Behaviour of Rock Masses in Victoria Power Tunnel

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Introduction

The Victoria Power Project encompasses a double curvature arch dam and power house of 3 x 70 MW capacity, linked by a 5.6 km long, 6.2 m dia concrete lined pressure tunnel.

Excavation of the tunnel commenced in September 1980 and by November 1982 the total length of tunnel was completed through 4 headings. Poor progress was recorded in some locations due to bad ground conditions and breakdown of mechanical plant. 84% of the total tunnel length was driven in a good quality rock with a maximum progress of 70 m/week.

From the data obtained it was revealed that one major fault and several minor faults could be predicted crossing the tunnel line. During excavation, cavity formation and rock fall took place in certain locations and the major fault resulted in a realignment of the orginal tunnel route.

Rock mass in the tunnel was classified in to four major categories based on two empirical systems (N.G.I & CSIR) for the purpose of designing of tunnel supports, reinforcement, lining and for grouting.

Geomechanical Characteristics

The Victoria tunnel intercepts high grade metamorphic rocks of Precambrian age, which have been folded & fractured in Pre Jurasic time. The rock mass is composed of gently dipping thickly foliated, gneisses, granulites quartzites and crystalling limestone. (Fig. 2)

There were three major types of discontinuities present in this tunnel, namely:

a). Joints (3 sers S1 S2 & S3)
b). Foliation
c). Faults (minor/major).

Intersection of joints and foliation planes are shown in isometric (Fig. 4)

The first 10 m distance from the downstream portal is in a blocky and fractured gneiss and supports were installed comprising 5 Nos steel arch ribs at 1 m centres along with 50 mm layer of shotcrete and 3 m long pattern rock bolts. (fig. 5). Some locations between Surge Chamber and Down Stream Portal the Presence of Smooth Joints and bedding planes caused excessive overbreak and formed diamond shaped profile after blasting (fig. 3). Minor faults that were found were broken and shared rock with clay zones of the order of 0.5 m wide with ground water inflow of 50 - 60 l/m. The existence of these fault zones contributed to the formation of unstable wedges and blocks. A predicted major fault was encountered at Ch. 2145 consisting of 2-3 m wide sheared rocks with clay zones. No major overbreak occured due to controlled blasting and it was supported by means of 100 mm thick shotcrete, reinforced with weld mesh as recommended by N.G.I. classification system.

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Fig 1

PLAN

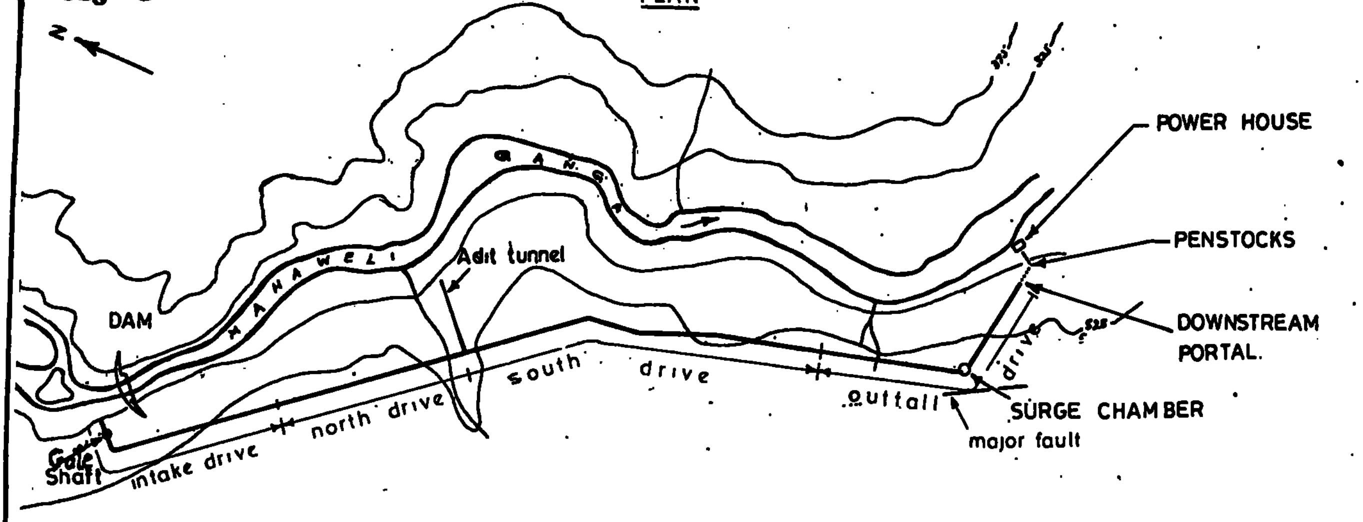
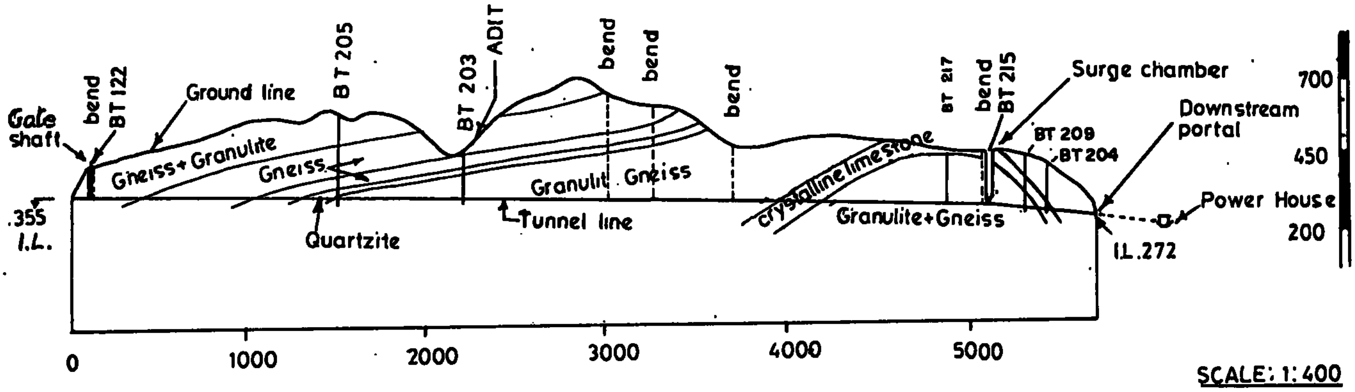


Fig 2

LONGITUDINAL SECTION OF TUNNEL



Influence of the geology in outfall drive heading - Typical sections

Fig 3

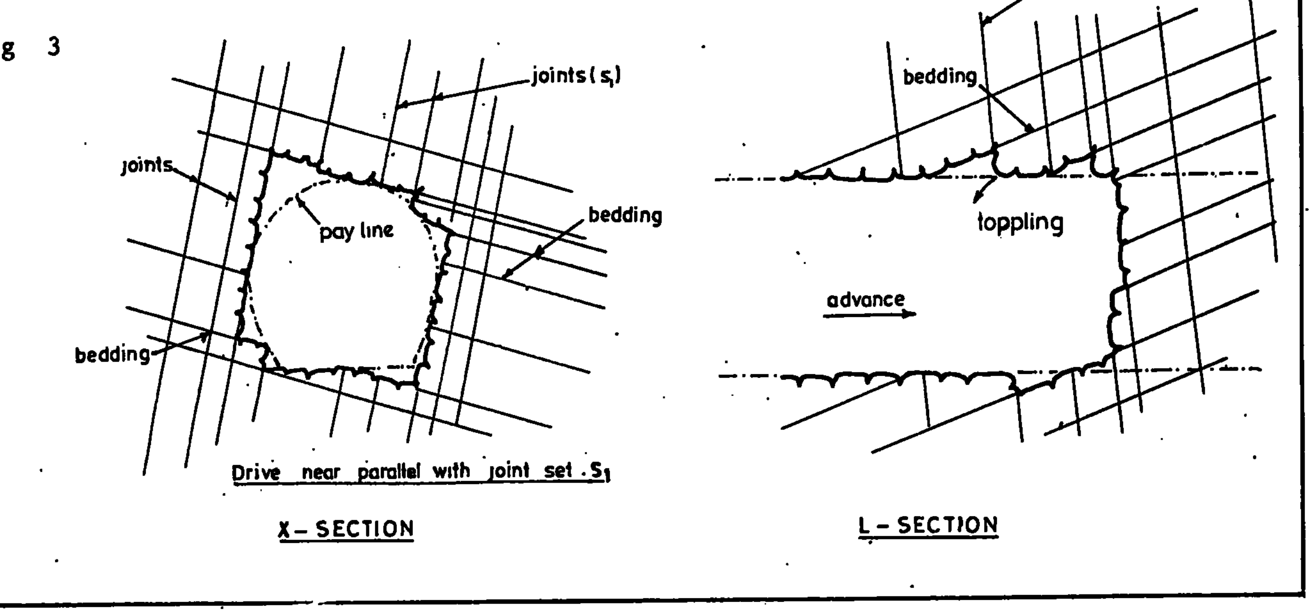


TABLE I

Rock Type	Rock Mass Quality	Tunnel Length (m)	Estimated Uniaxial Compresive Strength (MPa)	Modulus of Rock (GPa)	Support Installed
1	100<Q<4	4765	100	>20	Occasional Spot Bolts
2	4<Q<100	727	40-20	30-20	3m long battern rock bolts 50 mm Shotcrete with or without mesh
3	0.1<Q<4	152	10-5	20-10	3-4m long pattern bolts occational 4m sbiles 50-100 mm schotcrete with mesh

TABLE II

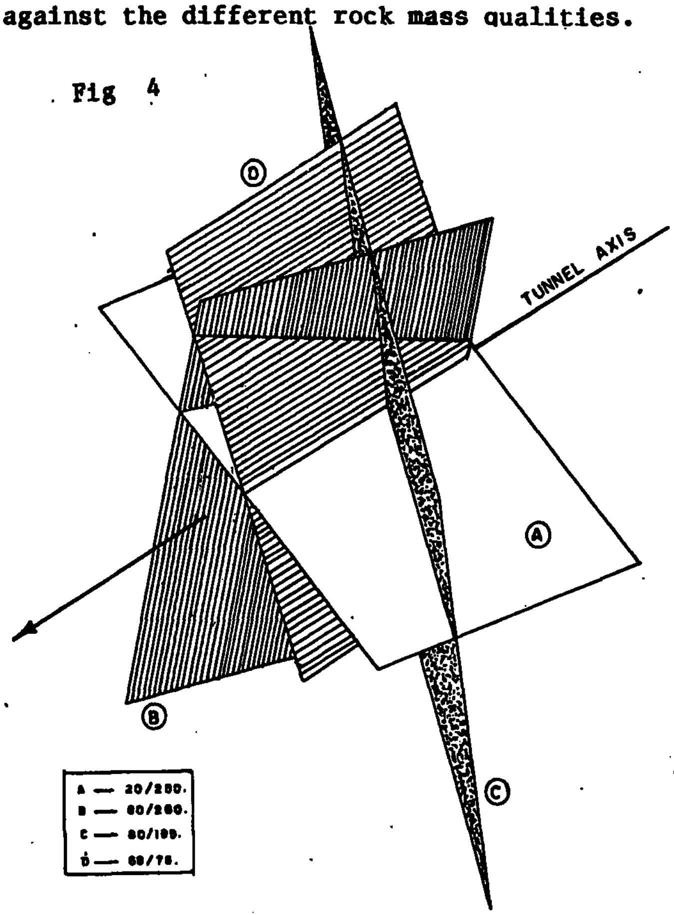
C. S. I. R. Classification			N. G. I. Classification	
Parameter	Value	Rate	Parameter	Value
U.C.S.	20-30 Mpa	4	R.Q.D.	40
R.Q.D.	40%	8	Jn	12
J. Sp	50-100 mm	7.5	Jr	1.5
J.Cond	Slickensided & infill	6	Jc	6
C.W.	Moderate Pressure	4	Ju	0.66
Rating		29.5	S.R.F.	7.5

TABLE III

Uniaxial compressive strength (σ_c)	40 MPa	30 MPa
Material constant for original rock mass (m)	0.750	0.320
Material constant for original rock mass (s)	0.0001	0.0001
Modulus of elasticity (E)	20×10^3 MPa	10×10^3 MPa
Poission's ratio (ν)	0.20	0.20
Material constant for brocken rock mass (m_r)	0.025	0.025
Material constant for brocken rock mass (s_r)	0	0
Unit weight of brocken rock mass (γ)	0.025 MN/m^3	0.025 MN/m^3
In situ stress magnitude (P_o)	3.75 MPa	3.7 MPa
Tunnel radius (r)	3.60 m	3.60 m

For the purpose of design of the supports, the reinforcement of the tunnel lining, and grouting, rock types have been classified into three major categories depending on the quality of the rock. Table 1 presents information on the installed rock support against the different rock mass qualities.

Fig 4



ISOMETRIC VIEW OF FOUR INTERSECTING PLANES
(NORTH DRIVE TUNNEL).

Stability Measures

Plain Failure

The downstream end of the Power Tunnel is located on a gently dipping slope, consisting of fractured gneiss with 1 m wide sub-vertical shattered and altered bands. The foliation is unfavourably oriented and the cross section of the tunnel portal showing a set of joints (S2) and foliation planes with thin layers of biotite schist (Fig. 5).

A slope stability analysis on the Class II rock has been carried out by using rock parameters given below. A planer failure was assumed along the biotite rich layers as shown in Fig. 6. Cohesion of the rock was assumed to be 20 - 30 kN/m² and friction angle of the order of 35°. The factor of safety was found to range from 2.42 to 2.21 under dry conditions. Under saturated conditions with water head varying up to 8 m, and factor of safety for the portal slopes varied from 0.28 to 0.99.

The critical stability of the slope cuts was recognised and the following stabilisation measures were undertaken immediately at the portal area before the tunnel excavation started.

a). Resin bonded 4 m long tensioned rock bolts across weaker planes with weld mesh and shotcrete.

b). Providing a number of weep holes for sub-surface drainage.

c). Diversion of surface run-off by providing a drain above the portal.

Wedge Analysis

Outfall Drive

Potential wedge failures in the tunnel drive were predicted by stereo plot analysis of joint sets and foliation plane. Orientation of these three planes are given in a stereo net (Fig. 7a)

Intersection lines of weak planes from a maximum of four wedges in different locations of the tunnel are shown in Fig. 7b. Analysis showed that of the four wedges defined as A,B,C & D, only A and B could fall under gravity. The wedge C cannot slide under gravity, while wedge D does not slide along the Plane 3, because the angle of friction is greater than the dip of Plane 3. In general, the gravity type wedge fails without any apparent sign of distress and it does so fairly quickly after blasting.

The size of the wedges are governed by the location and orientiation of joint

Fig 5 CROSS SECTION OF THE DOWNSTREAM PORTAL

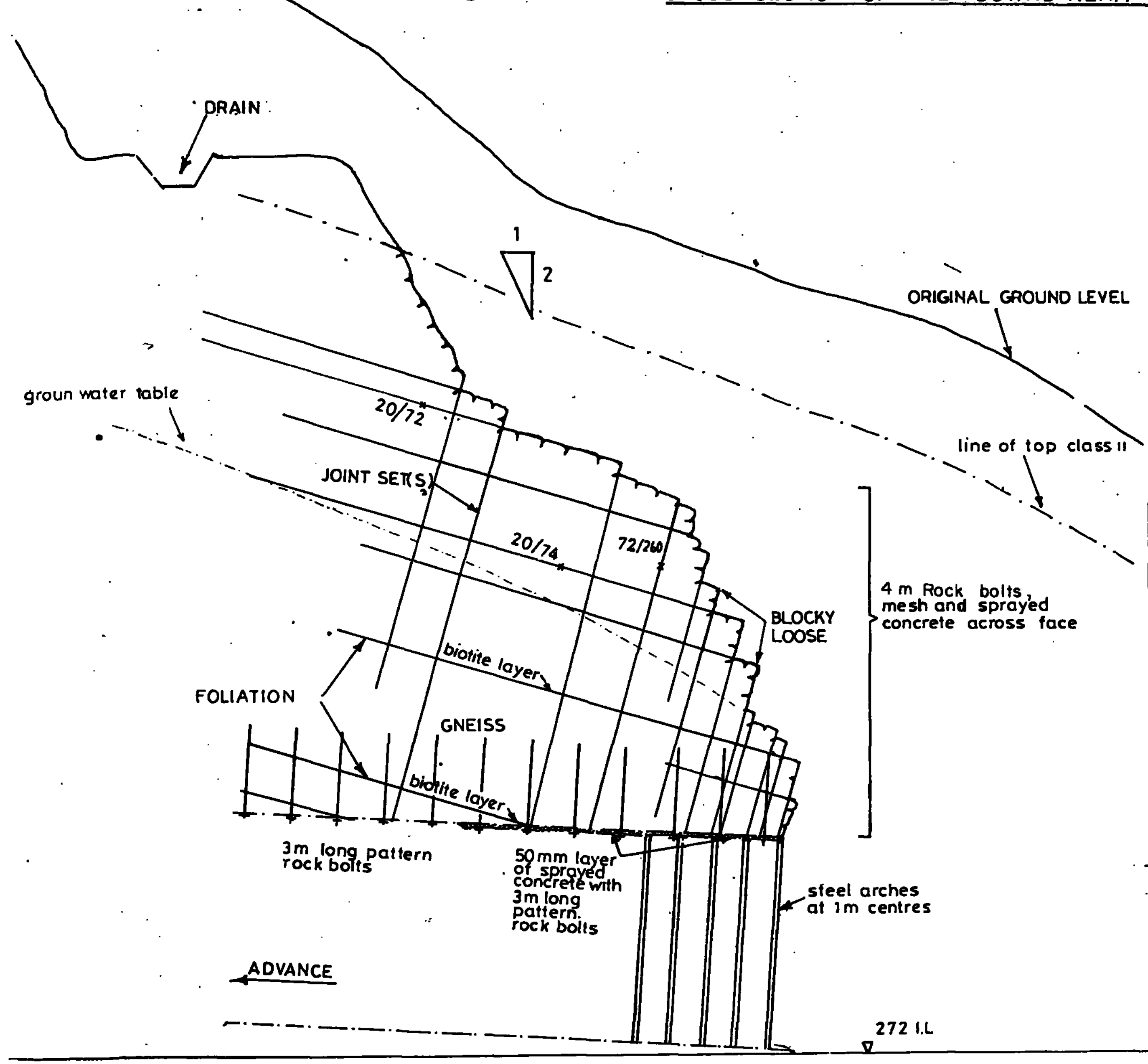


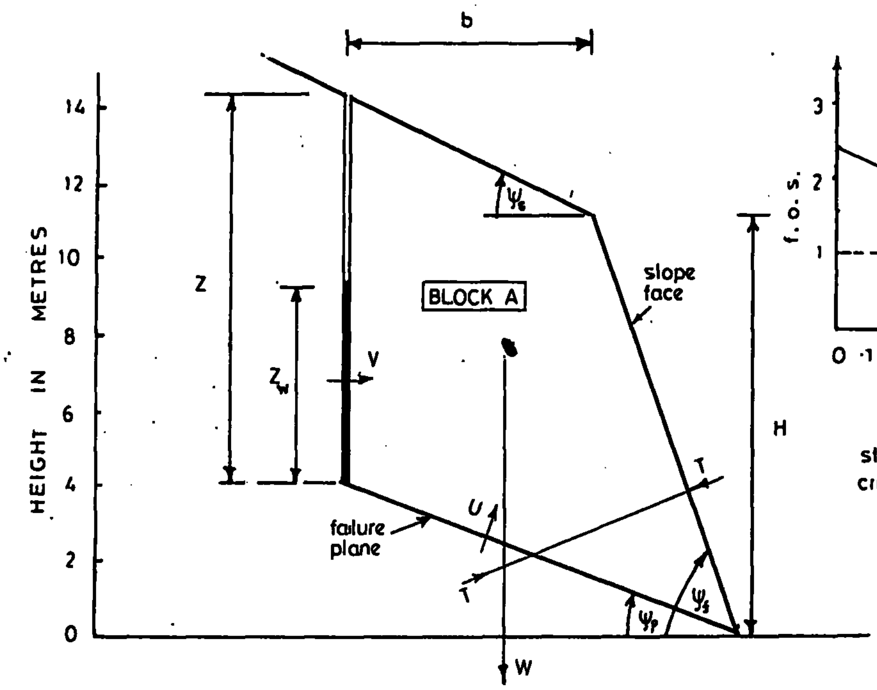
TABLE IV

μ_i (M)	P_{roof} (MPa)	P_{Wall} (MPa)	P_{Floor} (MPa)
0.0500	0.4320	0.02	-0.392
0.0360	0.3860	0.05	-0.286
0.0330	0.3780	0.06	-0.258
0.0280	0.3680	0.08	-0.208
0.0260	0.3650	0.09	-0.185
0.0140	0.3630	0.10	-0.163
0.0053	0.5720	0.50	+0.128
0.0031	0.7350	0.70	0.665
0.0025	0.8214	0.80	0.770
0.0014	1.0020	1.00	0.990
0.0082	1.100	1.10	1.100

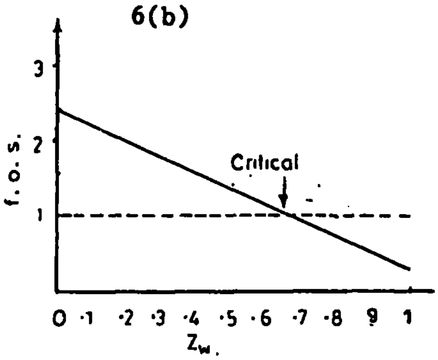
μ_i (M)	P_{roof} (MPa)	P_{Wall} (MPa)	P_{Floor} (MPa)
0.1430	0.9027	0.10	-0.702
0.0790	0.7690	0.20	-0.369
0.0490	0.7330	0.30	-0.133
0.0330	0.7401	0.40	+0.059
0.0230	0.7720	0.50	0.228
0.0175	0.8197	0.60	0.880
0.0130	0.8780	0.70	0.821
0.0100	0.9440	0.80	0.655
0.0078	1.0170	0.90	0.782
0.0062	1.0900	1.00	1.910
0.0004	1.2500	1.20	1.140

SCALE 1 : 20

Fig 6(a) ANALYSIS OF PLANE FAILURE IN ROCK SLOPE DOWNSTREAM PORTAL



6(b)



stable if no water, critical when saturated,

INPUT DATA:

- H - height of slope face
- ψ_f - inclination of slope face
- ψ_s - inclination of upper slope face
- ψ_p - inclination of failure plane
- Z_w/Z - depth of water in tension crack / depth of tention crack
- T - tensio in rock bolts
- θ - inclination of force T to normal of failure plane
- C - cohesive strength of failure surface.
- ϕ - angle of friction on failure surface
- b - distance of tension crack behind slope crest

Factor of safty:

$$F = \frac{C.A + (W\cos\psi_p + T\cos\theta - U - V\sin\psi_p)\tan\phi}{W\sin\psi_p + V\cos\psi_p - T\sin\theta}$$

$H = 11\,m$, $\psi_f = 70^\circ$, $\psi_s = 25^\circ$, $\psi_p = 20$, $Z_w/Z = 0-1$

$T = 0.\,kg$, $\theta = -$, $C = 20-30\ kN/m^2$, $\phi = 35$, $\gamma = 2500\ kg/m^3$., $b = 7m$

b (M)	F. O. S. if, $Z_w/Z=0$	F. O. S. if, $Z_w/Z=1$
0	2.42	0.281
2	2.29	0.59
4	2.25	0.77
6	2.23	0.90
8	2.21	0.99

for $c = 20\ kn/m^2$, $b = 0$

where:

$Z = H + b\tan\psi_s - (b + H\cot\psi_f)\tan\psi_p$,

$W = 2/2\ \gamma\,(H^2\cot\psi_f + 2bH + b^2\tan\psi_s - (H\cot\psi_f + b)^2\tan\psi_p)$

$A = \dfrac{H\cot\psi_f + b}{\cos\psi_p}$,

$U = 1/2\ \gamma_w . Z_w . A$,

$V = 1/2\ \gamma_w Z_w^2$,

planes with respect to the excavation profile. A wedge formed by Joint Sets S1, S2 and the foliation plane is gravity type and it can fall from the roof (Fig. 8). The volume of the largest wedge which

STEREOPLOT SHOWING THE WEDGE FORMED BY JOINT SET S_2 S_1 AND FOLIATION PLANE

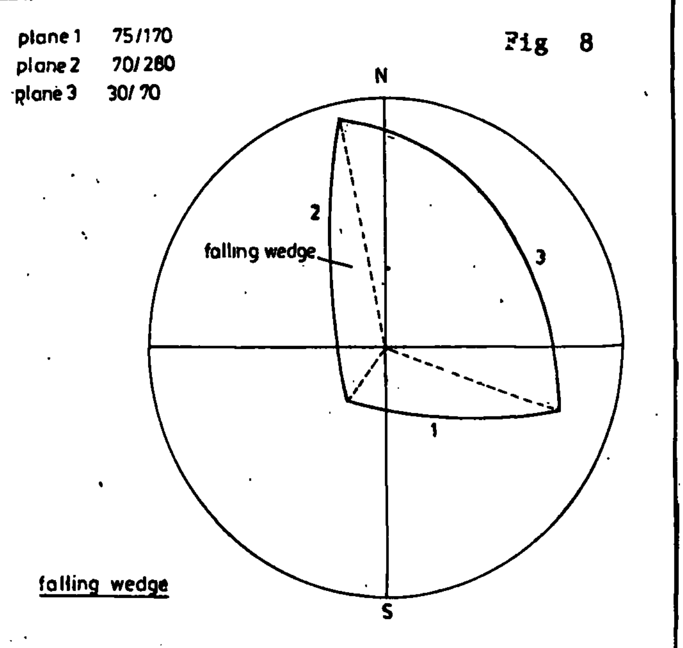


Fig 8

can move under gravity into the tunnel as estimated is shown in Fig. 9. The length of the anchors was determined by considering the height of the wedge. In this case the maximum height of the wedge is 1.1 m which determines the maximum length of the bolt at 7.1 m plus the bond length. At some places where joint spacing is closer, small wedges could fail. from this analysis it was found that no extra bolting is required other than the standard pattern tensioned bolting of 1.5 x 1.5 m spacing up to 3 m in depth. It was calculated that the support pressure required to stabilise the rock wedges in the tunnel is 9.85kN/m². A factor of safety of 1.5 was obtained by the standard pattern rock bolting.

Intake Drive

Stereographic projection of joint data in the intake drive are plotted on the polar equal area net and the orientation of these joints is likely to establish a potential wedge in the tunnel roof. the most probable wedge configurations are shown in Fig. 10. Small wedges failed at some locations where support was required Analysis showed that the requirement for standard 3 m long rock bolts on the grid of 1.5 x 1.5 m was justified. The weight of the gravity type wedge is calculated to be of the order of 75 tonnes, and it can be easily retained with standard pattern bolting (F.O.S. 1.5) as shown in the Fig. 10.

Evaluation of rock conditions and support measures in the faulted area Ch. 2147-2128 m

The two empirical systems. C.S.I.R. and N.G.I. have been used to evaluate the rock conditions in the fault zones, Table 2.

C.S.I.R.

A rate of 29.5 classified the rock conditions as poor with an average stand up time of 5 hours for a 1.5 m span. An estimated cohesion of 100 - 150 MPa, a friction angle of 30°-35° and a rock modulus of 5 - 10 GPa was estimated to be representative for these faulted areas.

N. G. I.

Using the N.G.I. classifications the value Q = 0.07 placed the conditions under category 33.

For an R.Q.D./Jn > 2 the recommended support is,

a. lm tensioned bolts.

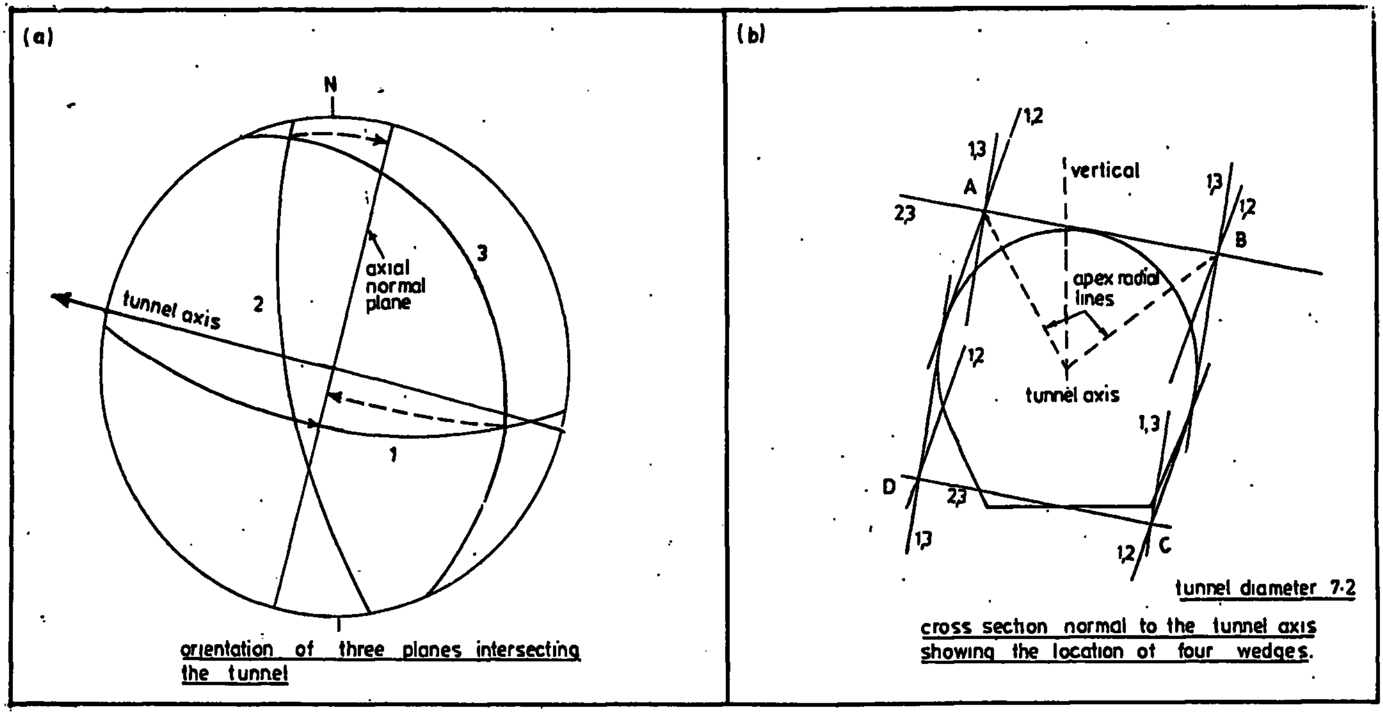
b. 25 - 100 mm shotcrete layer and reinforced with mesh.

Rock Support Interaction Analysis

Based on observations in the tunnel and on the results obtained from the two classification systems, the following rock parameters were taken to be representive for the fault zone.

The results of the analysis in Table

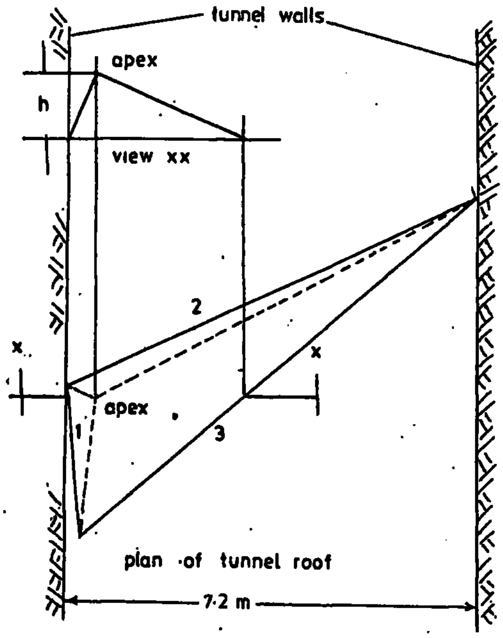
Fig 7



(a) orientation of three planes intersecting the tunnel

(b) cross section normal to the tunnel axis showing the location of four wedges.

Fig 9



Block cornar	Plunge degrees	Plan length (m)	Apex height (m)
I_{12}	68°	0·5	1·20
I_{13}	25°	2·3	1·07
I_{23}	8°	7·4	1·04

mean apex height = 1.10 m

base area of wedge = 9·34 m^2

wedge volume = 3.42 m^3

Fig. 9 Determination of the volume of a wedge projected parallel to tunnel roof

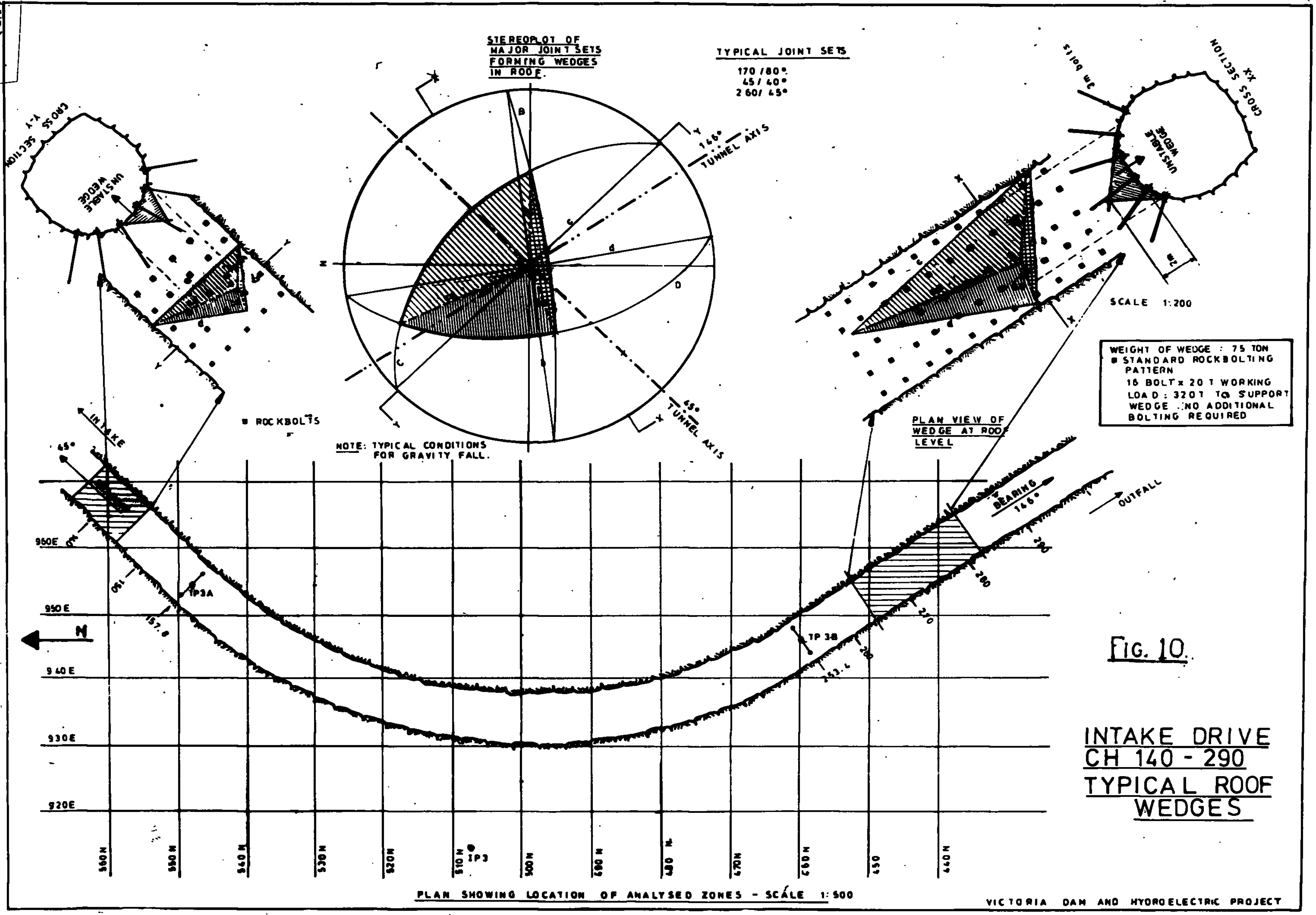


FIG. 10.

INTAKE DRIVE
CH 140 - 290
TYPICAL ROOF WEDGES

4. were plotted in Fig. 11. The support line for 4 m rock bolts and 100 mm thick shotcrete are represented by line 1, 2 & 3. Fig. 11 shows that rock bolts alone are not sufficient as a support system and therefore, a combination of rock bolts (1 m spacing and 4 m long) and 100 mm shotcrete provide an adequate support system in the fault zone. (Support line No. 3)

Concluding Remarks

The paper described the different types of stability analysis to evaluate the required support pressures to restablish the tunnel stability in different quality of rock. the actual supports installed were based on the above analysis during the time of excavation. Unstable blocks were able to be predicted through graphic solution in different locations of the tunnel.

Instrumentation in the tunnel is most essential to ensure that the actual rock pressures do not exceed the support capacity during tunnel construction. Rock mass behaviour and support system should be monitored by instrumentation to update the design parameters to ensure safe and economic construction.

Acknowledgement

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Fig 11