of static penetration by a rigid wedge into a brittle material", AIME Transactions, Vol 232, Dec, pp 372-383.

Pirrie, N D, 1973, "The use of rock tunnel machines", Proc Instn Civil Engrs, Vol 54, London, paper 7577.

Prandtl, L, 1921, "Eindringungsfestigkeit und Festigkeit von Schneiden, Zeit Fur Math und Mech, Vol 1, pp 15-20.

Protodyakonov, MM, 1963, "Mechanical properties and drillability of rocks", Proc 5th Symp Rock Mech, Minnesota, May 1962, pp 103-118.

Rad, P F, 1975, "Importance of groove spacing in tunnel boring machine operations", ASCE J Geotechnical Engr Div, Vol 101, No GT9, pp 949-962.

Rad P F and F J McGarry, 1971, "Thermally assisted cutting of granite", Proc 12th Sym on Rock Mech, Univ of Missouri, Nov 1970, pp 721-757.

Rae, D, 1964, SMRE Research Report 223.

Reichmuth, D R, 1963, "Correlation of force-displacement data with physical properties of rock for percussive drilling systems", Proc 5th Symp on Rock Mech, pp 33-59.

Rittinger, P von, 1867, Lehrbuch der Aufbereitungskunde, Berlin.

Robbins, J S, 1956, "New concepts in drilling and boring rock", 6th Drilling and Blasting Symp, Univ of Minnesota, Oct, pp 92-97.

Robbins, J S, 1976, "Mechanized tunnelling - progress and expectations", Tunnelling 1976, Inst Min Met, March, London.

Robberts, A, 1977, Geotechnology Pergamon Press.

Ross, N and W Hustrulid, 1972, "Development of a tunnel boreability index", USBR, Report REC-ERC-72-7, Feb, Denver.

Roxborough, F F, 1969, "Rock cutting research for the design and operation of tunnelling machines", Tunnels and Tunnelling, Vol 1, No 3, pp 125-126.

Roxborough FF and H R Phillips, 1975, "Rock excavation by disc cutter", Int J Rock Mech Min Sci and Geomech Abstr, Vol 12, pp 361-366.

Sasaki, et al, 1972, "Studies on the cutting resistance of rock", trans from Japanese by Kojima and Hustrulid, USBM Contract H0210043, ARPA monitor, order no 1579.

Schimazek, J and H Knatz, 1970, "Der einfluss des gesteinsaufbaus auf die schnittgeschwindigkeit und den meisselverschleiss von streckenvortriebsmaschinen", Gluckauf, Vol 106, No 6, pp 274-278.

Selmer-Olsen, R and O T Blindheim, 1970, "On the drillability of rock by percussive drilling", Proc 2nd Cong, Int Soc Rock Mech, Belgrade, Vol 3, paper 5-8.

Selmer-Olsen, R and E Broch, 1977, "General design procedure for underground openings Norway", First Int Symp on Storage in Excavated Rock Caverns, Rockstore 77, Vol 2, pp 11-18, Stockholm, Sept 5-8.

Shepherd, R, 1950, "Physical properties and drillability of mine rocks", Colliery Engineering; Vol 27, No 322, pp 468-470; Vol 28, No 323, pp 28-34; Vol 28, No 324, pp 51-56; Vol 28, No 325, pp 121-126.

Siever, H, 1950, "Die bestimmung des bohrwiderstandes von gesteinen", Gluckauf, Vol 86, No 37-38, pp 776-784.

Sikarskie, D L and N J Altiero, 1973, "The formation of chips in the penetration of elastic-brittle materials (rock)", J Applied Mech, Vol 40, No 3, Sept.

Simon, R, 1963, "Energy balance in rock drilling", Proc 1st Conf Drilling and Rock Mech, Univer, Texas, Jan.

Simon R, 1967, "Rock fragmentation by concentrated loading", Eighth Symp on Rock Mech, Univer Minnesota, Sept 15-17, 1966, pp 440-454.

Snowdon, R A, 1979, "Performance of full face tunnelling machine in Coal Measures rocks", M Sc thesis, Univ Newcastle-upon-Tyne, England.

Sperry, P E and R E Heuer, 1972, "Excavation and support of Navajo Tunnel No 3", 1st N American Rapid Excavation and Tunnelling Conf, Proceedings, June, Chicago, pp 539-571.

Stimpson B, 1965, "Index tests for rock", M Sc thesis, Imperial College, London.

Sugden, D B, 1975, "Tunnel boring machines and systems: a survey", J Inst Engr, Australia, Nov - Dec, pp 23-31.

Swain, M V and B R Lawn, 1976, "Indentation fracture in brittle rocks and glass", Int J Rock Mech Min Sci and Geomech Abstr, Vol 13, pp 311-319.

Tarkoy, P J, 1973, "A study of rock properties and tunnel boring machine advance rates in two mica schist formation", 15th Symp Rock Mech, South Dakota, Sept.

Tarkoy, P J, 1974, "Predicting TBM penetration rates in selected rock types", Proc 9th Canadian Rock Mech Symp, L'Ecole Polytechnique de Montreal, Dec, pp 263-274.

Tarkoy, P J, 1975, "Rock hardness index properties and geotechnical parameters for predicting tunnel boring machine performance", Ph D thesis, Univ Illinois at Urbana-Champaign, NSF Research Grant GI-36468.

Teale, R, 1965, "The concept of specific energy in rock drilling", Int J Rock Mech Min Sci, Vol 2, pp 57-73.

Terzaghi, K, 1946, in: Rock Tunnelling with Steel Supports, Proctor and White ed, Commercial Shearing and Stamping Co, Youngstown, Ohio.

Timoshenko, S P and J N Goodier, 1951, Theory of Elasticity, 2nd Edn, McGraw Hill, New York.

Walsh, T and A R Biggart, 1976, "The bentonite tunnelling machine at Warrington", Tunnelling 1976, Inst Min Met, pp 209-218, London.

Wang, F D, R Miller and L Ozdemir, 1974, "Improving hard rock tunnelling through comparison of laboratory and field tunnel boring studies", 2nd N American Rapid Excavation and Tunnelling Conf, Vol 2, pp 1741-1757.

Wang, J K and T F Lehnhoff, 1976, "Bit penetration into rock - a finite element study", Int J Rock Mech Min Sci and Geomech Abstr, Vol 13, pp 11-16.

Wanner, H, 1975, "Bibliography on tunnel boring machines", Bull Int Ass Engr Geology, No 12, Krefeld.

Wanner, H, 1975, "On the influence of geological conditions at the application of tunnel boring machines", Bull Int Ass Engr Geology, No 12, Krefeld.

Weber, E, 1972, "Practical experience in rock behaviour in tunnelling, Proc 8th Canadian Rock Mech Symp, Toronto, pp 187-201.

West, G (ed), 1975, "Probing ahead for tunnels: a review of present methods and recommendations for research", Suppl Report 171UC, Transport and Road Research Laboratory, Crowthorne, England.

West, G and D A Barratt, 1976, "Rock cutting profiles of the sandstone tunnel face at chainage 366 m in the N Wear drive, Kielder Water Scheme", Transport and Road Research Laboratory, Working Paper

No 7, Sept, Crowthorne, England.

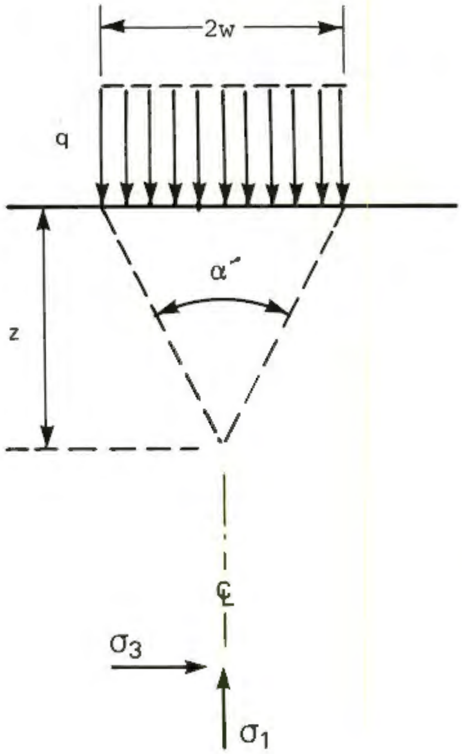
West, G and D A Barratt, 1977, "Rock cutting profiles of the tunnel face at chainage 868 m in the N Wear drive, Kielder Water Scheme", Transport and Road Research Laboratory, Working Paper No 18, Oct, Crowthorne, England.

Whitbread, J E, 1960, Colliery Engineering, Vol 37, No 25.

APPENDIX I

INITIAL FAILURE BENEATH A UNIFORMLY LOADED AREA

A. UNIFORMLY LOADED STRIP -



q = uniform pressure

w = strip half width

z = depth beneath surface

σ_1, σ_3 = principal stresses

C_o = unconfined compressive strength

ϕ = internal friction angle

μ = Poisson's ratio

stress state,

$$\sigma_1 = q(\alpha' + \sin\alpha')/\pi$$

$$\sigma_3 = q(\alpha' - \sin\alpha')/\pi$$

Mohr-Coulomb failure criterion

$$\sigma_1 = C_o + Q\,\sigma_3 \;,\; Q = \tan^2(45 + \phi/2)$$

$$q(\alpha' + \sin\alpha')/\pi = C_o + qQ(\alpha' - \sin\alpha')/\pi$$

$$q = \pi\, C_o\, [\alpha' + \sin\alpha' - Q\alpha' + Q\sin\alpha']^{-1}$$

minimize q,

$$\frac{dq}{d\alpha'} = 1 + \cos\alpha' - Q + Q\cos\alpha' = 0$$

$$\alpha' = \cos^{-1}\left[\frac{Q - 1}{1 + Q}\right] \;,\; z = w/\tan(\alpha'/2)$$

B. UNIFORMLY LOADED CIRCULAR AREA OF RADIUS a -

stress state along centre line,

$$\sigma_1 = q\ (1 - b^3)$$

$$\sigma_3 = \frac{q}{2}\left[(1 + 2\mu) + b^3 - 2(1 + \mu)b\right]$$

Mohr-Coulomb failure criterion,

$$\sigma_1 = C_o + Q\,\sigma_3$$

$$q\ (1 - b^3) = C_o + \frac{qQ}{2}\left[(1 + 2\mu) + b^3 - 2(1 + \mu)b\right]$$

$$q = 2\,C_o\left[2(1 - b^3) - Q(1 + 2\mu) - Qb^3 + 2Q(1 + \mu)b\right]^{-1}$$

from $\frac{dq}{db} = 0,$

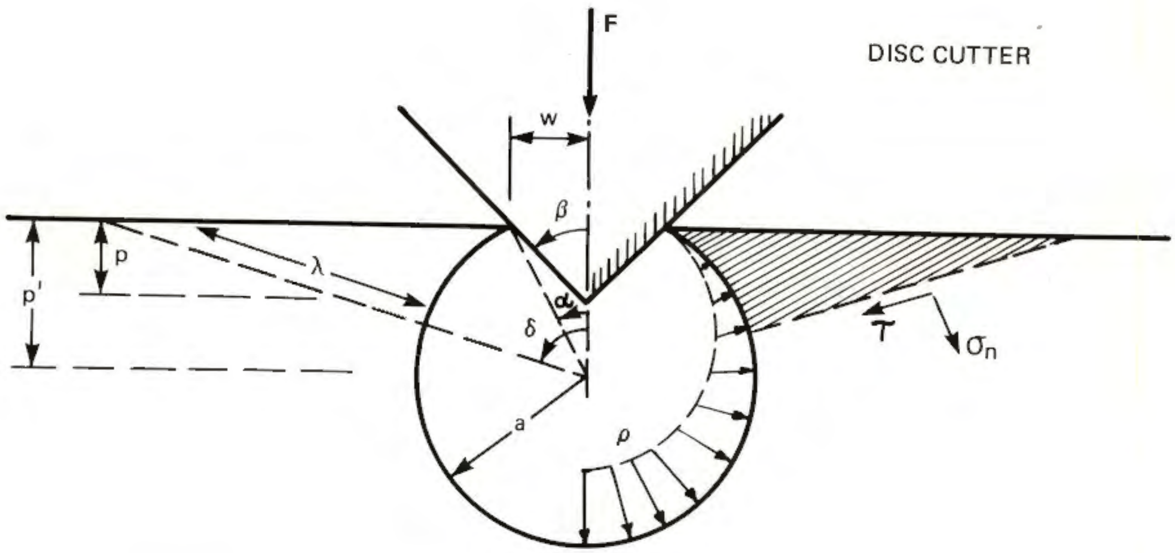
$$b^2 = \frac{2Q(1 + \mu)}{6 + 3Q} \quad \text{for a minimum } q$$

where $z = \frac{ab}{\sqrt{1 - b^2}}$

APPENDIX II

PRESSURE BULB MODEL

A. SURFACE CHIP FORMATION -



assumptions:

(a) linear failure surface of length λ

(b) uniform stress distribution on failure surface

(c) circular or spherical shaped pressure bulb

(d) linearly increasing pressure, ρ, within bulb; zero at surface, maximum ρ_m at base

(e) centre of pressure located at initial failure position (Appendix I)

(f) linear failure criterion

geometry of pressure bulb:

$\alpha = \frac{\alpha'}{2}$ from Appendix I

$\sin\alpha = \frac{w}{a}$, $\tan\beta = \frac{w}{p}$

pressure distribution within bulb , $\alpha \leqslant \theta \leqslant \pi$:

$$\rho(\alpha) = 0 \quad , \quad \rho(\pi) = \rho_m$$

$$\rho(\theta) = \rho_m \frac{(\theta - \alpha)}{(\pi - \alpha)}$$

equilibrium of vertical forces within pressure bulb:

$$P_v^* = \int_{\theta=\alpha}^{\pi} a \rho \cos\theta \, d\theta = \int_{\theta=\alpha}^{\pi} a \rho_m \frac{(\theta - \alpha)}{(\pi - \alpha)} \cos\theta \, d\theta$$

$$P_v^* = \frac{a \rho_m}{(\pi - \alpha)} \Big[\cos\theta + (\theta - \alpha)\sin\theta\Big]_{\theta=\alpha}^{\pi} = \frac{-a \rho_m}{(\pi - \alpha)} (1 + \cos\alpha)$$

$$F = 2 P_v^*$$

$$\rho_m = \frac{-F(\pi - \alpha)}{2a(1 + \cos\alpha)}$$

vertical force on surface chip, $\alpha \leqslant \theta \leqslant \delta$:

$$P_v = \int_{\theta=\alpha}^{\delta} \rho a \cos\theta \, d\theta = \int_{\theta=\alpha}^{\delta} a \rho_m \frac{(\theta - \alpha)}{(\pi - \alpha)} \cos\theta \, d\theta$$

$$P_v = \frac{a \rho_m}{(\pi - \alpha)} \Big[\cos\delta + (\delta - \alpha)\sin\delta - \cos\alpha\Big]$$

$$P_v = \frac{-F}{2(1 + \cos\alpha)} \Big[\cos\delta + (\delta - \alpha)\sin\delta - \cos\alpha\Big]$$

horizontal force on surface chip, $\alpha \leqslant \theta \leqslant \delta$:

$$P_h = \int_{\theta=\alpha}^{\delta} \rho \ a \sin\theta \ d\theta = \int_{\theta=\alpha}^{\delta} a \ \rho_m \frac{(\theta - \alpha)}{(\pi - \alpha)} \sin\theta \ d\theta$$

$$P_h = \frac{a \ \rho_m}{(\pi - \alpha)} \left[\sin\delta + (\alpha - \delta) \cos\delta - \sin\alpha \right]$$

$$P_h = \frac{- F}{2 \ (1 + \cos\alpha)} \left[\sin\delta + (\alpha - \delta) \cos\delta - \sin\alpha \right]$$

resolving P_v and P_h to shear, P_s, and normal, P_n, forces along chip failure surface:

$$P_s = P_v \cos\delta + P_h \sin\delta$$

$$P_n = P_v \sin\delta - P_h \cos\delta$$

failure criterion:

$$\tau = \frac{S_o}{T_o} \sigma + S_o \quad \text{where}$$

S_o = shear strength

T_o = tensile strength

substitution of normal and shear stresses:

$$\tau = -\frac{P_s}{\lambda} \quad , \qquad \sigma = -\frac{P_n}{\lambda}$$

$$P_s = -\lambda S_o + \frac{S_o}{T_o} P_n$$

$$P_v \cos\delta + P_h \sin\delta = -\lambda S_o + \frac{S_o}{T_o} (P_v \sin\delta - P_h \cos\delta)$$

$$\frac{F}{2 \ (1 + \cos\alpha)} \left[1 - \cos(\delta - \alpha) - \frac{S_o}{T_o} (\delta - \alpha) + \frac{S_o}{T_o} \sin(\delta - \alpha)\right]$$

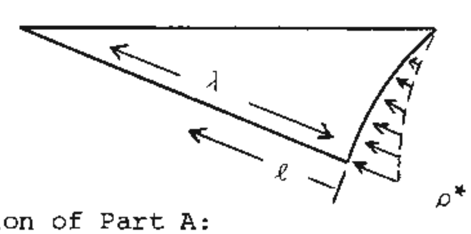
$$= S_o \lambda$$

$$\text{where} \quad \lambda = a \left[\frac{\cos\alpha}{\cos\delta} - 1\right]$$

minimized solution to F when $\frac{dF}{d\delta} = 0$ found numerically and presented graphically in Figure AII - 1

B. SURFACE CHIP FORMATION ASSUMING A NONLINEAR SHEAR STRESS DISTRIBUTION ALONG THE FAILURE SURFACE

$$\tau = -\rho^* \left(\frac{\lambda - \ell}{\lambda}\right)^m$$



on substitution into the formulation of Part A:

$$F' = \frac{F}{m + 1}$$

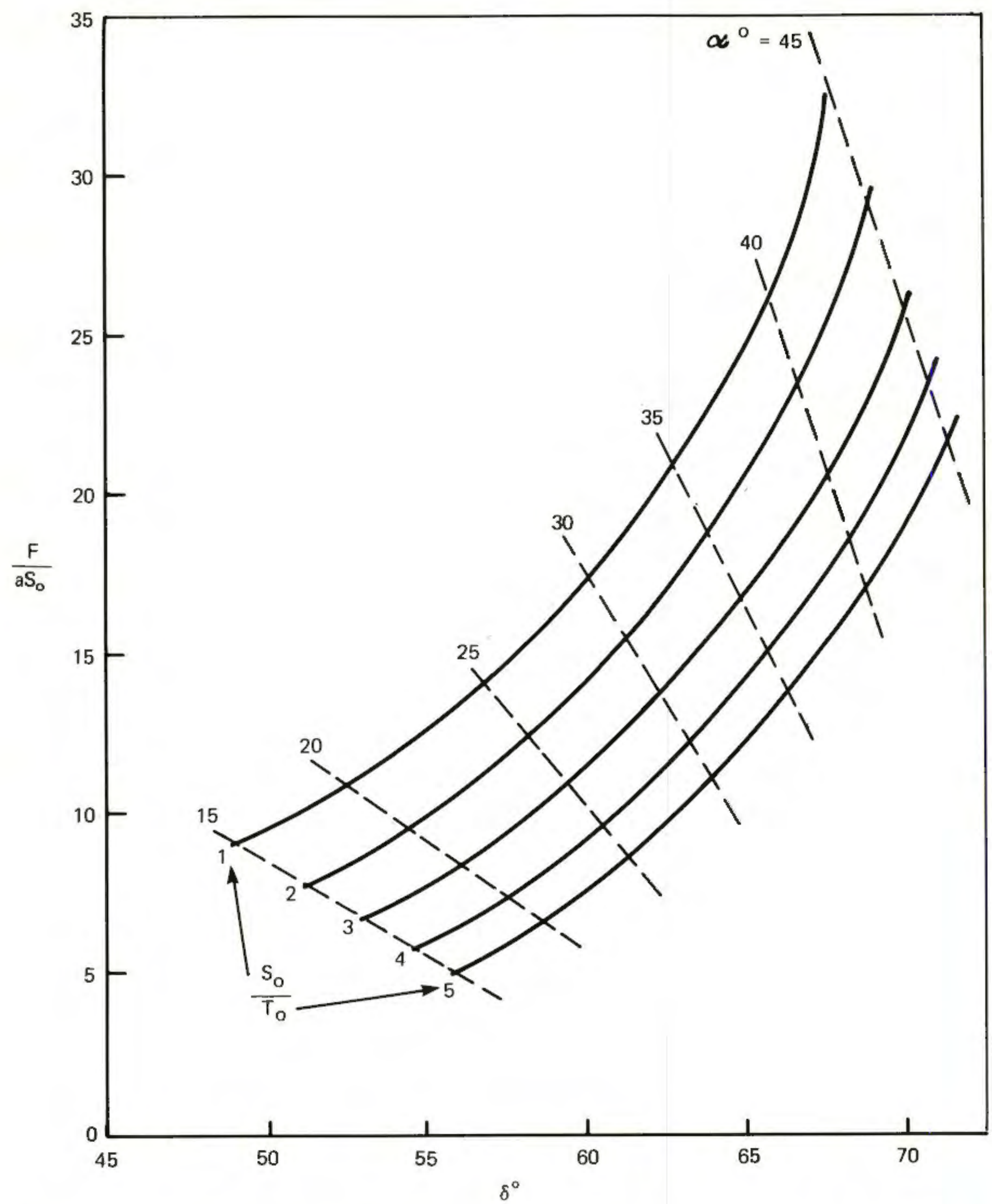
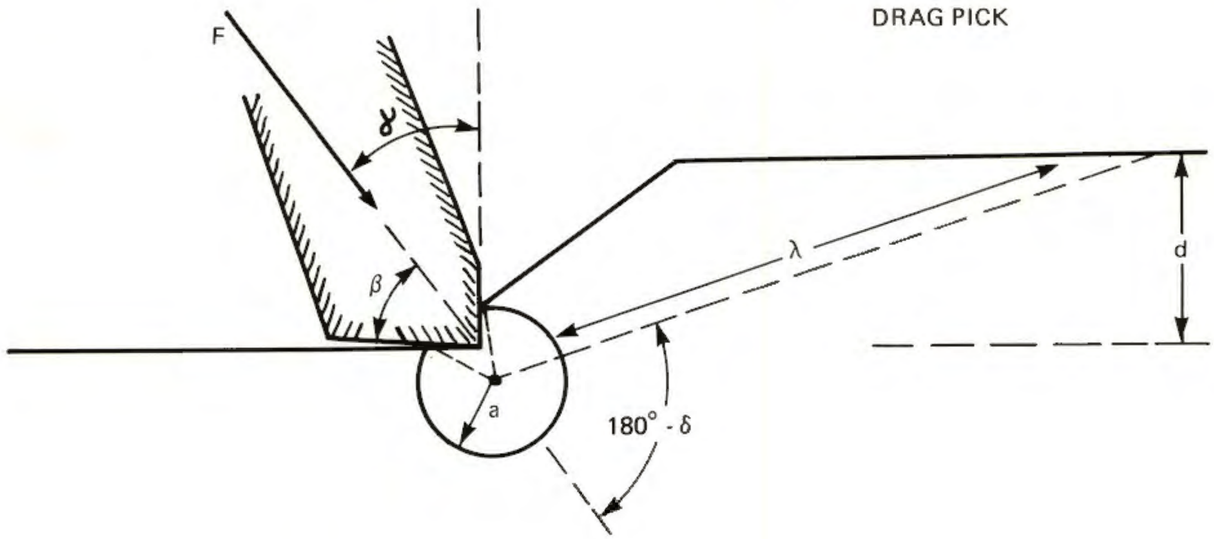


Fig. AII–1 MINIMIZED FORCE FOR SURFACE CHIP FORMATION AT BREAK OUT ANGLE δ

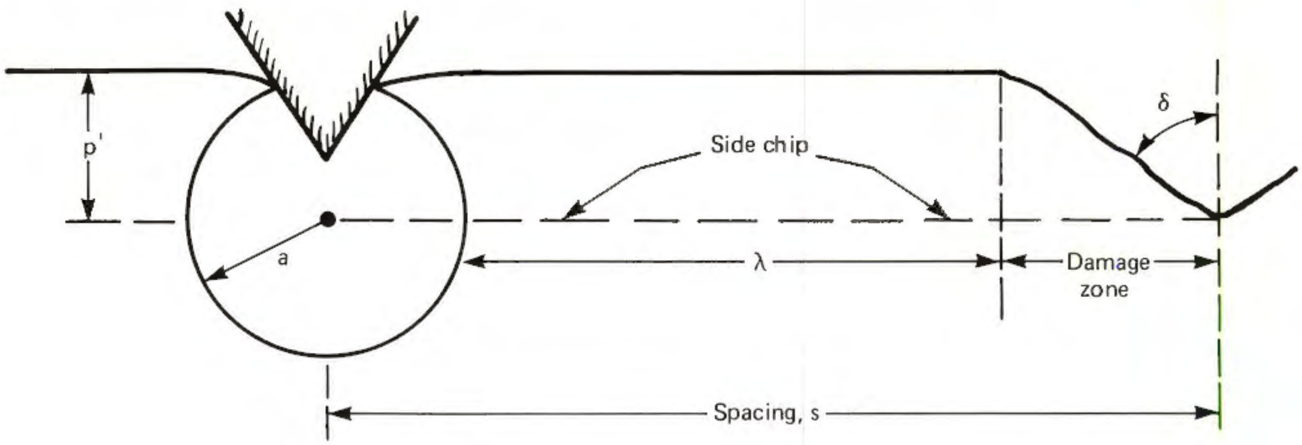
C. SURFACE CHIP FORMATION BY PICK



$$\lambda = \frac{d + a\left[\cos\alpha - \frac{\sin\alpha}{\tan\beta}\right]\cos\gamma}{\cos(\delta - \gamma)} - a$$

same formulation as Part A employing the failure surface length λ given above

D. SIDE CHIP FORMATION

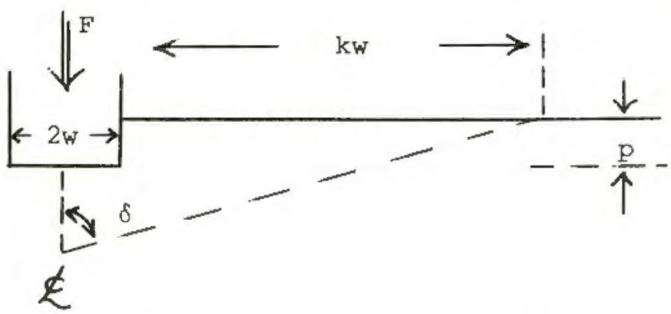


$$\lambda = s - p'\tan\delta - a\sin\delta$$

same formulation as Part A employing the failure surface length λ given above

APPENDIX III

SPECIFIC ENERGY - MATERIAL YIELD RELATION



work by flat indentor of unit length,

$$W = Fp \qquad \text{with the}$$

size effect $\quad F \propto \sqrt{w} \quad ; \quad W \propto p\sqrt{w}$

(a) specific energy, S E, for a material yield, V, proportional to the penetration squared

$$V = p^2 \tan\delta$$

$$SE = \frac{W}{V} \propto \frac{\sqrt{w}}{p \tan\delta}$$

(b) specific energy for yield proportional to indentor width squared

$$V = \frac{(kw)^2}{\tan\delta}$$

$$SE = \frac{W}{V} \propto pw^{-3/2} \tan\delta$$

APPENDIX IV

CASE HISTORIES FROM EUROPE

The following case histories were largely derived from site visits made between Fall 1977 and Winter 1979 while participating in the US DOT/TRRL tunnelling research and development exchange program. Not all of the histories deal specifically with TBM operations, but provide a general background of underground construction. As compared with a full face machine operation, alternative methods and techniques such as NATM, modern drill and blast, and excavation by roadheader are often extremely competitive, and therefore, require full consideration before selection of the construction method.

Table of Contents

Kielder Water Scheme

Antwerp Pre Metro

Arc Isere Hydroelectric Scheme

Arlberg Road Tunnel

Bochum Underground

Born Railroad Tunnel

Frejus Road Tunnel

Lyon Metro, Cremaillere Tunnel

Oslo Sewer Tunnels

Paijanne Water Tunnel

Pfander Road Tunnel

Sellrain-Silz Hydroelectric Project

Underground Design and Construction Practice in Scandinavia

Victoria Collieries

KIELDER WATER SCHEME

The Kielder scheme is designed to transfer water from the River Tyne to the Rivers Wear and Tees in north-east England. Of the 40 km transfer distance, the 28 km tunnel section between the Rivers Derwent and Tees was the major tunnel contract, Figure AIV-1. For the most part, the 3.5 m diameter openings traverse a gently dipping sedimentary series of limestone, mudstone, siltstone and sandstone of Carboniferous age (Carter and Mills, 1976). Rock compressive strengths largely range between low and medium values, 30 to 150 MN/m^2, with the exception of dolerite intrusives, 350 MN/m^2.

Aside from the sections of dolerite (400 m total) and several fault zones on the order of tens of meters wide, the tunnel excavation was well suited to full face tunnel boring machines. This was illustrated by the 4 to 2 distribution of proposed machine drives to conventional drill and blast tenders received (Berry and Brown, 1977). Prior to this project the use of full face machines for rock excavation in Britain was rare as compared to continental Europe. Consequently it was of little surprise when an Anglo-German consortium was awarded the 19 M pound ($38M) contract in May 1975.

Originally it was intended to drive the tunnels from four headings with two Demag machines specially purchased for the project. However, slow progress during the early stages of work forced the contractor to rent a third machine manufactured by Robbins. The introduction of the Robbins machine resulted in considerable interest as its design both in terms of cutter head and back-up systems differed in many aspects from that of the two Demag machines. Although the operation of each machine was influenced by the specific geology encountered along their alignment, many areas were similar allowing for a general comparison of cutting or penetration and progress rate.

Tunnel Boring Machines

Basic machine layouts, Figures AIV-2 and -3, are used to illustrate

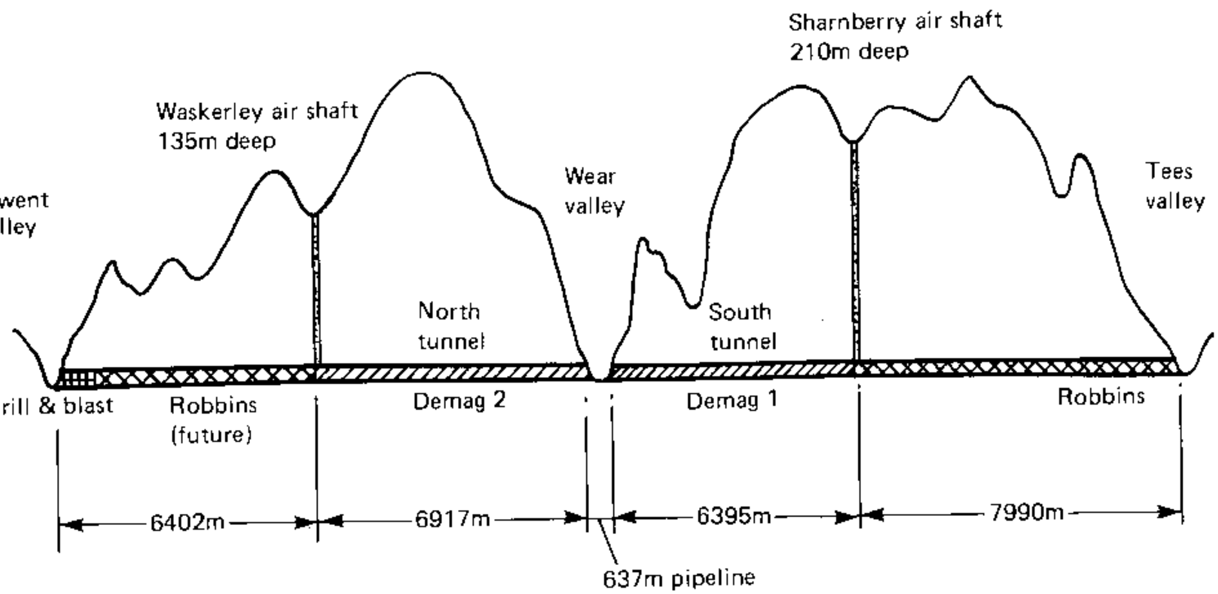


Fig. AIV–1 SECTION SHOWING DRIVAGE (BROWN and MILOW, 1979)

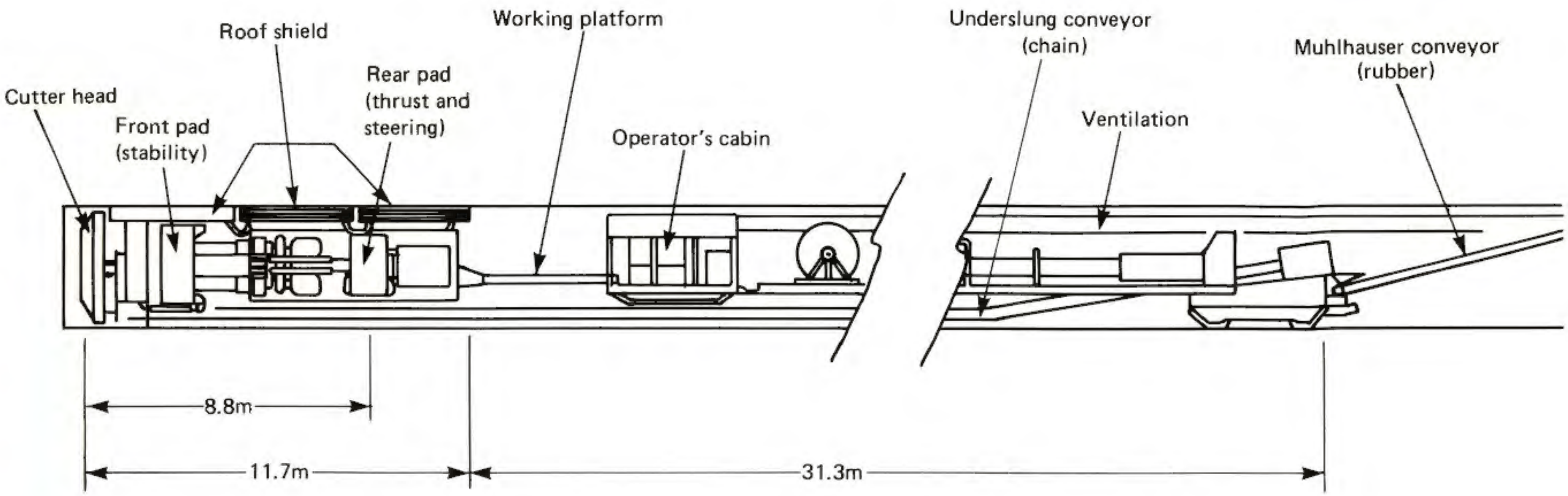


Fig. AIV–2 TUNNELLING MACHINE DEMAG T.V.M. 34–38

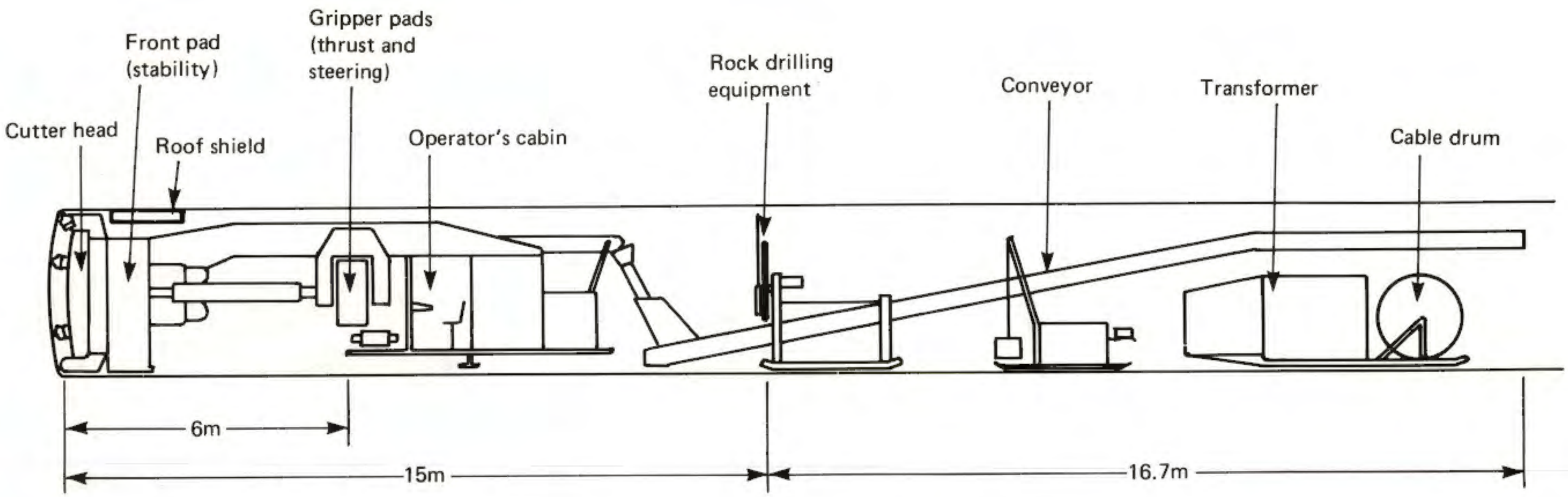


Fig. AIV–3 TUNNELLING MACHINE ROBBINS MODEL 123–133

several important design similarities and differences. Both machines steer and obtain forward thrust reaction through the system of rear gripper pads. Their position with respect to the distance from the face, 6 m and 8.8 m for the Robbins and Demag respectively, influenced performance on passing from weak ground into more competent rock as will be shown. Front pads or braces were employed to stabilise the head during excavation.

The most significant factor controlling performance in difficult ground was the ability to place support as close to the tunnel face as possible. Severely limited working space along the first 12 m from the front of the Demag cutter head made it impractical to carry out support operations at any point other than the "working platform", designed for that purpose, PlateAIV-1. This required the opening to remain essentially self-supporting over the time necessary to advance 3.5 diameters if tunnelling operations were to proceed unimpeded. Small fallouts in the crown were contained by the roof shield. Unfortunately, no protection against sidewall fallout was provided; this being the weakest geologically related link in the system. A small fallout could easily wedge between the machine and tunnel wall while larger overbreaks resulted in loss of gripper pad bearing. It should be emphasised that this loss of bearing was rarely related to a bearing capacity failure of the ground, but to a lack of sidewall material to react against.

Although the rock drill and support platform were located 15 m behind the front of the Robbins machine, the capability to install support within one tunnel diameter of the face (2.5-3 m) was realized by the open space provided between the cutter head and gripper pads, Plate AIV-2. This proved to be a major advantage in difficult ground, however, somewhat of a hindrance in more competent regions with only minor stability problems. All potential fallouts near the face, minor or major, basically required the same preventative measures, steel sets with mesh or sheet lagging, Plate AIV-3. The effort required to place this support was considerably greater than that needed to work

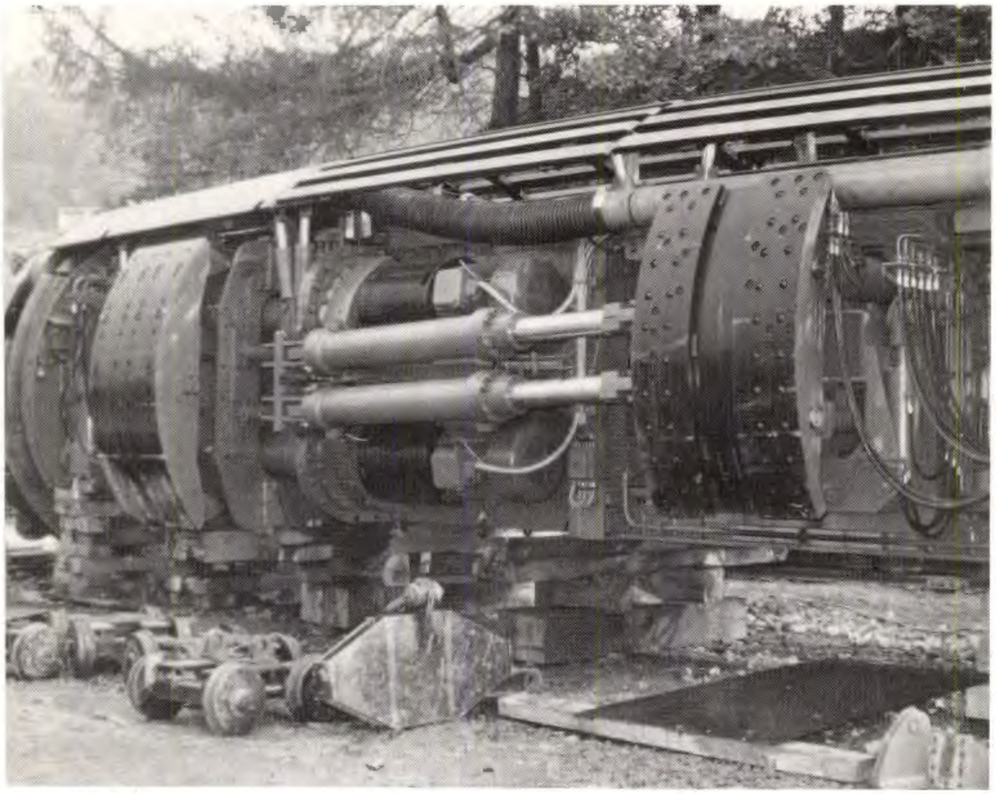


PLATE AIV-1. Demag TBM between cutter head and rear gripper pad; note partial shield and limited working space

PLATE AIV-2. Robbins TBM; note open working space behind cutter head for support installation

PLATE AIV-3. Steel sets and wire mesh used to support blocky mudstone in crown, Tees station 4050 to 4060 from the support platform. If left unsupported, unexpected fallouts could and occasionally did plague workers leading to the installation of an extended canopy over the operators cabin. Rock bolts and mesh, used extensively in blocky mudstone on those drives employing Demags, were impractical to use at locations other than the drill platform due to the difficulties in drilling holes above the spring line. Several examples in the form of machine performance comparisons can be used to demonstrate the consequences of design.

Summarised in Tables AIV-2 and -3 under major categories of tunnelling operations are the average times required for one meter of advance through blocky mudstone (average joint spacing 0.5 to 1 m). With arches installed behind the cutter head of the Robbins machine, spaced at 2 m, delays associated with supporting totalled slightly over one hour per meter (Table AIV-2). Clearing loosened rock from the Demag shield and installing rock bolts and mesh only required 0.30 hours (Table AIV-3). The additional work needed to clear the tunnel sidewalls (0.21 hours per meter), a direct result of delayed support, should be added for a total delay of approximately one half hour per meter. Consequently, for the relatively minor stability problems caused by blocky mudstone, supporting delays for the Robbins drive were double that for the Demag. This apparent advantage, however, was largely cancelled by the 1.5 to 3 cubic meter overbreak per meter of tunnel created on removing the loosened rock prior to bolting.

In difficult ground such as fault zones composed of shattered mudstone, the advantages of immediate support were clearly demonstrated. Steel sets placed behind the Robbins cutter head resulted in delays of 1 to 3 hours (Table AIV-9) per meter depending on spacing and ground conditions. Accountable delays on a Demag drive in similar ground were 4.7 hours per meter, Table AIV-10 (Supporting plus Bearing). In terms of overall average progress excluding "other" non-machine or ground related delays (ie, scheduled maintenance, waiting for muck disposal, labour breaks and travel, etc) the Robbins rate was roughly 60 per cent greater than the Demag. The reduced progress was not the only penalty because of the additional overbreak created by delayed

TABLE AIV - 2

Performance between Tees station 966 and 979, Robbins TBM operating in blocky mudstone

Operation (hr/m)	Station (m) 966-979
Driving	.33
Supporting	1.08
Bearing	.00
Mucking	.00
Other	.48
Progress rate (hr/m)	1.89 (1.41)
(m/hr)	0.53 (0.71)
Cutting rate (m/hr)	3.05
Type support near face	arches spaced at 2m

() excludes "Other"

TABLE AIV - 3

Performance between North Wear station 1480 and 1505, Demag TBM operating in blocky mudstone

Operation (hr/m)	Station (m) 1480-1505
Driving	.57
Supporting	.30
Bearing (clearing sidewalls)	.21
Mucking (blocked chutes)	.15
Other	.53
Progress rate (hr/m)	1.76 (1.23)
(m/hr)	0.57 (0.81)
Cutting rate (m/hr)	1.75
Type support	wire mesh w 8 bolts/m

TABLE AIV - 4

Performance between Tees station 649 and 765, Robbins TBM operating in blocky ground with inflowing water

Operation (hr/m)	Station (m) 649-765
Driving	.51
Supporting	.79
Bearing	.04
Mucking	1.19
Other	1.55
Progress rate (hr/m)	4.08 (2.53)
(m/hr)	0.25 (0.40)
Cutting rate (m/hr)	1.96
Type support near face	16 arches

support, exceeding 5 cubic meters per meter of tunnel.

Not all of the problems of slow progress were associated with inefficient support. The design of the buckets on the Demag cutter head for debris removal and that of the muck chute to receive the debris, were not compatible. Basically, the buckets could receive larger fragments or blocks of rock than the opening could dispose. In blocky ground up to one half hour of delay per meter was attributed to clearing a blocked chute. On occasion a build-up of slurry would also cause blockage. Problems persisted further along the spoil removal system with severe wear of the under slung chain conveyor, particularly in sandstone. On one machine it was changed to a rubber conveyor, however, in slurry conditions the system was not as effective as the chain and reversion was made (Berry and Brown, 1977). The under slung system proved considerably more difficult to clean and maintain than the overhead system employed on the Robbins.

For both types of machine mucking and conveyor systems, the removal of slurry produced by the mixing of inflowing or ponding water and debris was most difficult and time consuming. The slurry tended to pond at the face of the cutter head where it was mixed and reground by the rotating head and discs. This reduced penetration and increased cutter wear. For example, on encountering inflowing water in blocky mudstone-sandstone the cutting rate dropped by over 40 per cent and tool changes increased 2 to 3 times the normal frequency. That material which was deposited on the conveyor system often spilled off into the invert, particularly the fines. Delays related to mucking and invert cleaning amounted to 1.19 hours per meter not considering the reduced cutting rate and tool life, Table AIV-4. Placing a water pump near the cutter head helped to reduce, but did not eliminate the problem.

Machine Performance in Difficult Ground

Overall machine performance in difficult ground depends not only on average or bulk rock mass strength, but on large lateral contrasts in strength (mixed face) and on transition in rock strength along the axis of the tunnel. The influence of water when superimposed on the above factors causes further deterioration of the ground, often enhancing contrasts and operational problems. Seven case histories of performance are presented to illustrate these points in the form of plots of cumulative working time, cutting rate, and geologic profile as a function of the tunnel face station. All of the cases except one were from the Robbins drive.

One of the most difficult regions encountered by the Robbins machine was a fault zone composed of massive intact blocks of sandstone and completely shattered mudstone, with little or no alteration, Figure AIV-4. The mudstone of dent quality was several orders of magnitude weaker than the sandstone resulting in a mixed face condition. To further complicate matters, small amounts of water inflow sufficient to weaken the mudstone were present and the ground below spring line most susceptible to bearing failure was largely mudstone. Once the machine penetrated the fault zone the cutting rate dropped as the thrust necessary to efficiently excavate the mixed face was lost. Slipping gripper pads could barely provide reaction for 20 per cent of the normal thrust employed in full face sandstone. Packing the tunnel sidewalls with timber and concrete often helped, but further reduced progress. Slow progress tended to increase the amount of ground deterioration and remoulding caused by the machine plus allowed time for the absorption of water. Finally, after progressing at 0.12 m/hr for 9 m the machine head started to sink and roll. With the heavy Robbins cutter head in a diving attitude and little control through slipping gripper pads, it was not possible to manoeuver the machine. After an additional 3 m drive, the heading was 0.3 m low. It subsequently required 300 hrs to place the machine on grade with the aid of a steel rail and concrete ramp constructed in front of the head. Explosives were used to correct the line and the sidewalls were concreted to increase bearing. On restarting, it was not until the gripper pads encountered more competent sandstone that cutting rates increased to over 2 m/hr.

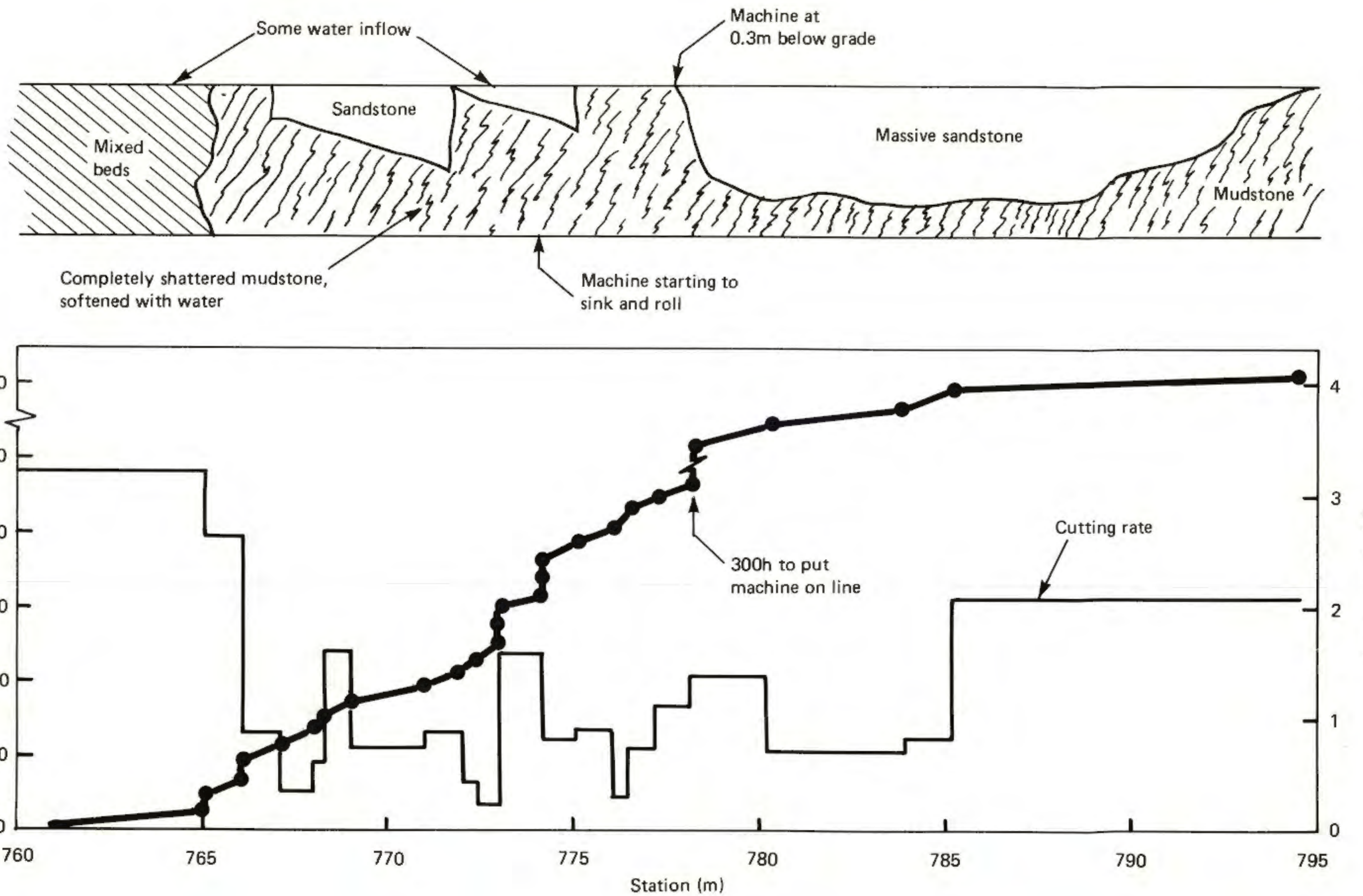


Fig. AIV–4 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN TEES STATION 760 AND 795, ROBBINS TBM

The case presented in Figure AIV-5 was basically an example for the appreciation of standup time. As shown, the heading was extended 10 m into the fault zone with few problems, however, left largely unsupported. Cutting rate was nearly 3 m/hr and progress was also good, 0.8 m/hr (first column, Table AIV-5). By the change of shift, the sidewalls started to deteriorate and ravel making it difficult to obtain gripper pad reaction. The next crew worked on this problem, but placed no support. Finally, after 18 hours attention turned to ground support as deterioration and fallouts spread. Over the next four working shifts efforts were mainly concentrated on supporting the region originally left unsupported, leaving little time for further advance (support delays of 6.11 hr/m over 4 m, Table AIV-5). The influence of delayed support on sidewall stability was also accountable for some proportion of the large effort required to obtain pad bearing (delay of 4.05 hr/m over 4 m). When the average progress rate for the 14 m wide zone, 0.17 m/hr, was compared to performance in other fault zones where support was placed concurrent with driving, it was at least 50 per cent less. Plates AIV-4 and-5 show the blocky sandstone and shattered mudstone in the tunnel sidewall just beyond the fault margin, station 940 to 945.

In situations where the fault zone gouge was reasonably compact and of uniform strength (not a mixed face) with negligible water inflow, passing through the difficult ground presented few problems. Very often cutting rates actually increased as the machine went from massive rock into the gouge, Figure AIV-6. Progress rates were slowed due to the requirement of supporting at or near the face, always good insurance. Once through the fault zone the major difficulty was in many cases passing back into the massive rock as illustrated in Figures AIV-6 and 7. In both examples, the faults were distinct zones (14 and 30 m wide) with little influence on the adjoining wall rock; transition from weak to strong was sudden, Plates AIV-6 and -7. On encountering the massive rock required thrust for penetration increased causing bearing failure beneath the gripper pads and a drop in cutting rate. Difficulties were acute whenever the rate fell below 1 m/hr. The simplest and most common solution was to pack the sidewalls, progress being dependent on

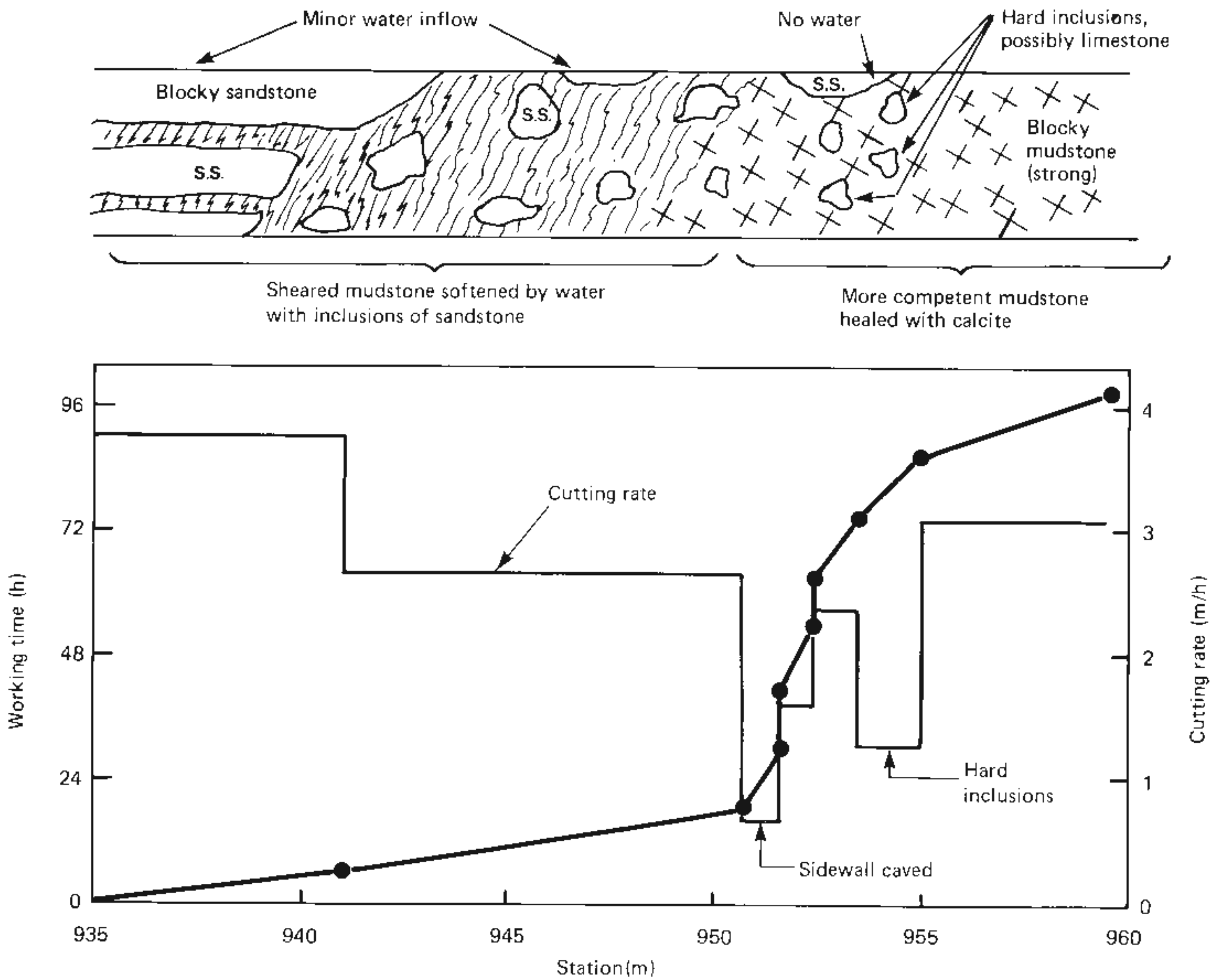


Fig. AIV–5 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN TEES STATION 935 AND 960, ROBBINS TBM

PLATE AIV-4. Intact blocky sandstone and shattered mudstone, Tees station 940 to 945

PLATE AIV-5. Shattered mudstone, Tees station 940 to 945

TABLE AIV-5

Performance between Tees station 935 and 960, Robbins TBM

Operation (hr/m)	Station (m)	
	941-951	951-955
Driving	.38	.81
Supporting	.35	6.11
Bearing	.00	4.05
Other	.51	4.03
Progress rate (hr/m)	1.24 (0.73)	15.0 (11.0)
(m/hr)	0.81 (1.37)	0.067 (.091)
Type support near face	3 bolts at station 939	8 arches at 942 to 950

TABLE AIV-6

Performance between Tees station 4578 and 4610, Robbins TBM

Operation (hr/m)	Station (m)	
	4578-4595	4595-4604
Driving	.37	.91
Supporting	.41	.00
Bearing	.00	3.68
Other	.85	2.86
Progress rate (hr/m)	1.63 (.78)	7.45 (4.59)
(m/hr)	0.61 (1.28)	0.13 (.22)
Type support near face	5 arches	packing sidewalls

() excludes "Other"

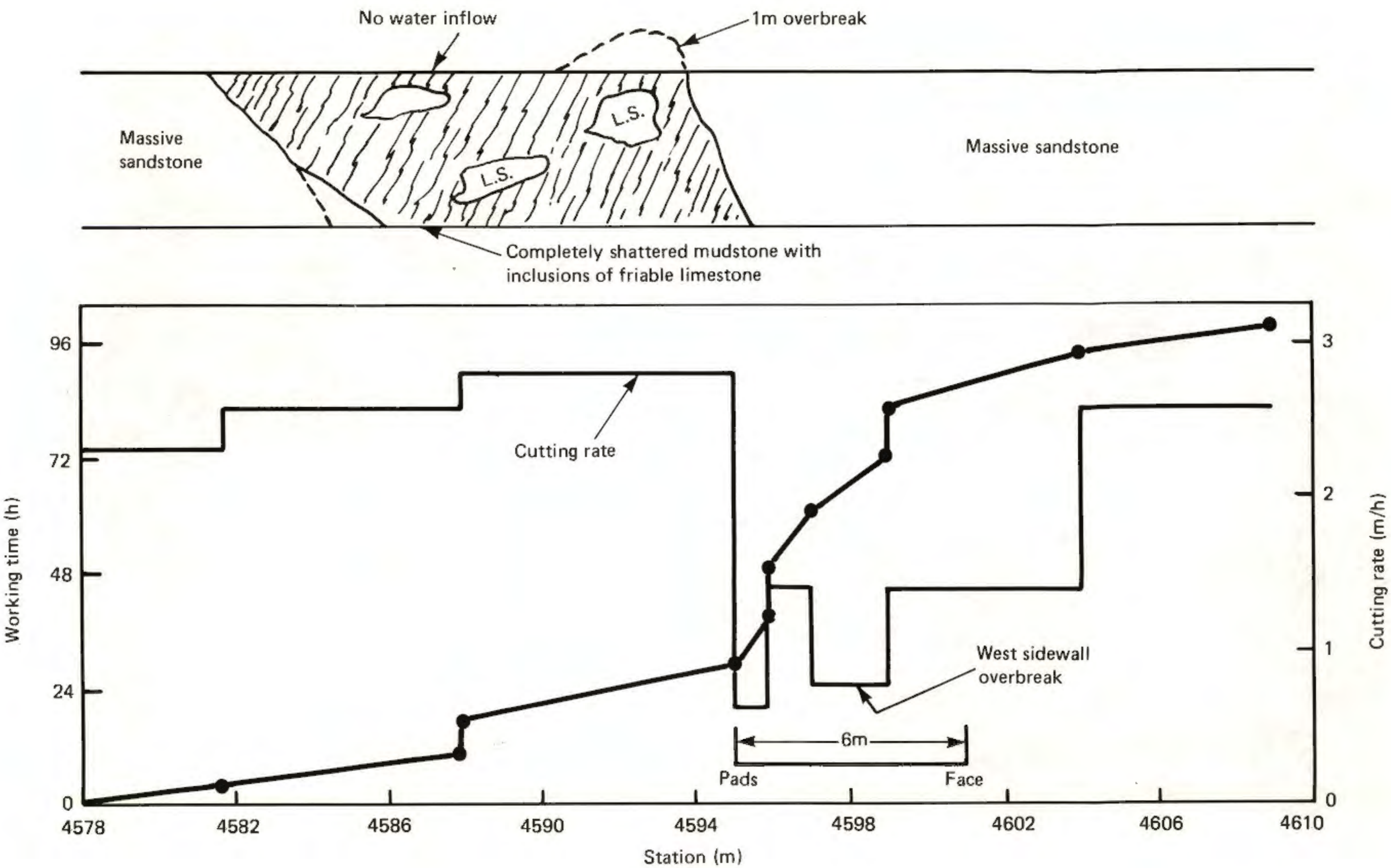


Fig. AIV–6 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN TEES STATION 4578 AND 4610, ROBBINS TBM

PLATE AIV-6. Transition between shattered mudstone and massive sandstone, Tees station 4594

PLATE AIV-7. Transition between fault gouge showing minor squeeze and massive limestone, Tees station 1295 the thrust mobilized. Only after the pads encountered massive rock did the cutting and progress rates return to normal. Once the bearing problem started it usually continued until the distance between the front of the cutter head and the gripper pads in full retracted position was covered. This was 6 m for the Robbins and 8.8 m on the Demag.

The moderate overbreak in the crown and west sidewall at the fault margin, Figure AIV-6, was largely the result of the failure to place support concurrent with advance. All five arches were installed after the incident. Subsequently, bearing problems in the west sidewall accounted for the major delay. Five shifts (over 60 hrs) were required to pass out of the fault zone, delays of 3.68 hrs/m over 9 m for bearing related work (Table AIV-6). Bearing related delays on exiting the fault shown in Figure AIV-7 were considerably less than the previous case, 0.62 hrs/m over 14 m (Table AIV-7). The difference was attributed to the decreased dip angle of the fault margin and the continuous placement of support behind the cutter head. Ground within the zone was a stiff gouge; following excavation 10 to 20 cm of closure was recorded as a result of squeeze (see Plate AIV-7; 60 m overburden).

When the transition in rock mass strength at the fault margin was less pronounced delays related to bearing were absent. The case illustrated in Figure AIV-8 is a good example. On departing from the zone of shattered mudstone, the fractured dolerite caused only a minor reduction in cutting rate (to less than 3 m/hr). Further, the transition from dolerite to more massive sandstone resulted in no change (Table AIV-8). There was a significant inflow of water, particularly within the fractured dolerite. Aside from waiting for flows to subside and placement of pumps, progress was not greatly hampered, attributable to the lack of influence of water on dolerite. If, however, the zone of shattered mudstone was longer the machine would most likely have dropped below grade on account of water softening the invert material. Despite the good progress through the 7 m zone (0.86 m/hr) the cutter head started to sink as the heading approached

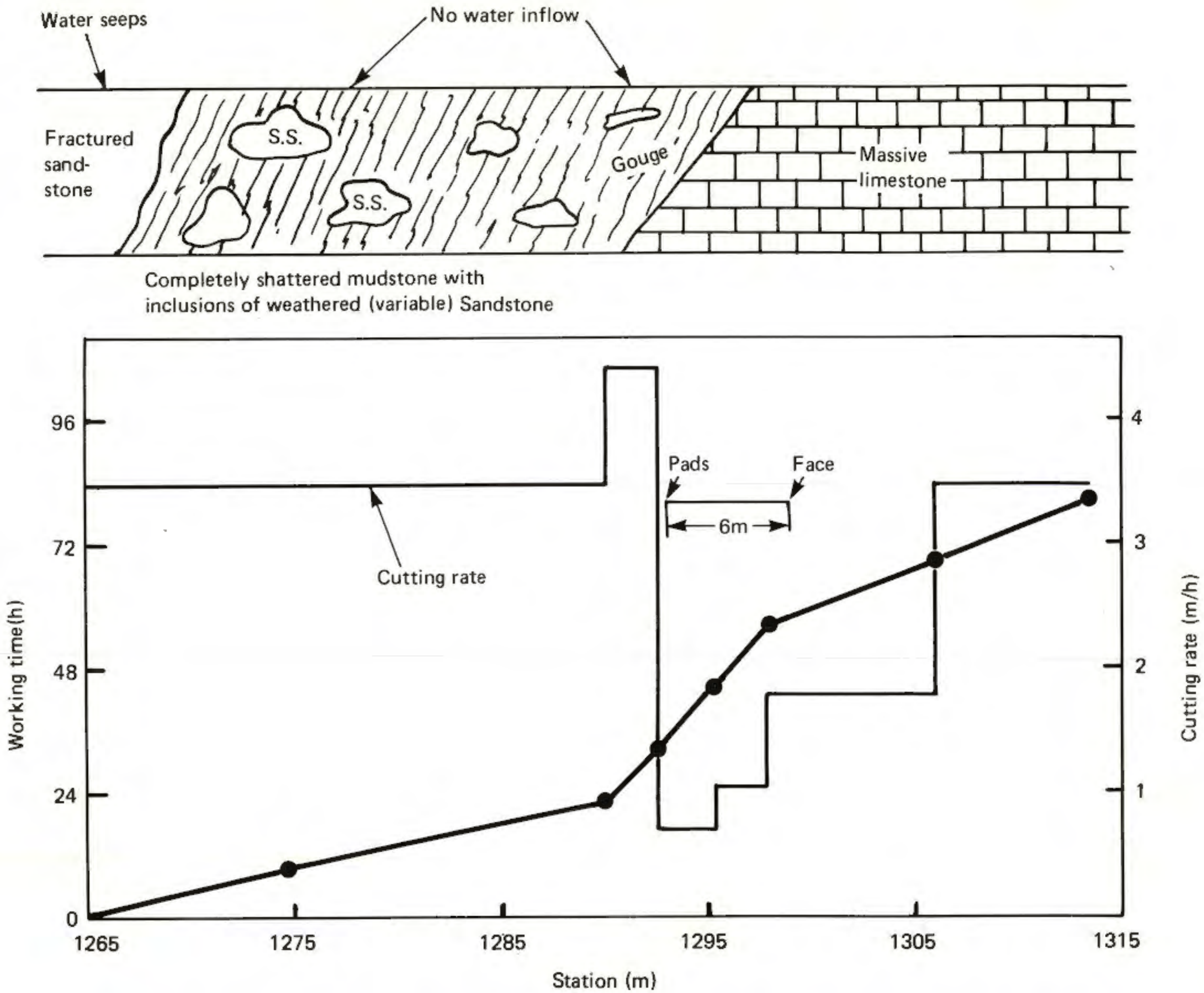


Fig. AIV–7 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN TEES STATION 1265 AND 1315, ROBBINS TBM

TABLE AIV - 7

Performance between Tees station 1265 and 1315, Robbins TBM

Operation (hr/m)	Station (m)	
	1265-1292	1292-1306
Driving	.28	.83
Supporting	.48	.84
Bearing	.00	.62
Other	.39	.34
Progress rate (hr/m)	1.15 (0.76)	2.63 (2.29)
(m/hr)	0.87 (1.32)	0.38 (0.44)
Type support near face	3 arches and bolts	6 arches

() excludes "Other"

TABLE AIV - 8

Performance between Tees station 4015 and 4050, Robbins TBM

Operation (hr/m)	Station (m) 4016-4045
Driving	.27
Supporting	.30
Bearing	.00
Other	1.19
Progress rate (hr/m)	1.76 (.57)
(m/hr)	0.57 (1.76)
Cutting rate (m/hr)	3.72
Type support near face	4 arches

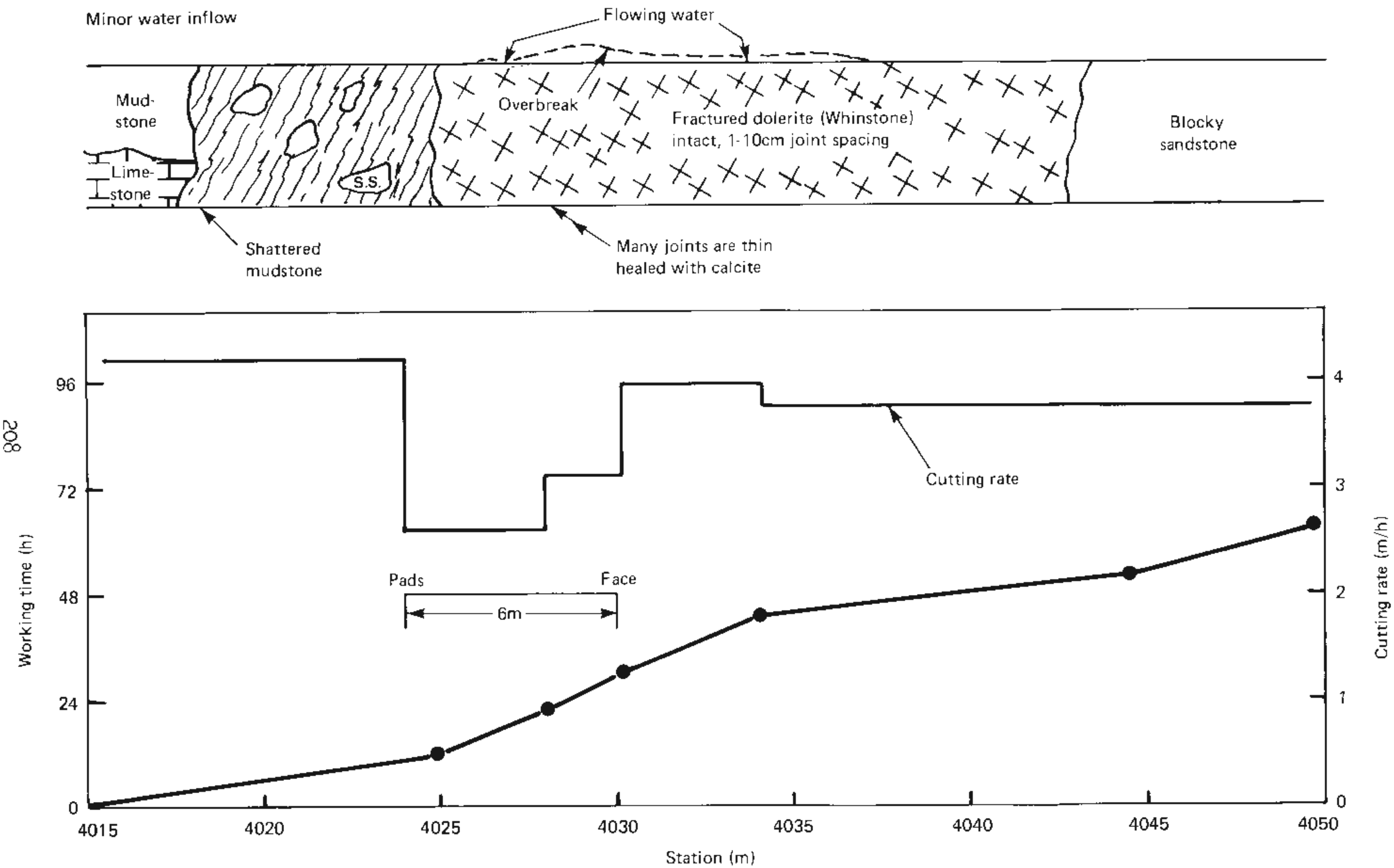


Fig. AIV–8 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN TEES STATION 4015 AND 4050, ROBBINS TBM

the margin.

Figure AIV-9 summarised a case representative of steady progress in difficult ground without ground water inflow. Standup time was such that continuous lagging and support, arches on 1 m centers, were required immediately behind the cutter head, Plates AIV-8 and -9. Average progress of 0.26 m/hr through the 18 m wide fault zone was reasonable even by conventional tunnelling standards. Some difficulty was evident on encountering the massive sandstone, although little time was lost. The friable sandstone had a higher bearing capacity than the shattered mudstone and sidewall integrity was maintained with immediate support. Most effort went to placing support resulting in delays of 2.97 hrs/m over 23 m, Table AIV - 9. This demonstrates the difficulty of installing arches and lagging behind the cutter head, but well worth the effort as previously illustrated.

Since it was not possible to install support close to the face of the Demag machine, fault zones of significant width inevitably caused severe problems and delays. Similar to the Robbins machine, as demonstrated in Figure AIV - 10, the Demag initially had few problems passing through the 12 m fault zone and cutting rates also increased. A few meters before the heading reached the fault margin the standup time was exceeded with collapse of roof and sidewall. From that point until the front 12 m of the machine passed well out of the difficult zone, major efforts went to removal of debris from sidewalls, packing walls for gripper pad reaction, clearing fallout from the roof shield, and installation of support. Placing 14 arches at 1 m centers plus work related to bearing resulted in delays of 4.69 hrs/m over 22 m (Table AIV - 10). As shown in previous cases, there was evidence that the Demag machine also had difficulty penetrating the more massive rock at the fault boundary (station 1524), although, this was further complicated by overbroken sidewalls. Only after advancing 19 m beyond the fault margin did sidewall stability cease to be a problem. Average progress was 0.74 m/hr up until the collapse and 0.16 m/hr after, Table AIV - 10.

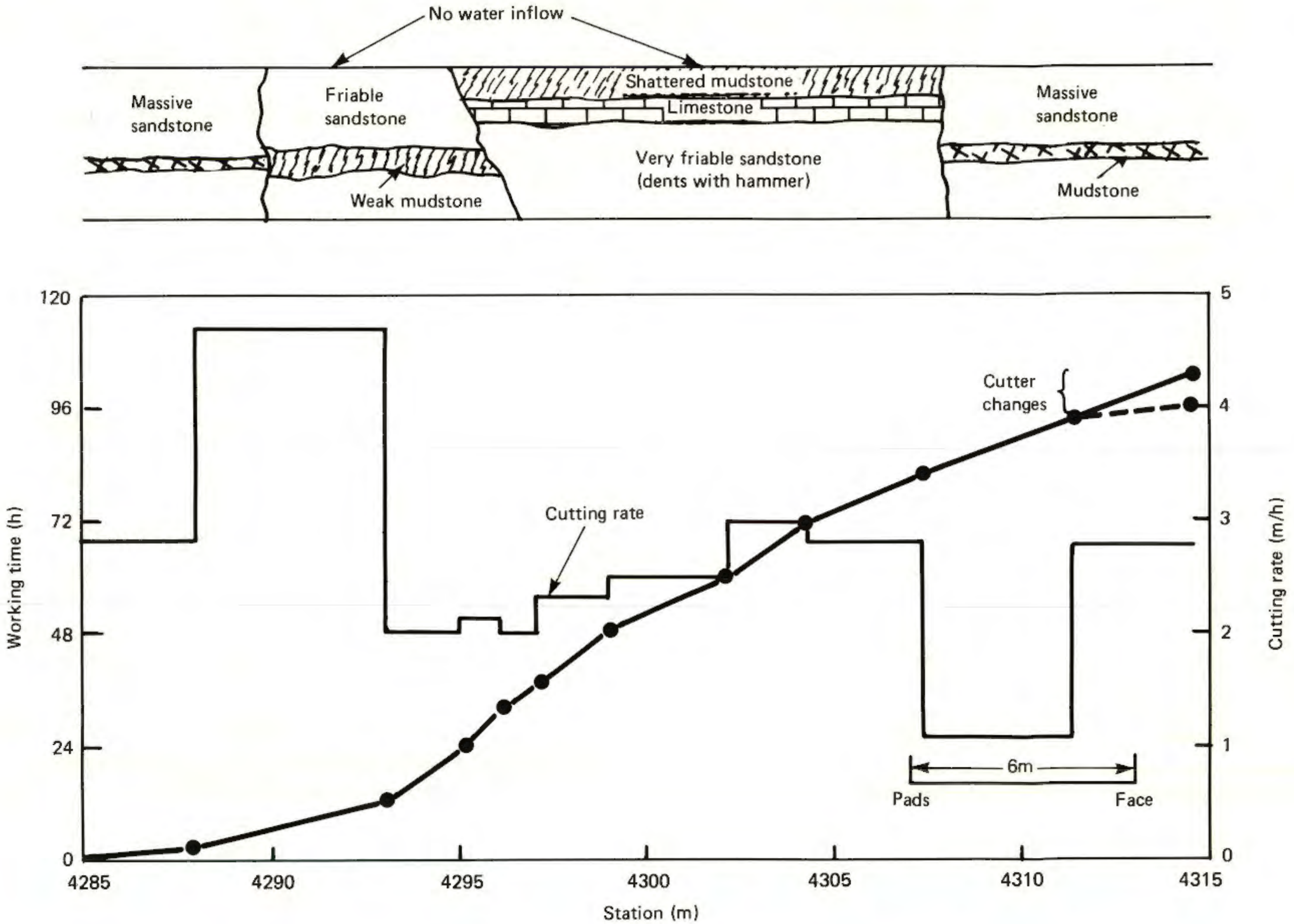


Fig. AIV–9 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN TEES STATION 4285 AND 4315, ROBBINS TBM

PLATE AIV-8. Shattered mudstone (top) and friable sandstone (bottom), Tees station 4300

PLATE AIV-9. Shear zone ground support of continuous lagging and circular steel sets spaced on one meter centers, Tees station 4303 to 4310

TABLE AIV - 9

Performance between Tees station 4285 and 4315, Robbins TBM

Operation (hr/m)	Station (m) 4288-4311
Driving	.46
Supporting	2.97
Bearing	.00
Other	.46
Progress rate (hr/m)	3.89 (3.43)
(m/hr)	0.26 (0.29)
Cutting rate (m/hr)	2.74
Type support near face	21 arches, ~1m spacing

() excludes "Other"

TABLE AIV-10

Performance between North Wear station 1505 and 1544, **Demag TBM**

Operation (hr/m)	Station (m)	
	1510-1522	1522-1544
Driving	.41	.61
Supporting	.00	3.00
Bearing (clearing sidewalls)	.00	1.69
Mucking (blocked chute)	.34	.34
Other	.61	.68
Progress rate (hr/m)	1.36 (0.75)	6.32 (5.64)
(m/hr)	0.74 (1.33)	0.16 (0.18)
Cutting rate (m/hr)	2.44	1.63
Type support	-	14 arches and bolts

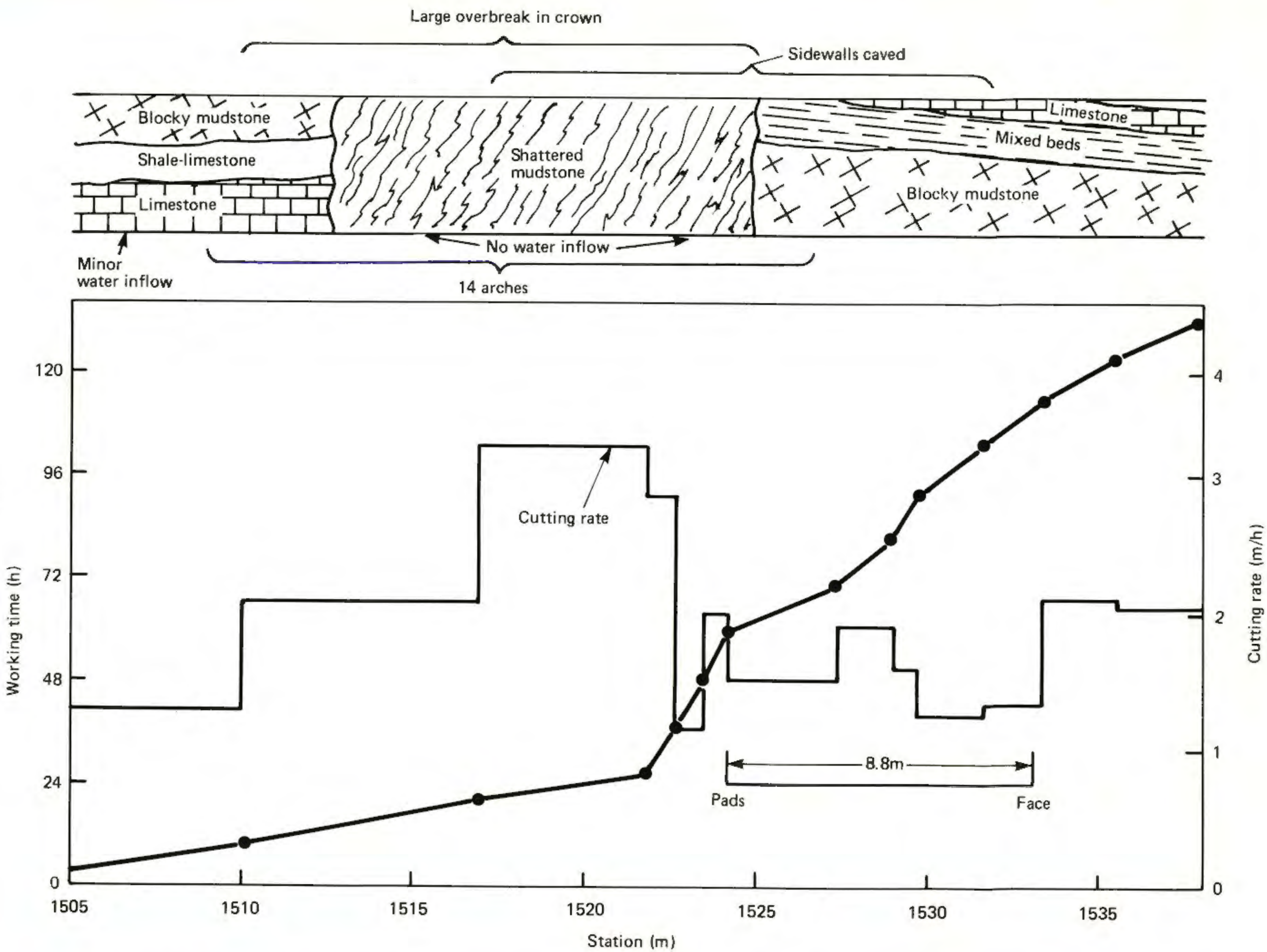


Fig. AIV–10 GEOLOGIC PROFILE, CUMULATIVE WORKING TIME AND CUTTING RATE BETWEEN N. WEAR STATION 1505 AND 1538, DEMAG TBM

Average Performance and Payment

From the start of tunnel construction up to September 1978, the average performance per month has been 160 m for Demag 1, 230 m for Demag 2 (only approximate), and 326 m for the Robbins drive. More detailed progress data by year are given in Table AIV - 1. Utilization of all machines has been between 20 and 30 per cent. Both higher cutting rates and immediate support capability account for some proportion of the difference in average progress, however, the three tunnel headings were not the same. Aside from the fault zones over the first 1.5 km, the Robbins drive had less difficult ground than the other two headings.

Recently, the North Wear drive (Demag 2, Figure AIV - 1) has entered a large zone of shattered mudstone in which it can not operate economically. After excessive overbreak exceeding the opening diameter and progress of less than 1 m per day, the machine drive has been replaced with conventional methods until better ground is reached. The machine will follow the heading and operate for mucking.

Payment on a per meter basis was dependent on the rock type. The five classes in increasing order of payment were limestone, sandstone, mudstone, highly fractured rock and fault gouge. A rock mass classification system was actually embodied within the scheme, as each rock type had relatively constant mass characteristics. In situations of mixed face the rock within the crown determined the class.

Fourth Tunnel Heading

The 6.4 km drive from the Derwent valley, Figure AIV - 1, was subcontracted and originally started by drill and blast methods. After 600 m a Titon (Paurat) roadheader was installed and operated for only 100 m after which poor performance forced its removal on encountering massive limestone (approximately 150 MN/m^2 compressive strength). Until the recent experience in the North Wear heading, the plan was to install the first available Demag as both machines were purchased by the contractor while the Robbins TBM is under lease. With large regions of shattered mudstone also predicted for the Derwent drive, the contractor has decided to rebuild and use the Robbins after the present operation is completed.

TABLE AIV - 1

Overall TBM progress, 1975 to 1978

Period	Machine progress (m/month)		
	Demag 1	Demag 2	Robbins
1975	133 (4 mo.)	-	-
1976	206	147 (11 mo.)	163 (4 mo.)
1977	180	287	313
1978*	85	stopped	416

* first 9 months; Demag 2 stopped in August

ANTWERP PRE METRO

Pre Metro construction for the city of Antwerp in Belgium has been under way since January 1970. The system is called the Pre Metro since it uses the existing street level tramway cars in the new underground stations and tunnels. Outside the city the tramway joins the surface system on a private right of way. In the future, with some minor adaptions, the system can be transformed into a full-scale Metro.

Parts of the East-West or First Axis of the Pre Metro are in operation with the remainder to be operational by 1980. Construction of stations included the cut and cover method employing a diaphragm wall to depths up to 30 metres. Average cost for the diaphragm wall system per square metre was between 5 and 10 thousand Belgium Francs ($150-300). The reinforced wall was used for permanent structural support and guaranteed water tight by the contractor for five years. Stations constructed by the method described cost between 150 and 180 million Belgium Francs (approximately 5 million dollars). Overall average construction cost was about one million Belgium Francs ($31,000) per metre of line at 1974-75 prices.

Construction of a Second Axis or line has been under way since March 1977. The initial phase consist of four stations and 3.8 km of running tunnel, all to be constructed underground. In contrast to stations built by the cut and cover method, the underground stations will cost four to five times as much, however, the additional expense was justified on considering the disruption to traffic and pedestrians. Total cost is 3 billion Belgium Francs (about $94 million) half of which is for construction of the running tunnels.

Station construction posed unique problems as a result of the shallow cover, 1.2 to 0.5 m, and the necessity to maintain normal traffic flow on the streets above. The station roof was formed by jacking 1.2 m diameter concrete pipes across the width of the future opening, Plate AIV - 10, one along side the other for its entire length.

PLATE AIV-10. Heading of pipe jacking operation for station roof

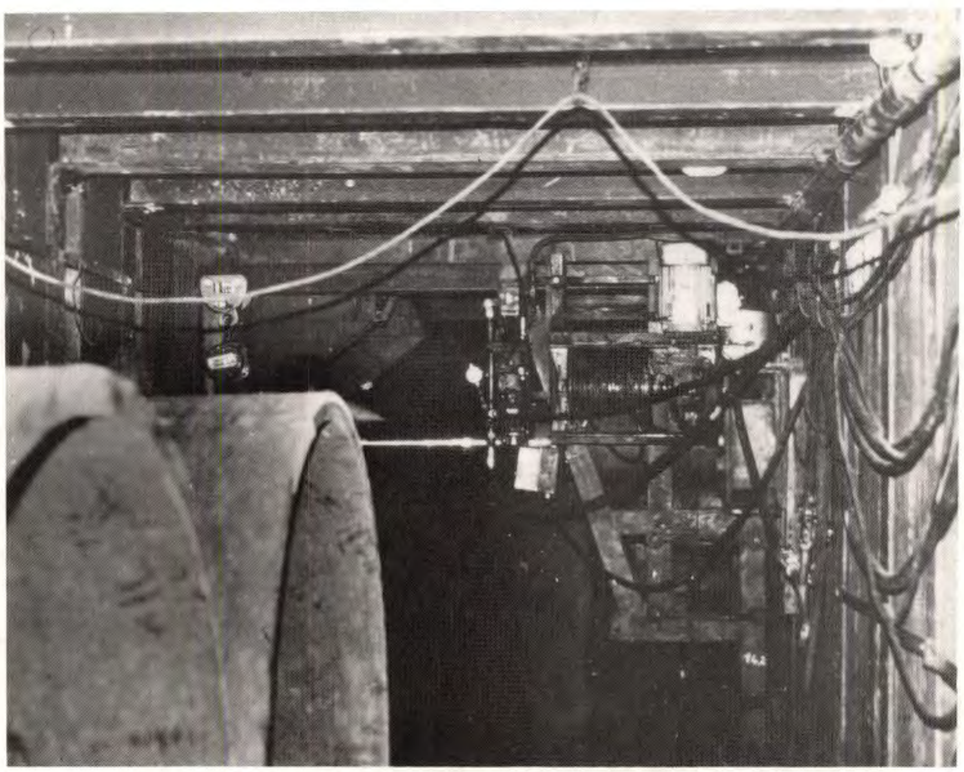


PLATE AIV-11. Pipe jacking from small drifts

Pipe jacking operations for the transfer station "Astrid", part of the First Axis, were carried out above ground and to one side of the station. The total length of jacked pipe ranged between 30 and 55 metres. A removable three piece cutting shield was fitted to the leading edge of the first pipe. Control on position was by laser guidance and steering was accomplished by wedging between the shield and pipe. A typical advance rate was 5 metres per day at a cost of 30 thousand Belgium Francs per metre (about $940). The actual price paid the subcontractor for the work was approximately 30 per cent less. Settlements at the surface ranged between 1 and 2 cm, well within the 4 to 5 cm allowed. Maximum allowable angular distortion was specified at 1 to 500.

When it was not possible to do the pipe jacking operation from the surface, as along the Second Axis, small underground galleries (2.5 by 2.5 m square) were first excavated along the length of the station boundary. The galleries were constructed in traditional fashion employing steel sets and forepoling with steel sheet. From within the openings the pipe jacking took place as previously described only using one metre long segments, Plates AIV - 11 and - 12. To complete the roof structurally, the pipes were fitted with a reinforcement cage and backfilled with concrete. Also, from within the galleries the side walls were constructed to a depth of 16 m below the surface, slightly above the ground water table. Upon backfilling the galleries with concrete, the upper portion of the station structure was completed and ready for excavation. To further extend the station to the design depth below the water table, diaphragm walls were constructed from within the upper level through the silt, into the impervious clay floor at a depth of more than 30 m.

The running tunnels are positioned below the ground water table within a deposit of fine sand containing lenses of clay (minimum 10 per cent minus 20 micron). Distance between the ground surface and the invert of the 6.45 m diameter tunnel is approximately 25 m. Maximum height of water above the invert as specified by the owner was 12 m.

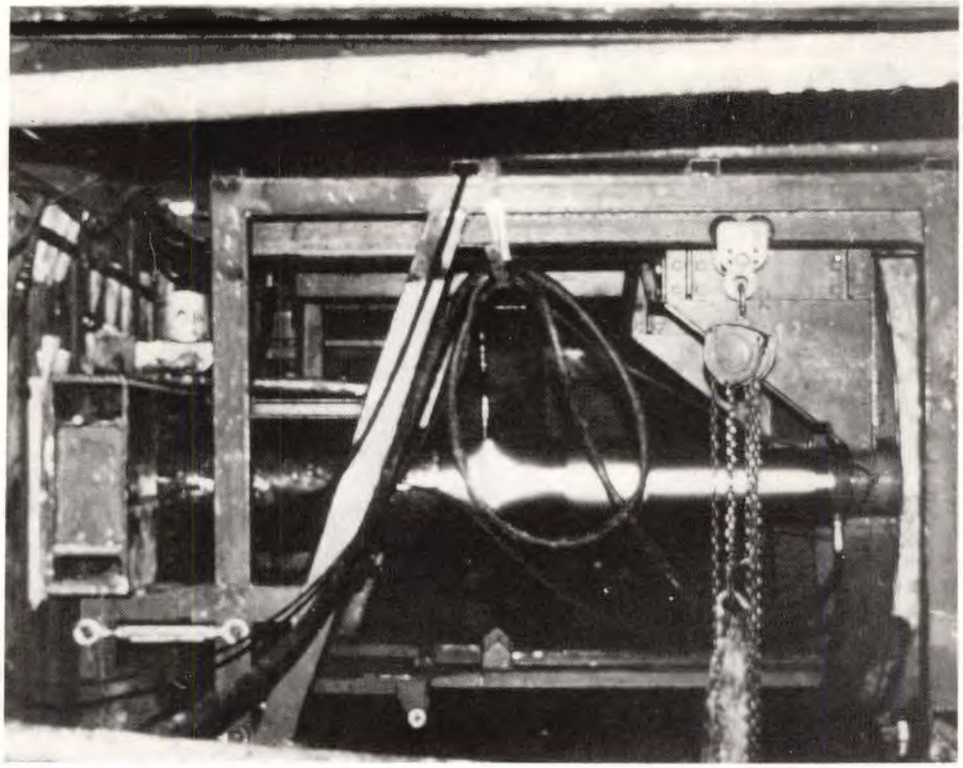


PLATE AIV-12. Specially designed ram for underground pipe jacking

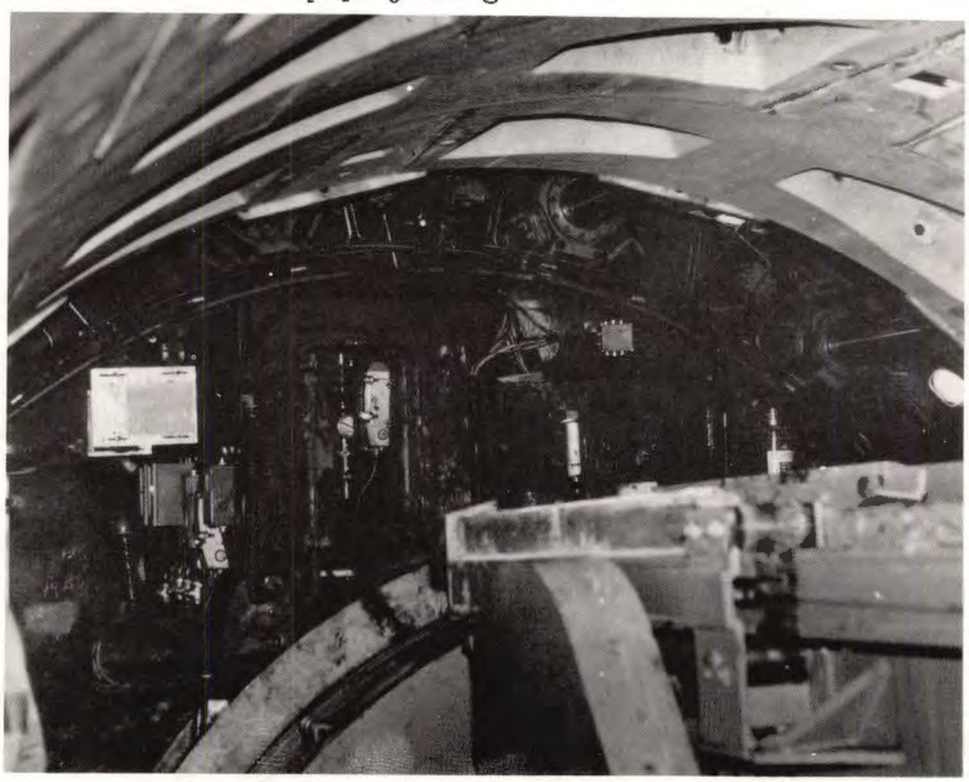


PLATE AIV-13. Wayss and Freytag bentonite shield and segmental concrete liner

Mecoma, the consortium of contractors awarded the project, is composed of among others Wayss and Freytag, the bentonite shield manufacturer. Their design differs from other systems in that the hydraulic pressure at the face is independent of the slurry flow to and from the plenum chamber. This is accomplished by dividing the pressure bulkhead into two sections with a partial diaphragm wall separating the slurry on the face side from a pressure controlled air cushion (Jacob, 1976). Another feature is the downward incline of the arrow-shaped cutting head axis and the position of the head fully within the protective cover of the shield. This ensures that no large areas of the face are undercut which differs considerably from the bentonite shield used at Warrington, England (Walsh and Biggart, 1976).

Much of the experience for the design of the shield, tailskin seal, and concrete liner segments was obtained at the Hamburg-Wilhelmsburg sewer tunnel (Jacob, 1976). Although compressed air was employed, eliminating the need for a tailskin seal, the manufacturer experimented with seals during the drive. A satisfactory design was achieved for a water pressure of 16 m lasting for between 0.5 to 1.0 km before requiring replacement. The maximum daily advance was 12 m or 15 rings of liner. On this basis the contractor expects to place between 10 and 15 rings per day. Since the shield has only recently turned under, 15 m total advance, there is as yet little experience.

No boulders or cobbles are expected, however, if encountered they are caught on an oblique grate in front of the outlet pipe. These are removed manually after filling the plenum chamber with compressed air. The pressurization operation only requires a few minutes.

One liner ring is composed of seven precast concrete segments with a cast-in water proof joint, Plate AIV - 13. The 1.1 m wide, 0.3 m thick segments were expanded with a key and bolted in all directions. Casting tolerances of 0.5 mm were required such that the neoprene sealing strips would form a tight contact for a water proof connection. The 4 cm void created by the tailskin is continuously grouted with cement.

ARC ISERE HYDROELECTRIC SCHEME

A major feature of the Arc Isere scheme was the 18.9 km water tunnel connecting the Glandon and Flumet valleys. Constructed from two headings, the 10 km drive from the Flumet valley employed a 5.8 m diameter Wirth 580H full face machine. Fitted with 38 cutting tools, all double discs except the four pilot bits, the machine was well suited for the excavation of medium strength rocks. Aside from 1 km of sedimentary rock and 700 m of granite-gneiss the remainder of the opening traversed crystalline schists, predominantly mica schist. From Glandon valley, the second heading was constructed by traditional methods using a Montaber jumbo fitted with six hydraulic drills. This approach was also well suited for the excavation of the high strength granite-gneiss encountered along the majority of the tunnel. Further complicating construction on both drives were several large shears, some with inflowing water, and a maximum overburden of 2 km.

Machine performance was largely influenced by the rock mass character and the ground water regime. In ground without significant stability problems the average progress was 16.3 m per day with peak rates up to 35.4 m per day (3-8 hour work shifts). Overall average for the entire drive was reduced to 10 m. The difference was largely the result of difficult ground, however, the relatively small amount encountered for the total length excavated, 300 m or 3 per cent, illustrates the vulnerability of full face machines. Over 25 per cent of the total construction effort was required to traverse the 13 shear zones, 10 to 50 m thick.

Eight major "accidents" or delays occurred when large fallouts of shattered rock completely blocked the heading. Water inflows were usually minor, although some squeeze of the sheared ground was present. On two occasions the machine head was so severely blocked that conventional methods were employed to pass through the zones and free the machine. A pilot tunnel was constructed around the trapped machine, through the shear zone into competent ground and finally, to a position on the main tunnel alignment. From this location, a second larger heading was driven through the shear toward the machine head. Each rescue caused three to five months delay. Overall progress for the 300 m of difficult ground was 0.85 m per day.

After passing through the first major fault a hydraulic drill for probing ahead was installed behind the cutter head. It could only be operated when the machine was stopped such as during tool changes. Although it was possible to drill 30 to 50 m ahead in two hours, continuous probing was not practical on account of interference with production.

Full face drill and blast operations in competent ground averaged 8.5 m per day, only half that of the machine heading in massive rock. In difficult ground of which there was nearly 900 m average progress, 2.7 m per day, was at least three times better than that by machine. Proportioned to length excavated, this 12 per cent of the drive required under 25 per cent of the total construction effort. It was also evident that shear zones encountered were further complicated by high water inflows, often in excess of 100 l/sec. In addition, when the overburden exceeded 1200 m systematic rock bolting was required to control ground deformations. Overall average progress for the entire heading driven conventionally was 7 m per day. Considering the larger proportion of difficult ground encountered the rate was high. Given the statistics of the machine drive, if the amount of difficult ground was greater, more than 7 per cent, the drill and blast operation would have produced the higher progress rate.

Aside from the machine performance in difficult ground the progress in the hard and abrasive granite-gneiss revealed additional problems. Fortunately, the massive rock was encountered over the remaining 700 m of the drive. Progress dropped to under 7 m per day (with 50 per cent utilization assumed, cutting rate was roughly 0.6 m/hr). Not considering tool wear, conventional methods progressed at a higher rate and were more economical.

As a part of the water conveyance system an additional 3.9 km length of tunnel was excavated by a second full face machine. The 8.1 m diameter Robbins encountered relatively massive calcareous and/or shaly sediments of medium strength (Lias). Average advance was 13 m per day and maximum progress was 30 m per day.

A nearby hydroelectric scheme, de la Coche, also employed a Robbins machine for a planned 7.9 km drive. The 3 m diameter machine was of a "Melbourne" type using a rail shield designed to allow for spiling at the face. Traversing a variety of formations of both loose and massive rock after 6.1 km the machine was inundated with water and debris on intersecting a major fault, Plate 3-1 (page 126). The cohesionless granular material (mostly limestone) completely covered the front half of the machine creating a void of 4 to 5 m high and several meters wide. As previously described, a pilot tunnel was driven around the machine to drain the fault and to continue excavation. On further inspection it was decided to remove the machine which up to that point had averaged better than 200 m per month. A McAlpine roadheader was installed to carry on with excavation, however, after 142 m it was also removed as a result of poor performance, only 60 m per month. The heading was subsequently completed by conventional methods with an average advance of 100 m per month.

ARLBERG ROAD TUNNEL

Constructed between mid 1974 and early 1979, the 14 km long tunnel was the third Austrian road tunnel to employ the "New Austrian Tunnelling Method" (NATM). In principle, NATM is an efficient construction method for openings situated in squeezing ground (ground with a significant time dependent response). The installation of reinforcement and support is coordinated with rock mass character, excavation sequence and measured opening response to control the rate and amount of ground deformation. By allowing for deformation the available shear strength of the rock mass is mobilized; controlling the deformation preserves the inherent strength by reducing rock mass deterioration. As a result, the self-stabilizing contribution of the ground is maximized, thereby, minimizing the required capacity of the installed stabilization system.

Cost savings were not only effected through reduced support, but also, by increased usable space. Out of the 90 to 103m^2 excavated section, 75 per cent of the area was available for ventilation and traffic. This can be compared with the nearly parallel Arlberg Railway Tunnel completed in 1884. Using a stiff, granite masonry support only 46 per cent of the 90 m^2 section remained as usable space (John, 1977).

The majority of the tunnel alignment encountered squeezing mica schist with zones of mylonite. In terms of material strength parameters, dry internal angle of friction was 15 to 20 degrees, 11 to 12 degrees wet and cohesion, 5 to 10 kg/cm^2. Maximum squeeze or opening closure was 0.7 m and average closure approximately one-half this amount. Position of maximum squeeze did not correspond to that of maximum overburden (800 m), however, to the point at which the ratio of initial stress to rock mass strength was greatest. Ground deformation at the opening boundary was not symmetrical. Large volumetric strain normal to the schistosity direction resulted in an increased proportion of displacement along the south wall.

Although the Arlberg Tunnel is continuous, it is composed of two tunnel sections separated by a valley. Consequently, it was possible to drive the openings from four headings. Depending on ground character, the excavation was driven full face or by top heading and bench (one or two benches), with a round length between 1 and 3 m. Advance rate, also influenced by geology, was nearly 200 m per month in ground with deformations up to 20 cm. In addition to the main excavation, two 9 m diameter ventilation shafts were constructed. The 736 m deep Albona shaft was excavated in full section from top to bottom. Because of its position and length, the 218 m Maienwasen shaft was constructed by raise bore and enlarged from the top down. Both shafts were provided with waterproof insulation and concrete lined.

Five rock classes were employed to differentiate the support-reinforcement measures and excavation procedure (John, 1976). In the most difficult ground stabilization measures consisted of : 21 kg/m yieldable steel sets spaced between 1.2 and 2.5 m, 10 to 15 cm layer of shotcrete with wire mesh, and up to 500 m of 9 to 12 m long rock bolts per meter of tunnel. After excavation the steel set and mesh were placed at the face and embedded in shotcrete. Fully cement grouted, untensioned rock bolts were installed approximately one round behind the face. Open gaps between the shotcrete or slots, 20 to 30 cm wide, circumferentially spaced every few meters were deliberately created to allow for the ground squeeze. Without the slots, a circumferential strain of up to six per cent would lead to significant shotcrete deterioration. The 24 mm diameter rock bolts were often forced beyond yield, although not always in direct tension. Large shear displacements on schistosity planes or within mylonitic zones caused some intersecting bolts to be sheared. Despite ground deformation control, the effect of local loosening (up to 3 m) was too great for complete transfer to the reinforcement system. Consequently, the steel sets (relatively light capacity) were used more than originally planned. Face stability was not a problem.

Ground and support measurements were vital to interpret opening response. A system of control measurements (horizontal convergence and crown settlement) and support measurements (extensometers, instrumented rock bolts and pressure cells) were carried out. In difficult ground control measurements were undertaken at intervals of 10 m. Based on experience, initial convergence rates of 2 to 3 cm per day indicated a stable opening. After one week this rate decreased to roughly 0.5 cm per day. For higher measured rates additional reinforcement was installed and round length was reduced to slow heading advance. The maximum measured convergence was on the order of 8 cm per day (John, 1977).

A final, unreinforced concrete liner with waterproof insulation was to be installed after the secondary ground deformation rate had decreased to less than 5 mm per month. Up to six months delay between excavation and liner placement was specified for this purpose. After the six month period, however, rates of up to 15 mm per month were recorded. In order to avoid construction delay, concrete strength specification was increased as a function of convergence rate. Between 5 and 12 mm per month compressive strength requirements were doubled (maximum to 400 kg/cm^2). As a further precaution, measurements of tangential stress were obtained with embedded pressure cells. Results indicated only the concrete self weight for rates up to 5 mm per month and stress between 5 and 10 kg/cm^2 for 10 mm per month. Diametric convergence of the 0.4 to 0.5 m thick liner has ranged from 1 to 5 mm.

The estimated total cost of the road tunnel at 1972 prices is 3520M Austrian schillings ($235M). Of this, construction cost is 2838M Austrian schillings ($190M).

BOCHUM UNDERGROUND

In many aspects, the design and construction of the Bochum underground are similar to other systems around the world. Both cut and cover as well as underground methods of construction are being employed for the 7 km line. Founded largely in marl and clay above the ground water table, tunnelling conditions could be classified as good with the exception that the marl quickly loses strength on exposure to air. Ground strength, although variable, was in the nebulous region between weak rock and strong soil, amenable to either soft ground tunnelling techniques or more traditional hard rock methods. The underground construction of the running tunnels and several stations presented a significant departure from soft ground methods. Rather than apply OETM (Old English Tunnelling Method), grouted precast concrete segments erected behind a shield, a system labelled NATM (New Austrian Tunnelling Method) was employed.

NATM is basically the installation of shotcrete, wire mesh and rock bolts in a coordinated manner so as to allow the mobilization of rock mass shear strength through controlled ground deformation. It was derived as a flexible system for squeezing ground conditions in Alpine tunnels. Even though the appropriate construction materials were employed, NATM, as applied in the Bochum underground was a misnomer. With only 3 to 8 m between opening crown and surface structures, the objective of the support system was to prevent as much ground deformation and hence settlement as possible. This requires the immediate installation of a relatively stiff support as compared to the stiffness o systems usually employed in squeezing ground where opening closure of up to 10 per cent is possible.

Construction was sequential as in traditional rock tunnelling methods. Excavation by Demag roadheader, Plate AIV - 14, provided the flexibility required to cope with changing cross sections, 35 to 95 meters square, and complicated bifurcations and intersections. Round length, up to 2 m, and heading attack, either full face or top heading

PLATE AIV-14. Demag roadheader at tunnel face and bench, depended on ground condition, Plate AIV - 15. Generally, the face was stable although it often required a layer of shotcrete to prevent deterioration. After an initial coat of shotcrete, wire mesh and lightweight steel sets were placed, followed by more shotcrete, Plate AIV - 16. In total, a very thick and stiff 25 to 35 cm layer of shotcrete was applied. Steel sets were only used to hold the mesh in place and profile the opening shape. Fully resin grouted rock bolts, installed in earlier tunnel sections (four to eight 3 m long bolts per advance), most likely contributed very little to the opening stabilization considering their stiffness relative to the thick layer of shotcrete.

A final concrete liner, nominally 30 cm thick, was designed for the full overburden with no consideration given to the contribution of the shotcrete. Earlier sections employed a layer of waterproofing material between the shotcrete and liner. This was subsequently replaced by a water resistant concrete mix and water stops at construction joints. The water resistant concrete has an increased water to cement ratio and a modified aggregate gradation to reduce permeability.

Underground station openings were simply constructed as the sum of two overlapping running tunnels driven side by side. Initially, an enlarged tunnel section was excavated and supported over the length of the station. A concrete liner was installed, however, the common intersecting wall was designed to serve as a pillar with provisions for passage ways. Finally, the second tunnel was excavated parallel to the first and lined to complete the total section: overall width 16.7 m, height 9 m.

Considering the large station spans and shallow cover the average settlements along the alignment were reasonable. The typical limit was 25 to 30 mm with a maximum of 40 to 50 mm at centerline. Favourable ground conditions have helped to limit the surface

PLATE AIV-15. Excavation of top heading

PLATE AIV-16. Application of shotcrete to top heading settlement trough width; consequently, no substantial damage to buildings has been reported.

Despite the considerable flexibility of the construction method, it was slow as compared with OETM and required a greater amount of control and workmanship. Average advance has been 2 m per shift with expectations of up to 3 m. Some controversy over the appropriateness of the method has emerged with the final analysis, as usual, left to economics (Leeney, 1976; Muller and Hereth, 1976; O'Reilly, 1977).

Civilworks cost of an 800 m section of running tunnel (70 to 90 square meter section), was 8 M marks ($4 M) or 10 M marks ($5 M) per kilometer. This was below the 3.5 M pounds (greater than $7.0 M at 1974 prices) cost per kilometer for the similar size Heathrow cargo tunnel in London clay, employing OETM with expanded concrete segments. Contributing to the low cost was the low material price of shotcrete and concrete, 70 marks ($35) per cubic meter, and the use of foreign labour.

BORN RAILROAD TUNNEL

Located in Solothurn, Switzerland, the 810 m long Born Tunnel has a 85 m^2 horseshce shaped section. Aside from landslide debris at each entrance and a 60 m length of limestone, the opening traverses Effinger marl. Although the marl is relatively massive and strong, the rock cover is marginal, less than 5 to 20 m, overlain by up to 30 m of loose landslide material. The anticipated shallow rock cover necessitated an extensive site investigation program and construction with minimum ground disturbance.

After exhaustive test borings from the surface and selection of the tunnel alignment, a full face Robbins tunnel boring machine was employed for further investigations. Positioned in the top heading, the 3.5 m diameter opening was driven the entire 810 m in two months. Despite high penetration, 1.5 cm per revolution, the average progress, 11 m per day, was low on account of machine back-up system problems (muck removal difficulties). When all systems were operating properly advance rate was up to 33 m per day (2-8 hr shifts per day). On completing the pilot tunnel, additional test borings from within the opening were made to delineate the rock-soil interface.

Excavation to full section was by top heading and bench. To minimize ground disturbance the top heading was driven by a Westfalia road header, Plate AIV - 17. Average excavation rate was 1 m per hour with a utilization of approximately 4 hrs per day. Cutter tool wear (picks) has typically ranged from 0.1 to 1 tool per cubic meter, although wear as low as 0.05 was recorded in the weaker marl. The high compressive strength of the marl, 170 MN/m^2 maximum, was responsible for increased wear and reduced excavation rate. For strengths in excess of 80 MN/m^2 the contractor received additional expenses. If not for the robust and stiff design of the Westfalia machine, the mechanical excavation of the strong marl would not have been possible. Beside

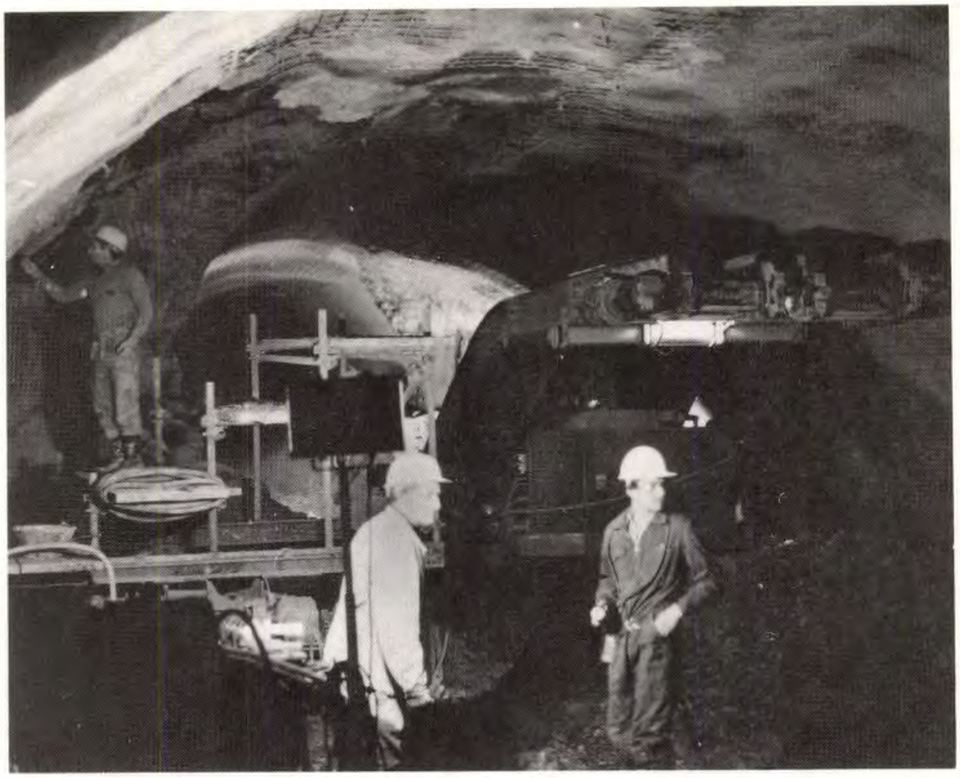


PLATE AIV-17. Construction of top heading with excavation by Westfalia roadheader; note pilot tunnel

PLATE AIV-18. Dust extraction through pilot tunnel problems associated with rock strength, the dust produced by point attack machines in similar ground was known to be excessive. This was another major factor influencing the decision to construct a pilot tunnel, Plate AIV - 18. Without the opening for exhaust ventilation at the face, health and safety regulations would have made it impractical to employ a road header. The bench will be removed by excavator and ripper.

Ground support and reinforcement was installed after each 2 to 4 m round. Rock bolts between 2 and 4 m long, spaced on 1.6 to 2 m centres, were resin anchored and fully cement grouted. Reinforcement was followed by wire mesh and shotcrete (less than 5 cm thickness). For permanent support the opening will be concrete lined and waterproofed with polyvinyl chloride insulation.

An excessive profile of saturated landslide debris necessitated freezing the ground along the first 75 m of tunnel from the north entrance. Freezing pipes for circulating a calcium chloride solution were installed from the surface in a fan shape around the tunnel opening. With a pipe spacing of 1 to 2 m the freezing of each section, three 25 m lengths, required three weeks. After excavation by top heading (road header) and bench (excavator) a reinforced concrete liner with insulation was installed.

Project duration is estimated at three years, 1976 to 1979, and total cost 19M Swiss francs (\$9M). Tunnel construction cost not including the frozen length and related cut and cover work was 12M Swiss francs (\$6M).

FREJUS ROAD TUNNEL

Preliminary work for the 12.9 km alpine road tunnel started in 1973-74. Two headings, one in France the other in Italy, are being driven simultaneously along with three ventilation shafts. Only the road heading from the French side, started in October, 1974, will be considered in the following discussion.

Except for a Triassic formation (anhydrite, dolomitic limestone, green shale) encountered for the first 450 m from the portal and another 230 m of anhydrite, 1500 m in, the remainder of the opening traverses a massive mica (glossy) schist. It is relatively impermeable resulting in a nearly dry tunnelling environment. The 10 m wide, 8 m high horseshoe shaped tunnel has a maximum overburden of 1730 m. At such great depth, the ground nearest the opening undergoes a large, time dependent, inelastic deformation or squeeze.

As a result of the fractured and faulted character of the Triassic formation the first 550 m of tunnel was excavated by crown drift, top heading, and two stages of bench. Ground support was by steel sets and concrete liner. It took 11 months to complete this section.

On penetrating the massive schist, the heading was advanced in full face. A relatively stiff system of rock bolts and 15 cm of shotcrete was used by the contractor to stabilize the opening. With the high overburden, exceeding 500 m, ground squeeze was significant leading to the failure of the shotcrete by shear. The contractor was advised to change his method of ground control. A system employing rock bolts and wire mesh without the brittle shotcrete was suggested. As a result of the large deformations, 20 to 30 cm closure, the contractor argued that it was not possible to employ such a system. Instead a request for a 90 per cent increase in bid price and a two year extension to complete the contracted works was offered as a solution. This was rejected and a new contractor has taken over construction.

The suggested system of rock bolts and wire mesh has been successfully adopted, Plates AIV - 19 and - 20. On the average, 20 bolts with lengths from 3 to 4.65 m were installed per meter of tunnel (approximately one bolt per square meter). Bolt lengths were varied to account for the asymmetrical closure of the tunnel cross section. This naturally resulted from the near parallel alignment of the tunnel axis and the strike of the schistosity with an oblique dip. Ground reinforcement cost was slightly more than 3000 French francs ($600) per meter of tunnel (about $8 per meter of bolt).

A typical bolt assembly is composed of a 20 mm diameter rod of mild reinforcement steel, mechanical point anchor, bearing plate, and spherical seat. The bearing plate, a small rectangular sheet of corrugated metal, in series with the spherical seat adds considerable flexibility to the reinforcement system. As the ground deformation leads to increased induced tension within the bolt, both the corrugations and seat are compressed flat before yield of the bolt steel. The compression amounts to 20 cm of displacement. If elongation of the bolt and seating of the anchor are considered the total displacement of ground beneath the bearing plate can reach 30 cm without failure of the reinforcement. This has been demonstrated in practice.

The excavation cycle was similar to most drill and blast operations with several important differences. Using a 0.5 m spacing of holes about the perimeter of the tunnel, the outside line was smooth blasted to assure a minimum of disturbance to ground nearest the opening. Debris from the 4 m round was quickly moved away from the face and stockpiled at a point 100 m back for later removal. As soon as possible the truck mounted bolting jumbos were brought to the face for immediate installation of the reinforcement. This effort was designed to minimise the amount of unrestrained deformation. Bolts were installed radially to within one meter of the face and pretensioned to 3 tons. Finally, wire mesh or chain link was placed and fixed with a second bolt plate and nut.

PLATE AIV-19. Tunnel heading and reinforcement system

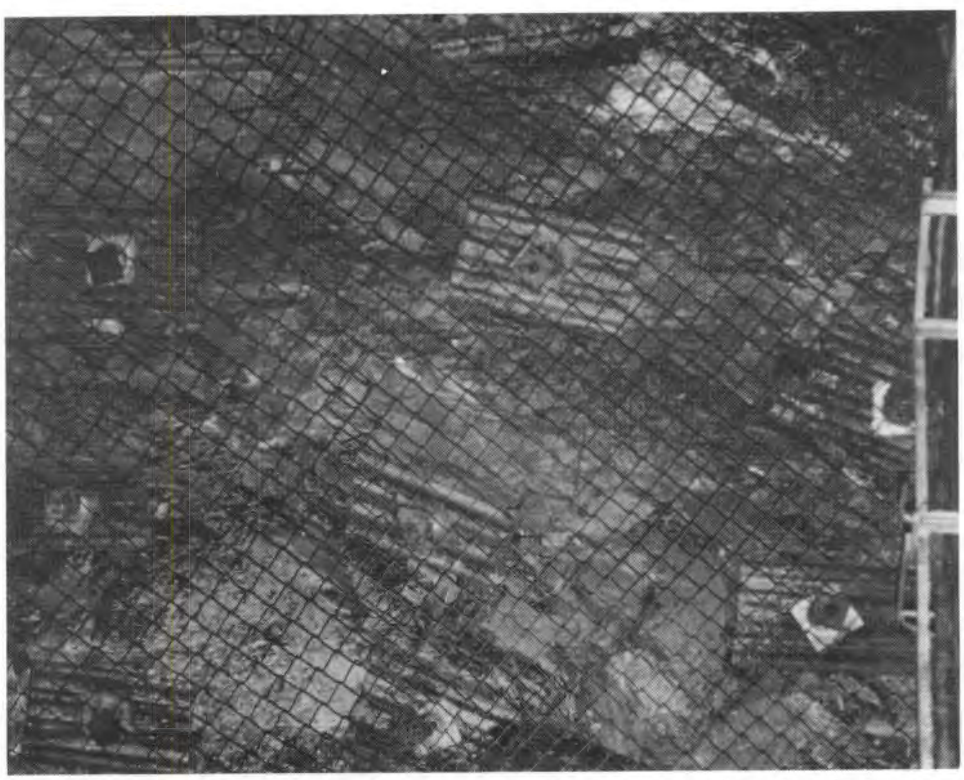


PLATE AIV-20. Wire mesh and rock bolts with flexible bearing plates

Despite the proximity of the reinforcement to the face, the bolts rarely loosened upon advancement of the heading. This was related to the careful control on blasting and continued loading of the reinforcement system with time and advancement of the face. The actual reinforced ground behaviour was determined by an instrumentation program, built into the construction cycle.

Four per cent of the total construction effort was devoted to monitoring ground behaviour. The majority of this effort went into the measurement of opening closure at stations spaced every 20 m along the tunnel profile. Each station was composed of 4 reference points installed in the wall, positioned so as to obtain the diametric closure. The reference point was simply a 1 meter length of steel rod fully grouted within a drilled hole.

Although it was of interest to record the amount of opening closure, presently between 5 and 35 cm, the rate of change of closure was the most significant parameter. On the basis of rate, the stability of the opening was ascertained along with the need for additional reinforcement. During the first week after excavation of the heading the closure rate was high. By the second week the rate normally decreased significantly approaching a steady state. Opening closure with time, and upon differentiation closure rate with time, were adequately described by the simple logarithmic decay

$$C = K \ln (1 + A t)$$

where C is the closure, t is time, K and A are constants. The range for the magnitude of the constants was derived from experience. If a particular station indicated an unusually high closure rate additional reinforcement was prescribed. A typical rate for a stable tunnel section was 1 mm per day after the first month and less than 0.5 mm per day after three months.

Closure rates were also used to determine the appropriate time for installation of the final concrete liner. Considering the maximum overburden, the 0.35 m thick liner could be damaged if placed before the majority of ground squeeze had occurred. If the ground was not allowed the necessary time to deform and mobilize shear strength, the liner would be forced to provide the resistance against ground deformation. To minimise this risk, a closure rate of less than 0.2 mm per day was required before placing the liner.

Measurements of opening closure were supplemented with data from multiple position extensometer installations. They indicate that approximately 70 per cent of all radial deformation or closure was restricted to ground within the first 2 m of the opening. It follows that the zone of major ground loosening was also limited to less than 2 m, most likely between 1 and 2 m of the opening.

Despite the amount of reinforcement installed, control on blasting, and instrumentation program the progress rates have not been adversely affected. The average rate of advance was two 4 m rounds per day or 200 m per month. In April 1978 the advance per month was up to 300 m.

LYON METRO, CREMAILLERE TUNNEL

With the exception of the 233 m long Cremaillere tunnel the Lyon Metro was constructed by cut and cover methods in soil. The tunnel construction presented special problems on account of the large section (7.3 m high by 10.4 wide), shallow depth of rock cover (2.5 to 15.5 m) and proximity to a densely populated region with very old structures. As the rock mass was a granite-gneiss, compressive strength 150 to 180 MN/m^2, the only practical method of excavation was by conventional drill and blast. Concern for the reduction of vibration due to blasting was necessary for damage and nuisance considerations as well as maintaining the integrity of the shallow rock cover. The rock margin between the massive granite-gneiss and the overlying alluvial deposits (3 to 15 m depth) was highly fissured and irregular in contour.

It is well known that the number of free surfaces adjacent to a region to be excavated by blasting influences the specific yield or consumption of explosives required. Furthermore, the quantity of charge per delay is directly proportional to the amplitude of the vibrations produced. In order to minimize the charge required and hence vibration, an additional free surface was created by the introduction of a pilot tunnel. Again, to reduce vibration a full face tunnel boring machine was employed for the excavation.

A 3.1 m diameter Wirth TB2H fitted with 20 multiple discs was purchased by the contractor. Average progress, 3.22 m per day (generally one shift), and cutting rates, 0.84 m/hr, were poor as would be expected for such a short drive. The main factor responsible for the low machine utilization, 25 to 45 per cent, was the delay related to ground support. Several zones of highly fractured and weathered granite necessitated the installation of steel sets as close to the face as possible. Severe space limitations between the machine and tunnel wall made the operation very difficult and time consuming.

Excavation for the full tunnel section was in four stages: two for the top heading which included the pilot bore, and two for the bench. On opening the top heading, steel sets were installed for immediate support followed by a concrete liner. Round length was limited to 1.5 m. This restriction was largely for stability considerations rather than vibration. Maximum velocity measured was on the order of 0.6 cm/sec, well below the 2 cm/sec specified limit.

One major fallout occurred while placing the steel set in the top heading. Unbeknown to the contractor, the contact between soil and rock was within several meters of the crown. Blasting the previous round apparently loosened the thin cover of very blocky granite starting a raveling process involving 300 cubic meters of material. Fortunately, the sink hole created at the surface did not involve any structures.

Remedial work consisted of backfilling the hole and prereinforcing the rock mass ahead of the top heading face. The pilot tunnel proved most useful for the latter operation. A total of 7 sets of 11 fully grouted dowels, 4 to 6 m long, were placed from within the pilot bore. Bolts were installed in a fan pattern over the future top heading (ie spiling reinforcement). On restarting excavation no additional stability problems were encountered.

OSLO SEWER TUNNELS

Geologically unique conditions in the Oslo region have made construction of the 34 km sewer tunnels ideally suited to tunnel boring machines. Bedrock is largely a massive shale and limestone formation with diabase intrusives. Above the bedrock is a clay layer up to several tens of meters thick. Many older buildings are founded on the clay which from previous experience has caused damaging settlements when allowed to consolidate (eg Oslo underground).

Aside from a short section, all excavation will occur within rock. Even though the majority of the rock formation is relatively massive, small included fissures produce excessive amounts of water. If damage associated with blasting loosened an additional one meter annulus around the 3 to 3.5 m diameter opening, the water inflow could be expected to increase by up to 70 per cent. This reflects the increased portion of surface area intersecting water bearing fissures.

Machine excavation would minimize ground disturbance and consequently, water inflow. It was also important to eliminate possibly damaging vibrations related to blasting, the main reason for specifying excavation by machine in the contract. The medium strength and low abrasive character of the rock formation coupled with the ability to leave the opening unsupported further favoured the use of machines.

As previously mentioned, the excavated opening would produce excessive water inflow even under ideal conditions. A specification defining the maximum allowable inflow was made variable depending on the location of the tunnel alignment with respect to the structures above and the height of rock cover. In sensitive areas the limit was 5 to 6 litres per minute per km of tunnel. To meet the requirement it was necessary to grout before excavation.

Considering the layer of clay and an overburden of around 35 m the prospect of grouting from the surface was not economical. The only reasonable solution, despite interference, was to grout at the face sequentially with excavation. A fan pattern with sixteen to twenty, 24 m long holes, equally spaced around the circumference was adopted. The pattern was repeated every 20 m providing 4 m of grout cover ahead of the face. Specific holes were pressure tested to estimate the water inflow and determine the grout requirements.

Chemical grouts were used in the shale with moderate success. At the maximum allowed pressure of 20 kg/cm^2 (285 psi) the quantity of grout delivered slightly exceeded the volume of the drilled holes. Despite the difficulties of sealing this rock the inflow requirement has been met. As would be expected, grouting operations in the limestone have been less troublesome. A less expensive mixture of cement and chemicals was used.

At present, three different contractors are working at four headings. Given the necessity to grout from the tunnel face and the requirement to excavate with a machine each contractor has produced a unique approach to the problem.

One contractor has employed a road header designed to operate in a full face mode. The French tunneller Bouygues TB 300C is a multiple point attack machine, Plate AIV - 21. Three separate arms fitted with 30 cm diameter discs oscillate within three concentric annuluses. Each arm is capable of up to 40 tons thrust and rotates at up to 48 RPM. The 5.6 M French franc (\$1.2 M) cost is considerably less than conventional full face machines, resulting in more favourable economics for shorter lengths of tunnel.

A typical construction sequence was to excavate for 20 m, pull the machine off the face, install a hydraulic drill rig to one arm of the machine, drill holes for grouting, pressure test, and grout. Approximately 2.5 shifts were used for excavation and 3.5 for grouting, an average of 33 m per week. The ability to obtain easy access to the face for drilling was the main advantage of the road header. It only

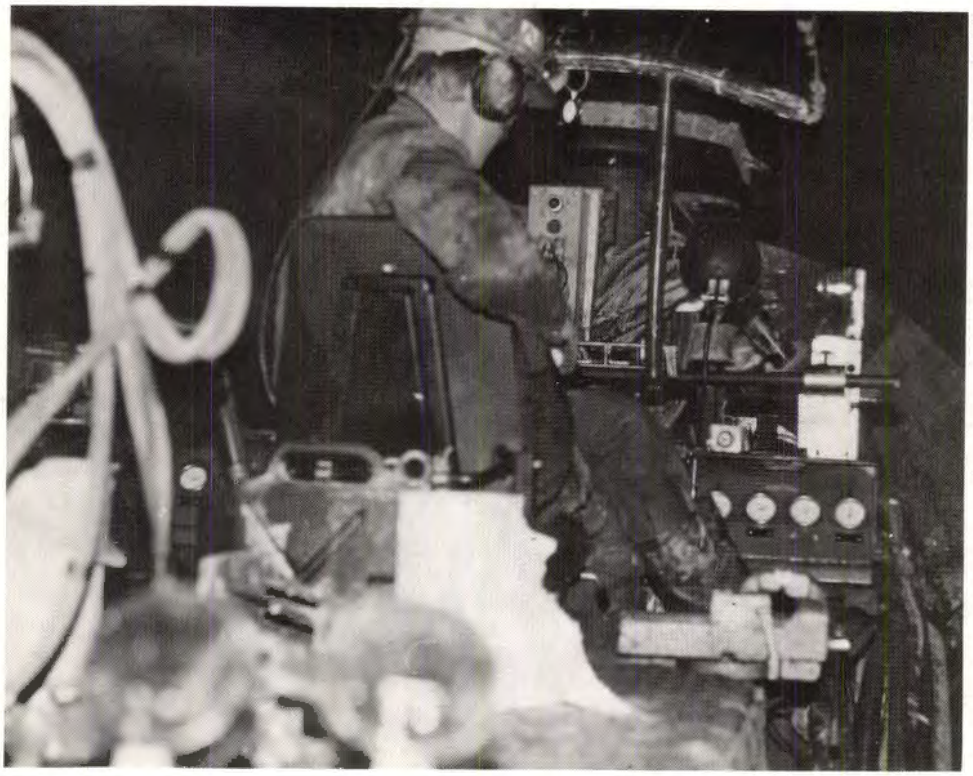


PLATE AIV-21. Bouygues TBM

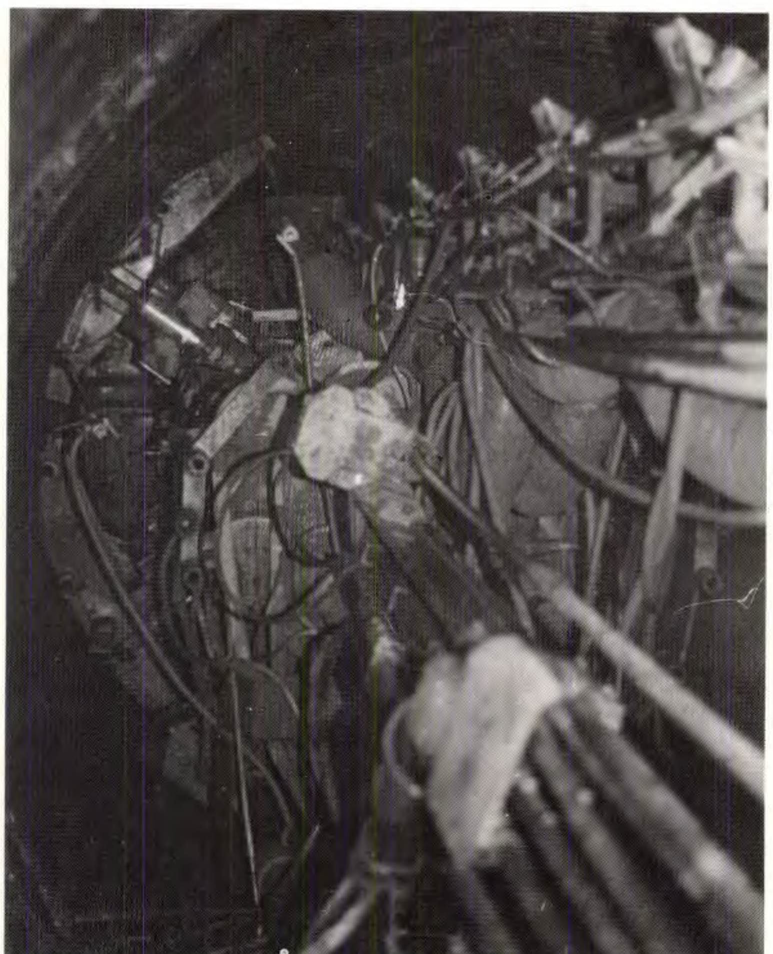


PLATE AIV-22. Hydraulic drill mounted on Wirth TBM required 1.5 hours to go from driving to drilling and provided the flexibility to employ any drill pattern or grout program desired. A disadvantage of the road header was the relatively slow average excavation rate of 1.25 m per hour as compared with conventional full face machines. A tunnel section completed earlier with a Robbins machine averaged 3.3 m per hour in similar ground. In general, a road header cannot excavate at the rate of a full face machine, but it is much more flexible.

A joint venture between a Norwegian and Swiss contractor had selected a 3.35 m diameter Wirth full face machine for their 7 km drive. Since it was not practical to pull the machine away from the face or work near the face, two hydraulic drill rigs were installed behind the last set of gripper pads, Plate AIV - 22. Ground water control was not as strict on this section of tunnel as for the previous contractor, consequently, a fixed pattern of 18 holes was adopted. The majority of holes were collared with four lengths of steel pipe installed within the base of each of the four gripper pads. Unfortunately, the back pads are 8 m from the face requiring that much additional length of hole.

On an average for the first kilometer, half the time was spent driving and half grouting. Excavation rate through the interbedded shale-limestone (less than 83 MN/m^2 compressive strength) has been 2.4 m per hour with 60 to 70 per cent utilization. The first set of 24 cutters were replaced after 700 m, most of which were double disc spaced at 4 to 5 cm.

Two full face Robbins machines have been purchased and modified by a second joint venture for the excavation of the two remaining 6 km tunnel sections. Modifications included the addition of two articulated arms for hydraulic drills, and increasing the stiffness of the cutter head support system.

Cutter head modifications were the result of similar changes made to a Robbins machine presently working on a hydro project in West Aurland, Norway. Much of the rock along the tunnel line is a massive high strength quartzite-gneiss with a quartz content between 30 and 40 per cent. Historically, Robbins machines have had difficulties in high strength rock, primarily low penetration and high cutter wear. Most of these problems are related to cutter head vibration or head stiffness. By increasing the stiffness of the head support pads (banana jack) with a larger steel section and one way action hydraulic pancake jacks much of the vibration was eliminated. Lateral vibration produces excess cutter wear and damages bearings, particularly those of the central cutters. Axial motion reduces penetration and is also detrimental to cutter life by increasing peak loads. With the improved system the excavation rate has averaged up to 1.8 m per hour at normal thrusts of 14 to 15 tons per cutter. Cutter wear in terms of cost has been moderate at 400 to 500 Norwegian kroner (about $80) per meter of tunnel or about $10 per cubic meter.

Both articulated arms for the drills are mounted on the underside of the main support frame between the operator's cabin and gripper pads, Plate 3 - 5 (page 136). This system will allow the grout holes to be collared within 2.5 m of the face. Since the machine has yet to start excavation, the overall system performance is not available. Estimated rate of advance is 4 m per hour with 70 per cent utilization.

Modifications made to the full face tunnelling machines to accommodate grouting operations ahead of the tunnel face have obvious application to questions of ground stabilization. If the ground near the face or ahead of it has too short a stand-up time to allow for normal excavation and support operation behind the face the ground could be prereinforced. This can either be accomplished by grouting, just as for water control, or by installing fully grouted steel dowels within the predrilled holes. The latter method, spiling reinforcement, has been shown to be effective in increasing opening stability (Korbin and Brekke, 1976 and 1978). If the ground is suitable for grouting, a fast setting cement or a chemical grout such as "magnesiabinder"

(100 kg/cm^2 compressive strength after 12 hours) can be used. This is likely, however, to be more expensive and slower than employing reinforcement. A system designed to allow for prereinforcement would be justified if more than 10 or 20 per cent of the tunnel was estimated to have stability problems near the face.

PAIJANNE WATER TUNNEL

When completed in 1982 the 120 km Paijanne water tunnel will be the longest continuous rock tunnel in the world. The 15.5 square meter horseshoe shaped section is being driven by drill and blast through relatively massive granite. Although the granite is slightly fractured in some regions, requiring shotcrete support, the majority of the tunnel length will be unsupported and unlined. With an overburden between 30 and 130 m, access tunnels to the upstream and downstream headings constructed by various contractors are spaced at distances of less than 10 km. Average cost for the tunnels at 1976 prices was about 2 million Finnish marks ($500,000) per km. Excavation started in 1973.

In one representative tunnel heading the average advance was five to six 3.5 m rounds in two 8-hour shifts or 1.25 m per hour. For a typical working week this amounted to 100 m of tunnel, a very good progress rate even by tunnel boring machine standards. Considering the rock type, size of work force (5 men per heading), number of headings, and rate of advance, a full-face machine was not likely to be economical as compared with the present operation.

The entire excavation cycle was well organised and as mechanized as possible. Hydraulic drill jumbos have increased drilling rates by over 50 per cent, furthering the economics of a smooth final line. Mechanization of the scaling operation has proved somewhat troublesome. A boom mounted hydraulic hammer or pick was found to remove too much material. Consequently, the head of the drill rail or poles were employed in traditional fashion.

As a result of the relatively massive character of the ground and the judicious use of internal support, only 7 per cent of the total water tunnel cost was for ground control. When required a layer of shotcrete between 6 and 12 cm thick was applied. The corresponding cost was 50 and 70 Finnish marks ($12 and $17) per square meter.

Adding wire mesh between two layers increased the price by approximately 50 Finnish marks ($12). The mesh was attached by small L-Shaped tie wires partially embedded within the first layer of shotcrete shortly after application. Payment was based on the average thickness of 10 core samples for every 200 square meters of supported surface area.

PFANDER ROAD TUNNEL

Started in February 1977, the 6.7 km Pfander tunnel, located near the Austrian-Swiss border, is expected to be operational in 1981. The cross section, 82 to 94 m^2, general layout and construction method are similar to the Arlberg road tunnel. It is the fourth Austrian road tunnel to be constructed by the "New Austrian Tunnelling Method" (NATM). Construction cost is estimated at 825M Austrian schillings ($55M).

Ground encountered along the tunnel alignment is a dipping sedimentary sequence of conglomerate (compressive strength 600-1000 kg/cm^2), sandstone, mudstone and marl (200-450 kg/cm^2). Aside from bedding planes and minor clay filled seams, the rock is relatively massive and has presented few problems to date. Ground squeeze in the weakest marl has been limited to less than 10 cm on account of the medium overburden, 350 m maximum. The use of NATM as a stabilization measure has largely prevented weathering of the mudstone and marl as well as controlled ground deformations. Considering the minor ground squeeze control measurements (horizontal convergence and crown settlement) were established at 50 m intervals. Instrumented reinforcement has indicated tensile forces between 7 and 20 tons with peak force positions near 1 and 2 m for the conglomerate - sandstone and marl respectively.

Excavation of the horseshoe-shaped opening was by drill and blast in full section. Two truck mounted hydraulic drill jumbos were employed for drilling the 2 to 4 m rounds and installing ground reinforcement, Plate AIV - 23. In the competent conglomerate-sandstone advance was 12 to 16 m per day and in marl, 6 to 8 m per day (based on three 8-hour shifts). Maximum progress to date has been 400 m per month.

Support measures were divided into six classes depending on rock type. Typically, after excavation and scaling a 4 to 5 cm thick layer of shotcrete was applied. Small wire ties were embedded in the shotcrete from which wire mesh as attached. After a cover layer of shot-

PLATE AIV-23. Drilling for next round

PLATE AIV-24. Pilot tunnel near invert of main heading crete, 7 to 20 cm, rock bolts were installed. Bolt lengths varied from 3 to 7 m and density from 8 to 119 m per meter of tunnel. Between the shotcrete and 25 cm thick concrete liner (specified thickness) a waterproof insulation was installed. It consisted of a 4 mm thick polypropylen sheet and 1.5 mm of polyvinyl chloride (PVC). The polypropylen was placed against the shotcrete to protect the PVC layer.

In addition to the main excavation, 8 m diameter ventilation shafts were constructed with a 1.5 m diameter raise bore and further enlarged from top to bottom. The 230 m deep shaft was enlarged by drill and blast at a rate of 1.5 to 3 m per day. For the first 200 m of the 315 m shaft a road header or point attack machine was employed for enlargement. The machine was held in position with gripper pads similar to those used on full face tunnel boring machines. Unfortunately, the overbroken sidewalls made it difficult to hold and on one occasion the machine fell 3 m. Strong sandstone further reduced progress. It required 10 months to enlarge the first 180 m, shortly after which, the machine was removed and work completed by drill and blast.

Site investigation for the Pfander tunnel included a 3.6 m diameter pilot tunnel driven by two full face tunnel boring machines, Plate AIV - 24. A pilot tunnel was considered advantageous for: reducing geologic uncertainties, estimation of support measures, ventilation shaft construction and ventilation of the main excavation. Both the Wirth and Robbins machines achieved maximum advance rates of 40 m per day and 450 m per month (5 day week). The Robbins machine provided greater access near the face for ground control in the mudstone and marl, however, the Wirth was easier to keep on line. Alignment was critical, as it was originally planned to enlarge the main excavation to full section with a reaming TBM. This machine uses the pilot tunnel for forward thrust, and consequently, is centered by the pilot. Only fiber glass rock bolts and mesh along with wood lagging were employed to stabilize the opening. Occasionally, the fiber glass mesh was not strong enough to contain loosened material between bolts. Total cost of the pilot tunnel was 90M Austrian schillings ($6M) or 11 per cent of the main excavation cost.

In considering the main excavation, all six joint ventures submitting bids tendered both full face machine and drill and blast methods of construction. Five of the six tenders revealed considerably lower cost by conventional methods despite allowances made for reduced ground disturbance on excavation by machine. This included a 40 per cent reduction in reinforcement and approximately 4 cm decrease in shotcrete thickness. Factors adversely influencing the cost of the road tunnel when excavated by full face machine were the more expensive circular cross section and the additional conventional excavation required for mechanical-electrical systems and vehicle turnouts.

SELLRAIN-SILZ HYDROELECTRIC PROJECT

Sellrain-Silz hydroelectric project in Austria is like many multifaceted water power schemes under construction in Europe. The two power stations have a total output exceeding 700 MW, one of which has a 720 million kWh pumped storage capacity. Of the 40 km of water tunnel only 9 km link power station turbines to reservoirs. The remaining tunnels convey water from numerous mountain valleys increasing the total catchment basin area to 140 square km. Because of the great length of tunnels required in the relatively short 4-year construction period, several fullface tunnel boring machines were employed. In addition, the circular section of full-face machines is hydraulically efficient and characteristically produces less overbreak, requiring less concrete for the final liner.

One of the catchment basin tunnels is being driven with a 3.8 m diameter Robbins full facer. The nearly-completed 4.5 km horizontal drive traverses a massive muscovite-granite-gneiss of medium strength. Without water inflow problems or the need for ground support, conditions were good for high rates of advance. Working three 8-hour shifts per day the overall average rate has been 35 to 36 m per day with a maximum of 67 m.

Construction of steeply inclined tunnels is one of the most dangerous and difficult types of underground work. The advantages of increased safety, not considering the increased production rates, has made full-face machines very popular in this area. Two examples were the head race tunnels for the two power stations.

On one heading, a 3.2 m diameter Wirth machine was used to construct the 2 km long, 39 degree inclined shaft. For excavation of the massive biotite-granite-gneiss and mica schist the machine head was fitted with strawberry button cutters. The heading was without support or water problems resulting in relatively high excavation rates considering the type of cutters employed. Average rates per hour were on the order of 1.2 m, occasionally up to 1.4 m.

The second inclined tunnel, at 23 degrees from the horizontal, also employed a Wirth machine only of larger diameter, 4.8 m. Payment for the 1 km drive was based on the percentage of support and hornblende within the rock mass. A schistose gneiss requiring no support with less than 10 per cent hornblende was the lowest category at 14 000 Austrian schilling (\$1000) per meter of tunnel. From 10 to 70 per cent hornblende the rock mass is described as a hornblende gneiss requiring a slight increase in payment. Above 70 per cent, an amphibolite was the top of the scale at 30 000 Austrian schilling (\$2000) per meter. Although hornblende, Moh's hardness 5 to 6, is not as abrasive as quartz the quantity in the rock had a large influence on cutter wear. With 30 cutters on the head at an average of over 6370 Swiss franc (\$3350) each, the question of wear largely influences the economics of the drive. Average cutter costs based on the first 700 m including depreciation ranged between 18 and 22 Swiss franc (about \$10) per cubic meter excavated.

Even though machine utilization was high, 70 to 90+ per cent, the rates of advance were low. In the schistose gneiss the average rate per hour was 0.6 to 0.7 m and in hornblende gneiss 0.4 to 0.35 m. This is equivalent to 0.7 to 1.5 mm penetration per revolution of the head Increasing the forward thrust above the normal average of 10 tons per cutter did not significantly increase penetration.

Aside from the low penetration rates, overall advance was further slowed by 1 m wide zones of mylonite within the otherwise massive schistose gneiss. They often led to fallouts close to the face and on one occasion ahead of the face resulting in a 3 m high by 6 m long overbreak. It was extremely difficult to remedy the problem on account of lack of working space near the face coupled with the incline of the machine. With great effort rock bolts and wire mesh or steel sheet were installed to control the ground.

Several different types of liner system will be used in the head race tunnels. Close to the water inlet, where pressure is low, the opening will simply be lined with concrete. For sections within 300 m of the power station, or where the pressure exceeds 400 m of head, the system is to be composed of an internal steel liner backfilled with concrete. The most interesting system is the transition section between the high and low pressure regions. One particular system consists of a prestressed concrete ring within a thin walled steel tube. The sequence of construction is to install the steel pipe, backfill the pipe with gravel, form and place the concrete liner within the pipe, and after curing, inject the backfill with cement grout. Injection pressures, as high as 50 bars (735 psi), will precompress the liner against internal water pressure and pre-load the rock nearest the opening.

Pre-loading the rock serves to increase the stiffness of the rock mass within the immediate vicinity of the tunnel, thus allowing for an increased contribution to the total system. As plate bearing tests reveal, the ground nearest the opening has relaxed resulting in an initial lower stiffness at low bearing pressures. With greater bearing pressure the small cracks and fissures are closed and stiffness increases considerably. By this procedure the liner and rock mass are made to act as one unit to resist the internal water pressure.

UNDERGROUND DESIGN AND CONSTRUCTION PRACTICE IN SCANDINAVIA

Scandinavia is formed by a massive Precambrian shield. With the exception of a portion of Norway, the dominant rock types are various competent gneisses and granite of medium to high strengths. Extensive and diverse use of underground space, since the early part of this century, has resulted in a history of experience. For example, in Norway over the past two decades 150 km of tunnels have been excavated annually for hydro power schemes (Selmer-Olsen and Broch, 1977). Finland has at present an underground storage capacity in excess of 9 million cubic meters. The massive character of the ground coupled with the construction experience has resulted in a hard rock tunnelling industry unique to Scandinavia.

Although the basic design considerations for underground openings are similar to those anywhere in the world, the role of numerical methods for the calculation of stress state, opening deformation or stability is of minor significance. This can be largely attributed to the ability to estimate performance from experience, thereby reducing the need for such analysis when balanced against the realistic predictive capability of most models. Numerical models, however, have been employed to determine the possible influence of high in-situ horizontal stress on large caverns or of steep valley sides. If problems related to stress can be identified at an early stage the systematic reinforcement expedients can be carried out conveniently during excavation rather than as remedial work (Anttikoski and Saraste, 1977).

Aside from specific problem areas, the majority of underground openings remain unsupported and unlined. This is not purely related to the favourable rock mass character, but to a philosophy which defines the problem in the context of the purpose for which the chamber or tunnel is to be used. For example, in a hydroelectric project the tail race tunnel has the highest factor of safety against fallouts since it would be the most difficult to drain and repair. The head race tunnel has a moderate factor of safety due to its importance while water collection tunnels external to the main system are designed to tolerate small failures. A block or wedge of rock less than a cubic meter in size will not adversely affect performance considering that most collector tunnels are dimensioned for construction convenience rather than hydraulic requirements.

Most problem areas are localized shear zones and seams of less than 10 m in width. They can be associated with a specific fault or a dike margin. Pegmatic regions high in potassium feldspar may be altered to the swelling clay, montmorillonite. These areas are often stable during excavation, but later ravel or fallout as the ground swells with loss of strength. A combination of shotcrete, wire mesh, or rock bolts are often employed to stabilize the ground. For permanent support, large shear zones are concrete lined with a two piece steel form or shield. Placement of a liner after excavation increases the construction cost by two to three times the unsupported amount.

In problem areas with short stand-up time the steel form will be installed at the face for liner placement concurrent with excavation. This operation adversely affects progress, increasing the construction cost by four to five times the unlined price. Occasionally, matters are further complicated by material falling on to the form prior to placement of the concrete. As an alternative, shotcrete can be applied, however, if the ground contains a large portion of clay or there is a significant water inflow it does not adhere. The U.S. tunnelling practice of employing steel sets and forepoles would be a logical solution. Since there is no tradition for the use of steel sets in Scandinavia this has not been attempted.

Openings with stress induced failure of the rock either as violent rock burst or the more passive rock slabbing are reinforced. A short length bolt, about one meter, is used on a regular pattern for control of the specific area involved. This surface or skin bolting does not stop the failure, but inhibits the progressive ravelling that would otherwise occur.

Excavation is largely by drill and blast employing typically three and up to four tunnellers per heading. Truck mounted hydraulic drill jumbos are popular due to high output and improved working environment, Plate AIV - 25. There is a substantial reduction in noise and practical elimination of all fog and dust compared to pneumatic drills.

Careful blasting techniques to reduce disturbance and overbreak are common. It is believed to be a good investment as compared with the expense of reinforcing damaged rock, increased scaling, and additional loading and transportation of overbreak material. The drilling pattern usually consists of a 100 to 200 mm diameter "burn hole" or several smaller unloaded holes. Final line of the opening is smooth blasted with a specific explosive consumption of 0.25 kg per cubic meter. Presplitting the final line is less frequently used as it is believed to cause increased disturbance by the opening of joints with trapped gas.

Construction of large underground caverns basically employs the same equipment and techniques as for tunnels only the excavation proceeds in stages. The size of the heading and benches are governed by equipment and the economics of drilling and blasting.

Brofjorden underground oil storage chambers cross section of 600 square meters are excavated in three stages. The 7 m high by 20 m wide top heading is driven full face, Plate AIV - 26, and subsequently, supported with several centimeters of shotcrete for safety. This is followed by an 8 m horizontal bench and another 15 m vertical bench. Changing from a horizontal to a vertical bench has the advantage of reducing the amount of drilling and specific explosive consumption. Overall averaged drilled meters per cubic meter and explosive consumption are both estimated at 0.6.

Under favourable conditions large underground storage projects have a capacity to excavate between 2000 and 3000 cubic meters per day. Costs usually range from $25 to $50 per cubic meter depending on the amount of ground support and grouting required. Unsupported and unlined

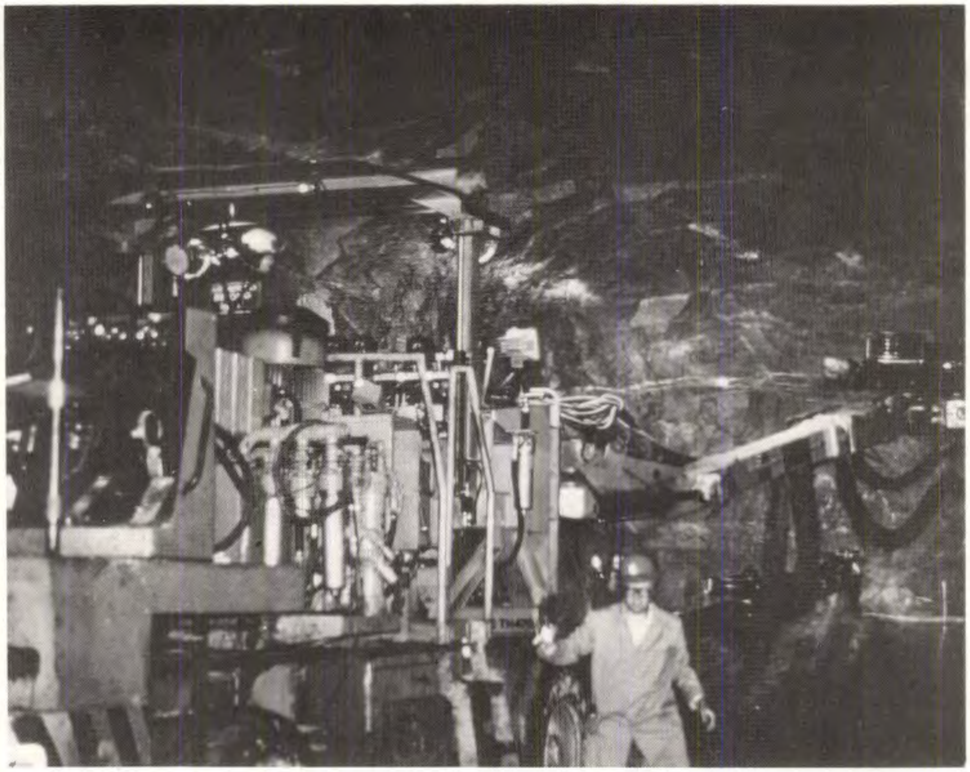


PLATE AIV-25. Truck mounted hydraulic powered drill jumbo

PLATE AIV-26. Top heading of Brofjorden oil storage chamber road and water tunnels are also of similar cost only closer to the lower bound.

Road tunnels differ from other underground openings in that they are often required to be water free. Water leaks accelerate deterioration of the road surface, decrease visibility and form ice in winter. The usual approach to solve these problems has been to place a double concrete liner with a waterproof layer in between. This is not economical when the openings rarely need support as in Scandinavia. In Norway, an economical system using corrugated aluminium sheets has proved successful for water shielding (Gronhaug, 1976). Where frost is a problem a built-up section of two aluminium sheets filled with a 100 mm thick layer of insulation prevents freezing and allows the water to be collected.

VICTORIA COLLIERIES

To expand present works and consolidate many smaller units, the German coal mining industry has undertaken a program designed to link many collieries together. It requires many tens of kilometers of development tunnel in a relatively short period of time. Realizing the limitations of traditional methods of excavation the industry has employed numerous full face tunnel boring machines. Since the early seventies over 25 km have been excavated at an average advance of nearly 12 m per day (Henneke, 1978).

Work being carried out at the Victoria Collieries was typical of present excavations. Operating in medium strength coal measures rock, the 6.1 m diameter Demag has completed 3 km of the 5.9 km drive. Average advance for the first 2.6 km was 13.1 m per day (three 6-hour work shifts, one 6-hour maintenance) and 20 m per day over the last 400 m. The maximum rate in any one day has been 30.2 m. Overall averages were low compared to machine capability on account of low utilization. This was primarily related to inadequate debris removal (limited hoist capacity) and second to installation of support. It took two labourers 20 minutes to install one steel set, however, it only required 12 and 20 minutes to excavate one 0.75 m stroke in shale and sandstone respectively. Set spacing was varied between .75 and 1.5 m, the latter being the maximum machine stroke or shove.

With a 1.1 km overburden, much of the ground nearest the opening exhibited some degree of stress or deformation induced spalling and squeeze. To stabilize the heading yieldable circular steel sets with heavy wire mesh lagging were placed immediately behind the cutter head concurrent with excavation. The segments forming a complete ring were assembled with the aid of a special erection system. Once assembled the set was jacked into place with an 8 ton preload. This entire operation occurred 1.7 m behind the face, reducing the unsupported span to an absolute minimum.

There was little question that without the ability to install support near the face additional time would have been lost on ground control. Occasionally, overbreak would occur at or ahead of the face requiring the use of shotcrete to stabilize the heading. In one situation a fractured zone was grouted from the face prior to excavation.

The cutter head employed a total of 33 triple discs. Normal operating thrust was 400 tons with a maximum 600 ton capacity. All cutter maintenace was subcontracted to Demag on the basis of rock type per cubic meter excavated. For shale the cost was 15 marks ($7.5) and for sandstone 40 marks ($20). Dust supression, aside from improving the environment, has been found to increase cutter life, most likely by cooling the metal discs.

Tunnel cost per meter was 5,750 marks ($2,900) for the 5.9 km drive based on 13 m per day, employing a work force of 11 to 12 per shift. A nearby conventional drill and blast operation with a similar ground, depth and size of opening was averaging 5 m per day at a cost of between 6 and 7 thousand marks (about $3,500) per meter excavated.

Several additional excavations are planned for the near future.

☆U.S. GOVERNMENT PRINTING OFFICE: 1979 O—631-873/9 REGION 3-1