CHAPTER VII

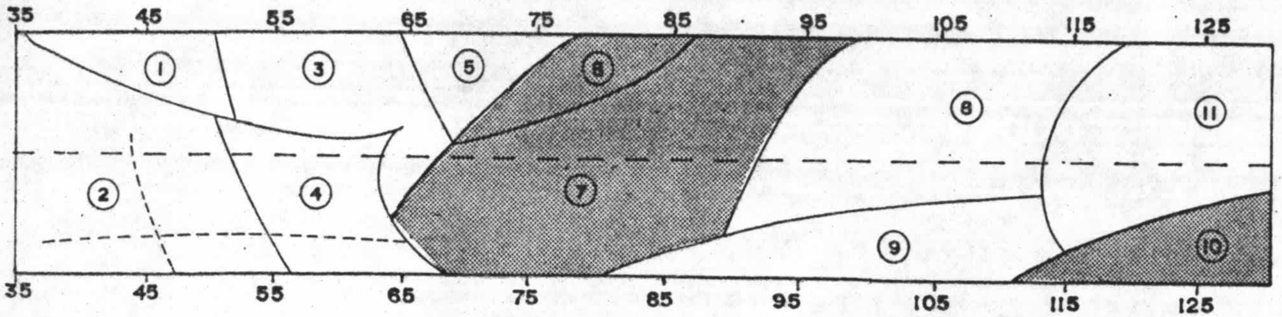
DIVERSION TUNNEL STABILITY ANALYSIS

The rock mass classification systems were applied to assess the rock mass quality along the diversion tunnel of Chiew Larn project. The data obtained from the joint survey and in-situ measurement provide the basic input data for two classification systems, namely, CSIR Geomechanics Classification (1979) and NGI Tunnelling Quality Index (1976).

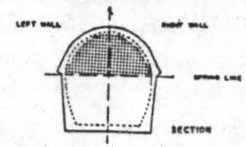
7.1 CSIR Geomechanics Classification

This classification of rock masses was carried out along the diversion tunnel and some portal slope faces. A geological logging was performed in order to obtain the structural rock masses regions. Fourty structural regions were identified and the results of the rock mass quality assessments were disclosed separately for the diversion tunnel in Figure 7.1. The results obtained from the classification can be concluded in terms of the rock mass quality related to the tunnel stability.

The assessments of tunnelling indices that the rock mass quality along the diversion tunnel varies from very poor to very good. The good rock mass is general predominant except in some structural regions where the low angle shear zone reigned. These



EXPLANATION.



- Very Poor Rock
- Poor Rock
- Fair Rock
- Good Rock
- Very Good Rock

Sys	App	Structural Regions / Rock Mass Parameters		1	2	3	4	5	6	7	8
Geomechanics Classification	A	Intact Rock Strength, MPa		3.60(7)	3.99(7)	3.86(7)	4.37(15)	3.06(7)	3.73(7)	3.34(7)	4.14(15)
		Rock Quality Designation-RQD, %		100(20)	92.46(20)	94.52(20)	91.73(20)	100(20)	98.74(20)	100(20)	100(20)
		Joint Spacing, m		0.6-2(23)	0.2-2(22.5	0.2-2(22.5)	0.2-2(22.5	0.2-2(22.5	0.2-2(22.5	0.6-2(23)	0.2-3(25)
		Persistence, m	Joint Conditions	2-3	2-3	2	2-3	2	2	2-3	2-3
		Filling		a	a	a	a	a	a	a	a
		Aperture, mm		3-4	3	2-3	2-3	3	3	2	3-4
		Roughness		4	3-4	3-4	3-4	3-4	3-4	4	3-4
		Average Rating		12	12	12	20	25	25	25	12
		Groundwater Inflow		Damp(10)	Damp(10)	Dry(15)	Damp(10)	Dry(15)	Dry(15)	Dry(15)	Damp(10)
		Parameter Rating		72	71.50	76.50	87.50	84.50	89.50	90	82
		Joint Orientation Rating		-5	-10	-10	-10	-5	-5	-5	-5
		Total Rating		67	61.50	66.50	77.50	79.50	84.50	85	77
		Rock Mass Class		II(Good)	II(Good)	II(Good)	II(Good)	II(Good)	I(V.Good)	I(V.Good)	II(Good)
	B	Joint Orientation Rating		-7	-15	-15	-15	-7	-7	-7	-7
		Total Rating		65	56.50	61.50	72.50	77.50	82.50	83	75
		Rock Mass Class		II(Good)	III(Fair)	II(Good)	II(Good)	II (Good)	I(V.Good)	I(V.Good)	II(Good)
Rock Mass Quality	A	Rock Quality Designation-RQD, %		100	92.46	94.52	91.73	100	98.74	100	100
		Joint Set Number, Jn		15	9	9	12	9	4	6	12
		Joint Roughness Number, Jr		1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
		Joint Alteration Number, Ja		1	1.5	1	1	1	1.5	1	1.5
		Joint Water Reduction Factor, Jw		1	1	1	1	1	1	1	1.5
		Stress Reduction Factor, SRF		1	1	1	1	1	1	1	1
		Tunnelling Index Q		10	10.27	15.75	11.47	16.67	24.68	25	16.67
		Rock Mass Quality		Good	Good	Good	Good	Good	Good	Good	Good

Note: a = Gouge Materials such that calcite, clay minerals, iron-oxides
b = No Gouge materials
c = Shear Zones
App = Application to; A = Tunnel, B = Foundation
Sys = System Type

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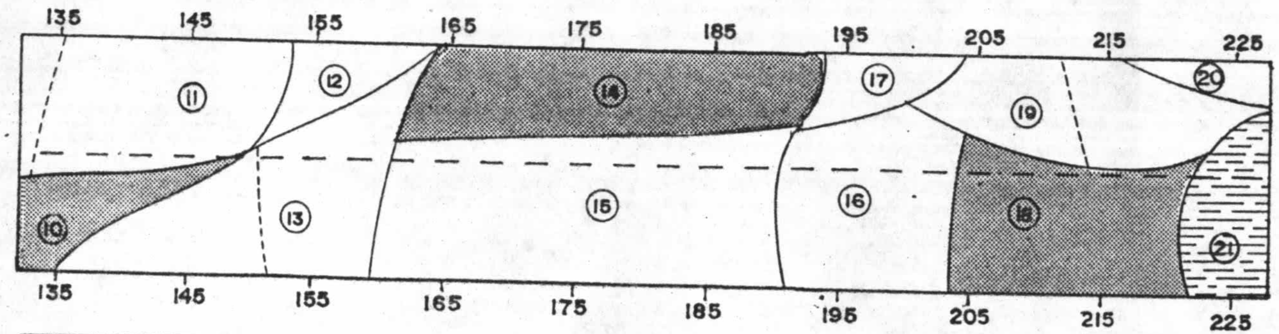
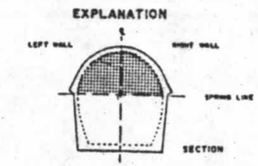
ROCK MASS CLASSIFICATION OF THE DIVERSION TUNNEL, CHIEW LARN DAM

UNDER SUPERVISION OF : DR. NOPADON MUANGNOICHAROEN
DR. WINNART SUNPERAYARNNONT

Figure 7.1

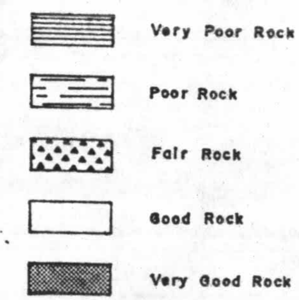
PURSUED BY : DANUPON TONNAYOPAS





Sys	App	Structural Regions / Rock Mass Parameters		9	10	11	12	13	14	15	16
Geomechanics Classification	A	Intact Rock Strength, MPa		3.47(7)	4.43(15)	3.51(7)	4.39(15)	5.44(15)	4.32(15)	4.29(15)	4.55(15)
		Rock Quality Designation-RQD, %		84.39(17)	88.35(17)	75.68(17)	92.93(20)	87.70(17)	85.51(17)	66.12(13)	90.60(20)
		Joint Spacing, m		0.2-0.6(15)	0.2-2(22.5	0.06-.6(10)	0.2-2(22.5	0.2-2(22.5	0.2-2(22.5	0.2-2(22.5)	0.2-2(22.5
		Persistence, m	Joint Conditions	2-3	2	2	2	2-3	2	2-3	2-3
		Filling		a	a	a	a	a	a	a	b
		Aperture, mm		2-4	2-3	2-10	2	2-3	2-3	2-3	3
		Roughness		2-4	3-4	3-4	2-4	2-4	3-4	2-4	4
		Average Rating		20	25	25	25	20	25	25	12
		Groundwater Inflow		Damp(10)	Dry(15)	Damp(10)	Dry(15)	Dry(15)	Dry(15)	Dry(15)	Dry(15)
		Parameter Rating		69	94.50	69	85	89.50	94.50	90.50	81.50
		Joint Orientation Rating		-2	-10	-5	-10	-10	-5	-10	-5
		Total Rating		67	84.50	64	75	79.50	89.50	80.50	76.50
		Rock Mass Class		II(Good)	I(V.Good)	II(Good)	II(Good)	II(Good)	I(V.Good)	II(Good)	II(Good)
	B	Joint Orientation Rating		-2	-15	-7	-15	-15	-7	-15	-7
		Total Rating		67	79.50	62	70	74.50	87.50	75.50	74.50
		Rock Mass Class		II(Good)	II(Good)	II(Good)	II(Good)	II(Good)	I(V.Good)	II(Good)	II(Good)
Rock Mass Quality	A	Rock Quality Designation-RQD, %		84.39	88.35	75.68	92.93	87.70	85.51	66.12	90.60
		Joint Set Number, Jn		9	12	9	9	12	9	15	12
		Joint Roughness Number, Jr		2	1.5	1.5	1.5	1	1.5	1.5	1
		Joint Alteration Number, Ja		1	0.75	1	1	3	1	1	4
		Joint Water Reduction Factor, Jw		1	1	1	1	1	1	1	1
		Stress Reduction Factor, SRF		1	1	1	1	1	1	1	1
		Tunnelling Index Q		18.75	14.72	12.61	15.49	2.40	14.25	6.61	1.89
		Rock Mass Quality		Good	Good	Good	Good	Poor	Good	Fair	Poor

Note: a = Gouge Materials such that calcite, clay minerals, iron-oxides
b = No Gouge materials
c = Shear Zones
App = Application to; A = Tunnel, B = Foundation
Sys = System Type



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ROCK MASS CLASSIFICATION OF THE DIVERSION TUNNEL, CHIEW LARN DAM

UNDER SUPERVISION OF Dr. NOPADON MUANGNOICHAROEN, Dr. ANWART SUMPERATANRONT

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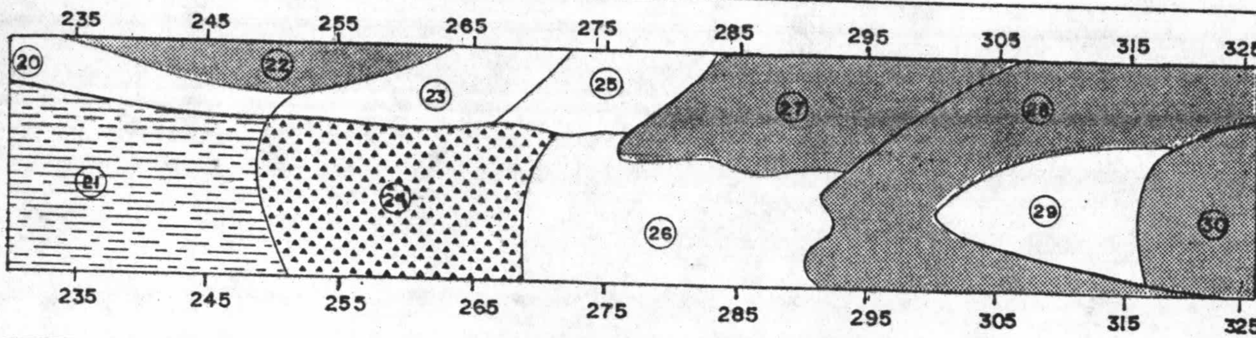
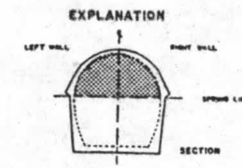
Figure 7.1 cont.





Sys	App	Structural Regions / Rock Mass Parameters		17	18	19	20	21	22	23	24
Geomechanics Classification	A	Intact Rock Strength, MPa		3.98(7)	4.95(15)	4.03(15)	4.57(15)	3.22(7)	2.15(7)	4.14(15)	5.31(15)
		Rock Quality Designation-RQD, %		88.60(17)	96.90(20)	100(20)	99.14(20)	25(3)	100(20)	78.14(17)	85.60(17)
		Joint Spacing, m		0.2-2(20)	0.2-2(20)	0.2-2(20)	0.2-2(20)	0.06(10)	0.2-0.6(20	0.2-2(20)	0.2-.6(10)
		Persistence, m	Joint Conditions	2-3	3	2	1-3	c	2	2-3	2-3
		Filling		a	a	a	b		a	a	b
		Aperture, mm		3-4	2	3	2-4		2	2-3	2-3
		Roughness		3-4	3-4	3-4	3-4		4	3-4	3-4
		Average Rating		12	25	20	20	0	25	20	15
		Groundwater Inflow		15	15	15	15	15	15	10	0
		Parameter Rating		71	95	90	90	35	87	82	54
		Joint Orientation Rating		-5	-5	-12	-10	-5	- 0	-10	-12
		Total Rating		66	90	78	80	30	87	72	45
		Rock Mass Class		II(Good)	I(V.Good)	II(Good)	II(Good)	IV(Poor)	I(V.Good)	II(Good)	III(Fair)
	B	Joint Orientation Rating		-7	-7	-25	-15	-7	-0	-15	-25
		Total Rating		64	88	65	75	28	87	67	32
		Rock Mass Class		II(Good)	I(V.Good)	II(Good)	II(Good)	IV(Poor)	I(V.Good)	II(Good)	IV(POOr)
Rock Mass Quality	A	Rock Quality Designation-RQD, %		88.60	96.90	100	99.14	25	100	78.14	85.60
		Joint Set Number, Jn		15	9	12	9	20	2	12	9
		Joint Roughness Number, Jr		1.5	1.5	1.5	1.5	1	1	1.5	1.5
		Joint Alteration Number, Ja		1	1	1	3	6	3	1	3
		Joint Water Reduction Factor, Jw		1	1	1	1	1	1	0.66	0.2
		Stress Reduction Factor, SRF		1	1	1	1	5	1	1	1
		Tunnelling Index Q		8.86	16.15	12.50	5.51	0.04	16.67	6.45	0.95
		Rock Mass Quality		Fair	Good	Good	Fair	Exc-Ext P	Good	Fair	Very Poor

Note: a = Gouge Materials such that calcite, clay minerals, iron-oxides
b = No Gouge materials
c = Shear Zones
App = Application to; A = Tunnel, B = Foundation
Sys = System Type

Very Poor Rock

Poor Rock

Fair Rock

Good Rock

Very Good Rock

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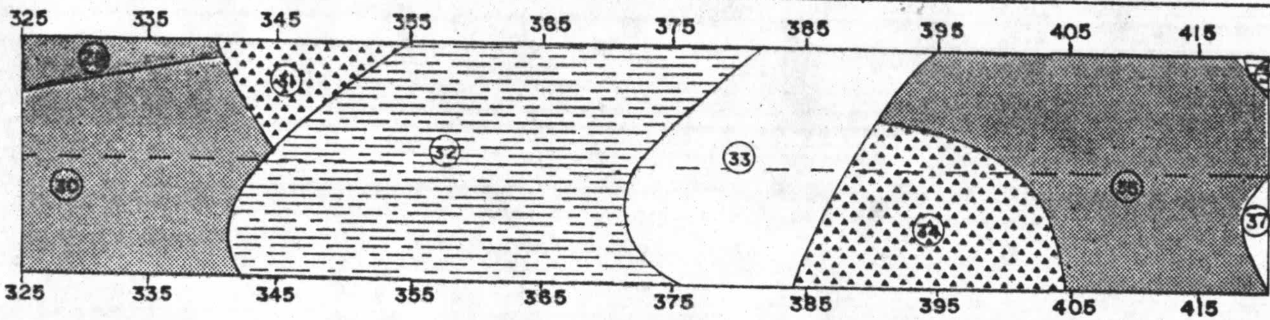
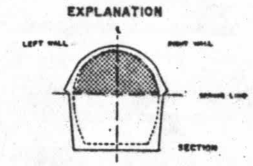
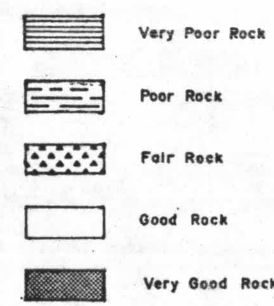
Figure 7.1 cont.







Sys	App	Structural Regions / Rock Mass Parameters		25	26	27	28	29	30	31	32
Geomechanics Classification	A	Intact Rock StrengtH, MPa		2.96(7)	4.86(15)	4.86(15)	4.12(15)	4.03(15)	4.26(15)	4.57(15)	4.43(15)
		Rock Quality Designation-RQD, %		100(20)	89.32(17)	97.29(20)	95.23(20)	90.70(20)	98.39(20)	70.13(13)	0-25(3)
		Joint Spacing, m		0.2-2(20)	0.2-2(22.5)	0.2-2(22.5)	0.2-2(22.5)	.06-.6(10)	0.2-.6(20	.6-2(15)	<.06(10)
		Persistence, m	Joint Conditions	2-3	2-3	2	2	2-3	2	2	
		Filling		a	a	a	a	a	a	a	
		Aperture, mm		2	3	2-3	2	3	2	2-3	o
		Roughness		4	3-4	4	3	4	3-4	3-4	
		Average Rating		25	20	25	25	25	25	12	0
		Groundwater Inflow		15	4	15	15	4	15	15	4
		Parameter Rating		87	78.50	97.50	97.50	74	95	70	32
		Joint Orientation Rating		-10	-10	-5	-10	-5	-5	-12	-5
		Total Rating		77	68.50	92.50	87.50	69	90	57	27
		Rock Mass Class		II(Good)	II(Good)	I(V.Good)	I(V.Good)	II(Good)	I(V.Good)	III(Fair)	IV(Poor)
	B	Joint Orientation Rating		-15	-15	-7	-15	-7	-7	-25	-7
		Total Rating		72	63.50	90.50	82.50	67	88	45	25
		Rock Mass Class		II(Good)	II(Good)	I(V.Good)	I(V.Good)	II(Good)	I(V.Good)	III(Fair)	IV(Poor)
Rock Mass Quality	A	Rock Quality Designation-RQD, %		100	89.32	97.29	95.23	90.70	98.39	70.13	0-25
		Joint Set Number, Jn		4	9	9	9	4	4	15	20
		Joint Roughness Number, Jr		1.5	1	1	1.5	1	1	1.5	1
		Joint Alteration Number, Ja		3	1	1	1	1	10.75	1	6
		Joint Water Reduction Factor, Jw		1	1	1	1	1	1	1	1
		Stress Reduction Factor, SRF		1	1	1	1	1	1	1	2.5
		Tunnelling Index Q		12.50	9.92	10.81	15.88	22.68	32.80	7.01	0.08
		Rock Mass Quality		Good	Fair	Good	Good	Good	Good	Fair	Exp Poor

Note: a = Gouge Materials such that calcite, clay minerals, iron-oxides
b = No Gouge materials
c = Shear Zones
App = Application to; A = Tunnel, B = Foundation
Sys = System Type

Figure 7.1 cont.

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ROCK MASS CLASSIFICATION OF THE DIVERSION TUNNEL, CHIEW LARN DAM

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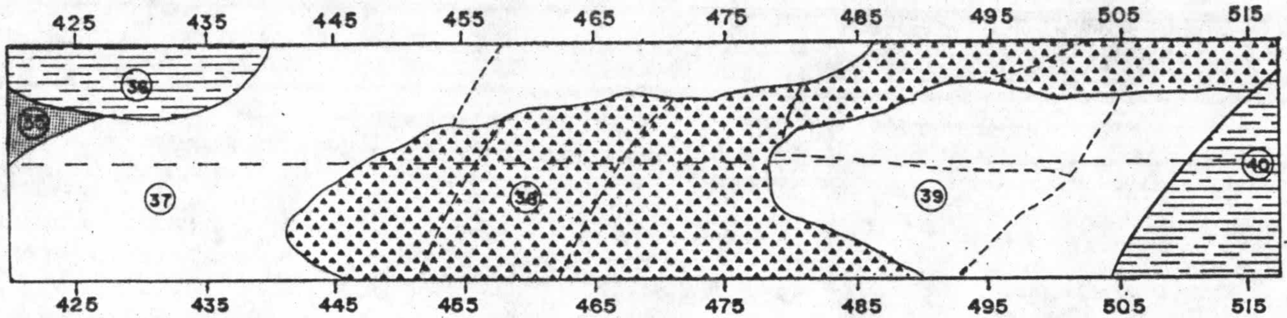

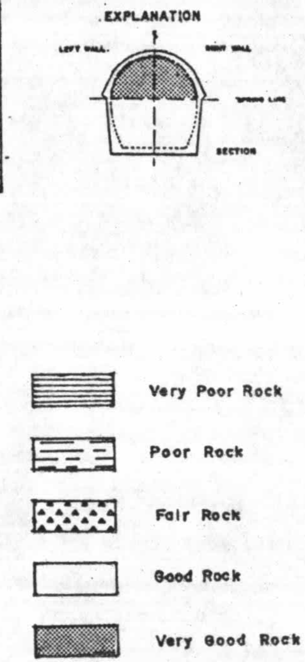

Sys	App	Structural Regions / Rock Mass Parameters	33	34	35	36	37	38	39	40
Geomechanics Classification	A	Intact Rock Strength, MPa	4.78(15)	5.61(15)	5.35(15)	5.74(15)	4.56(15)	5.68(15)	4.79(15)	4.62(15)
		Rock Quality Designation-RQD, %	82.02(17)	25-50(8)	76.72(17)	0-25(3)	91.28(20)	20-30(8)	79.51(17)	15-25(3)
		Joint Spacing, m	0.2-.6(10)	.06-.2(10)	.2-.6(15)	<0.06(10)	0.2-2(22.5	.06-.2(10	.2-2(22.5)	<0.06(10)
		Joint Conditions: Persistence, m	2		2		2		1-2	
		Joint Conditions: Filling	a	c	a	c	a	c	a	c
		Joint Conditions: Aperture, mm	2-3		2-3		2-3		2-4	
		Joint Conditions: Roughness	3-4		3-4		3-4		3-4	
		Joint Conditions: Average Rating	20	6	25	0	20	12	25	0
		Groundwater Inflow	15	15	15	15	15	15	0	7
		Parameter Rating	77	54	87	40	92.50	60	79.50	35
		Joint Orientation Rating	-12	-5	-5	-10	-10	-10	-5	-2
		Total Rating	65	49	82	30	82.50	50	74.50	33
		Rock Mass Class	II(Good)	III(Fair)	I(V.Good)	IV(Poor)	II(Good)	III(Fair)	II(Good)	IV(Poor)
	B	Joint Orientation Rating	-25	-7	-7	-15	-15	-15	-7	-2
		Total Rating	52	47	80	25	77.50	45	72.50	31
		Rock Mass Class	III(Fair)	III(Fair)	II(Good)	IV(Poor)	II(Good)	III(Fair)	II(Good)	IV(Poor)
Rock Mass Quality	A	Rock Quality Designation-RQD, %	82.02	25-50	76.72	0-25	91.28	20-30	79.51	15-25
		Joint Set Number, Jn	9	15	9	20	9	15	15	15
		Joint Roughness Number, Jr	1.5	1.5	1.5	1	1.5	3	1.5	1
		Joint Alteration Number, Ja	1	6	1	6	3	3	1	4
		Joint Water Reduction Factor, Jw	1	1	1	1	1	1	0.33	1
		Stress Reduction Factor, SRF	1	2.5	1	2.5	1	2.5	1	5
		Tunnelling Index Q	13.67	0.33	12.79	0.08	5.07	0.80	2.62	0.08
		Rock Mass Quality	Good	Very Poor	Good	Exp Poor	Fair	Very Poor	Poor	Exp Poor

Note: a = Gouge Materials such that calcite, clay minerals, iron-oxides
b = No Gouge materials
c = Shear Zones
App = Application to; A = Tunnel, B = Foundation
Sys = System Type

Figure 7.1 cont.

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Dr. WINMART SUMPERAYARNNONT

PURSUED BY : DANUPON TONNAYOPAS

areas tend to have a lower rock mass quality ranging from very poor to poor. The distribution of the rock mass quality is presented by the histograms illustrated in Figure 7.2.

The evaluation of the rock mass quality depends on the classification parameters with a personal judgement. A discussion on these parameters is below.

7.1.1 Strength of Intact Rock Material

In order to determine the uniaxial compressive strength of the intact rocks indirectly in the field, the Schmidt rebound hammer test and point-load index test were applied. The wall strength of every joint in each point set in a structural region was determied using a Schmidt rebound hammer type L, and the average uniaxial compressive strength and their deformation moduli were obtained. Furthermore, the irregular-shape samples were randomly collected from each structural region for the testing in the field-laboratory at the site.

The effect of anisotropy, due to the orientation of cleavage in the specimens, on the uniaxial compressive strength of the rocks was also determined.

The average point-load strength indices, excluding the extreme values in MPa unit, were used to estimate the strength of the rock elements along the diversion tunnel and at its portals.

7.1.2 Rock Quality Designation (RQD)

The RQD value along the diversion tunnel varies due to the structural distribution. Since only a few borehole-rock cores were available along the tunnel length, more available along the main dam axis foundation, RQD was estimated from the number of joints per cube meter (Jv), when the total number of joints of each set are counted perpendicularly to the relevant joint set. The RQD value was then computed using Equation 6.2 mentioned previously. The RQD values in forty structural regions along the diversion tunnel, together with other classification parameters, are summarized in Figure 7.1. It can be noted that the quality of rock mass depends on the values of RQD. The good to excellent quality (RQD = 75-100 %) slighty-jointed rock masses are predoninant in the are while the very-poor to poor quality orushed or sheared rock masses are occasionally found (Figure 7.3).

In this study, RQD was estimated from the volumetric joint count on the exposures instead of determined from the borehole rock cores. Thus, in order to check the validity of this estimation, RQD from the previous core logs were plotted against the upper and lower limit of joint frequencies measured per meter in the regular length of RQD determination. The technique is to approximate the joint frequencies measured per unit length of borehole to be equivalent to the volumetric joint count (Jv) obtained by the linear empirical regression. The results are as follow also shown in Figure 7.4.

$$\text{Upper limit} \quad RQD = 105 - 3.5Jv \quad \ldots\ldots\ldots\ldots \quad (7.1)$$

$$\text{Mean} \quad RQD = 105 - 4.5Jv \quad \ldots\ldots\ldots\ldots \quad (7.2)$$

$$\text{Lower limit} \quad RQD = 105 - 5.2Jv \quad \ldots\ldots\ldots\ldots \quad (7.3)$$

The relationship used to estimate in the field RQD (ISRM, 1981) is expressed in equation 6.2

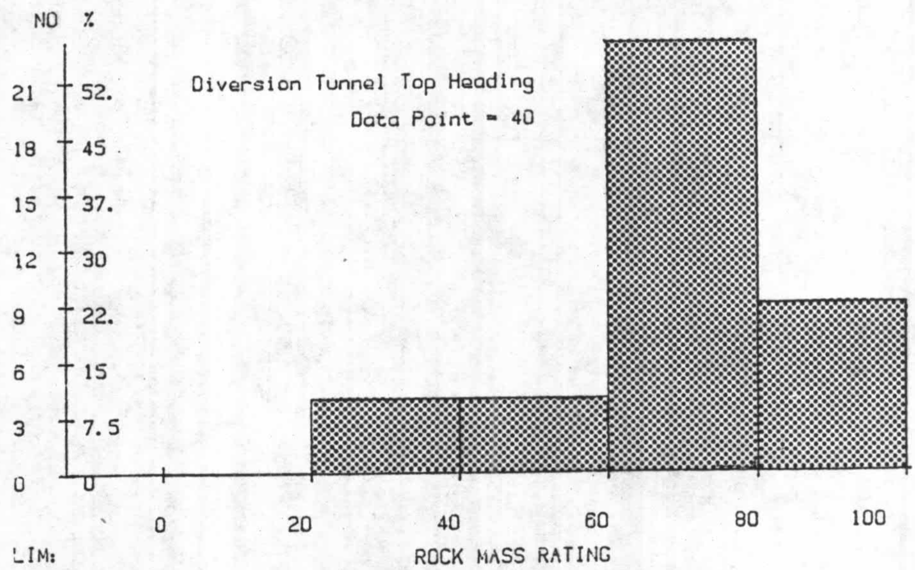


Figure 7.2 Histogram showing distribution of rovk mass rating along the diversion tunnel.

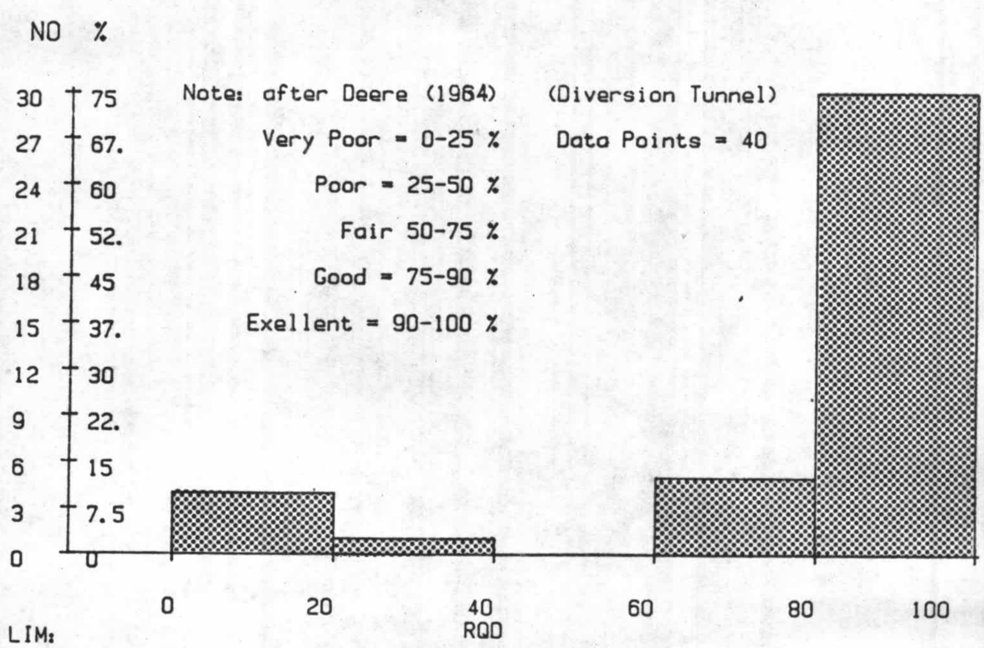


Figure 7.3 Histogram shwing distribution of RQD derived from volumetric joint count (J_v).

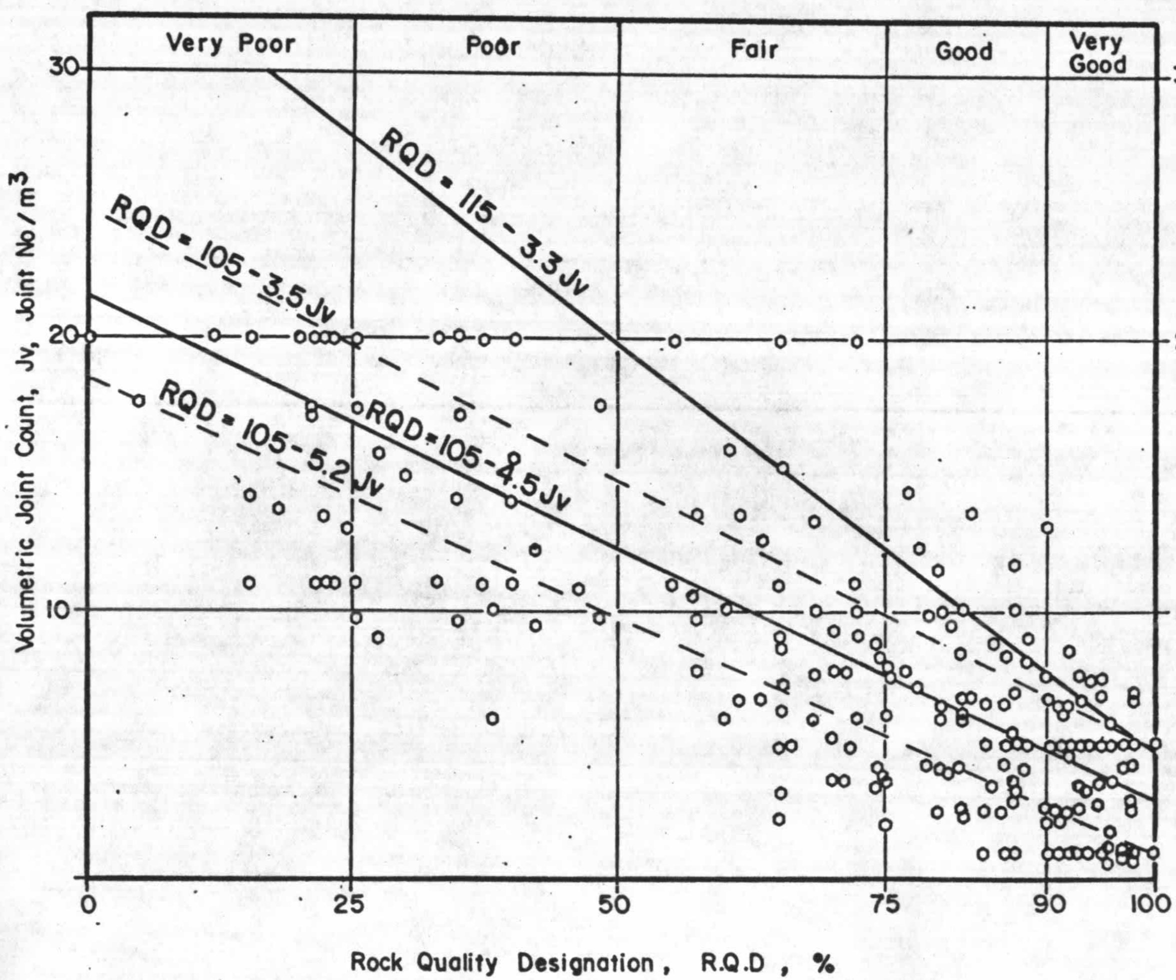


Figure 7.4 Correlation between rock quality designation (RQD) and volumetric joint count (Jv).

The above comparison indicates that RQD determined from the borehole rock core is slightly less than that estimated in the exposures. This might be because the number of joints per unit length of the borehole core are measured in only one direction while they were measured in 3 dimensions in an exposure. So some joints are not found in the direction of the drillhole axis, while for RQD estimated on the exposures, all joints are counted perpendicular to the relevant sets. Thus more joints were observed.

7.1.3 Joint Spacings and Block Sizes

The joint spacing was measured using a 2-meter measuring tape. Because of a limited vertical distance of the sidewalls to the top heading to the diver.ion tunnel, about 5.6 m high, and the predominance of the vertical joints, only the joint continuity of more than 1.5 m was studied. The majority of these joints are wide to moderately wide (Figure 7.5). The relative block size and shape was also determined using the histogram of the value of the volumetric joint counts, Jv (Figure 7.6). The histogram indicated that the block dimensions are generally medium-sized (Jv = 3-10) blocky shaped.

It should be noted that the spacing of joint was measured relative to block size as mentioned above. These results were determined only from the ordinary jointed rock masses excluding the crushed zones which were counted as the very closely-spaced jointed zones.

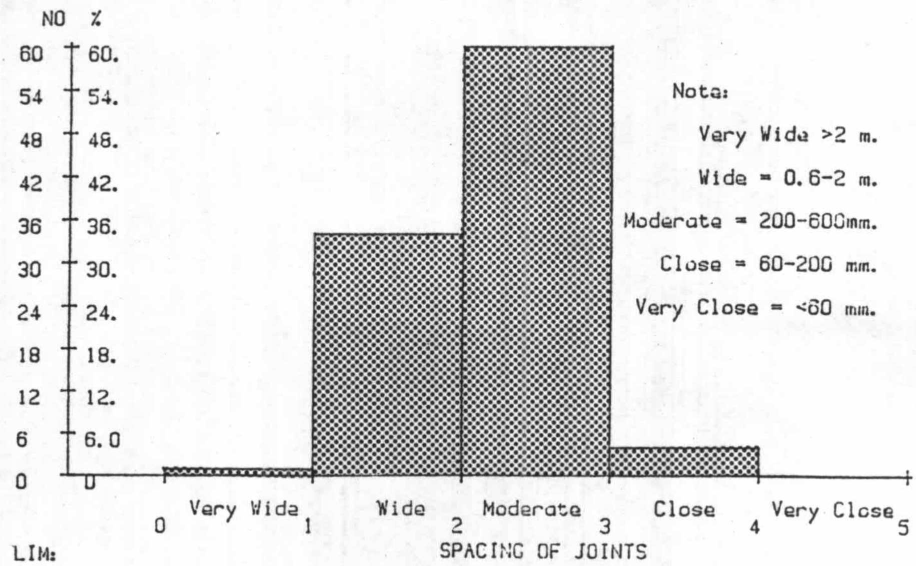


Figure 7.5 Histogram showing distribution of joint spacing.

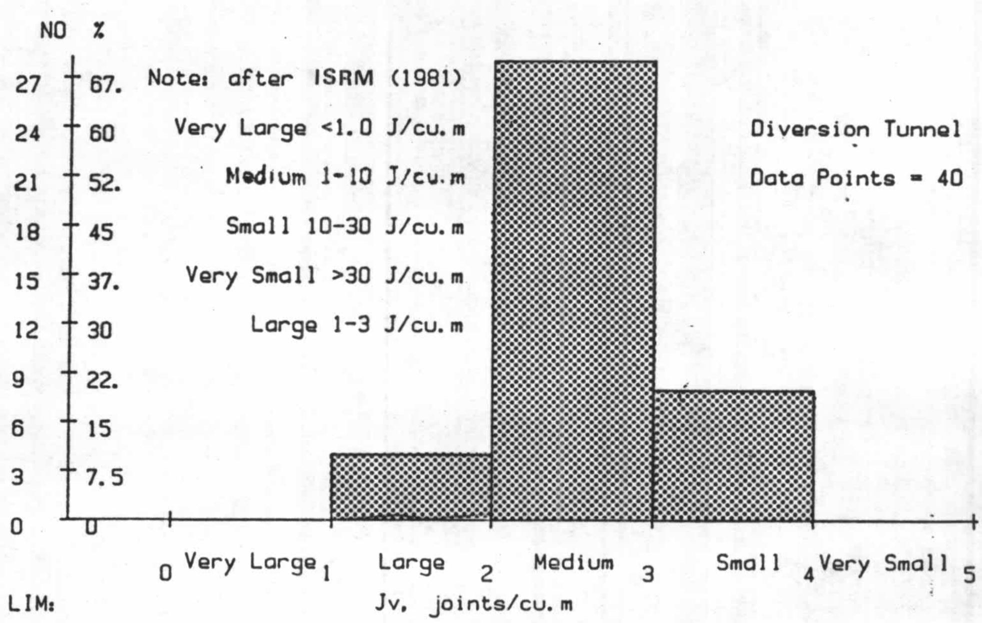


Figure 7.6 Histogram showing distribution of block size.

7.1.4 Joint Conditions

The condition of joints is defined as the joint continuity, separation, surface roughness, infilled materials and joint wall alteration. IN this study, the joint persistence is generally low to medium (1-3 m to 3-10 m) with the independence of sheared planes (shear joints and faults) which have high to very high persistence (>10 m) (Figure 7.7).

The separation of joints was observed directly Figure 7.8, using the description of this parameter given in the CSIR Geomechanics classisifcation input-data form (Appendix A-3). The histogram of jointsurface roughness indicates that the parameter varies from smooth planar surface to smooth undulating surface, the former is predominant (Figures 7.9 and 7.10).

The filling materials were determined in terms of type, thickness, strength and seepage condition. The seepage condition was determined using the description of the filled discontinuities as suggested by ISRM (1981), and is exhibited in Table 6.13.

Brekke and Howard (1973) proposed a discrimination of discontinuity infilling materials (on the behavior of tunnel stability) into five classes (of fault filling or gouge materials as listed in Table 7.1. The swelling clay filling are potentially the most troublesome class. Other clay fillings, the sheet-silicate minerals, e.g. chlorite, and other micas, talc, serpentine, etc., and graphite, can introduce an extremely low shear strength, particularly if the thickness of filling is greater than the roughness amplitude.

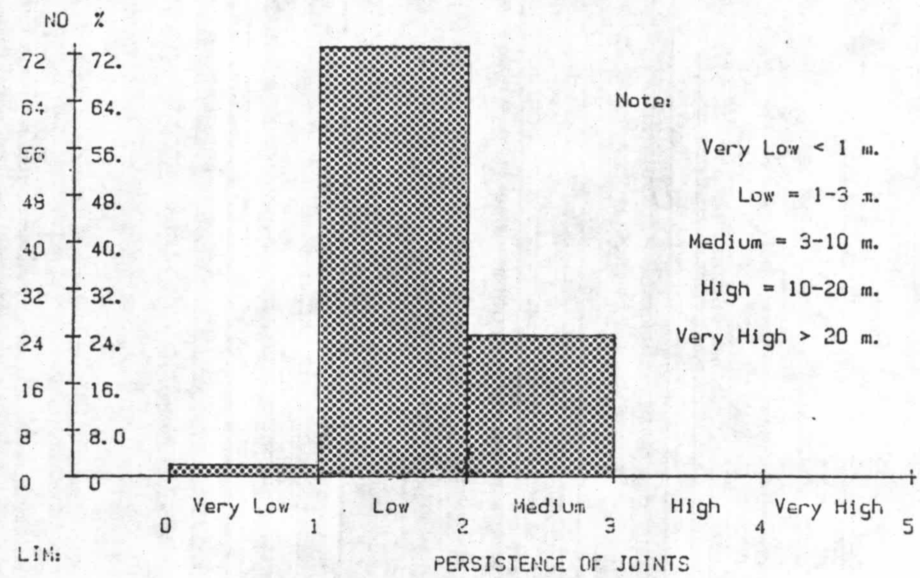


Figure 7.7 Histogram showing distribution of joint persistence.

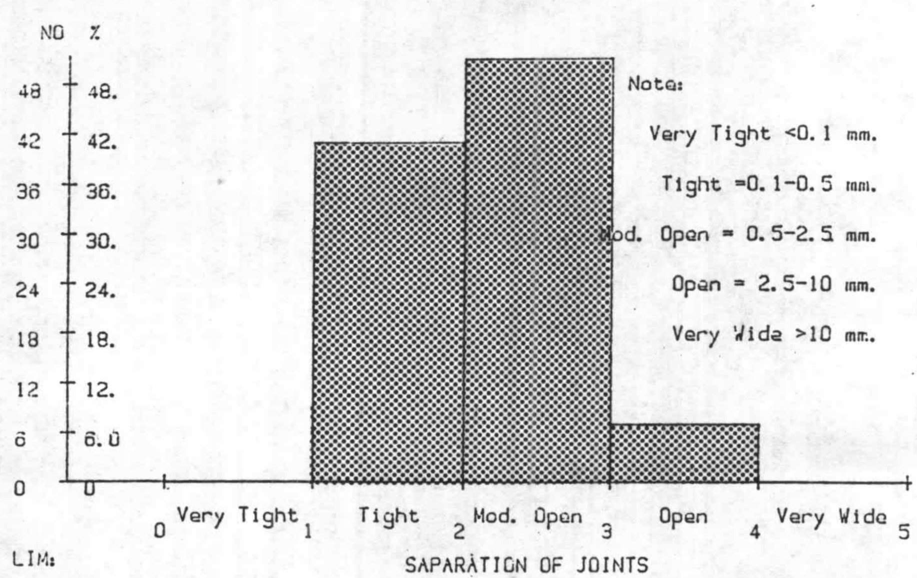


Figure 7.8 Histogram showing distribution of joint spacing.

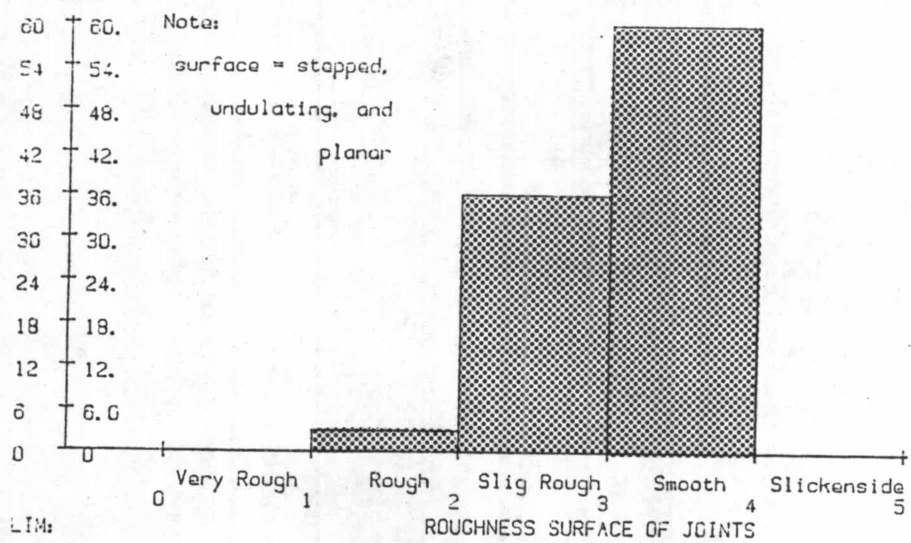


Figure 7.9 Histogram showing distribution of joint roughness surface.

Figure 7.10 Close up of joint conditions on the left wall between the extensometer Section no. 1 and no. 2.

Table 7.1 Classes of discontinuity infilling upon the behavior of tunnel (after Brekke and Howard, 1973)

Dominant Material in Gouge	Potential Behavior of Gouge Material in Tunnels	
	At Tunnel Face	Later
Swelling clay	Free swelling, sloughing. Swelling pressure and squeeze on shield	Swelling pressure and squeeze against support or lining; free swell with down-fall or wash-in if lining inadequate
Inactive clay	Slaking and sloughing caused by squeeze; heavy squeeze under extreme con conditions	Squeeze on supports of lining where unprotected slaking and sloughing due to environmental changes
Chlorite, talc, graphite or serpentine	Ravelling	Heavy loads may develop due to low strength, in particular when wet
Crushed rock fragments or sandlike gouge	Ravelling; standup time may be extremely short	Loosening loads on lining; running and ravelling if unconfined
Porous or flaky calcite, gypsum	Favourable condition	May dissolve, leading to instability of rock mass

The crushed rock fillings, and incomplete quartz or calcite fillings offer a potentially high permeability. They can erode and undermine the adjacent rock in the exoeptional cases. This is particularly troublesome in an unlined water tunnel (Goodman, 1976).

In the diversion tunnel at the Chiew Larn dam project area. The infilled materials are generally veneer of calcite, limonite and very small amount of pyrite and crystalline calcite.

The joint wall condition is largely unaltered with only some surface staining. It can be concluded that the joints condition of the rock masses in the investigation area are commonly good enough for the excavation works except in the crushed or sheared Zone where some problems might be anticipated.

7.1.5 Groundwater Inflow

The groundwater inflow was estimated by uning the seepage description (Table 6.14) suggested by ISRM (1981). This seepage description was also correlated with the description of groundwater condition in the Bieniawski's (1979) input-data form. The frequency of jointing of the rock masses in the study area is rather high, thus tends to increase the permeability of the rock masses. These may be the cause of a continucus groundwater inflow into the tunnel throughout the rainy season. The seepage of groundwater usually occurs in the crushed zones and in the highly persistent joints with open separations. This is a stability problem in the diversion tunnel, rock foundation works as well as the slopers.

Contrarily, the groundwater inflow during the dry season is generally small and it may not affect the operation except in a deep excavation opening such as power house, etc.

7.1.6 Joint Orientations and Inclinations

A Clar geological compass was used to measure the attitude of the discontinuities. The pole concentation and representative planes of joint sets as well as the angular relationship of these representative planes and the tunnel axis were determined in an equalarea stereographic net plot (Figure 7.11). The favourability adjustment in joint orientation and inclination for the tunnel was then determined and analysed for the stability of the construction.

7.2 NGI Tunnelling Quality Index

Barton et al. (1974) proposed an index quality for the determination of the tunnelling quality of a rock mass. This system is based on the following paremeters.

(i) Rock quality designation (RQD)

(ii) Joint set number (Jr)

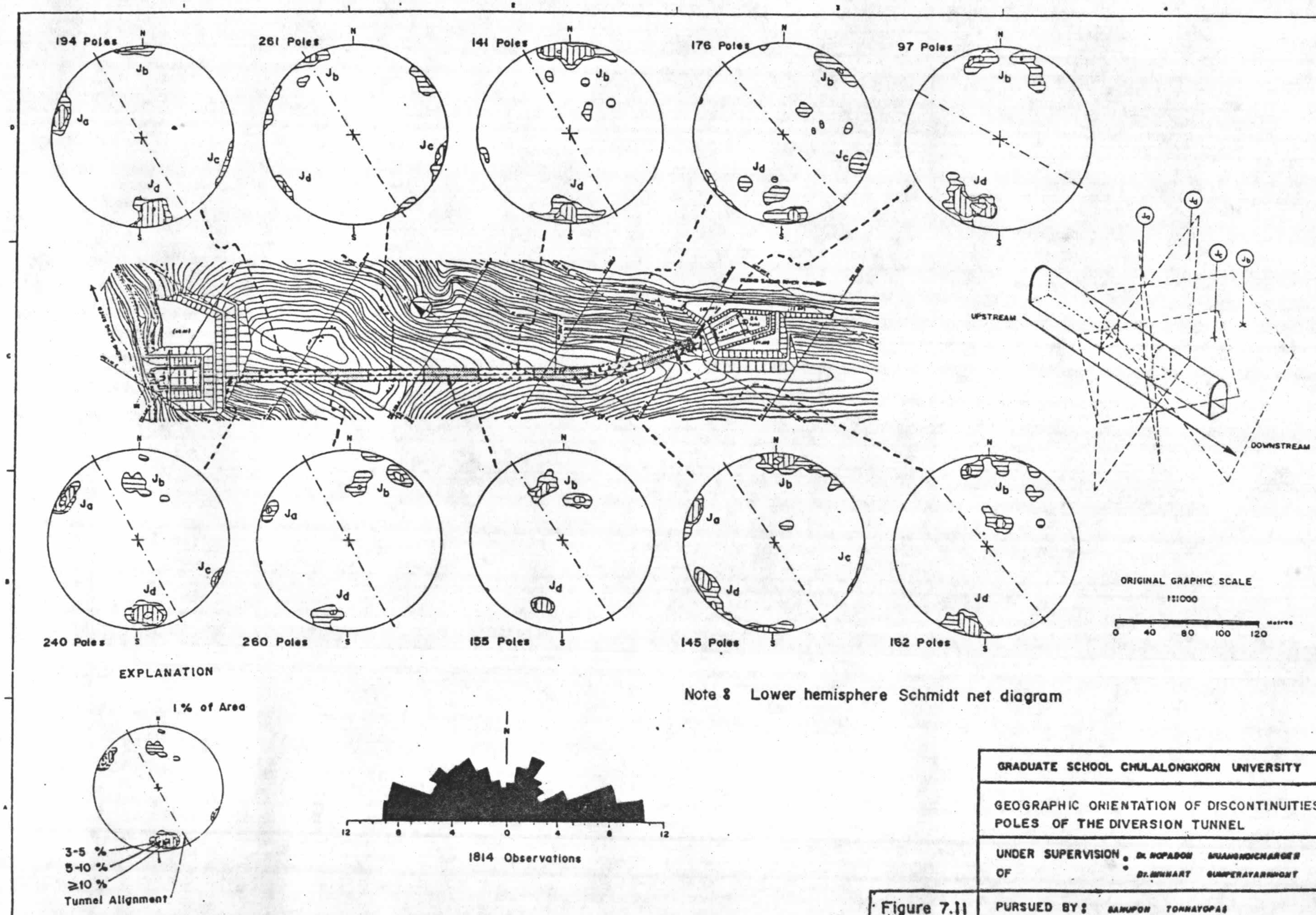
(iii) Joint roughness number (Jr)

(iv) Joint alteration number (Ja)

(v) Joint water reductuon factor (Jw)

(vi) Stress reduction factor (SRF)

These parameters are determined from the joint survey data which follows the description given by Barton (1976) as exhibited in Table 6.15. The determination of RQD, joint set number (Jn), joint



194 Poles
261 Poles
141 Poles
176 Poles
97 Poles
240 Poles
260 Poles
155 Poles
145 Poles
162 Poles
N
S
Ja
Jb
Jc
Jd
UPSTREAM
DOWNSTREAM
ORIGINAL GRAPHIC SCALE
1:1000
0 40 80 100 120
metres
EXPLANATION
1 % of Area
3-5 %
5-10 %
≥10 %
Tunnel Alignment
12 8 4 0 4 8 12
1814 Observations
Note : Lower hemisphere Schmidt net diagram
GRADUATE SCHOOL CHULALONGKORN UNIVERSITY
GEOGRAPHIC ORIENTATION OF DISCONTINUITIES POLES OF THE DIVERSION TUNNEL
UNDER SUPERVISION OF :
PURSUED BY : SANUPON TONNAYOPAS

Figure 7.11

roughness (Jr), joint alteration number (Ja) and joint water reduction factor (Jw) are performed by a similar procedure to that of Bieniawski's (1979) classification. In the heavily jointed rock masses or shattered zones and crushed or sheared zones the above parameters are assessed by assuming no rock wall contact when sheared.

The assessment of these controlling factors on the rock mass behavior is difficult, particularly in a heterogenous rock mass i.e., pebbly graywackes and subarkosic sandstones on the portal slopes. It must also be kept in mind that an index is only an average value predicting the rock mass behavior and there can be a great variation in any parameter measured in the diversion tunnel.

7.2.1 Determination of RQD

As a limited RQD information was available, the concept of a fracture frequency was used. The results of a linear joint survey parallel to the diversion tunnel traverse line were used to calculated the number of joints per cubic meter, Jv. The value substituted into Barton et al.'s (1974) formula, RQD = 115-3.3Jv, which gives the reasonable RQD values.

7.2.2 Joint Sets Number

This is defined relative to the size and shape of the blocks and as such is a problem of excavation. The problem is to account for a variability in the joint sets since in each locality anumber of values observed can be different from stereographic interpretation.

7.2.3 Measurement of Groundwater Inflow

In the Chiew Larn tunnel top heading, a crude but effective mean of estimating the groundwater inflow was to record the time for a container of known dimensions to be filled. Knowing the area of the tunnel from which water was being pumped out, the volume of flow per unit length (of the tunnel) was estimated.

Generally, the tunnel excavation seemed to be dry and no groundwater problems were anticipated at this stage, although a series of perched water tables were suspected in the pebbly gray-wackes. It was expected that the local flows would diminish with time.

7.2.4 Joint Roughness and Joint Alteration Number

Both parameters to be used in Barton's (1974) classification require a considerable experience of the observer to be determined accurately since they are assessed by a visual examination. But to place then into any category requires a full understanding of the difference betweenthose categories.

7.2.5 Stress Reduction Factor

The last parameter, the stress reduction factor (SRF), was also determined separately for the heavily jointed rock mass, shattered zones, crushed or shear zones, and the ordinary jointed rock masses. This complex factor, also for Barton et al.'s (1974) classification, involves a considerable degree of sophistication and requires at least some basic experience. Eighteen detailed divisions are applicable. There is a significant range for the SRF values, yet the decision as to the type of rock stress action and its magnitude is difficult to determine especially on a rapid visual inspection. In the rock stress problem of the competent rock, the major principal stress (σ_1) was assumed to be equal to the load of overburden per unit area. Thus, the ratio of compressive strength (σ_c) obtained from the point-load index tests and the major principal stress (σ_1) could be estimated.

7.3 Results and Discussions

NGI tunnelling quality indices of 40 structural regions along the tunnel were performed and the results are depicted in Figure 7.1.

The results exhibited that the rock mass quality vary from extremely poor to good, but fair and good quality rock masses are eminent as demonstrated by a plot of histogram in Figure 7.12. The rock quality index values obtained can be used to predict the maximum unsupported excavation spans and the support requirements (Figure 7.13). The results indicated that the maximum unsupported span varied from 5.6 to 24 m.

Rock mass rating results are also illustrated in Figure 7.2, following the Geomechanics Classification guide for excavation and support in rock tunnels in Table 7.2. The rating results indicate that this tunnel generall requires no support except for the occasional spot bolting in crown. The bolts are suggested to be 3 m long with a space of 2.5 m and with wire mesh.

It was found that the CSIR Geomechanics Classification was better applicable in the field than Barton's Q-index. The partially

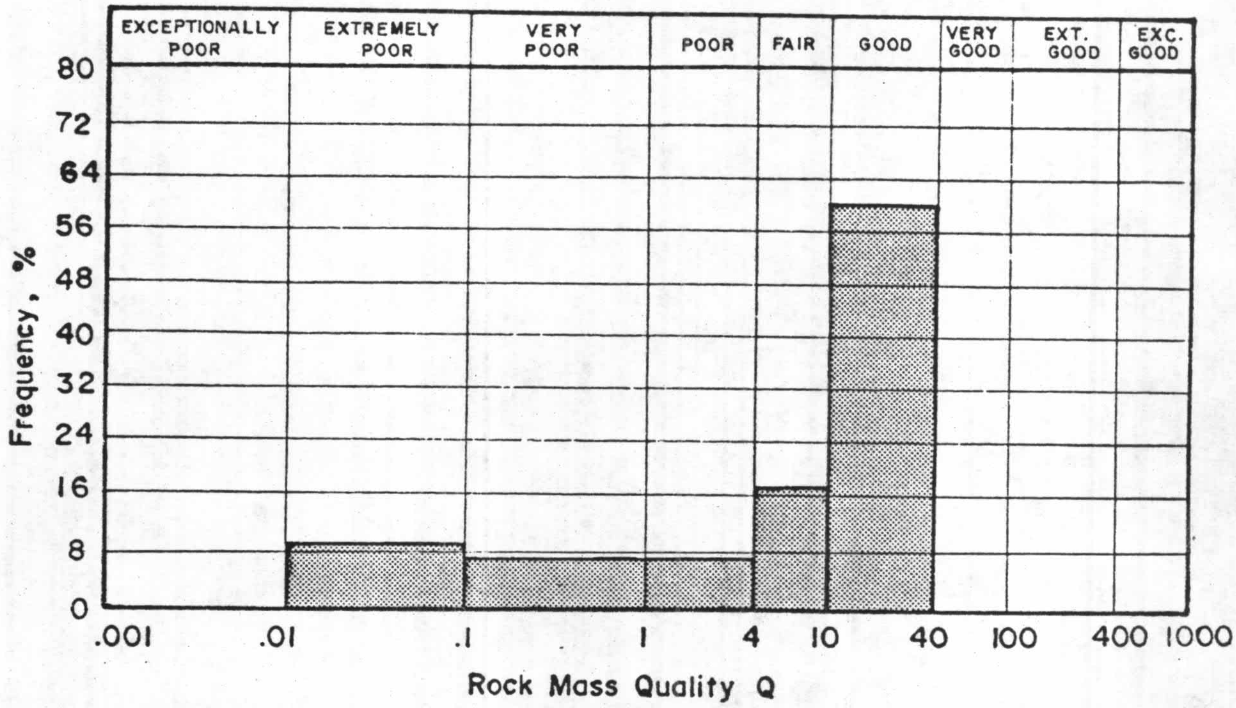


Figure 7.12 Histogram of rock mass quality Q showing distribution of rating.

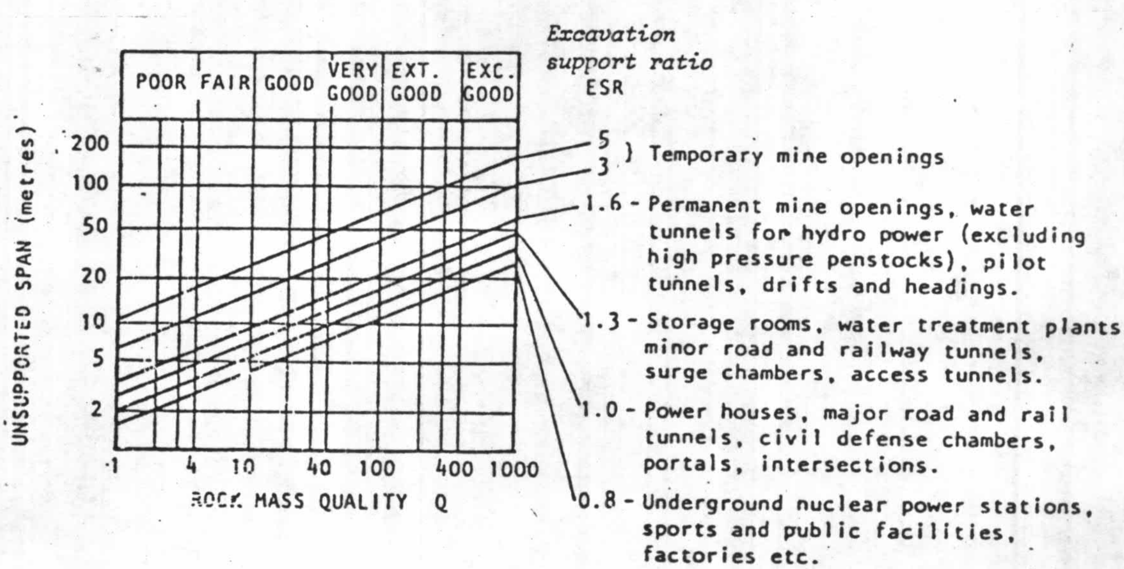


Figure 7.13 Recomended maximum unsupported excavation spans for different quality rock mass (after Barton, 1976).

Table 7.2 Geomechanics guide for excavation and support in rock tunnels (after Bieniawski, 1979).

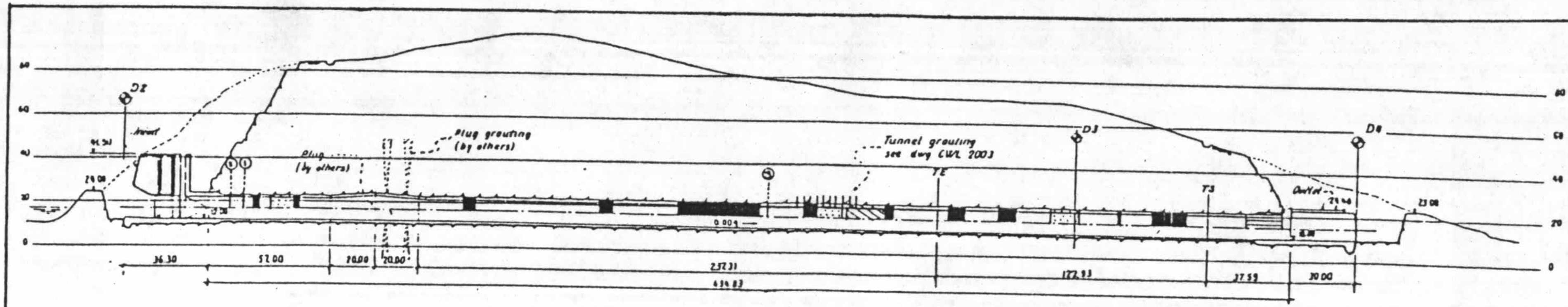
Rock mass class	Excavation	Support		
		Rockbolts (20 mm dia., fully bonded)	Shotcrete	Steel sets
Very good rock I RMR: 81-100	Full face. 3 m advance	Generally no support required except for occasional spot bolting		
Good rock II RMR: 61-80	Full face. 1.0-1.5 m advance Complete support 20 m from face.	Locally bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None
Fair rock III 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 m - 2 m in crown and walls with wire mesh in crown.	50 - 100 mm in crown and 30 mm in sides.	None
Poor rock IV 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.
Very poor rock V RMR: <20	Multiple drifts. 0.5 - 1.5 m advance in top heading. Instal 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 fore-poling if required. Close invert.

Note : shape: horseshoe; width; 10 m; vertical stress: below 25 MPa; construction: drilling and blasting reflects the way by which the basic factors are defined. Bieniawski (1979) inflexibly classed the rock mass divisions using a straight forward definition. With Barton's system, several points are noted. Firstly, the scales defining the parameters is generally more sophisticated. This is reflected in the range of values assigned and the number of divisions produced. It is by no means an error, but an increased sophistication implies a fuller understanding of the rock mass behavior, therefore, a considerable field experiencs. Besides, in defining his parameters, Barton (1975) assumed a considerable amount of engineering geological knowledge, which was not available at the early stages of the Chiew Larn project, when it is required to apply to the rock mass classification.

It is also worth nothing that despite the increased sophistication of the Barton's system there is no significant difference between the ratings made by it and the CSIR Geomechanics classification.

7.4 Rock Mass Detormation Measurements

The measurements of the movement of the rock walls and roof of the tunnel were begun when the excavation of the "top heading" was almost complete. The extensometers were anchored in the rock close to the working face as the construction would permit, generally in the range of 1 m. Immediately after the installation of these extensometers the movement was measured as displayed in Figures 7.14 and 7.15. The rock movement was re-measured periodically until the



LONGITUDINAL PROFILE

EXPLANATION

① EXTENSOMETER SECTION 1 (5 SET) AT STATION 56.00 m.

② EXTENSOMETER SECTION 2 (3 SET) AT STATION 46.00 m.

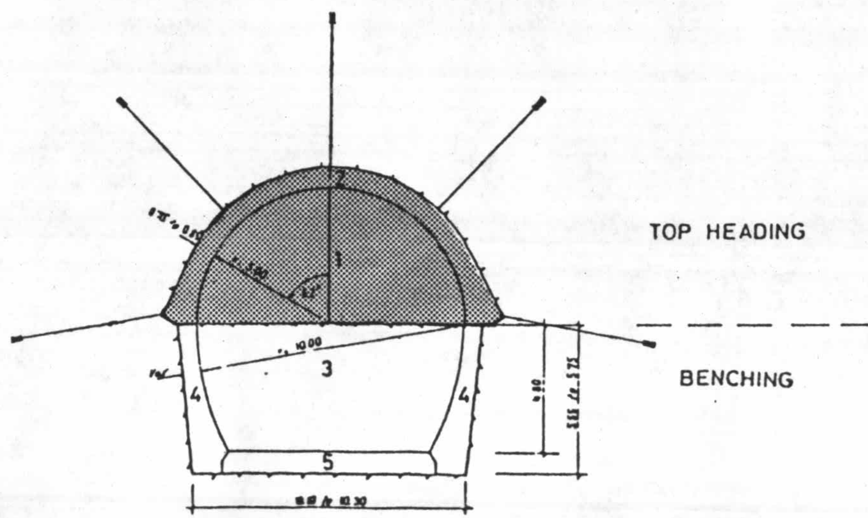
1 Excavation

2 Concrete lining

3 Excavation

4 Wall lining

5 Floor lining



SECTION

GRAPHIC SCALE

GRADUATE SCHOOL CHULALONGKORN UNIVERSITY

MONITORING ARRANGEMENTS AT MEASURING STATION OF THE DIVERSION TUNNEL

UNDER SUPERVISION OF : Dr. NOPADON MUANGNOICHAROEN
Dr. WINNART SUMPERAYARNMONT

Figure 7.14

PURSUED BY : DANUPON TONNAYOPAS

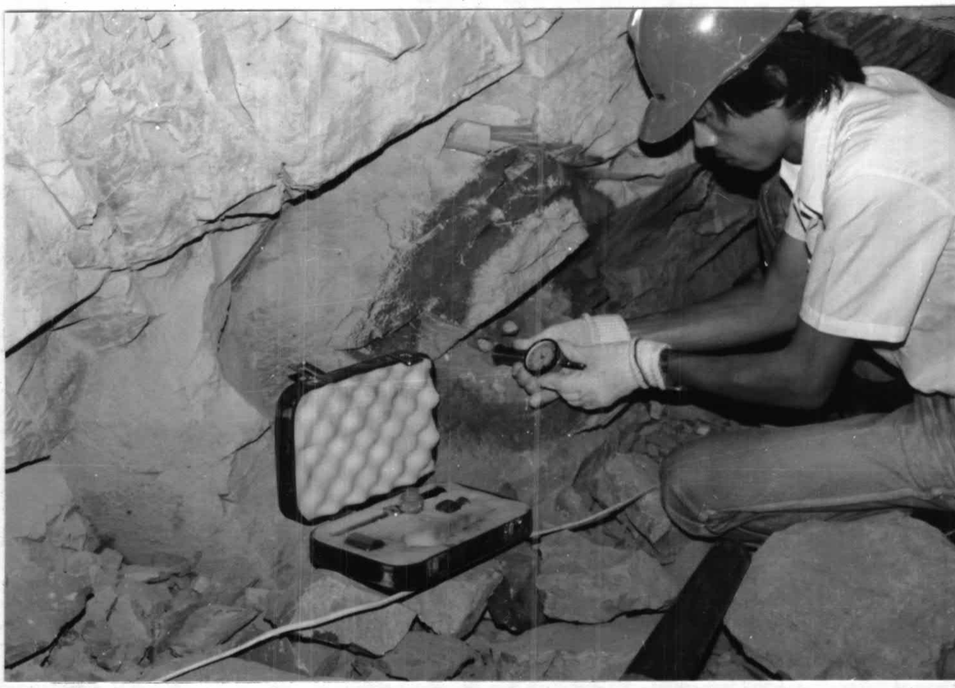


Figure 7.15 Dial gage mounted on the borehole extensometer in pebbly graywackes of diversion tunnel

concrete lining was placed. The manual reading with dial gage was employed and after that the displacement was measured using an electrical remote reading.

Typical curves of the movement of the walls for three stations in the diversion tunnel are illustrated in Figures 7.16 to 7.18. In this instance the measured were started after each round (blast) for 20 m until the face is advanced sufficiently from the section. Thereafter, it was read daily for one week and weekly reading afterward. As the heading moved onwed onward, the measurement of the inward movements of the tunnel walls at the orown of section 1, at chanaige 46.50 m, on three installed extemsometers.

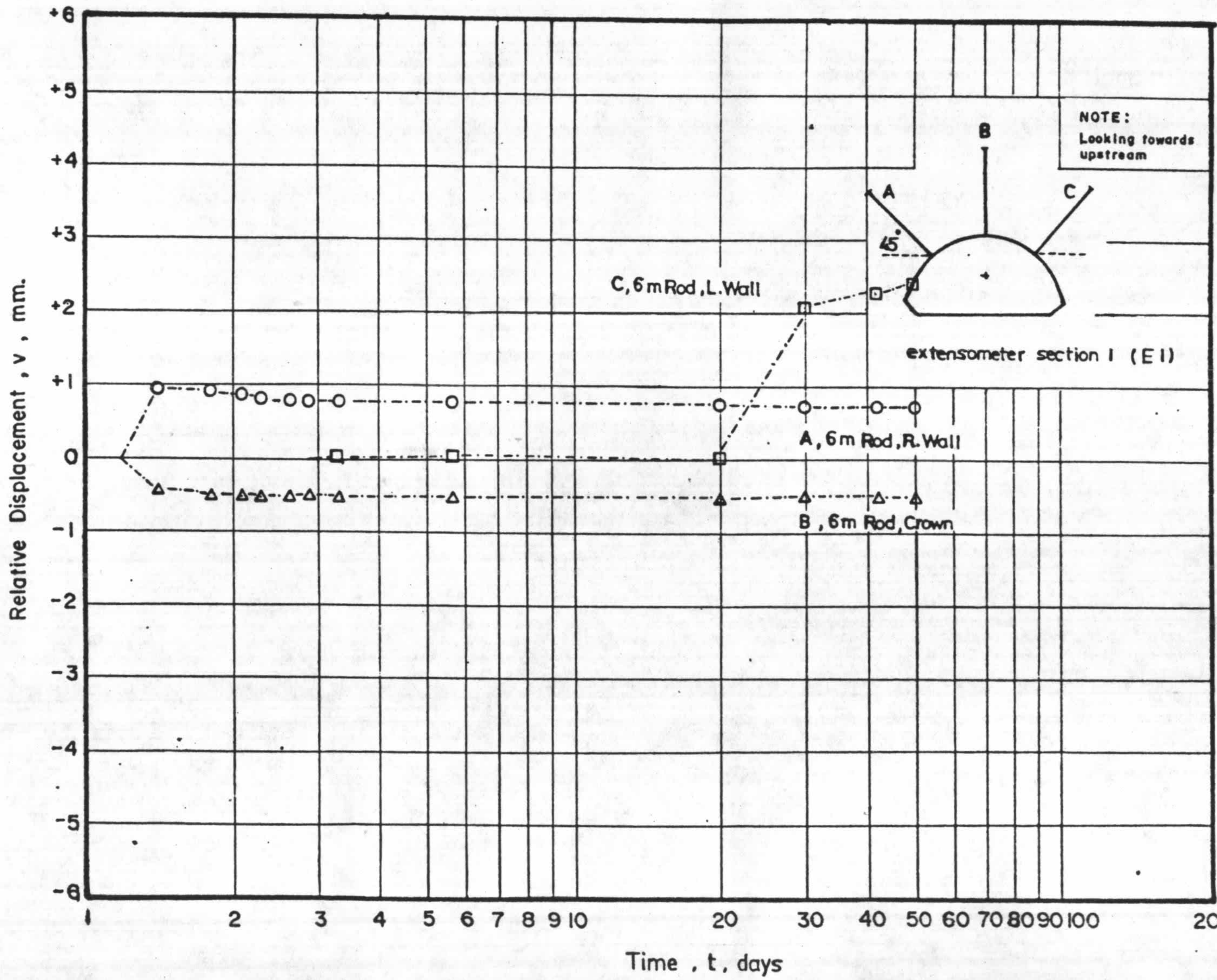


Figure 7.16 Long term displacement measurements by means of extensometers in diversion tunnel Section 1 at chanaige 46.50 m.

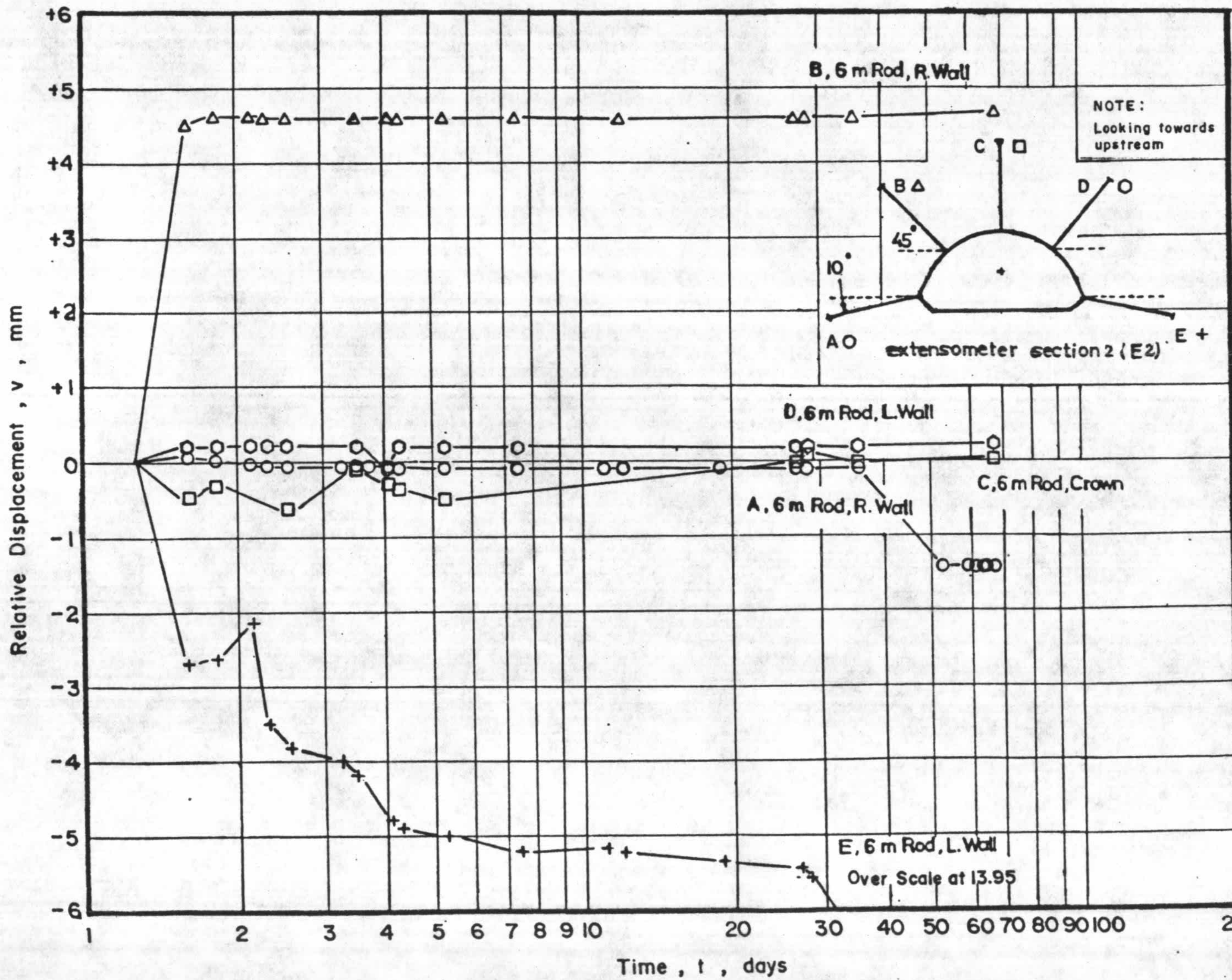


Figure 7.17 Long term displacements by single point borehole of extensometers in diversion tunnel top heading section 2 at chanaige 56.50 m.

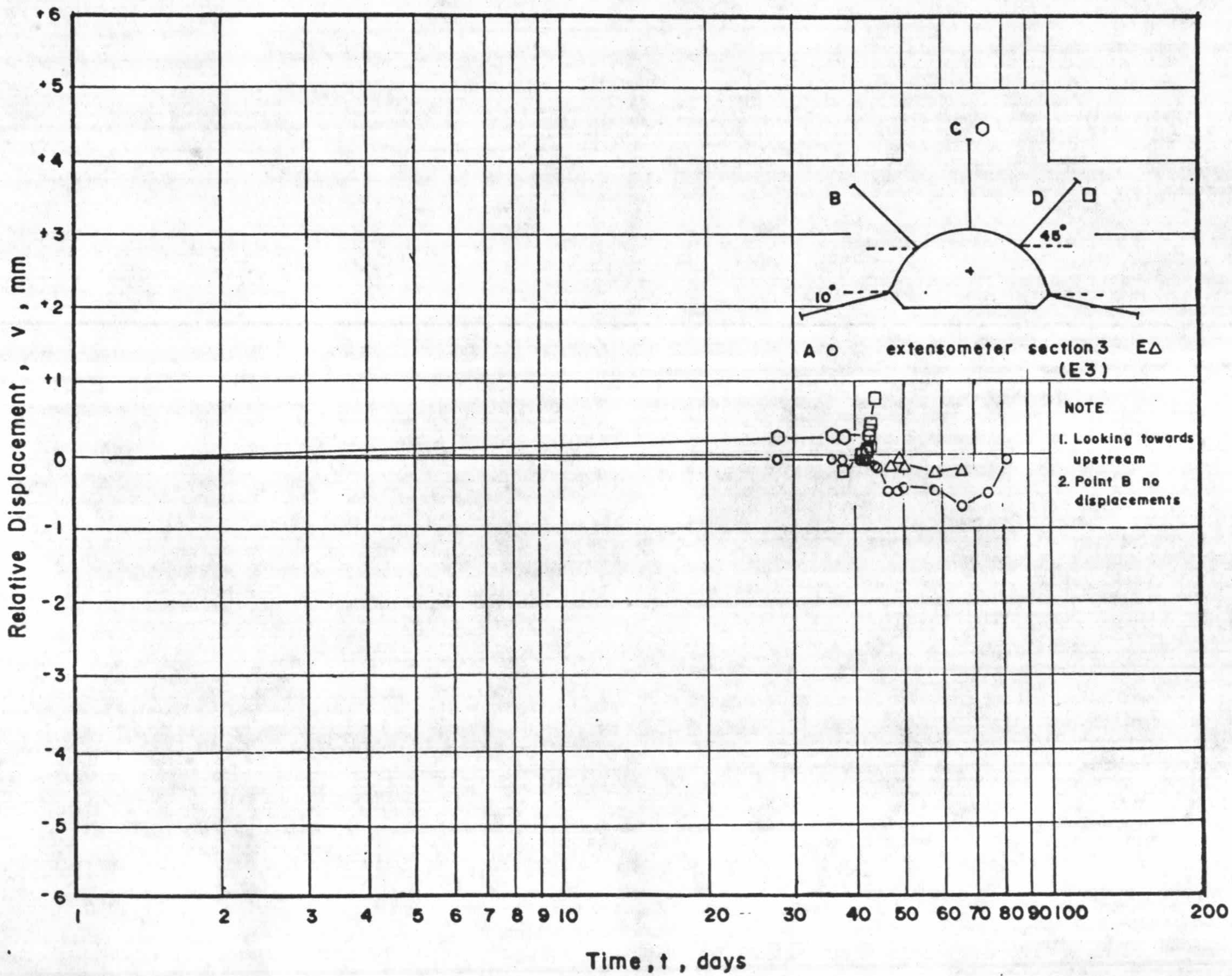


Figure 7.18 Long term displacements by single-point borehole of extensometers in diversion tunnel top heading Section 3 at chaniage 329.00 m.

The position A indicated a compression about 0.75 mm followed by an inward "creep" which brought the position B to move 0.50 mm in 30 days. After the removal of the position C, an additional movement of about 2.45 mm was observed.

The movement study at Section 2, chanaige 56.50 m, was taken on five positions. Three extensometers were installed in the crown and both sides. The horizontal inward movement at the position A indicated an extension of 1.42 mm. Position B showed a constant closure of 4.5 mm since the begining. The vertical movement of the crown at position C showed a somewhat erratic behavior the maximum elongation in order of 0.5 mm, the minimum in the order of 0.05 mm.

The extensometer at the position D indicated a constant shortening (compression) of 0.23 mm since the begining of the measurements. The extensometer at the position E exhibited a large extension, up to 7 mm, with the widening of the top heading.

After the concrete lining was placed, the borehole extensometers were installed at Section 3. The position A and B had a small value of movement, in the order of 0.08 and 0.02 mm respectively. Since these movements were so small, they were not possible to obtain a definite information of the time or rate of the rock movement. The displacement of the positions C, D and E was respectively 0.27, 0.75 and 0.21 mm in only 82 days.

If the rock movement either before or after the bench removal is plotted against the logarithm of time, a stratght line results, suggesting that this oreep is a logarithmic function of time as has been found in some other cases (e.g. the Overvaal railway tunnel, Bieniawski, 1975). Accepting this, it is then possible to predict an unrestrained movement which would occur in the tunnel at any time in the future by extending the plotted straight line.

The analysis of overburden stress was performed using the equations

$$\sigma_v = Z \qquad \text{..........} \quad (7.4)$$

and

$$\sigma_v = 0.027\ Z \qquad \text{..........} \quad (7.5)$$

where σ_v = vertical stress

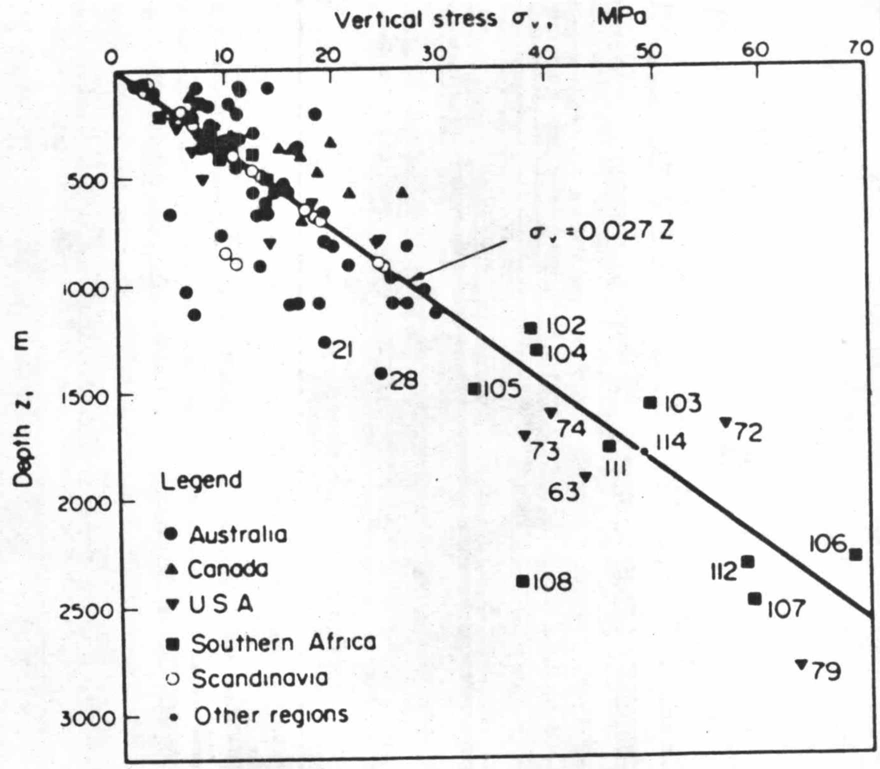
γ = unit weight

Z = depth.

These empirical equations have been supported by numerous measurements (Figure 7.19a) and are ones of the reliable formulas to determine the in-situ stresses (Goodman, 1980). It appears that the overburden at Section 1 which the excavated surface is 30 m, has an overburden stress of 0.80 MPa. There may lready be a pronounced effect of the open excavation. Whereas at Section 2, the overburden is 62 m, thus resulting then in an overburden stress of 1.65 MPa.

Brown and Hoek (1978) examined a number of published values of in-situ stress (Figure 7.18b) and independently discerned a hyperbolic relation for the limits of k(Z). Thus, even without measurements one can estimate, within broad limits, the variation of horizontal stress with depth.

a)



b)

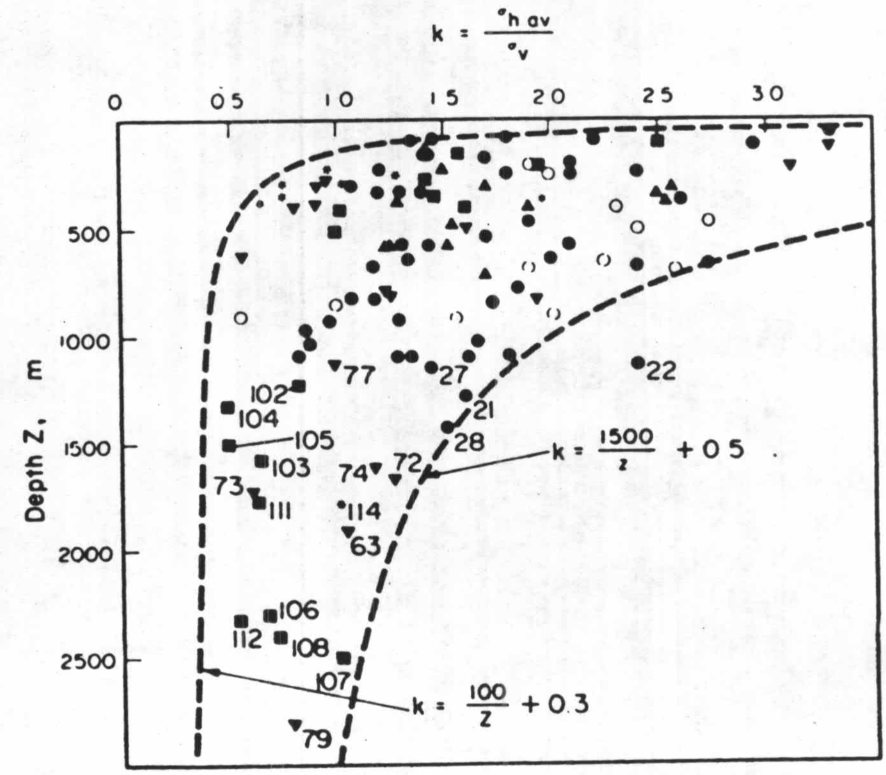


Figure 7.19 Results of stress measurements. a) Vertical stresses. b) Average horizontal stresses (after Brown and Hoek, 1978).

The horizontal stress ratio (k) has been obtained from both the relationship in Figure 7.18b and assuming a circular opening in an elastic ground and obtaining the ratio of elongation of an extensometer at the spring line to one in the crown (Figures 7.16 and 7.17). The resulting horizontal stress ratio lies in the range of $k = 0.77$ to 2.84.