CHAPTER VI

GEOTECHNICAL ROCK CLASSIFICATION SYSTEMS

The geotechnical rock classification is essential for the tunnel stability study. The classification is also applicable to the other stability of slope, foundation, etc.

A system for the general rock classification proposed by the Canada Advisory Committee on Rock Mechanics appeared in the "Canadian Mannal on Foundation Engineering" (1975) is considered to be the most complete one at present. This system comprises three classifications as follow.

(i) Geological classification systems or classification according to the geological condition, i.e, igneous, sedimentary, and metamorphic origin.

(ii) Intact rock classification systems or classification according to the rock strength. The rock classes are defined in terms of their limited unconfined compressive strength. It should be noted that the limiting values of compressive strength are selected on an arbitrary basis and may have to be changed when dealing with some specific problems.

(iii) Rock mass classification systems or classification according to rock discontinuities.

Many workers have adopted the mentioned classification system into a practical one. The classifications to be used in this study are illustrated in Tables 6.1 to 6.3 and Figures 6.1 to 6.2.

6.1 Geological Classification Systems

The most widely-used geologic classification fo rocks is the three-family grouping of igneous, sedimentary, and metamorphic rocks, based on a blend of the genetic and descriptive factors.

6.1.1 Field Classification of Sedimentary Rocks

In an early study, Krynine (1948) had classified the sedimentary rocks based on the petrographic characteristics, and further into members on the basis of mineral composition of the detrital fractions, i.e. (1) quartz plus chert, (2) feldspar plus kaolin, and (3) micas plus chlorite. These fractions may occour as rock fragments, large, loose, individual flakes or as a micaceous clayey paste (Figure 6.1a). Krynine (1948) proposed a further classification as the normal sedimentary rocks suitable for an observative megascopic and field use as the detrital and chemical rocks. The rocks were classified as the quarizites, graywackes or arkosic series according to the composition of the medium-grained clastic silicate fraction in Table 6.1.

Lundegard and Samuels (1980) proposed a field classification of the fine-grained sedimentary rocks, based on the features of genetic significance, grain size and stratification. They used Folk's (1954) textural categories with the exception of silt-shale, that is almost identical to that proposed by Potter et al. (1980) (Tables 6.2 and 6.3).

6.1.2 Petrographic Classification of Sandstones

Many schemes of classification of sandstones have been drawn up, some based on the theoretical principles, the other emphasized the mineral composition or devised empirically for convenience

Table 6.1 Megascopic classification of normal sedimentary rocks (after Krynine, 1948).

(Most of the concentrates and placers are omitted or left undifferentiated in this table)

	TEXTURES: MAIN TEXTURE	TEXTURES: SUB TEXTURE	BONDING: CHEMICAL CEMENT OR MATRIX		COMPOSITION OF RECOGNIZABLE CLASTIC SILICATE FRACTION: QUARTZ ± CHERT — QUARTZITE SERIES	QUARTZ + ROCK FRAGMENTS + CHERT + MICACEOUS OR CHLORITIC CLAY — GRAYWACKE SERIES: − feldspar (<10%)	+ feldspar (>10%)	QUARTZ + FELDSPAR (>20%) ∓ KAOLINITIC CLAY ∓ impurities (<20%) — ARKOSE SERIES
DETRITAL ROCKS (>50% clastic silicates)	COARSE CLASTIC (average >2.0 mm) Conglomerate and Breccia class	Boulder Conglomerate (>64 mm)	− cement (<10%)	Loose	Quartz or Chert Gravel	Graywacke Gravel (Till— Boulder Clay)		Feldspathic Gravel (Feldspathic Till)
		CONGLOMERATE OR BRECCIA (4-64 mm)		Consolidated A, M-C	QUARTZ OR CHERT CONGLOMERATE	Common	Feldspathic	ARKOSIC CONGLOMERATE OR BRECCIA
		Fine Conglomerate (<4 mm)	+ cement (>10%)	SiO_2		Graywacke Conglomerate or Breccia		
		Sandy, Silty, or Clayey Conglomerate (>20% sand, silt, or clay)		CO_3		Fanglomerate or Tillite		Fanglomerate or Tillite
				Fe,G,P				
	MEDIUM CLASTIC (0.0625–2.0 mm) Sandstone class	Conglomeratic Sandstone (>20% pebbles)	− cement (<10%)	Loose	Quartz Sand	Quartz–Chert and Mica Sand ("Pepper and salt sand")		Feldspathic or Arkosic Sand
		Pebbly Sandstone (>10% pebbles)		Consolidated A, M-C	"Sandstone" rare, usually passes into	gray or red Common (low rank)	black or dark gray Feldspathic (high rank)	gray or red
		SANDSTONE	+ cement (>10%)	SiO_2	QUARTZITE (Gannister)			ARKOSE (<20% impurities)
		Silty Sandstone (>20% silt)		CO_3	IMPURE (CO_3,Fe) QUARTZITE (50–75% SiO_2 in cements); QUARTZITIC SANDSTONE (25–50% SiO_2 in cements)	GRAYWACKE		
		Clayey Sandstone (>20% clay)		Fe,G,P	QUARTZOSE SANDSTONE (<25% SiO_2 in cements)			Sub Arkose (20–40% impurities)
	FINE CLASTIC (average <0.0625 mm) Siltstone — Shale class	Sandy Siltstone (>20% sand)	− cement (<10%)	Loose	(Laterite) ← Silt and Clay → (Loess) (Quartzose, micaceous–chloritic, or feldspathic–kaolinitic)			
		SILTSTONE (gritty) a siltstone proper b micro-conglomerate or micro-breccia		Consolidated	SILTSTONE OR SHALE (Quartzose, micaceous–chloritic, or feldspathic–kaolinitic)			
			+ cement (>10%)	SiO_2	Siliceous, Opaline, or Chalcedonic Shale or Siltstone			
		Silty Shale (semi-gritty)		CO_3	← MARL →			
		SHALE (smooth)		Fe,G,P	Ferruginous, Glauconitic or Phosphatic Siltstone or Shale			
CHEMICAL ROCKS (<50% clastic silicates)	SANDY (5–50% clastic silicates)	CLASTIC coarse: >2.0 mm. medium: 0.0625–2.0 mm. fine: <0.0625 mm		SiO_2	Sandy and Oolitic Cherts	(rare)	Sandy Bedded Volcanic Cherts	Sandy Dolomites
				CO_3	SANDY LIMESTONE AND DOLOMITE (± ← oolitic → ∓)			
				Miscellaneous	SANDY: Gypsum, Anhydrite, Glauconite, Phosphates, Chemical Iron Ores, Salt (rare)			
	PURE (not sandy) (<5% clastic silicates)	I Clastic or arenitic if more than 10% of fragmental particles of any kind (sizes as above) II. CRYSTALLINE coarse: >4.0 mm medium: 1.0–4.0 mm fine: <1.0 mm		SiO_2	CHERT (nodular + bedded)	(rare)	CHERT (bedded)	Dolomite
				CO_3	LIMESTONE AND DOLOMITE (clastic or crystalline)			
				Miscellaneous	SALT, GYPSUM, ANHYDRITE, PHOSPHATES, CHEMICAL IRON ORES, COAL			

SYMBOLS

+ and − signs indicate frequency of occurrence and are quantitatively defined in the text

1 CHEMICAL CEMENTS		2 MATRIX
Fe — Ferruginous	SiO_2 — Siliceous	A — Argillaceous
G — Glauconitic	CO_3 — Calcareous or Dolomitic	M-C — Micaceous or chloritic
P — Phosphatic		

Table 6.2 Field classification of mudrock (after Lundegard and Samules, 1980).

		SILT FRACTION		
		(>2/3)	2/3 – 1/3	(<1/3)
INDURATED	non-laminated	SILTSTONE	MUDSTONE	CLAYSTONE
INDURATED	laminated	SILTSTONE	MUDSHALE	CLAYSHALE

Table 6.3 Classification shale (More than 50 % grains less than 0.062 mm) (after Potter et al., 1980).

Percentage clay-size constituents			0-32	33-65	66-100
Field Adjective			Gritty	Loamy	Fat or Slick
NONINDURATED	Beds	Greater than 10 mm	BEDDED SILT	BEDDED MUD	BEDDED CLAYMUD
NONINDURATED	Laminae	Less than 10 mm	LAMINATED SILT	LAMINATED MUD	LAMINATED CLAYMUD
INDURATED	Beds	Greater than 10 mm	BEDDED SILTSTONE	MUDSTONE	CLAYSTONE
INDURATED	Laminae	Less than 10 mm	LAMINATED SILTSTONE	MUDSHALE	CLAYSHALE
METAMORPHOSED		Degree of metamorphism LOW	QUARTZ ARGILLITE	ARGILLITE	
METAMORPHOSED			QUARTZ SLATE	SLATE	
METAMORPHOSED		HIGH	PHYLLITE AND/OR MICA SCHIST		

in the field and laboratory description. A useful summary of the sandstone classifications was given by McBridge (1963) from which it can be observed that the current tendencies are to subdivide sandstones on the basis of their mineral content, mineral and textural attributes, or together mineral, texture and structure attributes.

Folk's (1954) classification is on the bassis of mineral attributes. It uses three end-member constituents which are the important composition in sandstones. The three end-member constituents are mica and metamorphic-rock fragments, feldspar and igneous-rocks fragments, and quartz and chert (Figure 6.1b).

Instead of using Folk's (1954) feldspar and igneous rock, Gilbert (1955) used feldspar as one of three end-member constitutes to classify the sandstones. Gilbert's classification is given in Figure 6.1c.

Pettijohn (1954) classified the sandstones according to the textural-mineralogical attributes. He took an account on the provenance, maturity and depositional current factors. The provenance is indicated by the relative proportion of sand-size feldspar and the rock fragments present; maturity by the proportion of quartz and chert to that of feldspar and other rock fragments; and depositional current activity by the relative amount of sand present to the amount of interstitial matrix (Figure 6.1d).

McBride (1963) modified Folk's (1954) scheme by rearranging the three end-members as (1) quartz plus chert and quartzite, (2) feldspar, and (3) fine-grained rock fragments exclusive of those under the particular heading above (Figure 6.1e).

Dott (1964) modified Pattijohn's (1954) scheme and defined several classes in terms of proportion of detrital quartz, feldspar, and rock particles and on the presence or absence of an interstitial matrix (Figure 6.2a). Those with 15 or more percent matrix constitute the "wackes" ; those with less matrix are the "ordinary" (ortho) sandstones. In Dott's class, three main families are defind : (1) those in which quartz forms 95 or more percent of the framework fraction, the quartz arenites (orthoquartzites); (2) those containing 25 or more percent feldspar which exceeds rock particles, the arkoses; and (3) those characterized by over 25 percent of rock particles, the lithic sandstones (lithic arenites). The subclasses are also defined between the major families. These are, for example, subarkoses, and sublithic sandstones or arenites. The lithic arenites class may by itself subdivided on the basis of the type of the rock fragments presented, as suggested in Figure 6.2A. The dominant rocks in the group are the graywackes, of which there are two further important subdivisions, the lithic graywackes in which rock particles exceed feldspars, and feldspatic graywackes if vice versa. The quartzwackes are a relatively minor and rare class within the wackes group.

Selly (1982) used quartz, feldspar and clay as the end members as shown in Figure 6.1f. His classification of sandstones is based on the use of clay as an indicator of textural maturity, and feldspar as an indicator of chemical maturity.

Many other sandstone classifications have been proposed that amplify, amond, extend or modify the earlier classifications. the proposals are shown in Table 6.4.

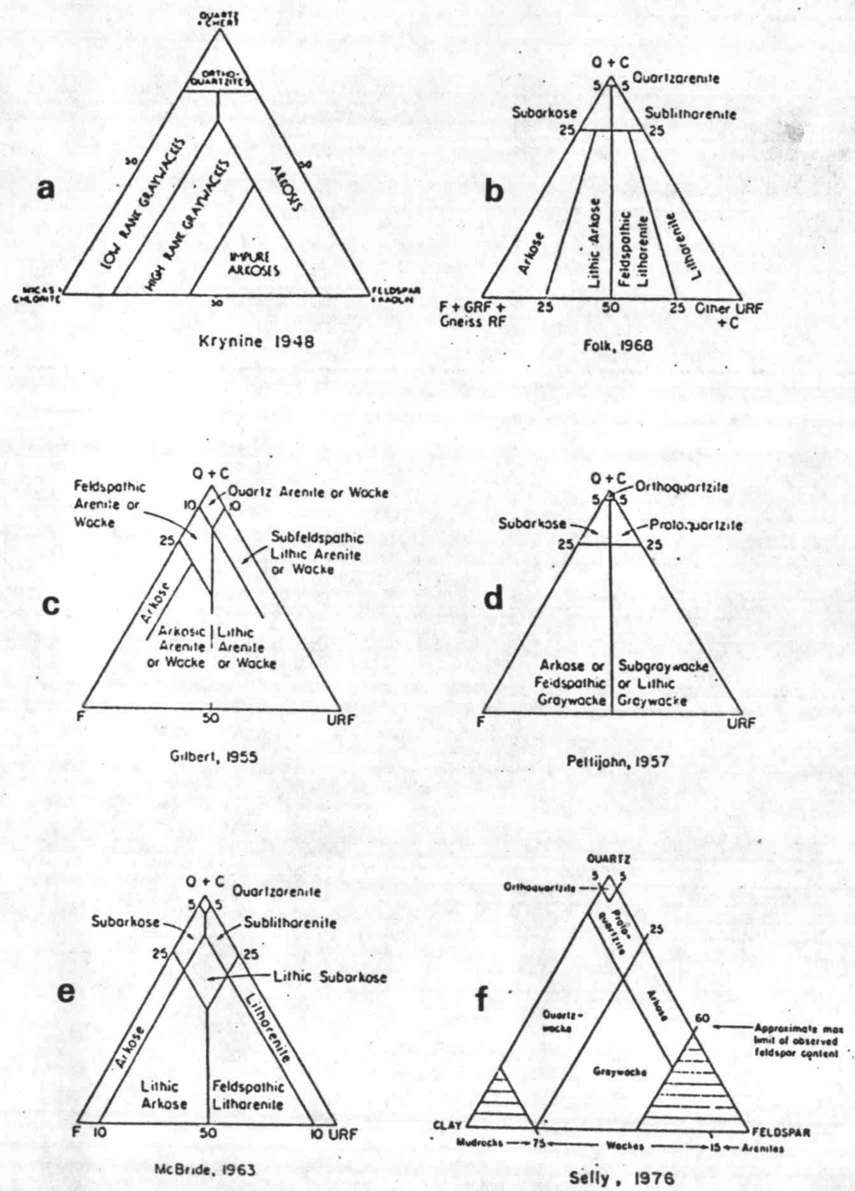


Figure 6.1 Some triangles in common use for the classification of sandstones.

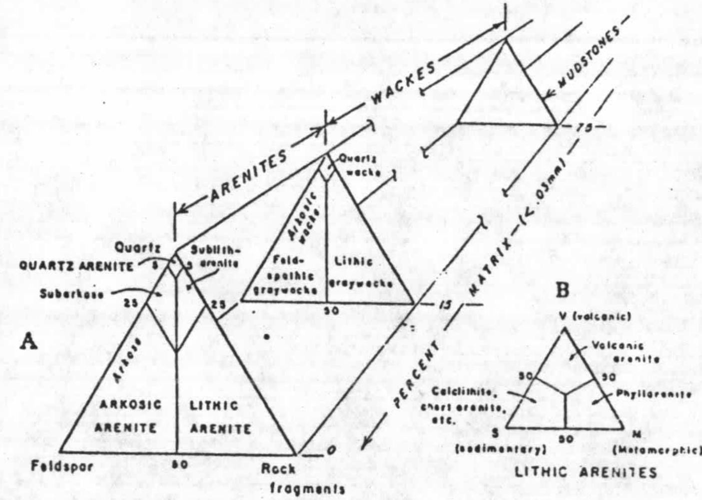


Figure 6.2 A : Classification of terrigeneous sandstones (modified from Dott, 1964); B : Subdivision of lithic arenites (after Folk, 1968).

Table 6.4 Summary of classifications of terrigenous sandstone (modified from McBride, 1963 and Pettijohn et al. 1972).

Reference	Basis of classification	End-members of classification			Comments
Fischer (1933)	Mineralogy	Quartz	Feldspar	Rock fragments	First use of triangular diagram for sandstone composition?
Krynine (1948)		Quartz	Feldspar and kaolin	Micas and chlorite	Ignores rock fragments, Graywacke based solely on mica and chlorite.
Folk (1954)		Quartz and chert	Feldspar and volcanic rock fragments	Metamorphic rock fragments, micas, metamorphic quartz	Graywacke based solely on metamorphic constituents. Sedimentary rock fragments ignored.
van Andel (1958)		Quartz	Feldspar	Rock fragments and chert	Graywacke based solely on rock fragments and chert.
Füchtbauer (1959)		Quartz	Feldspar	Rock fragments and chert	Recognizes clay-rich and clay-poor sandstone types.
Hubert (1960)		Quartz, chert and metaquartzite	Feldspar and feldspathic crystalline rock fragments	Micas and micaceous rock fragments	Non-micaceous rock fragments are not treated as a major constituent. Classification designed originally for feldspathic rocks.
Fujii (1962)		Quartz plus chert	Feldspar	Rock fragments	Divides triangle into five fields.
McBride (1963)		Quartz, chert and quartzite	Feldspar	Rock fragments	Ignores large micas. Has 8 classes.
Shutov (1967)		Quartz	Feldspar	Rock fragments	Divides triangle into 12 fields forming three major groups; graywacke based solely on rock fragment content.
Teodorovich (1967)		Quartz	Feldspar and mica/chlorite	Rock fragments	Eighteen subdivisions of triangle; addition of pyroclastic material requires tetrahedral representation and 11 additional subclasses.
Tallman (1949)	Texture and Mineralogy	Quartz	Feldspar		Graywacke based solely on matrix content. Rock fragments ignored.
Dapples, Krumbein, and Sloss (1953)		Quartz and chert	K and Na feldspar	Rock fragments and matrix	Graywacke based on sum of rock fragments and matrix. Ca feldspar ignored.
Williams and others (1954), Dott (1964)		Quartz, chert, quartzite	Feldspar	Unstable fine-grained rock fragments	Recognizes two suites on basis of > or < 10% matrix. Graywacke used as special rock type and not part of classification.
Bokman (1955)		Quartz	Feldspar and rock fragments	Clay	Graywacke based solely on clay content. Feldspar and rock fragments not differentiated.
Pettijohn (1957)		Quartz and chert	Feldspar	Rock fragments	Clay matrix is most important property of graywacke.
Sahu (1965)		Stable grains (Quartz, chert, quartzite, plus tourmaline, etc.)	Unstable grains (Feldspar, rock fragments, and micas)	Matrix	Eight clans of sandstone.
Krumbein and Sloss (1966)		Quartz	Feldspar	Clay, sericite, and chlorite	Ignores rock fragments. Graywacke based on clay, sericite, chlorite, and feldspar content.
Boggs (1967)		Siliceous resistates	Feldspar	Labile grains	Ten principal and 10 subclasses.
Packham (1954)	Structure, Texture, and Mineralogy	Quartz and chert	Unstable minerals and rock fragments	Matrix	Recognizes two suites. Graywacke based on deposition by turbidity current.
Crook (1960)		Quartz and chert	Unstable minerals and rock fragments	Matrix	Recognizes three suites. Graywacke based on deposition by turbidity current.

Others papers dealing with sandstone classification, not represented in this table, include those of Michot (1958), Kossovskaya (1962), Shutove (1965), Chab (1967), Chang (1967), Wang (1967), Chen (1968), Konta (1969), and Travis (1970).

In the present study, the petrographic determination of the test specimens and samples includes the microscopic measurements of the average grain size, average fracture opening, average fracture intensity and Modal analysis of mineral constituents and observations of textural characteristics. The study results using the classification of Pettijohn (1957) and Potter et al. (1980) are shown as Table 4.2.

6.1.3 Quantitative Classification of the State of Alteration

No single index derived from the simple field observations or laboratory tests can be expected to apply appropriately to all materials in the vast range of weathering products derivable from intermediate stages of decomposition of rocks. Several approaches useful in the particular rock types are offered as the examples to be emulated in principle or in detail as the case warrants.

Ege (1968) used four classification indices, expressed by estimating the percent of altered minerals in the rock, without reference to any unweathered standard. The rock is classed as unweathered, sligthly weathered, moderately weathered or severely weathered as the amount of the altered minerals is in the respective ranges 0-10%, 10-25%, 25-75%, and 75-100%. Deere and Patton (1971) reviewed the classification of the weathered rock mass of different rock types and suggested a standard terminology based on the approach used by Moye (1955), Ruxton and Berry (1957) for granites. These suggestions, as well as the works of Little (1969), Fookes and Horswill (1970), Dearman (1976), and others are listed in the Table 6.5.

The rock masses along the diversion tunnel, portals and other study area are classified for the weathering grade following to Bieni-

Table 6.5 Comparison of some engineering rock weathering classification.

MOYE (1955)			RUXTON & BERRY (1957)			
Grade	Degree of Deecomposition	Diagnostic features	Zone	Character	% age of solid rock	Grade
VI	True or mature residual soil	No orginal fabric. Humus and plant roots présent in surface layers.	I	Residual debris structureless sandy clay or clayey sand, 1 m to 25 m thick; up to 30% clay, dominantly quarts and kaolin, reddish brown when very clayey; light brown or orange when less clayey.	Usually Zero	VI
V	Completely decomposed	The texture still recognisable. Ingranitic rocks,original felespars completely decomposed to clay minerals. Disintigrates in water. Cannot be recovered as cores by ordinary drilling methods,	IIa	Residual debris with core stones which are subordinate, rounded and free;equal amounts of gruss and debris; less than 5% clay but planty of clay-forming minerals- sericite and kaolin; light colour; less than 10% core stones;deposit up to 60 m thick	less than 10	V
			IIb	As IIa but 10% to 50% core stones		
IV	Highly decomposed	Fairly large pieces can be broken and crumble in the hands. Hammer blow penetrates surface. Cores can be obtained only by careful diamond drilling.	III	Core stones with residul debris; core stones dominant, rectangular and locked together; most comminuted material is gruss; deposit 7 m to 17 m thick	50 to 90	IV
III	Moderately decomposed	Considerably weathered throught. large picees (e.g.NX drill core) cannot be broken by hand. Hammer blow-drummy sound.	IV	Partially weathered rock; minor residual debris along major structuralplanes but more than 50% may be iron stained indicating significant chemical decomposition and breakdown of biotite; 3 to 30 m thick	greater than 90	III
II	Slightly decomposed	Distinctly weathered throughout the rock fabric with slight Limonite staining. Strength approaching that of fresh rock. Hammer blow-dull note.	Bedrock:	Fresh unweathered granite; medium grained, light grey; two sets of vertical joints spaced 0.5 m to 12 m	c. 100	II
I	Fresh rock	Some limonite stained joints may present. Hammer blow-rings.				I

Table 6.5 (cont.)

LITTLE (1969)		
Degree of Decomposition	Field recognition	Engineering properties
Soil	Surface layer contains humus and plant roots no recognisable rock texture; unstable on slopes when vegetable cover destroyed.	Unsuitable for important foundations; unstable on slopes when cover is destroyed.
completely weathered	rock completely decomposed by weathering in place but texture still recognisable; in type of granite origin feldspars completely decomposed to clay minerals; cores cannot be recovered by ordinary rotary drilling methods; can be excavated by hand.	can be excavated by hand or riping without use of explosives. Unsuitable for foundations of concrete dams or large structures; may be suitable for foundations of earth dams and for fills; unstable in high cuttings at steep angles; requires erosion protection.
highly weathered	rock so weakened by weathering that fairly large pieces can be broken and crumbled in the hands, sometimes recovered as core by careful rotary drilling; stained by limonite	similar to grade V; unlikely to be suitable for foundations of concrete dams; erratic presence of boulders makes it an unreliable foundation stratum for large structure.
moderately weathered	considerably weathered; possessing some strength. Large pieces cannot be broken by hand; often limonite stained; difficult to excavate without use of explosives.	excavated with difficulty without use of explosives; mostly crushes under bulldozer tracks; suitable for foundations of concrete structures and rockfill dams; may be suitable for semi-pervious fill; stability in cuttings depends on structural features, specially joint attitudes.
slightly weathered	distinctly weathered with slight limonite staining, some decomposed feldspar in granites, strength approaching that of fresh rock; explosives required for excavation	requires explosives for excavation; suitable for concrete dam foundations; high permeability through open joints; often more permeable than the zones above or below; questionable as concrete aggregate.
fresh rock	fresh rock may have some limonite stained joints immediately beneath weathered rock	staining indicates water percolation along joints; individual pieces may be loosened by blasting or stress relief and support may be required in tunnals and shafts

Table 6.5 (comt.)

CHANDLER (1969)		VARGAS et al (1965), DEERE & PATTON (1971)		
Zone	Description	Zone	Description	Engineering properties
fully weathered IV a	matrix only; distinguisable from solifluction or drift by absence of pebbles; plastic slightly silty clay; may be fissured	I	Upper zone near tne surface; mature residual soil or/and colluvial soil. Difficult to distinguish one from the other. Consist of red brown or yellow porous clay or sandy clay soil.	Medium low k, low streng.h very compressible.
IV b	matrix with occational clay stone pellets less than 1/8 inch diameter but more usually coarse sand size; little or no trace of zone I structure; permiability less than underlying layers.	II	Intermediate zone; red brown or yellow stiff or hard clay or clayey sand; meadum to compact. consist of limonite conections or hard layers formed by the precipitation of coloidal material from the upper layer.	Medium k, low to medium strength RQD=0
partly weathered III	matrix with frequent lithcrelicts upto 1 inch; as weathering progress,lithorelicts become less angular; water content of matrix greater than that of lithorelicts.	III	Residual soil; Gravelly, sandy or clayey with relict structure. Eventually with boulders or layers of decomposed rock.	High k RQD=0-50%
II	angular blocks of marl; first indications of chemical weathering; matrix starting to encroach along joints leading to spheroidal weathering.	IV	Fresh weathered zone composed of weathered rock, boulders or layers of intact rock mixed together with clayey or sandy soils.	Meadium to high k; RQD=50%-75%
un-weathered I	mudstone (often fissured); water content varies due to different lithology.	V	Sound rock substratum which in most cases fissured to a certain depth.	Very high strength RQD>75%

Table 6.5 (cont.)

FOOKES & HORSWILL (1969)					DEARMAN (1976)		
Term	Grade	Abbreviation	Soils (i.e. soft rocks)	Rocks (i.e. hard rocks)	Degree of composition	Grade	Nature of weathered product
True residual soil	VI	Rw	the material is completely changed to a soil of new structure and composition in harmony with existing ground surface conditions	the rock is discoloured and is completely changed to a soil with the original fabric completely destroyed	Residual soil	VI	All rock materials converted to soil. The mass structure and material fabric are destroyed. There is large change in volume but the soil has not been significantly transported.
Completely weathered	V	Cw	the material is altered with no trace of original structure	the rock is discoloured and is externally changed to a soil, but the original fabric is mainly preserved; the properties of the soil depend in part on the nature of the parent rock	Completely weathered	V	All rock materials is decomposed and/or disintegrated to soil. The original mass structure and material fabric are still largely intact
Highly weathered	IV	Hw	the material is mainly altered with occasional small lithorelicts of original soil; little or no trace of original structure	the rock is discoloured; discontinuities may be open and the fabric of the rock near to the discontinuities is altered; alteration penetrates deeply inwards, but lithorelicts are still present	Highly weathered	IV	More thanhalf of the rock material is decomposed and or disintegrated into a soil. Freshor discoloured rock is present either as a discontinuous framwork or as core stone
Moderately weathered	III	Mw	the material is composed of large discoloured lithorelicts of original soil separated by altered material	the rock is discoloured discontinuities may be open and surfaces will have greater discolouration with the alteration penetrating inwards; the intact rock is noticeably weaker as determined in the field than the fresh rock	Moderately weathered	III	Less tha half of rock material is decomposed and or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as a core stone
Slightly weathered	II	Sw	the material is composed of angular blocks of fresh soil, which may or may not be discoloured; some altered starting to penetrate inwards from discontinuities separating blocks	the rock may be slightly discoloured; discontinuities may be open and have slightly discoloured surfaces; the intact rock is not, as determined in the field, weaker than the fresh rock	Slightly weathered	II	Discolouration indicater weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.
Fresh	I	Fr	the parent soil shows no discolouration, loss of a strength or any other effects due to weathering	the parent rock shows no discolouration, loss of strength or any other effects due to weathering	Fresh	I	No visible sign of rock material weathering. Perhaps slight discolouration on major discontinuity.

Table 6.5 (cont.)

Bieniawski (1973), ISRM (1981)			Engineering Group Work Party (1977)		
Term	Description	Grade	Term	Description	Grade
Fresh	No visible sign of rock material weathering: perhaps slight discolouration on major discontinuity surfaces.	I	Fresh	No visible sign of rock material weathering.	IA
			Faintly weathered	Discoloration on major discontinuity surfaces.	IB
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition.	II	Slightify weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker than in its fresh condition.	II
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III	Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV	Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is resent either as a discontinuous framework or as corestones	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V	Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destoyed. There is a large change in volume, but the soil has not been significantly transported.	VI	Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

awski (1973) and ISRM (1981) classifications which described in Table 6.5. The pebbly graywackes and subarkoses in the diversion tunnel and its portal are generally slightly to moderately weathered with occasional highly to completely weathered in the upper-portal slone faces closed to ground surface and in some excavation open cut area.

6.2 Engineering Classification Systems

The engineering classifications of rocks are directed toward the potential engineering use of the rocks or of underground excavation in the rocks. The compilation of the data and the application of classification criteria must take into account the way in which each datum point has been obtained from the intact rock material or the rock mass, or both (Table 6.7)

Franklin (1970) and Bieniawski (1974) considered the following prerequirities to be of an importance for any rock classification system employed on a routine basis.

(1) It should be based on the measurable parameters which can determined by the relevant, quick and cheap field tests.
(2) It should involve only the rapid testing techniques which are able to cope routinely with a large number of samples.
(3) The testing techniques should be simple enough to be carried out even by the semi-skilled field laboratory staff.
(4) The range of test result values should allow a sufficient power of discrimination when applied to the various test samples.

Some classification systems are listed below.

Table 6.6 Some engineering classification systems for rock.

Objective	For general purpose	For a special purpose
Intact Rock	Coates (1964) Deere and Miller (1966) Underwood (1967) Duncan (1969) Deere and Gamble (1971) Morgenstern and Eigenbrod (1974) Wood and Dev (1975) Fourmaintraux (1976)	Bergh-Christensen and Selmer-Olsen (1970) - resistence to blasting Selmer-Olsen and Blindheim (1970) - drillability Olivier (1979) - stability of tunnel
Rock Mass	John (1962) Deere (1964) Hansagi (1965) Onodera (1970) Iida et al. (1970) Muller and Hoffman (1970) Franklin et al. (1971) Sjogren et al. (1979)	Terzaghi (1946)- tunnels Stini (1950) - tunnels Lauffer (1958)- tunnels Carterpillar Tractor Co. (1966) - rippability Obert and Duvall (1967) - mining Ege (1968) - tunnels Kruse et al. (1970) - tunnel Wickham et al. (1972) - tunnels Bieniawski (1973)- tunnels Barton et al. (1974) - tunnels

6.2.1 Intact Rock Classification Systems

A systematic approach to the intact rock substance classification could invovle the characterization on the properties of rock that are routinely measured, usually in the laboratory.

6.2.1.1 Coates's Rock Mechanics Classification

Coates (1964) proposed a comprehensive classification with five characteristics, the first three to be applied to the intact rock substance, and the other two to the rock mass. The five parameters are the uniaxial compressive strength, pre-failure deformation, failure characteristics, gross homogeneity, and continuity of rock material in the rock mass. Burton (1965) extended Coates's criteria for the rock mass from "massive" and "layered" to "homogeneous", "heterogeneous" and "heterogeneous-welded" and classified the continuity of rock material in the rock mass to become more specifically as "intact", "tabular", "columnar", "blocky", "fissured or seamy" and "crushed".

Coates and Parsons (1966) subsequently modified the previous proposal to include the geological name of the rock and to alter the boundary between weak and strong rocks to a uniaxial compressive strength of 10,000 psi (70 MPa). In addition, instead of trying to distinguish between the elastic and viscous substances and plastic and brittle failure based on the deformation characteristics, the substance was simply classified either as elastic or yielding. The term yielding means a certain minimum time-dependent strain rate or a certain proportion of total strain being permanent.

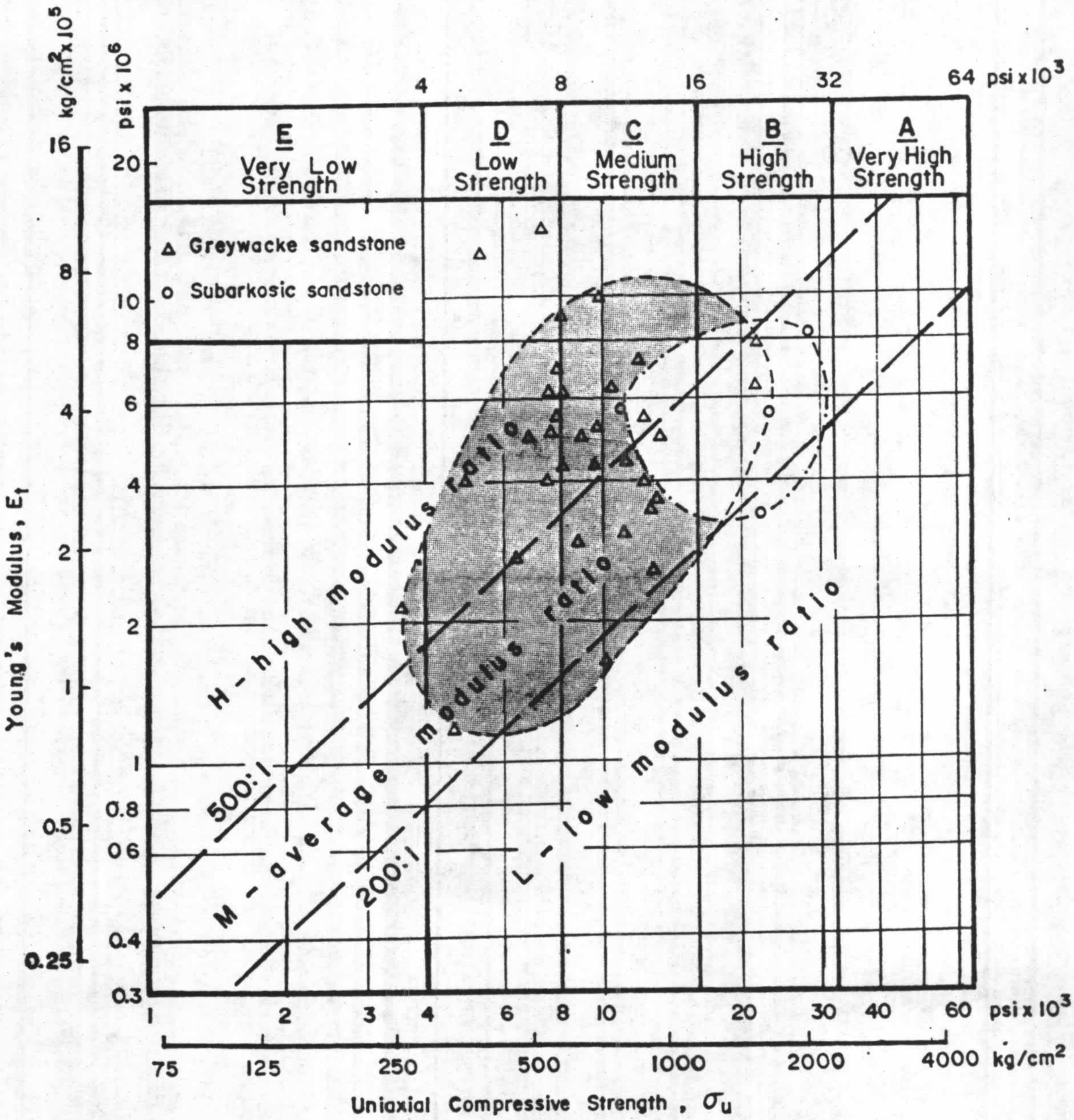
Stapledon (1968) also discussed Coates's clsddification at the "strong" level. He then added the "medium-strong", "weak", "very weak" classes, roughly corresponding to the range of compressive strength.

6.2.1.2 Modulus-Strength Classification of Deere and Miller

Deere and Miller (1966) had classified the intact rocks based jointly on the uniaxial compressive strength and elastic modulus. They also presented the data showing the relationship between the strength and elastic modulus. In practice, the modulus of elasticity and the uniaxial compressive strength of the rocks are plotted on the log-log grid and their properties are classified according to the position of the point plotted (Figure 6.3). The complete classification should also include the lithologic description.

Following the classification system of Deere and Miller's (1966) mentioned above, the test of the pebbly graywacke specimens can be classified as medium strength, wilh an average to high modulus ratio (CH to CM), medium-grained, dense, homogenous and the subarkosic sandstones as high strength, with an average modulus ratio (BM), medium-grained, very dense, and homogeneous.

The major limitation of the Deere-Miller system as pointed out by Olivier (1978) is its inability to differentiate the rocks of a variable durability as the swelling properties were not taken into consideration.



Note: (1) E_t = tangent modulus at 50 % ultimate strength

(2) Classify rock as AM, BH, BL, etc

Figure 6.3 Engineering classification for intact rock samples from Chiew Larn damsite (37 specimens, 95 % of points) after the classification system of Deere and Miller's (1966).

A general comparison of the various classes with numerical values of uniaxial compressive strength is given in Table 6.8.

6.2.1.3 Durability-Plasticity Classification by Deere and Gamble

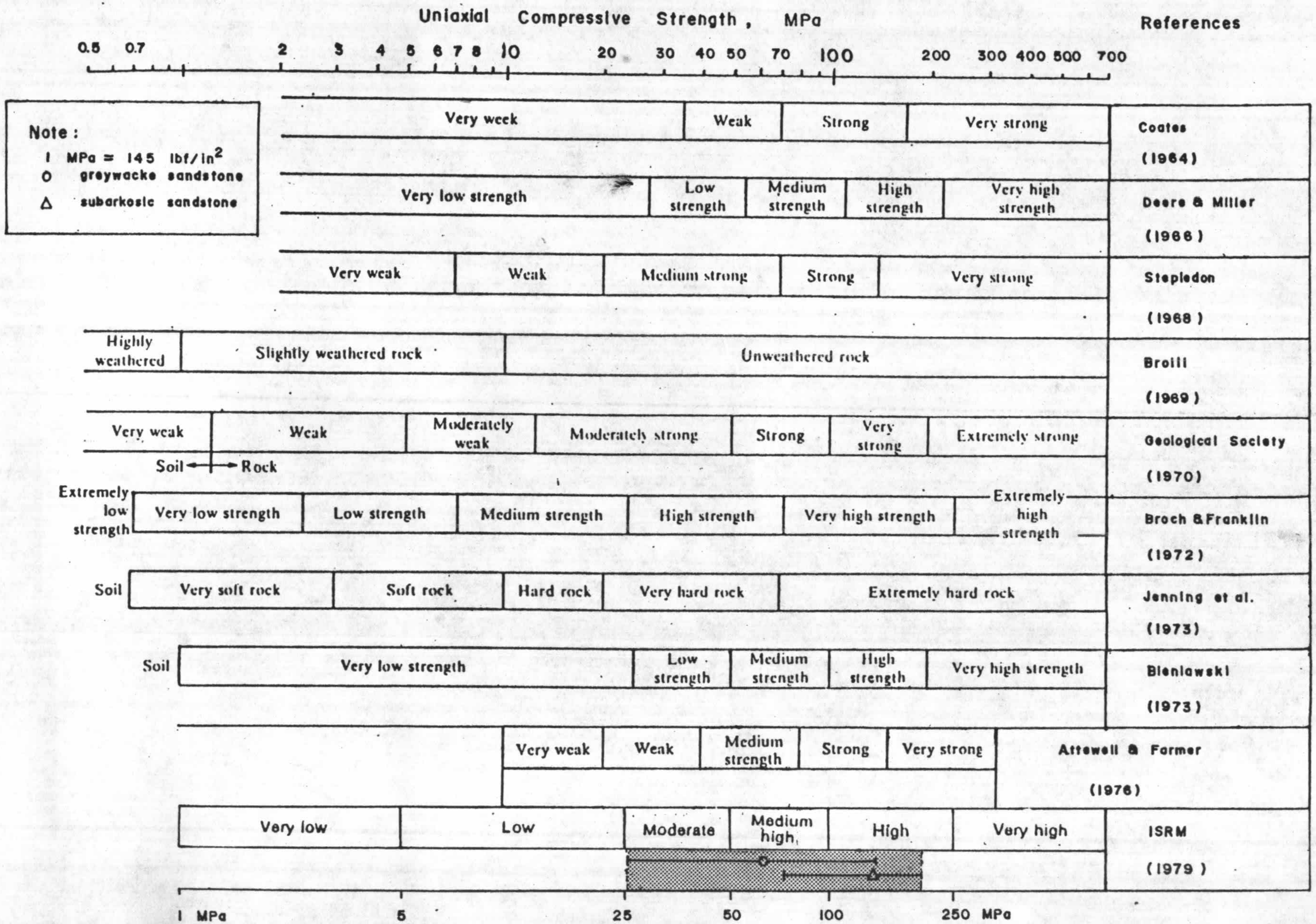
Deere and Gamble (1971) gave a modified version of Gamble's (1971) earlier work. That classification is described as the slake durability index (I_d). The values of I_d vary over the range from 0 to 100%. There is no discernible connection between the durability and geological age but the durability increases linearly with the density and inversely with the natural water content (Goodman, 1960). Olivier (1978) reported his results of two-cycle slake durability tests carried out on the selected rock samples using the apparatus developed by Franklin and Chandra (1972), then classified according to this classification. His report is presented in Figure 6.4.

6.2.1.4 Fissuring Classification

The degree of fissuring should be a basic component of any rock classification scheme. It can be characterized through a microscopic observation, or more simply through the index tests.

Tourenq et al. (1971) offered a approach based on a comparison of the actual and theoretical wave propagation velocities. The low velocity ratio of the actual to the theoretical values indicates a significant fissuring degree. They defined the ratio of measured-to calculated longitudinal wave velocities in term of a quality index IQ.

Table 6.7 Uniaxial compressive strength classification for intact rock (modified from Bieniawski, 1981)



The results of the pebbly graywacke are classified according to the fissuring classification of Fourmaintraux (1976), are presented in Figure 6.5. This phenomenon appears to be slightly-to non-fissured rock.

6.2.2 Rock Mass Classification Systems

The engineering classification of rook masses is neccessary adjunct for assessing the rock mass conditions for the engineering purposes in both design and construction works. It is believed that this approach is a suitable, practicable and highly promising mean for the engineering work during preliminary planning and construction (Bieniawski, 1976, 1979). The successive olassification systems proposed by the previous workers are as follow.

6.2.2.1 Terzaghi's Rock Load Classification

Terzaghi (1946) was the first to attempt any type of rock classification for the engineering purposes. His classification is a simple system for use in estimating the loads to be supported by the steel arches in tunnels. He recognised the significance of discontinuities, their spacing, and infilling materials, and used this information to define the certain types of ground which would influence the load imposed on the steel arches in tunnel. The classification is according to the tunnelling terms i.e., intact, stratified, moderately jointed, blocky and seamy, crushed, squeezing, and swelling rocks (Geological Society of Engineering Group Working Party, 1977 and Hoek and Brown, 1980).

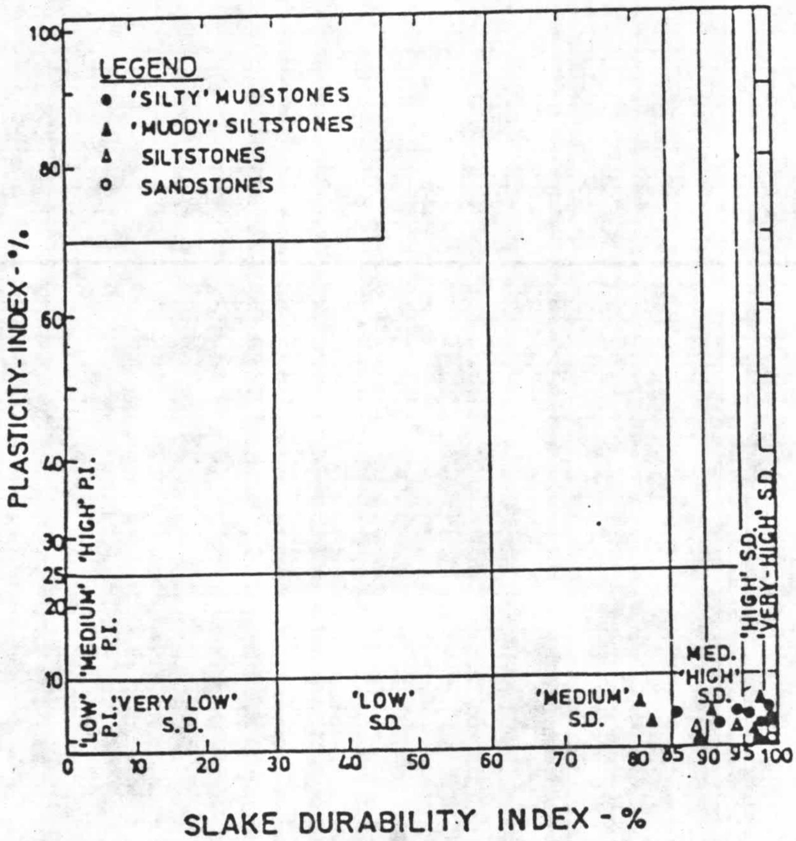


Figure 6.4 Deere-Gamble durability-plasticity classification of the intact rock (modified after Olivier, 1978).

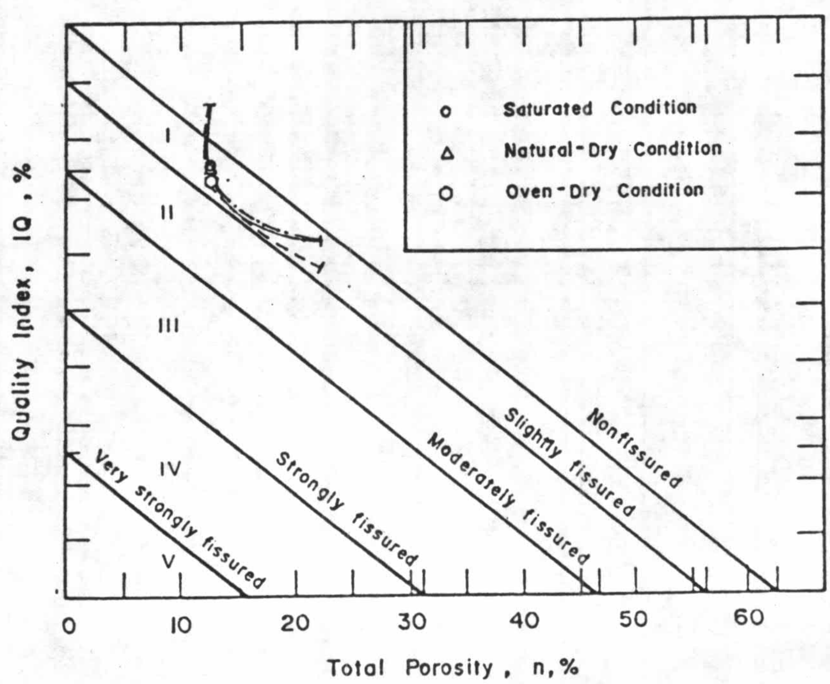


Figure 6.5 Classification scheme for fissuring in 37 Chiew Larn rock specimens (modified after Fourmaintraux, 1976).

6.2.2.2 Classifications of Stini and Lauffer

Stini (1950) proposed another rock mass classification which emphasized the importance of structural defects in the rock masses and stressed the need to avoid tunnelling paralled to the strike of steeply-dipping discontinuities. Lauffer (1958) emphasized the concept of the active unsupported rock spans and the corresponding stand-up time which are very relevant parameters for determining the type and amount of support in tunnels. He suggested that the stand-up time for any given active span was related to the rock mass characteristics, and by this way the rock was classified into seven categories from a very good rock to very poor rock as displayed in Figure 6.6.

6.2.2.3 Deere's Rock Quality Designation (RQD)

Rock quality designation (RQD) was proposed in 1964 by Deere. It is a measure of drill core quality as obtained from the borehole according to the degree of fracturing and the amount of weathering (Bieniawski, 1974). RQD is a quantitative index based on a modified core recovery procedure which incorporated only those pieces of hark, sound pieces which are 10 cm or longer in length. Generally RQD should be determined on a core of at least 50 mm in diameter and is expressed as

$$RQD = \frac{\text{Length of core in pieces} > 100 \text{ mm}}{\text{Total of core length}} \times 100 \quad ....(6.1)$$

Deere (1968) developed the relationship between the numerical value of RQD and the engineering quality of the rock from very poor to very good, as shown in Table 6.8.

Table 6.8 Deere's RQD quality bands (after Farmer, 1983).

Deere's RQD Classification	Description	Terzaghi Classification
90 - 100	Excellent	1 - 3
75 - 90	Good	3 - 4
50 - 75	Fair	5
25 - 50	Poor	5 - 6
0 - 25	Very Poor	6 - 7

The rock mass classification system used in this study follows that of Deere's (1964) as the system suggests a measurable quantity rather than the personal experience in the systems of Terzaghi (1964) and Stini (1950). The RQD values of this study area obtained from the boreholes showed that the pebbly graywackes are fair to good (50 to 100 %) except in strong weathering and shear zones where very poor to poor (0-50 %). On the other hand, that of the subarkoses are generall good to very good (75-100 %) with the lower RQD where the rock contact with the weathered cobbly graywackes or are near ground surface. These results illustrated in Figure 6.7.

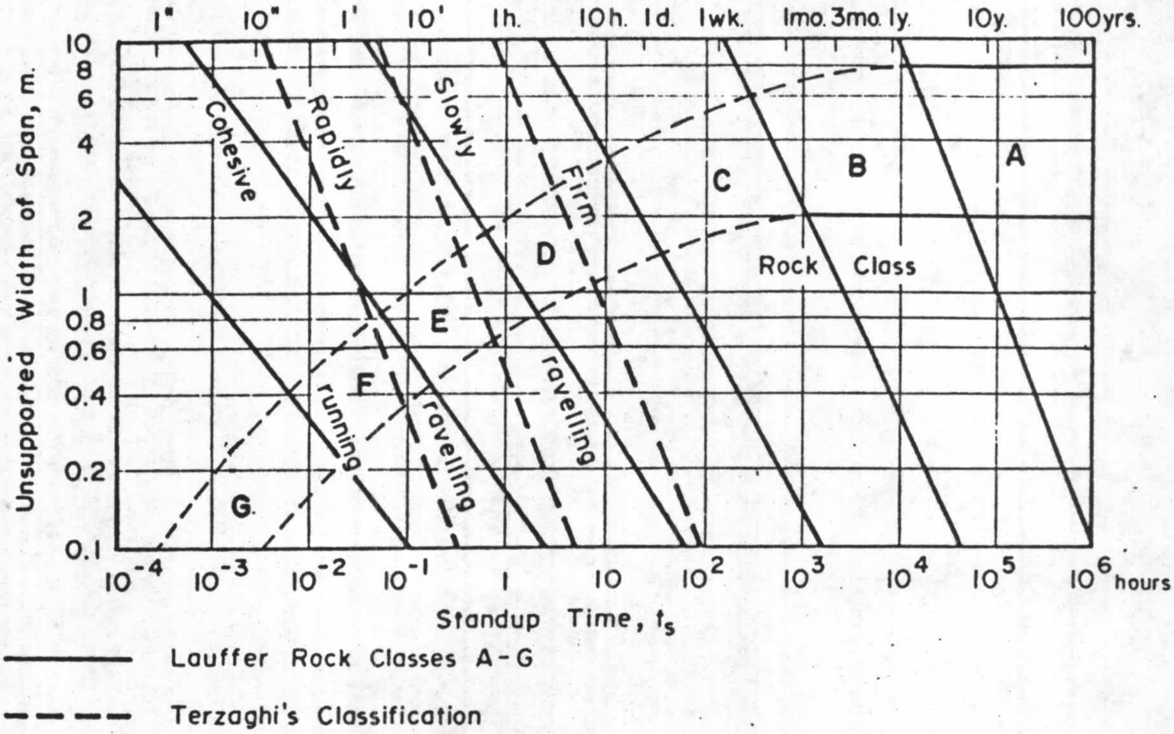


Figure 6.6 Standup time as a function of rock class and undupported width of tunnel roof (modified from Lauffer, 1958)..

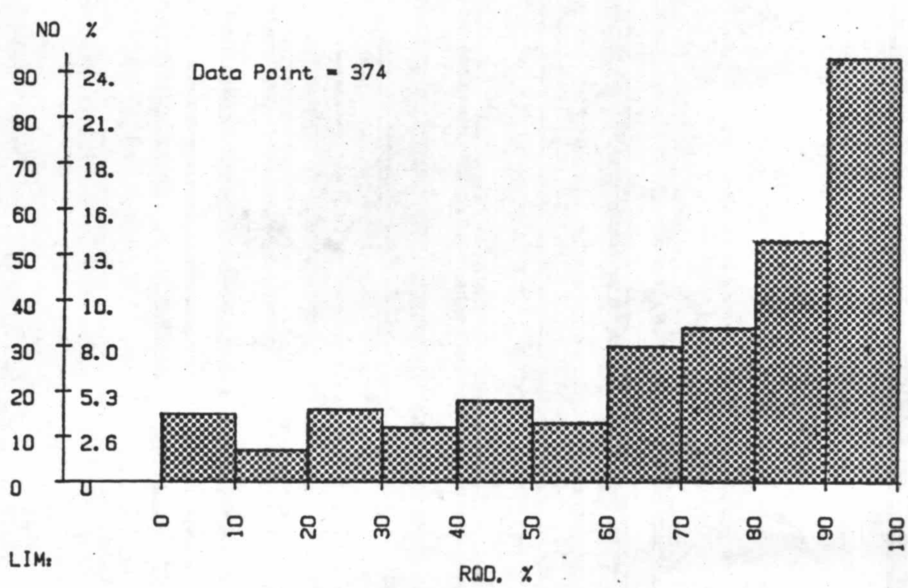


Figure 6.7 Histogram plot of the frequency distribution of the rock quality designation of Chiew Larn rock.

6.2.2.4 Rock Structure Rating (RSR)

Wickham et al. (1974) presented a ground-support prediction model intended to provide a common method for appraising and rating, on a numerical scale, the competency or physical quality of a rock structure, with respect to its need for structural support around the tunnel openings. It is based on an index called the Rock Structure Rating (RSR) which is defined as the sum of the values of three parameters A, B and C. Parameter A represents the general geology of the rock mass, including the effects of rock type, strength and the geological defects. Parameter B represents the joint pattern, including the joint spacing and thickness, the attitude (strike and dip) and the tunnel alignment as related to joint orientations. Parameter C represents the groundwater and joint conditions. The RSR will vary from the lowest possible value of 19 for the worst rock condition to a maximum of 100 for an ideal condition.

The classification system was not further studied in the present work since the system is to be equipped with some complicate parameters, yet the study result will be similarly to that obtained from the RQD system.

6.2.2.5 Geomechanics Classification of Jointed Rock Masses

Bieniawski (1973) introduced a Geomechanics Classification similar to Wickham's RSR concept. The original system of Bieniawski had eight parameters, namely the rock quality designation (RQD), degree of weathering, intact rock strength, joints spacing, joints separation, joint continuity, groundwater inflow, joint orientation (strike and dip). Bieniawski (1974, 1975, 1976, 1979) had revised his previous classification system by eliminating some parameters such and separation of joints together and adding the point-load uniaxial compressive strengths. He also changed the weighting of the parameters. The five basic classification parameters, having been revised are the strength of intact rock material, rock quality designation, joint spacing, joint condition and groundwater condition. The latest Bieniawski's (1979) system is shown in Table 6.10.

Bieniawski (1973) also recommended that, for an application, the following six parameters were most significant in the behavior of rock mass. The ratings for parameter 1 to 5 in part A of Table 6.10 are summed and then corrected by the rating adjustment for the joint orientations given in part B to give a parameter called the rock mass rating, or RMR system. Part C on the table shows the class and description give to the rock masses with various rock mass rating. The interpretation of these ratings in terms of stand-up times for the underground excavations and rock mass strength parameters is given in part D of the table and also illustrated in Figure 6.8. These classification parameters can all be observed and measured in the field (Bieniawski, 1974).

(a) <u>Uniaxial Compressive Strength of Intact Rock Material</u>

The properties of the intact rock material is important if the discontinuities are not persistance, thus a specimen of rock material represets a microscopic-scale model of the rock mass as the rock have been subjected to the same geological process in every scale.

Table 6.9 Geomechanics Classification of jointed rock mass (after Bieniawaki, 1979).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGES OF VALUES				
1	Strength of intact roeck 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 compressive 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		Very rough surfaces. Not continuous No seperation 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. Continous
	Rating		30	25	20	10	0
5	Ground water	Inflow per 10 m tunnel length	None	<10 litres/min	10-25 litres/min	25 - 125 litres/min	> 125
			OR	OR	OR	OR	OR
		Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0	0.0-0.1	0.1-0.2	0.2-0.5	> 0.5
			OR	OR	OR	OR	OR
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100← 81	80 ← 61	60 ← 41	40 ← 21	< 20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CLASSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

Table 6.10 Assessment of joint orientation favourability upon stability of a) tunnels (after Bieniawski, 1979), b) dam foundations (after Bieniawski, 1975).

a)

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

b)

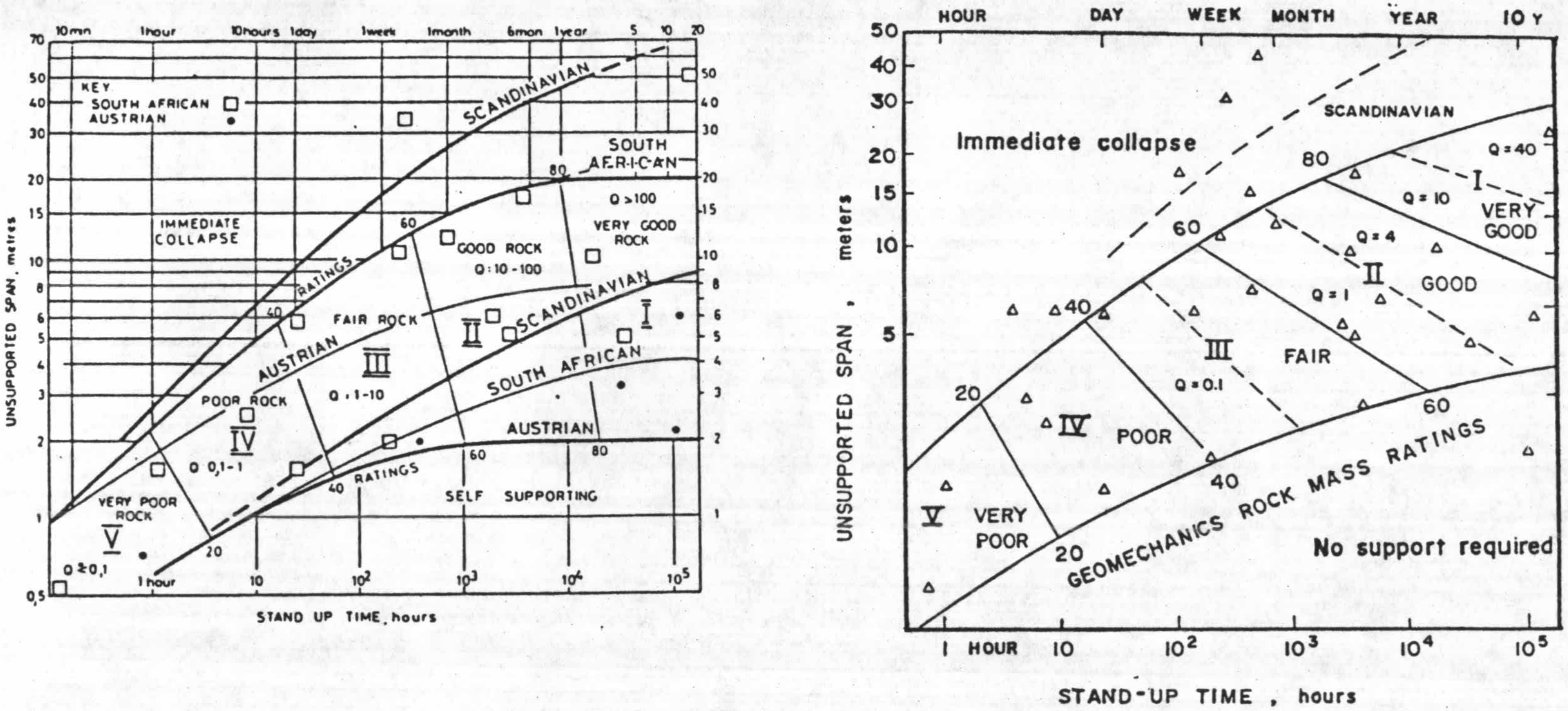
Dip 0°-10°	Dip 10°-30°		Dip 30°-60°	Dip 60°-90°
	Dip direction			
	Upstream	Downstream		
Very favourable	Unfavourable	Fair	Favourable	Very unfavourable

Note :

This table is based on experience and on considerations of stress distributions in foundation rock masses [28] as well as on an assumption that in a dam structure both the arch and the gravity effects are present.

The initial *in-situ* state of stress is not considered here as in dam foundations *in-situ* stresses are mainly important when considering grouting, drainage curtains and the excavation sequence of the foundations. For this last aspect recent evidence [29] shows that high horizontal stresses may be expected in near-surface rock masses.

This table is tentative only and represents simplifications of a problem which is justified for rock mass classification purposes only. The authors would welcome comments and discussion on this table.



a)

b)

Figure 6.8 a) Comprison of output data from classifications, b) Relationships between stand up time and unsupported span for various rock mass classes (after Bieniawski, 1979, 1981).

The uniaxial compressive strength can be determined by both in-situ and laboratory test methods. However, since the usual laboratory tests require a careful specimen preparation and rather expensive testing apparatus, it is recommended that the determination in the field from the point-load strength index which requires a simple portable equipment and involves testing on site of unprepared rock core of irregular chump is a good substitution (Bieniawski, 1974). The relationship between the uniaxial compressive strength, the point-load strength classification of the rock material is given in Table 6.11. The table also includes the development introduced by Bieniawski (1981) to conform with the values of the SI metric units.

b) <u>Drill Core Quality</u> (RQD)

The previously reviewed RQD quality (Deere, 1964) is very practical and can be used by a simple approach with a considerable potential to relate the value to the tunnel support as well as estimating deformability of rock masses (Bieniawaki, 1979). However, if a borehole core is unavailable, RQD may be estimated from the number for each joints per unit volume, in which the number of joints per meter for each joint set are added. A simple relation can be used to correct this number to get RQD in the case of clay-free rock masses (Barton et al., 1974). The calculation is as follow.

$$RQD = 115 - 3.3\, Jv \quad \text{(approx)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (6.2)$$

where Jv = total number of joint per m^3.

Thus, $RQD = 100$ for $Jv \; 4.5$

Table 6.11 Uniaxial compressive strength classification for intact rock (modified from Bieniawski, 1981).

Designation	ISRM		Deere and Miller (1966)		Bieniawski (1979)
	Commission on Rock Classification (1977)	Commission on Standardization (1979)	Strength		Point-Load Strength Index
	MPa	MPa	MPa	x 100 lbf/in^2	MPa
Very high	> 200	> 250	> 200	> 32	10
High	60-200	100-250	100-200	16-32	4-10
Medium	-	50-100	50-100	8-16	2-4
Moderate	20-60	25-50	-	-	1-2
Low	6-20	5-25	25-50	4-8	< 1
Very low	< 6	1-5	< 25	< 4	

Note : 1 MPa = 145 lbf/in^2

(c) Spacing and Orientation of Joints

As a geotechnical point of view, the term "joints" includes the discontinuities that resemble the joints, e.g. the bedding planes and other surfaces of weakness. The spacing and orientation of joints are of an importance in the stability of structure in the jointed rock masses. The presence of joints reduces the strenght of the rock mass and the joint spacing as well as their dip and strike govern the degree of such reduction. The spacing of joints means the average distance between two planes of weakness in the rock mass measured in the direction perpendicular to the planes. The classification according to the spacing and orientation of joints proposed by Deere (1968) is illustrated in Table 6.12.

Table 6.12 Discontinuity spacing classification (after Deere, 1968).

Description	Discontinuity Spacing (m)	Fracture Index (m^{-1})	Terzaghi Classification	Rock Mass Designation
Very wide	> 3		1	Solid
Wide	1 - 3	>1	3	Massive
Moderately	0.3 - 1	1 - 3	4 - 5	Blocky/Seamy
Close	o.05 - 0.3	3 - 20	6	Fractured
Very close	< 0.05	>20	7	Crushed/ Shattered

(d) Condition of Joints

The condition of joints includes the joint roughness, separation, and continuity, and the infilling or gouge material. The tight joints with a rough surface and without gouge have a rather high strength while the open continuous joints facilitate an unrestricted inflow of groundwater. ISRM (1981) suggested a quantitative description on the basis of roughness and infilled materials as exhibited in Figure 6.9 and Table 6.13.

(e) Groundwater Inflow

This parameter has an important effect on the behavior of the jointed rock masses. In the case of a tunnel, the rate of inflow of groundwater, measured in litre per minute, is indicated to be the governing factor (Wickham et al., 1972). ISRM (1978) preposed the classification of this parameter as shown in Table 6.14.

6.2.2.6 NGI Tunnelling Quality Index (Q)

The classification was proposed by Barton et al. (1974) and was known as the Q-system of the rock mass classification and support design. The system is based on Barton's (1976) suggestion of a numerical assessment of the rock mass quality using six parameters, i.e., RQD, number of joint set (Jn), roughness of the most unfavourable joint or discontinuity (Jr), degree of water inflow (Jw), and stress condition (SRF). The numerical values of these parameters are shown in Table 6.15 and the overall quality Q is expressed in the following equation,

$$Q = (RQD/Jn) \times (Jr/Ja) \times (Jw/SRF) \qquad \text{.............(6.3)}$$

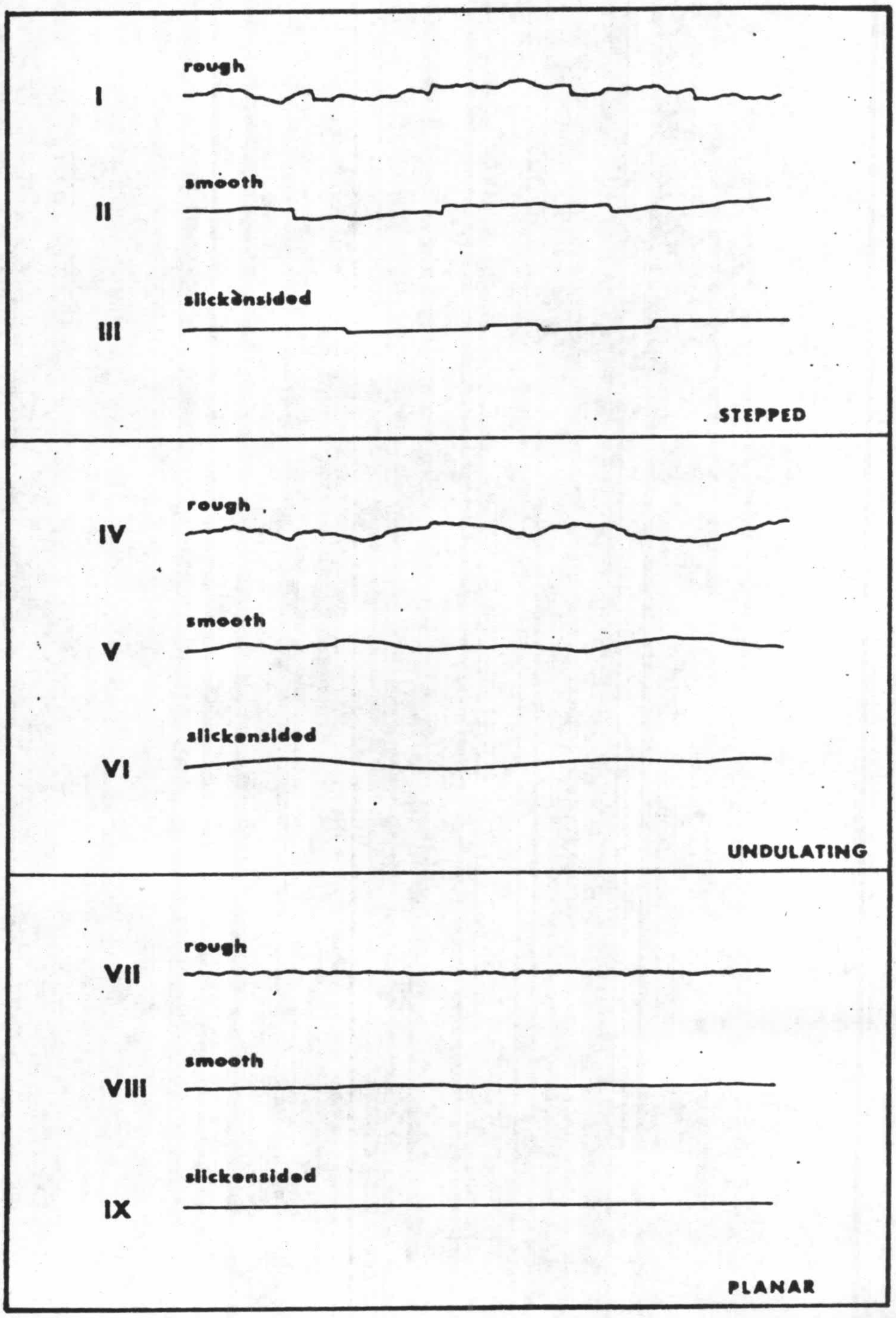


Figure 6.9 Typical roughness profiles and suggested nomenclature. The length of each profile is in the range 1 to 10 metres. The vertical and horizontal scales are equal (after ISRM, 1981).

Table 6.13 Description increments for filled discontinuity (after ISRM, 1981).

(a) Geometry:	width wall roughness field sketch	W1 The filling materials are heavily consolidated and dry, significant flow appears unlikely due to very low permeability.
(b) Filling type:	mineralogy particle size weathering grade soil index parameters swelling potential	W2 The filling materials are damp, but no free water is present. W3 The filling materials are wet, occasional drops of water. W4 The filling materials show signs of outwash, continuous flow of water (estimate litres/minute).
(c) Filling strength:	manual index (S1–S6) shear strength over-consolidation ratio displaced/undisplaced	W5 The filling materials are washed out locally, considerable water flow along out-wash channels (estimate litres/minute and describe pressure i.e. low, medium, high).
(d) Seepage:	water content (rating as W1–W6) permeability quantitative data	W6 The filling materials are washed out completely, very high water pressures experienced, especially on first exposure (estimate litres/minute and describe pressure).

Grade	Description	Field identification	Approx. range of uniaxial compressive strength (MPa)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025
S2	Soft clay	Easily penetrated several inches by thumb	0.025–0.05
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05–0.10
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10–0.25
S5	Very stiff clay	Readily indented by thumbnail	0.25–0.50
S6	Hard clay	Indented with difficulty by thumbnail	>0.50
R0	Extremely weak rock	Indented by thumbnail	0.25–1.0
R1	Weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0–5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0–25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25–50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50–100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100–250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

Note. Grades S1 to S6 apply to cohesive soils, for example, clays, silty clays and combinations of silts and clays with sand, generally slow draining. Some rounding of the strength values has been made when converting to S.I. units

Table 6.14 Description increments for seepage (after ISRM, 1981).

Unfilled discontinuities

Seepage rating	Description
I	The discontinuity is very tight and dry, water flow along it does not appear possible.
II	The discontinuity is dry with no evidence of water flow.
III	The discontinuity is dry but shows evidence of water flow, i.e. rust staining, etc.
IV	The discontinuity is damp but no free water is present.
V	The discontinuity shows seepage, occasional drops of water, but no continuous flow.
VI	The discontinuity shows a continuous flow of water. (Estimate l/min and describe pressure i.e. low, medium, high).

Filled discontinuities

Seepage rating	Description
I	The filling materials are heavily consolidated and dry, significant flow appears unlikely due to very low permeability.
II	The filling materials are damp, but no free water is present.
III	The filling materials are wet, occasional drops of water.
IV	The filling materials show signs of outwash, continuous flow of water (estimate l/min).
V	The filling materials are washed out locally, considerable water flow along out-wash channels (estimate l/min and describe pressure i.e. low, medium, high).
VI	The filling materials are washed out completely, very high water pressures experienced, especially on first exposure (estimate l/min and describe pressure).

Rock mass (e.g. tunnel wall)

Seepage rating	Description
I	Dry walls and roof, no detectable seepage.
II	Minor seepage, specify dripping discontinuities.
III	Medium inflow, specify discontinuities with continuous flow (estimate l/min/10 m. length of excavation).
IV	Major inflow, specify discontinuities with strong flows (estimate l/min/10 m. length of excavation).
V	Exceptionally high inflow, specify source of exceptional flows (estimate l/min/10 m. length of excavation).

Table 6.15 Classification of individual parameters used in NGI tunnelling quality infex (after Barton et al., 1974)

Description	*Value*	*Notes*
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	
G. Three joint sets plus random	12	1. For intersections use ($3.0 \times J_n$)
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	
E. Rough or irregular, planar	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. $J_r = 0.5$ can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
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	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
a. Rock wall contact.		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	

	J_a	ϕ_r (approx.)	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)	
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)	
b. Rock wall contact before 10 cms shear.			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)	
c. No rock wall contact when sheared.			
K, Zones or bands of disintegrated L. or crushed rock and clay (see M. G,H and J for clay conditions)	6.0 8.0 8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
O. Thick, continuous zones or P. bands of clay (see G, H and R. J for clay conditions)	10.0 - 13.0 13.0 - 20.0	(6° - 24°)	
5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (Kgf/cm²)	
A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D. Large inflow or high pressure , considerable outwash of fillings	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10	

Table 6.15 (cont.)

6. STRESS REDUCTION FACTOR

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

	SRF
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)	5.0
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)	2.5
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)	5.0
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)	2.5
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)	5.0

b. Competent rock, rock stress problems

	σ_c/σ_1	σ_t/σ_1	SRF
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure

	SRF
N. Mild squeezing rock pressure	5-10
O. Heavy squeezing rock pressure	10-20

d. Swelling rock, chemical swelling activity depending upon presence of water

	SRF
P. Mild swelling rock pressure	5-10
R. Heavy swelling rock pressure	10-20

1. Reduce these values of SRF by 25 - 50% if the relevent shear zones only influence but do not intersect the excavation.
2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
3. Few case records available where depth of crown below surface is less than span width. Suguest SRF increase from 2.5 to 5 for such cases (see H).

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :
   $\boxed{RQD = 115 - 3.3J_v \text{ (approx.)}}$ where J_v = total number of joints per m^3
   ($RQD = 100$ for $J_v < 4.5$)
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the *weakest significant joint set or clay filled discontinuity* in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q . *The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.*
4. When a rock mass contains clay, the factor SRF appropriate to *loosening loads* should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

Again these parameters which come in 3 pairs are found to be a crude estimation of the relative block size (RQD/Jn), interblock shear strength (Jr/Ja), and active stress (Jw/SRF). In order to relate their tunnelling quality index Q to the behavior and support requirement of an underground excavation, Barton et al. (1976) proposed an assumed maximum design span for the man-made opening based on the case history records of permanently unsupport excavations in a variety of rock masses as illustrated in Figure 6.10. And in order to estimate the maximum unsupport span for the difference of the excavation support ratio, ESR, as redrawn in Figure 6.11. The relationship is given in following equation.

$$\text{span} = 2.\text{ESR}.Q^{0.4} \qquad \text{.................(6.4)}$$

or

$$Q = (\text{span}/2.\text{ESR})^{2.5} \qquad \text{.................(6.5)}$$

6.2.3 Parameters Affecting Rock Mass Classification

The parameters which affect the values, to be used in the rock mass classification, are listed in Table 6.16. The assessment of these parameters, having been performed in the present study, is described below.

6.2.3.1 Joint Surveys

The joint survey is planned according to the size of the excavation and the scale of the being-considered fractures. Usually the measurement of the size, orientation, etc, of the joints is done on all joints, no matter whether they belong to the same systematic joint sets nor be randomly scattered. Certainly, the joint-set members will acquire an average value of orientation which is

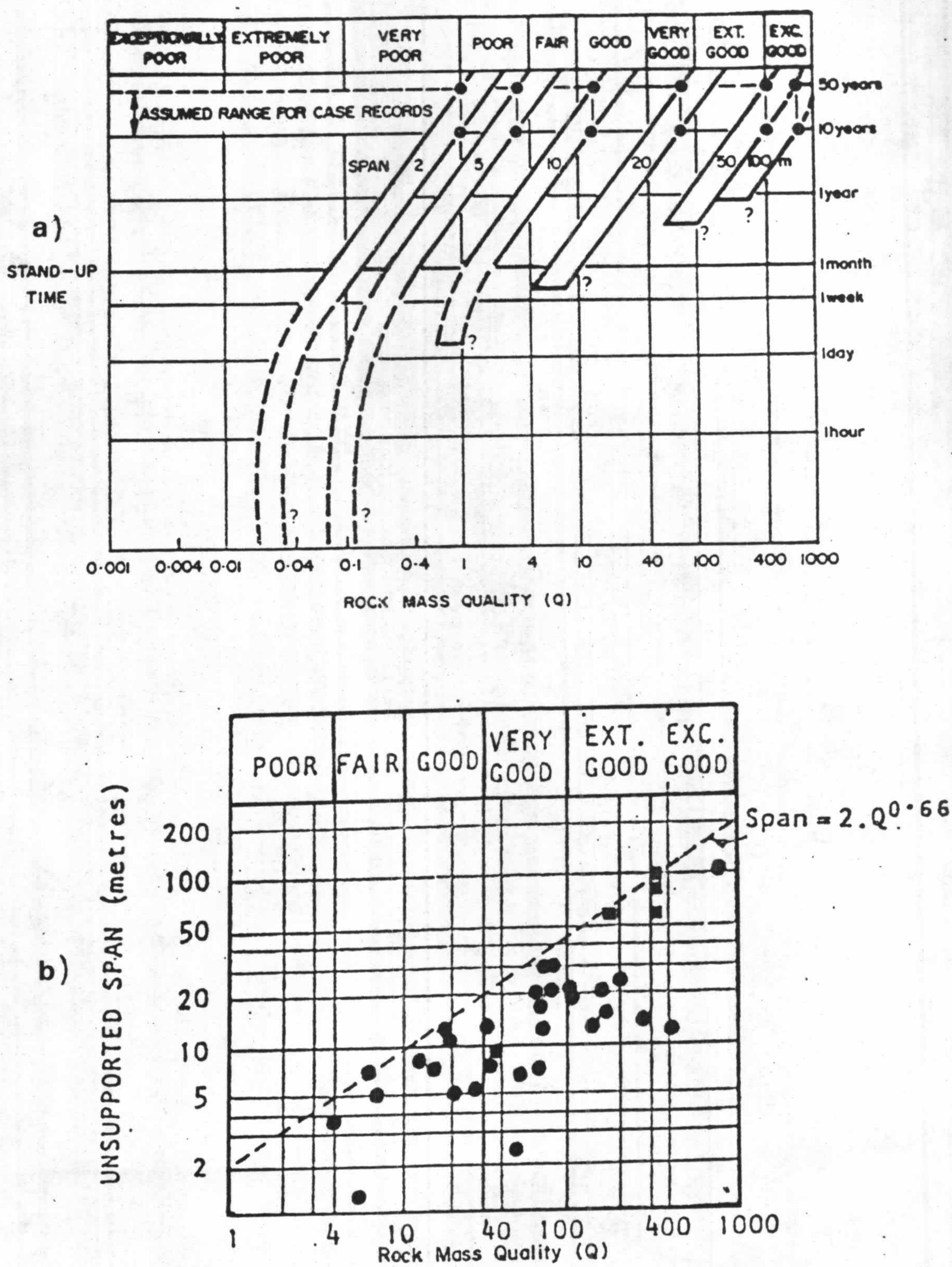


Figure 6.10 a) Relationship between span, standup time and the rock mass quality (Q), b) Man-made and natural unsupported excavation indifferent quality rock mass (after Barton, 1976).

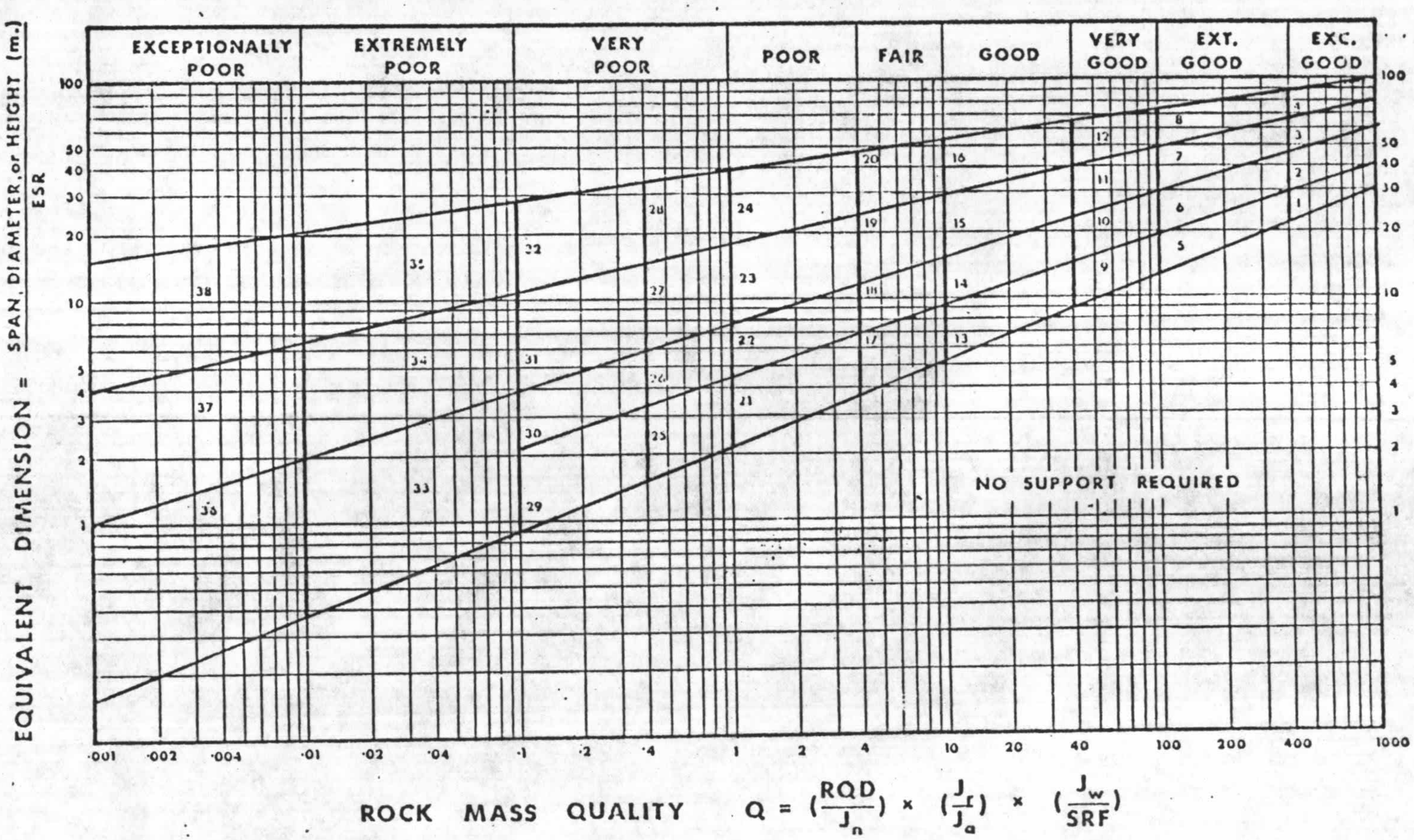


Figure 6.11 Relationship between the maximum equivalent dimension of an unsupported underground excavation and the NGI rock mass quality index Q (after Barton et al., 1974).

Table 6.16 Approximate equation for principal stress relationship and Mohr envelops for intact rock and jointed rock masses (after Hoek and Brown, 1980).

	CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS AND METAMORPHIC CRYSTALLINE ROCKS *amphibolite, gabbro, gneiss, granite, norite and quartz-diorite*
INTACT ROCK SAMPLES *Laboratory size rock specimens free from structural defects* *CSIR rating 100+, NGI rating 500*	$\sigma_{1n} = \sigma_{3n} + \sqrt{7\sigma_{3n} + 1.0}$ $\tau_n = 0.816(\sigma_n + 0.140)^{0.658}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{10\sigma_{3n} + 1.0}$ $\tau_n = 0.918(\sigma_n + 0.099)^{0.677}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{15\sigma_{3n} + 1.0}$ $\tau_n = 1.044(\sigma_n + 0.067)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{17\sigma_{3n} + 1.0}$ $\tau_n = 1.086(\sigma_n + 0.059)^{0.696}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{25\sigma_{3n} + 1.0}$ $\tau_n = 1.220(\sigma_n + 0.040)^{0.705}$
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints spaced at ± 3 metres* *CSIR rating 85, NGI rating 100*	$\sigma_{1n} = \sigma_{3n} + \sqrt{3.5\sigma_{3n} + 0.1}$ $\tau_n = 0.651(\sigma_n + 0.028)^{0.679}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{5\sigma_{3n} + 0.1}$ $\tau_n = 0.739(\sigma_n + 0.020)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{7.5\sigma_{3n} + 0.1}$ $\tau_n = 0.848(\sigma_n + 0.013)^{0.702}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{8.5\sigma_{3n} + 0.1}$ $\tau_n = 0.883(\sigma_n + 0.012)^{0.705}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{12.5\sigma_{3n} + 0.1}$ $\tau_n = 0.998(\sigma_n + 0.008)^{0.712}$
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints spaced at 1 to 3 metres.* *CSIR rating 65, NGI rating 10*	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.7\sigma_{3n} + 0.004}$ $\tau_n = 0.369(\sigma_n + 0.006)^{0.669}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.0\sigma_{3n} + 0.004}$ $\tau_n = 0.427(\sigma_n + 0.004)^{0.683}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.5\sigma_{3n} + 0.004}$ $\tau_n = 0.501(\sigma_n + 0.003)^{0.695}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.7\sigma_{3n} + 0.004}$ $\tau_n = 0.525(\sigma_n + 0.002)^{0.698}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{2.5\sigma_{3n} + 0.004}$ $\tau_n = 0.603(\sigma_n + 0.002)^{0.707}$
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1 metre.* *CSIR rating 44, NGI rating 1.0*	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.14\sigma_{3n} + 0.0001}$ $\tau_n = 0.198(\sigma_n + 0.0007)^{0.662}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.20\sigma_{3n} + 0.0001}$ $\tau_n = 0.234(\sigma_n + 0.0005)^{0.675}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.30\sigma_{3n} + 0.0001}$ $\tau_n = 0.280(\sigma_n + 0.0003)^{0.688}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.34\sigma_{3n} + 0.0001}$ $\tau_n = 0.295(\sigma_n + 0.0003)^{0.691}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.50\sigma_{3n} + 0.0001}$ $\tau_n = 0.346(\sigma_n + 0.0002)^{0.700}$
POOR QUALITY ROCK MASS *Numerous weathered joints spaced at 30 to 500mm with some gouge filling / clean waste rock* *CSIR rating 23, NGI rating 0.1*	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.04\sigma_{3n} + 0.00001}$ $\tau_n = 0.115(\sigma_n + 0.0002)^{0.646}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.05\sigma_{3n} + 0.00001}$ $\tau_n = 0.129(\sigma_n + 0.0002)^{0.655}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.08\sigma_{3n} + 0.00001}$ $\tau_n = 0.162(\sigma_n + 0.0001)^{0.672}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.09\sigma_{3n} + 0.00001}$ $\tau_n = 0.172(\sigma_n + 0.0001)^{0.676}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.13\sigma_{3n} + 0.00001}$ $\tau_n = 0.203(\sigma_n + 0.0001)^{0.686}$
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced less than 50mm with gouge filling / waste rock with fines* *CSIR rating 3, NGI rating 0.01*	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.007\sigma_{3n} + 0}$ $\tau_n = 0.042(\sigma_n)^{0.534}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.010\sigma_{3n} + 0}$ $\tau_n = 0.050(\sigma_n)^{0.539}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.015\sigma_{3n} + 0}$ $\tau_n = 0.061(\sigma_n)^{0.546}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.017\sigma_{3n} + 0}$ $\tau_n = 0.065(\sigma_n)^{0.548}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.025\sigma_{3n} + 0}$ $\tau_n = 0.078(\sigma_n)^{0.556}$

different from that of the combined joint set members and randomly orientated fractures, but the occurence of the random joints must be respected. Also, the joint condition, or the joint quality, will be less significant of only RQD and joint spacing values are being considered, unlike when all joint parameters are to be used. The joint condition in the latter case depicts a real behavior of the joint orientation to be used was that of the combined joint-set members and random fractures which were observed in and around each location of the major engineering structures.

6.2.3.2 Strength of Rock Substances

The strength of the rock is considered to be the most important parameter and is thus given a high rating in both RMR and Q-systems. Unfortunately, a problem which occurs is the variation of the rock strength obtained in both point-load index tests and uniaxial compression tests, due to an anisotropy of the rock substances. When only a wide range of the strength is available for an assessment of the rock mass quality, especially in the Geomechanics Classification, the problems are rather critical when the rating changes from one class to another. So in the case of the anisotropy should be remarked for the corresponding rating considered in the assessment of the rock mass quality. It may be concluded that the value of strength is the lowest when the direction of applied load is parallel to the weakest discontinuity plane of anisotropy, and can be considered as the worst condition of the rock mass.

6.2.3.3 Degree of Persistance

The limitation of the determination of the joints persistance degree depends largely on the availability of the joint information. It can be concluded that the variation in persistance depends partly on the exposures at each locality. Thus the range of values may not reflect the exact degree of persistance of the entire joints, but rather of the joints at those exposures.

6.2.3.4 Discontinuity Condition

The condition of discontinuity is the most important rating parameter for the Geomechanics Classification, as it effect the total rating value. This parameter consists of the persistance, separation, roughness and filling material. The classification of the joint condition in the RMR rating is rather limited as it considers only a fraction of the data obtained, for example, the persistance of joint is defined as "not continuous", "continuous", "with gouge" and "without gouge", etc. Hence, it inadequate and is difficult for a field appraisal of the joint conditions.

6.2.3.5 Measurement of Groundwater Inflow

In general, a crude estimation of the groundwater inflow is to record the time for the seepage to fill a container of known demensions. Knowing the area of tunnel from where the water was being pumped out, the volume of flow per unit time per unit horizontal length of along or across the tunnel can be estimated. For a small amount of groundwater inflow this method is not appropriate especially at the tunnel wall where the groundwater flows on the rock surface. Moreover, such an estimation may not represent the average flow because it represents only the locality where the measurement is performed, whereas the variation of seepage may be distributed in several localities along the entire tunnel length. This method may however be used in measuring the groudwater flow from the top heading. In a case of estimating the groundwater inflow for the surface exposures the flow should be estimated according to the regional hydrology which will be difficult in terms of practicality for field classification of the rock masses.

6.2.3.6 Joint Roughness (Jr) and Joint Alteration Number (Ja)

For the NGI classification, these parameters are assessed on a visula examination, the assessment requires an understanding of the difference between the classifying criteria which again depend on the personal judgement and experiences.

6.2.3.7 Stress Reduction Factor (SRF)

The type of rock stress and its magnitude are difficult to determine especially by field observation and classification Hoek and Brown (1980) suggested the basis of the strength of rock masses (Table 6.16) which gives a set of empirical equations defining the relationship between principal stresses and the Mohr failure envelops of the rock mass related to the Geomechanics Classification rating, RMR and rock mass quality, Q. The principal stresses relationship is presented as followed.

$$\sigma_{1n} = \sigma_{3n} + m\sigma_{3n} + 5 \qquad \ldots\ldots\ldots\ldots\ldots\ldots(6.6)$$

where m, s = the material constants

σ_{1n}, σ_{3n} = the normalised principal stress σ_1/σ_c and σ_3/σ_c, being the uniaxial compressive strength of the rock materials

6.2.4 Correlation of Different Rock Mass Classification Systems

The correlation of some vital classification systems was prepared bt Rutledge (1977), as shown in Table 6.17. The correlation between the systems of Wickham et al. (1972), Bieniawski (1973), and Barton et al. (1974) will be emphasized because they provide a mean for the most effective communication. Bieniawski (1976) attempted to correlate his RMR system with the Q system. The following relationship was employed.

$$RMR = 9 \ln Q + 44 \qquad \ldots\ldots\ldots\ldots\ldots\ldots(6.7)$$

Rutledge (1977 , 1978) endeavored to correlate three system, RSR, RMR and Q, using a number of case studies in New Zealand. From the regression analyses the following empirical relationships were derived.

$$RSR = 0.77\ RMR + 12.4 \qquad \ldots\ldots\ldots\ldots\ldots(6.8)$$

$$RSR = 13.3\ \mathrm{Log}Q + 46.5 \qquad \ldots\ldots\ldots\ldots\ldots(6.9)$$

$$RMR = 13.5\ \mathrm{Log}Q + 43.0 \qquad \ldots\ldots\ldots\ldots\ldots(6.10)$$

Table 6.17 Factors considered or included in various rock classification systems (after Rutledge, 1977)

	Factors Considered	Terzaghi	Wickham	Bieniawski	Barton	Laufer & Linder	Ikedu	Deere	Cecil
OBSERVABLE ROCK PROPERTIES	1 Specific for tunnels	yes	yes	yes	yes	yes	yes	yes	yes
	2 Geological rock type	qualitative	specu	not incl.	not incl.	not incl.	specific	not incl.	not incl.
	3 Rock hardness	not incl.	not incl.	not incl.	not incl.	not incl.	not incl.	not incl.	not incl.
	4 Rock Quality Designation (RQD)	not incl.	not incl.	specific	specific	not incl.	not incl.	specific	specific
	5 Degree of Weathering	qualitative	specific	not incl.	specific wrt joints	not incl.	affects 16	not incl.	noted
	6 Joints (a) pacing	qualitative	spacific	specific	no (in 4)	not incl.			
	(b) orientation	discussed	specific	specific	discussed	qualitative			
	(c) condition (ø)		specific s	specific s				not incl.	
	(d) number of sets	qualitative		not incl.			said to to be in	discussed	
	(e) continuity	not incl.	not incl.	specific s		not incl.	16	May be	not incl.
	(f) waviness			not incl.	specific			incl. in	
	(g) width-filling	qualitative	specific s	specific s				future	
	7 Weakness zones (faulting)	specific		not incl.					
	8 Geologic structure		specific	specific		link to 6b			
	9 Water inflow/pressure	discussed				not incl.	qualitative		
	10 Position of water table	specific	not incl.	not incl.	not incl.		not incl.		

Table 6.17 (cont.)

	Factors Considered	Terzaghi	Wickham	Bieniawski	Barton	Lauffer & Linder	Ikedu	Deere	Cecil
MEASURABLE PROPERTIES	11 In situ rock stresses	discussed	not incl.	to be 25 MPa	specific			specific	
	12 Unconfined compressive strength		specific	specific	not incl.			link to 11	
	13 ϕ and C	not incl.			for joints				not incl.
	14 Young s Mod. and Poisson s ratio		indirect use		not incl.	not incl.	specific		
	15 Rock density	specific	specific	not incl.	could adjust			not incl.	
	16 Seismic velocity	not incl.	indirect use						alt. to 4
	17 Slake durability	qualitative	not incl.		not incl.		not incl.		not incl.
TUNNEL SUPPORT	18 Sup. variation with tunnel size	specific wrt B&E	specific wrt B&H	only given for B=10 m	specific	no change	specific	no change	specifc
	19 Sup. limit of applicability	none	none		none	Up to B=10m	30-60 m^2	6.5-13 m	C-13 m
	20 Cover above tunnel	specific	no restr.	no restr.		no restr			
	21 Prediction of rock loads on support	specific wr		not given		not given	specific	specific wr	not given
	22 Factor of Safety used	specific						implied	
	23 Support Types (a) Rock bolts		specific	specific*s	specific				specific s
	(b) Shotcrete			specific*s		specific*	not given		
	(c) Steel sets and shotcrete	not given	not given	not given				specific*s	not given
	(d) Steel sets&timb	specific	specific	general*s	not given	not given	specific	general*s	
	(e) Permanent support	not given	not given	same as temp.?	specific	same as temp.		not given	
	24 Support stiffness in (21)	not incl.	not incl.	not incl.	not incl.			not incl.	not incl.

Table 6.17 (cont.)

	Factors Conditions	Terzaghi	Wickham	Bieniawski	Barton	Lauffer & Linder	Ikedu	Deere	Cecil
MISC.	25 Concept of stand up time	not qualif.	not incl.	yes	yes	main parameter	yes	no	no
	26 Method of excavation considered	not spec.	machine, D&B	D & B	D & B	qualitative	predicts	machine, D & B	D & B
	27 Advance rate				not qualif.				
	28 Swelling/squeezing ground covered	specific	not appl.	not appl.	specific	not appl.	qualitative	specific s	not appl.
	29 No. of tunnel in study	not known	53	not known	over 200	not known	70	not known	14
	30 Location of tunnels in study	European Alps	West Mid-W,USA		Scandinavia bias	mainly Europe	Japan	probably USA	Scandinavia

Notes: D & B = Drill & Blast
sup. = support
s = simple
wrt = with respect to
restr = restriction
appl = applicable
alt = alternative
qualif = qualified
Support given only for one tunnel size or same support specified for all sizes
wr = wide range
reqd = required
B = tunnel width
H = tunnel height

In the present study, the results of rock mass rating using the RMR-and Q systems of 40 structural regions along the top heading diversion tunnel were plotted in Figure 6.12 and the relationship was obtained.

$$RMR = 8.56 \ln Q + 53.7 \qquad \text{.............} \quad (6.11)$$

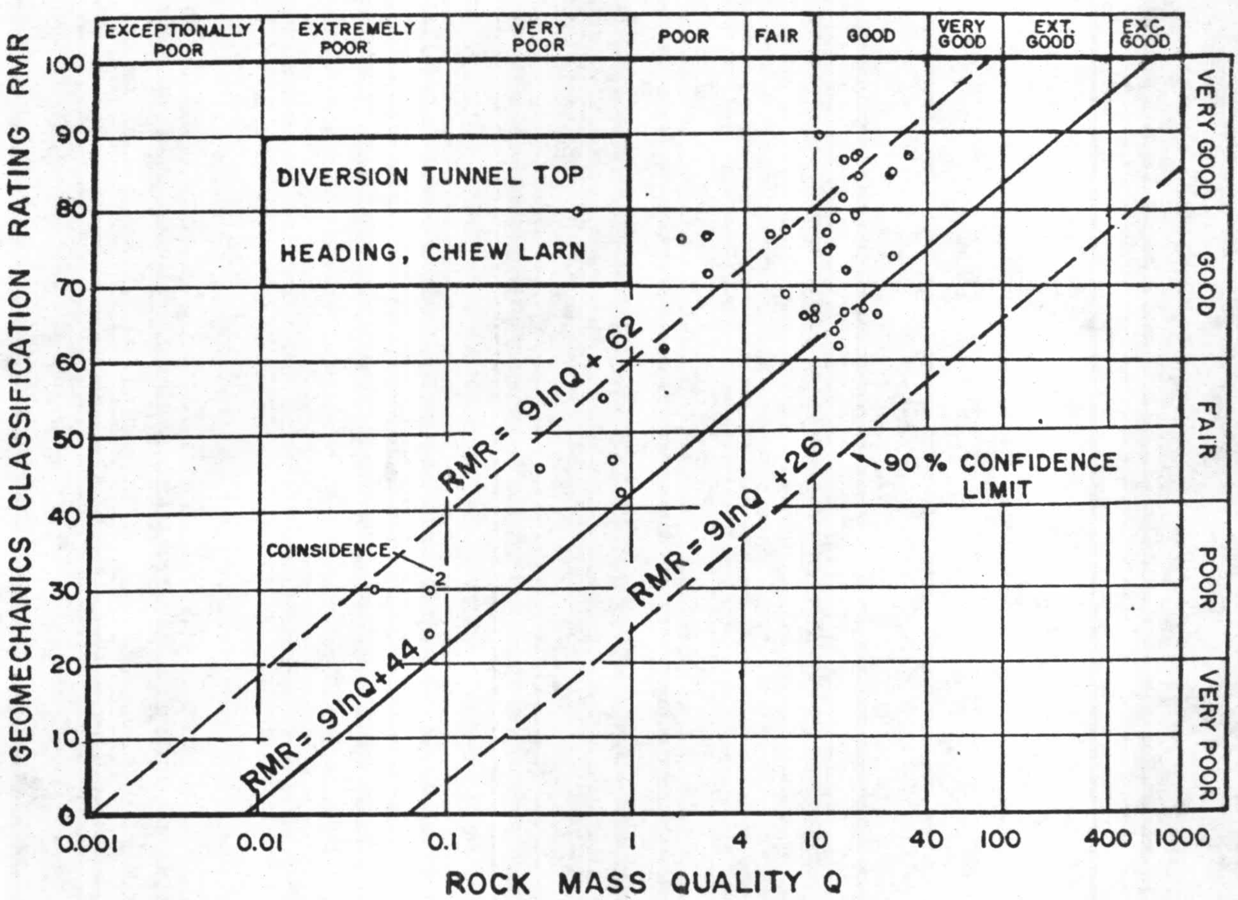


Figure 6.12 Scatter diagram display correlation between Geomechanics Classification and logarithm of number of rock mass quality Q of the diversion tunnel.

6.2.5 Range of Application

The rock mass classifications currently in use have reached a high level of development that enables the application to a wide range of engineering problems. Both the Geomechanics Classification and Q-system provide a sound basis for an engineering assessment of the rock masses and are suitable for a determination of the rock-bolt and shotcrete-support systems. The Geomechanics Classification is somewhat easier to be used and has been widely used in both tunnelling and mining (Bieniawski, 1981). The Q-system is particularly suitable for the cases of hard rock tunnels and large chambers (Barton et al., 1974).

(a) Tunnels and Chambers

The main application of rock mass classification has traditionally been for the tunnelling works. The RSR concept, the Geomechanics Classification, and the Q-system have all been applied extensively to the numerous tunnel types and large rock chambers. The output of rock mass classifications for tunneling is the stand-up time of an unsupported roof span, as depicted in Figure 6.12, as well as the support guidelines to be developed according to the case histories. Bieniawski (1976) reported a Geomechanics Classification based on 49 case histories and 200 for the Q-system. The different rock mass classification systems have clearly affected the selection of stand-up time and length of unsupported spans in the different rock conditions and tunnel practice. As delineated in Table 6.18, in the Overvaal tunnel case, the estimations for the support requirement using six different classification systems were compared.

(b) Rock Slopes

For the rock slopes, the output from the Geomechanics Classification is the cohesion and friction data for five rock-mass classes. In 1976, Steffen classified the slopes by using the Geomechanics classification. He calculated the factors of safety and plotted the results in the form of histograms that showed the frequency of the occurrance of slope failures versus the factor of safety.

(c) Rock Foundations

For a design of rock foundations, the knowledge of the modulus of rock mass deformability is the prime importance. The Geomechanics Classifications were found useful for estimating the in-situ deformability of the rock masses. This will be demonstrated in detailing in Chapter 9.

Besides, the Geomechanics Classification can also be applied to foundation bearing pressure, ground rippability, mining application, assessing of mine roof stability, and construction aspects (Bieniaeski, 1979).

Table 6.18 Comparison of rock mass classification applied at the Overvaal tunnel (width 5.5 m) (after Bieniawski, 1976).

Locality	GEOMECHANICS CLASSIFICATION (Bieniawski, 1973)		NGI CLASSIFICATION : Q-SYSTEM (Barton, 1974)		RSR CLASSIFICATION (Wickham, 1972)	
	Class	Support	Class	Support	Class	Support
H 6	I Very good rock RMR = 83	Occasional spot bolting.	Good rock Q = 33,0	Spot bolting only	RSR = 68	Bolts 25 mm dia. at 2 m (length not given)
H 4	II Good rock RMR = 67	Locally, grouted bolts (20 mm dia.) spaced 2-2,5 m, length 2,5 m plus mesh; shotcrete 50 mm thick if req.	Good rock Q = 12,5	Systematic grouted bolts (20 mm dia.) spaced 1 m - 2 m; length 2,8 m.	RSR = 60	Bolts spaced 1,4 m, shotcrete 35-45 mm or medium ribs at 2 m
H 2	III Fair rock RMR = 52	Systematic grouted bolts spaced 1,5-2 m, length 3 m plus mesh and 100 mm thick shotcrete.	Fair rock Q = 8,5	Systematic grouted bolts spaced 1,5 m, length 2,8 m; and mesh	RSR = 57	Bolts spaced 1,2 m and 50 mm shotcrete or ribs 6H20 at 1,7 mm
H 3	IV Poor rock RMR = 29	Systematic grouted bolts spaced 1-1,5 m, length 3 m, mesh plus 100-150 mm shotcrete (ribs at 1,5 m).	Poor rock Q = 1,5	Shotcrete only: 25-75 mm thick or bolts at 1 m, 20-30 mm shotcrete and mesh.	RSR = 52	Bolts spaced 1 m and 75 mm shotcrete or ribs 6H20 at 1,2 m.
H 5	V Very poor rock RMR = 15	Systematic grouted bolts spaced 0,7-1 m, length 3,5 m, 150-200 mm shotcrete and mesh plus medium steel ribs at 0,7 m. Closed invert.	Extremely poor rock Q = 0,09	Shotcrete only: 75-100 mm thick or tensioned bolts at 1 m plus 50-75 mm shotcrete and mesh.	RSR = 25	N/A

Locality	RQD CLASSIFICATION (Deere, 1969)		AUSTRIAN CLASSIFICATION (Rabcewicz/Pacher, 1974)		FRENCH CLASSIFICATION (Louis, 1974)	
H 6	Excellent RQD > 90	Occasional bolts only.	I Stable	Bolts 26 mm dia., 1,5 m long spaced 1,5 m in roof plus wire mesh.	A	50 mm shotcrete or 3 m long bolts at 3,1 m.
H 4	Good RQD: 75-90	Bolts 25 mm dia., 2 m-3 m long spaced 1,5-1,8 m and some mesh or 50-75 shotcrete or light ribs.	II Over-breaking	Bolts 2-3 m long spaced 2-2,5 m, shotcrete 50-100 mm with mesh.	B	100 mm shotcrete with mesh and 3 m bolts at 2,8 m.
H 2	Fair to good RQD: 50-90	Bolts 2 m-3 m long at 0,9-1 m plus mesh or 50-100 mm shotcrete or light/medium ribs at 1,5 m.	III Fractured to very fractured	Perfo-bolts 26 mm dia., 3-4 m long spaced 2 m plus 150 mm shotcrete plus wire mesh and steel arches TH16 spaced 1,5 m.	C	150 mm shotcrete with mesh and 3 m bolts at 2,5 m.
H 3	Poor RQD: 25-50	Bolts 2 m-3 m long at 0,6-1,2 m with mesh or 150 mm shotcrete with bolts at 1,5 m or medium to heavy ribs.	IV Stressed rock	Perfo-bolts 4 m long, spaced 1 m by 2 m plus 200 mm shotcrete plus mesh plus steel arches TH21 spaced 1 m. Concrete lining 300 mm.	D	210 mm shotcrete with mesh and 3 m bolts at 2 m and steel ribs.
H 5	Very poor RQD < 25	150 mm shotcrete all around plus medium to heavy circular ribs at 0,6 m centres with lagging.	V Very stressed rock	Perfo-bolts 4 m long spaced 1 m plus 250 mm shotcrete plus mesh and steel arches TH29 spaced 0,75 m. Closed invert. Concrete lining 500 mm	E	240 mm shotcrete with mesh and 3 m bolts at 1,7 m; steel ribs at 1,2 m. Closed invert.