# KBS TEKNISK RAPPORT

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# Groundwater movements around a repository

## Rock mechanics analyses

Joe L Ratigan

Hagconsult AB september 1977

KÄRN - BRÄNSLE - SÄKERHET

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GROUNDWATER MOVEMENTS AROUND A REPOSITORY

ROCK MECHANICS ANALYSES

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I slutet av rapporten har bifogats en förteckning över av KBS hittills publicerade tekniska rapporter i denna serie.

# TECHNICAL REPORT 4

## ROCK MECHANICS ANALYSES

KBS - Kärnbränslesäkerhet

GROUNDWATER MOVEMENTS AROUND A REPOSITORY

Phase 2. Technical report 4. Rock Mechanics Analyses

Hagconsult AB
in association with
Acres Consulting Services Ltd
RE/SPEC Inc.

## FOREWORD

This report was prepared as one of a series of Technical reports wihtin a study of the groundwater movements around a repository for radioactive waste in the precambrian bedrock of Sweden. The contract for this study was between KBS - Kärnbränslesäkerhet (Project Fuel Safety) and Hagconsult AB of Stockholm, Sweden. RE/SPEC Inc. of Rapid City, SD/USA and Acres Consulting Services Ltd of Niagara Falls, Ontario/Canada acted as subconsultants to Hagconsult AB.

The principal author of this report is Mr. Joe L. Ratigan of RE/SPEC inc. Review was provided by Dr. Ulf E. Lindblom of Hagconsult AB, Dr. Paul F. Gnirk of RE/SPEC Inc., and Dr Robin G. Charlwood of Acres Consulting Services Ltd.

The opinions and conclusions in this document are those of the author and should not be interpreted as necessarily representing the official policies on recommendations of KBS.

Stockholm September 1977

Ulf E. Lindblom
Study Director
Hagconsult AB

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# ROCK MECHANICS ANALYSES OF A RADIOACTIVE WASTE REPOSITORY

## 1. INTRODUCTION

The determination and rational assessment of groundwater flow around a repository depends upon the accurate analysis of several interdependent and coupled phenomenological events occuring within the rock mass.

In particular, the groundwater flow pathways (joints) are affected by the excavation and thermomechanical stresses developed within the rock mass, and the properties of the groundwater are altered by the temperature perturbations in the rock mass. A detailed discussion of the interrelated coupling phenomena is presented in (1)[x] and will not be repeated in its entirety in this report.

The objective of this report is to present the results of the rock mechanics analysis for the repository excavation and the thermally-induced loadings. These results will be used in subsequent studies to assess the perturbations in the groundwater flow pathways resulting from both the excavation and thermally induced loadings and unloadings occuring in the rock. Qualitative analysis of the significance of the rock mechanics results upon the groundwater flow is provided in this report whenever such an analysis can be performed. A quantitative assessment of the rock mechanics results as regards flow permeability perturbations is the subject of a subsequent report (2).

[x]Numbers in paranthesis refer to references at end of text.

# 2. ROCK MECHANICS MODELING

## 2.1 Material characterization

In an actual rock mass with thermal induced loadings, the material response is non-linear and involves time-dependent deformation and fracture, temperature- dependent thermal, mechanical and hydrological parameters, and intimate coupling of the thermal, mechanical and hydrological occurances. However, a solution of the total coupled system is difficult, and not practical at the current time. Therefore, in this study, the individual mathematical couplings are being treated separately in an effort to rationally assess the importance of each "coupling", as regards a practical assessment of the groundwater movements around a repository.

In the rock mechanics analysis, the rock properties have been assumed to be temperature and time independent and the interaction of the groundwater pressures within the joints is not considered.
A detailed discussion of the mechanical properties of granitic bedrock, including the expected temperature dependence, is provided in (3).

For the purposes of this analysis, the rock mass is assumed to be well characterized by a linear Mohr Coulomb strength failure criterion. Additionally, the rock is assumed to possess orthogonal joint planes or planes of weakness, failure of which is also of the Mohr Coulomb type. Details of the material characterization employed in this study have been previously described in (4) and (5).

For both the intact rock and the joints, any strength failure is followed by an assignment of a residual strength to the region of failure. For the joints, the residual strength is characterized by a reduced cohesion and internal angle of friction.

For the purposes of this study the strength failure of the rock mass has been assumed to occur on the joint planes or planes of weakness rather than within the intact rock mass. Previous studies (4 and 5) have shown that the assumption of failure along the joints is more conservative than failure within the intact rock.

## 2.2 Input Data

In order to utilize the material characterization previously described, certain rock mass properties must be selected and in situ conditions assumed. These parameters include rock density, Youngs Modulus, of elasticity, Poisson's ratio, and the coefficient of thermal expansion. In situ conditions which must be selected include the initial virgin stress state, joint orientation and the parameters for the Mohr-Coulomb failure criterion.

The numerical values of these parameters are presented in Table 1. As can be seen in the table all parameters are given only one numerical value with the exception of joint plane orientation which has been given two. Whereas the joints can be expected to be orthogonal and aligned vertically and horizontally, the inclined joint set orientation has been analyzed to illustrate the effects of deviation from vertical and horizontal jointing.

The ratio of virgin horizontal and vertical stresses has been taken to be constant with depth in this study. In fact, this is not the situation indicated in a variety of field investigations, as stated in (3). Generally, field investigations have shown that the horizontal stresses are greater than the vertical stresses at shallow depths. With increasing depth, the ratio of horizontal stress to vertical stress decreases. Therefore, the assumption of a constant ratio of the two stresses results in a conservative assumption due to the fact that horizontal confinement at shallow depths is less in the assumed model than is actually the case in the field.

## 2.3 Repository Modeling

As was the case in other reports within this study, the modeling of the rock mass containing the repository has been divided into two distinct areas viz; local or near field modeling and global or far field modeling. The local rock mechanics modeling is essential in determining the stress states in the near vicinity of the storage tunnels and canister drillholes and also in determining the extent of regions of rock which have experienced strength failure due to the excavation and thermal loadings.

The results of this type of modeling can be used directly in determining storage tunnel support requirements (4 and 5). However, in this study the local modeling is being performed for the explicit purpose of assessing the perturbations in the hydraulic permeabilities as a consequence of the excavation and thermomechanical stress states.

The global or far field rock mechanics analysis has been done in an effort to identify areas of potential fracture initiation, and to assess the changes in the hydraulic permeability due to induced thermal stresses.

Both the local and global models are discussed in more detail in further sections of this report, and the finite element meshs are presented in the Appendix. All analysis was performed assuming plane strain in the direction normal to the plane of the model.

## 3. LOCAL MODELING RESULTS

### 3.1 Model and Assumptions

The model used for the local rock mechanics analysis is presented in Figure 1. The model has been developed by assuming symmetry about vertical lines through the storage tunnel center and the pillar center. The upper and lower boundaries are positioned at such a distance so as to provide negligible influences upon the local deformations.

Two material types are actually utilized in the analysis of this model. In the region identified as the canister region, the coefficient of thermal expansion is given a zero value, whereas external to this region, the coefficient of thermal expansion is that value given in Table 1. This material differentiation is necessary due to the fact that the temperature fields utilized in determining the thermal loadings are taken directly from those presented in (6). In (6) the actual canister material is modeled in the region identified as the canister region; however, in the rock mechanics analysis, this region must by necessity be the rock situated between adjacent drillholes.

All local rock mechanics analyses were performed without modeling of any storage tunnel backfill material. In this regard, the local analyses is conservative since the backfill may possibly provide some confinement to the storage room periphery.

### 3.2 Pre-emplacment

The stress states and regions of strength failure which arise due to the excavation of the storage room are obtained by negating the normal stress and tangential shear stress on the storage tunnel periphery which existed prior to excavation. The stress states which then arise in excess of the assumed strength relationship are transfered to adjacent regions which have not yet experienced failure. This process is repeated until all stress states are within a certain tolerance of the residual or intact strength depending on whether the material has failed or not. This "stress-transfer" technique is discussed in (7) and utilized in (4) and (5).

The pre-emplacement analysis was performed for joint sets or planes of weakness at 0 and 90 degrees, and joint-sets at 45 and -45 degrees. Figures 2 and 3 present the regions which have experienced strength failure for each of the respective joint orientations. As can be seen in the figures the two cases do not have a significant amount of "failed region" in common. Therefore, one can conclude that the joint set orientation is an important parameter in the assessment of the pre-emplacement rock mechanics. Storage tunnel support requirements are also highly dependent upon the joint set orientation. Storage room support may be adequately obtained with such measures as shotcret in the case of the vertical and horizontal joints, whereas the inclined joint set may require rock bolting.

The principle stresses due to the in situ virgin stress and the stresses due to excavation are illustrated in Figures 4 and 5, for both of the respective joint orientations. The resulting stress states for the two orientations are quite similar. Even though the principle stress states are similar, the regions of strength failure differ due to the orientation of the joint sets.

These stress states will be used in subsequent studies to predict the changes in hydraulic permeability caused by the excavation-induced stress changes.

## 3.3 Post-Emplacement

The post-emplacement stress states and regions of strength failure are determined from both the excavation loadings and the thermal loadings resulting from the radiogenic heat disssipation. In other words, the stress states along with the failed regions from the pre-emplacement analysis are the initial conditions for the calculation of total stresses resulting from the superposition of the incremental thermal loadings. In this regard, the post-emplacement rock mechanics analysis is "quasi-static" and only includes the time dimension implicitly since the incremental thermal loadings represent the various temperatures resulting in time In particular, the thermal loadings are imposed upon the rock mechanics model as temperatures representative of 0.1, 1, 4, 10, 40, 100 and 1000 years after waste emplacement.

As was the case with the calculation of the excavation stresses, the total stresses resulting from the addition of thermal loading are checked against the strength failure criterion and stress states in excess of the strength are transfered to adjacent locations which have not yet experienced failure. Through this procedure, the progressive strength failure around the storage tunnel can be examined.

Figures 6 and 7 illustrate the progressive strength failure around a storage tunnel for the 0 and 90 degree joint sets and the 45 and -45 degree joint sets, respectively. For both joint set orientations, the additional strength failure due to the thermal loading is not as great as the area of failure due to excavation.

In addition to observing the areal extent of the strength failure one should also note the locations of the strength failure. In the case of the vertical and horizontal joint sets, the strength failure arising from the addition of thermomechanical stress is in the region of the tunnel floor and rib-floor intersection. In the case of the inclined joint sets, the strength failure in excess of that due to excavation occurs for the most part above the tunnel springline. In general, this behavior can be mainly attributed to the shape of the storage tunnel cross section. In other words, the strength failure occurs in regions where the tunnel periphery (and subsequently, principle stresses) are aligned with the joint set orientation. Thus, the geometry of the storage tunnel should be considered once the in situ joint characterization has been established in order to control the strength failure in locations where it may be undesireable. Since the regions near the tunnel periphery which undergo strength failure will undergo dilatation, the hydraulic permeability in these regions could be expected to be much greater than that in the unfailed regions. The porosity could also be expected to be greater.

Figures 8, 9 and 10 illustrate the horizontal and vertical stresses in the roof, rib and floor. The in situ, post-excavation and total stresses after 40 years without ventilation are displayed in the figures. From observation of these figures, qualitative observations of hydraulic permeability changes can be made. Specifically, in the roof, the horizontal permeability near the tunnel will increase due to the low vertical stress and the vertical permeability will decrease due to the increased horizontal stress. The converse is true at the pillar rib. In the region of the floor, the vertical permeability will probably not decease as much as will be the case in the roof. The fact that the horizontal stress increase in the floor is not as great as in the roof can again be attributed to the cross sectional shape of the storage tunnel.

## 3.4 Ventilation Effects

In the previous discussion of post-emplacement stress states and strength failure, the thermal loading was extracted from (6) for the case of no ventilation of the storage tunnels. In this section of the report the thermal loading will be that which is presented in (6) for the case of 30 year ventilation.

In analyzing ventilation from a rock mechanics viewpoint, the primary concern is with the associated strength failure for both the ventilated and unventilated cases. In the present study, the concern in analyzing both ventilated cases and unventilated cases is identical. The regions experiencing strength failure are important in accessing hydraulic permeability changes. Therefore, it is important to identify whether the lack of ventilation results in excessive strength failure or whether the "double thermal cycle" (6) resulting from a finite period of ventilation is, in fact, more detrimental to the tunnel periphery.

Figures 11 and 12 illustrate the progressive strength failure which occurs with 30 years of ventilation for the 0 and 90 degree joint sets and the 45 and -45 degree joint sets. The general features displayed in these figures are similar to those shown in Figures 6 and 7; however, the areal extent of the regions experiencing failure is less with ventilation than without. This is particularly the case with the inclined joint set wherein, the roof experiences considerably less strength failure with ventilation than without. In the case of the horizontal and vertical joint sets, the effect of ventilation is not nearly as pronounced.

## 3.5 Practical Considerations

In assessing the local response of a rock mass to excavation and thermally-induced loading, physical situations and conditions which are not always amenable to mathematical modeling must be realized when analyzing the results of a mathematical model. For example, rock damage due to excavation methodology is not taken into account in this present study. Additionally, dissimilarity of joint response and strength is not accounted for. Rather, each joint plane is assumed to repond to load in exactly the same manner as each of the other joints and also to have the same strength characteristics.

The exclusion of the later two physical situations (blasting damage and joint set characterization) in the mathematical modeling is natural when site specific properties and conditions are not available. When these properties and conditions do become available, the analysis must be examined and performed again if the investigated physical situations are not within the bounds of the previous modeling.

# 4. GLOBAL MODELING RESULTS

## 4.1 Model and Assumptions

Three distinct models were examined for the global rock mechanics analysis. These models are illustrated in Figure 13. Models A and B represent symmetric half sections of repositories situated at 500 meters deep and 1000 meters deep, respectively. The thermal loading for these two models is calculated with the temperatures resulting from instantaneous waste emplacement (6). The temperatures resulting from a 30 year linear waste emplacement were utilized in determining the thermal loading for model C.

Each of the models is homogeneous and the detail of storage room excavation is not provided in the analysis due to the large extent of the models.

## 4.2 Pre-Emplacement

Since the repository storage tunnel excavation is not modeled in the global analysis, the pre-emplacement stress states are merely the in situ stress states. Determination of the in situ stress state is, however, not necessarily straightforward. Results of several field investigations are presented in (3) which show a tendency for the horizontal stresses to be greater than the vertical for shallow depths and approximately equal to the vertical at much greater depth. The vertical stress is generally assumed to be that which results from assuming a gravitating medium (density times depth). In this analyses the vertical in situ stress is taken to be density times depth and the horizontal in situ stress is assumed to be a constant multiple (=2.0) times the vertical.

## 4.3 Post-Emplacement

The post-emplacement stress states and regions of strength failure in the global analysis are determined from the superposition of the in situ stresses and those resulting from the respective thermal loadings. As was the case with the local rock mechanics analysis, the time dimension is only implicitly incorporated in the analysis by virtue of the transient thermal loading. In the case of the global analysis, the thermal loadings are imposed upon the rock mechanics model as temperatures representative of 1, 10, 40, 70, 100 and 1000 years after emplacement. Analysis of Model C also involved the thermal loading represented by the temperature field at 5000 years after emplacement. This temperature distribution was calculated using the analytical representation of the heat generating function from (6) for times greater than 1000 years.

In four of the six cases analyzed (3 models x 2 joint sets), minor strength failure occurred at the ground surface above the repository. Model B did not experience any strength failure. However, this strength failure is not of a great concern since the confining horizontal stress which can be expected to exist in the Fennoscandinavian Shield (3) was not modeled. Rather, the ratio of in situ horizontal to in situ vertical stress is taken to be constant at all depths. If the confining horizontal stress were incorporated into the models, no strength failure would be exhibited in any of the global models. Therefore, in a practical sense, the global response of the rock mass to the radiogenic heat dissipation and associated thermal loading is elastic and reversible as far as rock mechanics is concerned.

Models A and C exhibited very similar response. The difference in the thermal loading associated with instantaneous waste emplacement and linear waste emplacement (6) is not significant in the global rock mechanics analysis for the gross thermal loading being considered in Sweden. The upward displacement across the earth's surface at various times for these two models is displayed in Figure 14. The transient displacement above the repository centerline is illustrated in Figure 15. The upward displacement depicted in these figures is not particularly significant. This is particularly true when one notes that the rate of surface uplift due to the rock expansion is about 0.3 mm/yr for the first 100 years and about 0.05 mm/yr from 100 to 1000 years.

These uplift rates can be compared to a current rate of ground surface uplift of 8 mm/year near Luleå and Piteå and 4mm/year near Stockholm (8).

Since the rock mass exhibits no strength failure, one can assume the stress increases to be relatively small. The maximum thermoelastic stresses exhibited in the 500 meter repository simulations are presented in Figure 16 as a function of depth. The thermoelastic stress increases in the 1000 meter repository model are similar to those in the 500 meter model. The displacements at the ground surface have a smoother variation across the ground surface due to a more uniform heating of the rock mass. The maximum uplift for the 1000 meter model is approximately 5 cm.

## 4.4 Practical Considerations

As was the case with the local analysis and excavation modeling, physical situations and conditions in the global repository domain are not amenable to modeling or prediction. In particular, major structural discontinuities which are site specific cannot be modeled without detailed knowledge of the site subsurface. Additionally, when one reports predictions for time domains of thousands of years, the boundary conditions will be somewhat altered in a relatively unpredictable manner.

Therefore, the global rock mechanics analysis can only be treated as an approximation to the mechanical response of the rock to the thermal loading and not as an exact representation of the rock mass after tens of thousands of years.

## 5. SUMMARY AND CONCLUSIONS

Non-linear rock mechanics calculations have been completed for the repository storage tunnels and the global repository domain. The rock mass has been assumed to possess orthoganol joint sets or planes of weakness with finite strength characteristics.

In the local analyses of the repository storage tunnels the effects of joint orientation and repository ventilation have been examined. The local analyses indicated that storage room support requirements and regions of strength failure are highly dependent upon joint orientation. The addition of storage tunnel ventilation was noted to reduce regions of strength failure, particularly during the 30 year operational phase of the repository. Examination of the local stresses around the storage tunnels indicated the potential for perturbed hydraulic permeabilities. The permeabilities can be expected to be altered to a greater degree by the stresses resulting from excavation than from stresses which are thermally induced.

The global rock mechanics did not incorporate the excavation of the storage tunnels due to the large areal extent of the models. Subsequently, due to the relatively small thermal stresses which develop with the low gross thermal loading being considered in Sweden, the global models exhibited a reversible elastic response to the radiogenic heat disipation. The thermal loading provided by the instantaneous waste emplacement resulted in stress states and displacements quite similar to those provided by the linear waste emplacement sequence.

The stress states observed in the local and global rock mechanics models will be used in subsequent studies to quantitatively analyse the perturbations of the hydraulic permeabilities.

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TABLE 1

ROCK MASS PROPERTIES AND

In Situ Conditions

| CASE No. | $\rho$ $(Kg/m^3)$ | $E^1$ $(Kg/cm^2)$ | $\nu$ | $\alpha$ $(^oC^{-1})$ | $\beta^2$ | $C_R^{1,3}$ $(Kg/cm^2)$ | $C_J^{1,4}$ $(Kg/cm^2)$ | $\phi_R^5$ | $\phi_J^6$ | $K_o^7$ |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2700 | $.214(10^6)$ $(2.1(10^4))$ | .25 | $8(10^{-6})$ | $0^o$ & $90^o$ | 10.2 (1) | 10.2 (1) | $34^o$ | $34^o$ | 2 |
| 2 | 2700 | $.214(10^6)$ $(2.1(10^4))$ | .25 | $8(10^{-6})$ | $45^o$ & $-45^o$ | 10.2 (1) | 10.2 (1) | $34^o$ | $34^o$ | 2 |

Notes: 1. Numbers in parenthesis are MPa.

2. Angles of joint set orientations
3. Cohesion or direct shear strength of intact rock
4. Cohesion or direct shear strength of joint (Residual strength = 3.3 $Kg/cm^2$ (.3 MPa))
5. Internal angle of friction for intact rock
6. Internal angle of friction for joint (Residual = $30^o$)
7. Ratio of in situ horizontal stress to vertical stress

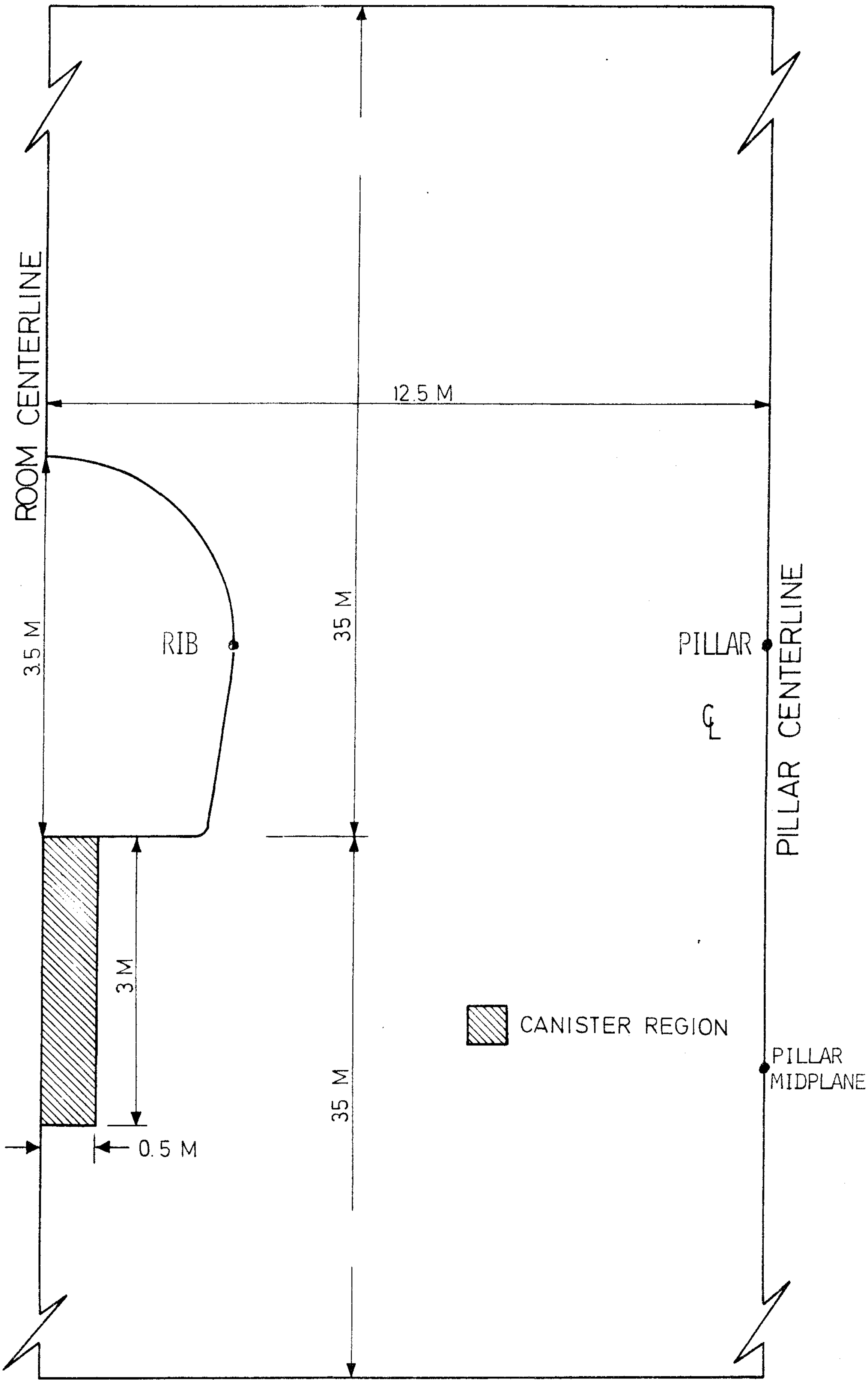



FIGURE 1. LOCAL ROCK MECHANICS MODEL

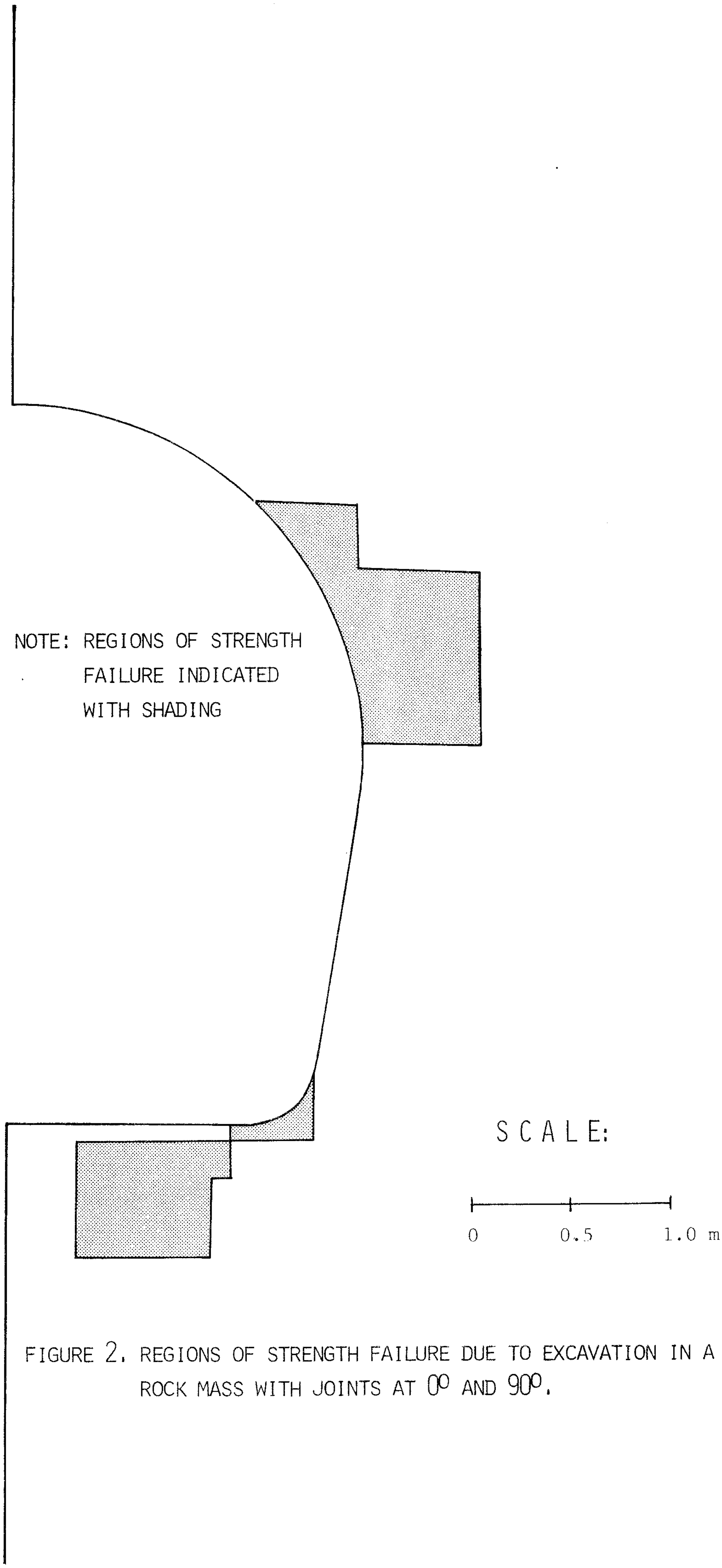



FIGURE 2. REGIONS OF STRENGTH FAILURE DUE TO EXCAVATION IN A ROCK MASS WITH JOINTS AT 0° AND 90°.

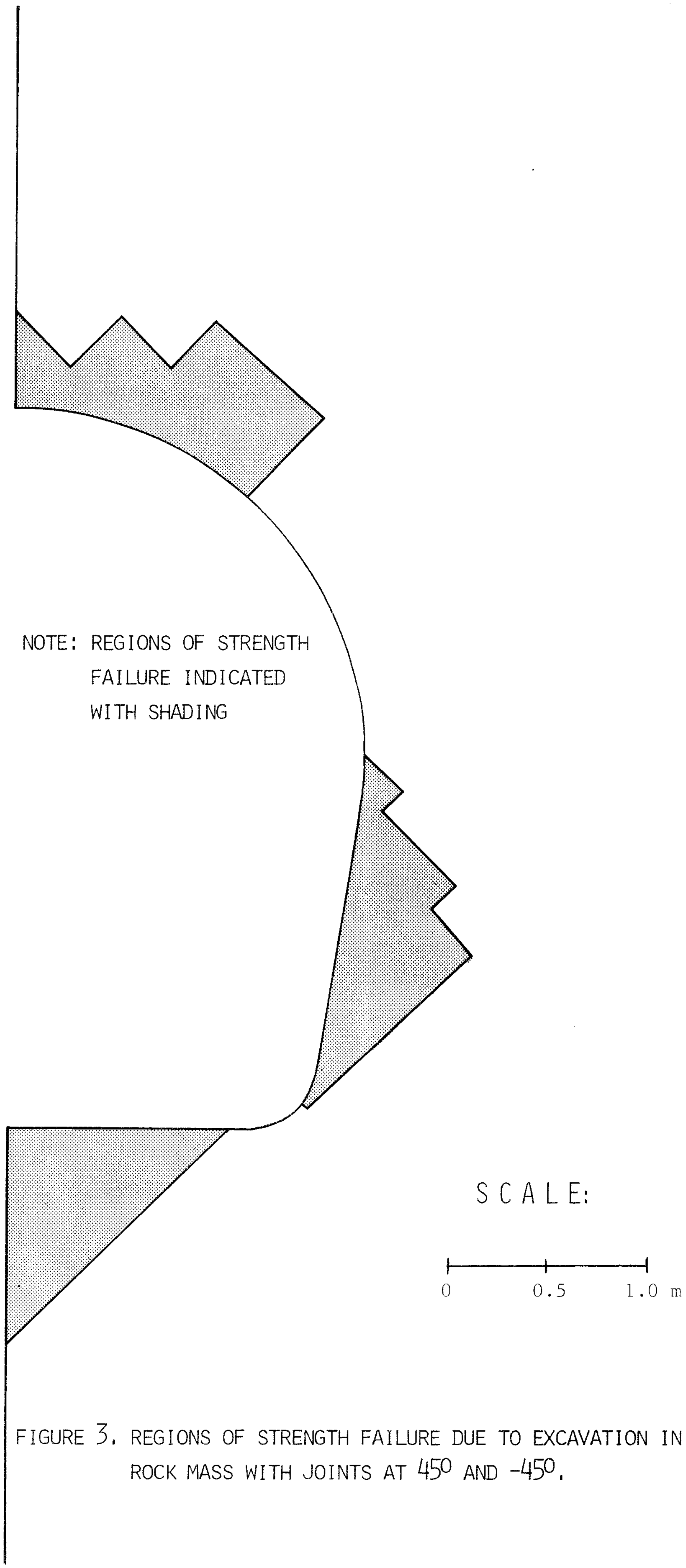



FIGURE 3. REGIONS OF STRENGTH FAILURE DUE TO EXCAVATION IN A ROCK MASS WITH JOINTS AT 45° AND -45°.

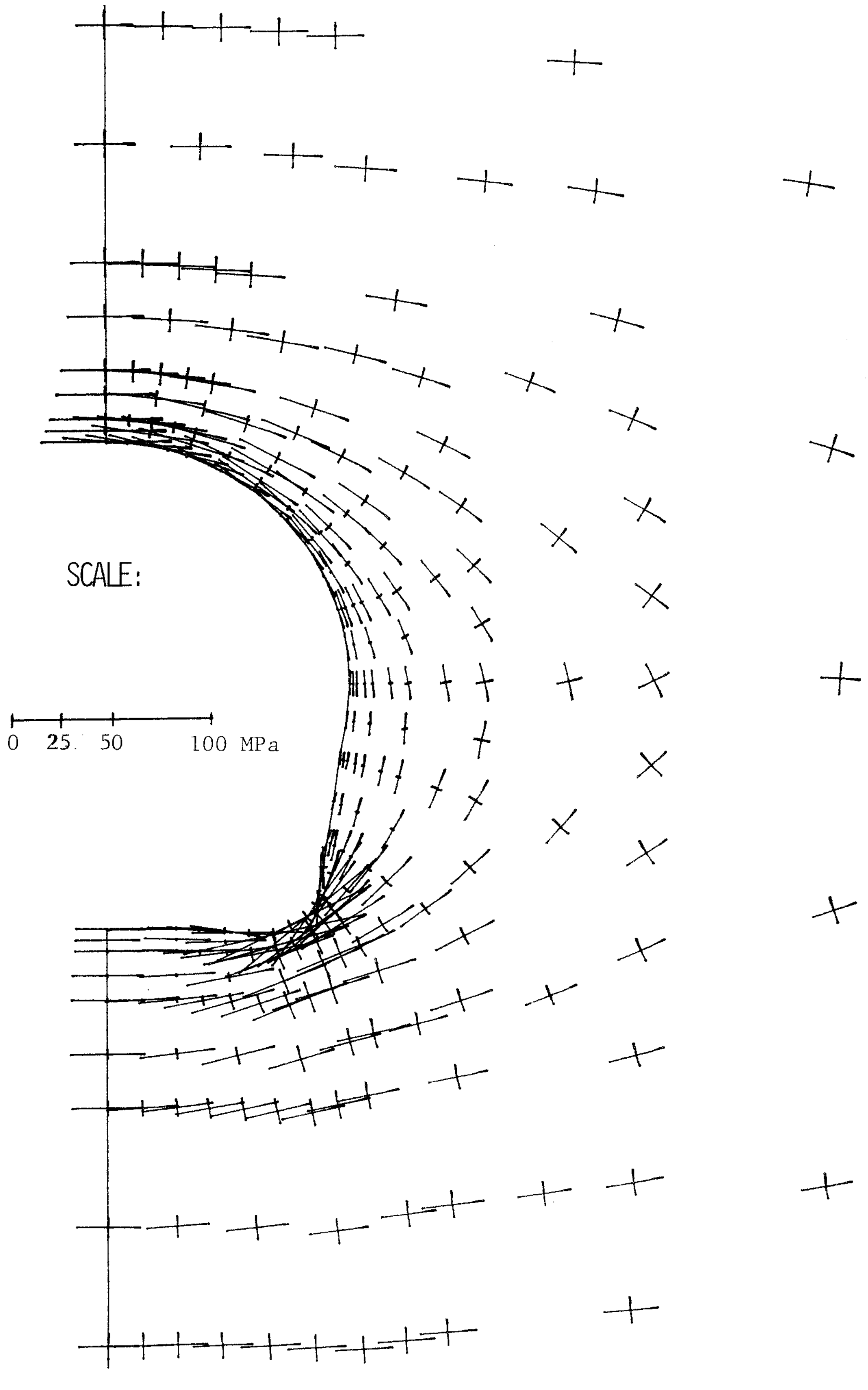



FIGURE 4. PRINCIPLE STRESSES RESULTING FROM EXCAVATION IN A ROCK MASS WITH $K_o = 2$ AND JOINTS AT $0^o$ AND $90^o$.

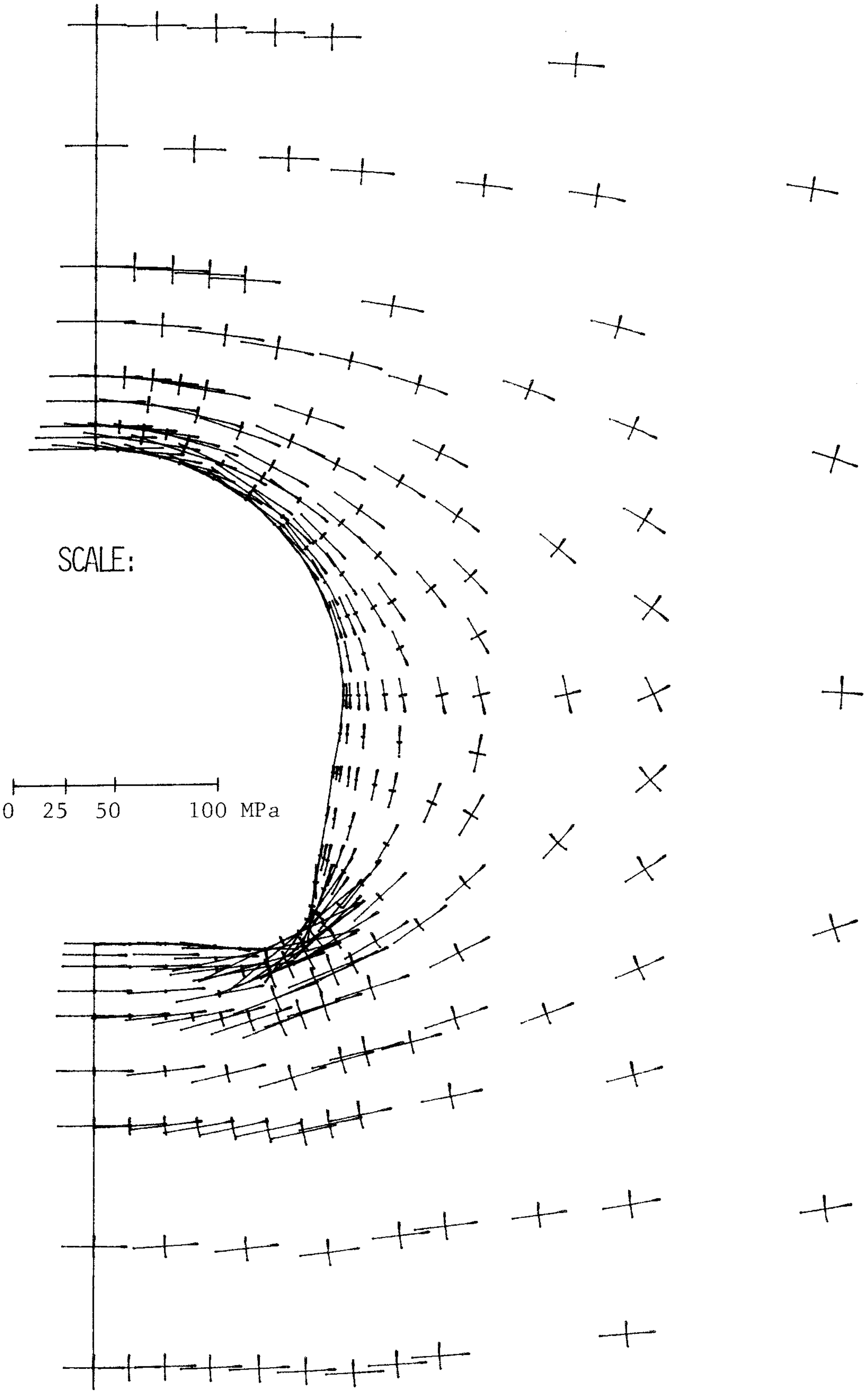



FIGURE 5. PRINCIPLE STRESSES RESULTING FROM EXCAVATION IN A ROCK MASS WITH $K_0 = 2$ AND JOINTS AT $45^0$ AND $-45^0$.

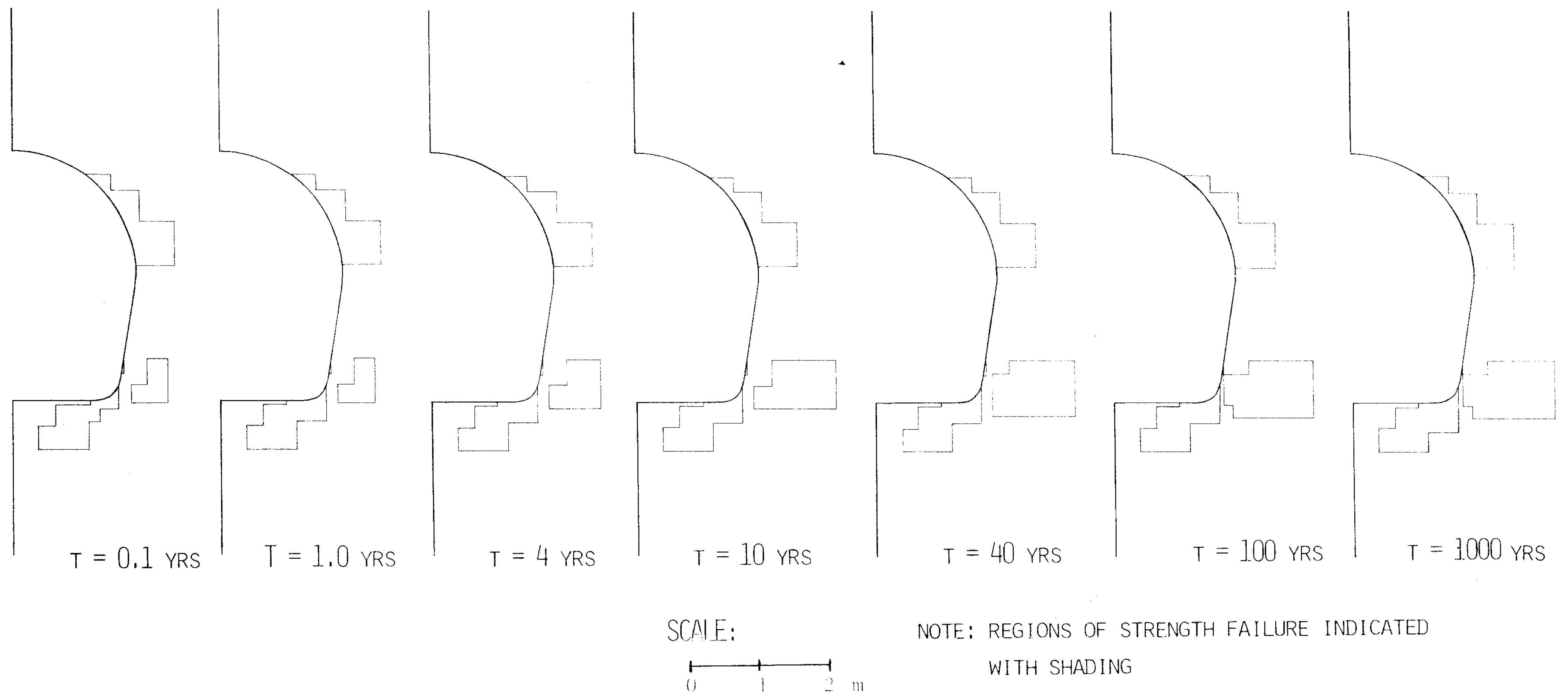



FIGURE 6. PROGRESSIVE STRENGTH FAILURE DUE TO EXCAVATION AND THERMOMECHANICAL STRESSES WITH JOINTS AT 0° AND 90°.

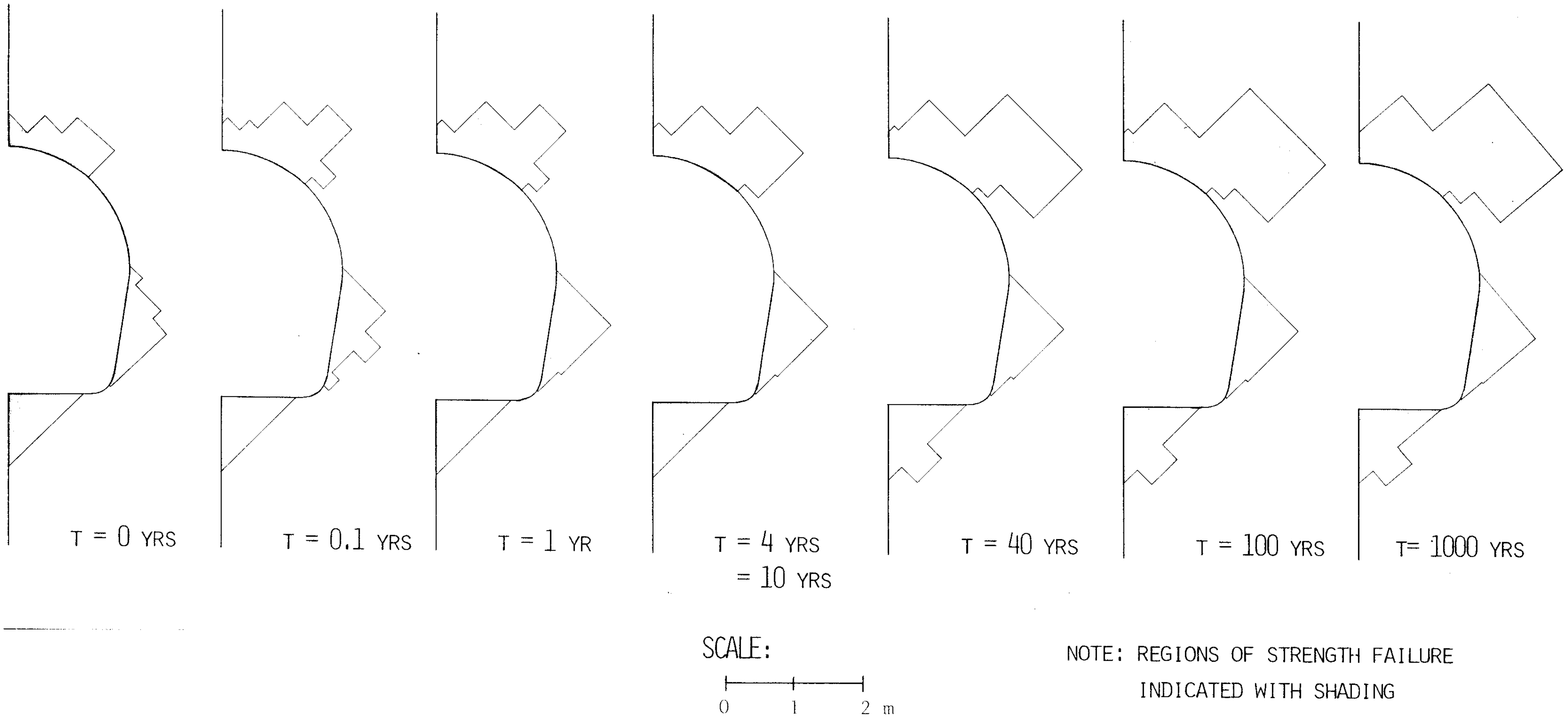



FIGURE 7. PROGRESSIVE STRENGTH FAILURE DUE TO EXCAVATION AND THERMOMECHANICAL STRESSES WITH JOINTS AT $45^{0}$ AND $-45^{0}$.

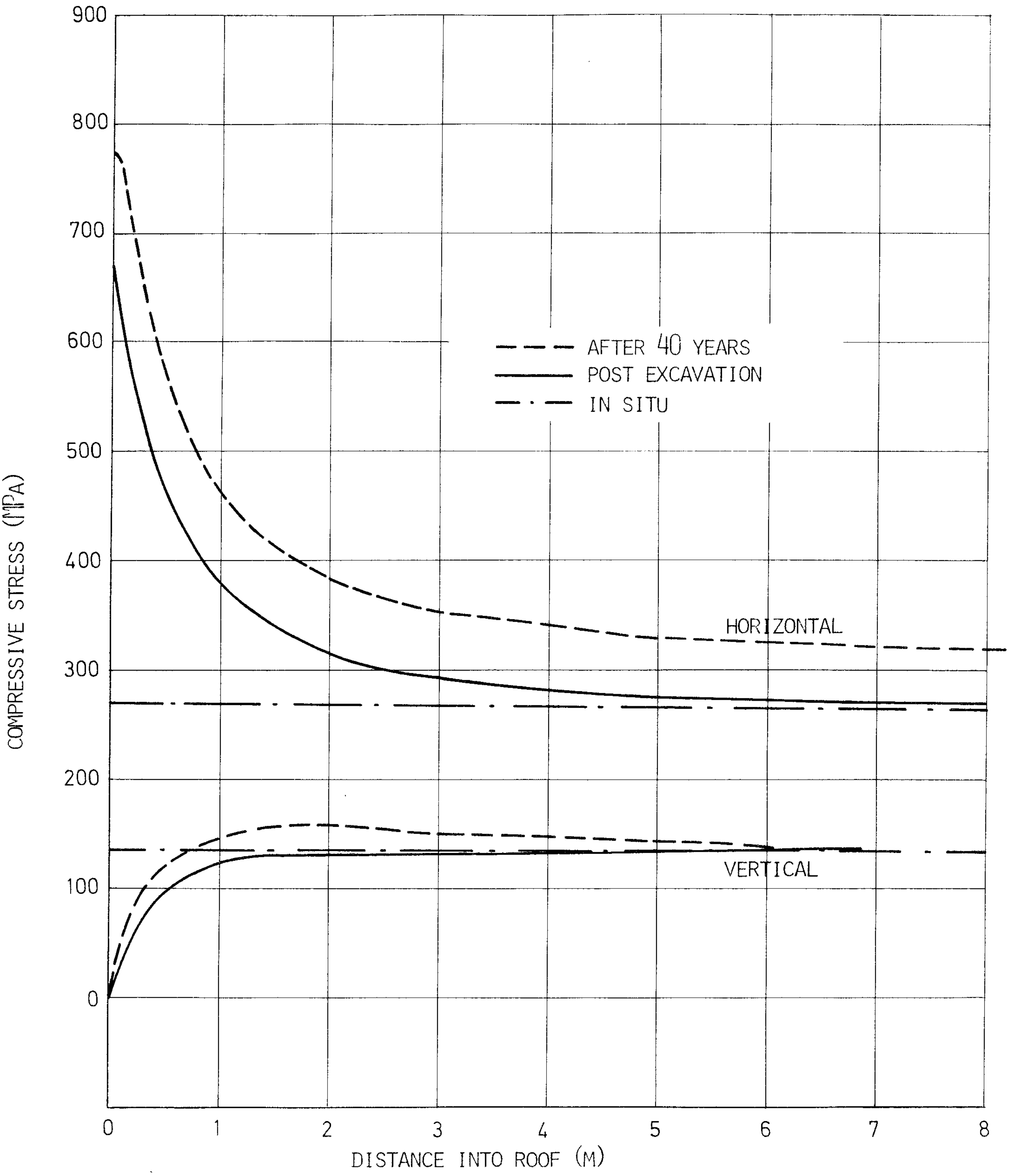



FIGURE 8. IN SITU, POST-EXCAVATION AND THERMOMECHANICAL STRESSES IN THE STORAGE TUNNEL ROOF.

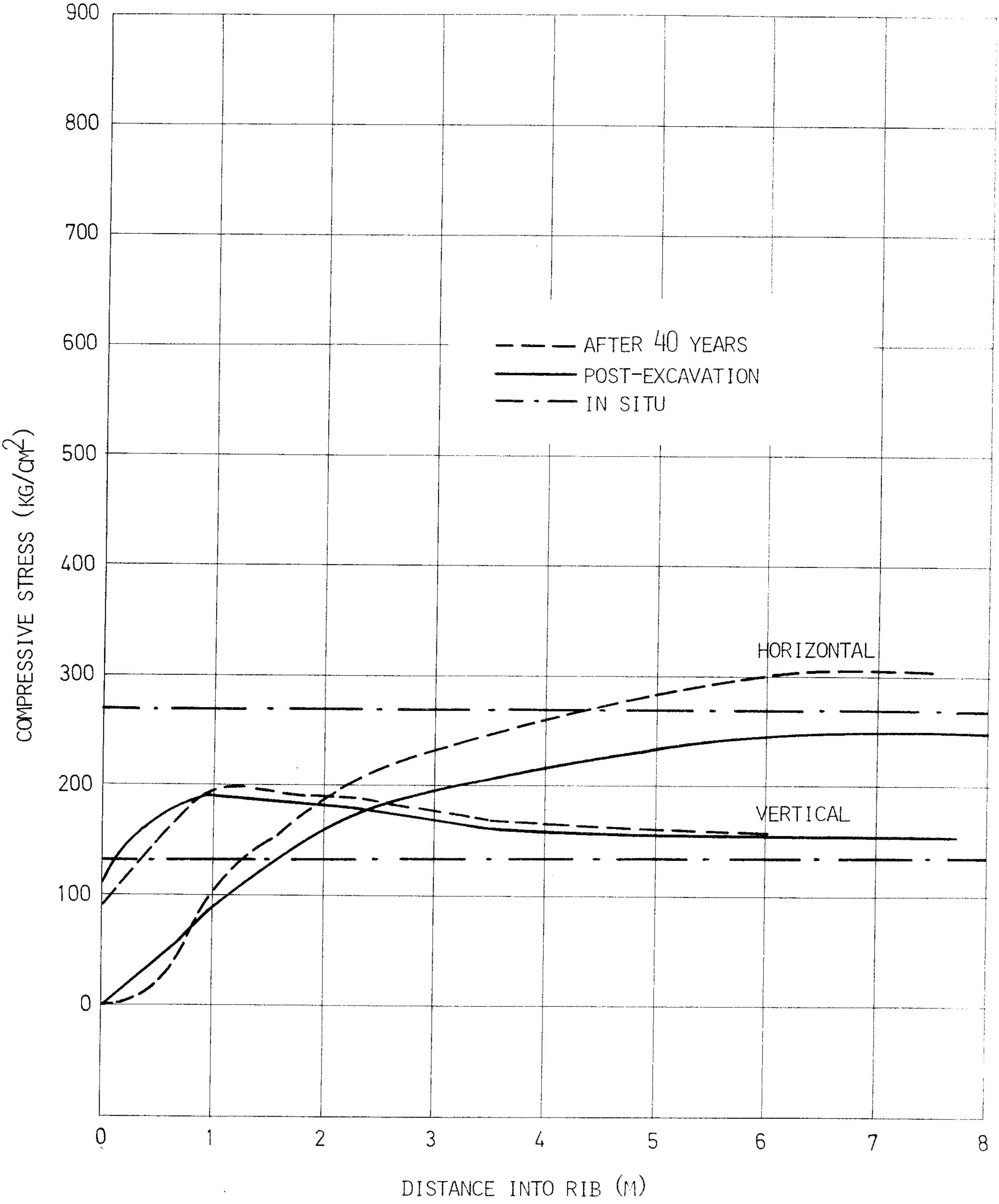



FIGURE 9. IN SITU, POST-EXCAVATION AND THERMOMECHANICAL STRESSES IN THE STORAGE TUNNEL RIB.

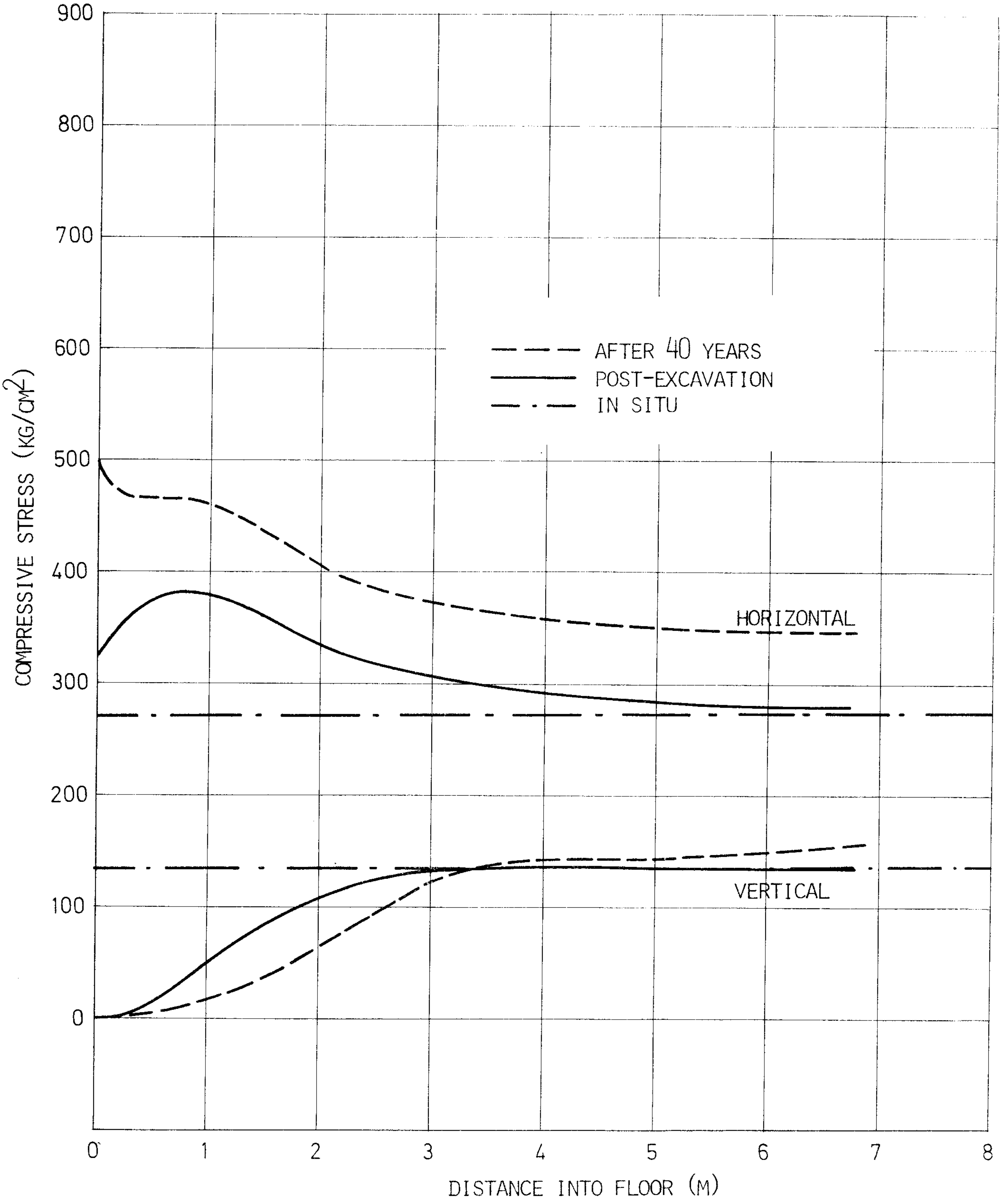



FIGURE 10. IN SITU, POST-EXCAVATION AND THERMOMECHANICAL STRESSES IN THE STORAGE TUNNEL FLOOR.

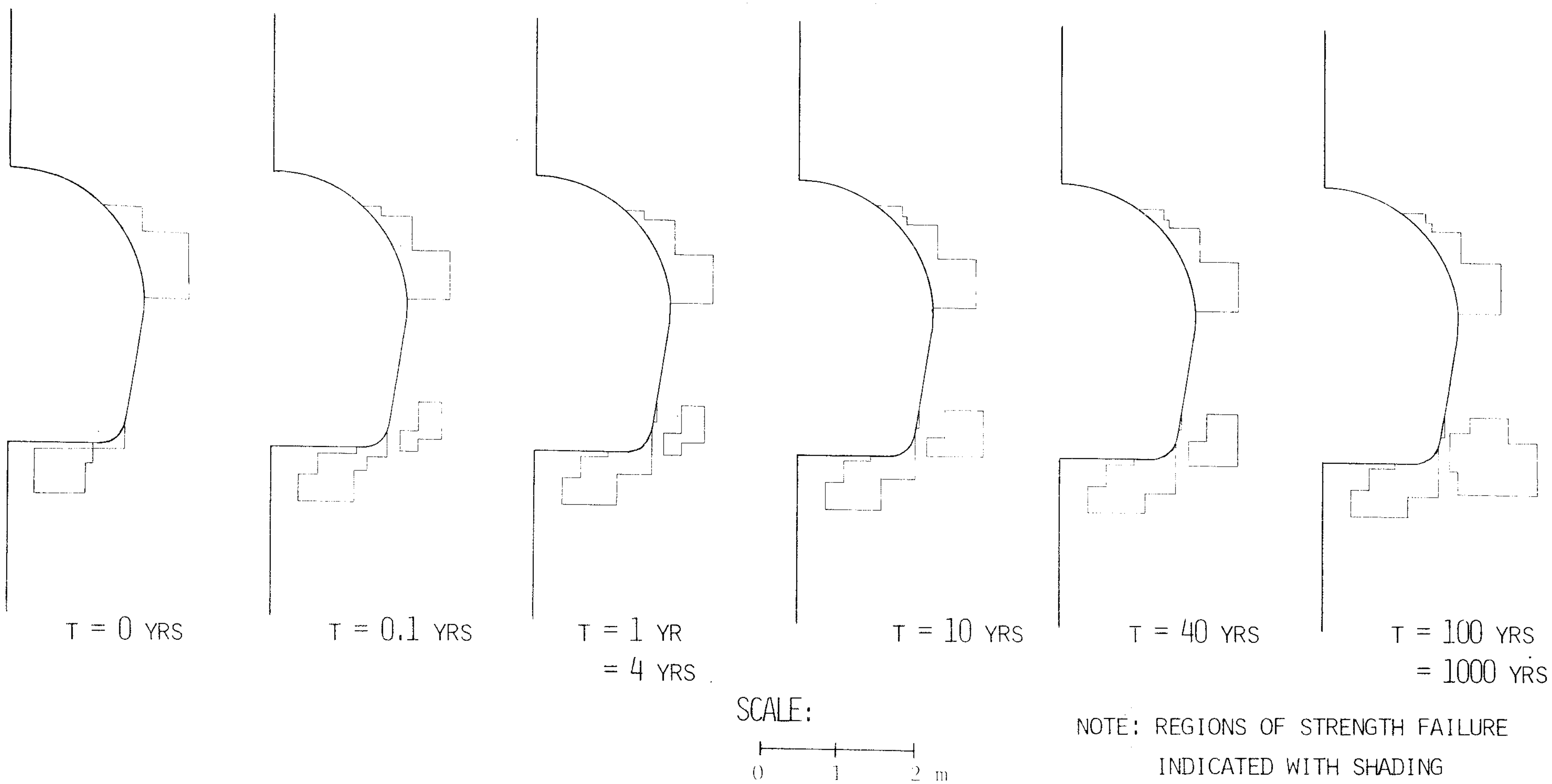



FIGURE 11. PROGRESSIVE STRENGTH FAILURE DUE TO EXCAVATION AND THERMOMECHANICAL STRESSES WITH JOINTS AT 0° AND 90° WITH 30 YEARS OF TUNNEL VENTILATION.

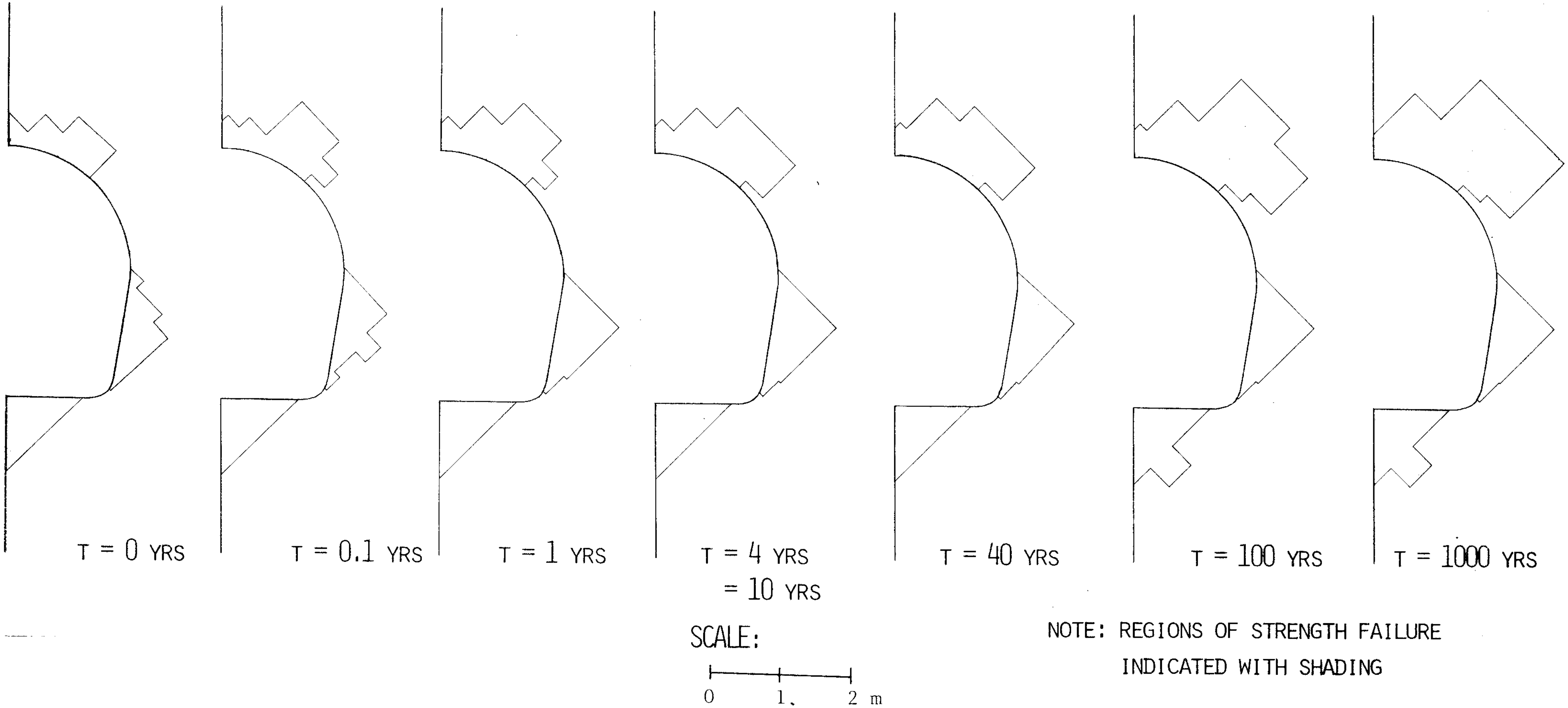



FIGURE 12. PROGRESSIVE STRENGTH FAILURE DUE TO EXCAVATION AND THERMOMECHANICAL STRESSES WITH JOINTS AT $45^{0}$ AND $-45^{0}$ WITH 30 YEARS OF TUNNEL VENTILATION.

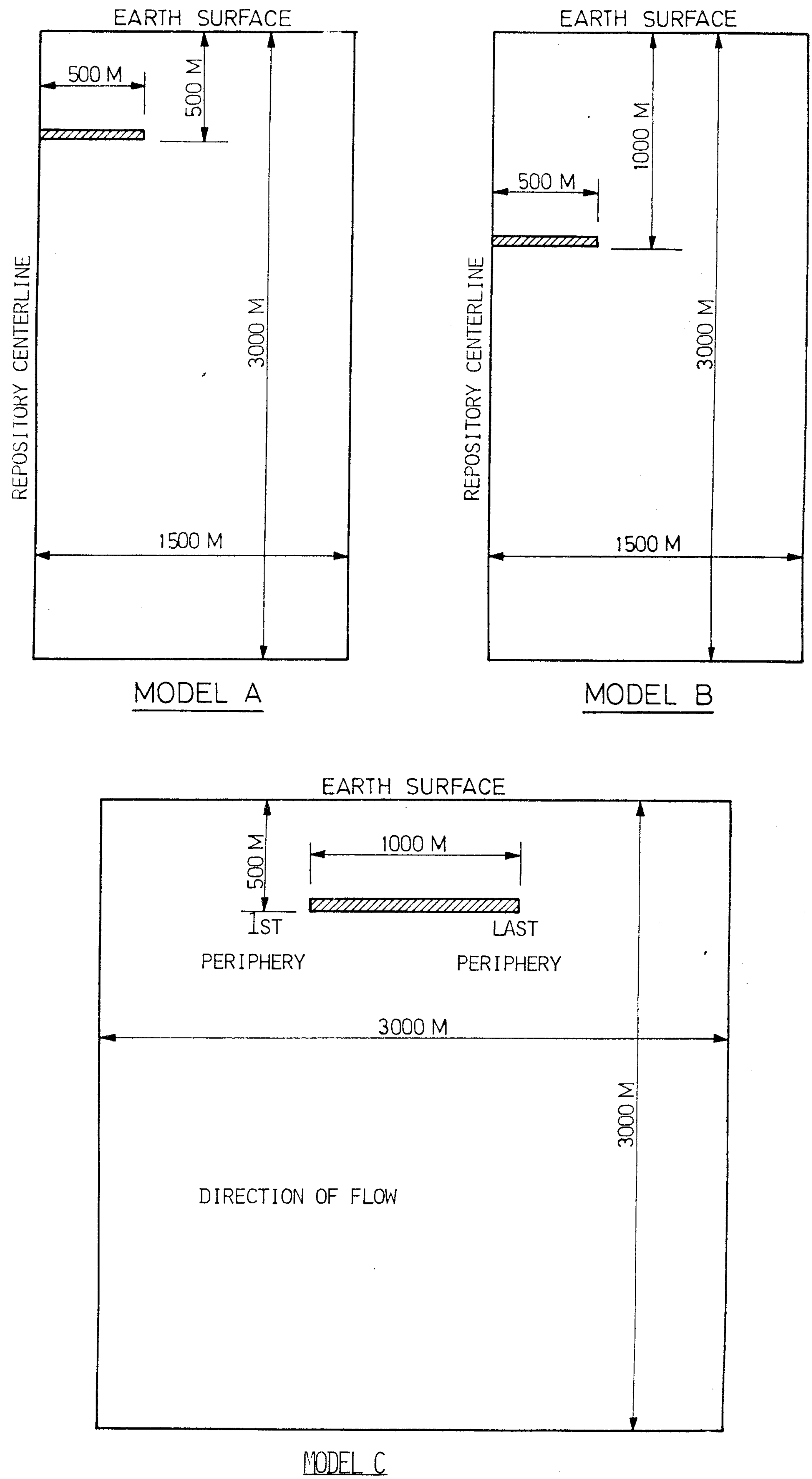



FIGURE 13. GLOBAL ROCK MECHANICS MODELS

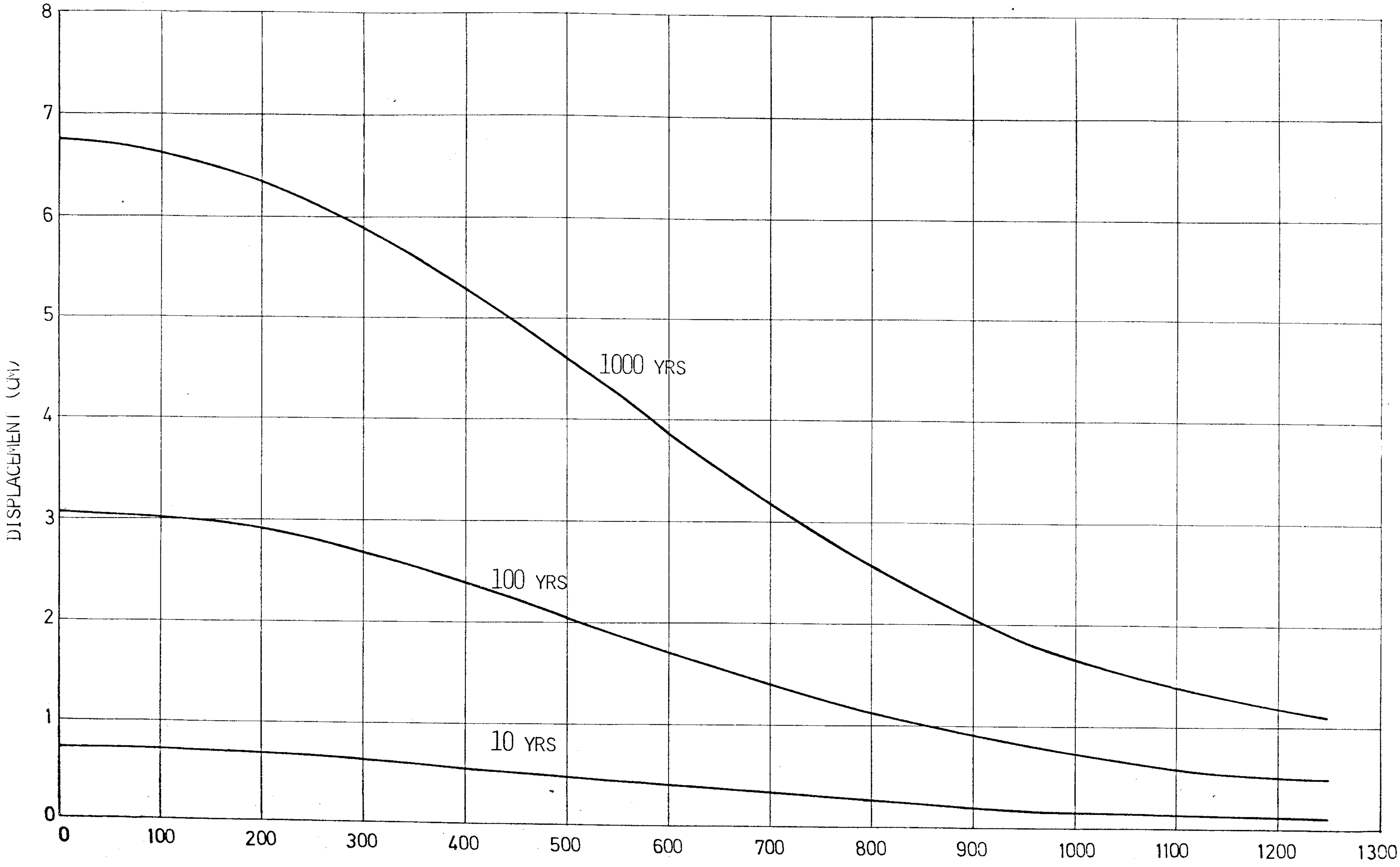



FIGURE 14. DISPLACEMENT ACROSS THE EARTH'S SURFACE DUE TO ROCK MASS EXPANSION.

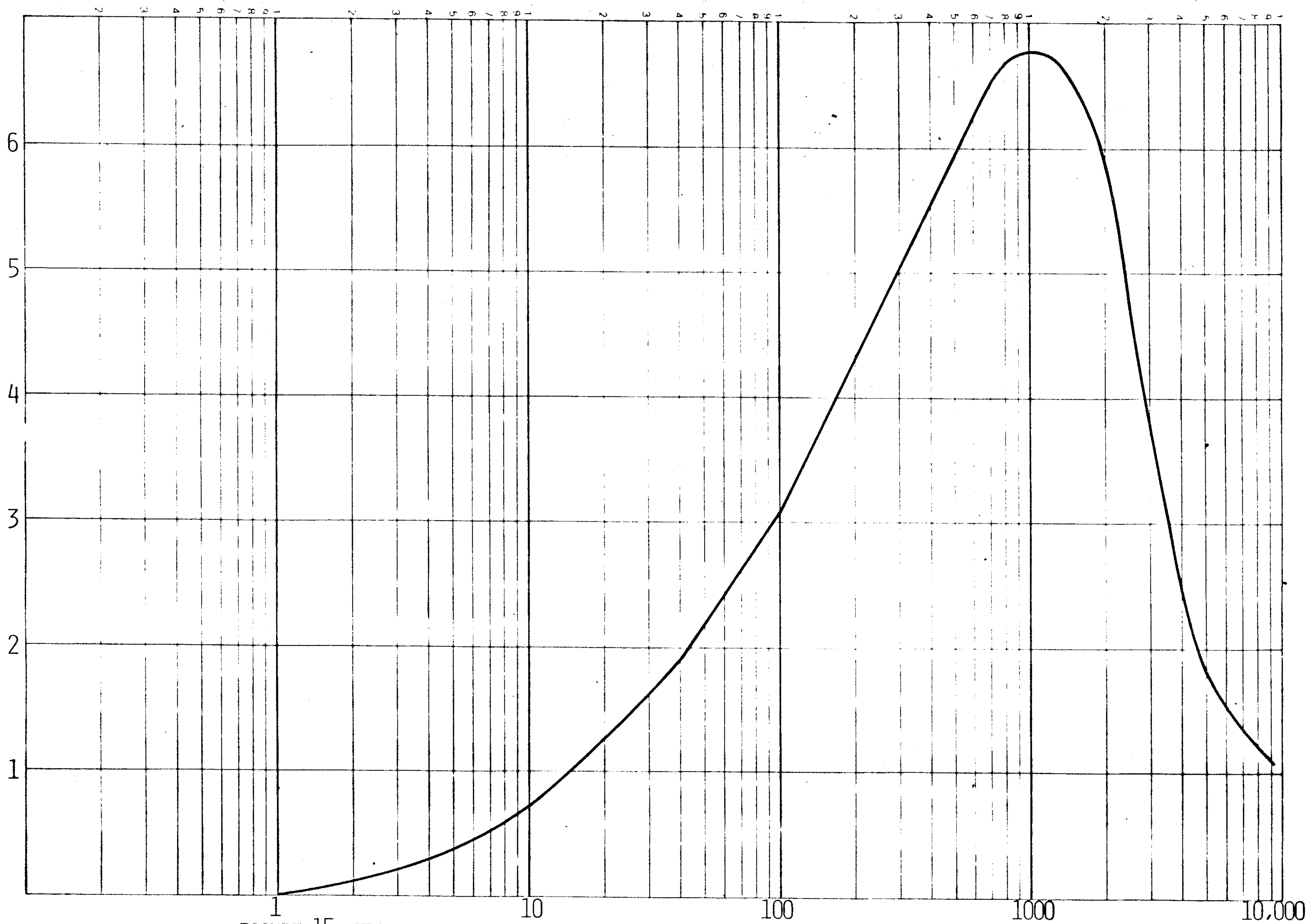


FIGURE 15. TRANSIENT DISPLACEMENT ABOVE THE REPOSITORY CENTERLINE

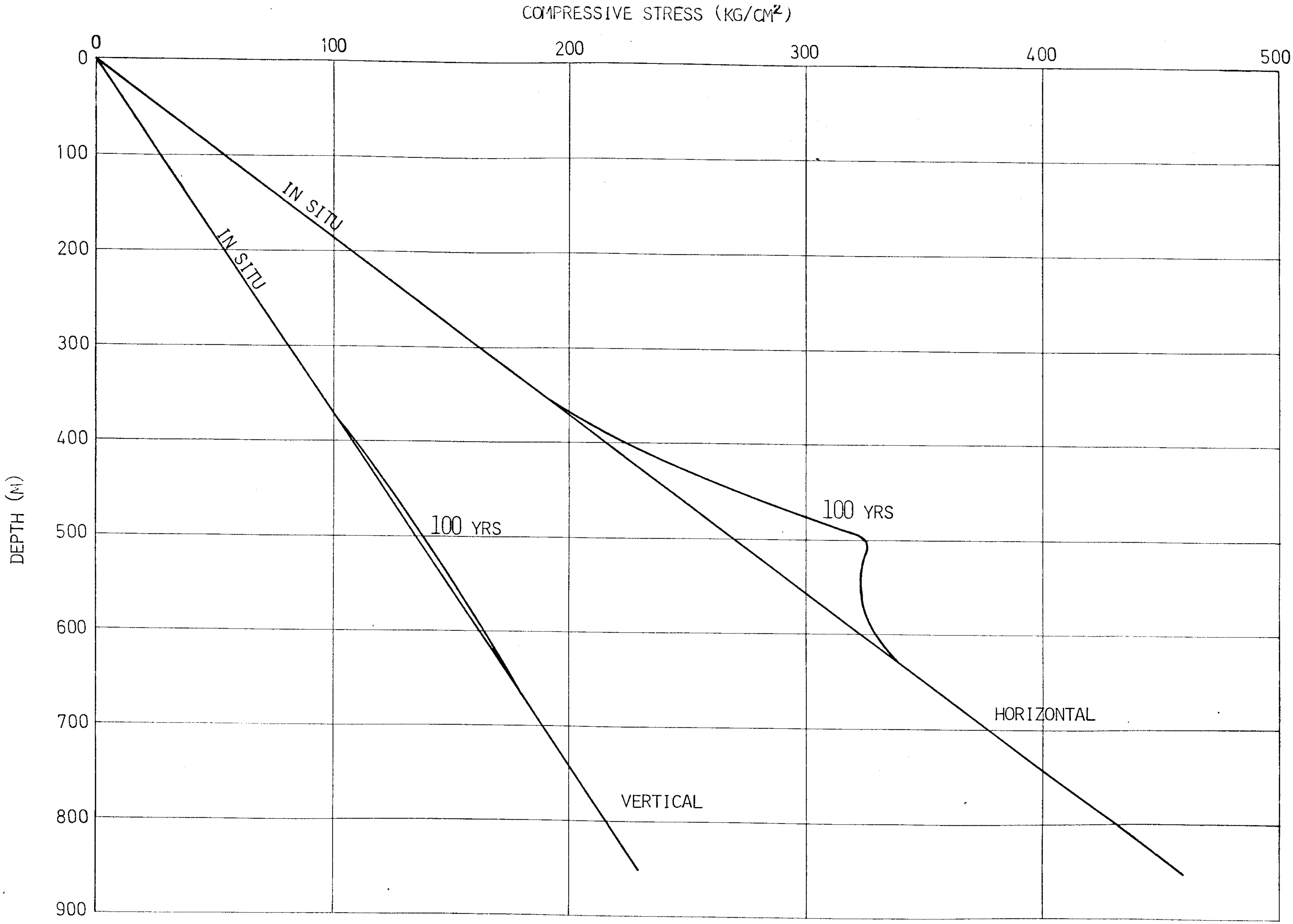



FIGURE 16. HORIZONTAL IN SITU AND THERMOMECHANICAL STRESSES THROUGH THE REPOSITORY CENTERLINE

# A P P E N D I X

## FINITE ELEMENT MODELS

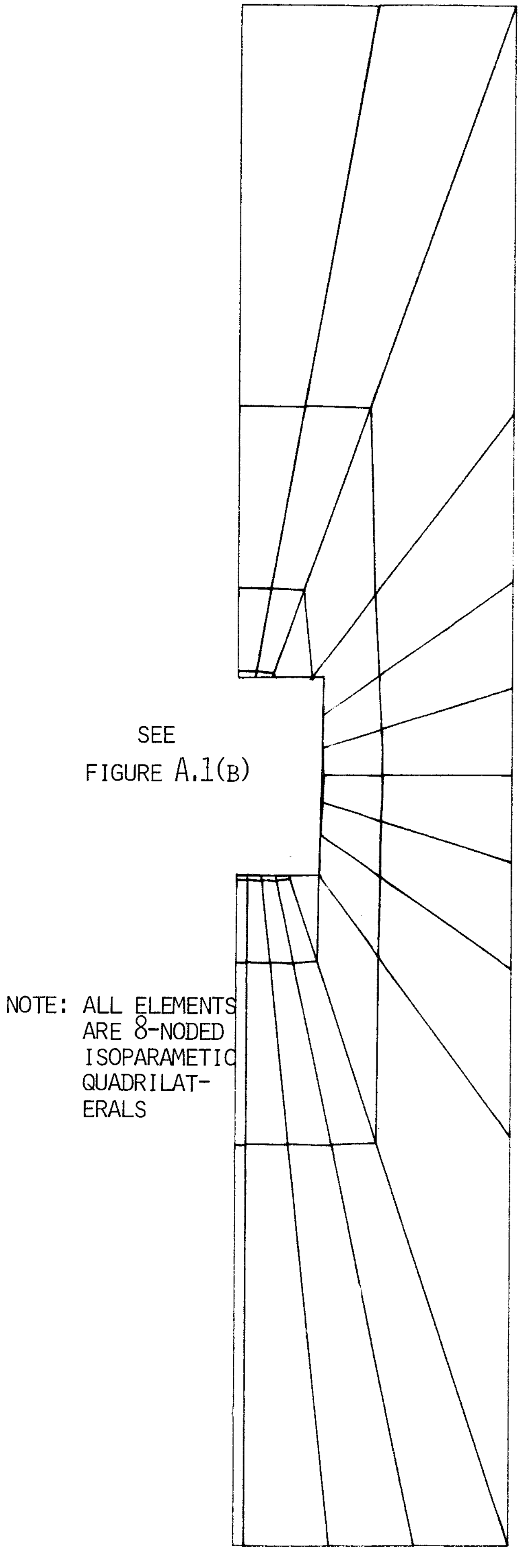



FIGURE A.1(A) FINITE ELEMENT MODEL FOR LOCAL ROCK MECHANICS ANALYSIS

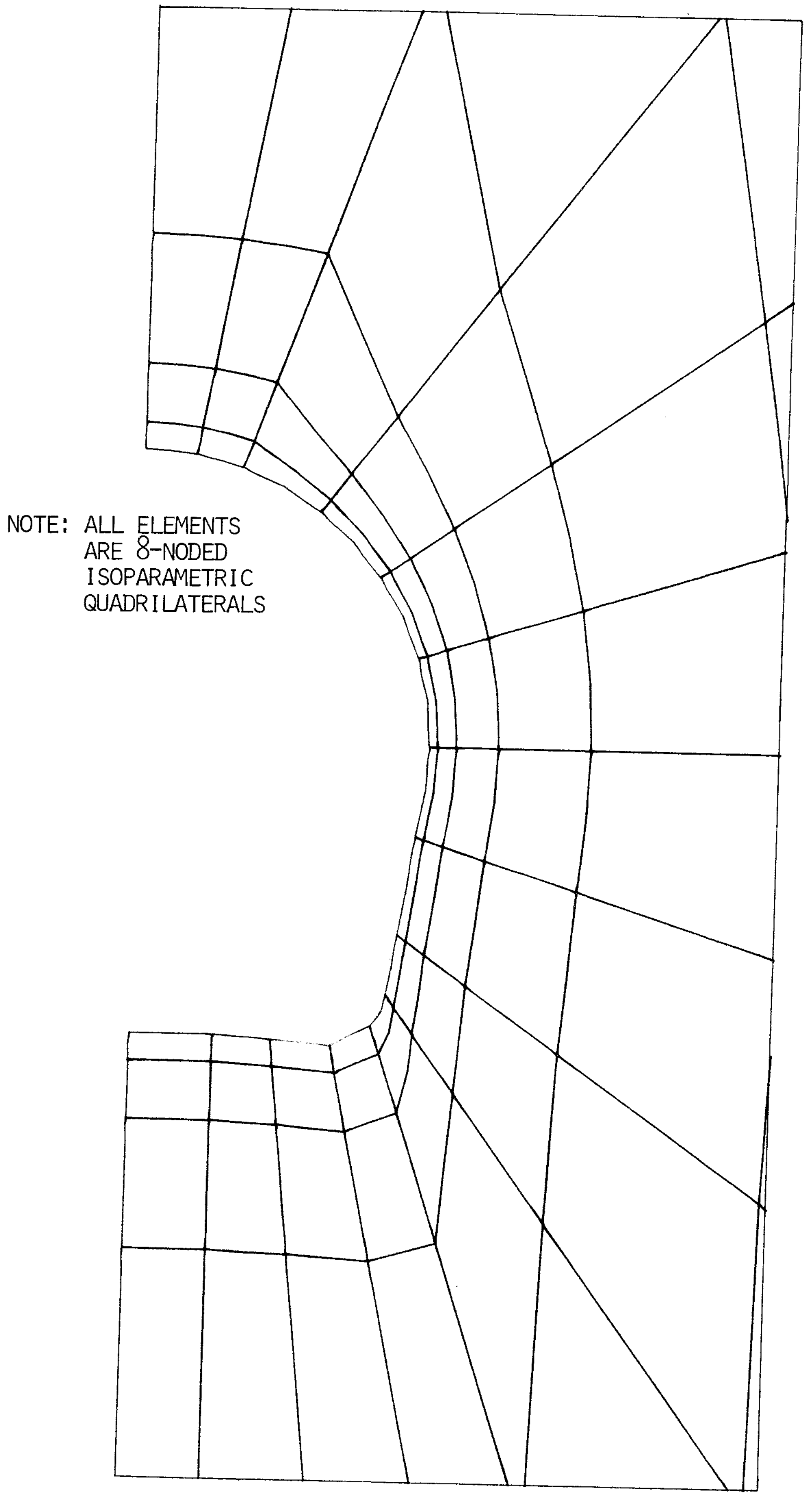
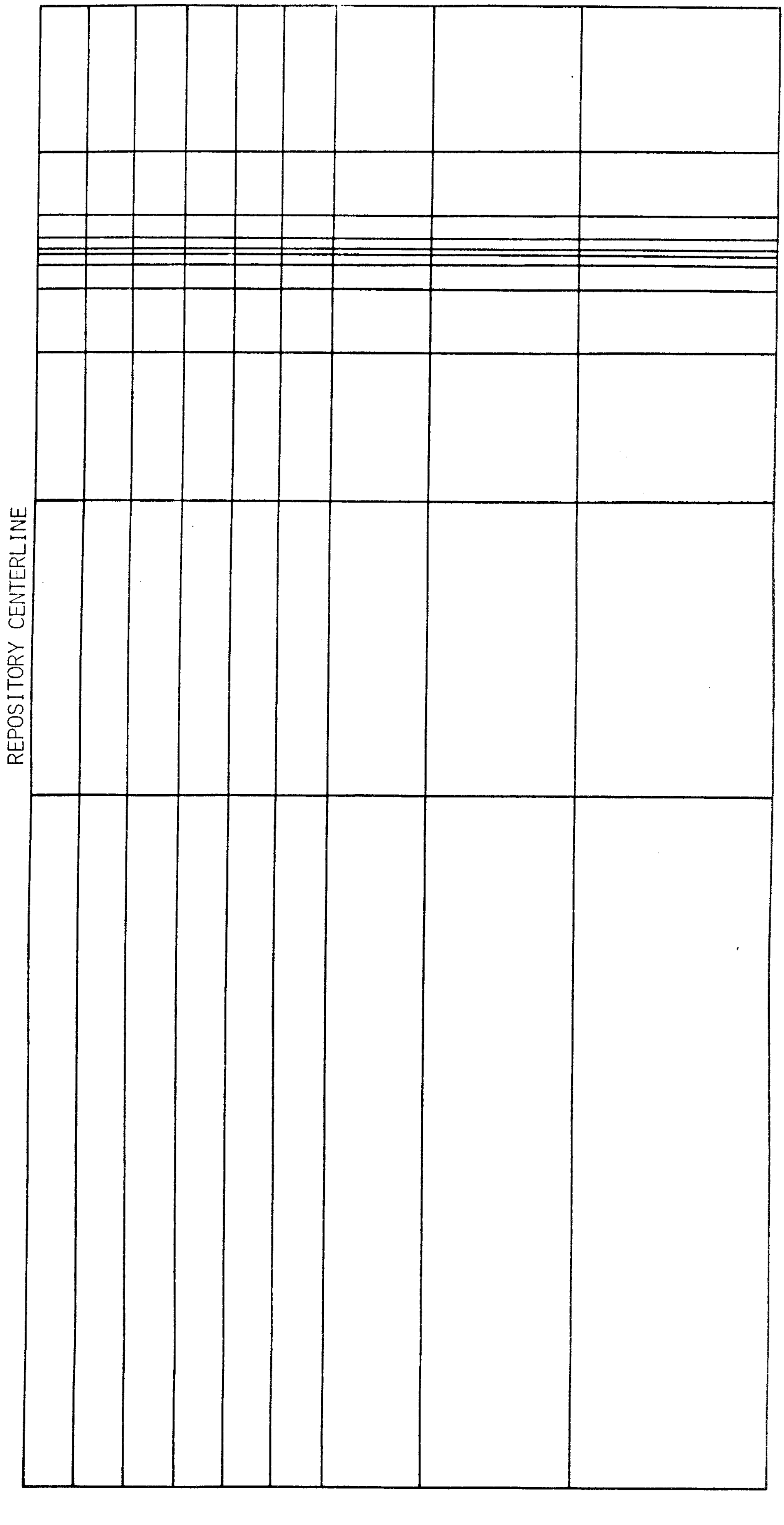



FIGURE A.1(B) INSERT OF LOCAL ROCK MECHANICS MODEL




NOTE:

ALL ELEMENTS
ARE 8-NODED
ISOPARAMETRIC
QUADRILATERALS

FIGURE A.2 FINITE ELEMENT MODEL FOR 500 M REPOSITORY GLOBAL ROCK MECHANICS ANALYSIS

NOTE:

(1) ALL ELEMENTS ARE 8-NODED ISOPARAMETRIC QUADRILATERALS

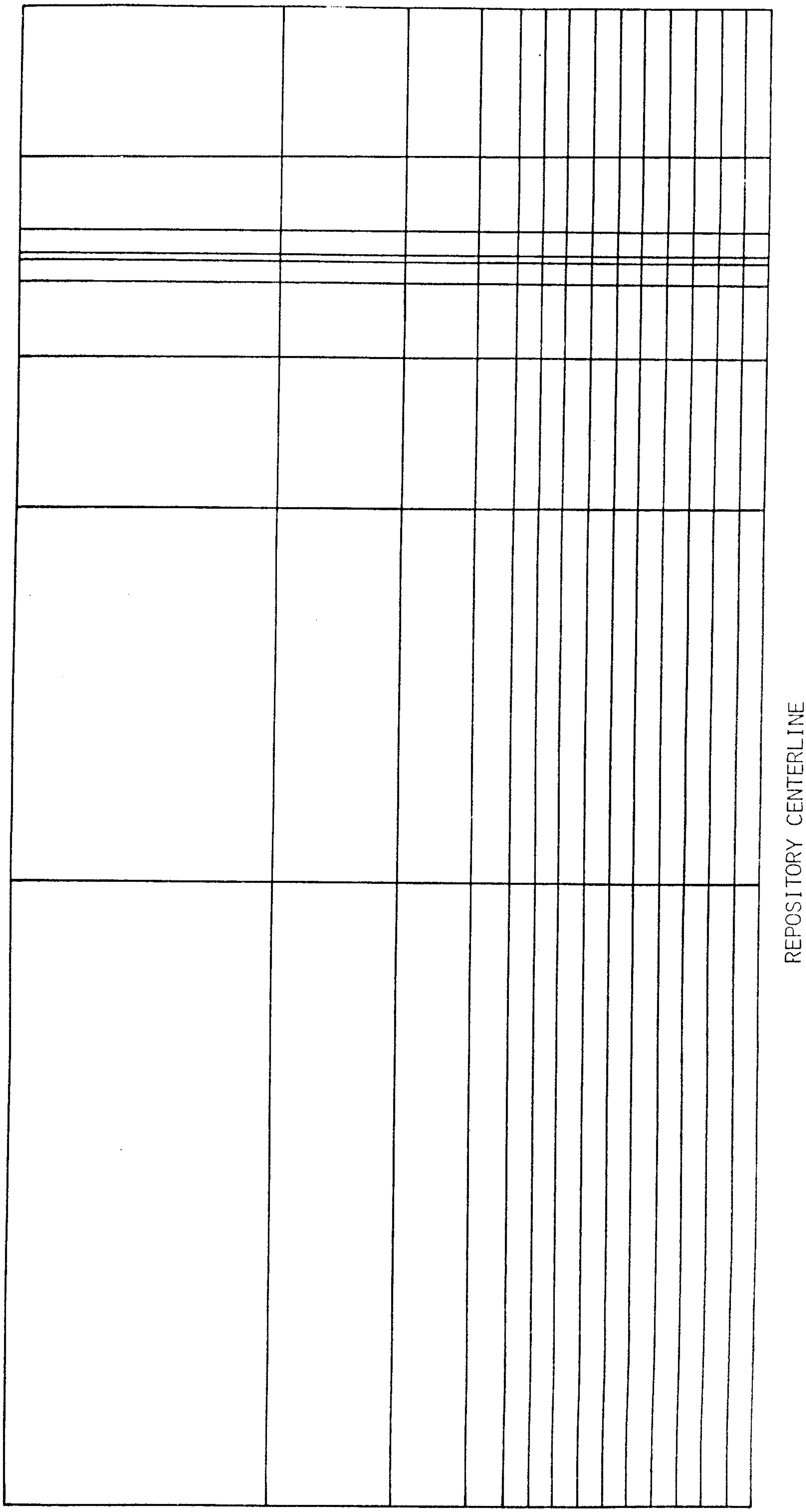



FIGURE A.4 SYMMETRIC HALF SECTION OF FINITE ELEMENT MODEL FOR 500 M LINEAR WASTE EMPLACEMENT REPOSITORY GLOBAL ROCK MECHANICS ANALYSIS.

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