Ground Reaction and NATM Concepts for the Design of Underground Excavations in Rock

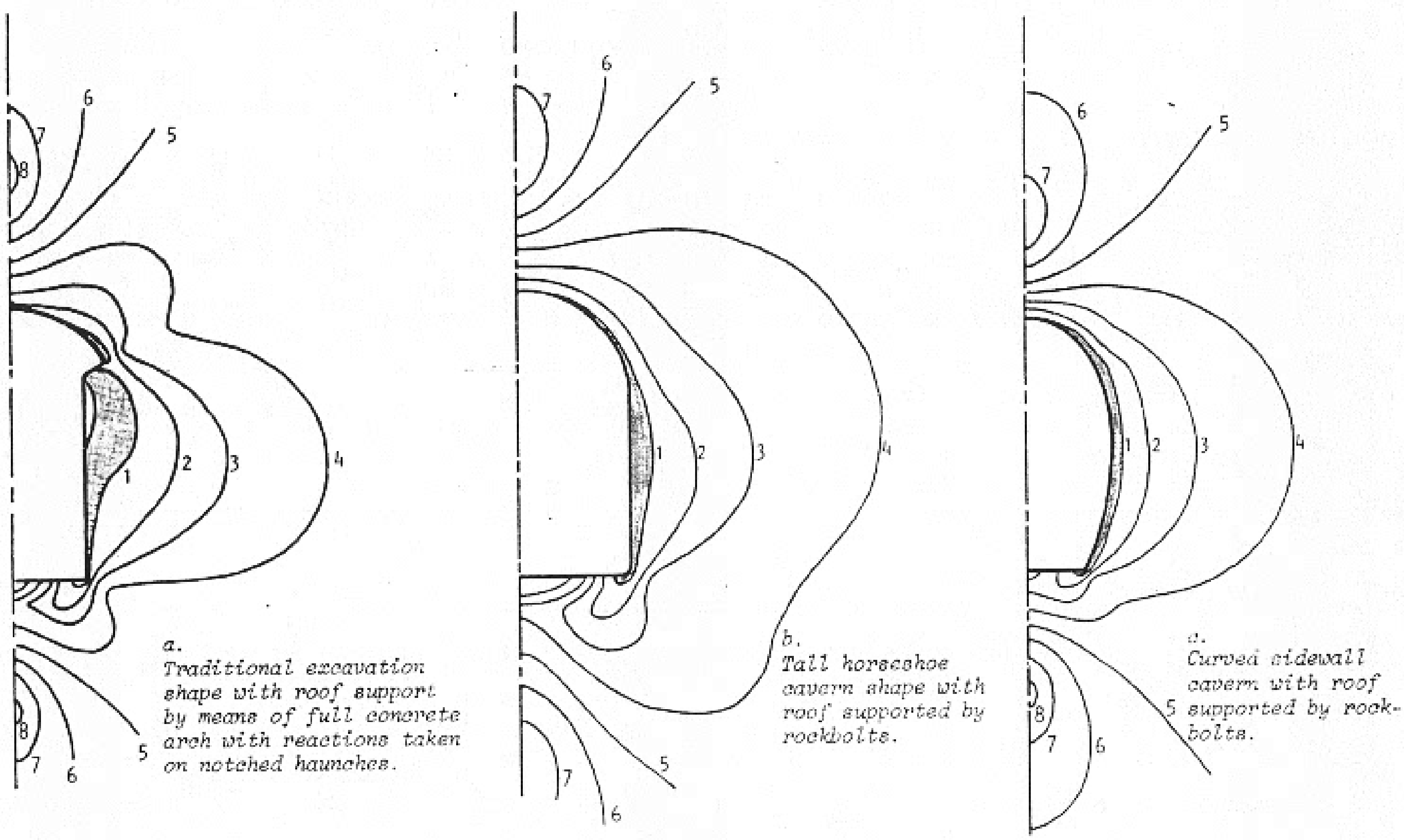


Figure 124 : Strength/stress ratio contours in the rock surrounding powerhouse caverns of different shapes.

(from Hoek & Brown, 1980)

$\sigma_H = 0.5\ \sigma_v$

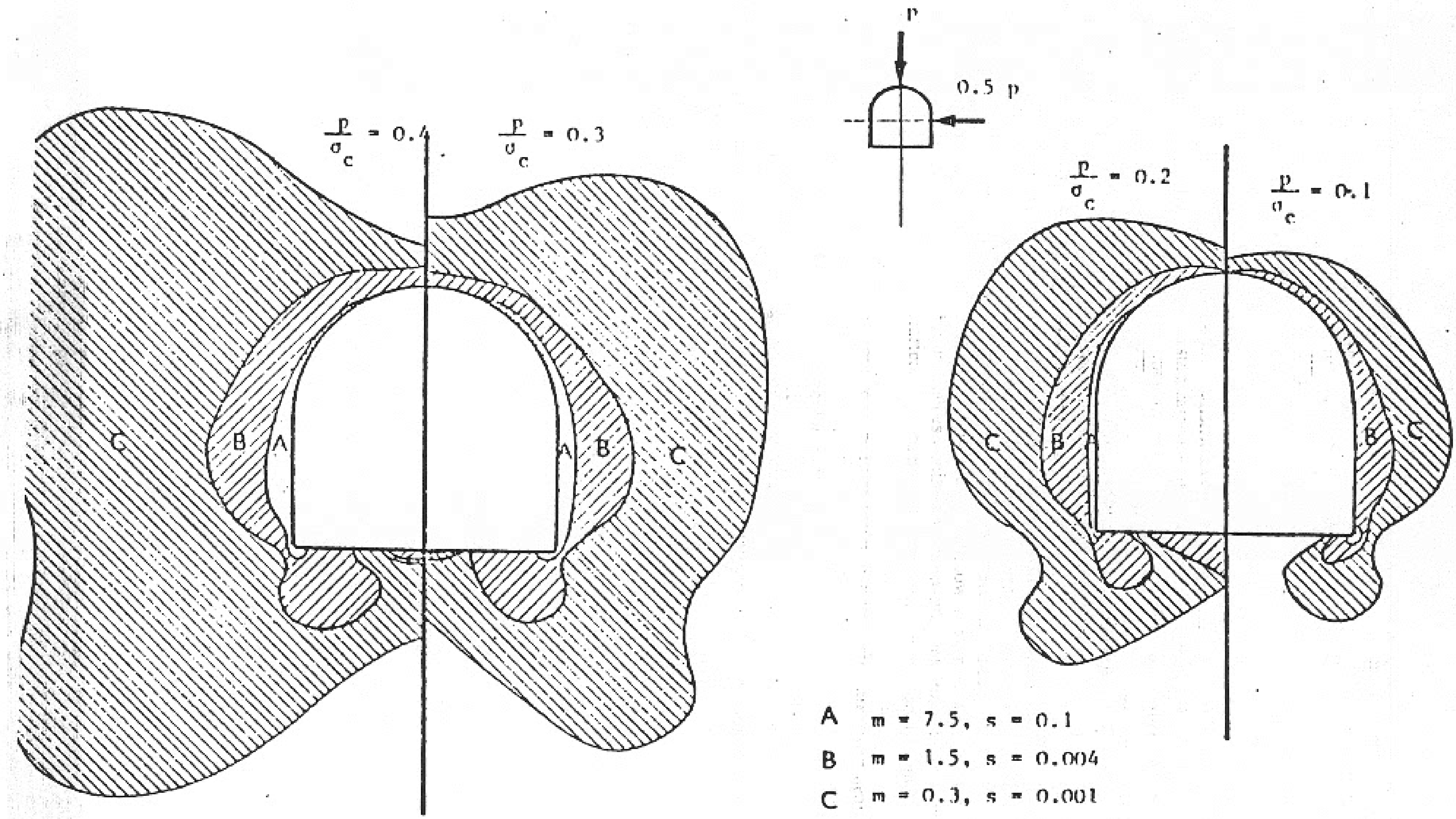
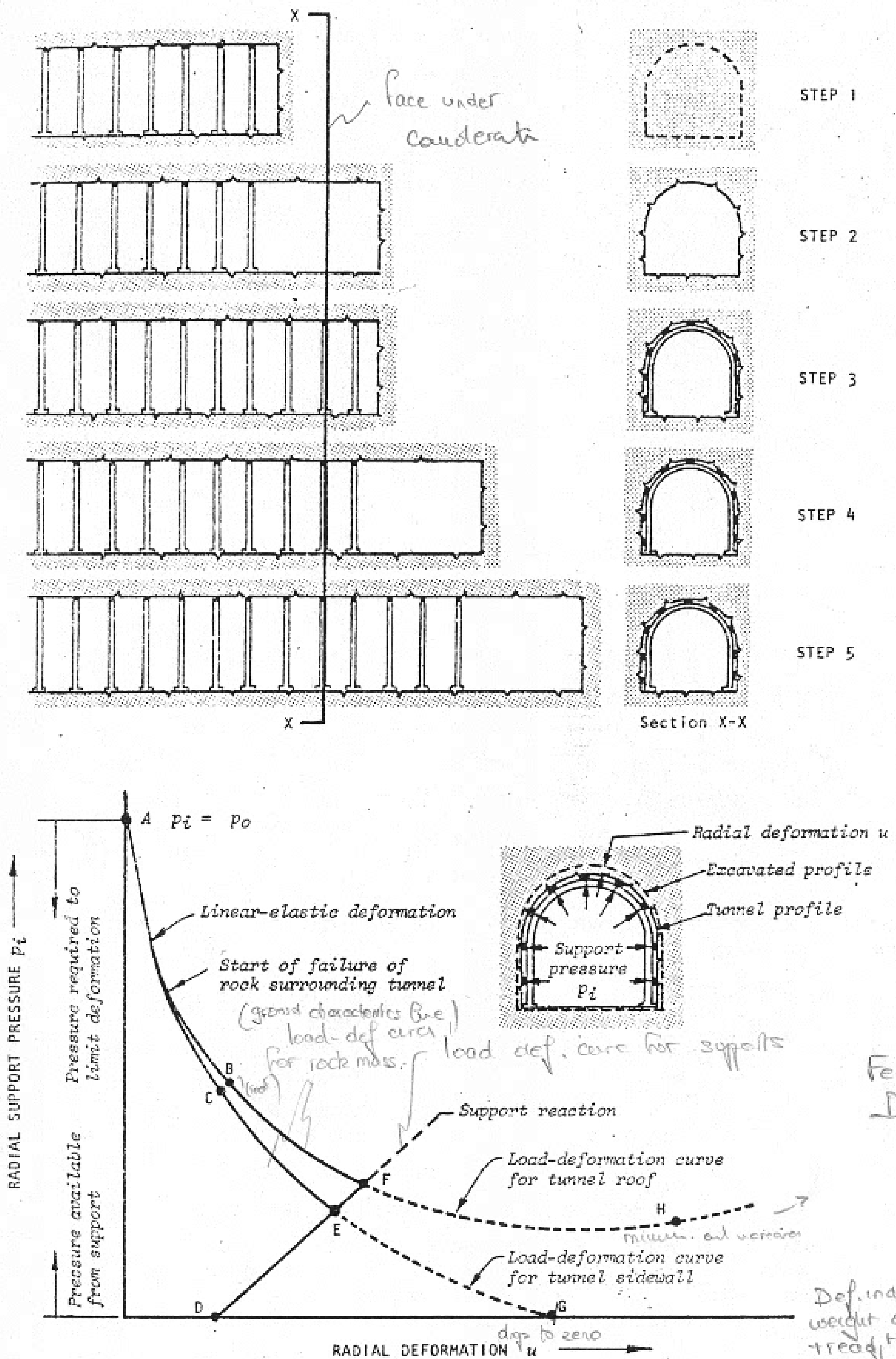


FIGURE 2.3.42 : POTENTIAL OVERSTRESSED ZONES AROUND A HORSESHOE OPENING IN BIAXIAL STRESS FIELD (k=0.5)

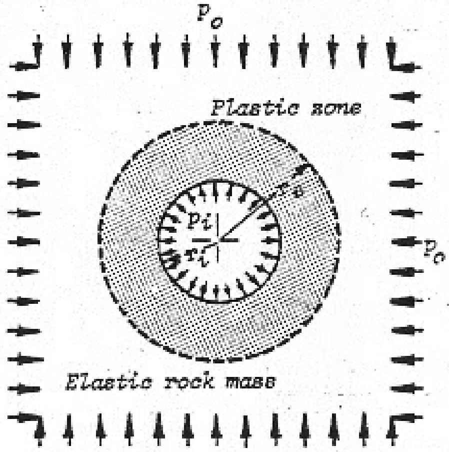


Fenner - Pacher Diagram.

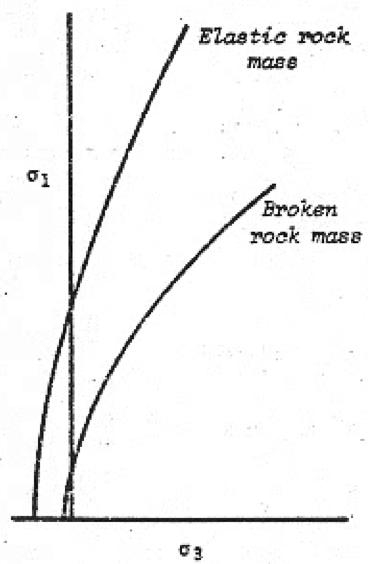
Def. induced by dead weight of loosened rock + read. + stress field in the rock around the excavation

Figure 129 : Hypothetical example of a tunnel being advanced by full face drill and blast methods with blocked steel sets being installed after each mucking cycle. The load-deformation curves for the rock mass and the support system are given in the lower part of the figure. (After Daeman[224]).

Concept of Ground - Support Interaction for the Design of Support Systems to Resist Deformations Associated with Stress Redistribution



Assumed tunnel geometry



Assumed failure criteria for original elastic and broken rock masses

Hoek and Brown Failure Criteria

- $\sigma_1 = \sigma_3 + (m\sigma_c\sigma_3 + s\sigma_c^2)^{1/2}$

  (intact rock)

- $\sigma_1 = \sigma_3 + (m_r\sigma_c\sigma_3 + s_r\sigma_c^2)^{1/2}$

  (broken rock)

m, s, m_r, s_r : empirical numbers

σ_c : unconfined compressive strength for intact rock.

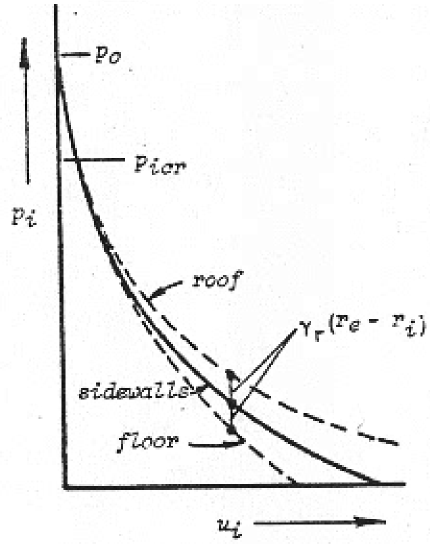
Hoek and Brown Failure Criterion.

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$$

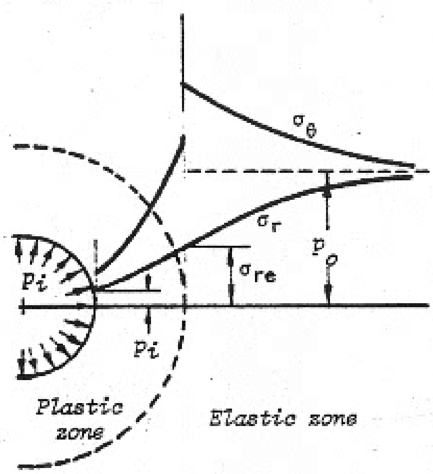
$$\underbrace{\frac{\sigma_1}{\sigma_c}}_{\sigma_{1n}} = \underbrace{\frac{\sigma_3}{\sigma_c}}_{\sigma_{3n}} + \sqrt{m\underbrace{\frac{\sigma_3}{\sigma_c}}_{\sigma_{3n}} + s} \quad \text{or} \quad \sigma_{1n} = \sigma_{3n} + \sqrt{m\sigma_{3n} + s}$$

TABLE 12 - APPROXIMATE EQUATIONS FOR PRINCIPAL STRESS RELATIONSHIPS AND MOHR ENVELOPES FOR INTACT ROCK AND JOINTED ROCK MASSES

	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*	$\sigma_{1N} = \sigma_{3N} + \sqrt{0.007\sigma_{3N} + 0}$ $\tau_N = 0.042(\sigma_N)^{0.534}$	$\sigma_{1N} = \sigma_{3N} + \sqrt{0.010\sigma_{3N} + 0}$ $\tau_N = 0.050(\sigma_N)^{0.539}$	$\sigma_{1N} = \sigma_{3N} + \sqrt{0.015\sigma_{3N} + 0}$ $\tau_N = 0.061(\sigma_N)^{0.546}$	$\sigma_{1N} = \sigma_{3N} + \sqrt{0.017\sigma_{3N} + 0}$ $\tau_N = 0.065(\sigma_N)^{0.548}$	$\sigma_{1N} = \sigma_{3N} + \sqrt{0.025\sigma_{3N} + 0}$ $\tau_N = 0.078(\sigma_N)^{0.556}$



Required support lines for the rock surrounding the tunnel



Notation for stresses around tunnel

Ground Response

5

TABLE 14 - CALCULATION SEQUENCE FOR ROCK-SUPPORT INTERACTION ANALYSIS

1. Required support line for rock mass

Input data required

σ_c = uniaxial compressive strength of intact rock pieces
m, s } material constants for original rock mass. (see table 12 on page 176)
E = modulus of elasticity of original rock mass
ν = Poisson's ratio of original rock mass
m_r, s_r } material constants for broken rock mass (see table 12 on page 176)
γ_r = unit weight of broken rock mass
p_o = in situ stress magnitude
r_i = radius of tunnel

Calculation sequence

a. $M = \frac{1}{2}\left(\left(\frac{m}{4}\right)^2 + m\,p_o/\sigma_c + s\right)^{\frac{1}{2}} - \frac{m}{8}$ (102)

b. $D = \dfrac{-m}{m + 4\left(m/\sigma_c(p_o - M\sigma_c) + s\right)^{\frac{1}{2}}}$ (101,116)

c. $N = 2\left[\dfrac{p_o - M\sigma_c}{m_r\sigma_c} + \dfrac{s_r}{m_r^2}\right]^{\frac{1}{2}}$ (105)

Input p_i

d. For $p_i > p_o - M\sigma_c$, deformation around tunnel is elastic

$\dfrac{u_i}{r_{io}} = \dfrac{(1+\nu)}{E}(p_o - p_i)$ (117)

e. For $p_i < p_o - M\sigma_c$, plastic failure occurs around tunnel

$\dfrac{u_e}{r_e} = \dfrac{(1+\nu)}{E} M\sigma_c$ (108)

f. $\dfrac{r_e}{r_i} = e^{N - 2\left(\frac{p_i}{m_r\sigma_c} + \frac{s_r}{m_r^2}\right)^{\frac{1}{2}}}$ (104)

g. For $r_e/r_i < \sqrt{3}$: $R = 2D \ln r_e/r_i$ (114)

h. For $r_e/r_i > \sqrt{3}$: $R = 1.1D$ (115)

i. $e_{av} = \dfrac{2(u_e/r_e)(r_e/r_i)^2}{\left((r_e/r_i)^2 - 1\right)\left(1 + 1/R\right)}$ (113)

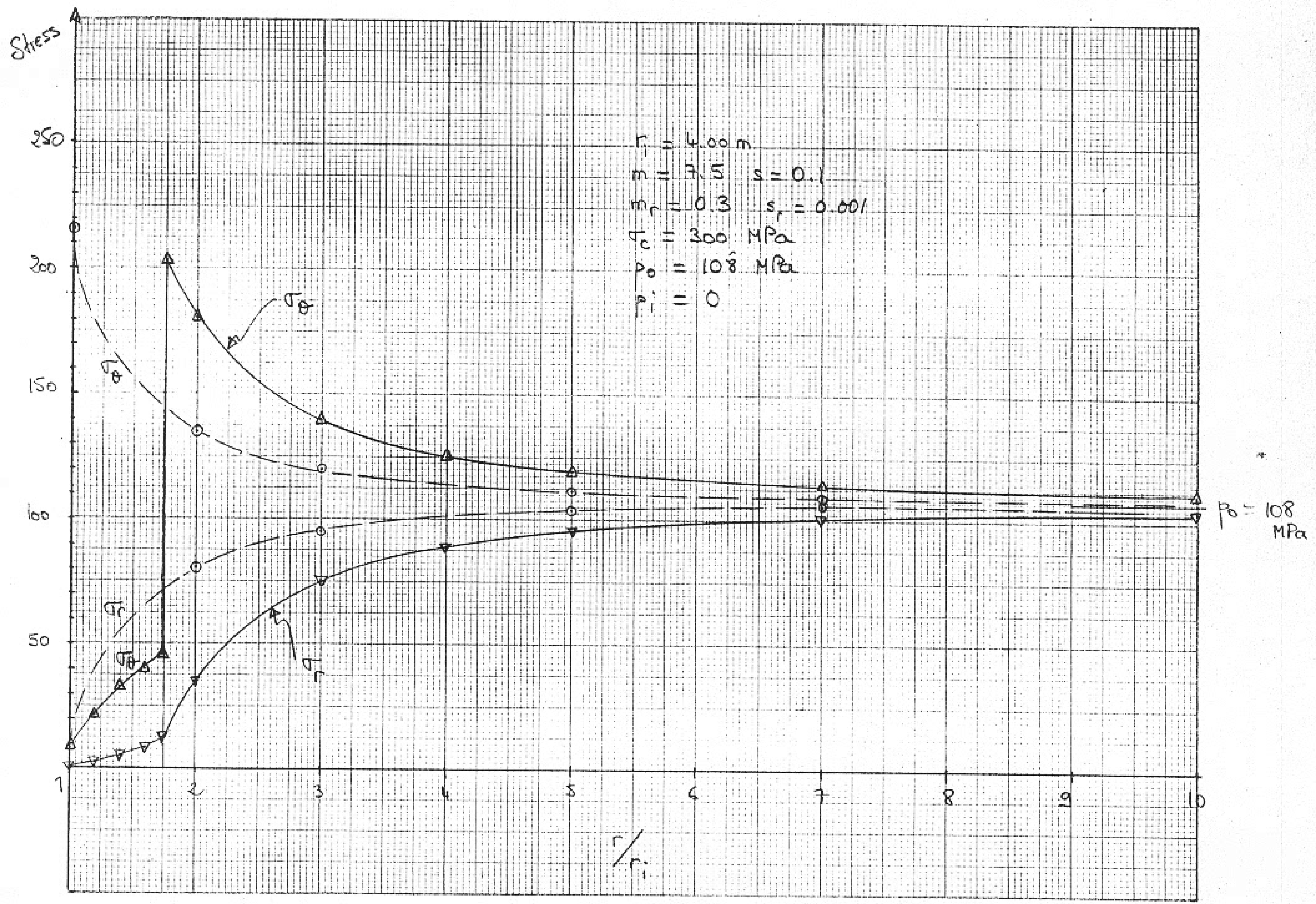
j. $A = (2u_e/r_e - e_{av})(r_e/r_i)^2$ (111)

k. $\dfrac{u_i}{r_{io}} = 1 - \left[\dfrac{1 - e_{av}}{1 + A}\right]^{\frac{1}{2}}$ (110)

l. For roof of tunnel, plot u_i/r_{io} against $\dfrac{p_i + \gamma_r(r_e - r_i)}{p_o}$

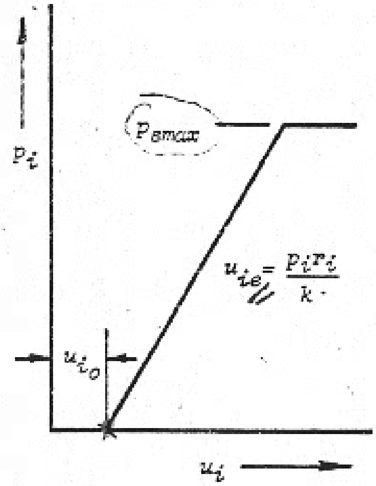
m. For sidewalls of tunnel, plot u_i/r_{io} against p_i/p_o

n. For floor of tunnel, plot u_i/r_{io} against $\dfrac{p_i - \gamma_r(r_e - r_i)}{p_o}$



Stress
250
200
150
100
50
1
2
3
4
5
6
7
8
9
10
r/r_i
r_i = 4.00 m
m = 7.5 s = 0.1
m_r = 0.3 s_r = 0.001
σ_c = 300 MPa
p_o = 108 MPa
p_i = 0
σ_θ
σ_r
σ_θ
σ_r
σ_θ
p_o = 108 MPa

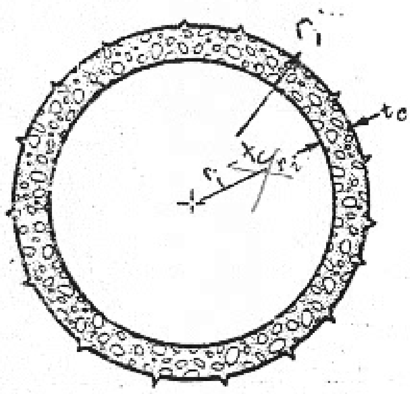
Support Response



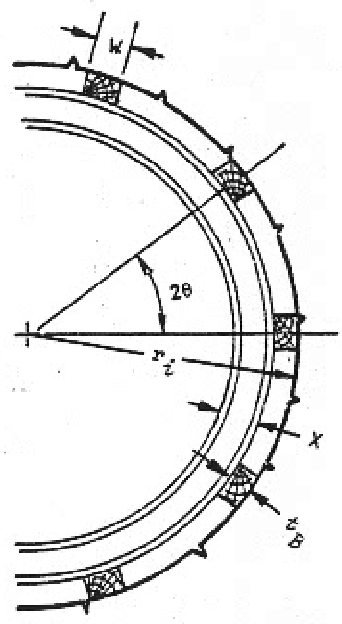
Available support curve

$$p_i = k \frac{(u_i - u_{io})}{r_i}$$

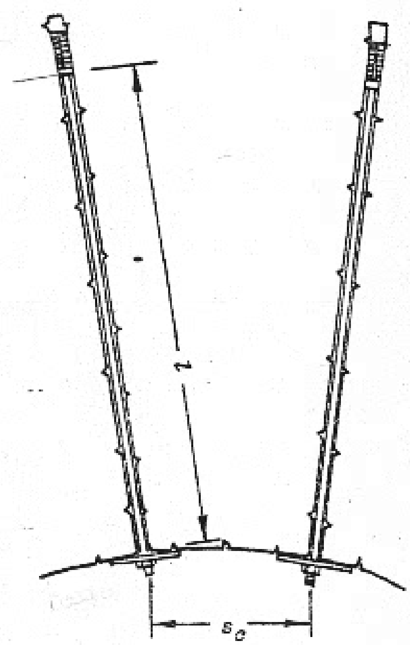
with $p_i < p_{smax}$



Concrete lining



Blocked steel set



Ungrouted mechanically anchored rockbolts

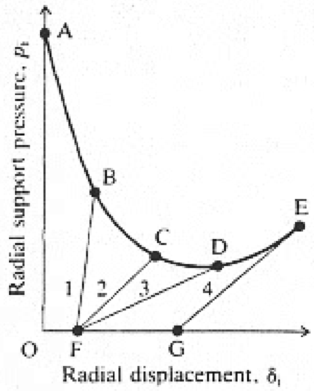


Figure 11.3 Illustration of the influence of support stiffness and of the timing of its installation on support performance.

important influence on this displacement control. Figure 11.3 shows a rock–support interaction diagram for a problem similar to that illustrated in Figure 11.1. The ground characteristic or required support line is given by ABCDE. The earliest practicable time at which support can be installed is after radial displacement of an amount OF has occurred.

Support 1 is installed at F and reaches equilibrium with the rock mass at point B. This support is too stiff for the purpose and attracts an excessive share of the redistributed load. As a consequence, the support elements may fail causing catastrophic failure of the rock surrounding the excavation.

Support 2, having a lower stiffness, is installed at F and reaches equilibrium with the rock mass at C. Provided the corresponding displacement of the periphery of the excavation is acceptable operationally, this system provides a good solution. The rock mass carries a major portion of the redistributed load, and the support elements are not stressed excessively. Note that if, as in the temporary/permanent support concept, this support were to be removed after equilibrium had been reached, uncontrolled displacement and collapse of the ~~rock mass~~ would almost certainly occur.

Support 3, having a much lower stiffness than support 2, is also installed at F but reaches equilibrium with the rock mass at D where the rock mass has started to loosen. Although this may provide an acceptable temporary solution, the situation is a dangerous one because any extra load imposed, for example by a redistribution of stress associated with nearby mining, will have to be carried by the support elements. In general, support 3 is too flexible for this particular application.

Support 4, of the same type and stiffness as support 2, is not installed until a radial displacement of the rock mass of OG has occurred. In this case, the support is installed too late, excessive convergence of the excavation will

Figure 11.4 Non-linear support reaction curves observed for some support types.

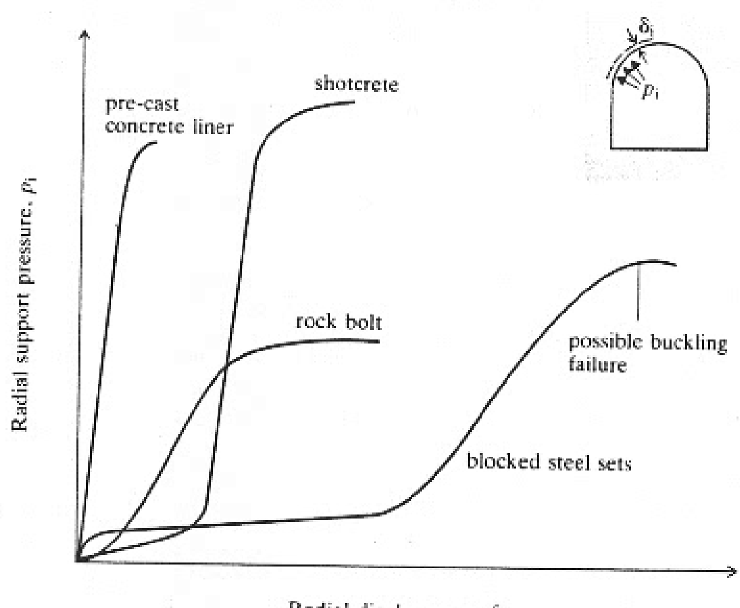


TABLE 14 - CALCULATION SEQUENCE FOR ROCK-SUPPORT INTERACTION ANALYSIS

2. Support stiffness and maximum support pressure for concrete or shotcrete lining

Input data required

E_c = modulus of elasticity of concrete or shotcrete
ν_c = Poisson's ratio of concrete or shotcrete
t_c = thickness of lining
r_i = tunnel radius
$\sigma_{c.conc.}$ = uniaxial compressive strength of concrete or shotcrete

Support stiffness and maximum support pressure

a. $$k_c = \frac{E_c\left(r_i^2 - (r_i - t_c)^2\right)}{(1 + \nu_c)\left((1 - 2\nu_c)r_i^2 + (r_i - t_c)^2\right)} \qquad (120)$$

b. $$p_{scmax} = \tfrac{1}{2}\sigma_{c.conc.}\left[1 - \cancel{\frac{r_i^2}{(r_i + t_c)^2}}\right] \quad \frac{(r_i - t_c)^2}{r_i^2} \qquad (121)$$

3. Support stiffness and maximum support pressure for blocked steel sets

Input data required

W = flange width of steel set
X = depth of section of steel set
A_s = cross-sectional area of steel set
I_s = moment of inertia of steel section
E_s = modulus of elasticity of steel section
σ_{ys} = yield strength of steel
r_i = tunnel radius
S = steel set spacing along tunnel axis
θ = half angle between blocking points (radians)
t_B = thickness of block
E_B = modulus of elasticity of block material

Support stiffness and maximum support pressure

a. $$\frac{1}{k_s} = \frac{S.r_i}{E_s A_s} + \frac{S.r_i^3}{E_s I_s}\left[\frac{\theta(\theta + \sin\theta\cos\theta)}{2\sin^2\theta} - 1\right] + \frac{2S.\theta.t_B}{E_B W^2} \qquad (122)$$

b. $$p_{ssmax} = \frac{3A_s I_s \sigma_{ys}}{2S.r_i.\theta\left[3I_s + XA_s\left(r_i - (t_B + \tfrac{1}{2}X)\right)(1 - \cos\theta)\right]} \qquad (123)$$

4. Support stiffness and maximum support pressure for ungrouted mechanically or chemically anchored rockbolts or cables

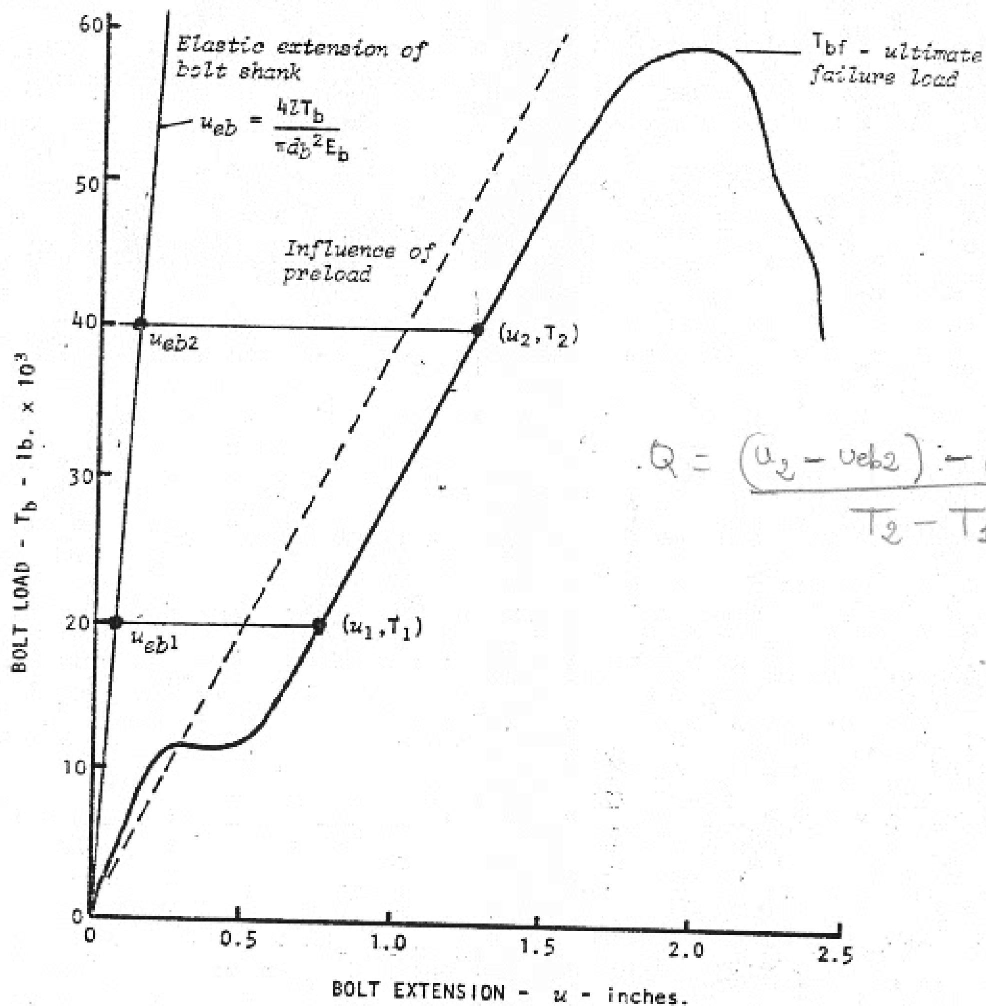
Input data required

l = free bolt or cable length
d_b = bolt diameter or equivalent cable diameter
E_b = elastic modulus of bolt or cable material
Q = load-deformation constant for anchor and head
T_{bf} = ultimate failure load from pull-out test
r_i = tunnel radius
s_c = circumferential bolt spacing
s_l = longitudinal bolt spacing

Support stiffness and maximum support pressure

a. $$\frac{1}{k_b} = \frac{s_c s_l}{r_i}\left[\frac{4l}{\pi d_b^2 E_b} + Q\right] \qquad (127)$$

b. $$p_{sbmax} = T_{bf}/s_c s_l \qquad (128)$$



$$Q = \frac{(u_2 - u_{eb2}) - (u_1 - u_{eb1})}{T_2 - T_1}$$

Figure 130 : Typical bolt load-extension curve determined by means of a pull-out test on a 1 inch diameter, 6 foot long bolt anchored by means of a 4 leaf Rawlplug expansion shell. After Franklin and Woodfield[226].

TABLE 13 - ULTIMATE STRENGTH AND LOAD-DEFORMATION CHARACTERISTICS OF TYPICAL ROCKBOLTS.

Bolt diameter in.	mm	Bolt length ft.	m	Anchor type	Rock type	T_{bf}* lb.	Q** in/lb	T_{bf} MN	Q m/MN
0.63	16.0	4.0	1.22	Expansion shell	Good rock	11 000	2.8×10^{-5}	0.049	0.160
0.63	16.0	6.0	1.83	Expansion shell	Shale	13 000	4.2×10^{-5}	0.058	0.241
0.63	16.0	4.0	1.22	Expansion shell	Unknown	9 000	9.0×10^{-6}	0.040	0.053
0.63	16.0	4.0	1.22	Expansion shell and resin	Unknown	14 000	5.0×10^{-6}	0.062	0.030
0.75	19.0	4.0	1.22	Expansion shell	Good rock	11 500	2.2×10^{-5}	0.051	0.126
0.75	19.0	6.0	1.83	Expansion shell	Unknown	20 000	4.0×10^{-6}	0.089	0.024
0.75	19.0	6.0	1.83	Expansion shell and resin	Unknown	22 000	4.0×10^{-6}	0.098	0.029
0.75	19.0	10.0	3.0	Slotted bolt and wedge	Unknown	22 000	1.3×10^{-5}	0.098	0.074
0.87	22.0	10.0	3.0	Expansion shell	Gneiss	48 000	5.5×10^{-6}	0.214	0.032
0.87	22.0	10.0	3.0	Expansion shell	Sandstone	44 000	7.3×10^{-6}	0.196	0.042
0.87	22.0	10.0	3.0	Expansion shell	Sandy shale	28 500	1.2×10^{-5}	0.127	0.069
0.87	22.0	10.0	3.0	Expansion shell	Shale	13 000	2.2×10^{-5}	0.058	0.126
1.00	25.4	19.7	6.0	Expansion shell	Massive gneiss	72 600	8.9×10^{-6}	0.323	0.051
1.00	25.4	6.0	1.83	Expansion shell	Granite	57 000	2.5×10^{-5}	0.254	0.143
1.00	25.4	6.0	1.93	Resin anchor	Granite	64 000	3.2×10^{-6}	0.285	0.018
1.00	25.4	4.0	1.22	Slotted bolt and wedge	Good rock	20 000	1.1×10^{-5}	0.089	0.064
1.00	24.5	6.0	1.83	Resin anchor	Shale	36 000	3.5×10^{-6}	0.160	0.020

* Ultimate pull out load determined in a field test.

** Defined by equation 126 on page 255.

Note : The values listed in this table have been determined from published test data and the authors cannot guarantee the accuracy of the results. For critical applications it is strongly recommended that pull-out and load-deformation characteristics be determined from field tests on the bolts to be used.

10

TABLE 14 - CALCULATION SEQUENCE FOR ROCK-SUPPORT INTERACTION ANALYSIS

5. Available support curve for a single support system

Input data required

k = stiffness of support system under consideration
p_{smax} = maximum support pressure which can be accommodated
u_{io} = initial tunnel deformation before installation of support

Available support curve

For $p_i < p_{smax}$: $$\frac{u_i}{r_i} = \frac{u_{io}}{r_i} + \frac{p_i}{k} \qquad (119)$$

6. Available support curve for a combined support system

Input data required

k_1 = support stiffness of system 1
p_{smax_1} = maximum support pressure for system 1
k_2 = stiffness for support system 2
p_{smax_2} = maximum support pressure for system 2
u_{io} = initial tunnel deformation before installation of support (Note that the two support systems are assumed to be installed at the same time and to start responding to tunnel deformation simultaneously)

Calculation sequence for available support curve

$U_{iueq} - U_{io}$

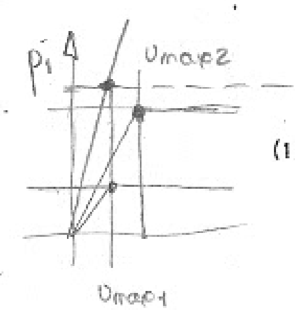
a. $u_{max_1} = r_i \cdot p_{smax_1}/k_1$ ✓

b. $u_{max_2} = r_i \cdot p_{smax_2}/k_2$ ✓

c. $u_{12} = r_i \cdot p_i/(k_1 + k_2)$ ✓

d. For $u_{12} < u_{max_1} < u_{max_2}$

$$\frac{u_i}{r_i} = \frac{u_{io}}{r_i} + \frac{p_i}{(k_1 + k_2)} \qquad (130)$$



e. For $u_{12} > u_{max_1} < u_{max_2}$

$p_{max_{12}} = u_{max_1}(k_1 + k_2)/r_i$ ✓

f. For $u_{12} > u_{max_2} < u_{max_1}$

$p_{max_{12}} = u_{max_2}(k_1 + k_2)/r_i$ ✓

TABLE 16 - TYPICAL INPUT DATA FOR ROCK-SUPPORT INTERACTION ANALYSIS.

Rock mass

Uniaxial compressive strength of intact rock	- σ_c -	see table 9 on page 141.
Material constant for original rock mass	- m -	see table 12 on page 176.
Material constant for original rock mass	- s -	see table 12 on page 176.
Modulus of deformation for rock mass	- E -	see table 15 on page 262.
Poisson's ratio for rock mass	- ν -	0.15 to 0.30.
Material constant for broken rock mass	- m_r -	see table 12 on page 176.
Material constant for broken rock mass	- s_r -	see table 12 on page 176.
Unit weight of broken rock mass	- γ_r -	see below

Sedimentary rocks γ_r - 0.025±0.003 MN/m³ - 155±19 lb/ft³
Igneous and metamorphic rocks γ_r - 0.030±0.003 MN/m³ - 187±19 lb/ft³
Monomineralic aggregates (ores) γ_r - 0.034±0.012 MN/m³ - 210±75 lb/ft³

Depending upon the degree to which the rock mass is jointed or broken, these values may be decreased by up to approximately 20%.

Concrete or shotcrete linings

Modulus of elasticity of shotcrete or concrete - E_c - 21±7 GPa - $3 \times 10^6 \pm 1 \times 10^6$ lb/in²
Poisson's ratio of shotcrete or concrete - ν_c - 0.25
Compressive strength of shotcrete or concrete - $\sigma_{c.conc.}$ - 35±20 MPa - 5000±3000 lb/in²
(Depending upon age and quality)

Blocked steel sets

		Light section 6 I 12		*Medium section 8 I 23*		*Heavy section 12 W 65*	
Flange width	W	0.0762m	2.0 inches	0.1059m	4.16 inches	0.3048m	12.0 in.
Section depth	X	0.1524m	6.0 inches	0.2023m	8.0 inches	0.3048m	12.0 in.
Section area	A_s	0.00228m²	3.52 in²	0.00433m²	6.71 in²	0.01233m²	19.12 in²
Moment of inertia	I_s	8.74×10^{-6}m⁴	21 in⁴	2.67×10^{-5}m⁴	64 in⁴	2.22×10^{-4}m⁴	534 in⁴

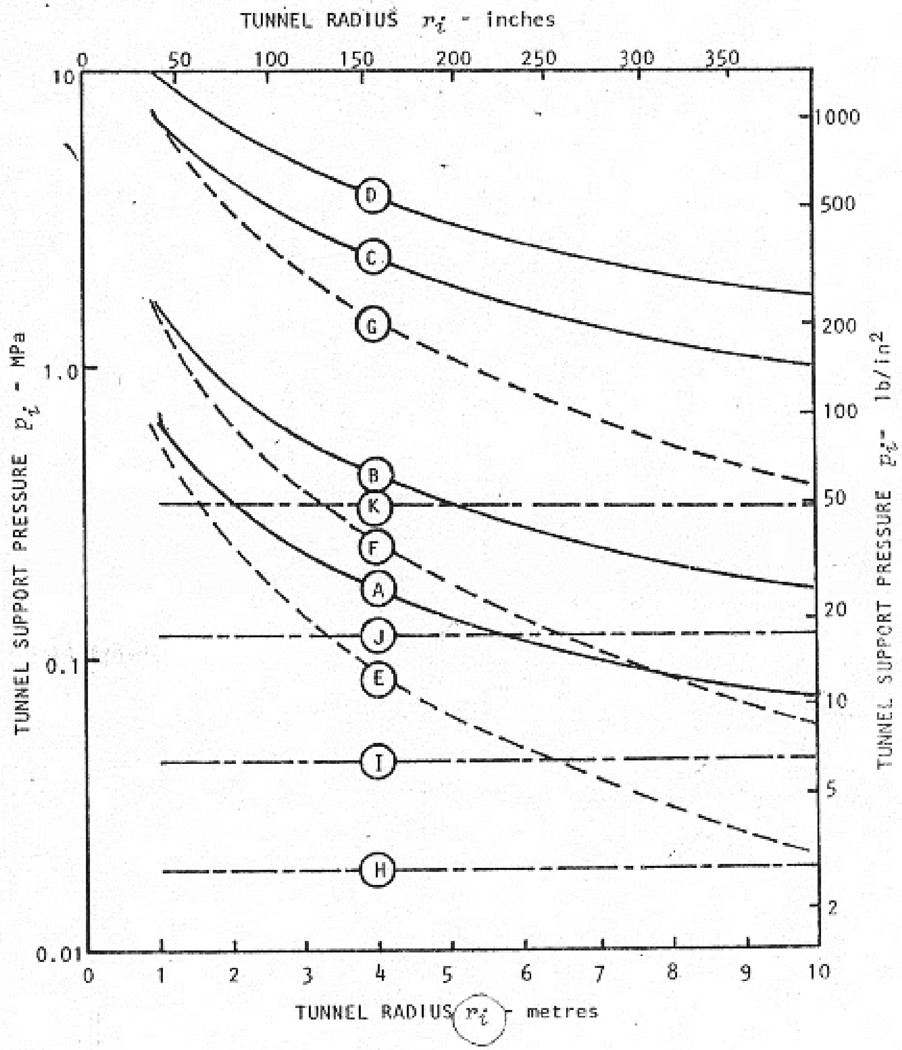
Young's modulus of steel - E_s - 207 GPa - 30×10^6 lb/in²
Yield strength of steel - σ_{ys} - 245 MPa - 36 000 lb/in²
Young's modulus of blocking material - E_B - *Stiff blocking* 10 000 MPa - 1.5×10^6 lb/in²
Soft blocking 500 MPa - 72 000 lb/in²

Rockbolts

Bolt diameter	- d_b -	16mm/⅝ in. - 19mm/¾ in. - 25mm/1 in. - 34mm/1⅜ in.
Young's modulus of bolts	- E_b -	207 GPa - 30×10^6 lb/in²
Anchor stiffness	- Q -	see table 13 on page 257.
Pull out strength	- T_{bf} -	see table 13 on page 257.

TABLE 17 - MAXIMUM SUPPORT PRESSURES FOR VARIOUS SYSTEMS.

Support system | *Tunnel radius*	r_i 1m 39in	r_i 2.5m 98in	r_i 5m 197in	r_i 10m 394in
A - SHOTCRETE - 5cm (0.05m)/ 2 inches thick shotcrete. $\sigma_{c.conc.}$ = 14 MPa/2000 psi after 1 day.	p_{smax} 0.65 MPa 95 psi	p_{smax} 0.27 MPa 39 psi	p_{smax} 0.14 MP 20 psi	p_{smax} 0.07 MPa 10 psi
B - SHOTCRETE - 5cm(0.05m)/ 2 inches thick shotcrete. $\sigma_{c.conc.}$ = 35 MPa/5000 psi after 28 days	1.63 MPa 236 psi	0.68 MPa 99 psi	0.34 MPa 50 psi	0.17 MPa 25 psi
C - CONCRETE - 30cm(0.30m)/ 12 inches thick concrete. $\sigma_{c.conc.}$ = 35 MPa/5000 psi after 28 days.	7.14 MPa 1036 psi	3.55 MPa 515 psi	1.93 MPa 279 psi	1.00 MPa 146 psi
D - CONCRETE - 50cm(0.50m)/ 19.5 inches thick concrete. $\sigma_{c.conc}$ = 35 MPa/5000 psi after 28 days.	9.72 MPa 1410 psi	5.35 MPa 775 psi	3.04 MPa 440 psi	1.63 MPa 236 psi
E - STEEL SETS - (6 I 12) space 2m/79 in.. Blocked 2θ=22½°, σ_{ys} = 248MPa/36 000 psi.	0.61 MPa 88 psi	0.18 MPa 27 psi	0.07 MPa 10 psi	0.02 MPa 3 psi
F - STEEL SETS - (8 I 23) space 1.5m/59 in. Blocked 2θ=22½°, σ_{ys} = 248MPa/36 000 psi.	1.59 MPa 230 psi	0.50 MPa 72 psi	0.18 MPa 27 psi	0.06 MPa 9 psi
G - STEEL SETS - (12 W 65) at 1m/39 in. Blocked 2θ=22½°, σ_{ys} = 248MPa/36 000 psi.	7.28 MPa 1055 psi	2.53 MPa 366 psi	1.04 MPa 150 psi	0.38 MPa 55 psi
H - VERY LIGHT ROCKBOLTS - 16mm/⅝in. Ø at 2.5m/98in. centres. Mechanical anchor. T_{bf} = 0.11MN/25 000 lb.	0.02 MPa 2.6 psi	0.02 MPa 2.6 psi	0.02 MPa 2.6 psi	0.02 MPa 2.6 psi
I - LIGHT ROCKBOLTS - 19mm/¾". Ø at 2.0m/79in. Mechanical anchor. T_{bf} = 0.18MN/40 000 lb.	0.045MPa 6.5 psi	0.045MPa 6.5 psi	0.045MPa 6.5 psi	0.045MPa 6.5 psi
J - MEDIUM ROCKBOLTS - 25mm/1" Ø at 1.5m/59in centres. Mechanical anchor. T_{bf} = 0.267MN/60 000 lb.	0.12 MPa 17 psi	0.12 MPa 17 psi	0.12 MPa 17 psi	0.12 MPa 17 psi
K - HEAVY ROCKBOLTS - 34mm/1⅜" at 1m/39in centres. Resin anchored. T_{bf} = 345 MN/ 150 000 lb.	0.34 MPa 49 psi	0.34 MPa 49 psi	0.34 MPa 49 psi	0.34 MPa 49 psi



7-1654 (7-73)
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COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	
DETAILS Numerical Example. Ground-Support Interaction.			

A 35 ft (10.7m) diameter highway tunnel is driven in fair quality gneiss at a depth of 400 ft (122m) below surface. The following data are required to calculate the required support lines for the rock mass surrounding the tunnel :

Uniaxial compressive strength of rock	σ_c = 10,000 lb/in^2	(69 MPa)
Material constants for original rock	m = 0.5	
mass	s = 0.001	
Modulus of elasticity of rock mass	E = 2 x 10^5 lb/in^2	(1380 MPa)
Poisson's ratio of rock mass	ν = 0.2	
Material constants for broken rock	m_r = 0.1	
	s_r = 0	
Unit weight of broken rock	γ_r = 0.074 lb/in^3	(0.02 MN/m^3)
in situ stress magnitude	p_o = 480 lb/in^2	(3.31 MPa)
Tunnel radius	r_i = 210 in	(5.33m)

17.5 ft

Follow calculation sequence Table 14. pp. 05.

$M = 0.038$, $D = -0.617$, $N = 0.632$

p_i (psi)	Radius of Plastic Zone r_e (ft)	$r_e - r_i$ (ft)	$\gamma(r_e - r_i)$ (f
0	32.9	15.40	~~14.4~~ 13.7
10	26.95	9.45	8.4
20	24.85	7.35	6.5
30	23.28	5.78	5.1
40	22.05	4.55	4.1
60	20.12	2.62	2.3
80	18.72	1.22	1.1
100	17.50	0	0

$p_{ic} = p_o - M\sigma_c = \underline{100 \text{ psi}}$ $\Rightarrow$ if $p_i \geqslant 100$ psi elastic

$p_i < 100$ psi plastic failure occurs around the tunnel).

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	
DETAILS			

Sidewalls. p_i (psi).	Roof $p_i + \gamma(r_e - r_i)$ (psi)	Floor $p_i - \gamma(r_e - r_i)$ (psi)	Displc^t u_i (in)
0	13.7	—	5.03
20	26.5	13.5	1.65
40	44.1	35.9	1.03
60	62.3	57.7	0.62
80	81.1	78.9	0.59
100	100	100	0.48 ✓

Elastic solution:

p_i (psi)	Displc^t $u_i = r_i(1+\nu)(p_o - p_i)/E$ (in)
0	0.60
20	0.58
40	0.55
60	0.53
80	0.50
100	0.48

15

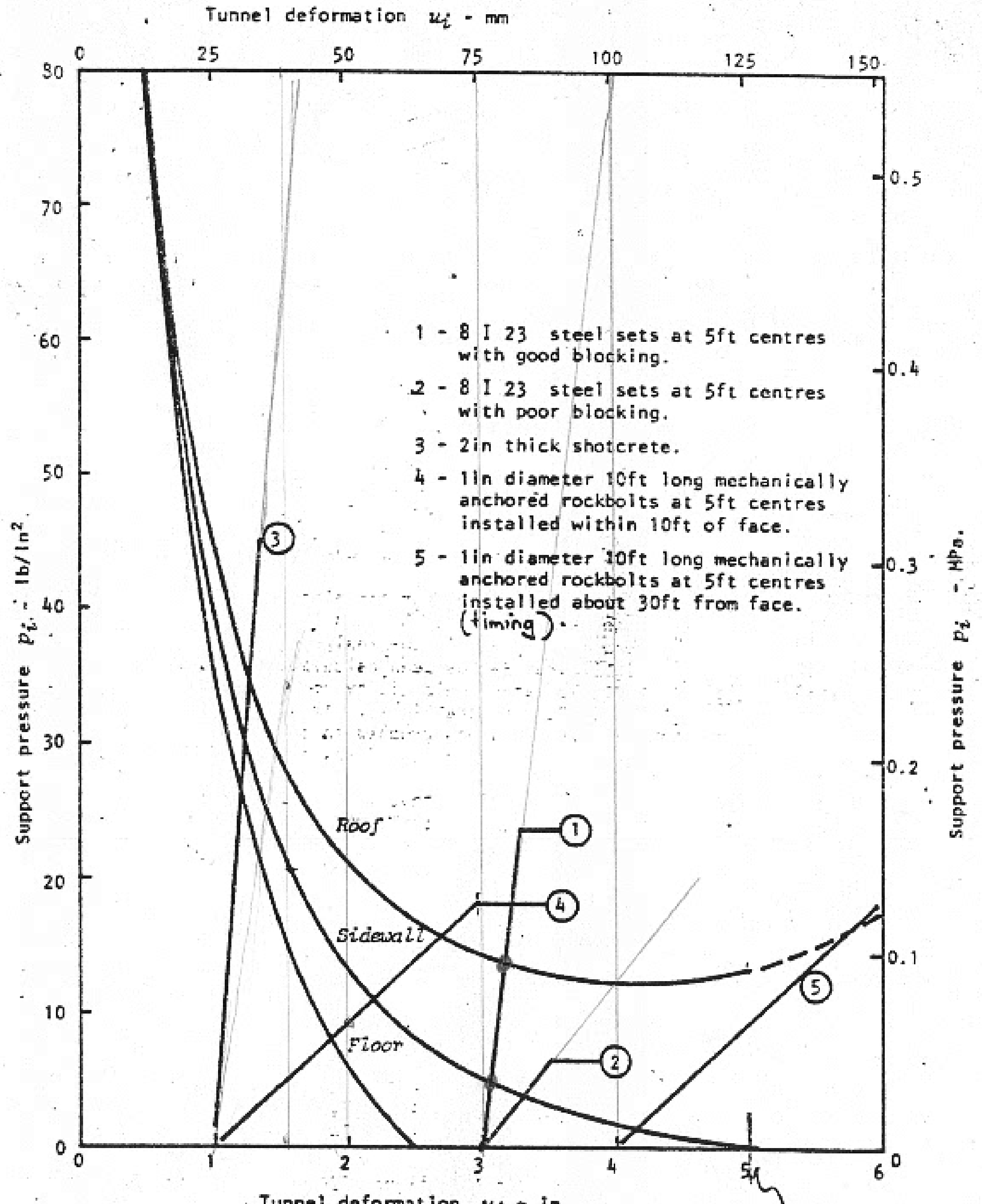

Figure 134 : Rock-support interaction analysis of a 35ft diameter tunnel in fair quality gneiss at a depth of 400 feet below surface.

7-1654 (7-73)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	
DETAILS	Support Curve Block Steel Sets.		

Block Steel Sets

2I 23 steel sets at 5 ft spacing

a)

Flange width of steel set	$W = 4.16$in	(0.1059 m)
Depth of section of steel set	$X = 8$in	(0.2023 m)
Cross-sectional area of steel	$A_s = 6.71$in^2	(0.0043 m^2)
Moment of inertia of steel section	$I_s = 64$in^4	(2.67×10^{-5}m^4)
Young's modulus of steel	$E_s = 30 \times 10^6$lb/in^2	(207000 MPa)
Yield strength of steel	$\sigma ys = 36000$lb/in^2	(245 MPa)
Tunnel radius	$r_i = 210$in	(5.33 m)
Set spacing	$S = 60$in 5ft	(1.52 m)
Half angle between blocking points	$\theta = 11.25°$	
Block thickness	$t_B = 10$in	(0.25 m)
Modulus of blocking material	$E_B = 1.5 \times 10^6$lb/in^2	(10,000 MPa) ← stiff blocking.
Deformation before support installation	$u_{io} = 3$in	(0.075 m)
In situ stress magnitude	$p_o = 480$lb/in^2	(3.31 MPa)

↳ From Table 14, pp. 67

$$\frac{1}{k_s} = \frac{60}{3\,10^7} \cdot \frac{210}{6.71} + \frac{60\,(210)^3}{3\,10^7 \times 64}\left[\frac{\pi}{180}\times 11.25\,\frac{\left(\frac{\pi}{180}\,11.25 + \sin 11.25\cos 11.25\right)}{2\sin^2 11.25} - 1\right]$$

$$+\ 2\times 60 \times \frac{\pi}{180}\times 11.25 \times 10 \Big/ 1.5\,10^{\cancel{7}\,6} \times (4.16)^2$$

$$\Rightarrow \underline{k_s = \cancel{1.37}\ 1.23\ 10^4 \text{psi}}$$

Maximum support pressure p_{smax} (from Table 17, pp) $p_{smax} \simeq 25$ psi

$$p_{smax} = \frac{3\times 6.71 \times 64 \times 36000}{2\times 60 \times 210 \times \frac{\pi}{180}\times 11.25\left[3\times 64 + 8\times 6.71\,(210-(10+4))(1-\cos 11.25\right.}$$

$$\underline{p_{smax} = 23.8 \text{ psi}}$$

Full face deformation: 5 in when face is between 1 and 1½ tunnel diameters e.g. 35 - 52.5 ft

⇒ 3 in. displacement will take place at a distance ranging between 21 - 31.5 ft from face.

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	
DETAILS			

b) Poor blocking $2\theta = 40°$ $\Rightarrow$ $\theta = 20°$

Modulus of blocking material $E_B = 72{,}000$ psi ~~soft blocking~~

$$\Rightarrow \frac{1}{k_s} = \frac{60 \times 210}{3\,10^7 \times 6.71} + \frac{60\,(210)^3}{3\,10^7 \times 64}\left[\frac{\pi}{180}\times 20\,\frac{\left(\frac{\pi}{180}20 + \frac{\sin 40}{2}\right)}{2\sin^2 20} - 1\right]$$

$$+\ 2\times 60\,\frac{\pi}{180}\times 20\times 10 \Big/ 72000\times(4.16)^2$$

$$\Rightarrow \underline{k_s \simeq 2014 \text{ psi}}$$

$$p_{s\,max} = \frac{3\times 6.71\times 64\times 36000}{2\times 60\times 210\times \frac{\pi}{9}\left(3\times 64 + 8\times 6.71\times 196\,(1-\cos 20)\right)}$$

$$\underline{p_{s\,max} = 6.4 \text{ psi}}$$

c) Note: Terzaghi classification. Moderately blocky and seamy.

$\Rightarrow$ rock load H_p: $0.25B - 0.35(B+H_t)$.

$B = H_t = 35$ ft $\Rightarrow$ H_p: 8.75 ft $- 24.5$ ft.

$\Rightarrow$ p_i: $7.8 - 21.8$ psi.

Compares well with support pressure in the roof predicted by ground-support interaction concept.

7-1654 (7-73)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	

DETAILS Support Curve 2 in Thickness shotcrete layer

Shotcrete

Modulus of elasticity of shotcrete	$E_c = 3 \times 10^6$ lb/in^2	(20700 MPa)
Poisson's ratio of shotcrete	$\nu_c = 0.25$	
Thickness of shotcrete layer	$t_c = 2$in	(0.05 m)
Tunnel radius	$r_i = 210$in	(5.33 m)
Compressive strength of shotcrete	$\sigma_{cc} = 5000$lb/in^2	(34.5 MPa)
Deformation before shotcrete placing	$u_{io} = 1$in	(0.025 m)
In situ stress magnitude	$p_o = 480$lb/in^2	(3.31 MPa)

↳ from Table 14, pp. 67

Deformation before shotcrete placing: 1 in. => shotcrete placed
↳ 10.5 ft behind the face of the tunnel.

$$k_c = 3\,10^6 \frac{\left((210)^2 - (208)^2\right)}{1.25\left(0.5(210)^2 + 208^2\right)}$$

$k_c = 3.07\ 10^4$ psi

$$p_{s\,max} = \frac{1}{2} \times 5000 \left(1 - \left(\frac{208}{210}\right)^2\right) = 47 \text{ psi}$$

$t_c = 1$ in $\rightarrow k_c = 1.53\ 10^4$

↳ $p_{s\,max} = 24$ psi

7-1654 (7-73)
Bureau of Reclamation

COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	
DETAILS	Support Curve Rock bolts. 1 in ø. Spacing 5 ft.		

Rock bolts

Rockbolt length	l = 120in	(3 m)	10 ft
Rockbolt diameter	d_b = 1in	(0.025 m)	
Modulus of elasticity of bolt steel	E_b = 30 x 10^6lb/in^2	(207,000 MPa)	
Anchor/head deformation constant	Q = 2.5 x 10^{-5}in/lb	(0.143 m/MN)	
Ultimate strength of bolt system	T_{bf}= 65,000lb	(0.285 MN)	
Tunnel radius	r_i = 210in	(5.33 m)	
Circumferential bolt spacing	s_c = 60in	(1.52 m)	
Longitudinal bolt spacing	s_l = 60in	(1.52 m)	
Deformation before bolt installation	u_{io}= 1in	(0.025 m)	
In situ stress magnitude	p_o = 480lb/in^2	(3.31 MPa)	

Ls from Table 14, pp. 07

Deformation before placing rock bolts 1 in. => bolts placed 7-10.5 ft behind the face of the tunnel

$$\frac{1}{k_b} = \frac{60 \times 60}{210}\left[\frac{4 \times 120}{\pi (1)^2 3 \, 10^7} + 2.5 \; 10^{-5}\right] \qquad \underline{k_b = 1938 \text{ psi}}$$

$$p_{smax} = \frac{65\,000}{60 \times 60} = \underline{18.06 \text{ psi}}$$

Deformation before placing rock bolts 4 in => bolts placed 28 - 42 ft behind the face

for In shotcrete.

Combinate shotcrete bolts.

$$k = k_b + k_s = 1938 + 15300 = \underline{17238 \text{ psi}}$$

7-1554 (7-73)
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COMPUTATION SHEET

BY	DATE	PROJECT	SHEET ___ OF ___
CHKD BY	DATE	FEATURE	
DETAILS			

Ground-Support Interaction - References -

1) Rabcewicz, L.V. The New Austrian Tunnelling Method. Water Power, vol. 16, 1964, pp. 453-457 and vol. 17, 1965, pp. 19-24.

2) Hoek, E. and Brown, E.T. Empirical Strength Criterion for Rock Masses, ASCE Jal. of Geotechnical Eng. Division, vol. 106, GT9, pp. 1013-1035.

3) Brown, E.T, Bray, J., Ladanyi, B., Hoek, E. Ground Response Curve for Rock Tunnels, ASCE Jal. of Geotechnical Eng. Division, vol. 109, No. 1, 1983.

4) Hoek, E. and Brown, E.T. Underground Excavations in Rock, IMM Publ, 1980. (Chapter 08).

NATM
New Austrian Tunneling Method

I. Definition (after Rabcewicz and Golser, 1973)

NATM is based on the principle that it is desirable to take utmost advantage of the capacity of the rock to support itself, by carefully and deliberately controlling the forces in the readjustment process which takes place in the surrounding rock after a cavity has been made, and to adapt the chosen support accordingly.

Generally two methods of support are carried out. The first is a flexible outer arch - or protective support - designed to stabilize the structure accordingly, and consists of a systematically anchored rock arch with surface protection mostly by shotcrete, possibly reinforced by additional ribs and closed by an invert.

The behavior of the protective support and the surrounding rock during the readjustment process is controlled by a sophisticated measuring system.

The second means of support is an inner arch consisting of concrete, and is generally not carried out before the outer arch has reached equilibrium. Its aim is to establish or increase the safety factors as necessary.

II. Major Elements of the NATM Philosophy (after Brown, 1981)

A. The inherent strength of the soil or rock surrounding the tunnel should be conserved and mobilized to the maximum extent possible.

B. Controlled deformation of the ground is required to develop its full strength safely. However, excessive deformation which will result in loss of strength or in unacceptable high surface settlements should be avoided.

C. These conditions may be achieved in a variety of ways, but generally a primary support system consisting of systematic rockbolting or anchoring and a thin semi-flexible shotcrete lining is used. Whatever support system is used, it is essential that it is placed and remains in intimate contact with the ground and deforms with it.

D. The timing of the placement of the support and of closing the initial shotcrete ring is of vital importance in controlling deformations and will vary from case to case.

E. The primary support will partly or completely represent the total support required. The dimensioning of the secondary support is based on an assessment of the results of systematic, measurements of stresses in the primary support elements and deformations of the tunnel surface and the ground surrounding the tunnel.

F. The length of tunnel left unsupported at any time during construction should be as short as possible. Where possible, the tunnel should be driven full face in minimum time with minimum disturbance of the ground by blasting.

G. All parties involved in the design and execution of project-design and supervisory engineers and the contractor's engineers and foremen-must understand and accept the NATM approach and adopt a co-operative attitude to decision making and the resolution of problems.

III. NATM References

- Rabcewicz, L. V. The New Austrian Tunnelling Method. Water Power, November 1964, pp. 453-457, December 1964, pp. 511-515, January 1965, pp. 19-24

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- Brown, E. T. Putting the NATM into perspective. Tunnels and Tunnelling, November 1981, pp. 13-17

- Golser, J. and Mussger, K. The NATM, Contractual Aspects, Tunneling in Difficult Ground, Kitamura Ed., Pergamon Press, 1978, pp. 387-392