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GUIDELINES FOR ROCK CUTTING PERFORMANCE PREDICTION USING ROCK MASS PARAMETERS

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ABSTRACT

The efficiency of access boring machines must be scientifically described. Until now, even though some of these machines have been used in hard rock settings, little research has been carried out in relation to machine performance aspects.

This report presents guidelines to predict access boring machine performance based on geomechanical aspects and machine advance data. Aspects related to intact rock (small scale), rock mass (large scale) and the combination of the two are discussed. A-priori data collection and validation of predictive models are discussed in relation to various performance elements to be evaluated.

KEYWORDS: Rock mass effects, intact rock effects, rock type, in-situ stresses, Schmidt hammer tests, joints, total hardness, critical energy release rate, crack formation, unconfined compressive strength, rock mass classification, access boring machine performance.

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LIGNES DIRECTRICES POUR LA PRÉDICTION DU DÉCOUPAGE DE ROC BASÉE SUR LES PARAMÈTRES DE MASSIFS ROCHEUX

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RÉSUMÉ

L'efficacité des excavatrices souterraines doit être décrite scientifiquement. Jusqu'à date, même si quelques unes de ces machines ont été utilisées en roches dures, très peu de recherches ont été faites au niveau de leurs performances.

Ce rapport introduit des lignes directrices pour la prédiction de performance de machines excavatrices basée sur les aspects géomécaniques et les données reliées à l'avancement de l'excavatrice. Les aspects reliés au roc intact (petite échelle), au massif rocheux (grande échelle) et aux combinaisons des deux sont discutés. La collecte de donnée, a-priori, et la validation de modèles de prédictions sont discutées en relation avec divers éléments de performances à être évalués.

MOTS CLÉS: Effets de la masse rocheuse, effets du roc intact, type de roc, contraintes en place, essais de marteau Schmidt, diaclases, dureté totale, taux de libération d'énergie, formation de brisures, résistance à la compression uniaxiale, classification de massif rocheux, performance d'excavatrices.

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CONTENTS

INTRODUCTION

The efficiency of access boring machines (TBM, road header, raise borers, etc.) is directly related to the rock properties encountered. Although these machines have been used in the Canadian Shield, there is still no scientific study available describing performance of such equipment in terms of rock mass parameters.

The "Foreuse Roger" has been used in mine trials, at the Belmoral, Bousquet and Kiena mines. This report outlines the cross section of rock mass parameters expected to affect the excavation efficiency of this boring machine. This consideration is based on its future application, for creation of mine level openings and/or stopes openings, in various rock types.

GENERAL SCIENTIFIC CONSIDERATION

In evaluating rock cutting machine performance, the scale of the cutting efficiency should be taken into account. Two scales are recognized: the rock mass (large) and intact rock (small) environments. The first relates to the influence of discontinuities, (orientation, density), large scale weaknesses, and stresses. The second relates to texture, mineralogy and their effect on abrasivity, unconfined compressive strength, tensile strength, modulus of elasticity and Poisson's ratio. The performance of these machines, for a given site, is related to the relative presence of the large scale effects: i.e. there will be a more difficult passage in solid, unfractured rock, compared to the same rock when suitably fractured.

Uniaxial compressive strength is the most widely used small scale parameter for predicting the performance of tunnelling machines, but it has been shown [1] that it may not be the most significant rock property available for predicting drillability in soft to medium rock formations. Additionally, results based on small sample size are not reflective of in-situ intact rock strength. There are several rock types which need specialized testing to obtain the correct parameters: schists, altered rock, weakly bedded lithologies.

There have been a number of non-Canadian case studies which have used other parameters to gauge the effects of large and small scale rock mass features.

With respect to the large scale, rock mass properties have been quantified to allow more accurate prediction of tunnelling machine performance. The Schmidt reduction index (related to Schmidt hammer measurements) used in the field, has been found to be related to excavation rates, while increases in these rates have been attributed to the fractured state of the rock mass [2].

With respect to the small scale, intact rock properties have been used to evaluate overall TBM performance [3] and cutter performance [4]. The conclusions of the former were that for massive, brittle materials, the critical energy release rate (a measure of the energy required to create new surface area):

$$G_K = \frac{K_{IC}^2 \left(1 - \nu^2 \right)}{E}$$

K_{IC} = fracture toughness
ν = Poisson's ratio
E = modulus of elasticity of intact rock

correlates very well with material total hardness:

$$H_T = H_R \sqrt{H_A}$$

H_A = Taber abrasion hardness
H_R = Schmidt hammer rebound hardness.

and machine advance rates.

The theory of positive rock-cutter tool interaction depends on the capacity of the rock to crack (in tension) and chip; such analysis has been covered on a theoretical and laboratory study basis [4]. In particular, penetration rates were found to be directly related to tensile strength of rocks, which are greatly affected by anisotropy orientation, e.g. much greater advance perpendicular to schistocity.

Several other physical properties usually obtainable from lab tests have been shown to influence penetration rates [1]. Bulk density, uniaxial compressive strength, apparent porosity, P-wave velocity and Schmidt hammer index have a direct relationship to penetration rates.

As the size of the machine produced chips increases, the new surface area created will diminish and less energy will be consumed. The presence of fractures extending through cutter paths will be beneficial to this situation.

The size of rock chips produced in boring machine excavation is one element that has been used to compare excavation efficiency and to optimize the performance of these machines. But it is only one side of the evaluation of performance. There are three steps to follow for this: first a performance model or models should be established for site specific material and machine used; second, geomechanical data, lab and field, must be gathered for input into the performance models established. Lastly, machine performance is assessed using as many machine activity indicators as possible, versus site conditions.

FORMULATION OF WORKING MODELS, APPLICATION OF VARIOUS PARAMETERS AND PERFORMANCE EVALUATION

Models, by their inherent nature, use a limited number of parameters. Depending on the scale at which the performance of the excavating machine is viewed, three model types are possible:

1) empirical;
2) analytical;
3) combined scale effects.

Empirical Models

Each empirical model uses a limited number of parameters. These parameters are not analytically based, but rather depend on visual or relative evaluation of in-situ, large scale conditions. Three levels of model complexity are presented here.

On the simplest level, individual parameters serve as models for machine performance; they depend solely on discontinuity characteristics:

RQD (1)
Fracture density (2)
Orientation (3)

An alternate way of assessing the combined effects of (1)-(3) would be to apply systematic and wide coverage Schmidt hammer tests as Young and Favell did with success [2].

In the case of a mass where plasticity of matrix material (schist, gouge, alteration product etc.) dominates, a scaled mass behaviour parameter ranging from very broken to mostly altered rock, here introduced as relative mass plasticity (Prm), can be used:

$$P_{rm} \propto kc \qquad (4)$$

c = material cohesion
k = disaggregating factor (k = 1 solid rock, k = 0 granular material).

A more complex level incorporates the effect of the existing stress field and its orientation/effects on the opening created. These can only be estimated versus joint family orientation and depth. In the best situation, the machine will advance parallel to a major joint family and parallel to the major principal stress. At worst, the joint family will be normal to machine advance and principal stress direction. In the latter scenario the blocks will not readily be plucked out, or helped by gravity.

Of course, the ideal situation exists when the stress field is too low to prevent block movement, but high enough to provide temporary opening stability. For joint family i, of orientation factor j (representing 0 to 180° to machine heading), and stress level (σ), and orientation (α) (0 to 180° to machine heading) of the major principal stress, the relationship to compare to machine performance is:

$$F_{ij} \ \sigma_1 \ \alpha_1 \qquad (5)$$

The third level empirical model incorporates several mass characteristics: brokeness, ease of block removal, helpful/detrimental effects. These can be found in some fashion in the NGI rock mass classification system [5]:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF} \qquad (6)$$

The Q value and the way it is calculated are not obviously set up for comparison to tunnel boring performance. Yet a low value from Q, especially if obtained primarily from the first two quotients, will be beneficial. The value for Q should be high enough to permit temporary self support of the opening. This situation is portrayed in Figure 1.

The first quotient calculates the effective creation of a discontinuous and blocky mass. The second addresses the potential for movement along joints. The last relates to environmental factors affecting the stability of the mass in a block movement sense: stress level and water inflow. Typical values are reported in Table 1.

High values for each of these quotients would not be beneficial for tunnelling machine performance.

By itself, the first quotient could be used as a first level empirical model of performance.

Analytical Models

Analytic models are based on simple theoretical relationships or single parameters. Unfortunately, since large scale behaviour does not fit elastic theory for example, analytic models apply only to intact rock characteristics. Thus, only the potential for evaluating rock breakage/chipping depending on strength parameters of intact rock can be evaluated, depending on strength parameters.

Each of the following rock properties can be used as a basis of performance comparison and are readily available from lab tests:

σ_c - unconfined compressive strength (6)
H_T - total strength (7)
G_c - energy release rate (8)

Combined Scale Effects Models

Any given rock face being bored presents a mixture of small and large scale effects. The end members will be on the one hand solid, undissected or unaltered rock, which is a worst case situation for machine advance, and totally disintegrating material requiring little cutting energy, only that necessary to make the mass crumble (assuming self-support of the opening, permitting the machine to function).

It is a question of combining the effects of the large and small scale for the entire path of the machine. The relative importance of each scale will vary according to the type of terrain, Figures 2 and 3.

Source of Data

There are two methods of collecting data to predict/explain machine performance. The first is to diamond drill holes along and in the volume to be excavated. A minimum of four holes, in each quadrant, are recommended, Figure 4, but any number can be used to cover the cross sectional area of the excavated face, Figure 5. These holes are drilled before the machine progresses along its designated path.

If these drill holes are parallel, then one or several holes angled to these is(are) necessary to pick up properties of joints which may parallel the holes drilled along the path of the machine.

Information from drill core discontinuities can be used for RQD, NGI and joint arrangement properties to describe large scale characteristics. The core itself can be used to perform tests yielding the small scale parameters.

The second method, the Schmidt hammer tests, has to be systematically performed, as per a grid system, Figure 6. But this requires stoppage of the machine, withdrawal of the cutters from the face and time allotted for completion of tests at the face.

Validating the Models

The last step in the process of establishing machine performance involves measurement of particular parameters to yield quantitative measure of performance.

Four elements can be used in such evaluations:

- machine advance rate
- cutter abrasion
- granulometry dispersion of the boring activity
- cutting head contact pressure (thrust).

Machine advance rate should be calculated for each geomechanically distinct rock mass zone. Cutter abrasion is performed on a relative scale and requires machine stoppage. Granulometric dispersion analysis must be based on large sample volume.

CONCLUSIONS

This report presents new and standard methods of measuring excavating machine performance. It is essential, however, to separate the program into three steps: data gathering, prediction of performance based on large and small scale rock mass parameters and validation of models thru machine performance.

In-situ tests (before and during excavating activity) and lab tests must be performed.

REFERENCES

1. Howarth, D.F., Adamson, W.R. and Berndt, J.R. "Correlation of model tunnel boring and drilling machine performances with rock properties"; Int. J. Rock Mech. Sci. & Geomech. Abstr.; 23 #2; pp. 171-175; 1986.

2. Young, R.P. and Favell, R.J. "Assessing rock discontinuities"; Tunnels and Tunnelling; June 1978.

3. Nelson, P.P., Ingraffea, A.R. and O'Rourke, T.D. "TBM performance prediction using rock fracture parameters"; Int. J. Rock Mech. Sci. & Geomech. Abstr.; 22 #3; pp. 189-192; 1985.

4. Sanio, H.P. "Prediction of the performance of disc cutters in anisotropic rock"; Int. J. Rock Mech. Sci. & Geomech. Abstr.; 22 #3; pp. 153-161; 1985.

5. Barton, N., Lien, R. and Lunde, J. "Engineering classification of rock masses for the design of tunnel support"; Rock Mech.; 6; pp. 189-236; 1974.

1. ROCK QUALITY DESIGNATION (*RQD*)

A.	Very poor	0— 25
B.	Poor	25— 50
C.	Fair	50— 75
D.	Good	75— 90
E.	Excellent	90—100

Note:
(i) Where *RQD* is reported or measured as ≦ 10 (including 0) a nominal value of 10 is used to evaluate *Q* in Eq. (1)
(ii) *RQD* intervals of 5, i. e. 100, 95, 90, etc. are sufficiently accurate

2. JOINT SET NUMBER (J_n)

A.	Massive, no or few joints	0.5—1.0
B.	One joint set	2
C.	One joint set plus random	3
D.	Two joint sets	4
E.	Two joint sets plus random	6
F.	Three joint sets	9
G.	Three joint sets plus random	12
H.	Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15
J.	Crushed rock, earthlike	20

Note:
(i) For intersections use ($3.0 \times J_n$)
(ii) For portals use ($2.0 \times J_n$)

3. JOINT ROUGHNESS NUMBER (J_r)

(a) *Rock wall contact* and
(b) *Rock wall contact before 10 cms shear*

A.	Discontinuous joints	4
B.	Rough or irregular, undulating	3
C.	Smooth, undulating	2
D.	Slickensided, undulating	1.5
E.	Rough or irregular, planar	1.5
F.	Smooth, planar	1.0
G.	Slickensided, planar	0.5

(c) *No rock wall contact when sheared*

H.	Zone containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)
J.	Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0 (nominal).

Note:
(i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m
(ii) $J_r = 0.5$ can be used for planar slickensided joints having lineations, provided the lineations are favourably orientated

4. JOINT ALTERATION NUMBER (J_a)

(a) *Rock wall contact*

		(J_a)	φ_r (approx.)
A.	Tightly healed, hard, non-softening, impermeable filling i. e. quartz or epidote	0.75	(—)
B.	Unaltered joint walls, surface staining only	1.0	(25°—35°)
C.	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock etc.	2.0	(25°—30°)
D.	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20°—25°)

Note:
(i) Values of $(\varphi)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present

Table 1. Description and typical ratings for the NGI rock mass classification (5).

E.	Softening or low friction clay mineral coatings, i. e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1—2 mm or less in thickness)	4.0	(8°—16°)
	(b) *Rock wall contact before 10 cms shear*		
F.	Sandy particles, clay-free disintegrated rock etc.	4.0	(25°—30°)
G.	Strongly over-consolidated, non-softening clay mineral fillings (Continuous, < 5 mm in thickness)	6.0	(16°—24°)
H.	Medium or low over-consolidation, softening, clay mineral fillings. (Continuous, < 5 mm in thickness)	8.0	(12°—16°)
J.	Swelling clay fillings, i. e. montmorillonite (Continuous, < 5 mm in thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8.0—12.0	(6°—12°)
	(c) *No rock wall contact when sheared*		
K, L, M.	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6.0, 8.0 or 8.0—12.0	(6°—24°)
N.	Zones or bands of silty- or sandy clay, small clay fraction (non-softening)	5.0	
O, P, R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10.0, 13.0 or 13.0—20.0	(6°—24°)

5.	JOINT WATER REDUCTION FACTOR	(J_w)	Approx. water pressure (kg/cm^2)
A.	Dry excavations or minor inflow, i. e. < 5 l/min. locally	1.0	< 1
B.	Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0— 2.5
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5—10.0
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5—10.0
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2—0.1	> 10.0
F.	Exceptionally high inflow or water pressure continuing without noticeable decay	0.1—0.05	> 10.0

Note:

(i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed

(ii) Special problems caused by ice formation are not considered

Table 1. (ctd)

6. STRESS REDUCTION FACTOR (*SRF*)

		σ_c/σ_1	σ_t/σ_1	(*SRF*)
	(a) *Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*			
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0
B.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation $\leqq 50$ m)			5.0
C.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation > 50 m)			2.5
D.	Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)			7.5
E.	Single shear zones in competent rock (clay free) (depth of excavation $\leqq 50$ m)			5.0
F.	Single shear zones in competent rock (clay free) (depth of excavation > 50 m)			2.5
G.	Loose open joints, heavily jointed or "sugar cube" etc. (any depth)			5.0
	(b) *Competent rock, rock stress problems*			
H.	Low stress, near surface	> 200	> 13	2.5
J.	Medium stress	200—10	13—0.66	1.0
K.	High stress, very tight structure (Usually favourable to stability, may be unfavourable to wall stability)	10—5	0.66—0.33	0.5—2.0
L.	Mild rock burst (massive rock)	5—2.5	0.33—0.16	5—10
M.	Heavy rock burst (massive rock)	< 2.5	< 0.16	10—20
	(c) *Squeezing rock;* plastic flow of *incompetent rock under the influence of high rock pressures*			
N.	Mild squeezing rock pressure			5—10
O.	Heavy squeezing rock pressure			10—20
	(d) *Swelling rock; chemical swelling activity depending on presence of water*			
P.	Mild swelling rock pressure			5—10
R.	Heavy swelling rock pressure			10—15

Note:

(i) Reduce these values of *SRF* by 25—50% if the relevant shear zones only influence but do not intersect the excavation

(ii) For strongly anisotropic stress field (if measured): when $5 \leqq \sigma_1/\sigma_3 \leqq 10$, reduce σ_c and σ_t to $0.8\,\sigma_c$ and $0.8\,\sigma_t$; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\,\sigma_c$ and $0.6\,\sigma_t$ where: σ_c = unconfined compression strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses

(iii) Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H)

Table 1. (ctd).

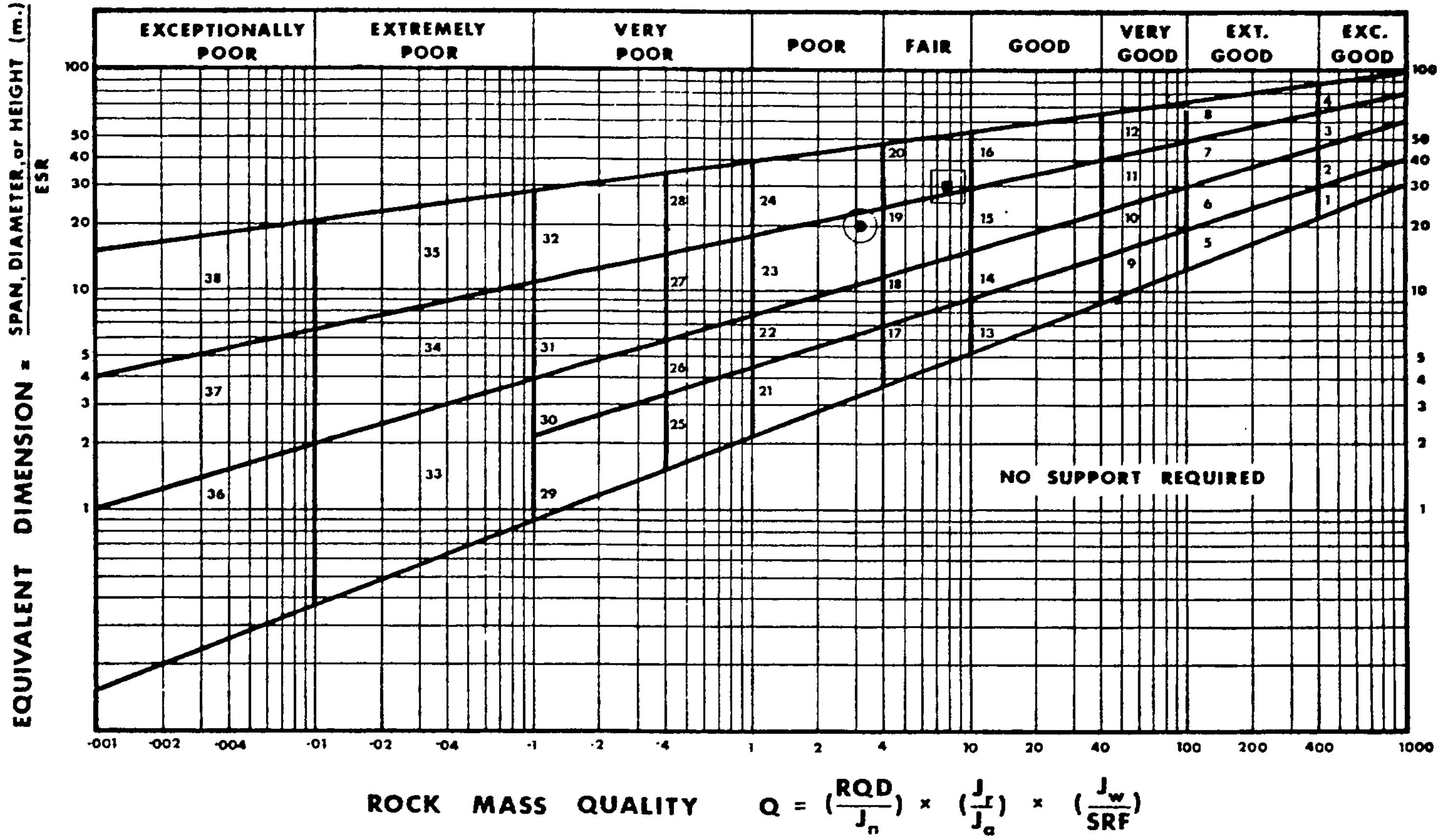

Figure 1. Support requirement categories, and area of no support requirement according to the NGI rock mass classification scheme. A value of 3-5 is used for ESR of temporary mine openings, 1.6 for permanent openings (5).

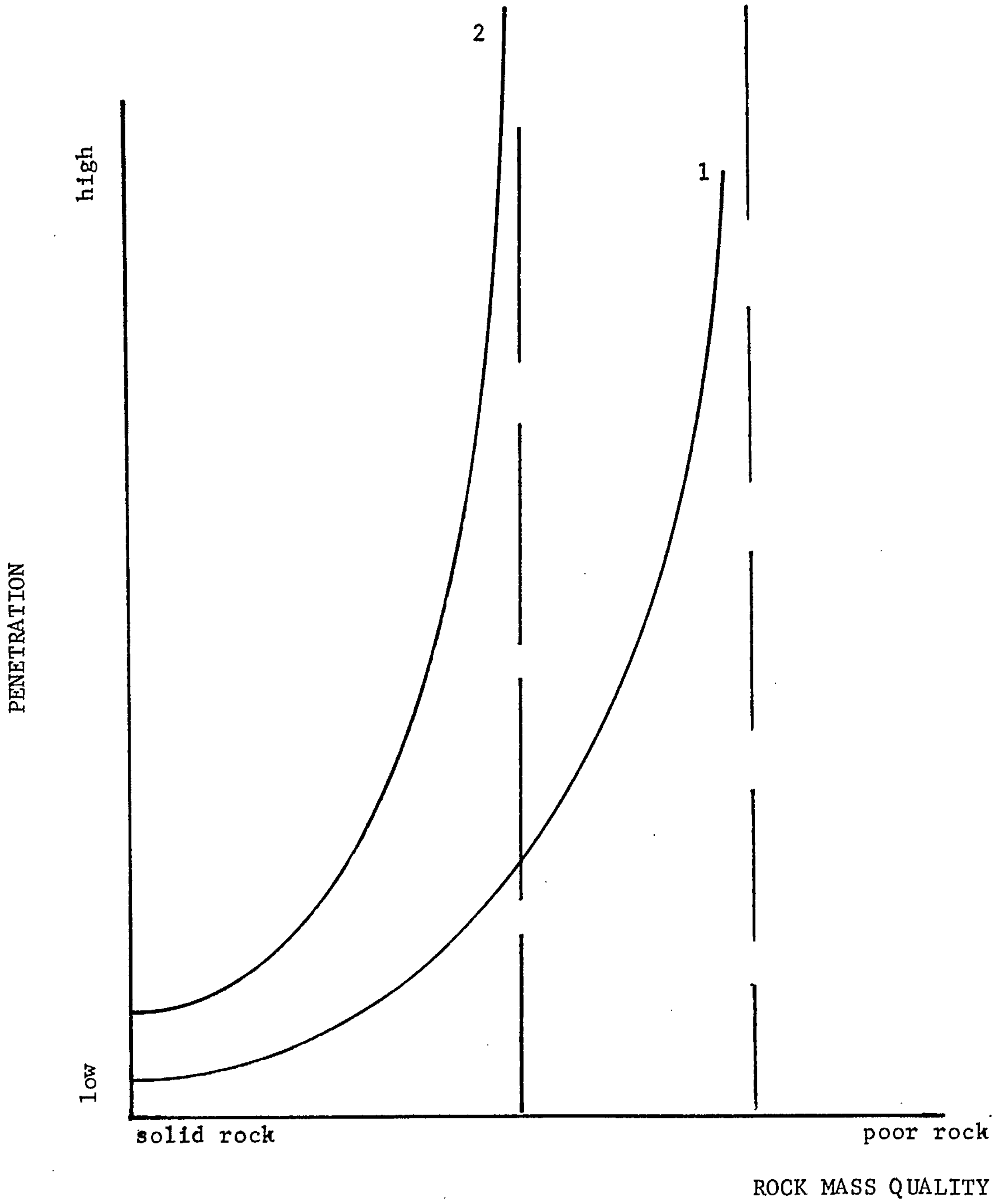

Figure 2. Effect of rock mass quality on excavating machine advance. 1= lower thrust, 2= higher thrust. Same rock type.

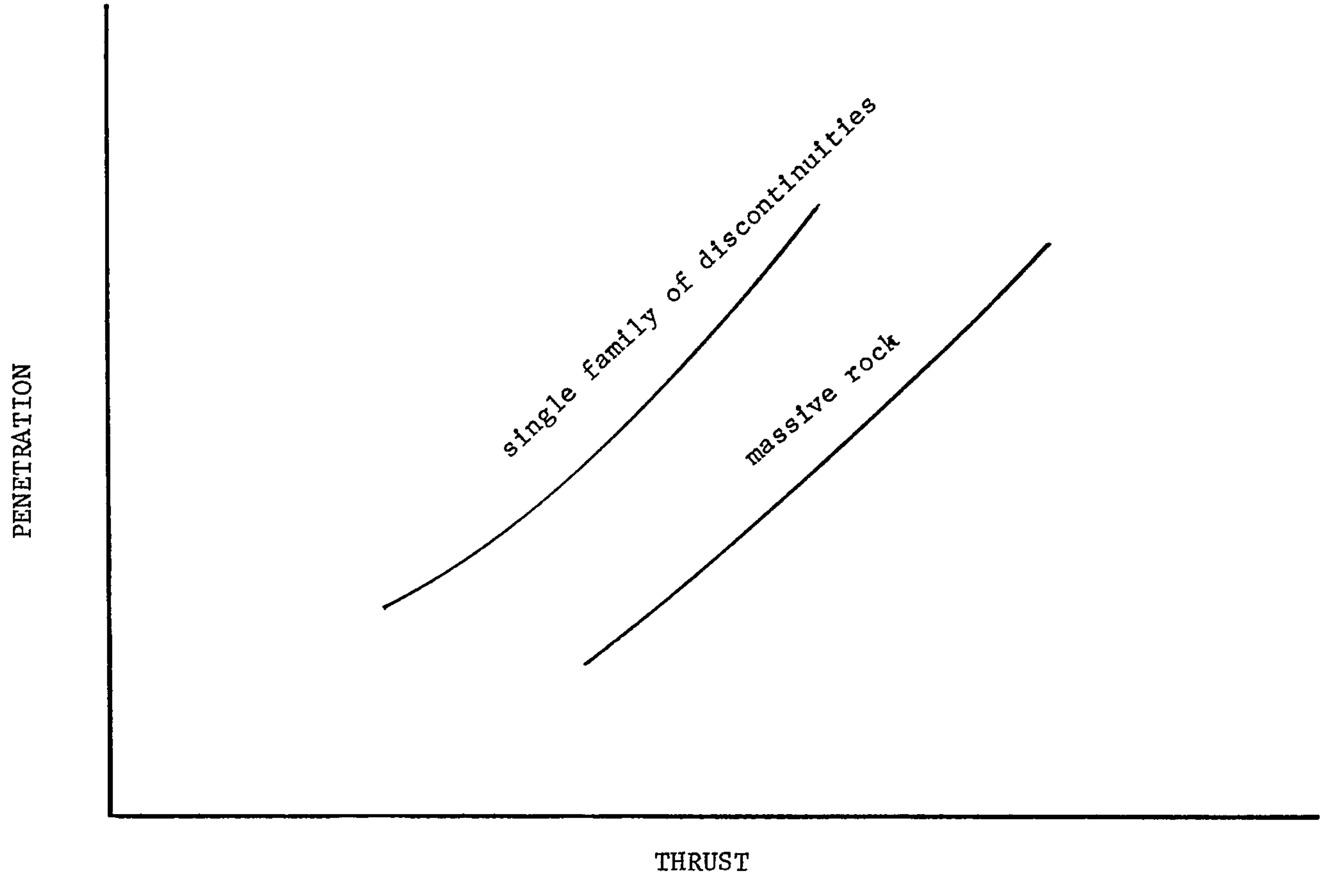

Figure 3. Effect of a family of discontinuities on thrust and penetration.

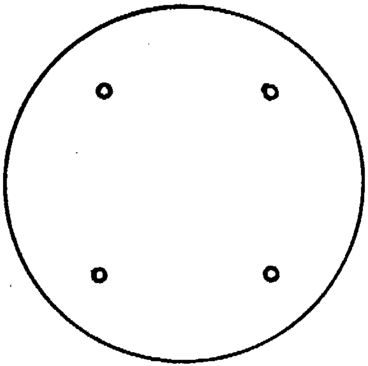

Figure 4. Vertical section showing limit of excavated tunnel with a simple pattern of data gathering diamond drill holes.

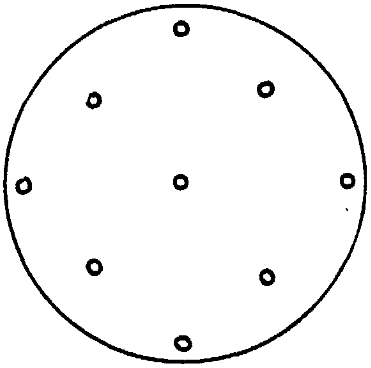

Figure 5. Vertical section showing limit of excavated tunnel and more complex pattern of data gathering diamond drill holes.

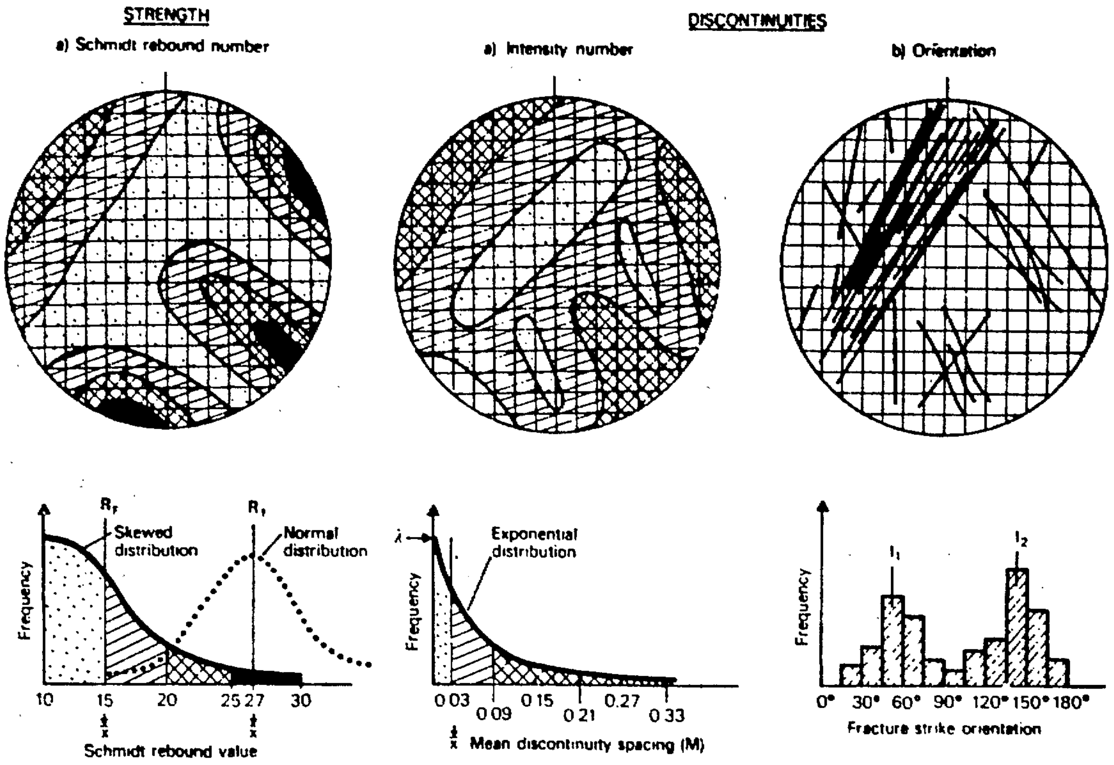

Figure 6. Application of Schmidt hammer rebound tests to a full face of a tunnel boring machine, using a grid system (2).