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POLITECNICO DI TORINO

Master in Tunnelling and tunnel boring machines

Post Graduated Master Course
TUNNELLING AND TUNNEL BORING MACHINES

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Subject of the lesson : GEOMECHANICAL CLASSIFICATIONS
Company / Affiliation : GEODATA ENGINEERING

EXTRACT

Endorsed by

ASSOCIATION INTERNATIONALE DES TUNNELS ET DE L'ESPACE SOUTERRAIN AITES | ITA INTERNATIONAL TUNNELLING AND UNDERGROUND SPACE ASSOCIATION

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Italian Tunnelling Society

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Outline of the lesson

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VIII	Excavation behaviour classification	42 (+5video)
	Total	200 (+5video)

EXTRACT

The design methods of underground constructions can be basically divided in [7]:

- **“analytical” methods** → mainly based on stress/strain analysis around the cavity (for example: numerical methods);
- **“observational” methods** → mainly based on behaviour monitoring during excavation, as well as on the the analysis of ground/support interation (for example, in general terms the NATM);
- **“empirical” methods** → mainly based on previous experiences of tunneling (for example, the geomechanical classifications)

The geomechanical classifications developed and widespread as a design empirical methods with the main purpose of [7]:

- subdividing the rock masses in geomechanical groups with similar behaviour;
- providing a valid base to understand the mechanical properties of rock masses;
- making the design easier, based on statistical analysis of precedent experiences;
- assuring a common language between different types of technicians involved in the design.

- According to the “Italian Guideline for Design, Tendering and Construction of Underground Works” [37] (LGP, fig.1), an exhaustive design should consider analytical (most important), empirical and observational components;

- following this approach, in the Italian current practice, the Geomechanical Classifications are only a part of a more complete design procedure, mainly useful for:
  - → the geomechanical zoning and the definitions of input parameters for the design analysis;
  - → the assessment of loading condition on structures;
  - → temporary support recommendations.

General setting (4 di 12)

Schematic structure of the Italian Guideline for Design, Tendering and Construction of Underground Works

Fig.1

It is conceptually important to distinguish [46]:

Geomechanical classes or groups (G.G.) ↓	→ Constituted by rock masses with comparable geomechanical characteristics (intrinsic properties)
Behavior categories (C.C.) ↓	→ Describe the deformation response of the cavity upon excavation, corresponding to different combinations of geomechanical and in-situ stress conditions
Technical Classes (C.T.)	→ Are directly associated with the different project solution (i.e. with the typical sections of excavation and support)

Main Classification Systems

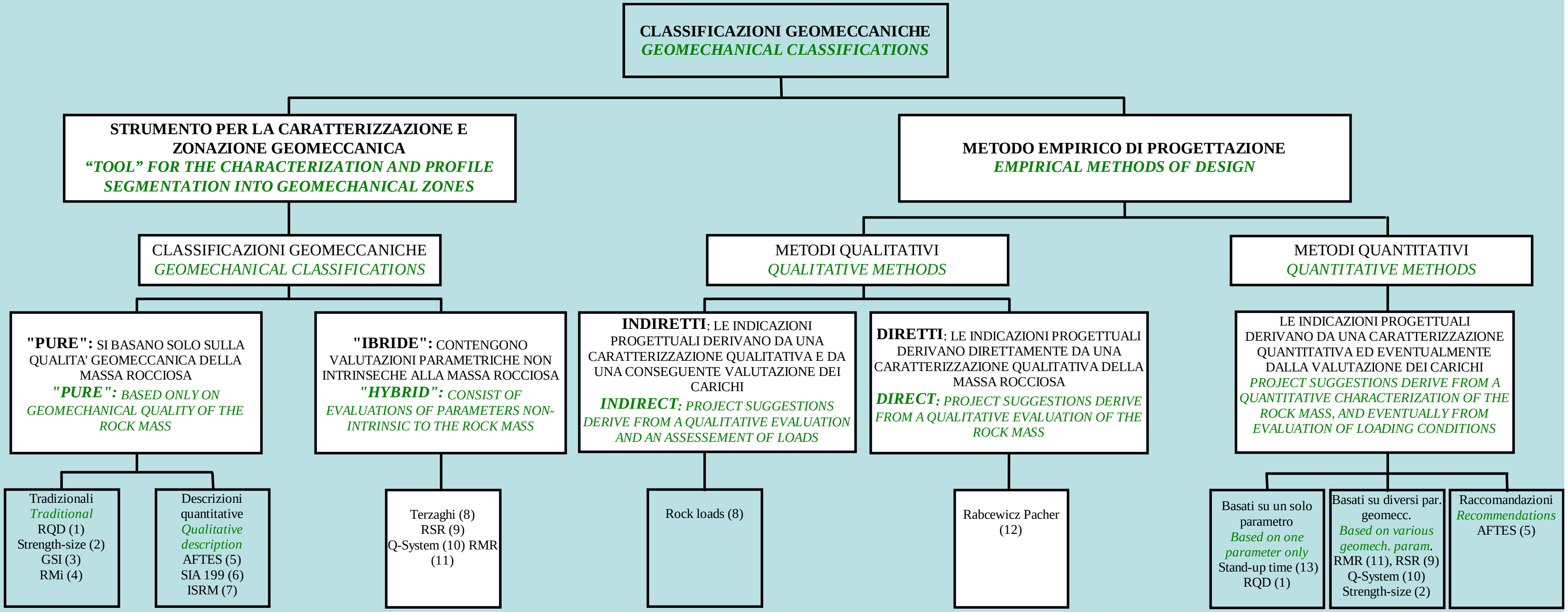
Method	Author	Year	G.G.	C.C.	C.T.
Rock loads (T)	Terzaghi	1946	(combined system)		indications
Stand-up time	Lauffer	1958÷1988		√	√
RQD system	Deere	1964	√		√
RSR system	Wickham	1972	√		√
RMR system	Bieniawski	1973÷1989	√		√
Lombardi	Lombardi	1974		√	
(R-P)	Rabcewicz-Pacher	1974	(combined system)		indications
Q system	Barton et al.	1974÷1999	√		√
Strength-size	Franklin	1975	√		√
RMi	Palmstrom	1995÷2000	√		√
GSI	Hoek et al.	1995÷2000	√		
Adeco-RS	Lunardi	1993		√	indications
GD Classification	Russo et al.	1998 ÷2007		√	indications

• As shown previously, (fig.2) the geomechanical classifications of an underground project can be used as:

- a method of assessment of input geomechanical parameters (equivalent-continuum model) about design analysis (→*GEO*) ;
- an empirical method of design (→*PRO*)

• Therefore, the choice of the most appropriate Geomechanical Classification is also a function of the foreseen usage:

- in the first case (→*GEO)* "pure quantitative" system are more advisable [for istance **GSI** (fabric index) **and RMi**];
- in the second case (→*PRO*) traditional "quantitative" systems are more indicate (such as **RMR, Q- System**, and even **RMi**).



Note: (1) Deere, 1964; (2) Franklin, 1975; (3) Hoek, 1994 and Hoek et al., 1995; (4) Palmstrøm, 1996; (5) 1993; (6) 1975; (7) 1981; (8) Terzaghi, 1946; (9) Wickham,1972; (10) Barton et al., 1974, 1994; (11) Bieniawski, 1973, 1989; (12) 1974; (13) Lauffer, 1958, 1988.

Per la bibliografia si veda Russo (1994) - *For bibliography refer to Russo (1994)*

Fig.2 [46]

	T	R-P	RSR	RMR	Q	RQD	GSI	RMi
Geomechanical quality ↓	√q_-	√q_-	√q^+*i*	√q^+*i*	√q^+*i*	√q^+*p*	√q^+*p*	√q^+*p*
Rock mass parameters ↓				√	√		√	√
Evaluation of the loads ↓	√		√	√	√	(√)		
Indications about support	√	√	√	√	√	(√)		√

Note: q_-/q^+=with qualitative/quantitative assessment; *p*/*i* = "pure"/ibrid index; () proposed by other authors.

Geomechanical Classifications limitations (1 of 3):

- As according to Guidelines (LGP), Geomechanical classifications cannot be the only means of design, particularly in more detailed phases and for permanent lining definition;
- often a problematic application to weak rocks (>>tendency of a geomechanical over evaluation of continuous rock masses) and/or to structurally complex rock formations (>> difficult parameter definition) [44];
- as an empirical method, they are generally more reliable for dimensioning radial stabilization measures in fractured rock masses, where mainly gravitational failures occur;

Geomechanical Classifications limitations (2 of 3):

- The limits of using only empirical method for the design are even more evident under difficult geomechanical conditions, where:
  - an analytical method of the ground-structure interaction is essential for structure dimensioning;
  - special interventions are often necessary, generally not proposed by classifications systems, whose definition varies from case to case (for example, the face and the profile preconfinement, presupport ("umbrella"), the rock mass improvement, etc.).

Geomechanical Classifications limitations (3 of 3):

- Hoek & Brown (1980) "recommend classification systems for general use in the preliminary design of underground excavations"

- Bieniawski (1997) is of the opinion that "rock mass classifications on their own should only be used for preliminary, planning purpose and not for final support"

- Stille & Palmstrom (2003) "strongly argued against using the existing classification systems as the only indicator to define the rock support or other engineering items"

QUALITATIVE INDIRECT METHODS

Basic scheme:

Qualitative rock mass characterization →

→ Definitions of structure loads →

→ Support dimensioning

ROCK LOAD CLASSIFICATION (Terzaghi,1946)

Main features:

- Formulated for the assessment of rock loads for dimensioning a support composed by steel ribs;
- N. 9 rock mass classes are defined (fig.4), with correlated rock load conditions (function of the tunnel dimensions), and indications about the expected behavior of the cavity are given;
- the rock load mobilization mechanism is showed in the figg. 3 and 5;
- the modification proposed by Deere (1970) is presented in fig.6

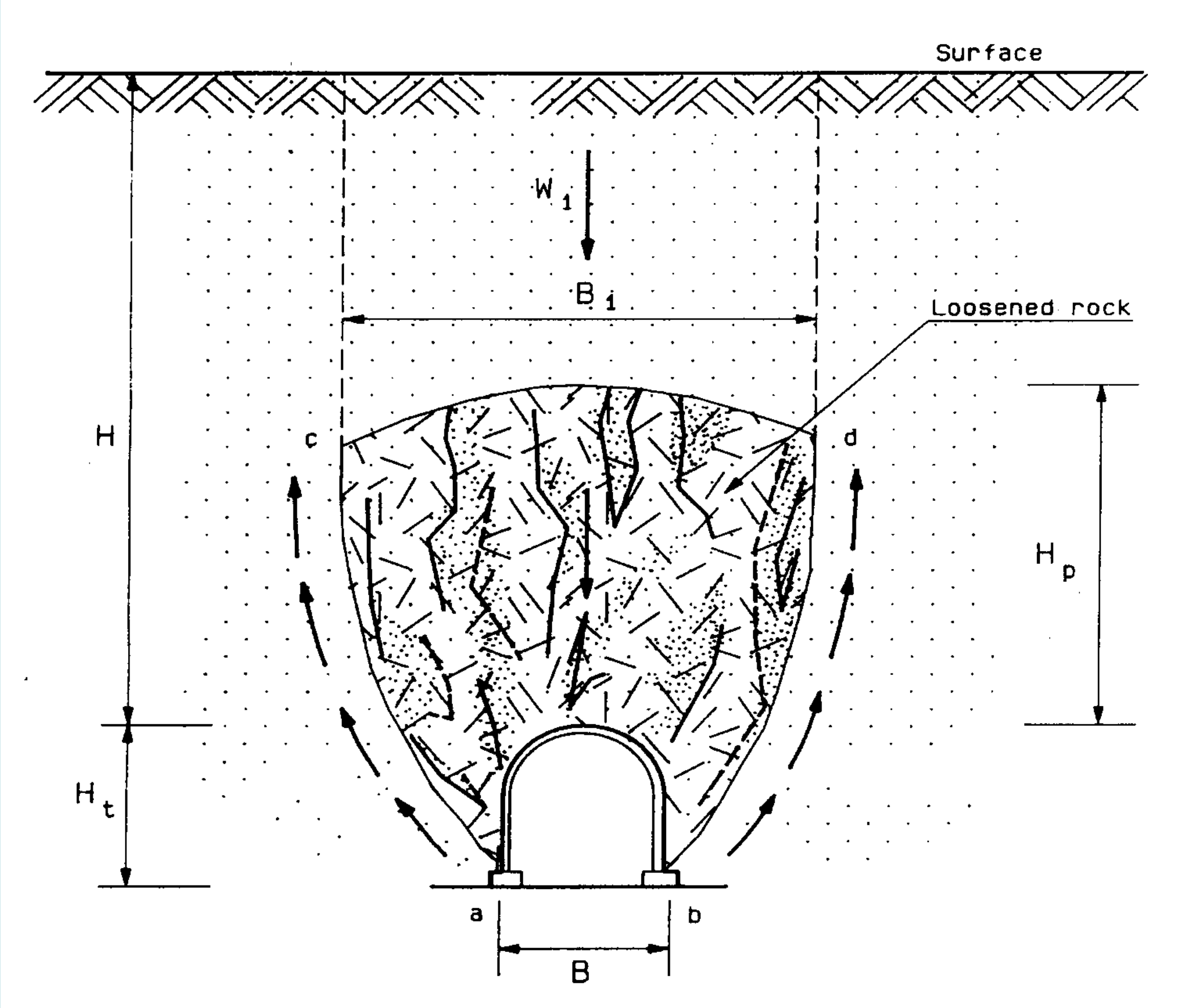


Fig.3: Load movement scheme on the tunnel (Terzaghi, 1946)[35]

Terzaghi's rock load classification of 1946.

Rock load H_p in feet of rock on tunnel roof with width B(ft) and height H_t(ft) at depth of more than $1.5(B + H_t)$.

Rock condition	Rock load H_p in feet	Remarks
1. Hard and intact	Zero	Light lining required only if spalling or popping occurs.
2. Hard stratified or schistose	0 to $0.5B$	Light support, mainly for protection against spalls. Load may change erratically from point to point.
3. Massive, moderately jointed	0 to $0.25B$	
4. Moderately blocky and seamy	$0.25B$ to $0.35(B + H_t)$	No side pressure.
5. Very blocky and seamy	$(0.35$ to $1.10)(B + H_t)$	Little or no side pressure.
6. Completely crushed	$1.10(B + H_t)$	Considerable side pressure. Softening effects of seepage towards bottom of tunnel require either continuous support for lower ends of ribs or circular ribs.
7. Squeezing rock, moderate depth	$(1.10$ to $2.10)(B + H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
8. Squeezing rock, great depth	$(2.10$ to $4.50)(B + H_t)$	
9. Swelling rock	Up to 250 feet, irrespective of the value of $(B + H_t)$	Circular ribs are required. In extreme cases use yielding support.

Definitions:

Intact rock contains neither joints nor hair cracks. Hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as a *spalling* condition. Hard, intact rock may also be encountered in the *popping* condition involving the spontaneous and violent detachment of rock slabs from the sides or roof.

Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the spalling condition is quite common.

Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both spalling and popping conditions may be encountered.

Blocky and *seamy* rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require lateral support.

Crushed but chemically intact rock has the character of a crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

Squeezing rock slowly advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.

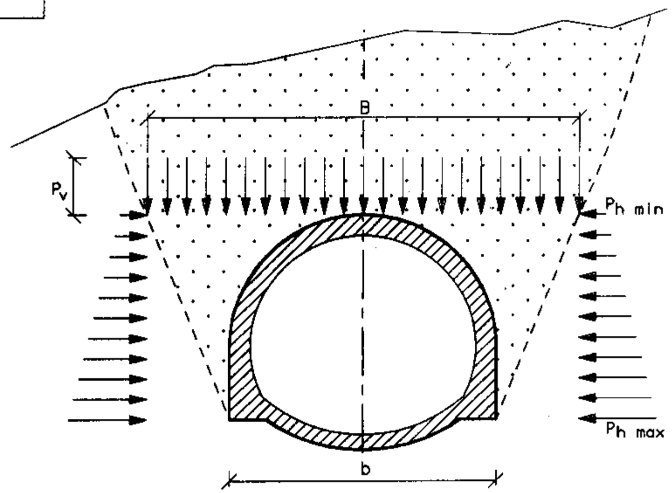
Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks which contain clay minerals such as montmorillonite, with a high swelling capacity.

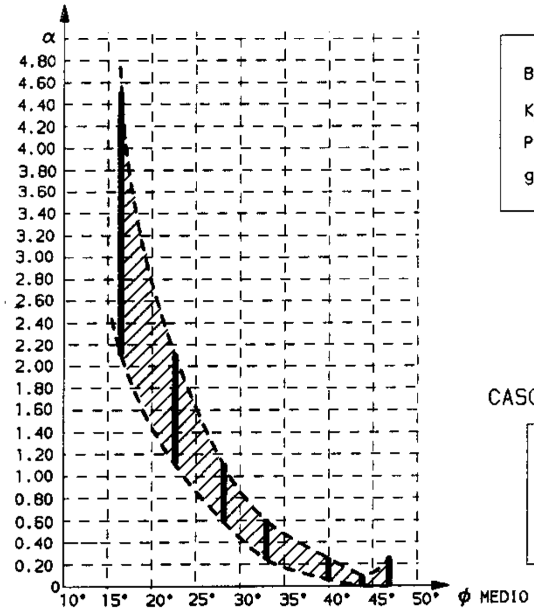
Fig.4 [7]

GEODATA

VALUTAZIONE DEI CARICHI SECONDO TERZAGHI

GALLERIE POCO PROFONDE (H<2.5 B)





Valutazione dei carichi secondo Terzaghi (1946), dopo modifiche di Deere (1969) e Rose (1982), per H>1.5(b+h).

$B = b + 2h\ tg\left(45 - \frac{\phi}{2}\right)$

$K_0 = 1 - \sin\phi$ (massiccio a riposo)

$P_h = K_0 \cdot Pv$

g=peso di volume

CASO 1 - ROCCE

$$P_v = \alpha\,(b+h)\,g$$

CASO 2 - TERRENI CON COESIONE

$$P_v = \frac{B\left(g - \frac{2c}{B}\right)\left(1 - e^{\frac{-2H\ tg\phi}{B}}\right)}{2tg\phi}$$

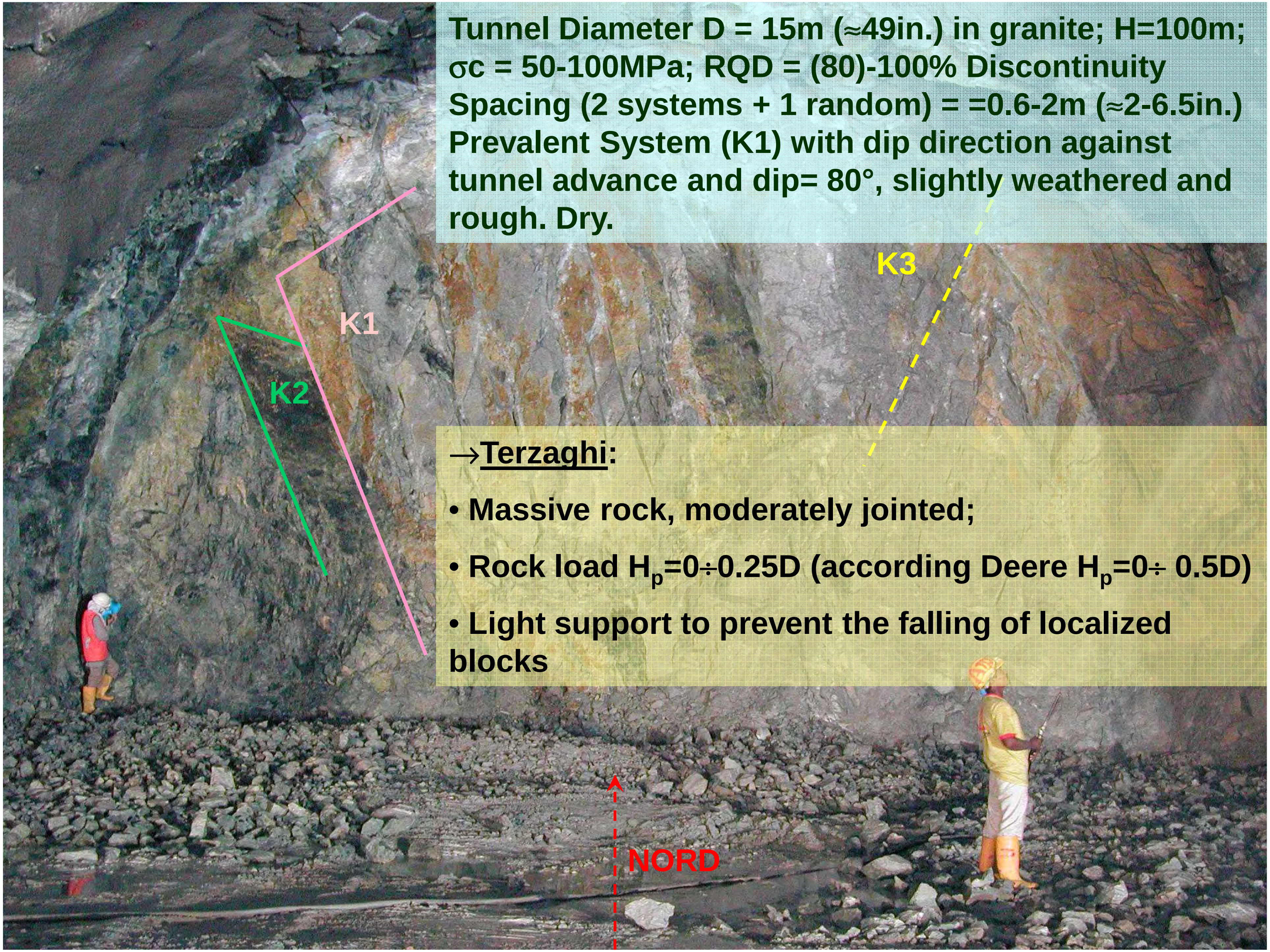
FIG. 5

Terzaghi's rock load classification as modified by Deere et al., 1970

Fracture spacing (cm)	RQD (%)	Rock condition	Rock load, H_p Initial	Rock load, H_p Final	Remarks
		1. Hard and intact	0	0	(rows 1–4:) Generally no side pressure. Erratic load changes from point to point. Lining only if spalling or popping
50	98, 95	2. Hard stratified or schistose	0	$0.25B$	Spalling common
	90	3. Massive, moderately jointed	0	$0.5B$	Side pressure if strata inclined, some spalling
20	75	4. Moderately blocky and seamy	0	$0.25B$ to $0.35C$	
10	50	5. Very blocky, seamy and shattered	0 to $0.6C$	$0.35C$ to $1.1C$	Little or no side pressure
5	25, 10	6. Completely crushed		$1.1C$	Considerable side pressure. If seepage, continuous support
2	2	7. Gravel and sand	$0.54C$ to $1.2C$	$0.62C$ to $1.38C$	Dense. Side pressure $P_h = 0.3\gamma(0.5H_t + H_p)$
			$0.94C$ to $1.2C$	$1.08C$ to $1.38C$	Loose
Weak and coherent		8. Squeezing, moderate depth		$1.1C$ to $2.1C$	Heavy side pressure. Continuous support required
		9. Squeezing, great depth		$2.1C$ to $4.5C$	
		10. Swelling		up to 250ft.	Use circular support. In extreme cases: yielding support

Notes:
1. For rock classes 4, 5, 6, 7, when above ground water level, reduce loads by 50%.
2. B is tunnel width, $C = B + H_t$ = width + height of tunnel.
3. γ = density of medium.

Fig.6 [7]



Tunnel Diameter D = 15m (≈49in.) in granite; H=100m; σc = 50-100MPa; RQD = (80)-100% Discontinuity Spacing (2 systems + 1 random) = =0.6-2m (≈2-6.5in.) Prevalent System (K1) with dip direction against tunnel advance and dip= 80°, slightly weathered and rough. Dry.

→Terzaghi:

- **Massive rock, moderately jointed;**
- **Rock load H_p=0÷0.25D (according Deere H_p=0÷ 0.5D)**
- **Light support to prevent the falling of localized blocks**

QUALITATIVE DIRECT METHODS

Basic scheme:

Rock mass qualitative characterization $\rightarrow$

$\rightarrow$ Support dimensioning/ Construction phases and procedures

RABCEWICZ-PACHER CLASSIFICATION (1974)

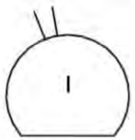
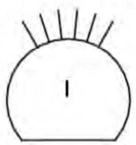
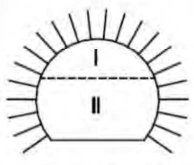
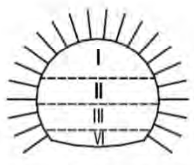
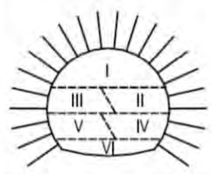
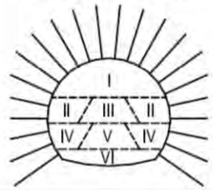
Main features

- Developed on the system base classification proposed by Lauffer[1] (1958) originating The New Austrian Tunnelling Method (NATM)
- n.6 rock classes are considered (fig.7), a qualitative description of the characteristics and the behaviour is associated to applicative procedures and support dimensioning
- For the mechanized excavation with TBM specific adaptation and development have been arranged, as proposed by the Austrian Norm (ONORM) 2203 (fig.8), furthermore modified in fig.9.

Note: 1 The classified method proposed by Lauffer will be shown in the “ direct quantitative methods” section

THE NEW AUSTRIAN TUNNELLING METHOD (NATM)

ROCK CLASSIFICATION ACCORDING TO RABCEWICZ-PACHER

ROCK CLASSES	I FROM STABLE TO SLIGHTLY BRITTLE	II VERY BRITTLE	III UNSTABLE TO VERY UNSTABLE	IV SQUEEZING	Va VERY SQUEEZING	Vb LOOSE MATERIAL
CHARACTERISTICS	COMPACT MATERIAL, SLIGHT TO MEDIUM FISSURING	HEAVY DIVISION INTO STRATA AND FRACTURING, SINGLE FISSURES ARE FULL OF CLAYEY MATERIAL; SCHISTOSE INTERCALATIONS	VERY HEAVY DIVISION INTO STRATA AND FRACTURING ON SEVERAL PLANES: FISSURES ARE FULL OF CLAYEY MATERIAL.	VERY WEATHERED ROCK: FOLDED AND SCHISTOSE; BANDSOF FAULTS; WELL CONSOLIDATED, COHESIVE, LOOSE MATERIAL	COMPLETELY MYLONITIZED AND WEATHERED REDUCED TO SCREE, NOT CONSOLIDATED, SLIGHTLY COHESIVE	LOOSE MATERIAL, NON-COHESIVE
CHARACTERISTICS	UNI AXIAL COMPRESSIVE STRENGTH σ_{gd} IS GREATER THAN THE TANGENTIAL STRESS σ_t; PERMANENT CONDITIONS OF EQUILIBRIUM OR GUARANTEED BY: MEASURES OF LOCAL PROTECTION	REINFORCEMENT OF THE RING OF LOAD BEARING ROCK IN THE CROWN	THE LIMIT STRENGTH OF THE ROCK IS REACHED AND EXCEEDED AROUND THE CROSS SECTION. SUPPORTS AND THE CREATION OF A RING OF LOAD BEARING ROCK ARE NECESSARY	THE TANGENTIAL STRESSES EXCEED THE STRENGTH OF THE ROCK. THE MATERIAL HAS PLASTIC BEHAVIOUR AND TENDS TO MOVE INTO THE CAVITY REDUCING THE CROSS SECTION; INTENSITY OF THE PHENOMENON: MEDIUM	STRONG LATERAL THRUSTS AND RAISING OF THE FLOOR. THE MOVEMENTS ARE WITHSTOOD BY THE FULLY CLOSED LOAD BEARING RING	SEE CLASS Va
INFLUENCE OF WATER	NONE	UNIMPORTANT	MAINLY ON THE CAVITY, OF THE FISSURES	FAIR	EVEN STRONG (THE MATERIAL TENDS TO BECOME SOAKED)	
EXCAVATION	FULL FACE 	FULL FACE 	TOP HEADING AND BENCH 	DIVISION OF FACE: I – IV 	DIVISION OF FACE: I - VI 	DIVISION OF FACE: I - VI 

ACTIVE STABILISATION CROSS SECTION TYPES FOR TUNNEL Ø ~ 10.00 m

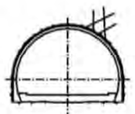
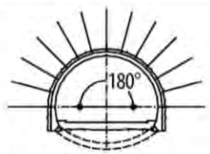
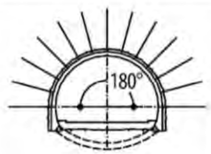
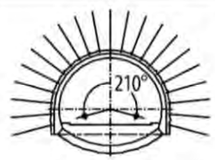
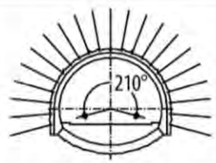
ROCK CLASSES	I	II	III	IV	V	VI
CROSS SECTION TYPES						
CHARACTERISTICS	ADVANCE STEP 3.5/4.5 m. LINING Thcknss=0+0+30=30 cm.	ADVANCE STEP 2.5/3.5 m. LOCALLY END ANCHORED ROCK BOLTS L=4.00 m. STEEL MESH REINFORCEMENT LINING Thcknss=5+5+30=40 cm.	ADVANCE STEP 1.5/2.5 m. SYSTEMATIC END ANCHORED ROCK BOLTS L=4.00 m. STEEL MESH REINFORCEMENT LINING Thcknss=10+10+35=55 cm	ADVANCE STEP 1.0/1.6 m. SYSTEMATIC FULLY BONDED ROCK BOLTS L=4.00 m. STEEL RIBS STEEL MESH REINFORCEMENT LINING Thcknss=10+15+40=65 cm. TUNNEL INVERT	ADVANCE STEP 0.8/1.2 m. SYSTEMATIC FULLY BONDED ROCK BOLTS L=4.00 m. STEEL RIBS STEEL MESH REINFORCEMENT LINING Thcknss=10+20+45=75 cm. TUNNEL INVERT Thcknss=0.75m.	ADVANCE STEP 0.5/1.0 m. SYSTEMATIC FULLY BONDED ROCK BOLTS L=4.00 m. STEEL RIBS STEEL MESH REINFORCEMENT Thcknss=10+20+50=85 cm. TUNNEL INVERT Thcknss=0.85m.

Fig.7 [derived from13]

Rock type	Rock behavior	Requirements for excavation and support measures for continuous advance
A Stable rock to rock liable to rockfall. This rock type includes all rock, which can bear loading without signs of rupture.	**A1** Stable Very quickly subsiding, low deformations, no chips falling after the removal of loose rock pieces.	Support is not necessary. Stand-up time: more than three weeks.
	A2 Liable to rockfall Very quickly subsiding, low deformations, isolated loosening of rock pieces from the crown and upper sides due to jointing.	Support is only necessary in the crown, abutment and upper area of tunnel sides to secure isolated blocks. Installation of support in working area 2 without interruption of the machine advance. Stand-up time: 3 weeks to 4 days.
B Fractured rock. This rock type covers all rock, which due to lack of bonding strength in the joints and/or lack of lack of interlocking, tends to loosen.	**B1** Fractured Very quickly subsiding, low deformations; low rock strength due to joints and blasting vibration cause loosening and de-bonding of the rock bonding in the crown and upper parts of the tunnel sides.	**B1.1** Systematic installation of support to a low extent, primarily in the crown, abutment and side areas in the work-free area 2, without interruption of the machine advance. Stand-up time: 2 to 4 days. **B1.2** Systematic installation of support in the crown, abutment and side areas in the working areas 1 and 2. The machine advance is affected by the installation of support. Stand-up time: 2 days to 10 hours.
	B2 Strongly fractured Deformations subside quickly; low rock strength determined by the joints, low interlocking, high movement of pieces and blasting leads to quick, deep loosening and collapsing of rock from free unsupported surfaces.	**B2.1** Systematic installation of support measures, beginning immediately behind the cutter head, the duration of support installation generally determines the advance rate. Cutting proceeds only in partial strokes. Stand-up time: 10 to 5 hours. **B2.2** Systematic preliminary support in cutter head area and systematic support installation to the entire working area in working area 1. Stand-up time: without preliminary support, 5 to 2 hours.

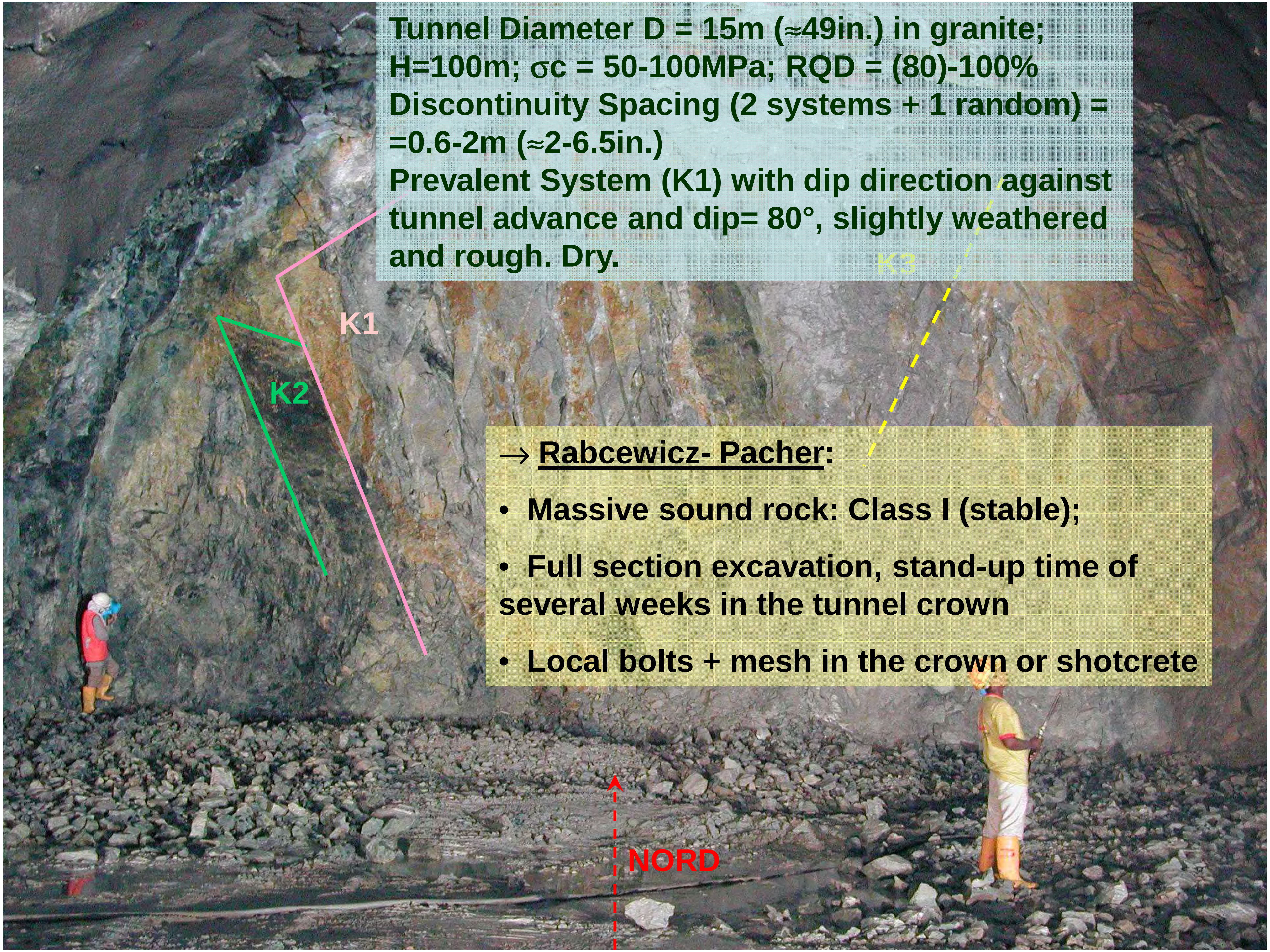
Austrian ÖNORM B 2203 (1994) for TBM (1 of 2)

Fig.8 [29]

Austrian ÖNORM B 2203 (1994) for TBM (2 of 2)

Rock type	Rock behavior	Requirements for excavation and support measures for continuous advance
	B3 Unstable On opening of only small partial sections, the rock trickles out. Absence of cohesion and missing toothing are the causes of insufficient stability.	Continuous advance with open tunnel boring machines is only possible with special measures. Stand-up time: less than 2 hours.
C Squeezing rock. This rock type includes all rocks, where the rock strength is deeply exceeded. Rock tending to collapse and rock with prominent swelling behaviour are included in this rock type.	**C1** Rock burst Elastic energy is stored in mostly heavy, hard and brittle rocks with high primary stress. Sudden conversion of this energy produces bursts of collapses with pieces of rock falling out. These pieces of rock thrown out of exposed surfaces are mostly chip shaped, the collapse only extends to shallow depths.	Installation of short, narrowly spaced anchors, if required with reinforcing mesh, in working area 1; the machine advance is not essentially hindered.
	C2 squeezing Prominent, long duration and slowly subsiding deformations. Development of ruptures or plastic zones in plastic, strongly cohesive soil.	**C2.1** systematic installation of support measures in crown, abutment and side areas. Stepwise installation of support measures in working areas 1 and 2; the machine advance is hindered by the installation of support; precautions have to be taken to prevent the jamming of the tunnel boring machine. Stand-up time: 2 days to 10 hours **C2.2** systematic installation of support, beginning immediately behind the cutter head; the duration of the installation of support generally determines the advance rate. Cutting is only possible in partial strokes; precautions have to be taken to prevent the jamming of the tunnel boring machine. Stand-up time: 10 to 5 hours.
	C3 Strongly squeezing Large, long duration and slowly subsiding deformations with high initial deformation speed. Development of deep ruptures or plastic zones.	**C3.1** systematic installation of support in crown, abutment and side areas; stepwise installation of support measures in working areas 1 and 2. The machine advance is hindered by the installation of support; precautions have to be taken to prevent the jamming of the tunnel boring machine. Stand-up time: 10 days to 2 hours. **C3.2** systematic installation of support measures, starting immediately behind the cutter head; the duration of support installation generally determines the advance rate. The cutting can only proceed in part strokes; precautions have to be taken to prevent the jamming of the tunnel boring machine. Stand-up time: 10 to 5 hours.
	C4 Flowing Very low cohesion and friction, weak plastic consistency of the soil leads to the soil flowing in, even with very small, only temporarily exposed and unsupported surfaces.	Advance with open tunnel boring machine is only possible with special measures. Stand-up time: shorter than 2 hours.
	C5 Swelling Soil types with mineral components, which depending on the relaxation through take up of water, experience an increase of volume, e.g. swellable clay minerals, salts, anhydrites.	Support measures effective in the long term accepting the swelling pressure or precautions enable the occurrence of swelling deformations without damage. Advance with an open tunnel boring machine is only possible with special measures. Stand-up time: no indication.

Fig.9 [29]


Tunnel Diameter D = 15m (≈49in.) in granite;
H=100m; σc = 50-100MPa; RQD = (80)-100%
Discontinuity Spacing (2 systems + 1 random) =
=0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against
tunnel advance and dip= 80°, slightly weathered
and rough. Dry.
K3
K1
K2
→ Rabcewicz- Pacher:
• Massive sound rock: Class I (stable);
• Full section excavation, stand-up time of
several weeks in the tunnel crown
• Local bolts + mesh in the crown or shotcrete
NORD

QUANTITATIVE DIRECT METHODS

Basic scheme:

Quantitative characterization of rock masses →

→ eventual derivation of geomechanical properties and/or load conditions →

→ support dimensioning/ Construction phases and procedures

There are methods based on a single parameter (such as "Stand-up time" by Lauffer and RQD by Deere) and methods based on the definition of more than one parameter (for example RSR, RMR, Q, RMi)

"**STAND-UP TIME" SYSTEM** (Lauffer, 1958÷1988)

Basic features

- Based on the following concepts of (fig.11):

  → Active unsupported span (lw) = Minor dimension between (1) the distance from tunnel face and the first installed support and (2) the width of the tunnel.

  → Stand-up time (ts) = Time in which the tunnel, for an active unsupported span, can remain stable after the tunnel excavation.

- 7 rock classes brought up to 9 in successive updating, are considered in the stand-up diagram.

GEODATA

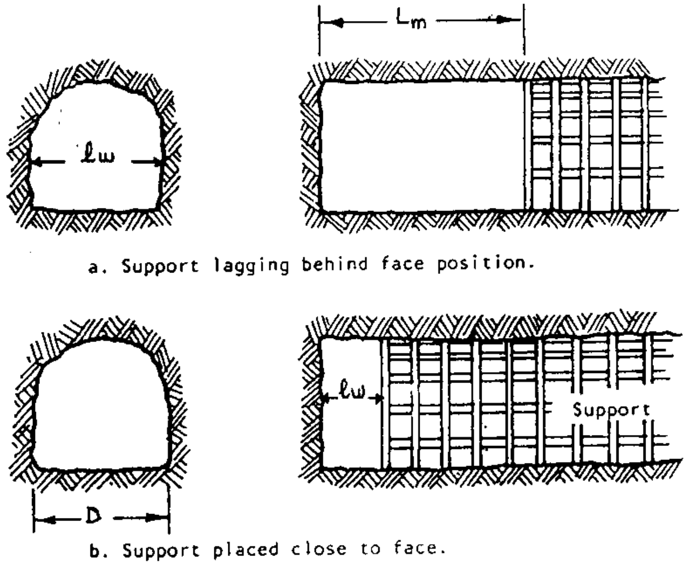
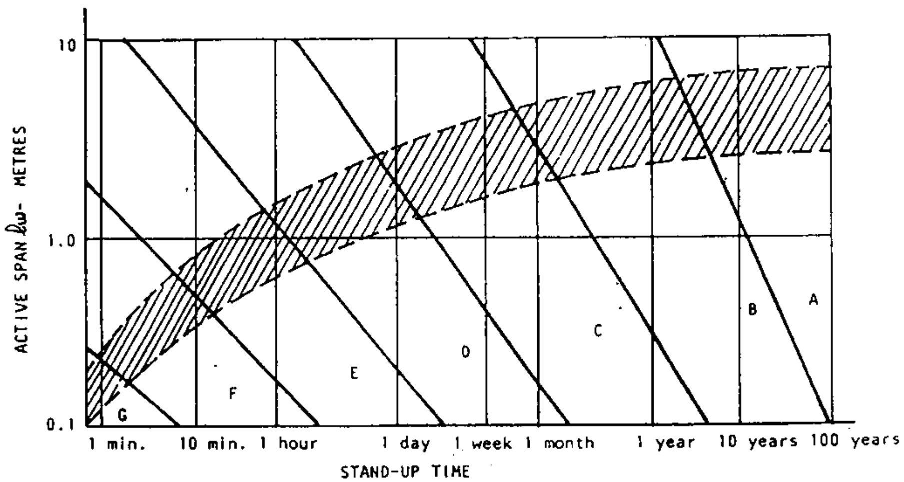


Fig. : Lauffer's definition of active span lw



Relationship between active span and stand-up time for different classes of rock mass. A - Very good rock, G - Very poor rock. (After Lauffer)

91709.gr.1

Fig.11 [21]

Features introduced in the up-dating of 1988 (fig.12,13,14):

- The stand-up diagram was modified, introducing the following expression:

$$\mathbf{t_s l_w^2 = 10^{8.9-1.7z}}$$

where:

z = stand-up coefficient associated to the rock mass characteristics, variable between 0 (superior limit class AA*) and 8 (limit between classes G/H*)

$$\mathbf{z = (8.9 - \log t_s - 2\log l_w)/1.7}$$

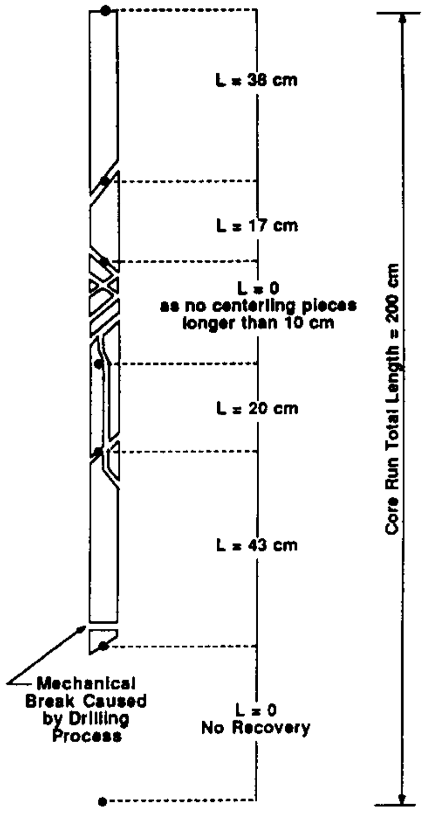
- new classes AA* e H*
- parallel lines spaced $1.7\log t_{s,}$ divide different stand-up classes
- to determine the active unsupported span, a corrective factor “x” is introduced to consider the three-dimensional face effect
- also the TBM characteristics are considered (fig.10)

RQD SYSTEM (Deere, 1964 and following)

Main features

- Based on the parameter Rock Quality Designation (RQD) defining 5 geomechanical classes (fig.15);
- Associated with these 5 classes, quantitative indications about necessary supports, are given, differing traditional and mechanized tunnelling with TBM (fig.16);
- As seen before (fig.6), Deere linked the index RQD to Terzaghi's classification.

RQD (%)	Rock Quality
<25	Very poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Excellent



$$RQD = \frac{\sum \text{Length of Core Pieces} > 10 \text{ cm (4 in.)}}{\text{Total Core Run Length}} \times 100\%$$

$$RQD = \frac{38 + 17 + 20 + 43}{200} \times 100\%$$

RQD = 59% (FAIR)

Procedure for measurement and calculation of rock quality designation. (After Deere, 1989.)

Fig.15 [8]

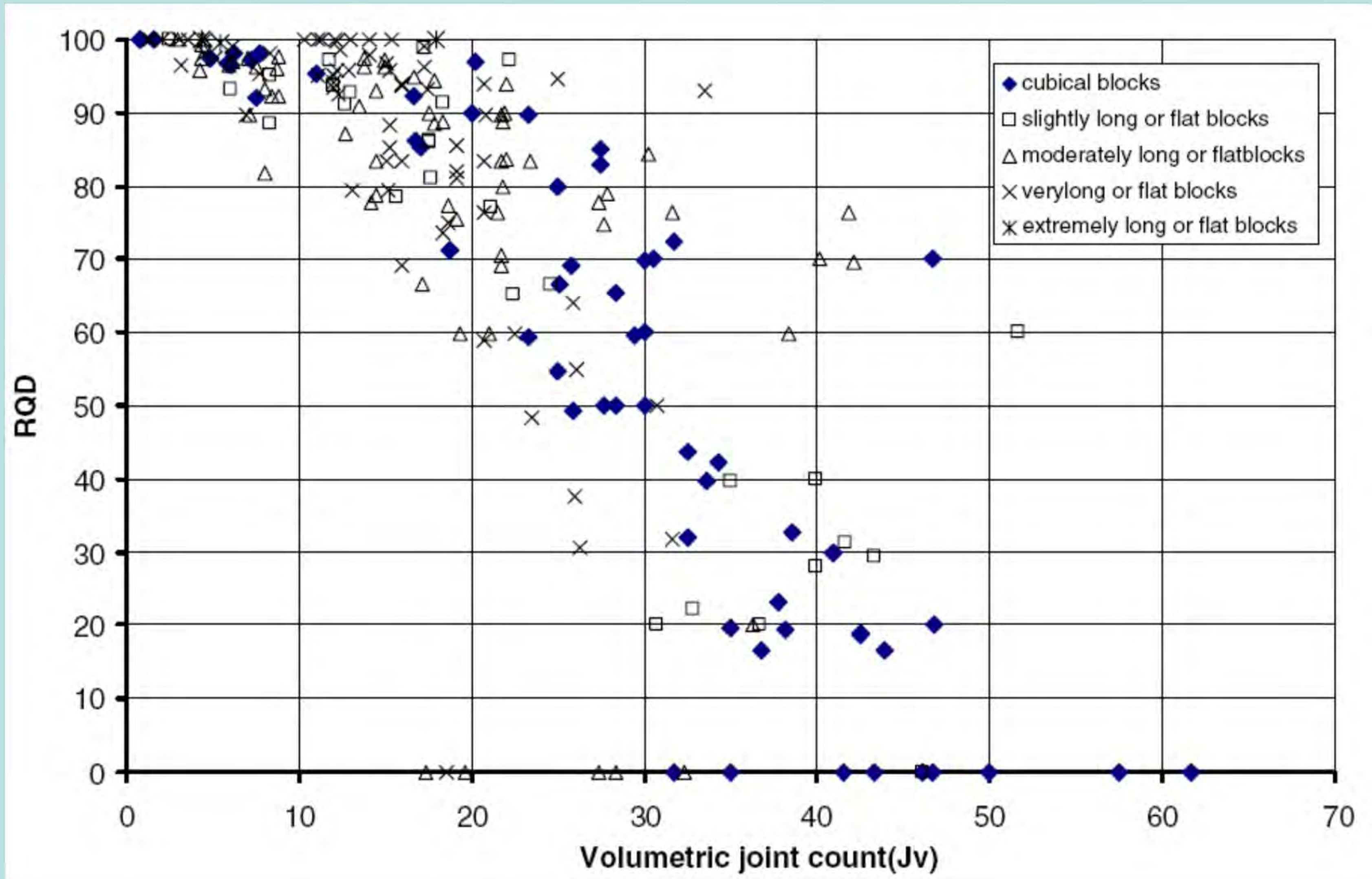
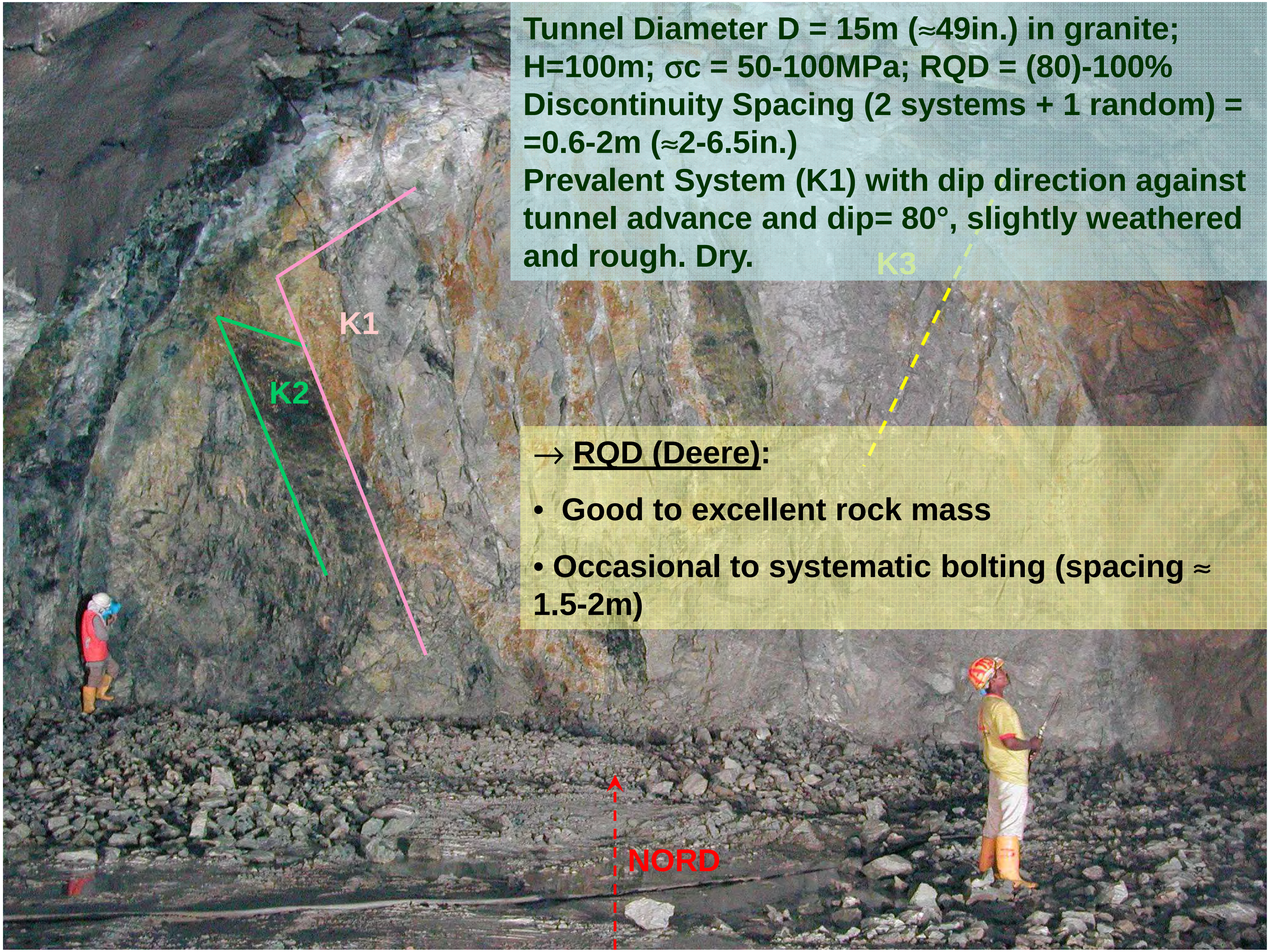


Fig.16bis [41quater]

Limits of RQD as fracturing index

Tunnel Diameter D = 15m (≈49in.) in granite; H=100m; σc = 50-100MPa; RQD = (80)-100%
Discontinuity Spacing (2 systems + 1 random) = =0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against tunnel advance and dip= 80°, slightly weathered and rough. Dry.



→ **<u>RQD (Deere)</u>:**

- **Good to excellent rock mass**
- **Occasional to systematic bolting (spacing ≈ 1.5-2m)**

RSR Concept (Wickham, 1972)

Basic features:

- Definition of a rock quality index RSR (Rock Structure Rating) derived from the sum of three geological and constructive parameters (fig.17)

$$RSR = A+B+C$$

A = General area geology

B = Joint pattern, direction of drive

C = ground water, joint condition.

Rock structure rating – Parameter A: general area geology (after Wickham et al., 1974)

	Basic rock type				Geological structure			
	Hard	Med.	Soft	Decomp.	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Igneous	1	2	3	4				
Metamorphic	1	2	3	4				
Sedimentary	2	3	4	4				
Type 1					30	22	15	9
Type 2					27	20	13	8
Type 3					24	18	12	7
Type 4					19	15	10	6

› Rock structure rating – Parameter B: joint pattern, direction of drive (after Wickham et al., 1974)

Average joint spacing	Strike ⊥ to axis					Strike ∥ to axis		
	Direction of drive					Direction of drive		
	Both	With dip		Against dip		Both		
	Dip of prominent joints*					Dip of prominent joints*		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
1. Very closely jointed < 2 in.	9	11	13	10	12	9	9	7
2. Closely jointed 2–6 in.	13	16	19	15	17	14	14	11
3. Moderately jointed 6–12 in.	23	24	28	19	22	23	23	19
4. Moderate to blocky 1–2 ft.	30	32	36	25	28	30	28	24
5. Blocky to massive 2–4 ft.	36	38	40	33	35	36	34	28
6. Massive > 4 ft.	40	43	45	37	40	40	38	34

Table 16c Rock structure rating – Parameter C: ground water, joint condition (after Wickham et al., 1974)

Anticipated water inflow (gpm/1000 ft)	Sum of parameters $A + B$					
	13–44			45–75		
	Joint condition**					
	Good	Fair	Poor	Good	Fair	Poor
None	22	18	12	25	22	18
Slight < 200 gpm (<13 ℓ/s)	19	15	9	23	19	14
Moderate 200–1000 gpm	15	11	7	21	16	12
Heavy > 1000 gpm (>63 ℓ/s)	10	8	6	18	14	10

*Dip: flat: 0–20 deg; dipping: 20–50 deg; and vertical: 50–90 deg.
**Joint condition: Good = tight or cemented; Fair = slightly weathered or altered; Poor weathered, altered, or open.

Fig.17 [7]

- The method experimentally developed for defining a support composed by steel arches, although there are suggested different correlations with other supports (bolts and shotcrete).
- To correlate the index RSR to the particular type of support, the "RIB RATIO“ (RR) was defined, so that different situations can be compared:

  RR = [theoretical spacing (Sd)/ real spacing (Sa)] * 100

- Each support with steel arches was related to a theoretical spacing (Sd, fig.18) calculated using Terzaghi’s expression to determine the loads in sandy grounds under water table.

Theoretical spacing of typical rib sizes for datum condition (spacing in feet). (Sd)

RIB SIZE	TUNNEL DIAMETER										
	10	12	14	16	18	20	22	24	26	28	30
4I7.7	1.16										
4H13.0	2.01	1.51	1.16	0.92							
6H15.5	3.19	2.37	1.81	1.42	1.14						
6H20		3.02	2.32	1.82	1.46	1.20					
6H25			2.86	2.25	1.81	1.48	1.23	1.04			
8WF31				3.24	2.61	2.14	1.78	1.51	1.29	1.11	
8WF40					3.37	2.76	2.30	1.95	1.67	1.44	1.25
8WF48						3.34	2.78	2.35	2.01	1.74	1.51
10WF49								2.59	2.22	1.91	1.67
12WF53										2.19	1.91
12WF65											2.35

Fig.18 [57]

- Empirically, the following expressions were derived:

(RR+ 70)(RSR+8)=6000

W_r = (D/302)*RR

W_r = (D/302)*[(6000/(RSR+8))-70]

S =24/W_r

t = 1+W_r/1.25 = D(65-RSR)/150

where

Wr = rock load (k_{ips}/ft^2 = 4.882t/m^2)

D = tunnel diameter (ft) (1ft=0.304m)

S = bolting spacing (ft) with elements of 25mm diameter and design load 24000lb (≈11t)

t = shotcrete thickness (ft)

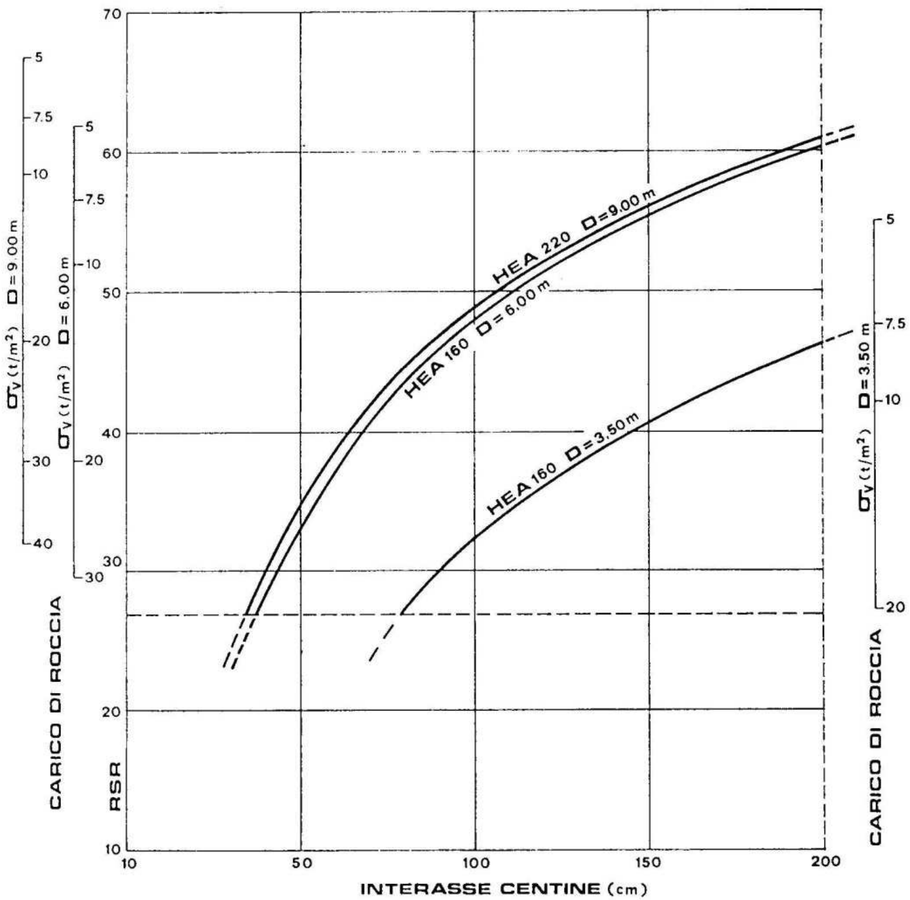
- Using the above formulas, diagrams were derived for the dermination of necessary support fig. 19-20
- In case of use of TBM a correction of the value of RSR is apported, as shown in fig.21.

II - RSR Concept (PRO→*C2a*)

- Correlazione tra RSR carico di roccia e diametro della galleria.

Diametro galleria D (m)	Carico di roccia σ_v (t/m²)					
	5	7,5	10	20,0	30,0	40,0
	Valori RSR					
3,5	55,0	47,7	41,9	27,2		
5,0	59,5	53,0	47,7	33,2	24,7	
6,0	62,5	56,8	51,9	38,0	29,4	
7,5	64,7	59,5	55,0	41,9	33,2	27,2
8,5	66,3	61,6	57,0	45,0	36,5	30,4
9,0	66,9	62,5	58,6	46,4	38,0	31,9

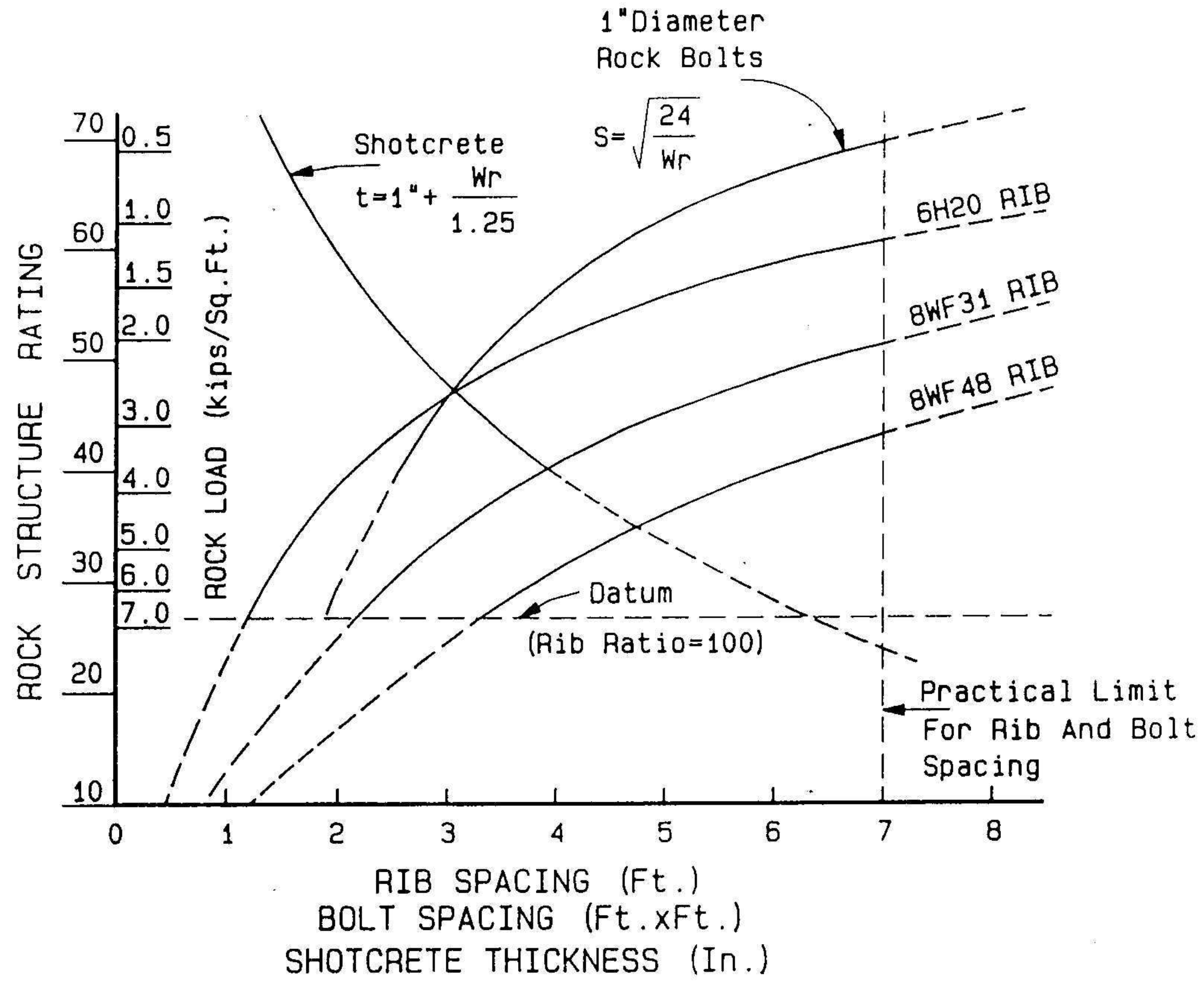
←Correlation between RSR, rock load and tunnel diameter



Dimensionamento centine per gallerie di 3,5 m, 6,0 m e 9,0 m di diametro.

←Steel ribs dimensioning for a tunnel with 3.5m, 6.0m and 9.0m of diameter

Fig.19 [13]



Support requirement for a 20 ft (6.1 m) diameter tunnel using the RSR concept (after Wickam et al., 1972).

Fig.20 [35]

Tunnel Diameter D = 15m (≈49in.) in granite;
H=100m; σc = 50-100MPa; RQD = (80)-100%
Discontinuity Spacing (2 systems + 1 random) = =0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against tunnel advance and dip= 80°, slightly weathered and rough. Dry.

→ RSR (Wickham):

- **Igneous rock of intermediate strength (Type 2);**
- **Geological structure: massive to slightly faulted (A = 20-27);**
- **B = 35 (Blocky to massive & Strike⊥axis, against dip)**
- **A+B = 55-62 → C = 22**
- **RSR = 77- 85; Wr ≈ 0.5t/m² → systematic support not required**

RMR SYSTEM (Bieniawski 1973, 1989)

Main features:

- Definition of a rock quality index RMR (Rock Mass Rating) derived from the sum of six geological-geomechanical and constructive parameters (fig.22):

RMR=a+b+c+d+e+f

a	intact rock compressive strength
b	RQD
c	Spacing of discontinuities
d	Condition of discontinuities
e	Ground water
f	Adjustment for discontinuity orientation

Fig.22: General table for RMR ratings [8].

Note:

- For a more detailed definition of the ratings recent diagrams of the same Author are used (1989) Fig.23÷27;
- when the characteristic conditions of the discontinuities result mutually exclusive (for example infilling and roughness) use A4 and not E.

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values				
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa / 1-5 MPa / < 1 MPa
	Rating		15	12	7	4	2 / 1 / 0
2	Drill core Quality *RQD*		90%-100%	75%-90%	50%-75%	25%-50%	< 25%
	Rating		20	17	13	8	3
3	Spacing of discontinuities		> 2 m	0.6-2 . m	200-600 mm	60-200 mm	< 60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous
	Rating		30	25	20	10	0
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10-25	25-125	> 125
		(Joint water press)/(Major principal σ)	0	< 0.1	0.1,-0.2	0.2-0.5	> 0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300-400	200-300	100-200	< 100
Friction angle of rock mass (deg)	> 45	35-45	25-35	15-25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1-3 m 4	3-10 m 2	10-20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1-1.0 mm 4	1-5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip-Dip 45-90°	Drive with dip-Dip 20-45°	Dip 45-90°	Dip 20-45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip-Dip 45-90°	Drive against dip-Dip 20-45°	Dip 0-20-Irrespective of strike°	
Fair	Unfavourable	Fair	

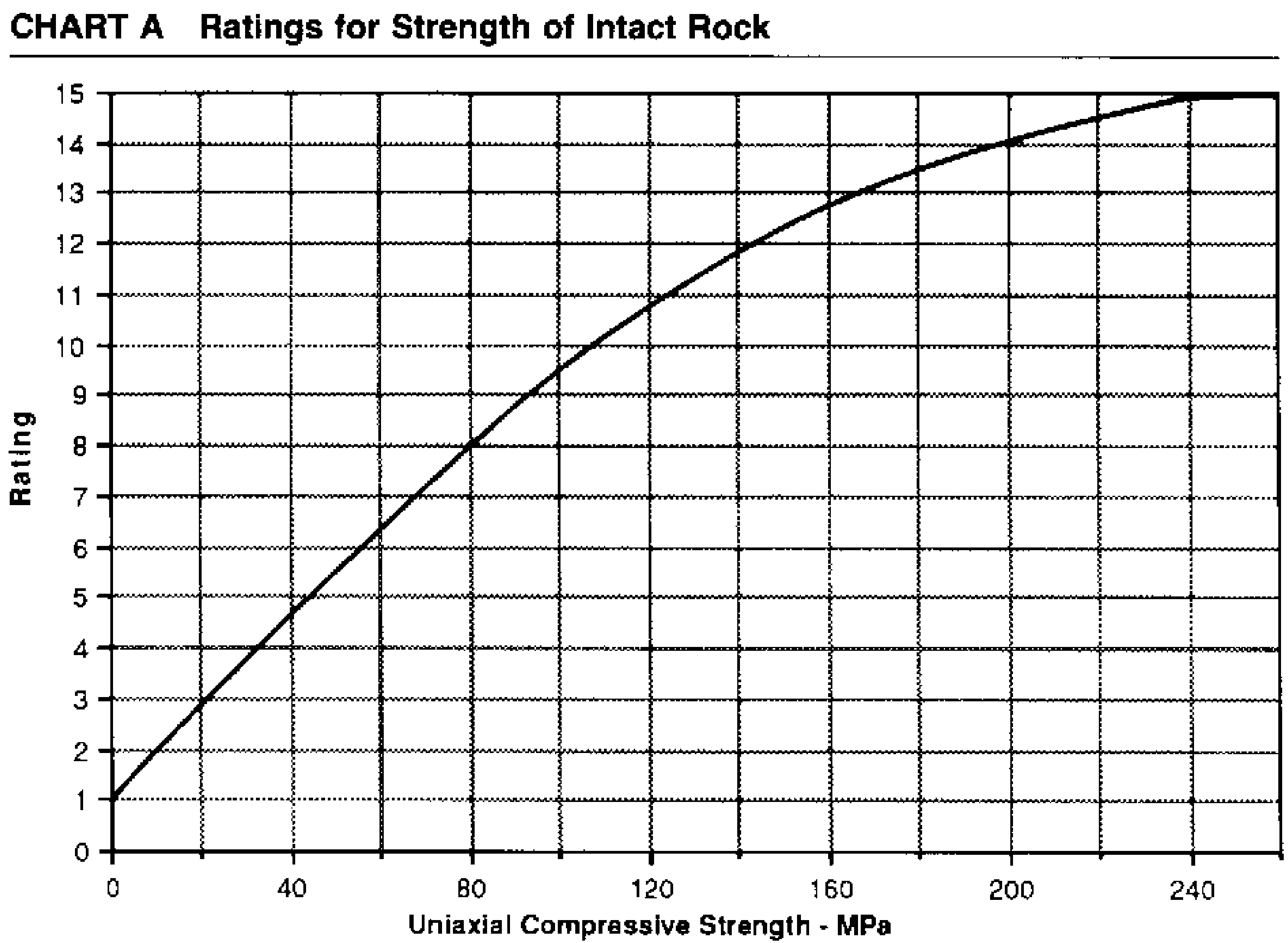


CHART A Ratings for Strength of Intact Rock
Rating
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
0
40
80
120
160
200
240
Uniaxial Compressive Strength - MPa

Fig.23 [8]

CHART B Ratings for RQD

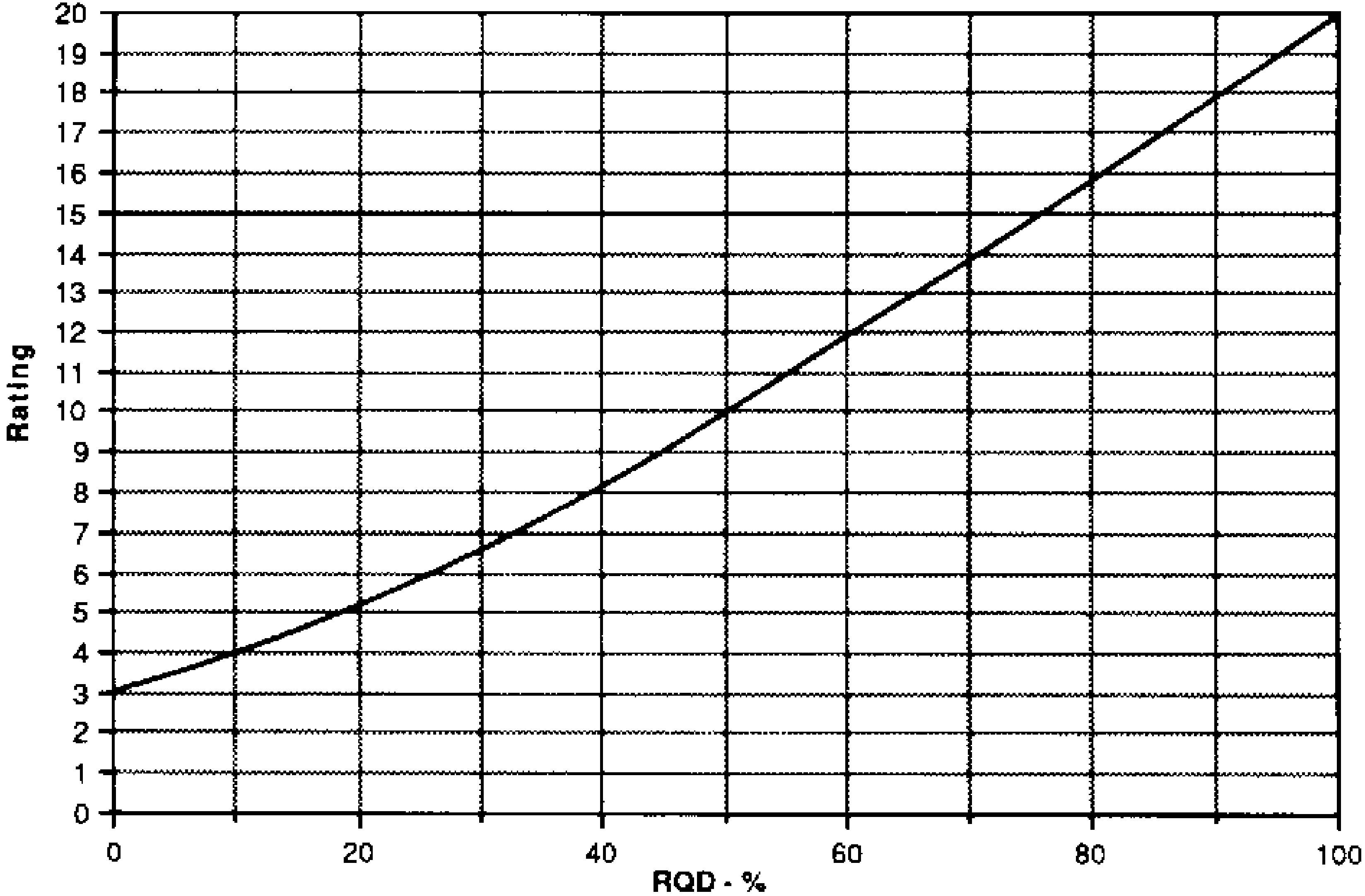


Fig.24
[8]

III - RMR System (PRO→*C2b*)

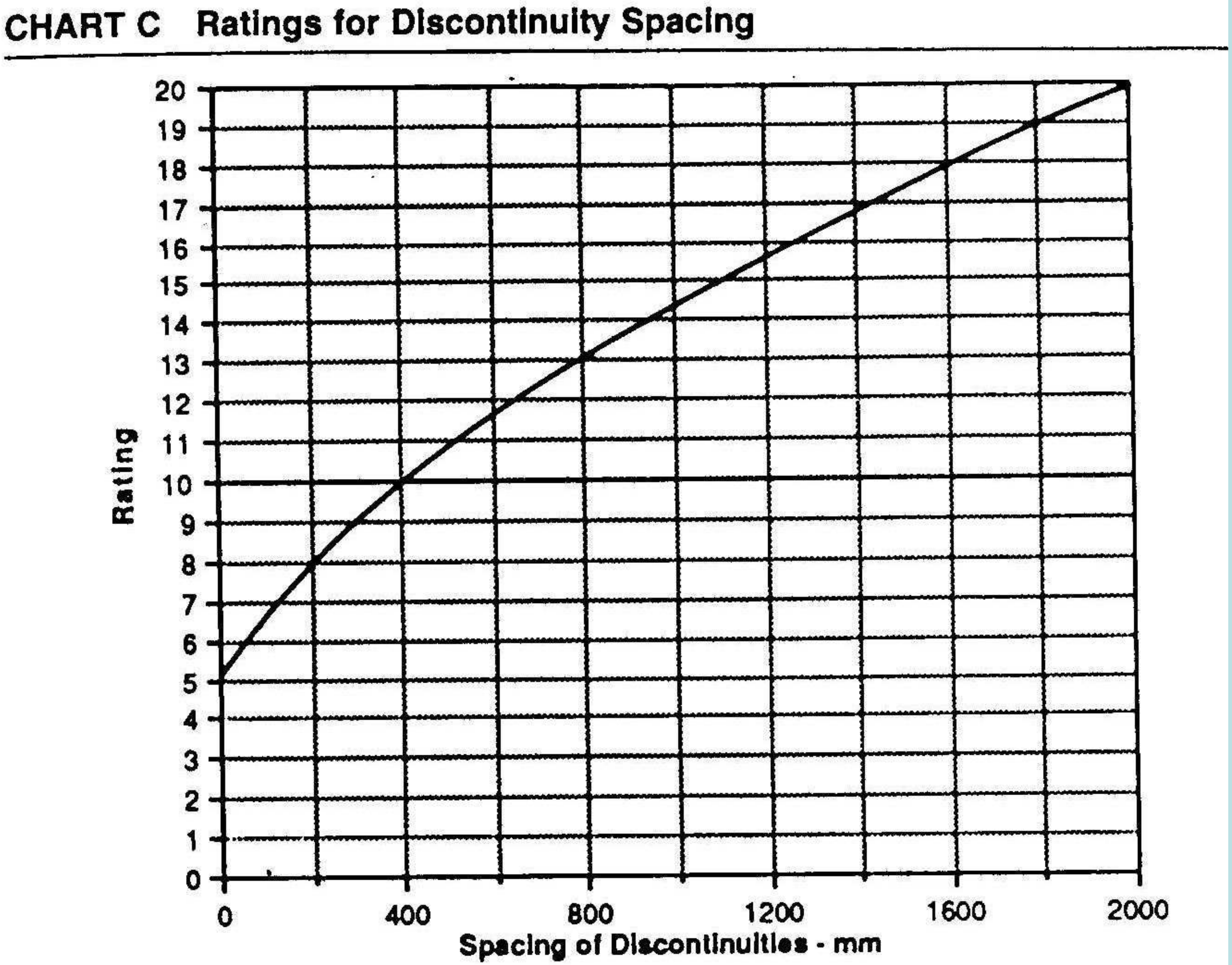
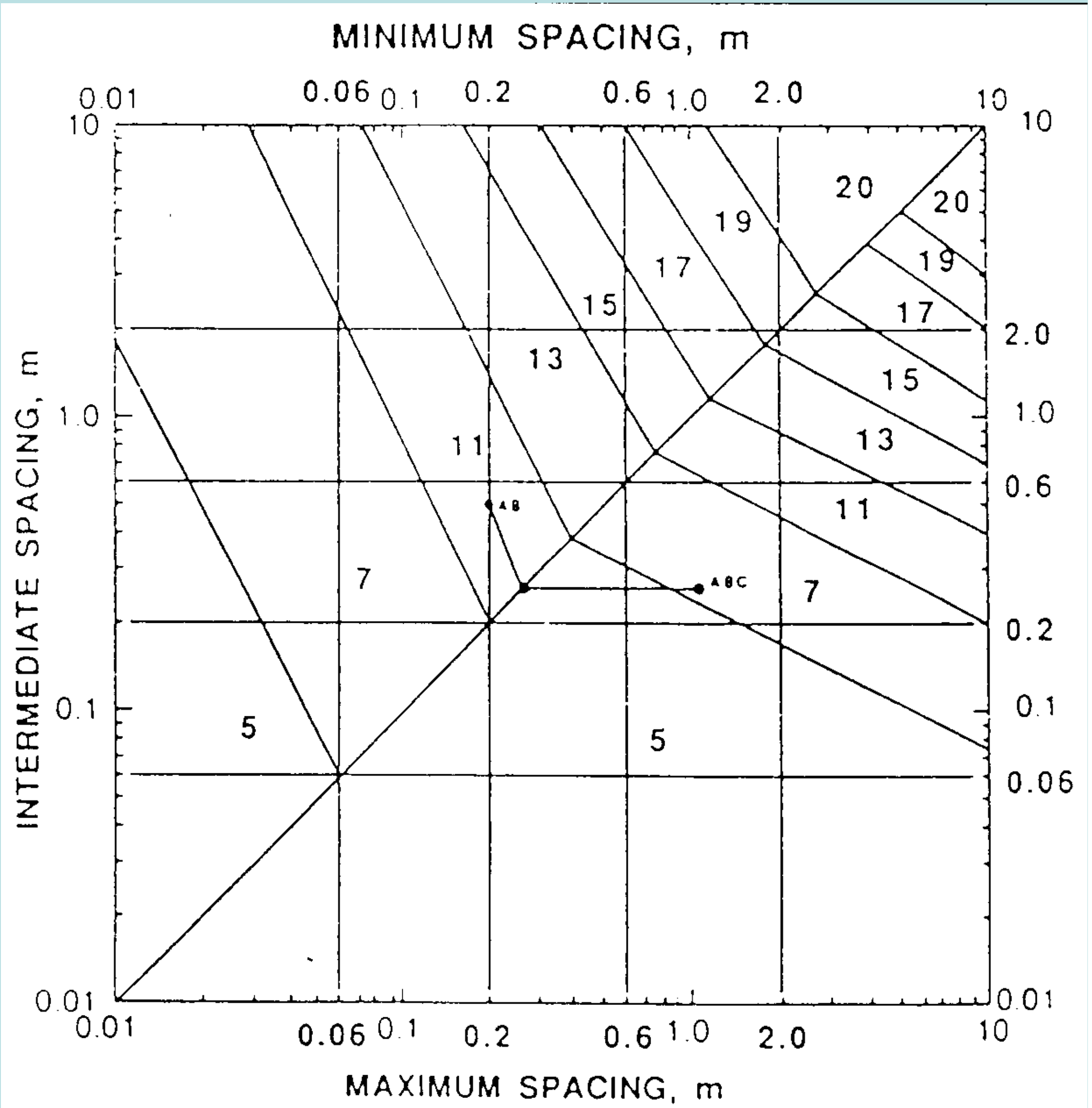


Fig.25 [8]

Figura 26 - Abacus for the rating for the Bieniawski classification to determine spacing parameter, in a rock presenting more than one discontinuity set [in the example A=0.2m, B=0.5m, C=1m from which derives a rating of 7.

[8] modified after Laubsher (1981) and Brook and Dharmaratne (1985).



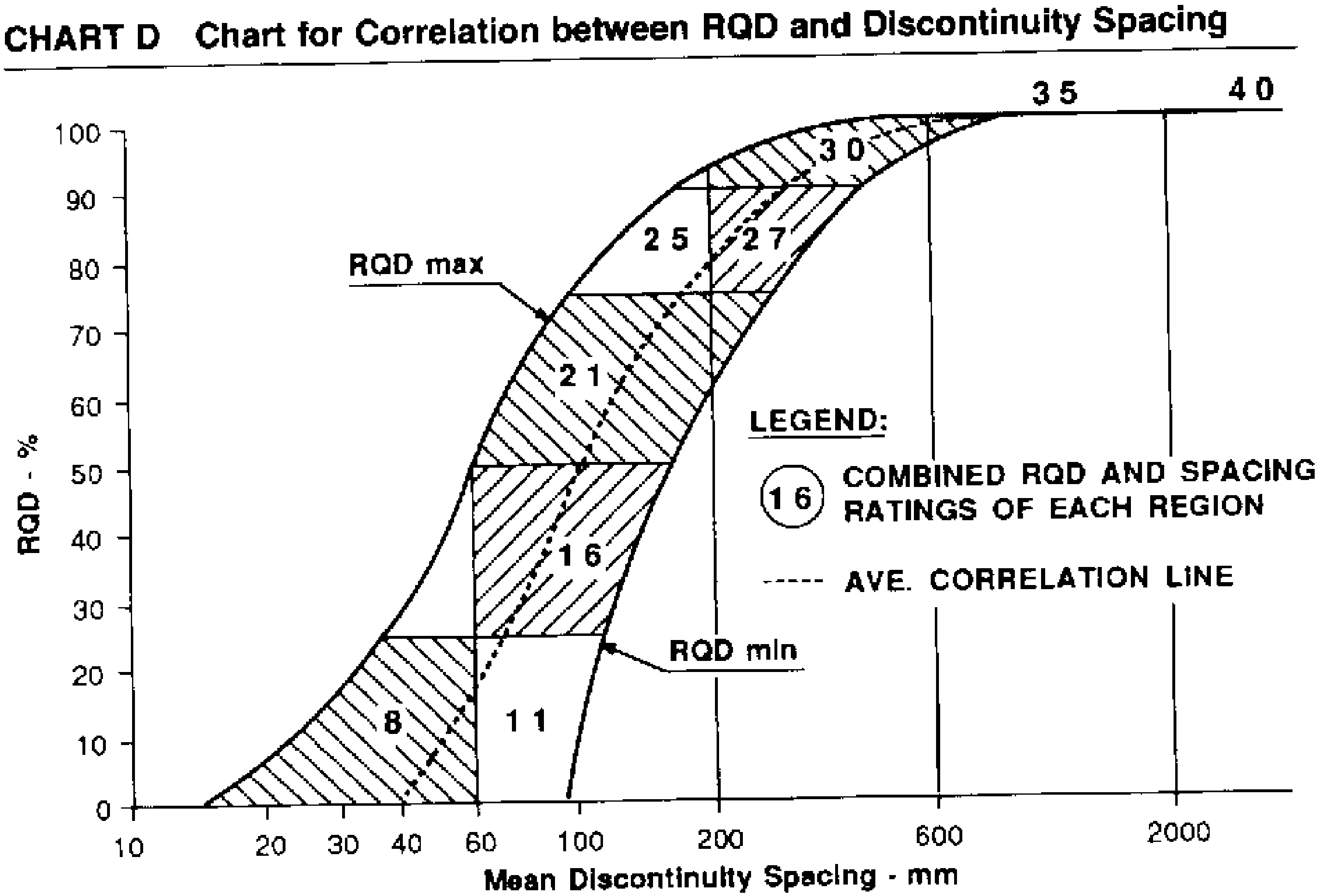


CHART D Chart for Correlation between RQD and Discontinuity Spacing
RQD - %
100
90
80
70
60
50
40
30
20
10
0
10
20
30
40
60
100
200
600
2000
Mean Discontinuity Spacing - mm
RQD max
RQD min
8
11
16
21
25
27
30
35
40
LEGEND:
16 COMBINED RQD AND SPACING RATINGS OF EACH REGION
----- AVE. CORRELATION LINE

Fig.27 [8]

Fig. 28: proposed diagram for the definition of the rating for the “f” parameter [45]

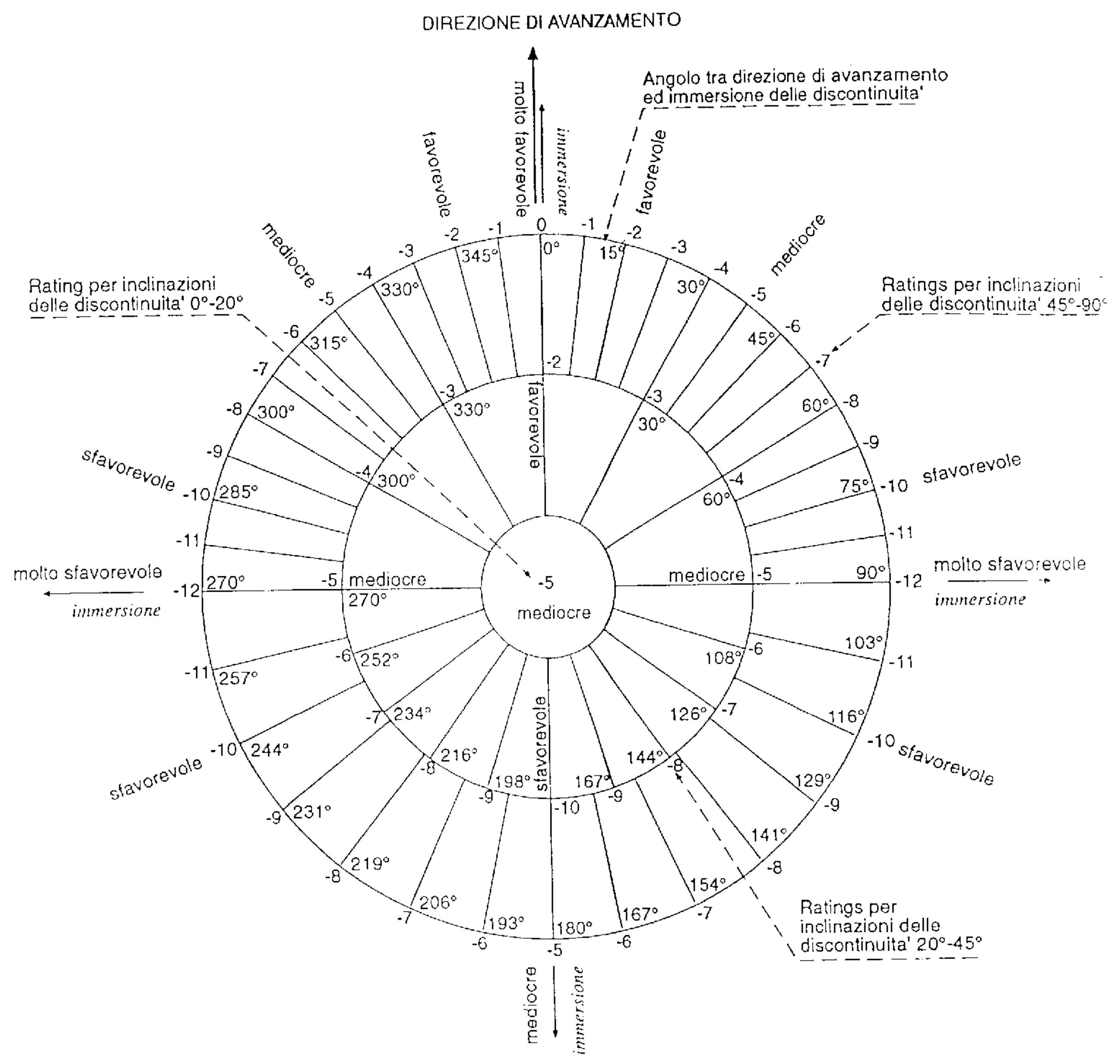


Fig. 28bis: recently some update was proposed [33b]

Use of Parameter RQD Is *Not* Recommended

This parameter was included originally among the six RMR parameters because the case histories collected in 1972 all involved RQD. Over the years it became apparent that RQD was difficult to determine at tunnel face, being directed to borehole characterization, and it was subsequently combined with parameter "discontinuity spacing" ("joint" spacing)—and named "spacing density" since the two are interrelated. For the best practical use, this led to the preferred use of "fracture frequency" as an invert of "fracture density"—as depicted in Figure 14. Neither of these approaches changed the basic allocation of rating values to these parameters.

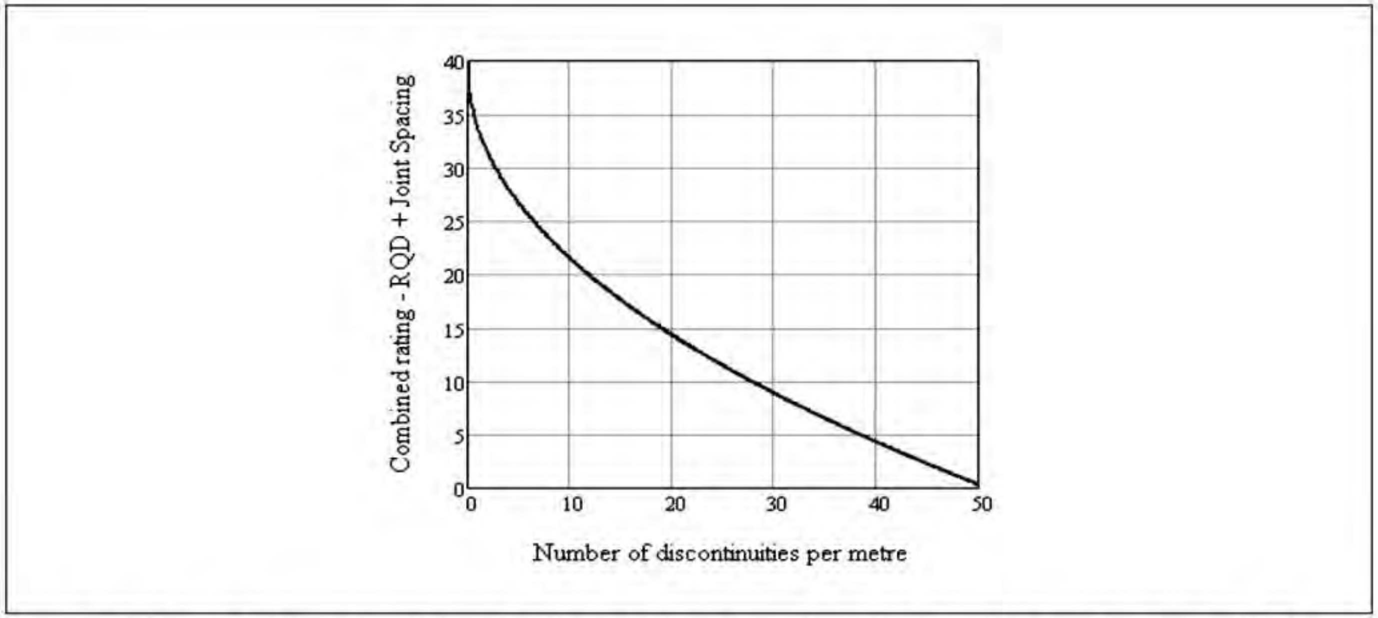


Figure 14. Chart D for combined rating of the discontinuity density parameters: RQD plus discontinuity spacing

rating (Spacing+RQD) =39.94-6.157*landa^0.476 (G.Russo,2014)

- In function of the RMR values 5 technical classes are defined from I (very good rock) to V (very poor rock).
- The sum of the first 5 parameters (except “f”) supplies **BMR** (Basic Mass Rating), connected to the main parameters of rock strength and deformability:

$$c = 5 \cdot BMR \ (kPa)$$

$$\varphi = 5 + BMR/2 \ (°)$$

$$E_d = 2 \cdot BMR - 100 \ (GPa,\ per\ BMR > 50)$$

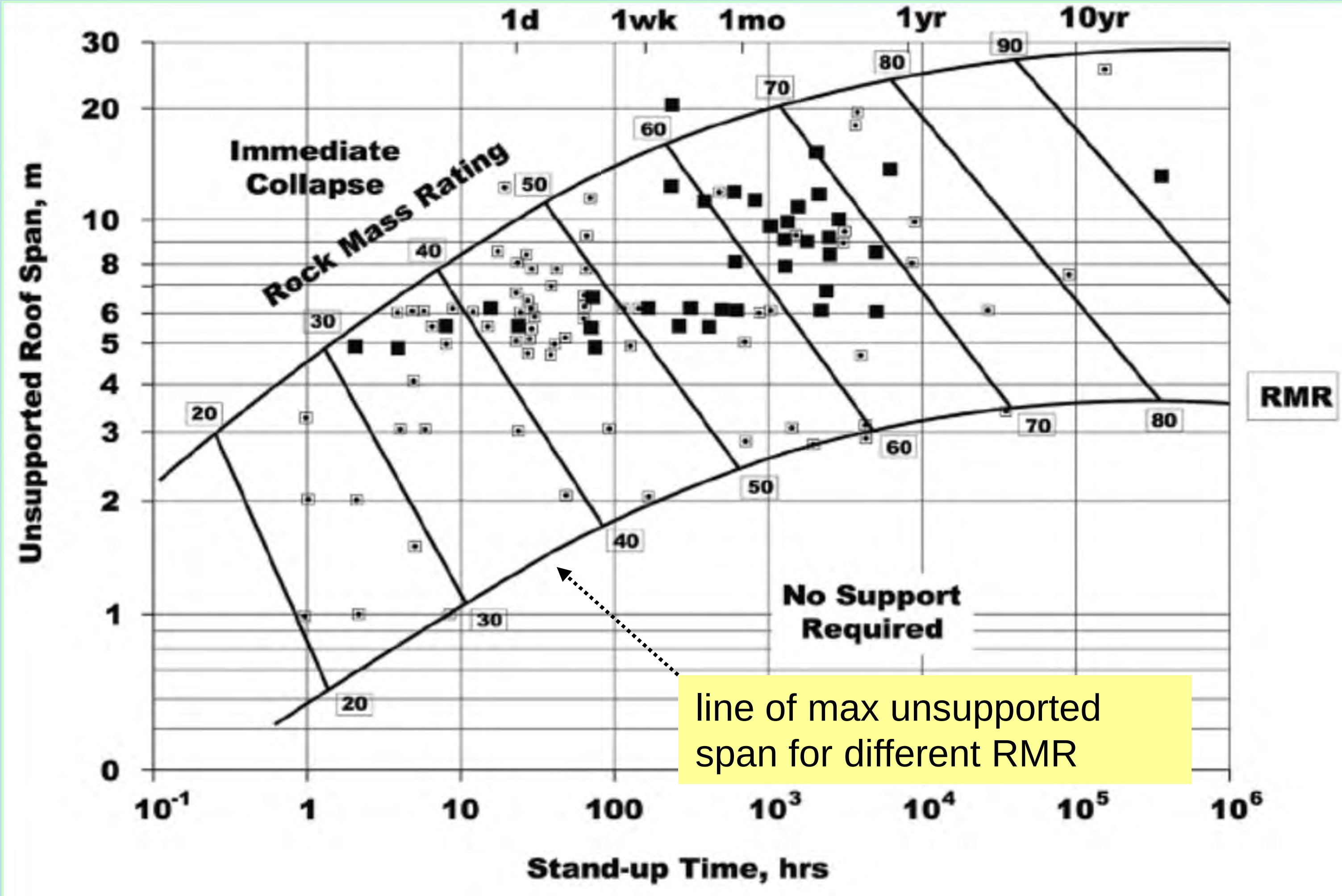
$$E_d = 10^{(BMR-10)/40} \qquad (1)$$

Note: (1) The original version of Serafim e Pereira (1983) considered the index of RMR. Other expressions, proposed for the determination of Hoek and Brown parameters, have been recently made with the GSI index and are regarded in a specific chapter.

Typical stand-up times for different roof spans of tunnel are proposed, according to the concepts proposed by Lauffer

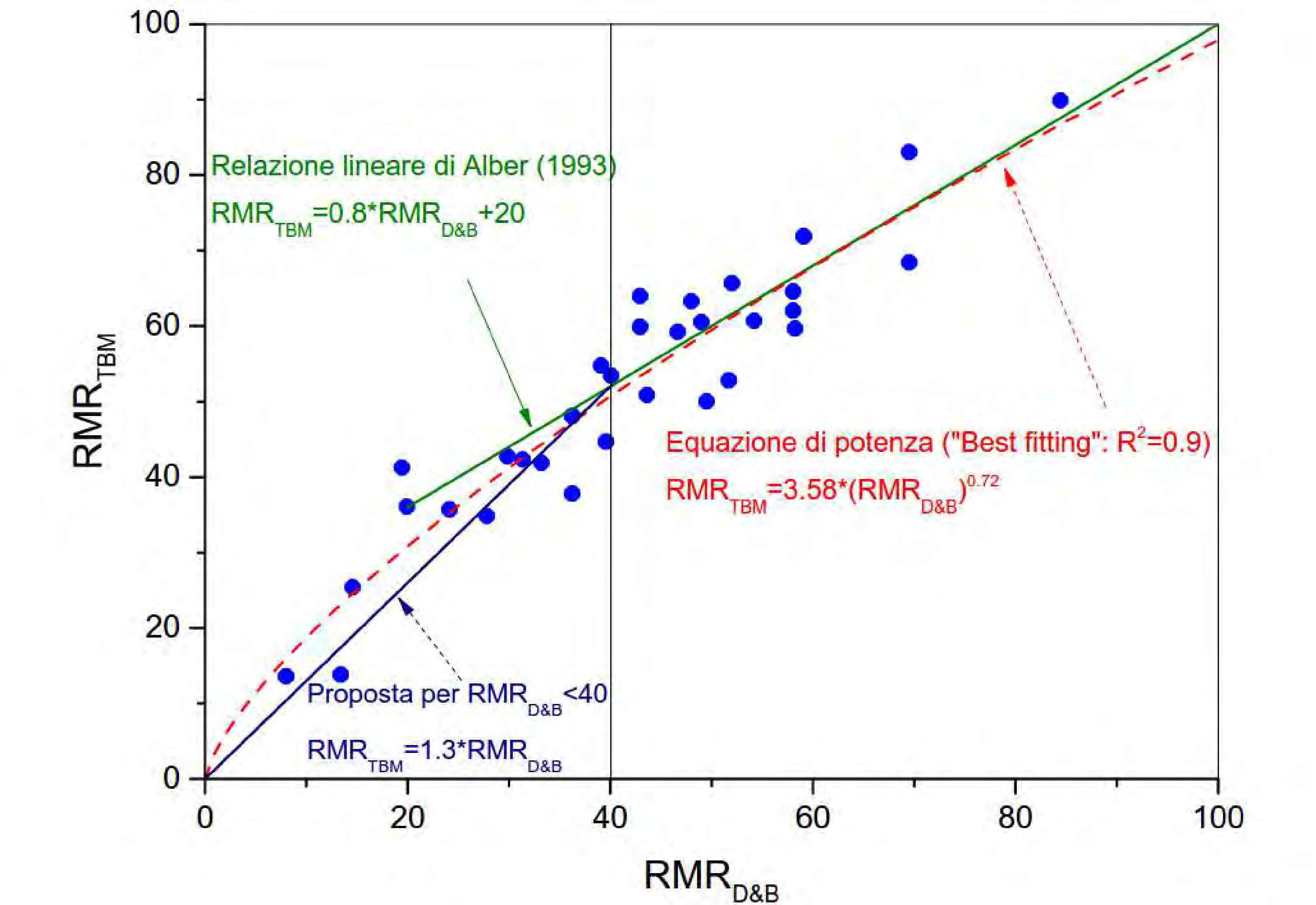
Fig.29 [7]: traditional excavation→

Note: The points represent collapse limit conditions registered



More properly, the following equation is proposed [10] in combination of the D&B chart: $RMR_{TBM}=0.8RMR_{D\&B}+20$

Anyway this is not on the safe side for RMR<40 and the following is preferred



G. Russo, 2014

- The load (P) on the support and the active rock height (h_t) can be derived by the following equations (Unal, 1983,[7])

$$P = \frac{100 - RMR}{100} * \gamma * B = \gamma * h_t \qquad h_t = \frac{100 - RMR}{100} * B$$

where

B= tunnel width (m)

γ = rock mass density (kg/m^3)

In the previously cited update [33b]:

Design Rock Load: $$P_r = \frac{100 - RMR}{100} .10m. \left(\frac{Span}{10m} \right)^{\frac{1}{2}} .\rho_r .\gamma_r$$

where γ_r is a partial factor and ρ_r is rock density

Associated to each class, quantitative indications about ways of tunnelling and which support is necessary are given (fig.31), with the hypothesis of:

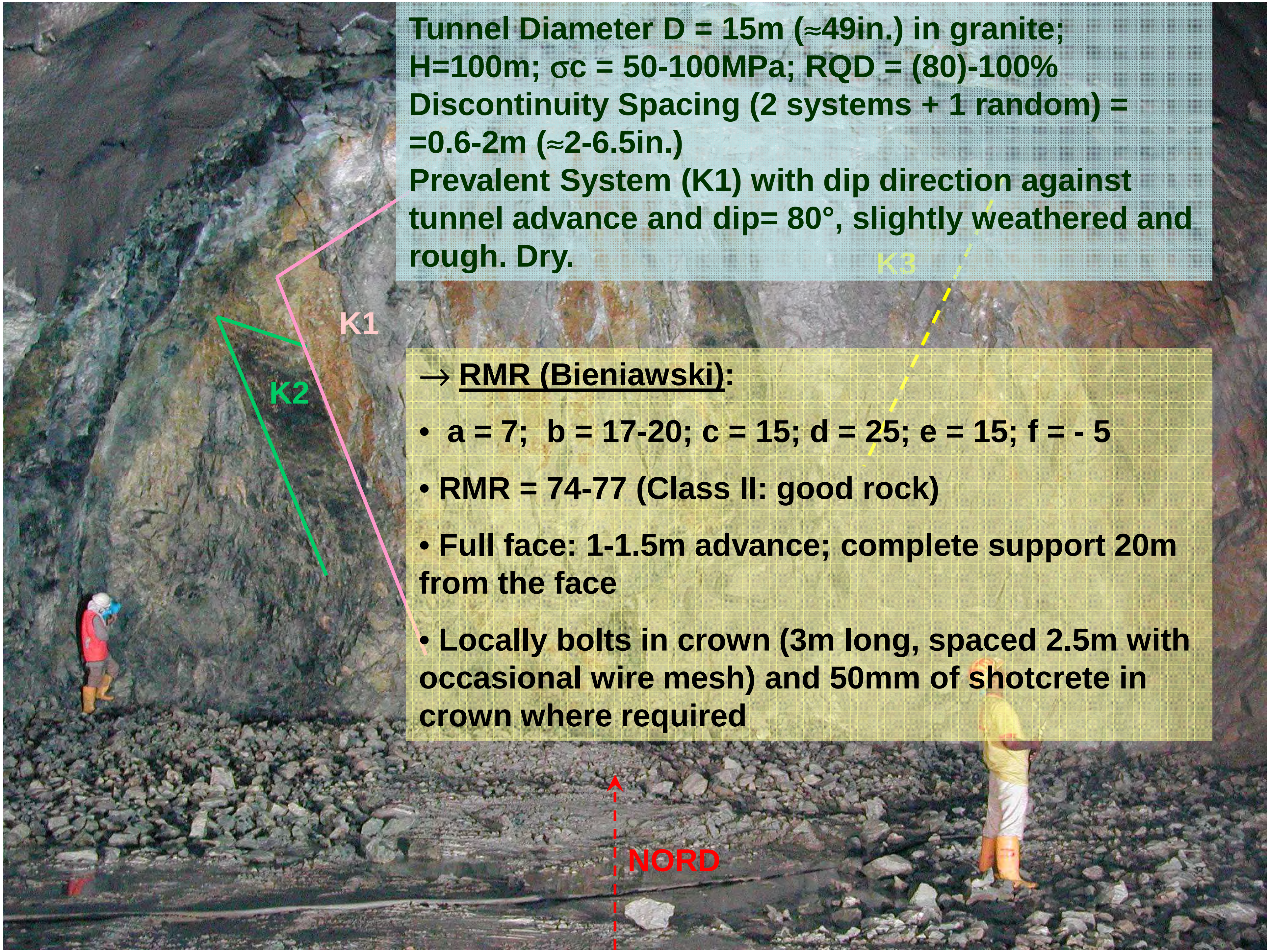
- “horse-shoe” shaped tunnel section
- tunnel width 10m
- vertical stress in situ less than 25MPa
- tunnelling with a traditional drill & blast method

III - RMR System (PRO→*C2b*)

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I – Very good rock *RMR*: 81-100	Full face, 3 m advance	Generally no support required except spot bolting		
II – Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III – Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown and 30 mm in sides	None
IV – Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
V – Very poor rock *RMR*: < 21	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert	150-200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert

Fig.31 [7]

Tunnel Diameter D = 15m (≈49in.) in granite;
H=100m; σc = 50-100MPa; RQD = (80)-100%
Discontinuity Spacing (2 systems + 1 random) =
=0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against tunnel advance and dip= 80°, slightly weathered and rough. Dry.



→ **RMR (Bieniawski)**:

- a = 7; b = 17-20; c = 15; d = 25; e = 15; f = - 5
- RMR = 74-77 (Class II: good rock)
- Full face: 1-1.5m advance; complete support 20m from the face
- Locally bolts in crown (3m long, spaced 2.5m with occasional wire mesh) and 50mm of shotcrete in crown where required

Applicative example

A tunnel with its axis orientated south-north should be excavated in a quartz rock mass, considering the entry data:

- B = 10m (section “horse-shoe”)
- σ_c = 80MPa
- RQD = 70%
- The most significant set of discontinuities is characterized by:
    - joint set orientation 45/70 (dip direction /dip)
    - persistence = 15m; aperture = 0.1-1mm;
    - slightly rough, without infilling , moderately alterated;
- wet rock mass.
- Tasks:

RMR, BMR, rock mass class, rock loads on the supports ed necessary stabilization interventions.

Solution

Parameter	Reference	Value	Rating
a	Fig. 23	σ_c=80MPa	8
b+c	Fig. 27	RQD=70%	21
d	Fig. 22		1+4+3+6+3
e	Fig. 22	wet	7
f	Fig. 28		-6
		RMR	47
		Class	III
		BMR	53
		ht = [(100-RMR)/100]*B	5.3m

Construction assessments (from fig.31):

- **Top heading and bench: 1.5-3m advance in top heading; commence support after each blast and complete support 10m from the face;**
- **Support: Systematic bolts (4m long, spaced 1.5-2m in crown and walls, with wire welded mesh in crown) and shotcrete (50-100 /30mm in crown/sides).**

Recently, the **Rock Mass Excavability (RME)** index was proposed by Bieniawski et al. [10bis] for estimating the performance of different types of TBM

The Rock Mass Excavability index is calculated on the basis of the following parameters:

- Uniaxial compressive strength of intact rock
- Drillability index
- Rock mass discontinuities
- Stand-up time
- Groundwater inflow

Uniaxial compressive strength of intact rock [0-25 points]					
σ_c (MPa)	<5	5-30	30-90	90-180	>180
Rating	4	14	25	14	0

Drillability [0-15 points]					
DRI	<80	80-65	65-50	50-40	<40
Rating	15	10	7	3	0

Discontinuities in front of the tunnnel face [0-30 points]										
Homogeneity			Number of joints per meter					Orientation with repect to tunnel axis		
	Homogeneous	Mixed	0-4	4-8	8-15	15-30	>30	Perpendicular	Oblique	Parallel
Rating	10	0	2	7	15	10	0	5	3	0

Stand up time [0-25 points]					
Hours	<5	5-24	24-96	96-192	>192
Rating	0	2	10	15	25

Groundwater inflow [0-5points]					
Liter/sec	>100	70-100	30-70	10-30	<10
Rating	0	1	2	4	5

Fig.31a: RME rating system

III - RME method (2008)

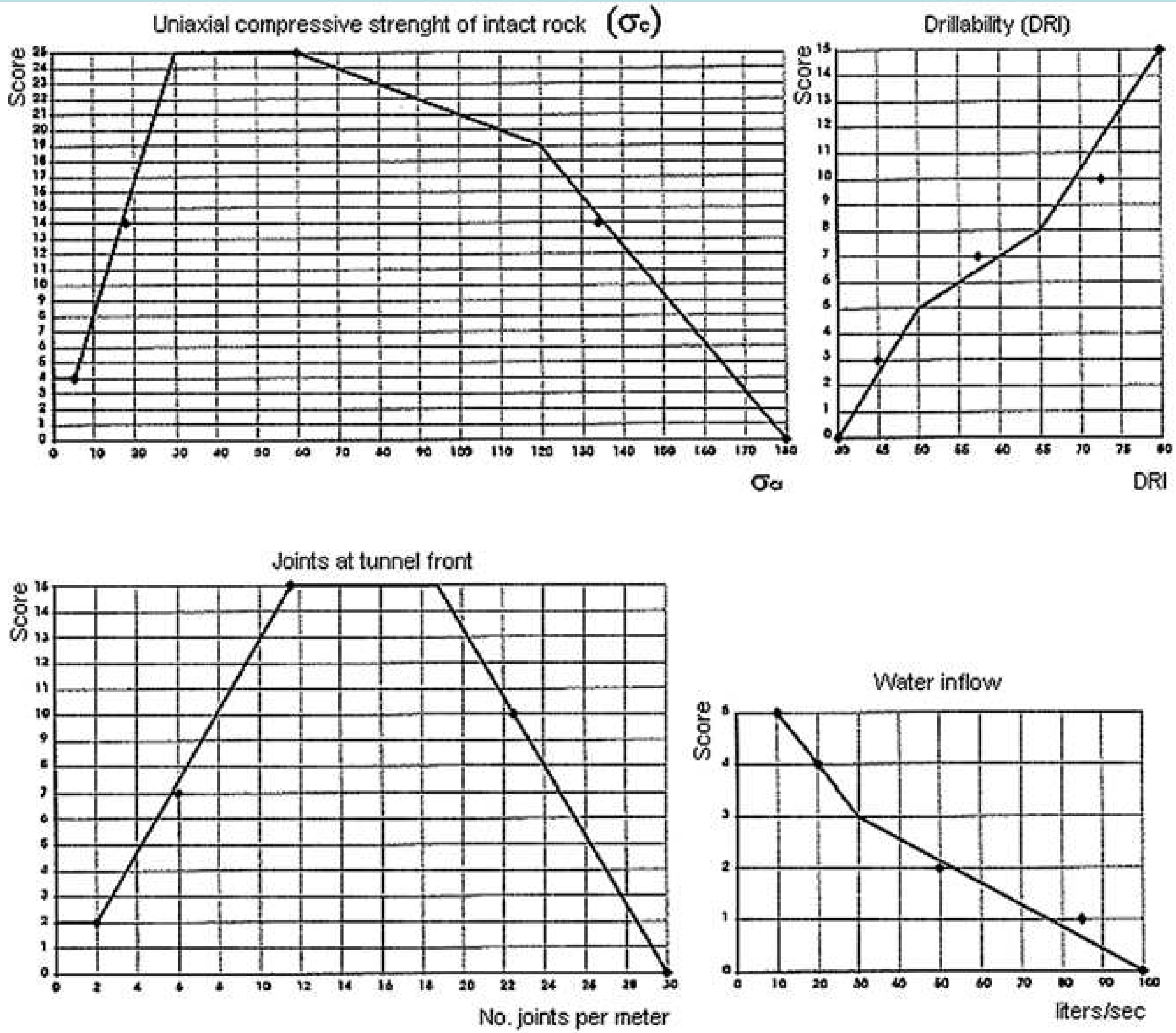
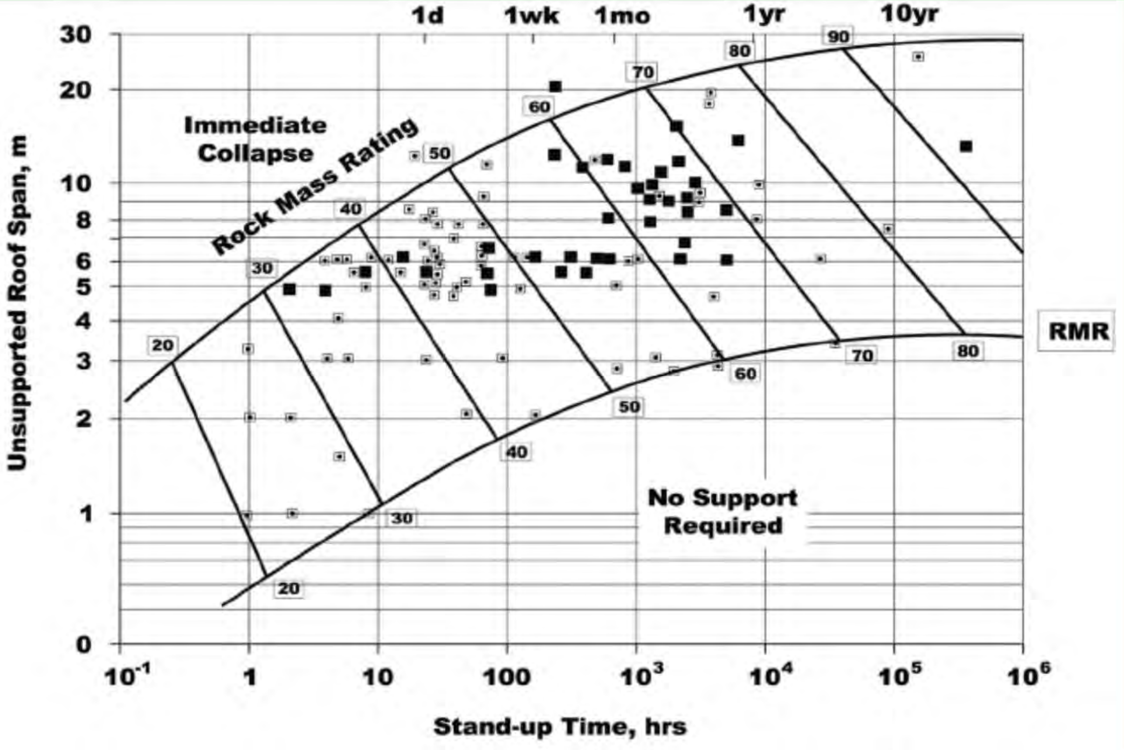


Fig.31b: detailed RME rating system



III - RME method (2008)

Fig.31c: Stand-up time rating

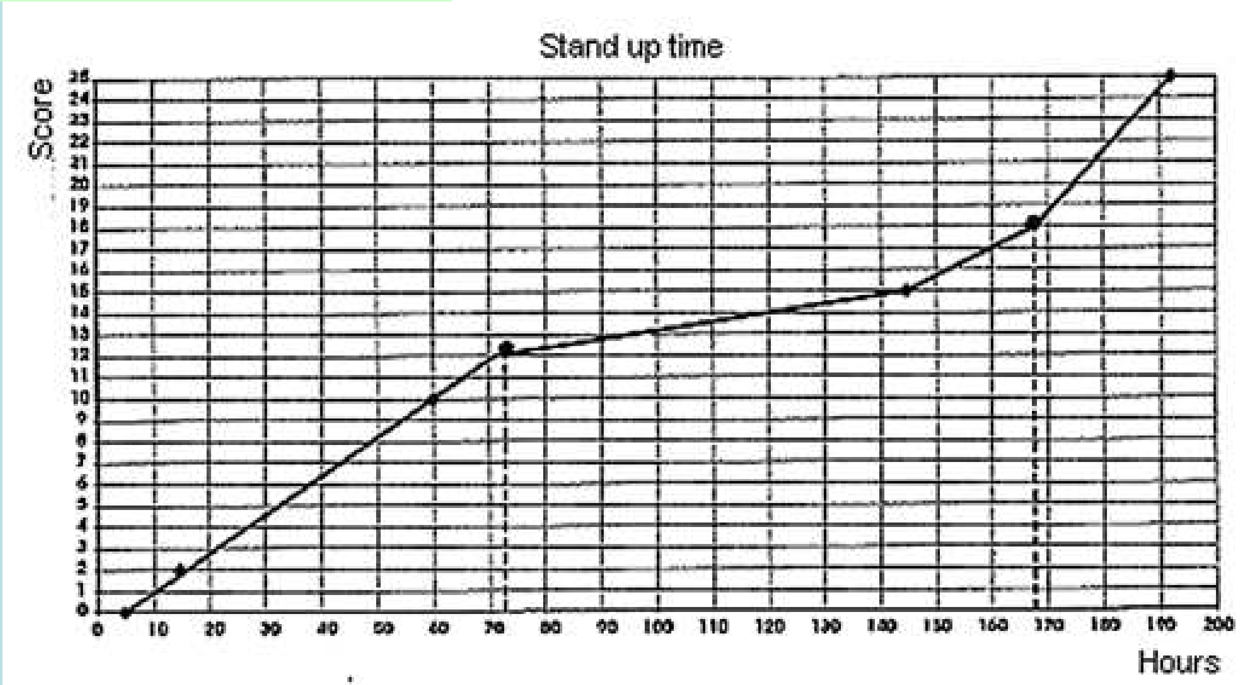
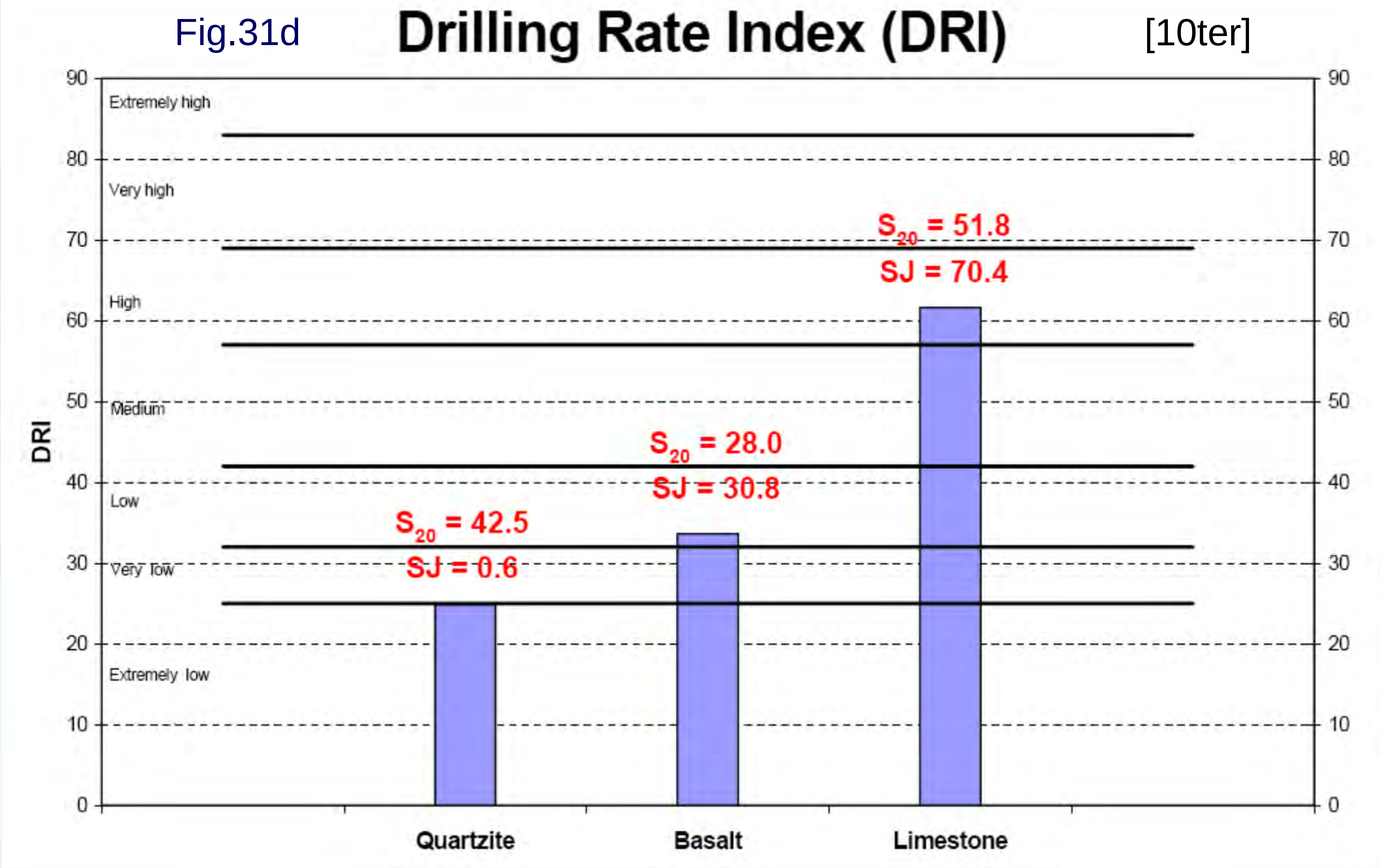


Fig.31d [10ter]



The following approximate correlation is reported by Palmstrom [40]:

$DRI \approx 1000 * \sigma_c^{-0.6}$ (with σ_c in MPa)

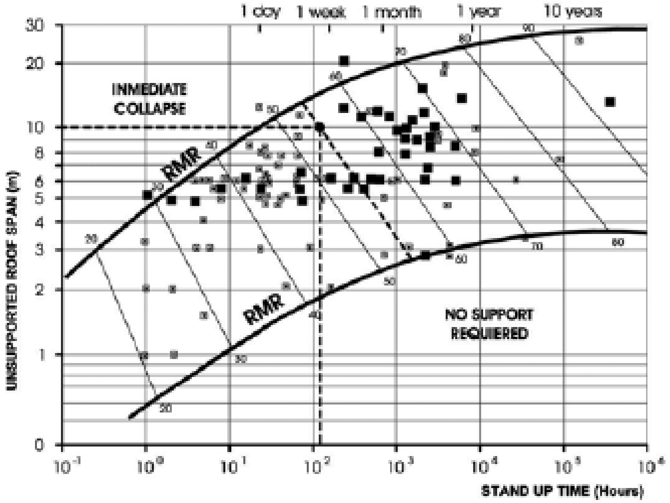
III - RME method (2008)

RME CALCULATION

TUNNEL: Abdalajís P.K.: 3+245 LITHOTYPE.: limestones

OVERBURDEN: 250 m R.M.R: 43 Ø EXCAVATION: 10,0 m

1.- Stand up time estimation.



RMR_{TBM}=0,80·RMR+20=...54... Stand up time: ...110...hours

2.- RME Parameters evaluation.

* Uniaxial compressive strenght of intact rock		σc=...67...MPa ⇒	24
* Drillability		DRI=...76... ⇒	13
* Joints			
Homogeneity:...Homogeneous... ⇒	10	⇒	25
Nº joints per meter:...21... ⇒	12		
Orientation with tunnel axis:... ⇒	3		
* Stand up time:		hours ...110... ⇒	14
* Water inflow at tunnel face:		l/s ...65... ⇒	1,5

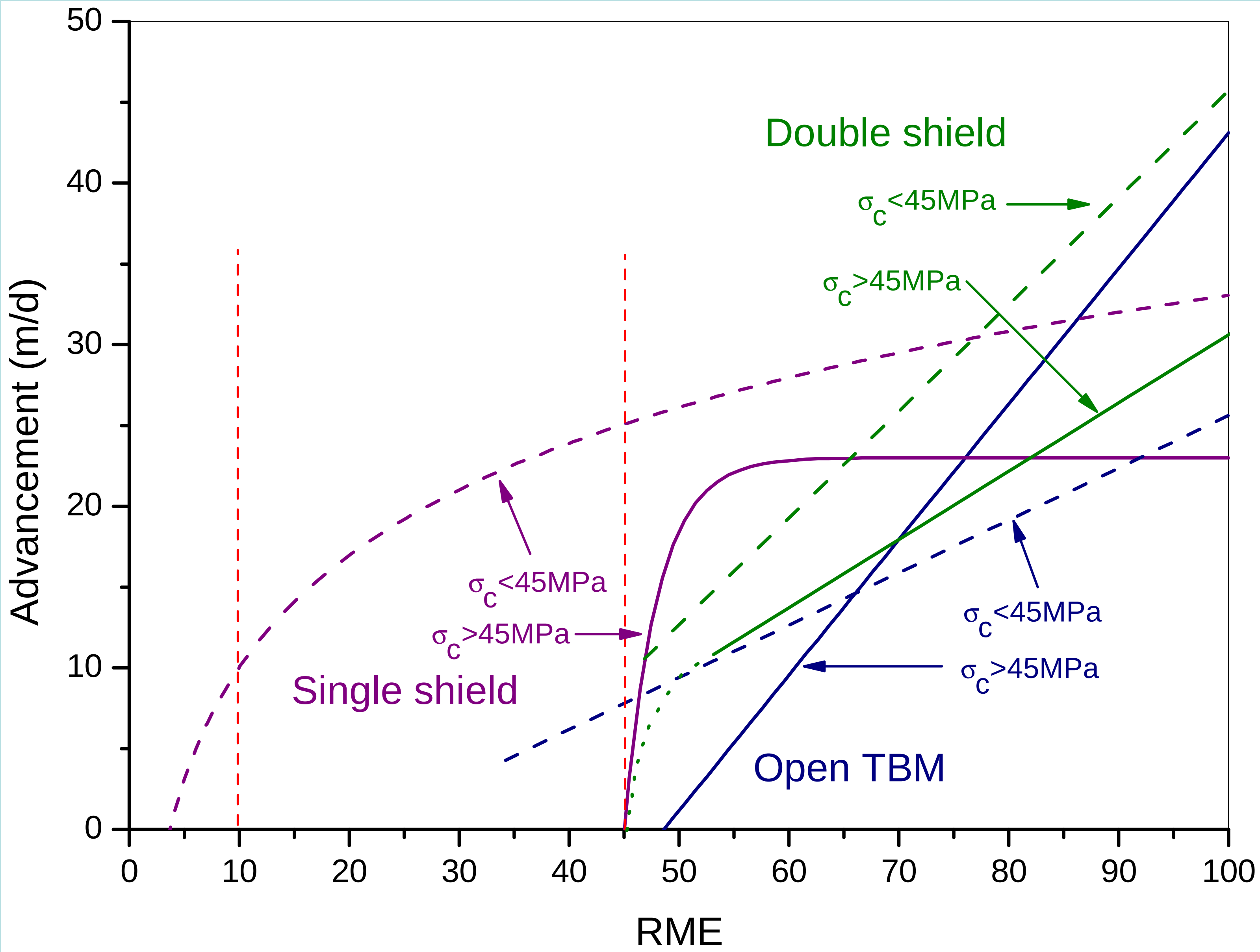
Calculated RME 07 | **77,5**

Fig.31e: example RME calculation (Bieniawski et al. 07)

- Mainly on the basis of practical experience, the RME index is correlated to the **Average Advance Rate (ARA)**
- In particular, one theoretical (t) and one real (r) ARA are considered, the latter taking into account some practical correction factors.
- The following correlations have been derived:

TBM type	n.	σ_c>45MPa	σ_c<45MPa
Open TBM	49	ARAt=0.839*RME-40.8 (R=0.763) → *limitation: no data for RME<35*	ARAt=0.324*RME-6.8 (R=0.729)
Single Shield	62	ARAt=23*[1-242$^{(45-RME)/17}$] (R=?)	ARAt=10LnRME-13 (R=0.784) →*limitation: few data for RME<35*
Double Shield (using grippers)	225	ARAt=0.422*RME-11.6 (R=0.658)	ARAt=0.661*RME-20.4 (R=0.867) → *limitation: only for RME>45*

Fig.31f: Graphical representation of RME-ARA correlation



RME

Note: The concept of "Optimized" Double shield may be considered considering for low RME values the single shield advance mode and for high values the double shield mode

The real Average Advance rate is calculated according the following equation:

$$ARAr = ARAt * F_E * F_A * F_D$$

Where

F_E= factor of crew efficiency = $0.7+F_{E1}+F_{E2}+F_{E3}$
F_A= factor of team adaption to the terrain
F_D= factor of tunnel diameter

Note: Remember correction in the text

III - RME method (2008)

Criteria for evaluation of coefficients
F_{E1}, F_{E2} and F_{E3} (after Grandori [5])

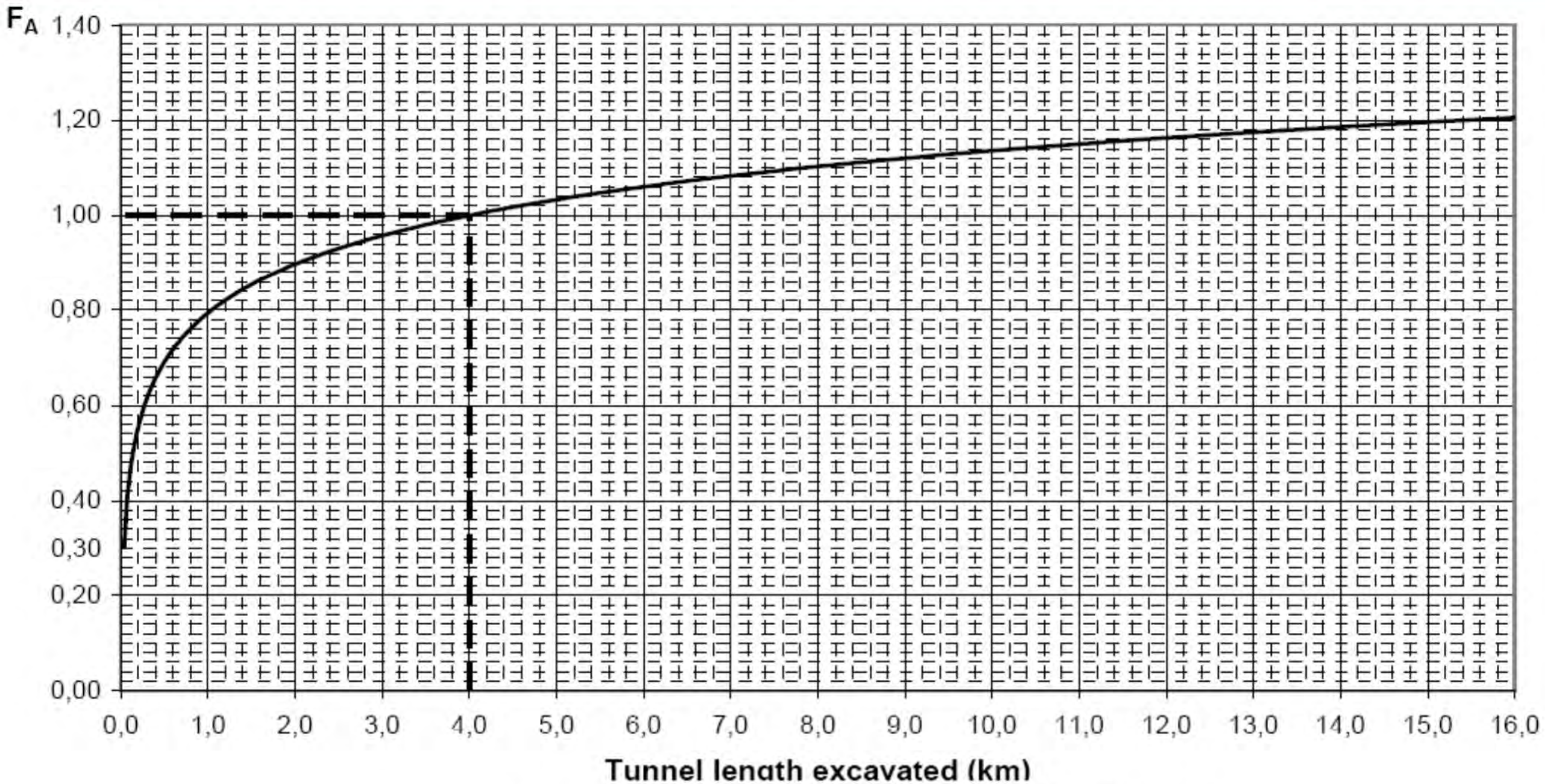
Contractor's TBM experience	No experience	1 to 5 tunnels built	6 to 10 TBM tunnels built	11 to 20 TBM tunnels built	>21 TBM tunnels built
Value of F_{E1}	0	0,05	0,10	0,15	0,2

Qualifications of the tunnelling crew	Little trained and none with TBMs	Trained but none with TBMs	Trained overall and with TBMs
Value of F_{E2}	0	0,1	0,15

Resolutions of disputes	TBM manufacturer rep on site	No TBM manufacturer rep on site	Time to resolve problems: < 1 month	Time to resolve problems: > 1 month
Value of F_{E3}	0.075	0	0.075	0

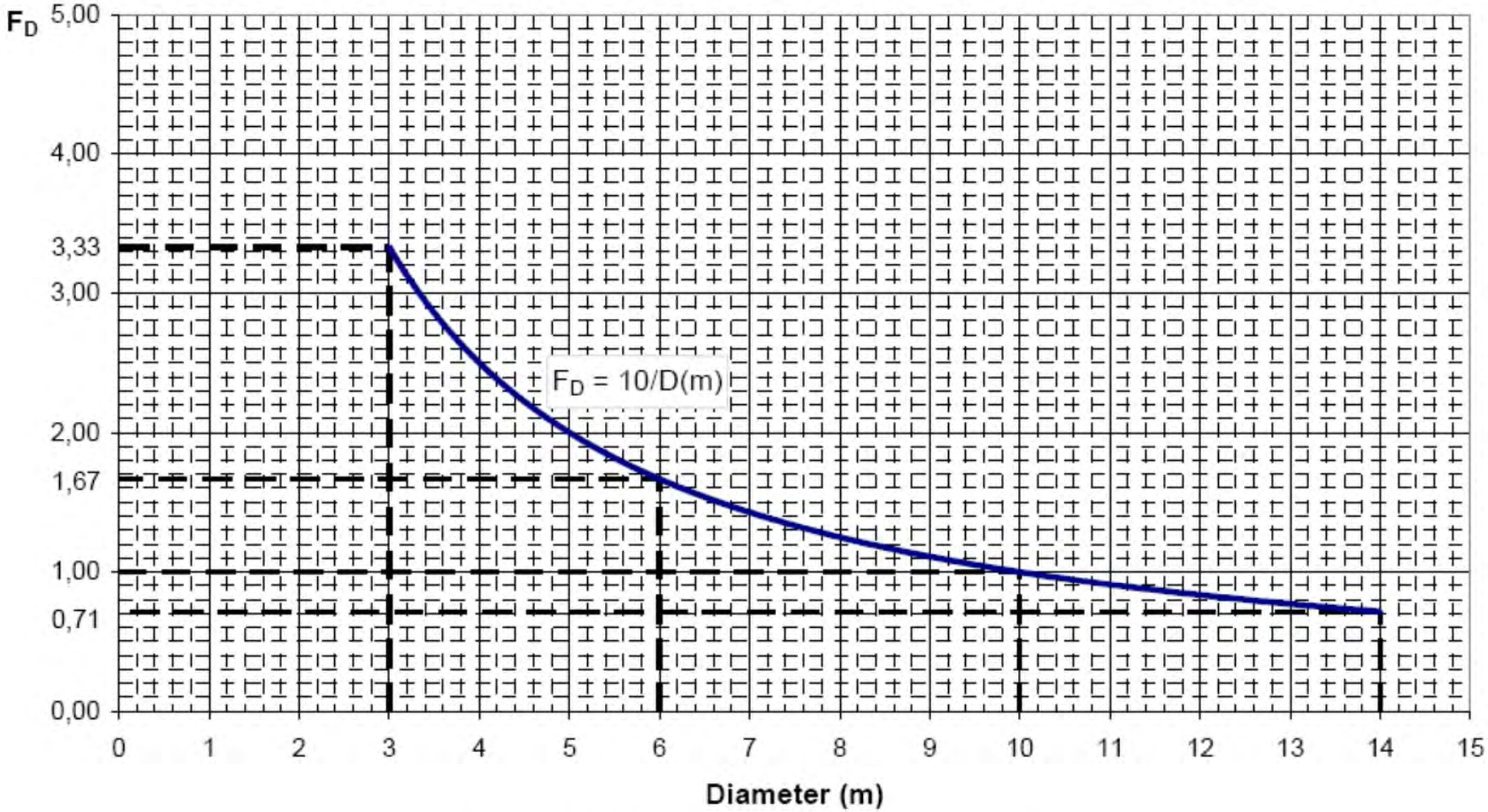
Fig.31g: F_E rating

$F_E = 0.7 + F_{E1} + F_{E2} + F_{E3}$



Variation of factor F_A with the excavated tunnel length

Fig.31h: F_A assessment



Variation of factor F_D with tunnel diameter

Fig.31i: F_D assessment

Q-SYSTEM (Barton et al., 1974-1999)

Main features:

- Rock mass quality index Q (variable from 0.001 to 1000) obtained by the following equation:

$$Q = \frac{RQD}{J_n} * \frac{J_r}{J_a} * \frac{J_w}{SRF}$$

RQD	Rock Quality Designation
Jn	joint set number
Jr	joint roughness number
Ja	joint alteration number
Jw	joint water reduction factor
SRF	joint stress reduction factor

Q is variable from 0.001 to 1000..

Q = 1000 (or better)

Q = 100/0.5 x 4/0.75 x 1/1

Q = 0.001 (or worse)

Q = 10/20 x 1/8 x 0.5/ 20

Fig.31 l

from Barton, 2006

IV: Q-System (PRO→*C2c*)

$\frac{RQD}{J_n}$	→*block size*
$\frac{J_r}{J_a}$	→ *inter-block shear strength*
$\frac{J_w}{SRF}$	→ *active stress*

The table on fig. 32 gives the classification of individual parameters used to obtain the Tunnelling Quality Index Q for a rock mass.

Fig.32 [20] : Q-System rating assessment table (1of2)

A. Rock quality designation (RQD)

Very poor	RQD = 0 - 25%
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

Notes:

(i) Where RQD is reported or measured as < 10 (including 0), a nominal value of 10 is used to evaluate Q

(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate

B. Classification with ratings for the Joint set number (Jn)

Massive, no or few joints	Jn = 0.5 - 1
One joint set	2
One joint set plus random	3
Two joint sets	4
Two joint sets plus random	6
Three joint sets	9
Three joint sets plus random	12
Four or more joint sets, heavily jointed, "sugar-cube", etc.	15
Crushed rock, earth-like	20

Notes: *(i) For tunnel intersections, use (3.0 x Jn); (ii) For portals, use (2.0 x Jn)*

C. Classification with ratings for the Joint roughness number (Jr)

a) Rock-wall contact, b) rock-wall contact before 10 cm shear	
Discontinuous joints	Jr = 4
Rough or irregular, undulating	3
Smooth, undulating	2
Slickensided, undulating	1.5
Rough or irregular, planar	1.5
Smooth, planar	1.0
Slickensided, planar	0.5

***Note**: i) Descriptions refer to small scale features, and intermediate scale features, in that order*

c) No rock-wall contact when sheared	
Zone containing clay minerals thick enough to prevent rock-wall contact	Jr = 1.0
Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0

Notes:

i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m

ii) Jr = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oreintated for minimum strength

D. Classification with ratings for the Joint alteration number (Ja)

	JOINT WALL CHARACTER		Condition	*Wall contact*
Contact between joint walls	CLEAN JOINTS:	Healed or welded joints:	filling of quartz, epidote, etc.	Ja = 0,75
		Fresh joint walls:	no coating or filling, except from staining (rust)	1
	JOINTS WITH COATING OF:	Slightly altered joint walls:	non-softening mineral coatings, clay-free particles, etc.	2
		Friction materials:	sand, silt calcite, etc. (non-softening)	3
		Cohesive materials:	clay, chlorite, talc, etc. (softening)	4

	FILLING OF:	Type	*Wall contact before 10 cm shear*	*No wall contact when sheared*
Partly or no wall contact	Friction materials	sand, silt calcite, etc. (non-softening)	Ja = 4	Ja = 8
	Hard cohesive materials	compacted filling of clay, chlorite, talc, etc.	6	5 - 10
	Soft cohesive materials	medium to low overconsolidated clay, chlorite, talc, etc.	8	12
	Swelling clay materials	filling material exhibits swelling properties	8 - 12	13 - 20

E. Classification with ratings for the Joint water reduction factor (Jw)

Dry excavations or minor inflow, i.e. < 5 l/min locally	p_w < 1 kg/cm^2	Jw = 1
Medium inflow or pressure, occasional outwash of joint fillings	1 - 2.5	0.66
Large inflow or high pressure in competent rock with unfilled joints	2.5 - 10	0.5
Large inflow or high pressure, considerable outwash of joint fillings	2.5 - 10	0.3
Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 - 0.1
Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1 - 0.05

Note: *(i) The last four factors are crude estimates. Increase Jw if drainage measures are installed*

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

Empirical methods (PRO): Q- System

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.
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 joints	3
D	Two joint sets	4
E	Two joint sets plus random joints	6
F	Three joint sets	9
G	Three joint sets plus random joints	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 $2.0 \times J_n$)

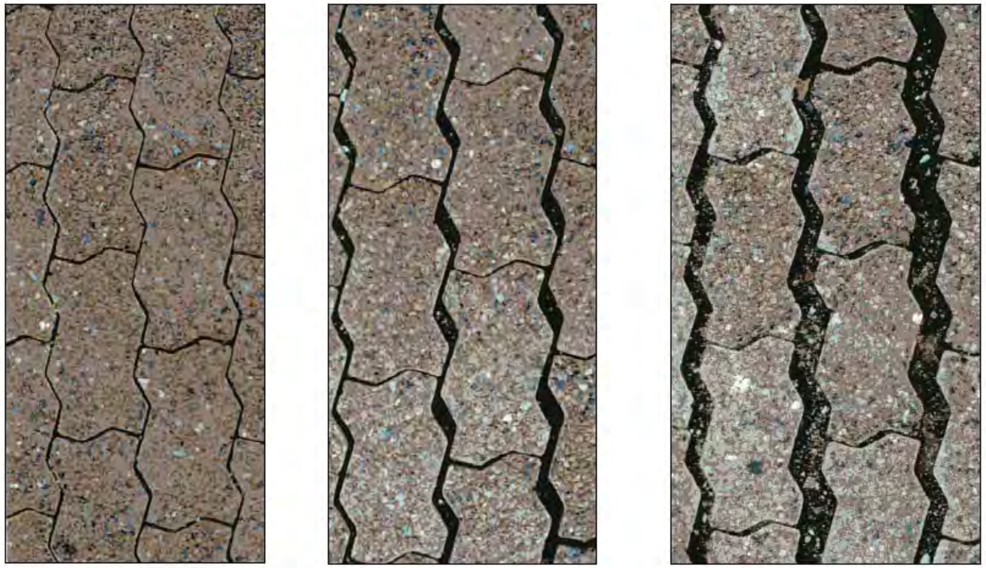
(2m window: RQD = 20 to 50 …. May need to measure)

(Jn = 4 → 6 → 9 ….a lot is blast damage)

Fig.32 a

Empirical methods (PRO): Q-System (Fig.32 b)

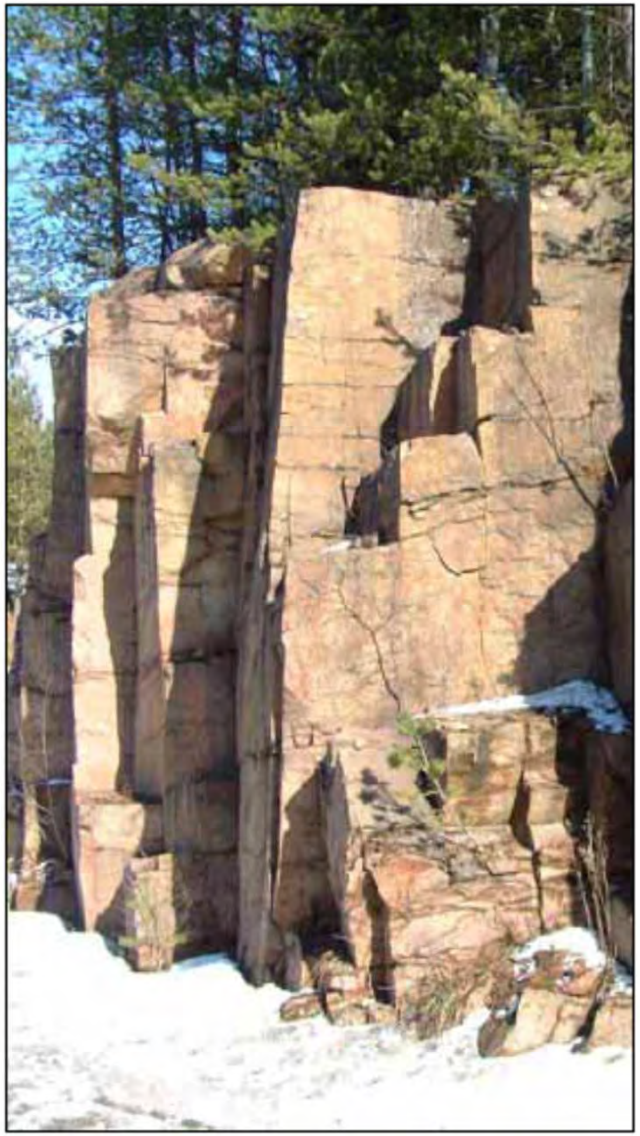
3. Joint Roughness Number		J_r
a) Rock-wall contact, and b) rock-wall contact before 10 cm shear		
A	Discontinuous joints	4
B	Rough or irregular, undulating	3
C	Smooth, undulating	2
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
Note: i) Descriptions refer to small scale features and intermediate scale features, in that order.		
c) No rock-wall contact when sheared		
H	Zone containing clay minerals thick enough to prevent rock-wall contact	1.0
J	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0
Note: i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. ii) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are oriented for minimum strength.		



a) rock-to-rock contact

b) rock-to-rock after shearing

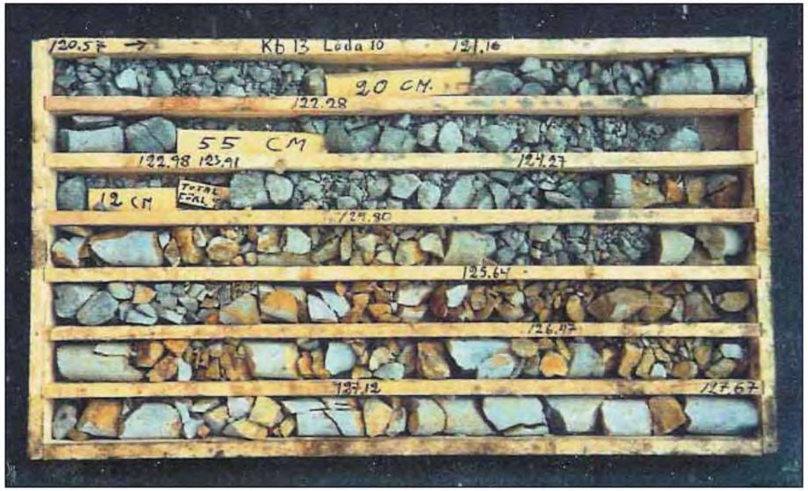
c) no rock-to-rock contact



Jr = 1.5 (joints in sun)

Empirical methods (PRO): Q- System

4. Joint Alteration Number		ϕ_r approx.	J_a
a) Rock-wall contact (no mineral fillings, only coatings)			
A	Tightly healed, hard, non-softening, impermeable filling, *i.e.*, quartz or epidote	-	0.75
B	Unaltered joint walls, surface staining only	25-35°	1.0
C	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, *etc.*	25-30°	2.0
D	Silty- or sandy-clay coatings, small clay fraction (non-softening)	20-25°	3.0
E	Softening or low friction clay mineral coatings, *i.e.*, kaolinite or mica. Also chlorite, talc, gypsum, graphite, *etc.*, and small quantities of swelling clays.	8-16°	4.0
b) Rock-wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, *etc.*	25-30°	4.0
G	Strongly over-consolidated non-softening clay mineral fillings (continuous, but <5mm thickness)	16-24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but <5mm thickness)	12-16°	8.0
J	Swelling-clay fillings, *i.e.*, montmorillonite (continuous, but <5mm thickness). Value of J_a depends on percent of swelling clay-size particles, and access to water, *etc.*	6-12°	8-12
c) No rock-wall contact when sheared (thick mineral fillings)			
KL M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6-24°	6, 8, or 8-12
N	Zones or bands of silty- or sandy-clay, small clay fraction (non-softening)	-	5.0
OP R	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6-24°	10, 13, or 13-20



Ja = ?? definitely 2 for weathered, maybe 4 or 6 for sandy or clay fillings

a) rock-to-rock contact

b) rock-to-rock after shearing

c) no rock-to-rock contact

Fig.32 c

Empirical methods (PRO): Q- System

5. Joint Water Reduction Factor		approx water pres. (kg/cm^2)	J_w
A	Dry excavations or minor inflow, *i.e.*, <5 l/min locally	<1	1.0
B	Medium inflow or pressure, occasional outwash of joint fillings	1-2.5	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	2.5-10	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	2.5-10	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	>10	0.2-0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>10	0.1-0.05

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.

Jw = 0.1 or 0.2

Jw = 1 or 0.66

Jw = 0.5

Jw = 0.2

Fig.32 d

F. Classification with ratings for the Stress reduction factor (SRF)

Weakness zones intersecting excavation	Multiple weakness zones with clay or chemically disintegrated rock, very loose surrounding rock (any depth)				SRF = 10
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50 m)				5
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)				2.5
	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)				7.5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation < 50 m)				5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation > 50 m)				2.5
	Loose, open joints, heavily jointed or "sugar-cube", etc. (any depth)				5
Note: (i)	*Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence, but do not intersect the excavation*		σ_c / σ_1	σ_θ / σ_c	SRF
Competent rock, rock stress problems	Low stress, near surface, open joints		> 200	< 0.01	2.5
	Medium stress, favourable stress condition		200 - 10	0.01 - 0.3	1
	High stress, very tight structure. Usually favourable to stability, maybe except for walls		10 - 5	0.3 - 0.4	0.5 - 2
	Moderate slabbing after > 1 hour in massive rock		5 - 3	0.5 - 0.65	5 - 50
	Slabbing and rock burst after a few minutes in massive rock		3 - 2	0.65 - 1	50 - 200
	Heavy rock burst (strain burst) and immediate dynamic deformation in massive rock		< 2	> 1	200 - 400
Notes: (ii)	*For strongly anisotropic stress field (if measured): when 5 < σ_1/σ_3 <10, reduce σ_c to 0.75 σ_c. When σ_1/σ_3 > 10, reduce σ_c to 0.5σ_c*				
(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 low stress cases*			σ_θ / σ_c	SRF
Squeezing rock	Plastic flow of incompetent rock under the influence of high pressure	Mild squeezing rock pressure		1 - 5	5 - 10
		Heavy squeezing rock pressure		> 5	10 - 20
Swelling rock	Chemical swelling activity depending on presence of water	Mild swelling rock pressure			5 - 10
		Heavy swelling rock pressure			10 - 15

Fig.32bis [20] : Q-System rating assessment table (2 of 2)

IV: Q-System (PRO→*C2c*)

- 9 classes are distinguished : from a “very poor” rock mass ($Q<0.01$) to an “excellent” rock mass ($Q>400$)
- In relating the value of the index Q (fig.34) to the stability and support requirements of underground excavations, an additional parameter is defined, called “Equivalent Dimension” of excavation, (De)

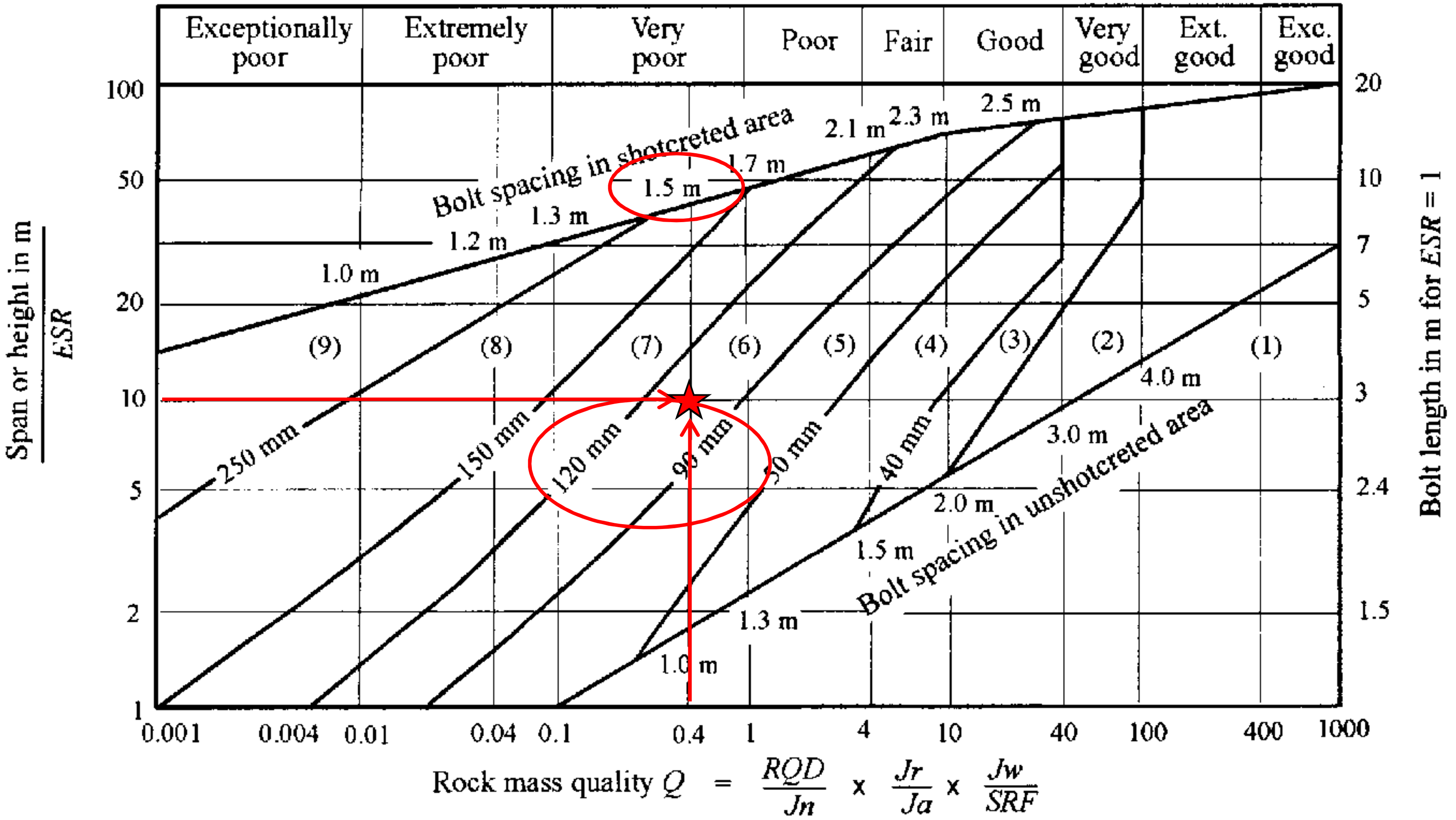
  De = Excavation span, diameter or height (m)/ESR

  where ESR= Excavation Support Ratio, is related to the degree of security which is demanded and has a similar meaning to the reciprocate of the safety factor (fig.33)
- For the determination of a temporary support could be used $Q_{(temp)}=5Q$ and $ESR_{(temp)}=1.5ESR$

IV: Q-System (PRO→*C2c*)

Type of Excavation		ESR (1994)	ESR (2014)
A	*Temporary mine openings, etc.*	ca. 2-5	*ca. 2 to 5*
B	*Permanent mine openings, water tunnels for hydropower (exclude high pressure penstocks), pilot tunnels, drifts and headings for large openings, surge chambers*	1.6-2.0	*1.6 to 2.0*
C	*Storage caverns,* water treatment plants, minor road and railway tunnels, *access tunnels*	1.2-1.3	0.9 to 1.1 *Storage caverns 1.2-1.3*
D	*Power stations,* major road and railway tunnels, *civil defence chambers, portals, intersections*	0.9-1.1	Major road and rail tunnels 0.5 to 0.8
E	*Underground nuclear power stations, railway stations, sports and public facilities, factories, major gas pipeline tunnels*	0.5-0.8	*0.5 to 0.8*

Fig. 33 [5b]



REINFORCEMENT CATEGORIES

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting with 40-100 mm unreinforced shotcrete
5) Fibre reinforced shotcrete, 50 - 90 mm, and bolting
6) Fibre reinforced shotcrete, 90 - 120 mm, and bolting
7) Fibre reinforced shotcrete, 120 - 150 mm, and bolting
8) Fibre reinforced shotcrete, > 150 mm, with reinforced ribs of shotcrete and bolting
9) Cast concrete lining

(50)

Fig.34
[20]

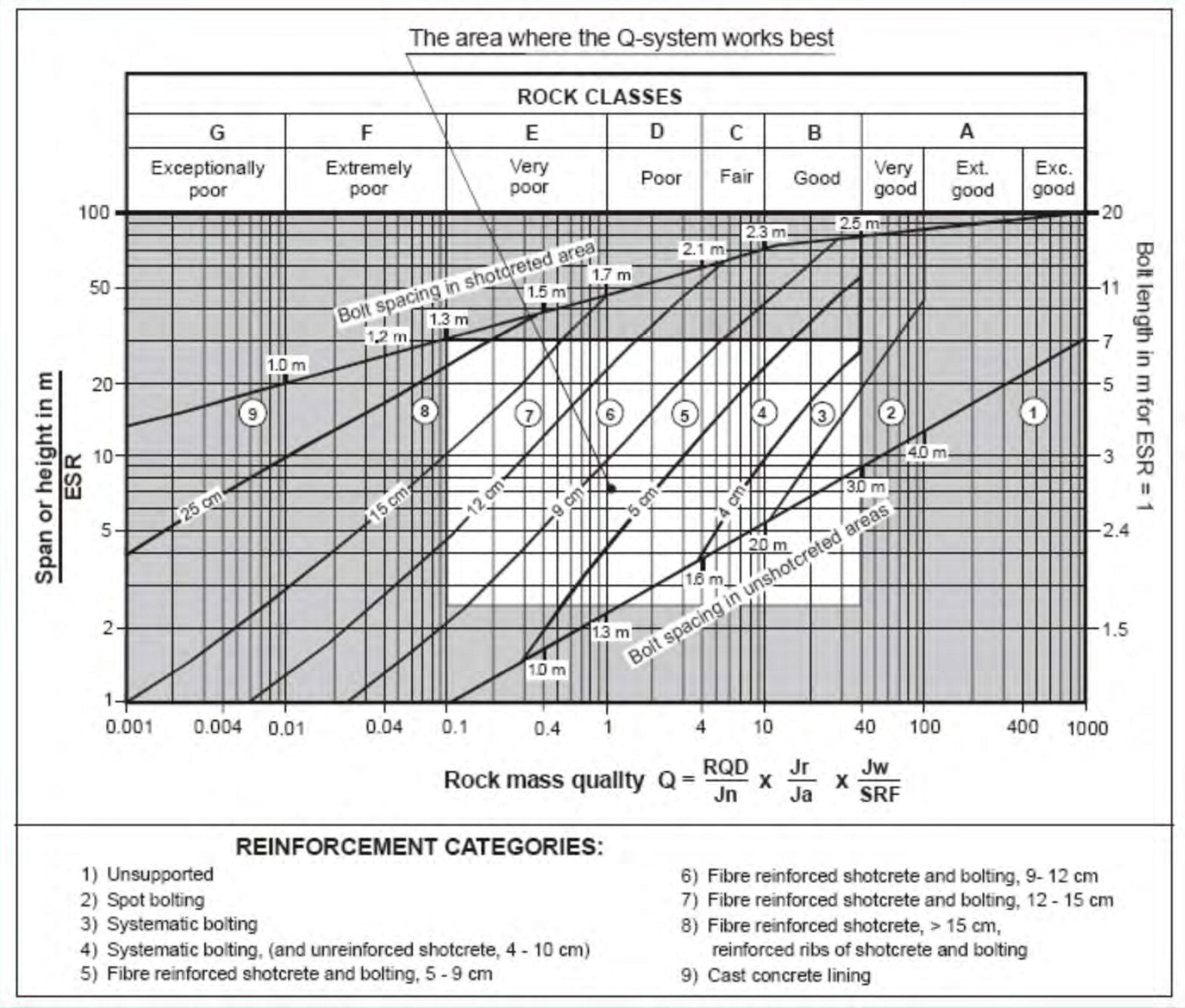


Fig.34bis [41ter]

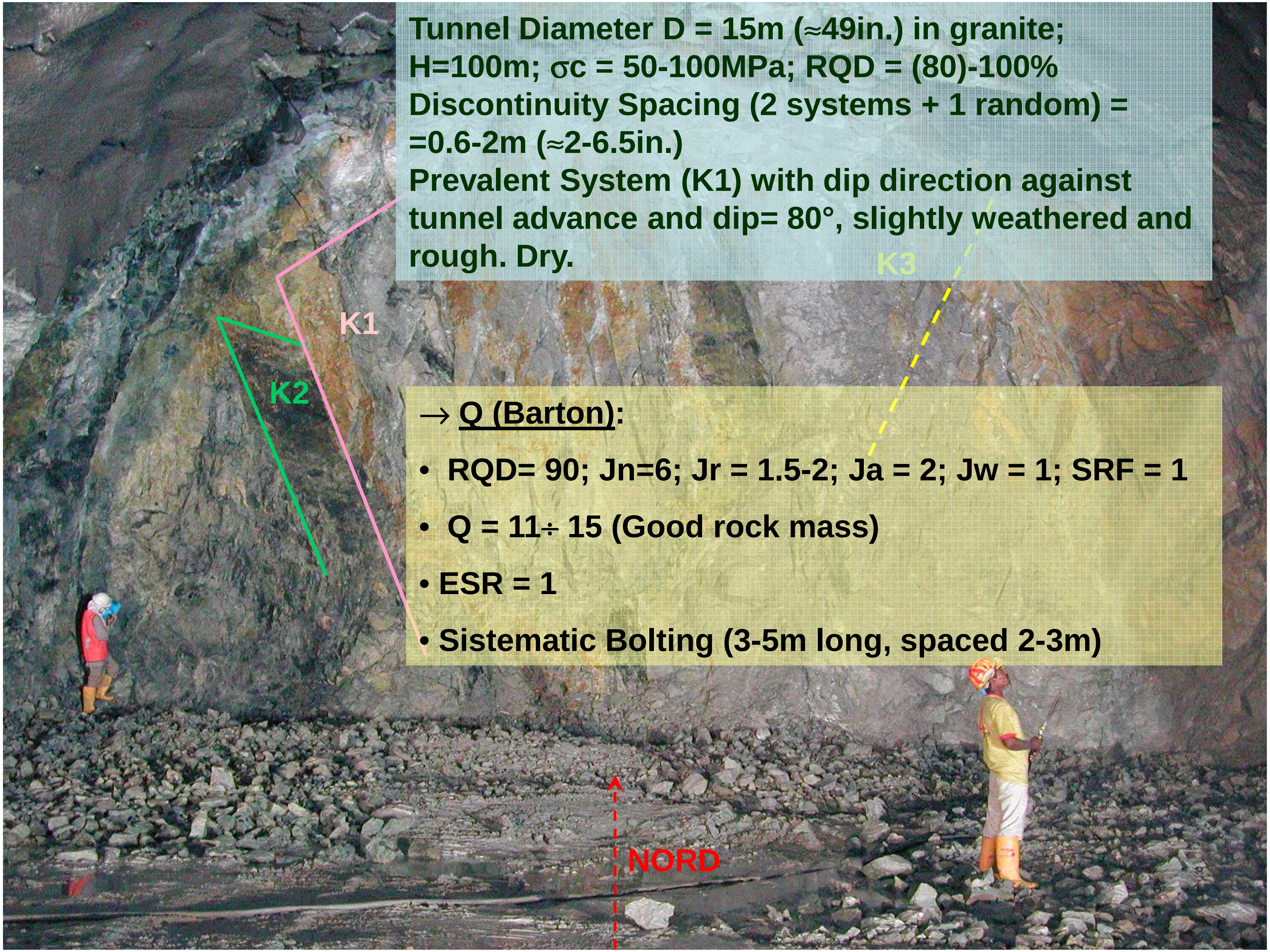
According Palmstrom and Broch [41ter], outside the unshaded area supplementary methods should be applied

IV: Q-System (PRO→*C2c*)

Empirical correlations with geomechanical parameters (see also fig.35)

Max unsupported span (*m*)	D_{max}	$2Q^{0.66}$ $2ESR*Q^{0.4}$
Radial pressure acting on support (*MPa*)	Pr	$\approx 0.1Q^{-1/3}$
Rock mass deformability modulus (*GPa*)	M	$\approx 10Q_c^{1/3}$
Longitudinal sismic waves velocity[1] P (*km/sec*)	V_p	$\approx 3.5+\log Q_c$
Tunnel radial displacement (*mm*)	Δ	$\approx$ D/Q
Lugeon Unit (*U.L.*)	L	$\approx 1/Q_c$

Note: $Q_c = Q*\sigma_c/100$; σ_c= intact rock strength (MPa); D = excavation dimension; [1]calculated with a refraction method with a maximum depth of 25m.


Tunnel Diameter D = 15m (≈49in.) in granite;
H=100m; σc = 50-100MPa; RQD = (80)-100%
Discontinuity Spacing (2 systems + 1 random) =
=0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against
tunnel advance and dip= 80°, slightly weathered and
rough. Dry.
K3
K1
K2
→ Q (Barton):
• RQD= 90; Jn=6; Jr = 1.5-2; Ja = 2; Jw = 1; SRF = 1
• Q = 11÷ 15 (Good rock mass)
• ESR = 1
• Sistematic Bolting (3-5m long, spaced 2-3m)
NORD

Proposed equation for TBM (figg. 36-37)

$$Q_{TBM} = Q_o * \frac{SIGMA}{F^{10}/20^9} * \frac{20}{CLI} * \frac{q}{20} * \frac{\sigma_\theta}{5}$$

Qo = **index calculated estimating RQD in the direction of the excavation and referring Jr/Ja to the joint set that mostly influences the tunnel excavation;**

SIGMA = rock compressive strength ($SIGMA_{cm}=5\gamma*Qc^{1/3}$) or tensile strength ($SIGMA_{tm}=5\gamma*Qt^{1/3}$), in case ($\sigma c/Is50$)>>25 and favorable orientation of the excavation;

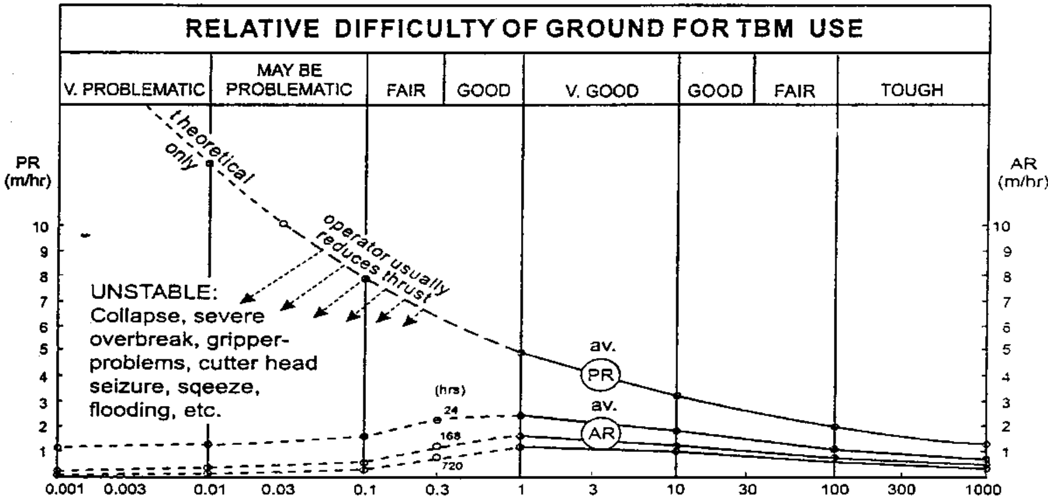
Qt=Q*Is50/4 where Is50 is the Point Load Test Index; γ= rock volume weight (g/cm3)
F/20 = thrust per cutter (t), normalized against 20;

CLI = Cutter Life Index

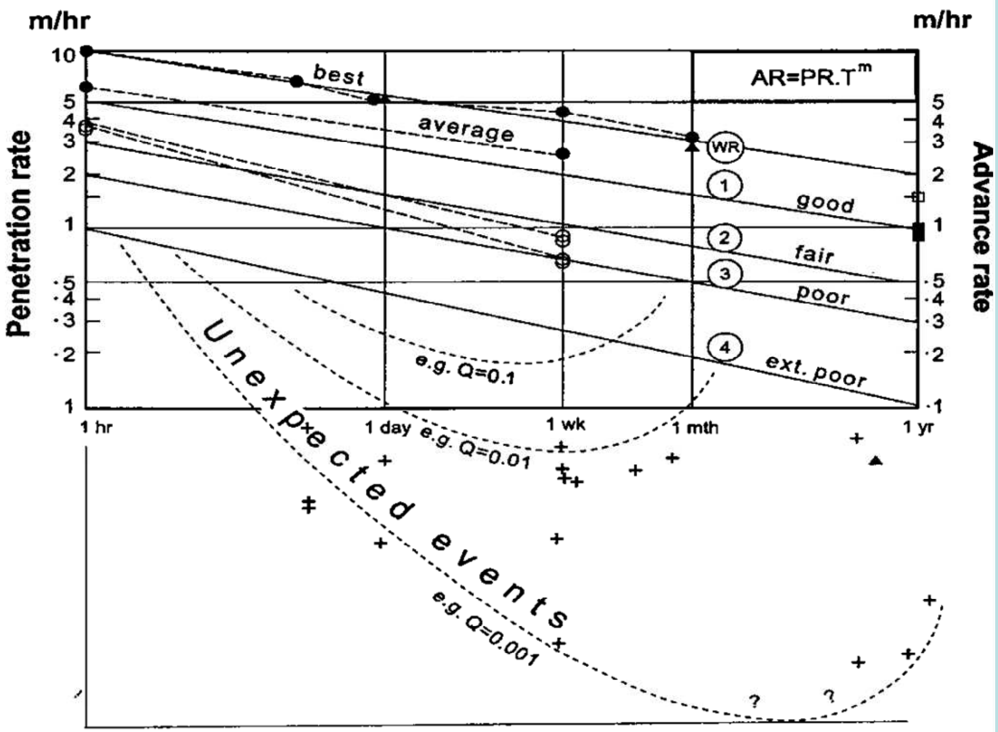
q = quartz content (%)

$\sigma\theta$/5 = average bi-axial stresses along the tunnel face MPa, normalized against a value corresponding to 100m depth.

IV: Q-System (PRO→*C2c*)



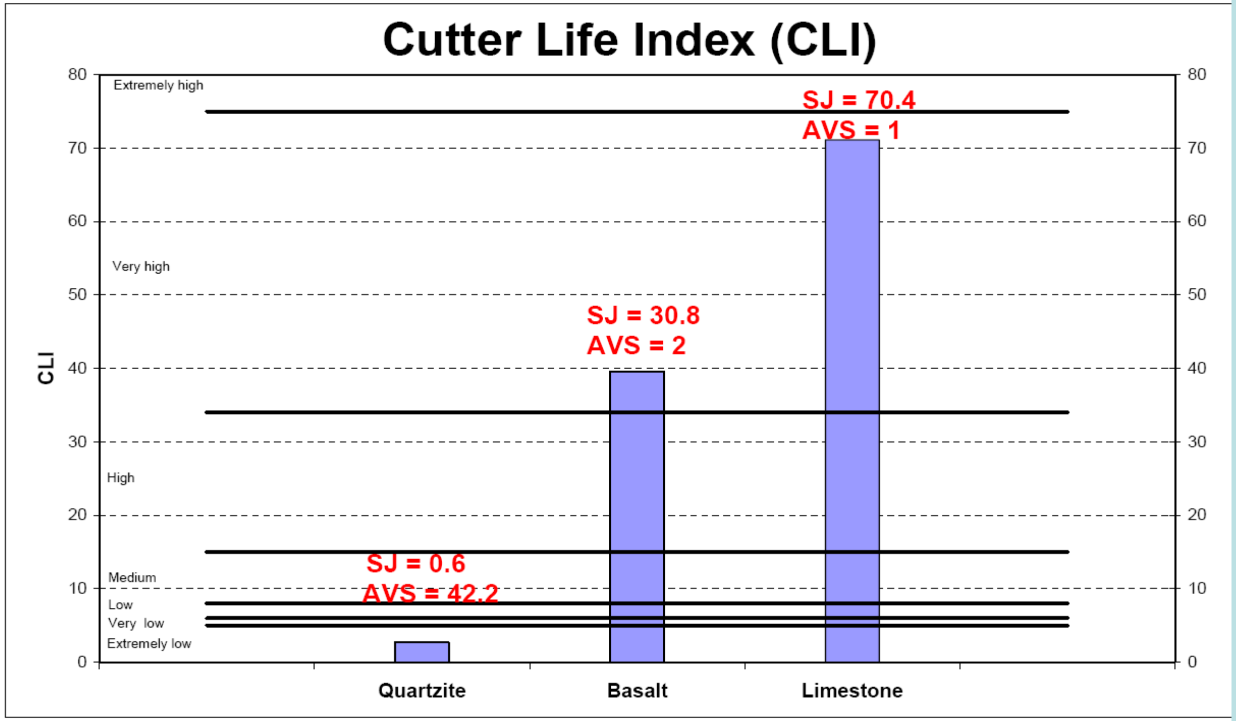
$$Q_{TBM} = \left[\frac{RQD_O}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \times \frac{SIGMA}{F^{10}/20^9} \times \frac{20}{CLI} \times \frac{q}{20} \times \frac{\sigma_\theta}{5} \right]$$



. General trends of deceleration from 145 TBM projects consisting of more than 1000 km of tunnels. (Barton, 2000b).

Fig.36 [3]

IV: Q-System (PRO→*C2c*)



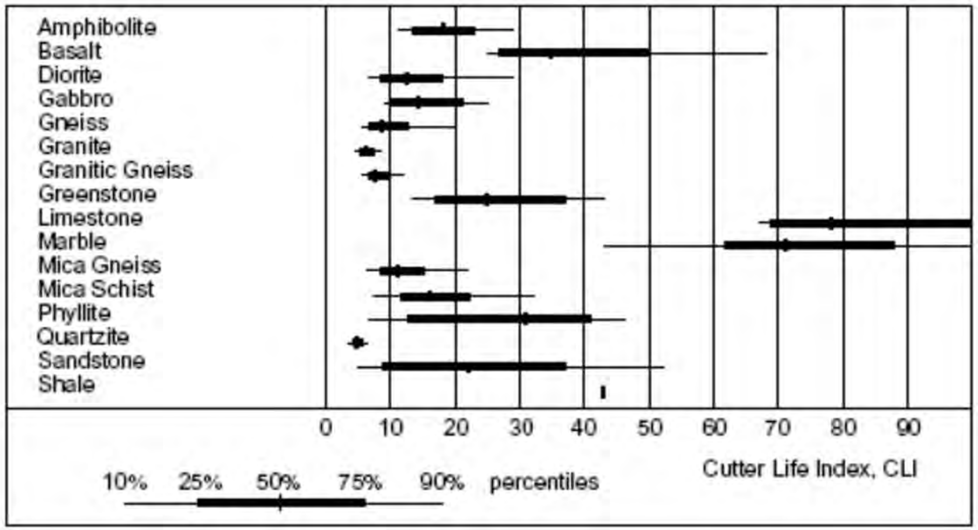


Figure 1.3 *Recorded Cutter Life Index for some rock types. Data from Project Report 13C-98 DRILLABILITY Statistics of Drillability Test Results.*

Fig.37 [10ter]

Q_{TBM} is correlated to the TBM advancement parameters

Penetration rate	PR *(m/h)*	$5Q_{TBM}^{-0.2}$
Advance rate	AR *(m/h)*	U*PR
Utilisation factor	U	T^m
Decelaration gradient (negative)	m *(m/h²)*	(*)

(*) $$m \approx m_1 * (D/5)^{0.20} * (20/CLI)^{0.15} * (q/20)^{0.10} * (n/2)^{0.05}$$

Q	0.001	0.01	0.1	1	10	100	1000
$m_1 \approx$	-0.9	-0.7	-0.5	-0.22	-0.17	-0.19	-0.21

note: T=time in hours; n = porosity (%)

Applicative example:

- Circular tunnel excavated by TBM
- L= 2500m; D = 8m; H = 300m; $k = (\sigma_h / \sigma_v) = 1$
- $\gamma \approx 2.5 g/cm^3$;
- RQD_o=15%;
- J_n=6;
- $J_r = J_a$=1;
- J_w=0.66;
- SRF=1
- $\sigma_c \approx$50MPa; $I_{50} \approx$0.5MPa;
- n ≈1%; q=20%
- F=15t; CLI=20

Tasks:

Q_o, Q_c, Q_{TBM}, the advance of the TBM in 2 months time and the time for completion

Result (1 di 2)

Parameter	Formula	Value
Q_o	(15/6)*(1/1)*(0.66/1)	1.65
σ_c/I_{50}	50/0.5	100
σ_θ	$2*\gamma*H = 2*0.025*300$	15MPa
Q_c	$Q_o*\sigma_c/100$	0.83
$SIGMA_{cm}$	$5\ \gamma*\ Q_c^{1/3}$	12MPa
Q_t	$Q_o*I_{50}/4$	0.21
$SIGMA_{tm}$	$5\ \gamma*\ Q_t^{1/3}$	7.4MPa
Q_{TBM}	**$1.65*[7.4/(15^{10}/20^{9})]*(20/20)*(20/20)*(15/5)$**	**≈ 33**

Solution (2 di 2)

- TBM advancement in 2 months ($L_{(2months)}$)

- Completing time ($T_{(end)}$)

PR	$5Q_{TBM}^{-0.2} = 5*33^{-0.2}$	2.5m/h
m_1	(from table)	-0.20
m	$-0.20*(8/5)^{0.20}*(20/20)^{0.15}*(20/20)^{0.10}*(1/2)^{0.05}$	-0.21
T	2*30*24	1440h
$U= T^m$	$1440^{-0.21}$	0.22
AR	PR*U = 2.5*0.22	0.55m/h
$L_{(2months)}$	0.55*1440	792m
$T_{(end)}$	$(L/PR)^{[1/(1+m)]} = (2500/2.5)^{[1/(1-0.21)]}$	6273h ≈ 9 months

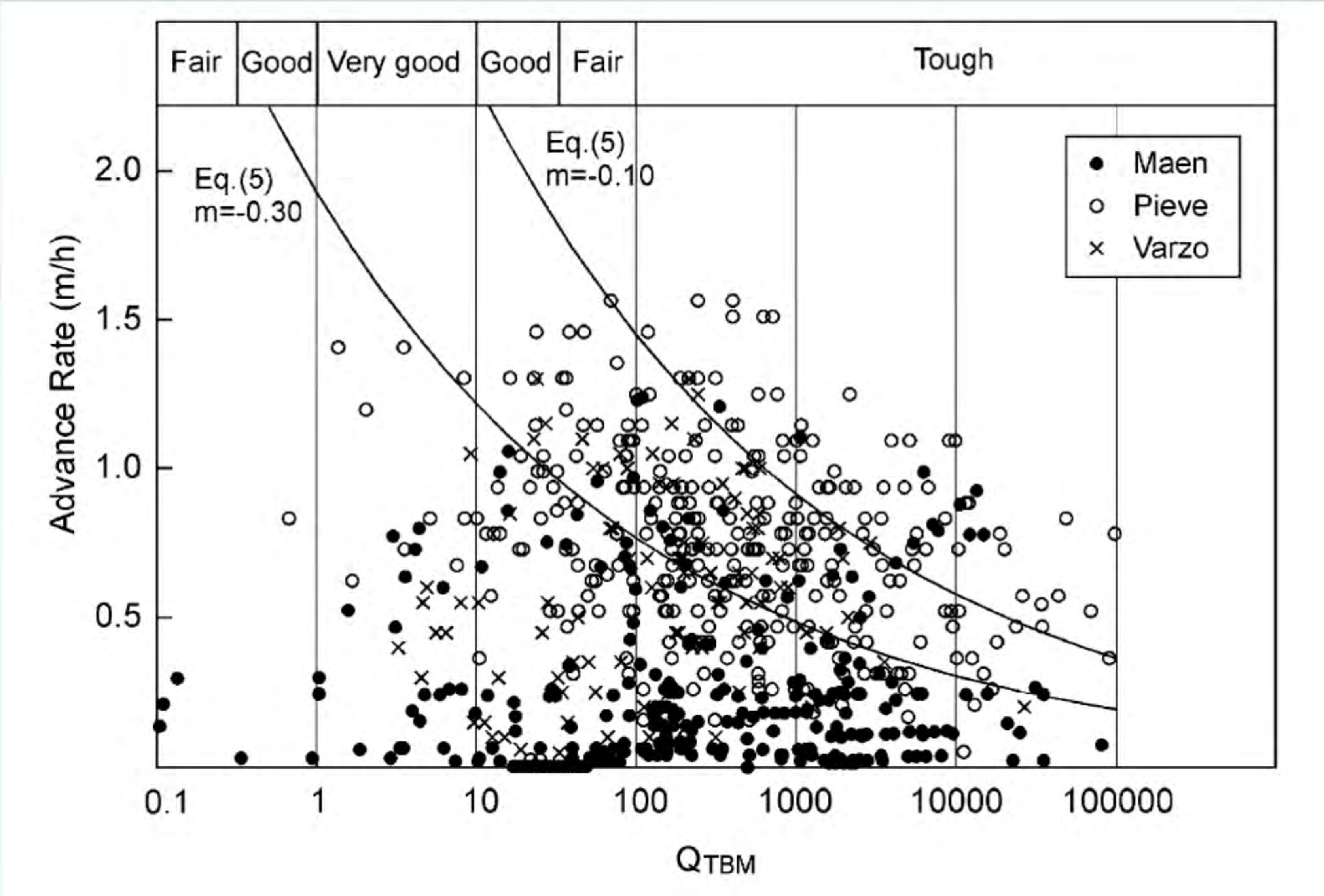


Fig.37bis
[41ter]

Q_tbm limits: Advance rate for three TBM plotted against Qtbm (Sapigni et al. in 41ter)

ROCK MASS index (RMi, Palmstrom, 1995÷2000)

- The RMi index expresses the quality and the geomechanical strength of rock mass (MPa) through the multiplication between the uniaxial intact rock compressive strength (σ_c) and a corrective factor (JP) depending on the geostructural conditions (fig.38)

→for jointed rock masses (JP< f_σ):

$$RMi = \sigma_c * JP = \sigma_c * 0.2\sqrt{jC} * Vb^D$$

→ for massive rock masses (JP>f_σ):

$$RMi = \sigma_c * f_\sigma = \sigma_c (0.05/Db)^{0.2} \approx 0.5\sigma_c$$

V - Rock Mass index (GEO→G3) / PRO →C2e

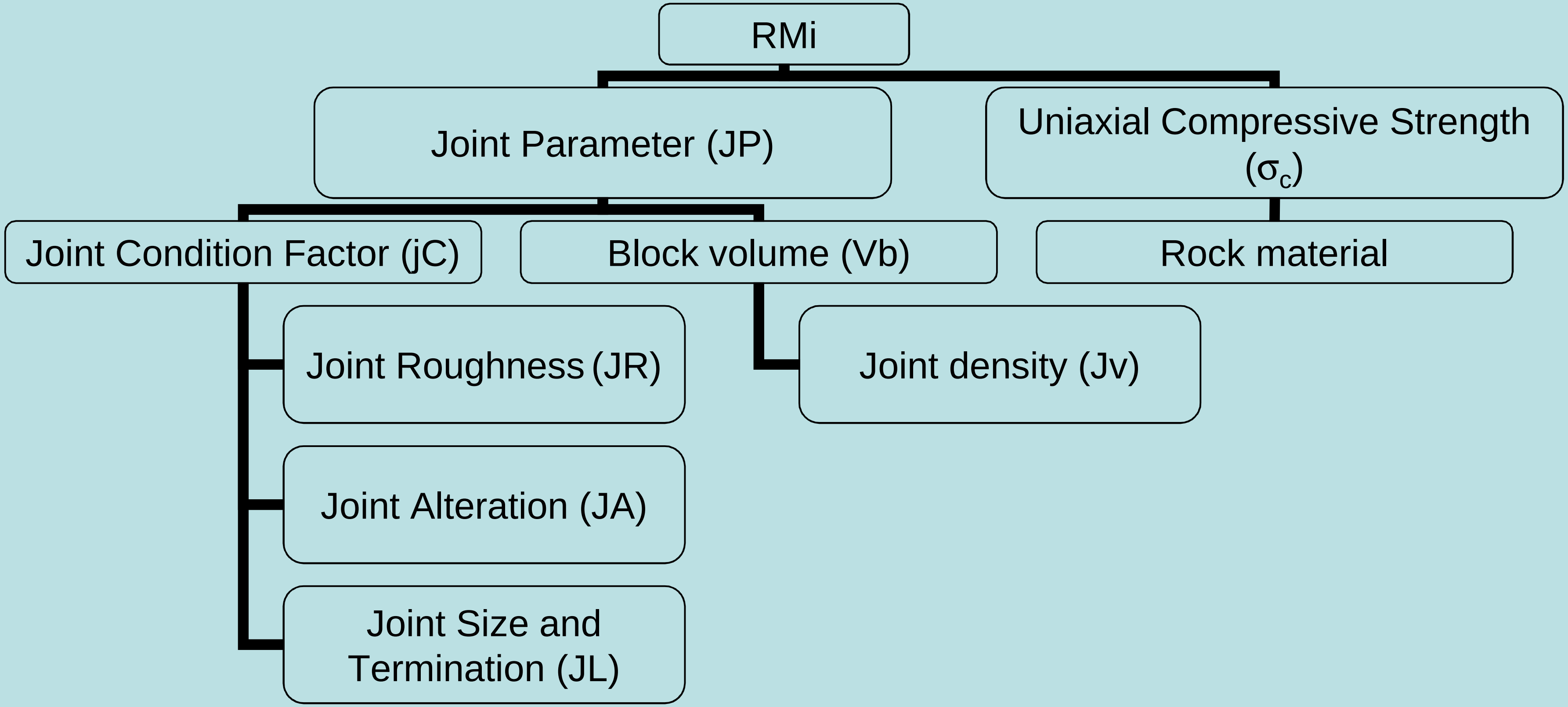


Fig. 38 [41]: RMi Conceptual scheme

- **JP** = Jointing Parameter, correlated to rock block size and to discontinuity properties. JP can vary from 0 (very fractured rock) to 1 (intact rock).

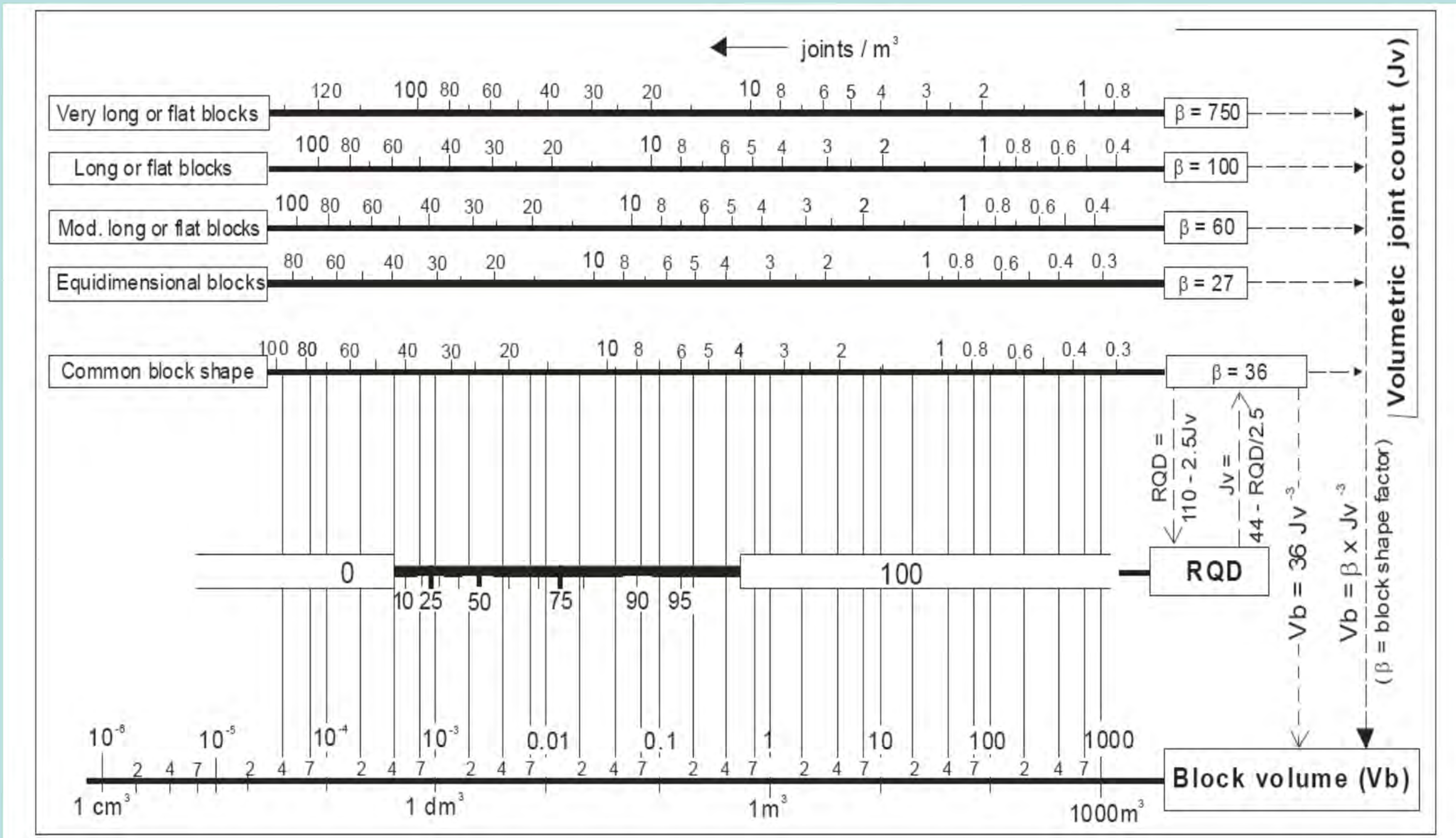
$$JP = 0.2 * \sqrt{jC} * Vb^{D} \qquad D = 0.37 jC^{-0.2}$$

- **jC** = Joint Condition factor = jL*(jR/jA)
- **jR** = Joint roughness factor, similar to Jr of Q-System
- **jA** = Joint alteration factor, similar to Ja of Q-System
- **jL** = Joint size and continuity factor: reflects the discontinuity persistence
- The criterion for assigning the rating are shown in fig. 40.

- Vb = volume of elementary blocks, expressed in m^3 (fig.39)
- Db = equivalent block diameter, for cubic block $Db = \sqrt[3]{Vb}$
- f_σ = massivity parameter [$f_\sigma = (0.05/Db)^{0.2}$]

  Generally, for a massive rock, Db>approx. 2m and so $f_\sigma \approx 0.5$.

Fig. 39 [41bis]: Correlations between diameter and volume of the rock block, and other parameters of fracturing



A. Uniaxial compressive strength of intact rock, σ_c	
Found from lab. tests, simple field hammer test or assumed from handbook tables	**value** of σ_c (in MPa)
B. Block volume, Vb	
Found from measurement at site or from drill cores, etc. (Vb can also be calculated from RQD or Jv)	**value** of Vb (in m³)

C. Joint roughness factor, jR (similar to Jr in the Q-system) jR = Jr = js × jw		
Small scale smoothness of joint surface	Very rough or interlocking	js = 3
	Rough or irregular	2
	Slightly rough	1.25
	Smooth	1
	Polished or slickensided *)	0.5 – 0.75
*) *For slickensided surfaces the ratings apply to possible movement along the lineations*		
Large scale waviness of joint plane	Planar	jw = 1
	Slightly undulating	1.4
	Moderately undulating	2
	Strongly undulating	2.5
	Discontinuous joints*)	6
*) *Discontinuous joints end in massive rock (For filled joints jR = 1)*		

Joint size (length) factor, jL	Length	Continuous joints	Discontinuous joints
Crack [1] (irregular, discontinuous break)	< ~0.3m	-	10
Parting (very short, thin joint)	< 1m	3	6
Very short joint	0.3 – 1m	2	4
Short joint	1 – 3m	1.5	3
Medium joint	3 – 10m	1	2
Long joint	10 – 30m	0.75	-
[1] Introduced 3 years ago (2004,n.d.r.)			

...ystem)		
..., etc.	jA =0.75	
...cept from staining (rust)	1	
...ration than the rock	2	
...eration than the rock	4	
...ithout content of clay	3	
	4	
	Thin filling (< ca. 5 mm)	Thick filling
...on-softening	jA = 4	jA = 8
	6	5 - 10
	8	12
Swelling clay materials — smectite, montmorillonite etc.	8 - 12	13 - 20

E. Joint size factor, jL (length of the joint) *discontinuous joints (earlier included here) have been included in the joint roughness*		
Bedding or foliation partings	length < 0.5m	jL = 3
Joints	with length 0.1 - 1m	2
	with length 1 - 10m	1
	with length 10 - 30m	0.75
Long joint, filled joint, seam or shear *)	length > 30m	0.5
*) *Often a singularity and should if it has a significant impact on stability, be treated separately*		

F. Interlocking (compactness) of rockmass structure, IL		
Very tight structure	Undisturbed rock mass, tightly interlocked	IL = 1.3
Tight structure	Undisturbed, jointed rock mass	1
Disturbed / open	Folded / faulted with angular blocks	0.8
Poorly interlocked	Broken with angular and rounded blocks	0.5

Fig.40 [41bis]

V - Rock Mass index (GEO→G3) / PRO →C2e

RMi [(-) (MPa)]	DESCRIPTION (-)	ROCK MASS (MPa)
<0.001	extremely low	extremely week
0.001-0.01	very low	very week
0.01-0.1	low	week
0.1-1	moderate	medium
1-10	high	strong
10-100	very high	very strong
>100	extremely high	extremely strong

Geomechanical correlations

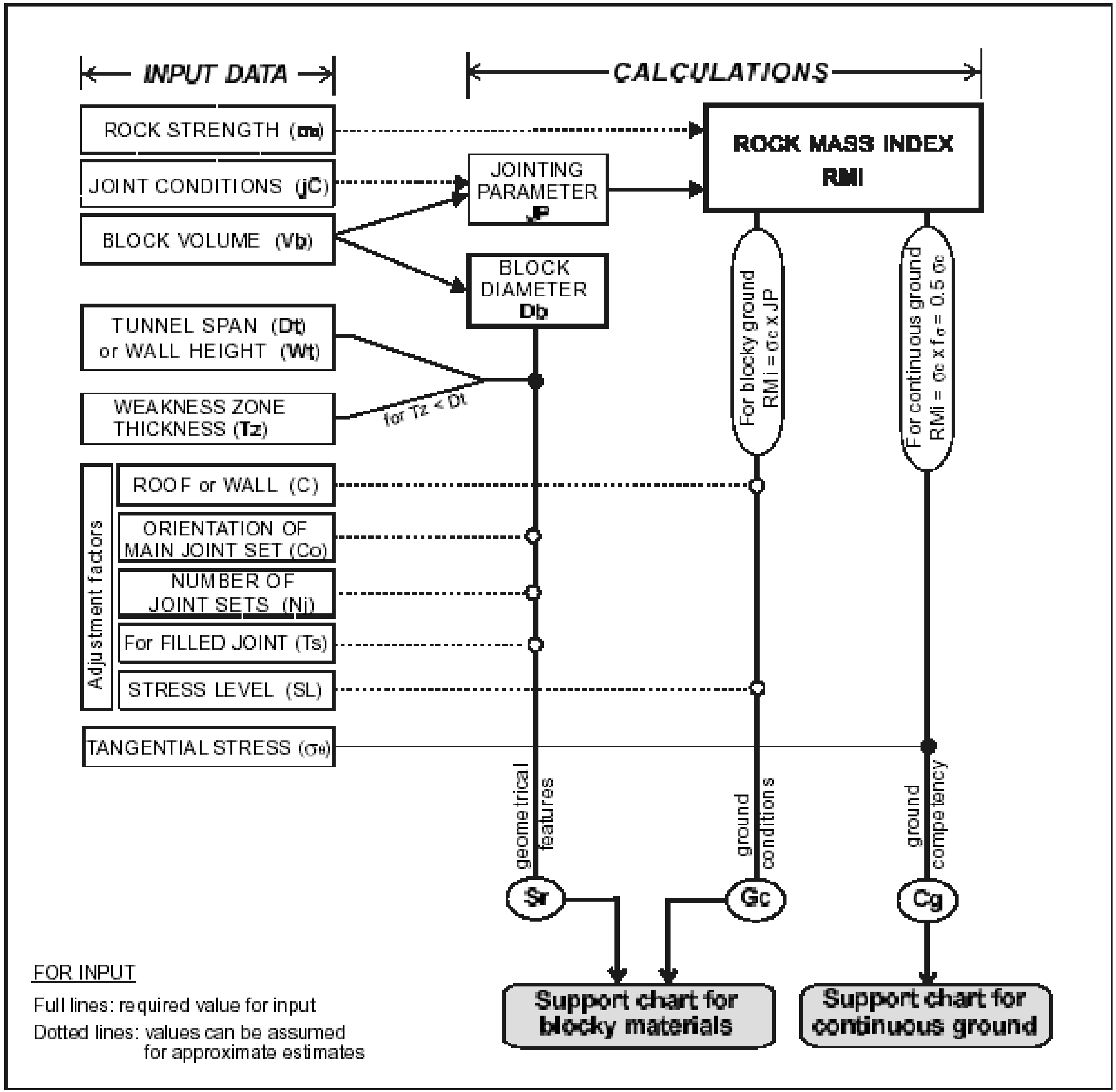
- $s = JP^2$
- $m_b = m_i * JP^{0.64}$ (undisturbed rock mass)
- $m_b = m_i * JP^{0.857}$ (disturbed rock mass)
- $E_d = 5.6 RMi^{0.375}$

where

m_i, m_b, s = Hoek and Brown costants, (1980);

σ_c , σ_{cm} = intact strength, rock mass strength

E_d = deformability modulus



INPUT DATA
CALCULATIONS
ROCK STRENGTH (σc)
JOINT CONDITIONS (jC)
BLOCK VOLUME (Vb)
JOINTING PARAMETER JP
ROCK MASS INDEX RMi
BLOCK DIAMETER Db
TUNNEL SPAN (Dt) or WALL HEIGHT (Wt)
WEAKNESS ZONE THICKNESS (Tz)
for Tz < Dt
For blocky ground RMi = σc x JP
For continuous ground RMi = σc x fσ = 0.5 σc
Adjustment factors
ROOF or WALL (C)
ORIENTATION OF MAIN JOINT SET (Co)
NUMBER OF JOINT SETS (Nj)
For FILLED JOINT (Ts)
STRESS LEVEL (SL)
TANGENTIAL STRESS (σθ)
geometrical features
ground conditions
ground competency
Sr
Gc
Cg
FOR INPUT
Full lines: required value for input
Dotted lines: values can be assumed for approximate estimates
Support chart for blocky materials
Support chart for continuous ground

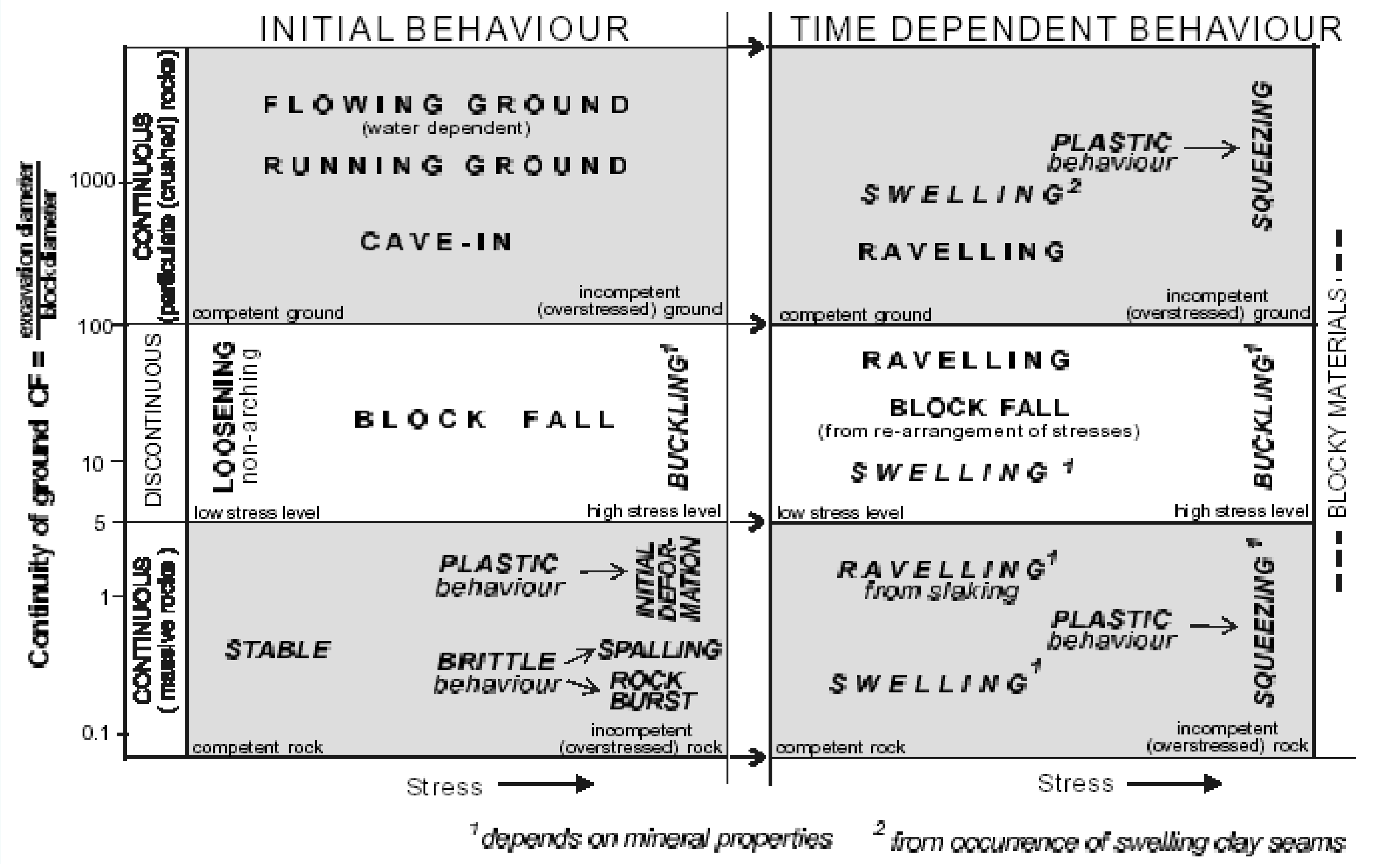
Fig.41 [41]

According to the flow chart of fig.41, for deriving the support required, the Continuity of Ground CF=Dt/Db (Dt, Db = tunnel, block diameter) must be before defined:

CF>100	CONTINUOUS: particulated (crushed) rocks
5>CF>100	DISCONTINUOUS: blocky rocks
CF<5	CONTINUOUS: massive rocks

V - Rock Mass index (GEO→G3) / PRO →C2e

Fig.42 [41]: Instability and rock mass behaviour



BLOCKY GROUND (DISCONTINUOUS)

For support definition, the **Ground Condition Factor (Gc) and the Size Ratio (Sr**) are defined:

- **Gc= RMi*(SL*C)=σ_c*JP*(SL*C)**
- **Sr = CF*(Co/Nj)=(Dt/Db)*(Co/Nj)** ([1])

where:

C= Gravity Adjustment Factor = 5-4cosδ [δ=angle (dip) of the opening surface measured from the horizontal]

SL= Stress Level Adjustment (from table in fig.43)

Co, Cos= Adjustment factor for the main joint orientation

Nj= Adjustment factor for the number (n_j) of joint sets (Nj=3/n_j)

Note: ([1]) for weakness zones of thickness Tz < Dt the equation Sr = (Tz/Db)*(Co/Nj) is used

V - Rock Mass index (GEO→G3) / PRO →C2e

Fig. 43 [41bis]:

GROUND WATER INFLOW, GW *)	
Dry excavation	GW = 1
Damp	
Wet	
Dripping *)	2.5
Gushing *)	5
Flowing, decaying with time	Outside limit of RMi
Heavily flowing, without noticeable decay	

*) *GW is related to groundwater's influence on rockmass stability*

INCLINATION OF TUNNEL SURFACE, C	
Horizontal (roof)	C = 1
30° inclination (roof in shaft)	1.5
45° inclination (roof in shaft)	2.2
60° inclination (roof in shaft)	3
Vertical (and steep walls)	5

STRESS LEVEL, SL		
Very low (in portals, etc.)	(overburden < 10 m)	SL = 0.1
Low	(overburden 10 - 35 m)	0.5
Moderate	(overburden 35 - 350 m)	1
High	(overburden > 350 m)	1.5 *)

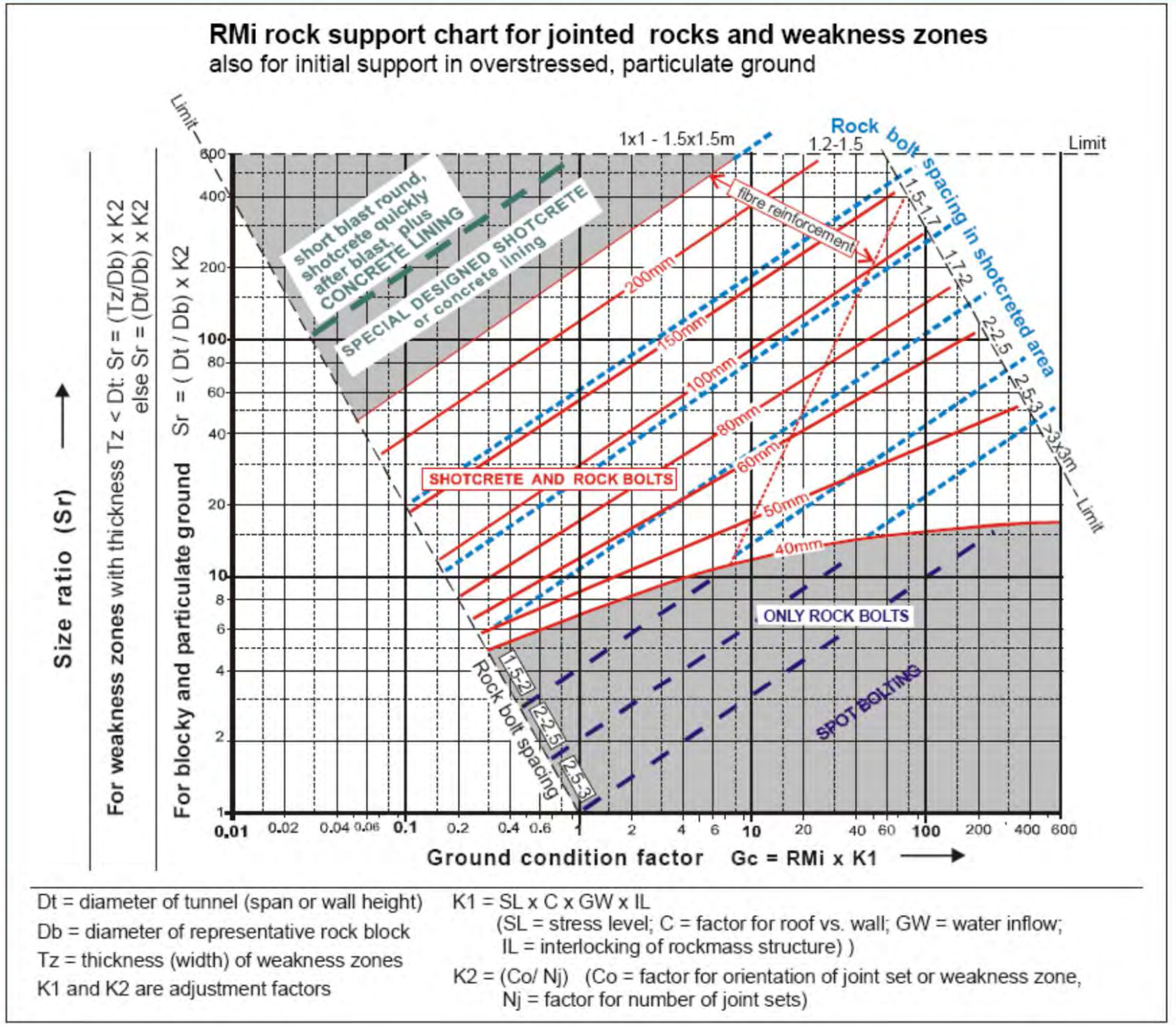
*) For stability in high *walls* a high stress level may be unfavourable. Possible rating, SL = 0.5 - 0.75

NUMBER OF JOINT SETS, Nj *)			
One set	Nj = 3	Three sets	Nj = 1
One set + random	2	Three sets + random	0.85
Two sets	1.5	Four sets	0.75
Two sets + random	1.2	Four sets + random	0.65

*) Means the number of joint sets, in the actual location only

ORIENTATION OF JOINTS AND ZONES, Co (related to the tunnel)			
Very favourable	Co = 1	Unfavourable	Co = 2
Favourable		Very unfavourable	3
Fair	1.5		

Fig.44 [41]: Rock support chart for blocky ground



V - Rock Mass index (GEO→G3) / PRO →C2e

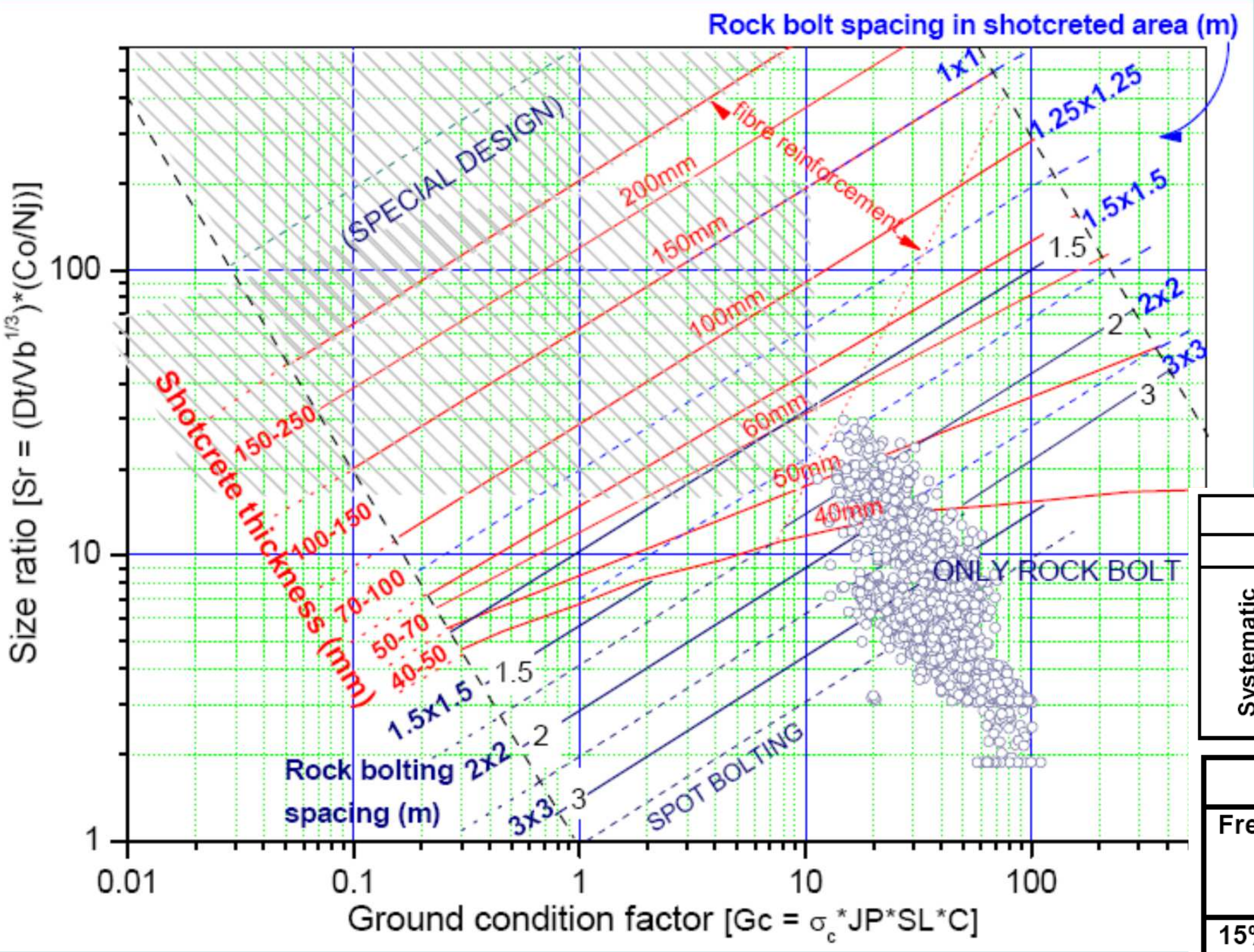


Fig.44bis: Example of implementation by probabilistic approach

WITHOUT SHOTCRETE		
Spot bolting		50.7%
Systematic bolting	3.0×3.0m	26.4%
	2.0×2.0m	7.5%
	1.5×1.5m	0.0%
	1.3×1.3m	0.0%

WITH SHOTCRETE		
Frequency	Shotcrete thickness (mm)	Bolts Patterns (m)
15%	45	2.0x2.0
0.3%	60	2.0x2.0
0.1%	60	1.5x1.5

CONTINUOUS GROUND

Since the tunnelling behaviour is influenced essentially by the stress conditions, the **Competency Factor** is considered (Cg=rock mass strength / stress condition):

- for massive rocks:

$$\mathbf{Cg = RMi/\sigma_\theta = f_\sigma * \sigma_c/\sigma_\theta \approx 0.5\sigma_c/\sigma\theta}$$

- for particulate rocks:

$$\mathbf{Cg = RMi/\sigma_\theta = JP * \sigma_c/\sigma_\theta}$$

Fig. 45 [41bis]: Chart for estimating support in continuous ground

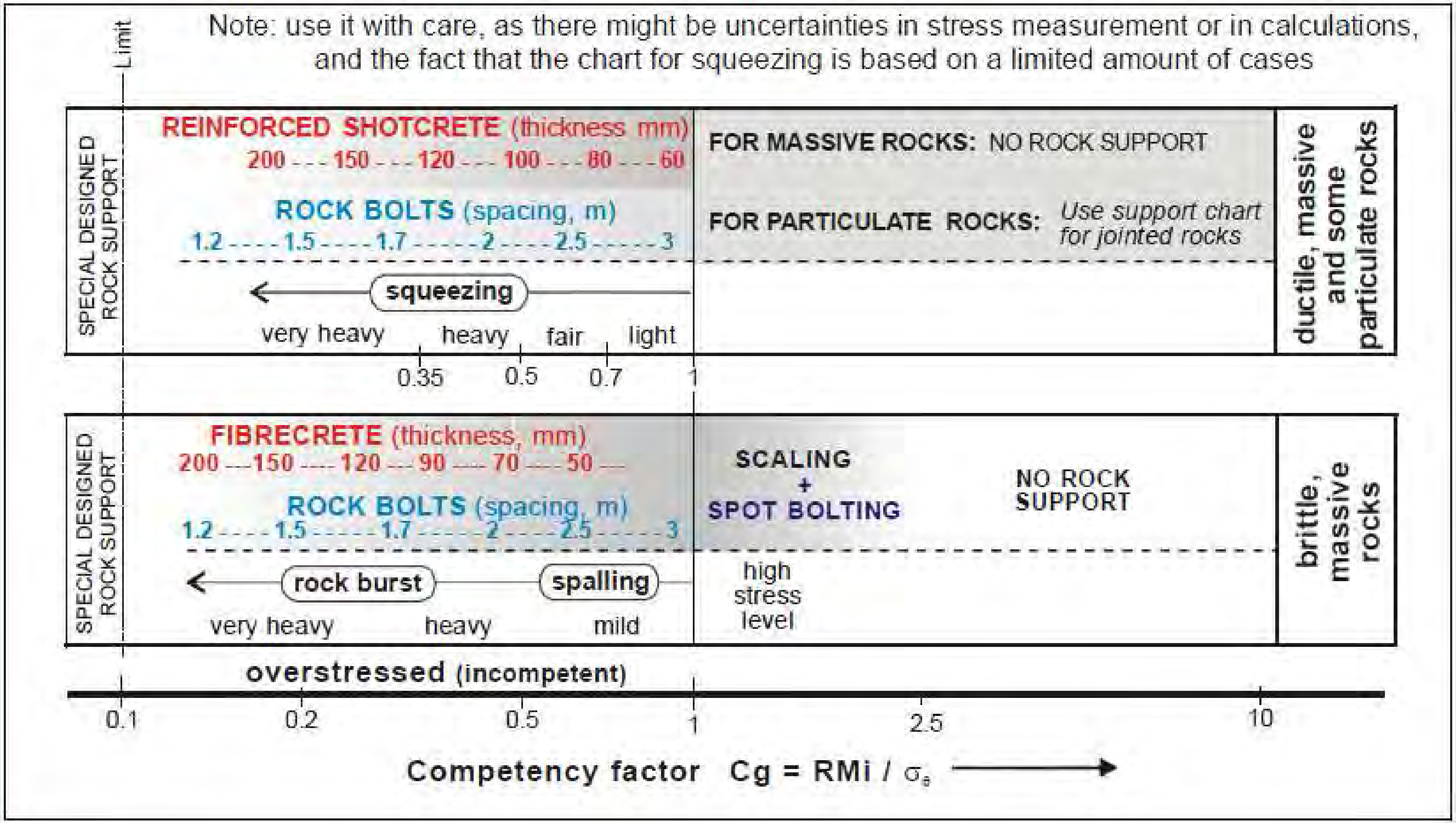
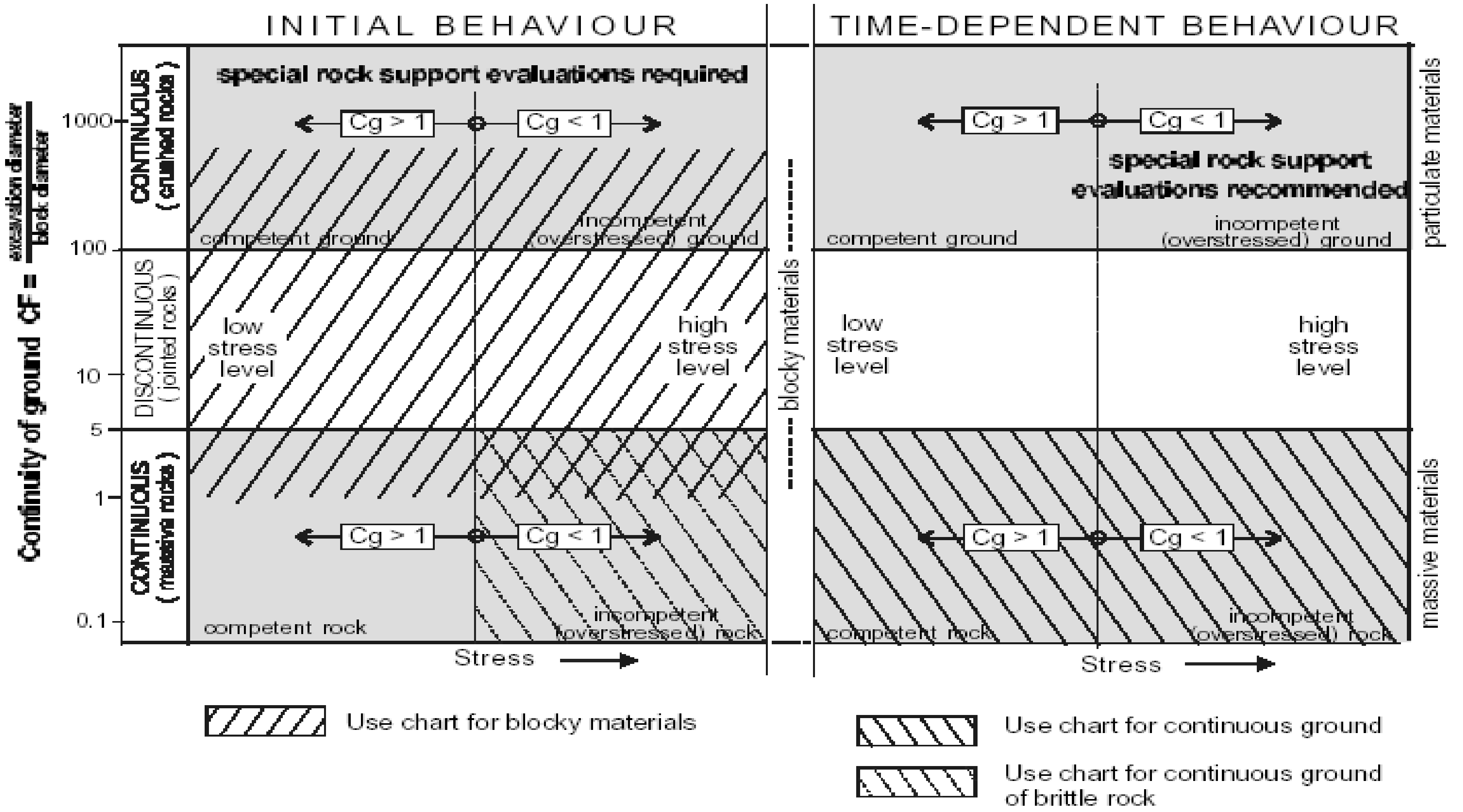
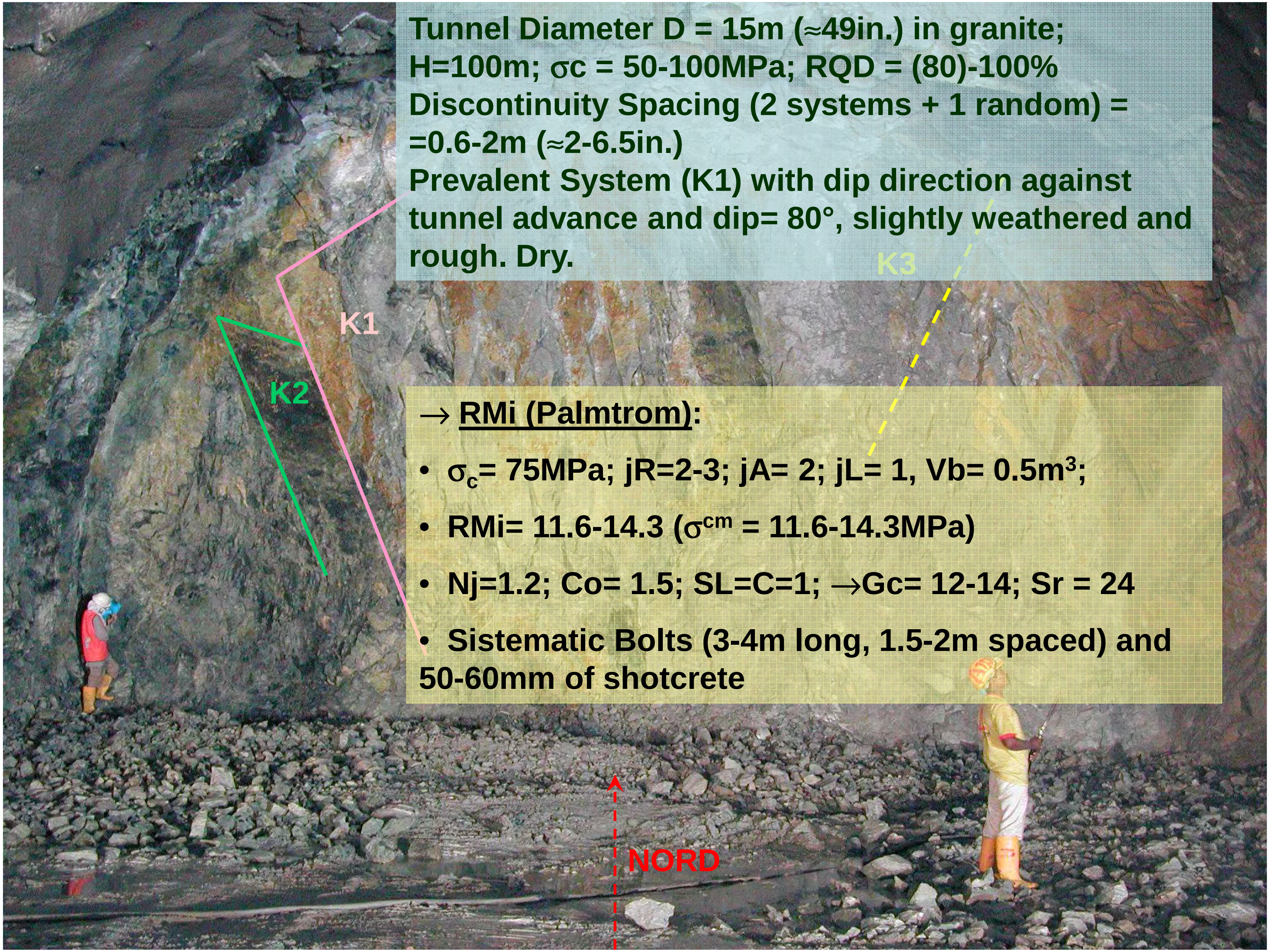


Fig. 46 [41]: Recommended application of the support charts





Tunnel Diameter D = 15m (≈49in.) in granite;
H=100m; σc = 50-100MPa; RQD = (80)-100%
Discontinuity Spacing (2 systems + 1 random) = =0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against tunnel advance and dip= 80°, slightly weathered and rough. Dry.

→ <u>RMi (Palmtrom)</u>:

- σ_c= 75MPa; jR=2-3; jA= 2; jL= 1, Vb= 0.5m^3;
- RMi= 11.6-14.3 (σ^{cm} = 11.6-14.3MPa)
- Nj=1.2; Co= 1.5; SL=C=1; →Gc= 12-14; Sr = 24
- Sistematic Bolts (3-4m long, 1.5-2m spaced) and 50-60mm of shotcrete

V- Table of Comparison

	Bolts (L/spacing) m	Shotcrete mm
Terzaghi	(light localized support)	
Rabcewicz-P.	Localized + wire mesh	(in alternative)
RSR-Concept	(no systematic support)	
RMR-System	3/2.5 + wire mesh	50 (eventual)
Q-System	$(3 \div 5)/(2 \div 3)$	
RMi	$(3 \div 4)/(1.5 \div 2)$	$50 \div 60$

GEOLOGICAL STRENGTH INDEX (GSI, Hoek et al., 1995÷2000)

- The GSI is introduced to better represent the rock mass structure, without to take into account other parameters such as intact strength, stress conditions, the orientation of discontinuity, the presence of water, etc.

- Initially, the Authors suggested to derive GSI:
  - →a) from a modified RMR
  - →b) from a modified Q-index
- In the following:
  - →c) from graphs (qualitative assessment: Figg.47,48,49) [26,36]
- More recently, other Authors proposed:
  - →d) from the same graph but with quantitative assessment (Figg.50a,b) [11]
  - →e) quantitative assessment by the same input parameters for the JP estimation of RMi system (Figg. 51a,b,c,d,e) [49,50]

a) **From $\mathbf{RMR_{(1989)}}$**

- A modified RMR is calculated (RMR') considering a dry condition (parameter *e*=15) and disregarding the adjustment for the orientation of discontinuities (*f* = 0).
- If RMR' ≥ 23 :

$$\mathbf{GSI = RMR' - 5} \qquad (*)$$

- If RMR' < 23 the GSI must be calculated using the Q-System.

Note:

- (*) if the 1976 RMR System version is used (max rating for water's parameter *e*=10), than GSI =$RMR'_{(1976)}$
- conceptual problem→ GSI is mainly used to scale intact rock properties to rock mass conditions and than should be a pure geostructural index: nevertheless, RMR includes intact rock strength and than the calculation of GSI by RMR does not appear a correct procedure.

b) From Q

- Analogously, a modified Q is calculated (Q'), with Jw/SRF = 1.

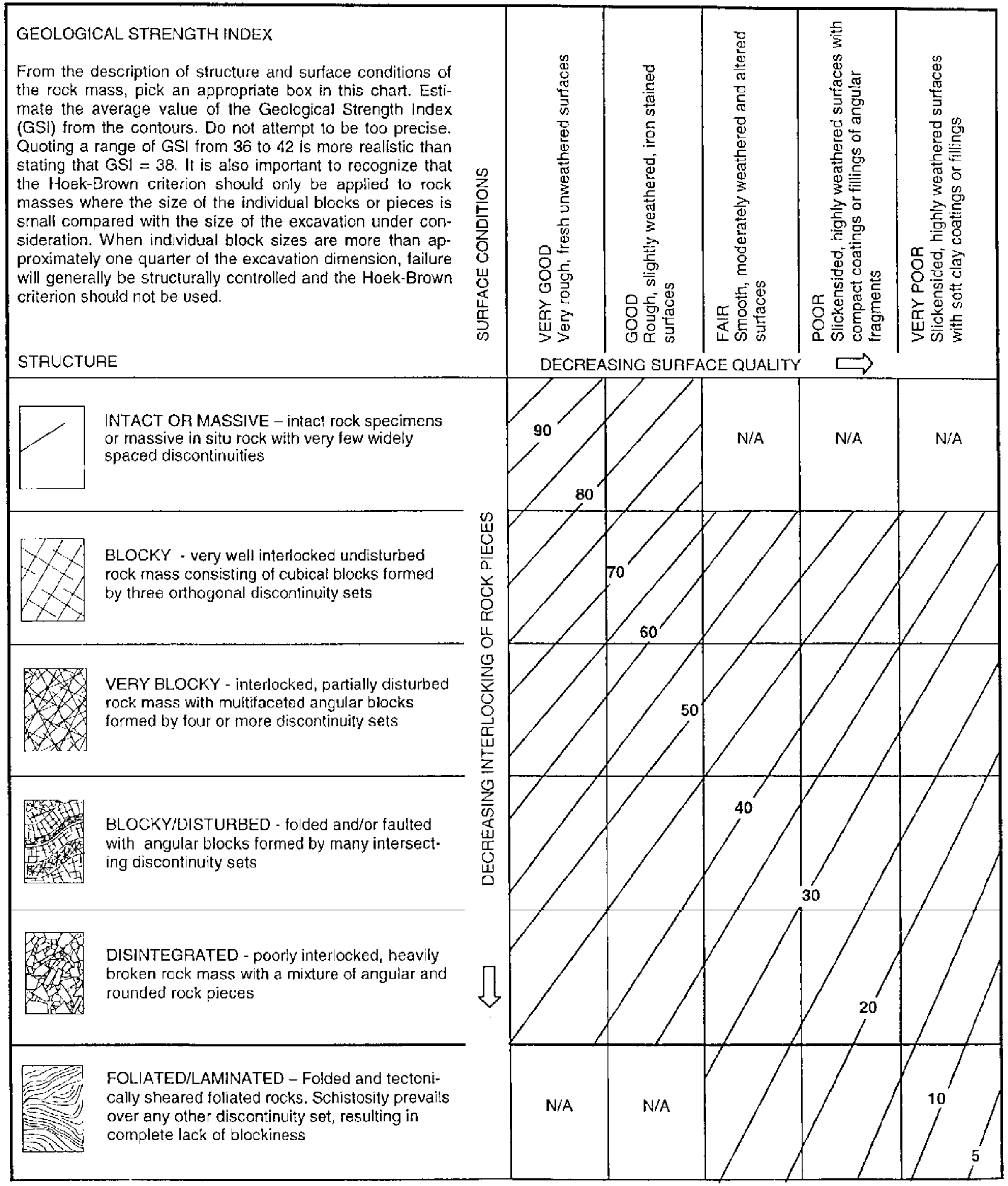
  Therefore:

  GSI = 9lnQ' + 44

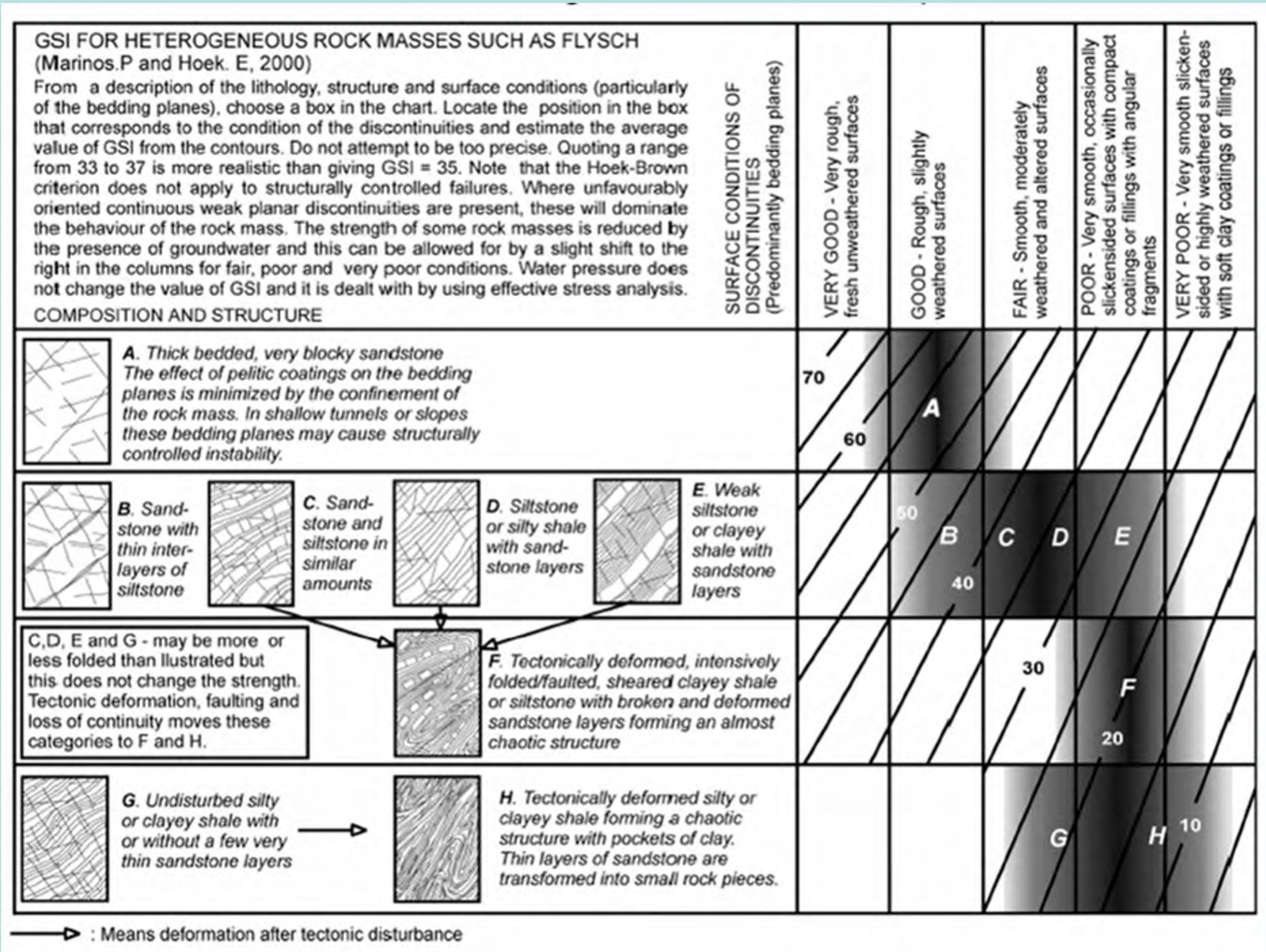
- Use this expression even when RMR'<23.

c)

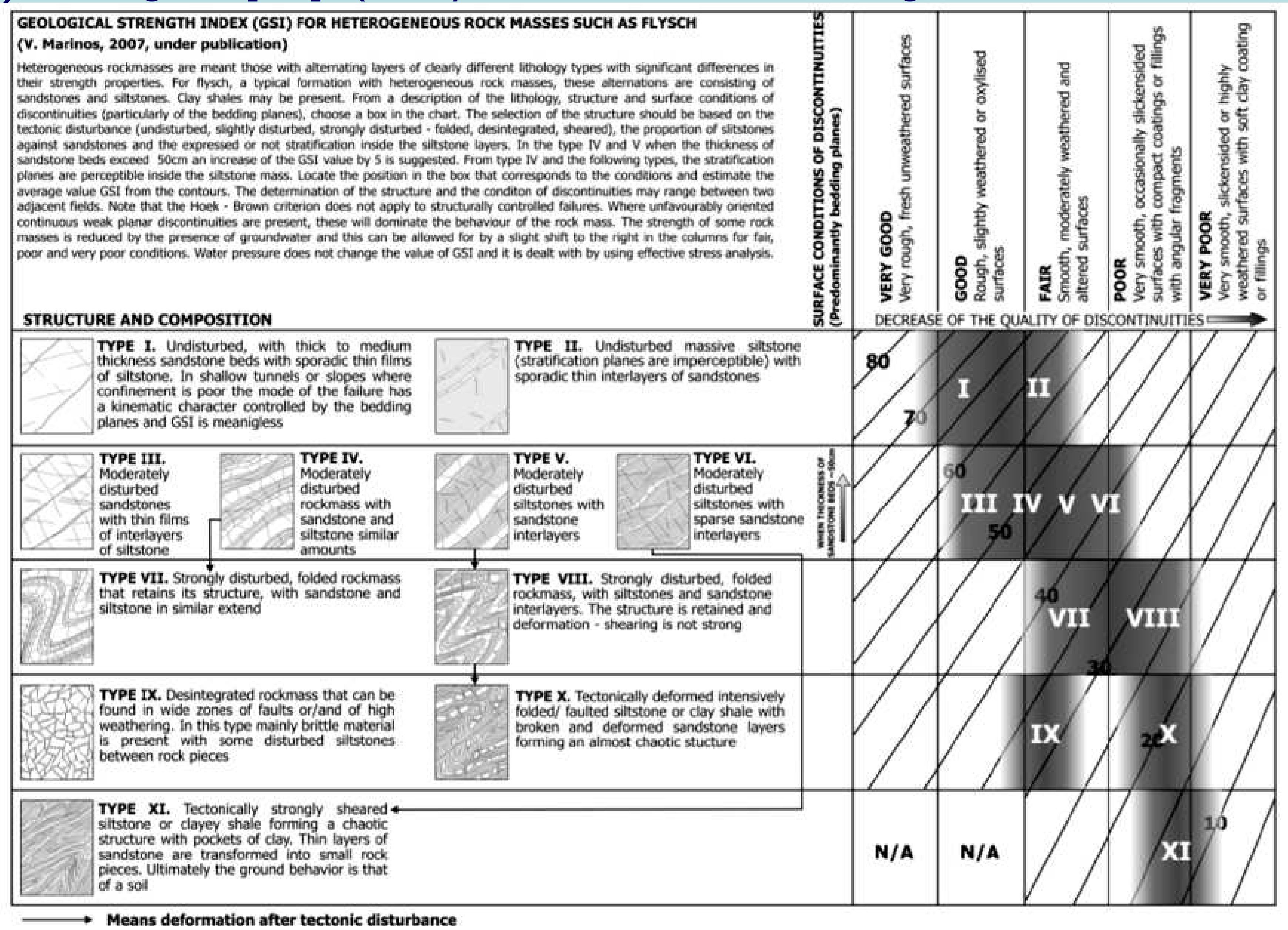
Fig.47 [26]: GSI Chart

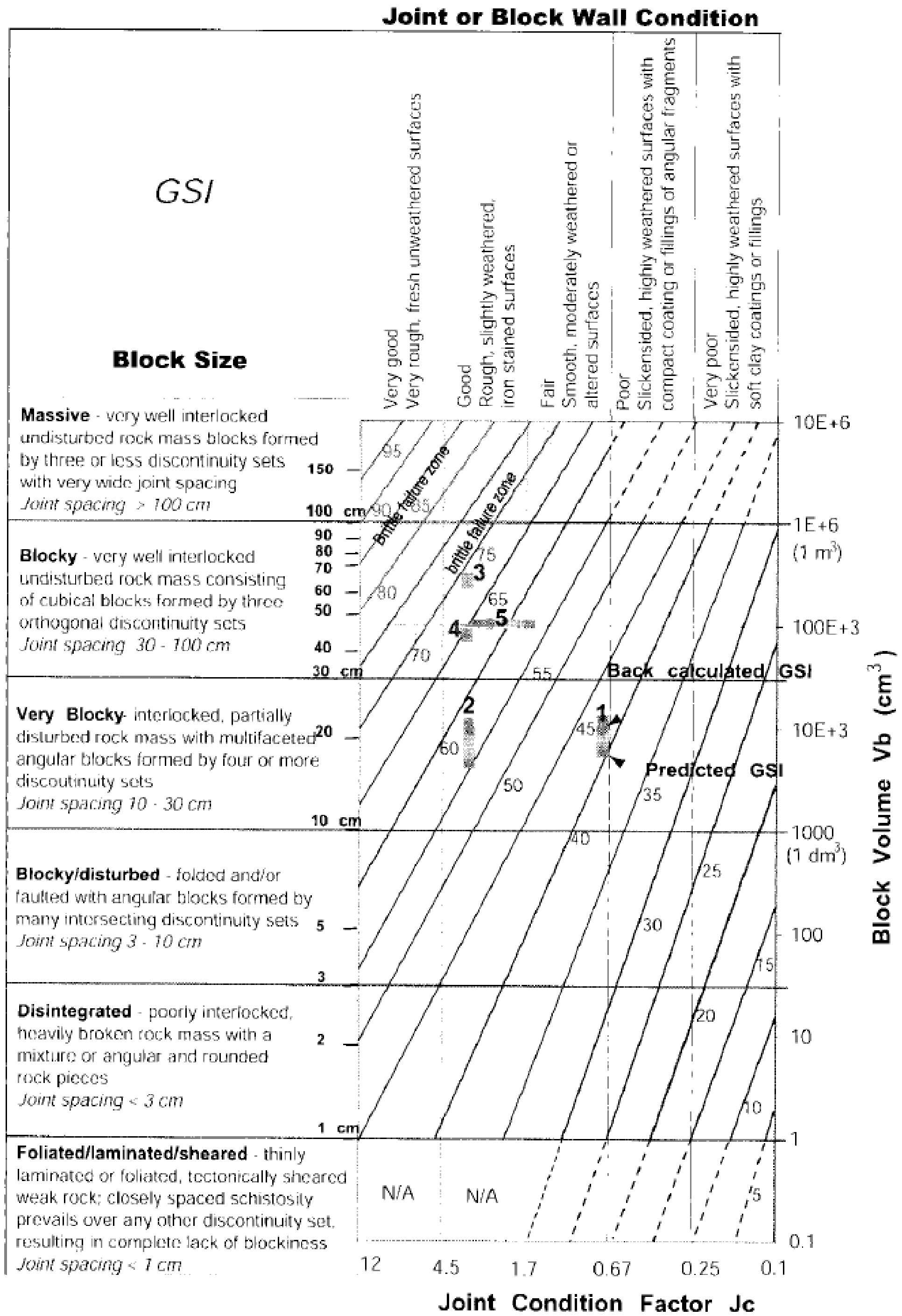


c) Fig.48 [26]: GSI chart for heterogeneous rock masses



c) Fig.49 [36]: (new) GSI chart for heterogeneous rock masses





VI - Geological Strength Index (GEO→*G2*)

d)

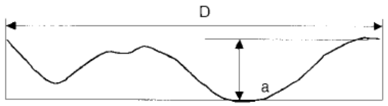
Fig.50a [11]: Modified GSI graph proposed by Cai et al. (2004)

Quantitative assessment of input parameters

Geological Strength Index (GEO→G2)

Table 2
Terms to describe large-scale waviness [27]

Waviness terms	Undulation	Rating for waviness J_W
Interlocking (large-scale)		3
Stepped		2.5
Large undulation	> 3%	2
Small to moderate undulation	0.3 3%	1.5
Planar	< 0.3%	1



Undulation = a/D
D - length between maximum amplitudes

Table 3
Terms to describe small-scale smoothness [27]

Smoothness terms	Description	Rating for smoothness J_S
Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface	3
Rough	Some ridge and side-angle are evident; asperities are clearly visible; discontinuity surface feels very abrasive (rougher than sandpaper grade 30)	2
Slightly rough	Asperities on the discontinuity surfaces are distinguishable and can be felt (like sandpaper grade 30 300)	1.5
Smooth	Surface appear smooth and feels so to touch (smoother than sandpaper grade 300)	1
Polished	Visual evidence of polishing exists. This is often seen in coating of chlorite and specially talc	0.75
Slickensided	Polished and striated surface that results from sliding along a fault surface or other movement surface	0.6–1.5

Table 4
Rating for the joint alteration factor J_A [4,27]

	Term	Description	J_A
Rock wall contact	*Clear joints*		
	Healed or "welded" joints (unweathered)	Softening, impermeable filling (quartz, epidote, etc.)	0.75
	Fresh rock walls (unweathered)	No coating or filling on joint surface, except for staining	1
	Alteration of joint wall: slightly to moderately weathered	The joint surface exhibits one class higher alteration than the rock	2
	Alteration of joint wall: highly weathered	The joint surface exhibits two classes higher alteration than the rock	4
	Coating or thin filling		
	Sand, silt, calcite, etc.	Coating of frictional material without clay	3
	Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals	4
Filled joints with partial or no contact between the rock wall surfaces	Sand, silt, calcite, etc.	Filling of frictional material without clay	4
	Compacted clay materials	"Hard" filling of softening and cohesive materials	6
	Soft clay materials	Medium to low over-consolidation of filling	8
	Swelling clay materials	Filling material exhibits swelling properties	8 12

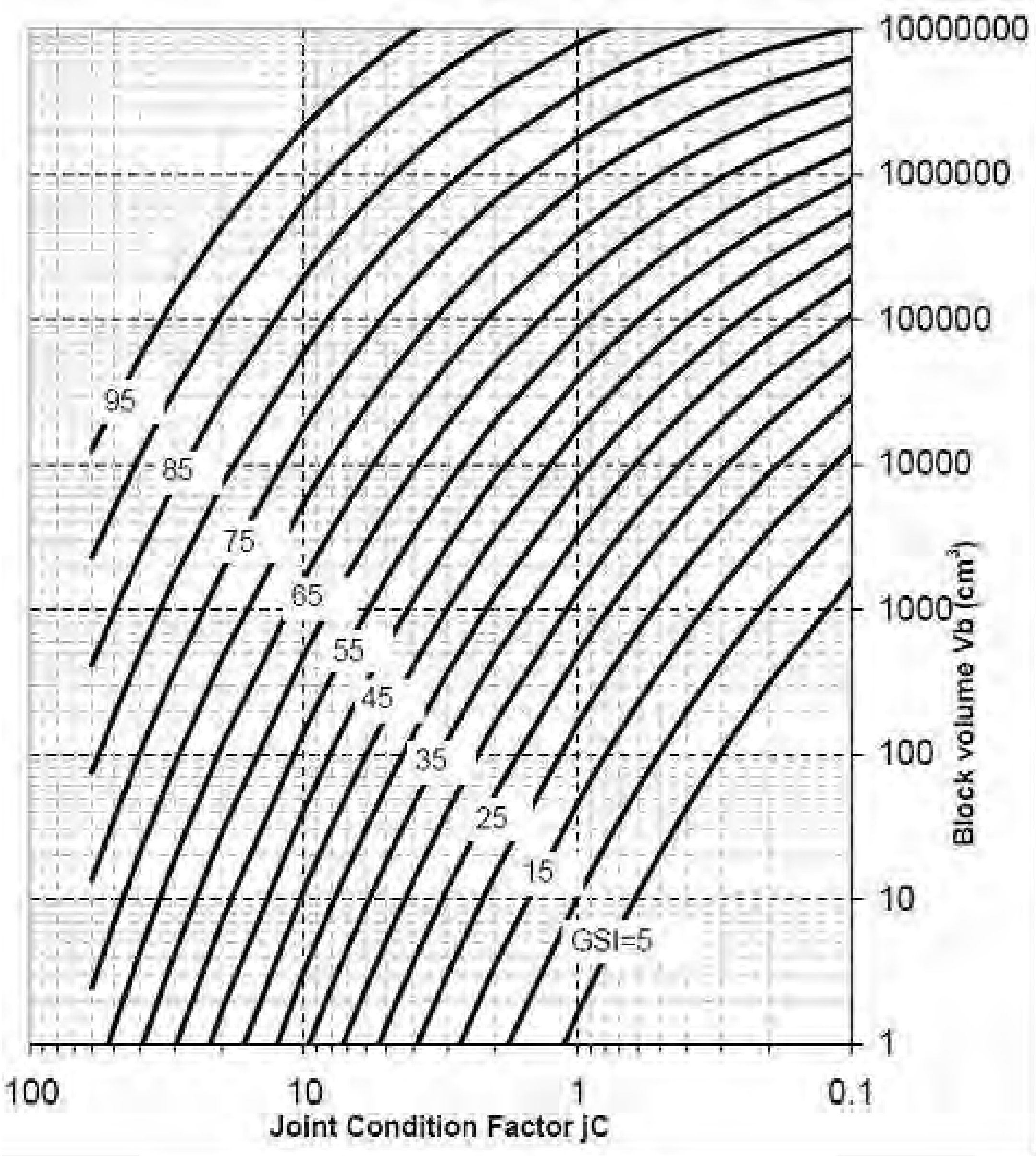
d)

Fig.50b [11]: Tables for evaluating the Joint Condition Factor J_C

$$J_C = J_W * J_S / J_A$$

e)

VI - GSI (GEO→G2)

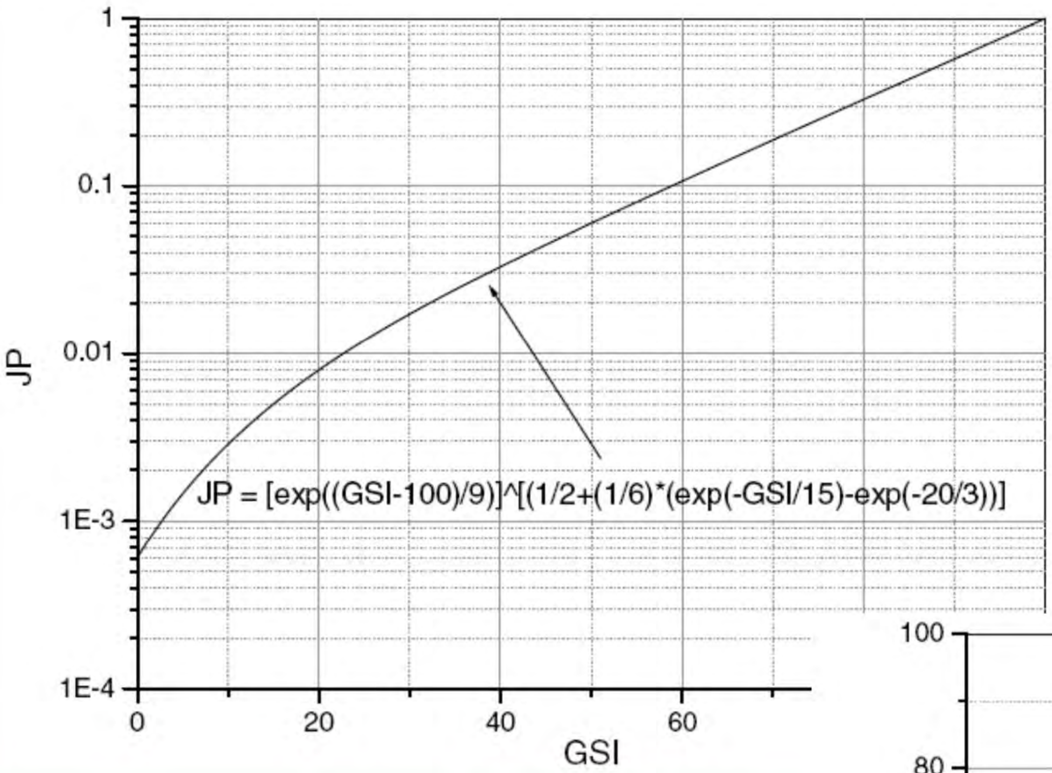


jC	TYPICAL CONDITIONS[1]
24	Discontinuous cracks
12	Small, rough fessures
6	Undulating, rough short joint
3	Undulating, rough joint
1.75	Slightly undulating, rough joint
1	Smooth, planar joint
0.5	Weathered joint wall
0.2	Clay coated joint
0.1	Filled joint

[1]Palmstrom, 2000 (refer to the original RMi tables for a more precise estimation of jC)

Fig.51a [49,50]: Integrated GSI-RMi system (GRs approach, 2007-2009) - Quantitative assessment of the same input parameters for estimating JP of RMi

e) VI - GSI (GEO→G2)



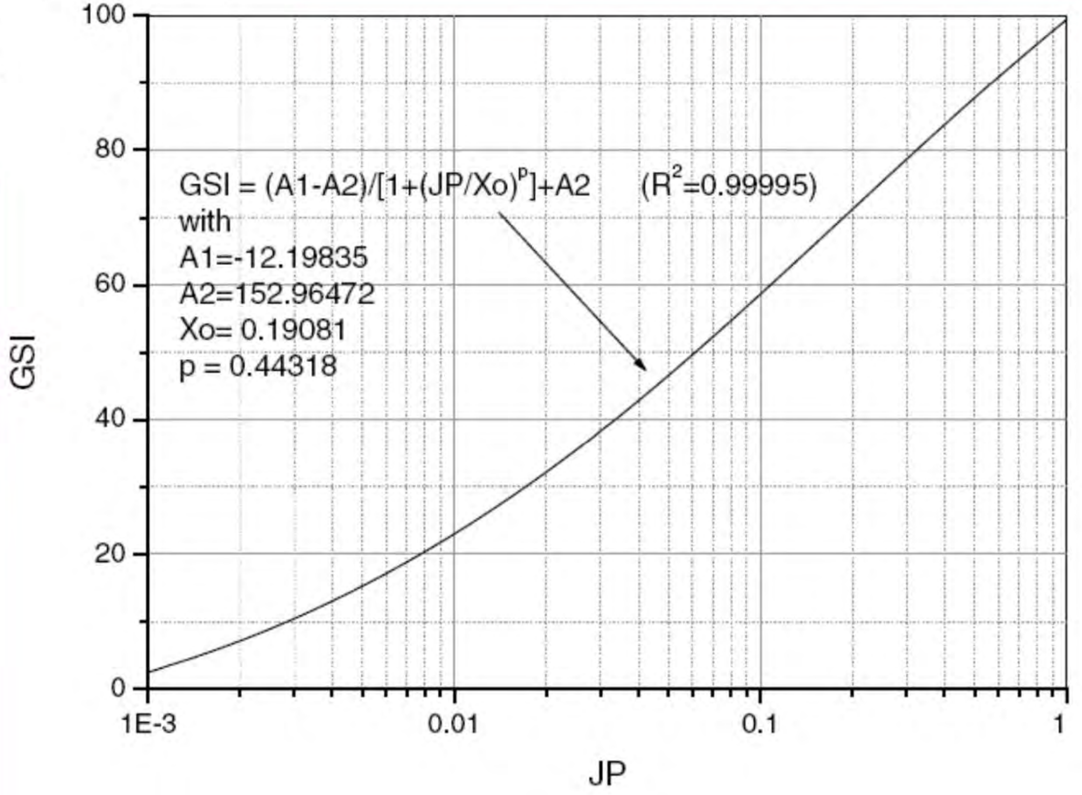
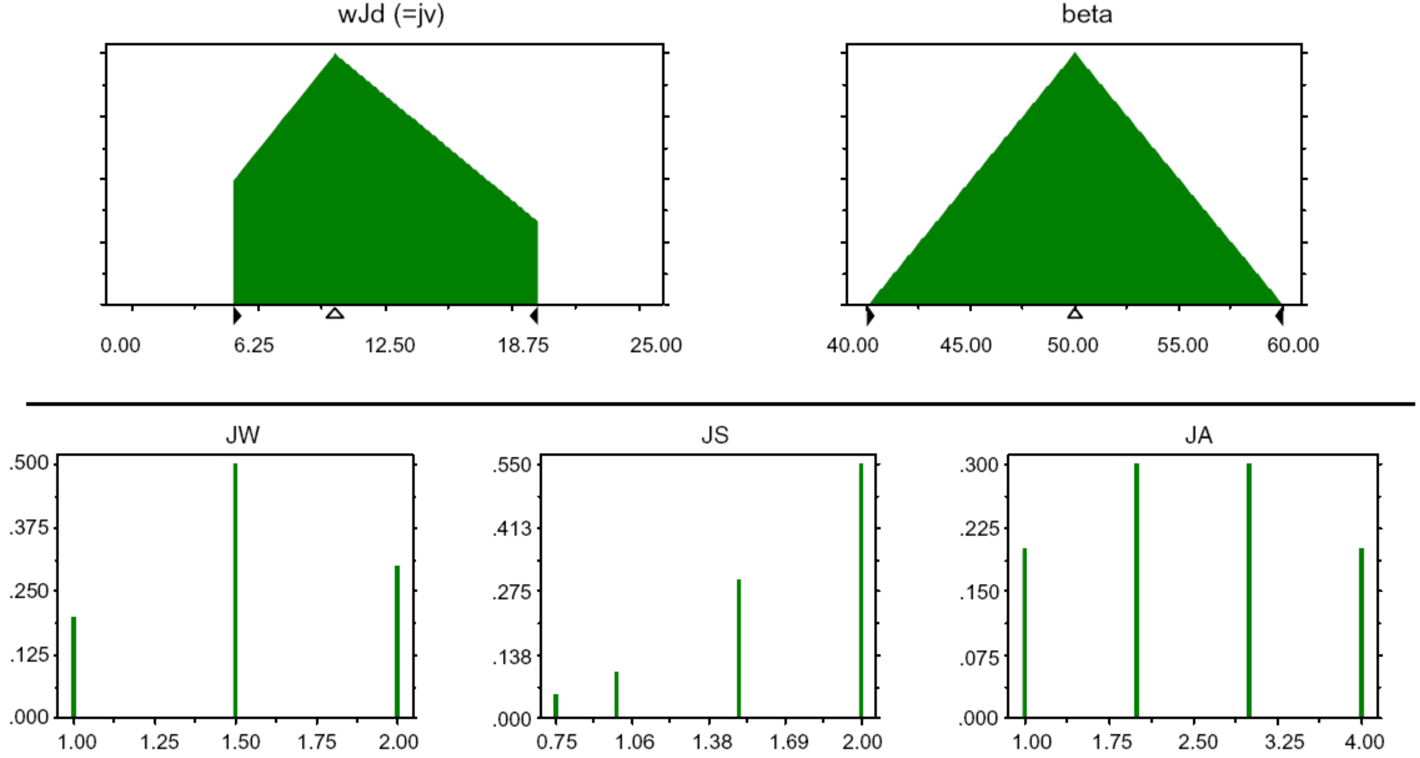


Fig.51b [50]: GRs approach, 2007: Relationship between GSI and JP



GSI (GEO→*G2*)

Fig.51c [49,50]: Probabilistic application of the GSI quantitative approaches

- Statistical analysis of available data (from boreholes, geostructural survey,..)
- Best fitting analysis and evaluation of the of the most appropriate probabilistic distribution (continuous or discrete) for each input parameter
- Definition of the eventual correlations among parameter
- Application of MonteCarlo sampling method to derive the possible GSI variability, as result of the variability of input parameters and their random combinations

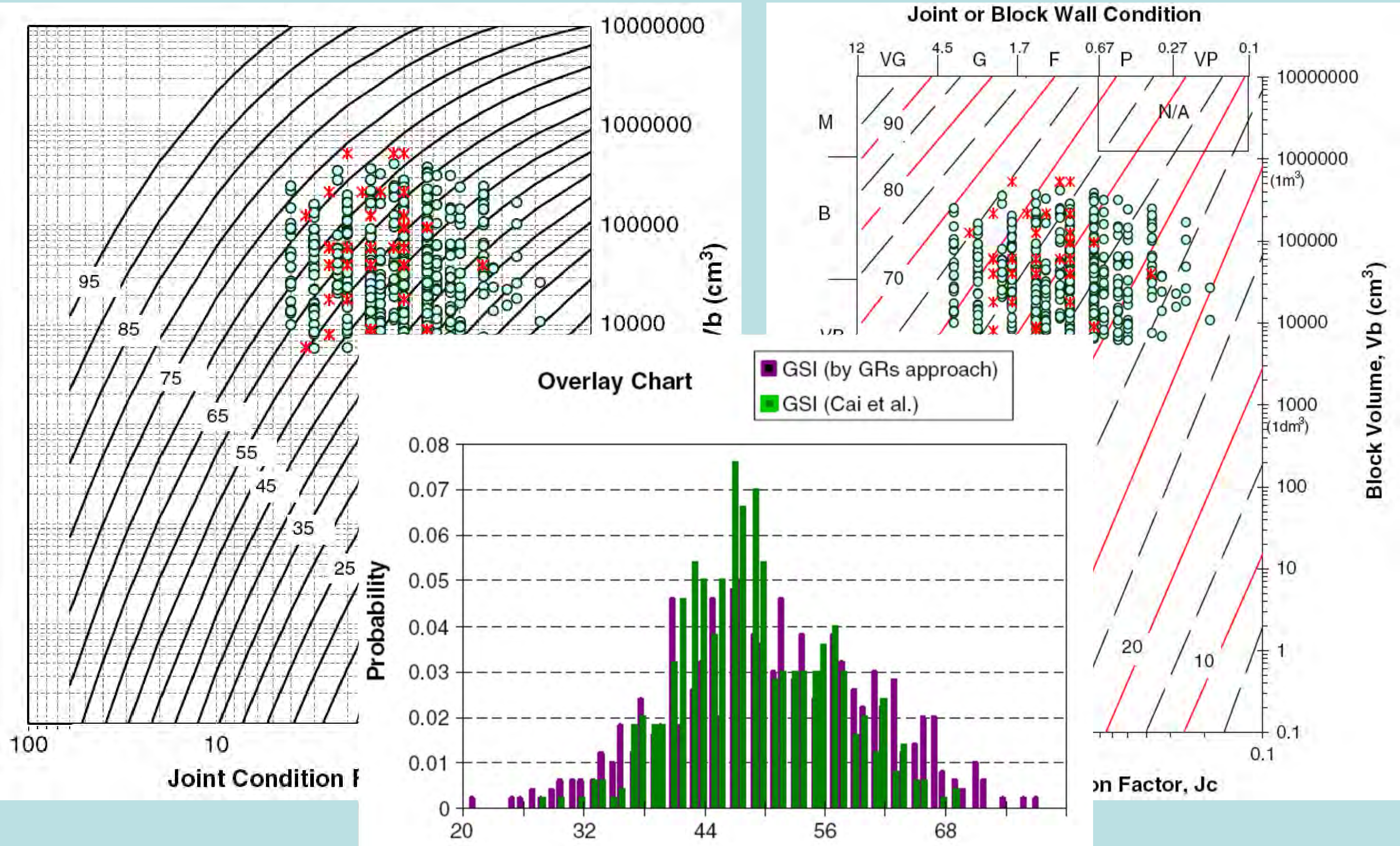


Fig.51d: Comparison between GRs (←) and Cai et al. (→) methods

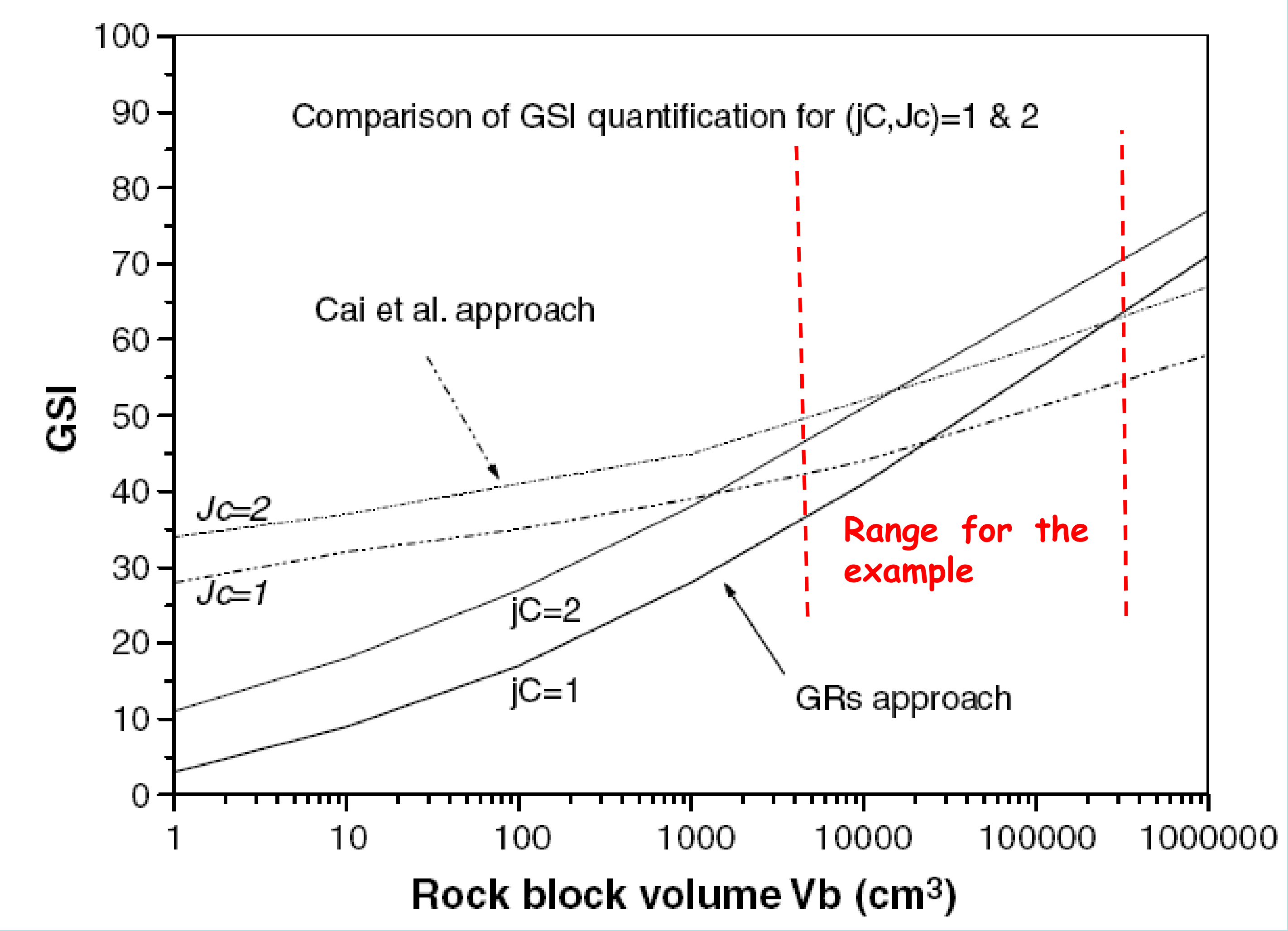


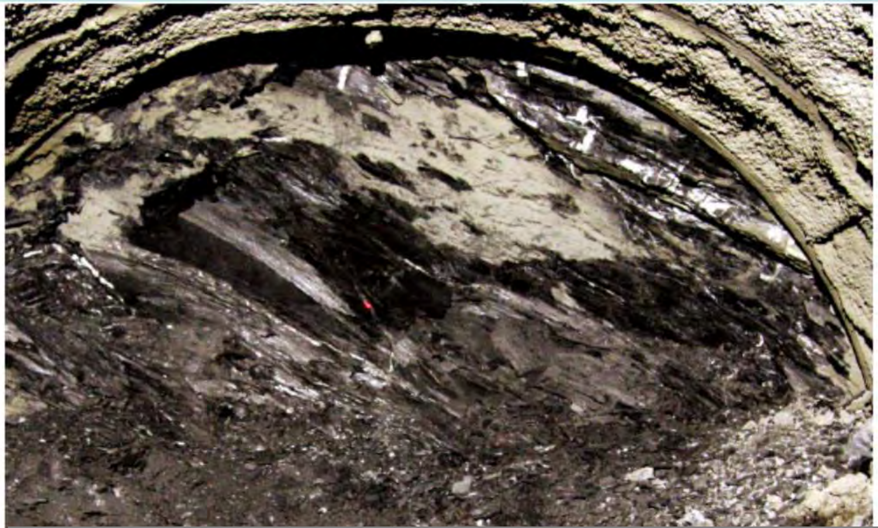
Fig.51e: differences between the Cai and GRs approaches [50]

Fig.51 f

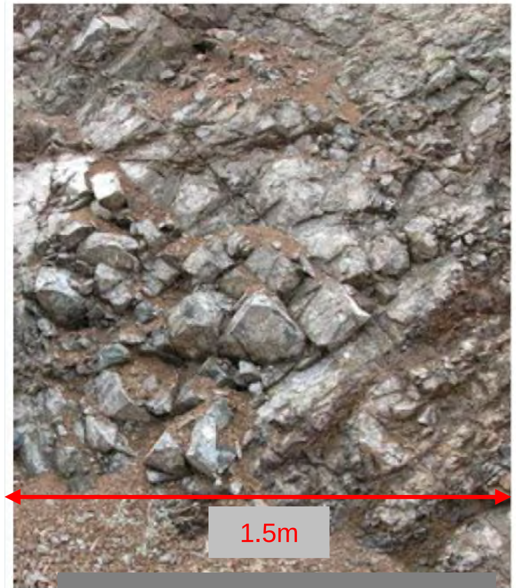
Geostructural index: GSI

Some H&M GSI estimates (1of2)

SMLP(LTF): graphitic schist GSI=25-40

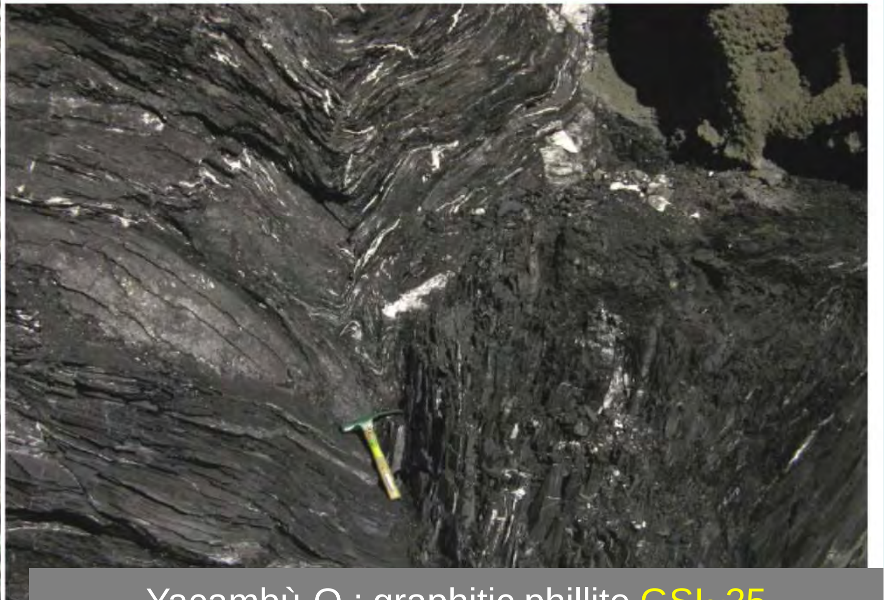


Yacambù-Q.: graphitic phillite GSI≈35



Peridotite GSI≈55

Volcanic rock GSI≈30



Yacambù-Q.: graphitic phillite GSI≈25

Geostructural index: GSI

Some H&M GSI estimates (2of2)

Subjectivity is a problem?

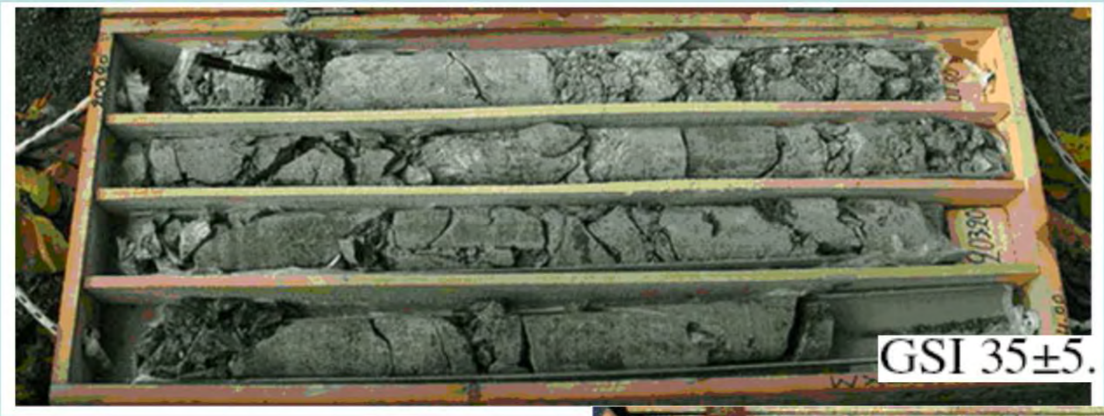


Fig.51 g

The GSI is correlated to the main geomechanical rock mass parameters (→equivalent-continuum modelling)

Shear strength

Referring to the generalized Hoek and Brown failure criterion [27]

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a$$

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D} \right)$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D} \right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3} \right)$$

σ_1', σ_3'	Effective principal stresses
m_b, s, a	Hoek and Brown rock mass constants[1]
D	Disturbancy factor (0→1)
[1]for intact rock: $m_b = m_i$; s=1; a=0.5	

Residual Shear Strength

GSIr= GSI*e(-0.0134*GSI)   [11b]

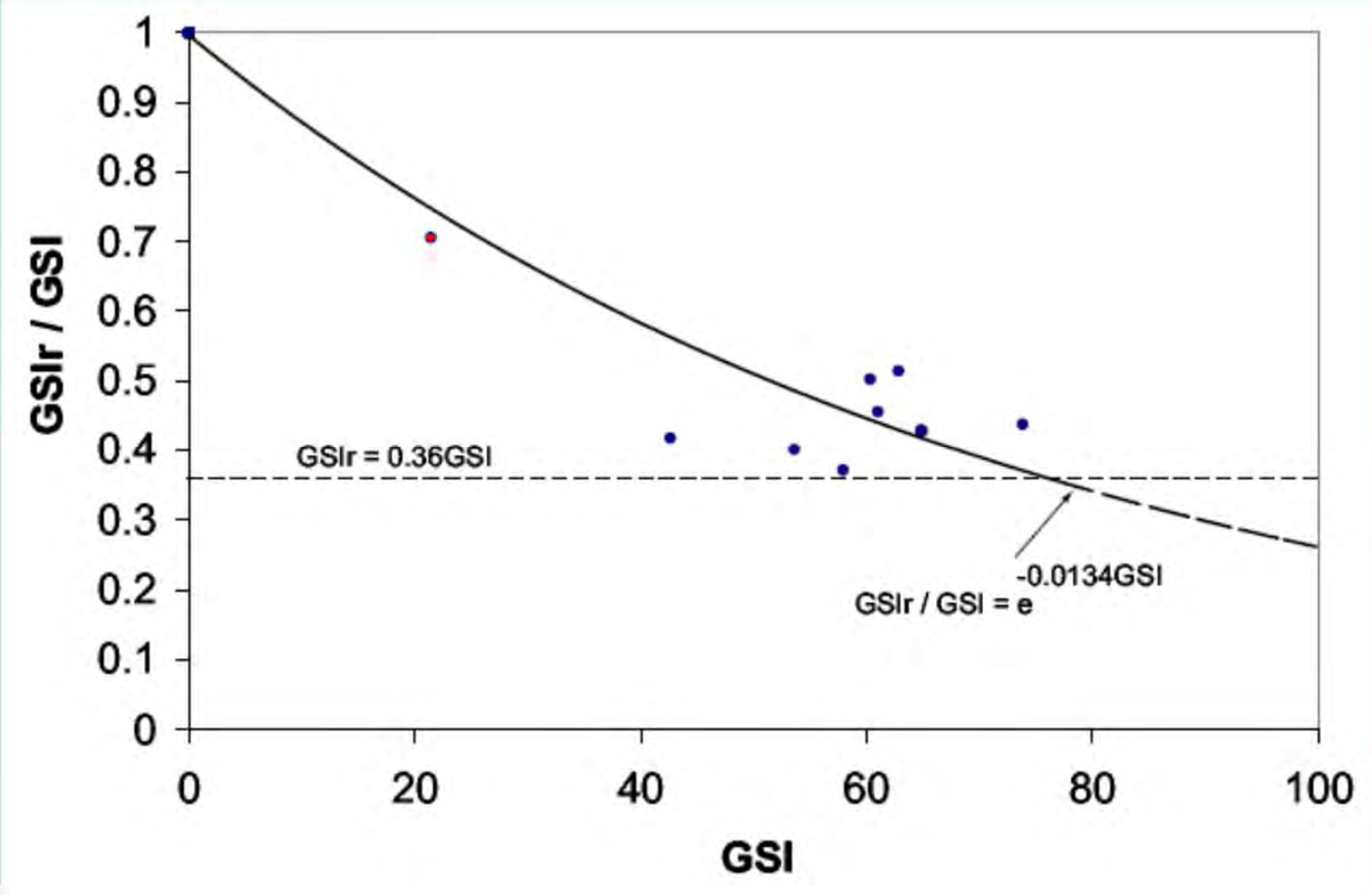


Fig. 52: relationship between the ratio GSIr/GSI and GSI [11b]
Note: the dotted linear equation has been previously proposed in [46]

Deformability

For $\sigma_c \leq 100$MPa→ $$E_d(GPa) = \left(1 - \frac{D}{2}\right)\sqrt{(\sigma_c/100)} * 10^{\frac{GSI-10}{40}}$$ [27]

For $\sigma_c > 100$MPa → $$E_d(GPa) = \left(1 - \frac{D}{2}\right) * 10^{\frac{GSI-10}{40}}$$ [27]

More recently, Hoek and Diederichs [25] have proposed:

Simplified formulation: $$E_d(GPa) = 100\left(\frac{1 - D/2}{1 + \exp((75 + 25D - GSI)/11)}\right)$$

Complete formulation: $$E_d = E_i\left(0.02 + \frac{1 - D/2}{1 + \exp((60 + 15D - GSI)/11)}\right)$$

where Ei= Elasticy modulus from laboratory test

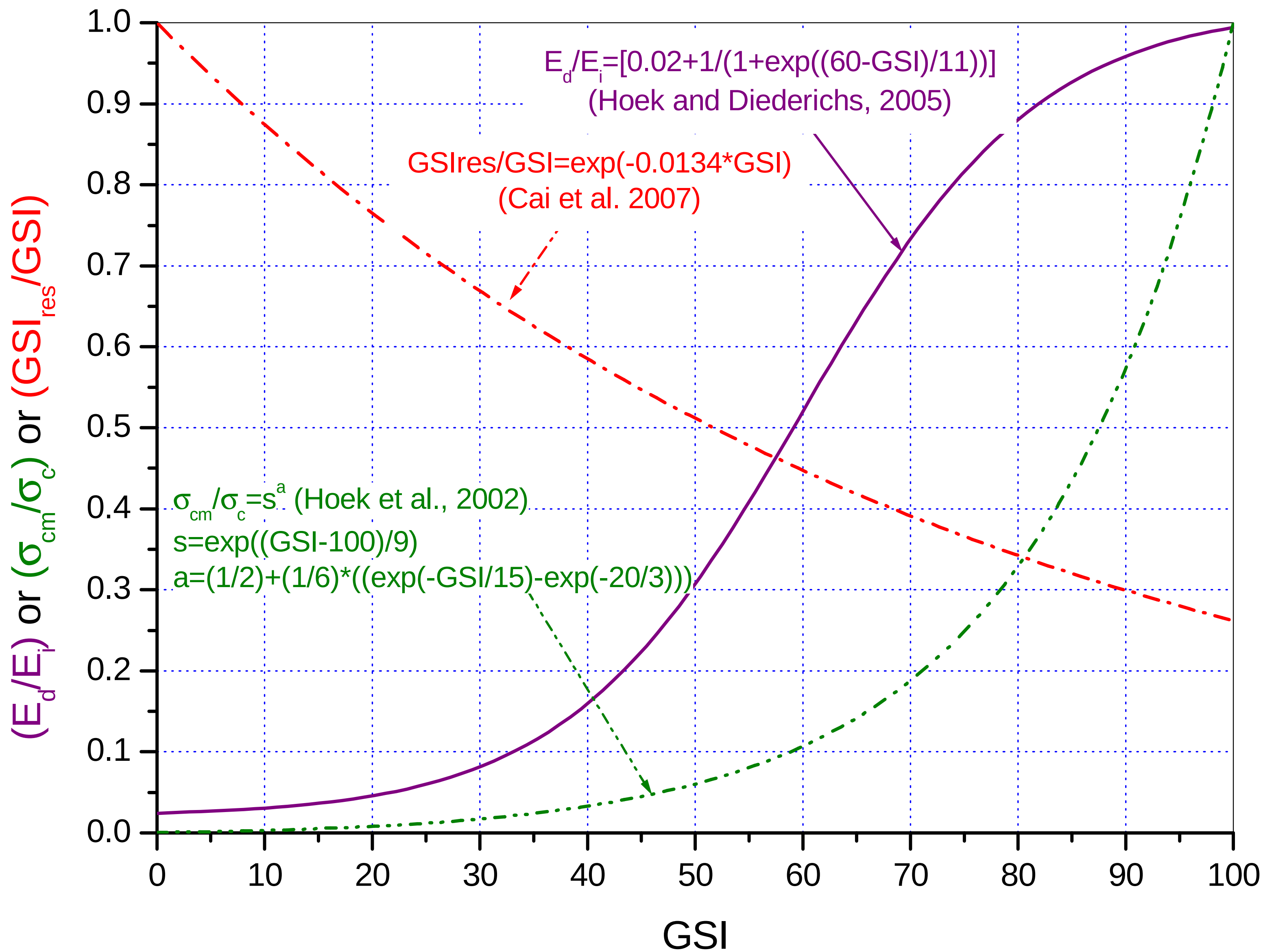


Fig.53: Equations based on GSI are used to derive the H&B rock mass constants (m,s,a) and the modulus of deformability (E_d)

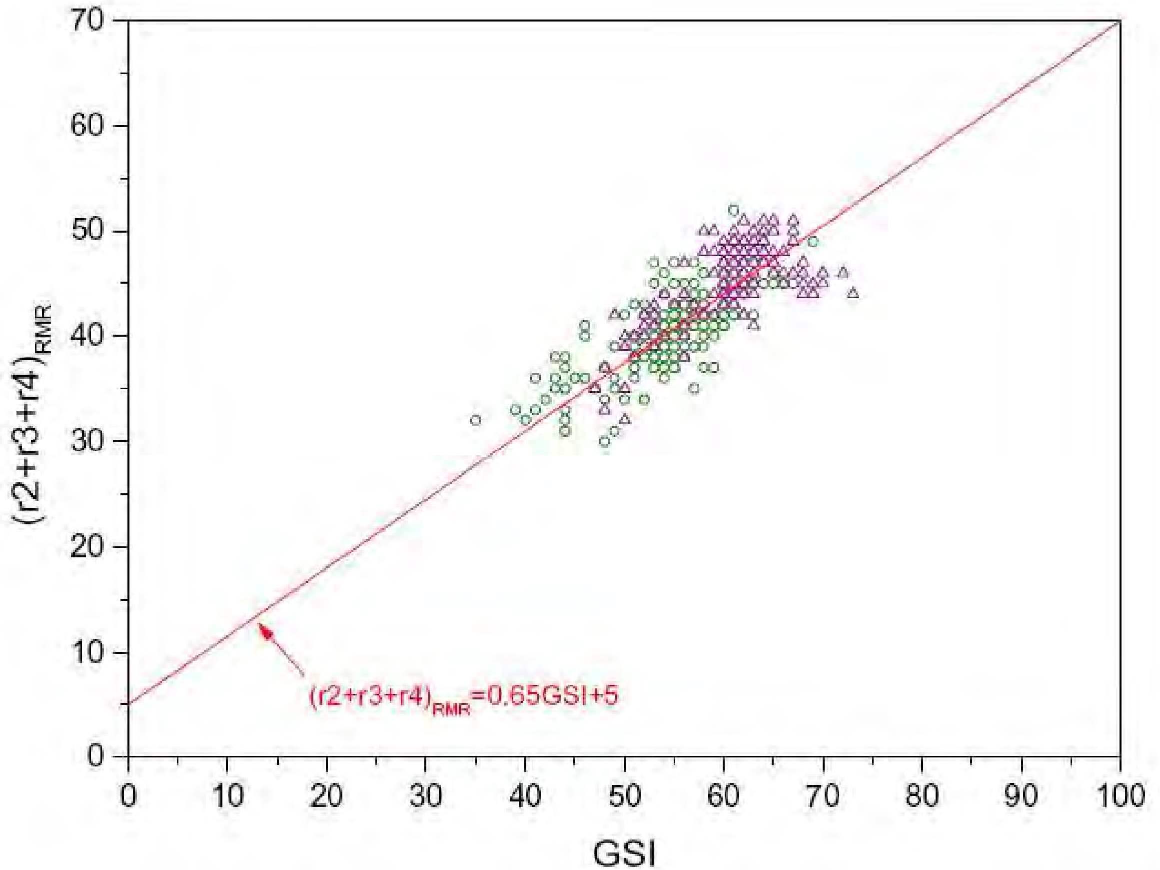
The parameters r2, r3 and r4 represent the geostructural component of RMR and their sum is therefore conceptually equivalent to the GSI ("fabric index"). Consequently, given that the possible ranges of variability are 8 to 70 and 5 to 100, respectively, the following approximate equation can be derived:

$$(r2+r3+r4) \approx 0.65GSI+5$$

or, more in general

$$RMR \approx 0.65GSI+5+r1+r5+r6$$

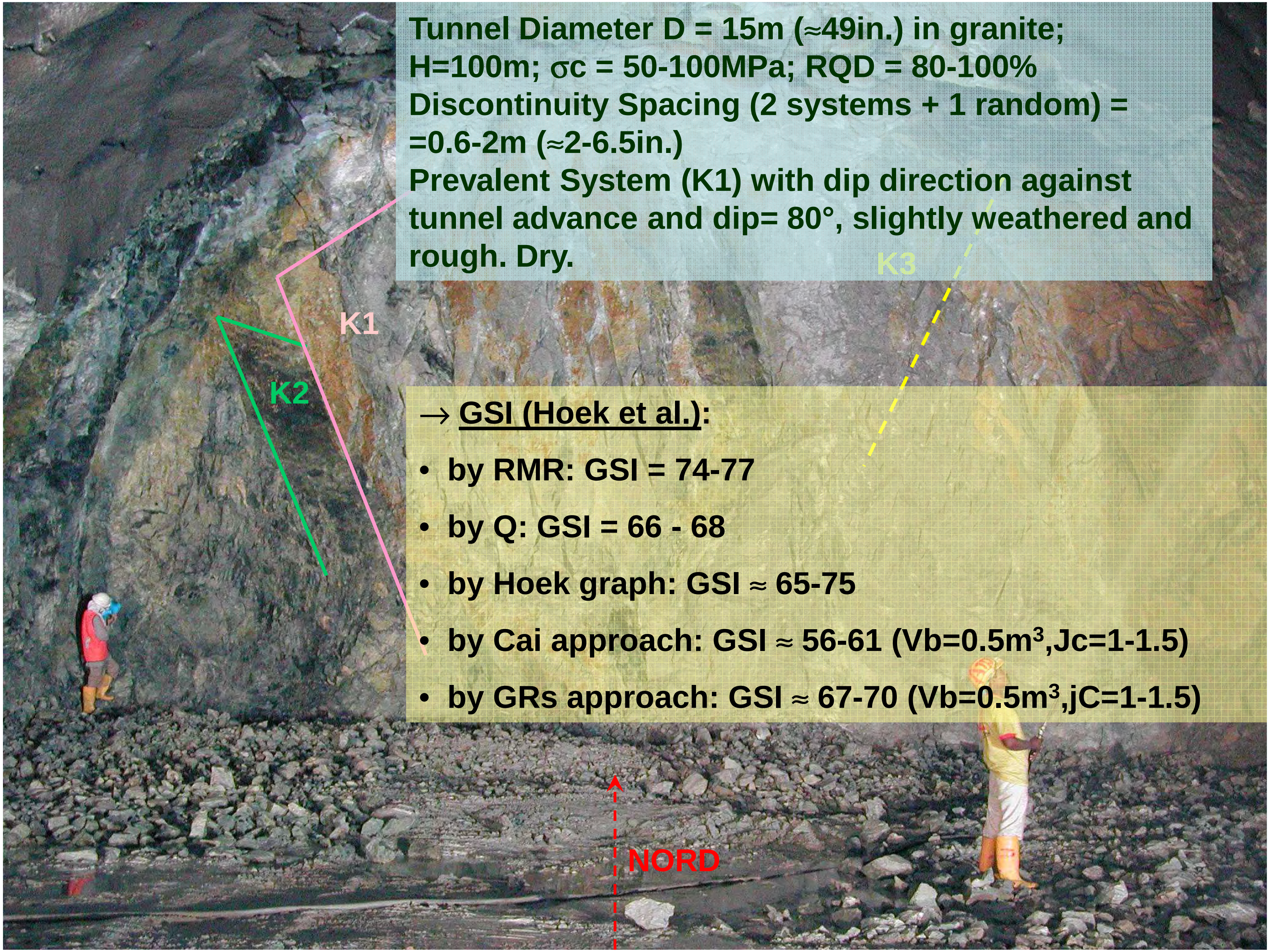
Note that according the RMR update [33b] r2+r3 is assigned by the number of disconinuities per meter



GSI and RMR parameters affinity

[ref. 51 ter]

Fig.53 a



Tunnel Diameter D = 15m (≈49in.) in granite;
H=100m; σc = 50-100MPa; RQD = 80-100%
Discontinuity Spacing (2 systems + 1 random) =
=0.6-2m (≈2-6.5in.)
Prevalent System (K1) with dip direction against tunnel advance and dip= 80°, slightly weathered and rough. Dry.

→ <u>GSI (Hoek et al.)</u>:

- **by RMR: GSI = 74-77**
- **by Q: GSI = 66 - 68**
- **by Hoek graph: GSI ≈ 65-75**
- **by Cai approach: GSI ≈ 56-61 (Vb=0.5m³,Jc=1-1.5)**
- **by GRs approach: GSI ≈ 67-70 (Vb=0.5m³,jC=1-1.5)**

Correlations between classification indexes

$RMR = 9\ln Q+44$	Bieniawski	1976
$RMR = 13.5\log Q+43$	Rutledge	1978
$RMR \approx 50+15\log_{10}Q$	Barton	1995
$RSR = 13.3\log Q+46.5$	Rutledge	1978
$RSR = 0.77RMR+12.4$	Rutledge	1978
RMi = 10^[(RMR-40)/15]	Palmstrom	1996
$GSI = 9\ln Q'+44$	Hoek et al.	1995
$GSI = 10\ln Q'+32$ (R^2=0.73)	Russo et al.	1998
$GSI \approx 153-165/[1+(JP/0.19)^{0.44}]$	Russo	2007

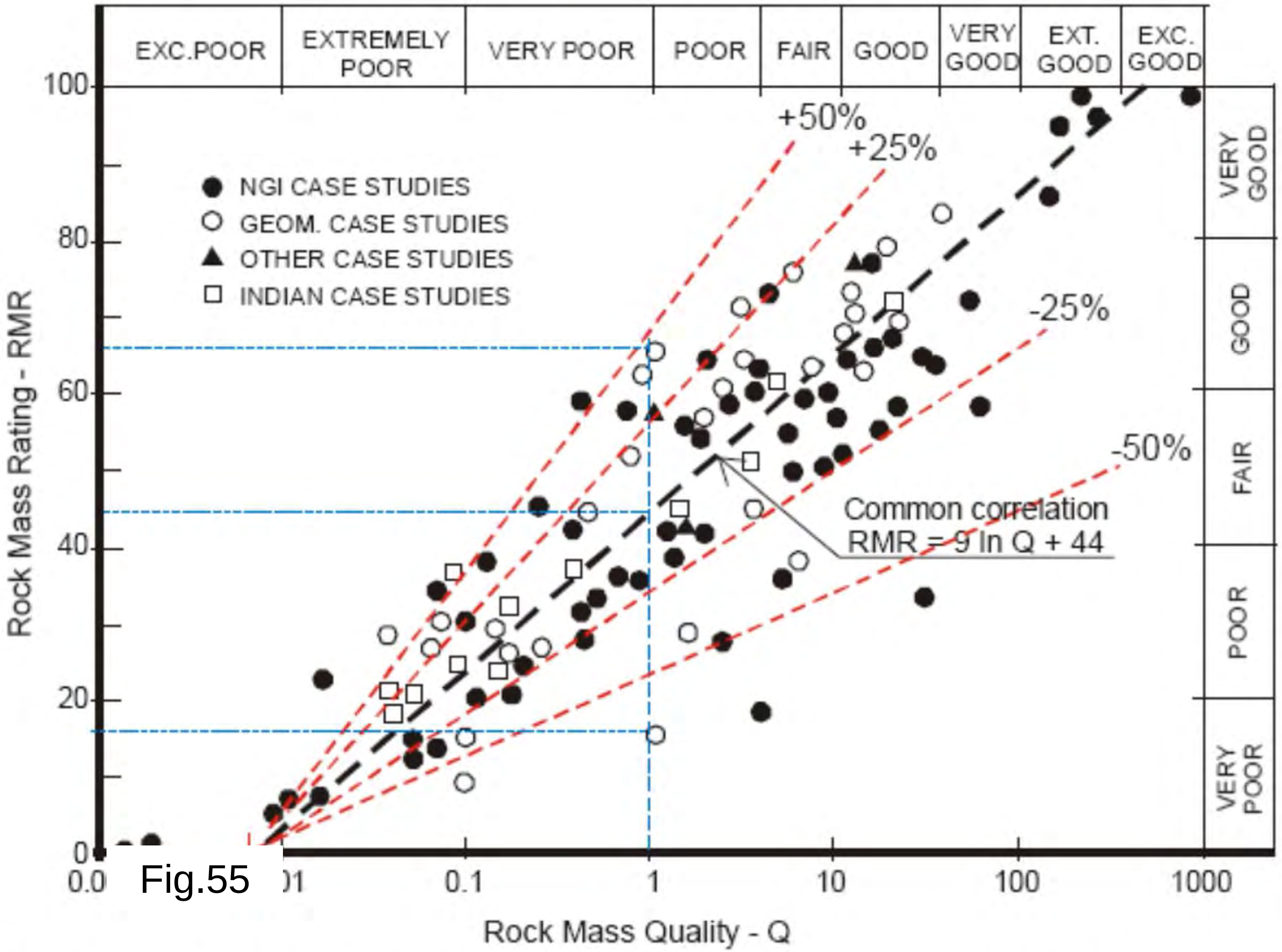


Fig.55

Figure 1: Correlation between the RMR and the Q-index with deviation from the common correlation. As seen, for Q = 1, RMR varies from less than 20 to 66. Note that the Q system applies logarithmic scale (After Bieniawski, 1976, and Jethwa et al., 1982).

[41bis]

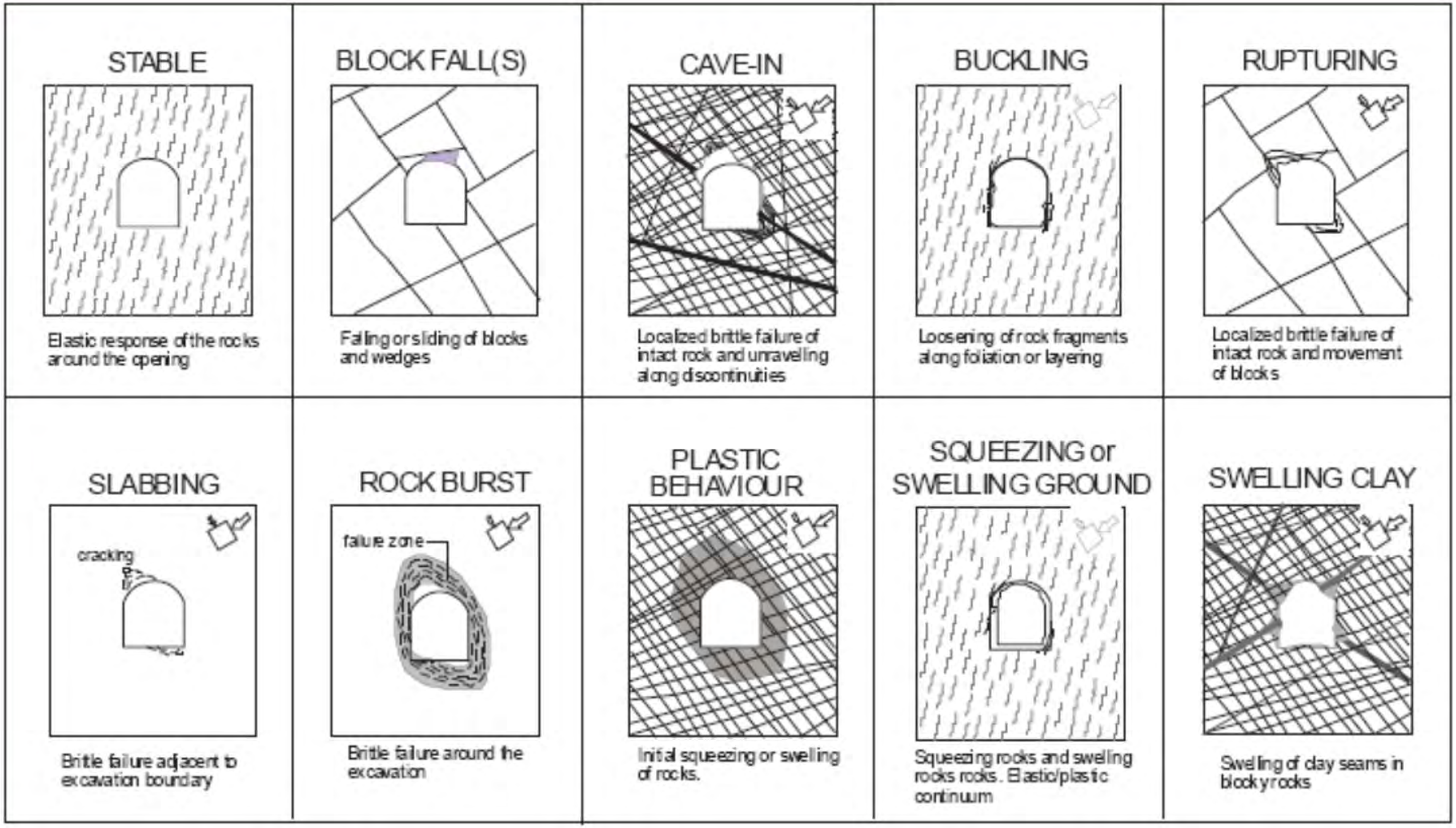


Fig.59: Some types of excavation behaviour (partly from Martin et al. 1999 and Hoek et al. 1995, as reported in [41quintum])

GROUP	GROUND BEHAVIOUR TYPE		DEFINITION	COMMENTS	
Type 1 Gravity driven	a. Stable		The surrounding ground will stand unsupported for several days or longer.	Massive, durable rocks at low and moderate depths.	
	b. Block fall(s)	of single blocks	Stable with potential fall of individual blocks	Discontinuity controlled failure.	
		of several blocks	Stable with potential fall of several blocks (slide volume < $10 m^3$).		
	c. Cave-in		Inward, quick movement of larger volumes (> $10\ m^3$) of rock fragments or pieces.	Encountered in highly jointed or crushed rock.	
	d. Running ground		A particulate material quickly invades the tunnel until a stable slope is formed at the face. Stand-up time is zero or nearly zero.	Examples are clean medium to coarse sands and gravels above groundwater level.	
Type 2 Stress induced	e. Buckling		Breaking out of fragments in tunnel surface.	Occurs in anisotropic, hard, brittle rock under sufficiently high load due to deflection of the rock structure.	brittle behaviour
	f. Rupturing from stresses		Gradually breaking up into pieces, flakes, or fragments in the tunnel surface.	The time dependent effect of slabbing or rock burst from redistribution of stresses.	
	g. Slabbing		Sudden, violent detachment of thin rock slabs from sides or roof.	Moderate to high overstressing of massive hard, brittle rock. Includes popping or spalling.[1)]	
	h. Rock burst		Much more violent than slabbing and involves considerably larger volumes (Heavy rock bursting often registers as a seismic event).	Very high overstressing of massive hard, brittle rock.	
	i. Plastic behaviour (initial)		Initial deformations caused by shear failures in combination with discontinuity and caused by overstressing	Takes place in plastic (deformable) rock from overstressing. Often the start of squeezing.	plastic behaviour
	j. Squeezing		Time dependent deformation, essentially associated with creep caused by overstressing. Deformations may terminate during construction or continue over a long period	Overstressed plastic, massive rocks and materials with a high percentage of micaceous minerals or of clay minerals with a low swelling capacity.	
Type 3 Water influenced	k. Ravelling from slaking		Ground breaks gradually up into pieces, flakes, or fragments.	Disintegration (slaking) of some moderately coherent and friable materials. Examples: mudstones and stiff, fissured clays.	hydrati-zation
	l. Swelling	of certain rocks	Advance of surrounding ground into the tunnel due to expansion caused by water adsorption. The process may sometimes be mistaken for squeezing.	Occurs in swelling of rocks, in which anhydrite, halite (rock salt) and swelling clay minerals, such as smectite (montmorillonite) constitute a significant portion.	swelling minerals
		of certain clay seams or fillings	Swelling of clay seams caused by adsorption of water. This leads to loosening of blocks and reduced shear strength of clay.	The swelling takes place in seams having fillings of swelling clay minerals (smectite, montmorillonite)	
	m. Flowing ground		A mixture of water and solids quickly invades the tunnel from all sides, including the invert.	May occur in tunnels below groundwater table in particulate materials with little or no coherence (and clay).	
	n. Water ingress		Pressurized water invades the excavation through channels or openings in rocks	May occur in porous and soluble rocks, or along significant openings or channels in fractures or joints.	

1) This term was often used by Terzaghi (1946) as synonymous with the falling out of individual blocks, primarily as a result of damage during excavation.

Fig.60: Main ground excavation behaviour [52bis]

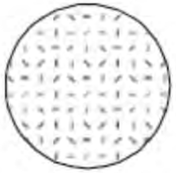
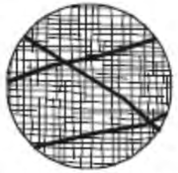
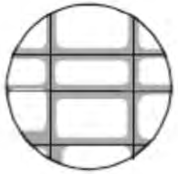
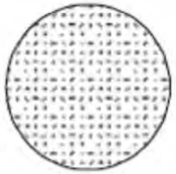
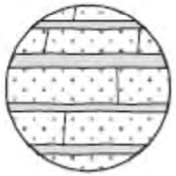
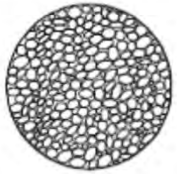
	MASSIVE ROCKS	JOINTED ROCKS or BLOCKY MATERIALS		PARTICULATE MATERIALS	SPECIAL MATERIALS
				occur often in weakness zones and faults	
	A	**B**	**C**	**D**	**E**
	Weak to strong rocks	**Rocks intersected by joints and partings**	**Jointed rocks intersected by seams or weak layers**	**Highly jointed or crushed rocks, and soil-like materials**	**Soft and weak materials**
1	 Brittle, homogeneous and foliated rocks (granite, gneiss, quartzite)	 Jointed homogeneous foliated and bedded rocks	 Jointed rocks intersected by seams (filled joints) (seamy and blocky ground)	 Highly jointed or crushed rocks with clay seams or shears	 Alternating soft and hard layers (as clay schist-sandstone-clay schist)
2	 Schistose (deformable) rocks rocks with high content of platy minerals	 Jointed, schistose rocks	 Prominent weathering along joints	 Highly jointed or crushed rocks (sugar-cube etc.) little clay	 Rock fragments with few contacts, in a matrix of soft (clayish) material
3	 Rocks with plastic properties (soapstone, rocksalt, many weathered rocks)	 Layered and bedded rocks with frequent partings (slate, flagstone)	 Jointed rocks with weak bedding layers	 Soil-like materials with friction properties (poorly cemented sandstones etc.)	 Soft or weak materials with plastic properties (mud-stone, clay-like materials)

Fig.61: Main types of rock mass compositions [52bis]

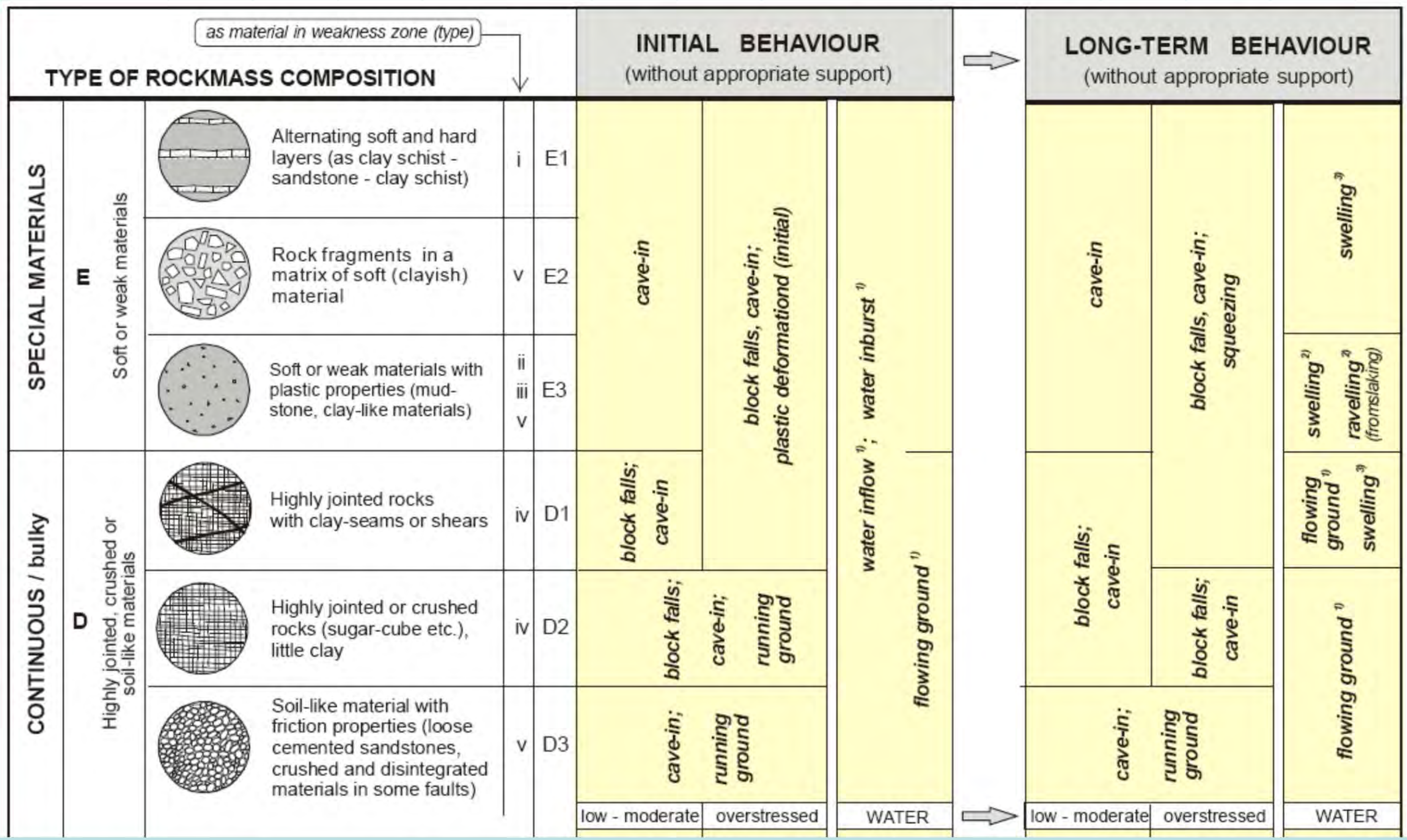


Fig.62a: Identification of excavation behaviour [52bis] (1 of 2)

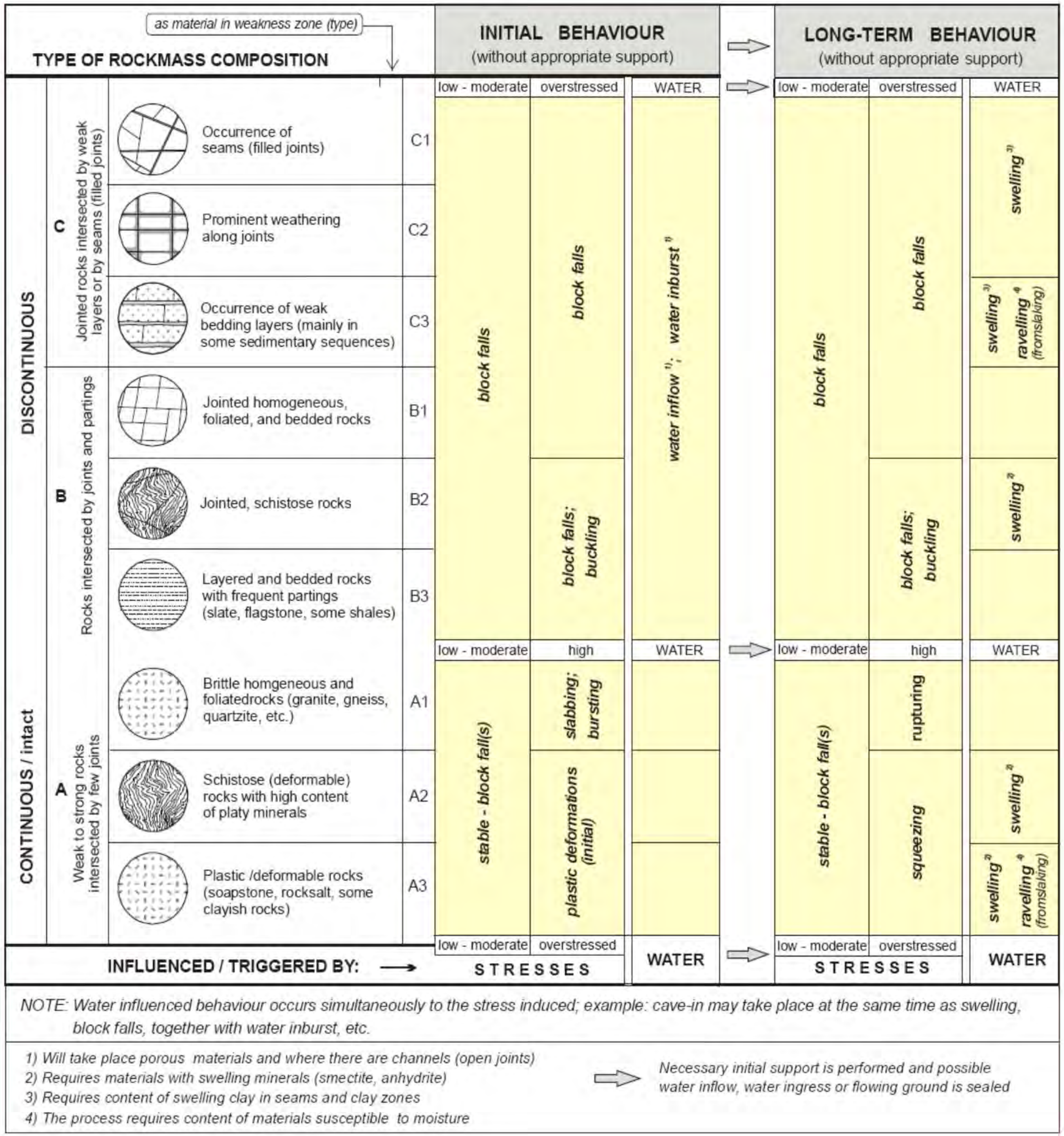


Fig.62b: Identification of excavation behaviour [52bis]

(2 of 2)

A quick overview on the Classification of the Behaviour of the excavation

It is possible to observe that there are methods based on

- stability of the cavity: for example the original Lauffer [30] system distinguished n.7 categories, from stable to very squeezing conditions
- stability of the tunnel face: for example Lunardi [34] proposed the Adeco RS approach, based on three categories: A (stable face), B (stable face in the short period) and C (unstable face)
- stability of both cavity and and tunnel face: for example, Lombardi [33] distinguished n.4 categories, taking into account all the possible combinations: from class I (face and cavity stable) to class IV (face and cavity unstable)

All these systems involve a qualitative assessment of the behaviour and therefore they are often open to individual interpretations. In the following a quantitative classification system developed in Geodata [46,47,48], based on deformation index of tunnel face, as well as of the cavity, is outlined.

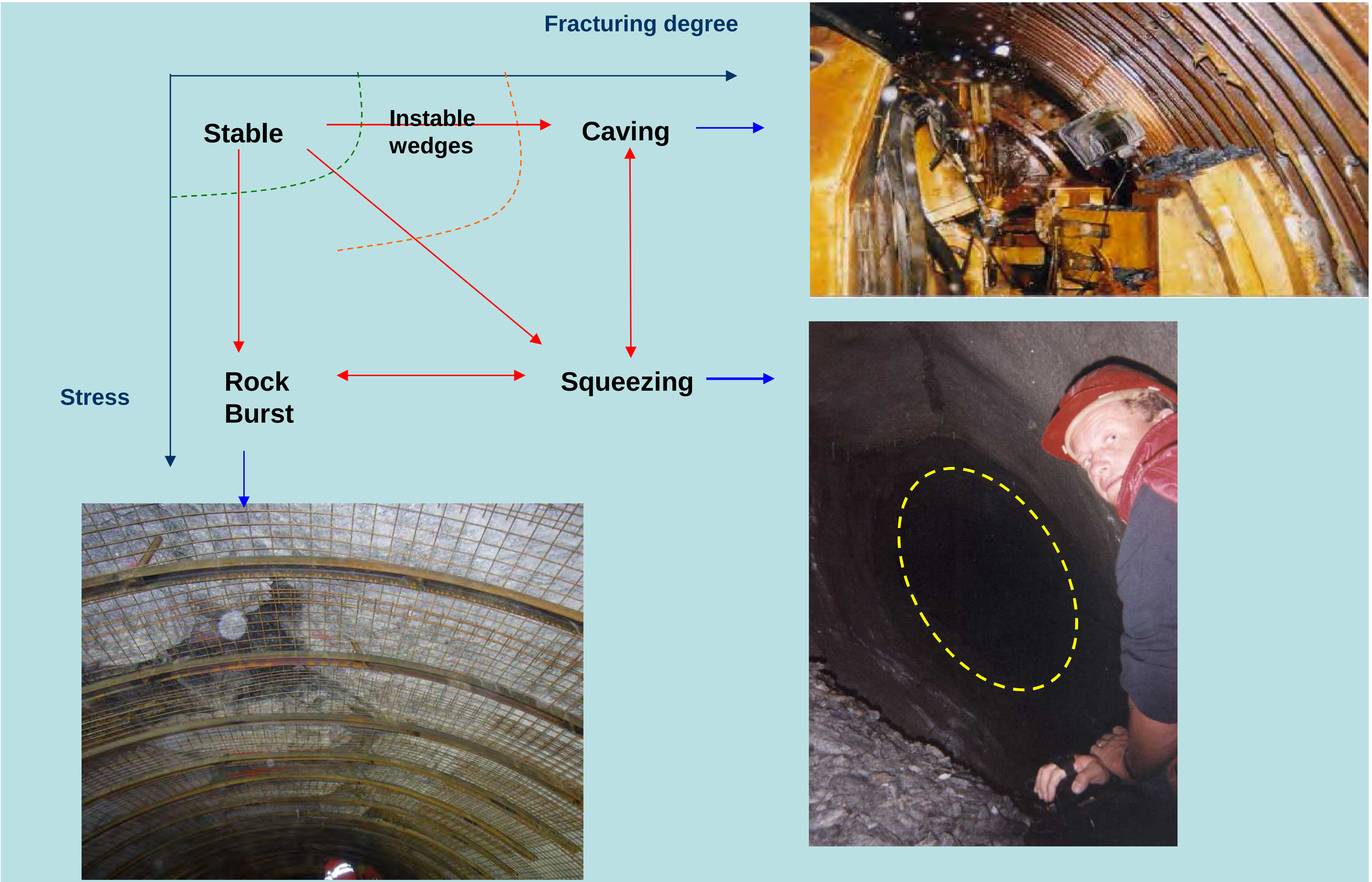
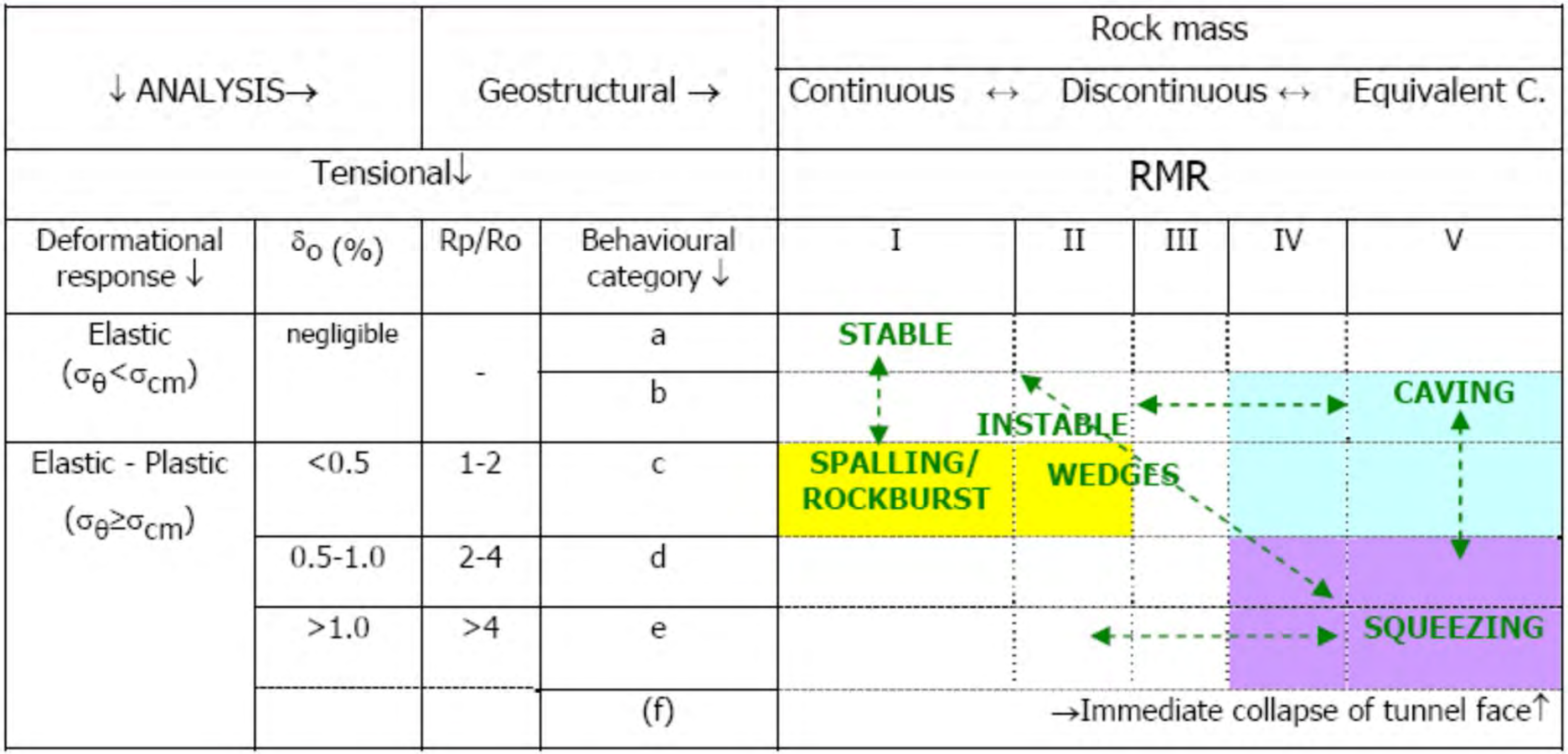


Fig. 63: General setting of behaviour of excavation by Geodata classification: Stress analysis + geo-structural conditions [47]

Fig.65: General scheme for the evaluation of the excavation behaviour [47,48]



Notes: δo=radial deformation at the face; Rp/Ro=plastic radius/radius of the cavity; σθ=max tangential stress; σcm=rock mass strength. The limits of shadow zones are just indicative

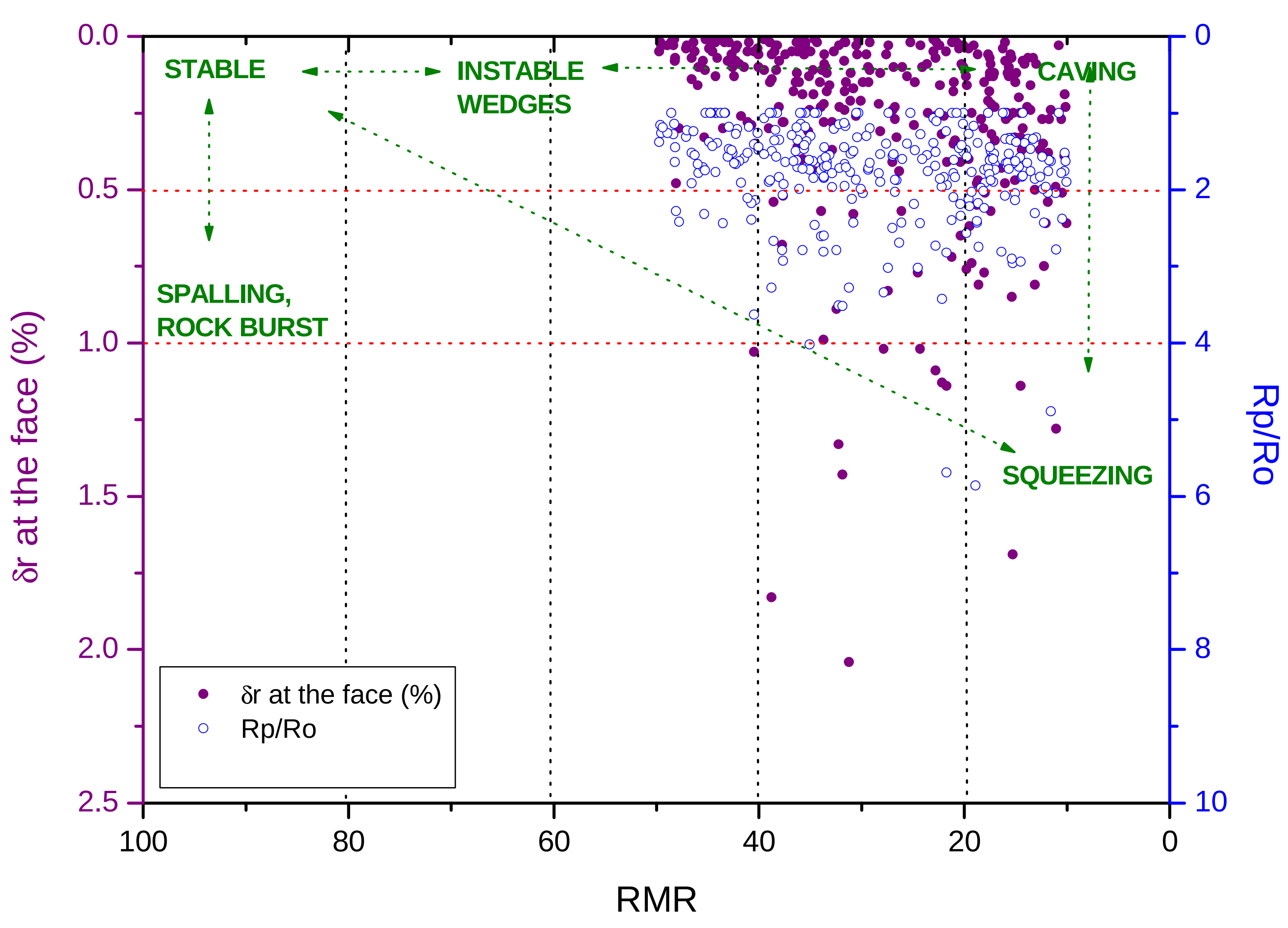
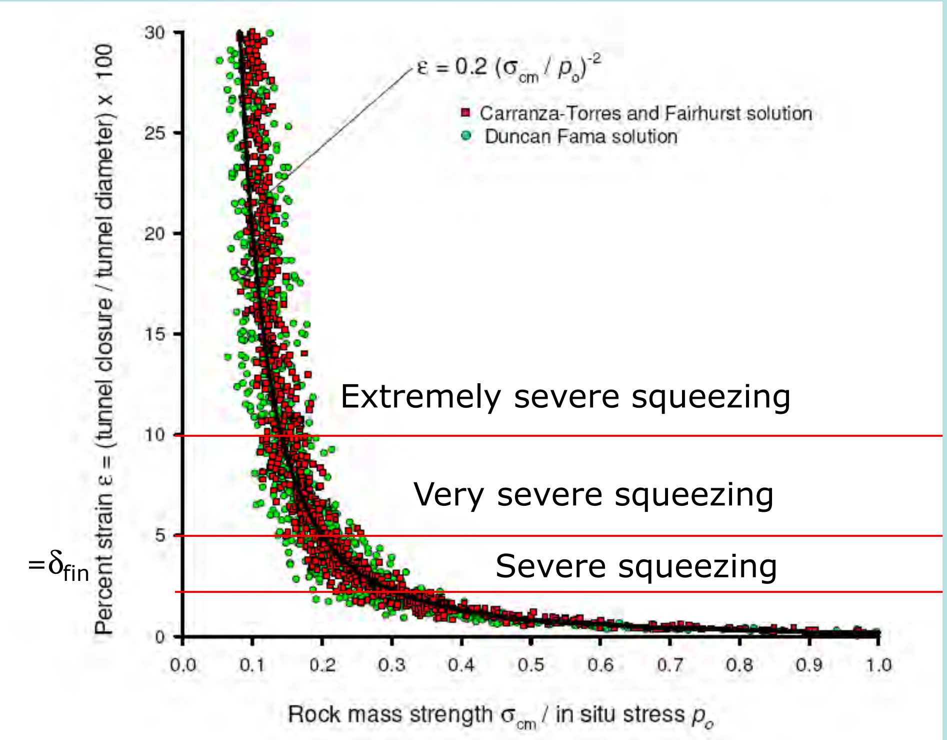


Fig.67: General scheme for the evaluation of the excavation behaviour: example of a probabilistic analysis for a relatively shallow tunnel in prevalent poor rock mass (Italian North Apennines)

Fig.67b: Additional considerations: Squeezing

Plastic deformations/Squeezing



Hoek & Marinos, 2000 [ref.26]

δ_o (%)	Rp/Ro	Behavioural category ↓	
negligible	-	a	
		b	
<0.5	1-2	c	
0.5-1.0	2-4	d	**Severe squeezing**
>1.0	>4	e	**Very severe squeezing**

Extract of GD classification

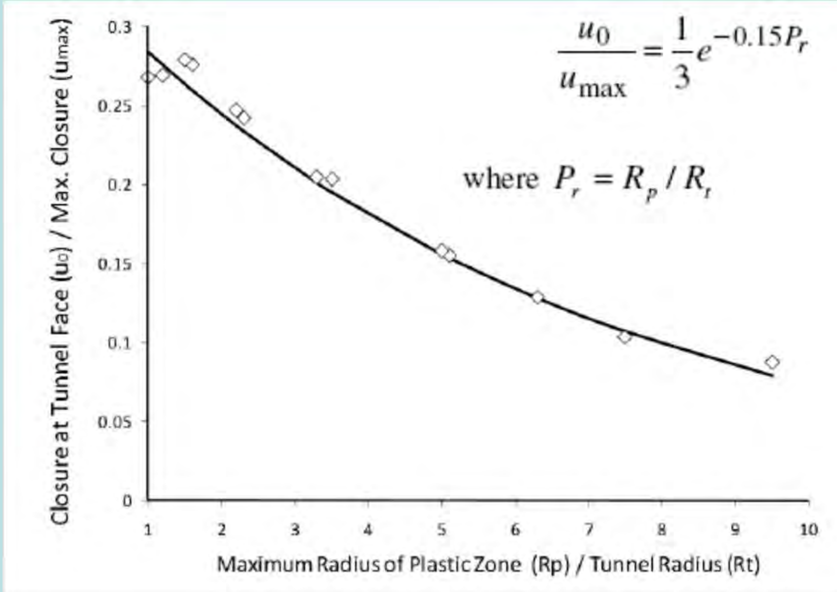
H&M classification based on δ_{final}

GD classification based on δ_o and Rp/Ro

Fig.67c: Additional considerations: Squeezing

Excavation behaviour: Squeezing

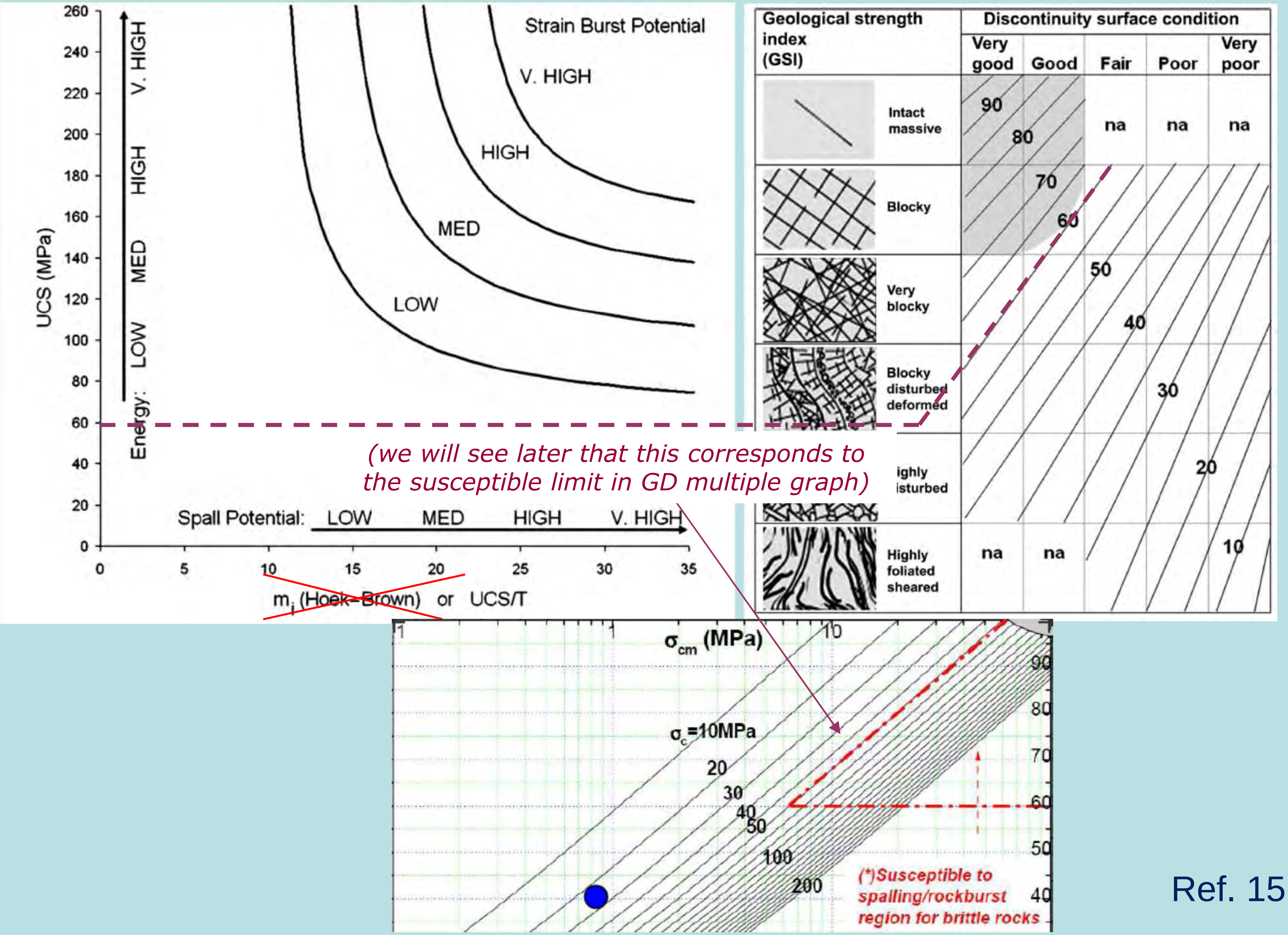
A relationship between δ_o, δ_{final} and Rp/Ro is proposed by Hoek et al. (2008; ref. 27b)



GD classification		Intensity of squeezing	GD classification by Hoek et al. formula	H&M classification
δ_o (%)	Rp/Ro		δ_{final} (%)	δ_{final} (%)
negligible	-	....	....	....
<0.5	1-2			
0.5-1.0	2-4	Severe	~2-5	2.5-5
>1.0	>4	Very severe	> ~5	5-10
		Extremely severe		>10

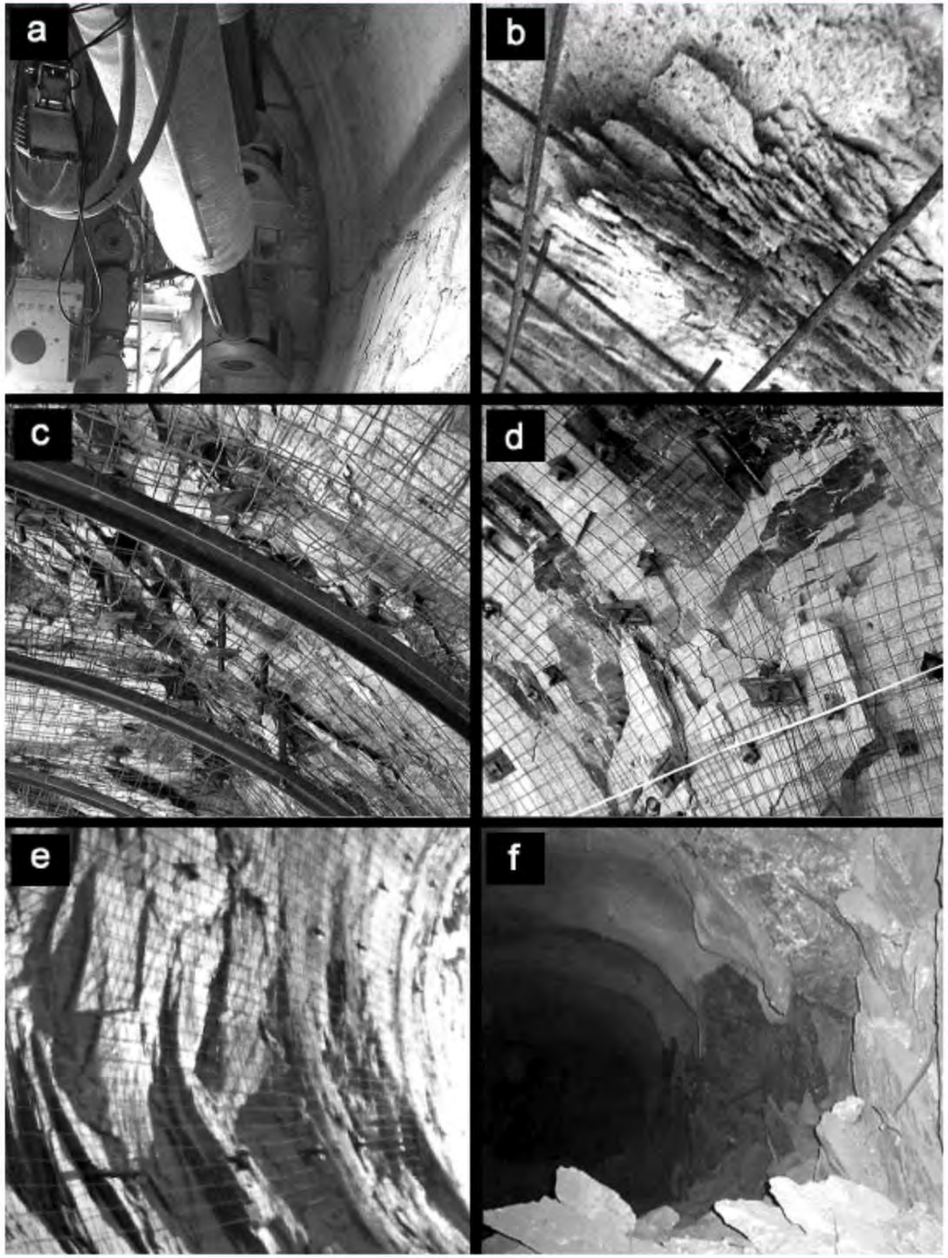
A reasonable agreement is observed..

Fig.67e: Additional considerations: Rockburst

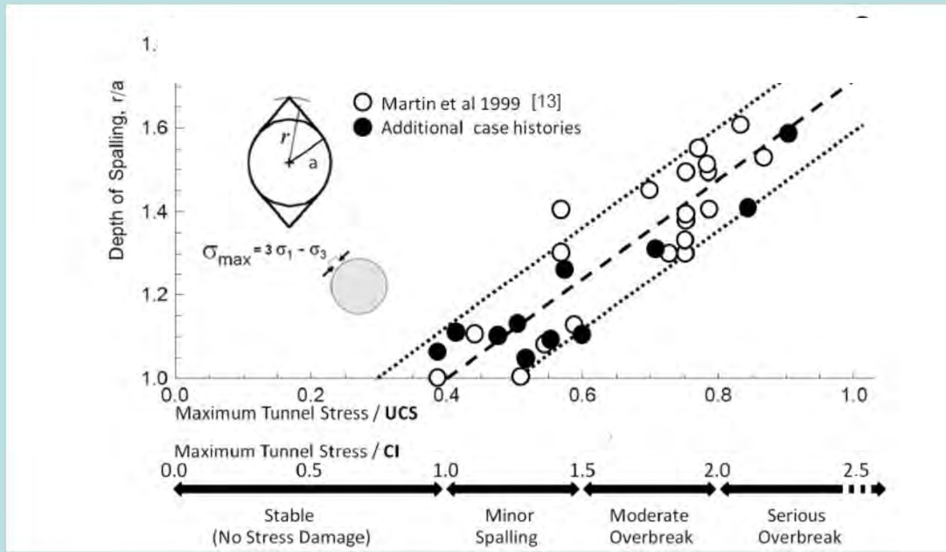


Ref. 15bis

Fig.67f: Additional considerations: Rockburst



It is observed that crack initiation threshold (CI) around a cavity occurs when $\sigma_{max} \approx 0.4\ (\pm 0.1)\ \sigma_c$

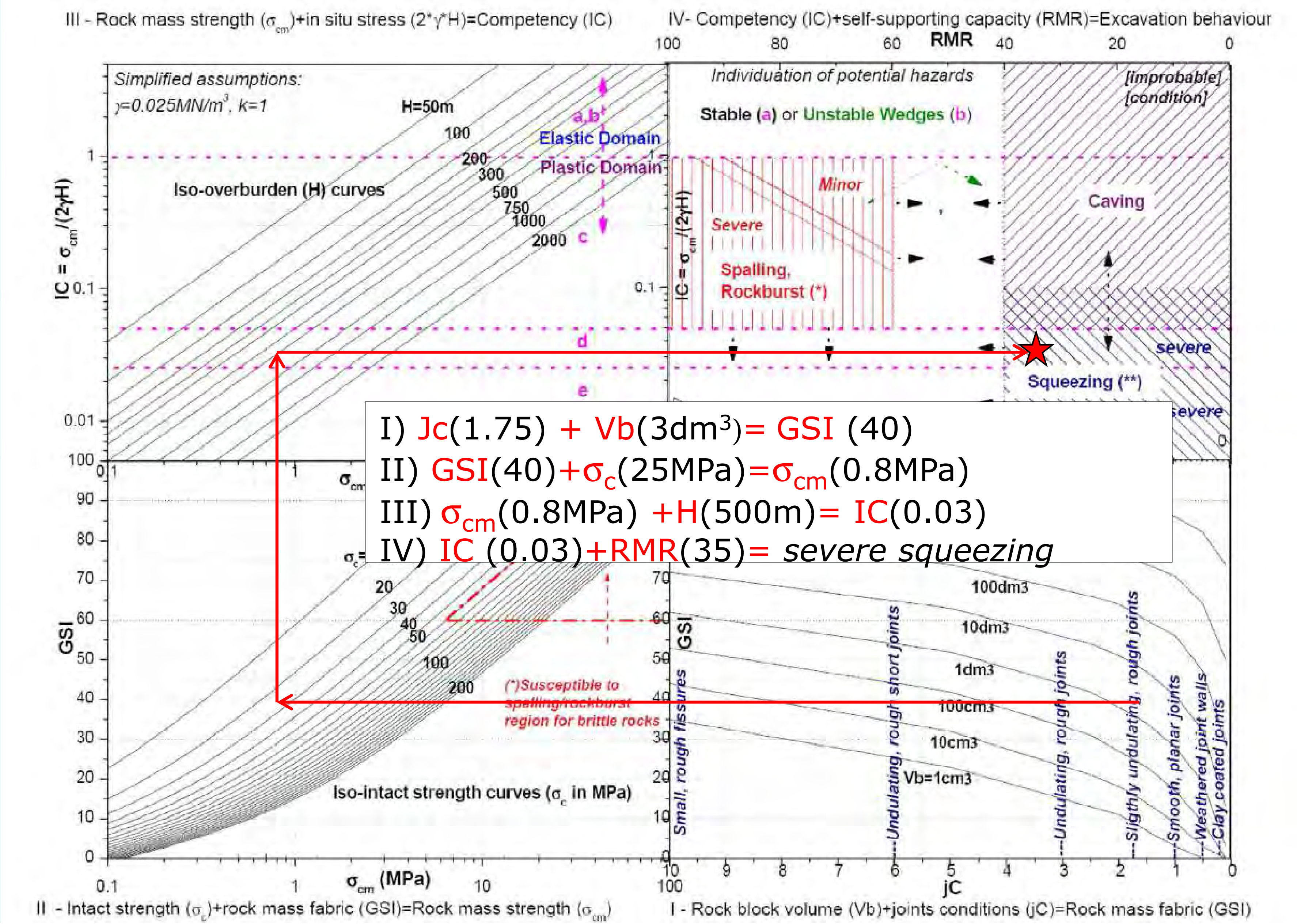


$$\frac{r}{a} = 0.5\left(\frac{\sigma_{max}}{CI} + 1\right) \qquad for\ \ \sigma_{max} > CI$$

a→f=increasing levels of spall damage

[Ref. 15bis, 15ter]

Fig. 68a: Simplified approach for a preliminary setting of excavation behaviour [51



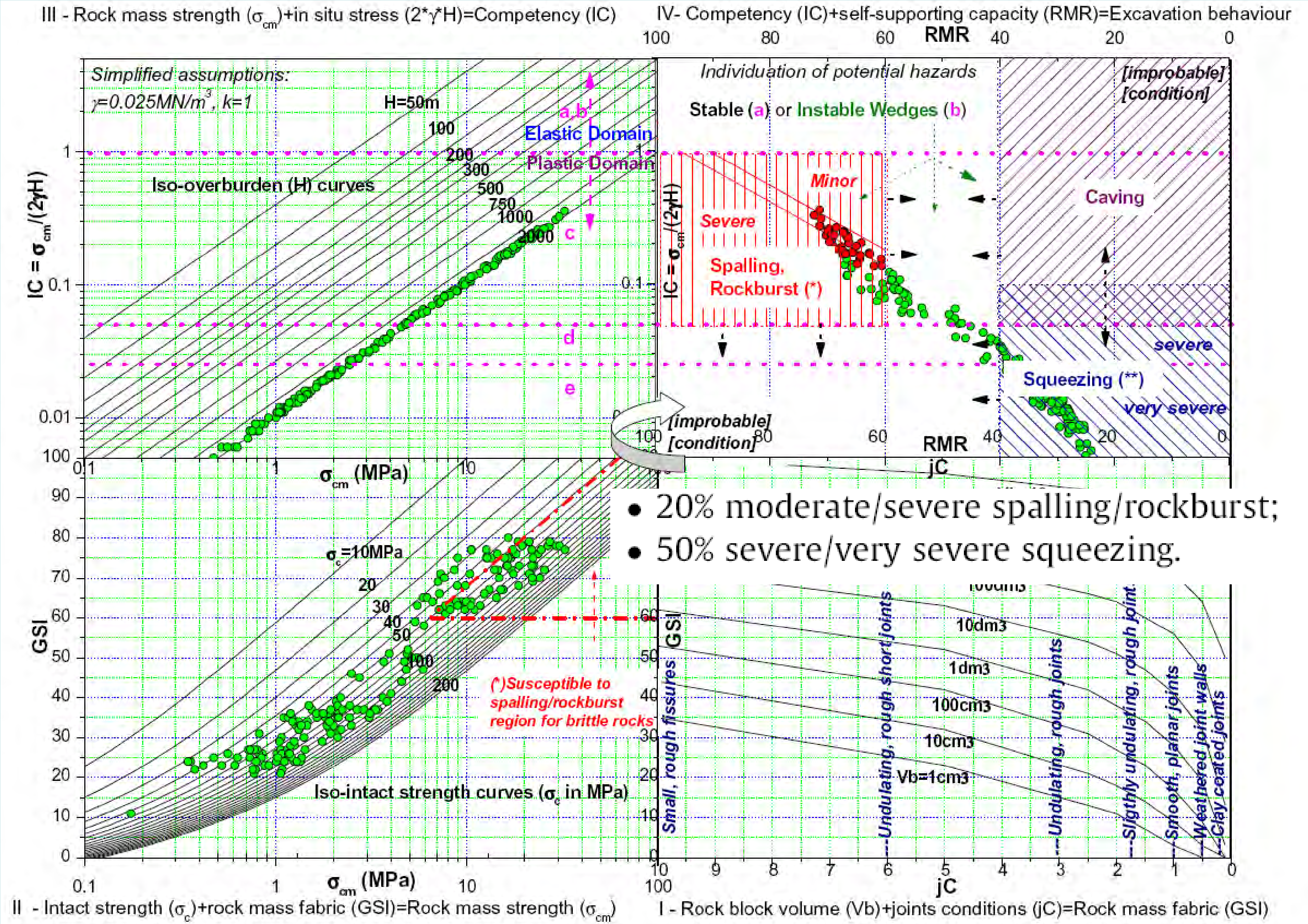
I) Jc(1.75) + Vb(3dm^3)= GSI (40)

II) GSI(40)+σ_c(25MPa)=σ_{cm}(0.8MPa)

III) σ_{cm}(0.8MPa) +H(500m)= IC(0.03)

IV) IC (0.03)+RMR(35)= *severe squeezing*

Fig. 68b: Simplified approach for a preliminary setting of excavation behaviour [51ter]



- 20% moderate/severe spalling/rockburst;
- 50% severe/very severe squeezing.

Fig.68 c [reference 51ter]

Depending on type and intensity of the hazards, mitigation measures are selected and support Section Types composed

IV – Design actions

Code	EXAMPLE OF RISK MITIGATION (STABILIZATION) MEASURES FOR TUNNEL [D&B]
	a) In advancement to the excavation
Ma1	Controlled drainage ahead the tunnel face/contour
Ma2	Pre-confinement/reinforcement of instable rock wedges (inclined bolts, spiling,..)
Ma3	Pre-confinement of excavation contour (reinforced grouting, jet grouting,..)
Ma4	Pre-reinforcement of rock mass contour (by fully connected elements)
Ma5	Pre-support of excavation contour (forepoling, umbrella arch,..)
Ma6	Tunnel face pre-reinforcement (injected fibreglass elements, reinforced grouting, jet gr..)
Ma7	Grouting for water-tightness
Ma8	De-stressing holes/blasting
…..	…..
	b) During the excavation
Mb1	Over-excavation to allow convergences (stress relief)
Mb2	Controlled de-confinement to allow convergences (sliding joints, deformable elements,..)
Mb3	Radial confinement of instable rock wedges
Mb4	Radial rock reinforcement (fully connected elements)
Mb5	Confinement by differently composed system (steel ribs, fbr shotcrete, bolts,..)
Mb6	High energy adsorbing composed system (steel mesh, yielding bolts, fbr shotcrete,..)
Mb7	Tunnel face protection
Mb8	Additional protective measures
…..	…..

Prevalent Hazard (Gravity driven / Stress induced)		GC		Excavation behaviour	ST	Typical mitigation measures
		GDE	RMR			
H1	Wedge instability/ Rockfall	a	I	Stable rock mass, with only possibility of local rock block fall; rock mass of very good quality with elastic response upon excavation	A	Ma1-Mb3
H1	Wedge instability/ Rockfall	b	II	Rock wedge instability; rock mass of good quality with elastic response upon excavation	B	Ma1-Mb3
H1	Wedge instability/ Rockfall	c	III	Pronounced tendency to rockfall; rock mass of fair quality, with possible occurrence of a moderate development of plastic zone	C1	Ma1-Mb5
H2	Spalling/ Rockburst	c	I-II	Mild brittle failure even associated to rock minor rock block ejection; overstressed hard, good rock mass (→Minor spalling/rockburst)		
H2	Spalling/ Rockburst	c	I-II	Sudden brittle failure; overstressed hard, good rock mass (→Moderate spalling/rockburst).	C3	Ma1-Mb6-Mb7
H2	Spalling/ Rockburst	c	I-II	Sudden and violent brittle failure, even associated to rock block ejection; highly overstressed hard, good rock mass (→Severe spalling/ heavy rockburst)	C4	Ma1-(Ma5) (Ma8)-Mb6-Mb7-Mb8
H3	Plastic deformations / Squeezing	d	III-IV-(V)	Development of plastic/viscous deformations; overstressed fair to poor rock mass, resulting in a significative extrusion of tunnel face and radial convergences (→Severe Squeezing)	D	Ma1-Ma5 (Ma6) (Mb4)-Mb5-Mb7
H3	Plastic deformations / Squeezing	e	III-IV-(V)	Intense development of plastic/viscous deformations; overstressed fair to poor rock mass, resulting in a large extrusion of tunnel face and radial convergences (→Very Severe Squeezing)	E	Ma1-Ma4 Ma6-Mb1-Mb2-Mb4-Mb5-Mb7
H4	Caving/ Flowing ground	c	IV	Gravity-driven instability; reduced self-supporting capacity of poor rock mass, generally associated to a moderate development of plastic zone	C2	Ma1 Ma5 (Ma6) Mb5-Mb7
H4	Caving/ Flowing ground	(e)/f	V	Severe gravity-driven instability, with immediate collapse of the tunnel face/excavation contour, including flowing ground; very poor quality, cataclastic rock mass, generally under conditions of high hydrostatic pressure/water inflow (fault zones, etc.)	F/ F_E	Ma1-Ma3 Ma5-Ma6 (Ma7) Mb5/(Mb2)-Mb7-Mb8

Note: GC=Geomechanical Classification; ST=Section Type

Fig. 68d: Simplified approach for a preliminary setting of excavation behaviour [51ter]

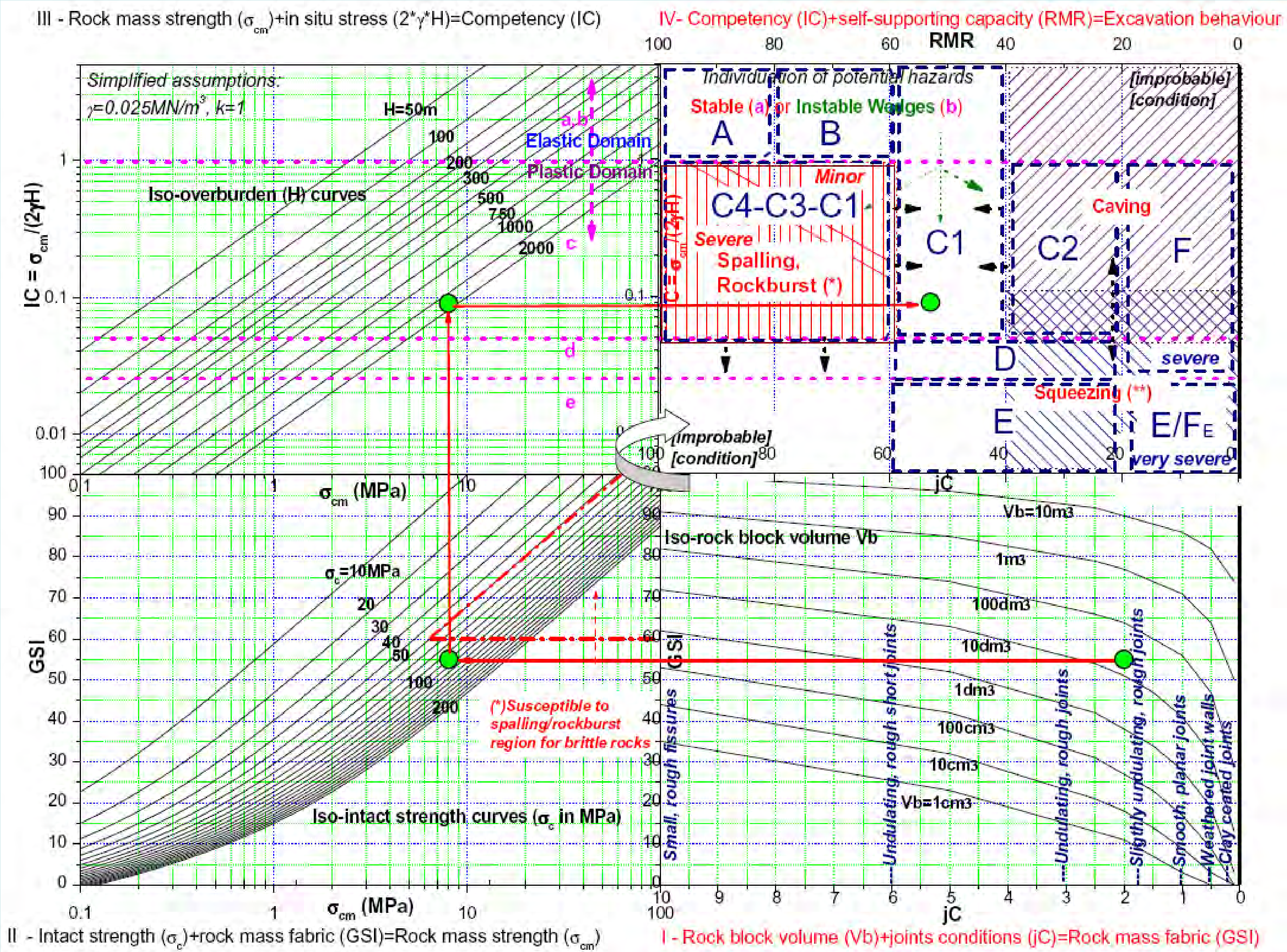
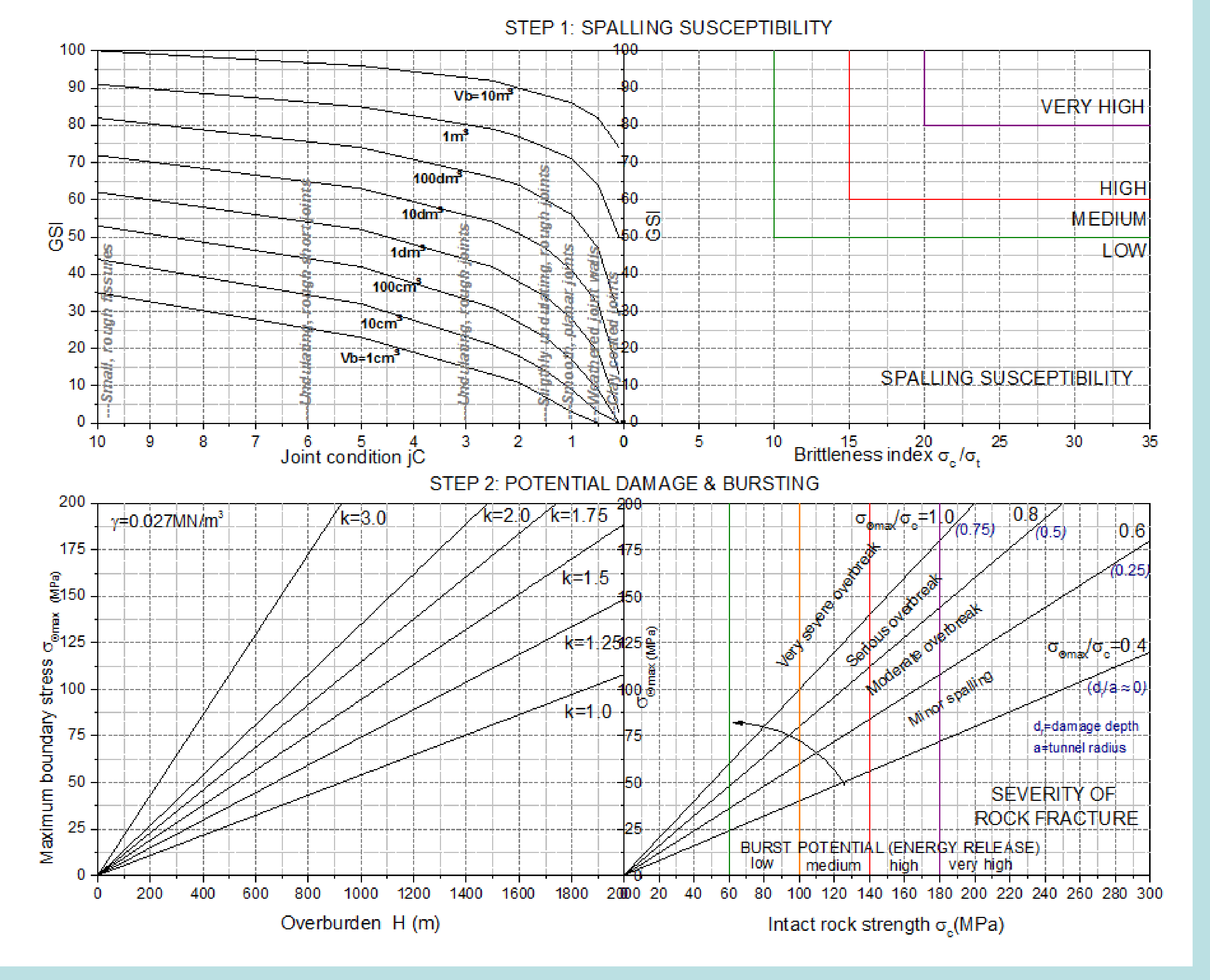


Fig.70bis: A composite graph for brittle failures..



G. Russo (2013) based on Diederichs (2007,2010) and Hoek (2010), modified

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