See Pocket 2 for encl.

DEPARTMENT OF MINING & GEOLOGICAL ENGINEERING
University of Arizona
Room 229, Mines
Tucson

6-19-86

MEMO TO: Bonad Jagannath

86 JUN 24 P5:18

SUBJECT: COMMENTS.

I enclose comments & will prepare more detailed comments on seals before Wednesday.

If you need to leave a message Try:
Quality Inn, Tuscaloosa,
(205) 556-9690
or Conference
(205) 348-3025

WM-RES
WM Record File B6934 BDM

WM Project 10, 11, 16
Docket No.
PDR ✓
LPDR B, N, S

Distribution:
x Jagannath

(Return to WM, 623-SS)

8607150302 860619
PDR WMRES EEIUARZ
B-6934 PDR

3160

See B6934 6/19/86
TO: B. Jagannath
Comments

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.15.
WM Branch.
Comment topic. Thermal properties

(B) Was the comment addressed in the Final EA? NO - unless the
Where? inclusion of the word generic p 3.54 para 5 indicates that most of the data is suspect. Apparently none of the cap rock (169 - 234m) and little of the rocksalt (234 - 387m) yielded good core. This is an important conclusion and should be addressed in the report. Shaft sinking thro' 140+m of plastic clays & 60+ m of broken cap-rock is a daunting prospect!

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed? No!
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 4.1
WM Branch.
Comment topic. Exploratory shaft facility.

(B) Was the comment addressed in the Final EA? NO
Where? There is a case where anomalous zones (edge and central) are known to exist (p 3.27) and the tunneling required to demonstrate their location is limited (say 5-6000 ft) for proving the lateral extent.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 4.2
WM Branch.
Comment topic. Final disposition

(B) Was the comment addressed in the Final EA? (NO)
Where?
unless Table 4.14 partially helps.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 4.10
WM Branch.
Comment topic. Ground water.

(B) Was the comment addressed in the Final EA? No
Where? Probably no longer relevant.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 4.13
WM Branch.
Comment topic. Exploratory shaft alternatives

(B) Was the comment addressed in the Final EA? YES
Where? p 4.30 – 4.57 This expanded action eliminates shaft drilling & discussed D&B sinking in detail. The rationale is still missing.

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 5.3.
WM Branch.
Comment topic. Shafts and facilities development.

(B) Was the comment addressed in the Final EA? Yes ?
Where? p 4.36 This is relevant, but the problem of damage of shaftwalls due to blasting is not generally addressed.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

This affects the "shaft disturbed zone" argument. In the present circumstance disturbance can occur
(a) through blasting
(b) If at any point $\sigma_1 \rightarrow 3\sigma_3$
The latter is possible at shallow depths in the caprock & must be addressed.
The former is probable under any conventional contract conditions. It can really only be avoided on a cost plus or cost sharing type of contract or with very close supervision

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 5.4
WM Branch.
Comment topic. Shaft & facilities development.

(B) Was the comment addressed in the Final EA? YES.
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 5.5
WM Branch.
Comment topic. Retrievability.

(B) Was the comment addressed in the Final EA? NO.
Where? This may be covered by discussions on p 6-165/166, and p 6.163/164 but there is limited discussion of salt fill behavior. This is a major factor & whilst

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

it is postponed by p 5.46 final para, this is a very open statement & not one which is included in Table 4.1. It does not seem to satisfactorily covered.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP, NNWSI, SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.9
WM Branch.
Comment topic. Evaluation process.

(B) Was the comment addressed in the Final EA? NO – presumably
Where? because there is an assumption that it is covered by new section 4.42–4.44. This is not satisfactory

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

Arguably a shaft in rocksalt (like a dam cut-off sealed with soluble silicate grouts) can never be sealed. In this case there will always be a head of ~250m of water above a seal which can never be watertight (condensation on salt surface, cement bleed, – anything will affect this)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.10
WM Branch.
Comment topic. Statement of qualifying conditions

(B) Was the comment addressed in the Final EA? Yes
Where? p 685 para 3 p 6.86/87

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information?
4) Other? (Specify) It is difficult to quarrel with the analysis as it now stands

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

(D) Was our comment addressed as we suggested? Yes.
If not, specify.

(E) Status.
1) Has the basis for our concern changed? ✓
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.14
WM Branch.
Comment topic. Analysis of favorable conditions

(B) Was the comment addressed in the Final EA? Yes.
Where? p 6.88 Evaluation paras 1-4
p 3.94
p 6-244 6246

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

(D) Was our comment addressed as we suggested? YES
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP, NNWSI, SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.21
WM Branch.
Comment topic. Statement of qualifying condition

(B) Was the comment addressed in the Final EA? Unsatisfactorily
Where? 6-100/101 Tables 3.8, 3.9.

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

This is unsatisfactory. There are errors in Table 6.9 (Test conditions are not discussed in source 3) and 3.6 (shear strength units W/m °C!) Why is the only relevant bit of data on Richton dome salt (unconfined compressive strength) missing. Is it because its too low @ 13 MN/m² ? If this value is correct (Table 3.8) as it stands, nothing will stand underground at 2,000 ft.

(D) Was our comment addressed as we suggested?
If not, specify.

Small point Table 3-8 footnotes
* MPa is a unit of fluid pressure
* MN/m², kN/m² are units of stress
It is not possible, mechanically, to have a deviatoric pressure!

(E) Status.
1) Has the basis for our concern changed? No.
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.22
WM Branch.
Comment topic. Rock characteristics

(B) Was the comment addressed in the Final EA? Yes
Where? Table 6-9

Note. Why is a range of strength given for caprock (anhydrite - an evaporite and also a creeping rock) and not for salt stock in this table?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.23.
WM Branch.
Comment topic. Analysis of favorable conditions

(B) Was the comment addressed in the Final EA? Yes
Where? p 6.102 and indirectly p 6.103.
All comments are indirect.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify) Not properly addressed. There is verbiage, but no data. The core must have been logged for RQD. However unsatisfactory this information would give information on the likely behavior of underground excavations constructed in the core rock and of rock mass behavior. This data should be included along with an engineering geological log.

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.24
WM Branch.
Comment topic. Statement of qualifying condition

(B) Was the comment addressed in the Final EA? Yes
Where? Footnote (e) Table 6.9 referring to Fig 3.21.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.26
WM Branch.
Comment topic. Analysis of favorable condition

(B) Was the comment addressed in the Final EA? Yep
Where? p 6.103 top line.
They have gone too far here. Either the thermal stress effects are still manageable (they are?) or they are not manageable in which case a favorable condition is not present. The effect of thermal expansion cannot just be ignored.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.27
WM Branch.
Comment topic. Analysis of favorable conditions

(B) Was the comment addressed in the Final EA? Not satisfactorily.
Where? p 6-103 paras 1, 2 additions

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify) This is a very contentious issue & is not properly addressed in the report.
Rocksalt at low confinement & moderate stress rate behaves in a brittle manner. This will happen around openings at depth. At moderate confinement it will behave in a ductile manner. There IS NO EVIDENCE that these conditions will exist in the filled openings.
There IS STRONG EVIDENCE (attached) that rocksalt has residual strength at low confinement & will resist the flow assumed.
The favorable condition is NOT present.

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. MAJOR COMMENT 1.
WM Branch.
Comment topic. FRACTURES (FAULTS, JOINTS) & ANOMALOUS ZONES.

(B) Was the comment addressed in the Final EA? YES.
Where? Fractures in caprock (p 3.23/3.26) Anomalous salt zones (p. 3.26/3.30) Spine shear zones (p 3.29) Faults in caprock (p 3.33/3.35 inc. Fig 3.15) Anomalous salt zones - mining (p 3.60/3.63) Anomalous zones - geohydrology (p.6.85) Anomalous zones - rock characteristics (p 6.102, 6.103) Anomalous zones - preclosure (p6.162) Joints etc. (p6.167)

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

New information was included, but principally from existing mines. These are mainly in the Coastal domes and there is a strong argument (assumption?) that such features are not found in Interior domes (3.60). Structures in the cap rock at Richton Dome are played down (3.26). When all else fails the blanket condition (p 6.162) that mining techniques can overcome everything is introduced.

(D) Was our comment addressed as we suggested?
If not, specify.

Probably yes. The elements of uncertainty requested are introduced and are considered in analysis, but findings are not revised. On balance, even with the special pleading introduced, the arguments are convincing.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

The arguments presented certainly justify SCP.

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. MAJOR COMMENT 5.
WM Branch.
Comment topic. EFFECTS OF HOST ROCK MASS HETEROGENEITY.

(B) Was the comment addressed in the Final EA? YES.
Where? Anomalous salt zones (p 3.26 - 3.30) Rock characteristics and anomalous salt (p 3.54 - 3.63) Geochemistry (p. 3.67/3.70)

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify) a bit irrelevant

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

Some new information on anomalous zones in rock salt was introduced but no new information on rock characteristics has been introduced. The conclusions are identical. The question of uncertainty is addressed on p 6-100 where the statement is made that evaluations are qualitative rather than analytical. p 6-102 contains the strange statement "Richton Dome samples are believed to be representative of the rock mass". Belief has no place in an engineering work!

(D) Was our comment addressed as we suggested?
If not, specify.
No. An element of uncertainty is acknowledged, but is not introduced into analysis. In particular the basis on which engineering is considered (p. 603 6.3.1.3.3) remains limited (see attached)

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

Basis of concern remains.

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. MAJOR COMMENT 7.
WM Branch.
Comment topic. SHAFT SEALING.

(B) Was the comment addressed in the Final EA? YES
Where? Shaft sinking methods and groundwater control (p 4.33 - 4.42)
Seal system (p 4.42/44) Postclosure seals (p 5.49 - 5-53)

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) ~~Other? (Specify)~~

New information for pre and post-closure seals was introduced & presented uncritically. The basis of this information seems to be: "These things have worked with varying degrees (unspecified) of success in dome salt mines. Therefore they will work for the proposed repository." At the very least causes of some seal failures should have considered.

(D) Was our comment addressed as we suggested?
If not, specify.
Conclusion (p. 6.244) remains exactly the same. Analysis remains exactly the same (p 6.169/170)

(E) Status.
1) Has the basis for our concern changed? No.
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.7.
WM Branch.
Comment topic. Salt dome development and geometry.

(B) Was the comment addressed in the Final EA? Yes.
Where? p 3.27-3.29.

(C) How was our Draft EA comment addressed?
✓1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
✓2) Same conclusions?
3) Other? (Specify)

The conclusion p 3.30 is that the anomaly if it exists will be of little consequence to mining operations. The consequence for lateral flexibility is considered in p 6.102 and not thought significant.

(D) Was our comment addressed as we suggested? Yes - although all evidence is indirect and in the case of Richton Dome incomplete (see Table 3.3).
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP? — Yes.
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.8
WM Branch.
Comment topic. Dissolution

(B) Was the comment addressed in the Final EA? Yes.
Where? p. 3.43 - Geomorphic evidence of dissolution.

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested? Yes.
If not, specify.

(E) Status.
1) Has the basis for our concern changed? Yes.
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.10
WM Branch.
Comment topic. Geomechanical properties of caprock & salt.

(B) Was the comment addressed in the Final EA? NO. On p 3.54 para 7.
Where? It is claimed that data for rock mass mechanical properties is provided. Although promised, the possible influence of discontinuities is not addressed. Bearing in mind the inadequacy of the data, the minimum requirement is that the core be logged to give RQD values. This is such basic information that its absence is unbelievable. A geotechnical or engineering geological log of MRIG 9 is essential. Include point load index as well as RQD to give an indication of mass behavior & possibly produce RMR or Q values.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.11.
WM Branch.
Comment topic. Geomechanical properties of caprock salt.

(B) Was the comment addressed in the Final EA? NO Table 3.8 replaces 3.7
Where? and includes two data at 100°C, 200°C. There are 2 paragraphs (p 3.54 bm/3.58 top) which discuss the effects of temperature in a general way. There is plenty of specific information available which can be included.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

Fig 3.19 is an unsatisfactory compromise.

eg. Eyring type equation (Jaeger & Cook)

$$\left(\frac{d\varepsilon}{dt}\right)_T = \left(\frac{d\varepsilon}{dt}\right)_0 \exp\left[-E/RT\right]$$

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.12
WM Branch.
Comment topic. Geomechanical properties of caprock & salt.

(B) Was the comment addressed in the Final EA? YES
Where? p 3.58 p 6.103 6.163 There is a problem – the whole discussion of creep, which was tenuous in the first place has been limited further. The best discussion is on p 6.103 – this would be more apposite in chapter 3 with extrapolation used to predict room deformations.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.13
WM Branch.
Comment topic. Geomechanical properties of caprock & salt

(B) Was the comment addressed in the Final EA? Yes. Both the comment
Where? (particularly) and the rewritten paragraph (p. 3.60) in the FEA are confusing. There are some mechanical reasons for arguing that the vertical stress is equal to overburden pressure. There are less sound reasons for any material capable of shearing for stating that stress is isotropic. This is concluded very strongly on p 6.104 2nd para.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

This whole paragraph is contentious – under some circumstances (despite what Lorenz may say) salt will act in a brittle manner & stay brittle. Thus a fractured opening (originally unconfined) will probably always maintain fractures in part of its volume after backfilling.

(D) Was our comment addressed as we suggested?
If not, specify. Yes, but maybe the comment was incomplete

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed? — possibly
4) Other? (Specify)

* Note. The data quoted from Hoek & Brown p. 98 is irrelevant. It is for a depth of 270m in the Winnfield LA Dome & the stress gradient is .020 MPa/m.

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 3.14
WM Branch.
Comment topic. Geomechanical properties of caprock & salt.

(B) Was the comment addressed in the Final EA? NO.
Where?
This omission may be critical. The reason is that the performance of seals & seal installation during shaft sinking in the caprock & at the caprock/salt boundary can depend (see p 4-42) on pressurization. If there is an in-situ stress tensor with $\sigma_1/\sigma_3 \rightarrow 3$ then pressurization without fracture is impossible. Hydrofracture associated with seal installation in non-isotropic stress field in rock salt was blame for a Avery Island seal failure. In the cap rock and edge rocks there will almost certainly be large deviatoric stress distributions. Unless these are addressed — even rhetorically — there is potential for disaster.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

TIME DEPENDENT STRENGTH REDUCTION OF ROCK SALT

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ABSTRACT

Time dependency in evaporites is commonly characterised, from the results of maintained load laboratory tests, as an empirical or phenomenological relation between strain and functions of time, deviatoric stress and temperature. An alternative approach, based on conventional triaxial testing techniques, using varying constant strain rates or programmed strain rate histories is suggested. It is argued that this approach more closely simulates the mechanics of rock deformation around an opening in a continuum, where a complex relation between stress and strain rate determines time dependent deformation and stress distribution.

INTRODUCTION

Time dependent effects are observed in the laboratory testing of most rocks. These vary with the rock material, confining and differential stresses and temperature. They may be characterised through laboratory experiments as:

a) Time dependent strain or "creep" under constant differential stress and temperature conditions.

b) Axial stress relaxation to maintain a constant specimen length under constant confining pressure and temperature conditions.

c) Relations between axial and confining stress and strain during triaxial testing at varying strain rates.

In rocks - such as evaporites - which demonstrate strong time dependency, it is common to describe this through empirical or phenomenological equations based on time dependent strain or stress relaxation. Jaeger and Cook (1968) give examples of the former; Fine et al (1979) give an example of the latter. It is relatively uncommon to describe time dependency on the basis of constant strain

rate tests, although the strain rate sensitivity of rock strength (see for instance Patterson, 1978) has long been recognised through recommended standard strain rates for rock testing. Since strain controlled triaxial tests are the most satisfactory way of demonstrating the mechanics of rock fracture and deformation there would be advantages if the results of such tests could be extended to provide a satisfactory description of time dependency.

CONSTANT STRAIN RATE TESTS

Triaxial tests on 75mm diameter x 150mm long rock salt specimens at a constant strain rate of $2.1 \times 10^{-5} sec^{-1}$ have been described by Price and Farmer (1980, 1981). These were carried out at a range of confining pressures between 0 and 42 MPa, using a cell designed to allow a high degree of axial and lateral deformation. Specimen dilation was measured from the volume of oil displaced through a relief valve preset at the cell confining pressure, and corrected for ram displacement and oil compressibility. The results are summarised as deviatoric stress - axial strain curves in Figure 1 and volumetric strain curves in Figure 2. They show a progressive overall change from strain softening

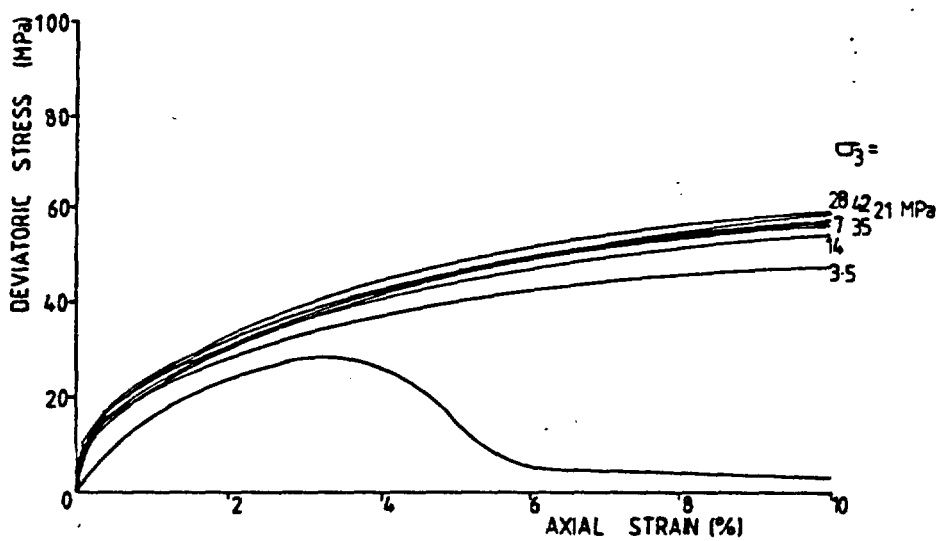

Figure 1 - Deviatoric stress - axial strain curves for Cheshire rock salt from Winsford Mine, Zone F, tested in triaxial compression at a constant strain rate of $2.1 \times 10^{-5} sec^{-1}$. (after Price and Farmer 1981).

behaviour at zero confining pressure to strain hardening behaviour at a confining pressure of 3.5 MPa and higher. Fractures are accompanied by strongly dilatant deformation in the former case and by mildly dilatant behaviour in the latter case, with dilation decreasing with increasing confining pressure. The transition from brittle to ductile

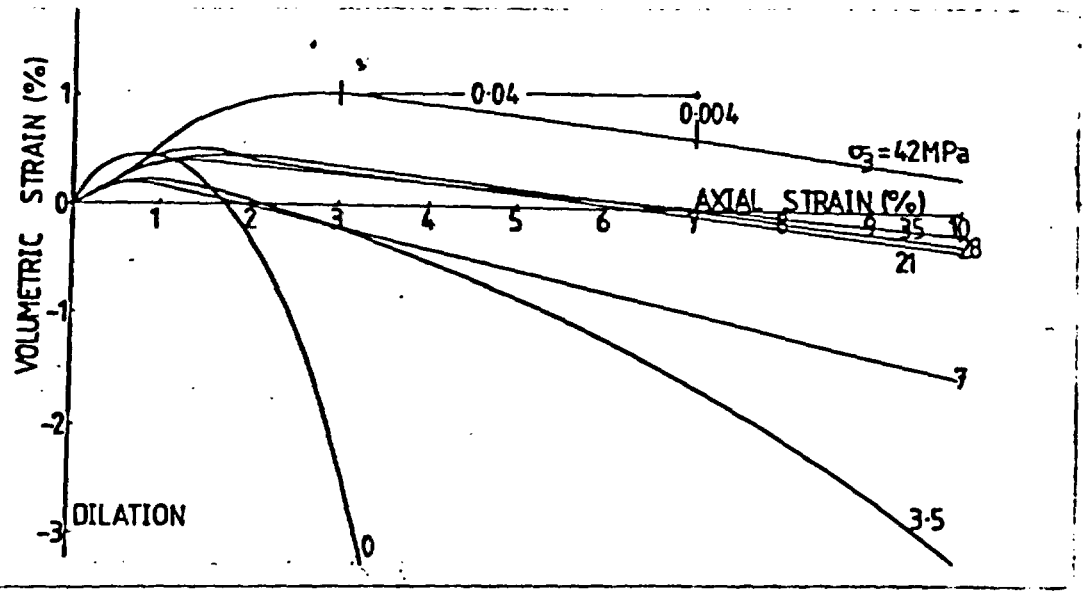

Figure 2 - Volumetric strain - axial strain curves from the test series in Figure 1.

behaviour is illustrated in Figure 3 by plotting the stress invariants $q(= \sigma_1 - \sigma_3 = 3/2\ \tau$ oct. where $\sigma_2 = \sigma_3)$, $p'(=1/3(\sigma_1 + 2\sigma_3) = \sigma$ oct. where $\sigma_2 = \sigma_3)$ at an arbitrarily selected 2% strain.

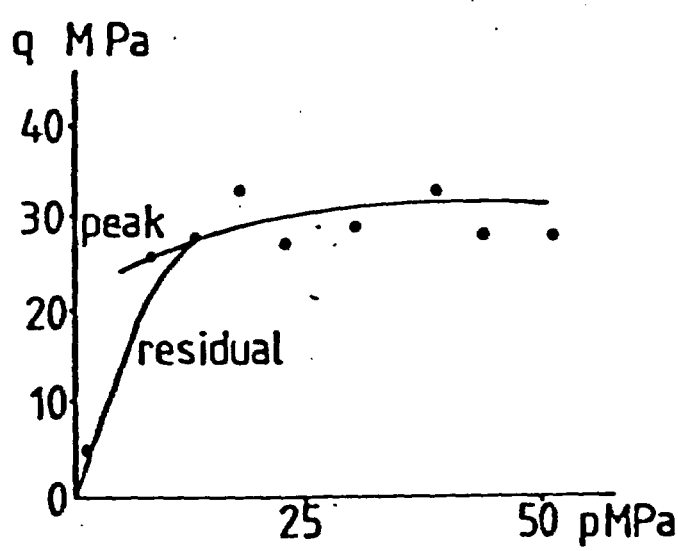

Figure 3 - Peak and residual strength envelopes for rock salt at 2% axial strain from the test results in Figure 1.

It is interesting to note that this general pattern of behaviour can be simulated in most rocks provided that the confining pressures and deviator stresses are high enough. It can form the basis for a

mechanical description of rock behaviour similar to the critical state approach to soil deformation (Schofield & Wroth 1968, Atkinson & Bransby 1978). In the critical state model for soils, the critical state line is defined as the single and unique line of failure points for both drained and undrained conditions. Failure is defined as the state at which large shear distortions occur with no change in stress or in specific volume. The complete critical state surface in q p' V_s space is defined by three state boundaries (Figure 4); the tension failure surface, Hvorslev surface and the Roscoe surface. The Roscoe surface, applies to highly compressible materials and has no engineering significance in rocks. The tension failure surface represents brittle fracture. The Hvorslev surface is the state boundary for over-consolidated materials.

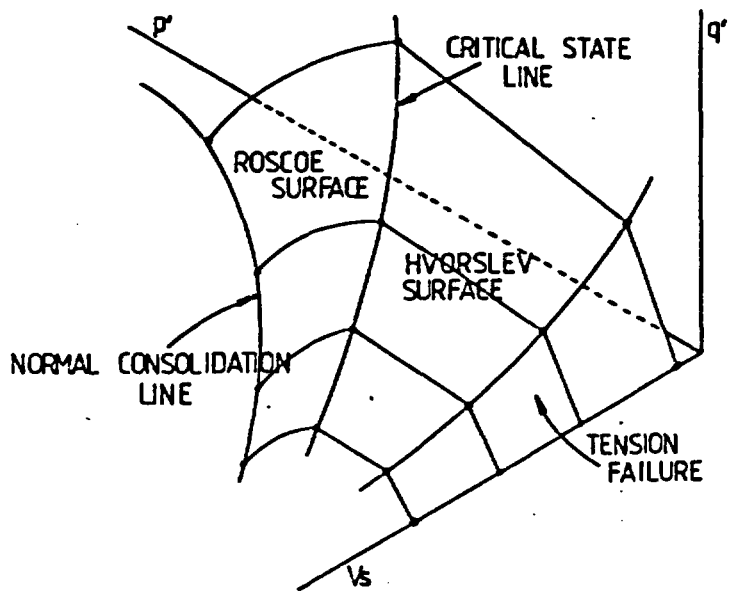

Figure 4 - The complete state boundary surface in q, p', V_s space (after Atkinson and Bransby, 1978). V_s is the specific volume.

The concept of stability is important in any consideration of specimen deformation in terms of the critical state model, and in particular of the Hvorslev surface. In terms of specimen behaviour two specific concepts of stability arise:

(a) Stability of the microstructure, which in most rocks involves microfractures, although in rock salt some deformations associated typically with crystal lattice imperfections may occur. In either case deformation is stable provided it is caused at a microstructural level by a small increment of stress increase leading to a new state of equilibrium

(b) Structural stability of the specimen, which exists if at a particular point in its stress-strain history, further deformation requires input of further energy from the test

system into the specimen.

In practice the second condition is restricted to strain-hardening materials. The change from structural instability to structural stability occurs at the brittle-ductile transition. Here the confining pressure works to suppress spontaneous microstructural deformation, or microcrack propagation, and the differential stress induces homogeneous deformation in the specimen. This is represented in Figure 3 by the junction between the peak and residual strength envelopes, and in Figure 1 by the change from strain softening behaviour.

In soil mechanics, the Hvorslev surface can be represented (Atkinson and Bransby, 1978) in a similar form to the stress-dilatancy equations of Taylor (1948) and Rowe (1962):

$$q/p' = M - \frac{\delta\varepsilon v}{\delta\varepsilon s} \qquad \ldots. (1)$$

where M is a frictional constant and εv and εs are respectively volumetric and shear strains, having different signs.

Evidence for this type of deformation in rock salt can be obtained by plotting the data from Figure 1 on q/p', $\frac{\delta\varepsilon v}{\delta\varepsilon s}$ axes in Figure 5.

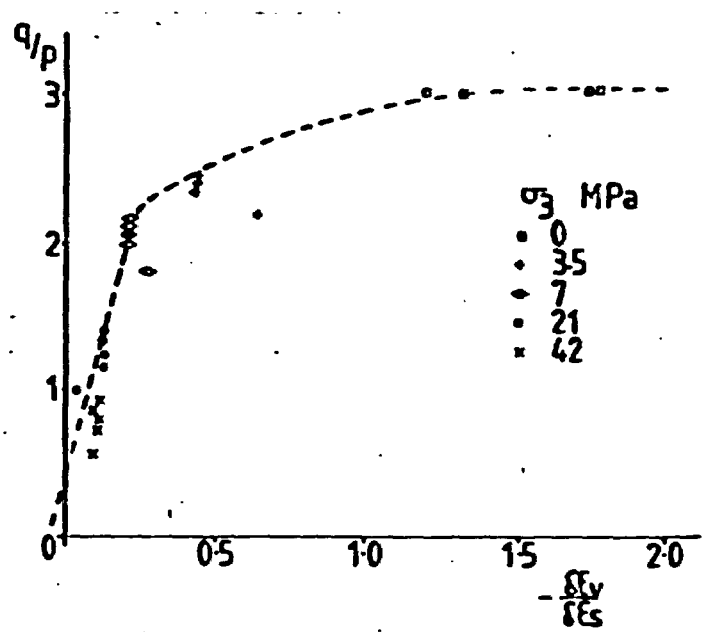

Figure 5 - Representation of the Hvorslev surface, based on the test results from Figures 1 and 2 (after Price and Farmer 1981).

Examples for other rocks are given in Price and Farmer (1981). Figure 5 indicates an equation of the form:

$$q/p' = M-K \frac{\delta\varepsilon v}{\delta\varepsilon s} \qquad \ldots. (2)$$

where K has a magnitude of about 8. The form of equation 1 assumes that all the work transferred to a unit volume of the specimen during stable, dilatant deformation is expressed as plastic frictional flow. Equation 2 implies, as already suggested by Gerogiannopoulos and Brown (1978) that other yield and deformation mechanisms are involved in rock salt deformation. For true frictional flow to occur in rocks, complete cataclasis in a homogeneously deformed specimen (as illustrated by Scholz, 1968) must occur. In rock salt the deformation process is more complex, with a probability that part of the deformation results from irregular intracrystalline movements which do not result in structural breakdown. This may explain the marked time-dependency of rock salt under some loading conditions. Nevertheless the concept of frictional resistance in rock salt - particularly at lower confining pressures as evidenced by the change in slope to near unity in Figure 5 - is strongly validated by the results.

EFFECT OF STRAIN RATE ON STRENGTH

In order to examine the degree of time dependency through the effect of strain rate on the strength envelope of the material illustrated in Figure 3, the tests were repeated - without volumetric measurements - at strain rates of $5 \times 10^{-3}sec^{-1}$, $2 \times 10^{-4}sec^{-1}$ and $2 \times 10^{-7}sec^{-1}$. The results are shown in Figures 6 - 8 as axial stress-strain curves and are summarised in Figures 9 - 11 where the relation between q and p' is plotted at axial strains of 2, 4 and 10%. Each of the curves is similar in form, comprising an origin in the brittle field, followed by an extended transition to ductile flow, reaching a critical state at q/p' values between 0.74 and 0.9. The effect of increased strain rate is to make the material appear more brittle, a feature which may be deduced from the state of the tested samples (Figures 12, 13) and from the extended transition zones at lower strain rates.

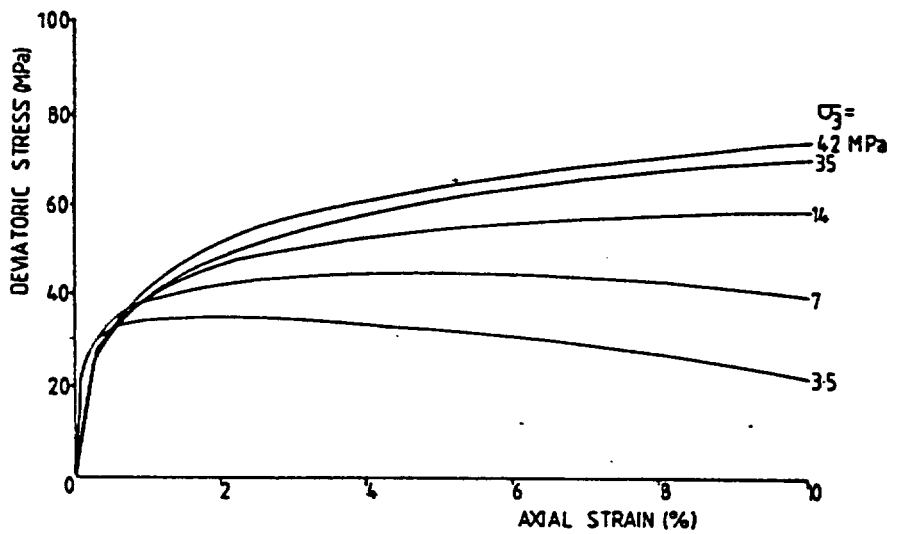

Figure 6

Deviatoric stress - axial strain curves for rock salt as in Figure 1 tested at a constant strain rate of 5×10^{-3} sec^{-1}

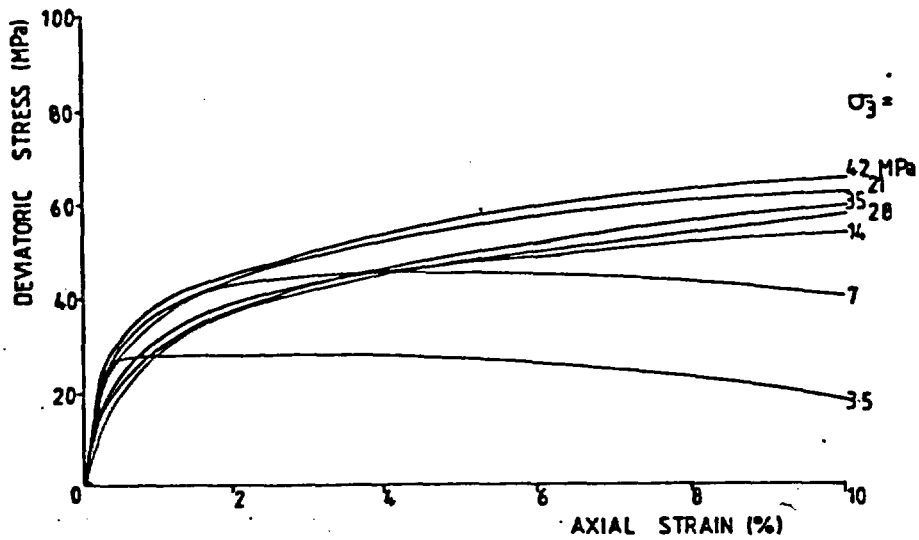

Figure 7 - Deviatoric stress - axial strain curves for rock salt as in Figure 1 tested at a constant strain rate of $2 \times 10^{-4} sec^{-1}$

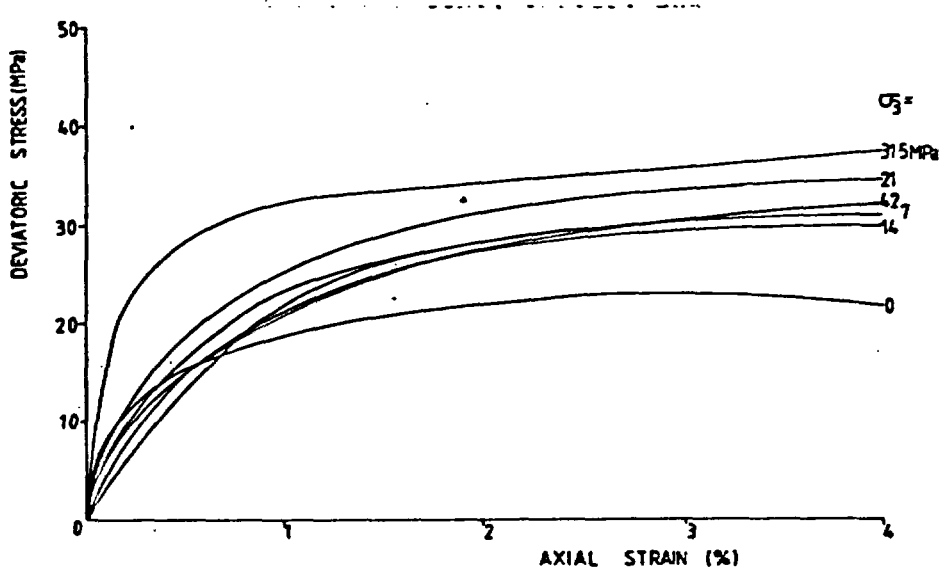

Figure 8 - Deviatoric stress - axial strain curves for rock salt as in Figure 1 tested at a constant strain rate of $2 \times 10^{-7} sec^{-1}$.

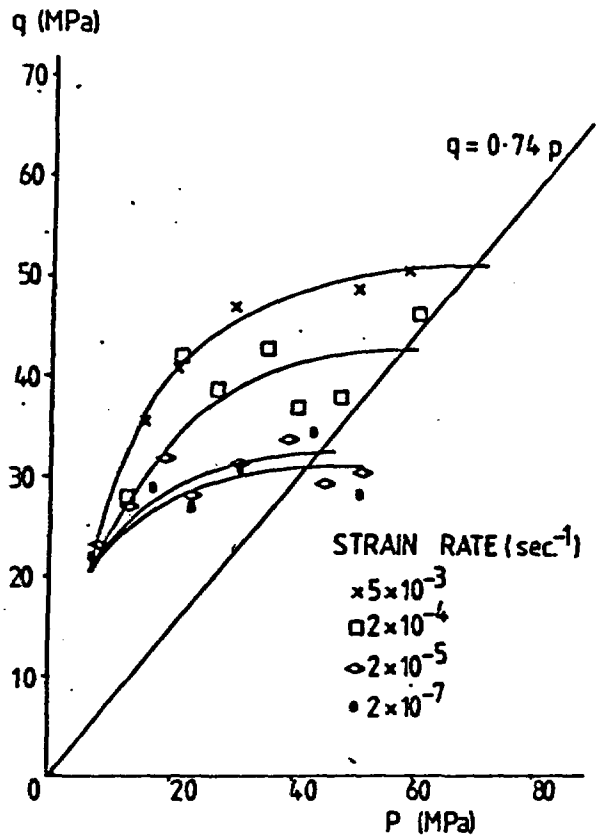

Figure 9 - Strength envelopes at 2% axial strain from the test results in Figures 1 and 6 - 8.

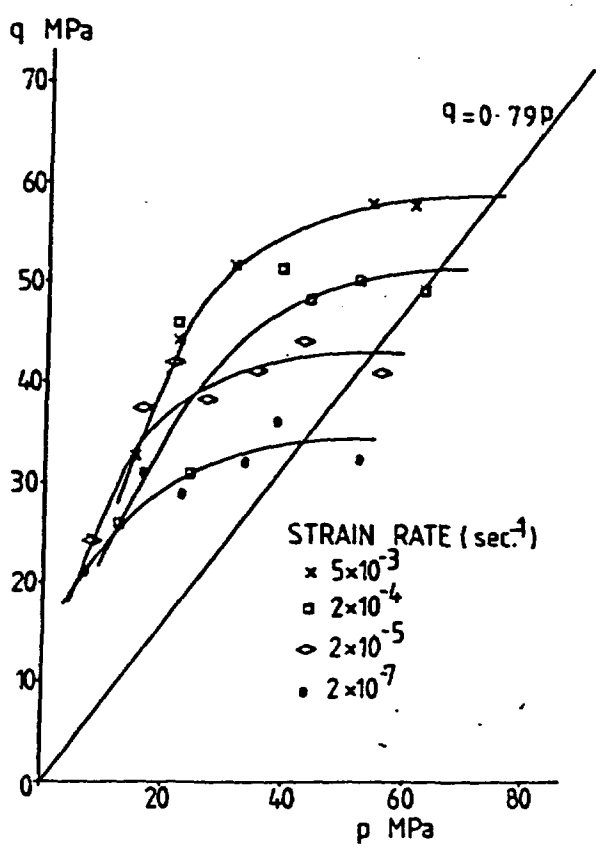

Figure 10 - Strength envelopes at 4% axial strain from the test results in Figures 1 and 6 - 8.

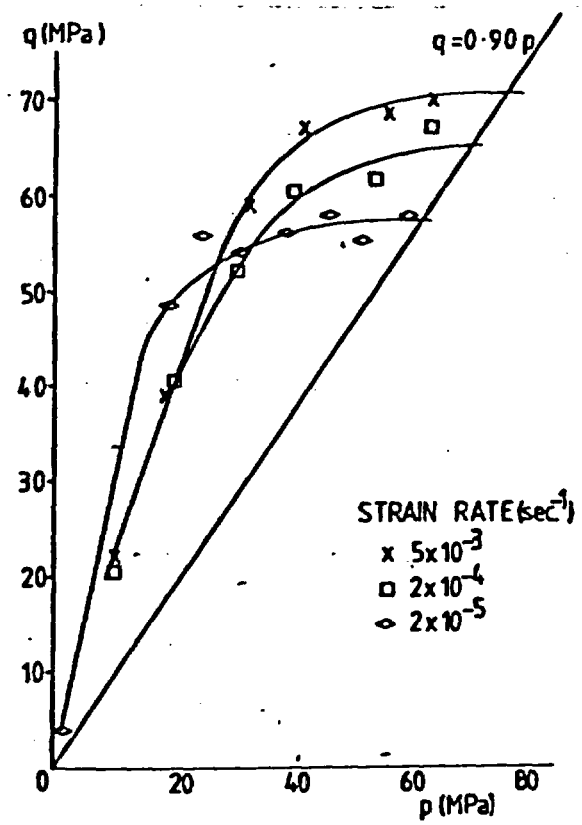

Figure 11 - Strength envelopes at 10% axial strain from the test results in Figures 1 and 6 - 8.

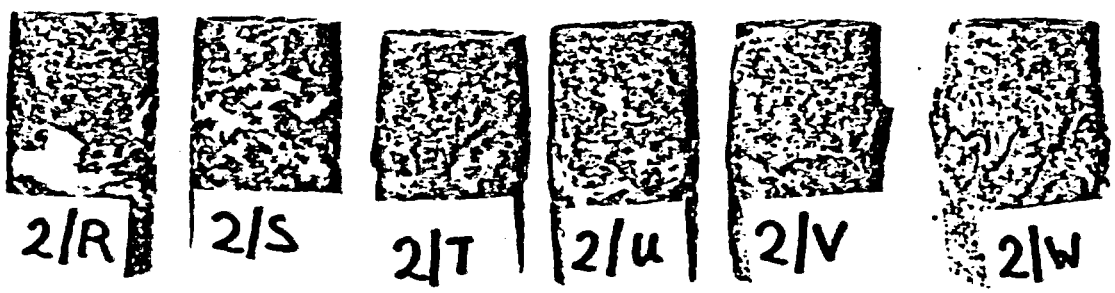

Figure 12 - Specimens of rock salt after testing at a constant strain rate of 5 x 10^{-3} sec^{-1}. Confining pressures were: 2R - 42 MPA, 2S - 35 MPa, 2T - 21 MPa, 2U - 14 MPa, 2V - 7 MPa, 2W - 3.5 MPa.

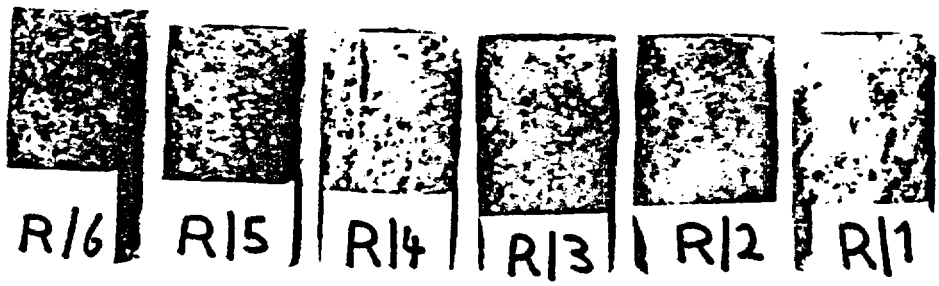

Figure 13 - Specimens of rock salt after testing at a constant strain rate of 2 x 10^{-5}sec^{-1}. Confining pressures were: R6 - 42 MPa, R5- 35 MPa, R4 - 28 MPa, R3 - 21 MPa, R2 - 14 MPa, R1 - 7 MPa.

If the deviatoric stress intercepts of the q,p' curves in Figures 9 - 11 with the critical state line are plotted against the logarith of strain rate as in Figure 14, the effect of strain rate on the strength of rock salt can be clearly demonstrated. This shows a linear relation, between q and $\dot{\varepsilon}$ in semi-logarithmic space.

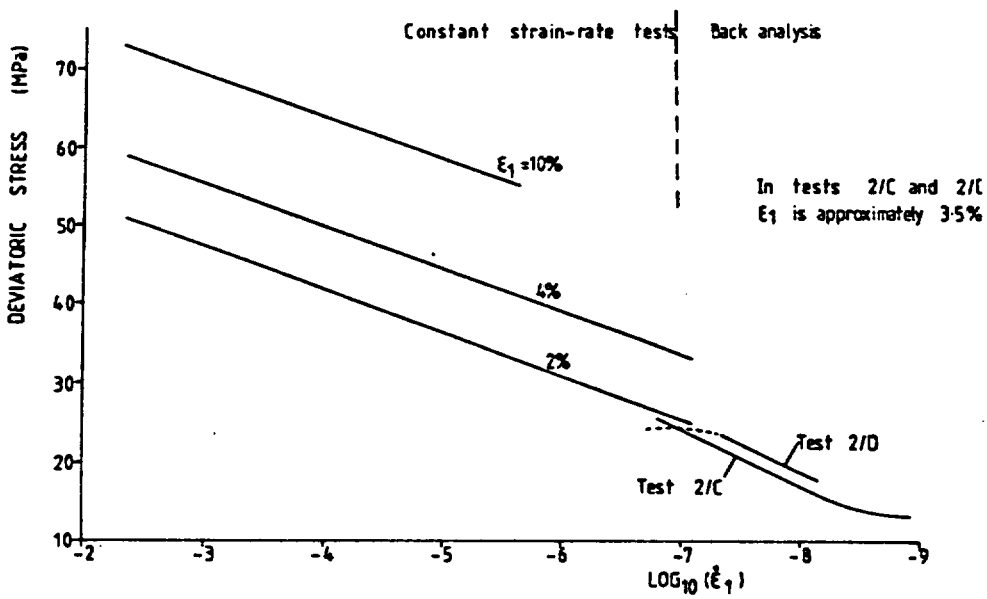

Figure 14 - Plot of the deviatoric stress intercept with the critical state line in Figures 9 - 11 against the strain rate for the series of constant strain tests in Figures 1 and 6 - 8. This is supplemented by data obtained from back analysis of programmed variable strain rate tests in Figure 19.

Since a relation exists between p' and q and q and $\dot{\varepsilon}$, it is possible to postulate a relation between the three variables in the form illustrated in Figure 15. This is only valid for deformation beyond the brittle-ductile transition. In less highly confined rocks, brittle phenomena will become more pronounced and affect both p', q and q, $\dot{\varepsilon}$ relations. If, however, stresses around an opening redistribute in a partly frictional manner (q = Mp') then it should be possible to predict the strain rate in an element confined to any stated confining pressure.

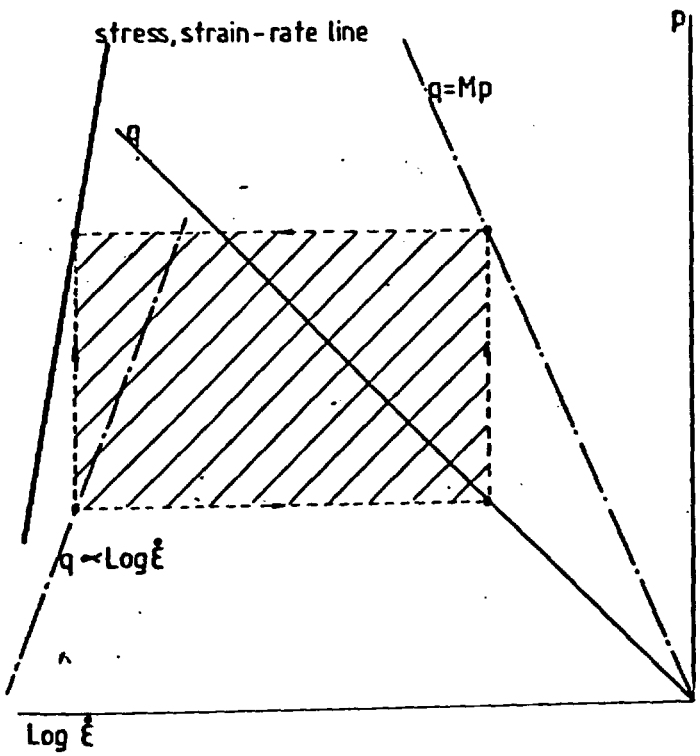

Figure 15.

Postulated stress-strain rate line in q, p', log $\dot{\varepsilon}$ space.

SIMULATION OF STRAIN RATE HISTORY AROUND AN OPENING

In-situ measurements to illustrate this hyposthesis are not readily available. The most detailed measurement programme is that obtained by Cook (1974) who measured, using an anchor extensometer system, deformations at depths of 0.6m, 1.5m, 3.0m and 4.5m from the unsupported wall of 7.6m diameter shaft in the centre of a 50m bed of halite at a depth of 1056m. These measurements allowed computation of a detailed strain rate - time history for the rock between anchors. For the purposes of laboratory simulation, the average strain rate history over the first 25 days (Figure 16) between the four sets of anchors 1.5 and 3m into the shaft wall were chosen (details are given in Gilbert, 1981). The initial elastic stresses were estimated from thick cylinder theory at a depth of 2.25m into the sidewall as σ_r = 13 MPa, σ_θ = 35 MPa and $\sigma_\theta - \sigma_r$ = 22 MPa. Tests were carried out on 75mm diameter by 150mm long cylindrical rock salt specimens consolidated initially to $\sigma_1 = \sigma_3$ = 13MPa for 5 days. The axial strain was then increased, using the servo-

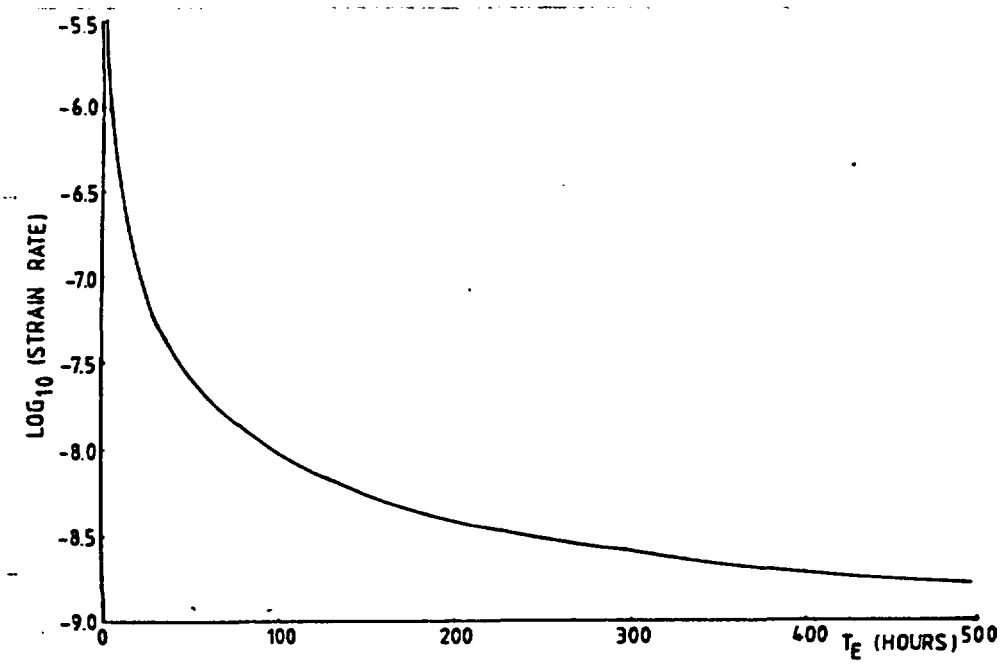

Figure 16 - Average strain rate history of a rock salt element between 1.5m and 3m into a shaft sidewall for the first 25 days after construction (from Cook, 1974).

control system, at a strain rate programmed to simulate the actual strain rate in Figure 16 starting at $\dot{\varepsilon} = 10^{-5.5}\text{sec}^{-1}$ and terminating at $\dot{\varepsilon} = 10^{-8.7}\text{sec}^{-1}$ after 486 hours in the case of Test 2C and 100 hours in Test 2D, when the testing machine developed a fault.

The results are presented in Figure 17 as stress-time and in Figure 18 as stress-strain curves. The stress-time curve can be considered in three regions:

(a) a loading curve to a peak q of 23-25 MPa after 9 - 30 hours

(b) a rapid unloading curve of about 105 hours during which the deviatoric stress drops to about 15 MPa - a drop of 40%

(c) a slow unloading curve during which the test appears to approach an equilibrium condition.

The increase in stress after 265 hours was associated with reprogramming of the testing machine.

In the stress-strain curves, both specimens have a "plateau" rather than a peak stress feature in the strain range 1.5 - 3% - considerably below any quoted strains associated with peak stress in rock salt at the confining pressure (13 MPa) used. This feature may be examined when the curves are expressed in stress-strain rate form in Figure 19.

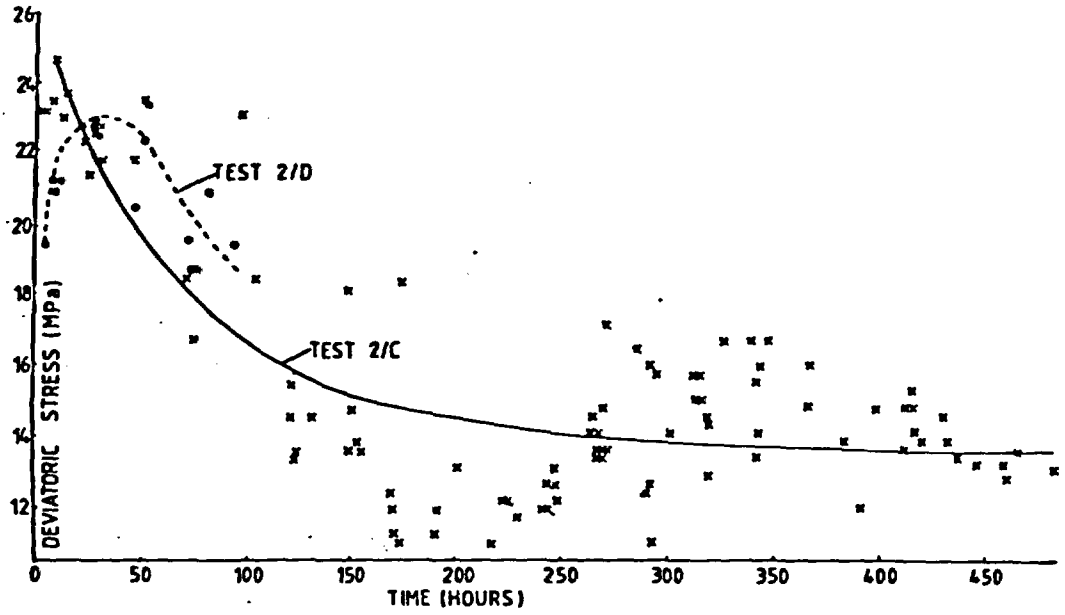

Figure 17 - Relation between deviatoric stress and time for two triaxial tests on specimens confined at 13 MPa and subjected to the axial strain rate history of Figure 16.

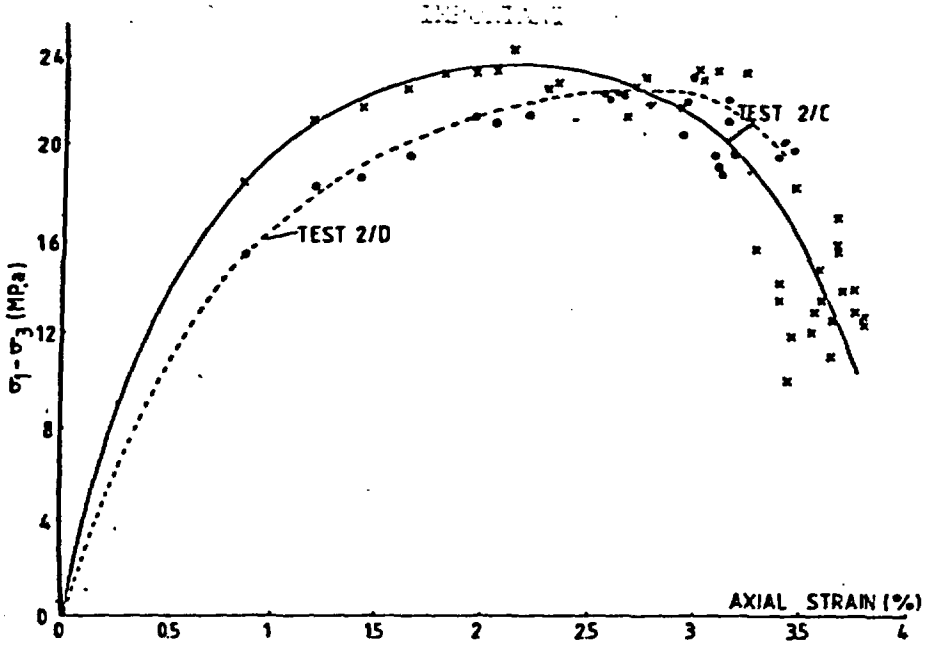

Figure 18 - Relation between deviatoric stress and time for the two triaxial tests in Figure 17.

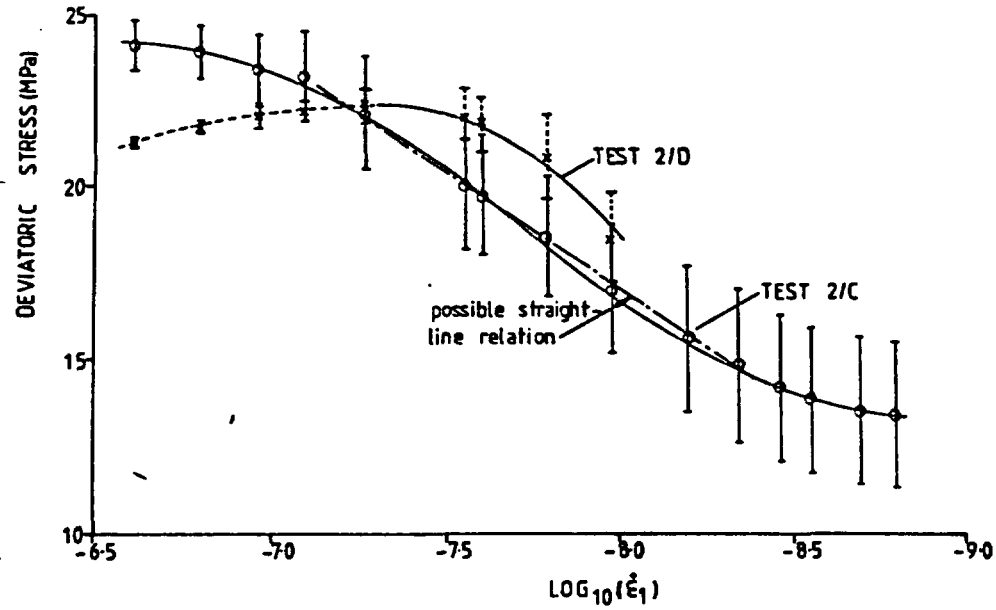

Figure 19 - Deviatoric stress - strain rate relations abstracted from the tests in Figure 17.

This indicates that in the first region - to peak stress or the end of the stress plateau - stress is dependant on strain rate and on cumulative strain, whilst in the second region, stress is primarily a function of strain rate. The relation between deviator stress and strain rate, defining a simple surface in q, $\dot{\varepsilon}$ has some similiarities with Heard's (1972) results from tests on polycrystalline salt - indicating that steady-state dislocation slip is a dominant mechanism within the deforming specimen. They can also be fitted to extend the curves in Figure 14.

The results illustrate that a particular applied strain rate under the test conditions will result in certain measured stress levels. If this approach is reversed so that the phenomena are considered in terms of the original prototype situation and the in-situ stresses involved in that situation, two important conclusions can be drawn:

(i) The movement of the rock around the shaft during the first month is associated with significant stress redistribution.

(ii) The strain rates measured in-situ appear to result from a fundamental relation between deviatoric stress and strain rate.

DISCUSSION

When describing time-dependancy in rock salt, it is usual to perform maintained load tests and to analyse the data using some model incorporating time-hardening or ageing. When constant and varying strain-rate tests are performed on rock salt specimens, a radically different view of the deformation process is obtained. There is no

evidence of hardening, and instead a relation between the stress in variants and the strain rate is indicated which is unique at the critical state line. When this approach is compared, by back analysis of data, with in-situ deformation measurements, it seems probable that in-situ deformation processes are the same as those operating in controlled strain-rate tests.

The mechanics of time-dependency in rock salt are less easily explained, and uncertainty about their exact nature has led to confusion in salt rheology. The evidence to support stress redistribution and the indications of dilation at lower confining pressures suggests a combination of brittle fracture phenomena and dislocation slip phenomena. The effects of elastic phenomena leading to initial stress redistribution should not be ignored, as they often are in conventional time-dependent models. An approach previously suggested by the authors (Gilbert 1981, Gilbert and Farmer 1981) based on Ladanyi's (1974) description of time dependant decay of strength parameters, following short-term stress redistribution may form the most satisfactory approach.

ACKNOWLEDGEMENTS

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BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.28.
WM Branch.
Comment topic. Analysis of favorable condition.

(B) Was the comment addressed in the Final EA? Yes.
Where? p 6.102 para 5
p 3.60 – 3.63

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

All the evidence is indirect. The conclusion should be deferred to SCP. The evidence at this stage is circumstancial & not conclusive.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6-29
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? Yes
Where? p 103-104.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify) There is a real problem here which has not been fully addressed. What are the depths of dome (or other) salt mines. My guess is less than 1,500ft. Below that solution mining must be more economical.

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

As depth increases, mining problems become more severe with roof outbursts a major problem at depths below 2000ft (see Gunn in 1964 Strata Control Conference). Roadway stability as pillar sizes increase is also a major problem (Serata in Stability in Mines 1983)

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

Anomalies are not as important as stress effects in weak rock. See attached for picture of potash mine roof after 2 months (Fig 7) at 2,900 ft. Potash strength 40MN/m².!

Rockbolt Support of Underground Structures – Design and Performance Studies

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University of Newcastle upon Tyne, England

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ROCKBOLT SUPPORT OF UNDERGROUND STRUCTURES—DESIGN AND PERFORMANCE STUDIES

I.W. FARMER* P.D. SHELTON*

SYNOPSIS

The design principles for underground rockbolt support systems are illustrated through three case histories where performance of rockbolts has been observed and analysed. These include (i) a shallow square section tunnel in strongly jointed Carboniferous mudstones where rockbolts were used to prevent sidewall wedge failure (ii) a medium depth circular tunnel in Carboniferous mudstones and sandstones where rockbolts successfully formed the major support system and (iii) a deep mine excavation in Permian evaporites where excessive yielding led to difficulties with rockbolts.

INTRODUCTION

The process of underground construction leads to a redistribution of stresses around the resultant excavation, and—particularly if drill and blast is used in construction—a loosening of blocks bounded by discontinuities in the excavation periphery. Where a combination of redistributed stresses and excavation geometry creates conditions for fracture of intact rock and sliding along discontinuity interfaces, then some form of support is required to prevent large displacements and ultimately collapse of peripheral rocks. Ideally for maximum efficiency and economy this support should utilise the considerable structural properties of the rock, and this is the ultimate aim of rockbolt support systems.

A pre-tensioned rockbolt applies an active force to the rock in which it is installed. If the rock comprises a newly excavated sidewall or roof of an underground opening and the rockbolts are installed in a direction normal to the exposed faces, then they will produce the following reactions:

(*a*) The rock influenced by the bolt will be placed in compression. If the bolts are closely spaced, a zone of peripheral compression can be formed around the excavation, and if the pre-stress and length of the bolts is correctly calculated, the stress distribution in the rock surrounding the opening can be restored to a level similar to that existing before excavation.

(*b*) Frictional resistance to sliding along discontinuities intersected by the bolts can be increased.

(*c*) Dilation of the rock whether through sliding along discontinuiies or yielding of intact rock will be inhibited, maintaining the shear strength of discontinuities and reducing peripheral deformation.

The resultant effect will be to minimise excavation closure and surface spalling, utilising as far as possible the inherent strength and shear resistance of the rock. Where rockbolts do not work successfully then, provided that the anchorage of the bolt, and the bolt itself, are strong enough to resist design loads, failure is invariably due to insufficient rock strength. This can result from weathering or clay infill along discontinuity interfaces, weathering of the rock itself, or stress concentrations which exceed the intact rock or discontinuity surface strength.

Design of rockbolt systems is discussed at length by Moy (1977), and some of his recommended design procedure is included in Table 1. His major recommendations are that emphasis should be placed on classification schemes for both feasibility decisions and preliminary design. There are essentially two major types of rockbolt design problem—those where the rock is of good quality and blocky and where design should concentrate on support of individual blocks likely to become detached from the excavation—a good example of this is the Dinorwic scheme (Anderson *et al* 1977). The other type is where the rock is of poor quality and the bolts are installed in an attempt to improve this overall quality by inhibiting rock dilation in order to maintain the inherent rock or discontinuity shear strength, and restrict the development of a plastic yield zone.

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TABLE 1 Rockbolt design checklist (after Moy 1977)

1. Feasibility

Determine whether rockbolt reinforcement is feasible by (*a*) simple two dimensional elastic stress analysis related to data on intact and discontinuity shear resistance or (*b*) by refrence to empirical rock classification schemes. Those of Bieniawski (1973) and Barton (1974) define respectively "Rock Mass Rating" or "Q-factor" based on rock mass, rock material and groundwater properties. They can be used to give an indication of whether the excavation will stand unsupported, whether continuous lining is required or the degree of reinforcement likely to be needed.

2. Preliminary design

Determine mode of failure whih the rockbolt system should withstand. Then

(*a*) carry out a discontinuity survey and using the methods of Moy (1977), Hoek (1978) and Goodman (1975) determine the size of blocks and wedges kinematically capable of falling or sliding from the roof and sidewalls bounded by the excavation geometry. or,

(*b*) In poor quality rock use Wilson's (1977) approach to estimate the size of plastic yield zone likely to be formed and the magnitude of reinforcement pressure to prevent excessive yield.

3. Detail design

(*a*) where the excavation is in good rock and individual blocks where movement is possible can be isolated, design individual rockbolts to restrain these.

(*b*) where the excavation is in blocky interlocking rock, Lang (1971) suggests on the basis of model tests that the ratio between bolt spacing (s) and block size (b), s/b should be between 3 and 4 and between bolt length (L) and (s), L/s should be greater than 2.

(*c*) various empirical rules for span and prestress (P_l) developed for different conditions give similar values

$L = 0.3\ B$ for excavation span B—Rabciewicz (1969)

$L = 6 + 0.004\ B^2$ dimensions in feet—Pender (1963)

$P_l = nB\gamma - c$ (roof), $P_l = mH\gamma$ (sidewall)—Cording *et al* (1971),

where γ is unit weight, c is cohesion and H excavation height. n has values from 0.1-0.25 and m, 0.05—0.13 each related to stability ratio.

The three case histories which are described in the present paper include one which fits the former design category (CASE A), one which fits the latter category (CASE C) and one which falls in between. Geotechnical and geometrical data for all three cases are summarised in Table 2.

SHALLOW TUNNEL IN CARBONIFEROUS LIMESTONE SERIES

Case A was a square section 3m high × 3.5m span main access tunnel in a fluorspar mine. The tunnel was excavated at a depth of 110 m by drlling and blasting in an interbedded shale-mudstone-sandstone unit of the Lower Carboniferous, immediately below the Four Fathom Limestone. Shortly after excavation it was found that along a 150 m length of the tunnel where the limestone-mudstone contact was 0.5 m from the roof, failure of the north-eastern tunnel sidewall was taking place by sliding of blocks of rock along joint planes. The limestone roof and the south-western tunnel wall were stable and unsupported. There were no signs of yield in the relatively strong roof and sidewall rocks.

TABLE 2 Excavation geometry and rock properties

	CASE A	CASE B	CASE C
Depth (m)	110	280	1100
Dimensions (m)	3×4.5 (span)	3.5 diameter	4×6 (span)
Method of excavation	Drill and Blast	Tunnel machine	Drill and Blast
Roof Rock	Limestone	Mudstone	Anhydritic shale
Strength (kN/m²)	80,000	35,000	12,000
Sidewall rock	Shale-sandstone layers	Silty Mudstone	Potash
Strength kN/m²	40,000	35,000	40,000
R.Q.D.%	95	40	—
Rock mass rating	53	33	—
Q—factor	2.6	0.29	—

A scan line survey was carried out along a 50 m length of the north-east wall at a height of 1.5 m above the floor. Stereographic projections of poles to the measured discontinuities, with the exception of bedding planes, have been plotted and contoured in Figure 1. The azimuth bearing of the horizontal tunnel centre-line was 305°. There were four major discontinuity sets with properties as follows:

*S*1	79°/150°	Average length 2 m spacing 1 to 2 m. Moderately rough, clean surfaces Not present in limestone
*S*2	80°/216° to 85°/036° predominantly vertical	Average length 2 m frequently continuous. Spacing 1m in mudstone, 4 m in limestone. Wavy surface, open 3 mm, occasionally calcite or clay filled.
*S*3	85°/258° to 85°/078° Predominantly vertical	Similar to *S*2, but average length 1m in roof
*S*4	15°/292° Bedding	1 m spacing. Wet and clay filled. Undulating

The average ϕ value of the surfaces was 37° and the unit weight of the intact rock 25.7kN/m³.

The excavation geometry and discontinuity planes are also plotted as stereographic projections in Figure 1. This suggests four possible modes of failure:

(*a*) Wedge failure of blocks bounded by *S1* and *S3* and released at the roof by *S4* can occur in the direction *F1* in the north-east wall.

(*b*) Plane failure of blocks along the parts of *S2* included at 80°/216° in the direction *F2* in the north-east wall.

(*c*) Plane failure of blocks along the parts of *S2* inclined at 80°/036° in the direction *F3* in the south-west wall.

(*d*) Vertical release of blocks bounded by *S1*, *S3* and *S4* in the roof.

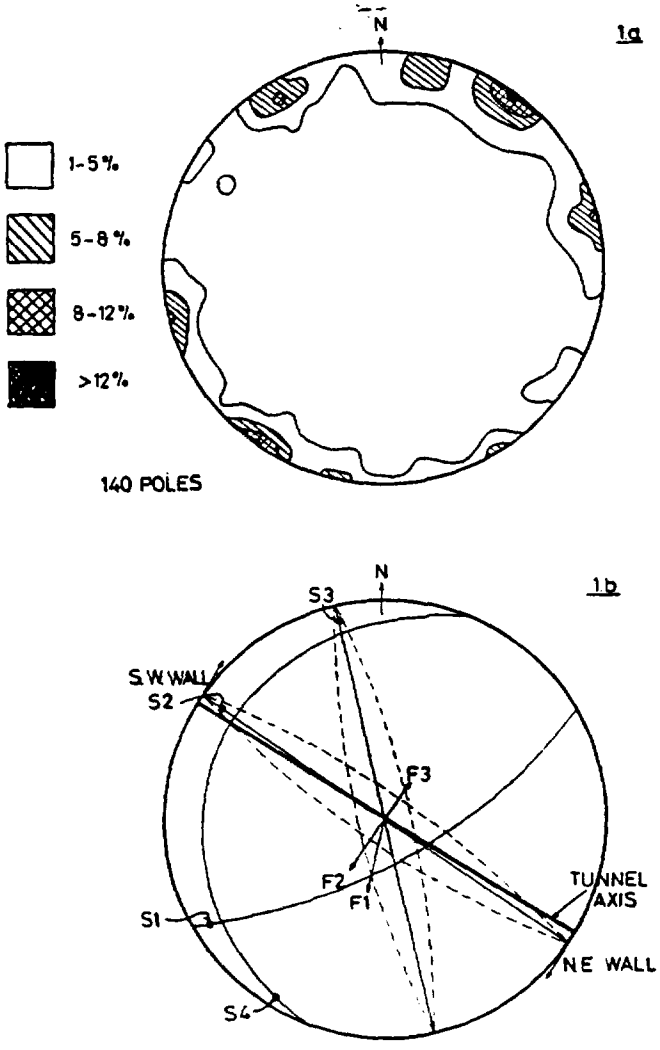

FIGURE 1 Countoured lower hemisphere projection of poles to discontinuities and stereographic projection of main discontinuity planes—CASE *A*

In fact failure occurred only in the first two modes and principally as wedge failure. In the limestone roof failure was inhibited by the absence of *S1* and the lack of continuity of *S3*. In the south-west wall, although parts of *S2* were apparently daylighting in the wall, the probable reason for stability was that the effective dip was nearer 80°/216°. The bias in the collected data was probably due to the waviness of the set. and the selected scan-line height, which was not fully compensated by circumferential scan-lines.

Stability analysis of the largest wedge bounded by *S1* and *S3* and largest block on *S2* projected onto the sidewall geometry indicated a required rockbolt pre-tension force of 7.5 kN for (*a*) and 18.2 kN for (*b*). Installation of 1.5m long resin-bonded bolts (see Figure 2) successfully stabilised the sidewall.

FIGURE 2 Rockbolted tunnel sidewall—CASE *A*

CIRCULAR TUNNEL IN MUDSTONE

Case B was a 200 m section of a total length of 32 km of 3.5 m diameter aqueduct tunnel, at a depth of 280 m in the section under consideration. It was driven by a full-face disc cutter tunnelling machine in Carboniferous Limestone strata, comprising in the section, moderately fissile silty mudstones and interbedded siltstone and mudstones. The tunnel was supported throughout by resin-bonded rockbolts at variable spacings depending on rock properties, but usually at about 1m centres in the roof quadrant of the tunnel. The installation was apparently highly successful, and

a series of instrumented bolts were installed in 3 locations to examine force build up and distribution in the rockbolts.

The instrumented bolts can be seen in Figure 3. They were placed in the crown and shoulders of the tunnel in between the permanent bolts. The instrumentation comprised four stainless steel wires anchored at intervals of 0.38 m from the base of 25 mm diameter hollow section bolt, 1.5 m long. Relative extension between the wires and the unstressed top of the anchor were measured using a clip-on extensometer. The 32 m diameter holes were resin filled to just below the upper wire anchorage. Laboratory calibration of the bolts showed they were sensitive to $\pm$ 1 k N of load.

A joint survey of the instrumented sections plotted as a contoured stereographic projection of poles to planes in Figure 4 shows the existence of numerous near vertical joint sets, the most marked of which, *S 1* and *S 3*, are closely spaced (0.1 m), largely non-continuous and are nearly normal to and parallel to the tunnel axis which has a direction due north. There is therefore a tendency for sliding of shoulder blocks into the tunnel as shown in Figure 3.

Typical examples of axial load computed from extension reading along central and shoulder rockbolts after installation are shown in Figure 5. In the case of the central bolt the major part of the bolt load is concentrated over the first part of the bolt and reduces towards the buried end. The peak load of 80 kN builds up rapidly and remains constant over 100 days reading. The ultimate stress pattern is similar to that demonstrated by Farmer (1975) for a fully bonded bolt. In the case of the shoulder bolt there is little load at the tunnel periphery and a rapid rise towards the buried end of the bolt. This is consistent with the observed tendency to detach and the position of detachment of the shoulder blocks.

Using Wilson's (1977) adaptation of the thick cylinder plastic yield zone analysis, the annular yield zone around the unsupported tunnel was estimated at 7 m wide. Deformation of the carcass was minimal at about 4 mm. The indications are that the bonded rocbolts have successfully resisted dilation of the yielding rock and have successfully preserved the inherent shear strength of the discontinuities in the rock mass.

DEEP EXCAVATION IN PERMIAN EVAPORITES

Case C was a 4 m high by 6 m wide mine roadway at a depth of 1100 m in a potash mine. The experimental work was carried out and the case described by Wiggett (1976) and by Dunham (1975). The excavation was formed by drilling and blasting in a sylvinite layer of

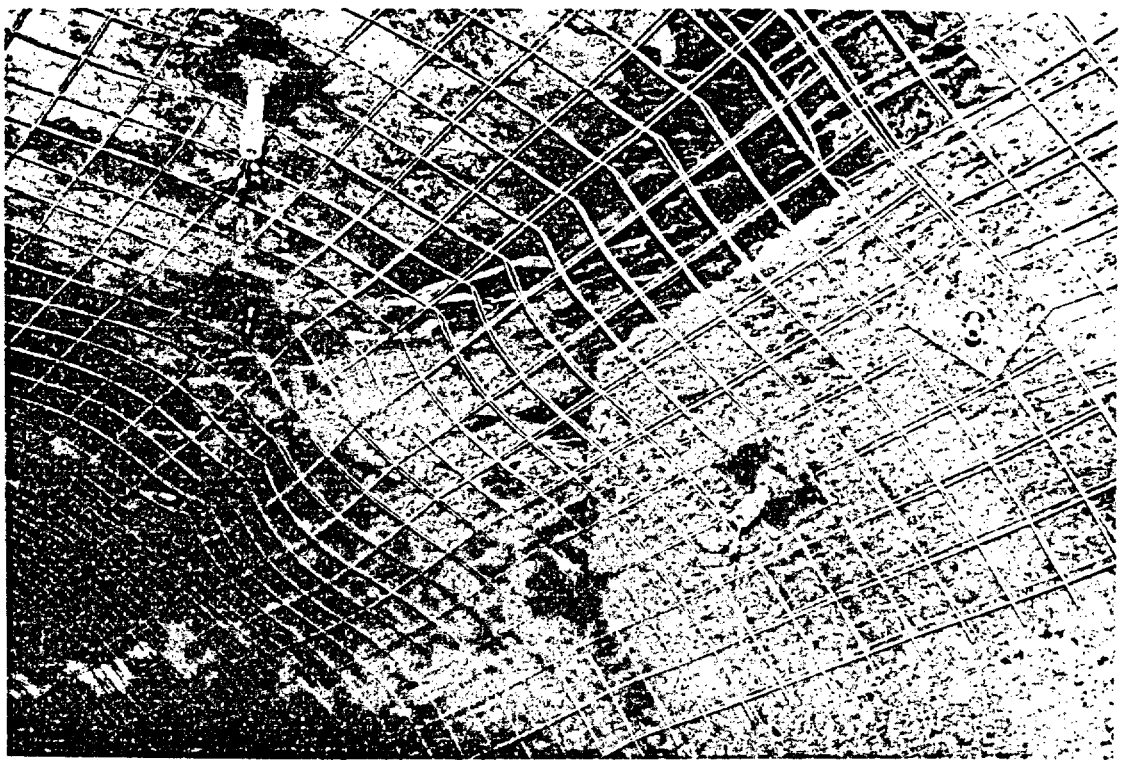

FIGURE 3 Instrumented rockbolts in a mudstone tunnel roof—CASE *B*

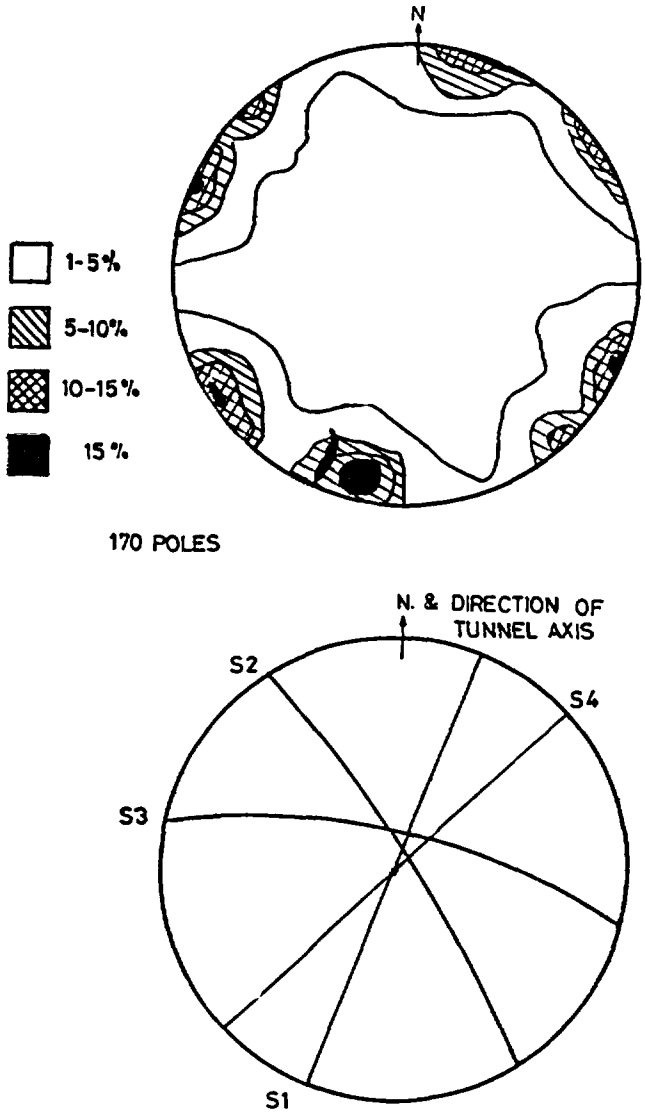

FIGURE 4 Contoured lower hemisphere projection of poles to discontinuities and stereographic projection of main discontinuity planes—CASE *B*

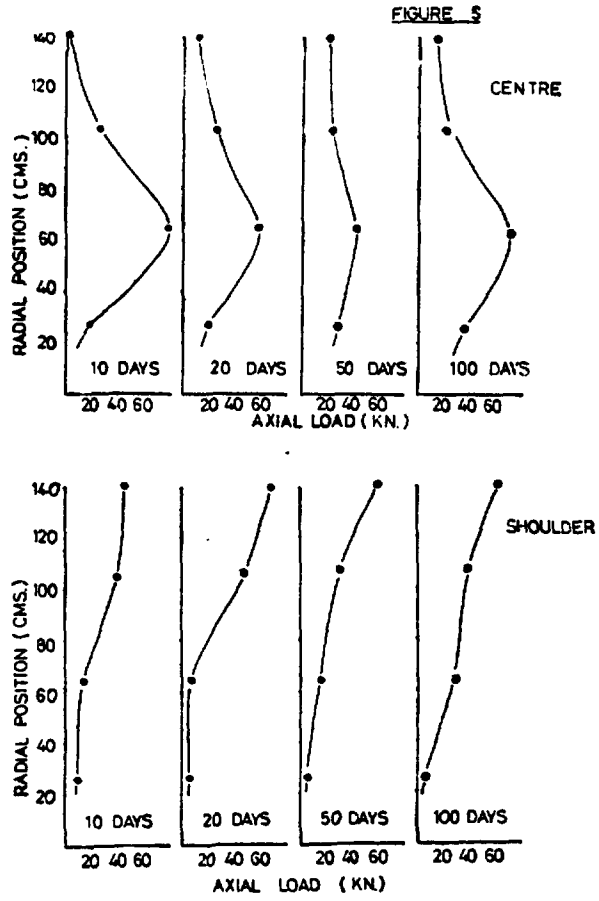

FIGURE 5 Variation of axial load along typical central and shoulder rockbolts—CASE *B*

average thickness 3 m, overlain by up to 15 m of anhydritic shale. The shale displays distinct time-dependent properties and observation have shown (Hebblewhite *el al* 1977) that movements occur in the roof to a height greater than 5m with initial closure rates of around 2mm/day. Floor and sidewall deformation are less intense. The geology and particularly the marl thickness tends to be variable.

The roof material comprised stratified and irregularly fractured anhydritic shale. The purpose of installing the bolts was to form a beam or arch of the roof layers capable of supporting the overlying strata. A staggered pattern of 1.5 m long fully resin-bonded bolts untensioned and in rows of 5 with all bolts vertical was installed. The rows were 1.2 m apart. Convergence and roof deformation using anchor extensometers to a depth of 3m was measured in the bolted zone.

The results in Figure 6 show that roof deformation due to creep and bed separation is high and in this particular rock the objectives sought by rockbolting were not achieved. The ultimate roof state after abandonment is illustrated in Figure 7. This is not altogether surprising considering the depth of the mine and the nature of the rock. To prevent yield an overall pressure of around 25,000 kN/m² would have to be applied to the roof which is not practicable, and some degree of yield is inevitable. A successful rockbolt system would be required to yield during the initial period of very high strain at the same time providing sufficient support to prevent spalling of small roof blocks. In the present case failure of bolts by necking, or of the bond was inevitable. Yielding bolts or wooden spiles or dowels might have proved a better solution.

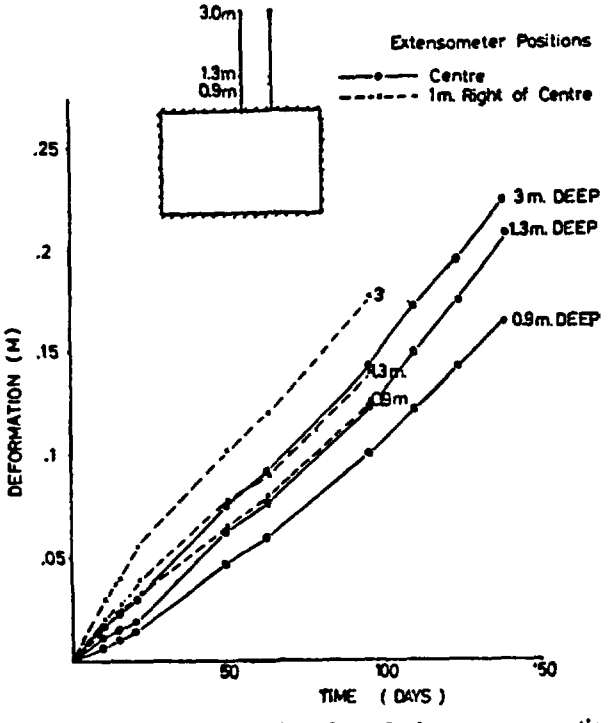

FIGURE 6 Deformation of bolted roof above an excavation—CASE *C* (after Wiggett 1976)

CONCLUSIONS

The case histories were chosen to illustrate three major aspects of rockbolt support. For the particular cases, they demonstrate

(1) in shallow strongly discontinuous rocks, stability can be obtained by designing and placing individual rockbolts to support blocks where movement is kinematically possible.

(2) in deeper discontinuous rocks, rockbolts can control deformation and prevent detachment of blocks.

(3) in very deep, weak rocks, rigid rockbolts do not satisfactorily control deformation, and flexible bolts must be adopted for successful support.

ACKNOWLEDGMENTS

The work was supported by Nobels Explosives Company Ltd., and the authors acknowledge the contribution of R.W. Beveridge of that Company.

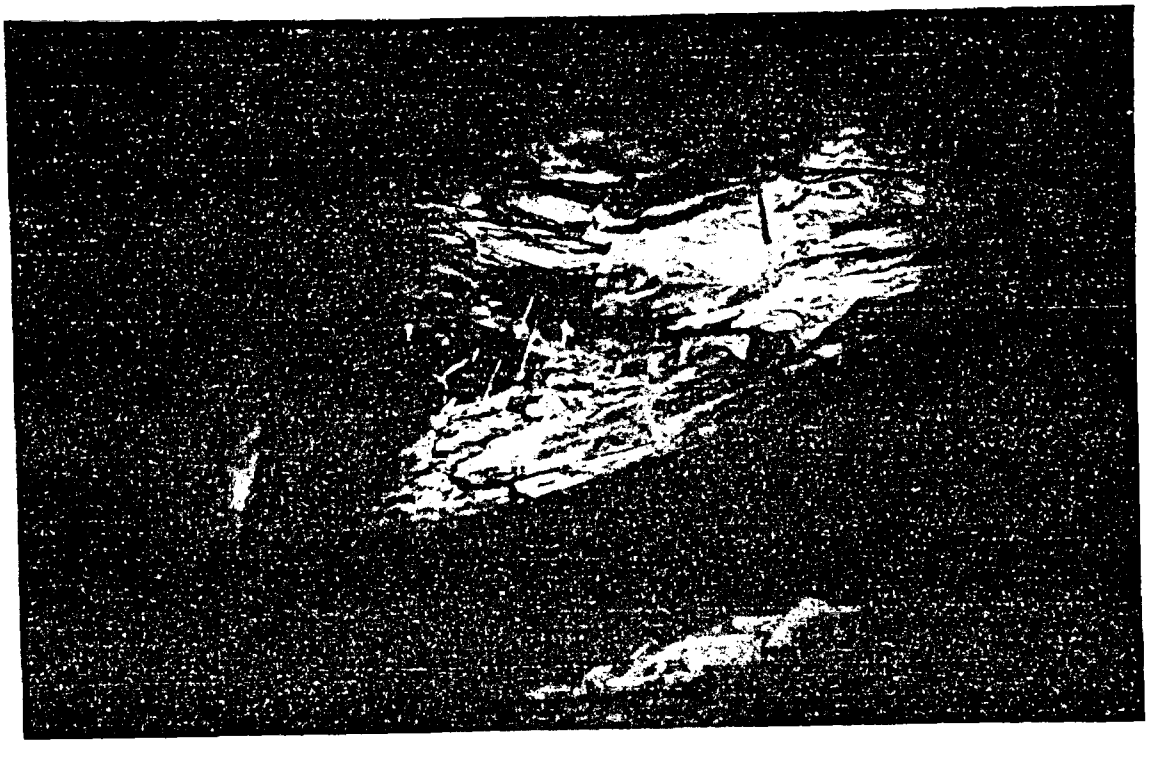

FIGURE 7 Ultimate state of roof after abandonment—CASE *C* (after Wiggett 1976)

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BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.30
WM Branch.
Comment topic. Analysis of potentially adverse conditions.

(B) Was the comment addressed in the Final EA? NO.
Where? [Maybe the original comment (p 6.211 and para 3) that work is planned during SC covers. It does not strike me as a major concern @ this stage]

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.31
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? NO
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

This is important. It is probable that in the case of openings in evaporites, the shaft pillar offers virtually no long term protection. Creep of shaft pillar will cause settlement and tilt of the shaft with potential for seal damage. This will be exacerbated with heating. [I know a case history, but it has not been publicised!]

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.32
WM Branch.
Comment topic. Conclusion (p 6.86)

(B) Was the comment addressed in the Final EA? NO (except, p 6.105 for removal of references to thermal expansion)
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

One of the problems of this report is the continual reference to mining in salt as proven. Shallow, high extraction mines in salt domes may be (~1000ft) in low extraction (cf WIPP) deeper openings the rock will fracture (see attachment to 6.29) after 4-8 weeks. This has led, in Canadian, U.K., European salt & potash mine to modifications of yield pillar techniques & rapid development & abandonment — none of which will work in a salt repository.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

* Unless arguments can be put forward the site should be disqualified.

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6-37
WM Branch.
Comment topic.

(B) Was the comment addressed in the Final EA? No
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

Damage to shaft & seals (see Dowding) will probably occur at 0.2g if shaft is fully restrained by the rock. If it is unrestrained (i.e. fracture zone, backwall cavity) and has natural frequency close to seismic wave frequency damaging vibration can occur at much lower levels ? 0.1g ? this should be addressed.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.4-7
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? YES.
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

This is not wholly relevant, but the site has 125m (410ft) of medium to high plasticity clays with high swelling potential. The possible effects of the latter are not addressed. It may be a real problem in shaft sinking & foundation design for heavy sensitive buildings.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.48
WM Branch.
Comment topic. Qualifying conditions.

(B) Was the comment addressed in the Final EA? YES.
Where? p 6.161 (bottom) - 6.162 (1st 3 paras)

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

The argument for representativeness of rock could be reinforced by RQD core data, but say it is acceptable. The third paragraph which quotes salt mining experience in domes is not acceptable without data on % extraction, depth, conditions &c. This applies to other sections cited also.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.49
WM Branch.
Comment topic. Qualifying condition

(B) Was the comment addressed in the Final EA? NO
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

Creep tests are irrelevant to design (see attached). The tests used are artificial. Under redistributed stresses rocksalt fractures and strength is the most important parameter. At 700m the stress around an opening here will be twice the unconfined strength measured in the laboratory.

FORGET CREEP!

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

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A time dependent model for lining pressure based on strength concepts

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SYNOPSIS

Weak rocks when subjected to moderately high confining and deviatoric stresses tend to exhibit time dependent deformation. This is most commonly characterised through phenomenological models of time-hardening strain under constant stress. An alternative description based on strength reduction with time is suggested. The implications of this approach for the design of shaft linings in evaporite rocks are discussed.

INTRODUCTION

Weak rocks when subjected to moderately high confining and deviatoric stresses continue to deform after initial application or redistribution of the stresses causing deformation. This time dependent deformation or creep is most commonly characterised through laboratory maintained load tests which predict a time-hardening strain rate of the form:

$$\varepsilon = At^{B}\sigma^{C}f(\theta) \qquad (1)$$

where t is time, σ is deviator stress, $f(\theta)$ is some function of temperature and A, B(<1) and C(3.5-4.3) are constants. This relation can be represented by rheological models of varying complexity, some of which are outlined by Jaeger and Cook (1968).

Such an approach does not explicitly consider the effect of stress redistribution. Around voids in a continuum, however, if the continuum material in its residual state is capable of mobilising shear, deformation must ultimately lead to stress redistribution. Thus equation 1 is only strictly applicable to isolated pillars.

If the effect of stress redistribution is significant, it represents a factor which has been ignored in the complex rheological models which have been developed for rock salt in particular. The most complex of these are the thermodynamic approaches of Biot (1954) and Fossum (1977). This work has often been based on developments in metallurgy where studies of micro-mechanistic phenomena (see Cittus, 1975) have been used to explain the creep of larger units. Although similar work on rock salt (see Le Compte, 1960 and Heard, 1972) has yet to yield any comparable success, probably because of the random nature of natural deposits, it does give a physical interpretation of the power law strain rate/stress relation of equation 1.

Rock salt does, however, exhibit classical strength behaviour and behaves as a shearing, C - ϕ material, when subjected to stress conditions similar to those existing in the vicinity of a void in a continuum during excavation. It is therefore sensible to suggest the existence of a zone of fractured or deformed rock capable of mobilising shear resistance under such conditions. This could be particularly important where a lining is introduced. Rheological models (see Gnirk and Johnson, 1964) suggest, ultimately, that full geostatic stresses are transferred to the lining as no other support mechanism is available. Shearing or yielding models suggest much lower stresses. Serata (1959) has suggested such a model, similar in concept to the critical state models for soil (Schofield and Wroth 1968, Atkinson and Bransby, 1979). The probability is that a true description of rock salt behaviour falls between the two extremes. To investigate this behaviour does, however, require a different approach to that used in time dependent deformation studies, involving an investigation of rock strength and involving the constant strain rate testing of rocks. This is not new, but

has not been applied on a large scale to rock salt. It is, however, an obvious approach to a redefinition of rock deformation.

CONSTANT STRAIN RATE TESTS

Triaxial tests on 75mm diameter x 150mm long rock salt specimens at a constant strain rate of 21 $\mu\varepsilon s^{-1}$ and confining pressures between 0 and 42 MPa have been described by Price and Farmer (1979, 1981). The confining pressures were selected to represent a reasonable range of stresses at sensible engineering depths. The results are summarised in Figure 1a as deviator stress against octahedral stress at an arbitrary 2% axial strain and in Figure 1b as the ratio between octahedral (p) and deviator (q) stress against the slope of the volumetric strain (ε_v) shear strain (ε_s) curve at the same strain. The relation between the latter two variables indicates a relation for the linear or ductile region of the curve in the form:

$$q/p = M - K \frac{\delta\varepsilon_v}{\delta\varepsilon_s} \qquad (2)$$

where M = 0.5 and K = 8. This equation is a modification of Taylor's (1948) equation for the dilation of sand, or for the Hvorslev surface of critical state soil mechanics - representing a state boundary for heavily over-consolidated materials. The linearity of the curve in Figure 1b confirms the essentially frictional or shearing nature of rock salt deformation at the stresses involved, but the high K value also indicates rather more complex non-frictional deformation processes.

The relation between q and p in Figure 1a shows that at low confining pressures the rock salt behaves in a strain-softening

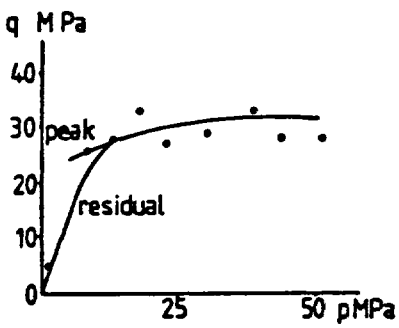

FIGURE 1a. Peak and residual strength envelopes for rock salt at 2% axial strain (after Price and Farmer, 1981).

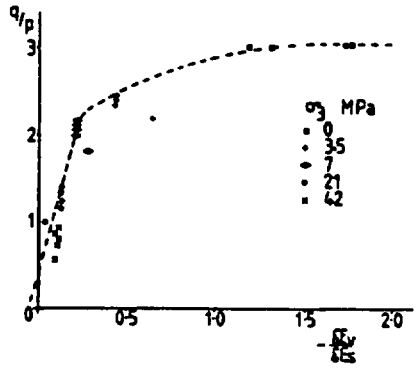

FIGURE 1b. Ratio of q/p plotted against $\delta\varepsilon_v/\delta\varepsilon_s$ for stress-strain data from Price and Farmer (1981).

manner having a peak and residual strength envelopes. At higher confining pressures, depending on the axial or shear strain, the rock salt exhibits ductile characteristics. The change from brittle to ductile deformation is represented by the change in slope of the curve.

In order to examine the effect of strain rate on this relation a series of additional tests were carried out on similar rock salt specimens at strain rates of $5 \times 10^{-3}s^{-1}$, $5 \times 10^{-4}s^{-1}$ and $2 \times 10^{-7}s^{-1}$ and at confining pressures between 0 and 42 MPa. In Figure 2 the relation between q and p, beyond the initial brittle-ductile transition is plotted at an arbitrary axial strain of 4%. It should be noted that whilst 4% strain was chosen to give the most complete curves, similar but less pronounced characteristics can be obtained at lower strains. Each of the curves follows the general pattern of Figure 1 reaching a plastic or critical state (represented by q = 0.79p) at a similar magnitude of q/p. The principal aspect of time dependency illustrated by the tests is however the reduction in shear resistance with reducing strain rate during testing - a feature noted by Bieniawski (1970) amongst others. This physical phenomenon is ignored in the phenomenological models discussed earlier. The relation can be characterised by abstracting a relation between the deviatoric stress (or shear resistance) at 4% axial strain at the critical state in Figure 3. This shows a reduction in strength of approximately 50% over the range of strain rates used in testing, and can be extrapolated to predict zero strength at a strain rate of 10^{-13} sec^{-1}. (The work of Munson (1979) suggests, however, there will be a change of mechanism, altering the slope before such a strain rate is reached.) Such a relation can be

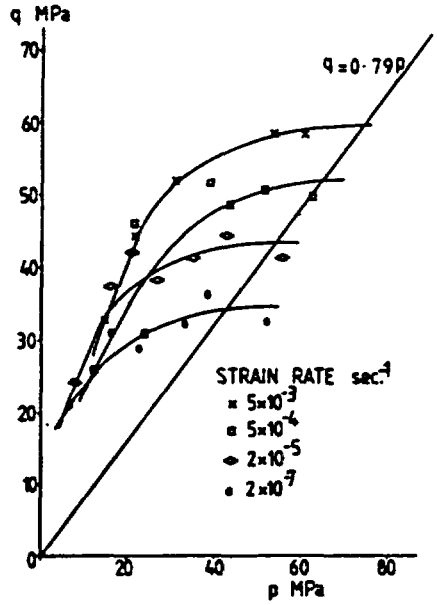

FIGURE 2. Strength envelopes for rock salt at 4% axial strain obtained at varying strain rates. The line q = 0.79p represents a limit at which the slope of individual curves approaches zero.

thought equivalent to a reduction in 'strength' with time and as such will only operate if the frictional resistance of the 'failed' material is not mobilised, as is the case with unlined voids in a continuum. Thus significant difference would be expected in the behaviour of lined and unlined voids. This can be illustrated by reference to two case histories.

UNLINED SHAFT IN ROCK SALT

Data published by Hebblewhite, Miller and Potts (1977) from a case history examined by Cook (1974) illustrates strains computed from anchor extensometer measurements in the sidewall of a 7.6m diameter shaft at the centre of a 50m bed of halite at a depth of 1056m. The sidewall was unsupported except for a layer of bagged vermiculite at the back of the 0.9m thick 5.5m internal diameter r.c. lining. The design resulted from maintained load tests which had indicated a creep function of the type :

$$\dot{\epsilon}_{rad} = At^{B}; \; \epsilon_{rad} = Ct^{(B+1)} \qquad (2)$$

This had suggested that a r.c. lining cast against the halite would quite rapidly be subjected to the full in-situ pressure from the deforming rock. Measurements of

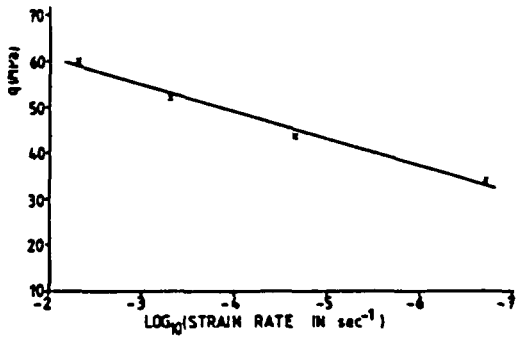

FIGURE 3. Relation between the deviatoric stress (q) at the critical state in Figure 2 and strain rate.

strains at anchors located in boreholes 0.6m, 1.5m, 3.0m and 4.5m from the unsupported shaft wall tended to support this hypothesis, and the laboratory creep functions fitted quite well the shape of the deformation curves obtained over a continuing observation period of 3 years. It was concluded that they confirmed the concept of time (strain) hardening. The results can however be explained just as well in terms of stress redistribution and an alternative equation of the form:

$$\dot{\epsilon}_{rad}(t) = D\sigma^{E} \qquad (3)$$

can be proposed, where D and E are constants and $\dot{\epsilon}(t)$ is the strain rate at a given time. This can be illustrated by a simple laboratory experiment designed to simulate the average short term strain history of an element between (say) the anchors 1.5m and 3.0m into the sidewall, illustrated in Figure 4.

The initial elastic stresses were estimated from thick cylinder theory at a depth of 2.25m into the sidewall as σ_r = 13 MPa σ_θ = 35 MPa and $\sigma_\theta - \sigma_r$ = 22 MPa. Tests were carried out in a servocontrolled test machine on a 75mm diamter by 150mm long cylindrical rock salt specimen consolidated initially to $\sigma_1 = \sigma_3$ = 13 MPa for 5 days. The axial stress, σ_1, was then increased at a strain rate programmed to simulate the average strain rate in Figure 4 - approximating closely the creep function described in equation 2.

The test results in Figure 5 are quite revealing, comprising a loading curve for 9 hours followed by a rapid reduction in deviator stress towards near stable level after about 300 hours at which time the strain rate is approximately 10^{-9} sec^{-1}. It would appear therefore, that the strain history is closely associated with stress

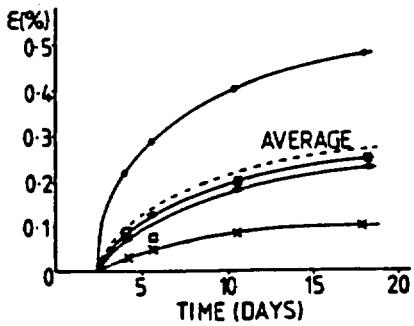

FIGURE 4. Computed strains from 4 instrumented boreholes containing fixed anchor extensometer points between 1.5m and 3m into an unsupported shaft sidewall in rock salt at a depth of 1056m (after Cook, 1974).

redistribution due to strength reduction, rapidly reaching a near stable level and thus is in close agreement with the theoretical model of Boresi and Deere (1963). The time hardening effects observed in this case therefore do not necessarily result from a maintained load, although they can be achieved by a maintained load test. Baar (1977) has in fact gone so far as to suggest that time (strain) hardening in rock salt is a purely laboratory phenomenon. The reduction in the deviator stress is consistent with the migration of a yield zone associated with the reduction of rock strength with time. This view is supported by volumetric analysis of the anchor data for the first

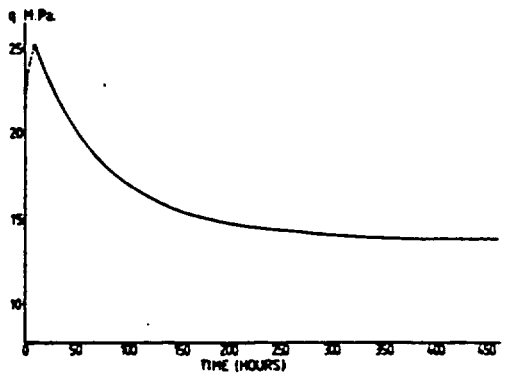

FIGURE 5. Variation of deviator stress (q) with time measured during a laboratory test designed to simulate the strain history in Figure 4.

20 days movement, which was not considered in detail by Hebblewhite, Miller and Potts (1977).

LINED SHAFT IN ROCK SALT

A recent experiment has been carried out in which borehole extensometers, diametral extensometers, vibrating wire strain gauges and hydraulic pressure cells were placed in and around a 7.6m internal diameter shaft with 1m thick lining at the centre of an 8m bed of rock salt at a depth of 326m. Tangential lining strains, and diametral measurements indicated that the most important factor affecting early lining deformation was concrete shrinkage. Results of radial pressure measurements at the lining - rock interface are illustrated in Figure 6. It can be seen that the average lining pressure (P_L) increases logarithmically with time in the form

$$P_L = 0.15 \ln(t) \qquad (4)$$

where t is in days.

Pressure change was not adequately described by a power law. Two things are apparent in the data in Figure 6. The lining stresses are significantly lower than the geostatic stress (estimated at 7.35 MPa) and they show no indication of approaching geostatic stress levels. The most realistic explanation for this phenomenom is that a yield zone, increasing

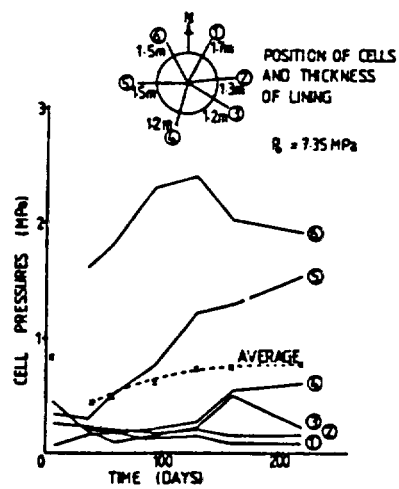

FIGURE 6. Measured radial stresses with time at a shaft lining/rock interface at a depth of 326m in rock salt.

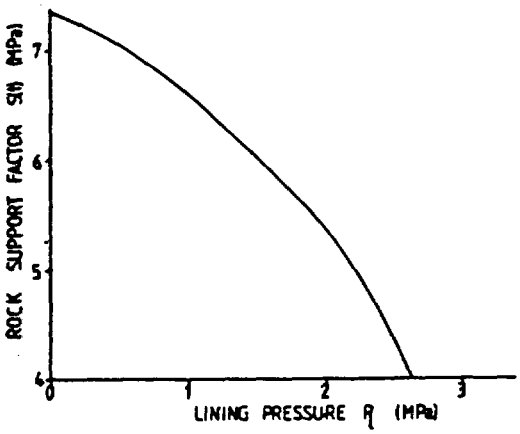

FIGURE 7. Relation between S(t) and lining pressure P(L) from equation 5.

slowly with time and encouraged by the shrinkage of the concrete is being formed behind the lining through reduction in strength of and redistribution of stress in the rock. Several models have been developed to describe this process, and one of the more recent, which allows for time dependency is that of Ladanyi (1974). If yielding rock is assumed to have no cohesion, Ladanyi's complex general equation for lining pressure (P_L) can be simplified in the form:

$$P_L = (Po - S(t))\left[\frac{2S(t)(1+\nu)/E - \varepsilon_v}{\dfrac{1 - \varepsilon_v}{1 - \Sigma(V_i/r_{io})^2} - 1}\right]^{\frac{(kp-1)}{2}} \qquad (5)$$

where Po is the geostatic stress

S(t) represents a proportion of the passive support provided by the rock mass to the rock/lining structure which varies with time. This can be called a <u>rock support factor</u>.

ν is Poisson's ratio

E is Young's modulus

Kp is the coefficient of passive earth pressure

ε_v is the average plastic dilation

ΣV_i is the sum of displacements

r_{io} is the excavated radius

Immediately before the lining is placed when $P_L = 0$, then either $P_o - S(t) = 0$ or $2S(t)(1+\nu)/E(t) - \varepsilon_v = 0$. Since ε_v is negative, $P_o = S(t)$ at $t = t_o$. If equation 5 is solved for various values of S(t) and using values of ε_v obtained from the data summarised in Figure 1 a relation between S(t) and P_L for the particular case studied can be plotted as in Figure 7. If a relation between S(t) and time can be estimated from the data in Figure 3 it should then be possible to estimate the strength reduction of the rock causing the increase in lining pressure in Figure 6. If q is linearly related to the logarith of strain rate, it follows that q will also be a linear function of the logarithm of time taken to achieve a specified axial strain. Hence a logarithmic decay of S(t) of the form:

$$S(t) = Po(1 - K_1 \log (t)) \qquad (6)$$

may be suggested.

Arbitrary selection of values of K_1 of 0.033, .066, .099 give corresponding reductions in S(t) of 15, 30 and 45% over 100 years. Values of P_L for these values of K_1 calculated from equation 5 are plotted in Figure 8, together with the measured data from equation 3. To allow for shrinkage effects, the increase on the 25 day values of P_L calculated from equation 5 is compared with the in-situ results. The comparison indicates that over the predicted life span of the lining, a 100 year time strength reduction of approximately 25% would fit the data best. The curvature of the solutions based on Ladayni's equation would be reduced if a linear function was substituted for a constant value of ε_v used in these calculations.

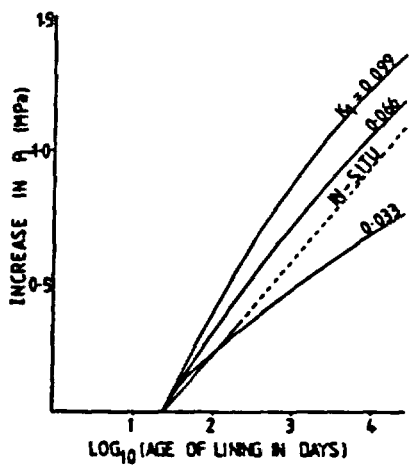

FIGURE 8. Comparison of predicted and measured increase in lining pressure with time at a shaft lining/rock interface at a depth of 326m in rock salt.

DISCUSSION

Constant strain rate tests have shown that rock salt can be described as a 'pseudo-frictional' material, whose response to stress involves additional complex non-frictional deformation mechanisms (equation 1). By varying the strain rate of triaxial tests, significant changes in apparent strength may be demonstrated. Thus laboratory work shows rock salt to be a complex material with a stress response which has both 'mechanical' and 'viscous' components.

During and immediately after the excavation of a cavity, the mechanical properties will be of a major importance, determining yield zone formation and ultimate stress distribution. Thus a large reduction in deviator stresses and deformation which is significantly non-isovolumetric are observed during the first 10-15 days after excavation of a deep shaft in rock salt. Beyond this period viscous phenomena tend to mask any residual primary mechanical behaviour, and 'secondary mechanical behaviour' such as passive support is all that remains of this component of the material stress response mechanism. Secondary mechanical behaviour, which is crucial to the lining pressure can only be inferred, however, in the cases considered.

ACKNOWLEDGEMENTS

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BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.50
WM Branch.
Comment topic. Statement of qualifying conditions

(B) Was the comment addressed in the Final EA? No - except by omission. We are left with statement on p 6.168 para 1 (end) which leaves the thing in the air. The probability is that
Where?

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

retrieval will be extremely difficult - almost impossible - & it should be addressed.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.53
WM Branch.
Comment topic. Analysis of favorable conditions.

(B) Was the comment addressed in the Final EA? YEL.
Where? p 6-163 para 1.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis? ✓
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

This seems to be adequately but horizontal drilling from underground during SC may be useful

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.54
WM Branch.
Comment topic. Analysis of favorable conditions

(B) Was the comment addressed in the Final EA? YES
Where? p 6.163/164 p 6.167 para 4.

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify) ✓ changed from present to not present

(D) Was our comment addressed as we suggested?
If not, specify.

As mentioned in 6.32 I think that a logical discussion of available data would lead to disqualification. Take in para 4 p 6.163 - if instead of the Wagner analysis a simple fracture zone analysis was carried out, it would indicate 40% closure after excavation (check)! These people are ignoring the effects of stress!

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.55
WM Branch.
Comment topic. Analysis of favorable conditions

(B) Was the comment addressed in the Final EA? YES
Where? p 6.163 paras 5,6,7

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify) ✓ changes to not present

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.56
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? YES
Where? p 6-164 last 4 paras

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

In One of the cases discussed, Boulby mine, a shaft, passing through Permo-Trias sandstone was treated by chemical grouting (@ 2,700 ft) rather than freezing. This shaft is watertight; the frozen shaft section has leaked badly (personal observation). Chemical grouting of the broken cap rock is feasible and would represent a superior alternative to freezing. It may take longer, but the finished job would have a lining, backwall grout, seal, rock system which was fully integrated & watertight. Shaft lining pressures may also be reduced (see attached)

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.57
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? YES
Where? p 6-163 top para and also 3-60-3.63

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

This looks thin, but the argument that Interior Domes have less anomalies the coastal domes is convincing. The margin is so slight however that something must be included in SP.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.58
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? YES
Where? p 6.165 para 3.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

(D) Was our comment addressed as we suggested? If not, specify.

Previously considered in 6.32, 6.54. Anomalous zones will probably not be a condition for major deformations. Conditions are likely to be worse than Gulf Coast mines. I would forsee heavily reinforced main access & ventilation roads with very short life (6-8 weeks) disposal rooms as a basic layout. This would necessitate radical redesign.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.59
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? YES
Where? p6.164 last 3 paras

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

See Comment 6.56

Freezing appears not to disturb strata (although the saturated clays in top 32-125 m may swelling present a challenge).

(D) Was our comment addressed as we suggested?
If not, specify.

The attached indicates the strengths & weaknesses of freezing. The KEY conclusion is that after thawing the full hydrostatic head of groundwater acts on the lining — usually through an annular cavity caused by cement bleed/hydration. Despite the conclusions, the outer layer may be weak. Anything that involves contact grouting is a problem/over

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

Sealing the shaft as in Fig 5.13-5.15 (post closure) or 4.13 (construction) may cause problems for different reasons. Consider a dam-cutoff. Would a short seal and no grouting be acceptable.

Grouting should be considered as an alternative

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.102
WM Branch.
Comment topic. Estimation of

(B) Was the comment addressed in the Final EA? YES
Where? previous paragraph

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

TEMPERATURE, STRESS AND STRAIN MEASUREMENTS DURING AND AFTER CONSTRUCTION OF CONCRETE LININGS IN FROZEN SANDSTONE

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ABSTRACT

Vibrating wire strain and temperature gauges and total stress cells and piezo ers were installed in a 7.3m inter diameter concrete shaft lining at a th of 232m in frozen saturated Bunter Sandstone. Observations of temperature changes during concrete hydration and stress changes during thawing of the icewall, led to the following main conclusions:

a) Rises in temperature during hydration of the cement in the concrete lining raised rock temperatures to a level where they would not affect hydration of the concrete at the rock-concrete interface.
b) Following thawing of the icewall, hydrostatic and total lining stresses were identical.

1. INTRODUCTION

When concrete shaft walls are cast against frozen strata during shaft construction, lining thicknesses are increased by 150mm (Auld, 1979; Weehuizen, 1959) to allow for incomplete hydration of the concrete in contact with the icewall. There is, however, no experimental evidence to justify the assumption that the temperature of the concrete adjacent to the icewall will be reduced during the initial stages of hydration to temperatures of around 4°C, where hydration is inhibited.

During sinking of shafts through frozen Bunter Sandstone in the Selby Coalfield, an opportunity arose to measure temperature in shaft linings and adjacent sidewalls at Whitemoor Mine. Instrumentation was designed and installed with the objective of measuring temperature variation in a nominal 600mm thick lining and at distances up to 1m into the frozen ground. In addition total stresses, strains and piezometric pressures were measured in the lining and at the lining/rock interface after placing of the lining and following the subsequent thaw.

2. STRATA AND SHAFT DETAILS

Whitemoor Mine shafts have a planned depth of 920m. The first 20m of shaft is through glacial drift overlying 260m of Bunter Sandstone. The remaining succession comprises Permian marls and limestones to 500m. These lie unconformably on the Middle Coal Measures. The surface level is 7m A.O.D. and the ground water level is approximately 10m below ground level. The Bunter Sandstone has high permeability and porosity. Packer permeability inflow borehole tests at an adjacent shaft site give permeability coefficients of:

Depth(m)	Permeability coefft. (m/s)	Estimated inflow to 10m shaft (l/m)
42.34- 52.03	5.01×10^{-6}	175
130.79-143.53	1.80×10^{-6}	205
200.94-210.63	2.10×10^{-6}	340

The average horizontal permeability coefficient of borehole specimens was 8.14×10^{-6} m/s and the average porosity

of borehole specimens was 34.4%. Other geotechnical average data from dry laboratory specimens were:

Uniaxial compressive strength	9.8	MN/m^2
Uniaxial deformation modulus	5.5	GN/m^2
Cohesion	2.8	MN/m^2
Coefficient of internal friction	0.50	
Dry unit weight	1.84	KN/m^3
Specific gravity	2.67	

Some tests were also carried out on frozen core specimens giving modified geotechnical data:

Uniaxial compressive strength	36.2	MN/m^2
Uniaxial deformation modulus	7.5	GN/m^2

During drilling of exploration boreholes there was low core recovery in the Bunter Sandstone, confirming the general picture of a weak high porosity sandstone in which the major part of the water flow was through the porespace. Selection of groundwater freezing to control water inflows and ensure sidewall strength at low depth was made at an early stage in the project. Detailed design of freezing is given by Wild and Forest (1981).

The shafts at Whitemoor Mine were 7.3m (24ft) internal diameter with nominal concrete design thickness of 0.6m, throughout the frozen zone. The specified concrete was 45 MN/m^2 sulphate resisting concrete to class 4 Building Research Establishment (U.K.) Digest 174.

3. INSTRUMENTATION

The basis of the instrumentation has been described by Altounyan (1982). It comprised vibrating wire temperature gauges, vibrating wire strain gauges and piezometers and stress cells with vibrating wire transducer outputs. These were multiplexed at source and monitored from the surface through a single shaft cable.

The general arrangement of the instrumentation layout at a depth of 232m is illustrated in Figure 1. At the level selected, overbreak had increased the lining thickness to an average of 950mm. Temperature gauges were installed at 90° intervals in the frozen rock and concrete. Boreholes were drilled 1.2m into the frozen rock and four temperature gauges mounted longitudinally in a p.v.c. tube were inserted so that the centre of the outer gauge was 50mm from the shaft sidewall. The other gauges were 320mm, 770mm and 1170mm from the sidewall. The hole was initially filled with grease to ensure optimum conductivity. Three temperature gauges were installed directly into the concrete, mounted on a rebar attached to the shuttering and hanging rods. The gauges were positioned (see Table under Figure 1) centrally and as close as practicable to the surface, backwall and axis of the lining. Strain gauges were located at 60° intervals. These were cast in pre-cured concrete briquettes and located on the shuttering and backwall to measure inner and outer lining hoop strains. Mercury filled pressure cells were located at three of these locations at the rock-lining interface to measure total radial stresses. Piezometers were located in 500mm boreholes backfilled with bentonite pellets at the same locations to measure piezometric pressures.

4. RESULTS

The results can be grouped in three categories: the change in temperature during cement hydration in the setting concrete; the change in temperature of the saturated/frozen sandstone during natural and forced thawing after cessation of the freeze, and the increase in ground water pressure and associated lining stresses and strains during the thawing of the icewall.

Continuous temperature observations were taken for 37 days after casting of the lining. The thaw was started 13 days after casting of the lining.

Figure 2 shows typical temperature measurements in the lining and icewall from one array (gauges 8-13) of temperature gauges. These are reproduced as an isometric projection in Figure 3. The following points may be noted:

a) A maximum temperature of 50.0°C at gauge 13 near the centre of the lining and of 49.8°C at gauge 12, 290mm from the rock concrete interface, occurred 29 hours after pouring of the concrete. A peak temperature of 38.4°C at gauge 14 occurred 29 hours after pouring of the concrete.

b) The effect of the exothermic reaction due to hydration is to increase temperatures in the icewall to above freezing point. At gauge 11, 50mm into the rock an initial temperature of -1.1°C, initially relatively high, due to the presence of warm air in the excavation, was raised to 0°C after 12 hours and reached a peak

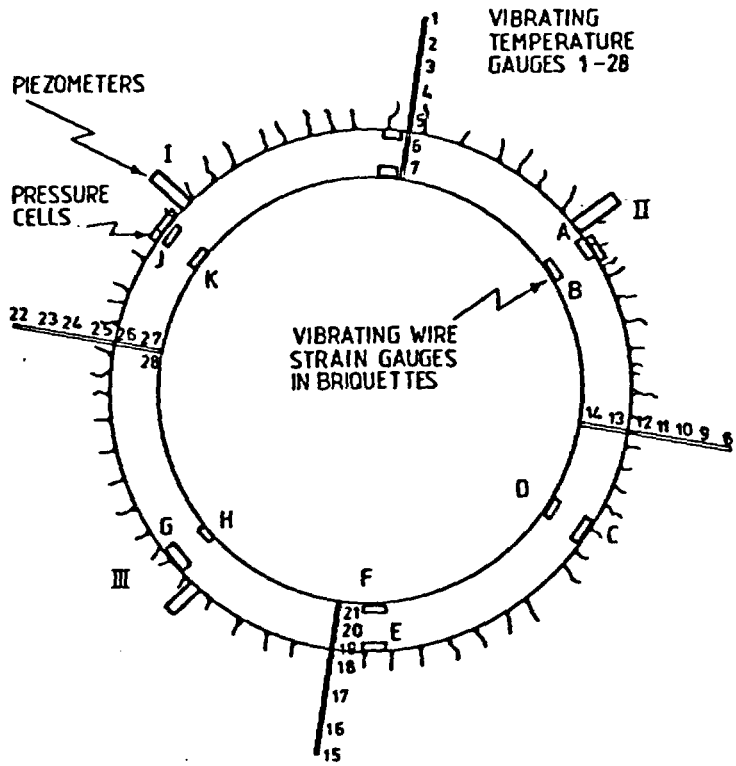

Figure 1. General layout of instrumentation around Whitemoor Mine No. 2 shaft at 232m in frozen Bunter Sandstone.

Position of temperature gauges

Gge No	Dist mm	Gge No	Dist mm	Gge No	Dist mm	Gge No	Dist mm	Gge No	Dist mm	Gge No	Dist mm	Gge No	Dist mm
7	115	6	340	5	555	4	790	3	1060	2	1510	1	1910
14	115	13	490	12	755	11	1095	10	1365	9	1815	8	2215
21	115	20	145	19	615	18	875	17	1145	16	1595	15	1995
28	110	27	465	26	770	25	925	24	1195	23	1645	22	2045

* Gauge Nos 4, 11, 18, 25 installed in the frozen ground 50mm from rock surface. Lining thicknesses, including overbreak, were 740mm at array 1-7; 1045m at 8-14; 825 at 15-21 and 875 at 22-28. Nominal shaft diameters were: inside 7320mm; outside 9090mm. Average freeze tube spacing was 687mm in a 1400mm diameter ring. Freeze tube steady state temperature was -30°C.

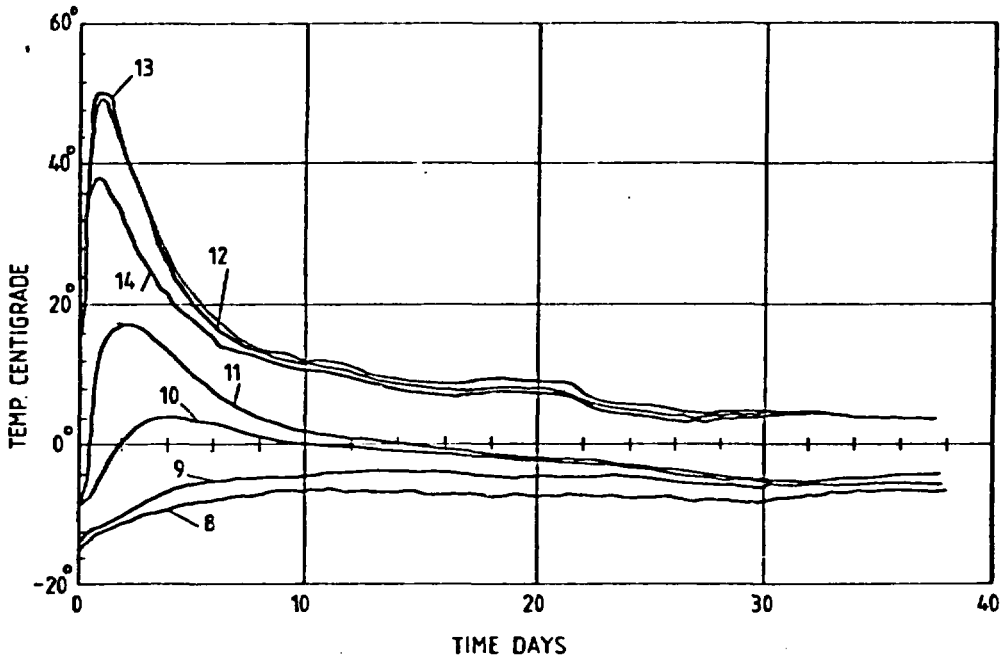

Figure 2. Change in temperature with time after pouring of concrete at gauges in array 8-14. Note position of gauges - 11, 50mm into icewall; 10, 320mm into icewall; 9, 720mm into incewall and 8, 1170mm into icewall. The concrete was 1045mm thick and gauge 12 was 290mm from the icewall; gauge 13 was 550mm from the icewall and 495mm from the shaft surface, and gauge 14, 115mm from the shaft surface.

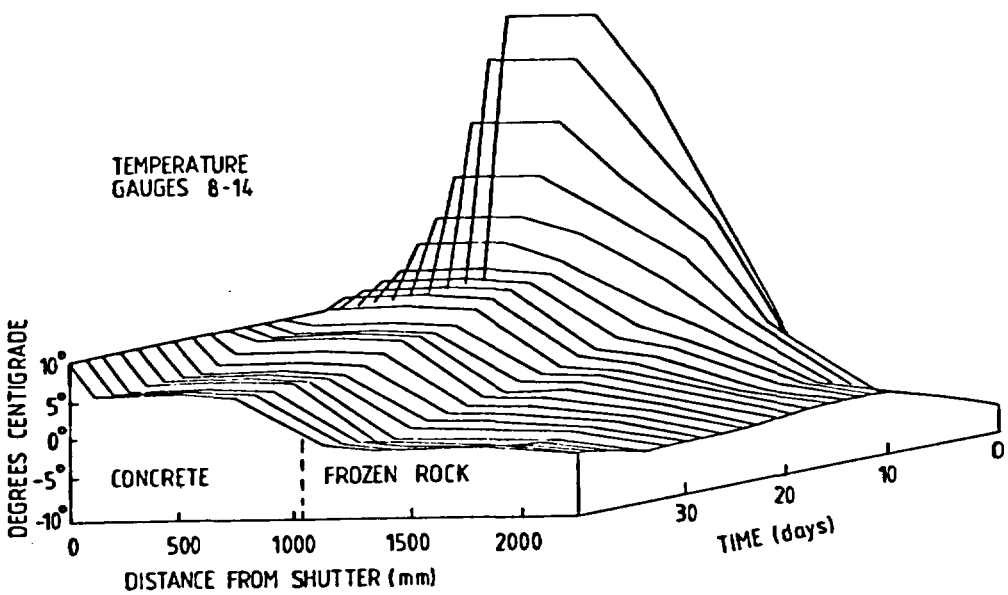

Figure 3. An isometric projection of the temperature profile with time through the shaft wall at array 8-14.

of 16.9°C after 54 hours. Gauge 10, 320mm into the rock recorded an initial temperature of -8.8°C, reached 0°C after 46 hours and a peak of 4.1°C after 98 hours.

c) Gauges 9 and 8, respectively 770mm and 1170mm into the rock recorded initial temperatures of -12.4°C and -14.2°C and although temperature rose significantly, it did not reach freezing point.

d) Gauge 11 did not monitor refreeze until 13 days after initial pouring of the concrete. None of the gauges in the concrete had recorded a temperature lower than 4°C when detailed observations ceased after 37 days.

Figure 4 plots change in temperature with time at the shaft wall/icewall interface, following cessation of freezing. Forced freezing started at -15°C and rose in stages to 0°C. The first major breach of the icewall occurred after 209 days (Table 1) and lining stresses - initially having a minor geostatic effective stress component - gradually rose to hydrostatic stress levels.

Table 1. Total stress and hydrostatic pressure readings.

Time after concrete pour(days)	Total stress-pressure cells (MN/m^2) I	II	III	Hydrostatic pressure-piezometers (MPa) I	II	III
15	0.37	0.40	0.33	0	0	0
200	0.40	0.42	0.36	0.03	0.05	0.04
209	0.67	0.66	0.63	0.50	0.53	0.56
217	0.68	0.69	0.65	0.48	0.38	0.49
300	2.20	2.11	2.17	2.22	2.20	2.21

From data on total stress and hydrostatic pressures obtained from pressure cells and piezometers in Table 1, the following points may be noted:

i) There is a close correlation between piezometric and total stresses . The average piezometric stress for three gauges after 300 days was 2.21 MN/m^2. This compares with a theoretical piezometric pressure of 2.18 MN/m^2 if the ground water level is assumed 10m below ground surface. The average total stress is 2.16 MN/m^2, indicating zero contribution to lining stresses from the back-wall rock - even though this was a relatively weak rock. This confirms previous observations by Altounyan and Farmer (1981) of the very low strata disturbance caused by groundwater freezing.

ii) The lining strains can be shown to exactly relate to the hydrostatic stresses in accordance with Lame's theorem, and a computed deformation modulus of 33.6 GN/m^2 compares with the design modulus range for 45 MN/m^2 concrete of 27-38 GN/m^2.

CONCLUSIONS

a) The effect of the exothermic reaction during freezing was to increase temperatures in the icewall to above freezing. Gauges 50mm into the icewall did not monitor freezing point until 13-14 days after pouring of concrete. It may be concluded that the presence of the ice wall did not inhibit hydration of the concrete at the rock-lining interface.

b) Following thawing, total and hydrostatic stresses on the lining at the rock-lining interface were identical, indicating zero geostatic component of radial lining stress, and minimum disturbance of strata by groundwater freezing.

ACKNOWLEDGEMENTS

The work was carried out as part of a research project supported by the National Coal Board and the Research Fund of the European Coal and Steel Community (ref. 7220-AC/84). The views expressed are solely those of the authors.

REFERENCES

ALTOUNYAN P.F.R. (1982) Measurement of stresses, strains and temperatures in concrete shaft linings. Proc

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6-101
WM Branch.
Comment topic. Estimation of extent

(B) Was the comment addressed in the Final EA? YES
Where? 6-A4/A5
top bottom

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis? ✓
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

Symp. Strata Mechanics, Newcastle upon Tyne, April 1982.

ALTOUNYAN P.F.R. and FARMER I.W. (1981) Tunnel lining pressures during groundwater freezing and thawing. Proc. 5th Rapid Exc. Tunn. Conf., San Francisco, 784-800.

AULD F.A. (1979) Design of concrete shaft linings. Proc. Inst. Civ. Engrs. Pt. 2 Res. & Theory, 67, 817-832.

WEEHUIZEN J.M. (1959) New shafts of the Dutch State Mines. Proc. Symp. Shaft Sinking & Tunnelling. London Inst. Min. Eng.

WILD W.M. & FORREST W. (1981) Application of the freezing process to ten shafts and two drifts at the Selby project. Mining Engineer, 140, 895-904.

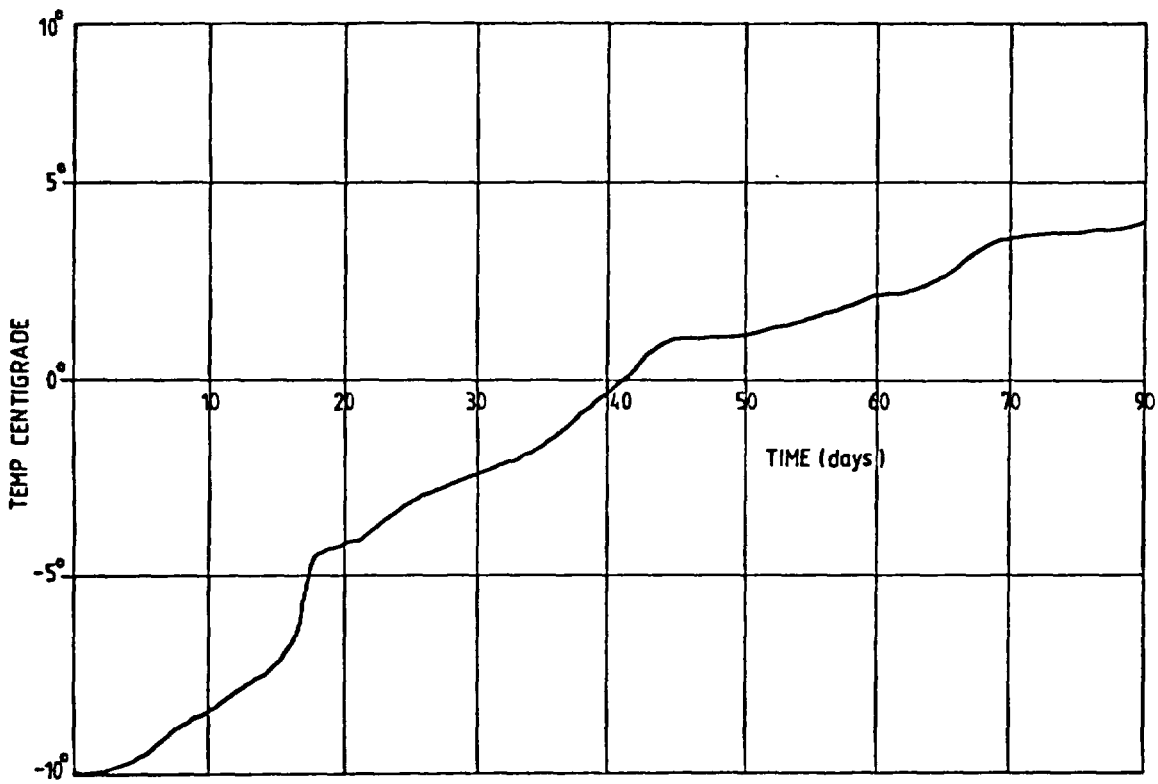

Figure 4. Change in temperature with time at the shaftwall-rock interface during thawing of the icewall. (Zero time is equivalent to 40 days on Figure 1)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.60
WM Branch.
Comment topic. Analysis of potentially adverse condition

(B) Was the comment addressed in the Final EA? NO
Where? Despite minor amendments & rewrites of paras 2, 3 (Evaluation p 6-165) and addition of p 6.166 para 2 this remains far too optimistic.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions? The heat effects are probably less significant than mechanical effects.
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.61.
WM Branch.
Comment topic. Analysis of potentially adverse condition

(B) Was the comment addressed in the Final EA? NO.
Where? [The conclusion p 6.166 para 2 essentially agrees with the comment]

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

rather than addressing it. The prospect of removing hot, radioactive cannisters from collapsed excavations at depth in weak rock is difficult to address.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.62
WM Branch.
Comment topic. Conclusion

(B) Was the comment addressed in the Final EA? NO.
Where?
The conclusion as it stands is incompatible with the conclusions at the top of 6.164 and 6.166 both of which are unfavorable. This is hopelessly optimistic bearing in mind the depth and weakness of the salt and the lack of knowledge of geotechnical and other data.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6-63
WM Branch.
Comment topic. Conclusion for qualifying condition

(B) Was the comment addressed in the Final EA? YES
Where? p 6.165 para 3.

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

This comment (6.63) was unfair. The paragraphs are similar & there was & remains little inconsistency. They are both based on incorrect assumptions of creep rather than fracture. Note: unconfined rock salt fractures See Table 3.8, top line for confirmation & attachment to 6.27 for strain-softening characteristic.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.64
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? NO
Where? [experience has increased to 100 years]
YES? 4.36 – 4.42 although no reference to this is included in analysis

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

(D) Was our comment addressed as we suggested?
If not, specify.

Grouting (which is mentioned & dismissed briefly on p 4.37 para 3 may be a better alternative for the fractured cap rock. See comment 6.57

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6-65
WM Branch.
Comment topic. Analysis of potentially adverse conditions

(B) Was the comment addressed in the Final EA? YES
Where? p 6-173 last 2 para

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information? ✓
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions? ✓
3) Other? (Specify)

See comment 6-37

(D) Was our comment addressed as we suggested?
If not, specify.

There is a strong possibility which has not been addressed of a disturbed zone due to shaft sinking in this very broken strata. It is difficult to see how this can be sealed using the method in 4.13.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6.99
WM Branch.
Comment topic. Geologic sub-system performance.

(B) Was the comment addressed in the Final EA? NO except
Where? indirectly 4.36 - 4.42

(C) How was our Draft EA comment addressed?
1) New information?
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions?
2) Same conclusions?
3) Other? (Specify)

See Comment 3.14.

(D) Was our comment addressed as we suggested?
If not, specify.

One failed (possibly) because of hydrofractures induced during seal emplacement (Avery Island). There are so many potential reasons that it can be argued that the question is not if but when.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)

BWIP,NNWSI,SALT
DRAFT EA/FINAL EA COMMENT FORM

NRC Compiler_____ Date_____ Contractor ( )

(A) Draft EA comment number. 6-100
WM Branch.
Comment topic. Estimation of the extent of the disturbed zone.

(B) Was the comment addressed in the Final EA? YES
Where?
p 6A-3 items 6-9

(C) How was our Draft EA comment addressed?
1) New information? ✓
2) New analysis?
3) Rewrite, with rationale, of previous information?
4) Other? (Specify)

1) Modified conclusions? ✓
2) Same conclusions?
3) Other? (Specify)

Q. Why are shafts excluded from this? (3 para). The disturbed zone will certainly exist.

(D) Was our comment addressed as we suggested?
If not, specify.

(E) Status.
1) Has the basis for our concern changed?
2) Resolution deferred by DOE to SCP?
3) Has a new NRC concern developed?
4) Other? (Specify)