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Numeriska beräknings- modeller för projektering av berganläggningar

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Numerical Modeling in the Geotechnical Design Process

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TRANSVERSE OPEN STOPING METHOD

Based on the orebody shape and grade, a longhole, transverse open stoping method is planned to extract the orebody. Initial stopes 20m wide by 40m high will be mined by blasting vertical faces drilled from an overcut drift. Pillars between the stopes are 20m in width. It is planned to mine a series of three stopes on the first sublevel, fill them, and mine a second series of stopes on the sublevel above, creating a total stope height of 80m in two lifts. The pillars are then to be mined by the same method between fill pillars. The question which must be asked is whether the pillars, as presently sized, will be stable and, if not, what must be done to provide a stable mining configuration.

As a first-cut analysis, a numerical model may be used to analyze this stoping sequence while examining the stability of the pillars. A stability analysis can be conducted using an elastic model and applying the calculated stresses to an empirical yield criterion such as the Hoek and Brown model (Hoek and Brown, 1980). In this case, a numerical model with a Mohr-Coulomb constitutive model is used directly to determine the induced pillar stresses and the yield state of the material at any stage of excavation.

The computer model FLAC, a PC-based explicit finite difference code, is used to provide a fast, gross, first-cut analysis of the stoping scheme. A finite difference grid for the analysis is shown in Fig. A1. The outer boundaries of the grid are placed approximately three stope diameters from the portion to be mined, and a pressure boundary is applied to maintain the in-situ stress at this position. The data file to set up this run is shown in Table A1. This data set is given to illustrate rather simple data input methodology possible with a "user-friendly" code.

The results of the sequential mining steps is illustrated in the series of plots given in Figs. A2-A8. These plots show the displacement vectors and x- and y-stress contours superimposed on a close-up view of the stope configuration. When the second stope is mined, the intervening pillar yields but comes to equilibrium. The yield behavior of any element can be traced using a state indicator, which allows the user to examine the location of the stress state of the element with respect to the yield surface. It is therefore possible, with an explicit code, to trace the evolution of the yield history of an element. An isolated section of the grid may be in active plastic flow while the overall grid is in global equilibrium.

This analysis indicates the 20m pillars become largely unloaded, particularly in the x-direction, as would be expected. The pillars yield, coming to equilibrium without backfill. It would be a simple matter to perform more detailed studies of varying pillar or stope spacings, rock mass properties, or backfill. This model used 400 finite difference zones and 441 gridpoints. On a standard IBM-PC/AT, each mining step requires about 6 hours to come to equilibrium. For a simulation requiring many mining steps, it may require several days for solution on a PC. Accelerator boards such as the Microway Number Smasher or the DSI-32 can increase the speed by a factor of 3 to 4. With this increased speed, the PC becomes competitive with a multi-user minicomputer environment.

JOB TITLE : Grid for example transverse stoping problem

FLAC (Version 1.01)

LEGEND

1/ 1/1980 0: 3
step 5000
-2.333E+02 < x < 4.333E+02
-1.833E+02 < y < 4.833E+02

Grid plot

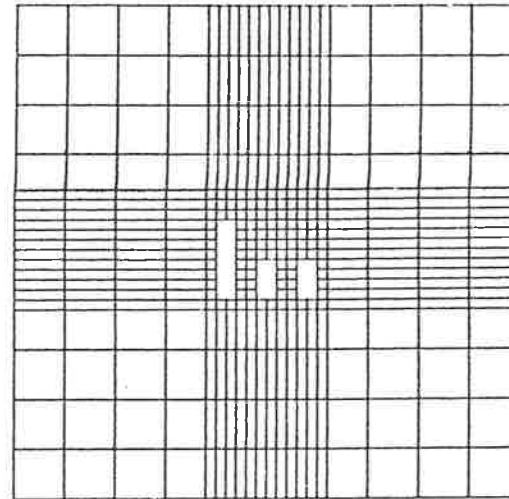


Fig. A1 Grid for Mining Example — 4 Stopes
Excavated at This Mining Stage

ITASCA Consulting Group, Inc.

Table A1

EXAMPLE DATA FILE FOR THE FLAC CODE FOR TRANSVERSE STOPING PROBLEM

```

*example problem of transverse open stoping
gri 20,20
*select mohr-coulomb model
mod mohr
*assign material properties
*E=3e6 psi, start with high cohesion
prop s=8.6e9 b=11.6e9 d=2000 fric=30 coh=1e10
*give the grid coordinates
gen 0,0 0,200 200,200 200,0
*deform edges of grid to simulate infinity
ini x=350 i=21
ini x=300 i=20
ini x=250 i=19
ini x=200 i=18
ini x=-150 i=1
ini x=-100 i=2
ini x=-50 i=3
ini x=0 i=4
ini y=350 j=21
ini y=300 j=20
ini y=250 j=19
ini y=200 j=18
ini y=-150 j=1
ini y=-100 j=2
ini y=-50 j=3
ini y=0 j=4
*create in situ stresses with pressure boundary cond.
ini sxx=-30e6 syy=-30e6
apply press=30e6 i=1 j=1,21
apply press=30e6 i=1,21 j=21
apply press=30e6 i=21 j=21,1
apply press=30e6 i=21,1 j=1
*save initial state
save stopel.sav
*use large strain logic
set large
*reset cohesion to lower level
prop coh=7e6
*mine out stope 1
mod null i=6,7 j=6,9
*time step to equilibrium
ste 1000
save stope2.sav

*reset displacements so we see only change not total
ini xd=0 yd=0
*mine stope 2
mod null i=10,11 j=6,9
*time step to equilibrium
ste 1000
save stope3.sav
ini xd=0 yd=0
mod null i=14,15 j=6,9
ste 1500
save stope4.sav
ini xd=0 yd=0
mod null i=6,7 j=10,13
ste 1500
save stope5.sav
ini xd=0 yd=0
mod null i=10,11 j=10,13
ste 1500
save stope6.sav
ini xd=0 yd=0
mod null i=14,15 j=10,13
ste 1500
save stope7.sav
return

```

JOB TITLE :

FLAC (Version 1.01)

LEGEND

26/11/1986 11.12

step 1000

$2.000E+01 < x < 1.800E+02$

$2.000E+01 < y < 1.800E+02$

Displacement vectors

Vector Length= $1.000E-02$

YY-stress contours

Contour interval= $2.000E+06$

(zero contour line omitted)

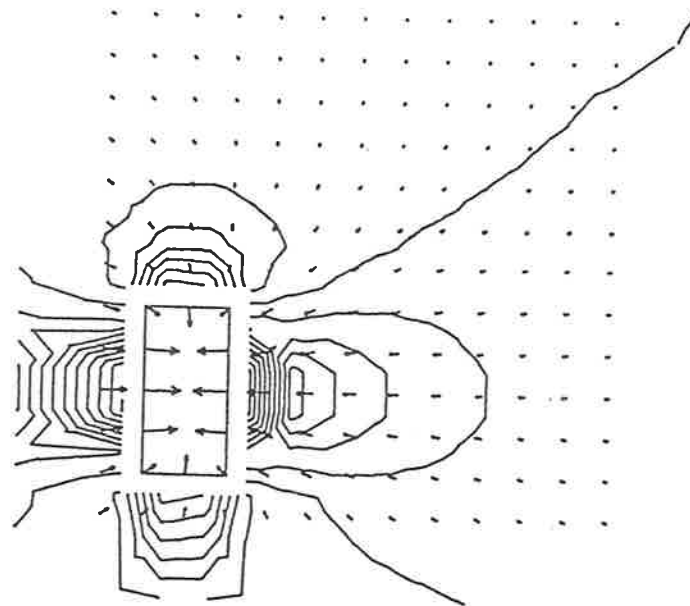


Fig. A 2

. Itasca Consulting Group, Inc.

JOB TITLE : Second stoper, first cut mined

FLAC (Version 1.01)

LEGEND

```
26/11/1986  11: 4
step 2000
2.000E+01 < x < 1.800E+02
2.000E+01 < y < 1.800E+02
```

Displacement vectors
Vector Length= 1.000E-02
YY-stress contours
Contour interval= 2.500E+06
(zero contour line omitted)

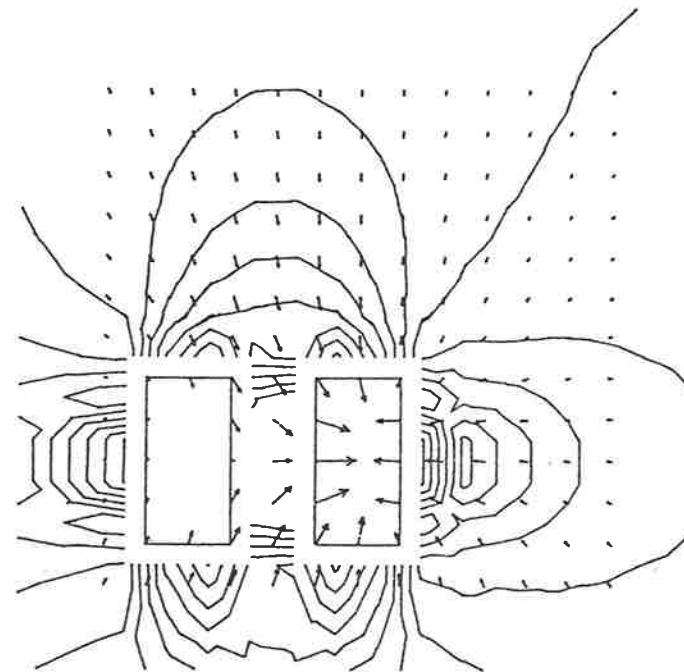


Fig. A3

JOB TITLE : First cut mined

FLAC (Version 1.01)

LEGEND

26/11/1986 10.55

#step 3500

2.000E+01 < x < 1.800E+02

2.000E+01 < y < 1.800E+02

Displacement vectors

Vector Length= 2.500E-02

YY-stress contours

Contour interval= 2.500E+06

(zero contour line omitted)

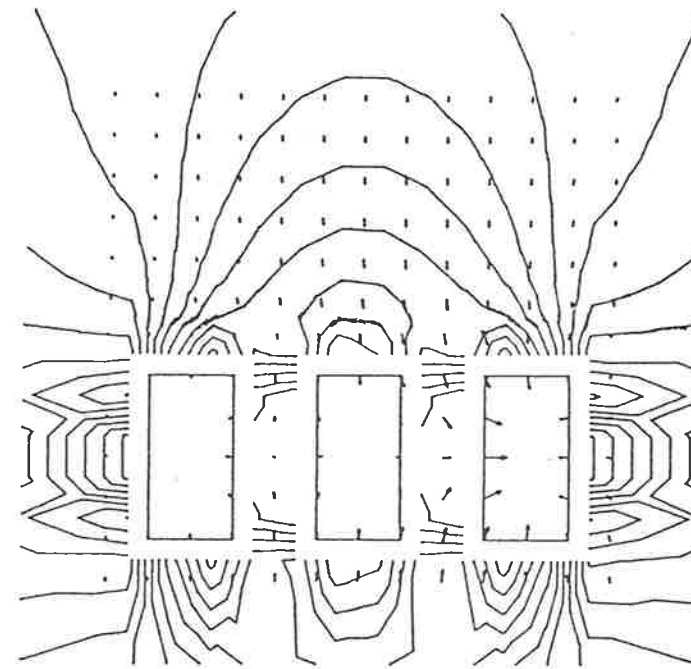


Fig. A4

Itasca Consulting Group, Inc.

JOB TITLE : Second lift on transverse slopes begin

FLAC (Version 1.01)

LEGEND

26/11/1986 10.30

step 5000

$2.000E+01 < x < 1.800E+02$

$2.000E+01 < y < 1.800E+02$

Displacement vectors

Vector Length= $2.500E-02$

YY-stress contours

Contour interval= $2.500E+06$

(zero contour line omitted)

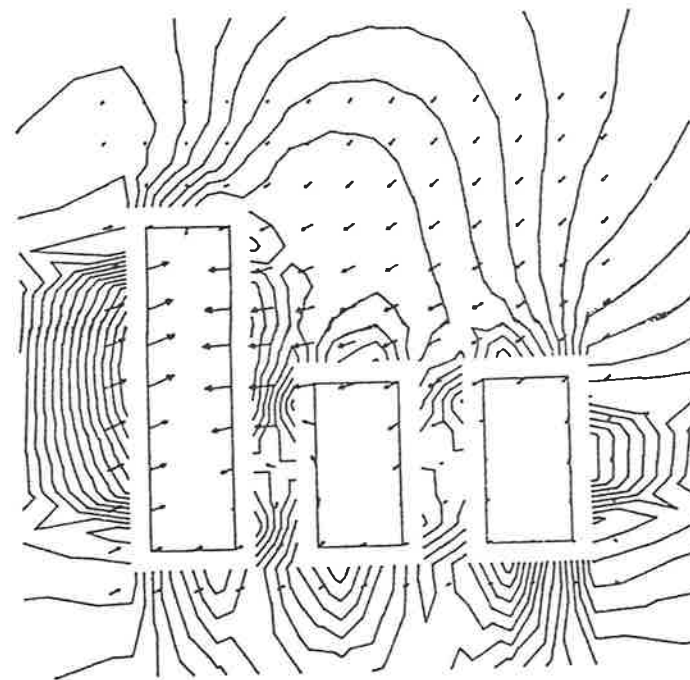


Fig. A5

Itoeco Consulting Group, Inc.

JOB TITLE : Second lift on transverse stopes begin

FLAC (Version 1.01)

LEGEND

26/11/1986 10.17

step 5000

$2.000E+01 < x < 1.800E+02$

$2.000E+01 < y < 1.800E+02$

Principal stresses

Max. Stress= $6.926E+07$

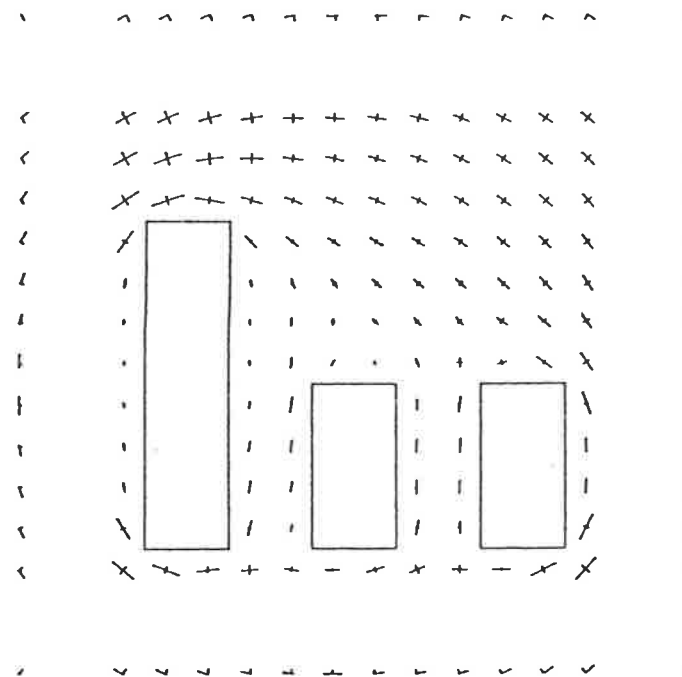


Fig. A6

JOB TITLE : Three transverse stopes mined

FLAC (Version 1.01)

LEGEND

25/11/1986 9:58

step 6600

$2.000E+01 < x < 1.800E+02$

$2.000E+01 < y < 1.800E+02$

Displacement vectors

Vector Length= $1.000E-02$

XX-stress contours

Contour interval= $5.000E+06$

(zero contour line omitted)

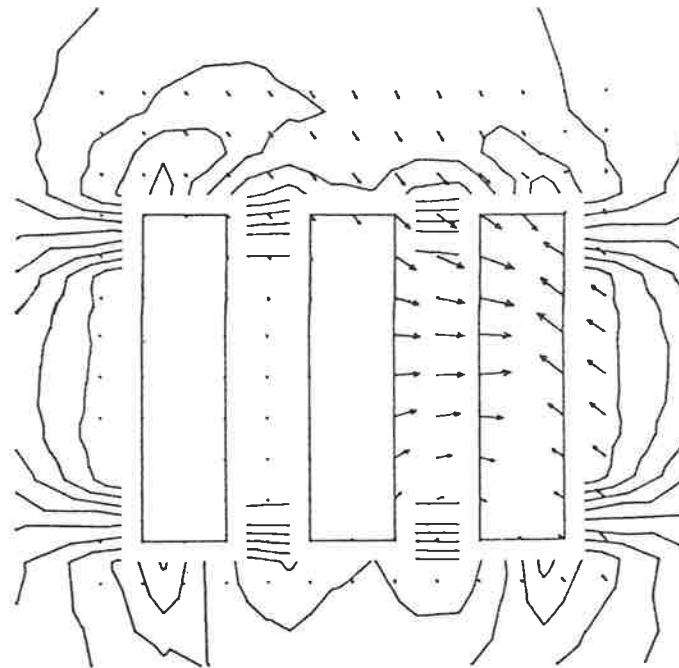


Fig. A7

. Itasca Consulting Group, Inc.

JOB TITLE :

FLAC (Version 1.01)

LEGEND

26/11/1986 11.12

step 1000

$2.000E+01 < x < 1.800E+02$

$2.000E+01 < y < 1.800E+02$

Displacement vectors

Vector Length= $1.000E-02$

YY-stress contours

Contour interval= $2.000E+06$

(zero contour line omitted)

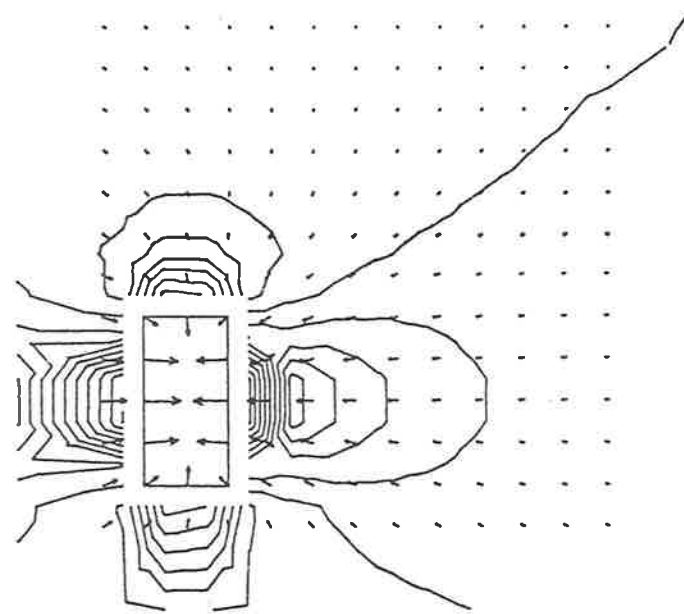


Fig. A8

Itasca Consulting Group, Inc.

SILO FÖR RADIOAKTIVT AVFALL

Följande exempel är hämtat från SFR, där man låtit bygga en anläggning för förvaring av låg- och medelaktivt reaktoravfall. Exemplet beskriver en stående cylinder (silo) med höjden ca 70 m och diametern ca 30 m. Modellsnittet är horisontellt genom silon och rådande initialspänningstillstånd har satts till 5 resp 10 MPa i x- respektive y-led. Beräkningen har utförts med FLAC version 1.00. Samtliga in-data är hämtade från SKB's arbetsrapport SFR 85-05, "FEM-analys av silo med anslutande tunnlar", H Stille m fl. Beräkningen med FLAC är utförd av VIAK AB, Falun, som en övning och test av FLAC's korrelation med FEM.

Modellen

Eftersom silon utgörs av en cirkel i horisontalplanet kan symmetrierna utnyttjas, vilket gör att endast en fjärdedel av geometrin behöver modelleras. Modellens dimensioner har valts till

90 x 90 m, vilket bör vara betryggande, eftersom silons radie är ca 15 m. En tumregel säger att ränderna bör ligga minst på ett avstånd som motsvarar tre ggr största öppningsmåttet.

Följande randvillkor har valts:

vänster (vertikal symmetrilinje) :	fri i y-led, låst i x-led
höger :	låst i både x- och y-led
nedre (horisontell symmetrilinje):	fri i x-led, låst i y-led
övre :	låst i både x- och y-led

Bergmassan har antagits uppträda elastiskt isotropt och ha elasticitetsmodulen $E = 20 \text{ GPa}$ och Poissons tal $\nu = 0,08$. Dessa numeriska värden måste räknas om till skjuvmodul och bulkmodul med följande formler:

$$\text{Bulkmodulen } B = \frac{E}{3(1-2\nu)} \quad \text{Skjuvmodulen } G = \frac{E}{2(1+\nu)}$$

Bergmassans densitet har satts till $\rho = 2670 \text{ kg/m}^3$.

I figur 1 visas en skiss över den använda modellen.

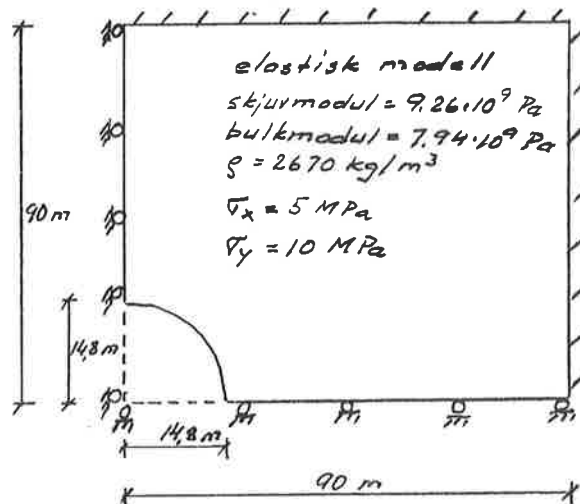


Fig 1. Den använda modellen.

Indata - körning

Det finita differensnätet har valts till 34×34 element med tätare elementindelning närmare siloväggarna (se figur 2).

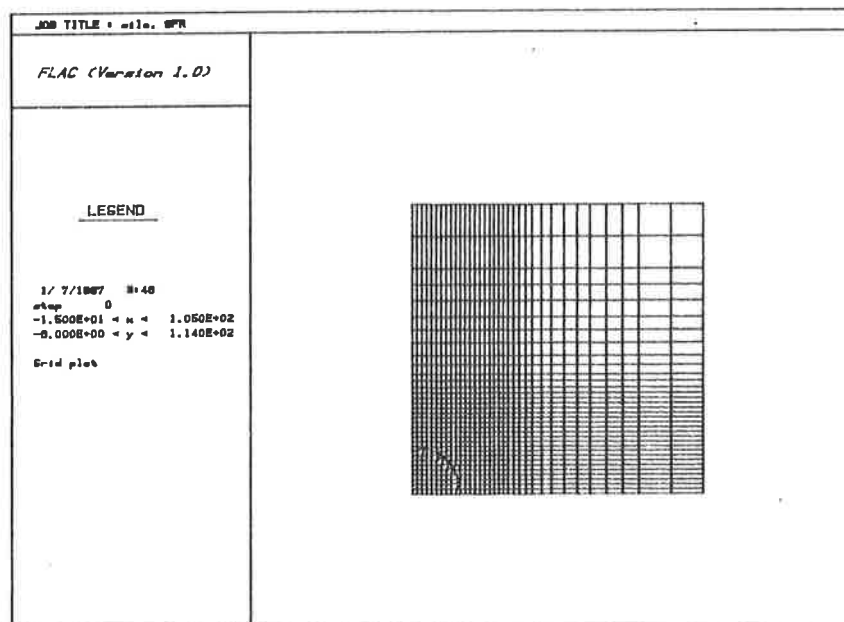


Fig 2. Finita differensnätet.

Följande lista (figur 3) visar de indata som modellen behöver samt hur körningen utförts. Indata kan antingen ges interaktivt eller via en i förväg iordninggjord indatafil.

```
*GRID
g 34,34

*ELASTISK MATERIALMODELL
model elas

*MATERIALPARAMETRAR
prop shear=9.26e9 bulk=7.94e9 dens=2670

*FORMA NATET
ini x=90 i=35
ini x=80 i=34
ini x=70 i=33
ini x=65 i=32
ini x=60 i=31
ini x=55 i=30
ini x=51 i=29
ini x=47 i=28
ini x=43 i=27
ini x=40 i=26
ini x=37 i=25
ini x=35 i=24
ini x=33 i=23
ini x=31.3 i=22
ini x=29.7 i=21
ini x=28.2 i=20
ini x=26.7 i=19
ini x=25.2 i=18
ini x=23.8 i=17
ini x=22.3 i=16
ini x=20.8 i=15
ini x=19.3 i=14
ini x=17.8 i=13
ini x=16.3 i=12
ini x=14.8 i=11
ini x=13.4 i=10
ini x=11.9 i=9
ini x=10.4 i=8
ini x=8.91 i=7
ini x=7.42 i=6
ini x=5.94 i=5
ini x=4.45 i=4
ini x=2.97 i=3
ini x=1.48 i=2
ini x=0.00 i=1

ini y=90 j=35
ini y=80 j=34
ini y=70 j=33
ini y=65 j=32
ini y=60 j=31
ini y=55 j=30
ini y=51 j=29
ini y=47 j=28
ini y=43 j=27
ini y=40 j=26
ini y=37 j=25
ini y=35 j=24
ini y=33 j=23
ini y=31.3 j=22
ini y=29.7 j=21
ini y=28.2 j=20
ini y=26.7 j=19

ini y=25.2 j=18
ini y=23.8 j=17
ini y=22.3 j=16
ini y=20.8 j=15
ini y=19.3 j=14
ini y=17.8 j=13
ini y=16.3 j=12
ini y=14.8 j=11
ini y=13.4 j=10
ini y=11.9 j=9
ini y=10.4 j=8
ini y=8.91 j=7
ini y=7.42 j=6
ini y=5.94 j=5
ini y=4.45 j=4
ini y=2.97 j=3
ini y=1.48 j=2
ini y=0.00 j=1

*FORMA TUNNELPERIFERIN
ini x=1.48 y=14.73 i=2 j=11
ini x=2.97 y=14.5 i=3 j=11
ini x=4.45 y=14.12 i=4 j=10
ini x=5.94 y=13.56 i=5 j=10
ini x=7.42 y=12.81 i=6 j=10
ini x=8.91 y=11.82 i=7 j=9
ini x=10.4 y=10.53 i=8 j=8
ini x=11.9 y=8.8 i=9 j=7
ini x=12.81 y=7.42 i=10 j=6
ini x=13.56 y=5.94 i=10 j=5
ini x=14.12 y=4.45 i=10 j=4
ini x=14.5 y=2.97 i=11 j=3
ini x=14.73 y=1.48 i=11 j=2
ini x=13.75 y=14.35 i=3 j=10
ini x=8.15 y=12.4 i=6 j=9
ini x=9.6 y=11.3 i=7 j=8
ini x=11.3 y=9.6 i=8 j=7
ini x=12.4 y=8.15 i=9 j=6
ini x=14.35 y=3.75 i=10 j=3

*PIXA TILL DET LITE UTANFÖR PERIFERIN
ini x=7.42 y=14.5 i=6 j=11
ini x=14.5 y=7.42 i=11 j=6
ini x=8.91 y=13.0 i=7 j=10
ini x=13.0 y=6.91 i=10 j=7

*RANDVILLKOR OCH INITIALSPANNINGAR
fix x i=1 j=1,35
fix y i=1,35 j=1
fix x y i=35 j=1,35
fix x y i=1,35 j=35
ini sxx=-5e6 syy=-10e6

*LOSNING AV INITIALSPANNINGSTILLSTADET
solve
save silo1.sav

*YTBRYTNING AV SILON
set large
ini xd=0 yd=0
model null i=1,10 j=1,2
modell null i=1,9 j=3,5
model null i=1,8 j=6
modell null i=1,7 j=7
modell null i=1,6 j=8
modell null i=1,5 j=9
modell null i=1,2 j=10

*KORNING AV TIDSTEGSSERIE 500, 2000, 3000, 5000
step 500
save silo2.sav
step 1500
save silo3.sav
step 1000
save silo4.sav
step 2000
save silo5.sav
return
```

Fig 3. Körningslista.

Först löses initialspänningstillståndet och sparas med fördel på en separat fil. Detta utförs med kommandona "solve" respektive "save" (se lista figur 3). Därefter stegas beräkningen fram i tidssteg med kommandot "step". För att kontrollera när beräkningen börjar närma sig jämvikt kan man t ex välja en nod (lämpligen en i vilken man kan förvänta sig relativt stora deformationer) och se hur förskjutningen i en riktning förändras med avseende på antal tidssteg (se figur 4).

tidssteg	förskjutning (mm)
500	7,7
2000	10,2
3000	10,4 (10,37)
5000	10,4 (10,39)

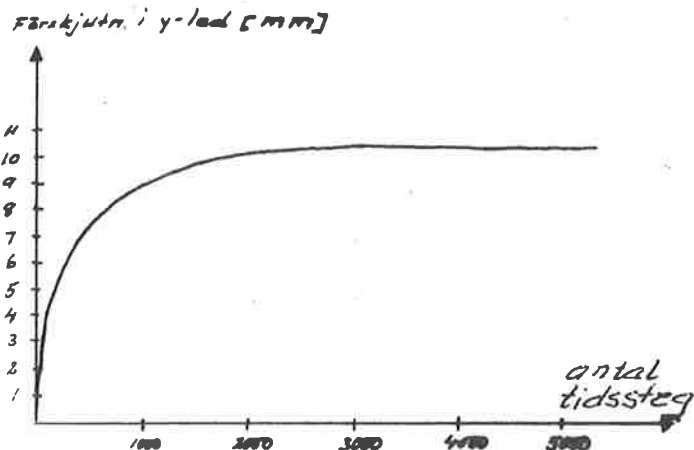


Fig 4. Förskjutning i y-led i noden $i = 1$, $j = 11$.

Man kan se att ändringen i förskjutningen de sista 2000 tidsstegen blev mycket liten, därför hade körningen kunnat stoppas redan efter ca 3000 tidssteg, utan att resultatet hade förändrats nämnvärt.

Utdata

Nedan visas exempel på lista för förskjutningarna i y-led efter 5000 tidssteg (figur 5). I figur 6-10 visas exempel på plottar för området närmast siloväggen.

y displ. (multiply values below by 10^{-2})

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
35	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
34	-0.051	-0.051	-0.050	-0.050	-0.049	-0.049	-0.048	-0.047	-0.047	-0.046	-0.045	-0.044	-0.043	-0.042	-0.041	-0.039	-0.038	-0.037	-0.036	-0.034	-0.033	-0.032	-0.030	-0.030
33	-0.108	-0.108	-0.107	-0.107	-0.106	-0.105	-0.104	-0.102	-0.100	-0.098	-0.096	-0.094	-0.092	-0.089	-0.087	-0.084	-0.081	-0.078	-0.075	-0.073	-0.070	-0.067	-0.063	-0.059
32	-0.140	-0.140	-0.139	-0.139	-0.137	-0.136	-0.134	-0.132	-0.129	-0.127	-0.124	-0.121	-0.117	-0.114	-0.110	-0.107	-0.103	-0.099	-0.095	-0.091	-0.087	-0.083	-0.079	-0.074
31	-0.175	-0.175	-0.174	-0.173	-0.171	-0.169	-0.167	-0.164	-0.160	-0.157	-0.153	-0.149	-0.144	-0.140	-0.135	-0.130	-0.125	-0.120	-0.115	-0.110	-0.105	-0.099	-0.094	-0.087
30	-0.214	-0.214	-0.213	-0.211	-0.209	-0.206	-0.203	-0.199	-0.194	-0.189	-0.184	-0.179	-0.173	-0.167	-0.160	-0.154	-0.147	-0.141	-0.135	-0.128	-0.122	-0.115	-0.108	-0.100
29	-0.249	-0.249	-0.247	-0.245	-0.242	-0.239	-0.234	-0.229	-0.223	-0.217	-0.211	-0.204	-0.196	-0.189	-0.181	-0.173	-0.165	-0.158	-0.150	-0.142	-0.134	-0.126	-0.118	-0.109
28	-0.288	-0.287	-0.285	-0.283	-0.279	-0.274	-0.269	-0.262	-0.255	-0.247	-0.239	-0.230	-0.221	-0.212	-0.202	-0.192	-0.183	-0.174	-0.164	-0.155	-0.146	-0.137	-0.127	-0.117
27	-0.331	-0.331	-0.328	-0.325	-0.320	-0.314	-0.307	-0.298	-0.289	-0.279	-0.269	-0.258	-0.246	-0.235	-0.223	-0.211	-0.200	-0.189	-0.178	-0.167	-0.156	-0.146	-0.135	-0.123
26	-0.368	-0.368	-0.365	-0.360	-0.354	-0.347	-0.338	-0.328	-0.316	-0.304	-0.292	-0.279	-0.266	-0.252	-0.238	-0.225	-0.211	-0.199	-0.187	-0.174	-0.163	-0.151	-0.139	-0.126
25	-0.410	-0.409	-0.406	-0.400	-0.392	-0.383	-0.372	-0.359	-0.346	-0.331	-0.317	-0.301	-0.285	-0.269	-0.253	-0.237	-0.222	-0.208	-0.194	-0.180	-0.168	-0.155	-0.142	-0.127
24	-0.441	-0.440	-0.436	-0.429	-0.420	-0.409	-0.397	-0.382	-0.366	-0.349	-0.333	-0.315	-0.297	-0.279	-0.262	-0.245	-0.228	-0.213	-0.198	-0.183	-0.170	-0.156	-0.142	-0.128
23	-0.475	-0.473	-0.468	-0.461	-0.450	-0.438	-0.423	-0.406	-0.388	-0.368	-0.350	-0.330	-0.310	-0.290	-0.270	-0.251	-0.233	-0.217	-0.201	-0.185	-0.171	-0.157	-0.142	-0.127
22	-0.506	-0.504	-0.499	-0.490	-0.478	-0.463	-0.446	-0.427	-0.406	-0.385	-0.364	-0.342	-0.320	-0.298	-0.277	-0.256	-0.237	-0.220	-0.203	-0.186	-0.171	-0.156	-0.142	-0.126
21	-0.538	-0.536	-0.530	-0.520	-0.506	-0.489	-0.470	-0.448	-0.425	-0.400	-0.377	-0.353	-0.329	-0.305	-0.282	-0.260	-0.240	-0.222	-0.203	-0.186	-0.171	-0.155	-0.140	-0.124
20	-0.571	-0.568	-0.561	-0.550	-0.534	-0.515	-0.492	-0.468	-0.442	-0.415	-0.390	-0.363	-0.336	-0.311	-0.286	-0.263	-0.241	-0.222	-0.203	-0.186	-0.169	-0.154	-0.138	-0.122
19	-0.606	-0.604	-0.595	-0.582	-0.564	-0.542	-0.517	-0.489	-0.460	-0.430	-0.402	-0.372	-0.343	-0.316	-0.289	-0.263	-0.242	-0.222	-0.202	-0.184	-0.167	-0.151	-0.136	-0.119
18	-0.645	-0.642	-0.632	-0.617	-0.596	-0.571	-0.542	-0.511	-0.478	-0.444	-0.413	-0.381	-0.349	-0.320	-0.292	-0.266	-0.242	-0.221	-0.200	-0.182	-0.165	-0.148	-0.132	-0.116
17	-0.685	-0.681	-0.670	-0.652	-0.628	-0.599	-0.566	-0.531	-0.495	-0.458	-0.423	-0.388	-0.354	-0.322	-0.293	-0.265	-0.240	-0.219	-0.198	-0.178	-0.161	-0.145	-0.129	-0.113
16	-0.731	-0.726	-0.713	-0.692	-0.665	-0.631	-0.594	-0.554	-0.513	-0.471	-0.433	-0.395	-0.358	-0.324	-0.292	-0.264	-0.237	-0.215	-0.194	-0.174	-0.157	-0.140	-0.124	-0.108
15	-0.782	-0.777	-0.761	-0.737	-0.704	-0.666	-0.623	-0.577	-0.530	-0.484	-0.442	-0.400	-0.360	-0.324	-0.290	-0.260	-0.233	-0.210	-0.188	-0.169	-0.151	-0.135	-0.119	-0.104
14	-0.838	-0.832	-0.814	-0.785	-0.747	-0.702	-0.652	-0.600	-0.547	-0.496	-0.450	-0.403	-0.361	-0.322	-0.287	-0.255	-0.227	-0.204	-0.182	-0.162	-0.145	-0.129	-0.114	-0.098
13	-0.900	-0.893	-0.872	-0.838	-0.794	-0.741	-0.683	-0.624	-0.564	-0.506	-0.455	-0.405	-0.359	-0.318	-0.281	-0.249	-0.220	-0.197	-0.174	-0.155	-0.138	-0.122	-0.107	-0.092
12	-0.967	-0.959	-0.935	-0.896	-0.844	-0.782	-0.715	-0.647	-0.579	-0.514	-0.458	-0.403	-0.354	-0.311	-0.273	-0.240	-0.211	-0.188	-0.166	-0.147	-0.130	-0.114	-0.100	-0.086
11	-1.039	-1.034	-1.019	-0.961	-0.900	-0.836	-0.750	-0.671	-0.593	-0.520	-0.458	-0.399	-0.347	-0.302	-0.263	-0.229	-0.200	-0.177	-0.156	-0.137	-0.121	-0.106	-0.093	-0.079
10	-1.003	-0.992	-0.953	-0.886	-0.794	-0.693	-0.604	-0.523	-0.455	-0.391	-0.337	-0.290	-0.247	-0.203	-0.168	-0.141	-0.119	-0.103	-0.088	-0.076	-0.066	-0.057	-0.049	-0.042
9	-0.856	-0.825	-0.717	-0.614	-0.522	-0.447	-0.379	-0.322	-0.274	-0.235	-0.201	-0.174	-0.152	-0.132	-0.115	-0.101	-0.088	-0.077	-0.065	-0.055	-0.045	-0.035	-0.025	-0.017
8	-0.789	-0.735	-0.619	-0.515	-0.433	-0.361	-0.302	-0.255	-0.216	-0.184	-0.157	-0.137	-0.119	-0.103	-0.088	-0.076	-0.066	-0.057	-0.049	-0.042	-0.035	-0.025	-0.017	-0.008
7	-0.670	-0.614	-0.528	-0.411	-0.337	-0.278	-0.231	-0.194	-0.164	-0.139	-0.121	-0.104	-0.090	-0.078	-0.068	-0.059	-0.050	-0.042	-0.035	-0.025	-0.017	-0.008	-0.000	-0.000
6	-0.585	-0.515	-0.397	-0.304	-0.247	-0.203	-0.168	-0.141	-0.119	-0.103	-0.088	-0.076	-0.066	-0.057	-0.049	-0.042	-0.035	-0.025	-0.017	-0.008	-0.000	-0.000	-0.000	-0.000
5	-0.416	-0.335	-0.263	-0.114	-0.273	-0.210	-0.165	-0.132	-0.108	-0.089	-0.075	-0.064	-0.054	-0.047	-0.040	-0.035	-0.030	-0.025	-0.020	-0.015	-0.010	-0.005	-0.000	-0.000
4	-0.257	-0.204	-0.147	-0.102	-0.076	-0.058	-0.046	-0.037	-0.031	-0.025	-0.022	-0.018	-0.016	-0.014	-0.012	-0.010	-0.008	-0.006	-0.005	-0.004	-0.003	-0.002	-0.001	-0.000
3	-0.102	-0.076	-0.058	-0.046	-0.037	-0.031	-0.025	-0.022	-0.018	-0.016	-0.014	-0.012	-0.010	-0.008	-0.006	-0.005	-0.004	-0.003	-0.002	-0.001	-0.000	-0.000	-0.000	-0.000
2	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Fig 5. Lista för förskjutningarna i y-led efter 5000 tidssteg.

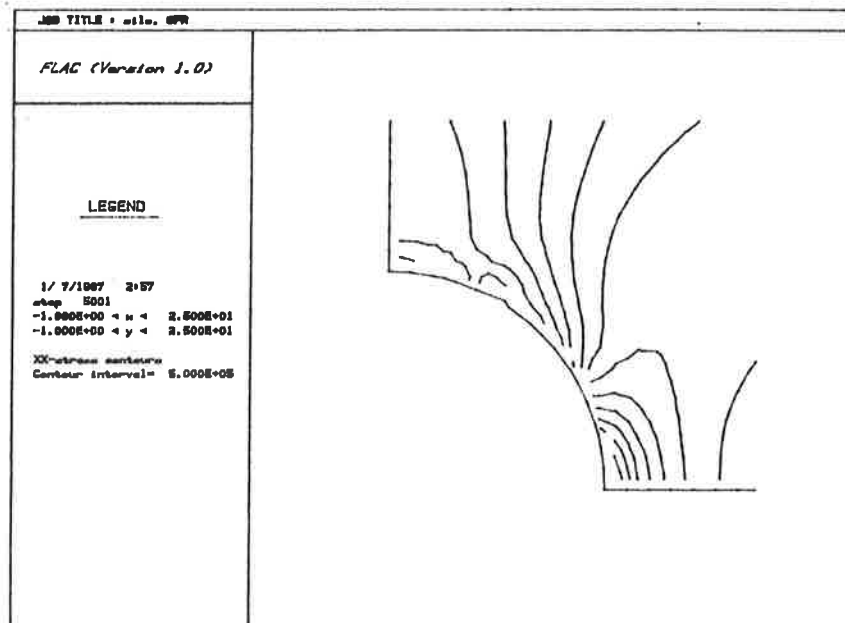


Fig 6. Isolinjer för spänningar i x-led.

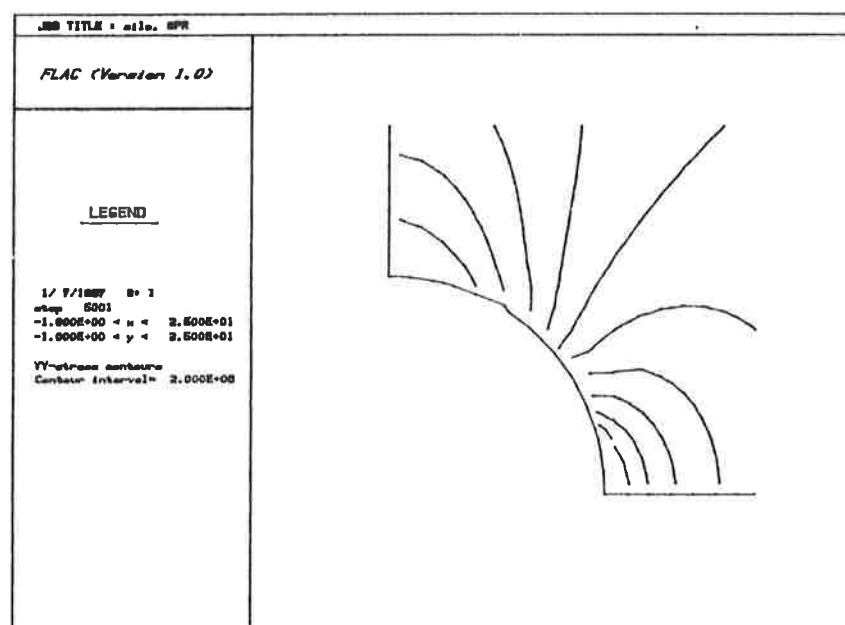


Fig 7. Isolinjer för spänningar i y-led.

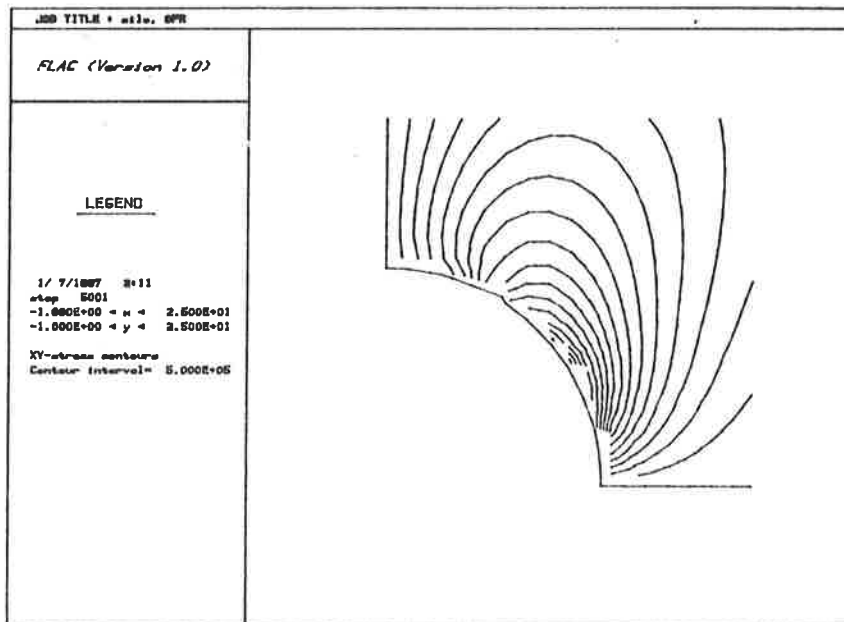


Fig 8. Isolinjer för skjuvspänningar.

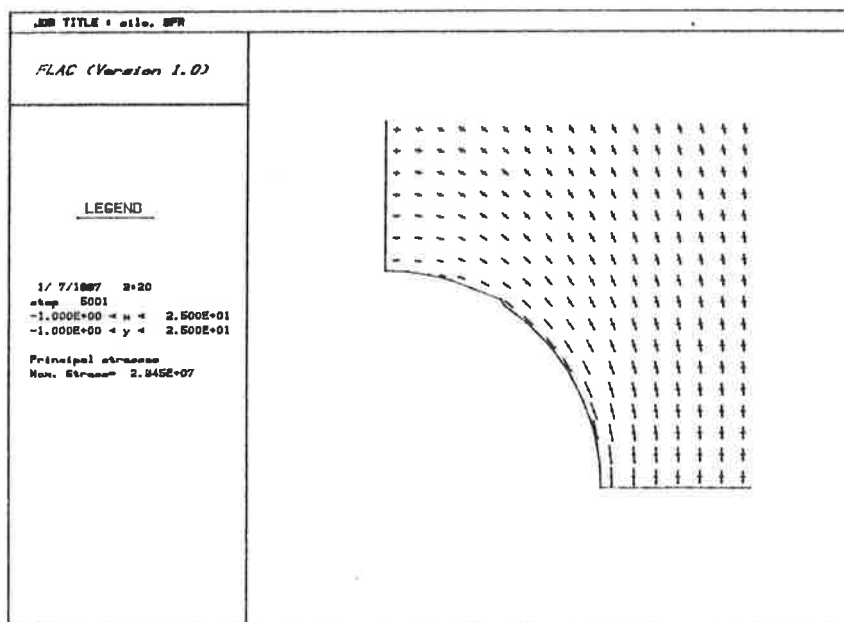


Fig 9. Huvudspänningsvektorer.

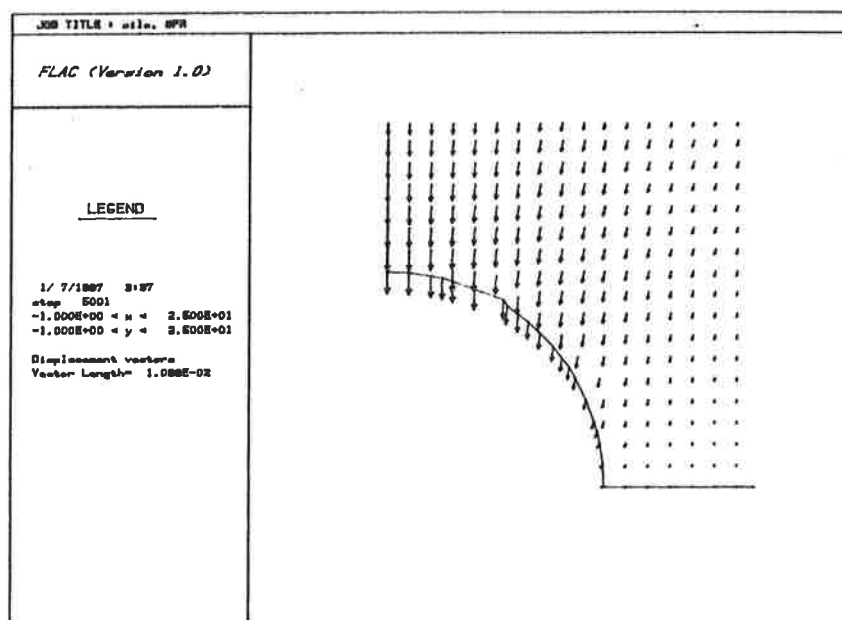


Fig 10. Förskjutningsvektorer

Jämförelse FLAC - analytisk lösning

För att få en uppfattning om hur väl FLAC stämmer med den analytiska lösningen av problemet har de radiella och tangentiella spänningarna beräknats utefter symmetrilinjerna från siloväggen och utåt. Härvid har Kirsch's ekvationer för en cirkulär öppning i ett oändligt medium utnyttjats. Den sekundära tangential- respektive radialspänningen i en punkt ges av:

$$\sigma_t = \frac{\sigma_x + \sigma_y}{2} \left(1 + \frac{a^2}{r^2}\right) - \frac{\sigma_x - \sigma_y}{2} \left(1 + \frac{3a^4}{r^4}\right) \cos 2\theta$$

respektive

$$\sigma_r = \frac{\sigma_x + \sigma_y}{2} \left(1 - \frac{a^2}{r^2}\right) + \frac{\sigma_x - \sigma_y}{2} \left(1 + \frac{3a^4}{r^4} - \frac{4a^2}{r^2}\right) \cos 2\theta$$

där

a = cirkelns radie

r = avståndet från cirkelns centrum till beräkningspunkten

σ_x = initialspänningen i x-led

σ_y = initialspänningen i y-led

θ = vinkeln mellan x-axeln och en linje mellan cirkelns centrum och beräkningspunkten

Om $\theta = 0^\circ$ är σ_t = sekundärspänningen i y-led och σ_r = sekundärspänningen i x-led. Då $\theta = 90^\circ$ blir förhållandet omvänt. I figur 11 visas jämförelsen mellan lösningen med FLAC och den analytiska lösningen.

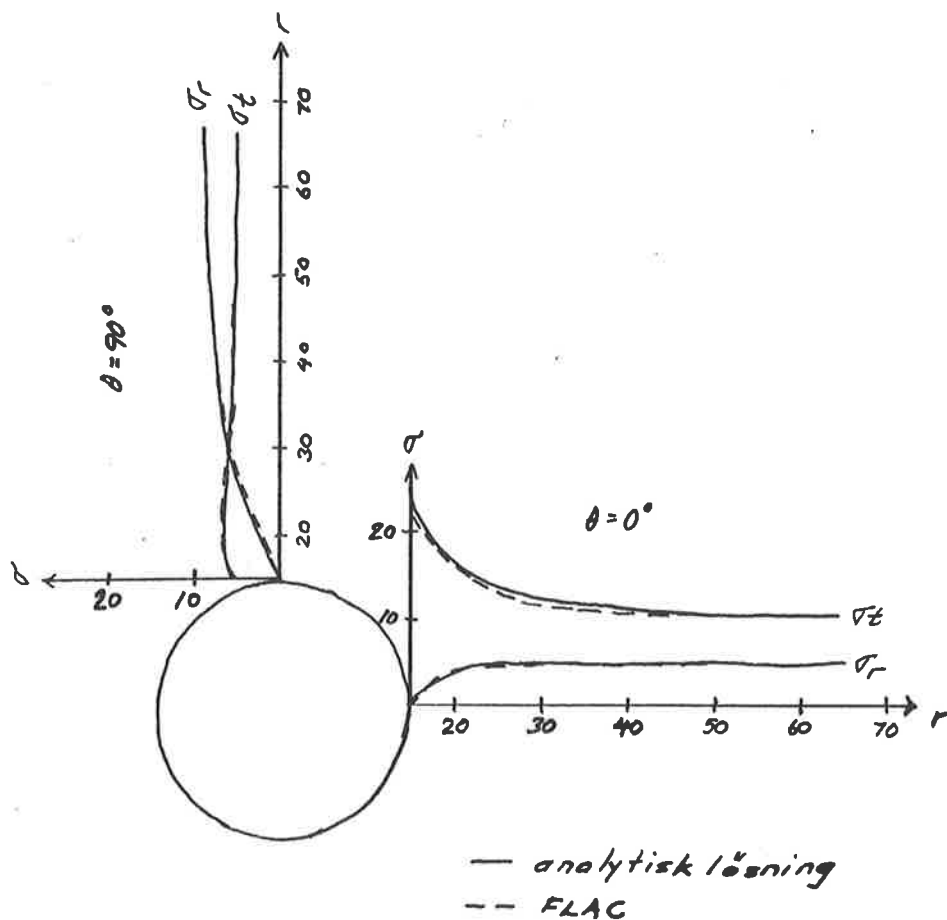


Fig 11. Jämförelse mellan analytisk lösning och FLAC.

EXEMPEL PÅ FINITA ELEMENTMETODEN I BERGMEKANIK

Problemställning :

Två tunnlar skall drivas nära varandra i ett område där horisontalspänningarna är 5 gånger större än vertikalspänningarna. Studera omfördelningarna av spänningar omkring och mellan tunnelarna.

Lösning :

Med GP4 som är en generell Pre- och PostProcessor för Finita Elementanalyser tillverkas indata till FEM programmet. GP4 använder den s.k. CAD-approachen, vilket innebär att endast ytterkonturer och begränsningslinjer behöver modelleras. På så sätt reduceras geometriska indata till ett minimum, samtidigt som fortfarande full flexibilitet för avancerade geometrier finns. Ett antal regioner definieras grafiskt med hjälp av linjer som i sin tur definieras från punkter. I dessa regioner kan elementnät genereras. Modellens ränder skall ligga tillräckligt långt från tunnelarna för att deras inverkan inte ska påverka resultatet.

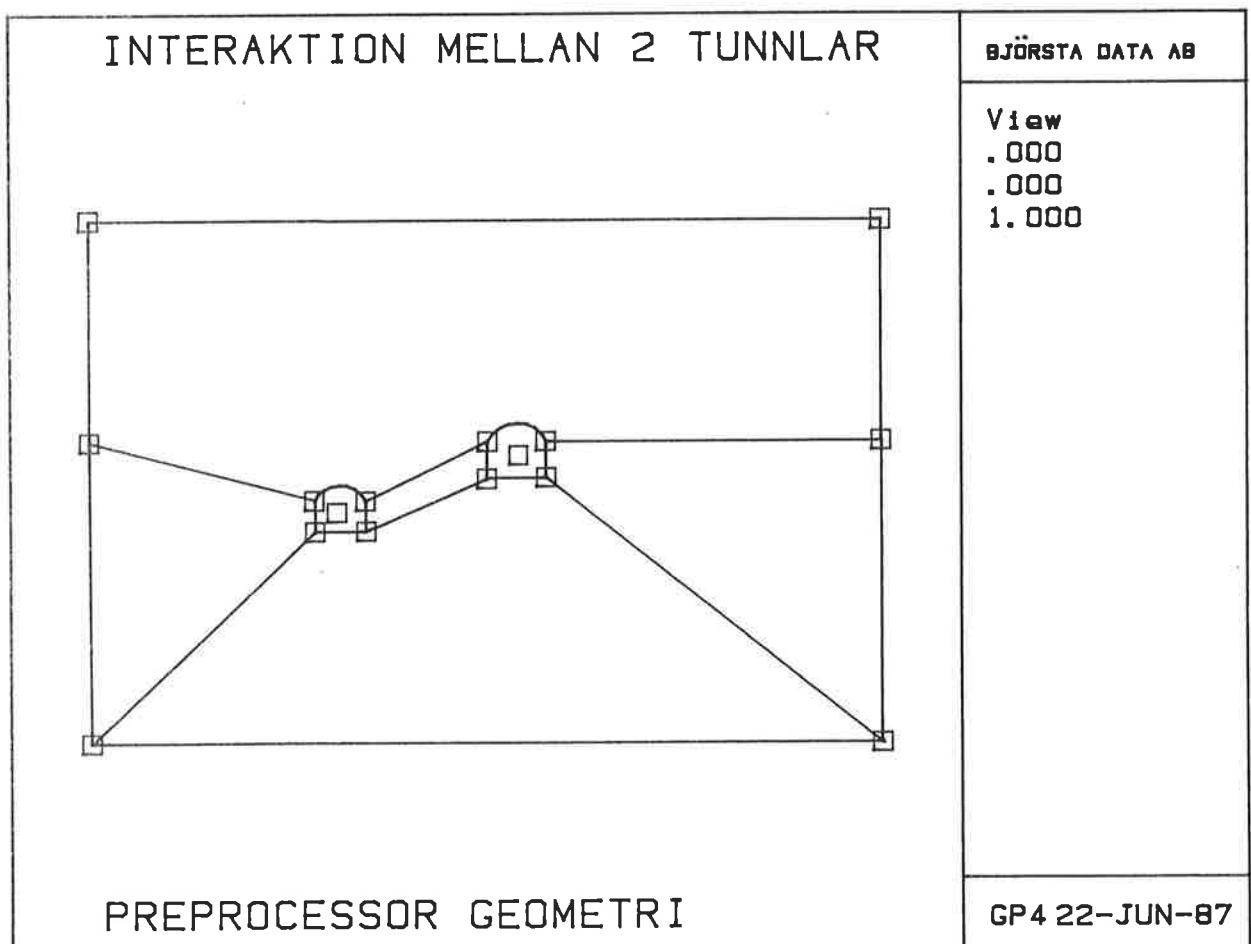


Fig. 1. Geometri för att generera Finita Element nätet.

För att få ett korrekt resultat i både deformationer och spänningar bör initialtillståndet där de vertikala spänningarna är 5 MPa och de horisontella är 20 MPa uppnås. Följande förfarande har använts :

1. Lås fast vänsterkanten på modellen för horisontella rörelser samt nedre kanten för vertikala rörelser.
2. Lägg på last motsvarande 20 MPa i horisontell led längs högerkanten och 5 MPa i vertikal led längs överkanten.
3. Ansätt en initialspänning i samtliga element på 20 MPa i horisontal led och 5 MPa i vertikal led.

Såväl laster som randvillkor genereras med referens till de yttre linjerna som GP4 modellen är uppbyggd av. Storleken på de uträknade nod-lasterna beror på storleken på elementen.

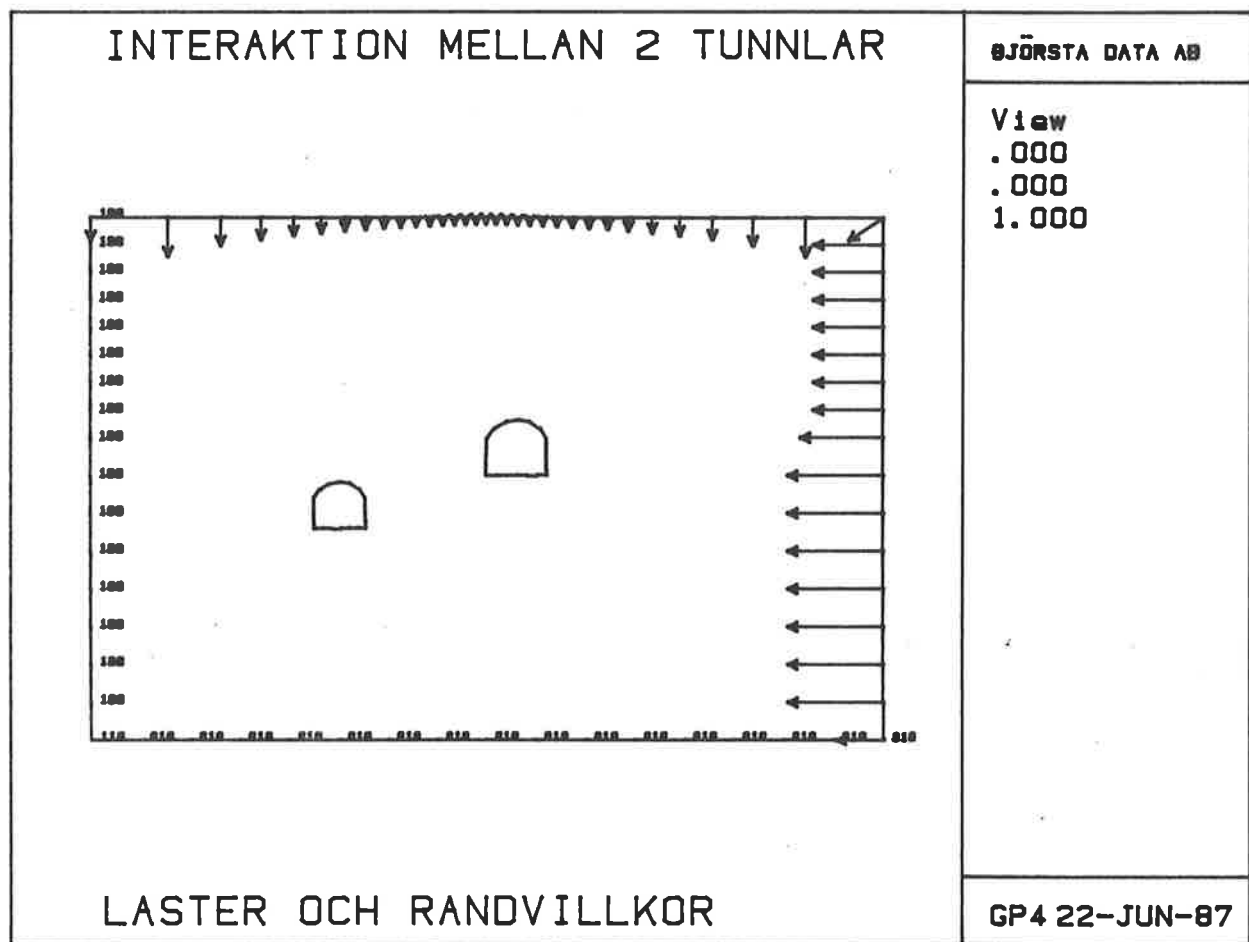


Fig. 2. Laster och randvillkor

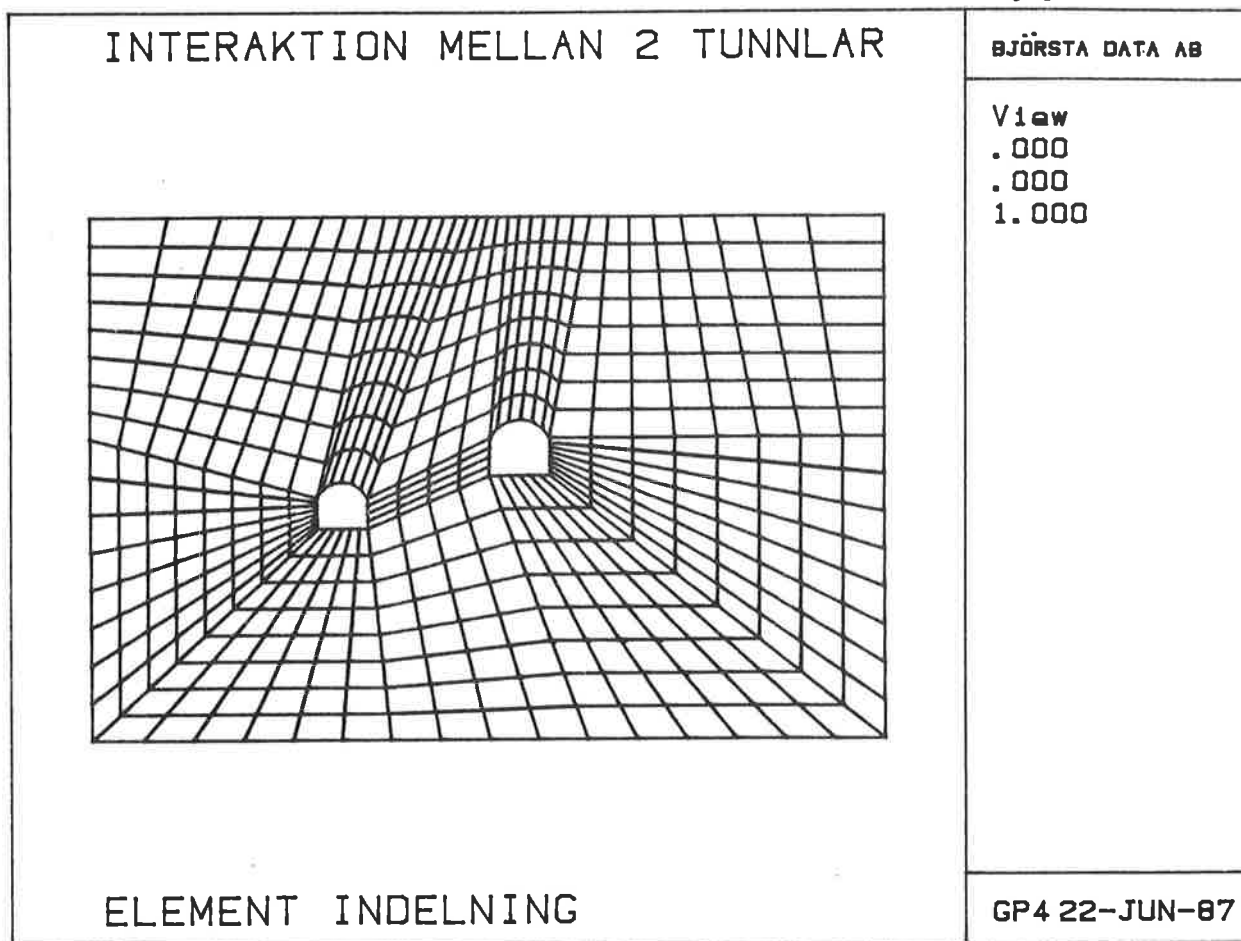


Fig. 3. Element nät

För själva analysen har PCFEMP använts. GP4 lämnar ifrån sig fullständiga data för noder, element, laster och randvillkor. Dessa kompletteras med materialdata och styrparametrar till PCFEMP.

MATE
1
50000. .3 0. 2 1 1 0. 1. 0

Material och elementdata

END
MACR
RNST MML.INI
TANG
FORM
SOLV
REDB ON
DISP
STRE
NSTR
END
STOP

Styrkommandon
Läs in initialspänningarna
Beräkna elementstyvheter
Assemblera
Lös ekvationssystemet
Slå på resultatdatabasen
Beräkna nod-deformationer
Beräkna element-spänningar
Beräkna nod-spänningar

Fig. 4. Indata till PCFEMP

Utvärdering av resultatet sker grafiskt. Både PCFEMP och GP4 kan användas med liknande resultat, GP4 med tyngdpunkt på färggrafik. Nedan har PCFEMP använts för att visa huvudspänningsriktningar och de omlagrade horisontal- och vertikalspänningarna.

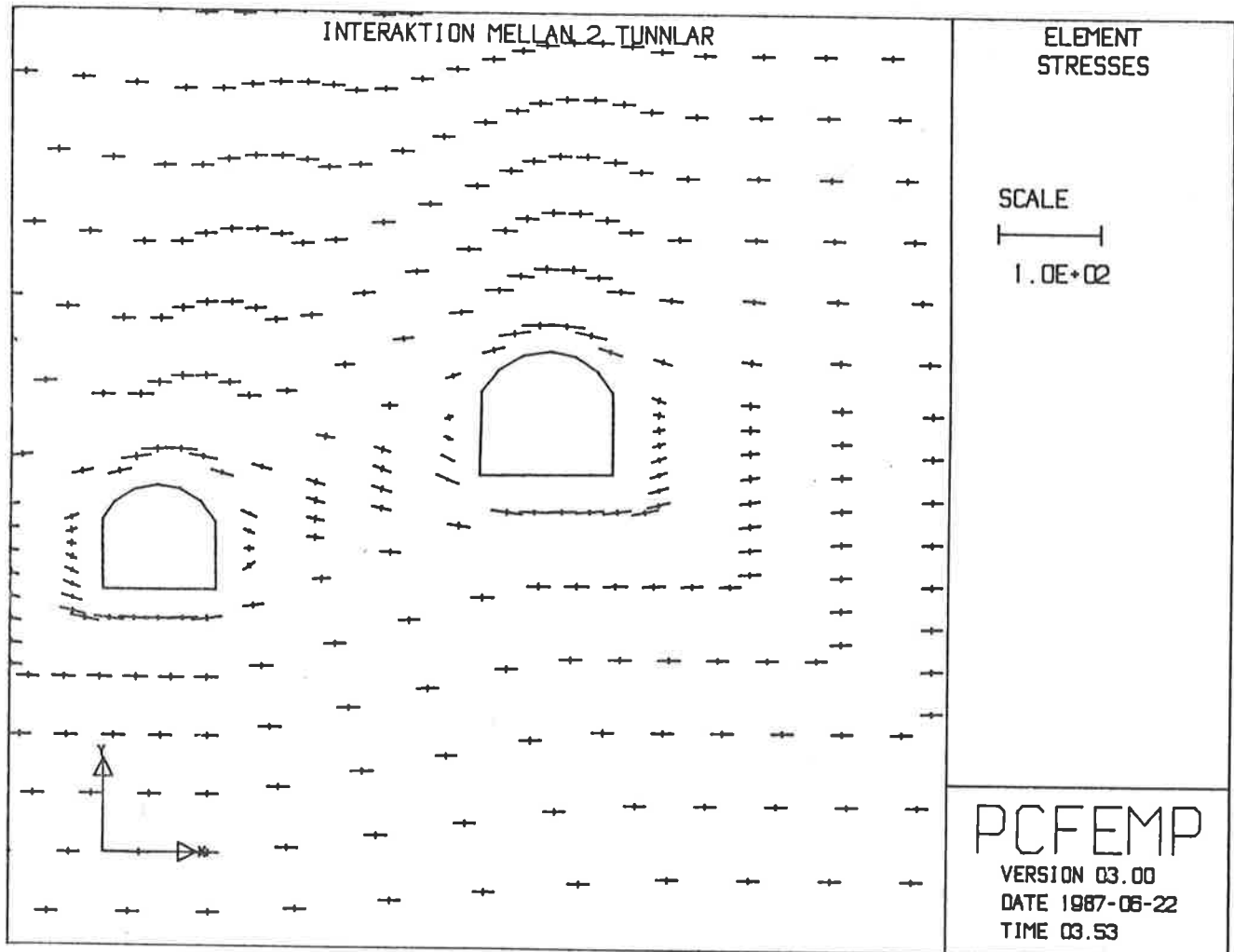


Fig. 5. Huvud-spänningar

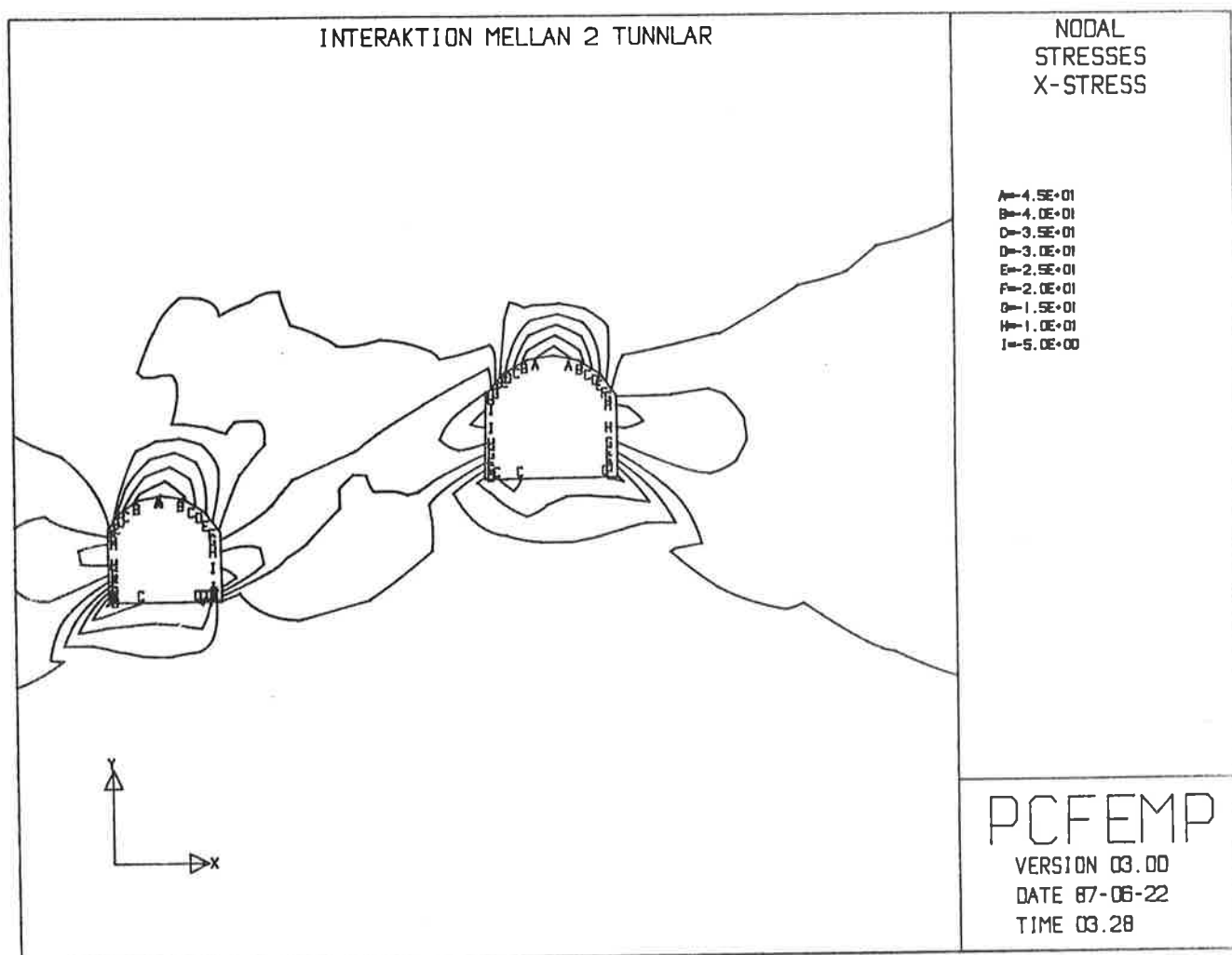


Fig. 6. Horisontal-spänningar

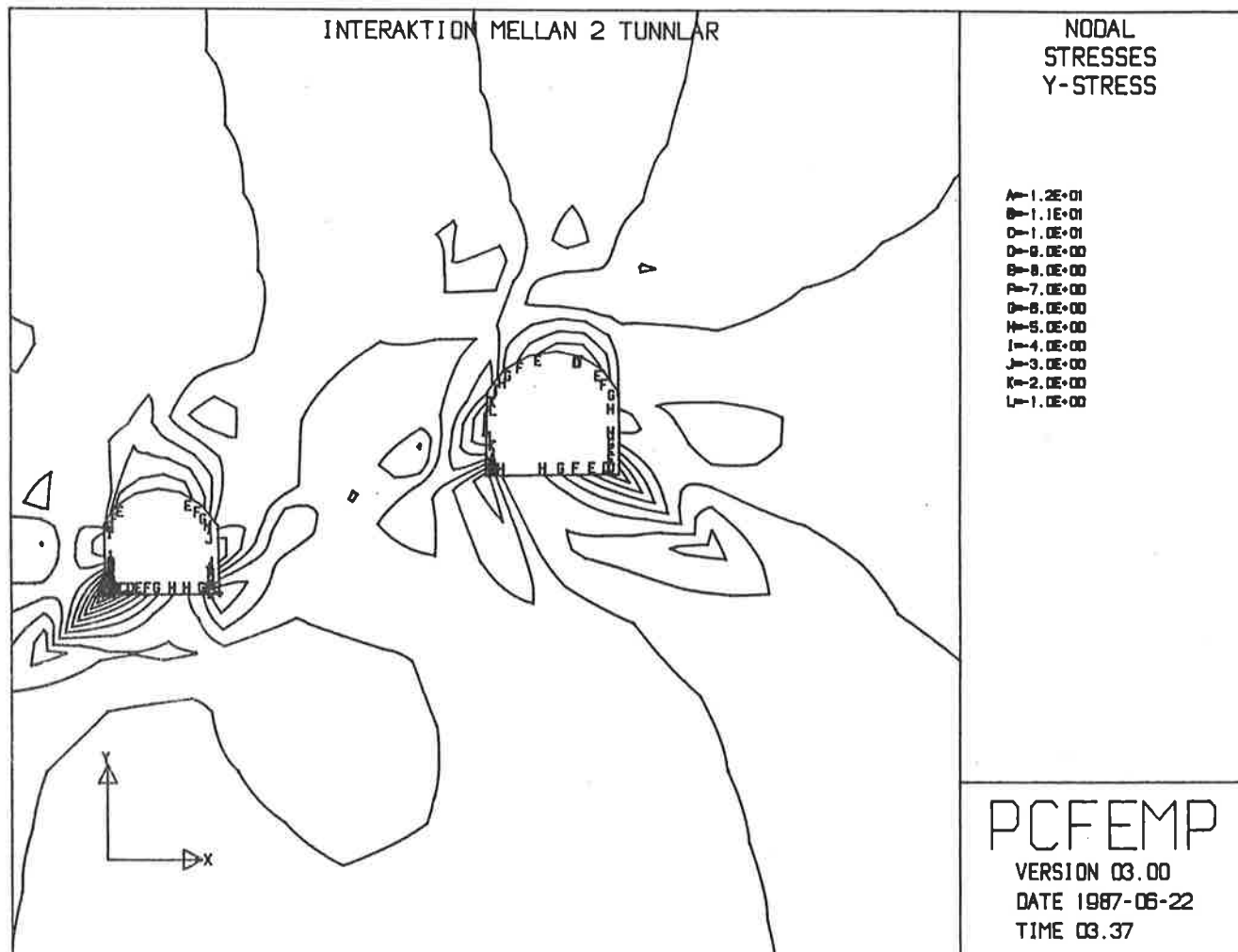


Fig. 7. Vertikalspänningar

EXEMPLE OF BOUNDARY ELEMENTS IN ROCK MECHANICS

Problem :

A tabular ore body has previously been open pit mined. Underground workings have started at several levels along the orebody. Calculate the interaction between the open pit and the underground workings.

Solution :

The BESOL system has been used. In the Boundary Element Method, only the boundaries need to be modelled, in this case the open pit and the orebody. Indata may be given interactively in a menu based program or as an indata file.

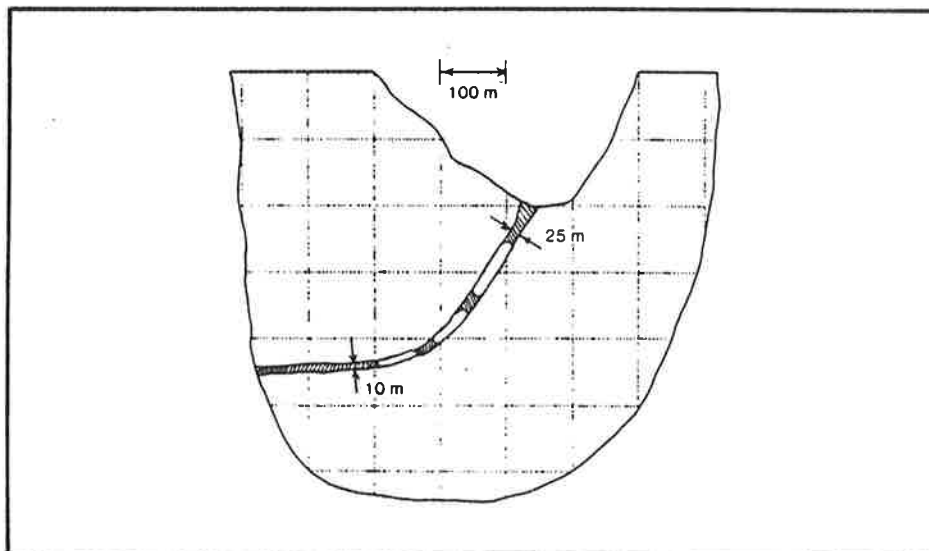


Fig. 2. Example problem for BESOL / P4002; interaction between underground workings and an open pit.

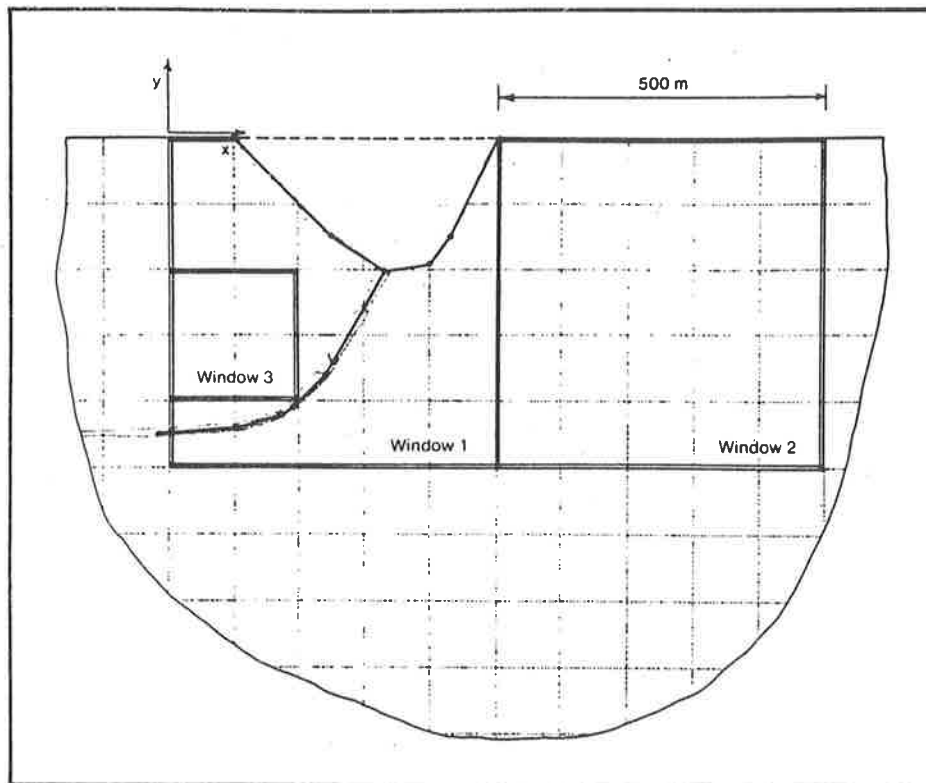


Fig. 3. Boundary element model of the problem depicted in Figure 2. The program BESOL/P4002 automatically includes the earth's surface (region code 002) and allows displacement discontinuity and fictitious stress elements to be used in the same problem (program series BESOL/P4000). The underground workings (excavations in a tabular orebody) are modeled by displacement discontinuity elements, while the open pit (which is an example of a massive excavation of irregular shape) is modeled by fictitious stress elements. The three "windows" indicated in Figure 3 are used to define grids of field points for post-processing.

EXAMPLE PROBLEM FOR BESOL/P4002; INTERACTION BETWEEN U/G AND SURFACE WORKINGS.

7 5 3 1 1 4

0.2 2E4 0 1000 0 0 0

5.0 -0.040 0.0 -0.027 0.0 0.020

5.0 20.0 30.0

10 6 -20.0 -450.0 100.0 -440.0 1E3 2E3
 6 1 100.0 -440.0 170.0 -420.0 0.0 0.0
 2 6 170.0 -420.0 190.0 -410.0 1E3 2E3
 6 1 190.0 -410.0 240.0 -360.0 0.0 0.0
 2 6 240.0 -360.0 250.0 -340.0 1E3 2E3
 8 1 250.0 -340.0 300.0 -260.0 0.0 0.0
 6 6 300.0 -260.0 330.0 -200.0 4E2 8E2

control parameters,
 elastic constants,
 initial stress state and
 failure properties.

displacement discontinuity
 elements.

18 1 100.0 0.0 250.0 -150.0 0.0 0.0
 8 1 250.0 -150.0 330.0 -200.0 0.0 0.0
 6 1 330.0 -200.0 400.0 -190.0 0.0 0.0
 4 1 400.0 -190.0 430.0 -150.0 0.0 0.0
 14 1 430.0 -150.0 500.0 0.0 0.0 0.0

fictitious stress elements.

0.0 0.0 500.0 2
 500.0 0.0 500.0 2
 0.0 -200.0 200.0 2

locations of windows
 of field points.

Fig. 4. The input data for the problem in Figure 3 is given in Figure 4. This file can be prepared using any convenient editor that produces "clean" ASCII text.

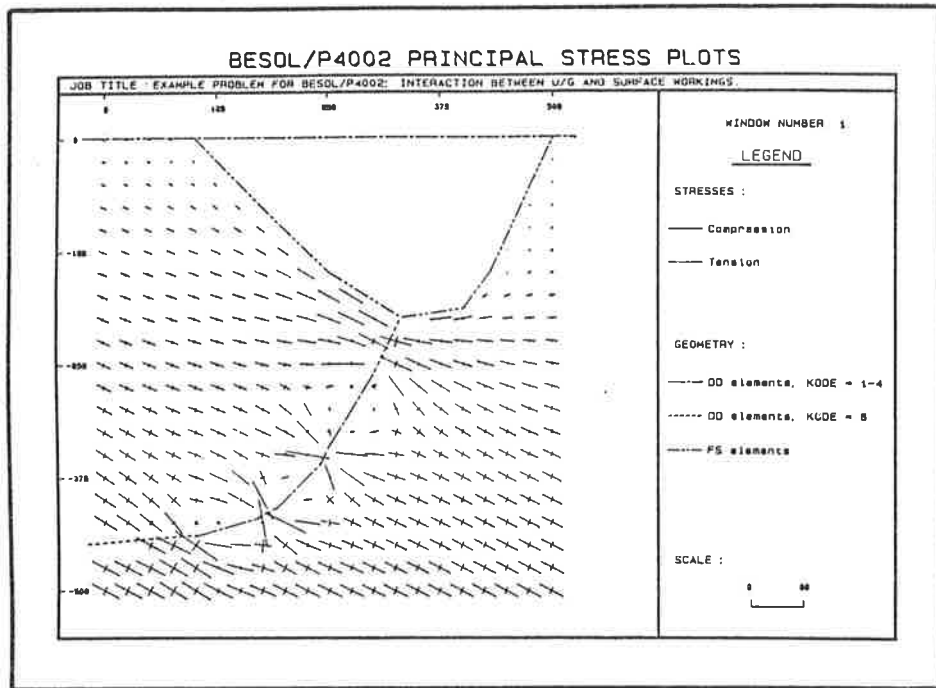
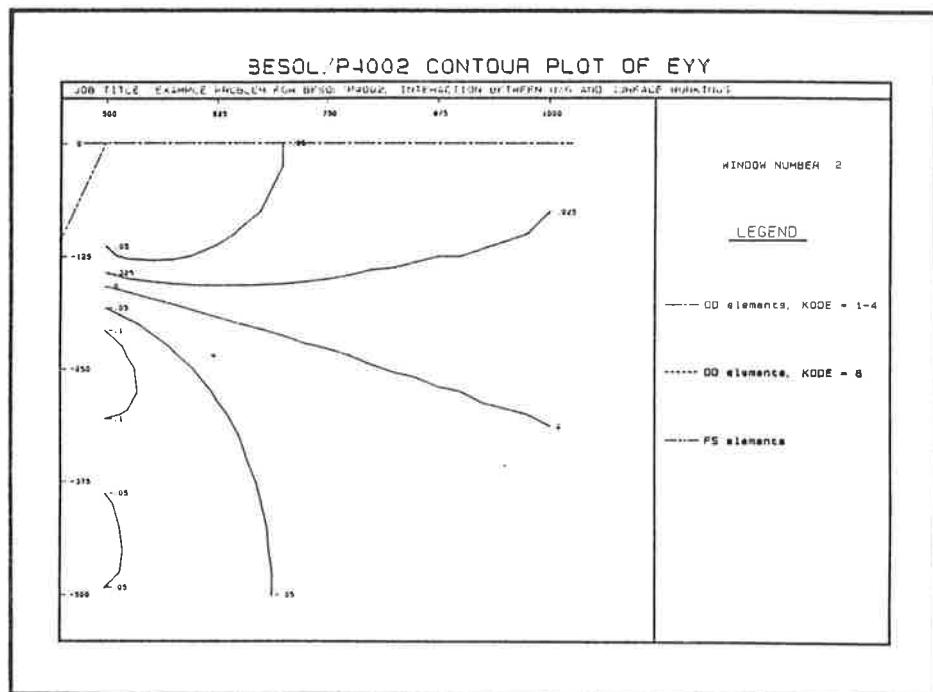


Fig. 5. Principal stresses and directions for field points in window 1 (see Figure 3 for the location of this window). Points inside the pit are automatically excluded by the program, as are field points that lie within a small distance from a boundary. The parameter KODE mentioned in the legend for this plot refers to particular kinds of boundary conditions for the displacement discontinuity elements; KODE = 1 and KODE = 6 represent, respectively, mined and unmined parts of the tabular orebody.



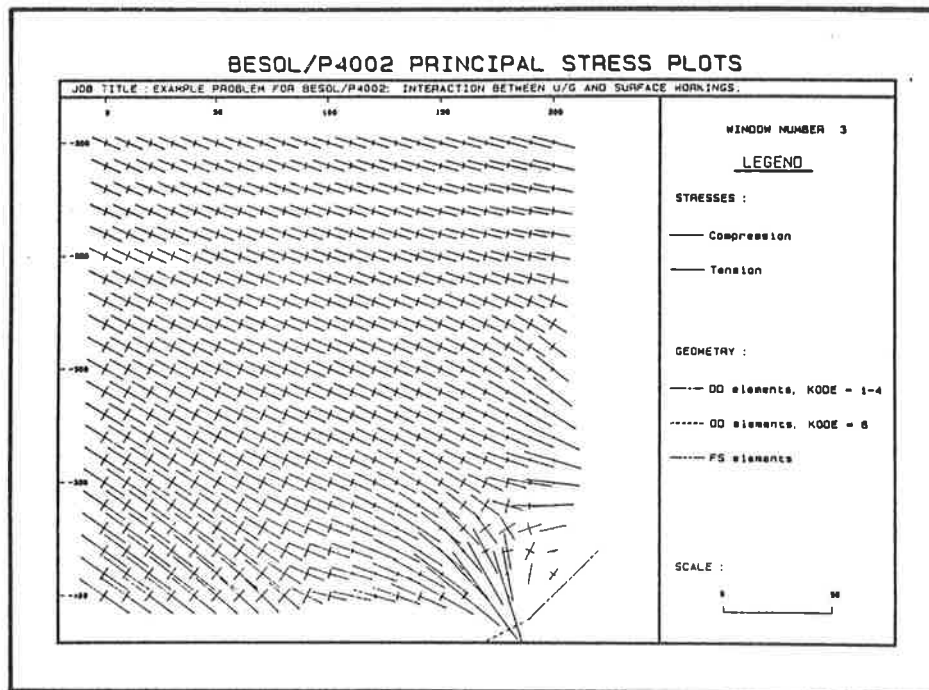


Fig. 7. Principal stresses and directions for points in window 3 (see Figure 3 for the location of this window).

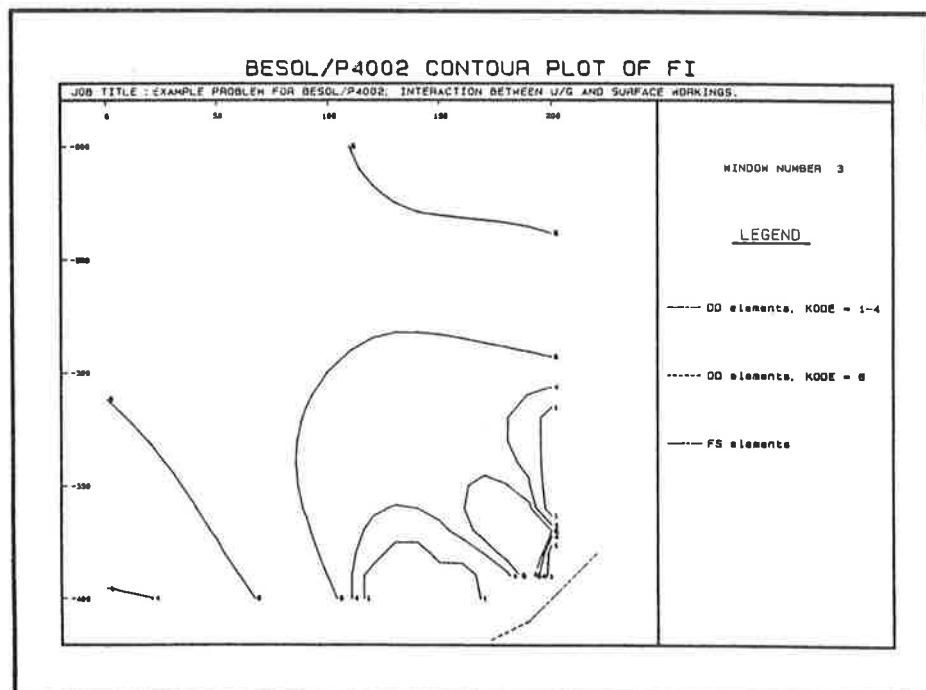


Fig. 8. Contours of the "failure index" (FI) for window 3 (see Figure 3 for the location of this window). The failure index represents the ratio of rock strength (as estimated from a simple failure criterion) to stress (as computed by BESOL/P4002). A failure index of 1 theoretically indicates failure.

BLASTHOLE OPEN STOPING BY VERTICAL RETREAT

In this simulation, a banded orebody with a weak footwall was examined for mining by vertical retreat. Of interest here is the stability of the crown pillar and stope back as the stope retreats. The PC distinct element code MUDEC is used to model widely-spaced joint sets and dipping orebody.

The model for this example is shown in Fig. 1. A steeply dipping orebody (average dip of 80°) is modeled between the 990m level and the 1190m level of the mine. A low-angle discontinuous joint set is also oriented at 10° dip and average spacing of 30m. The average thickness of the orebody is 14m. The upper stope, above the 1090m level, is mined first; then, mining of the lower level is completed, leaving a 10m crown pillar.

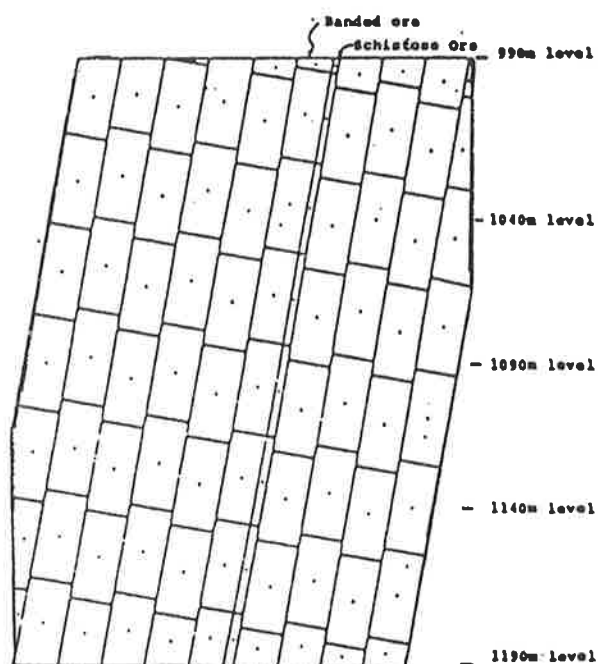


Fig. 1 Initial Problem Geometry for Blasthole Open Stoping Problem

BLASTHOLE OPEN STOPING BY VERTICAL RETREAT

Four rock types are defined for the model: the hanging wall quartzite, the footwall quartzite, the banded ore, and the weaker schistose ore. Based on the average laboratory test values, the following properties for these rock types were assumed:

	<u>Hanging Wall</u>	<u>Banded Ore</u>	<u>Schistose Ore</u>	<u>Footwall</u>
Young's Modulus (MPa)	62	56	40	67
Poisson's Ratio	0.29	0.28	0.33	0.28
Unconfined Compressive Strength (MPa)	186	168	96	198

The following joint properties were estimated:

Joint Normal Stiffness	5000 MPa/m
Joint Shear Stiffness	5000 MPa/m
Joint Friction Angle	27°
Joint Cohesion	0

The pre-mining state of stress was estimated to be hydrostatic with 33 MPa at the 1190m level.

After model consolidation, the mining progressed in five stages. First, the upper level blocks were removed for a stope height of 45m. The lower stope was then mined in four stages of 17m, 15m, 15m and 18m, leaving the 10m crown pillar. The final stress concentration is depicted in Fig. 2. At this stage, most of the stress is transferred to the abutments. The stress buildup in the crown pillar is shown in Fig. 3. Note that, at the final stage of the mining sequence, the crown pillar fails and the stresses drop to a residual value. Although backfilling was not simulated in this example, the model can simulate backfill emplacement after excavation and stress histories in the backfill can be monitored similar to that shown in Fig. 3 for the crown pillar.

BLASTHOLE OPEN STOPING BY VERTICAL RETREAT

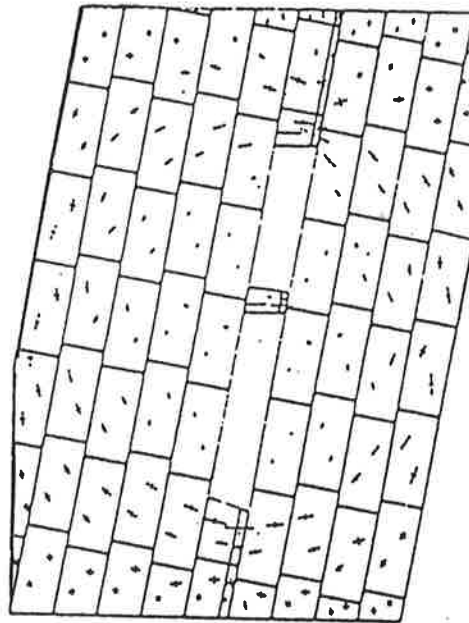


Fig. 2 Plot of Principal Stresses at End of Mining Sequence

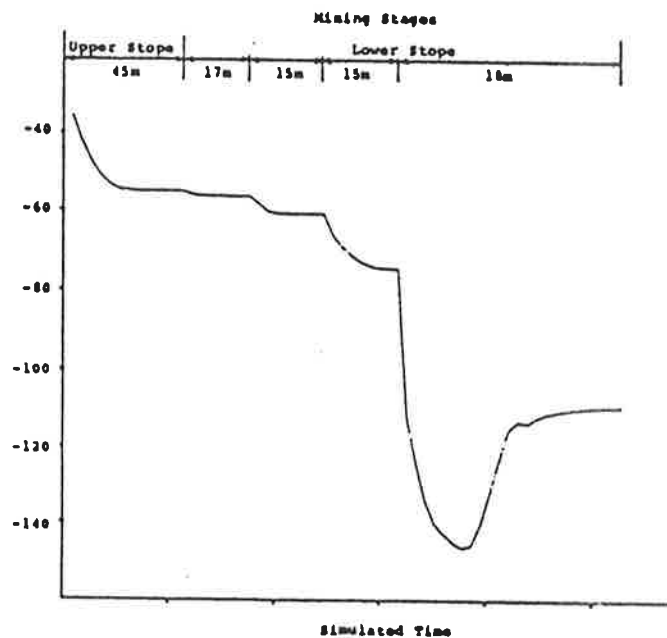


Fig. 3 Changes in Horizontal Stresses in Crown Pillar with Mining Stages (note: compressive stresses negative)

BLASTHOLE OPEN STOPING BY VERTICAL RETREAT

INPUT COMMANDS FOR OPEN STOPING USING VERTICAL RETREAT EXAMPLE

```

START
HEAD
EXAMPLE -- OPEN STOPING USING VERTICAL RETREAT
WI 0 200 0 200
* ESTABLISH INITIAL GEOMETRY
BLOCK 0 0 0 200 150 200 150 0
JREG 0 0 0 200 150 200 150 0
JSET 80,0 300,0 0,0 14,0 0 0
DEL 0 200 0 200 1210
SPLIT 67.864 0 104.129 200
* ASSIGN MATERIAL PROPERTIES HERE
CHANGE MAT=2 CON=3
CHANGE 86 87 0 200 MAT=4 CON=3
CHANGE 75 85.6 0 200 MAT=3 CON=3
CHANGE 87 200 0 200 MAT=5 CON=3
* PUT IN DISCONTINUOUS CROSS JOINTS
JREG 0 0 0 200 150 200 150 0
JSET -10 0 14 0 14 0 30 0 0 0
JSET -10 0 14 0 14 0 30 0 49.48,200
SAVE MINDOL1.SAV
* PUT IN JOINTS NEEDED FOR LATER EXCAVATIONS
split 74 100 89 100
split 84 155 99 155
split 63 40 78 35
split 65 52 81 52
split 68 67 83 67
split 71 82 86 82
* CREATE FINITE DIFFERENCE TRIANGLES IN ALL BLOCKS
gen 0 200 0 200 auto 100
* SPECIFY STRESS FIELD
bound lstr -33e6 0 -33e6 0 0 0 2e4 0 2e4
insitu str -33e6 0 -33e6 ygrad 2e4 0 2e4
grav 0 -10
* JOINT PROPERTIES
prop mat=1 d=2000 g=26e9 k=44e9
prop mat=1 jkn=5e9 jks=4e9 jfric=0.5
prop mat=1 kn=5e9 ks=5e9 fric=0.5
change jcon=2 jmat=1
* HANGING WALL ROCK
prop mat=2 d=2000 g=24e9 k=48e9 coh=93e6
* BANDED ORE
prop mat=3 d=2000 g=22e9 k=42e9 coh=84e6

```

BLASTHOLE OPEN STOPING BY VERTICAL RETREAT

```
* SCHISTOSE ORE
prop mat=4 d=2000 g=15e9 k=38e9 coh=48e6
* FOOT WALL
prop mat=5 d=2000 g=26e9 k=51e9 coh=99e6
* CONDITIONS DURING SOLUTION
mscale on 2.836e5 5.642e5
damp auto
* BEGIN EXCAVATION SEQUENCE WITH UPPER STOPE
del 78 98 110 155
hist n=50 sxx 80 105 type 1
hist syy 80 105
cyc 600
pr max
save mindola3.sav
* EXCAVATE LOWEST PART OF LOWER STOPE
del 63 81 35 52
cyc 400
pr max
save mindola4.sav
* EXCAVATE NEXT PART OF LOWER STOPE
del 65 83 52 67
cyc 400
pr max
save mindola5.sav
* EXCAVATE NEXT PART OF LOWER STOPE
del 67 86 67 82
cyc 400
pr max
save mindola6.sav
* EXCAVATE LAST PART OF LOWER STOPE
del 71 89 82 100
cyc 1200
pr max
save mindola9.sav
pr his 1 2
stop
```


SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

This problem examines the excavation of two tunnels mined in a Morh-Coulomb material at a depth of roughly 70 meters beneath sea level. The problem analyzes the continuum behavior of the rock mass and the loads developed in a concrete liner from one tunnel as the other is mined. The problem is analyzed with the PC distinct element code MUDEC. This example illustrates the versatility of the MUDEC code in analyzing a continuum problem by not allowing failure on joints.

PROBLEM GEOMETRY

The idealized geometry of the tunnel system is shown in Fig. 1. The system consists of two tunnels on 12-meter centers at a depth of roughly 70 meters (centerline) beneath the sea bed. The water level is approximately 110 meters above the tunnel centerline. The small (service) tunnel is 5.24 meter diameter with a 37cm-thick sectional concrete liner. The main tunnel is 8.22 meter diameter with a 46cm-thick sectional concrete liner. The service tunnel is driven and lined prior to excavation and lining of the main tunnel.

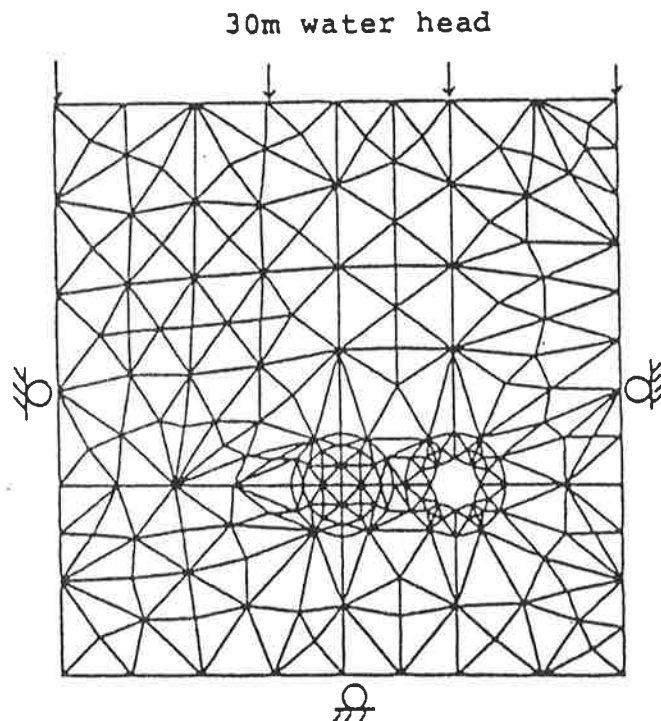


Fig. 1 Initial Mesh with Service Tunnel Excavated

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Page 2

MATERIAL PROPERTIES

Chalk

The blue chalk has the following material properties:

instantaneous deformation modulus	=	900 MPa
Poisson's ratio	=	0.35
uniaxial compressive strength	=	8 MPa
cohesion	=	1 MPa ⁺
density	=	2600 kg/m ³

The chalk exhibits significant creep-compaction when dewatered, as well as creep under constant applied stress. In an attempt to simulate this behavior, an equivalent elastic reduction in stiffness has been defined. The reduced value for Young's modulus is referred to as the "deferred" modulus which is 133 MPa—or, roughly, 6 times less than the instantaneous value.

Concrete Liner

The elastic modulus for the concrete liner is 24 GPa.

CONSTRUCTION STAGES

The sequence of construction activities are:

- (1) excavation of service tunnel;
- (2) lining of service tunnel;
- (3) excavation of main tunnel; and
- (4) lining of main tunnel.

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

NUMERICAL MODEL

Two cases were analyzed with MUDEC:

- (1) rock modulus = instantaneous = 900 MPa until final tunnel mined, deferred modulus of 133 MPa thereafter; and
- (2) rock modulus = lowest deferred value = 133 MPa

The tunnel simulation was run with the steps shown in Fig. 2, below.

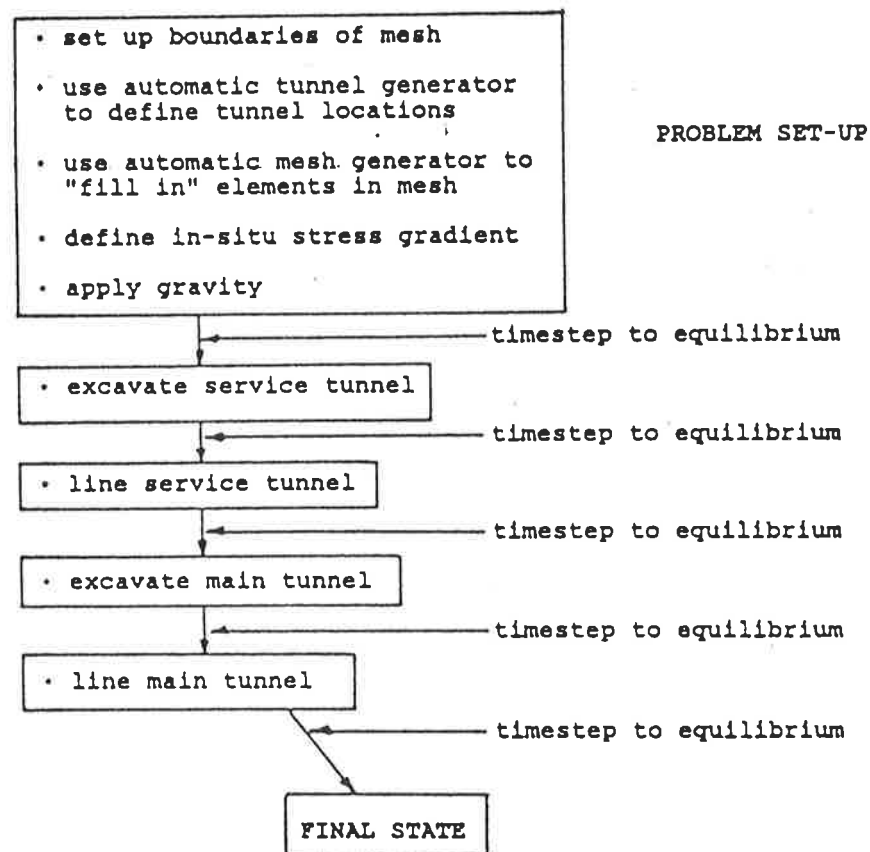


Fig. 2 Flow Chart for Problem Set-Up and Operation

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

The mesh used in the calculations is shown in Fig. 1. The lower and side boundaries are fixed with rollers. The weight of the seawater above the seabed is modeled by applying the equivalent pressure of 30 meters of water head to the top surface of the mesh. Since the tunnels are lined with a waterproof segmented liner, there is no need to perform a transient groundwater flow analysis. The pore water pressure was accounted for by setting the unit weight of the chalk to the submerged unit weight. The vertical to horizontal stress ratio was assumed to be hydrostatic.

After gravity stresses have been initialized in the body, the service tunnel is mined and timestepping is run until equilibrium is achieved. The resulting elastic displacements are given in Fig. 3. Next, 16 beam elements are used to model the concrete liner. In the dynamic formulation, each beam is considered to be a mass which is coupled to the rock via a shear and normal stiffness (Fig. 4). Each mass has three degrees of freedom—two displacement components and one rotation. As the rock mass deforms, displacements are transferred to the mass in accordance with the interface stiffness. Adjacent beam elements transfer shear and normal forces across their contacts. The axial and shear force and the moment at each grid connection are determined by using the standard stiffness matrix for a beam element:

$$\{F\} = [K] \{\Delta\}$$

The MUDEC program performs all calculations internally; the user need only define the geometry and properties of the support.

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

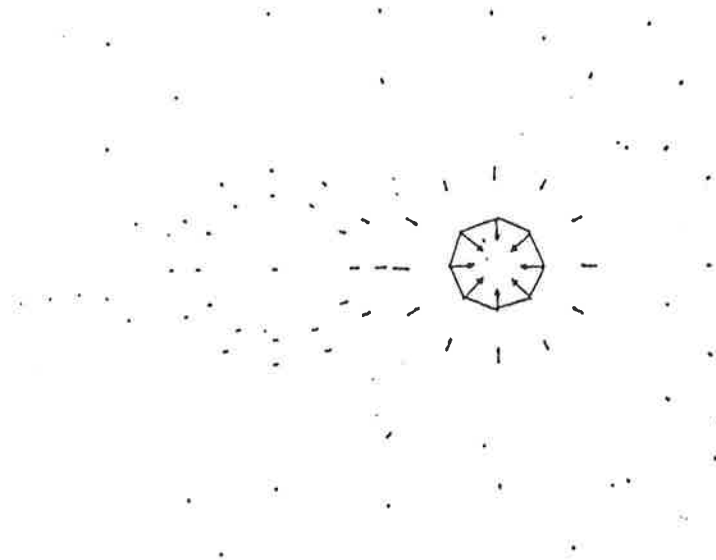


Fig. 3 Elastic Displacements Due to Excavation of Service Tunnel

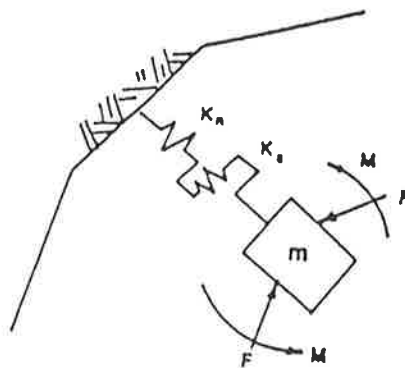


Fig. 4 Beam Element as Lumped Mass Connected to Rock via Shear and Normal Springs

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Figure 5 is a plot of the finite difference zones and the beam element geometry. The second, main, tunnel is next excavated, which loads the supports in the service tunnel. Finally, the modulus of the rock mass is reduced simultaneously with the installation of support in the main tunnel. Timestepping of the solution then continues until equilibrium. Figures 6 and 7 illustrate the displacements and principal stress distributions resulting from excavation of the main tunnel. Note that the entire service tunnel translates toward the main tunnel.

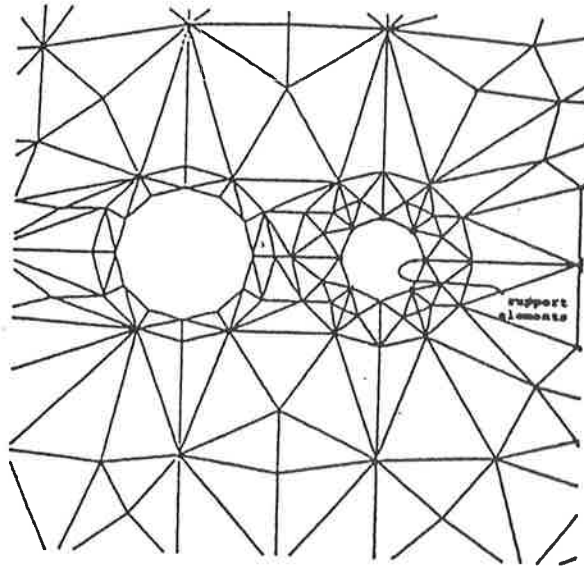


Fig. 5 Close-Up View of Tunnels and Support in Service Tunnel

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

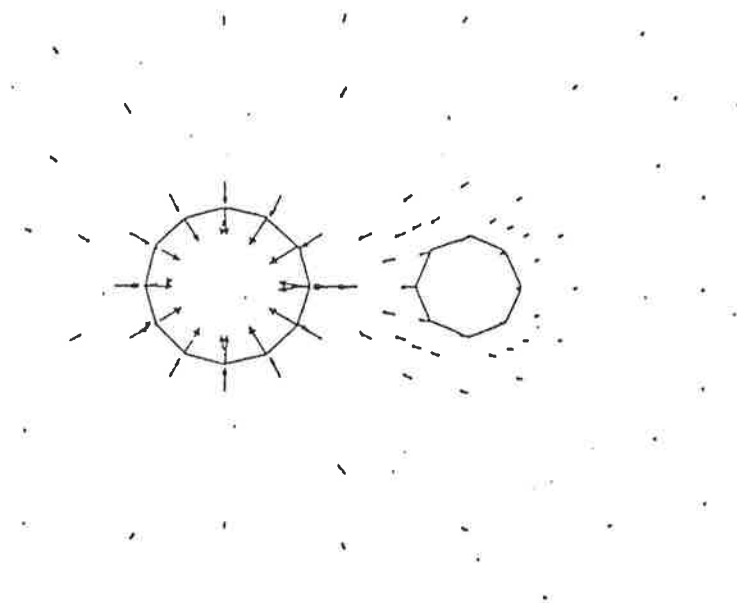


Fig. 6 Displacements After Mining Main Tunnel

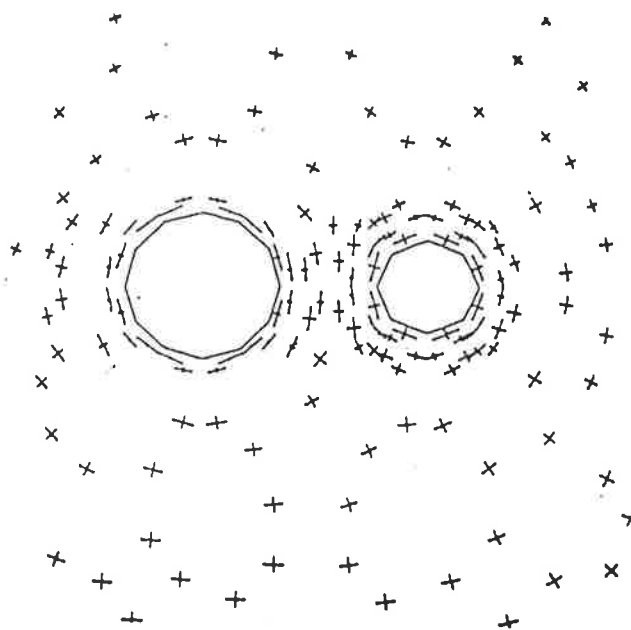


Fig. 7 Principal Stress Distribution After Mining of Main Tunnel

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Figure 8 is a polar plot of the final axial forces for Case 1. The greatest axial loads for both liners occur at the horizontal midplanes of the tunnels. At these positions, the thrust is $1.0 \times 10^5 \text{ N}$, or an axial stress of 0.32 MPa in the service tunnel. The maximum thrust in the large tunnel is $1 \times 10^5 \text{ N}$, or an axial stress of 0.22 MPa. The axial and shear forces and moment distribution for each beam element is given in Table 1.

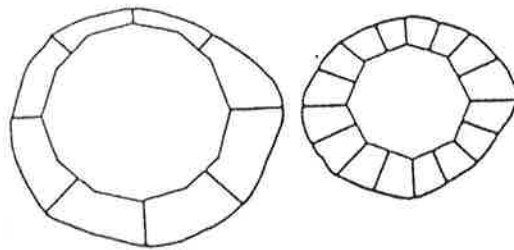


Fig. 8 Polar Plot of Axial Forces Induced in Concrete Liner, Case 1

Table 1

FORCES AND MOMENTS IN LINERS FOR CASE 1

LARGE TUNNEL

<u>ELE.</u>	<u>MIDPOINT X,Y COORDS</u>	<u>FORCE</u>		<u>MOM1</u>	<u>MOM2</u>
		<u>axial</u>	<u>shear</u>		
2	3.261E+01 -7.262E+01	1.212E+05	2.362E+04	3.657E+04	3.572E+04
3	3.000E+01 -7.370E+01	9.656E+04	-9.704E+03	-3.024E+04	5.363E+02
4	2.739E+01 -7.262E+01	9.274E+04	-1.986E+03	-1.297E+04	6.894E+03
5	2.630E+01 -7.001E+01	6.330E+04	-1.029E+04	1.297E+03	-3.280E+04
6	2.739E+01 -6.739E+01	5.257E+04	1.496E+04	3.688E+04	8.926E+03
7	3.000E+01 -6.631E+01	2.537E+04	7.007E+03	-1.115E+04	3.259E+04
8	3.261E+01 -6.740E+01	2.933E+04	-1.217E+04	-3.338E+04	-3.878E+03

SMALL TUNNEL

<u>ELE.</u>	<u>MIDPOINT X,Y COORDS</u>	<u>FORCE</u>		<u>MOM1</u>	<u>MOM2</u>
		<u>axial</u>	<u>shear</u>		
1	4.442E+01 -7.000E+01	9.882E+04	4.685E+04	8.017E+04	-3.510E+04
2	4.423E+01 -7.093E+01	7.867E+04	3.244E+04	3.478E+04	-3.569E+03
3	4.371E+01 -7.171E+01	7.396E+04	3.923E+04	2.963E+03	3.478E+04
4	4.292E+01 -7.223E+01	5.368E+04	2.233E+03	-3.435E+04	3.650E+04
5	4.200E+01 -7.242E+01	7.998E+04	-9.176E+03	-3.543E+04	2.661E+04
6	4.107E+01 -7.224E+01	8.072E+04	-1.378E+04	-2.637E+04	1.311E+04
7	4.029E+01 -7.171E+01	1.040E+05	-3.837E+04	-1.380E+04	-2.311E+04
8	3.977E+01 -7.093E+01	1.001E+05	-2.060E+04	2.347E+04	-4.328E+04
9	3.958E+01 -7.000E+01	9.784E+04	8.608E+03	4.338E+04	-3.510E+04
10	3.977E+01 -6.908E+01	7.911E+04	2.611E+04	3.472E+04	-9.606E+03
11	4.029E+01 -6.829E+01	6.632E+04	4.190E+04	9.499E+03	3.081E+04
12	4.107E+01 -6.777E+01	4.890E+04	1.082E+04	-3.107E+04	4.148E+04
13	4.200E+01 -6.758E+01	5.517E+04	-1.273E+04	-4.147E+04	2.922E+04
14	4.293E+01 -6.777E+01	6.173E+04	-2.594E+04	-2.847E+04	3.511E+03
15	4.371E+01 -6.829E+01	8.230E+04	-1.938E+04	-3.079E+03	-1.557E+04
16	4.423E+01 -6.908E+01	8.473E+04	-6.018E+03	1.499E+04	-2.078E+04

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Figure 9 illustrates the final liner axial force distribution for Case 2. The peak axial support forces occur at the tunnel midline. The maximum service tunnel axial load is $1.9 \times 10^5 \text{ N}$ (0.41 MPa axial stress); the maximum tunnel axial load is $1.5 \times 10^5 \text{ N}$ (0.3 MPa axial stress). The final force and moment distribution for both liners is given in Table 2.

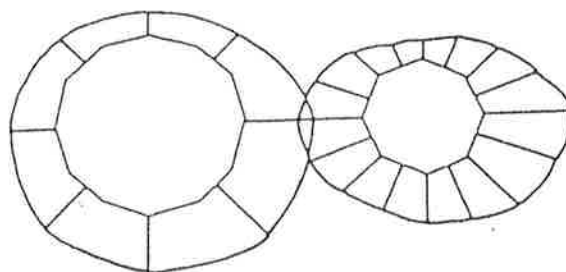


Fig. 9 Polar Plot of Axial Forces Induced in Concrete Liner, Case 2

Table 2

FORCES AND MOMENTS IN LINERS FOR CASE 2

LARGE TUNNEL

ELE.	MIDPOINT X,Y COORDS		FORCE		MOM1	MOM2
			axial	shear		
1	3.368E+01	-7.001E+01	1.482E+05	-1.298E+04	6.116E+03	-4.569E+04
2	3.260E+01	-7.261E+01	1.570E+05	2.672E+04	4.572E+04	3.570E+04
3	3.000E+01	-7.368E+01	1.201E+05	-4.614E+03	-3.568E+04	2.163E+04
4	2.740E+01	-7.260E+01	1.266E+05	-1.385E+04	-2.150E+04	-2.065E+04
5	2.633E+01	-7.001E+01	9.142E+04	-2.596E+03	2.082E+04	-2.871E+04
6	2.740E+01	-6.741E+01	6.318E+04	1.836E+04	2.881E+04	2.707E+04
7	3.000E+01	-6.633E+01	4.400E+04	-4.121E+03	-2.705E+04	1.450E+04
8	3.260E+01	-6.741E+01	6.886E+04	-5.101E+03	-1.445E+04	-1.096E+03

SMALL TUNNEL

ELE.	MIDPOINT X,Y COORDS		FORCE		MOM1	MOM2
			axial	shear		
1	4.440E+01	-7.000E+01	1.912E+05	3.639E+03	9.120E+04	-8.772E+04
2	4.422E+01	-7.092E+01	1.791E+05	3.629E+04	8.772E+04	-5.298E+04
3	4.370E+01	-7.170E+01	1.607E+05	9.748E+04	5.298E+04	4.033E+04
4	4.292E+01	-7.222E+01	9.977E+04	5.219E+04	-4.033E+04	9.027E+04
5	4.199E+01	-7.241E+01	9.733E+04	8.405E+03	-9.027E+04	9.831E+04
6	4.107E+01	-7.222E+01	8.514E+04	-4.480E+04	-9.831E+04	5.546E+04
7	4.029E+01	-7.170E+01	1.137E+05	-1.080E+05	-5.546E+04	-4.780E+04
8	3.977E+01	-7.092E+01	1.387E+05	-5.277E+04	4.780E+04	-9.827E+04
9	3.959E+01	-7.000E+01	1.400E+05	4.055E+03	9.827E+04	-9.439E+04
10	3.977E+01	-6.908E+01	1.199E+05	5.442E+04	9.439E+04	-4.235E+04
11	4.029E+01	-6.830E+01	8.410E+04	9.015E+04	4.235E+04	4.393E+04
12	4.107E+01	-6.778E+01	4.405E+04	4.288E+04	-4.393E+04	8.497E+04
13	4.199E+01	-6.760E+01	3.713E+04	1.943E+04	-8.497E+04	1.036E+05
12	4.292E+01	-6.778E+01	5.700E+04	-6.754E+04	-1.036E+05	3.893E+04
13	4.370E+01	-6.830E+01	1.004E+05	-8.409E+04	-3.893E+04	-4.152E+04
14	4.422E+01	-6.908E+01	1.342E+05	-5.122E+04	4.152E+04	-8.053E+04

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

TOTAL LINER STRESS

The total stress in the liner is determined by adding the ground-induced load with that induced by the hydrostatic water pressure around the tunnel. The ground loading is calculated by adding the stress induced from axial and bending stresses:

$$\sigma_{\text{ground}} = \frac{T}{A} + \frac{Mc}{I} \quad (1)$$

where T = thrust (axial force)

A = cross-sectional area

M = bending moment

c = distance from outer fiber to neutral axis

I = second moment of area = $(1/12) bt^3$

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Case I: $E_{\text{instantaneous}}$ initially, E_{deferred} After Mining Main Tunnel

Service Tunnel: $T = 1.0 \times 10^5 \text{ N}$

$$A = 0.37\text{m} \cdot 1\text{m} = 0.37\text{m}^2$$

$$M = 4.33 \times 10^4 \text{ N-m}$$

$$c = 0.185\text{m}$$

$$I = (1/12)bt^3 = (1/12) \cdot 1(0.37)^3 = 4.22 \times 10^{-3} \text{ m}^4$$

$$\sigma_{\text{ground}} = \frac{1 \times 10^5 \text{ N}}{0.37 \text{ m}^2} + \frac{(4.33 \times 10^4 \text{ N-m}) \cdot (0.185 \text{ m})}{4.22 \times 10^{-3} \text{ m}^4}$$

$$\approx 0.3 \text{ MPa} + 1.9 \text{ MPa} = 2.2 \text{ MPa}$$

hydrostatic pressure (assume average depth of 110m)

$$\sigma_{\text{water}} = P_{\text{hydrostatic}} \cdot r/t$$

where $P_{\text{hydrostatic}}$ = hydrostatic water pressure

r = radius

t = thickness of liner

$$\sigma_{\text{water}} = 9.8 (\text{KPa/m}) \cdot 110\text{m} \times (2.62\text{m}) / (0.37\text{m})$$

$$= 1.068 \text{ MPa} \cdot (2.62\text{m} / 0.37\text{m}) = 7.6 \text{ MPa}$$

$$\sigma_{\text{total}} = \sigma_{\text{ground}} + \sigma_{\text{water}} = 2.2 \text{ MPa} + 7.6 \text{ MPa} = 9.8 \text{ MPa}$$

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Main Tunnel: $T = 1.2 \times 10^5 \text{ N}$

$$A = 0.46 \text{ m} \cdot 1 \text{ m} = 0.46 \text{ m}^2$$

$$M = 3.6 \times 10^4 \text{ N-m}$$

$$c = 0.23 \text{ m}$$

$$I = (1/12)bt^3 = (1/12) \cdot 1(0.46)^3 = 0.008 \text{ m}^4$$

$$\begin{aligned} \sigma_{\text{ground}} &= \frac{1.2 \times 10^5}{0.46} + \frac{(3.6 \times 10^4 \text{ N-m}) \cdot (0.23 \text{ m})}{8 \times 10^{-3} \text{ m}^4} \\ &= 0.26 \text{ MPa} + 1.03 \text{ MPa} = 1.3 \text{ MPa} \end{aligned}$$

hydrostatic pressure .

$$\begin{aligned} \sigma_{\text{water}} &= 1.078 \text{ MPa} \cdot (4.11 \text{ m} / 0.46 \text{ m}) \\ &= 9.6 \text{ MPa} \end{aligned}$$

$$\sigma_{\text{total}} \approx 1.2 + 9.6 \approx 10.8 \text{ MPa}$$

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

Case 2: Edeferred Always

Service Tunnel: $T = 1.9 \times 10^5 \text{ N}$

$$M = 9.12 \times 10^4 \text{ N-m}$$

$$\sigma_{\text{ground}} = \frac{1.9 \times 10^5 \text{ N}}{0.96 \text{ m}^2} + \frac{(9.12 \times 10^4 \text{ N-m} \cdot (0.185 \text{ m}))}{4.22 \times 10^{-3} \text{ m}^4}$$

$$= 0.41 \text{ MPa} + 4.0 \text{ MPa} = 4.4 \text{ MPa}$$

$$\sigma_{\text{total}} \approx \sigma_{\text{ground}} + \sigma_{\text{water}} \approx 4.4 \text{ MPa} + 7.6 \text{ MPa} \approx 12 \text{ MPa}$$

Main Tunnel: $T = 1.5 \times 10^5 \text{ N}$

$$M = 4.5 \times 10^4 \text{ N-m}$$

$$\sigma_{\text{ground}} = \frac{1.5 \times 10^5 \text{ N}}{0.46 \text{ m}^2} + \frac{(4.5 \times 10^4 \text{ N-m}) \cdot (0.23 \text{ m})}{8 \times 10^{-3} \text{ m}^4}$$

$$= 0.32 \text{ MPa} + 1.3 \text{ MPa} = 1.6 \text{ MPa}$$

$$\begin{aligned} \sigma_{\text{total}} &\approx \sigma_{\text{ground}} + \sigma_{\text{water}} \\ &\approx 1.6 \text{ MPa} + 9.6 \text{ MPa} \\ &\approx 11.2 \text{ MPa} \end{aligned}$$

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

DISCUSSION

The thrust distribution varies with angle around the tunnels since the shearing stiffness between the rock and liner was made very large to simulate a "locked" interface. The thrust distribution is consistent with the fact that the entire service tunnel is translating toward the main tunnel. The significant displacement differences around the periphery of the service tunnel leads to larger moments than in the case of the main tunnel, which experiences fairly uniform radial displacements. The compressive stresses resulting from the ground movement are much smaller than those from the hydrostatic water pressure.

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

INPUT COMMANDS FOR TUNNEL SUPPORT LOADING EXAMPLE

```

start
*
*   tunnel support loading
*
block 0 -30 60 -30 60 -90 0 -90
wi -1 91 -91 1
round 0.1
split 0 -70 60 -70
split 30 0 30 -90
split 42 0 42 -90
tun 30 -70 4.11 12
tun 42 -70 2.62 8
tun 30 -70 5.5 12
tun 42 -70 5.5 12
gen 0 90 -50 0 auto 10.0
gen 0 90 -90 -75 auto 10.0
gen 0 90 -75 -50 auto 3.0
bound lstr 1.02e5 0 1.02e5 0 0 0 1.34e4 0 1.34e4
insitu stress 1.02e5 0 1.02e5 ygrad 1.34e4 0 1.34e4
* rock properties
*prop mat=1 d=1340 g=.33e9 k=.99e9 coh=1e6 fric=0.577
prop mat=1 d=1340 g=.05e9 k=.15e9 coh=1e6 fric=0.577
change mat=1 con=3
* elastic joint properties
prop mat=2 jkn=1e9 jks=1e9 kn=1e6 ks=1e6 jfric=10.0 fric=10.0
jtens=1e6
prop mat=2 tens=1e6
change jmat=2 jcon=2
damp auto
gravity 0 -10
cyc 10
bound -1 1 -91 0 xvel=0.0
bound -1 90 -91 -89 yvel=0.0
bound 59 61 -91 0 xvel=0.0
del 40 44 -72 -68
save chan1.sav

```

SUB-SEABED TUNNEL EXCAVATION AND SUPPORT LOADING

```

* histories around tunnel 1
hist n=20 ydis 42 -67 sxx 42 -67 type 1
hist ydis 42 -73 sxx 42 -73
hist xdis 39 -70 syy 39 -70
hist xdis 45 -70 syy 39 -70
* histories around tunnel 2
hist ydis 30 -65.0 sxx 30 -65.0
hist ydis 30 -75.0 sxx 30 -75.0
hist xdis 25.0 -70 syy 25.0 -70
hist xdis 35.0 -70 syy 35.0 -70
reset disp jdisp
cyc 600
save chan2.sav
pr max
struct 42 -70 11.25 3.0 16 5 0.37
prop mat=5 d=2400 g=10e9 k=13e9 kn=1e8 ks=1e7 fric=10.0
del 28 32 -72 -68
hist type 15
reset disp
cyc 5000
save chan6.sav
pr max
pr ste stc lm
struc 30 -70 22.5 4.3 8 5 0.46
prop mat=1 d=1340 g=.05e9 k=.15e9 coh=1e6 fric=.577
his type 15
reset disp
cyc 5000
pr max
pr ste stc lm
save chan7.sav
* stop
* return
rest chan1.sav
* rock properties
prop mat=1 d=1340 g=.05e9 k=.15e9 coh=1e6 fric=0.577
* histories around tunnel 1
hist n=20 ydis 42 -67 sxx 42 -67 type 1
hist ydis 42 -73 sxx 42 -73
hist xdis 39 -70 syy 39 -70
hist xdis 45 -70 syy 39 -70

```

SEISMIC-INDUCED GROUNDFAIL

A demonstration simulation of a seismic-induced groundfall was performed with MUDEC to illustrate the use of the code for analyzing rockburst potential. The model shown in Fig. 1 was developed based on the configuration and dimensions of the 34-1-554 overcut shown on a section drawing for Fraser Mine at Falconbridge. A two-dimensional, plane strain representation was chosen normal to the axis of the overcut. The overcut was modeled as being 5m high and 10m wide.

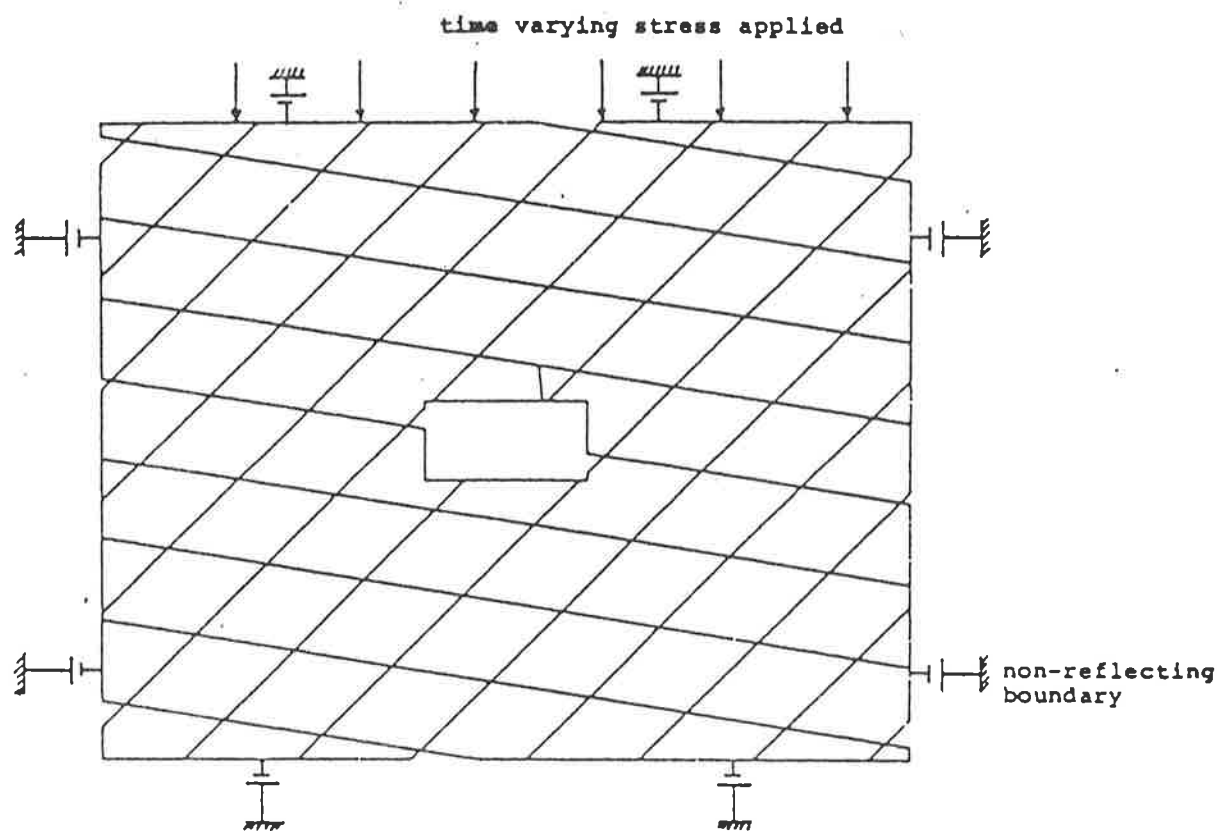


Fig. 1 MUDEC Model

SEISMIC-INDUCED GROUNDFAIL

It was assumed that two continuous joint sets intersect the plane of analysis; one with an orientation of 45° and the other with an orientation of 9° . Both sets have a joint spacing of 5m. A near vertical joint was also added to the block in the roof of the excavation, based on observations.

From the average laboratory test values provided for the intact rock, the following material properties were assumed for the rock blocks:

density	$0.003 \times 10^6 \text{ kg/m}^3$
Young's Modulus	75,000 MPa
Poisson's Ratio	0.18

The blocks were assumed to behave elastically only. Mohr-Coulomb behavior was assumed for the joints and typical textbook values were chosen for joint properties:

joint normal stiffness	20,000 MPa/m
joint shear stiffness	20,000 MPa/m
friction angle	30°
cohesion	0

The in-situ stress state was estimated to be isotropic at 24 MPa (assuming vertical loading due to overlying rock at a depth of approximately 800m).

The modeling sequence was performed in three stages. First, the model, without the overcut excavation, was consolidated under the in-situ stresses. Next, the excavation was introduced and the model cycled to an equilibrium state. The stress distribution around the overcut at this stage is illustrated in Fig. 2. The blocks immediately above and below the overcut have slipped and then stabilized.

In the third stage, two different seismic events with different peak velocities were evaluated. For all seismic simulations, viscous boundaries were introduced around the outer perimeter of the problem domain to eliminate wave reflections, thereby simulating an infinite rock mass. Seismic events were represented by a sinusoidal y-directed stress wave applied at the top of the model. The applied stress wave was superimposed on the existing in-situ stresses.

SEISMIC-INDUCED GROUNDFAIL

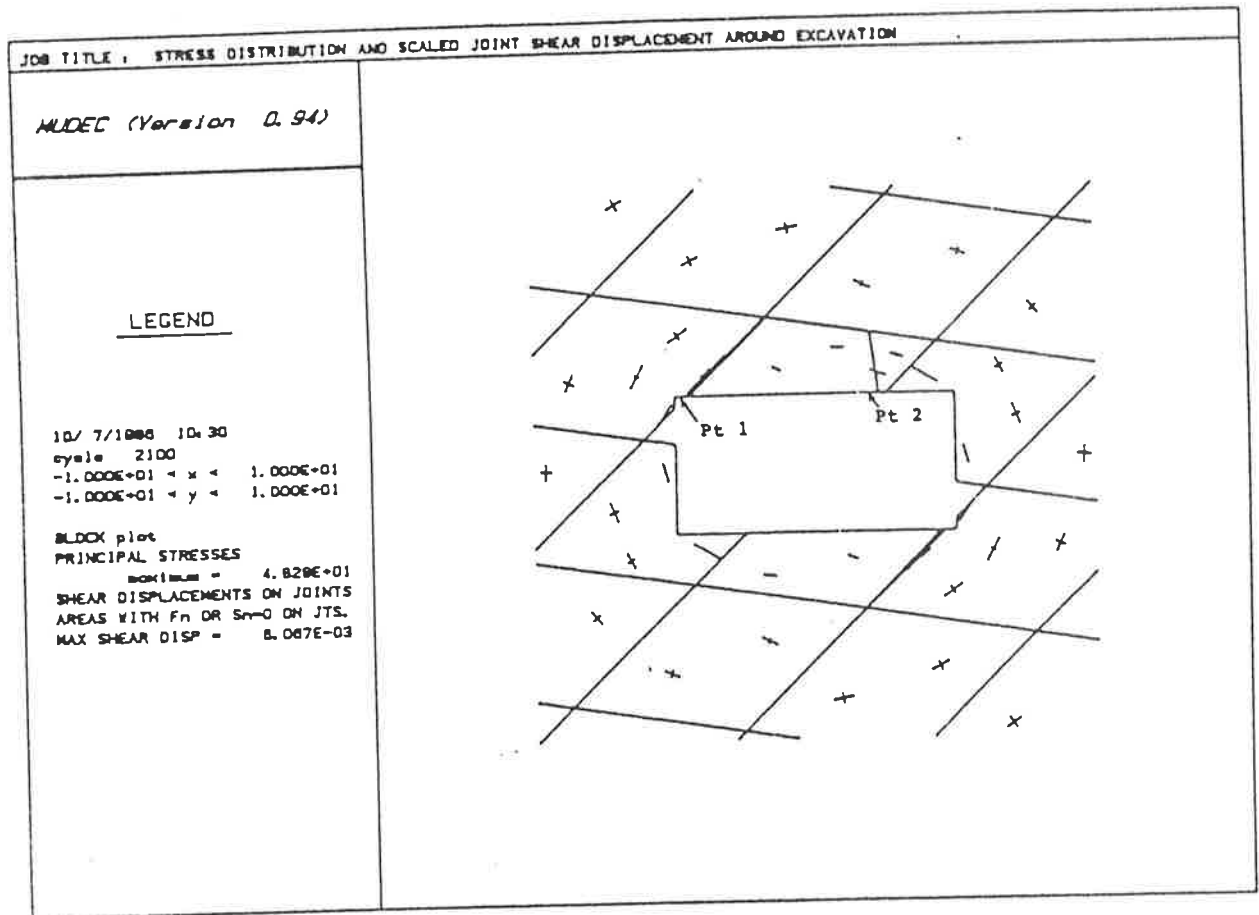


Fig. 2 Stress Distribution Around Excavation at End of Excavation Stage

In the first simulation, a peak stress of 1.25 MPa was applied. It should be noted that, due to the viscous boundary conditions in effect at the top of the model, the "effective" applied stress is $1.25 \text{ MPa}/2$, or 0.625 MPa . The stress distribution in the roof of the excavation after 0.02 seconds is shown in Fig. 3. Displacements were monitored at two points (shown in Fig. 2). Point 1 is located in the left corner of the excavation; Point 2 is located at the right corner of the roof block. Displacement versus time plots (Fig. 4) for these points essentially show an elastic response.

SEISMIC-INDUCED GROUNDFAILL

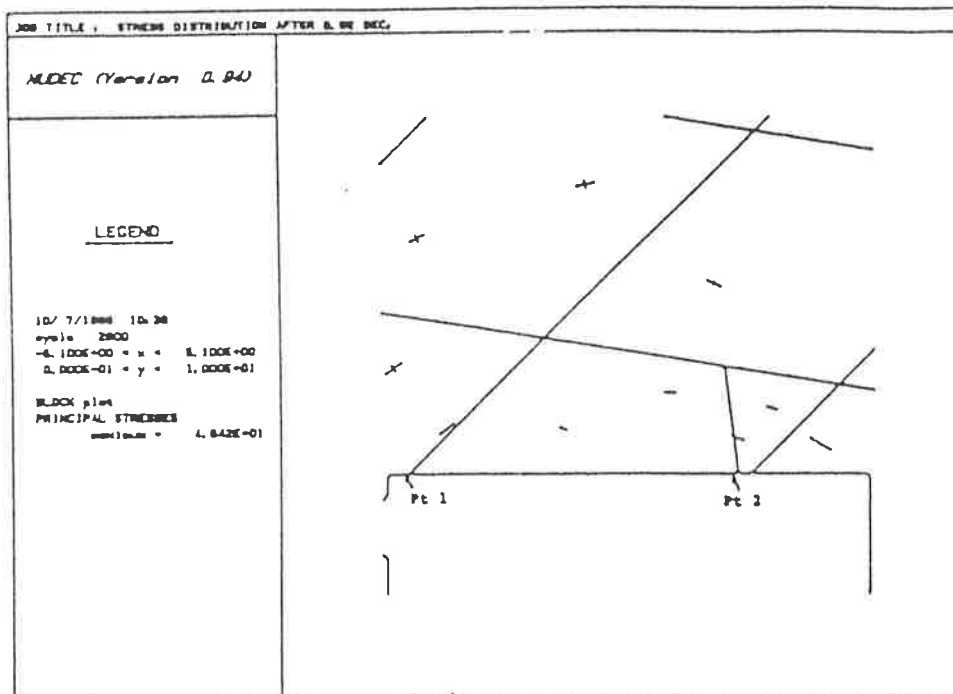


Fig. 3 Stress Distribution in Roof of Excavation After 0.02 Seconds [applied stress = $1.25x\cos(2\pi 100t)$]

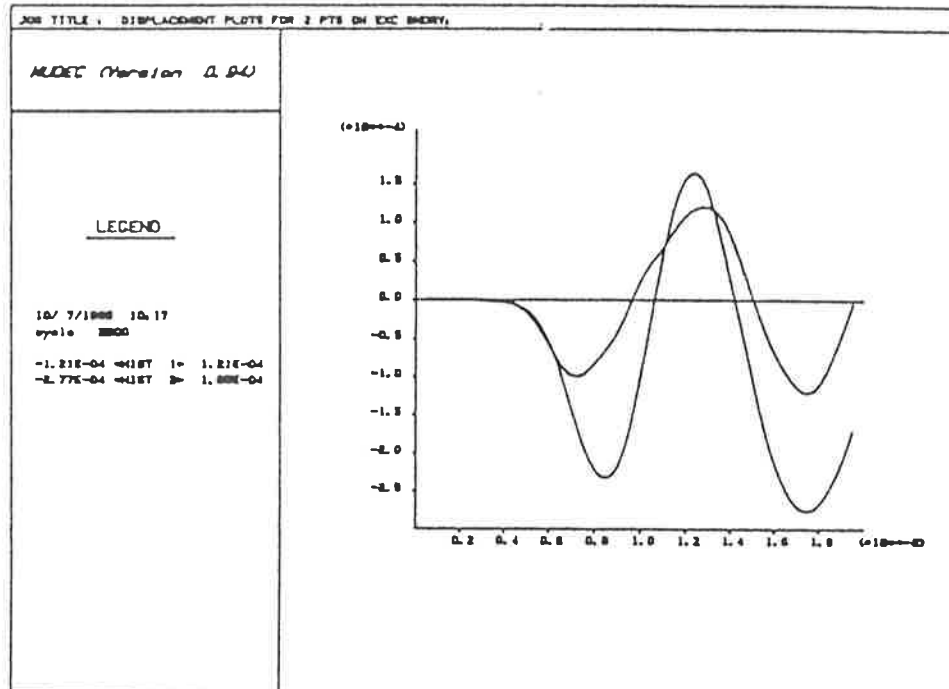


Fig. 4 Y-Displacement Histories for Two Points on Excavation Boundary [applied stress = $1.25x\cos(2\pi 100t)$]

SEISMIC-INDUCED GROUNDFAIL

It is interesting to compare estimated applied velocities with calculated velocities at the top of the model. The following equation can be used to estimate the applied velocity:

$$V = \frac{\sigma}{2(\rho C_p)}$$

$$\text{where } C_p = \frac{K + (4/3)G}{\rho}$$

Using this equation, the applied maximum velocity would be about 0.04 m/sec. Figure 5 shows a peak velocity of less than 0.06 m/sec. Differences between estimated velocities and measured velocities result from using the intact rock modulus rather than the equivalent deformation modulus which takes into account the joint deformation.

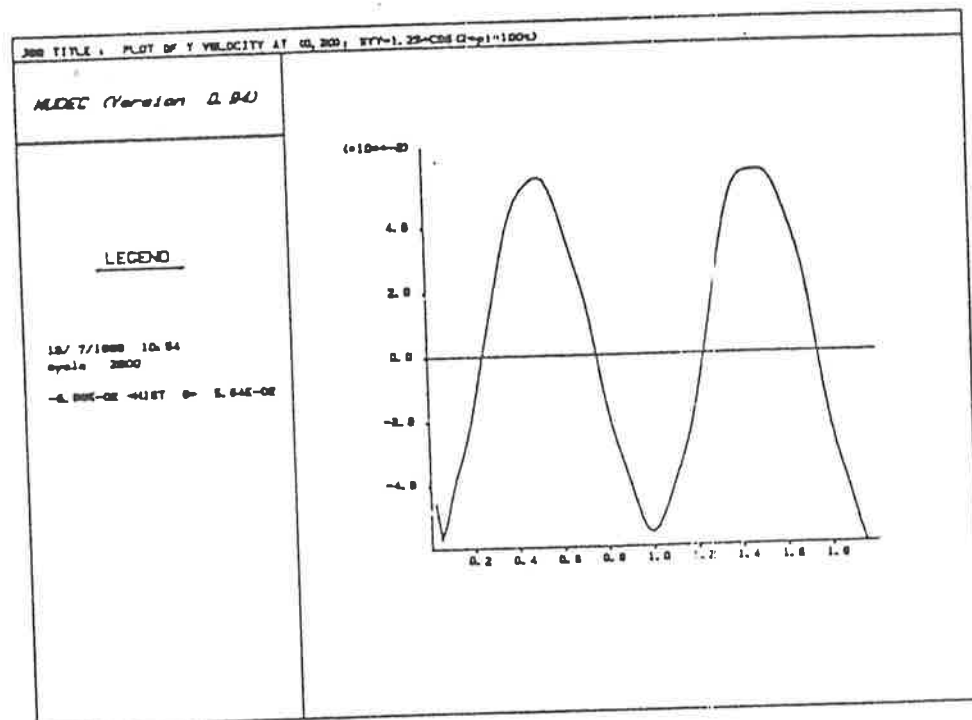


Fig. 5 Plot of y-Velocity at Top of Model for Applied Stress = 1.25xcos(2π 100t)

SEISMIC-INDUCED GROUND FALL

In the second example, a stress wave with peak stress of 12.5 MPa ("effective" stress = 6.25 MPa) was applied. The stress distribution in the roof of the excavation after 0.02 seconds is shown in Fig. 6. This figure shows that the roof block is unstressed, indicating that the block has loosened. Displacement versus time plots (Fig. 7) also indicate that the block has loosened and is falling. As a matter of interest, the problem geometry and stress distribution at three later times is presented in Figs. 8-10.

The predicted velocity (from the equation above) at the top of the problem is 0.4 m/sec. The actual recorded velocity is shown in Fig. 11. Again, differences between predicted and measured velocities result from using intact rock modulus rather than rock mass deformation modulus.

SEISMIC-INDUCED GROUND FALL

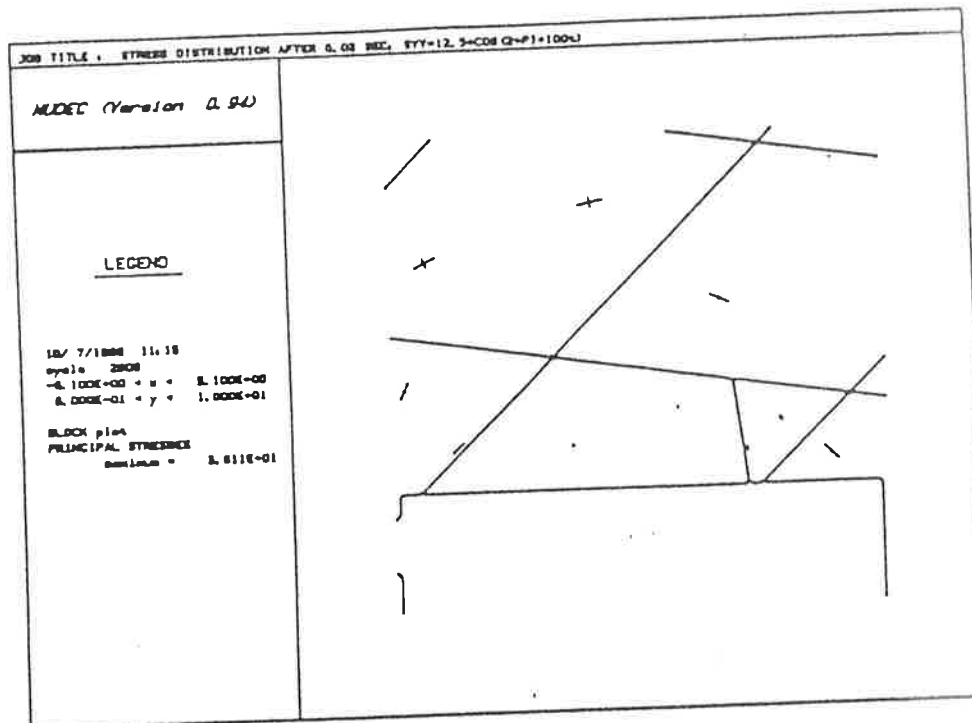


Fig. 6 Stress Distribution in Roof of Excavation After 0.02 Seconds [applied stress = $12.5 \times \cos(2\pi 100t)$]

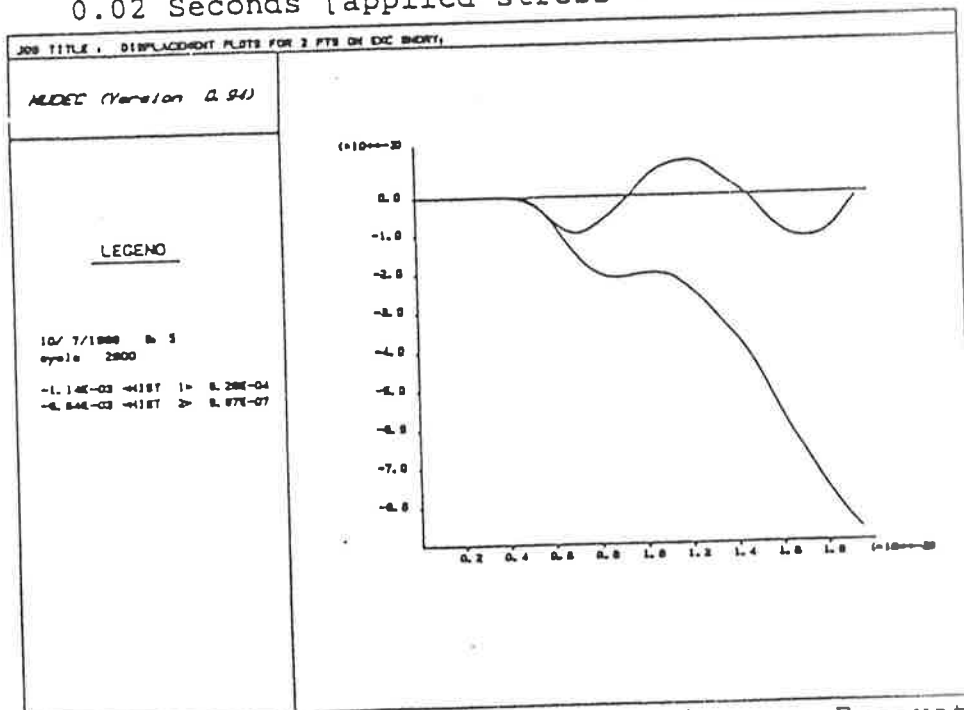


Fig. 7 Y-Displacement Histories for Two Points on Excavation Boundary [applied stress = $12.5 \times \cos(2\pi 100t)$]

SEISMIC-INDUCED GROUNDFAILL

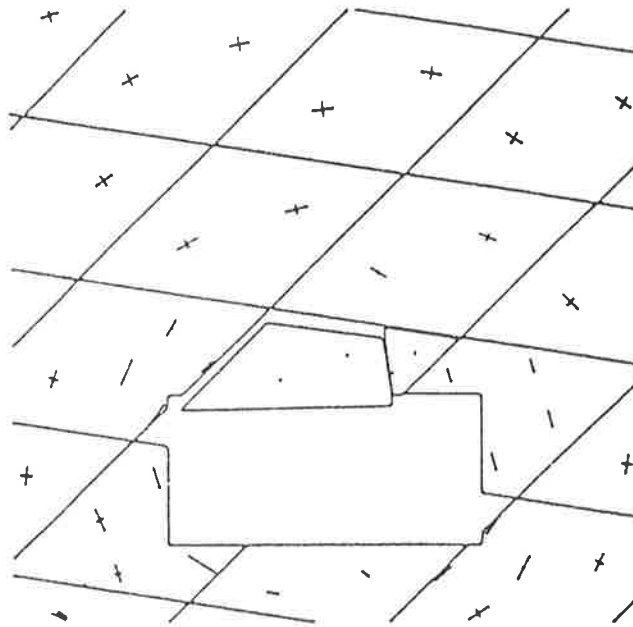


Fig. 8 Stress Distribution Around Excavation After
0.26 Seconds [applied stress = $12.5x\cos(2\pi 100t)$]

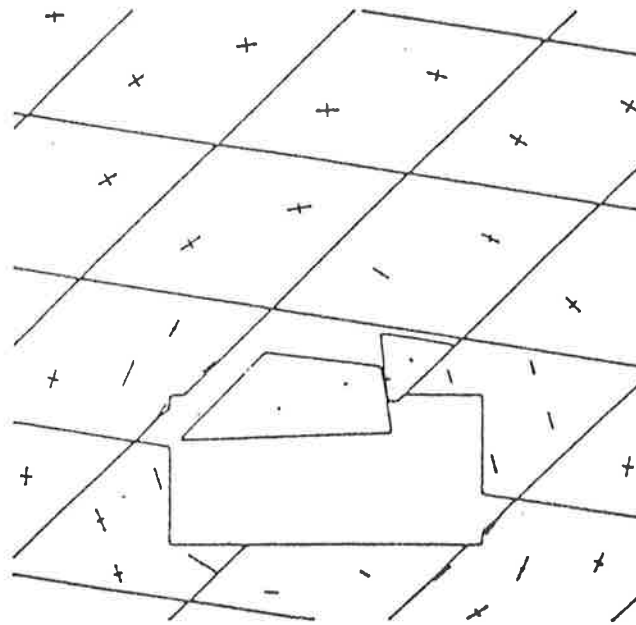


Fig. 9 Stress Distribution Around Excavation After
0.51 Seconds [applied stress = $12.5x\cos(2\pi 100t)$]

- The design of the model should be driven by the questions that the model is supposed to answer rather than the details of the system that is being modeled. This helps to simplify and control the model.
- It might even be appropriate to build a few very simple models rather than one complex model; the simple models would either relate to different aspects of the problem, or else address the same questions from different perspectives.
- Instead of trying to validate a model, one should aim to gain confidence in it and to modify it as one uses it. One's approach to the model should be like that of a detective rather than a mathematician.
- The purpose of modeling data-limited problems is to gain understanding and to explore potential trade-offs and alternatives, rather than to make absolute predictions.
- One progresses, slowly and painfully, from region 4 of Holling's diagram towards region 3 by a kind of 'bootstrap' operation. First one builds a simple model and exercises it in a conjectural way. That process almost always suggests new ways of obtaining data, or new ways of interpreting available data. New data, in turn, suggest improvements to the model or ideas for new models. Implementing those improvements leads to new data requirements or insights, and so on.

How do these ideas apply to rock mechanics? We believe they can be converted into a preliminary set of guidelines to modeling.

GUIDELINES TO MODELING IN ROCK MECHANICS

1. Be sure, before you start, that you are quite clear about **why** you are building a model and **what** questions you are trying to answer.
2. Use a model at the earliest possible stage in a project to generate both data and understanding. Do not delay while waiting for field data. You need a conceptual model [3] in place as soon as possible. A good conceptual model can lead to savings in time and money on field tests that are better designed.
3. Look at the **mechanics** of the problem. Try to identify important mechanisms, modes of deformation and likely modes of failure.
4. Think of experiments one would like to perform on the model and try to visualize, qualitatively, what the answers might be. In particular, if you have two or more conflicting ideas about what is going on in the field, propose simple numerical experiments to eliminate one or more of them (i.e. use models to falsify hypotheses).
5. Design or borrow the simplest model that will allow the important mechanisms to occur, and could serve as a laboratory for the experiments you have in mind.
6. Implement the model, choose your simplest experiment, and run it.

- If the model ties in with your expectations, proceed to more complex experiments; if not, identify the weaknesses in your thinking (or the weaknesses in the model) and remedy them before continuing. Take a similar approach to the other experiments: explore possibilities and be critical!
7. If your only available model has weaknesses that you cannot remedy (e.g. you only have a two-dimensional model for a situation that is clearly three-dimensional), make a series of simulations that will bracket the true case. In many instances the bounds will be sufficiently close so that useful insight will be obtained for the case that could not be simulated.
 8. Once you have learned all you can from the simple model or models, you may want to run more complex models to explore those neglected aspects of the geology that are most likely to affect the behavior of the simple model. You may also want to do one or two design runs using more complex models, but these will often be more cosmetic than illuminating; it is often better, for design, to develop simple equations based on the mechanisms revealed by the models. These equations are then used for design rather than numerical models.

Note that steps 4 through 6 are crucial. They may be implemented in a number of different ways. For example, you might want to test a series of simple alternative models rather than just one model. In running the model (or models) there are bound to be parameters whose values you will have to guess at. Be sure to run the model for ranges of plausible values. Where the model output is relatively insensitive to parameter values, you are safe; where it is sensitive, the form of sensitivity should suggest limits to the parameter values or field experiments that must be performed.

Visualizing and anticipating solutions before running a model is an important discipline. Hardy Cross [4] has this to say about structures: "The ability of a designer of continuous structures is measured chiefly by his ability to visualize the deformation of the structure under load. If he cannot form a rough picture of these deformations when he begins the analysis he will probably analyze the structure in some very awkward and difficult way; if he cannot picture these deformations after he has made the analysis, he doesn't know what he is talking about."

It should also be noted that implicit in the above guidelines is a rule that rock mechanics models should never be run only once; it is in the sensitivity of the results to changes in parameters and assumptions that the model is most informing.

The above guidelines are not presented as a panacea for rock mechanics modeling; they are presented as an approach that exemplifies an underlying philosophy. People commonly say "you only get out of a computer what you put into it" or "the results are only as good as the data". At one level these comments are of course true, but at another they are misleading. Modeling in a cautious and considered way leads to new knowledge or, at the least, fresh understanding. Even the

DATA LIMITED PROBLEMS

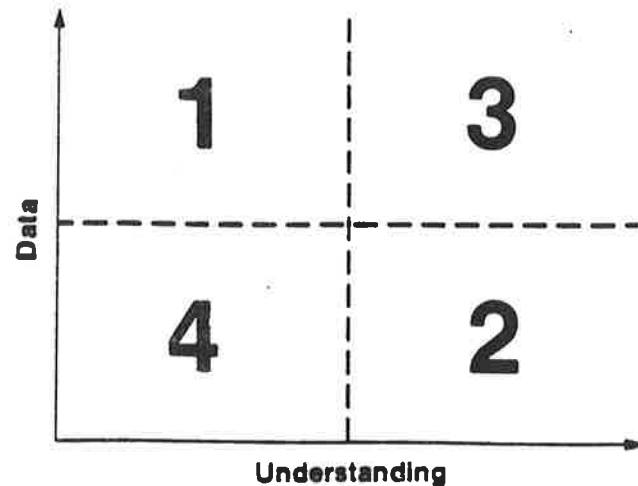


Fig. 1. Holling's [1] classification of modeling problems.

Fig. 1 introduces a classification, due to Holling [1], that modelers in ecology have found to be useful. Holling introduces two axes, one that is a measure of the quality and/or quantity of available data, while the other measures the understanding of the problem to be solved. He then divides the quadrant between the axes into four regions. In region 1 there are good data but little understanding; this is where statistics is the appropriate modeling tool. In region 3 one has both the data and the understanding; this is where models can be built, validated and used with conviction. Regions 2 and 4 relate to problems that are data-limited in the sense that the relevant data are unavailable or cannot easily be obtained.

We would argue that problems in rock mechanics usually fall in the data-limited category; one seldom knows enough about a rock mass to model it unambiguously. Many of the problems in other branches of engineering mechanics, however, belong in region 3. Some of the old arguments against modeling in rock mechanics were, in essence, based on the recognition that the rationale for modeling in region 3 did not necessarily apply to the more amorphous data-limited problems of regions 2 and 4.

The modern trend is to hope to collect sufficient data to move rock mechanics into region 3, and so use models in a conventional way. Our hypothesis is that neither the old arguments nor modern goals are useful. The old argument is easily disposed of: if modeling in the conventional way is inappropriate to rock mechanics, that only creates the need for more thoughtful, unconventional modeling. The modern trend is more pernicious; it leads, as we pointed out in the Introduction, to more and more complex models and more expensive site evaluations without a concomitant improvement in either understanding or design. There are two salient arguments against it: the first is that it is futile ever to

expect to have sufficient data to model rock masses in the conventional way, and the second is that as one includes more and more detail, one loses intellectual control of the model and so it becomes less instead of more effective.

What is useful is to ask how modeling tools developed for the well-posed problems in region 3 can be applied to rock mechanics problems in regions 2 and 4. We need to accept that rock mechanics does not fit the more conventional mould and develop a philosophy of modeling that fits rock mechanics instead of trying to fit rock mechanics to the prevailing philosophy.

MODELING GUIDELINES

Starfield and Bleloch [2], in the context of ecological modeling, list some of the characteristic differences between problems in region 3 and those in regions 2 and 4 of Holling's diagram:

- The fundamental problem is the question of resolution; in region 3 one knows what level of detail is necessary to solve a problem and when simplifying assumptions are appropriate, whereas in regions 2 and 4 one is nervous of over-simplifying the problem. As a result, modelers tend to concentrate on detail and build complex, unwieldy models.
- Problems in regions 2 and 4 are often ill-posed, leading to difficulties in interpreting the results and the nagging question of whether or not the correct problem has been modeled.
- Validation of models in region 3 is a well understood process; in regions 2 and 4 it may be impossible to validate a model, but instead of recognizing that impossibility and developing pragmatic approaches that address it, modelers pay lip-service to the process of validation.
- Models in region 3, once validated, can be used, almost routinely, for prediction. Models in regions 2 and 4 can never be used routinely; they have to be used far more cautiously and thoughtfully.

Anybody who has built a rock mechanics model will recognise that these points are as pertinent to rock mechanics as they are to ecology.

Starfield and Bleloch also suggest ways of thinking about models that address the differences between the well-posed problems of region 3 and the data-limited problems of regions 2 and 4:

- A model is a simplification of reality rather than an imitation of reality. It is an intellectual tool that has to be designed or chosen for a specific task.

Towards a Methodology for Rock Mechanics Modeling

A.M. STARFIELD

P.A. CUNDALL

Rock mechanics models fall into the class of 'data-limited problems;' one seldom knows enough about a rock mass to model it unambiguously. Modelers are beginning to realise that data-limited problems require a very different modeling approach from that developed in, for example, electrical or aerospace engineering.

It follows that one cannot use models in rock mechanics in a conventional way, and that there is a need to adopt a distinctive and appropriate methodology for rock mechanics modeling. Some guidelines and heuristics, which may be considered as the first steps towards developing such a methodology, are presented. Three case studies are then used to illustrate the application, in practice, of these ideas.

INTRODUCTION

Perspectives on modeling in rock mechanics have changed dramatically during the past quarter of a century. There was a time when the focus of attention in rock mechanics was on laboratory and field measurement, and models (mathematical or computational) were generally thought to be either irrelevant or inadequate. Modelers spent a large part of their efforts trying to persuade skeptics that modeling was a useful engineering exercise.

The focus has shifted from measurement to computation; nowadays everybody builds models.

It would be comforting to believe that this popularity has been

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hard won, that modelers in rock mechanics have converted the erstwhile skeptics. While this is partly true, there are probably three more important reasons for the upsurge in modeling:

- ease of access to versatile and powerful computer packages;
- a dramatic increase in the ability to include geological detail in the construction of a model;
- the manifest success of modeling in other branches of mechanics.

Each of these reasons carries with it both a positive and negative connotation and highlights why there is a need to develop an explicit methodology for modeling in rock mechanics.

Ease of access to computer packages provides the rock engineer with a kit of computational tools. The problem is in knowing how to use the kit effectively and in having an understanding of both the strengths and weaknesses of the tools. Rock mechanics may be passing through the phase where the tool is perceived as a solution rather than as a means to a solution. We need to develop the intuition of the craftsman who knows the inherent limitations of the materials he works with, and is intrigued but not bedazzled by new and more versatile tools.

The lack of geologic detail was a major stumbling block to early acceptance of modeling in rock mechanics; models appeared to be such gross over-simplifications of the geology that few stopped to ask whether they might nevertheless be useful. The ability to include more detail is welcome, but only up to a point. As a carryover from the past we still seem to have an implicit credo that more detail implies a better model. It is an addictive credo: the modeler becomes hooked on bigger and 'better' models and these in turn need more data, leading to more field and laboratory measurements. At best these efforts are a waste of time and resources; at worst they are counter-productive, concealing the wood for the trees. After all, we build models because the real world is too complex for our understanding; it does not help if we build models that are also too complex. The art of modeling lies in determining what aspects of the geology are essential for the model. The challenge is to turn that art into a methodology.

Finally, the success of modeling in other engineering fields has been a spur to modelers in rock mechanics, but we must be wary of emulation because the differences between rock mechanics and, for example, aerospace or even structural mechanics, may be more important than the similarities. Modeling techniques and approaches, as well as expectations of what modeling can achieve, that are appropriate to the one may not be appropriate to the other. The challenge to modelers in rock mechanics is to recognise these differences and to develop a distinctive modeling methodology that is both purposeful and effective.

In this paper we will try to identify some of the distinctive features of rock mechanics modeling and to suggest useful approaches and perspectives.

SEISMIC-INDUCED GROUND FALL

```

* track the damping, x-displacement, and y-displacement over time
HIST N=10 DAMP XDIS=0,7 YDIS=0,7
* dump out the XDIS-value to screen every 10th timestep during cycle
CYCLE 100
* save consolidated state
SAVE ROOFG.SAV
PRINT MAX
*****
* make excavation
DELETE -5,5 -2.5,2.5
* hold y-velocity to zero to keep block assemblage from rising due
to
* unbalanced reactive forces now that we have removed mass
BOUND -26,26 -21,-19 YVEL 0.0
BOUND -26,26 19,21 YVEL 0.0
HIST N=50
CYCLE 2000
* save excavated state
SAVE ROOF1.SAV
PRINT MAX
*****
* Apply seismic load from top (PEAK VELOCITY=0.04 m/sec)
*
* set up nonreflecting boundary
BOUND MAT=1
BOUND -26 -23 -21 21 XVIS
BOUND 23 26 -21 21 XVIS
BOUND -26 26 -21 -19 XVIS YVIS
BOUND -26 26 19 21 XVIS YVIS
* apply sinusoidal stress wave
BOUND -26 26 19 21 STRESS 0 0 -1.25
BOUND YHIST=COS(100.0,0.02)
PRINT BOUND
DAMP 0.1 1.0 MASS
RESET TIME HIST DISP ROT
HIST N=10 YDIS (-4.48,2.57)
HIST YDIS (0,2.57) YVEL (0,2.57) YVEL (4,2.57) YVEL(-4.48,2.57)
* more history traces to check exactly what is being applied at
boundary
* history numbers 6-16.
hist Yvel (0,20) Yvel (25,10) Yvel (25,-10) Yvel (0,-20)
hist Yvel (-25,-10) Yvel (-25,10)
hist SXX (25,10) SXX (25,-10) SXX (-25,-10) SXX (-25,10)
HIST SYX (0,20)
CYC 800
PR MAX
SAVE ASEISMIC.SAV
*****

```

SEISMIC-INDUCED GROUND FALL

```
*   Apply seismic load from top (PEAK VELOCITY=0.4 m/sec)
*
RESTART ROOF1.SAV
* set up nonreflecting boundary
BOUND MAT=1
BOUND -26 -23 -21 21 XVIS
BOUND 23 26 -21 21 XVIS
BOUND -26 26 -21 -19 XVIS YVIS
BOUND -26 26 19 21 XVIS YVIS
* apply sinusoidal stress wave
BOUND -26 26 19 21 STRESS 0 0 -12.5
BOUND YHIST=COS(100.0,0.02)
PRINT BOUND
DAMP 0.1 1.0 MASS
RESET TIME HIST DISP ROT
HIST N=10 YDIS (-4.48,2.57)
HIST YDIS (0,2.57) YVEL (0,2.57) YVEL (4,2.57) YVEL(-4.48,2.57)
* more history traces to check exactly what is being applied at
boundary
* history numbers 6-16.
hist Yvel (0,20) Yvel (25,10) Yvel (25,-10) Yvel (0,-20)
hist Yvel (-25,-10) Yvel (-25,10)
hist SXX (25,10) SXX (25,-10) SXX (-25,-10) SXX (-25,10)
HIST SYX (0,20)
CYC 800
PR MAX
SAVE BSEISMIC.SAV
STOP
```

SEISMIC-INDUCED GROUNDFAIL

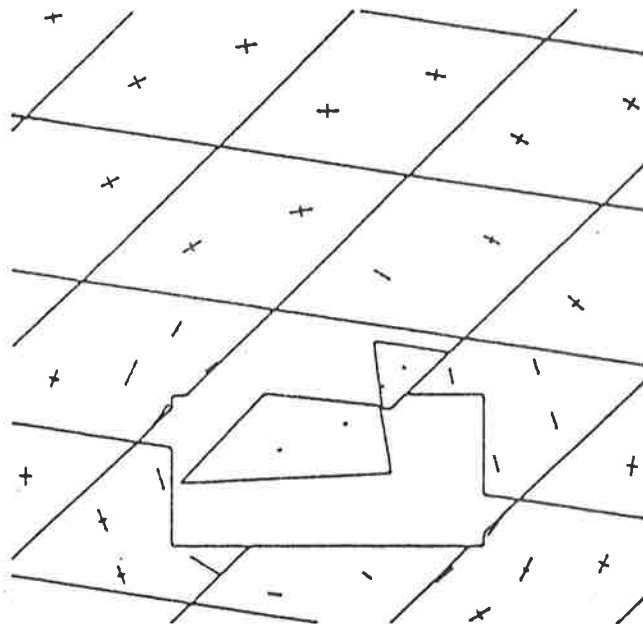


Fig. 10 Stress Distribution Around Excavation After 0.75 Seconds [applied stress = $12.5x\cos(2\pi 100t)$]

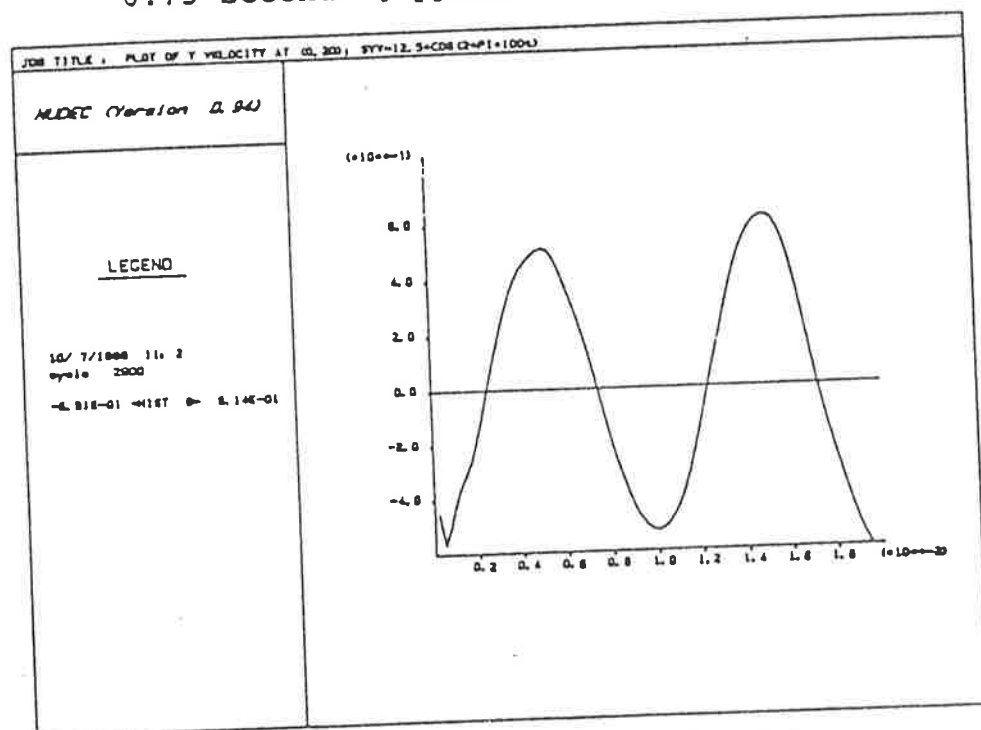


Fig. 11 Plot of Y Velocity at Top of Model [applied stress = $12.5x\cos(2\pi 100t)$]

SEISMIC-INDUCED GROUNDFAIL

INPUT COMMANDS FOR SEISMIC-INDUCED GROUNDFAIL EXAMPLE

```

START
HEADING
(Roof) Example of seismic induced roof collapse
*
ROUND 0.01
WINDOW -30,30 -30,30
* define original boundary of modeled region
BLOCK -25,-20 -25,20 25,20 25,-20
* generate joint pattern over entire original region
JREGION -25,-25 -25,25 25,25 25,-25
JSET 45,0 200,0 0,0 5.0,0
JSET -9,0 200,0 0,0 5.0,0
* put in joints needed for the later excavation
CRACK -5.01,-2.51 5.01,-2.51
CRACK -5.01, 2.51 5.01, 2.51
CRACK -5,-2.5 -5,2.5
CRACK 5,-2.5 5,2.5
CRACK 2.25,2.5 1.93,5.0
* delete small blocks to improve timestep
DELETE -30,30 -30,30 2.0
*
*
* generate FDEF zones and assign joint properties (MAT=1 &
JMAT=1;default)
GENERATE -30,30 -30,30 AUTO 9.0
CHANGE -30,30 -30,30 JCONS=2
PROP MAT=1 D=0.00300 K=39060 G=31780
PROP MAT=1 KN=5000 KS=5000 JKN=20000 JKS=20000
PROP MAT=1 F=0.577 JF=0.577
ROUND 0.1
*
*
* apply boundary conditions and initial conditions to
* consolidate model under field stresses
BOUND LSTRESS=-24.0, 0.0, -24.0 0 0 0 -.03 0 -.03
INSITU STRESS=-24.0, 0.0, -24.0 YGRAD=-.03 0 -.03
GRAV 0.0 -10.0
*
*
* specify conditions during solution
DAMP AUTO
*
*
```

writer of a modeling program learns new things when running his program; a system of interacting parts often behaves in ways that are surprising to those who specified the rules of interaction. Exploring and explaining those interactions is a form of learning; the model becomes a laboratory for those who built it.

The philosophy that underlies the above guidelines is that models should be used in rock mechanics with the same caution and curiosity that exemplifies good laboratory work. In the next section we use three case studies to illustrate this approach.

THREE CASE STUDIES

1. The mystery of the staggered path

A large number of micro-seismic events were recorded during fluid injection 2000m below surface in a geothermal energy project in Cornwall. The locations of events indicated a general trend in the spread (apparently of the injecting fluid) that was, surprisingly, not in the direction of the major joint sets, but rather in the direction of major principal stress [5], [6].

At first sight this result is puzzling; injection pressures were too low to create new fractures, so the fluid must flow along existing joints. The rock/fluid system was modeled with program FRIP [7], although the problem may seem to be a poor candidate for numerical modeling in view of the data deficiency: there is little hard evidence on joint persistence, joint properties, and the initial fluid and mechanical state. However the model is conceptually simple: the intact rock is represented as a linear, elastic material. The joints deform linearly in shear, until the shear stress becomes equal to the shear strength, when Coulomb slip occurs. Since shear strength is equal to the friction coefficient multiplied by the effective normal stress, the effect of fluid pressure is to decrease shear strength and allow slip. Joints are programmed to dilate at a constant angle when sliding: this dilatation increases the permeability of the joint in proportion to the square of the aperture.

Treating the model as a laboratory, one can make a series of guesses about joint persistence, joint properties, etc. When these guesses are made, the model reveals a mechanism that explains how the general direction of propagation can differ from the joint directions. Fig. 2 shows the results from a typical run: there is not one single fluid pathway, but a complex series of staggered paths that transport the fluid in a general direction that differs from the individual joint directions. This "en echelon" effect is even more pronounced when the model contains non-continuous joints.

The explanation for the behavior is as follows. Increased fluid pressure causes slip on the one joint that intersects the injection point and is also a member of the set (call it the "primary set") that has the lower normal stress. At each end of

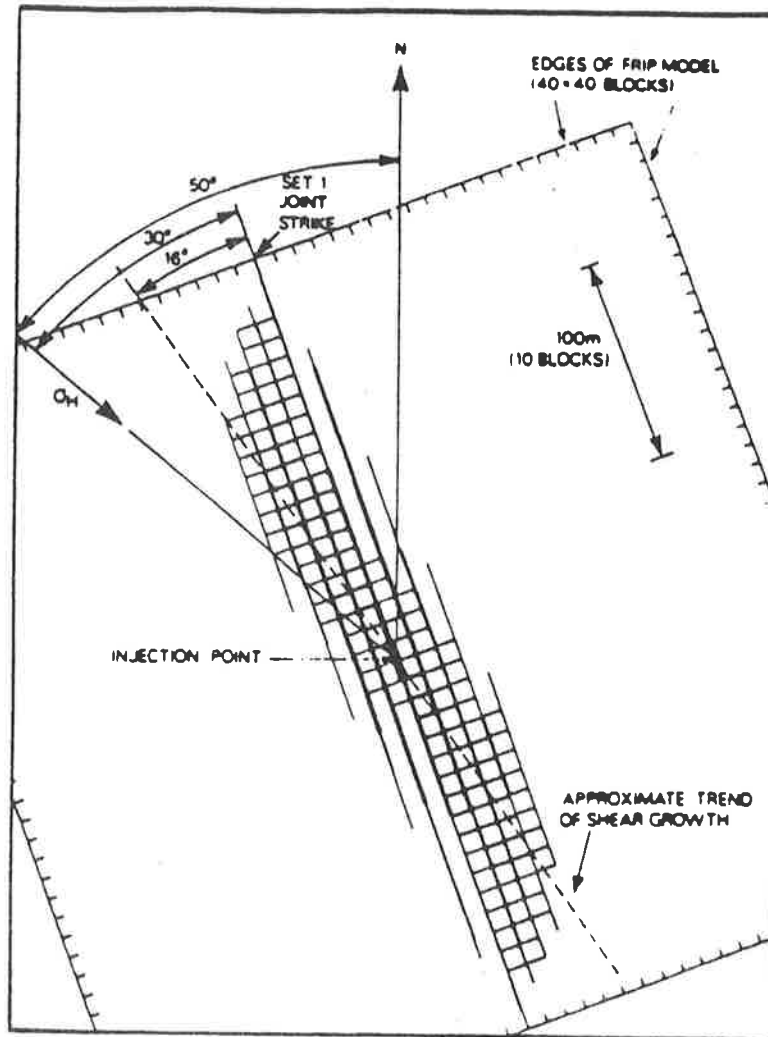


Fig. 2. Plan view of dilated joints, from a simulation by the FRIP program; after Pine (1983).

this slipped section, the in-situ stresses parallel to the joint are thereby increased on one side and decreased on the other. On the sides with the decreased stresses, some of the cross-joints are enabled to slip because their normal stresses are reduced. Fluid is then able to migrate to the end of the cross-joint (owing to slip-induced dilation), and the whole process continues, since fluid has now found its way to the next joint in the primary set.

Once we understand this basic mechanism, we can exercise the model with various parameters so as to discover what conditions are necessary to bring about the observed behavior. In a sense the computer supplies some of the missing field data, since the model only exhibits the observed pattern of fluid migration for a rather restricted set of data on joint geometry, in-situ conditions and material properties.

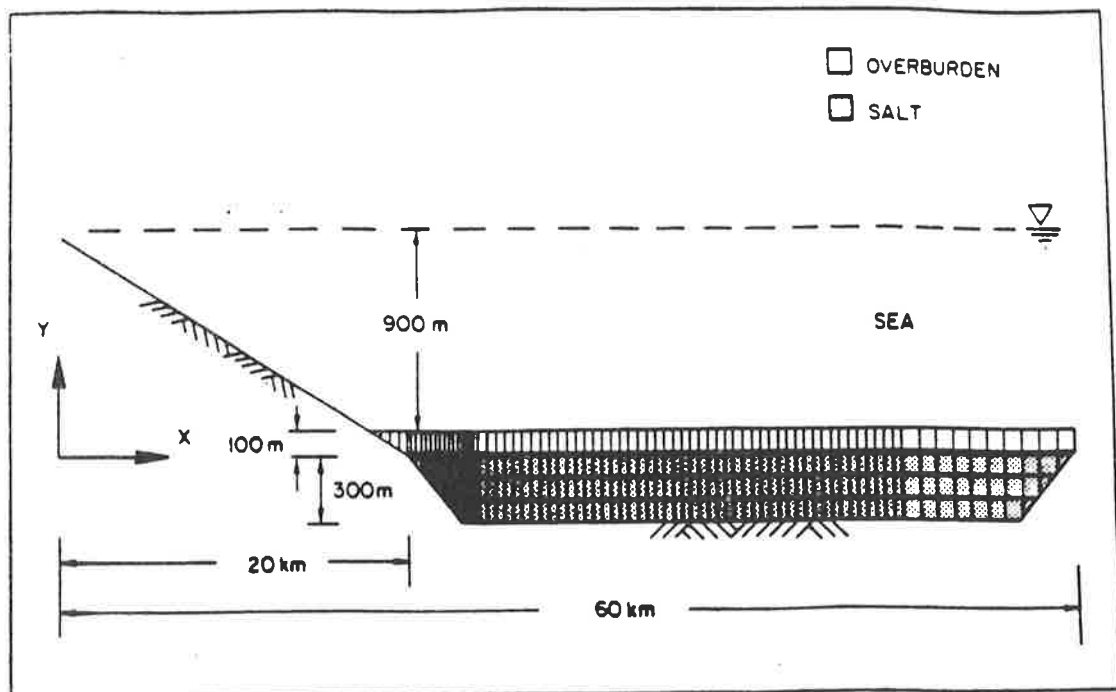


Fig. 3. Initial geometry of a numerical grid, showing 300m of salt under 100m of overburden. The program adds sloping layers of overburden from the left; after Last (1986).

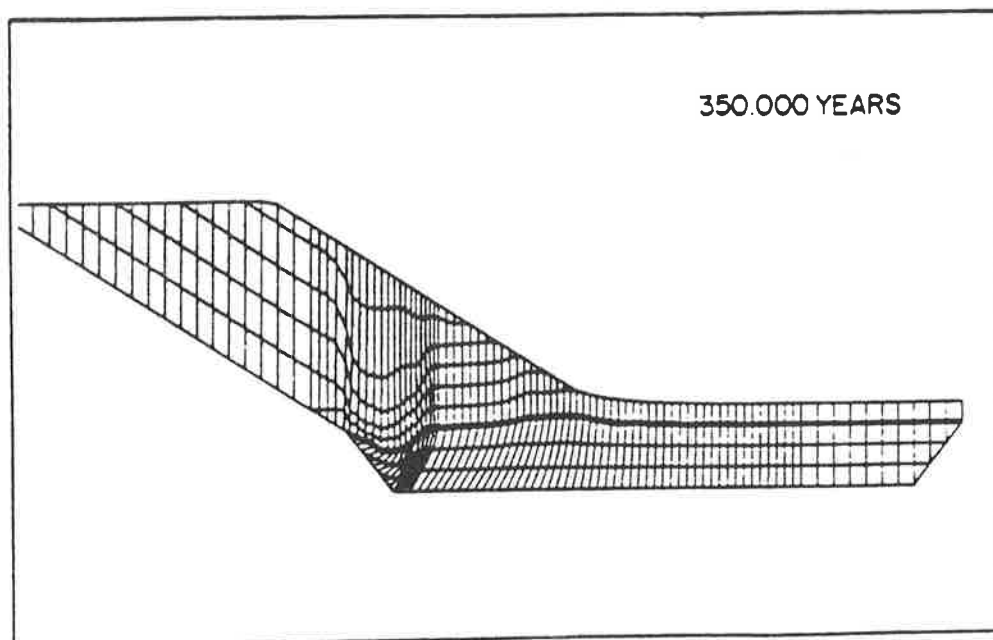


Fig. 4. Numerical grid after simulated time of 350,000 years: a basin has formed in the new overburden, and a salt dome is beginning to form at the toe of the slope; after Last (1986).

2. A study of creeping sediment

In sediment layers deposited over thousands of years by large rivers, complex structures such as faults and salt domes are often seen but not often understood, partly because of their apparent complexity. Since there are almost no data on properties, boundary conditions or initial conditions, geologists are uninhibited when

inventing scenarios, but one prime suspect is that the weight of overburden causes an underlying layer (of salt, for example) to creep, and thereby induce distortions into the overburden.

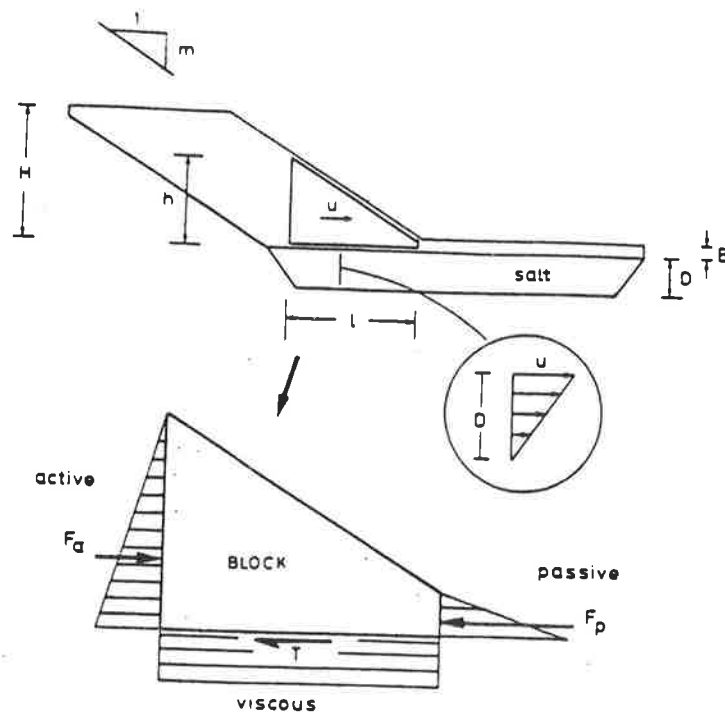
A computer model is valuable in two ways. First it provides some specific examples that serve to keep our imaginations within realistic bounds. Second, by exercising some of the simpler cases covered, it is able to test the theories critically. Last [8] uses a large-strain finite difference model to represent a layer of viscous salt over which frictional material is progressively deposited. Fig. 3 illustrates the initial state of the model, and Fig. 4 the state at 350,000 years. Even this simple model produces quite complicated patterns of distortion: a subsidence basin has formed to the left, and the beginnings of a salt dome are forming on the right. These types of structure are certainly observed in the field. The model has thus provided a picture of the distortions that can be induced when salt creeps outwards in response to an overburden load.

This is not the end of the story. As Last points out, the region between the basin and the "dome" seems to be moving as a rigid body, while the left and right sides are in the active and passive plastic states. Considering horizontal equilibrium, a simple equation can be written that relates many of the important parameters in the problem (see Fig. 5).

The numerical model has suggested a simple mechanism that was obscured by the apparent complexity of the field data. The mechanism is then described by an equation that can readily be applied to future field observations. The conceptual model and the equation act as filters that help us to interpret incoming evidence by reducing data overload.

3. The paradoxical rock slope

A planned rock slope was to be constructed in sedimentary rock. The bedding planes dip steeply, so the designer cleverly arranged for the slope to dip at the same angle as the bedding (see Fig. 6). In this way he hoped to avoid failure, since potential failure planes do not daylight in the slope. When the slope was constructed however, the face was found to break up continuously, with sheets of rock sliding from the slope. On closer inspection some horizontal fracture planes were noticed, but it was not immediately evident how these could be associated with the instability since rock does not normally move horizontally.



Horizontal Equilibrium:

$$F_a - F_p = T$$

$$\frac{1}{2} (\rho g) [k_a h^2 - k_p B^2] = \eta \left(\frac{u}{D} \right) l$$

Fig. 5. A simple formula that expresses the simplified mechanics of the salt/sediment system; after Last (1986).

A numerical simulation was made with the distinct element program UDEC [8], using various values of friction for the bedding planes and the fracture planes. It is found that the horizontal planes are indeed the culprits since they allow the rock blocks to rotate (see Fig. 7, which shows successive plots from the computer simulation). By making several runs with different properties, we define, empirically, the conditions under which the failure can occur, and can design remedial measures.

Perhaps more importantly, once the computer model has suggested (or confirmed) the mechanism of failure, we can try to develop a simple formula based on equilibrium of forces. For example, an equation can be derived for a single, thin slab that relates friction angle to slope angle, at limiting equilibrium.

The failure mechanism seems obvious after it has been identified, but in the field things are not often so clear, as one can be overwhelmed by extraneous factors.

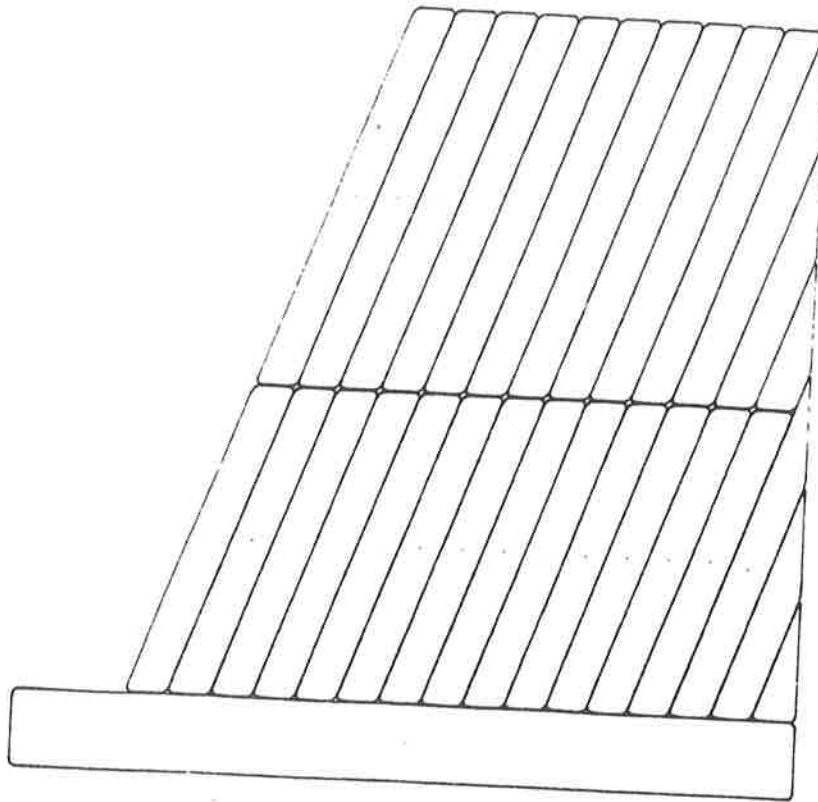


Fig. 6. Initial geometry of a rock slope, in which the slope face is parallel to the bedding planes. Note that two horizontal faults are also present.

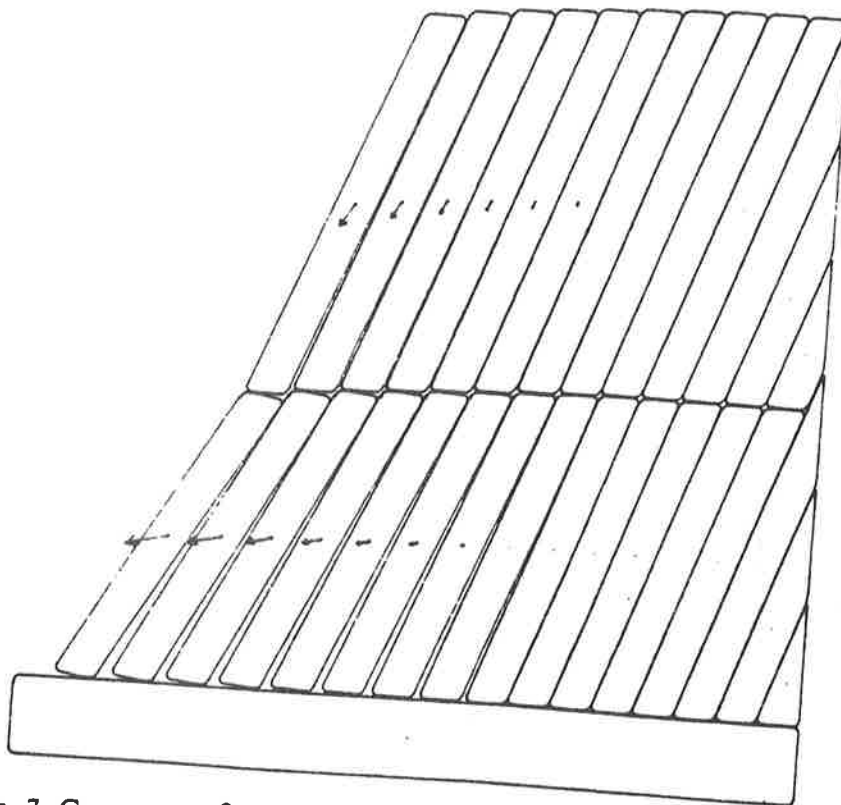


Fig. 7. Geometry of rock blocks after some time has elapsed. The calculation was made by program UDEC for a friction coefficient of 0.15. The horizontal faults allow block rotations to occur.

The example illustrates the importance of selecting a model that is capable, in principle, of reproducing the suspected mechanisms. We might have tried using, as an alternative, a transversely isotropic material to represent the effect of the bedding planes, but a slope constructed of this material would not be able to fail in a mode that involves rotation.

Notice how each of the case studies described above was motivated by puzzling field observations. Notice too that in each case there were very few mechanical or geometrical data; all three fall in the category of data-limited problems. We have deliberately presented the cases so as to resemble detective stories, precisely because the modeler should be playing the role of detective. The modeling exercises led, in all three cases, to plausible mechanisms that explained the puzzling observations, and from there to further investigations and understanding. In at least one case, once the mechanisms were highlighted by the model, simpler analytic solutions could be found to 'explain' the modeling results even more succinctly. All three cases illustrate how the computer can be used as a 'laboratory' and exemplify the 'bootstrap' approach of Starfield and Bleloch [2].

It is often tempting, once the modeling process is complete and new understanding has been acquired, to point out that the mechanisms revealed are 'obvious' and that the modeling was not really necessary. This is easy to say with hindsight; those who read detective novels also tend to say: 'I knew who did it all along,' but only **after** they have finished the book!

CONCLUDING REMARKS

Historically, modeling in rock mechanics has been driven by a perceived need to include more geological realism. That drive has led to large, clumsy and complex computer models that people build and run, not because they are an integral part of the design process, but because it is considered irresponsible not to bolster a design with plots of stress contours and the like.

We hope that we have convinced at least some of our readers that detail can smother a model, and that simplification is a crucial part of rock mechanics modeling. There is a dialectic between geological detail and engineering understanding.

We also hope that we have shown how a model is an aid to thought, rather than a substitute for thinking.

Rock mechanics is an experimental science if only because its subject, rock, cannot be described in a meaningful way without measurement. Rock engineering is the discipline that exploits the science for design purposes, and so it too must be based on measurement. Yet, paradoxically, measurement and experiments in rock masses are costly, difficult to perform or even impossible to carry

out. It follows that modeling, as an ersatz form of experimentation, is an essential ingredient of both the science and the design discipline. We suggest that it is both necessary and constructive to think of rock mechanics modeling in this way; the first rule of a modeling methodology is one that says "plan the modeling exercise in the same way as you would plan a laboratory experiment."

Methodolgy is bound to become a vital area of debate in rock mechanics. In the first place there is an urgent need for it, if only because the ability to communicate, argue from and defend a model will be put to the test in large engineering projects such as the underground storage of high-level radioactive waste. In the second place, the burgeoning field of expert systems and knowledge engineering provides a 'language' for talking about, teaching and implementing methodology.

This paper provides a pragmatic philosophy and some robust heuristics for modeling. It is conceived as one of the opening statements in the upcoming debate. Methodology is not a subject that lends itself to research proposals, higher degrees or even papers in engineering journals, but it is however a part of everything we do. Methodology will develop if everybody who builds a model pauses to think about why they are building it at all, why they built one model rather than another, whether they could have built a more useful model, and how their model has influenced their understanding or design. The purpose of this paper is to encourage people to do this, and to think, talk and write about it.

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