A CONTRIBUTION TO THE DESIGN OF
TUNNELS IN ARGILLACEOUS ROCK

by

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ABSTRACT

Tunneling in argillaceous rocks is still one of the unresolved problems in underground construction. This is not only due to the large magnitude of both forces and displacements encountered, but also to the lack of knowledge about the underlying mechanisms and to the variety of conditions associated with swelling and creep phenomena. This research proposes a new design method, based on the "Stress Path" method, to simulate the behavior of the rock in the vicinity of a tunnel.

The research program consists of major components: (1) review of design methods and investigation of the behavior of tunnels in argillaceous rocks, (2) design and construction of a new triaxial system with development of software for automated testing and data acquisition via a personal computer, (3) extensive calibration of the new triaxial system, (4) analysis and synthesis of the results of undrained triaxial tests on argillaceous rocks under stress conditions which simulate those around a tunnel located in an initially isotropic state of stress, and (5) design methodology to obtain the pore pressure and strain distributions around the springlines of a tunnel just after excavation.

Results from experiments show that the non-cemented argillaceous rocks tested behave similarly to either overconsolidated clays or jointed rocks. For the rocks which behave similarly to overconsolidated clays, preliminary results suggest that a normalizable behavior can be observed with a reasonable accuracy for engineering purposes. This is surprising for rocks with an initial water content ranging from one half to two thirds of their respective shrinkage limits, and which have been subjected to very complex stress history.

Thesis supervisor: Dr. Herbert H. Einstein
Title: Prof. of Civil Engineering
DEDICATION

To my Parents
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A new program on argillaceous rocks was entirely initiated at MIT by the author. This, however, would never have been possible without the help and support of numerous people and organizations. Their support and contributions have been very much appreciated. In particular, I would like to express my heartfelt thanks to:

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FOREWORD

This thesis is the result of several years of research and work in the area of swelling rocks and related problems. It started in 1981 when I was taking a special course on Underground Construction at the Swiss Institute of Technology in Lausanne. The problems encountered in tunneling through swelling rocks impressed me greatly, and I found them very challenging.

From 1982 to 1984, I worked with Motor Columbus AG Consulting Engineers (now Gaehler und Partners AG) in Baden, Switzerland. There I was confronted with the problem of the design of the Bözberg tunnel (between Baden and Basel in Switzerland), which crosses jurassic formations containing swelling rocks. In 1983 I extended a method based on previous work of Gysel (1977) for the design of tunnel support in swelling rock; this extension has been published by Gysel (1987). At that time I realized that such a model had many limitations and was not representative of the swelling process. In 1985, upon proposition of Professor Einstein (Chairman of the Commission on Swelling Rock of the International Society of Rock Mechanics), I went to the Massachusetts Institute of Technology in Cambridge (USA). After a period of nine months as a visiting engineer, I started working towards a Ph.D in the fall of 1985. This work is presented hereafter.

The main task of my thesis was to initiate a completely new project in the area of argillaceous rocks. First, I developed a new design approach for tunnels in argillaceous rocks. Second, I built a new, computer-controlled triaxial apparatus, in which: (1) in-situ conditions or close to in-situ conditions can be reproduced, (2) pore water pressure is monitored, and (3) direct accurate volumetric change is recorded. Third, I performed undrained tests on three types of argillaceous rocks coming from two tunnel projects in Switzerland: the Transjurane and the Wisenberg. The obtained results open new fields in the understanding of the behavior of argillaceous rocks which show a strong similarity with overconsolidated clays.
The results presented in this thesis represent a first step towards a better understanding of the behavior of tunnels in argillaceous rocks. Due to time considerations, it has not been possible for me to investigate the behavior of argillaceous rocks in the drained phase. I am aware that some aspects of argillaceous rocks are not considered in this thesis, but the main goals pursued throughout this work are to stimulate new interests in this area, and to provide a new, more rational, design approach for tunnels in argillaceous rocks.
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NOTATION

**Symbols**

- $a_c$: corrected area of cross-section of the specimen at axial strain $\varepsilon_a$
- $a_i$: area of cross-section of the specimen at the end of saturation, before shearing
- $a_0$: original cross-sectional area of specimen
- $\#b$: number of bytes
- $c'$: drained cohesion
- $c'j$: drained cohesion along a joint
- $c$: generalized consolidation coefficient (3-D)
- $d_i$: initial diameter of the specimen after saturation, before shearing
- $d_{im}$: initial (unstretched) diameter of the membranes
- $d_0$: original diameter of specimen
- $e$: solid volumetric strain ($= e_{kk}$)
- $e'$: void ratio
- $e_i$: initial void ratio after saturation
- $e_{ij}$: solid strain tensor, positive in compression
- $e_0$: original void ratio
- $h$: depth of tunnel
- $h_i$: initial height of the specimen after saturation
- $h_m$: maximum overburden
- $h_0$: original height of the specimen
- $h_w$: height of column of water above the tunnel axis before excavation
- $i$: angle of roughness
- $k$: intrinsic permeability (in units of length squared)
- $m$: y-intercept in hyperbolic model, or exponent for OCR in empirical relationship for undrained strength, depends only on material
- $n$: porosity or slope of straight line in hyperbolic model
- $n_0$: initial porosity
- $p$: mean total stress, $p = (\sigma_1 + \sigma_3)/2$
\( p' \) mean effective stress, \( p' = (p-u_o)-\Delta u \)
\( p_0 \) isotropic component of total stress, \( p_0 = (\sigma_{v_0}+\sigma_{h_0})/2 \)
\( q \) maximum shear stress, \( q = (\sigma_1-\sigma_3)/2 \)
\( q_1 \) specific discharge vector
\( q_0 \) initial maximum shear stress, \( q_0 = (\sigma_1-c)/2(\sigma_{rr,c}+\sigma_{\theta\theta,0})^2+\sigma_{r\theta,0})^21/2 \)
\( q_c' \) drained uniaxial strength
\( r \) radial coordinate
\( s \) undrained shear strength for normally consolidated material, constant for each material
\( s_u \) undrained shear strength, function of OCR
\( t \) time
\( t_e \) time at end of consolidation/swelling
\( t_1 \) unit reference time
\( u \) pore water pressure
\( u_o \) initial pore water pressure
\( w \) water content
\( w_c \) water content at end of consolidation/swelling
\( w_e \) final water content
\( w_i \) initial water content at end of saturation
\( w_l \) liquid limit
\( w_0 \) original natural water content
\( w_p \) plastic limit
\( w_s \) shrinkage limit
\( A \) Skempton's pore pressure parameter, \( A = (\Delta\sigma_1-\Delta\sigma_3)/(\Delta u-\Delta\sigma_3) \)
\( A_{rd} \) area of rolling diaphragm
\( A_3 \) area of cross-section of specimen
\( B \) Skempton's pore pressure coefficient, \( B = \Delta u/\Delta\sigma_3 \)
\( B_{corr} \) corrected \( B \)-value
\( B_{obs} \) observed \( B \)-value
\( B_{used} \) used \( B \)-value
\( BV \) ball valve
\( C \) bulk compressibility of the rock skeleton, \( C=1/K \)
\( C_{fl} \) fluid compressibility
\( C_1 \) compressibility of the drainage lines
\( C_a \) compressibility of the pore pressure measuring element
(pressure transducer)

$C_g$ compressibility of the solid material of the skeleton

$C_w$ compressibility of water

$CF$ clay fraction (<2μm) or calibration factor

$CF_1$ calibration factor of proximity sensor #1

$CF_2$ calibration factor of proximity sensor #2

$CP$ confining pressure

$C(t)$ time function of tunnel convergence

$D$ diameter of dummy concrete specimen

$D/A$ digital/analog

$E^*$ extension modulus of the rubber membrane

$ESP$ effective stress path

$G$ shear modulus

$Int$ function returning greatest integer less than the input number

$J_a$ joint alteration number

$J_n$ joint set number

$J_r$ joint roughness number

$J_w$ joint water reduction number

$K$ drained bulk modulus of the rock skeleton

$K_c$ concrete bulk modulus

$K_{f1}$ inverse of fluid compressibility $C_{f1}$, $C_{f1}=1/K_{f1}$

$K_0$ coefficient of lateral stress at rest, defined in terms of effective stress, $K_0 = \sigma_{ho}/\sigma_{vo}$

$K_0(OC)$ coefficient of lateral stress at rest, for overconsolidated material (OCR > 1.0), defined in terms of effective stress ($K_0 = \sigma'_{ho}/\sigma'_{vo}$)

$K_p$ coefficient of lateral stress at passive failure

$K_g$ modulus of solid grains

$LC$ axial force measured by the submerged load cell

$MMV$ micrometer valve

$N_1$ initial number of microsteps between the $i$th and $(i+1)$th-targets

$NV1$ needle valve #1

$NV2$ needle valve #2
OCR  overconsolidation ratio
PI  plasticity index
Q  total rating parameter (Barton et al., 1974)
R₀  nominal radius of tunnel opening
RQD  rock quality design
S  degree of saturation
S₁  initial degree of saturation, after saturation
S₀  original degree of saturation
SRF  stress reduction factor
T*  characteristic parameter of time
T₀  time factor, T₀ = c₀t/R₀²
TSP  total stress path
TSP-u₀  total stress path minus initial pore water pressure
U₃  deviatoric component of the radial displacement
Uᵢ  average solid displacement vector in direction i,
i=r → radial direction, i=θ → tangential direction
Uᵣ  volumetric component of the radial displacement
VDC  volts direct current
V  volume of the rock specimen
V₁  volume of specimen at end of saturation, before shearing
Vᵢ  volume of the fluid in the water lines
V₀  original volume of the rock specimen
Vpin  volume of the micrometer valve pin
W  swelling material constant
X  coefficient, extrapolated strain at zero stress-intercept of
the log ɳᵢ - Δ relationship
ɑ  R₀/ᵣ or Biot coefficient of effective stress
β  stress exponent, slope of the log ɳᵢ - Δ relationship
δ  partial derivation
δₐ  axial displacement
δₑₚ  axial displacement of piston
δₑₑ  change in distance "probe-target" with a dummy
concrete specimen
δₑₛ  change in diameter of the dummy concrete specimen
δ₁₁  diameter change
δ₁₂  Kronecker delta
\( \delta_{ae}(CP) \) deformation of the moving support due to changes in the confining pressure CP

\( \varepsilon_a \) axial strain (positive in compression, negative in extension)

\( \dot{\varepsilon}_a \) axial strain rate

\( \varepsilon_{a,i} \) axial strain between the \( i^{th} \) and \( (i+1)^{th} \)-targets

\( \varepsilon_a^{\text{tot}} \) total axial strain, \( \varepsilon_a^{\text{tot}} = \varepsilon_a^\sigma + \varepsilon_a^\nu \)

\( \varepsilon_a^\nu \) axial strain due to water absorption

\( \varepsilon_a^\sigma \) axial strain due to deformation of the material's matrix structure

\( \varepsilon_d \) diameter strain

\( \varepsilon_r \) radial strain

\( \varepsilon_{\text{vol}} \) volumetric strain upon shearing

\( \gamma \) shear strain, \( \gamma = \varepsilon_a - \varepsilon_d \)

\( \gamma' \) buoyant specific gravity of material (soil, rock)

\( \gamma_s \) specific gravity of solid grains

\( \gamma_t \) specific gravity of material (soil, rock)

\( \gamma_w \) specific gravity of water

\( \theta \) angular coordinate, measured in positive degrees

\( K \) swelling coefficient

\( K_p \) coefficient of permeability, \( K_p = k/\mu \)

\( \lambda \) time exponent, slope of the log \( \varepsilon_a \) -log t relationship

\( \lambda_0 \) coefficient of lateral stress at rest, defined in terms of total stresses, \( \lambda_0 = \sigma_{ho}/\sigma_{vo} \)

\( \mu \) viscosity of water

\( \nu \) drained Poisson's ratio

\( \nu_u \) undrained Poisson's ratio

\( \zeta \) variation of fluid content per unit reference volume

\( \phi' \) drained friction angle

\( \phi'_{ij} \) drained friction angle along a joint

\( \phi'_{ir} \) drained residual friction angle

\( \rho \) coefficient of correlation of linear regression analysis

\( \sigma \) normal stress

\( \sigma_a \) axial total stress

\( \sigma_{ac} \) correction term for the axial total stress

\( \sigma_c' \) isotropic consolidation effective stress

\( \sigma_{ho} \) initial horizontal total stress
\( \sigma_{ij} \) total stress tensor, positive in compression

\( \sigma'_{ij} \) effective stress tensor, definition in soil mechanics:
\[ \sigma'_{ij} = \sigma_{ij} - u \]

\( \sigma''_{ij} \) effective stress tensor, definition in rock mechanics:
\[ \sigma''_{ij} = \sigma_{ij} - \alpha u \]

\( \sigma_0 \) initial state of total stress

\( \sigma_{oct} \) octahedral total stress,
\[ \sigma_{oct} = (\sigma_1 + \sigma_2 + \sigma_3) / 3 = (\sigma_{rr} + \sigma_{\theta \theta} + \sigma_{zz}) / 3 \]

\( \sigma_{oct,o} \) initial octahedral total stress

\( \sigma'_{oct} \) octahedral effective stress

\( \sigma_{om} \) initial confining stress due to the rubber membranes

\( \sigma_r \) radial total stress

\( \sigma_{rr} \) principal radial total stress

\( \sigma_{rr,o} \) initial principal radial total stress

\( \sigma_{r\theta} \) shear stress

\( \sigma_s(M) \) swelling pressure determined with the method of Hudler and Amberg (1970)

\( \sigma_s(K) \) swelling pressure determined with the method of Kovari et al. (1981)

\( \sigma_s(M) \) swelling pressure determined with the method of Madsen (1976)

\( \sigma'_{vc} \) vertical consolidation effective stress

\( \sigma'_{vm} \) maximum vertical past effective stress (1-D model)

\( \sigma_v \) initial vertical total stress

\( \sigma_{zz} \) principal longitudinal total stress parallel to the tunnel axis

\( \sigma_{zz,o} \) initial principal longitudinal total stress parallel to the tunnel axis

\( \sigma_0 \) tangential total stress

\( \sigma_{\theta \theta} \) principal tangential total stress

\( \sigma_{\theta \theta,o} \) initial principal tangential total stress

\( \sigma_1 \) major principal total stress

\( \sigma_{ma} \) correction to the major principal stress due to the strength of the rubber membranes

\( \sigma_2 \) intermediate principal total stress

\( \sigma_3 \) minor principal total stress

\( \tau \) shear stress

\( \Delta \) stress level with respect to failure, \( \Delta = q / q_f \)

\( \Delta g \) change in maximum shear stress
\( \Delta u \)  
excess pore water pressure

\( \Delta u(i) \)  
excess pore water pressure associated with mode \( i \)

\( \Delta u_{\text{norm}}(i) \)  
excess pore water pressure associated with mode \( i \)

\( \Delta w \)  
change in water content

\( \Delta CP \)  
change in confining pressure

\( \Delta LC \)  
change in reading of axial force measured in the submerged load cell, due to change in confining pressure \( \Delta CP \)

\( \Delta N_i \)  
increment (positive or negative) between the \( i \)th and \((i+1)\)th-targets

\( \Delta V \)  
change in voltage with respect to initial reading value

\( \Delta V_1 \)  
change in voltage with respect to initial reading value across the proximity sensor \#1

\( \Delta V_2 \)  
change in voltage with respect to initial reading value across the proximity sensor \#2

\( \Delta V_3 \)  
change of volume of specimen upon shearing

\( \Delta V_w \)  
change in volume of water of the specimen

\( \Delta \sigma_h \)  
change in horizontal total stress

\( \Delta \sigma_{\text{oct}} \)  
change in octahedral total stress

\( \Delta \sigma'_{\text{oct}} \)  
change in octahedral effective stress

\( \Delta \sigma_h \)  
change in horizontal total stress

\( \Delta \sigma_1 \)  
change in major principal total stress

\( \Delta \sigma_3 \)  
change in minor principal total stress

\( \nabla^2 \)  
Laplace operator

\( \Omega \)  
compressibility of the pore pressure apparatus

\( 1-D \)  
uni-dimensional

\( 2-D \)  
bi-dimensional

\( 3-D \)  
tri-dimensional

**Subscripts**

ave  
average

obs  
observed

f  
at failure

100  
at 100% consolidation

**Superscripts**

\( 0^+ \)  
at very short times, \( t \approx 0 \)

\( \infty \)  
at very large times, \( t \rightarrow \infty \)
CHAPTER 1

INTRODUCTION

1.1 FRAMEWORK

Tunneling in argillaceous rock is still one of the most challenging and unresolved problems in underground construction. This is not only due to the large magnitude of both forces and displacements encountered, but also due to the lack of knowledge about the underlying mechanisms and the variety of conditions associated with swelling and creep phenomena. Case histories, for instance, show that geological and hydrogeological conditions may vary, as do the in-situ initial state of stress, the excavation methods used, and finally, the rigidity of the tunnel supports provided.

Tunnel design in argillaceous rocks has relied mostly on past experience, in which qualitative and empirical methods have played a major role. In the last two decades, however, analytical models have been developed as an alternative approach to treat the problem in a more quantitative manner.

This thesis is a contribution to a new quantitative approach to the design of tunnels in argillaceous rocks. The new concepts presented here are based on experimental evidence and represent a first step towards a better design of tunnels in argillaceous rock.

The second section of this chapter defines important concepts and establishes the terminology used throughout this thesis. The third section briefly describes the problems encountered in tunneling through argillaceous rocks. Finally, the two last parts of this chapter present a concise overview of the entire thesis.
1.2 PROBLEM DEFINITION

1.2.1 GEOMECHANICS OF POROUS MEDIA

The relation between changes (decrease or increase, and/or rotation) in the applied stresses and the corresponding excess pore water pressures, and subsequent water inflow or outflow, has long drawn the attention of geotechnical engineers. This interest is justified because the stress-strain characteristics of saturated elements of soil (or rock) depend on the adjustment of the water content to the applied state of stress (e.g. Terzaghi, 1925; Casagrande, 1934).

Terzaghi and Peck (1967) recognized two extreme conditions: a) drained conditions, under which the changes in stress are applied sufficiently slowly, with respect to the ability of the ground to drain, that no excess pore water pressures develop; and b) undrained conditions, under which the stresses are changed so rapidly in relation to the ability of the ground to drain that no dissipation of the excess pore water pressure takes place. These extreme conditions are rarely fully realized in the field. They can be produced in the laboratory, however, and because they represent limiting conditions, they are valuable guides to understanding the ground behavior.

For a given problem, a build-up of excess pore pressures (positive or negative) is observed within the undrained phase. In contrast, the drained phase is characterized by the dissipation of excess pore water pressures over time. One also observes a simultaneous corresponding effect on the volumetric behavior of the ground. If water flows toward an element or a zone in the ground, one speaks of swelling to describe the volume change, as opposed to consolidation, when water outflow takes place. Consolidation and swelling are therefore two opposite phenomena.

Practically, the drained phase can be further divided into two subphases: (1) consolidation/swelling and (2) creep. These subphases are defined below:
(1) **Consolidation/swelling** (also called primary consolidation/swelling when referring to 1-D process)

This subphase involves dissipation of the excess pore water pressures through a hydrodynamic diffusion process. During this process, the effective stresses are altered and a volume change results.

(2) **Creep** (also called secondary consolidation/swelling when referring to the 1-D process)

In this subphase, the effective stresses as well as the pore water pressures and the total stresses are essentially constant. However, strain and volume changes can still occur due to time-dependent properties of the skeleton.

An important controversial point still exists: the chronological ordering of the subphases creep and consolidation/swelling. The unanswered question is whether these subphases are simultaneous or consecutive. In this research, we assume that creep takes place after consolidation/swelling as hypothesized and supported by experimental work at MIT and at the University of Illinois at Urbana-Champaign (e.g. Taylor, 1942; Mesri et al., 1983). For further information concerning the theory of primary consolidation/swelling, the reader is referred to Lambe and Whitman (1969).

### 1.2.2 SWELLING ROCK

The Commission on Swelling Rock of the International Society for Rock Mechanics (ISRM) has defined the swelling of rock as a time-dependent volume increase involving physico-chemical reactions with water (ISRM, 1983). This definition was kept general because it relates to all rocks showing time dependent volume increase. The most recent attempts to classify such swelling mechanisms in rock have been made by Einstein and Bischoff (1975) and Lindner (1976). Although there is no one-to-one correspondence between their classifications (Table 1-1), they agree that the most significant mechanism is the one which describes the time-dependent interaction between stress changes (decrease and/or rotation) and increase in water content. This mechanism occurs in materials
consisting primarily of stiff to very stiff and hard clays, whether or not they are cemented. These materials are called argillaceous rocks in this thesis. Usually, they have low shear strength, low durability and high swelling or rebound potential (Gamble, 1971).

The term "swelling rock", generally used to denote rocks that undergo significant time-dependent volume increase, is misleading as there are actually two drained subphases, swelling and creep, although they can be difficult to separate.

Mechanisms involving chemical reactions, such as for example, hydration of anhydrite, oxidation of pyrite, or precipitation of gypsum (ISRM, 1983), are not considered in this work.

1.3 PROBLEM STATEMENT

The excavation of a tunnel produces large changes in the state of stress and in pore pressure distributions, which considerably affect the behavior of the surrounding rock. During the undrained phase, the rock may undergo not only severe shearing with development of high excess pore pressures, but also dramatic changes in its original mechanical properties (e.g. shear modulus). These properties may then further change during the drained phase, when a new flow pattern compatible with the tunnel opening is established. If the excess pore pressures are initially positive, water is expelled from the pores of the rock as it consolidates. In contrast, if excess pore pressures are initially negative, water flows into the pores of the rock as it swells. It is known from experience that this can take many years (e.g. Terzaghi, 1936; Sulem et al., 1987a). After the end of this diffusion process, creep strain and volume changes under a constant state of stress can occur.

Because of the relatively low strength of argillaceous rocks, a plastic zone, consisting of decompressed and broken rock, may develop around the tunnel. Consolidation/swelling and creep phenomena can lead to the development of a plastic zone or vice-versa.

All the subsequent considerations assume initially 100% saturated
rocks, unless explicitly specified otherwise.

1.4 THESIS SCOPE AND OBJECTIVES

The primary objectives of this research are: (1) to develop a new framework for the design of tunnels in argillaceous rocks, (2) to conduct undrained tests on various argillaceous rocks using a new triaxial system, and finally, (3) to develop a new methodology for the design of tunnels in argillaceous rock. To satisfy these objectives, the following tasks were undertaken:

- design and construction of a new triaxial system, together with the development of software for automated testing and data acquisition;

- analysis and synthesis of the results of undrained tests on three types of argillaceous rocks, under stress conditions which simulate those encountered within the rock mass surrounding a tunnel;

- development of a new approach to tunnel design in argillaceous rock.

1.5 ORGANIZATION OF THESIS

Chapter 2 presents brief descriptions of case histories, and of existing analysis and design approaches for underground facilities in argillaceous rock. Six case histories are described which illustrate particular aspects of swelling and creep. Existing design approaches for underground facilities in swelling rock are analyzed and conclusions regarding their advantages and limitations are drawn.

In Chapter 3 existing analytical models for the design of underground facilities in porous media are critically reviewed. This review identifies the key parameters needed for the design.

Chapter 4 discusses the experimental requirements for simulating the stress conditions within the ground due to the excavation of a circular
tunnel. This Chapter also describes the new triaxial system and outlines the specimen preparation and testing procedures. Detailed information concerning the new triaxial system can be found in Appendix D. Finally, the computer control software for the triaxial system and its data acquisition system is provided in Appendix E.

In Chapter 5 the testing program is outlined, and the tests results are discussed. A new approach for the design of tunnels in argillaceous rock is proposed in Chapter 6, and finally in Chapter 7 the thesis is summarized, and conclusions and recommendations are given.
### Table 1-1

<table>
<thead>
<tr>
<th>Causes</th>
<th>Effects</th>
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<tbody>
<tr>
<td>1 Change (relief or rotation) in stress state</td>
<td>Volume increase similar to particle rebound on much smaller scale</td>
</tr>
<tr>
<td>2 Adsorption or absorption of water</td>
<td>Volume increase due to increase in water content</td>
</tr>
<tr>
<td>3 Stress changes lead to adsorption and/or adsorption of water</td>
<td>Volume increase due to stress changes and increase in water content</td>
</tr>
<tr>
<td>4 Adsorption and/or absorption of water lead to stress changes</td>
<td>Volume increase due to increase in water content and stress changes</td>
</tr>
<tr>
<td>5 Creep</td>
<td>Adsorption of water and associated weakening of bonds with reduction of shear strength and further increase in volume</td>
</tr>
</tbody>
</table>

### Correspondence Table

**from Einstein & Bischoff to Lindner**

<table>
<thead>
<tr>
<th>Einstein &amp; Bischoff</th>
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**from Lindner to Einstein & Bischoff**

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<th>Einstein &amp; Bischoff</th>
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</tr>
<tr>
<td>5</td>
<td>not discussed</td>
</tr>
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CHAPTER 2

BACKGROUND

This chapter deals with two aspects of tunneling in argillaceous rock. First, six case histories are presented which the author considered the most interesting of those reported in the literature. This is followed by a critical review of the state-of-the-art of designing underground facilities in argillaceous rocks.

2.1 FIELD EVIDENCE: ANALYSIS OF ILLUSTRATIVE CASE HISTORIES

Although these case histories are, with some exceptions, qualitative, they are very useful because they report valuable observations which emphasize some particular aspects of the in-situ behavior of argillaceous ground.

Of particular interest are:

- the observations related to water inflow into the tunnel,
- the time-dependent radial deformations of the tunnel lining,
- the time-dependent stresses on/in the tunnel lining,
- other possible mechanisms which are related to swelling in the field such as the development of a plastic zone around the tunnel, and the deterioration of the rock mass.

2.1.1 CASE HISTORIES

Three previous studies serve as guideline, namely Golta (1967), Einstein (1979), and Steiner and Metzger (1988). Observations in tunnels that have been well documented and reported form the basis for the following comments. The case histories discussed in this section are listed in Table 2-1. Each case is very briefly described, and the
relevant observations (in the author’s opinion) discussed. For more detailed information, the reader is referred to to the original papers listed in Table 2-1. The major observations drawn from the six case histories are summarized in Table 2-2.

2.1.1.1 Case A: Boetzberg tunnel (Northwestern Switzerland)

This two-track railroad tunnel of 2.5 km length is located in marl, clayshale and anhydrite under a maximum overburden of 250 m (Figure 2-1). It was built between 1871 and 1875. Invert heave and abutment convergence occurred just after completion. The abutments in some sections were reconstructed several times. Significant reconstruction was undertaken between 1903 and 1905, involving the construction of invert arches in the most severely damaged sections (Figure 2-1). These, however, were rapidly destroyed. The drainage channel was frequently destroyed as well. Since 1923 regular crown and invert displacement and convergence measurements were made; the results are plotted in Figures 2-1 and 2-2. No measurements were taken after 1954 due to heavy damage to the invert where the reference points were located. In these zones new invert arches were constructed between 1963 and 1967 (Figure 2-1). A cross-section of the tunnel is displayed in Figure 2-3, which illustrates the inward displacement of the original open arch support and the reconstructed section with an invert arch.

Relevant observations (in the author’s opinion)

(1) After a complete check of the tunnel conditions in 1923, measuring points were installed in the invert at locations where invert arches had been heavily damaged before 1903 and reconstructed between 1903 and 1905. These measuring points were thus installed 48 years after the end of the tunnel construction. They were destroyed by 1954, 31 years later, due again to significant heaving. The measurements made during this period were reported in the literature on a arithmetic time scale (Figure 2-2). When reporting these measurements on a logarithmic time scale, one obtains results which are more interesting from an engineering viewpoint. Figure 2-4
illustrates the three "heave-log t" curves of points located in the argillaceous rock (marl and clayshale). In this Figure, the three curves have a similar shape, in which two parts can be distinguished:

- from 1924 to 1939: the heave is increasing linearly with the logarithm of time, and
- from 1939 to 1954: the heave suddenly increases with very large deformations.

(2) During the reconstruction period of 1963 to 1967, large voids of 30 to 50 cm depth and loose material were found behind the tunnel lining in the springlines of tunnel sections where large deformations occurred (Figure 2-3) (Beck and Golta, 1972).

2.1.1.2 Case B: Belchen tunnel (Northwestern Switzerland)

This is a 3.2 km long highway tunnel, consisting of two parallel two-lane tubes, which is located in marl, clayshale and anhydrite under a maximum overburden of 300m (Figure 2-5). Built between 1963 and 1970, the tunnel was constructed by excavating two invert drifts in each tube and then enlarging them to the full cross-section. After the enlargement to the full cross-section in the Keuper formation, the invert level rose 0.9 m within a few months. The drainage pipe system rose too, and was laterally compressed (Figure 2-6). An invert arch was then built with a radius of R=10.4 m and a thickness of t=0.45 m (Figure 2-7). This however was destroyed shortly after construction and a further 0.60 m of invert heave occurred (Figure 2-8), requiring the construction of an invert arch with radius of 8.12 m and up to 0.85 m thick (Figure 2-7). In 1968, it was decided to monitor the stresses and the deformations in the lining. Furthermore, extensometers were added in 1986 to monitor deformations inside the rock (Kovari et al., 1987).

Relevant observations (in the author's opinion)

(1) The time-dependent stresses in the invert arch and under the invert arch are reported in Figure 2-9. The radial stresses are
measured at the contact between the ground and the lining. The tangential stresses are measured in the concrete lining. Results show an increase of both radial and tangential stresses with time in a linear stress-log t plot.

(2) Further, a plot of the mean tangential stress, σθ, versus the mean radial stress, σr, shows that they are extremely well correlated (coefficient of linear regression, ρ=0.977) (Figure 2-10). Unfortunately, the radius of the invert arch is not known, so that it is not possible to check if the hoop stress formula holds.

(3) In 1986, ten years after completion of the tunnel, extensometers were placed in the invert and in the springlines of the tunnel. One year later, in 1987, measurements clearly showed that deformations still occur over a zone approximately 10 m thick, i.e. approximately one tunnel diameter (Figure 2-11).

(4) Water required for swelling seems to have come from inflow either perpendicular or parallel to the tunnel. In the case of inflow parallel to the tunnel, water came from adjacent aquifer rocks (limestone, sandstone) (Grob, 1972).

2.1.1.3 Case C: San Donato tunnel (Northeastern Italy)

The San Donato tunnel is a high-speed railway tunnel, approximately 11 km long. Its cross-section ranges from 90 to 120 m², depending on the ground conditions at the face. The tunnel was excavated from North and South portals simultaneously, at a maximum depth of 250 m below the surface (Figure 2-12). It crosses a wide range of geological formations varying from soil-like to rock-like conditions.

The encountered sedimentary rocks form an anticline with the Macigno formation (sandstones, siltstones) in the center, surrounded by the Argille Scaglioise complex (clayshales) and finally, the Alberese formation (marly limestones), as illustrated in Figure 2-12. These rocks have been found to be highly tectonized, with nearly vertical faults being always present. The Argille Scaglioise complex exhibits two quite
different textures, namely a disarranged texture, with curved surfaces, often contorted and twisted together, and a texture with strong preferred orientation of the clay minerals (Figure 2-12). The latter is found in particular in the proximity of the Alberese formation (swelling zone, Figure 2-12), where the clay minerals are parallel to a nearly vertical fault. The Alberese and Macigno formations are jointed rock masses with three well marked discontinuity sets. Because of their fissured nature, they contain considerable quantities of water and act as aquifers. During the excavation, medium to large rates of groundwater inflow were encountered, primarily in the Alberese formation and in the transition zone between the Argille Scagliese and Macigno formations.

In the difficult ground conditions of the Argille Scagliese, the tunnel was advanced by a heading and bench excavation procedure with a single heading and a bench consisting of three sections, and finally the invert (Figure 2-13). The initial support, consisting of 4.5 m long fully grouted resin rock bolts, steel ribs (2NP 180/spaced at 0.8m) and 25 cm thick shotcrete, was put in place immediately after excavation. This heavy support formed a closed ring within a distance of 2.0 to 2.5 tunnel diameters behind the face. The construction methods used on the North side and on the South side were the same. When excavation took place from the North side, no problem occurred; however significant damage occurred on the South side (Figure 2-12). Heave up to 1.0 m and horizontal convergence of 1.2 m took place (Figures 2-14). The invert support was heavily damaged and the steel ribs and the shotcrete lining were breaking down over extended distances. With the invert support failing in most cases, the deformations around the tunnel increased dramatically over time (Figure 2-14).

Relevant observations (in the author's opinion)

(1) The considerable convergence and invert heave which took place on the South side, were attributed to the swelling characteristics of the Argille Scagliese, and to the presence of water below the invert (Barla et al., 1986). As a matter of fact
water was recorded on the South side in boreholes driven below the invert arch. As the tunnel was excavated downdip within the Argille Scagliose, water from the Alberese formation was running by gravity towards the tunnel face. On the North side, no water inflow was observed. The conclusion that the water played the major role is based on the fact that damage occurred on the South side only, while an analysis of the mineral content showed that a more significant swelling potential existed on the North side (Table 2-3).

(2) Horizontal convergence versus time curves of four steel sets (No. 180, 181, 186, 206) have been reported in semi-log plots in Figure 2-15. These curves show that convergence can be approximated by two sets of lines. Typically, each curve consists of several "linear" sections displaying slow rates of deformation, interspersed with "linear" sections representing rapid acceleration. Superposition of these four curves in Figure 2-16 shows that all the curves can be approximated with two sets of lines only. The author believes that these two sets of lines are representative of swelling (steeper set of lines) and creep (flatter set of lines) which took place in the rock after excavation, due to the associated stress relief. Indeed, for a simple excavation event, swelling first occurs at small times and then creep takes place. During the excavation process where successive excavation events take place, it is possible to observe successive phases of swelling and creep in the same measuring section as shown in Figures 2-15 and 2-15.

(3) Results of a theoretical analysis (performed by Barla et al., 1987) of the rock behavior show that a plastic zone should develop, extending approximately one diameter outside the tunnel perimeter.

2.1.1.4 **Case D: Test tunnel at the South Saskatchewan River Dam (Canada)**

In the third Terzaghi’s Lecture, Bjerrum (1967) presented mechanisms of progressive failure of overconsolidated plastic clays and clay
shales. He discussed this case history to show the effects of weathering on a plastic clay shale, the Bearpaw Shale. This is a fully supported test tunnel, located at 40 m depth, which crosses three distinct zones, namely, an unweathered zone, a zone showing medium weathering, and a completely weathered zone. Observations of natural water content and of the convergence of the tunnel walls clearly show the effects of weathering on the diagenetic bonds and consequently on the swelling of the ground.

Relevant observations (in the author's opinion)

(1) Gradual destruction of the diagenetic bonds is accompanied by non-uniform swelling, probably due to local variations in the mineralogical composition of the material, resulting in an increase in the joint intensity inside the ground. This is reflected in the great variation of the natural water content from point to point, which is much greater in the disintegrated ground than in the unaltered one (Figure 2-17).

(2) The convergence of the tunnel walls in the horizontal plane shows an increase when progressing from the unweathered to the completely weathered zone (Figure 2-18). The convergence in the unweathered zone is relatively small, 2 to 8 cm, while it increases from 27 to 53 cm in the completely weathered zone. A few days after excavation, the convergence curves can be very well approximated by straight lines in a plot of the horizontal convergence versus the logarithm of time.

2.1.1.5 Case E: Clay mine in Provins, near Paris (France)

A stratum of stiff overconsolidated clay of approximately 35 m thickness is mined for commercial purposes. It is covered by 20 m of water-bearing silt and sand. The galleries of this mine have a square cross-section of 1.85 x 1.85 m, and are located at an average depth of 35 m. The clay stood without any support immediately after excavation. After about one week, swelling started and approximately three months later, significant damage involving the crushing of timber supports was
observed. A test gallery was constructed in an untouched section of the clay deposit.

Relevant observations (in the author's opinion)

(1) The natural water content of 35% increased to approximately 70% in the clay directly adjoining the tunnel springlines.

(2) The horizontal stresses in the ground are greater than the vertical stress. This observation is confirmed by the following computations (made by the author).

(3) The following information is available from Terzaghi, 1936, p.154, and Proctor and White, 1946, p.79:

- Angle of friction : $\phi ' = 7$ to $10^o$ (CIDC(L) tests)
- Uniaxial drained strength : $q_c ' = 1.6$ to $1.8$ ksc
  
  $= 0.16$ to $0.18$ MPa
- Maximum past pressure : $\sigma_{vm} ' = 60$ tsf = 60 ksc = 6.0 MPa
- Depth of mine tunnel : $h = 35$ m
- Natural water content : $w_0 = 35%$

The following assumptions are made:

- The ground water table is at the surface
- The ground is 100% saturated, $S = 100% = 1.0$
- Specific gravity of solid grains, $\gamma_s = 27$ KN/m$^3 = 0.027$ MPa/m
- Coefficient of lateral stress at rest, for OCR=1, $K_0(NC) = 0.60$ (plastic clay)
- 1-D mechanical unloading of the clay layer
- Non-cemented clay

Analysis

- From $S$, $w_0$ and $\gamma_s$  : $e_0 = \gamma_s w_0 / S = 0.945$
- From $w_0$, $e_0$, $\gamma_s$ and $\gamma_w$  : $\gamma_t = (1+w_0)\gamma_s \gamma_w / (1+e_0) = 0.019$ MPa/m
  
  $\gamma' = \gamma_t - \gamma_w = 0.009$ MPa/m
  
  $\sigma_{vo} ' = \gamma' h = 0.305$ MPa
- From $\phi '$ and $q_c '$ : $c' = 0.069$ to $0.077$ MPa (Figure 2-19)
From $\phi'$ 

\[
K_p = \frac{(1 + \sin\phi')}{(1 - \sin\phi')}
\]

= 1.28 to 1.42

OCR = $\sigma_{vm}' / \sigma_{vo}' \approx 19.7$

Assuming 1-D mechanical unloading:

- $K_0(OC) = K_0(NC) (OCR)^{0.4} = 1.98 \approx 2.0$ (Ladd et al., 1977, p. 442)
- $\sigma_{ho}' = K_0(OC) \times \sigma_{vo}' \equiv 0.60 \text{ MPa}$

(2-1)

Assuming passive failure (Figure 2-19):

- $\sigma_{ho}' = \sigma_c' + K_p \times \sigma_{vo}' \equiv (0.16 \text{ to } 0.18) + (1.28 \text{ to } 1.42) \times 0.52$

= 0.55 to 0.61 MPa

(2-2)

Comparison of $\sigma_{ho}'$ obtained from empirical correlation through OCR (Equation 2-1) and $\sigma_{ho}'$ obtained assuming passive failure (Equation 2-2) shows that the ground is very close to or in passive failure (Figure 2-19) \(^1\).

2.1.1.6 Case F: Tunnel in Paris (France)

This tunnel with a diameter of approximately 12 m is located in a slightly overconsolidated clay \(^2\) at very shallow depth of 25 to 30 m (Figure 2-20). Construction started by excavating the clay within the shaded area shown in Figure 2-20. Very soon after excavation was finished, the clay began to expand greatly. It was thus necessary to excavate between 10 to 15% in excess of the quantity required by the final cross-section of the tunnel. During this process, the water content of the clay adjoining the tunnel changed considerably.

---

\(^1\) Terzaghi (in Proctor and White, 1946, p. 79) stated: "It was estimated that the pressure required to crush the timbering is about 25 tons per square foot, which is more than three times the overburden pressure. This observation indicates that the horizontal pressure in the clay is very much greater than the corresponding overburden pressure."

\(^2\) With empirical correlation based on OCR (Brooker and Ireland, 1965), an estimated $K_0$-value of 0.9 was found by the present author. Computations to estimate $K_0$ are similar to those shown in detail in Section 2.1.1.5.
Relevant observation (in the author's opinion)

(1) As illustrated in Figure 2-21, excavation changed the initial water content of 56% in the clay directly adjoining the tunnel to 90-130%, (+34-74%). It caused a decrease from 56% down to 46%, (-10%), at a distance of 4 m (approximately one third of the tunnel diameter) inside the clay mass (Terzaghi, 1936).

(2) Undisturbed samples of clay, kept in a closed container with a relative humidity approaching 100%, dried out. Terzaghi concluded that this observation disproves the hypothesis that an appreciable part of the surplus water in the expanded clay was provided by the tunnel air, but, on the contrary, proves that water was coming from the clay located beyond the zone of swelling.

2.1.2 DISCUSSION AND CONCLUSIONS

Table 2-2 summarizes the relevant observations by case histories, whereas Table 2-4 summarizes them according to specific issues. These issues are: (1) the initial state of stress in the ground, (2) the development of a plastic zone around the tunnel, (3) the presence of free water inside or in the immediate vicinity of the tunnel, (4) the disintegration of the rock due to weathering, (5) the evolution over time of the stresses acting on/in the lining, and finally, (6) the evolution over time of the radial deformations at the tunnel wall or inside the rock mass. These will now be discussed in more detail.

2.1.2.1 Initial state of stress in the ground

The knowledge of the initial state of stress in the ground is one of the key elements in designing tunnels. This is particularly true for tunnels excavated in argillaceous rock because they have in general low shear strength. In some cases, the natural ground may be close to or at failure. Examples are zones close to the surface which have experienced strong unloading or heavily tectonized zones. High lateral stresses may be locked in the ground \((K_0>1)\). In such cases, failure conditions, and
possibly residual conditions, in the ground prior to excavation are not unlikely. The clay mine of Provins (Section 2.1.1.5) is a case where the ground was estimated to be in a state close to or at failure prior to excavation.

2.1.2.2 Plastic zone

The creation of an opening in a prestressed medium is associated with a large increase in the deviator stress (See also Section 3.1), which can lead to the development of a plastic zone around the tunnel. This plastic zone is characterized by a strong unloading, along with significant weakening of the rock. Consequently, large deformations at the tunnel wall arise. Swelling is often associated with development of a plastic zone, as illustrated in the case history of the San Donato tunnel (Section 2.1.1.3), where the extension of the plastic zone has been theoretically estimated to one tunnel diameter.

2.1.2.3 Water

The availability of water is the most important element to consider when tunneling in argillaceous rock. Whether a rock element will swell or not depends on the availability of water. If water is not available, the rock element will not swell. Experience from various case histories has shown that swelling is always related to water flow toward the tunnel. This water can be provided by inflow parallel or perpendicular to the tunnel. The rock directly adjacent to the tunnel shows a large increase in water content. It is, however, very unlikely that water is provided by the air in the tunnel. In this respect, the experiment carried out by Langer and reported by Terzaghi (1936) in the case history of the tunnel in Paris (Section 2.1.1.6) disproves the common belief that water can be provided by condensation along the walls of the tunnel, as hypothesized by Wittke and Pierau (1976) for instance.

2.1.2.4 Disintegration

Prior to excavation, the argillaceous rock might have been subjected to tectonic forces or unloading movements which can have produced
considerable structural changes. Situations can be found in the field where the rock is now either in, or close to a failure state (e.g. clay mine in Provins near Paris, Section 2.1.1.5) or in a residual state if the deformations have been very large.

When a tunnel is excavated in such a rock, plastic zones appear in the direct vicinity of the tunnel, with highly non-uniform stress fields. The ground mass next to the tunnel reaches its maximum strength and experiences large deformations, while away from the tunnel the stress field is almost unchanged and deformations are small. Thus non-uniform strain field appear, favoring progressive failure. Gradual disintegration of diagenetic bonds is accompanied by non-uniform swelling, probably also due to a variation of the mineralogical composition of the material (Bjerrum, 1967). This results in an increase of the joint intensity and a non-uniform increase in the water content of the rock adjoining the tunnel, as reported by Bjerrum (1967) (Figure 2-17) and Terzaghi (1936) (Figure 2-21). The volume change, associated with the progressive failure phenomenon, is very strongly dependent on the strength of the diagenetic bonds. If the bonds are strong, the volume change will be much smaller than if the bonds were nonexistent. Thus, it is expected that the swelling characteristics of the rock are strongly related to the strength of the diagenetic bonds.

2.1.2.5 Stress-time relationship

Measurements of the radial and tangential stresses acting on/in the lining show a continuous increase with time. The data from the Belchen tunnel (Figure 2-9) demonstrate that equilibrium has not been reached as of today, i.e. 20 years after completion of the excavation. The measured tangential stresses tend to show that the lining loads increase with time. More data are, however, needed in order to further confirm this point.

2.1.2.6 Radial deformation-time relationship

The relationship of radial deformation of the tunnel wall versus time is by far the most valuable information (quantitatively speaking) that
has been recorded in underground construction over the past 100 years. Lo et al. (1978) reported results of many case histories in a plot of the deformations at the tunnel wall versus logarithm of time (Figure 2-22). Results of the present study have also been reported in the same plot. A similar pattern is found in all these cases, namely a simple linear relationship between the deformations at the tunnel wall and the logarithm of time. These deformations are attributed to creep occurring inside the rock mass. However, as illustrated in Figure 2-4, this linear relationship can be altered and a sudden increase in deformation can occur. This has also been observed in several other cases (e.g. test tunnel at the Saskatchewan river dam, Figure 2-18, and the case histories analyzed by Semple et al. (1973)). Such sudden changes in behavior are thought to be due to local failures occurring inside the rock mass surrounding the tunnel. These changes are sudden accelerations in the phenomenon of progressive failure (Bjerrum, 1967).

2.2 EXISTING DESIGN APPROACHES OF UNDERGROUND FACILITIES IN ARGILLACEOUS ROCK - STATE-OF-THE-ART REVIEW

The existing design approaches generally take into consideration swelling, creep, or both. They range from pure empiricism to more sophisticated methods, such as the finite element method. They are described hereafter in order to point out the assumptions and limitations inherent to their methods.

2.2.1 EMPIRICAL APPROACH

The empirical approach is used primarily before construction, when limited geological information is available, or during construction when time is limited. Amongst the many existing empirical methods developed (See Steiner (1980) for a complete review), two are often used in the design of tunnel supports in swelling rock, namely those by Terzaghi.

3 The swelling phase does not appear in Figure 2-22. The Author believes that the swelling phase was completed before the first deformation measurements were made. Thus, the curves in Figure 2-22 should be observed by keeping in mind that the swelling phase has been completed before the creep phase has been taking place.
(1946) and by Peck (1969).

Terzaghi (1946) gave special recommendations for the design of tunnels with steel supports in swelling rock: "Class 9, rock load equivalent to 250 feet (≈75 m), independent of tunnel dimensions" (Table 2-5). It seems that this rockload recommendation is based on literature review, though detailed sources cannot be determined (Steiner, 1980). Terzaghi, nevertheless, stated (in Proctor and White, 1946, p. 82):

"In shallow tunnels, the ultimate pressure on the tunnel lining may be considerably higher than the overburden pressure 4."

"In deep tunnels through swelling rock, pressure of 10 tons per sq. feet (1.0 MPa) are not uncommon. Exceptionally, pressures as high as 20 tons per sq. feet (2.0 MPa) have been encountered. A pressure of 20 tons per sq. feet is equivalent to the weight of a layer of rock with a thickness of not more than about 270 feet (≈82 m)."

In addition to the rockload factor, Terzaghi gave recommendations for the construction of tunnel support, proposing the use of narrow-flanged strong steel sets, which would be able to sustain large pressures. Since the flanges of the steel sets would be small, the rock could flow around the steel sets without damaging them (Figure 2-23a). To prevent rock from falling into the tunnel, lagging placed on the interior flange was proposed (Figure 2-23b). As an alternative, Terzaghi also recommended placing compressible spacers in the ribs to allow the rock to squeeze in. Finally, he proposed taking load measurements, in order to obtain a better understanding of the actual loads.

Following Terzaghi's line of thought, Peck (1969), in his review of tunneling in soft ground, also stated that the swelling pressure can exceed the overburden pressure. He suggests an equivalent support load corresponding to the pressure of "(σ_vo+σ_ho)/2" rather than to the overburden pressure, σ_vo, itself; σ_ho denotes the initial horizontal

---

4 With particular reference to the case history of the clay mine in Provins, near Paris, France. Terzaghi (in Proctor and White, 1946, p. 79) stated: "It was estimated that the pressure required to crush the timbering is about 25 tons per square foot, which is more than three times the overburden pressure. This observation indicates that the horizontal pressure in the clay is very much greater than the corresponding overburden pressure."
stress in the ground. This load assumes no radial deformation of the rock mass due to tunnel excavation, and thus represents the most unfavorable case of the immediate installation of a perfectly rigid support.

One should note that both methods provide design radial stresses on the tunnel support which are independent of the rock properties and the tunnel dimensions.

Other empirical approaches have been developed in the past few years: e.g. Brekke and Howard (1973), Barton et al.(1974), (See also Louis (1974), Franklin (1976) and Barton (1976)). Brekke and Howard (1973) developed a functional classification of gouge materials from seams and faults and discussed the appropriate tunneling method in these materials. Severe swelling problems can be encountered in tunneling if swelling pressures are greater than 0.25 MPa. The swelling pressure can be obtained either from a direct measurement of the swelling pressure by a laboratory swelling pressure test, or possibly from correlation with liquid limit test, or from correlation with the percentage of imbibed water under 100% relative humidity. Barton et al.(1974), developed a total rating parameter, Q, to describe the rock mass quality of the ground, which together with the equivalent dimension (the actual dimensions of the opening divided by the excavation support ratio, ESR), defines a rock class (Figure 2-24). The Q-parameter is obtained from a combination of six parameters as:

\[
Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF}
\]  \hspace{1cm} (2-3)

where RQD is the rock quality design, J_n is the joint set number, J_r is the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, and SRF is the stress reduction factor. In the tables proposed by Barton et al.(1974), description and rating with respect to swelling can be found for the parameters J_a and SRF. For swelling rock, the rating of both these parameters increases considerably, and the parameter Q decreases accordingly. Swelling rock is generally rated with "extremely poor" to "exceptionally poor" rock
mass quality parameter, \( Q \) (Figure 2-24). Louis (1974) and Franklin (1976) developed similar methods for preliminary design of underground openings. The proposed methods consider the swelling pressure as a parameter to determine a ground class; they are, however, not aimed at the design of tunnel support in swelling rock.

2.2.2 ANALYTICAL APPROACH

The analytical approach provides models which describe the behavior of the rock mass and its interaction with the tunnel support. These models are built within the framework of continuum mechanics and take into account the tunnel geometry, the initial state of stress, the rock properties obtained either from laboratory tests or from in-situ measurements, and possibly the excavation sequences. Four types of models can be distinguished: (1) models based on a swelling law, (2) rheological models, (3) derived rheological models, and (4) diffusion models.

2.2.2.1 Models based on swelling law

The models based on swelling law have been especially developed for tunnel design in swelling rock. They incorporate a swelling law in a classical linear elastic model. The swelling law assumes that, from a given pressure, called the swelling pressure, the material increases in volume as the applied stresses decrease. The main features of these models are summarized in Table 2-6. The swelling law and swelling pressure are obtained either with the methods of Huder and Amberg (1970) or Kovari et al. (1981) from tests performed in the 1-D oedometer (Appendix A).

In order to model swelling around tunnels, Grob (1972) approximated field situations with 1-D conditions; laboratory tests were performed accordingly in the 1-D oedometer. Based on tests results obtained after the method of Huder and Amberg (1970), Grob found a relationship between the axial strain, \( \varepsilon_a(\%) \) and the axial stress, \( \sigma_a \) (Figure 2-25):
\[ \sigma_a = W - K \log_{10} \sigma_a \]  

where \( W \) and \( K \) are material constants of the swelling rock.\(^5\)

The floor heave is computed following essentially an inverse 1-D settlement procedure.

Simultaneously, Einstein et al. (1972) hypothesized that the first invariant of the total stresses controls the volumetric swell deformations. They assumed that the swelling rock behaves as an isotropic linear elastic material, and the stress distribution is obtained accordingly for the oedometer. Results of 1-D tests are extrapolated to 3-D with the following relationship:

\[ \sigma_r = (\nu/(1-\nu)) \sigma_a \]  

where \( \nu \) is the Poisson ratio and \( \sigma_r \) and \( \sigma_a \) are, respectively, the radial and axial stress in the oedometer.

The 3-D in-situ state of stress around the tunnel is also determined with linear elasticity. The floor heave is evaluated following an inverse settlement computation which relates the change in the first invariant of stresses to the volumetric swell deformation.

More recently, the interaction between the tunnel support and the swelling rock was studied more closely and various methods of analysis were proposed using the finite difference method, the finite element method (Wittke and Rissler, 1976; Gysel, 1977; Kovari et al., 1983; Schweisig and Duddeck, 1985; Fröhlich, 1986) or closed form solutions (Gysel, 1987b). Hypotheses were made in order to subdivide the volumetric strain into principal strains. Several strain distributions have been proposed, namely isotropic and anisotropic.

The models which do consider 3-D states of stress (Table 2-6) can be characterized as follows:

\(^5\) which are dependent on the chosen units for the axial stress, \( \sigma_a \).
- only the strain state corresponding to the final equilibrium can be computed,

- the methods are restricted to linear elasticity,

- the hypothesis of Einstein et al. (1972), based on linear elasticity, is inconsistent since the behavior of the material in an oedometer swell test in non-linear,

- if the initial state of stress in the ground is isotropic ($\lambda_0=1$), the models predict that no swelling occurs,

- the 3-D state of stress, obtained by extrapolation from the oedometer, is independent of the initial state of stress in the ground.

2.2.2.2 Rheological models

Because of the observed time-dependency of the swelling and creep processes and the difficulties of understanding these mechanisms, a phenomenological approach, in the form of rheological models, appeared to be attractive. Rheological models are scale-independent representations of material behavior. They are essentially stress-strain-time models which can describe various types of rock behavior such as instantaneous and viscous behavior. They are composed of three basic elements, namely the Hookean (spring) element, the Newtonian (dashpot) element, and the St. Venant (slider) element (Figure 2-26). The constitutive laws represented by these models depend on the type of elements contained in the model and on the manner in which these elements are combined. To illustrate this, Figure 2-27a represents a model used to describe three types of creep observed in rock materials, namely, primary, secondary and tertiary creep. Figure 2-27b illustrates the displacements with time obtained with such a model. Several models have been applied in practice to describe the observed behavior of swelling rocks (e.g. Lo et al., 1978; Gaudin et al., 1981; Lombardi, 1974; Sun Jun et al., 1984; Nguyen Minh and Habib, 1988). Panet (1979) reported the limitations of rheological models used in underground works.
It has been recognized that most of the rheological models developed until now deal only with the deviatoric component of the total behavior, implying that the volumetric component is not taken into account (Table 2-7). These models are adequate to describe the phenomenon of creep, but not swelling. Swelling around tunnels is time-dependent volume increase of the ground produced by the adsorption of water in the zones directly adjacent to the excavation (Terzaghi, 1936; Terzaghi, 1946). If one wants to be consistent, one should also incorporate in these rheological models the time-dependent volumetric contribution to tunnel deformations. This volumetric contribution has been omitted in most previous studies. It has been introduced, however, in viscoelastic models by Sakurai (1977) and Lo and Yuen (1981). A solution for viscoplastic models has been presented by Lombardi (1984), in which the amount of volumetric strain due to swelling is associated to a change of the stress field which corresponds to a final plastic state of the rock mass directly adjoining a tunnel (Figure 2-28).

A rheological model for the phenomena of swelling and creep has been developed in a separate study (Aristorenas, 1989). Swelling is introduced through a variable bulk modulus, K, which is assumed to be finite (compressible material) and to vary over time whereas in conventional rheological (creep) models the bulk modulus, K, is assumed to be infinite (incompressible material). This new model has the capability of describing three stages, namely primary, secondary and tertiary, with both volumetric and deviatoric components. It is represented by a Burger model joined in series with an arrangement of a Bingham model (Figure 2-29). The results can be summarized as follows:

- In the viscoelastic domain, the contribution of the volumetric component is small compared to the deviatoric component of the radial displacement (Figure 2-30); e.g. for θ=90°, λ₀=0.5 and ν=0.25, it represents only 15%. Other values are reported in Table 2-8.

- In the case of viscoplasticity with perfectly plastic flow, the volumetric and deviatoric contributions are proportionally related by the term tg²φ' (Figure 2-31). For values of φ' larger
than 25°, the volumetric contribution becomes important with respect to the deviatoric one; i.e. \( \tan^2 \phi > 20\% \) (Figure 2-32).

In conclusion, the viscoelastic range of rheologic behavior can be modeled by existing approaches, which do not take the volumetric component into consideration. This is no longer the case for the viscoplastic range, where the volumetric component becomes important.

2.2.2.3 Derived rheological models

Derived rheological models are models incorporating laws which are obtained through curve fitting to results from either laboratory creep tests or in-situ convergence measurements.

Semple et al. (1973) developed a creep equation for altered rocks, similar to that of Singh and Mitchell (1969), expressed as:

\[
\varepsilon_a = X \exp (\beta \Delta) \left( \frac{t}{t_1} \right)^\lambda
\]  

(2-6)

where: \( \varepsilon_a \): axial strain (%),
\( \Delta \): stress level with respect to failure (%), \( q/q_f \)
\( X \): coefficient, extrapolated strain at zero stress-intercept of the log \( \varepsilon_a - \Delta \) relationship,
\( \beta \): stress exponent, slope of the log \( \varepsilon_a - \Delta \) relationship,
\( t \): elapsed time since application of the load corresponding to \( \Delta \),
\( t_1 \): unit reference time, generally \( t_1 \) is taken equal to one day,
\( \lambda \): time exponent, slope of the log \( \varepsilon_a - \log t \) relationship.

The drained creep behavior of undisturbed specimens of the overconsolidated (OCR=30) London Clay, monitored over a period of 3 1/2 years, was reported by Bishop and Lovenbury (1969). The test data are plotted in Figure 2-33, along with the line representing Equation 2-6 for the London Clay parameters, which best fit the test data. Semple et al. (1973) further incorporated their creep equation (Equation 2-6) in a model which is used to study the effect of time-dependent properties of altered rock on tunnel support requirements. A computer program was developed to solve the problem of an unlined or lined circular opening instantaneously created in an infinite, homogeneous, isotropic,
incompressible, creep-sensitive medium under an initially isotropic state of stress. The medium was assumed to be linear elastic with superimposed creep as given by Equation 2-6. The predicted radial stress on a tunnel lining with time is shown in Figure 2-34(a). For the same initial conditions, Figure 2-34(b) illustrates an example of predicted convergence of an unlined tunnel with time. The authors reported that a reasonable agreement is found between their theoretical results and the data obtained from published case histories.

Sulem (1983), Sulem et al. (1987a, 1987b) analyzed measurements of tunnel convergence with time and distance from tunnel face. The derived time function of tunnel convergence, obtained through curve fitting analysis, is given as:

\[ C(t) = 1 - \left( \frac{T^*}{t + T^*} \right)^n \]  (2-7)

where \( T^* \) is a characteristic parameter of the time dependent properties of the ground (analogous to relaxation time), \( t \) is the time and \( s \) is a parameter essentially equal to 0.3 (Sulem et al., 1987a).

This time function is represented in a semi-log t plot for different values of the characteristic parameter \( T^* \) (Figure 2-35). We observe that these curves (C(t)-curves) are identical, but they are shifted horizontally by one log t cycle, for each increase in \( T^* \) by one order of magnitude. Their shape suggests that they can be decomposed into two parts, namely a S-shape curve typical of consolidation/swelling, and a straight line typical of creep. Tangents are drawn to the two straight-line portions of the observed curves. The intersection of these lines defines a point which is assumed to be that of the end of consolidation/swelling, \( t_s \). Curves obtained from theory of 1-D consolidation/swelling have been reported on the same plot. Their time at the end of consolidation/swelling, \( t_{100} \), matches the \( t_s \) of the C(t)-curves. The strong similarity between these two sets of curves suggests that the observed behavior in the field is characterized by a consolidation/swelling process. Moreover, the straight lines of the C(t)-curves in Figure 2-35, which were attributed to creep effect, are
very similar to the theoretical creep curves obtained by Semple et al. (1973) in Figure 2-34b. This suggests that the observed tunnel convergences are primarily due to consolidation/swelling, and followed by creep.

2.2.2.4 Diffusion models

The two classes of models presented so far model swelling without considering the essential element of swelling: the water. The diffusion models directly incorporate the effect of water and thus provide a more rational approach.

The mechanical response of a fluid-saturated porous material is characterized by deformation-diffusion processes. These are consolidation or swelling depending on the relative volumetric change of the material. Consolidation is used to describe a volume decrease, while swelling is defined as a volume increase.

Diffusion models describe mechanical swelling as opposed to physico-chemical swelling. Mechanical swelling is modeled by a diffusion law and obeys only the laws of mechanics. Physico-chemical swelling, which is due to microscopic electrical and chemical interactions between the water and the clays particles, is not considered here.

The processes, which are caused by the presence of pore water, can be summarized as follows:

- excess pore water pressures are generated by externally applied loads,

- the effective stress governs the volumetric deformation of the material,

- the gradient of the pore water pressure acts like a body force which has to be taken into account in the equilibrium equations, and

- the excess pore water pressure is dissipated following a diffusion law.
There are two groups of theories in which laws of deformation (mechanics) and diffusion (hydraulics) have been incorporated, namely: theories with uncoupled models and theories with coupled models. The uncoupled models were developed first by Terzaghi (1923) in 1-D and extended to 3-D by Rendulic (1935). In these models, the solutions are obtained by solving the mechanical part independent of the hydraulic part.

In coupled models, however, the mechanical and the hydraulic parts are solved interactively. The simplest consistent theory which accounts for these coupled mechanical-hydraulic processes is the Biot theory of poroelasticity (e.g. Biot, 1941; Biot, 1956). It differs from the uncoupled theories in the following respects:

- a mechanism for the generation of pore water pressure, characterized by the Skempton B-coefficient (Skempton, 1954) is present,

- the effective stress governing the deformation of the porous solid is characterized by \( \sigma_{ij}' = \sigma_{ij} - \alpha u \delta_{ij} \), where \( \sigma_{ij}' \) is the effective stress, \( \sigma_{ij} \) is the total stress, \( \alpha \) is the Biot coefficient (0 ≤ \( \alpha \) ≤ 1) \(^6\), \( u \) is the pore pressure, and \( \delta_{ij} \) is the Kronecker delta, and

- the diffusion law for the pore water pressure is coupled to the rate of change of the volumetric deformation.

For the sake of clarity, the elastic models are treated separately from the plastic models.

**Elastic models**

Although the uncoupled and coupled consolidation/swelling theories have existed for several decades, their application to tunnels or related problems (e.g. boreholes) occurred only very recently.

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\(^6\) \( \alpha = 1 \) in the theories of Terzaghi (1923) and Rendulic (1935).
The first attempt was made by Carter and Booker (1982), who used a coupled model with incompressible constituents and developed solutions for the displacements and stress changes around a long circular opening in a saturated isotropic linear elastic medium. The authors showed that time-dependence of stress distribution and of associated displacements is due to the two-phase nature of the saturated elastic medium considered.

Detournay and Cheng (1988) extended the solution of Carter and Booker (1982) by using a coupled model with compressible constituents. They developed a complete set of transient solutions for the displacements and stress changes around a borehole in a non-hydrostatic stress field, in a saturated isotropic linear elastic medium. Their solutions are consistent with the partial solutions obtained by Rice and Cleary (1976). These authors provided solutions to the stress-diffusion problem around a circular hole for short-term and long-term behavior.

Elastoplastic models

Carter (1988) presented a semi-analytical solution for the dissipation of pore pressure around a vertical hole in a hydrostatic stress field. He used an isotropic linear elastic, perfectly plastic, porous medium with incompressible constituents. The author showed that yielding of the ground simultaneously induces excess negative pore pressures.

2.3 SUMMARY, DISCUSSION AND CONCLUSIONS

Two types of approaches, empirical and analytical, used to design tunnels in swelling rock have been presented. The empirical approach demonstrates that quantitative design parameters are very difficult to obtain. The analytical methods, despite a more "rational" approach, still display a lack of complete understanding of the phenomena of swelling and creep.
The consideration of water as an independent parameter is the essential difference between the models based on a swelling law and the rheological models on the one hand, and the diffusion models on the other. In particular, the first two models assume a one-phase non-porous medium, whereas the latter models consider saturated two-phase porous media.

In models based on swelling law, swelling is considered as a property of the material (not a characteristic of the material). It can be viewed as an inviscid property of the material since only the final, swelled state is taken into account. During unloading, the material changes its behavior and swells under a given state of stress governed by the swelling pressure. In other words, swelling depends on the state of stress. Thus, from a previously assumed isotropic or anisotropic linear elastic behavior, the material starts to expand or to swell following a given swelling law. This swelling law is determined experimentally in the oedometer and is characterized by three parameters, namely the swelling pressure and two swelling characteristics representing the rate of swelling with respect to changes in the stress state. These models are restricted to elasticity and are inconsistent with the observed swelling behavior.

In rheological models, swelling is modeled by assuming a degradation of the shear modulus with time. This represents a fundamental inconsistency because swelling, which is after all a volumetric change, is modeled with a shear parameter. The model developed by Aristorenas (1989) (with both deviatoric and volumetric components) indicated, that results obtained with models having only a deviatoric component are acceptable in viscoelasticity but are questionable in viscoplasticity. The good correspondence between the observed behavior and the behavior described by the model is inherent in the procedure. In fact, parameters of the model are found, from measured data sets, through curve fitting techniques. They are then introduced in the model, which duplicates the observed behavior. The accuracy obtained is a function of the number of parameters which enter the model. Thus, by choosing the appropriate model, it is not difficult to duplicate the measured data. However, not
much can be said about their capability to explain fully the physics behind the phenomenon these models are describing. In particular, it is very difficult to conduct laboratory experiments which can isolate the behavior of each component entering the model. Panet (1979) and Nguyen Minh and Habib (1984) report the difficulty in obtaining "true" parameters from classical laboratory tests. The applicability of the model in predicting how swelling will occur in another tunnel is thus uncertain, due to differences in the initial stress and geological conditions.

Derived rheological models have provided interesting results. The analysis of tunnel convergence and pressure against tunnel linings with time suggests that the excavation of a tunnel is followed first by a phase of consolidation/swelling and then by a phase of creep. The good agreement obtained by Semple et al. (1973) between the test data and the creep equation (Equation 2-6) on the one hand, and the predicted and observed convergence and radial stresses on tunnel linings on the other hand, are encouraging.

In diffusion models, the interaction between water and the solid skeleton of the porous material is modeled. Swelling is described by a diffusion law and obeys the laws of mechanics. These models describe mechanical swelling only. The interaction of water with the solid skeleton produces coupling effects which have been predicted by the theory and observed experimentally, primarily in soil mechanics. Because of the rationale of the model, it is easy to get parameters which have a physical meaning and can be, hopefully, used in field situations.

It is therefore recommended to describe swelling and creep in the direct vicinity of tunnels driven in argillaceous rock with, respectively, diffusion and creep models.

The existing diffusion models and creep models are discussed in more detail in Section 3.2.
<table>
<thead>
<tr>
<th>Case history</th>
<th>Reference</th>
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<tr>
<td>Boezberg Tunnel (Switzerland)</td>
<td>Beck and Golta (1972)</td>
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<tr>
<td></td>
<td>Grob (1976)</td>
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<tr>
<td></td>
<td>Einstein (1979)</td>
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<tr>
<td></td>
<td>Steiner and Metzer (1988)</td>
</tr>
<tr>
<td>Belchen Tunnel (Switzerland)</td>
<td>Grob (1972)</td>
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<td></td>
<td>Grob (1976)</td>
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<td></td>
<td>Einstein (1979)</td>
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<td>Kovari et al. (1987)</td>
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<td>Steiner and Metzger (1988)</td>
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<td>San Donato Tunnel (Italy)</td>
<td>Barla et al. (1986)</td>
</tr>
<tr>
<td>Test Tunnel at the South Saskatchewan River Dam (Canada)</td>
<td>Peterson (1954)</td>
</tr>
<tr>
<td></td>
<td>Peterson et al. (1960)</td>
</tr>
<tr>
<td></td>
<td>Bjerrum (1967)</td>
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<td>Clay Mine in Provins (France)</td>
<td>Terzaghi (1936)</td>
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<td></td>
<td>Proctor and White (1946)</td>
</tr>
<tr>
<td>Tunnel of Paris (France)</td>
<td>Terzaghi (1936)</td>
</tr>
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</table>

Table 2-1 References of Case Histories on Underground Works in Argillaceous Rocks
<table>
<thead>
<tr>
<th>Case history</th>
<th>Main observations</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Case A:</strong> Bözberg Tunnel (Switzerland)</td>
<td>1. Heave-log time curves show two distinct segments: a first linear segment with constant slope, a second segment with sharp increase in deformations (Fig.2-4). 2. Large voids and loose material observed behind tunnel lining along springlines in heavily damaged areas.</td>
<td>2.1.1.1</td>
</tr>
<tr>
<td><strong>Case B:</strong> Belchen Tunnel (Switzerland)</td>
<td>1. Stress-log time curves show two distinct segments: a first linear segment with constant slope, a second segment with a sharp increase in stresses (Fig.2-9). 2. Mean radial stresses and mean tangential stresses well correlated (Fig.2-10). 3. Extensometer measurements show that deformations occur inside rock mass at a distance of one tunnel diameter below the invert arch (Fig.2-11). 4. Water provided by inflow parallel and perpendicular to tunnel.</td>
<td>2.1.1.2</td>
</tr>
<tr>
<td><strong>Case C:</strong> San Donato Tunnel (Italy)</td>
<td>1. Water from aquifers responsible for damage observed on south side. 2. Horizontal convergence-log time curves approximated by two sets of lines thought to be representative of the swelling (steep lines) and creep phenomena occurring in rock mass (Fig.2-15, 2-16). 3. Plastic zone extending one diameter outside tunnel (theoretical estimation).</td>
<td>2.1.1.3</td>
</tr>
<tr>
<td><strong>Case D:</strong> Test Tunnel at the South Saskatchewan River Dam (Canada)</td>
<td>1. Gradual disintegration of diagenetic bonds accompanied by nonuniform swelling, resulting in an increase in joint intensity, reflected by the much greater variation in water content in disintegrated than in unaltered ground. 2. Horizontal convergence-log time curves show essentially one linear segment with constant slope (Fig.2-18).</td>
<td>2.1.1.4</td>
</tr>
<tr>
<td><strong>Case E:</strong> Clay Mine of Provins (France)</td>
<td>1. Excavation produces increase in water content from 35 to 70% in clay directly adjoining the tunnel springlines. 2. Horizontal stresses in the ground are larger than vertical stress. Backcalculation shows that ground is in or close to a state of passive failure.</td>
<td>2.1.1.5</td>
</tr>
<tr>
<td><strong>Case F:</strong> Tunnel of Paris (France)</td>
<td>1. Excavation produces changes in water content: large increase at exposed surface, and decrease further inside ground mass (Fig.2-21). 2. Experiment shows that it is unlikely that water is provided by the tunnel air.</td>
<td>2.1.1.6</td>
</tr>
</tbody>
</table>

**Table 2-2** Relevant Observations (in the Author's Opinion) from Case Histories of Underground Works in Argillaceous Grounds. Classification According to Case Histories
<table>
<thead>
<tr>
<th>Mineralogical Composition</th>
<th>South Side</th>
<th>North Side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>n</td>
<td>X</td>
</tr>
<tr>
<td>Quartz</td>
<td>14</td>
<td>16.5</td>
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<td>Feldspar</td>
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<tr>
<td>Carbonate</td>
<td>14</td>
<td>13.3</td>
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<td>Clay Minerals</td>
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<td>Chlorite</td>
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<td>Chlorite-Vermiculites</td>
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<td>9.6</td>
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<td>Vermiculites</td>
<td>14</td>
<td>11.4</td>
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<tr>
<td></td>
<td></td>
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<tr>
<td>Vermiculites-Montmorillonite</td>
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<td>-</td>
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<tr>
<td>Montmorillonite</td>
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<td>-</td>
</tr>
<tr>
<td>Illite-Montmorillonite</td>
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<td>17.5</td>
</tr>
<tr>
<td>Illite</td>
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<td>18.6</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>14</td>
<td>34.3</td>
</tr>
</tbody>
</table>

**Symbols used in Table:**

n: Number of samples studied  
X: Mean  
s: Standard deviation

**Table 2-3** Case C: San Danato Tunnel - Mineralogical Composition of the Clayshale of the Argille Scagliese  
(from Barla et al., 1986)
<table>
<thead>
<tr>
<th>Specific Issue</th>
<th>Main Observations</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial state of stress in the ground</td>
<td>1. Horizontal stresses in ground are larger than vertical stresses. Backcalculation shows that ground is in or close to a state of passive failure (Fig.2-19).</td>
<td>2.1.1.5</td>
</tr>
<tr>
<td>Plastic zone</td>
<td>1. Plastic zone extending one diameter outside tunnel (theoretical calculation).</td>
<td>2.1.1.3</td>
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<tr>
<td>Water</td>
<td>1. Water provided by inflow parallel and perpendicular to tunnel.</td>
<td>2.1.1.2</td>
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<td></td>
<td>2. Water from aquifers responsible for large damages observed on south side.</td>
<td>2.1.1.3</td>
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<td></td>
<td>3. Excavation produces increase in water content in clay directly adjoining the tunnel springlines.</td>
<td>2.1.1.5</td>
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<tr>
<td></td>
<td>4. Excavation produces changes in water content: large increase near to exposed surface, and decrease further inside ground mass (Fig.2-21).</td>
<td>2.1.1.6</td>
</tr>
<tr>
<td></td>
<td>5. Experiment shows that it is unlikely that water is provided by the tunnel air</td>
<td>2.1.1.6</td>
</tr>
<tr>
<td>Disintegration</td>
<td>1. Gradual disintegration of diagenetic bonds accompanied by nonuniform swelling, resulting in an increase in the joint intensity, reflected by the much greater variation in water content in disintegrated than in unaltered ground (Fig.2-17)</td>
<td>2.1.1.4</td>
</tr>
<tr>
<td>Stress-time relationship</td>
<td>1. Stress-log time curves show two distinct segments, a first linear segment with constant slope, a second segment with a sharp increase in stresses (Fig.2-9).</td>
<td>2.1.1.2</td>
</tr>
<tr>
<td>Deformation-time relationship</td>
<td>1. Heave-log time curves show two distinct segments: a first linear segment with constant slope, a second segment with sharp increase in deformations (Fig.2-4).</td>
<td>2.1.1.1</td>
</tr>
<tr>
<td></td>
<td>2. Extensometer measurements show that deformations occur inside rock mass at a distance of one tunnel diameter below the invert arch (Fig.2-11).</td>
<td>2.1.1.2</td>
</tr>
<tr>
<td></td>
<td>3. Horizontal convergence-log time curves approximated by two sets of lines thought to be representative of the swelling (steep lines) and creep phenomena occurring in rock mass (Fig.2-15, 2-16).</td>
<td>2.1.1.3</td>
</tr>
<tr>
<td></td>
<td>4. Horizontal convergence-log time curves show essentially one linear segment with constant slope (Fig.2-18).</td>
<td>2.1.1.4</td>
</tr>
</tbody>
</table>

**Table 2-4** Relevant Observations (in the Author's Opinion) from Case Histories of Underground Works in Argillaceous Grounds. Classification According to Specific Issues
Rock load $H_p$ in feet of rock on roof of support in tunnel with width $B$ (ft) and height $H_r$ (ft) at depth of more than 1.5 ($B + H_r$).\(^\text{1}\)

<table>
<thead>
<tr>
<th>Rock Condition</th>
<th>Rock Load $H_p$ in feet</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Hard and intact</td>
<td>zero</td>
<td>Light lining, required only if spalling or popping occurs.</td>
</tr>
<tr>
<td>2. Hard stratified or schistose(^\text{2})</td>
<td>0 to 0.5 $B$</td>
<td>Light support.</td>
</tr>
<tr>
<td>3. Massive, moderately jointed</td>
<td>0 to 0.25 $B$</td>
<td>Load may change erratically from point to point.</td>
</tr>
<tr>
<td>4. Moderately blocky and seamy</td>
<td>0.25 $B$ to 0.35 ($B + H_r$)</td>
<td>No side pressure.</td>
</tr>
<tr>
<td>5. Very blocky and seamy</td>
<td>(0.35 to 1.10) ($B + H_r$)</td>
<td>Little or no side pressure.</td>
</tr>
<tr>
<td>6. Completely crushed but chemically intact</td>
<td>1.10 ($B + H_r$)</td>
<td>Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs</td>
</tr>
<tr>
<td>7. Squeezing rock, moderate depth</td>
<td>(1.10 to 2.10) ($B + H_r$)</td>
<td>Heavy side pressure, invert struts required. Circular ribs are recommended.</td>
</tr>
<tr>
<td>8. Squeezing rock, great depth</td>
<td>(2.10 to 4.50) ($B + H_r$)</td>
<td></td>
</tr>
<tr>
<td>9. Swelling rock</td>
<td>Up to 250 ft. irrespective of value of ($B + H_r$)</td>
<td>Circular ribs required. In extreme cases use yielding support.</td>
</tr>
</tbody>
</table>

1. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty per cent.

2. Some of the most common rock formations contain layers of shale. In an unweathered state, these shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

---

**Table 2-5** Terzaghi’s Rockload Recommendations
(from Proctor and White, 1946)
<table>
<thead>
<tr>
<th>Model</th>
<th>Laboratory test 1-D oedometer</th>
<th>Field situation swelling zone</th>
<th>Analytical method</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stress state</td>
<td>Swelling law</td>
<td>Strain</td>
<td>Displacement</td>
</tr>
<tr>
<td>Grob (1972)</td>
<td>X</td>
<td>H-A</td>
<td>semi-log.</td>
<td>X</td>
</tr>
<tr>
<td>Einstein et al. (1972)</td>
<td>X</td>
<td>H-A</td>
<td>semi-log.</td>
<td>X</td>
</tr>
<tr>
<td>Wittke &amp; Rissler (1976)</td>
<td>X</td>
<td>H-A</td>
<td>semi-log.</td>
<td>X</td>
</tr>
<tr>
<td>Gysel (1977)</td>
<td>X</td>
<td>H-A</td>
<td>semi-log.</td>
<td>X</td>
</tr>
<tr>
<td>Kovari et al. (1983)</td>
<td>X</td>
<td>K&amp;al.</td>
<td>semi-log. linear polynomial</td>
<td>X</td>
</tr>
<tr>
<td>Schwesig &amp; Duddeck (1985)</td>
<td>X</td>
<td>H-A</td>
<td>nonlinear</td>
<td>X</td>
</tr>
<tr>
<td>Froehlich (1986)</td>
<td>X</td>
<td>H-A</td>
<td>semi-log.</td>
<td>X</td>
</tr>
</tbody>
</table>

**Symbols used in Table:**

* assuming an isotropic linear elastic material except for Froehlich (1986) who assumed an anisotropic linear elastic material.

# but discussed briefly.

H-A Hudig and Amberg (1970) testing procedure (Figure A-7)
The swelling pressure is denoted by $\sigma_s(H)$.

K&al. Kovari et al. (1981) testing procedure (Figure A-3)
The swelling pressure is denoted by $\sigma_s(K)$.

FEM Finite Element Method.
<table>
<thead>
<tr>
<th>Model</th>
<th>Reference</th>
<th>Volumetric contribution</th>
<th>Initial state of stress</th>
<th>Solution as</th>
<th>Lining as</th>
<th>Case history</th>
<th>Rock type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pointing-Thomson</td>
<td>Gill et al. (1970) Sakurai (1977)</td>
<td>No</td>
<td>(\lambda_0\neq1)</td>
<td>CF</td>
<td>Yes</td>
<td>Sakurai (1977)</td>
<td>Basaltite and liparite</td>
</tr>
<tr>
<td></td>
<td>Berset and Nguyen Ming (1983)</td>
<td>No</td>
<td>(\lambda_0=1)</td>
<td>FEM</td>
<td>Yes</td>
<td>Nguyen Ming and Habib (1985)</td>
<td>Marls</td>
</tr>
<tr>
<td></td>
<td>Nguyen Ming and Habib (1985)</td>
<td>No</td>
<td>(\lambda_0=1)</td>
<td>NA</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Derived laws</td>
<td>Ayers (1969) Semple et al. (1973)</td>
<td>No</td>
<td>(\lambda_0=1)</td>
<td>CF</td>
<td>Yes</td>
<td>Semple et al. (1973)</td>
<td>Altered magmatic, metamorphic &amp; sedimentary rocks</td>
</tr>
<tr>
<td></td>
<td>Sulem et al. (1987b)</td>
<td>No</td>
<td>(\lambda_0=1)</td>
<td>FD</td>
<td>No</td>
<td>Sulem et al. (1987a)</td>
<td>Marly soil</td>
</tr>
<tr>
<td></td>
<td>Ladanyi and Gill (1984)</td>
<td>No</td>
<td>(\lambda_0=1)</td>
<td>CF</td>
<td>Yes</td>
<td>Sulem et al. (1987b)</td>
<td>Shistose rock</td>
</tr>
</tbody>
</table>

**Symbols used in table:**
- **CF**: Closed form
- **FD**: Finite difference
- **FEM**: Finite Element Method
- **NA**: Not available
- \(\lambda_0\): Coefficient of internal stress at rest
<table>
<thead>
<tr>
<th>$\theta$</th>
<th>$\lambda_0$</th>
<th>$\nu$</th>
<th>$u_v/u_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0°</td>
<td>0.5</td>
<td>0.15</td>
<td>-0.609</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>-0.400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.35</td>
<td>-0.222</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.45</td>
<td>-0.069</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>0.15</td>
<td>0.144</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>0.105</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.35</td>
<td>0.065</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.45</td>
<td>0.022</td>
</tr>
<tr>
<td>90°</td>
<td>0.5</td>
<td>0.15</td>
<td>0.209</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>0.154</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.35</td>
<td>0.095</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.45</td>
<td>0.033</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>0.15</td>
<td>-0.264</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>-0.182</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.35</td>
<td>-0.105</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.45</td>
<td>-0.034</td>
</tr>
</tbody>
</table>

$U_d$ deviatoric component of the radial viscoelastic displacement
$U_v$ volumetric component of the radial viscoelastic displacement
$\lambda_0$ coefficient of lateral stress at rest
$\nu$ Poisson's ratio
$\theta$ angular coordinate

**Table 2-8** Ratio of the Volumetric to the Deviatoric Component of the Radial Viscoelastic Displacement
Figure 2-1  Case A: Bözberg Tunnel - Longitudinal Profile and Deformation Measurements from 1923 to 1954
(from Grob, 1976)
Figure 2-2  Case A: Bösberg Tunnel – Invert Heave from 1923 to 1954
(from Grob, 1976)
Figure 2-3  Case A: Bözberg Tunnel - Left Side of Figure: Abutment Convergence and Void Behind Abutment, Right Side of Figure: Reconstructed Support with Invert Arch
(from Beck and Golta, 1972)
Figure 2-4  Case A: Bözberg Tunnel - Plot of Heave versus Log-Time from 1923 to 1954 for Measuring Sections in Swelling Rock (Marl and Opalinus Clayshale) (Data from Grob, 1976; see also Figure 2-2)
Figure 2-5  Case B: Belchen Tunnel - Longitudinal Profile and Invert Heave (from Grob, 1972)
Figure 2-6  Case B: Belchen Tunnel - Drainage Pipe Lifted and Laterally Compressed
Figure 2-7 Case B: Beichen Tunnel - Cross-section with Original (1) and Reconstructed (2) Invert Arches
Figure 2-8 Case B: Belchen Tunnel - Destroyed Invert Arch [The thick concrete slab on the righthand side was to serve as a base for the roadway. This was also cracked in some locations.] (Photo by G. Amberg, ETH-Z)
Figure 2-9 Case B: Behavior Tunnel - Plot of Radial and Tangential Stresses versus Log-Time for Measuring Section in Argillaceous Rock (Oppitz and Traeger 1980) (Data from Stelner and Heizger, 1988)
Figure 2-10 Case B: Belchen Tunnel - Plot of Mean Tangential Stress versus Mean Radial Stress for Measuring Section in Argillaceous Rock (Opalinus Clayshale) (Data from Grob, 1972; and Steiner and Metzger, 1988)
Figure 2-11 Case B: Belchen Tunnel - Extensometer Measurements in Invert and Springlines. Radial Swelling Strain in the Adjoining Swelling Zone Around the Tunnel (From Kovari et al., 1987)
Argille Scagliose Complex
Clay particle arrangement

disarranged, with curved surfaces, contorted and twisted

preferred orientation of the clay minerals parallel to nearly vertical fault plane

North

South

Geological Conditions:
(1) Soil-like
(2) Rock-like

Legend:
- Lacustrine deposits:
- Albarene formation:
- Argille Scagliose complex:
- Macigno formation:

sand and clay layers
mostly marly limestones with clay shale layers and conglomerates; jointed rock mass, highly tectonized, aquifer
marly clay and clay shales; jointed rock mass, highly tectonized
thick bedded sandstones and siltstones, with shale and silty shale interbeds; jointed rock mass, highly tectonized; aquifer

Figure 2-12 Case C: San Donato Tunnel - Geological Longitudinal Profile (from Barla et al., 1986)
Figure 2-13 Case C: San Donato Tunnel - Construction Method in Poor Ground Conditions (from Barla et al., 1986)
Figure 2-14 Case C: San Donato Tunnel - Plot of Horizontal Convergence versus Time of Steel Sets (from Barla et al., 1986)
Figure 2-15 Case C: San Donato Tunnel - Plot of Horizontal Convergence versus Log-Time of Four Steel Sets (from Figure 2-14, data from Barla et al., 1986)
Figure 2-16 Case C: San Donato Tunnel - Plot of Horizontal Convergence versus Log-Time of Four Steel Sets. Superposition of Figures 2-7a through 2-7d
Figure 2-17 Case D: Test Tunnel at the South Saskatchewan River Dam - Variation in Water Content and Atterberg Limits in Unweathered and Partially Weathered Bearpaw Shale as Observed in a Section of the Test Tunnel (from Bjerrum, 1967)
Figure 2-18 Case D: Test Tunnel at the South Saskatchewan River Dam - Horizontal Convergence versus Log-Time for Unweathered, Partially Weathered, and Completely Weathered Bearpaw Shale (from Bjerrum, 1967)
Figure 2-19 Case E: Clay Mine in Provins, near Paris - Estimated State of Stress in the Ground is Close to or in a State of Passive Failure in a $t-\sigma$ Plot
Figure 2-20 Case F: Tunnel In Paris - Cross-Section
(from Terzaghi, 1936)
Note: \( w_0 \): natural water content before excavation

\( w_1 \): water content after excavation

Figure 2-21 Case F: Tunnel In Paris - Observed Changes in Water Content of the Clay Adjoining the Tunnel (from Serzaghi, 1936)
Figure 2-22 Comparison of Rates of Deformation of Various Tunnels in Argillaceous Rocks (from Prairie Farm Rehabilitation Administration (PPRA), 1951; Einstein et al., 1972; Grob, 1976; Ward et al., 1976; Manton, 1978; Pantel, 1979; Nguyen Hing et al., 1985)

Saskatchewan: completely weathered Bearpaw shale
(Sec.2.1.1.4)

Saskatchewan: partially weathered Bearpaw shale
(Sec.2.1.1.4)

Switzerland (NAGRA): marlstone

Bötzberg: Opalinus clayshale and (Sec.2.1.1.1) marl
marn

Bötzberg: marl (Sec.2.1.1.1)

Saskatchewan: unweathered (Sec.2.1.1.4) Bearpaw shale

Ontario: Sandstone, limestone and shale

Ontario: Dundas shale

Note: The relative slopes of the lines represent the percentage reduction of tunnel dimension per log-cycle of time

1 log cycle of time
In stiff swelling ground it is necessary to allow the ground to squeeze some undetermined amount to soften it. If the ground is not sufficiently soft to extrude between the ribs when the shrinkage provided in the crush lattices (Fig. 113) is used up, slots are excavated beyond the ribs, as shown here. This is repeated until extrusion between the ribs is established. The squeeze is then allowed to run its course before concreting.

(b) Over-mining in stiff swelling ground to induce softening

Figure 2-23 Ground Support Behavior in Swelling Rock
(from Proctor and White, 1946)
Figure 2-24 Classification Diagram (from Barton et al., 1974)
First dry loading
Dry unloading
Second dry loading
Addition of water under constant axial stress $\sigma_a$, possible swelling strain
Swell unloading, final swelling strain as function of axial stress

$K$, $W$: characteristic constants of the rock (Equation 2-4)

N.B.: the swelling pressure $\sigma_s(H)$ is defined by the intersection of the extrapolated second dry loading curve (3) with the prolonged swelling curve (5)

**Figure 2-25** 1-D Swelling Law (from Grob, 1972) and Determination of the Swelling Pressure in the Huder and Amberg (1970) Swelling Test Procedure
Figure 2-26 Basic Rheological Elements: (a) Hookean, (b) Newtonian, and (c) St. Venant
Figure 2-27 Three Creep Stages with Combined Burger and Bingham Models: (a) Schematic Diagram and (b) Convergence-Time Curve
Figure 2-28 Volumetric Strain due to Swelling Associated to a Change of the Stress Field Corresponding to the Plastification of the Rock Directly Adjoining a Tunnel (from Lombardi, 1984)
Figure 2-29 Viscoelastic Volumetric and Deviatoric Components in a Burger Model
Figure 2-30 Viscoelastic Radial Displacements: (a) $\lambda_0 = 1.0$ and 
(b) $\lambda_0 = 2.0$
\[ \dot{\varepsilon}_{vp_{rd}}: \text{deviatoric component of the viscoplastic radial strain rate} \]

\[ \dot{\varepsilon}_{vp_{rv}}: \text{volumetric component of the viscoplastic radial strain rate} \]

\[ \varepsilon_{vp_{r}}: \text{viscoplastic radial strain rate} \]

\[ \varepsilon_{vp_{r}} = \varepsilon_{vp_{r1}} + \varepsilon_{vp_{r2}} = \left[ (\varepsilon_{vp_{rd}})^2 + (\varepsilon_{vp_{rv}})^2 \right]^{1/2} \]

**Figure 2-31** Components of the Viscoplastic Radial Strain Rate
Figure 2-32 Variation of $\text{tg}^2 \phi$ as a function of $\phi$
Figure 2-33 Creep Curves for Drained CIDC(L) London Clay on Undisturbed Specimens (OCR=30). Comparison of Observed and Predicted Creep Curves (Data from Bishop and Lovenbury, 1969)
Figure 2-34 Example of Theoretical Results Obtained by Semple et al. (1973): (a) Predicted Radial Stress on a Tunnel Support with Time, (b) Predicted Ratio of Inwards Displacement to the Initial Radius for an Unsupported Tunnel with Time (Both Figures from Semple et al., 1973)
Figure 2-35 Time Function of Tunnel Convergence Obtained by Sulem (1983) and Sulem et al. (1987a) Compared with 1-D Consolidation Curve of Terzaghi
CHAPTER 3

INVESTIGATION OF TUNNEL BEHAVIOR IN ARGILLACEOUS ROCKS

The understanding of tunnel behavior requires knowledge of the short-term and long-term behavior of the surrounding rock mass. This chapter presents the results of a preliminary investigation which has been conducted in order to get an overall picture of the behavior of the saturated porous rock mass around a tunnel. In the first section, a conceptual model is proposed as a basis for further discussion. Existing coupled diffusion and creep models, developed for modeling the rock mass around circular openings (not necessarily tunnels) are studied in the second section. Final comments in the third section provide the basis for the subsequent steps to be undertaken.

3.1 NEW CONCEPTUAL MODEL

This original conceptual model was presented by the author at a special workshop on Swelling Rock during the 5th International Congress on Rock Mechanics in Montreal. It has been published in the Proceedings of the Congress by Bellwald and Einstein (1987). It will be discussed very briefly here; the reader is referred to Appendix B for further details.

The original conceptual model presents a new approach for the design of tunnels in argillaceous rocks. It uses the "Stress Path" method (Lambe, 1967) and assumes that the argillaceous rock behaves like an uncemented overconsolidated clay.
The "Stress Path" method approach consists of the following three steps:

(1) Obtain the elastic/poroelastic solutions to estimate the instantaneous changes in stress (and pore water pressure) due to the excavation. In our case, this is done at various elements in the rock mass located in the springlines and crown/invert.

(2) Impose these same changes in stress (and pore water pressure) to the rock elements in laboratory tests and then measuring the resulting strains.

(3) Integrate the strains to get displacements at various locations around the tunnel.

Three phases have been recognized and should be studied when designing a tunnel in argillaceous rock: (1) the original (natural) phase with the initial state of stress, (2) the undrained phase, and (3) the drained phase. The drained phase should be subdivided into two subphases, namely, consolidation/swelling, and creep.

3.2 LITERATURE REVIEW OF PREVIOUS ANALYSES

This section discusses the existing solutions for modeling time-dependent behavior around circular openings in continuous, homogeneous, saturated and porous media (Section 2.2.2). For the sake of clarity, the diffusion models, which model consolidation/swelling, are studied separately from the creep models. Also, the diffusion models which deal only with linear elastic behavior of the rock are treated separately from those involving plastic behavior.

3.2.1 CONSOLIDATION/SWELLING

3.2.1.1 Elastic models

The general theory of poroelasticity introduced first by Biot (1941) and then reexamined from various perspectives (e.g. Biot, 1955;
Biot, 1956; Rice and Cleary, 1976), serves as a basis for the elastic models. The fundamental assumptions of Biot theory have been summarized in Section 2.2.2.4. The approach followed here is that of Rice and Cleary (1976), who have provided an elegant reformulation of the Biot theory in terms of easily identifiable quantities and material constants. This approach has the additional advantage that it allows one to interpret directly the short- and long-term behavior of a poroelastic system.

Detournay and Cheng (1988) used a coupled model of poroelasticity and developed solutions for the displacements and stress changes around a long vertical borehole in a saturated isotropic linear elastic medium. The rock is assumed to behave as a poroelastic material with compressible constituents, following the Biot theory. The initial in-situ state of stress is assumed to be anisotropic in the horizontal plane. The pore pressure is assumed to be equal to $u_0$ at the depth considered. It is further assumed that the borehole perimeter constitutes a drainage boundary. This situation is identical to that of a tunnel at large depth excavated in a porous medium subject to an anisotropic state of stress (with a total vertical stress, $\sigma_{v0}$, and a total horizontal stress, $\sigma_{h0}$, acting in the plane of the tunnel section), and an initial pore water pressure, $u_0$, at the level of the tunnel axis.

Carter and Booker (1982) presented a special case of the solution of Detournay and Cheng (1988) for incompressible constituents. They developed solutions for the displacement and stress changes around a long circular opening in a saturated isotropic linear elastic medium. The tunnel is located at a depth, $h$, which is large when compared with the nominal radius, $R_0$, of the tunnel opening ($R_0/h << 1$). The initial in-situ state of stress may therefore be idealized as uniform with a total vertical stress, $\sigma_{v0}$, and a total horizontal stress, $\sigma_{h0}$, in all horizontal directions. The pore water is assumed to have an initial pressure $u_0$ at the level of the tunnel axis. Carter and Booker (1982) presented solutions for two extreme drainage conditions at the tunnel perimeter, $r=R_0$: (1) permeable tunnel; i.e. with drainage at the tunnel
perimeter, and (2) impermeable tunnel; i.e. without drainage at the tunnel perimeter.

**Governing equations**

The total stresses, $\sigma_{ij}$ (positive in compression), and the pore water pressure, $u$ (positive), are chosen as the basic kinetic variables. The corresponding conjugate kinematic quantities are the solid strain, $e_{ij}$ (positive in compression), derivable from an average solid displacement vector, $U_i$, and the variation of fluid content per unit reference volume, $\xi$.

The constitutive equations can be written in terms of the above quantities as follows:

$$\sigma_{ij} = 2G e_{ij} + \frac{2G\nu}{1-2\nu} e_{ij} + \alpha u \delta_{ij} \quad (3-1)$$

and:

$$u = \frac{2G(1+\nu)}{3(1-2\nu)} e - \frac{2G(1-2\nu)(1+\nu)^2}{9(\nu-u)(1-2\nu)} \xi \quad (3-2)$$

in which $G$ is the shear modulus, $e=\varepsilon_{kk}$ is the solid volumetric strain, $\alpha$ is the Biot coefficient of effective stress (Biot, 1941), $\delta_{ij}$ is the Kronecker delta, $B$ is the Skempton pore pressure coefficient (Skempton, 1954), and $\nu_u$, $\nu$ are the undrained and drained Poisson's ratios, respectively.

The Biot coefficient of effective stress, $\alpha$, is defined in terms of drained parameters only, as (Biot, 1941):

$$\alpha = 1 - \frac{K}{K_a} \quad (3-3)$$

where:  $K$: drained bulk modulus of the rock skeleton,

Figure 3-1 illustrates the variation of $\alpha$ as a function of the drained bulk modulus, $K$, assuming a value for the solid particles $K_a=37.037\times10^5$MPa (Bishop, 1973; Coynor, 1984 (experimental values for sandstones)). In Figure 3-1 the range of $\alpha$-values to be expected in argillaceous rocks used in this study is indicated. It will be noted
that for very small values of \( K \), say \( K<500 \) MPa, the values of \( \alpha \) are, for all practical purposes, equal to unity.

Assuming that all pores are connected, the Skempton \( B \)-parameter can be expressed as (e.g. Bishop, 1973):

\[
\frac{1}{B} = 1 + n_0 \frac{K(1-K_{f1}/K_0)}{K_{f1}(1-K/K_0)} = 1 + 4.5308 \times 10^{-4} \times n_0 \frac{K}{\alpha} \quad (3-4)
\]

where: \( n_0 \): initial porosity,
\( K_{f1} \): inverse of fluid compressibility,
\( K_{f1}=2.083 \times 10^3 \) MPa = \( 1/C_{f1} \), where \( C_{f1}=48 \times 10^{-5} \) MPa\(^{-1} \).

Figure 3-2 illustrates the variation of \( B \) as a function of the drained bulk modulus, \( K \), for various values of the initial porosity, \( n_0 \). Assuming that argillaceous rocks which usually cause problems in tunneling have a bulk modulus \( K<15000 \) MPa and an initial porosity \( n_0<30\% \), allows one to determine a corresponding range of \( B \)-values (Figure 3-2). For an argillaceous rock with an initial porosity of 15\%, the \( B \)-value ranges from 1.0 to 0.4 for a drained bulk modulus between, say, 500 MPa (very soft rock) and 15000 MPa (very stiff rock).

In soil mechanics, where the compressibility of the soil skeleton is large relative to those of the solid particles and of the water, one can assume that \( \alpha=1 \). This value of \( \alpha=1 \) implies that \( B=1 \) for all practical purposes (Equations 3-3 and 3-4).

Equation (3-2) has been written, for convenience, in terms of \( \nu_u \), the undrained Poisson's ratio. Bishop and Hight (1977) discussed this ratio and showed that it can be expressed in terms of drained parameters as:

\[
\nu_u = \frac{3\nu + B(1-2\nu)\alpha}{3 - B(1-2\nu)\alpha} \quad (3-5)
\]

where \( \nu \) is the drained Poisson's ratio, \( \alpha \) is the Biot coefficient of stress, and \( B \) is the Skempton parameter.

Figure 3-3 illustrates the relationship between \( \nu_u \) as a function of \( B \) for different values of \( \nu \) for \( \alpha=1 \), \( \alpha=0.8 \) and \( \alpha=0.6 \). \( \alpha=1 \) corresponds to the soil mechanics case, and \( \alpha=0.6 \) corresponds to that of a stiff
cemented rock. It will be noted that \( v_u = 0.5 \) when \( B = 1.0 \) and \( \alpha = 1 \) for any value of \( v \).

Biot and Willis (1957) demonstrated that by the use of the effective stress \( \sigma_{ij}^* = \sigma_{ij} - \alpha \sigma \delta_{ij} \), known as the Biot effective stress, the problem of deformation in a linear porous material with pore pressure can be reduced to a linear elastic problem in a non-porous material (Equation 3-1). Equation 3-2 indicates that the pore pressure depends linearly on both the deformation of the porous solid and the variation of fluid content.

In the case where the material is incompressible \((v_u = 0.5, B = 1.0)\), Equations 3-1 and 3-2 uncouple, and Equation 3-2 yields:

\[
e = \zeta
\]  

(3-6)

Equation 3-6 means physically that the volume change is equal to the variation of fluid content per unit reference volume. This one-to-one relationship is expected because of the incompressibility of the constituents of the porous medium.

For both coupled models with incompressible or compressible constituents, a complete description of the governing equations include:

- the equilibrium equation:

\[
\sigma_{ij,j} = 0
\]  

(3-7)

- Darcy’s law:

\[
q_i = -k u_{,i}
\]  

(3-8)

where \( q_i \) is the fluid flux, \( k \) the permeability coefficient; the comma implies derivation with respect to the variable following it.
and the continuity equation of the fluid phase:

\[
\frac{\partial \xi}{\partial t} + q_{i,1} = 0
\]  

(3-9)

where \( q_{i} \) is the specific discharge vector, and \( k \) the coefficient of permeability. It will be noted that for incompressible material \( \frac{\partial \xi}{\partial t} = 0 \).

**Field equations**

The basic Equations 3-1, 3-2, and 3-7 to 3-9 can be combined to yield various field equations for the solution of initial- or boundary-value problems. For the present purpose, they are presented as a Navier equation with a coupling term and a diffusion equation in terms of the displacement vector, \( U_i \), and variation of the fluid content, \( \zeta \).

\[
\nabla^2 U_i + \frac{1}{1-2\nu_u} e_{i,1} - \frac{2B(1+\nu_u)}{3(1-2\nu_u)} \zeta_{,1} = 0
\]  

(3-10)

\[
\frac{\partial \zeta}{\partial t} - c\nabla^2 \zeta = 0
\]  

(3-11)

where \( c = K_p \left[ \frac{2G(1-\nu)}{1-2\nu} \right] \left[ \frac{3B^2(1+\nu_u)^2(1-2\nu)}{9(1-\nu_u)(\nu_u-\nu)} \right] \) is a generalized consolidation coefficient (Rice and Cleary, 1976). \( K_p = k/\nu_u \), where \( k \) is the coefficient of permeability, and \( \nu_u \) is the unit weight of the pore fluid (water). For incompressible constituents, the term in the last bracket is unity, and \( c = \frac{2K_p G(1-\nu)}{1-2\nu} \), which is the 1-D coefficient of consolidation (McTigue et al., 1986).

Alternatively, Equation 3-10 can be written in terms of excess pore pressure, \( \Delta u \):

\[
\frac{\partial \Delta u}{\partial t} - c\nabla^2 \Delta u = \frac{2B(1+\nu_u)}{3(1-2\nu_u)} \frac{\partial e}{\partial t}
\]  

(3-12)
For boundary-value problems characterized by the application of constant boundary conditions, the initial \((t=0^+)\) and the final \((t=\infty)\) solutions are simply obtained by solving an elasticity problem with undrained and drained elastic constants, respectively.

Consider first the conditions at \(t=0^+\). Initially, upon the application of the boundary conditions, fluid has not yet escaped from the pores; i.e. \(\zeta=0\), and Equation 3-10 reduces to the classic Navier equation with undrained Poisson's ratio, \(\nu_u\):

\[
\nabla^2 u_1 + \frac{1}{1-2\nu_u} e_{r_1} = 0
\]

(3-13)

Combination of Equations 3-1 and 3-2 shows that the initial excess pore pressure \(\Delta u^{0+}\) is directly proportional to the change in octahedral stress, \(\Delta\sigma_{\text{oct}} = \Delta\sigma_{kk}/3\), at short times:

\[
\Delta u^{0+} = \frac{B}{3} \Delta(\sigma_{kk})^{0+}
\]

(3-14)

where repeated indices imply summation.

At time \(t=\infty\), the excess pore pressure has dissipated; i.e. \(\Delta u^\infty = 0\). Then according to Equation 3-2 and using Equation 3-5:

\[
\chi^\infty = \frac{3(\nu_u - \nu)}{B(1-2\nu)(1+\nu_u)} \sigma^\infty = \xi e^\infty
\]

(3-15)

where \(\nu_u\) and \(\nu\) are the undrained and drained Poisson's ratios, respectively.

Substitution of the derivative of the above expression for \(\chi^\infty\) into Equation 3-10, yields the classic Navier equation with the drained Poisson’s ratio, \(\nu\):

\[
\nabla^2 u_1 + \frac{1}{1-2\nu} e_{r_1} = 0
\]

(3-16)
Solutions to the tunnel problem

The solutions of both models of Carter and Booker (1982) and Detournay and Cheng (1988), which describe the behavior of a poroelastic medium around a circular cavity, are reported here. The geometry of the problem considered is illustrated in Figure 3-4. The approach followed to solve the problem is identical in both models; one assumes an instantaneous tunnel excavation (undrained phase) followed by time effects (drained phase). The stresses acting along the boundary of the tunnel opening are decomposed into isotropic and anisotropic components as follows:

- Total stresses: isotropic and anisotropic
- Pore water pressure: isotropic

The isotropic and anisotropic components of the total stresses are defined respectively as: \( p_0 = (\sigma_{v0} + \sigma_{h0})/2 \) and \( q_0 = |\sigma_{v0} - \sigma_{h0}|/2 \), where \( \sigma_{v0} \) is the initial vertical total stress and \( \sigma_{h0} \) the initial horizontal stress. The initial pore pressure is denoted \( u_0 \).

Solutions are found for three fundamental loading modes (Figure 3-5):

- **Mode 1**: axisymmetric. Removal of the isotropic component of the total stresses at the tunnel boundary, \( p_0 = (\sigma_{v0} + \sigma_{h0})/2 \)

- **Mode 2**: axisymmetric. Removal of pore water pressure at the tunnel boundary, \( u_0 \)

- **Mode 3**: antymmetric. Removal of the anisotropic component of the total stresses at the tunnel boundary, \( q_0 = |\sigma_{v0} - \sigma_{h0}|/2 \)

For every mode, solutions are obtained in closed form in the Laplace transform space for each stress or displacement component and are then inverted using a numerical algorithm. The final solution is obtained by invoking the principle of superposition and adding the solutions of each mode. The main results for each mode can be summarized as follows:
Mode 1 and mode 2

Computation of the displacements, stresses and excess pore pressure induced by the fundamental loading mode 1 and mode 2 can be carried out in a parallel manner. Indeed, it can be demonstrated (Sills, 1975; Rice and Cleary, 1976) that, in the case of a tunnel in an infinite domain and under axisymmetric loading, Equation 3-12 uncouples to give a homogeneous diffusion equation.

\[
\frac{\partial^2 \Delta u}{\partial r^2} + \frac{1}{r} \frac{\partial \Delta u}{\partial r} = \frac{1}{c} \frac{\partial \Delta u}{\partial t} \tag{3-17}
\]

The pore pressure can therefore be solved independently of other quantities such as strains or stresses. More specifically, we have:

In mode 1, solutions for displacements and stresses can be expressed in terms of normalized parameters as function of \( r/R_0 \) as shown in Table 3-2. There is no mechanism for pore pressure generation because the rock deformations are entirely associated with the deviatoric strain; therefore no volume change but only pure shear mechanisms are involved. Solutions are independent of the bulk modulus of the rock skeleton, \( K \), and of the drained Poisson’s ratio, \( \nu \). Changes in displacement and stress field occur instantaneously; i.e. they are independent of time. Finally, solutions for incompressible or compressible constituents are identical.

In mode 2, solutions for displacements and stresses can be expressed in terms of normalized parameters as function of \( r/R_0 \) and \( T_c = c t/(R_0)^2 \) as shown in Table 3-3. The time factor, \( T_c \), is a non-dimensional time which is introduced for convenience. For the model with incompressible constituents, solutions can easily be found by setting \( \alpha = 1 \) (Equation 3-5).

Mode 3

The antimetric mode 3 is the only loading mode which involves coupling between deformation and diffusion processes. More specifically, the solutions for displacements and stresses can be expressed in terms of normalized parameters as functions of \( r/R_0 \), \( T_c \), \( V \), \( \nu_u \), \( B \), and \( \theta \),
where \( \theta \) is the angular coordinate. They are given in Table 3-4. For material with incompressible constituents, solutions can be also found by setting \( V_u=0.5 \) and \( B=1.0 \) \((a=1.0)\). For a set of values of \( V \), \( V_u \), \( B \), and \( \theta \), they can be expressed either as functions of \( r/R_0 \) or as functions of \( T_c \).

Note that all the solutions are expressed in terms of easily determinable parameters. In particular, the distribution and history of stresses are given in terms of total stresses and pore pressure.

Relevant results of the solutions to the tunnel problem

The most important results for the case of a permeable tunnel, obtained by Carter and Booker (1982) and Detournay and Cheng (1988) are discussed here. The solution of Carter and Booker (1982) represents a special case of the more general solution obtained by Detournay and Cheng (1988) (Table 3-2). As there are no mechanisms for pore pressure generation in mode 1, displacements, stresses and pore pressure can be found easily with the classical solution in elasticity. This solution also shows no time dependency (Table 3-2). The solution is trivial and will not be discussed further. Results for mode 2 and mode 3 are by far more interesting.

**Mode 2:**

- There is no movement of the tunnel wall although there is a non-zero, inward radial displacement inside the medium (Figure 3-6).

- The distribution of the excess pore water pressure can be found with solutions to the axisymmetric diffusion (heat flow) problem which have been reported by Jaeger (1956). The isochrones of excess pore water pressure are approximately linear functions of the logarithm of the radius (Figure 3-7(b)). The history of the excess pore pressure is plotted in Figure 3-8 for various distances from the tunnel perimeter. The initial flat parts of the curves suggest that drainage
propagates very slowly inside the rock mass, as can be observed in Figure 3-7.

- At and near the tunnel wall, the tangential total stress becomes instantaneously a tensile stress. For any other point inside the medium, the tangential total stress is first compressive and then tensile (Figure 3-9).

**Mode 3:**

- The history of the radial displacement at the boundary point $\theta=0$ is plotted in Figure 3-10. The radial displacement varies from $(3-4\nu_u)R_0q_0/2G$ at short times (perfectly undrained), to $(3-4\nu)R_0q_0/2G$ at long times (perfectly drained). Figure 3-11 illustrates the evolution of the radial displacement at the tunnel wall as a function of the $\theta$-angle. Since the radial displacement is proportional to $\cos 2\theta$ (Table 3-5), it can be seen that the existence of a far-field stress deviator produces a time-dependent outward displacement in the direction of the minimum compressive in-situ stress as consolidation occurs, and a time-dependent inward displacement in the direction of the major compressive in-situ stress as swelling occurs (Figure 3-11). Finally, it should be noted that the magnitude of the consolidation/swelling displacements are of the same order as those of the initial undrained displacements as reported in Table 3-5. For instance, for $\nu_u=0.4$ and $\nu=0.25$, the consolidation/swelling displacements represent 71.4% of the displacements taking place during the undrained phase; for $\nu_u=0.5$ and $\nu=0.25$, the consolidation/swelling displacements are equal to those occurring in the undrained phase.
The distribution of the excess pore pressure is given by Detournay and Cheng (1988):

\[
\Delta u^0 = \frac{4}{3} q_0 B (1 + \nu_u) \left( \frac{R_0}{r} \right)^2 \cos 2\theta \\
= \left[ 2q_0 \left( \frac{R_0}{r} \right)^2 \cos 2\theta \right] \times \frac{2B(1+\nu)}{3-B(1-2\nu)\alpha} 
\]

(3-18)

classic poroelastic elasticity effect

which is plotted in dashed line in Figure 3-12. Equation 3-18 indicates that the largest change in pore water pressure occurs for incompressible material \((\alpha=1, B=1)\) for which the term of the poroelastic effect is maximum and equal to unity. Figure 3-12 illustrates Equation 3-18 for \(\nu=0.2, \nu_u=0.4\) and \(B=0.8\). Because of the imposed boundary condition, \(u=0\), at the tunnel perimeter, there is a steep radial gradient of the excess pore water pressure at early times, which is associated with a rapid drainage of fluid at the tunnel perimeter. Since the initial excess pore pressure is proportional to \(\cos 2\theta\), its sign changes between the springlines \((\theta=0^0, \theta=180^0)\) and the crown \((\theta=90^0)\) or the invert \((\theta=270^0)\). Pressure gradients, parallel to the tangential direction, which exist between the positively or negatively pressurized zones located in the immediate vicinity of the tunnel, produce tangential flows.

Rapid drainage of the rock mass has a direct impact on the stress concentration. Indeed, at the tunnel wall, the rock is characterized by the drained elastic modulus while the rock further inside the medium has a stiffer undrained modulus. As a result of this stiffness contrast, the tunnel is partially shielded from the stress concentration at early times. Figure 3-13 illustrates this phenomenon where the isochrones of the tangential stress varying with radial distance have been plotted. For example, the normalized tangential stress reaches a maximum inside the rock mass at \(r/R_0 = 1.03\), for \(T_c=10^{-4}\) and not at the tunnel wall.
- Detournay and Cheng (1988) showed that for constant boundary conditions, mode 3 is the only fundamental loading mode to introduce time-dependent variation of the stress concentration. Figure 3-14 illustrates the fact that the short-term stress concentration at the tunnel wall, \( \sigma_{00}/g_0 \cos \theta \), is \( 4(1-v_u)/(1-v) \), and not 4 as predicted by the elastic analysis. As a result, at very short time \( T_c = \text{ct}/(R_0)^2 < 10^{-2} \), the peak of the tangential stress is actually located inside the rock mass and not at the tunnel wall (Figure 3-13). Rice and Cleary (1976) reached the same conclusion in their analysis of the stress diffusion solutions around a borehole in a saturated porous medium. Note, however, that significant deviation of \( \sigma_{00} \) from the elastic distribution takes place only in a small region near the tunnel wall. For practical purposes, this region can be assumed to have an extent of approximately \( 0.15 \times R_0 \), for all \( T_c \); that is, for \( R_0 = 4 \) to 6 m, this zone extents 0.6 to 0.9 m from the tunnel perimeter. Detournay and Cheng (1988) describe this poroelastic mechanism as a skin effect. Finally, one should keep in mind that the elastic solution gives always an upper bound for the stress concentration at the tunnel wall (Figure 3-14).

- Detournay and Cheng (1988) further showed that the poroelastic mechanisms associated with the fundamental loading mode 3 are responsible for a delayed tunnel failure. In addition they show that shear failure can be initiated at a small distance inside the rock mass, along a radial direction perpendicular to the in-situ major compressive stress, rather than at the tunnel wall as predicted by the theory of elasticity. This can be seen in Figure 3-13 which illustrates the fact that the tangential stress (major principal stress) is maximum inside the rock mass, and not at the tunnel wall.

Before closing this section on elastic models, we devote some attention to water, as it is the key parameter in this study. Indeed, knowledge of the change in pore water pressure generated during the sudden excavation of the tunnel is very important as consolidation,
respectively swelling, will take place in the zones which have initially positive, respectively negative, excess pore water pressure. From an engineering point of view, we are primarily interested in the time- and space-distributions of the excess pore water pressure.

It can be deduced from the normalized plots given in Figures 3-7 and 3-12 that the magnitude of the excess pore pressure depends on: (1) the initial pore water pressure at the level of the tunnel axis, \( u_c \), (2) the anisotropic component of the initial state of stress, \( \sigma_{d0} \), and (3) the location around the tunnel, or more specifically, on the polar coordinates \( r \) and \( \theta \).

The excess pore pressure can be expressed as:

\[
\Delta u = \Delta u(1) + \Delta u(2) + \Delta u(3) \tag{3-19}
\]

where \( \Delta u(i) \) is the excess pore water pressure associated with mode \( i \).

**Mode 1 does not generate excess pore pressure, therefore:**

\[
\Delta u(1) = 0 \tag{3-20}
\]

From Figure 3-7, the excess pore pressure of mode 2 can be expressed as:

\[
\Delta u(2) = \Delta u_{\text{norm}}(2) \times u_0 = \Delta u_{\text{norm}}(2) \times \gamma_w h_w \tag{3-21}
\]

where:
- \( \Delta u_{\text{norm}}(2) \) is read from Figure 3-7,
- \( \gamma_w \): specific gravity of the water, \( \gamma_w = 10 \text{ KN/m}^3 \),
- \( h_w \): height of the column of water above the tunnel axis before excavation.

From Figure 3-12, the excess pore water pressure of mode 3 is written as:

\[
\Delta u(3) = \Delta u_{\text{norm}}(3) \times q_0 \cos \theta
= \Delta u_{\text{norm}}(3) \times 0.5 \left| 1 - \lambda_0 \right| \sigma_{d0} \cos \theta \tag{3-22}
\]

where:
- \( \Delta u_{\text{norm}}(3) \) is read from Figure 3-12, which has been obtained for \( V=0.2 \), \( V_u=0.4 \) and \( B=0.8 \).
Assuming that the specific gravity of the rock, \( \gamma_t \), is 2.4 times that of the water, \( \gamma_w \), \( \sigma_{vo} = 2.4 \gamma_w h \), where \( h \) is the tunnel depth (measured at the level of the tunnel axis). Putting this value of \( \sigma_{vo} \) in Equation (3-22) gives:

\[
\Delta u(3) = 1.2 \Delta u_{\text{norm}}(3) \times |1-\lambda_0| \gamma_w h \cos \theta \tag{3-23}
\]

Introducing Equations 3-20, 3-21 and 3-23 in Equation 3-19, the following relationship is obtained for the excess pore pressure:

\[
\Delta u = \Delta u_{\text{norm}}(2) \times \gamma_w h_w + 1.2 \Delta u_{\text{norm}}(3) \times |1-\lambda_0| \gamma_w h \cos \theta
= \gamma_w h_w [\Delta u_{\text{norm}}(2) + 1.2 \Delta u_{\text{norm}}(3) \times |1-\lambda_0| (h/h_w) \cos \theta] \tag{3-24}
\]

Equation 3-24 can be normalized with respect to the initial pore water pressure \( u_0 = \gamma_w h_w \):

\[
\Delta u / u_0 = [\Delta u_{\text{norm}}(2) + 1.2 \Delta u_{\text{norm}}(3) \times |1-\lambda_0| (h/h_w) \cos \theta] \tag{3-25}
\]

Equation 3-25 has been solved for various values of \( h/h_w \) (1.0, 2.0, 4.0) and \( \lambda_0 \) (0.5, 1.0, 2.0), for two different locations around the tunnel: in the springlines (\( \theta=0^\circ, \theta=180^\circ \)) and in the crown/invert (\( \theta=90^\circ, \theta=270^\circ \)). Results are illustrated in normalized plots of \( \Delta u/u_0 \) versus \( r/R_0 \) in Figures 3-15, 3-16 and 3-17.

It can be seen from Figure 3-15 to Figure 3-17 that, at long times (\( T_0 > 1 \)), the pore pressure distribution is practically the same for all cases studied. This means that the pore pressure distribution is governed by mode 2 at long times, while mode 3 is dominant at short times. Further, it can be seen that the relative importance of mode 3 with respect to mode 2 is increasing as the product \( |1-\lambda_0| (h/h_w) \) increases. For example, consider a tunnel located at a depth of 100 m. Mode 3 will be relatively more important than mode 2 if the water table is located at, e.g. 25 m \( (h/h_w=4) \) above the tunnel axis, than if it is located at the surface \( (h/h_w=1) \); for \( \lambda_0=2.0 \) and \( \theta=0^\circ \) (Figure 3-17), the negative pore pressure generated at the tunnel wall at time \( T_0=0^+ \) is \(-8.2u_0\) for \( h/h_w=4 \), while it is only \(-2.8u_0\) for \( h/h_w=1.0 \). Note, finally, from Figure 3-15 to Figure 3-17, that the extent of the zone where the
pore pressure changes are significant is limited to approximately three to four tunnel radii.

The permeable or impermeable drainage boundary conditions at the tunnel perimeter affect the evolution of consolidation/swelling of the rock with time. Carter and Booker (1982) and Detournay and Cheng (1988) reported that the characteristic time factor at the end of consolidation/swelling, $T_c$, is approximately equal to one ($T_c \approx 1$) if the tunnel perimeter is permeable. The same authors estimate that this characteristic time factor can be increased by a factor of as much as ten ($T_c \approx 10$) if the tunnel perimeter is impermeable.

3.2.1.2 Elasto-plastic models

Carter (1988) presented a semi-analytical solution for the dissipation of pore water pressure around a freshly created vertical hole. It is assumed that the ground behaves as an isotropic linear elastic perfectly plastic material, with incompressible constituents; i.e., $B=1.0$, $G=1.0$. The initial state of stress is assumed to be hydrostatic in the horizontal plane. The initial pore pressure is assumed to be equal to $u_0$ at the depth considered. Both permeable and impermeable borehole interfaces have been considered, together with either no support for the hole or partial support provided by hydrostatic pressure within the hole. It is assumed that immediately following the creation of the cylindrical cavity, yielding, defined by Tresca's criterion ($\tau_c = 0$, $\Phi_i = 0$), takes place. Yielding of the ground simultaneously induces excess negative pore pressures. As a matter of fact, assuming that the octahedral effective stress remains unchanged ($\Delta\sigma_{oct} = 0$), the excess pore water pressure, $\Delta u$, generated during yielding is equal to the change in mean total stress, $\Delta\sigma_{oct}$, which takes place in the plastic zone around the tunnel (Figure 3-18). During swelling, however, the rock is assumed to behave elastically. Carter made this assumption to get an analytical solution. Therefore, the governing equations and field equations obtained for the elastic model with incompressible constituents hold.
Computation of time-dependent stresses and excess pore pressures induced by the fundamental mode 1 and mode 2 (mode 3 does not exist in this case; the initial state of stress is hydrostatic in the horizontal plane) is identical to the previous elastic case. However, for mode 1 the initial excess pore water pressure is no longer zero, but a quantity which depends on the excess negative pore pressure generated immediately after the creation of the cavity. Mode 2, however, which describes the release of the pore water pressure at the tunnel boundary, is not affected by the occurrence of a plastic zone around the cavity. The solution of Carter (1988) only deals with the stress and pore pressure distributions. No attempt is made to estimate the radial deformations at the wall of the borehole.

The solution of Carter (1988) represents a first attempt in solving the problem of swelling around a circular hole in a ground which has reached its yield strength. Although the solution is not complete, it shows that yielding and swelling are closely related.

3.2.2 CREEP

3.2.2.1 Introduction

Semple et al. (1973) studied the time-dependent nature of the deformation and support loads in squeezing ground conditions in altered hard-rock tunneling. Their study suggested that the creep behavior of the material is the significant engineering property controlling the in-situ rock behavior. The authors presented an experimental and analytical study that deals with soil creep and its application to the prediction of tunnel support requirements.
Semple et al. (1973) developed the following creep equation for altered rock:

$$e_a = X \exp (\beta \Delta) \left( t/t_1 \right)^\lambda$$  

(3-19)

where: $e_a$: axial strain (%),
$\Delta$: stress level with respect to failure (%), $q/q_f$
$X$: coefficient, extrapolated strain at zero stress-intercept
of the log $e_a - \Delta$ relationship,
$\beta$: stress exponent, slope of the log $e_a - \Delta$ relationship,
t: elapsed time since application of the load corresponding to $\Delta$,
t_1: unit reference time, generally $t_1$ is taken equal to one day,
$\lambda$: time exponent, slope of the log $e_a - \log t$ relationship.

This equation is similar to that proposed by Singh and Mitchell (1968). It was obtained with tests performed under undrained conditions, as are most triaxial creep tests reported in the literature. The use of undrained conditions is questionable when the objective is long-term field application of laboratory creep measurements. Fortunately, however, Equation 3-19 is, for all practical purposes, valid for drained tests (CIDC[L]) too (Singh and Mitchell, 1968). Comparison of Equation 3-19 with data of drained tests on undisturbed specimens of highly overconsolidated (OCR=30) London Clay reported by Bishop and Lovenbury (1969) shows a good agreement. These test data have been plotted in Figure 2-33, along with Equation 3-19 (=Equation 2-6) with the parameters for London Clay, which best fit the test data.

It will be noticed, however, that the data of Bishop and Lovenbury (1969) are the only published data on long-term drained creep! Although the tests were conducted for three and a half years and thus represent very valuable information on the long-term behavior of soils, they should be regarded with caution, primarily because of the lack of other experimental data to compare with.

Semple et al. (1973) further solved their nonlinear empirical creep equation directly with the appropriate boundary conditions, by using the finite difference method. They developed a computer program for the analysis of an unlined or lined circular opening, instantaneously created in an infinite, homogeneous, isotropic, incompressible, creep-sensitive medium under initial isotropic state of stress. The medium was
assumed to be linear elastic with superimposed creep as given by Equation 3-19.

3.2.2.2 Relevant results in the study of Semple et al. (1973)

Analysis of the coefficients in Equation 3-19 reveals some interesting features:

- The coefficient X is dependent on the stress parameter $\beta$ as shown on Figure 3-19.

- The time parameter $\lambda$ is dependent on the plasticity index, PI(%) (Figure 3-20) and on the water content, w(%) (Figure 3-21). These results are important for practical purposes. As a matter of fact, the higher the plasticity index or the water content of the material, the higher the coefficient $\lambda$. Therefore, highly plastic materials will creep at a faster rate than materials of low plasticity. On the other hand, the magnitude of the coefficient $\lambda$ will be affected by the water content of the rock at the end of the consolidation/swelling phase. Thus, in the zones around the tunnel where consolidation took place, the rate of creep will be smaller than in the zone where swelling took place. This is due to the fact that in the zone of consolidation, the initial water content prior to excavation will decrease, whereas it can increase significantly in the zone of swelling (See for instance the case history of the tunnel de Paris, Section 2.1.1.6, Figure 2-21).

Semple et al. (1973) reported good agreement between their theoretical predictions and in-situ measurements. Typical plots of the radial stress on tunnel support with time and of the ratio of the radial inward displacement to the initial tunnel diameter with time are shown in Figures 3-22 and 3-23, respectively.

- Curves of radial stress on a rigid support display an S-shape in a semi-logarithmic plot (Figure 3-22), with a continuous increase with time. In other words, there is no upper limit to
the radial stress on support as time goes on. The rate of increase slows down with time, but never stops completely.

- Curves of the ratio of the radial inward displacement to the initial tunnel diameter time also show a continuous increase over time in a semi-logarithmic plot (Figure 3-23). Again, there is no upper limit to the diametric deformation as time passes.

3.2.3 DISCUSSION

Existing models which describe the behavior of a porous homogeneous saturated medium around a circular opening have been described briefly. The diffusion models show that negative pore pressures can be generated during sudden tunnel excavation and that mechanisms of swelling can be observed upon release of these negative pore pressures. Further, the creep model suggests that creep might be an important aspect of the long-term behavior of excavated tunnels.

In diffusion models, assuming that the medium behaves as a linear elastic perfectly plastic material, it has been demonstrated that excess pore water pressures are due to three causes:

- drawdown of the initial pore water pressure in the vicinity of the tunnel (mode 2),

- anisotropic initial state of stress (mode 3),

- development of a plastic zone accompanied by decompression of the rock mass in the vicinity of the tunnel.

It has been further shown with models for isotropic linear poroelastic materials that the anisotropic component of the initial state of stress is the only one which involves a coupled deformation-diffusion process inside the rock mass. Finally, the drainage interface between the tunnel and the ground is expected to strongly affect the duration of consolidation and swelling, and the magnitude of the
associated deformations. Impermeable interfaces tend to increase both the duration of consolidation/swelling, and the tunnel deformations.

The creep model shows that neither the radial stress nor the ratio of the radial inward displacement to the initial tunnel diameter reaches an upper limit. This continuous process can lead to serious problems, especially in highly plastic materials or in zones where the initial water content of the rock increases after excavation (swelled zone).

It will be noticed that the models discussed in this section do not account for the "real" material behavior of argillaceous rocks. On the one hand, the diffusion models found in the literature are based on very little experimental evidence (Detournay and Cheng, 1988). B-values, for instance, are rarely reported in the literature for argillaceous rocks. On the other hand, these models have important limitations which must be eliminated in order to be used reliably for practical purposes. Amongst the most obvious are:

- \textit{Mode 1} (pure shear) predicts no generation of excess pore pressure (Table 3-2, Equation (6)).

- In Table 3-5, the radial displacements occurring during the undrained and consolidation/swelling phases are of the same magnitude

- A mechanism such as dilation upon shear, which can induce drastic pore pressure changes during the undrained phase, is never taken into account.

The experimentally based creep model is not fully satisfactory either. As already mentioned, long-term drained behavior is not well documented, if at all. Furthermore, data have been obtained for soils only, although the models have also been used for altered hard rocks.

Thus, in order to proceed further in the developments of new models, the behavior of the argillaceous rocks in both the undrained and drained phases must be better understood.
3.3 **FINAL COMMENTS**

In the present chapter and its companion appendix B, a new approach to the design of tunnels in argillaceous rocks has been presented, along with a literature review of previous analyses for tunnels in porous media.

It can be observed from this review that:

(1) Theoretical solutions for the elastic cases with compressible (Detournay and Cheng, 1988) and incompressible (Carter and Booker, 1982) constituents do exist and are complete for stress distributions as well as for displacements.

(2) For the elastoplastic case (Carter, 1988) the solution is not complete and should be extended to obtain a solution for the displacements. This solution should also be extended for the case of initial anisotropic state of stress.

(3) Any further developments of theoretical models must involve a better understanding of the behavior of the rock at hand under undrained and drained (consolidation/swelling and creep) conditions, thus,

(4) Experimental work is needed in the undrained phase as well as in the drained phase.

In the following part of this thesis, the undrained behavior of three types of argillaceous rocks found in the Jura Chain of Switzerland will be investigated.
<table>
<thead>
<tr>
<th>Model with constituents</th>
<th>Material behavior</th>
<th>Boundary condition at the tunnel wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>pervious</td>
</tr>
<tr>
<td>compressible</td>
<td>linear elastic</td>
<td>Detournay &amp; Cheng (1988)</td>
</tr>
<tr>
<td></td>
<td>plastic/Mohr-Coulomb</td>
<td>Detournay &amp; Cheng (1988)*</td>
</tr>
</tbody>
</table>

*Partial results discussed*
\textit{Radial displacement} \quad \frac{2GU_L}{P_0R_0} = \frac{-R_0}{r} = f\left(\frac{r}{R_0}\right) \quad (1)

\textit{Tangential displacement} \quad \frac{2GU_B}{P_cR_0} = 0 \quad (2)

\textit{Radial stress} \quad \frac{\sigma_{rr}}{P_0} = \left(\frac{R_0}{r}\right)^2 = f\left(\frac{r}{R_0}\right) \quad (3)

\textit{Tangential stress} \quad \frac{\sigma_{\theta\theta}}{P_0} = \left(\frac{R_0}{r}\right)^2 = f\left(\frac{r}{R_0}\right) \quad (4)

\textit{Shear stress} \quad \frac{\sigma_{r\theta}}{P_0} = 0 \quad (5)

\textit{Pore water pressure} \quad \frac{u}{P_0} = 0 \quad (6)

\textbf{Note:} All terms are defined in Figure 3-4

\begin{table}[h]
\centering
\begin{tabular}{|c|c|}
\hline
\textbf{Table 3-2} & Coefficients with Normalized Parameters for Displacements and Stresses of Node 1 (modified from Detournay and Cheng,1988) \\
\hline
\end{tabular}
\end{table}
Radial displacement
\[ \frac{2G \omega R}{\eta u_0 R_0} = \frac{f}{R_0 T_c} \]  
(1)

Tangential displacement
\[ \frac{2G u_0}{\eta u_0 R_0} = \frac{f}{R_0 T_c} \]  
(2)

Radial stress
\[ \frac{\sigma_{rr}}{\eta u_0} = \frac{f}{R_0 T_c} \]  
(3)

Tangential stress
\[ \frac{\sigma_{\theta\theta}}{\eta u_0} = \frac{f}{R_0 T_c} \]  
(4)

Shear stress
\[ \frac{\tau_{\theta r}}{\eta u_0} = \frac{f}{R_0 T_c} \]  
(5)

Pore water pressure
\[ \frac{u}{u_0} = \frac{f}{R_0 T_c} \]  
(6)

Note: All terms are defined in Figure 3-4

- \( \eta = \frac{\alpha \frac{1-2v}{1-v}}{\frac{L^2}{1-V}} \)
- \( T_c = \frac{ct}{(R_0)^2} \)

Table 3-3 Coefficients with Normalized Parameters for Displacements and Stresses of Mode 2 (modified from Detournay and Cheng, 1988)
Radial Displacement  \[ \frac{2GU_x}{q_0R_0} = f\left(\frac{x}{R_0}, T_c, v, \nu_u, B, \theta\right) \]  (1)

Tangential displacement  \[ \frac{2GU_\theta}{q_0R_0} = f\left(\frac{x}{R_0}, T_c, v, \nu_u, B, \theta\right) \]  (2)

Radial stress  \[ \frac{\sigma_{xx}}{q_0} = f\left(\frac{x}{R_0}, T_c, v, \nu_u, B, \theta\right) \]  (3)

Tangential stress  \[ \frac{\sigma_{\theta\theta}}{q_0} = f\left(\frac{x}{R_0}, T_c, v, \nu_u, B, \theta\right) \]  (4)

Shear stress  \[ \frac{\sigma_{x\theta}}{q_0} = f\left(\frac{x}{R_0}, T_c, v, \nu_u, B, \theta\right) \]  (5)

Pore water pressure  \[ \frac{u}{q_0} = f\left(\frac{x}{R_0}, T_c, v, \nu_u, B, \theta\right) \]  (6)

Note:  - All terms are defined in Figure 3-4

\[ T_c = \frac{ct}{(R_0)^2} \]

Table 3-4  Coefficients with Normalized Parameters for Displacements and Stresses of Mode 3 (modified from Detournay and Cheng, 1988)
Radial displacement at time $t=0^+$

$$u_r^{0+} = \frac{(3-4\nu_u)R_o q_o \cos \theta}{2G}$$  \hspace{1cm} (1)

Radial displacement at time $t=\infty$

$$u_r^\infty = \frac{(3-4\nu)R_o q_o \cos \theta}{2G}$$  \hspace{1cm} (2)

Radial displacement during consolidation/swelling

$$u_r^\infty - u_r^{0+} = \frac{R_o q_o \cos \theta}{G} 2(\nu_u-\nu)$$  \hspace{1cm} (3)

Ratio of the radial displacement occurring during consolidation/swelling to the radial displacement occurring at time $t=0^+$ i.e. $(3)/(1)$

$$\frac{u_r^\infty - u_r^{0+}}{u_r^{0+}} = \frac{4(\nu_u-\nu)}{3-4\nu_u}$$  \hspace{1cm} (4)

<table>
<thead>
<tr>
<th>$\nu_u$</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>0.25</td>
</tr>
<tr>
<td>0.40</td>
<td>0.15</td>
</tr>
</tbody>
</table>

$$\frac{u_r^\infty - u_r^{0+}}{u_r^{0+}} = 1.000$$

$$\frac{u_r^\infty - u_r^{0+}}{u_r^{0+}} = 0.714$$

$u_r (\theta=\pi/2)$

inwards for $\lambda<1.0$

$R_o$ \hspace{1cm} $\theta$

$u_r (\theta=0)$

outwards for $\lambda>1.0$

$\sigma_{ho} = \lambda_0 \sigma_{vo}$

Table 3-5  Radial Displacements at Tunnel Wall During Consolidation/Swelling (modified from Detournay and Cheng, 1988)
Figure 3-1  Plot of $\alpha$ versus $K$
Figure 3-2  Plot of $B$ versus $K$ for $K < 15000$ MPa and $n_0 < 30\%$
Figure 3-3  Plot of $B$ versus $V_u$ for Various Values of $V$ and the Biot Coefficient of Effective Stress $A=1$, $A=0.8$, and $A=0.6$

(Equation 3-5)
Rock Properties

Shear modulus: $G$
Bulk modulus: $K$
Biot coeff. of stress: $\alpha$
Undrained Poisson’s ratio: $\nu_u$
Drained Poisson’s ratio: $\nu$

$$p_0 = (\sigma_{vo} + \sigma_{ho}) / 2$$
$$q_0 = (\sigma_{vo} - \sigma_{ho}) / 2$$

Figure 3-4  Problem Definition and Coordinate System
Boundary conditions at the tunnel wall

**Mode 1**

\[ \Delta \sigma_{rr}(1) = -p_0 \]
\[ \Delta \sigma_{rr}(1) = 0 \]
\[ \Delta u(1) = 0 \]

**Mode 2**

\[ \Delta \sigma_{rr}(2) = 0 \]
\[ \Delta \sigma_{rr}(2) = 0 \]
\[ \Delta u(2) = -u_0 \]

**Mode 3**

\[ \Delta \sigma_{rr}(3) = -q_0 \cos 2\theta \]
\[ \Delta \sigma_{rr}(3) = q_0 \sin 2\theta \]
\[ \Delta u(3) = 0 \]

**Figure 3-5** Three Fundamental Loading Modes
(from Carter and Brooker, 1982)
Note: \[ \eta = \frac{\alpha}{1 - 2\nu} \]

\[ T_c = \frac{ct}{(R_0)^2} \]

**Figure 3-6** Isochrones of the Coefficient of Radial Displacement for Mode 2 (from Carter and Brooker, 1982)
Figure 3-7  Isochrones of the Coefficient of Excess Pore Pressure for Mode 2: (a) with Arithmetic Scale (from Carter and Brooker, 1982) and (b) with Semi-Logarithmic Scale (modified from Jaeger, 1956)
\[ T_c = \frac{ct}{(R_0)^2} \]

**Figure 3-8** Time-History of the Coefficient of Excess Pore Pressure at Various Radii for Node 2 (from Detournay and Cheng, 1988)
Note: \[ \eta = \alpha \frac{1-2\nu}{1-\nu} \]

\[ T_c = \frac{ct}{(R_0)^2} \]

**Figure 3-9** Isochrones of the Coefficient of Tangential Total Stress for Node 2 (from Detournay and Cheng, 1988)
Figure 3-10 Time-History of the Radial Displacement at Tunnel Wall for Mode 3 with $v=0.2$, $V_u=0.4$, and $B=0.8$ (adapted from Detournay and Cheng, 1988)
(a) $\lambda_0 < 1$ \hspace{1cm} ($\sigma_n < \sigma_v$)

$g_0 = \sigma_v(1-\lambda_0)$

Outwards displacement in the direction of the minor in-situ stress

(b) $\lambda_0 \geq 1$ \hspace{1cm} ($\sigma_n > \sigma_v$)

$g_0 = \sigma_n(1-\lambda_0)$

Inwards displacement in the direction of the major in-situ stress

**Figure 3-11** Time-Dependent Inwards/Outwards Displacements due to Initial Far-Field Stress Deviator for Mode 3
Figure 3-12 Isochrones of the Coefficient of Excess Pore Pressure for Mode 3 with $v=0.2$, $v_u=0.4$, and $S=0.8$ (from Detournay and Cheng, 1988)

$$T_c = \frac{ct}{(R_0)^2}$$
Strong deviation from elastic solution due to poroelastic effect

Little deviation from elastic solution

![Graph showing normalized stress vs normalized radial distance with various Tc values.]

\[ T_c = \frac{ct}{(R_o)^2} \]

**Figure 3-13** Isochrones of the Coefficient of Tangential Total Stress for Mode 3 with \( V=0.2 \), \( V_u=0.4 \), and \( B=0.8 \) (from Detournay and Cheng, 1988)
Figure 3-14 Time-History of the Coefficient of Stress Concentration for Mode 3 with $V=0.2$, $V_d=0.4$, and $S=0.8$ (from Detournay and Cheng, 1988)
Figure 3-15 Isochrones of Normalized Pore Pressure versus Normalized Radial Distance for Modes 2 and 3 for $\lambda_o = 0.5$ and $h/h_o = 1.0, 2.0, \text{ and } 4.0$
Figure 3-16 Isochrones of Normalized Pore Pressure versus Normalized Radial Distance for Modes 2 and 3 for $\lambda_0=1.0$ and all $h/h_y$-values
Figure 3-17 Isochrones of Normalized Pore Pressure versus Normalized Radial Distance for Nodes 2 and 3 for $\lambda_0=2.0$ and $h/h_0=1.0, 2.0, \text{and} 4.0$
Figure 3-18 Pore Pressure Distribution Just After Excavation, Assuming an Isotropic Linear Elastic Perfectly Plastic Material with Tresca Failure Criterion
(modified from Carter, 1988)
Figure 3-19 Coefficient $X$ versus Stress Parameter $\beta$

(from Semple et al., 1973)
Figure 3-20 Slope of Logarithmic Creep Curves as a Function of Water Content (from Semple et al., 1973)
Figure 3-21 Slope of Logarithmic Creep Curves as a Function of Water Content (from Semple et al., 1973)
Figure 3-22 Radial Stress on Tunnel Support with Time
(from Semple et al., 1973)
Figure 3-23 Inwards Displacement of Unsupported Tunnel with Time
(from Semple et al., 1973)
CHAPTER 4

EXPERIMENTAL REQUIREMENTS,
DESCRIPTION OF THE NEW TRIAXIAL SYSTEM,
AND TESTING PROCEDURES

4.1 INTRODUCTION

The purpose of any laboratory investigation is to predict in-situ behavior, e.g. the volume change of rock when subjected to changes in the applied state of stress. The removal of samples from the ground may produce irreversible structural damages, which affect the in-laboratory-observed properties of the rock. Argillaceous rocks are most probably sensitive to extraction from the ground and for this reason great care should be taken to: (1) minimize sample disturbance, and (2) re-establish as closely as possible the initial in-situ stress and volumetric conditions.

The second section of this chapter provides a general guide to minimizing sample disturbance. The third section discusses an evaluation of the existing method used to re-establish the initial in-situ conditions for underground works. The fourth section presents and discusses the "Stress Path" method which is used to estimate the history and variation of stress for elements inside the rock mass. The last two sections discuss the testing equipment and experimental procedures. The testing procedures are presented in the fifth section. The new triaxial system, which has been developed for this study and used to test specimens under undrained conditions only, is also described in the fifth section. Finally, drained testing is discussed in the sixth section.
Before proceeding further, it should be noted that very little has been reported in the literature concerning the undrained/drained behavior of argillaceous rocks. The following is a list of a few of the available references.

Undrained: Mesri and Gibala (1971)
Morgenstern and Eigenbrod (1974)
Chiu et al. (1983)

Drained: Casagrande (1949)
Lo et al. (1978)
Morgenstern and Balasubramonian (1980)
Cepeda Diaz (1987)

As these papers report results which cannot be integrated directly into this thesis, they will not be discussed further. The reader may find it interesting to refer to them in order to learn about other interesting features of argillaceous rocks.

4.2 SAMPLE DISTURBANCE

Sample disturbance can occur during sampling, handling and specimen preparation. It should be minimized because of its pernicious influence on the measured properties of the material.

Generally, samples are obtained from deep boreholes. In order to get samples of good quality, boring with a double-tube core barrel (e.g. Terzaghi and Peck, 1967; Winterkorn and Fang, 1975) or with a triple-tube core barrel (e.g. Meigh, 1976), with diameter between 60 and 100 mm, is required. Preferably, the boring should be performed dry, generally with air-pressure cooling. If this is not possible, an anti-swelling mixture (e.g. Antisol ¹, which increases the viscosity of the drilling fluid, and thus reduces its interaction with the natural water of the rock) should be mixed with the drilling fluid (ISRM, 1987).

¹ See ISRM (1987).
During sampling, some sample disturbance is unavoidable and results in an instantaneous expansion (volume increase), which is due to various mechanisms, such as: (1) undrained release of shear distortion (a volume change of the specimen can take place during the undrained phase as the Skempton B-parameter can be less than one; $B<1$), (2) cracking, (3) retreat of the capillary menisci, (4) possible cavitation, and (5) any process which leads to an increase in microfissuring (e.g. vibrations, shocks, etc, ...). All of these mechanisms tend to allow air to enter the discontinuities and reduce the original degree of saturation. At the Swiss Institute of Technology of Zürich (ETH-Z) observations made on 132 samples of argillaceous rocks have shown that the initial degree of saturation of initially 100% saturated specimens obtained from deep boreholes is between 60 to 95%, with an average value of 87% $^2$.

Samples must be cleaned, wrapped in a plastic aluminium foil and sealed with paraffin wax immediately after extraction from the ground, in order to maintain their natural water content throughout the process of transportation and storage. Personal experience has shown that decrease in the initial natural water content of the material often leads to microfissuring and opening of discontinuities, which in turn may produce an overall volume increase of the specimen. On the other hand, intact materials will have the tendency to decrease in volume due to an increase in the negative pore pressure upon drying (Cepeda Diaz, 1987). Thus, depending on the initial conditions of the rock, the sample will contract or expand. Wetting is expected to have opposite effects.

Handling and specimen preparation (trimming, face cutting and setting) in the laboratory should be done as rapidly as possible in order to keep disturbance to a minimum. In particular, the natural water content should be kept unchanged during specimen preparation. Therefore, the operations of cutting and facing should be done with air-cooling, and not with cooling water (ISRM, 1987). Our experience has shown that working with cooling water and an anti-swelling admixture (Antisol) is not effective in the laboratory. Specimens break along pre-existing

$^2$ See Section A.5 in Appendix A.
weakness planes during the operations of recoring, cutting, facing. The addition of anti-swelling admixture is not useful. Samples should not be recored, because of the large specimen losses. Time and effort can be saved by adapting the testing apparatus to the rock specimen obtained from borings.

4.3 RE-ESTABLISHMENT OF THE INITIAL IN-SITU CONDITIONS

In most previous investigations, undrained conditions with zero volume change of the specimen were assumed to prevail during sampling, handling and specimen preparation when attempting to re-establish the field volumetric and stress conditions.

For near surface foundation design, laboratory tests have been developed to simulate the 1-D field conditions under large slats. A rock specimen is placed in the oedometer, and its initial volume is maintained constant after the specimen is given access to water by increasing the axial stress correspondingly. The axial stress which prevents any volume change has been called the swelling pressure. Cepeda Diaz (1987) concluded in his thesis that the initial void ratio of an undisturbed specimen, \( e_i \), is expected to be greater than the void ratio of the material in the ground, \( e_0 \), because of the instantaneous expansion upon sampling and associated sample disturbance. This instantaneous expansion considerably reduces the magnitude of the swelling pressure (Section A.3.2).

For tunneling design, which can be characterized as deep foundation, the same 1-D approach has been used. The maximum pressure that the foundation has to sustain in the field is assumed to equal the swelling pressure. In the existing design methods for tunnels in swelling rock (e.g. Wittke and Rissler, 1976; Gysel, 1977; Kovari et al., 1981; Schweisig and Duddeck, 1985; Gysel, 1987b), the swelling pressure is used in the computations (Section 2.2.1.2). Three laboratory methods for the

---

3 Personal experience with specimens of Oxfordian shales from the reconnaissance gallery of the "Transjurane" (Mt Terry, Switzerland, JU), where more than 95% of losses occurred during undercoring and facing.
determination of the swelling pressure have gained general acceptance. These are the methods of: Huder and Amberg (1970), Madsen (1976, 1979) 4, and Kovari et al. (1981). The values of the swelling pressure obtained using these methods differ significantly and generally lead to important design adjustments of tunnel linings. These methods are discussed in detail in Appendix A, where it is shown that the swelling pressures differ primarily because of the consideration given to sample disturbance. These tests should, therefore, be used as index tests only, as suggested by the ISRM (ISRM, 1983). A better approach needs to be developed.

The recompression technique developed by Bjerrum (1973) is used for overconsolidated, structured and cemented clays. It consists of reconsolidating the specimen to exactly the same pressures to which it was subjected in the field. This technique requires an estimate of the in-situ K₀-value, but Jamiołkowski et al. (1985) believe that errors in the K₀-value are probably not significant compared to potential adverse effects of disturbance in highly overconsolidated, structured and cemented clays, such as destruction of original cemented bonds (LaRochelle et al., 1981). It is believed that this technique is the most appropriate for argillaceous rocks, which are similar to highly overconsolidated, structured and cemented clays. The recompression technique has two beneficial effects which are: (1) to replace field stresses in the laboratory with an "identical" set of effective stresses, and (2) to saturate specimens which lost water during sampling, handling and specimen preparation (Section 4.2).

4.4 "STRESS PATH" METHOD

4.4.1 BRIEF DESCRIPTION

The "Stress Path" method, developed by Lambe (1967) and further discussed by Lambe and Marr (1979), is an approach to stability and

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4 This laboratory method has been adopted by the Commission on Swelling Rock of the International Society of Rock Mechanics (ISRM) (ISRM, 1987).
deformation in soil mechanics. In general, it consists of the following two steps:

(1) estimate the history and variation of stress for one or more elements of soil in actual field structure, and

(2) use soil tests (laboratory or field, or both) and analytical techniques that approximate the field stress and strain conditions after construction.

"The basic philosophy of the "Stress Path" method is: because strain, pore water pressure, and strength of a soil element depend on the stress path, the engineer should consider the field stress paths (or strains, or both) in selecting the soil-testing procedure and method of analysis for any given problem. Frequently, the stress paths and the behavior of the soil in actual problem are such that conventional testing techniques are sufficient. These situations can be recognized readily from the stress paths for the problem at hand and from the results of tests on particular soil involved." (Lambe, 1967)

4.4.2 APPLICATION

The "Stress Path" method has been applied to estimate the history and variation of stress for elements of argillaceous rocks located in the vicinity of a tunnel opening. An approach has been used for the case of a circular tunnel excavated in a homogeneous, linear elastic medium subjected to an isotropic state of stress ($\lambda_0 = 1.0$). An analysis has been performed and detailed results are presented in Appendix C. Results obtained with the "Stress Path" method show that a complete testing program should investigate the effect of principal stress rotations (Appendix C). A simplified solution is proposed to study the behavior of argillaceous rocks in two zones located in the crown/invert and in the springlines (Figure 4-1). For these particular zones, there is no rotation of the principal stresses, and any stress path can be followed easily in a triaxial apparatus: pure shear ($\Delta\sigma_{oct} = 0$) compression in the springlines and pure shear extension in the crown/invert.
It should be recalled here that the stress path which is proposed here models exactly mode I, as defined in poroelastic models (e.g. Detournay and Cheng, 1988) (Section 3.2.1.1).

4.5 TESTING

4.5.1 INTRODUCTION

After construction and calibration of the new triaxial system, it was only possible to test six specimens under undrained conditions in the time frame given to complete this thesis. Undrained tests have been performed in the new triaxial system, which consists of a computer-controlled large triaxial apparatus. It can produce a confining pressure up to 12 MPa and an axial stress up to 15 MPa. Specimens can be tested in compression as well as in extension. The internal set-up can be easily adapted to specimens with diameters varying from five to nine cm, and with a corresponding height of two times the diameter.

The most important feature of this system is that diameter changes can be monitored directly with proximity probes. Experience has shown that conventional strain gauges are not satisfactory when testing argillaceous material (e.g. Chiu et al., 1983). This is due to the difficulty in bonding strain gauges directly to the wet surface of the rock specimen.

4.5.2 OVERVIEW OF THE NEW TRIAXIAL SYSTEM

The main features and capabilities of the new triaxial system are reported here. A detailed description of the system along with its experimental evaluation is given in Appendix D.

The triaxial system consists of: (1) a large triaxial apparatus, (2) two loading devices: one regulating the confining pressure, the other controlling the axial load, (3) a pore pressure apparatus and, (4) a system control unit: a personal computer and a data acquisition system. Figure 4-2 presents a schematic view of the entire system.
4.5.2.1 *Large triaxial apparatus*

Large cell

The cell (designed by K.A.Soon (1987) and fabricated in-house) was transformed in order to fit the new type of experiments to be performed. The cell was therefore partially redesigned for: (1) a new specimen size, (2) saturation of specimens, (3) measurement of pore water pressure, (4) measurement of volume change (increase or decrease) under undrained or drained conditions. A general view of the cell is given in Figure 4-3.

Inside the cell, we can find the rock specimen [22] and all the measuring devices which have been placed around it (Figure 4-4). The rock specimen, at present 84 mm in diameter and 170 mm high, is placed on a removable bottom cap [19] screwed to a fixed pedestal [18], which is in turn attached to the aluminium base [12]. The axial force applied by the loading piston [28] is measured by an internal submerged load cell [27] and then transmitted to the rock specimen through the loading cap [25] and the top cap [24]. The loading cap is not removable, but the top cap is. All the parts described (pedestal, bottom cap, top cap, and loading cap) are made of stainless steel because they supply water to the specimen (ISRM, 1983). Note that the bottom cap, the top cap and the loading cap have the same diameter as the specimen. All the surfaces of the parts which act as seals are highly polished. O-ring seals are provided at the interfaces loading cap -- top cap and pedestal -- bottom cap in order to prevent any water leakage and oil penetration into the pore water pressure apparatus or into the rock specimen. The pedestal also provides a support for the microstepping motor [10] which drives the measuring device monitoring the diameter changes [21] of the rock specimen. Electrical signals from inside the cell chamber are transmitted to the outside through four electrical feedthroughs [15] installed at the base of the cell.

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5 The numbers in brackets [] refer to the parts illustrated in Figure 4-4.
Axial displacement

The axial displacement is monitored with a DCDT placed outside of the cell, parallel to the loading piston [28]. Corrections have been made to take into account the vertical compliance of the triaxial apparatus.

Diametric deformation measuring device

Particular care has been devoted to measuring diameter changes. The measuring device consists of: (1) two proximity sensors [6], (2) a moving support [21], (3) a lead screw [5], and (4) a computer-controlled microstepping motor [10]. Each pressure-insensitive proximity probe is mounted on a support which can move along the specimen. The moving support is actuated by the computer-controlled microstepping motor through the lead screw. The proximity probes can scan the full height of the rock specimen, plus 20 mm at each end of the specimen.

The two diametrically opposite proximity probes [6] measure the diameter change of the rock specimen [22] (Figure 4-4). Curved steel targets [4] are directly attached to the surface of the specimen. The monitored voltage across each probe varies proportionally with the distance between the probe and the target. The diameter change of the specimen can therefore be measured by monitoring the voltage changes across the probes. The true diameter change between the proximity probes and the boundary of the rock specimen has been found to be accurate within ±10 μm for any confining pressure, taking into account the characteristics of both the proximity sensors and the moving support; i.e. non-linearity, repeatability, hysteresis, and long term stability. The diameter strain for a specimen of 84 mm can be expected to be measured with an accuracy of ±0.012%. This accuracy is less than that obtained for a DCDT: ±0.025%. This experimental result is judged to be exceptionally good. The set-up is thus well suited to perform K₀-tests for example, for which great accuracy in monitoring diameter change is required.
Temperature control unit

Temperature variations produce dramatic changes in pore pressure during undrained tests (Lambe and Whitman, 1969). The entire triaxial apparatus is therefore enclosed in a box made of polystyrene in which the temperature is kept constant within ±0.2 °C. In the triaxial cell the temperature remained constant within ±0.1 °C.

4.5.2.2 Loading devices

There are two separate and independent loading devices, namely a confining pressure device and an axial loading device. These loading devices control the state of stress applied to the rock specimen, via the confining pressure and the axial stress, respectively. Both devices are computer-controlled. The axial load is monitored by a load cell [27] inside the cell chamber, so that the measured axial stress is free from piston friction effects (Figure 4-4). The precision of the cell pressure is ±0.003 MPa and that of the axial stress is ±0.019 MPa.

4.5.2.3 Pore pressure apparatus

During the saturation and swelling processes, water must be provided to the rock specimen. The pore water pressure (backpressure) in the specimens is supplied by air to a maximum pressure of 1.8 MPa. The pore pressure apparatus consists of: (1) an air regulator, (2) an air-water interface with a volume change device, (3) a pressure transducer and, (4) a set of lines and valves (Figure 4-5). The pore water pressure is measured at a unique location, at the bottom of the cell, underneath the pedestal (Figure 4-5).

Reliable measurement of pore water pressure in rocks is a very difficult task because the magnitude of the pore water pressure is a function of the relative compressibility of the porous rock skeleton, of the pore water and of the solid material comprising the skeleton. It is also influenced by the porosity and possibly the intergranular contact area (Section D.2.4.2. in Appendix D) (Bishop, 1973; Bishop, 1976; Mesri et al., 1976; Chiu et al., 1983). Furthermore, pore pressure measurements can be strongly influenced by the compressibility of the device.
(Bishop, 1976). Thus, great care has been taken to build a pore pressure apparatus with low compressibility and to calibrate it precisely. The calibration tests have shown that the pore pressure response, \( B = \Delta u / \Delta \sigma_{\text{act}} \), of the rock specimen can be measured within 10% of its actual value for the entire range of moduli of the argillaceous rocks investigated in this study, i.e. up to, say, 8000 MPa. A plot of the observed \( B \)-value, \( B_{\text{obs}} \), versus the drained bulk modulus of the rock skeleton, \( K \), has been developed in order to assess the pore pressure response. The result is presented in Figure 4-6, where data of tests which will be discussed in Chapter 5 have been plotted. If the point representing the observed \( B \)-value is measured data \((B_{\text{obs}}, K)\) falls inside the area drawn in Figure 4-6, then the correction to the observed \( B \)-value, which is made to obtain the "true" \( B \)-value, is less than 10%. The corrected \( B \)-value, \( B_{\text{corr}} \), is computed from the following formula, which was derived for the new triaxial system in Section D.2.4.4 6:

\[
B_{\text{corr}} = \frac{1}{\left[ \frac{1}{B_{\text{obs}}} - \Omega \right]}
\]

(4-1)

where \( \Omega \) is the compressibility of the pore pressure apparatus.

4.5.2.4 System control unit

The "heart" of the new triaxial system consists of two components, namely a personal computer (IBM, model PC-XT) and a data acquisition system (Fluke, model 2240-A) (Figure 4-2). This forms the control unit.

4.5.3 TESTING PROCEDURES

4.5.3.1 Test preparation

The preparation of the specimen, except for the first operation (cutting and facing), which is done in a room with a relative humidity of approximately 70%, is performed in a humid room to prevent the specimen from drying. Test preparation consists of the steps given

---

6 See Appendix D, Equation D-30' for more details.
hereafter. A detailed view of the final specimen set-up can be seen in Figure 4-7.

1/ Cut and face off both ends of the sample with a saw with air cooling (no water!). The final dimensions of the specimen are: diameter=8.2 cm (the diameter of the extracted sample), height=17.0 cm (approximately two times the diameter, when possible).

2/ Glue ten (2x5) targets to the specimen (Devcon, type Super Glue). The targets are made of magnetic stainless steel (type #304). They are 0.5 mm thick squares (2.5 cm x 2.5 cm) and have a radius of curvature equal to that of the rock specimen; i.e. approximately 4.2 cm.

3/ Measure the initial mass of the specimen, as well as the initial height and initial diameter.

4/ Place a thin film of high vacuum silicone grease (Dow Corning, High Vacuum Grease) around both the bottom cap and the top cap.

5/ Place two filter layers on the bottom cap. The filter layers are made of a very fine mesh 0.114 mm thick geotextile (Tetko, Inc., type PeCap 7-105/52).

6/ Place rock specimen on the bottom cap with the two filter layers.

7/ Place two other filter layers on the top of the rock specimen.

8/ Place the top cap on the two filter layers.

9/ Place three layers of stainless steel foil 0.025 mm thick around both (bottom and top) cap -- rock specimen contacts. This is done to prevent puncturing of the membranes. The sharp edges of the stainless steel foil are covered with electrical tape in order to protect the rubber membranes.

10/ Place the first rubber membrane and the two first sets of O-rings (2x3).

11/ Coat the outside of the first rubber membrane with a thin layer of high vacuum silicone grease.

12/ Place the second rubber membrane and the two second sets of O-rings (2x3). A second rubber membrane is placed around the specimen to prevent damage by silicon oil and to have a redundant protection against puncturing of the first membrane at high cell pressure.

At this stage, the specimen is ready to be placed in the cell in the testing room at a relative humidity of approximately 70%.

14/ Screw the moving support [21] on the ball nut of the leadscrew [5].

15/ Adjust the proximity probes [6] on the moving support [21] and connect them to their respective probe drives (an initial probe response of approximately 7V is recommended).

16/ Set the metal frame [1] in place and attach it into the base [12]. A torque of approximately 200 Nm should be applied to each nut.

17/ Slide the loading piston [28] through the hole of the metal frame [1], and connect it to the load cell [27]. The zero value of the load cell before and after attachment to the piston should be approximately 0.15 mV.

18/ Connect the two water lines [14] to/from the top cap to the metal frame [1].

19/ Check all electrical connections (load cell, pressure transducers, DCDTs, proximity probes, thermistor, input voltages, microstepping motor and the two switches of leadscrew).

20/ Check manually the voltage readings of all channels on the data acquisition system.

21/ Close the cell with the vessel [17] and screw the closure plate (Figure 4-3) onto the metal frame [1].

22/ Attach the air cylinder to the top plate of the loading frame, and connect it to the air system.

23/ Connect the piston of the air cylinder to the axial piston of the triaxial device.

24/ Place the DCDT on the closure plate. This DCDT monitors the axial displacement of the specimen.

25/ Fill the cell with silicone oil (Dow Corning, type Fluid 200 20 cst).

The rock specimen is set-up and the test can start. One should, however, check again manually all the electrical connections and voltage readings of all channels on the data acquisition system.

7 The numbers in brackets [] refer to the parts illustrated in Figure 4-4.
4.5.3.2 Testing

The first testing operation is aimed at re-establishing the estimated in-situ state of effective stress which was acting on the rock specimen. This is done by decomposing the effective stress into total stress and pore water pressure. In order to speed up the saturation process and ensure the full saturation of the specimen, a backpressure of at least 1.0 MPa is recommended in the laboratory. The state of stress to apply to the specimen should therefore be chosen accordingly in order to obtain the required state of effective stress (Appendix C).

Dry-loading

The state of total stress is applied under dry conditions, or in other words, at constant (natural) water content.

An interactive program has been developed for this operation. It is listed in Appendix E. Practically, the operator has to enter the total stress path to be followed by inputting a set of data for the axial stress and the corresponding confining stress. The program will follow the desired stress path by applying the stresses incrementally to the specimen.

Saturation

This procedure describes the successive steps that should be followed in order to achieve saturation of the specimen. The backpressure applied can be chosen at will by the operator. It is, however, recommended to apply a backpressure of at least 1.0 MPa in order to speed up the saturation process. The total stress applied should be chosen accordingly.

1/ Prepare at least two liters of desired water and keep it under vacuum.

2/ Close off the ball valve (BV) of the pore water pressure apparatus (Figure 4-5).

3/ Fill the volume change device with desired water.
4/ Bring the water in the volume change device to a pressure of approximately 1.0 MPa.

5/ Open the micrometer valve (MMV) and the two needle valves (NV1 and NV2) (Figure 4-5).

6/ Apply vacuum at the end of the plastic tubing connected to the micrometer valve for at least five minutes.

7/ Close off the needle valves.

8/ Open the ball valve. Water flushes through the entire pore pressure apparatus.

9/ Open and close the needle valves in an alternating manner. The water is thus forced to flow through the top or bottom of the specimen.

10/ Refill the volume change device when needed.

11/ Repeat steps 8/ and 9/ until a total volume of water equivalent to four times the volume of the volume change device has run through the pore pressure apparatus.

12/ Repeat steps 8/ to 10/ until no air bubbles appear at the downstream end of the micrometer valve.

13/ Refill the volume change device completely.

14/ For saturation, let the specimen sit for at least one week. A ten day period is desirable.

It should be noted that if saturation is difficult to achieve, one has to flush the pore pressure apparatus with desired water once every day in order to drive the air out of the apparatus.

**B-check**

Once the saturation process is completed, one proceeds to a B-check. The procedure followed is that proposed by Wissa (1969). It consists of increasing the confining and axial stress isotropically in successive loading steps while recording the response of the pore pressure. If the ratio of pore pressure to cell pressure increase remains constant for each loading step, the specimen is considered to be saturated. In other words, if $B$ is constant and independent of the loading step, the specimen is considered to be saturated. If, on the other hand, the $B$-value increases during one loading step, the specimen is not yet saturated and the saturation process has to be continued. Note that if a
large increase or decrease in the $B$-value occurs during this operation, the operator should check for internal or external leakage, respectively.

An interactive program has been developed for this $B$-check. It is listed in Appendix E. Practically, the operator has to enter the number of successive loading increments (maximum of five increments) and the total change in isotropic confining stress (0.5 to 1.0 MPa). Generally, three stress increments of 0.2 MPa each have been used. The computer controls the operations automatically during the loading and unloading operations.

After the $B$-check, the specimen is left overnight under the saturation stress (at initial $\sigma'_{c}$ before measuring the $B$-value) in order to equilibrate the pore pressure before shearing.

Shearing

Undrained and drained tests can be performed with the new triaxial system. Stress paths can be followed as desired, such as compression unloading, extension loading or extension unloading (Lambe and Whitman, 1969). Interactive programs have been developed for the shearing tests. They are given in Appendix E.

Practically, the operator has simply to enter the desired stress path. All other operations, including acquisition of data, are controlled by the computer.

In this study, only undrained tests have been performed in the triaxial system because of time considerations. The purpose of undrained tests is to simulate the short-time behavior of the rock just after excavation. The tests are stress controlled, but the stress increments were applied only once the measures excess pore water pressure was constant. All the specimens tested were sheared with an axial strain rate between 0.5 and 1.0 % per hour (Table 5-6).
4.5.4 DATA ACQUISITION

The data acquisition system collects all the necessary information needed for the interpretation of the tests. Analog signals generated by various measuring devices (load cell, pressure transducers, DCDTs, proximity probes, thermistor and input voltages) are converted into digital signals and then stored by the computer. Table 4-1 gives a summary of the data recorded.

4.5.5 CONTROL SOFTWARE

Software programs have been developed to control the operations of dry loading, saturation, B-check and shearing. They are listed in Appendix E. These programs are user-friendly interactive programs. As soon as the operator starts running the main program called "LOAD&SAT" (which stands for "LOADing and SATuration"), all operations are controlled by the computer. The operator’s only duty is to enter the data asked for by the program in order to proceed further. At any time, the operator can choose to halt the operation which is under way by simply stopping the program and rebooting it from the beginning. This method has been found to be extremely simple to use and very flexible.

Test data are recorded simultaneously on the computer hard disk and on a diskette located in the computer drive A. During the operations of dry loading, data are recorded at each loading step. During the saturation process, data are recorded approximately every 30 minutes. For the B-check, 75 data sets are recorded for each stress increment, which lasts for approximately 25 minutes. During shearing, test data are recorded every 30 seconds.
<table>
<thead>
<tr>
<th>Channel #</th>
<th>Description</th>
<th>Measurement of...</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>DCDT of volume change device</td>
<td>Volume change of water, ΔV_w</td>
</tr>
<tr>
<td>5</td>
<td>DCDT for axial displacement</td>
<td>Axial deformation of the rock specimen, δ_a</td>
</tr>
<tr>
<td>6</td>
<td>Proximity sensor #1</td>
<td>Diameter change of rock specimen, δ_d</td>
</tr>
<tr>
<td>7</td>
<td>Proximity sensor #2</td>
<td>Diameter change of rock specimen, δ_d</td>
</tr>
<tr>
<td>8</td>
<td>Pressure transducer of confining pressure</td>
<td>Confining pressure, CP</td>
</tr>
<tr>
<td>9</td>
<td>Not used</td>
<td>--</td>
</tr>
<tr>
<td>10</td>
<td>Pressure transducer of pore water</td>
<td>Pore water pressure, PP</td>
</tr>
<tr>
<td>11</td>
<td>Thermocouple</td>
<td>Temperature, T</td>
</tr>
<tr>
<td>12</td>
<td>Load cell</td>
<td>Axial force, AF</td>
</tr>
<tr>
<td>13</td>
<td>Control of input voltage for proximity sensors</td>
<td>Input voltage ±15 VDC, VPS</td>
</tr>
<tr>
<td>14</td>
<td>Control of input voltage for load cell</td>
<td>Input voltage +12 VDC, VLC</td>
</tr>
<tr>
<td>15</td>
<td>Control of input voltage for DCDT’s and pressure transducers</td>
<td>Input voltage +5.5 VDC, V</td>
</tr>
</tbody>
</table>

**Table 4-1** Channel Allocation of the Data Acquisition System
Figure 4-1 Locations of the Sections Studied: Invert/Crown and Springlines
List of symbols

AP : Air pressure
BV : Ball valve
CP : Cell pressure
DC : Direct current
DP : Driver of proximity probe
LC : Load cell
MV : Microstepping motor
NV : Needle valve
PP : Pore pressure
TR : Thermistor
VD : Vertical displacement
VW : Volume change of water

Figure 4-2 Schematic View of the New Triaxial System
Figure 4-3  General View of the Large Cell
Figure 4-4  Detail View of the Inside of the Large Cell with all Measuring Devices
List of Symbols:

BV  Ball Valve
PT  Pressure Transducer
NV1, NV2 Needle Valves 1 & 2, respectively
MMV  Micrometer Valve

Figure 4-5  Schematic View of the Pore Pressure-Measuring Apparatus
If the point defined by the measured values \((B_{obs}, K)\) falls within the bounded area, the correction to the observed \(B\)-value, \(B_{obs}\), which is made in order to get the "true" \(B\)-value, is less than 10%.

**Note:** The data points correspond to the observed values for tests presented in Chapter 5.

**Figure 4-6** \(B_{obs}-K\) Diagram. Experimental Check-Diagram of the Observed \(B\)-value, \(B_{obs}\), versus the Drained Bulk Modulus of the rock, \(K\), for a Ratio of \(B_{obs}/B_{corr}\leq 90\%\).
**Figure 4-7**  Detail View of the Specimen Set-Ur

- **Top cap**
- **Second set of O-rings**
- **Second membrane**
- **First set of O-rings**
- **First membrane**
- **Electrical tape**
- **Stainless steel foil (3 layers, 0.025 mm thick)**
- **Thin film of high vacuum silicone grease**
- **Filter layers (2) (geotextile)**
- **Rock Specimen**
CHAPTER 5

TEST RESULTS

5.1 BACKGROUND AND PURPOSE OF THE EXPERIMENTAL INVESTIGATION

The tests discussed here were conducted in conjunction with two Swiss tunnel projects, namely, the Transjurane, with two tunnels under Mont Russelin and Mont Terry, and the Wiesenber tunnel. Six triaxial tests were performed on intact specimens of three types of argillaceous rocks: three tests on Oxfordian shale, one on Opalinus shale, and two on Lias Alpha shale. These argillaceous rocks from the Jura Chain of western and north-western Switzerland have a reputation for causing large deformations in tunnels.

The purpose of this experimental investigation is to simulate the instantaneous excavation of a circular tunnel in a medium assumed to be subjected to an isotropic initial state of stress.

5.2 DESCRIPTION OF ARGILLACEOUS ROCKS TESTED

5.2.1 GENERAL

Information on specimen characteristics, intact conditions, and index properties of the specimens tested can be found in Table 5-1.

A detailed mineralogical analysis of the rock samples tested was not performed, but some generally available mineralogical information has been summarized in Table 5-2.

Since the degree of induration of the argillaceous rocks after a particular sample preparation procedure is expected to have a significant effect on physical properties such as liquid limit and clay fraction (%<2μm), it was decided to "free" the clay mineral particles
through a maximum disaggregation process. No chemical dispersing agents were used, however. The natural argillaceous rock samples were air-dried at room temperature (20°C) and then broken by hand until all the particles seemed disaggregated to the naked eye. The pulverized rocks were then thoroughly rehydrated with distilled water, using tempering times of at least six weeks. This procedure was used for preparing samples for the measurements of the liquid limit, \( w_l \), the plastic limit, \( w_p \), the shrinkage limit, \( w_s \), the clay fraction, CF, the drained friction angle, \( \phi' \), the drained residual friction angle, \( \phi'_r \), and the specific gravity of solid grains, \( G_s \) (Table 5-1). The \( \phi \)-angles were determined in a direct shear test. The air-drying of the rock specimens as a part of the sample preparation procedure was considered to be appropriate because irreversible dehydrator, due to the presence of organic matter or clay minerals such as halloysite, was expected to have at most a minor effect on the argillaceous rocks investigated.

The plasticity of the argillaceous rocks investigated here ranges from very low to low on the plasticity chart, as illustrated in Figure 5-1. The materials constituting the Oxfordian and Opalinus shales can be classified as inorganic clays of low to medium plasticity, CL, whereas the material constituting the Lias Alpha shale is an inorganic silt of low plasticity, ML, as defined in the Unified Soil Classification System (USCS).

5.2.2 OXFORDIAN SHALE (TRANSJURANE)

The rock samples were obtained from an horizontal borehole, driven in the springline of the niche at chainage 0.474 Km of the reconnaissance gallery for the Transjurane tunnels, as illustrated in Figure 5-2. At this location, the reconnaissance gallery crosses the center of the overturned fold of the anticline of Mont Terry.

The Oxfordian shale investigated is of marine origin, with low plasticity (Figure 5-1). The dominant clay minerals are kaolinite and chlorite, as listed in Table 5-2. The rock is dark-grey. Upon immersion in distilled water, air-dried samples first displayed a fissile (defined
in Figure 5-3) behavior before readily disintegrating into mud. This observation corresponds to the classification of Cepeda Diaz (1987) based on Atterberg limits and initial water content (Figure 5-3). Significant slaking was observed during the process of disintegration. Cementation does not seem to be likely in the samples tested because of their rapid disintegration in water.

The three samples were contiguous in the ground, as shown in Figure 5-2. The specimens had weakness surfaces which were glossy and had lineations in the direction of the dip. Some of these weakness surfaces were not visible when the specimens were at their natural water content, but become visible upon drying, after a short period of exposure to air in the laboratory. Figure 5-4 illustrates these surfaces on specimen TR2#H5 before testing. They are inclined at angles of 20° to 70° relative to the specimen axis. During sampling, the samples broke into pieces 15 to 40 cm long, along these natural weakness surfaces. The author believes that these weakness surfaces are slicksided and developed when the anticline was folding during the formation of the Alps.

5.2.3 OPALINUS SHALE (WISENBERG)

The Opalinus shale (RB 23-1) sample was obtained from a vertical borehole (Diepfingen RB 23) driven from the surface. The sample was taken at a depth of 23 m.

The Opalinus shale is of marine origin, with low plasticity (Figure 5-1). The dominant clay minerals are illite and kaolinite, as listed in Table 5-2. The rock is dark-grey. Air-dried samples quickly disintegrated into a pile of thin, small plates, with minor surface slaking when immersed in distilled water. The specimen seemed to be massive at first glance, but became fissile, with small cracks appearing parallel to the bedding, upon a few minutes' exposure to air in the laboratory. This observation corresponds to the classification of Cepeda Diaz (1987) based on Atterberg limits and initial water content (Figure 5-3). The immersed small plates were reduced to mud by very little mechanical work. The rock had little, if any, cementation, as small
plates remained unchanged even after the rock had been immersed in water for many days.

5.2.4 LIAS ALPHA SHALE (WISENBERG)

The samples of Lias Alpha shale were obtained from the same vertical borehole as the Opalinus shale (Diepfingen RB 23). The samples, RB 23-9 and RB 23-10, were taken at depths of approximately 73 and 77 m, respectively.

The Lias Alpha shale is of marine origin, with very low plasticity (Figure 5-1). The dominant clay mineral is illite, as listed in Table 5-2. The rock is grey. Air-dried samples seemed at first to be little altered when immersed in water. Nevertheless, after a few hours, they did break into chunks, with preferential orientation of rupture surfaces parallel to the bedding and a few random vertical cracks. This observation corresponds to the classification of Cepeda Diaz (1987) based on Atterberg limits and initial water content (Figure 5-3). The rock had little, if any, cementation, as chunks remained unchanged even after the rock had been immersed in water for more than one month.

The sample RB 23-9 had not been well-protected when it was packed on site, so that significant drying occurred before testing.

Between extraction and testing, small pin-head sized secretions developed on the surfaces of samples RB 23-9 and RB 23-10.

5.3 TEST RESULTS

5.3.1 ISOTROPIC CONSOLIDATION

Once a specimen of rock has been placed in the triaxial apparatus, it is isotropically loaded under dry conditions, as described in Section 4.5.3.2. As the exact initial state of stress in the ground is not known, the standard, and most reasonable, assumption to make is $\lambda_0=1.0$ (Appendix B). The pore water apparatus is then flushed with desired water, and the specimen is saturated for a period of at least ten days.
For instance, the specimen RB 23-10 of Lias Alpha shale, which had dried out before testing, was saturated for more than 50 days. During this period of time, most specimens tested had an initial negative pore pressure, causing intake of water, and swelled, as shown in Table 5-3. An exception to this is the Opalinus shale, which was consolidated far beyond the in-situ state of stress, so that water was actually expelled from the specimen (Table 5-3).

The change in water content of the specimen, $\Delta w = w_c - w_0$, is not entirely due to the change from the in-situ state of stress to the applied state of stress in the laboratory, but depends also on the original degree of saturation of the specimen, $S_0$. For this reason, the saturation process has not been interpreted in detail in this investigation. Table 5-4 summarizes the specimen characteristics in their initial conditions and at the end of the saturation process.

### 5.3.2 B-CHECK

Once saturation was complete, a B-check was performed. The B-check procedure is described in Section 4.5.3.2. Results are reported in Table 5-5.

The observed B-values, $B_{\text{obs}}$, reported in Table 5-5 are mean values of the three loading increments performed. The corrected B-values, $B_{\text{corr}}$, have been evaluated using Equation 4-1:

$$B_{\text{corr}} = \frac{1}{1 - \frac{1}{B_{\text{obs}}}} \Omega$$

(5-1)

where $\Omega$ is the apparatus compressibility, with corresponding drained bulk modulus, $K$, for each specimen. The drained bulk modulus was determined from the measured shear modulus at small strains, $G$, and the drained Poisson's ratio $V$, which was assumed to be equal to 0.3 $^1$. Values of $B_{\text{corr}}$ are reported in Table 5-5. The correction for apparatus compressibility which must be made to the observed B-value, $B_{\text{obs}}$, to

---

$^1 K = [2(1+V)/(3(1-2V))]G = 2.167 \ G$
obtain the "true" $B$-value, $B_{corr}$, is less than 10% for all tests, except for TR#H5 (Figure 4-6).

The value $B_{used}$ is the $B$-value which was used to reduce the shearing test data. It was assumed that $B_{used}=1$ in all tests (Table 5-5). This approximation is believed to have a negligible effect on the final results.

5.3.3 SHEARING

5.3.3.1 General

All the specimens were tested in pure shear ($\Delta \sigma_{oct}=0$) in the triaxial apparatus described in Chapter 4. The test results of the six tests, five in triaxial compression and one in triaxial extension, are summarized in Table 5-6 and Figures 5-5 through 5-16. The shear tests were stress-controlled, but were conducted at a slow incremental stress rate, so that the measured overall axial strain rate, $\varepsilon_a$, never exceeded 1% per hour (Table 5-6).

In Figures 5-5 to 5-16, one can find plots of normalized stress paths, normalized maximum shear stress versus shear strain, normalized excess pore pressure versus shear strain, and normalized shear modulus versus shear strain. In these plots the consolidation effective stress, $\sigma'_c$, has been chosen as the normalizing parameter, so that comparisons between results with different consolidation stresses can be made easily. The basic results of tests in the Oxfordian shale of the Transjurane are presented in Figure 5-5 through 5-8, while those for the Wisenburg are illustrated in Figures 5-9 through 5-12 (Opalinus shale) and 5-13 through 5-16 (Lias Alpha shale). Each set of figures is discussed separately.

5.3.3.2 Oxfordian shale (Transjurane)

The specimens TR#H3 and TR#H4 were isotropically consolidated at $\sigma'_c=2.0$ MPa and were sheared in pure shear compression and pure shear extension, respectively. The specimen TR#H5 was isotropically
consolidated at $\sigma'_c=3.5$ MPa and sheared in pure shear compression. These consolidation stresses represent, respectively, one-third and two-thirds of the estimated in-situ vertical effective stress as given in Table 5-3. Details concerning the consolidation stress and the mode of shearing are given in Table 5-6.

TR#H3

Before failure

The considerable influence of the shear strain, $\gamma$, on the magnitude of normalized excess pore pressure is illustrated in Figure 5-5. For $\gamma<1.25\%$, the normalized excess pore pressure, $\Delta u/\sigma'_c$, generated was positive, with a maximum positive value at approximately $\gamma=0.75\%$. For $\gamma>1.5\%$, the normalized excess pore pressure became negative and reached its value at failure, $\Delta u_f/\sigma'_c=-0.175$, at a small shear strain of 2.06%. This behavior indicates that the material first contracted and then dilated until failure occurred.

The plot in Figure 5-6 shows a monotonic increase in maximum shear stress with increasing shear strain. The curve is slightly concave.

The effective stress path in Figure 5-7 is concave down. Upon shearing, the ratio $\sigma'_o/\sigma'_c$ first decreased because of the positive excess pore pressure generated, and then increased sharply when the excess pore pressure became negative.

The normalized shear modulus decreased continuously with increasing shear strain (Figure 5-8). This behavior indicates that the material degraded considerably upon shearing, as is usual in clays. The curve for TR#H3 in Figure 5-8 is slightly concave, but can be approximated by a straight line for all practical purposes.

---

2 One of the initial goals of this investigation was to study the effect of consolidation stress on the behavior (in compression and in extension) and on the strength of the rock. This goal has been only partially attained in the results presented here, due to sampling problems which occurred in the reconnaissance gallery. The author received only three specimens, instead of the minimum of ten initially forseen.
At failure

The specimen broke into two pieces along a well-defined shear plane, as shown in Figure 5-17. The angle between the shear plane and the direction of the major principal stress (axial stress) was found to be equal to 25°. The shear surface was glossy even when dry and had lineations in the direction of the dip of the failure surface. After failure the specimen displayed a marked strain-softening behavior, which could not be measured, due to the stress-controlled nature of the test procedure.

Stress paths and failure conditions

The stress path and failure conditions of the specimen TR#H3 are reported in Figure 5-18. One can observe that, at low shear stresses, the specimen contracted and positive pore pressure were generated up to point A. At this point, dilation took place, and, consequently, excess pore pressure decreased until failure.

After the point B, the effective stress path, ESP, makes an angle of 45° with horizontal, meaning that the change in pore water pressure was equal to the change in the minor principal stress, i.e. the cell pressure. Failure of the specimen took place when the maximum shear stress \( q_f = s_u = 3.0 \text{ MPa} \). The excess pore water pressure at failure was negative, \( \Delta u_f = -0.304 \text{ MPa} \).

In Figure 5-18, the line originating at the pole and parallel to the measured inclination of the failure plane cuts the Mohr circle at point T, which is the tangent point of the failure envelope.

Two possible explanations of the failure mechanism are:

(a) **failure in intact material**: If one assumes that failure took place in the intact material, the parameters of the Mohr-Coulomb envelope, obtained from Figure 5-18, are: \( \phi' = 40° \) and \( c' = 1.2 \text{ MPa} \).

(b) **failure along a joint with no cohesion**: Assuming that failure took place along a pre-existing natural weakness surface without
cohesion (c'j=0), then the friction angle along the joint, \( \phi'_{j} \), satisfies \( \phi'_{j}=59.5^\circ \) (Figure 5-18).

Discussion

As shown in Figure 5-17, the failure surface was not planar. The average angle of roughness, \( i \), was measured to be 11^\circ. The angle of friction of the material was thus: \( \phi'=\phi'_{j}-i=59.5^\circ-11^\circ=48.5^\circ \). Because this angle is clearly quite large for the material in consideration, there must have been some cohesion along the joint. Thus neither explanation provides much guidance in deciding if failure occurred along a pre-existing weakness surface or in the intact material.

TR#H4

Before describing the results of this test, it should be noted that:

1. The rock specimen had a horizontal discontinuity at mid-height, and
2. The test TR#H4 could not be conducted up to failure because of a breakdown of the load cell. This breakdown occurred while the specimen experienced a shear strain, \( \gamma \), equal to approximately 0.15%. For \( \gamma>0.15\% \), the results showed some inconsistencies in the applied axial stress, so that their interpretation is not possible. For \( \gamma<0.15\% \), however, the test results are believed to be reliable.

The significant influence of the shear strain on the shear-induced pore pressure is illustrated in Figure 5-5. Because only negative excess pore pressures were induced, we can conclude that the rock specimen only dilated upon shearing in extension. The normalized excess pore pressure decreased monotonically, and almost linearly, with increasing shear strain. It is believed that this is a direct consequence of the pressure of the horizontal discontinuity in the specimen. The normalized negative excess pore pressure generated at very small strain (\( \gamma=0.15\% \)) was very high, \( \Delta u/\sigma'_{c}\varepsilon=0.4 \).

The plot of normalized maximum shear stress versus shear strain in Figure 5-6 shows a monotonic increase in maximum shear stress with
increasing shear strain. This relationship is linear for all practical purposes.

In Figure 5-5, the extension test TRH4 displays large negative shear-induced pore pressure, which corresponds to a large increase in the value of $\sigma'_{\text{oct}}/\sigma'_{c}$ in Figure 5-7. At the end of the test $\sigma'_{\text{oct}}/\sigma'_{c}$ had increased by more than 40%.

The normalized shear modulus decreased continuously with increasing shear strain, as illustrated in Figure 5-8. The curve for TRH4 in Figure 5-8 is concave down.

TRH5

Before failure

The considerable influence of the shear strain, $\gamma$, on the magnitude of normalized excess pore pressure is illustrated in Figure 5-5. For $\gamma < 0.08\%$, the normalized excess pore pressure increased. For larger values of the shear strain, the normalized excess pore pressure decreased, reaching its value at failure, $\Delta u_t/\sigma'_{c} = -0.303$, at a small shear strain, $\gamma = 1.37\%$. This behavior indicates that the specimen contracted at first, and then dilated until failure occurred.

The plot in Figure 5-6 shows a monotonic increase in normalized maximum shear stress with increasing shear strain. This relationship can be approximated by two linear segments: one segment with a steep slope, where $\gamma < 0.08\%$, and one segment with a flatter slope, where $\gamma > 0.08\%$.

The effective stress path in Figure 5-7 is concave down. Upon shearing, the ratio $\sigma'_{\text{oct}}/\sigma'_{c}$ decreased at first, because of the positive excess pore pressure generated as a result of the shearing operation. It then increased sharply when the excess pore pressure became negative.

The normalized shear modulus decreased continuously with increasing shear strain, as illustrated in Figure 5-8. The curve for TRH5 is
concave, but can be approximated by two linear segments which intersect at \( \gamma \approx 0.08 \% \).

**At failure**

The specimen broke into two pieces along a well-defined shear surface. The angle between the shear surface and the direction of the major principal stress (axial stress) was estimated at 22.5°. The shear surface had the same characteristics as in specimen TR#H3; i.e. a glossy surface with lineations in the direction of the dip of the shear surface. It is important to note, however, that the specimen did not break along one of the visible pre-existing weakness surfaces (Figure 5-4), but broke along an average conjugate surface. After failure, the specimen displayed a marked strain-softening behavior, which could not be measured due to the stress-controlled nature of the test procedure.

**Stress paths and failure conditions**

The stress path and failure conditions of the specimen TR#H5 are reported in Figure 5-19. One can observe that, at low shear stresses, the specimen contracted and positive pore pressure was generated up to point A. At this point, dilation took place, and consequently excess pore pressure decreased until failure.

After point B, the effective stress path, ESP, makes an angle of 45° with horizontal, meaning that the change in pore water pressure was equal to the change in the minor principal stress, i.e. the cell pressure. Failure of the specimen took place when the maximum shear stress \( q_f = s_u = 3.5 \) MPa. The excess pore water pressure at failure was negative, \( \Delta u_f = -1.060 \) MPa.

In Figure 5-19, the line originating at the pole and parallel to the measured inclination of the failure plane cuts the Mohr circle at point T, which is the tangent point to the failure envelope.
Two possible explanations of the failure mechanism are:

(a) **failure in intact material**: When failure takes place inside the intact material, the Mohr circle is tangent to the failure envelope. Recall that: (1) the cohesion intercept is given by the intersection of the failure envelope with the \( T \)-axis and (2) the \( T \)-intercept of the Mohr-Coulomb envelope always provides an upper bound for the possible values of the cohesion intercept.

In Figure 5-19, the Mohr-Coulomb envelope would be the tangent line to the Mohr circle at \( T \), if failure occurred in the intact material. Upon inspecting Figure 5-19, one observes that the \( T \)-value of the intercept of this tangent line is negative. Thus, the cohesion intercept must be negative as well, which is physically impossible. Therefore, failure could not have occurred in the intact material.

(b) **failure along a joint with no cohesion**: Assuming that failure took place along a pre-existing natural weakness surface without cohesion \((c' = 0)\) the friction angle along the joint, \( \phi'_j \), satisfies \( \phi'_j = 37.7^0 \) (Figure 5-19).

**Discussion**

The failure surface of TR#H5 was nonplanar. The measured average roughness angle, \( i \), was \( 13^0 \). The drained angle of friction of the material was thus: \( \phi' = \phi'_j - i = 37.7^0 - 13^0 = 24.7^0 \). This value seems to be resonable for this material. Since failure cannot take place in the intact material, the failure must have occurred along a pre-existing weakness surface, along which there might have been no cohesion \((c' = 0)\).

It is important to recall, however, that the specimen did not break along any of the visible pre-existing weakness surfaces, but along an average conjugate surface. Thus explanation (b) does not provide much guidance in determining the degree of cohesion along the failure surface.
5.3.3.3 *Opalinus shale (Wisniben)*

The specimen RB 23-1 was isotropically consolidated at $\sigma'_c=5.0$ MPa and sheared in pure shear compression. During consolidation and saturation the specimen lost approximately 14% of its natural water content. This means that the specimen was subjected to stresses which were greater than the in-situ state of stress. The consolidation (vertical) effective stress was about 11 times larger than the estimated in-situ vertical effective stress, as reported in Table 5-3. Details of the consolidation stress and mode of shearing are given in Table 5-6. Test results are reported in Figures 5-9 to 5-12.

**Before failure**

The effect of shearing on the amount of excess pore pressure can be observed in Figure 5-9. The excess pore pressure increased monotonically with increasing shear strain, for $\gamma$ less than approximately 1.1%. It then decreased slightly until failure. An overall positive excess pore pressure was generated at failure, which occurred at $\gamma_f=2.37\%$. In other words, the net reaction of the specimen during shearing was to contract.

The plot in Figure 5-10 illustrates a monotonic increase in maximum shear stress with increasing shear strain. The curve is markedly concave until failure.

The effective stress path plotted in Figure 5-11 is at first slightly convex, and then concave. Upon shearing, the ratio $\sigma'_\text{oct}/\sigma'_c$ first decreased because of the positive excess pore pressure generated. It then increased, while the positive excess pore pressure decreased.

The normalized shear modulus decreased continuously with increasing shear strain, as illustrated in Figure 5-12. This behavior indicates that the material degraded considerably upon shearing, as is usual in clays.
At failure

The specimen broke into small pieces, so that it was not possible to observe any shear plane. Furthermore, as only one specimen of Opalinus shale was investigated in this study, it was not possible to obtain the parameters for the failure envelope.

5.3.3.4 Lias Alpha shale (Wisenberg)

The specimens RB 23-9 and RB 23-10 were isotropically consolidated at $\sigma'_c=2.5$ MPa and sheared in pure shear compression. The consolidation stress applied was about twice as large as the in-situ estimated effective vertical stress, as shown in Table 5-3. Details of the consolidation stress and mode of shearing are given in Table 5-6. The swelling of the specimens during saturation is attributed to the fact that: (1) the specimens might have dried out before testing (for example, RB 23-9 was extremely dry; its original degree of saturation was $S_o=9.6\%$!) and (2) the in-situ $K_o$-value was most probably larger than one ($K_o>1.0$), so that the original octahedral effective stress in the ground was larger than that applied in the triaxial apparatus. Negative excess pore pressures thus existed in the specimens at the beginning of saturation.

Figures 5-13 through 5-16 show that both tests yielded essentially identical results.

It should be noted for test RB 23-10 that cavitation occurred in the water lines of the pore water apparatus because of the significant negative pore pressure generated when approaching failure. The measured pore pressure dropped to 0.0 MPa so that the effective stress path, ESP, close to failure is parallel to the total stress path, TSP-$u_0$, as illustrated in Figure 5-15. It is unlikely, however, that cavitation took place inside the pores and other very small discontinuities of the specimen (Chenevert, 1957), so that the pore pressure is assumed to have continued to decrease. The curves for RB 23-10 in the Figures 5-13 through 5-15 have been extrapolated in order to account for this experimental limitation.
Before failure

The significant influence of the shear strain, $\gamma$, on the amount of normalized excess pore pressure is illustrated in Figure 5-13. For $\gamma < -0.9\%$, the normalized excess pore pressure generated was positive, with a maximum positive value of approximately 0.10 when $\gamma = 0.3\%$. For $\gamma > 0.9\%$, the normalized excess pore pressure was negative and reached a maximum value at failure of $-0.48$ for RB 23-9, and $-0.66$ (extrapolated value) for RB 23-10. This behavior indicates that the specimens contracted first and then dilated until failure occurred. The rate of increase in normalized negative excess pore pressure with increasing shear strain was approximately $-0.85/$(% shear strain) for both specimens. In other words, the specimens generated negative excess pore pressures at a significant rate of $2.1$ MPa/(% shear strain) upon shearing, when approaching the failure envelope.

The plot in Figure 5-14 shows a monotonic increase in maximum shear stress with increasing shear strain. The curves are slightly concave, but can be approximated by linear relationships for all practical purposes. The high values of the maximum shear stress at failure are primarily due to the effect of the significant negative excess pore pressure.

The effective stress path in Figure 5-15 is concave. It has a marked curvature for values of the normalized maximum shear stress, $q/\sigma'_c$, less than one. For $q/\sigma'_c < 0.5$, the material contracted, causing generation of positive excess pore water pressures. For $q/\sigma'_c > 0.5$, however, the material dilated, and negative excess pore water pressures were generated. Once the normalized maximum shear stress was greater than one, the material dilated rapidly, until it reached failure.

A direct consequence of the negative excess pore pressure generated by material dilation was an increase in the ratio $\sigma'_0/\sigma'_c$ to 1.45 and 1.70 at failure for RB 23-9 and RB 23-10, respectively (Figure 5-15).

The normalized shear modulus decreased continuously with increasing shear strain, as illustrated in Figure 5-16. This behavior indicates
that the material degraded considerably upon shearing, as is also observed in clays. The curve for both specimens is slightly concave, but can be well approximated by a straight line for all practical purposes.

At failure

The specimen RB 23-9 broke into several small pieces so that it was not possible to observe any shear surface. The specimen RB 23-10 broke along two conjugate shear surfaces, with average inclinations of 21° with respect to the direction of the major principal stress (axial direction). At failure, both specimens displayed a significant softening behavior, which could not be measured due to the stress-controlled nature of the test procedure.

5.4 DISCUSSION AND SYNTHESIS OF THE TEST RESULTS

For convenience, the Opalinus and Lias Alpha shales are discussed separately from the Oxfordian shale.

5.4.1 OPALINUS AND LIAS ALPHA SHALE

The plots in Figure 5-9 to 5-16 show strong similarities with the behavior observed in uncemented (lightly and highly) overconsolidated clays. In this discussion we will further attempt to establish relationships between clays and the argillaceous rocks tested.

5.4.1.1 A-parameter

Undrained behavior of clays is strongly affected by the excess pore water pressure generated during shear. This is particularly true for overconsolidated clays, for which the excess pore pressure is known to control failure to a far greater extent than the cohesion or the angle of friction does (Ladd, 1985).

It is common practice to measure the shear-induced pore pressure, \( \Delta u \), with respect to a given stress path using the A-parameter
(Skempton, 1954), defined as:

\[ A = \frac{\Delta u - \Delta \sigma_3}{\Delta \sigma_1 - \Delta \sigma_3} \]  \hspace{1cm} (5-2)

where \( \Delta \sigma_1 \) is the change in major principal stress and \( \Delta \sigma_3 \) is the change in minor principal stress.

The \( A \)-parameter must be measured in the laboratory. Test results plotted in Figure 5-20 show that the \( A \)-parameter is linearly related to the shear strain. For the Opalinus shale, the \( A \)-parameter is constant for all practical purposes, since the slope of the linear relationship is so small. It is decreasing with increasing shear strain for the Lias Alpha shale. Figure 5-21 illustrates the same relationship but in a semi-logarithmic plot. It is interesting to note that the \( A \)-value at very small strain (\( \gamma \leq 0.001\% \)) can be extrapolated to 1/3. This value is equal to the \( A \)-value for an isotropic linear elastic material, sheared in triaxial compression. Results further show that, at very small strains (\( \gamma < 0.001\% \)), the specimen already behaves strongly non-linearly.

### 5.4.1.2 Hyperbolic model

In clays, the stress-strain curve can be very well approximated by a hyperbolic curve, which can be expressed in our case as (Figure 5-22):

\[ \frac{2q}{\sigma'_{c}} = \frac{\gamma}{m+n\gamma} \]  \hspace{1cm} (5-3)

where: \( q \) is the maximum shear stress, \( \sigma'_{c} \) is the isotropic consolidation stress, \( \gamma \) is the shear strain, and \( m \) and \( n \) are coefficients equal to the ordinate-intercept and the slope, respectively, of the straight line in the figure.

Results for the Opalinus shale and Lias Alpha shale are reported in Figure 5-23. For the Opalinus shale, the fit is remarkably good with coefficients: \( m=0.38 \) and \( n=0.42 \). For the Lias Alpha shale, the fit is not as good, although the trends are still quite clear. For small shear strain, the model is not very accurate; but for shear strains larger
than, say, $\gamma = 0.25\%$, the model is good for all practical purposes. The parameters for the Lias Alpha shale are: $m = 0.30$ and $n = 0.14$.

### 5.4.1.3 Undrained shear strength

In this section, we assume that the argillaceous rocks (Opalinus and Lias Alpha) behave like overconsolidated low plasticity clays. Measured values of undrained shear strength are compared with values estimated from empirical relationships known to hold for soils with normalizable behavior.

For isotropically consolidated undrained triaxial compression (CIUC) soil tests, the undrained strength, $\sigma_u$, can be estimated with the following empirical relationship (Ladd et al., 1977):

$$\frac{\sigma_u}{\sigma_c} = s (OCR)^m$$  \hspace{1cm} (5-4)

where:
- $\sigma_c$: isotropic consolidation stress
- OCR: overconsolidation ratio
- $s$: parameter, $s = 0.35 \pm 0.05$
- $m$: parameter, $m = 0.8 \pm 0.11$

The value of OCR has to be known in order to use Equation 5-4. For isotropically consolidated tests, one can define OCR by (Ladd et al., 1977):

$$OCR = \frac{\sigma'_{cm}}{\sigma'_c}$$  \hspace{1cm} (5-5)

where $\sigma'_{cm}$ is the maximum past isotropic effective stress, and $\sigma'_c$ is the isotropic consolidation effective stress.

Determination of $\sigma'_{cm}$ is easy for remolded soil specimens tested in the triaxial apparatus, as it is equal to the maximum isotropic effective stress applied to the specimen during the test. For argillaceous rocks which have an unknown stress history, $\sigma'_{cm}$ can be more complicated to determine. No attempt has been made to determine this parameter experimentally in this thesis, but the effect of varying it will be discussed below.
For a given $\sigma'_{\text{cm}}$ and $\sigma'_{\text{c}}$, OCR and $s_u/\sigma'_{\text{c}}$ ($=q'_{\text{f}}/\sigma'_{\text{c}}$) can be computed for any test and the corresponding points (OCR,$s_u/\sigma'_{\text{c}}$) plotted in a log-log plot. This has been done in Figure 5-24 for each test performed in this investigation, assuming initially that $\sigma'_{\text{cm}}=9.0$ MPa for all specimens (Lias Alpha and Opalinus).

The lower and upper boundaries of Equation 5-4, at $s=0.30$ and $m=0.70$ and for $s=0.40$ and $m=0.90$, respectively, are also plotted in Figure 5-24. Furthermore, results of CIUC tests RB 23-5 and RB 23-6 (Opalinus) performed by Aristorenas (1991) have been added to complement the data obtained by the author.

From Figure 5-24 it is clear that:

- The trends observed for the argillaceous rocks tested are similar to those for soils.

- Most of the data points lie above the range for soils (Ladd, 1985).

- Results of the tests of Lias Alpha shale are consistent with those of Opalinus shale.

- A straight line approximation drawn through these data points yields the values $s=0.74$ and $m=0.60$.

The question now arises of what effect of $\sigma'_{\text{cm}}$ has on the values obtained for $s$ and $m$. This can be answered easily by choosing another value of $\sigma'_{\text{cm}}$, e.g. $\sigma'_{\text{cm}}=20.0$ MPa, and replotting the data on the same log-log plot. This was done in Figure 2-25, where one observes that all data points are horizontally shifted by the same amount from their previous positions. In the case $\sigma'_{\text{cm}}=20.0$ MPa, one obtains $s=0.45$ and $m=0.60$.

In general, changing $\sigma'_{\text{cm}}$ only shifts the plot horizontally. Thus the value of $\sigma'_{\text{cm}}$ is related to the value of $s$, but not to that of $m$. This result is extremely interesting, as it implies that the undrained shear strength of the rock, $s_u$, can be estimated for any $\sigma'_{\text{c}}$, as long as one uses the same value of $\sigma'_{\text{cm}}$ in the tests and in-situ.
5.4.2 OXFORDIAN SHALE (TRANSJURANE)

The plots in Figure 5-5 to 5-8 present results which do not differ significantly from those obtained for the Lias Alpha shale. These results are, however, interpreted separately because of: (1) their different origin and (2) the uncertainty concerning failure, i.e. whether it occurred in the intact material or along pre-existing natural weakness surfaces. For practical reasons, the test TR#H4, which was conducted in extension and was stopped at very small strain, will not be further discussed.

The discussions of the stress paths and failure conditions for TR#H3 and TR#H5 lead to the conclusion that there is no general pattern for failure in Oxfordian shale. The failure surface of specimen TR#H3 could have developed either in the intact material or along a pre-existing weakness surface with some cohesion, while on the other hand, the failure of specimen TR#H5 occurred along a weakness surface with unknown cohesion.

5.4.2.1 $A$-parameter

Figure 5-26 presents the A-parameter as a function of the shear strain. We first observe an increase in the A-parameter for small shear strain, and then a practically linear decrease until failure occurs. The curves plotted do not reveal differences when compared with the results obtained for the Lias Alpha shale and reported in Figures 5-20(b) and 5-21(b).

5.4.2.2 Hyperbolic model

It can be seen in Figure 5-27 that the approximation by a hyperbolic model is good for the specimen TR#H3. The model parameters can be estimated as: $m=0.38$ and $n=0.16$. These values do not differ much from the values obtained for the Lias Alpha shale. As already observed in Figure 5-23(b) for the Lias Alpha shale, the model is not very accurate at small strain ($\gamma<0.25\%$), but becomes more accurate past this strain
value. For the specimen TR#H5, however, the model is less satisfactory because of the presence of a pre-existing weakness surface which governs the behavior of the specimen.

5.4.3 FINAL COMMENTS

Although this study was performed on a limited set of rock specimens, the results obtained show interesting trends which integrate soil and rock behavior. For the Oxfordian shale, evidence has been presented of failure both in the intact material and along pre-existing weakness surfaces. More tests are needed, however, to establish a distinction in behavior between specimens with and without weakness surfaces.

For the Lias Alpha and Opalinus shales, the results are consistent enough to indicate that strong similarities do exist between the uncremented homogeneous shales tested and overconsolidated clays. In particular, the measured undrained strength in the shales tested was well approximated by the undrained strength calculated from an empirical relationship very similar to that applied to overconsolidated clays. One assumption underlying the application of this relationship to a material is that it exhibits normalizable behavior with respect to the consolidation stress $\sigma'_c$. Normalizable behavior is unexpected for shales because: (1) their water content at the end of consolidation is less than one-half the shrinkage limit, and (2) they have been subjected to a stress history which may have involved very complex mechanisms, such as rotation of stresses.

If this experimental evidence is further confirmed, the range of materials for which normalizable models are valid will be considerably broadened, since the shales tested have an initial water content which is far beyond the previously known limits within which materials could be assumed to have normalizable behavior.
<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Specimen Name</th>
<th>Depth (m)</th>
<th>$d_0$ (cm)</th>
<th>$h_0$ (cm)</th>
<th>$b_0$ (%)</th>
<th>$e_0$ (1/1)</th>
<th>$S_0$ (%)</th>
<th>$w_1$ (%)</th>
<th>$w_p$ (%)</th>
<th>$w_s$ (%)</th>
<th>PI (%)</th>
<th>CF (%)</th>
<th>PI/CF (%)</th>
<th>$\phi'$ (deg)</th>
<th>$\phi''_c$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxfordian shale</td>
<td>TRH3</td>
<td>160</td>
<td>8.019</td>
<td>13.580</td>
<td>6.08^1</td>
<td>0.244</td>
<td>68.1</td>
<td>43.6</td>
<td>23.6</td>
<td>14.4</td>
<td>23.0</td>
<td>35</td>
<td>0.66</td>
<td>—</td>
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<td></td>
<td>TRH4</td>
<td>160</td>
<td>8.064</td>
<td>13.404</td>
<td>8.51</td>
<td>0.247</td>
<td>95.4</td>
<td>44.1</td>
<td>19.9</td>
<td>16.7</td>
<td>24.9</td>
<td>39</td>
<td>0.64</td>
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<td>—</td>
</tr>
<tr>
<td></td>
<td>TRH5</td>
<td>160</td>
<td>7.967</td>
<td>13.588</td>
<td>8.20</td>
<td>0.243</td>
<td>92.0</td>
<td>50.4</td>
<td>21.0</td>
<td>16.8</td>
<td>29.4</td>
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<td>17.1</td>
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<td>RB 23-1</td>
<td>21.80</td>
<td>7.864</td>
<td>16.579</td>
<td>9.56</td>
<td>0.269</td>
<td>99.9</td>
<td>31.5</td>
<td>18.6</td>
<td>16.2</td>
<td>12.9</td>
<td>30</td>
<td>0.43</td>
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<tr>
<td>Lias Alpha shale</td>
<td>RB 23-9</td>
<td>72.70</td>
<td>7.872</td>
<td>15.758</td>
<td>0.82^2</td>
<td>0.159</td>
<td>9.6</td>
<td>28.6</td>
<td>23.7</td>
<td>15.6</td>
<td>4.9</td>
<td>35</td>
<td>0.15</td>
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<td></td>
<td>RB 23-10</td>
<td>77.20</td>
<td>7.875</td>
<td>15.036</td>
<td>6.16</td>
<td>0.175</td>
<td>99.5</td>
<td>28.2</td>
<td>23.5</td>
<td>15.8</td>
<td>4.7</td>
<td>32</td>
<td>0.14</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

1. significant drying before testing
2. from EPFL-LMR (1987)
3. from Balzari and Schudel (1989)
4. determined in direct shear box
### Table 5-2
Mineral Composition of Argillaceous Rocks Used in this Study

<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Clay Minerals</th>
<th>Non-clay Minerals</th>
<th>Clay fraction ((&lt;\mu m))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Smectite (%)</td>
<td>Illite (%)</td>
<td>Illite &amp; Smectite (%)</td>
</tr>
<tr>
<td>Oxfordian shale 1</td>
<td></td>
<td>10-18</td>
<td></td>
</tr>
<tr>
<td>Opalinus shale 2</td>
<td></td>
<td>26-22</td>
<td>10-18</td>
</tr>
<tr>
<td>Lias Alpha shale 2</td>
<td></td>
<td>28-30</td>
<td>8-16</td>
</tr>
</tbody>
</table>

1 from boreholes in vicinity of experimental niche (from EPFL-LMR, 1987), Transjurane
2 from borehole RB 23 (Diepflingen), (from Balzari and Schudel, 1989), Wisenberg
<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Specimen Name</th>
<th>$\sigma'_{vo}$ (MPa)</th>
<th>$\sigma'_{vc}$ (MPa)</th>
<th>$\sigma'<em>{vc}/\sigma'</em>{vo}$</th>
<th>$w_0$ (%)</th>
<th>$w_i$ (%)</th>
<th>$\Delta w$ (%)</th>
<th>Consolidation or Swelling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxfordian shale</td>
<td>TR/H3</td>
<td>5.67</td>
<td>1.99</td>
<td>0.35</td>
<td>6.08₁</td>
<td>9.74</td>
<td>+3.66</td>
<td>Swelling</td>
</tr>
<tr>
<td></td>
<td>TR/H4</td>
<td>5.67</td>
<td>1.98</td>
<td>0.35</td>
<td>8.51</td>
<td>9.12</td>
<td>+0.61</td>
<td>Swelling</td>
</tr>
<tr>
<td></td>
<td>TR/H5</td>
<td>5.67</td>
<td>3.50</td>
<td>0.62</td>
<td>8.20</td>
<td>9.09</td>
<td>+0.89</td>
<td>Swelling</td>
</tr>
<tr>
<td>Opalinus shale</td>
<td>RB 23-1</td>
<td>0.45</td>
<td>4.99</td>
<td>11.09</td>
<td>9.56</td>
<td>8.23</td>
<td>-1.33</td>
<td>Consolidation</td>
</tr>
<tr>
<td>Liassic Alpha shale</td>
<td>RB 23-9</td>
<td>1.15</td>
<td>2.53</td>
<td>2.20</td>
<td>0.82₁</td>
<td>5.32</td>
<td>+4.50</td>
<td>Swelling</td>
</tr>
<tr>
<td></td>
<td>RB 23-10</td>
<td>1.36</td>
<td>2.51</td>
<td>1.85</td>
<td>6.16</td>
<td>6.41</td>
<td>+0.25</td>
<td>Swelling</td>
</tr>
</tbody>
</table>

1 significant drying before testing

**Table 5-3**  Change in Water Content During Saturation
<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Specimen Name</th>
<th>Depth (m)</th>
<th>$d_0$ (cm)</th>
<th>$h_0$ (cm)</th>
<th>$\nu_0$ (%)</th>
<th>$e_0$ (%)</th>
<th>$\lambda_0$ (%)</th>
<th>$S_0$ (%)</th>
<th>$d_1$ (cm)</th>
<th>$h_1$ (cm)</th>
<th>$\omega_1$ (%)</th>
<th>$e_1$ (%)</th>
<th>$S_1$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxfordian shale</td>
<td>TRA#3</td>
<td>160</td>
<td>8.019</td>
<td>13.580</td>
<td>6.08$^3$</td>
<td>0.244</td>
<td>68.1</td>
<td></td>
<td>8.042</td>
<td>13.624</td>
<td>9.74</td>
<td>0.265</td>
<td>101.8</td>
</tr>
<tr>
<td></td>
<td>TRA#4</td>
<td>160</td>
<td>8.064</td>
<td>13.404</td>
<td>8.51</td>
<td>0.247</td>
<td>2.77$^1$</td>
<td></td>
<td>8.089</td>
<td>13.365</td>
<td>9.12</td>
<td>0.251</td>
<td>100.7</td>
</tr>
<tr>
<td></td>
<td>TRA#5</td>
<td>160</td>
<td>7.967</td>
<td>13.588</td>
<td>8.20</td>
<td>0.243</td>
<td>92.0</td>
<td></td>
<td>7.966</td>
<td>13.521</td>
<td>9.09</td>
<td>0.240</td>
<td>104.9</td>
</tr>
<tr>
<td>Opalinus shale</td>
<td>RB 23-1</td>
<td>21.80</td>
<td>7.864</td>
<td>16.579</td>
<td>9.56</td>
<td>0.269</td>
<td>2.81$^2$</td>
<td></td>
<td>7.837</td>
<td>16.378</td>
<td>8.24</td>
<td>0.231</td>
<td>100.2</td>
</tr>
<tr>
<td></td>
<td>RB 23-9</td>
<td>72.70</td>
<td>7.872</td>
<td>15.758</td>
<td>0.82$^3$</td>
<td>0.159</td>
<td>2.82$^2$</td>
<td></td>
<td>7.886</td>
<td>15.564</td>
<td>5.31</td>
<td>0.150</td>
<td>99.8</td>
</tr>
<tr>
<td></td>
<td>RB 23-10</td>
<td>77.20</td>
<td>7.875</td>
<td>15.036</td>
<td>6.16</td>
<td>0.175</td>
<td>2</td>
<td></td>
<td>7.866</td>
<td>14.945</td>
<td>6.41</td>
<td>0.180</td>
<td>100.3</td>
</tr>
</tbody>
</table>

1 from EPFL-LMR (1987)
2 from Balzari and Schudel (1989)
3 significant drying before testing
<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Specimen Name</th>
<th>Step 1</th>
<th>Step 2</th>
<th>Step 3</th>
<th>Mean</th>
<th>$B_{corr}$</th>
<th>$B_{used}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxfordian shale</td>
<td>TR#H3</td>
<td>0.89</td>
<td>0.90</td>
<td>0.90</td>
<td>0.90</td>
<td>0.93</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>TR#H4</td>
<td>0.92</td>
<td>0.93</td>
<td>0.92</td>
<td>0.92</td>
<td>0.99</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>TR#H5</td>
<td>0.88</td>
<td>0.85</td>
<td>0.96</td>
<td>0.86</td>
<td>0.96</td>
<td>1.00</td>
</tr>
<tr>
<td>Opalinus shale</td>
<td>RB 23-1</td>
<td>0.80</td>
<td>0.81</td>
<td>0.81</td>
<td>0.81</td>
<td>0.84</td>
<td>1.00</td>
</tr>
<tr>
<td>Lias Alpha shale</td>
<td>RB 23-9</td>
<td>0.96</td>
<td>0.96</td>
<td>0.94</td>
<td>0.95</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>RB 23-10</td>
<td>0.92</td>
<td>0.93</td>
<td>0.92</td>
<td>0.92</td>
<td>0.99</td>
<td>1.00</td>
</tr>
</tbody>
</table>

1 using Equation 5-1 and the corresponding drained bulk modulus for each specimen.

Table 5-5  Observed, Corrected, and Used $B$-Values
<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Specimen Name</th>
<th>True Values</th>
<th>Normalized Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Simulation in Field</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stress/strain</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Specimen</td>
<td>5% o.d.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stress/strain</td>
<td>5% o.d.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specimen</td>
</tr>
</tbody>
</table>

Table 5-6 Summary of Test Results: (a) True Values and (b) Normalized Values
1 test stopped at 0.15% shear strain
2 extrapolated value (cavitation)
3 significant drying before testing
<table>
<thead>
<tr>
<th>Geological Formation</th>
<th>Specimen Name</th>
<th>$s_u/\sigma'_c$ computed</th>
<th>$s_u/\sigma'_c$ measured</th>
<th>Relative Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Opalinus shale</td>
<td>RB 23-1</td>
<td>1.05</td>
<td>1.05</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>RB 23-5$^1$</td>
<td>2.36</td>
<td>2.53</td>
<td>+6.7</td>
</tr>
<tr>
<td></td>
<td>RB 23-6$^1$</td>
<td>2.36</td>
<td>2.15</td>
<td>-9.8</td>
</tr>
<tr>
<td>Lias Alpha shale</td>
<td>RB 23-9$^2$</td>
<td>1.58</td>
<td>1.48</td>
<td>-6.8</td>
</tr>
<tr>
<td></td>
<td>RB 23-10</td>
<td>1.58</td>
<td>1.68</td>
<td>+6.0</td>
</tr>
</tbody>
</table>

Data from Figure 5-24.

$^1$ tests performed by Aristorenas (1991)

$^2$ significant drying before testing

NB: The relative error is computed as:

$$\left\{\left(\frac{s_u/\sigma'_c}_{\text{meas}} - \frac{s_u/\sigma'_c}_{\text{comp}}\right) / s_u/\sigma'_c_{\text{meas}}\right\} \times 100\%$$

Table 5-7  Comparison between Measured and Computed Normalized Undrained Shear Strength
Unified Soil Classification System (USCS)

Oxfordian shale → CL
Opalinus shale → CL
Lias Alpha shale → ML

Figure 5-1 Plasticity Chart
**Mont Terry**

**a) General view**

**b) Detailed view**

---

**Figure 5-2** Sketch of an Experimental Niche and Reconnaissance Gallery of Mont Terry in the Swiss Jura
**Class:** F: Fissile  
 M: Massive  

**Subclass:**  
1: deteriorates upon wetting  
2: deteriorates upon drying or wetting  
3: deteriorates upon drying

**Fissile:** deterioration along planes of stratification and other planes, followed by splitting of thin slabs in direction perpendicular to planes (Cepeda Diaz, p.602)

**Massive:** no preferred orientation of discontinuities, breakage into chunks and chips along random cracks, fractures and slickenside surfaces (Cepeda Diaz, p.603)

**Figure 5-3** Characterization of Shale Deterioration  
(from Cepeda Diaz, 1987)
Figure 5-4  Oxfordian Shale - Pre-existing Natural Weakness Planes

Before Testing in Specimen TR#H5
Figure 5-5  Oxfordian Shale - Plot of Normalized Excess Pore Pressure versus Shear Strain

Figure 5-6  Oxfordian Shale - Plot of Normalized Maximum Shear Stress versus Shear Strain
Figure 5-7  Oxfordian Shale - Plot of Normalized Maximum Shear Stress versus Normalized Octahedral Effective Stress

Figure 5-8  Oxfordian Shale - Logarithmic Plot of Normalized Shear Modulus versus Shear Strain
Figure 5-9 Opalinus Shale - Plot of Normalized Excess Pore Pressure versus Shear Strain

Figure 5-10 Opalinus Shale - Plot of Normalized Maximum Shear Stress versus Shear Strain
Figure 5-11 Opalinus Shale - Plot of Normalized Maximum Shear Stress versus Normalized Octahedral Effective Stress

Figure 5-12 Opalinus Shale - Logarithmic Plot of Normalized Shear Modulus versus Shear Strain
Figure 5-13 Lias Alpha Shale - Plot of Normalized Excess Pore Pressure versus Shear Strain

Figure 5-14 Lias Alpha Shale - Plot of Normalized Maximum Shear Stress versus Shear Strain
Figure 5-15 Lias Alpha Shale - Plot of Normalized Maximum Shear Stress versus Normalized Octahedral Effective Stress

Figure 5-16 Lias Alpha Shale - Logarithmic Plot of Normalized Shear Modulus versus Shear Strain
Figure 5-17 Oxfordian Shale - Shear Plane in TR#H3
Figure 5-18 Oxfordian Shale – Total and Effective Stress Paths and Failure Conditions for TR#53
Figure 5-19 Oxfordian Shale - Total and Effective Stress Paths and Failure Conditions for TRSH5
Figure 5-20 Plot of the $A$-Parameter versus Shear Strain: (a) Opalinus Shale and (b) Lias Alpha Shale
Figure 5-21 Semi-Logarithmic Plot of the A-Parameter versus Shear Strain: (a) Opalinus Shale and (b) Lias Alpha Shale
Figure 5-22 Schematic Plot of the Hyperbolic Model
Figure 5-23 Hyperbolic Model: (a) Opalinus Shale and (b) Lias Alpha Shale
Note: Results for specimens RB-5 and RB-6 provided thanks to Mr. George Aristomenas.

**Figure 5-24** Experimental Data for Opalinus and Lia Alpha Shales Compared with the Experimental Relationship of the Normalized Undrained Shear Strength versus the Overconsolidation Ratio.

**Empirical Relationship:**

\[ \frac{s_u}{\sigma'_c} = S \ (OCR)^m \]  

(for soil)  

where \( S = 0.35\pm0.05 \)  

\( m = 0.80\pm0.10 \)
Note: Results for specimens RB-5 and RB-6 provided thanks to Mr. George Aristorenas.

Figure 5-25 Effect of $\sigma'_cm$ on Experimental Relationship of Normalized Undrained Shear Strength versus OCR
Figure 5-26 Oxfordian Shale – Plots of the A-Parameter versus Shear Strain: (a) Arithmetic Scale, and (b) Semi-Logarithmic Scale
Figure 5-27 Oxfordian Shale - Hyperbolic Model
CHAPTER 6

DESIGN METHODOLOGY

6.1 INTRODUCTION

Results obtained in Chapter 5 show that the behavior of argillaceous rocks may include characteristics of both soil and rock mechanics. In rock mechanics, the presence of joints in the material may be of decisive importance, but there is still too little experimental data to provide a thorough understanding of the undrained behavior of rock with weakness surfaces. This type of rock will therefore not be considered further in this chapter. In the case of continuous argillaceous rocks, however, although the set of data is still quite limited, consistent results have been obtained indicating behavior which is similar to that observed in clays, in particular overconsolidated clays. In the following, we will concentrate on argillaceous rocks which behave similarly to Opalinus and Lias Alpha shales.

This chapter will first present a methodology to follow in order to obtain design parameters for further studies. An example will then be presented, describing the instantaneous effect of the excavation of a tunnel on the behavior of the rock in its vicinity.

6.2 DESIGN METHODOLOGY

Test results analyzed in Chapter 5 show that normalized behavior for the argillaceous rocks tested can be reasonably well predicted for engineering practice. In this experimental investigation, the in-situ state of stress was assumed to be isotropic; i.e. $K_0=1.0$. This is not generally the case, however, and in practice, evaluation of the in-situ state of stress and stress history is required in order to generate the necessary normalized parameters.
6.2.1 EVALUATION OF THE IN-SITU STATE OF STRESS AND STRESS HISTORY

The recompression technique, which was discussed in Section 4.3, has been proposed as a laboratory technique to minimize the effect of disturbance. This technique requires an estimation of the in-situ $K_0$-value, but errors in the $K_0$-value are probably not significant compared to adverse effects of disturbance, as pointed out by Jamiolkovski et al. (1985). The measurement of $K_0$ can be done in-situ with methods such as hydraulic fracturing, overcoring or undercoring. Additional information about the in-situ state of stress can be obtained by measuring the apparent maximum past pressure in the oedometer or the apparent isotropic maximum past pressure, using relationships between equilibrium water content and relative humidity (Cepeda Diaz, 1987).

The combined measurements of the in-situ state of stress in the ground, together with laboratory tests, should provide an estimate of the stress history of the ground. Profiles of the in-situ state of stress and the maximum past history can be elaborated along boreholes and tunnel axes (Figure 6-1). However, these profiles must always be carefully evaluated in light of the local geological and hydrogeological conditions. In fact, the initial effective state of stress can be derived only upon knowledge of the initial pore water pressure existing in the ground. It is therefore important to know whether it is statical or artesian, for example. Evaluation of these natural conditions is usually difficult to carry out, primarily due to the low permeability of most argillaceous rocks. Since the long term behavior of the rock mass surrounding a tunnel depends greatly on the in-situ hydrogeological conditions, this evaluation should be carried out according to state-of-the-art practice.

---

1 See Goodman (1980) for further discussion of these methods.
2 In Chapter 5
3 See Section
6.2.2 GENERATION OF NORMALIZED PARAMETERS

Once the in-situ state of stress is known, one should run experiments to obtain plots of normalized parameters such as:

\[ \left( \frac{s_u(TC)}{\sigma'_{VC}} \right) \text{ versus } \log \text{ OCR} \]

and

\[ \left( \frac{s_u(TE)}{\sigma'_{VC}} \right) \text{ versus } \log \text{ OCR} \]

where \( s_u(TC), s_u(TE) \) are the undrained shear strengths in triaxial compression and extension, respectively; \( \sigma'_{VC} \) is the in-situ vertical effective stress at the considered depth; and OCR is the corresponding overconsolidation ratio. This is done to model rock behavior in the springlines and crown/invert of a tunnel.

These experiments, which have been discussed in detail in Section 5.3, are composed of three steps each.

1. The specimens are first consolidated to the appropriate in-situ conditions. The state of stress to which the specimens are consolidated should approximate as closely as possible the in-situ vertical effective stress, \( \sigma'_{VC} \), the in-situ horizontal effective stress, \( \sigma'_{HC} \), and the in-situ pore pressure \( u_0 \).

2. After a B-check, the specimens are sheared under conditions which simulate the excavation of a tunnel.

3. Experimental results are reported in normalized plots, so that design parameters can be estimated.

6.2.3 UTILIZATION OF THE DATA

Once normalized plots have been obtained for a given rock layer of a tunnel section in which geological and hydrogeological conditions can be assumed to be uniform, specific design parameters can be obtained. This is easily done once the in-situ OCR is known by (1) selecting a normalized parameter and (2) by multiplying it with the in-situ vertical effective stress, \( \sigma'_{VC} \).
6.3 **EXAMPLE**

This section provides an example how to use test data obtained with the triaxial apparatus to determine the excess pore pressure distribution, as well as changes in shear strain and shear modulus in the springlines of the tunnel. Let us consider the following situation: a 10 m diameter tunnel excavated at a depth of 160 m in a homogeneous layer of the Lias alpha shale \(^3\) (Figure 6-2). The ground water table is located 15 m below the ground surface, and the shale has an average specific gravity of 0.025 MN/m\(^3\). The initial state of stress is assumed to be isotropic; i.e. \(\lambda_0 = K_0 = 1.0\), where \(\lambda_0 = \sigma_{ho}/\sigma_{vo}\) and \(K_0 = \sigma'_{ho}/\sigma'_{vo}\).

The following state of stress is estimated at the depth of the tunnel axis:

\[
\begin{align*}
\sigma_{vo} &= \sigma_{ho} = 0.025 \times 160 = 4.0 \text{ MPa} \\
u_0 &= 0.010 \times (160 - 15) = 1.45 \text{ MPa} \\
\sigma'_{ho} &= \sigma'_{vo} = 4.0 - 1.45 = 2.55 \text{ MPa}
\end{align*}
\]

**Note:** Based on test results, the Terzaghi stress law is assumed to hold, i.e. \(\sigma = \sigma' + u_0\).

When the tunnel is excavated, elements along the horizontal radial distance in the springlines (\(\theta=0^0\) and \(\theta=180^0\)) are subjected to pure shearing (See Appendix C). Tests which simulate this situation are reported in Chapter 5. The behavior of the Lias Alpha shale in the springlines is assumed to be modeled by the behavior observed in the specimen RB 23-10 and reported in normalized plots in Figures 5-13 and 5-16. This test was run with a consolidation effective stress of \(\sigma'_{c}=2.50 \text{ MPa}\), which corresponds to \(\sigma'_{ho}=\sigma'_{vo}\), assumed above.

### 6.3.1 EXCESS PORE PRESSURE DISTRIBUTION

The normalized stress path of test RB 23-10 is illustrated in Figure 5-15. The normalized shear-induced excess pore pressure corresponding to a normalized shear value \(q/q'_{c}\) can be read as the horizontal distance between the effective stress path, ESP, and the total stress path minus the initial pore pressure, TSP-\(u_0\). The distribution of the excess pore
pressure in the springlines, along the radial distance from the tunnel, can be easily determined by the following procedure.

In Appendix C, it was shown that the change in the maximum shear stress, $\Delta g$, is given by (Equation B-27):

$$\Delta g = \sigma_{v_0} \alpha^2$$  \hspace{1cm} (6-1)

where $\sigma_{v_0}$ is the initial vertical stress acting at the level of the tunnel axis, and $\alpha$ is the normalized radial coordinate, $\alpha = R_0/r$, with $R_0$ the nominal tunnel radius and $r$ the radial coordinate.

It can be seen from Equation 6-1 that a value of $\Delta g/\sigma_{v_0}$ corresponds to each normalized coordinate $\alpha$. This ratio can be expressed as $q/\sigma_c$ since $\Delta g = q - q_0$, $q_0 = 0$, given the assumption of $\sigma_{v_0} = \sigma_{n_0} = \sigma_c$. Thus Equation 6-1 is equivalent to:

$$q = \sigma_c \alpha^2$$  \hspace{1cm} (6-2)

Considering Figure 6-3, where the various stress paths (TSP, TSP-$u_0$, and ESP) of test RB 23-10 have been schematically reproduced, we see that the normalized $(q/\sigma'_c)$-axis can be rescaled to a normalized $(q/\sigma_c)$-axis, via multiplication by the ratio $(\sigma'_c/\sigma_c)$. This ratio can be read on the normalized $(\sigma'_{oct}/\sigma'_c)$-axis and is equal to the ratio of the abscissa of the total stress path, TSP, reported, to the abscissa of the total stress path minus the initial pore pressure, TSP-$u_0$. For our example, the ratio $\sigma_c/\sigma'_c$ is 4.0/2.55=1.659, and thus, $\sigma'_c/\sigma_c$=0.637. The ordinates in the normalized $(q/\sigma'_c)$-plot must therefore be multiplied by 0.637. This has been done in Figure 6-4, which illustrates the rescaled plot of $(\Delta g/\sigma_c)$ versus $(\sigma'_{oct}/\sigma'_c)$.

Now, for a given value of $\alpha$, one computes $\alpha^2$, which is equal to $q/\sigma_c$ (Equation 6-2). The normalized excess pore pressure in the springline at the radial coordinate $r=R_0\alpha$ is given in the plot of Figure 6-4 as the horizontal distance between ESP and TSP-$u_0$, at the corresponding value of $\alpha$. Table 6-1 provides the numerical values evaluated for eleven values of $\alpha$ between 0.0 and 1.0.
Figure 6-5 illustrates the excess pore pressure distribution due to the excavation, assuming that the initial pore pressure remains constant in the ground around the tunnel, i.e. that the drawdown of the ground water table due to the tunnel excavation is not taken into account. It can be seen in this figure that large negative pore pressures are generated in the immediate vicinity of the tunnel. At the tunnel wall, the excess pore pressure attains its minimum, approximately -1.5 MPa. The excess pore pressure increases monotonically from the tunnel wall out to a distance of approximately 3.5 m from the wall (8.5 m from center), or \( \frac{r}{R_0} \leq 1.7 \), where it attains an absolute maximum of approximately +0.26 MPa. For \( r \geq 8.5 \) m, the excess pore pressure decreases monotonically. It becomes negligible at a distance of approximately two tunnel diameters. The point at which the excess pore pressure passes from negative to positive is located at \( r = 6.5 \) m or, equivalently, \( \frac{r}{R_0} \leq 1.3 \). Contractant behavior would be observed in the shale for \( r \geq 8.5 \), whereas dilatant behavior would occur in the region bounded by \( r = 5.0 \) m (tunnel wall) and \( r = 8.5 \) m.

It was seen in Chapter 3, however, that the excavation of a circular tunnel produces drawdown of the ground water table (mode 2) (Detournay and Cheng, 1988) and thus excess pore pressures in the direct vicinity of the tunnel. The solution to mode 2 at short times has been plotted in normalized terms, i.e. \( \frac{\Delta u}{u_0} \), in Figures 3-7 and 3-16. For all practical purposes, the normalized excess pore pressure due to mode 2 is assumed to vary linearly with the normalized radial coordinate, from -1.0 when \( \frac{r}{R_0} = 1.0 \) (tunnel wall) to 0 when \( \frac{r}{R_0} = 1.1 \).

The excess pore pressures resulting from shearing and from drawdown of the ground water table must be superimposed in order to model correctly the effects of the tunnel excavation. This has been done in Table 6-2, where numerical values are reported. The radial distribution of the excess pore pressures in the springlines of the tunnel (\( \theta = 0^\circ \) and \( \theta = 180^\circ \)) is illustrated in Figure 6-6.
6.3.2 STRAIN DISTRIBUTION

The same procedure used for the excess pore pressure distribution can be applied to obtain the distribution of the shear strain in the springlines. For values of \( q/Q_c (=q^2) \) between 0.0 and 1.0, one can estimate the shear strain, \( \gamma \), from Figure 6-7. Numerical values are reported in Table 6-1. Figure 6-8 presents the shear strain distribution as a function of the radial coordinate, \( r \). Note that \( \gamma \), which is approximately equal to 1.8% at the tunnel wall, is slightly less than the shear strain at failure (\( \gamma_f = 1.9\% \)). Thus, in our example, the rock at the tunnel wall is close to failure but has not failed yet.

6.3.3 VARIATION IN SHEAR MODULUS

Once the shear strain, \( \gamma \), is known, one can obtain the variation of the shear modulus, \( G \), with the shear strain from Figure 6-9. Then, by using Figure 6-8, one can plot the variation of the shear modulus as a function of the radial coordinate, \( r \), as shown in Figure 6-10. It can be observed that the shear modulus decreases considerably over a zone of approximately one tunnel diameter. Indeed, in our example it decreases by a factor of three; i.e. from 740 MPa at \( r=15 \) m to approximately 240 MPa at \( r=R_0=5 \) m. Numerical values can be found in Table 6-1.

6.4 DISCUSSION

Thus far, one has described how to use the results of undrained tests which simulate the behavior of rock elements surrounding a tunnel, upon excavation of the opening. In particular, we have discussed triaxial compression tests simulating the behavior of rock elements located in the springlines of a circular tunnel excavated in a homogeneous, continuous layer of Lias Alpha Shale subjected to an initial isotropic state of stress. Although the proposed example is limited to the case of the springlines, the same approach can be used for the crown/invert when using results of undrained tests run in triaxial extension.
The main results of this investigation can be summarized as follows:

- Large excess pore pressure are generated in the vicinity of the tunnel. These excess pore pressures are negative near the tunnel wall and positive inside the rock mass.

- The shear strain decreases with the distance from the tunnel wall.

- The shear modulus increases with the distance from the tunnel wall.

It should be recalled here that the stress path to which RB-23 was subjected models exactly mode 1, as defined in poroelastic models (e.g. Detournay and Cheng, 1988). These models predict that (Section 3.2.1.1): (1) there should be no excess pore pressure generation due to tunnel excavation, and (2) the shear modulus should be constant, independent of the shear strain. The test results thus indicate that poroelastic models are of very limited use for describing the natural shale considered.

The changes in pore pressure and shear modulus occur primarily within a zone which extends approximately one tunnel diameter into the rock mass from the tunnel perimeter. Two aspects of rock mass behavior in the springlines of a tunnel are important:

(1) The excess pore water pressures which are generated due to the contractant/dilatant behavior of the rock establish an instantaneous natural water flow from the zone with positive excess pore pressure to the zone with negative excess pore pressure. This will take place over a period of time. In the zone with initially positive excess pore pressure, water will escape the pores; the water content of the rock will decrease; and consolidation will gradually take place. On the contrary, in the zone where the excess pore pressure is initially negative, water will penetrate the pores, and the water content of the rock will increase over time; and swelling will occur.

It should be noted, finally, that the situation described in our example leads to observations which are very similar to those
reported by Terzaghi (1936) from the tunnel de Paris (Case F, Section 2.1.1.6, Figures 2-20 and 2-21).

(2) Upon inspecting Figure 6-10, one observes that the excavation of the tunnel creates a weakened zone of fairly large extent in the direct vicinity of the tunnel. In our example, this zone, which extends over a distance of one tunnel radius (~5 m) has an average shear modulus of 300 MPa, approximately one third of its initial value.
<table>
<thead>
<tr>
<th>$\alpha$ = $R_o/r$</th>
<th>$\gamma$ = $R_o/\alpha$</th>
<th>$\alpha^2$ = $q/\sigma_c$</th>
<th>$\Delta u/\sigma_c$</th>
<th>$\Delta u$ [MPa]</th>
<th>$\gamma$ [l/l]</th>
<th>$g$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>---</td>
<td>0.00</td>
<td>0.000</td>
<td>0.000</td>
<td>0</td>
<td>--</td>
</tr>
<tr>
<td>0.316</td>
<td>15.8</td>
<td>0.10</td>
<td>+0.043</td>
<td>+0.110</td>
<td>0.049</td>
<td>320</td>
</tr>
<tr>
<td>0.447</td>
<td>11.2</td>
<td>0.20</td>
<td>+0.083</td>
<td>+0.212</td>
<td>0.179</td>
<td>190</td>
</tr>
<tr>
<td>0.548</td>
<td>9.1</td>
<td>0.30</td>
<td>+0.098</td>
<td>+0.250</td>
<td>0.344</td>
<td>140</td>
</tr>
<tr>
<td>0.632</td>
<td>7.9</td>
<td>0.40</td>
<td>+0.102</td>
<td>+0.260</td>
<td>0.511</td>
<td>130</td>
</tr>
<tr>
<td>0.707</td>
<td>7.1</td>
<td>0.50</td>
<td>+0.059</td>
<td>+0.150</td>
<td>0.711</td>
<td>110</td>
</tr>
<tr>
<td>0.775</td>
<td>6.5</td>
<td>0.60</td>
<td>+0.018</td>
<td>+0.046</td>
<td>0.961</td>
<td>100</td>
</tr>
<tr>
<td>0.837</td>
<td>6.0</td>
<td>0.70</td>
<td>-0.087</td>
<td>-0.222</td>
<td>1.133</td>
<td>98</td>
</tr>
<tr>
<td>0.894</td>
<td>5.6</td>
<td>0.80</td>
<td>-0.226</td>
<td>-0.576</td>
<td>1.318</td>
<td>98</td>
</tr>
<tr>
<td>0.949</td>
<td>5.25</td>
<td>0.90</td>
<td>-0.400</td>
<td>-1.020</td>
<td>1.512</td>
<td>97</td>
</tr>
<tr>
<td>1.00</td>
<td>5.00</td>
<td>1.00</td>
<td>-0.583</td>
<td>-1.487</td>
<td>1.793</td>
<td>96</td>
</tr>
</tbody>
</table>

**Note**: $R_o = 5.0$ m  
$\sigma_c = 4.00$ MPa  
$\sigma'_c = 2.55$ MPa

**Table 6-1** Numerical Values of Excess Pore Pressure, Shear Strain, and Shear Modulus, Determined along the Radial Direction in the Tunnel Springlines ($\theta=0^\circ$ and $180^\circ$)
<table>
<thead>
<tr>
<th>$\alpha$</th>
<th>$\alpha^2$</th>
<th>(1)</th>
<th>(2)</th>
<th>Superposition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mode 2 $\Delta u$ [MPa]</td>
<td>Stress Path $\Delta u$ [MPa]</td>
<td>$(1) + (2)$</td>
</tr>
<tr>
<td>--</td>
<td>0.0</td>
<td>0.00</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>15.8</td>
<td>0.1</td>
<td>0.00</td>
<td>+0.110</td>
<td>+0.110</td>
</tr>
<tr>
<td>11.2</td>
<td>0.2</td>
<td>0.00</td>
<td>+0.212</td>
<td>+0.212</td>
</tr>
<tr>
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<td>0.3</td>
<td>0.00</td>
<td>+0.250</td>
<td>+0.250</td>
</tr>
<tr>
<td>7.9</td>
<td>0.4</td>
<td>0.00</td>
<td>+0.260</td>
<td>+0.260</td>
</tr>
<tr>
<td>7.1</td>
<td>0.5</td>
<td>0.00</td>
<td>+0.150</td>
<td>+0.150</td>
</tr>
<tr>
<td>6.5</td>
<td>0.6</td>
<td>0.00</td>
<td>+0.046</td>
<td>+0.046</td>
</tr>
<tr>
<td>6.0</td>
<td>0.7</td>
<td>0.00</td>
<td>-0.222</td>
<td>-0.222</td>
</tr>
<tr>
<td>5.6</td>
<td>0.8</td>
<td>0.00</td>
<td>-0.576</td>
<td>-0.576</td>
</tr>
<tr>
<td>5.25</td>
<td>0.9</td>
<td>-0.725</td>
<td>-1.020</td>
<td>-1.745</td>
</tr>
<tr>
<td>5.00</td>
<td>1.0</td>
<td>-1.450</td>
<td>-1.487</td>
<td>-2.937</td>
</tr>
</tbody>
</table>

**Note:** $u_0 = 1.45$ MPa  
$R_0 = 5.0$ m

**Table 6-2** Numerical Values of the Superposition of the Excess Pore Water Pressure due to Mode 2 and to Shearing along the Radial Direction in the Tunnel Springlines ($\theta=0^\circ$ and $180^\circ$)
Note:  
\( \sigma_{vo} \) : original vertical total stress  
\( (\sigma_{ho})_{ave} \) : average original horizontal total stress  
\( u_0 \) : original pore water pressure  
\( \sigma_{vo}' \) : original vertical effective stress  
\( (\sigma_{ho}')_{ave} \) : average original horizontal effective stress  
\( \sigma_{va}' \) : maximum past pressure as obtained in the 1-D oedometer test

Figure 6-1 Profiles of In-Situ State of Stress and Maximum Past Pressure along Boreholes and Tunnel Axes
Lias Alpha Shale

\( \gamma = 0.025 \, \text{kN/m}^3 \)
\( \gamma' = 0.015 \, \text{kN/m}^3 \)
\( \lambda_0 = K_0 = 1.0 \)

\[ \sigma_{vo} = \sigma_{ho} = 0.025 \times 160 = 4.0 \, \text{MPa} \]
\[ \mu_0 = 0.010 \times 145 = 1.45 \, \text{kPa} \]
\[ \sigma'_{vo} = \sigma'_{ho} = 4.0 - 1.45 = 2.55 \, \text{MPa} \]

**Figure 6-2** General Situation of the Tunnel Excavated in the Lias Alpha Shale
Figure 6-3 Schematic of the Transformation from \((q/\sigma'_c)\)-Axis to \((q/\sigma_c)\)-Axis by Multiplying \((q/\sigma'_c)\) by \((\sigma'_c/\sigma_c)\), Which is Read on \((\sigma'_{oct}/\sigma'_c)\)-Axis
Figure 6-4  Lias Alpha Shale - Rescaled Plot of Normalized Maximum Shear Stress versus Normalized Octahedral Effective Stress; \( \frac{q}{\sigma_c} \) versus \( \frac{\sigma_{oct}}{\sigma_c} \)
Figure 6-5  Radial Distribution of the Excess Pore Pressure in the Tunnel Springlines ($\theta = 0^\circ$ and $180^\circ$). Drawdown of Ground Water Table is not Taken into Account
Figure 6-6  Radial Distribution of the Excess Pore Pressure in the Tunnel Springlines ($\theta = 0^\circ$ and $180^\circ$). Drawdown of Ground Water Table is Taken into Account
Figure 6.7
Lias Alpha Shale - Rescaled Plot of Shear Stress versus Shear Strain, \(q/\sigma_c\) versus \(\gamma\)

\[ \gamma(q/\sigma_c = \alpha^2 = 0.50 \rightarrow r = 7.07 \text{ m}) \approx 0.70\% \]
Figure 6-8  Radial Distribution of the Shear Strain in the Tunnel Springlines ($\theta = 0^\circ$ and $180^\circ$)
Figure 6-9  Lias Alpha Shale - Logarithmic Plot of Normalized Shear Modulus versus Shear Strain
Figure 6-10 Variation in Shear Modulus in the Radial Direction in the Tunnel Springlines ($\theta = 0^\circ$ and $180^\circ$)
CHAPTER 7

THESIS SUMMARY, RECOMMENDATIONS, AND SUGGESTIONS FOR FUTURE STUDIES

7.1 THESIS SUMMARY

This thesis is a contribution to a new quantitative approach to the design of tunnels in argillaceous rock. New concepts are presented based on experimental evidence which represent a first step towards more rational tunnel design for argillaceous rock.

The thesis begins with an investigation of the observed behavior of argillaceous rocks in the vicinity of tunnels and an analysis of current design methods. Existing theoretical models of rock behavior around tunnels are critically reviewed, and key parameters for tunnel design are identified. One conclusion drawn from this study is that tunnel designers must take into account the two-phase reaction of argillaceous rock upon tunnel excavation. The interaction of water with the rock mass is of central importance in both phases. In the first phase, the undrained phase, there is an instantaneous pore water pressure change due to excavation of the tunnel opening. The second phase, the drained phase, consists of two subphases, (a) consolidation/swelling and (b) creep, during which one observes the effects of the release of the excess pore pressure and of time, respectively, on the volumetric behavior of the rock surrounding the tunnel.

The primary focus of the thesis is the undrained phase, about which very little is known. In the author's opinion, this phase is extremely important. What occurs during the undrained phase determines the outcome
of the first drained subphase, i.e. whether the rock consolidates or swells.

In order to study the undrained phase, the "Stress Path" method is used as a simplified approach to simulate tunnel excavation. In this thesis, the "Stress Path" method is applied to the problem of a tunnel excavated in a medium subjected to an isotropic state of stress. The relevant stress paths can be shown to be in pure shear: pure shear compression in the springlines of the tunnel, and pure shear extension in the crown and invert.

Six specimens of argillaceous rock (Opalinus shale, Oxfordian shale, and Lias Alpha shale) were tested in a triaxial cell, following the guidelines of the "Stress Path" method. The cell is part of a new computer-controlled triaxial system, designed by the author as one of the main tasks for this thesis. Five of the six specimens were tested in pure shear undrained compression. The sixth specimen was tested in pure shear undrained extension, with results which could not be satisfactorily interpreted, due to the presence of a discontinuity in the specimen and to the breakdown of the load cell.

All specimens tested in compression displayed identical behavior. The specimens contracted first, and then dilated as they approached the failure envelope. Excess pore pressures were generated accordingly, i.e. positive upon contraction and negative upon dilation. This behavior resembles that observed in overconsolidated clays. In most tests, it has been shown that a relationship, similar to that found in soil mechanics, exists between the undrained shear strength, $s_u$, and the overconsolidation ratio, OCR. It can be expressed mathematically by:

$$
s_u/\sigma'_{c} = s \ (OCR)^m
$$

(7-1)

where: $s$: undrained shear strength for normally consolidated material, dependant only on the material,
$s_u$: undrained shear strength, function of OCR,
$m$: exponent depending only on the material,
OCR: overconsolidation ratio.
$\sigma'_{c}$: isotropic consolidation effective stress
In this thesis, the "Stress Path" method is further applied to an example tunnel situation: a 10 m diameter circular tunnel excavated at a depth of 160 m in a homogeneous layer of Lias Alpha shale. Experimental results obtained with the specimen RB 23-10 are assumed to be typical for the layer of Lias Alpha shale. This example focuses on determination of the pore pressure, shear strain, and shear modulus distributions along the radial direction in the springlines of the tunnel.

An analysis shows that the existing poroelastic models do not model these distributions correctly. In particular, the very large excess pore pressures and considerable decreases in shear modulus which arise due to the contractant/dilatant behavior of the shale are not predicted by these models. It is very important to account for these phenomena, as they lead to consolidation and/or swelling.

7.2 RECOMMENDATIONS AND SUGGESTIONS FOR FUTURE STUDIES

7.2.1 ROCK MECHANICS

In order to be able to design a tunnel properly, an engineer needs to have broad knowledge of the properties and behavior of the surrounding rock. In the case of tunneling in argillaceous rock, a thorough understanding of rock mechanics in both the undrained and drained phases is required. Towards this end, the author recommends the following topics for future research:

(1) In Chapter 5, the existence of normalized behavior is hypothesized for continuous argillaceous rock. This hypothesis should be further confirmed before being applied in practice. Experiments should be performed in triaxial compression and extension, under both undrained and drained conditions, to test the hypothesis.

(2) Tests should also be run to determine the effects of the initial $K_0$-value on the behavior and strength of the rock. There are four combinations of conditions under which these tests should
be performed: undrained compression, undrained extension, 
drained compression, and drained extension.

(3) The effects of initially high cohesion (due to cementation 
and/or aging) should be investigated. In particular, one should 
determine how closely argillaceous rocks resemble 
overconsolidated clays, for which models have already been 
developed (e.g. MIT-E3 (Whittle, 1987)). This can be done by 
testing first argillaceous rock specimens, and then 
reconsolidated soil specimens originating from the rock 
material. Comparison of the outcome of these experiments should 
reveal much about the similarities and differences between 
argillaceous rocks and overconsolidated clays.

(4) For discontinuous rocks, the role of discontinuities with 
respect to excess pore water pressure, stiffness (shear and 
bulk), and permeability should be examined.

7.2.2 THEORETICAL MODELS

As discussed in Section 6.4, linear poroelastic models for continuum 
are quite limited. More sophisticated models should therefore be 
developed to describe better the undrained phase, and possibly the 
drained phase as well, in continuous rocks.

The proposed simplified approach based on the "Stress Path" method 
predicts the initial conditions leading to swelling/consolidation around 
the tunnel and is thus more satisfactory than linear poroelastic models. 
It is necessary, however, to develop more general models, which account 
for, e.g. high initial cohesion of the rock, dilation upon shear, and 
decrease in shear modulus. This development should occur in conjunction 
with extensive experimentation. A recommended first step towards such a 
general model is the elaboration of an effective-stress model for 
argillaceous rocks, based on the presently available version of the 
"MIT-E3" model (Whittle, 1987).
7.2.3 PRACTICE

Good quality in-situ measurements of stresses, pore water pressures, and displacements are usually very difficult and expensive to carry out. Nevertheless, they are absolutely essential for integration of the predictions obtained with models using laboratory data into particular field situations, in order to progress towards a safer and more economical design of tunnels in argillaceous rocks.

It is important to emphasize here that partial measurements of stresses, pore water pressures or displacements only are not very useful. An integrated approach is required for measuring:

- the initial state of stress in the ground before excavation,
- the past stress history of the rock mass along various profiles (Figure 6-1),
- the initial water content and the void ratio of the rock along various profiles (Figure 6-1),
- the initial pore water pressure in the ground before excavation and, possibly, whether it is static or artesian.

Particular care should be devoted to:

1. placing the initial state of stress in the context of the past history of the rock mass. For this reason, it is important to determine the maximum past pressure, void ratio, and water content, both in the vicinity of the tunnel and along various profiles, such as boreholes (Figure 6-1).

2. measuring the initial pore water pressures, which take a long time to stabilize after insertion of the piezometer into the ground.
After excavation, one should monitor:

- radial/tangential stresses against/in the lining,
- pore water pressures in the vicinity of the tunnel,
- rock displacements with extensometer measurements,
- lining displacements with convergence measurements, and finally
- displacements of the tunnel axis with general surveying methods.

Measurements of stresses, pore water pressures, and rock, lining, and tunnel displacements after excavation should be performed as soon as possible behind the tunnel face. Such measurements should then be carried out regularly for many years.

7.2.4 CONCLUSION

Laboratory testing, theoretical modeling, and in-situ measurements are all elements of vital importance in state-of-the-art design of tunnels in argillaceous rock.
CHAPTER 8

REFERENCES

Note: ARPA Advanced Research Projects Agency
ASCE American Society of Civil Engineers
ASTM American Society for Testing and Materials
CGT Canadian Geotechnical Journal
ICSMFE International Conference on Soil Mechanics and Foundation Engineering
IJNAMG International Journal for Numerical Methods in Geomechanics
IJSS International Journal of Solide Structures
ISRM International Society of Rock Mechanics
ISSMFE International Society for Soil Mechanics and Foundation Engineering
JSMFD Journal of the Soil Mechanics and Foundation Engineering
JGED Journal of the Geotechnical Engineering Division
MIT Massachusetts Institute of Technology
NRC National Research Council
STP Special Technical Publication


Lefèbre, G., (1990). Measurement of In-Situ K0 from Field and Laboratory Testing. Special Seminar presented at MIT (01/29/90)


Taylor, D.W., (1942). Research on Consolidation of Clays. MIT.


APPENDIX A

SWELLING PRESSURE

A.1 INTRODUCTION

When a rock specimen is placed in the oedometer and is given access to water, its initial volume tends to increase. The axial stress which should be applied in order to prevent any volume change of the specimen is called the swelling pressure. In the existing methods for designing tunnels in swelling rock, which use models based on swelling law (Wittke and Rissler, 1976; Gysel, 1977; Wittke and Pierau, 1979; Kovari et al., 1981; Gysel, 1987b), the swelling pressure is an important parameter which enters the computations. Three laboratory methods for the determination of swelling pressure on rock specimens have gained general acceptance. They are the methods of:

- Huder and Amberg (1970)
- Madsen (1976, 1979) ¹
- Kovari et al. (1981)

In all three methods, tests are run on intact rock specimens in the oedometer. This testing device was primarily developed to simulate the drained 1-D behavior of soils under large surface foundations.

The interest of this appendix lies in the fact that all of these methods are currently used in practice, despite the fact that they provide very different magnitudes of swelling pressure.

The purposes of this appendix are first to analyze these methods with respect to the effects of sample disturbance (Section 4.2), and second

¹ This procedure has been adopted by the Commission on Swelling Rock of the International Society on Rock Mechanics (ISRM, 1987)
to draw conclusions on the use of these methods for practical purposes, in particular for tunneling.

The following procedure is used. First, each method is briefly presented and analyzed. Then, practical conclusions on the use of particular methods are outlined and finally various issues affecting the design of underground works are discussed.

A.2 PRESENTATION OF THE METHODS

A.2.1 HUDER AND AMBERG (1970)

The test procedure used is illustrated in Figures A-1. The swelling pressure, $\sigma_s(H)$, is defined to be the axial total stress which prevents any volume change when the rock specimen is submerged in water. Generally, this stress is not or cannot be reached in the laboratory test. It has to be determined graphically by extrapolation. This is done by intersecting the extrapolated second dry-loading curve (3) with the prolonged swelling curve (5) (Figures A-1).

A.2.2 MADSEN (1976, 1979)

The procedure used is illustrated in Figure A-2. The swelling pressure, $\sigma_s(M)$, is defined to be the vertical axial total stress needed to maintain the initial height of the rock specimen. When micro-heaves occur (0.01 to 0.02 mm), the initial height of the specimen is re-established by increasing the axial total stress. Equilibrium is reached when no further tendency to heave is observed. The corresponding applied axial stress, $\sigma_a$, is called the swelling pressure, $\sigma_s(M)$ (Figure A-2).

A.2.3 KOVARI ET AL. (1981)

The procedure used is illustrated in Figures A-3 and A-4. The swelling pressure, $\sigma_s(K)$, is defined to be the axial total stress which causes a strain, called $\varepsilon^s$, that exactly cancels the strain due to water adsorption, $\varepsilon^w$. $\varepsilon^s$ is due to deformation of the material's matrix structure. The following relationship is proposed for the total axial
strain: \( \varepsilon^{\text{tot}} = \varepsilon^c + \varepsilon \). The swelling pressure, \( \sigma_s(K) \) is the axial total stress which produces \( \varepsilon^{\text{tot}} = 0 \). Its value is obtained graphically by reversing the initial dry-loading curve, defined as \( -\varepsilon^c \), and by intersecting it with the swelling curve. Hence, \( \varepsilon^{\text{tot}} = 0 \) and \( -\varepsilon^c = \varepsilon \) (Figure A-4).

A.3 ANALYSIS OF THE METHODS

It is assumed that "state-of-the-art" tests are run in each method and that even minor volume changes of the rock specimen dependent on the testing apparatus are eliminated. It has been recognized in soil mechanics that the swelling pressure is very sensitive to minor changes such as: (1) omission of compensation for apparatus deformation (Fredlund, 1969), and (2) use of an oedometer with a flexible confining ring (Alpan, 1957; Franklin, 1985). Moreover it is assumed that a "perfect" contact exists between the specimen and the confining ring. This is known to be extremely difficult to obtain in practice, in particular with stiff fissile materials.

A.3.1 HUDER AND AMBERG (1970)

In this method, the first dry-loading (1) and the cycle of dry-unloading (2) -- dry-loading (3) is performed to reduce the effects of undrained expansion and sample disturbance as discussed in Section 4.2 (Figure A-1). The slopes and hysteresis of these curves are expected to be strongly related to the degree of sample disturbance: the larger the sample disturbance, the steeper the slopes and the greater the hysteresis. The water content of the specimen is assumed to remain constant during the steps (1) to (3). However, upon loading, the degree of saturation of the specimen increases because of the decrease in its overall volume, or equivalently, because of the decrease in its overall void space. Physically, this implies a closure of the discontinuities created by sample disturbance, and an advance of the capillary menisci towards the outer boundaries of the specimen. This is accompanied by a decrease of the negative excess pore water pressure, initially generated by the removal of the in-situ confining stresses, when the applied axial total stress is increased. The remaining negative excess pore water
pressure, if any, is released when water is given access to the specimen (4), and the specimen swells (Figure A-1).

In this test, there are no conditions imposed on the vertical strain; the test is thus axially stress-controlled.

The swelling pressure, \( \sigma_s(H) \), is the axial stress which prevents any entry of water in the specimen. For this stress level, the water content of the rock remains constant; i.e. undrained conditions are met. Hudler and Amberg (1970) believed that \( \sigma_s(H) \) is the swelling pressure which occurs in-situ when the effects of sample disturbance have been removed.

A.3.2 MADSEN (1976, 1979)

The swelling pressure, \( \sigma_s(M) \), is equal to the axial total stress needed to maintain the initial height of the unloaded specimen of rock. When water is supplied, it fills the discontinuities created by sample disturbance. Simultaneously, the negative excess pore water pressure decreases as water enters the intact materials. Drained conditions occur as a result of the increase in water content of the specimen.

The release of the negative excess pore pressure produces a tendency for the specimen to swell. This is prevented by increasing the applied axial stress. The test is truly strain-controlled because zero volume change conditions are imposed: laterally the ring confines the specimen and no axial deformation is allowed. The swelling pressure obtained is thus a function of the excess negative pore pressure, and of the increase in void space due to sample disturbance.

Cepeda Diaz (1987) performed constant volume tests in the oedometer on two intact specimens of Taylor shale to study the effect of sample disturbance. The first specimen was set unloaded in the oedometer and then maintained at constant initial volume upon immersion in water. The initial void ratio of this specimen was \( e_o=0.407 \). The second specimen was set in the oedometer, and then submitted to one cycle of dry-loading -- dry-unloading in order to reduce sample disturbance. The maximum axial stress applied was 0.45 MPa, with a corresponding void ratio \( e=0.401 \). Then, the specimen was unloaded without noticeable volume
change. Finally, upon access to water, the volume of the specimen was maintained constant by increasing the axial stress. Results of this tests are reported in Figure A-5. Beside the fact that there are different rates in the development of swelling pressure, it will be noted that the magnitude of swelling pressure is strongly affected. The specimen with the higher sample disturbance shows the lower swelling pressure.

A.3.3 KOVARI ET AL. (1981)

The test is performed similarly to that of Hudar and Amberg (1970) except that there is no cycle of dry-unloading (2) -- dry-loading (3) (Figure A-1). Again, control of the test is obtained through application of axial stress. The test is thus stress-controlled.

The swelling pressure, \( \sigma_3(K) \), is obtained as the intersection of the reversed dry-loading curve (1) with the swelling curve (3) (Figure A-4). The dry-loading curve is known to be strongly affected by instantaneous expansion and sample disturbance. The proposed method makes these effects more predominant.

For the stress level defined by \( \sigma_a = \sigma_3(K) \), drained conditions do occur with increase in the water content of the specimen. If large sample disturbance has taken place, the specimen becomes more deformable and the dry-loading curve displays a large slope. In turn, the reversed dry-loading curve is steeper, and thus its intersection with the swelling curve, which is the obtained swelling pressure, \( \sigma_3(K) \), can be considerably underestimated.

A.4 COMMENTS ON THE METHODS

If three identical, intact specimens of rock are tested, using a different method for each specimen, the result will be three different values of swelling pressure. This is illustrated in Figure A-6, assuming that the magnitude of swelling pressure is not affected by the stress path followed to achieve final equilibrium. The methods differ in their emphasis (deliberate or not on the part of the author(s) of the methods)
on sample disturbance. These effects are reduced to a minimum in the method of Huder and Amberg (1970); they are discarded in the method of Madsen (1976,1979); finally, they are overemphasized in the method of Kovari et al.(1981), in particular if sample disturbance is important.

A.5 DISCUSSION

The water content of the specimen increases after immersion, even under no volume change conditions, because of the effects of sample disturbance. Based on 132 test results \(^2\), obtained with the method of Madsen (1976,1979), it was observed that the initial water content, \(w_0\), is different from the final water content, \(w_c\), (Figure A-7). Usually, \(w_c\) is larger than \(w_0\). For the stress level defined by \(q_a=q_s(M)\), which prevents any volume change upon immersion in water, drained conditions do occur in the test because water enters the specimen to complete specimen saturation until a new equilibrium is reached. This initially unsaturated state can be confirmed by observing the time-dependent developments of swelling pressure in constant volume tests run in the oedometer (Figure A-8). In these tests, the swelling pressure develops with time in a manner similar to the swell-time curves observed in drained tests. Water enters the specimen and increases the degree of saturation from its original value up to or close to 100%. Assuming that \(w_c\) corresponds to 100% saturation of the specimen, it is found, in the set of data reported in Figure A-7, that the initial degree of saturation is equal to approximately 87%. This average value corresponds well with other values reported in the literature, where the degree of saturation generally lies between 70 and 95% (e.g. Cepeda Diaz,1987). Both these results confirm that sample disturbance generally produces an increase in the volume of the specimen and, therefore, in its overall void ratio. Cepeda Diaz (1987) showed that this increase in void ratio considerably reduces the magnitude of the swelling pressure (Figure A-5).

\(^2\) These results come from unpublished reports from ETH-Z, which are listed in Figure A-7.
Cepeda Diaz (1987) noted that not only the initial void ratio, $e_0$, but also the stress path followed to achieve the final equilibrium affect the magnitude of the swelling pressure. Therefore, it seems that, depending on the stress history followed, no unique void ratio-swelling pressure relationship exists but instead there is a range of possible equilibrium void ratios $^3$.

These observations raise the fundamental question of the usefulness of the swelling pressure tests run in the oedometer with regard to practical tunneling applications. Thus, swelling pressure tests as performed currently should be used as index tests only for a rough estimate of the swelling pressure in argillaceous rocks.

A.6 CONCLUSIONS

The three methods which are currently used for the determination of the swelling pressure in argillaceous rocks in the oedometer have been analyzed. The removal of the sample from the ground produces excess negative pore pressure and sample disturbance, which alter the overall initial void ratio, $e_0$, of the specimen. The methods differ in the treatment of the sample disturbance, which is responsible for the differences observed in the magnitude of the swelling pressure.

Since it is also true that the swelling pressure is stress path dependent, it is concluded that the currently used 1-D swelling tests should be used as index tests only in saturated argillaceous rocks, and in particular, in the future, for tunneling application.

$^3$ See Cepeda Diaz (1987, p.93) for further comments.
(a) Testing procedure

1. First dry loading
2. Dry unloading
3. Second dry loading
4. Addition of water under constant axial stress $\sigma_a$, possible swelling strain
5. Swell unloading, final swelling strain as function of axial stress

(b) Test result

Axial strain $e_a$ (%)

Axial stress $\sigma_a$ (MPa)

Logarithmic scale

Figure A-1 Swelling Test after Huder and Amberg (1970)
(a) Testing procedure

(b) Test result

Figure A-2  Swelling Test after Madsen (1979) (from ISRM, 1987)
(a) Testing procedure

(b) Test result

Figure A-3  Swelling Test after Kovari et al. (1981)
Axial strain $\varepsilon_a$ (%)

$\sigma_a$ (K)

Reversed dry loading curve

$\varepsilon_a$:

axial strain

$\varepsilon_a^g$:

axial strain due to deformation of the material's matrix structure

Note: The swelling pressure $\sigma_a(K)$ is defined by the intersection of the reversed dry loading curve 1 with the swell unloading curve 3.

Figure A-4 Determination of Swelling Pressure after Kovari et al. (1981)
Figure A-5  Rate of Development of Swelling Pressure for Intact Taylor Shale at Various Constant Initial Conditions (from Cepeda Diaz, 1987)
Symbols:

$\sigma_s(H)$: Swelling pressure after Huder and Amberg (1970)

$\sigma_s(M)$: Swelling pressure after Madsen (1976, 1979)

$\sigma_s(K)$: Swelling pressure after Kovari et al. (1981)

Note: For numbered sequences in Figure, see Figure A–1.

Figure A–6 Comparison of Swelling Pressure Obtained with the Three Methods, Assuming No Stress Path Effect
Figure A-7  Plot of Final Water Content versus Initial Water Content of Intact Specimens Tested after the Method of Madsen (1979) (Data from unpublished reports from ETH-Z, which are listed in the figure)
Figure A-8  Plot of Swelling Pressure versus Time for Intact Argillaceous Rocks Tested after the Method of Madsen (1976, 1979)
APPENDIX B

ELASTO-PLASTIC CONSTITUTIVE
MODEL FOR ARGILLACEOUS ROCKS

This appendix consists of a paper which was presented by the author at a special workshop on Swelling Rock during the 5th International Congress on Rock Mechanics in Montreal (Canada) in 1987. This paper has been published by Bellwald and Einstein (1987) in the Proceedings of the Congress.

The original paper is reproduced here below. The numbering of paragraph titles and figure captions does not correspond to that of the thesis but should be read as, e.g. B.2.1, instead of 2.1, and Figure B-2, instead of Figure 2, etc.
ELASTO-PLASTIC CONSTITUTIVE MODEL

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H.H. Einstein (USA)

ABSTRACT: In this paper swelling in shales is treated analogously to a consolidation problem. Three main features have to be considered when designing underground structures in swelling shales: (1) the initial state of stress in the ground, (2) the behavior of the swelling shales during the undrained phase and, (3) the behavior of the swelling shales during the drained phase. Yielding can occur during either the undrained phase or the drained phase, leading to a plastic behavior of the swelling shales. In order to better understand this behavior it is necessary to perform undrained and drained laboratory testing under 3-d conditions.

1 INTRODUCTION

The swelling shale problem will be considered in the context of analyzing the behavior of an underground opening in such a medium. Specifically the behavior of swelling rock around an opening will be discussed first. This will be followed by a review of the current laboratory practice with emphasis on its drawbacks. Finally suggestions will be made on further investigations.

2 MECHANICAL BEHAVIOR OF SWELLING SHALES AROUND UNDERGROUND OPENINGS

Swelling in shales is treated analogously to a consolidation problem. Three main phases have to be considered when designing underground structures in swelling shales: (1) the original (natural) phase with the initial state of stress in the ground, (2) the behavior of the swelling shales during the undrained phase and, (3) the behavior of the swelling shales during the drained phase.

The stress path method of Lambe (Lambe and Whitman, 1969) will be used to describe what goes on in each of these phases.

2.1 Initial state of stress

The knowledge of the initial state of stress is one of the key issues in the prediction of the rock mass behavior around an excavation. For our discussion we consider a homogeneous isotropic plate of unit thickness (2-d) (Figure 1).

![Figure 1. 2-dimensional model. Homogeneous isotropic plate of unit thickness.](image)

This plate is subjected to the initial state of stress which is assumed to be completely known at the level of the future excavation. The principal stress directions are defined in Figure 1. Three assumptions are made in this discussion:

1. The computations below will not include the principal stress \( \sigma_0 \) acting perpendicular to the plate. This is done to simplify the discussion, and does not affect the validity of the conclusion.

2. The initial coefficient of lateral stress at rest \( K_o = \frac{\sigma_v}{\sigma_h} \) is assumed to be 1.5, a value which can often be observed in sedimentary rocks, and

3. The future opening lies at a depth of 100m below the surface and 40m below the ground water table (Figure 2).

![Figure 2. p-q diagram. Initial state of stress.](image)

These specific values have been chosen to provide meaningful examples, any other values could have been selected, however.

Figure 2 shows the initial state of effective stress in a p-q diagram where \( p \) is the mean total stress \( \frac{\sigma_x + \sigma_y}{2} \), \( p' \) is the mean effective stress \( \frac{\sigma_x + \sigma_y - 2\sigma_h}{2} \) and \( q \) is the deviatoric stress \( \frac{\sigma_x - \sigma_y}{2} \).

The Mohr-Coulomb envelope is defined by 2 parameters, \( c' \) and \( \phi' \) which are directly obtained from the mechanical parameters \( c' \) and \( \phi' \). \( c' \), the drained cohesion is assumed to be 2.0 MN/m² and \( \phi' \), the drained internal angle of friction is 30°.
2.2 Undrained phase

The excavation of an underground opening changes the stress distribution inside the rock mass. The low permeability of the rock (10^-10 to 10^-12 m/s) and the relatively high rate of excavation (3 to 10 m/day) cause the rock mass to behave in an undrained manner just after excavation. Undrained behavior means that no change in the water content (constant), but a change in pore pressure will occur.

The stress paths of two points located at the circumference of a circular opening; point I in the invert and point S in the springline, are shown in Figure 3 for purposes of providing an example. The total stress paths represent vertical extension loading in the invert and vertical compression unloading in the springline (Lambe and Whitman, 1969). The effective stress paths are assumed to follow the general trends observed in undrained tests on highly overconsolidated uncemented clays (Parry, 1960). The shear induced pore pressure changes δu are negative at both points I and S because of the dilatant behavior of the material upon loading or unloading. The exact form of the stress paths depends on the particular case but two generally valid points can be made:

1. If the effective stress path for either point I or S or both touch the Mohr-Coulomb envelope, yielding will take place: yielding in extension in the invert and yielding in compression in the springline.

2. Cavitation may develop when the pore pressure drops below zero absolute pressure. High negative pressures can occur as shown by Chenetert (Chenevert, 1957), who reported negative pressures of several MN/m² in shales of low porosity. Cavitation would lead to an instantaneous partial drainage at the particular point of the rock mass.

2.3 Drained phase/consolidation and swelling

Unbalanced pore water pressures in the rock mass, the drainage effect of the opening and inflow of external water (from construction operations or air humidity) will change the initial water content of the rock mass with time. If the excess pore pressures are positive, consolidation will take place, while swelling will occur where the excess pore pressures are initially negative. This drained phase will last until the entire rock mass reaches a new equilibrium. One knows, from practical experience, that this can go on for many years.

The following discussion will concentrate on the zone in the invert in which most of the swelling will take place because of the large negative pore pressures developed during the undrained phase (Figure 3). The release of negative excess pore pressures during the drained phase will decrease the effective stress drastically and swelling will occur.

![Figure 3. p-q diagram. Stress path of points I and S during undrained phase.](image)

In a p-q diagram the point I can only move inside the hatched area towards the origin (Figure 4). At present one does not know the exact form of this path. Laboratory testing is needed to obtain a better understanding of the swelling behavior during the drained phase.

Nevertheless, two conclusions can be drawn from the preceding considerations:

1. The horizontal effective stress σʰ remains always larger than the vertical effective stress σᵥ because the point I moves below the p-axis.

2. Once the stress path of point I touches the Mohr-Coulomb envelope, yielding occurs; swelling will thus lead to failure.

![Figure 4. p-q diagram. Stress path of point I inside hatched area during drained phase.](image)
An example of yielding occurring during swelling is illustrated in Figure 5 in which the result of an oedometer test on an undisturbed swelling clay-shale are presented (Sun Jun et al., 1984). During this test, the vertical and horizontal stresses were recorded.

Figure 5a presents the raw data. The same data are plotted in a p-q diagram in Figure 5b which shows that yielding take place followed, in this particular case, by a progressive reduction of the mechanical parameters \( a' \) and \( \phi' \), thus of \( c' \) and \( \phi' \); \( c' \) goes to zero while \( \phi' \) decreases towards its residual value \( \phi'_{res} \). Ultimately, the rock will reach this residual shear resistance.

![Figure 5a](image1)

**Figure 5a.** 1-dimensional swelling test. Vertical and horizontal stresses as recorded during test (after Sun Jun et al., 1984).

![Figure 5b](image2)

**Figure 5b.** p-q diagram. 1-dimensional swelling test (after Sun Jun et al., 1984).

3. PRESENT LABORATORY PRACTICE

As shown in the preceding discussion, undrained and drained tests should be performed to predict the complete swelling behavior.

3.1 Undrained tests

To the author's knowledge, no systematic research has been undertaken to understand better the undrained behavior of swelling shales. As one can see in Figure 3, important stress changes occur inside the rock mass when an opening is excavated. In particular, we note a dramatic increase in the mean effective stress. Undrained tests should tell us how important this increase will be. The estimation of the state of effective stresses in the ground just after excavation is important because it is the starting point of the effective stress path of the drained phase and thus considerably affects the behavior of the rock during the drained phase.

3.2 Drained tests

Drained tests are mostly, if not exclusively, performed with the oedometer by following the Huder-Amberg procedure (Figure 6) (Huder and Amberg, 1970). As can be seen in this figure the resulting swelling curve relates the axial vertical strain \( \varepsilon_v \) to the axial vertical stress \( \sigma_v \). The major drawback of this test is its inability to record the radial horizontal stress (except for the version developed by Sun Jun et al., 1984), an issue which will be addressed in section 4 below.

It would be desirable to have testing equipment and procedures allowing one to conduct both drained and undrained tests in a consistent manner and under 3-d conditions. At MIT, we have therefore developed a new computer controlled triaxial cell in which undrained and drained tests on swelling shales will be performed along predetermined stress-or strain paths.

![Figure 6](image3)

**Figure 6.** 1-dimensional swelling test (after Huder and Amberg, 1970).
4 THREE DIMENSIONAL BEHAVIOR

In order to extend the 1-d swelling law as obtained with the oedometer tests to three dimensions, one generally assumes that the horizontal stress is proportional to the vertical stress by the coefficient \( v/(1-v) \) where \( v \) is the Poisson’s ratio. This expression is derived from the theory of elasticity (Einstein et al., 1972). To check this assumption, the recorded values of \( K_0 \) obtained by Sun Jun et al. (1984) have been plotted versus the vertical strain \( \epsilon_v \), and they are compared with \( K_0 \) as one would obtain based on Poisson’s ratio as described above (Figure 7). By making the assumption of proportionality, one greatly underestimates the \( K_0 \)-value both in the elastic and in the plastic ranges. 3-d behavior therefore needs to be considered not only in the analysis but by conducting appropriate laboratory tests. The new MIT approach is to perform pseudo-oedometer tests in the triaxial cell by controlling the strain path of the specimen during swelling.

\[
K_0 = \frac{q_h}{q_v}
\]

\[1-d \text{ swelling test}\]

**Figure 7.** \( K_0 \)-value as a function of the vertical strain. Comparison between measured and \( K_0 \)-computed values (after Sun Jun et al., 1984 for measured data).

5 CONCLUSIONS

It was demonstrated that in addition to the initial state of stress, two basic phases should be considered and analyzed when excavation in swelling shales is performed: an undrained phase and a drained phase. In order to obtain a good understanding of the behavior of swelling shales around underground openings, laboratory tests need to be performed for these two phases and under 3-d conditions.

ACKNOWLEDGEMENTS

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APPENDIX C

PRACTICAL AND THEORETICAL CONSIDERATIONS FOR THE
DETERMINATION OF THE TESTING PROGRAM

C.1 "STRESS PATH" METHOD

The "Stress Path" method, developed by Lambe (1967) and further discussed by Lambe and Marr (1979), is an approach to stability and deformation problems in soil mechanics. In general it consists of the following two steps:

(1) estimate the history and variation of stress and strain for one or more elements of soil in the actual field structure, and

(2) use soil tests (laboratory or field, or both) and analytical techniques that approximate the field stress and strain conditions before and after construction.

"The basic philosophy of the "Stress Path" method is: because strain, pore water pressure, and strength of a soil element depend on the stress path, the engineer should consider the field stress paths (or strains, or both) in selecting the soil-testing procedure and method of analysis for any given problem. Frequently, the stress paths and the behavior of the soil in actual problem are such that conventional testing techniques are sufficient. These situations can be recognized readily from the stress paths for the problem at hand and from the results of tests on the particular soil involved....". (Lambe, 1967).

A stress path is a line drawn through points on a stress diagram. Each point is defined by the maximum shear stress of a given stress state, as illustrated in Figure C-1. Thus, a stress path is the locus of points of maximum shear stress experienced by an element in going from one state of stress to another.
The three stress paths basic to the "Stress Path" method are (Figure C-2) ¹:

**Total Stress Path: TSP**

\[
\frac{\sigma_1 - \sigma_3}{2} \text{ versus } \frac{\sigma_1 + \sigma_3}{2} \text{ or } q \text{ versus } p
\]  
(C-1)

This is the stress path applied to a soil element.

**Path of total stress minus static pore pressure: TSP-\(u_0\)**

\[
\frac{\sigma_1 - \sigma_3}{2} \text{ versus } \left(\frac{\sigma_1 + \sigma_3}{2} - u_0\right) \text{ or } q \text{ versus } (p-u_0)
\]  
(C-2)

This is the stress path applied to a fully saturated element of soil, taking into account the initial static pore pressure, \(u_0\), acting before shear.

**Effective Stress Path: ESP**

\[
\frac{\sigma_1 - \sigma_3}{2} \text{ versus } \left(\frac{\sigma_1 + \sigma_3}{2} - u_0\right) - \Delta u \text{ or } q \text{ versus } p' = (p-u_0) - \Delta u
\]  
(C-3)

This is the stress path followed by a fully saturated element of soil when sheared.

where: \(\sigma_1\) : major principal total stress,

\(\sigma_3\) : minor principal total stress,

\(u_0\) : initial static pore pressure,

\(\Delta u\) : excess pore pressure generated during shear,

\(q\) : maximum shear stress, \(q = \frac{\sigma_1 - \sigma_3}{2}\),

\(p\) : mean total stress, \(p = \frac{\sigma_1 + \sigma_3}{2}\),

\(p'\) : mean effective stress, \(p' = p - u_0 - \Delta u\).

The "Stress Path" method ignores the intermediate principal stress, \(\sigma_2\). Justification for ignoring \(\sigma_2\) has been given by Lambe and Marr (1979) as follows:

---

¹For further information on the "Stress Path" method, refer to Lambe and Whitman (1969), pp.112-115.
- the triaxial apparatus has proven the most useful equipment for performing stress path tests. In the triaxial apparatus, \( \sigma_2 \) must equal either the major principal stress, \( \sigma_1 \), or the minor principal stress, \( \sigma_3 \); i.e. one cannot control \( \sigma_2 \) independently of \( \sigma_1 \) and \( \sigma_3 \).

- usually \( \sigma_2 \) only slightly affects the strength and stress-strain behavior of soil.

- an important attribute of the stress path method is that it furnishes a simple framework for understanding a geotechnical situation. Trying to incorporate \( \sigma_2 \) would detract from the simplicity of the stress path method.

The original philosophy of the "Stress Path" method would, therefore, lead to following stress paths imposed by the simultaneous variations of \( \sigma_1 \) and \( \sigma_3 \) only; i.e. \( \Delta \sigma_1 \) and \( \Delta \sigma_3 \).

For drained tests, there is no practical difficulty in following the stress paths which simulate field conditions. For undrained tests, however, excess pore pressures generated during shear should be taken into consideration. In the triaxial apparatus, the excess pore pressure, \( \Delta u \), during undrained shear can be expressed as (Skempton, 1954):

\[
\Delta u = B \left( \Delta \sigma_3 + A \left( \Delta \sigma_1 - \Delta \sigma_3 \right) \right)
\]  
(C-4)

where: \( A, B \): pore pressure parameters,
\( \Delta \sigma_1, \Delta \sigma_3 \): change in major and minor principal stress, respectively.

Equation C-4 consists of two terms: (1) \( B \Delta \sigma_3 \), which expresses the change in isotropic pressure and (2) \( BA(\Delta \sigma_1 - \Delta \sigma_3) \), which describes the dependence of \( \Delta u \) on the incremental deviator stress \( \Delta \sigma_1 - \Delta \sigma_3 \).

The order of magnitude of the stress changes in the rock surrounding a tunnel excavation is several MPa. Thus, the excess pore pressure is expected to be large. As an example, we consider a tunnel excavated in a linear elastic isotropic medium subjected to an initial isotropic state of stress \( \sigma_0 \). At the tunnel wall, we have the following stress changes (e.g. Obert and Duvall, 1967):
\[
\Delta \sigma_1 = \sigma_0 \\
\Delta \sigma_3 = -\sigma_0
\]  

To follow such a stress path in a triaxial apparatus would involve very large negative excess pore pressures because of: (1) unloading of the isotropic stress, and (2) the expected strong dilation of the material upon shearing. More specifically, if we assume that the rock behaves as a heavily overconsolidated clay, the excess negative pore pressure at failure, with \( B_f = 1.0 \) and \( A_f = 0 \) to \(-0.5\) (Skempton, 1954), is expected to be:

\[
\Delta u_f = -2.0 \sigma_0 \text{ to } -1.0 \sigma_0
\]  

For example, if one wants to simulate the stress path around a tunnel located at a depth of 200 m, the negative excess pore pressure generated is expected to be in the order of \(-10 \) to \(-5\) MPa. This is a tremendous negative excess pore pressure, which is difficult to measure in the laboratory. In order to measure such high negative pore pressures, one has to apply an initial backpressure, \( u_0 \), to avoid cavitation of the water during the test; i.e. \( u = \Delta u_f + u_0 > 0 \). For such conditions, special equipment is needed because the cell pressure and the axial stress should be increased corresponding to the applied initial backpressure.

As the excess pore pressure is the major element of interest in this study, it has been decided to modify the "Stress Path" method. The change in octahedral stress, \( \Delta \sigma_{oct} \), and the maximum deviator stress, \( \Delta q \), are used as variables instead of \( \Delta \sigma_1 \) and \( \Delta \sigma_3 \). The total stress paths proposed can differ appreciably from the one originally proposed by the "Stress Path" method, but it is known from soil mechanics that the TSP has virtually no influence on the effective stress path, ESP; i.e. there is a unique ESP for all combinations of TSP as long as distinction is made between compression and extension (Lambe and Whitman, 1969).
Equation C-4 can be rewritten as (Skempton, 1954):

\[
\Delta u = B \left[ (\Delta \sigma_1 + 2 \Delta \sigma_3) / 3 + (3A-1)(\Delta \sigma_1 - \Delta \sigma_3) / 3 \right] \quad (C-7a)
\]

or \[
\Delta u = B \left[ \Delta \sigma_{\text{oct}} + 2(3A-1)\Delta q / 3 \right] \quad (C-7b)
\]

Equations C-7a and C-7b are equivalent.

The different stress paths have been redefined as (Figure C-3):

**Total Stress Path: TSP**

\[
\frac{(\sigma_1 - \sigma_3)}{2} \text{ versus } \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} \quad \text{or } q \text{ versus } \sigma_{\text{oct}} \quad (C-8)
\]

This is the stress path applied to a soil element.

**Path of total stress minus static pore pressure: TSP\(_u_0\)**

\[
\frac{(\sigma_1 - \sigma_3)}{2} \text{ vs } \left( \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} - u_0 \right) \quad \text{or } q \text{ vs } (\sigma_{\text{oct}} - u_0) \quad (C-9)
\]

This is the stress path applied to a fully saturated element of soil, taking into account the initial static pore pressure, \(u_0\), acting before shear.

**Effective Stress Path: ESP**

\[
\frac{(\sigma_1 - \sigma_3)}{2} \text{ vs } \left( \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} - u_0 \right) - \Delta u \quad \text{or } q \text{ vs } (\sigma_{\text{oct}} - u_0) - \Delta u \quad (C-10)
\]

This is the stress path followed by a fully saturated element of rock when sheared.

where:

- \(\sigma_1\) : major principal total stress,
- \(\sigma_2\) : intermediate principal total stress,
- \(\sigma_3\) : minor principal total stress,
- \(u_0\) : initial static pore pressure,
- \(\Delta u\) : excess pore pressure generated during shear,
- \(q\) : maximum shear stress, \(q=(\sigma_1 - \sigma_3)/2\),
- \(\sigma_{\text{oct}}\) : octahedral total stress, \(\sigma_{\text{oct}}=(\sigma_1 + \sigma_2 + \sigma_3)/3\).

We examine now the total stress path (TSP) for the case of a circular tunnel excavated in a homogeneous, isotropic elastic medium. The initial, or prior-excavation, state of stress and the post-excavation
state of stress are studied, assuming that the excavation can be simulated in 2-D; i.e. in plane strain excavation.

At the time at which testing was performed in this study, the initial in-situ states of stress, at the locations where the rock specimens were taken, were not known. Therefore, it has been assumed that the initial in-situ state of stress is isotropic, i.e. $\lambda_0=1.0$, which is the standard assumption when the initial in-situ state of stress is not known.

C.2 INITIAL OR PRIOR-EXCAVATION STATE OF STRESS

The initial, or prior-excavation, state of stress is assumed to be isotropic, i.e. the initial horizontal and vertical stresses are identical: $\sigma_{ho} = \sigma_{vo}$. It can be expressed in cylindrical coordinates as shown in Figure C-4 as:

\[
\begin{align*}
\sigma_{rr.0} &= \sigma_{vo} \\
\sigma_{\theta\theta.0} &= \sigma_{vo} \\
\sigma_{zz.0} &= \sigma_{vo} \\
\sigma_{r\theta.0} &= 0
\end{align*}
\]  

(C-11a) \hspace{1cm} (C-11b) \hspace{1cm} (C-11c) \hspace{1cm} (C-11d)

where: $\sigma_{rr.0}$ : initial radial stress,  
$\sigma_{\theta\theta.0}$ : initial tangential stress,  
$\sigma_{zz.0}$ : initial longitudinal stress parallel to the tunnel axis,  
$\sigma_{r\theta.0}$ : initial shear stress,  
$\sigma_{ho}$ : initial horizontal stress,  
$\sigma_{vo}$ : initial vertical stress.

It will be noted that the initial stress distribution is independent of any material properties.

The initial octahedral normal stress is:

\[
\sigma_{oct.0} = (\sigma_{rr.0}+\sigma_{\theta\theta.0}+\sigma_{zz.0})/3 = \sigma_{vo}
\]  

(C-12)
The initial maximum shear stress is (Figure C-5):
\[
\sigma_0 = (1/2)(\sigma_{rr,0} - \sigma_{\theta\theta,0})^2 + (\sigma_{r\theta,0})^2)^{1/2} = 0 \tag{C-13}
\]

C.3 POST-EXCAVATION STATE OF STRESS

C.3.1 Basic Equations

Assuming plane strain conditions during the unloading process due to excavation, the post-extraction state of stress written in cylindrical coordinates is (Figure C-4):
\[
\begin{align}
\sigma_{rr} &= \sigma_v(1-\alpha^2) \tag{C-14a} \\
\sigma_{\theta\theta} &= \sigma_v(1+\alpha^2) \tag{C-14b} \\
\sigma_{zz} &= \sigma_v \tag{C-14c} \\
\sigma_{r\theta} &= 0 \tag{C-14d}
\end{align}
\]
where:
- $\sigma_{rr}$: radial stress,
- $\sigma_{\theta\theta}$: tangential stress,
- $\sigma_{zz}$: longitudinal stress parallel to the tunnel axis,
- $\sigma_{r\theta}$: shear stress,
- $\sigma_v$: initial vertical stress,
- $\alpha = R_0/r$: ratio of the radius of the tunnel, $R_0$, to the radial coordinate, $r$, $0.0 \leq \alpha \leq 1.0$. 

$\alpha \to 0$ corresponds to a far-field condition, whereas $\alpha=1.0$ represents a location at the wall of the excavation (Figure C-4).

It will be noted that the post-extraction state of stress is independent of any material properties.

The post-extraction octahedral normal stress is:
\[
\sigma_{\text{oct}} = (\sigma_{rr} + \sigma_{\theta\theta} + \sigma_{zz})/3 = \sigma_v \tag{C-15}
\]
The post-excavation maximum shear stress is (Figure C-5):
\[ q = \{(1/2)^2(\sigma_{rr} - \sigma_{\theta\theta})^2 + (\sigma_{r\theta})^2\}^{1/2} = \sigma_v \alpha^2 \]  \hspace{1cm} (C-16)

C.3.2 Definition of compression and extension zones around the opening

The maximum shear stress, \( q \), is non-negative by definition. Therefore it is impossible to distinguish compression from extension. In practice, however, it is important to know if a particular point inside the rock mass experiences compression or extension, because the respective rock behavior and the amount of excess pore pressure (positive or negative) generated during the undrained phase following the excavation are expected to be different.

In a \( \sigma_{oct} - q \) plot, compression and extension are defined as shown in Figure C-6. Mathematically, one of the following conditions has to be fulfilled:

for compression

\[ (1) \; \Delta \sigma_v > 0 \; \text{and} \; \Delta \sigma_h < 0 \]  \hspace{1cm} (C-17a)
\[ (2) \; \Delta \sigma_v > 0 \; \text{and} \; \Delta \sigma_h > 0 \; \text{and} \; \Delta \sigma_v > \Delta \sigma_h \]  \hspace{1cm} (C-17b)
\[ (3) \; \Delta \sigma_v < 0 \; \text{and} \; \Delta \sigma_h < 0 \; \text{and} \; \Delta \sigma_v > \Delta \sigma_h \]  \hspace{1cm} (C-17c)

for extension

\[ (1) \; \Delta \sigma_v < 0 \; \text{and} \; \Delta \sigma_h > 0 \]  \hspace{1cm} (C-18a)
\[ (2) \; \Delta \sigma_v > 0 \; \text{and} \; \Delta \sigma_h > 0 \; \text{and} \; \Delta \sigma_v < \Delta \sigma_h \]  \hspace{1cm} (C-18b)
\[ (3) \; \Delta \sigma_v < 0 \; \text{and} \; \Delta \sigma_h < 0 \; \text{and} \; \Delta \sigma_v < \Delta \sigma_h \]  \hspace{1cm} (C-18c)

In the following, formulae for \( \Delta \sigma_v \) and \( \Delta \sigma_h \) are developed as functions of \( \alpha \) and \( \theta \). The post-excavation vertical and horizontal stresses, \( \sigma_v \) and \( \sigma_h \) respectively, are obtained from basic transformations (rotation of frame) in a Mohr circle:

\[ \sigma_v = (\sigma_{rr} + \sigma_{\theta\theta})/2 - ((\sigma_{rr} - \sigma_{\theta\theta})/2)\cos 2\theta + \sigma_{r\theta} \sin 2\theta \]  \hspace{1cm} (C-19)
\[ \sigma_h = (\sigma_{rr} + \sigma_{\theta\theta})/2 + ((\sigma_{rr} - \sigma_{\theta\theta})/2)\cos 2\theta - \sigma_{r\theta} \sin 2\theta \]  \hspace{1cm} (C-20)
Using the relationships established in Equation C-14 for $\sigma_{rr}$, $\sigma_{\theta\theta}$ and $\sigma_{r\theta}$, Equations C-19 and C-20 yield:

$$\sigma_v = \sigma_{v_0} (1 + \alpha^2 \cos 2\theta) \quad \text{(C-21)}$$

$$\sigma_h = \sigma_{v_0} (1 - \alpha^2 \cos 2\theta) \quad \text{(C-22)}$$

Further simple computations lead to the following relationships for the two first conditions in Equation C-17 and C-18 for the change in stresses in the vertical and horizontal directions:

$$\Delta \sigma_v = \sigma_v - \sigma_{v_0} = \sigma_{v_0} \alpha^2 \cos 2\theta \quad \text{(C-23)}$$

Similarly,

$$\Delta \sigma_h = \sigma_h - \sigma_{h_0} = -\sigma_{v_0} \alpha^2 \cos 2\theta \quad \text{(C-24)}$$

Subtracting Equation C-24 from Equation C-23 gives an expression for the third condition in Equation C-17 and C-18:

$$\Delta \sigma_v - \Delta \sigma_h = 2 \sigma_{v_0} \alpha^2 \cos 2\theta \quad \text{(C-25)}$$

Figure C-7 illustrates the zones in compression and in extension around the opening as defined by Equation C-17 or C-18 with Equation C-23, C-24 and C-25. It will be noticed that compression is found in the springlines, whereas extension develops in the crown/invert.

C.4 CHANGE BETWEEN PRIOR- AND POST-EXCAVATION STATES OF STRESS

Equations C-11 and C-14 express respectively the prior- and post-excavation states of stress around the circular opening, assuming plane strain conditions during the unloading process due to excavation. Nevertheless, of primary interest are the change in the octahedral normal stress, $\Delta \sigma_{oct}$, and the change in the maximum shear stress, $\Delta \sigma$. $\Delta \sigma_{oct}$ is obtained by subtracting Equation C-12 from Equation C-15:

$$\Delta \sigma_{oct} = \sigma_{oct} - \sigma_{oct,0} = \sigma_v - \sigma_{v_0} = 0 \quad \text{(C-26)}$$
The change in maximum shear stress, $\Delta q$, is obtained by subtracting Equation C-13 from Equation C-16:

$$\Delta q = q - q_0 = \sigma_v \alpha^2 \quad (C-27)$$

The sign of $\Delta q$ ($\Delta q = (\Delta \sigma_v - \Delta \sigma_h)/2$) is determined by the sign of Equation C-25 and can readily be obtained from Figure C-7.

C.5 **STRESS PATH**

For an initial isotropic state of stress, the following observations can be made:

1. **Any element in the ground is subjected to a stress path in pure shear plane strain ($\Delta \sigma_{oct} = 0$, $\Delta q = f(\theta)$) (Figure C-8).**

2. **The maximum shear stress generated during the excavation process is maximum at the wall of the opening and decreases proportionally to $\alpha^2$. At a distance of approximately 1.4 times the radius of the excavation ($\alpha = 0.70$), it decreases by a factor of one half.**

3. **For any element in the ground, the state of stress can be found using the Mohr circle. Figures C-9 and C-10 illustrate the fact that the excavation process produces a rotation of the principal stresses. For all $\theta$-angles, we observe plane strain pure shear conditions with rotation of the principal stresses. Two locations are of particular interest:**

   a. **The springlines ($\theta = 0^\circ, \theta = 180^\circ$) where any element in the ground experiences a plane strain pure shear in compression** (Figure C-9), and

   b. **The crown ($\theta = 90^\circ$) and the invert ($\theta = 270^\circ$) where the stress path can be described as a plane strain pure shear in extension** (Figure C-9), for any element in the ground.
C.6 DISCUSSION AND CONCLUSIONS

In this study, it has been decided to test specimens of argillaceous rock in a triaxial apparatus. This versatile apparatus, which is widely used in practice, cannot simulate rotation of the principal stresses. It has been shown in Figure C-9 and C-10 that rotation of the principal stresses due to tunnel excavation occurs around the opening except in two perpendicular sections: crown (θ=90°)/invert(θ=270°) and springlines (θ=0° and θ=180°). We will therefore consider only these sections (crown/invert, springlines) in the following discussion. In both sections the stress paths are in pure shear (Figure C-7): (1) pure shear in extension in the crown/invert, (2) pure shear in compression in the springlines. It is therefore recommended to run two undrained tests in pure shear, one in compression and the other in extension in order to simulate the stress path followed in the field. These undrained tests are run in a triaxial apparatus as follows:

Pure shear compression: \[ \Delta \sigma_v = -2\Delta \sigma_h \quad \text{with} \quad \Delta \sigma_v > 0 \] (C-28)

Pure shear extension: \[ \Delta \sigma_v = -2\Delta \sigma_h \quad \text{with} \quad \Delta \sigma_v < 0 \] (C-29)

As a first step, it has been assumed that the initial in-situ state of stress is isotropic with \( \lambda_0=1.0 \). However, similar reasoning can be followed in order to develop solutions for an initial anisotropic state of stress characterized by \( \lambda_0=1.0 \).
Figure C-1  Stress Point Used in the "Stress Path" Method
(from Lambe, 1967)
Figure C-2  Effective Stress Path (ESP), Total Stress Path (TSP), and Path of Total Stress minus Pore Pressure (TSP-\(u_0\)) in a \(p-q\) Diagram
Figure C-3  Effective Stress Path (ESP), Total Stress Path (TSP), and Path of Total Stress minus Pore Pressure (TSP-\(u_0\)) in a \(\sigma_{oct}-q\) Diagram
Assumptions: \( \sigma_{rr,0}, \sigma_{ho}, \) and \( \sigma_{zz,0} \) are the principal stresses.

\( \sigma_{zz,0} \) is the intermediate principal stress

\[ \sigma_{ho} \leq \sigma_{zz,0} \leq \sigma_{rr,0} \] or \[ \sigma_{rr,0} \leq \sigma_{zz,0} \leq \sigma_{ho} \]

Definition: \( \alpha = \frac{R_0}{r} \)

Figure C-4 Definition of the Problem
Figure C-5  Mohr Circle. Definition of the Maximum Shear Stress, $q$
Figure C-6  Definition of Compression and Extension
Figure C-7  Compression and Extension around a Circular Opening
Figure C-8  Stress Paths in $\Delta \sigma_{oct}$-$\Delta q$ Diagram as a Function of $\alpha=R_d/r$ for all $\theta$-Values
Figure C-9  Rotation of the Principal Stresses Due to Change of State of Stress during the Excavation Process
$\delta$: angle of rotation of the principal stress with respect to the initial vertical stress (assumed to be the initial principal stress)

Figure C-10 Stress Paths with Representation of the Rotation of the Principal Stresses
APPENDIX D

DESCRIPTION AND EXPERIMENTAL EVALUATION OF

THE NEW TRIAXIAL SYSTEM

This appendix presents the new triaxial system, a computer-controlled large triaxial apparatus, which has been used to run most of the tests reported in this study (Table D-1). In the first section, a thorough description of each component of the new triaxial system is given. Results of calibration tests for each component of interest are reported. At the end of the section the most important components and their manufacturers are listed.

The second section describes typical experiments run with the new triaxial system. Each experiment performed is described in detail, the results are analyzed and practical consequences for the interpretation of the results are discussed.

D.1 DESCRIPTION

The new triaxial system consists of: (1) a large triaxial apparatus (2) two loading devices: one regulating the confining pressure, the other controlling the axial load, (3) a pore pressure apparatus and, (4) a system control unit: a personal computer and a data acquisition system. A schematic of the new triaxial system is shown in Figure D-1.

D.1.1 LARGE TRIAXIAL APPARATUS

The large triaxial apparatus consists of: (1) a large cell, (2) an axial displacement measuring device, (3) a diameter change measuring device, and (4) a temperature control unit.
D.1.1.1 Large cell

The cell (designed by K.A. Soon (1987) for concrete testing and fabricated in-house) was transformed in order to suit the new type of experiments to be performed. The cell was therefore partially redesigned for: (1) a new specimen size, (2) saturation of specimens, (3) measurement of the pore water pressure, (4) measurement of the volume change (increase or decrease) under undrained or drained conditions. Since the cell had to be partially redesigned and was intended to serve more than one purpose, it does not fulfill the optimum requirements for every type of measurement. Nevertheless, due to its great versatility, it is a very useful device for the type of applications described above.

As illustrated in Figure D-2, this cell consists of a 5 cm thick aluminum base with a metal frame attached, a 25 cm outer diameter hollow steel cylinder with a 1 cm thick wall and a 4 cm thick aluminum closure plate (Figure D-2). The metal frame consists of four 2.5 cm diameter tie-rods and a 6.5 cm thick aluminum top plate. It is attached to the aluminum base with nuts, which are tightened with a torque-meter. The torque applied equally on each of the four nuts is on the order of 200 Nm. The cell is assembled by sliding the hollow steel cylinder along the metal frame into position on the base, then by fastening the aluminum closure plate onto the aluminum top plate using eight 1 cm socket-head cap screws. An axial loading piston of 5 cm diameter inserted through the top opening completes the assembly. O-ring seals are provided to prevent any pressure leakage from the cell. A dual purpose safety valve located at the top of the cell acts as a vent hole as well as a pressure relief valve. It aids in the filling and draining of the cell fluid, and also provides relief from excessive build-up of pressure in the cell.

The cell was designed for a pressure of 14.0 MPa. It has been proof-tested by K.A. Soon (1987) up to 11.0 MPa. In our investigation, the maximum confining pressure used is 10.0 MPa. The dual purpose safety valve discharges excessive internal pressure into the atmosphere when the pressure in the cell chamber exceeds 12.5 MPa. The weakest links of the cell are at the threaded ends of the four tie-rods. These locations represent regions of high stress concentrations, and substantial
elongations of the tie-rods are expected under excessively high pressure.

Inside the cell, we find the set-up for the rock specimen \{22\} \(^1\) and all the measuring devices which have been placed around it as shown in Figure D-3. In this subsection, only the set-up is discussed. The description of the measuring devices can be found in the following subsections.

The rock specimen \{22\} 84 mm in diameter and 170 mm high is placed on a removable bottom cap \{19\} screwed to a fixed pedestal \{18\}, which is in turn attached to the aluminum base \{12\}. The axial force applied by the loading piston \{28\} is measured by an internal submerged load cell \{27\} and then transmitted to the rock specimen through the loading cap \{25\} and the top cap \{24\} (Section D.1.2.2). The loading cap is not removable from the cell, but the top cap is. All the parts described (pedestal, bottom cap, top cap and loading cap) are made of stainless steel because they supply water to the specimen. Note that the bottom cap, the top cap, and the loading cap all have the same diameter as the specimen. All the surfaces of those parts which are acting as seals are highly polished. O-ring seals are provided at the interfaces loading cap--top cap and pedestal--bottom cap in order to prevent any water pressure leakage and oil penetration into the pore pressure apparatus or into the rock specimen. The pedestal also provides support for the microstepping motor \{10\} which drives the measuring device monitoring the diameter change \{21\} of the rock specimen.

Electrical signals from inside the cell are transmitted to the outside through four electrical feedthroughs \{13\} installed at the base of the cell.

\(^1\) The numbers in brackets\{\} refer to the parts illustrated in Figure C-3.
D.1.1.2 Axial displacement measuring device

Description

Axial (or vertical) displacements are monitored with a DCDT (Direct Current Displacement Transducer) (Hewlett-Packard, model 7DCDT-500). This measuring device is placed at the outside of the cell, parallel to the loading piston (Figure D-4). It actually measures the axial relative displacement of the loading piston with respect to the aluminum closure plate.

The axial displacement, $\delta_a$(cm), is computed from the relationship:

$$\delta_a = CF \times \Delta V$$  \hspace{1cm} (D-1)

where:  
- $CF$: calibration factor of the DCDT (cm/V) \(^3\),
- $\Delta V$: voltage change with respect to the initial value (V).

The axial strain, $\varepsilon_a(\%)$, is computed as follows:

$$\varepsilon_a = 100 \times \frac{\delta_a}{h_0}$$  \hspace{1cm} (D-2)

where:  
- $h_0$ is the initial height of the specimen after saturation (cm).

Calibration

The calibration factor of the DCDT is $CF = 0.377592$ (cm/V), thus Equation D-1 yields:

$$\delta_a = 0.377592 \times \Delta V$$  \hspace{1cm} (D-3)

\(^2\) Another method for measuring the axial deformation is presented in Section C.1.1.3 under the heading "Special Application".

\(^3\) The DCDT was calibrated for 5.5 VDC. Any fluctuation of the input voltage during the test is taken into account during the data reduction operation, so that $\delta_a$ is a corrected value for a constant 5.5 VDC input voltage.
D.1.1.3 **Diametric deformation measuring device**

**Description**

The measuring device for the diameter change consists of: (1) two proximity sensors \(4\), (2) a moving probe support \(21\), (3) a leadscrew \(5\) with ball nut and support \(8\), and (4) a computer-controlled microstepping motor \(10\). Each proximity sensor consists of a proximity probe \(6\) and a probe driver (an oscillator/demodulator). Figures D-3 and D-6 present views and details of this measuring device, while a general schematic set-up is shown in Figure D-5.

Two diametrically opposite proximity probes (Indikon, model SP-I AP-1351) measure the diameter change of the rock specimen (Figure D-6). Steel targets having the same curvature as the specimens are directly attached to the surface of the specimen. The proximity probe consists of an eddy-current probe (Figure D-7). It has a tip containing an inductance element which is excited by a high frequency current from its associated oscillator/demodulator. The resulting alternating current magnetic field at the probe tip induces eddy-currents in the steel target. These eddy-currents in turn change the impedance of the probe, thereby changing the voltage across it. Since the magnitude of the eddy-current interaction is dependent upon the distance between the probe and the steel target opposite it, the voltage across the probe varies with the distance "probe-target".

The proximity probes are mounted on a moving support built from plexiglas (Figures D-3 and D-6). They are insensitive to changes in confining pressure and can thus be placed inside the cell. Two coaxial cables (one per probe, and each with 1 signal wire and 1 common wire), connect the probes to their associated probe driver. Each cable forms a flexible spiral around the specimen and the pedestal down to the base of the cell. Their length (about 60 cm) is adjusted to fit the requirements imposed by the geometry of the set-up. The signals from inside to outside the cell chamber are transmitted through two electrical feedthroughs (EnviroCon, model LSG-6-BG) installed at the base of the

\(^4\) The numbers in brackets\{\} refer to the parts illustrated in Figure C-3.
cell. In order to reduce the noise in the signal and the possible changes in the capacitance of the electrical system, the probe drivers are fixed to the bottom plate of the loading frame. The demodulator of the probe driver transforms the high frequency current into a direct current which is linearly proportional to the distance "probe-target" between the proximity probe tip and the steel target.

The moving support (21), which is attached to a ball nut, is driven by a leadscrew (NSK, model W1003-10P-C3Z4) as illustrated in Figure D-3. This leadscrew is held vertically. It is rigidly fixed at both ends by a ball screw support (8) at the bottom and a ball bearing at the top. The ball screw support (8) (NSK, model WBK-10-01) is screwed into the pedestal (18). The ball bearing is inserted in a stainless steel ring which forms, with the 3 vertical shafts screwed in the pedestal, a very rigid frame (26). The 3 shafts (Thomson, solid 60 case hardened and ground shaft, class L) are made of stainless steel and are hardened.

The schematic in Figure D-5 illustrates the components of the diametric deformation measuring device. The microstepping motor (Compumotor, model NEMA 17) is computer-controlled. Commands are sent from the personal computer (IBM, model PC-XT) indexer card (Compumotor, model PC-21) to the drive. These commands are transformed into electrical pulses which are transmitted to the microstepping motor. To each pulse corresponds a microstep. The vertical displacement of the moving support is directly proportional to the rotation of the shaft of the microstepping motor \(^5\). The leadscrew is connected to the shaft of the microstepping motor by a flexible coupling (Helical, model 6005-12-6). Figure D-6 illustrates a detailed view of the moving support for the proximity probes. The lateral arm which connects the ball nut to one of the 3 vertical shafts of the frame prevents the moving support from rotating in the horizontal plane \(^6\). The sliding connection between the

---

5 In the present configuration, each microstep corresponds to a 0.16 \( \mu \text{m} \) vertical displacement of the proximity probes.
Microstepping motor: 25000 steps/turn
Leadscrew: lead = 4 mm; i.e. 4 mm/turn = 4000\( \mu \text{m} / \text{turn} \)
Vertical increment per microstep: 4000 \( \mu \text{m} / 25000 \text{ steps} = 0.16 \mu \text{m/step} \).

6 This arm consists of two separate aligned pieces which are free when moving in the axial direction but are rigid in the lateral direction.
arm and the vertical shaft is built of two ball bushing bearings (Thomson, type XA-61014). Two limit switches have been mounted at each end of the leadscrew in order to prevent overrun. When the switches are activated, the microstepping motor is stopped instantaneously.

The proximity probes can scan the full height of the rock specimen, plus 20 mm at each end of the specimen. A scan record of one side of the rock specimen can be seen in Figure D-8. When the proximity probe is located outside of the target, it essentially "reads" +13 VDC. When the proximity probe moves step by step directly opposite to the target, the voltage response drops from about +13 VDC, reaches a minimum and then increases to about +13 VDC. The voltage response describes a parabolic curve as a function of the path followed. The extreme values of +13 VDC on the parabolic curve correspond to the edges of the target, whereas the minimum value is reached at the center of the target. This minimum value is of practical importance because it gives a representative value of the distance between the probe and the target, or in other words the distance between the probe and the rock specimen. It will be noted that a control command flag has been installed in the program which drives the microstepping motor in order to ensure synchronization between the positioning of the proximity probes and the recording of the voltage response across the probes. This is done to prevent the proximity probes from moving when data recording of the voltage response occurs. Thus the voltage response is free from all vibrations which can occur during the positioning of the proximity probes.

The true diameter change of the rock specimen, $\delta_d$, is equal to the sum of the changes in distance between the proximity probes and the boundary of the rock specimen. It is computed from the relationship:

$$\delta_d = CF_1 \Delta V_1 + CF_2 \Delta V_2 + \delta_m \text{(CP)}$$

(D-4)

where: $\delta_d$: diameter change ($\mu$m),

$CF_1$, $CF_2$: calibration factors of the proximity sensors ($\mu$m/V) \(^7\),

(Figure D-9),

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\(^7\) Each proximity sensor consists of a proximity probe and a corresponding probe driver which should not be interchanged.
ΔV₁, ΔV₂: voltage changes with respect to the initial reading value across each proximity probe (V),

δₘ₃(CP): deformation of the moving support due to changes in the confining pressure, CP(μm) ⁸.

The diametric strain, ξ₅(%), is computed as follows:

$$ \xi₅ = 100 \times \delta₅/d₅ $$  (D-5)

where d₅ is the initial diameter of the specimen after saturation (μm).

Calibration

The proximity probes have been calibrated while already mounted in the cell; i.e. adjusted cable lengths, electrical connections and feedthroughs are taken into account in the calibration curves. The relationship between the voltage and the distance "probe-target" is linear in the range of 0 to 6.5 mm (Figure D-9). The characteristics of the proximity sensors, provided by the manufacturer, are as follows:

<table>
<thead>
<tr>
<th>Linearity</th>
<th>±18 μm → 0.3% of full scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Repeatability</td>
<td>±1 μm</td>
</tr>
<tr>
<td>Hysteresis</td>
<td>±1 μm</td>
</tr>
<tr>
<td>Long term stability</td>
<td>±6 μm → 0.1% of full scale</td>
</tr>
<tr>
<td>Temperature stability</td>
<td>±0.5 μm/°C</td>
</tr>
</tbody>
</table>

Changes in confining pressure produce a change in the shape of the moving support which holds the proximity probes. The voltage response across the proximity probes is therefore biased and should be corrected by means of the term δₘ₃(CP) in Equation D-4. An experiment was carried out in order to evaluate the correction term, δₘ₃(CP). The results, which are presented in Section D.2.1.2, show that δₘ₃(CP) is a linear function of the confining pressure, CP. The correction curve, δₘ₃(CP) (Equation D-18 in Section D.2.1.2) is accurate within ±10 μm for any

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⁸ Changes in the cell pressure produce a change in the shape of the moving support which holds the proximity probes. The voltage response across the proximity probes is therefore biased and should be corrected. This is discussed in Section D.2.1.
confining pressure, taking into account non-linearity, repeatability and hysteresis of the proximity probes and of the moving support.

For the proximity sensors used, the true diameter change \( \delta_d \) (\( \mu m \)), given in Equation (D-4) is:

\[
\delta_d = 514.04785x\Delta V_1 + 514.74450x\Delta V_2 - (10+10.25xCP)
\]  \hspace{1cm} (D-6)

where: \( \Delta V_1, \Delta V_2 \): voltage changes with respect to the initial reading value across each proximity probe (V),

\( CP \): confining pressure (MPa).

The true diameter change is accurate within \( \pm 10+2\times(\pm 6) = \pm 22 \, \mu m \) \(^9\) for any confining pressure, taking into account the characteristics of both the proximity sensors and the moving support; i.e. non-linearity, repeatability, hysteresis, and long term stability when using Equation D-6. The diametric strain, \( \varepsilon_d \), may be expected to be measured with an accuracy within \( \pm 0.026\% \). This experimental result is judged to be exceptionally good. The set-up is thus very well suited to perform \( K_0 \)-tests for example, where great accuracy in monitoring the diameter change is required.

**Special application**

The measuring device for diameter change can also be used for recording the axial displacement of the specimen in the following way. Figure D-8a illustrates a scan record of the rock specimen on which five targets have been attached diametrically on the specimen surface. Figure D-8b presents a detailed view of the voltage change as a function of the vertical position (in microsteps) when one of the proximity probe moves along a particular target. It can be seen that when the proximity probe is located outside the target, it essentially "reads" +13 VDC. When the

---

\(^9\) The non-linearity, repeatability and hysteresis of the proximity sensors is taken into account in the true correction curve of the moving support, \( \delta_{true}(CP) \), given by Equation D-18 in Section D.2.1.2. The value of \( 2\times(\pm 6) \) accounts for long term stability of the proximity sensors, which cannot be measured during the short period of time that the calibration test lasts, but which is given by the manufacturer (Indikon).
proximity probe moves step by step opposite to the target, the voltage response drops from +13 VDC, reaches a minimum and then increases to +13 VDC. The voltage response describes a parabolic curve as a function of the path followed. The extreme values of +13 VDC (Points A & B on Figure D-8b) on the parabolic curve correspond to the edges of the target. These edges are very clearly defined as shown in Figure D-8b. Records of the target edges as a function of the vertical position (in microsteps) of the proximity probes allow one to monitor the distance in-between the successive targets and therefore to make estimates of the axial deformation at a particular location of the rock specimen. In the present configuration of the microstepping motor, a vertical displacement of 0.16 μm corresponds to one microstep. The axial strain between each target can therefore be easily obtained as:

\[ e_{a,i}(\%) = 100 \times \frac{N_i + \Delta N_i}{N_i} \]  \hspace{1cm} (D-7)

where \( e_{a,i}(\%) \) is the axial strain between the \( i^{th} \) - and the \( (i+1)^{th} \) - targets, and \( N_i \) and \( \Delta N_i \) are, respectively, the initial number of microsteps between the \( i^{th} \) - and the \( (i+1)^{th} \) - targets and the increment (positive or negative) of microsteps between the \( i^{th} \) - and the \( (i+1)^{th} \) - targets. The accuracy obtained can be evaluated to two microsteps out of approximately 70000 (which is about the initial number of microsteps between any \( i^{th} \) - and any \( (i+1)^{th} \) - targets, \( N_i \)); i.e. 0.003%.

Because of time considerations regarding the completion of this thesis, it has not been possible to use this measuring system in drained tests, but it has been tested with a wooden dummy and gave full satisfaction.

D.1.1.4 Temperature control unit

Description

Temperature variations produce dramatic changes in pore pressure during undrained tests (Lambe and Whitman, 1965). The entire triaxial apparatus is enclosed in a box made of polystyrene in which the temperature can be kept constant. During the tests, temperature is
recorded inside the triaxial chamber with a thermistor to ensure that constant temperature requirements are met.

It will be noted that the microstepping motor, which is submerged in silicone oil, does not produce any measurable temperature changes inside the cell chamber for two reasons:

- the microstepping motor is moving at very low speed, therefore losses due to friction are very small,

- the microstepping motor is disconnected during the time between two sets of measurements, so that no additional heating caused by the motor can take place.

**Calibration**

Inside the polystyrene box, the temperature can be kept constant within ±0.2°C, whereas inside the cell chamber, the temperature remains constant within ±0.1°C (Figure D-10).

**D.1.2 LOADING DEVICES**

There are two separate and completely independent loading devices, namely a confining pressure device, and an axial loading device. These loading devices control the state of stress applied to the rock specimen.

**D.1.2.1 Confining pressure device**

The confining pressure is applied to the specimen hydraulically with silicone oil (Dow Corning, 200 Fluid 20 cst). This clear, light yellowish hydraulic oil is electrically non-conducting so as to prevent interference with the electrical devices (microstepping motor, proximity sensors, thermistor and load cell) placed inside the cell chamber. The very low vapor pressure (0.015 mmHg) makes this fluid very interesting for long duration application at high pressure, because the quantity of permeant (air, water) driven through the membranes which enclose the
rock specimen is nil (Section D.2.2.2). Finally, the low viscosity of 20 centistokes of this hydraulic oil has two advantages: (1) it helps speed up the filling and draining of the cell in each test (typically one needs 20 minutes for filling and 15 minutes for draining), and (2) it allows the support which holds the two proximity probes for monitoring the diameter change to move at a reasonable speed of 0.10 mm/sec, even under pressures as high as 10.0 MPa, without generating problematic oscillations or vibrations.

Description

The confining pressure device consists of: (1) a computer-controlled piston, (2) a gear box, (3) a leadscrew, and (4) a pressure transducer. The general schematic set-up is presented in Figure D-11. This apparatus works in the following manner:

The required confining pressure, which is set by the operator (generally a statement command in a control program run on the personal computer), is transformed into a continuous analog signal through a control board (Metrabyte, model DAC-02) inserted in one of the slots in the back of the personal computer. This signal is a reference control signal for the proportional controller and can be set between -10 and +10 VDC.

The pressure transducer monitors the actual confining pressure in the cell chamber. Its output signal ranges between 0 and 150 mV for pressures ranging from about 0 to 10 MPa. The signal is consecutively amplified by a factor 50 and inverted, so that the signal then ranges between 0 and -7.50 VDC. In the proportional controller, the reference control signal from the computer and the amplified inverted signal from the pressure transducer are added. Their sum should be equal to zero if the apparatus is in equilibrium; i.e. if the pressure monitored is the one required by the operator. If this is not the case, the proportional controller sends a current to the DC-motor, which moves in response. The clockwise or counterclockwise rotational motion is transformed into a linear motion of the piston through a gear box and a leadscrew. The movement of the piston (Parker, model C2HLTS24, serial# 152692) produces
an increase or a decrease of the confining pressure until the required pressure is reached. Two limit-switches have been mounted at each end of the piston stroke in order to prevent overrunning the end. When the switches are activated, the DC-motor is turned off.

The control board consists of two separate double-buffered 12 bit multiplying D/A (digital/analog) channels plus interface circuitry. The D/A channel converters have been used with a fixed ±10 VDC reference. The positive range (0 to +10 VDC) has been reserved to control the pressure while the negative range (-10 to 0 VDC) is used to generate signals to drive the piston "back" for refilling or emptying the piston chamber. Since data is 12 bits, data is written to each D/A in 2 consecutive bytes. The first byte is the least significant and contains the 4 least significant bits of data. The second byte is the most significant and contains the most significant 8 bits data. The least significant byte is usually written first and stored in an intermediate register in the D/A, having no effect on the output. When the most significant byte is written, its data is added to the stored least significant data and presented "broadside" to the D/A converter, thus assuring a single step update. This process is known as double buffering. Due to the analog inversion in the bipolar output ranges (±10 VDC), data coding is complementary offset binary; i.e. zero digital corresponds to -10 VDC and 4095 digital corresponds to +10 VDC.

Calibration

After initial calibration of the control board (Metrabyte, model DAC-02), and for the pressure transducer (Data Instruments, AB 1000 Series, #C47257) used, the following relationship has been found between the number of bytes \( n \) to be sent to the control board and the confining pressure, \( CP \), monitored by the pressure transducer: 11

---

10 The volume of the piston chamber is small (100 cm\(^3\)) and refilling or emptying of the chamber is needed during the test. This operation occurs when large pressure changes are applied, primarily because of the compliance observed in the line connecting the piston chamber to the cell chamber.

11 If another pressure transducer is used, one should recalibrate.
\[ \#b = \text{Int}(2045 + 162.504062 \times \text{CP}) \]  

(D-8)

where: \( \#b \) must be an integer, \( \text{Int} \); and \( \text{CP} \) is in (MPa).

Note that in order to avoid damage to the pressure transducer and/or to the triaxial apparatus, the control board should not receive a value of \( \#b \) exceeding \( \leq 700 \) (CP=10.2 MPa).

Table D-2 presents experimental results obtained with the confining pressure device. Comparison of the pressure required by the operator and the actual pressure monitored by the pressure transducer shows excellent agreement (Coefficient of correlation of the linear regression \( \rho = 0.999996 \)).

It has been found that hysteresis and repeatability were dependent on the operation sequence ("loading" = increase in pressure or "unloading" = decrease in pressure) and on the prevailing temperature in the laboratory. An accuracy within 0.003 MPa (±1/2 byte) can nevertheless be achieved easily in the following manner: at any time, the operator can enter the zero-value of \( \#b \) of Equation D-8; i.e. a value of approximately 2045 in order to get a monitored pressure closer to the one required when the system is at equilibrium. For example, it has been found that a value of 2046 or 2047 is well suited for "loading" operations, while a value of 2043 or 2044 is more appropriate for "unloading" operations.

D.1.2.2 Axial loading device

The large cell is placed on a fixed frame primarily because drained tests can last up to 5 weeks and thus do not allow one to use a conventional loading frame, such as, for example, a MTS hydraulically powered press.

The frame consists of: (1) four vertical 2.5 cm O.D. threaded steel rods, (2) a 5 cm thick aluminum bottom plate, (3) a 5 cm thick aluminum top plate (Figure D-12). The base of the cell rests on the bottom plate. All the water lines run through the base of the cell and the bottom plate. Because of this arrangement the cell is connected to the bottom
plate and therefore to the frame. The top plate is removable and must be taken away in order to open the cell chamber. The position of both aluminum plates can be adjusted at will along the vertical rods.

Description

The axial loading device is attached to the frame in order to provide an axial stress independent of the confining pressure. The axial loading device consists of: (1) a computer-controlled air regulator, (2) a double-acting cylinder, (3) a loading piston with a linear alignment coupler, (4) an internal submersible load cell and (5) a loading cap and a top cap. The general schematic set-up is shown in Figure D-13. The air pressure provided to the cylinder can be adjusted between 0 and 0.85 MPa by a computer-controlled air regulator. The air cylinder (Lin-Act, model# A2FH-8.0x5-N-2-N/R) is double-acting, meaning that it can be driven up and down, or if one prefers as in our case, it can be used and controlled in compression ("push") and in tension ("pull"). The piston of the air cylinder and the loading piston of the cell are connected by a linear alignment coupler (Lin-Act, part# LC-1-16) which can adjust for misalignments of up to 0.16 cm axial float and 1.0° in spherical movement. This device can reduce enormously, if not eliminate completely, any eccentric force in the loading piston of the cell, which would have the 3 following negative effects:

- bending of the loading piston,
- introduction of large friction forces between the loading piston and the aluminum closure top plate of the cell, and
- introduction of non-uniform stresses in the rock specimen.

The axial force is measured inside the cell chamber by a submersible load cell (Hydronics, model TH-LB1B, capacity 0-25000 lbs, serial# 029387) in order to obtain the actual axial load applied to the rock specimen, free from the unavoidable friction forces between the loading piston and the aluminum closure top plate of the cell.

The axial force is transmitted to the rock specimen through the loading cap and the top cap. The top cap has the same diameter as the
rock specimen; i.e. 84 mm. The loading cap is fixed, whereas the top cap is removable, in order to allow one to prepare the rock specimen outside of the triaxial cell (Section 4.5.3.1). The actual axial stress applied to the rock specimen consists of two components due to:

(1) the axial force measured by the load cell, and
(2) the confining pressure acting on the top of the loading cap (Figure D-14).

The actual axial stress, $\sigma_a$ (MPa), applied to the rock specimen is given by:

$$\sigma_a = \frac{(1000 \times LC)}{A_s} + CP$$  \hspace{1cm} (D-9)

where: $A_s$: cross-sectional area of the specimen, $A_s = 5518.0$ mm$^2$,
LC: axial force measured by the load cell, in KN,
CP: confining pressure, in MPa or in N/mm$^2$,

Numerically, Equation D-9 yields:

$$\sigma_a = \frac{LC}{5.518} + CP$$

or more conveniently for the subsequent developments:

$$LC = 5.518 \times (\sigma_a - CP)$$  \hspace{1cm} (D-10)

The axial force loading system works in the following manner.

The axial stress, $\sigma_a$, set by the operator (generally a statement command in a control program run on the personal computer), is converted into an axial force (Equation D-10)\textsuperscript{12}, and is then transformed into a continuous analog signal through a control board (Metrabyte, model DAC-02) inserted in one of the slots in the back of the personal computer. This signal is a reference control signal for the proportional controller and can be set between 0 and +10 VDC.

\textsuperscript{12} The confining pressure required by the operator, CP, is plugged into Equation D-10.
The load cell monitors the actual force acting in the loading piston. Its output signal ranges between -200 and +400 mV for loads of respectively about -10 to +30 KN. The signal is subsequently amplified by a factor 25 and inverted, meaning that the signal then ranges between -5.0 and +10.0 VDC. In the proportional controller these two signals, the reference control signal from the computer and the amplified inverted signal from the load cell, are added. Their sum should equal zero when the apparatus is in equilibrium; i.e. when the force monitored is the one required by the operator. If this is not the case, the proportional controller sends a current to the DC-motor which controls the air regulator. The clockwise or counterclockwise rotational motion produces an increase or a decrease of the air pressure until the pressure inside the cylinder will produce the required force. Two limit-switches have been mounted in order to prevent overrun. When the switches are activated the DC-motor is turned off.

The control consists of two separate double buffered 12 bit multiplying D/A (digital/analog) channels plus interface circuitry. The D/A channel converters have been used with a fixed 0 to +10 VDC reference. Since data are 12 bits, data are written to each D/A in two consecutive bytes. The first byte is the least significant and contains the four least significant bits of data. The second byte is the most significant and contains the most significant eight bits data. The least significant byte is usually written first and stored in an intermediate register in the D/A, having no effect on the output. When the most significant byte is written, its data is added to the stored least significant data and presented "broadside" to the D/A converter thus assuring a single step update. This process is known as double buffering. For unipolar outputs (0 to +10 VDC), data coding is true binary; i.e. zero digital corresponds to 0 VDC and 4095 digital corresponds to +10 VDC.

Calibration

The load cell (Hydronics, model TH-LB1B, capacity 0-25000 lbs, serial# 029387) has been calibrated in compression. The following characteristics of the load cell can be emphasized:
Linearity : ±0.03 mV → 0.10% of full scale
Repeatability : ±0.015 mV → 0.05% of full scale
Hysteresis : ±0.03 mV → 0.10% of full scale
Long term stability : ±0.015 mV → 0.05% of full scale
Calibration factor : 3.71470 KN/mV

The axial stress, \( \sigma_a \), due to the applied axial force only (no effect of the confining pressure) is accurate within ±0.013 MPa, taking into account non-linearity, repeatability and long term stability. It is important to report here that the load cell has been tested and found insensitive to confining pressure up to 10.0 MPa (Section D.2.3).

After initial calibration of the control board (Metrabyte, model DAC-02), the following relationship has been found for the load cell used between the number of bytes \( \#b \) to be send to the control board and the axial force, LC, monitored by the load cell: 13

\[
\#b = \text{Int}(360+146.983313\times LC)
\]

(D-11)

where: \( \#b \) must be an integer, Int; and LC is in KN

Note that in order to avoid damage to the load cell and/or to the large triaxial apparatus, a maximum value of \( \#b=4750 \) (LC=+30 KN) in compression and \( \#b=1850 \) (LC=−10 KN) in extension should not be exceeded.

Table D-3 presents experimental results obtained with the axial loading device. Comparison can be made between the load required by the operator and the actual load monitored by the load cell. The quality of the results is excellent (Coefficient of correlation of the linear regression \( \rho=0.999998 \)).

It has been found that hysteresis and repeatability were dependent on the operational sequence ("loading" = increase in axial stress or "unloading" = decrease in axial stress) and on the temperature in the laboratory. An accuracy within 0.006 (±1/2 byte) can nevertheless be easily achieved in the following manner: at any time, the operator can

13 If another load cell is used, one should recalibrate.
enter the zero-value of \( b \) of Equation D-11; i.e. enter a value of approximately 360 in order to get a monitored pressure closer to the one required when the system is at equilibrium. For example, it has been found that a value of 361 or 362 is well suited for "loading" operations, while a value of 358 or 359 is more appropriate for "unloading" operations.

The axial stress, \( \sigma_a \), due to the applied axial force and the confining pressure is accurate within \( \pm 0.019 \) MPa. This maximum value is reached when the confining pressure is null. The minimum value of inaccuracy is reached when the axial force is null.

D.1.3  PORE PRESSURE APPARATUS

D.1.3.1 Description

During the saturation and swelling processes, water must be provided to the specimen. Figure D-3 presents a view of the lines supplying water to the specimen. The pore pressure apparatus consists of: (1) an air regulator, (2) an air-water interface with volume change device, (3) a pressure transducer, and (4) a set of lines and valves. A schematic of the pore pressure apparatus is shown in Figure D-15.

Air is provided to the pore pressure apparatus from a main compressor under a constant pressure of 0.85 MPa. An air regulator (Fairchild, model 10), manually driven, controls the pressure. At the interface, the air pressure in the cylinder (Bellofram, model S9F) is transmitted to the water through a piston with a rolling diaphragm (Bellofram, class 4). There, the water pressure is 1.85 times the air pressure because of the change in area between the air cylinder and the piston with the rolling diaphragm. This means that the maximum water pressure which can be obtained in the pore pressure apparatus is 1.85\times 0.85 \text{ MPa}. A volume change device is part of the interface. It consists of a hollow stainless steel cylinder closed at one end, inside which the piston with the rolling diaphragm moves. Volume change of water of the specimen can be monitored during both saturation and swelling. The volume of water
flowing in and out of the specimen, \( \Delta V_y \), can be easily computed from the following relationship:

\[
\Delta V_y = A_{rd} \cdot \delta_{ap} = A_{rd} \cdot CF \cdot \Delta V
\]  

(D-12)

where: \( A_{rd} \): area of the rolling diaphragm \(^{14} \), \( A_{rd} = 29.290 \text{ cm}^2 \),

\( \delta_{ap} \): vertical displacement of the piston (cm), \( \delta_{ap} = CF \cdot \Delta V \),

\( CF = 0.609657 \text{ cm/V} \) \(^{15} \),

\( \Delta V \): voltage change with respect to the initial value (V).

At the downstream side of the volume change device, one can find the following elements (Figure 6-15): (1) a ball valve, (2) a pressure transducer, (3) two separate water lines to the specimen, one to the top and one to the bottom, (4) two separate water lines from the specimen, one from the top and one from the bottom, (5) one needle valve on each water line, and finally (6) a micrometer valve.

The ball valve (Withey, model SS-43S4) is used to disconnect the volume change device from the rest of the pore pressure apparatus. This valve is closed for the B-check at the end of saturation and during undrained tests. The pressure transducer (Data Instruments, AB 200 series, #C03199) measures continuously the water pressure of the specimen. There are no separate measurements of the water pressure at the top and at the bottom of the specimen. Both the bottom cap and the top cap have two drainage lines in order to achieve quick and complete saturation of the pore pressure apparatus. The saturation process is discussed further below. The needle valves NV1 and NV2 are needed during the first phase of the saturation process. Without them, it is practically impossible to saturate the pore pressure apparatus and, in particular, the drainage lines. The micrometer valve has been used to measure the compressibility of the pore pressure apparatus (Section

\(^{14} \) The area of the rolling diaphragm has been calibrated in the laboratory and found to be within 0.85% accuracy of the data given by the manufacturer (Bellofram).

\(^{15} \) The DCDT was calibrated for 5.5 VDC. Any fluctuation of the input voltage during the test is taken into account during the data reduction operation, so that \( \delta_{ap} \) is a corrected value for a constant 5.5 VDC input voltage.
D.2.4). Otherwise, it has no function except to back up the two needle valves, if needed.

All the dimensions of the elements used in the pore pressure apparatus are shown in Figure D-16. All elements which are in contact with the rock specimen or the water are made of stainless steel. All the surfaces of the parts, such as the bottom cap and the top cap which act as seals against the membranes, are highly polished. Sharp bends and intermediate connections in the water lines are reduced to a minimum in order to prevent the possibility of trapping air bubbles. All intermediate connections are silver-soldered. Connections between the loading cap and the top cap, and between the bottom cap and the pedestal, are sealed with O-rings.

At the early stage of the saturation process (specimen and pore water apparatus), the pore pressure apparatus must be completely filled with desired water first. This is achieved as follows:

1/ The ball valve is closed and the volume change device is filled with desired water;

2/ The rock specimen is confined dry, up to a given state of stress at which saturation starts (there is no water in the drainage lines);

3/ Vacuum is applied to the pore pressure apparatus from the outlet of the micrometer valve. A negative pressure of approximately 690 mm Hg (≈-0.9 atm) is applied for a few minutes;

4/ The ball valve is opened and water flushes the entire apparatus;

5/ Vacuum is applied until at least 100 cm$^3$ of water have run through the entire apparatus, more specifically, from the volume change device to the micrometer valve. During this operation, the volume change device needs to be refilled with desired water four or five times.

Thorough saturation of the apparatus can only be achieved if water is forced to run through either one of the lines to the top and to the
bottom of the specimen. This is done first by closing the valve NV1 and
opening the valve NV2 (Figure D-15). In this arrangement, no flow takes
place in the bottom cap and water is forced to flow through the top cap.
After a few minutes, one closes the valve NV2, and simultaneously, one
opens the valve NV1. No flow takes place in the top cap and water is
forced to flow through the bottom cap as shown in Figure D-15. This
operation is repeated four to five times during the saturation phase of
the pore water apparatus.

D.1.3.2 Calibration

When drained tests are performed, the volume of water which enters
the specimen must be recorded accurately. The calibration factor of the
DCDT (Hewlett-Packard, model 7DCDT-500) for the volume change is
CF=0.609657 cm/V. The volume change is given by Equation D-12, which
yields:

$$\Delta V_w = 17.855684x\Delta V$$  \hspace{1cm} (D-13)

where: $\Delta V_w$: volume change of water,
$\Delta V$: voltage change with respect to the initial value (V).

When undrained tests are performed, the pore pressure apparatus must
be completely leakless, and its compliance must be known. Two
calibration tests have been conducted with great care in order to
address these two important issues.

Leakage proof-test

The pore pressure apparatus is transformed into a rigid-confined
apparatus: the drainage lines of the top cap are interconnected by a
small piece of stainless steel tubing of 1/8 inch O.D., which has been
silver-soldered on the top cap as shown in Figure D-17. The same thing
is done for the bottom cap.

The pore pressure apparatus is saturated following the procedure
described previously. The water pressure is then increased up to 1.5
MPa. (maximum water pressure that is used in testing with specimens),
and the ball valve is closed. The pore pressure apparatus is completely closed, disconnected from any pressure or water supply. If any leakage occurs, the pressure drops immediately. It has been computed that a decrease of 0.07% of the volume of water inside the apparatus would cause the pressure to drop from 1.5 MPa to 0! The water pressure has been monitored under constant temperature conditions for more than one week; it remained absolutely constant.

**Apparatus compressibility**

Reliable measurement of the pore pressure is a very difficult task under undrained conditions (Bishop, 1973; Bishop, 1976; Mesri et al., 1976; Chiu et al., 1983). This so because the magnitude of the pore water pressure is dependent on the compressibility of the porous rock skeleton, on the pore water and on the solid material comprising the skeleton. It is also influenced by the porosity and possibly the intergranular contact area, as discussed in detail in Section D.2.4.1. Furthermore, pore pressure measurements can be strongly influenced by the compressibility of the device (Bishop, 1976). Thus, great care has been taken to build a pore pressure apparatus with low compressibility and to calibrate it precisely. The calibration test performed requires a detailed study, which is presented in Section D.2.4.

The results of this test show that the pore pressure response \( B = \Delta u / \gamma_{oct} \) of the rock specimen can be measured within 10% of its actual value. A plot of the observed \( B \)-value, \( B_{obs} \), versus the drained bulk modulus of the rock skeleton, \( K \), has been developed in order to assess the accuracy of the pore pressure response. This plot is given in Figure D-18. If the point representing the observed data \( (B_{obs}, K) \) falls in the unshaded area drawn in Figure D-18, then the observed \( B \)-value is measured within 10% of its true value. In any case, the observed \( B \)-value should be corrected to account for the compressibility of the pore pressure apparatus. This is done by applying the following formula
(Section D.2.4):

\[ B_{corr} = \frac{1}{\Omega} \left[ \frac{1}{B_{obs}} \right] \]  

(D-14)

where: \( \Omega = 2.16480 \times 10^{-5} / ((1/K) - 2.7 \times 10^{-5}) \): compressibility of the pore pressure apparatus,

\( K \): drained bulk modulus of the rock skeleton, in MPa.

D.1.4 SYSTEM CONTROL UNIT

The "heart" of the new triaxial system, called the system control unit, consists of two components, namely a personal computer (IBM, model PC-XT) and a data acquisition system (Fluke, model 2240-A). Its role is to control the triaxial apparatus and to collect the test data. A schematic of the system control unit integrated in the new triaxial system is shown in Figure D-1.

D.1.4.1 Personal computer

The personal computer used is a common personal computer (IBM, model PC-XT), the capabilities of which have been enhanced in order to control the large triaxial apparatus. The configuration of the personal computer is described hereafter and summarized in Table D-4.

Control boards and communication adapters have been plugged into the slots available for such purposes in the back of the computer. They are:

1. an indexer card (Compumotcr, model PC-21), which controls the vertical position of the proximity probes by rotation of the microstepping motor (Figure D-5),

2. a control card (Metrabyte, model DAC-02), which simultaneously controls the two loading apparatus: the confining pressure apparatus (Channel #0) (Figure D-11) and the axial loading apparatus (Channel #1) (Figure D-13),
(3) a parallel printer adapter (IBM, model PAC), which connects the personal computer to the switches mounted on the leadscrew to prevent overrun of the moving support (Figure D-5),

(4) an asynchronous communications adapter (IBM, model ACA) used as input port for data from the data acquisition system to the personal computer, and

(5) an asynchronous communications adapter (IBM, model ACA) used as output port for data from the personal computer to the printer (Digital, model Decwriter II).

A user-friendly program has been set up in order to run the tests. Interactive programs have been developed in Basic to control each operation. The tests are conducted by an operator, who enters commands which are either executed by the computer, or sent to the various control boards and communications adapters. The operator can intervene at any time, thus ensuring great flexibility in the testing procedure. A copy of the computer control software is provided in Appendix E. During a test, data are collected every two seconds and then directly printed in engineering units on the computer screen. Every 30 seconds, a data sample containing the monitored values of each device listed in Table D-5 is stored in a sequential file on a diskette and also printed out as a back-up copy on the hard disk of the computer. The test data are reduced and interpreted at the very end of the test.

D.1.4.2 Data acquisition system

The data acquisition system (Fluke, model 2240-A) collects all the necessary information to interpret the tests. It transforms the analog signals generated by the various measuring devices (DCDTs, pressure transducers, load cell, proximity sensors and input voltages) into digital signals, which can be read, sorted, and stored by the personal computer. Eleven channels are monitored; the channel allocation on the data acquisition system is reported in Table D-5.
D.1.5 LIST OF ITEMS AND MANUFACTURERS

The new triaxial system is a prototype which has been fabricated in-house, but with various components bought elsewhere. This section gives a list the most important items which were described before (Sections D.1.1 to D.1.4). The items are listed in alphabetic order. The appropriate model is given as well as the names and addresses of their respective manufacturers or distributors.

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<thead>
<tr>
<th>Item</th>
<th>Model</th>
<th>Name and Address</th>
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<td>Sprague Air Controls Hingham, MA</td>
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<td><strong>Air cylinder</strong> (volume change device)</td>
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<td><strong>Data acquisition system</strong></td>
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D.2 EXPERIMENTAL EVALUATION OF THE NEW TRIAXIAL SYSTEM

The required great versatility of the new triaxial system has imposed severe design constraints: e.g. measurement of small diameter change, measurement of precise axial stress over a large confining pressure range, measurement of the "true" pore pressure response and reduction of leakage rate through the membranes. All these points are critical in order to provide good quality test results, and great care has been devoted to finding the best solution.

Evaluation through experimental measurements is required in order to assess the reliability and the accuracy of the test results. This section presents in detail each of the critical problems and discusses the results of the experimental evaluation, together with the practical consequences on the interpretation of the test results on rock specimens.

D.2.1 INFLUENCE OF THE CONFINING PRESSURE ON THE DIAMETERIC DEFORMATION MEASURING DEVICE

The change in diameter of the rock specimen is measured with a system composed of proximity sensors mounted on a plexiglas support (Figure D-6). This system is located inside the cell chamber and thus subject to changes in the confining pressure. Due to changes in confining pressure, the support deforms, and consequently the measurement of the diameter change is altered. It is therefore very important to evaluate what part of the measurement is due to the deformation of the support and what part of the measurement is actually due to the diameter change of the rock specimen.

The purpose of this experiment is to evaluate the deformation of the support. The diameter change of a dummy specimen of known modulus of deformation is computed as a function of the confining pressure and then compared to the experimental results. This subsection describes the experiment carried out with a concrete specimen and discusses the
results and the practical consequences for the interpretation of the diameter change of the rock specimen.

D.2.1.1 Description of the experiment

A cylindrical concrete specimen 84 mm in diameter and 170 mm high was placed inside the cell chamber. The initial distances between probe and target, i.e. between the concrete specimen and the proximity probes, were monitored while the cell chamber was still empty. The cell chamber was then filled with a non-conductive silicone oil (Dow Corning, type 200 Fluid 20 cst) and pressurized. The relative distances "probe-target" were recorded at different confining pressures. In the experimental procedure used, three calibration cycles with complete loading and unloading were performed. The loading, respectively unloading, increments are 0.10 MPa in the range of confining pressures of 0 to 1.0 MPa, and then 0.40 MPa in the range of confining pressures of 1.0 to 10.0 MPa.

D.2.1.2 Discussion of the results

The results of the three calibration cycles are shown in Figure D-19. Of primary interest are the Figures D-19(c), D-19(f) and D-19(i), which present the sum of the changes in the "probe-target" distance of the probes 1 and 2 with the confining pressure. From these Figures, the following conclusions can be drawn:

- Submersion of the probes in silicone oil does not alter the zero reading values.

- The change in distance "probe-target" is almost linearly dependent on the confining pressure.

- Good repeatability in the measurements is achieved within ±10 μm in the three tests.

- There is a small hysteresis of ±5μm between the "loading" and the "unloading" curves.
It will be noted in Figure D-19 that the changes in distance "probe-target" of the probes 1 and 2 are not similar, neither in shape nor in magnitude. This is due to the asymmetric configuration of the moving support, illustrated in Figure D-6. The lateral arm which is attached to one of the three vertical shafts produces a change in stiffness of the moving support in the horizontal plane, resulting in a rotation of the proximity probes. Nevertheless, a unique calibration curve can be obtained from the results presented in Figures D-19(c), D-19(f) and D-19(i), as Figure D-20 demonstrates.

From Figure D-20 the mathematical description of this unique calibration curve can be given as follows:

$$\delta_{cc} = 10 + \left( \frac{(125-10)}{10} \right) x CP = 10 + 11.5 x CP \quad \text{(D-15)}$$

where $\delta_{cc}(\mu m)$ is the change of distance "probe-target" with the dummy, and CP(MPa) the confining pressure.

Equation D-15 gives the relative deformation of the moving support with respect to the concrete specimen. It does not take into account the diameter change of the concrete specimen, which also deforms upon changes in the confining pressure. The diameter change of the concrete specimen, $\delta_{cs}(\mu m)$, has been evaluated using the following formula based on linear elasticity:

$$\delta_{cs} = \frac{2D}{3K_C} x CP \quad \text{(D-16)}$$

where: $D$: diameter of the concrete specimen, $D=84 \text{ mm}=84000 \mu m$,
$K_C$: concrete bulk modulus, $K_C=45000 \text{ MPa}$,
$CP$: confining pressure, in MPa.

Numerically, one obtains:

$$\delta_{cs} = 1.244 x CP \quad \text{(D-17)}$$

Thus, the calibration term should be altered to get a true correction term of the moving support, $\delta_{ms}(\mu m)$, free from the deformation of the dummy concrete specimen (Figure D-20):
\[ \delta_{ms} = \delta_{cc} - \delta_{cs} \]
\[ \delta_{ms} = \delta_{ms}(CP) = 10 + (11.5 - 1.24) \times CP = -10 + 10.25 \times CP \] (D-18)

The true correction term, \( \delta_{ms}(CP) \) in Equation D-18, is accurate within \( \pm 10 \ \mu m \) for any confining pressure, CP(MPa), as shown in Figure D-21. This relationship takes into account non-linearity, repeatability and hysteresis of the measuring device.

D.2.1.3 Conclusions

For all practical purposes, the true correction term of the moving support, \( \delta_{ms}(CP) \), is given by Equation D-18. This correction term, which is directly dependent on the confining pressure, has been found to be accurate within \( \pm 10 \ \mu m \) for any confining pressure, as shown in Figure D-21. This relationship takes into account non-linearity, repeatability and hysteresis of the measuring device.

D.2.2. LATEX MEMBRANES

In the triaxial tests, two membranes surround the rock specimen and isolate it from the confining fluid of the cell chamber. Such a set-up may influence the test results, and in particular the following factors should be taken into account:

- the strength of the membranes, and
- the leakage through the membranes.

D.2.2.1 Corrections for strength of the membranes

During a triaxial test, substantial restraints might be imposed on the specimen by the two membranes enclosing it and corrections of the measured stresses applied to the specimen should generally be made. These corrections depend on: (1) the elastic properties of the membranes, (2) the initial geometric characteristics of the membranes with respect to those of the rock specimen, and finally (3) the behavior of the specimen at failure. In this section the measured properties of the membranes are presented. The corrections for the strength of the membranes are then discussed.
Membrane properties

The latex membranes used for the tests are commercially available. They are made of natural rubber and have an unstretched diameter of 77 mm and a thickness of 1.1 mm.

The extension modulus, $E^*$ (in N/mm), of the membranes has been measured following the procedure described by Bishop and Henckel (1957). It consists of suspending a circumferential strip 120 mm wide between two steel rods covered with talc and then measuring its extension for different loads (Figure D-22). Two cycles of loading-unloading are performed with strains up to 25%. Figure D-23 presents the experimental results. The extension modulus decreases with increasing strains; e.g. at very low strains, say 1%, the extension modulus is higher than at larger strains. Nevertheless, for all practical purposes, the linear force (per unit thickness)-strain curve can be approximated by a straight line for strains up to 16%. The slope of this line gives an average value for the tangent extension modulus, $E^*$. A linear regression analysis performed on the two loading-unloading cycles yields $E^*=1.212$ N/mm. This value agrees well with the published data for this type of rubber membrane (Figure D-24).

Membrane correction for initial confining stress

Both membranes must be stretched during the initial set-up in order to enclose the rock specimen, thus producing an initial lateral confining stress. This confining stress is a function of the extension modulus of the membranes, and of the initial diameters of the membranes and of the specimen. The initial confining stress caused by the membrane, $\sigma_{om}$ (MPa), can be calculated using the hoop stress formula:

$$\sigma_{om} = 2E^*\frac{(d_1-d_{im})}{d_1d_{im}}$$  \hspace{1cm} (D-19)

where: $E^*$ : extension modulus of the membranes, $E^*=1.212$ N/mm,
$d_1$ : diameter of the specimen at the end of saturation, before shearing, $d_0=84$ mm,
$d_{im}$ : initial (unstretched) diameter of the membranes, $d_{im}=77$ mm.
Numerically, Equation D-19 yields:

\[ \sigma_{om} = 2.6 \times 10^{-3} \text{ N/mm}^2 = 2.6 \times 10^{-3} \text{ MPa} \]  

(D-20)

Thus, for two membranes:

\[ \sigma_{om} = 5.3 \times 10^{-3} \text{ N/mm}^2 = 5.3 \times 10^{-3} \text{ MPa} \]  

(D-21)

**Conclusion**

The value of the initial confining stress for the two membranes, \( \sigma_{om} = 5.3 \times 10^{-3} \text{ MPa} \), is negligible for all practical purposes and **no correction will be made for the initial confining stress due to the stretching of the two membranes**.

**Membrane correction during shearing**

Three limiting cases of failure are observed in triaxial testing: (1) bulging failure (in compression), (2) failure on a single shear plane (in both compression and extension), and (3) necking failure (in extension). Bulging and necking failures occur for ductile failures, whereas failure on a shear plane is associated to a mechanism of brittle failure (Figure D-25). Membrane corrections should be made in accordance with the failure mechanism observed during the test. The membrane correction for ductile failures is generally accepted, whereas the membrane correction for the brittle failures is still controversial. Both cases are discussed hereafter.

**Ductile failure**

At zero cell pressure, the membrane, which is thick, is held firmly against the specimen and does not buckle. For this case, following LaRochelle et al. (1986), the correction for the strength of the membrane, \( \sigma_{1m} \), to be applied to the major stress, \( \sigma_1 \), should be
expressed as:

\[ \sigma_{1m} = \sigma_{cm} + (\pi d_i E^* \varepsilon_a)/a_c \]  \hspace{1cm} (D-22)

where: \( \sigma_{cm} \): initial confining stress
\( d_i \): diameter of the specimen at the end of saturation, before shearing, \( d_0 = 84 \text{ mm} \).
\( E^* \): extension modulus of the membrane, \( E^* = 1.212 \text{ N/mm} \).
\( \varepsilon_a \): axial strain in (%) (positive in compression, negative in extension),
\( a_c \): corrected cross-sectional area of the specimen at axial strain \( \varepsilon_a(\%) \).

The corrected cross-sectional area of the specimen for bulging and necking failures is given by (Figure D-25):

\[ a_c = a_1 \frac{(1+\Delta V/V_1)/(1-\varepsilon_a)}{(1+\varepsilon_{vol})/(1-\varepsilon_a)} \]  \hspace{1cm} (D-23)

where: \( a_1 \): cross-sectional area of the specimen at the end of saturation, before shearing, \( a_0 = \pi d_0^2/4 = 5542 \text{ mm}^2 \).
\( \Delta V \): volume change upon shearing,
\( V_1 \): volume of the specimen at the end of saturation, before shearing, \( V_1 = h_1 x a_1 \),
\( \varepsilon_{vol} \): volumetric strain upon shearing, \( \varepsilon_{vol} = \Delta V/V_1 \).

The second term of the righthand side of Equation D-22 becomes by using Equation D-23:

\[ X = \pi d_i E^* \varepsilon_a (1-\varepsilon_a)/a_1 (1+\varepsilon_{vol}) \]  \hspace{1cm} (D-24)

**Extreme** values for \( \varepsilon_a \) and \( \varepsilon_{vol} \), which can be obtained at failure in testing soils or very soft rocks, can be plugged into Equation D-24 in order to evaluate the order of magnitude for the maximum membrane correction required:

**Bulging** (for drained test in compression): \( \varepsilon_a = +10\% \)
\( \varepsilon_{vol} = -15\% \)
Numerically, Equation D-24 yields then: \( x=6.1 \times 10^{-3} \) MPa. Thus, for two membranes, \( x=12.2 \times 10^{-3} \) MPa.

**Necking** (for drained test in extension):  
\[ c_a = -5\% \]
\[ c_{vo1} = -10\% \]

Numerically, Equation D-24 yields: \( x=-3.3 \times 10^{-3} \) MPa. Thus, for two membranes, \( x=-6.7 \times 10^{-3} \) MPa.

With the numerical values obtained above, the membrane correction, \( q_{1m} \), for two membranes (Equation 7-22) is equal to:

**Bulging**: \( 5.3 \times 10^{-3} + 12.2 \times 10^{-3} = 17.5 \times 10^{-3} \) MPa  \((D-22a)\)

**Necking**: \( 5.3 \times 10^{-3} - 6.7 \times 10^{-3} = 1.4 \times 10^{-3} \) MPa  \((D-22b)\)

**Conclusions**

The maximum membrane corrections for two membranes for ductile failures are given by Equations D-22'a and D-22'b. For all practical purposes, these values are negligible. Therefore **no correction will be made for the major principal stress, \( q_1 \), due to the strength of the two membranes.**

**Brittle failure**

The membrane correction for brittle failures is derived from two sets of experimental results obtained with the testing arrangement shown in Figure D-26 (Symons,1967, LaRochelle et al.,1986). Although the experimental results seem coherent, the membrane correction has been found to be very large. This result is quite unlikely when compared to the correction obtained for the case of ductile failures. The reason for such discrepancies can be found by looking at the Figure D-26. We observe that:

- the shear plane is quite thick (\( \approx 3.2 \) mm), and
- the top cap is free to move horizontally and free to rotate at the point of contact between the piston and the steel ball.
These two observations call for the following remarks:

- The thickness of the shear plane is thought to have an influence on the results. This has unfortunately not been investigated.
- In our large triaxial apparatus, the top cap is rigidly attached to the loading piston. Thus, no horizontal displacement and no rotation are allowed.

It is thought that the membrane correction for brittle failures as proposed by LaRochelle et al. (1986) is inappropriate in our case. It has been decided, considering the very low values of the membrane correction for ductile failures, that no correction will be made for the strength of the two membranes for brittle failures.

Conclusions

It has been decided, based on the discussion above and considering the very low values of the membrane correction for ductile failures (Equations D-22'a and D-22'b), that no correction will be made for the strength of the two membranes for brittle failures.

D.2.2.2 Leakage through the membranes

An important problem, especially for long term tests, is leakage through the membranes. For 100% saturated specimens, leakage may result from the combined effects of the total (or hydraulic) pressure gradient and the mole fraction (or osmotic pressure) gradient between the pore pressure and the confining fluid of the cell chamber through the membranes. Depending on the chemical compositions of the pore fluid and of the confining fluid, and on the applied effective stress, inflow of the confining fluid into the specimen or outflow of pore water may occur. One possibility to limit or avoid flow of pore water towards the cell chamber and flow of confining fluid towards the specimen would consist of selecting a confining fluid in which the water would not be miscible and which would have molecules larger than the pores of the membranes. For testing 100% saturated specimens, the two most important requirements for the confining fluid are: (1) low solubility in water,
and (2) low permeability of the membranes to the confining fluid (Poulos, 1964).

Various types of confining fluids have been proposed to this end: liquid paraffin, glycerin (Poulos, 1964; Leroueil et al., 1986), castor oil (Poulos, 1964), transformer oil (Poulos, 1964), kerosene (Leroueil et al., 1986), silicon oil (Poulos, 1964; Leroueil et al., 1986), and mercury (Poulos, 1964; Mesri et al., 1976).

Leroueil et al. (1986) showed that silicon oil is ideally suited for use as a confining fluid for two reasons: (1) very small rates of water outflow from the specimen were observed (tests run with a confining pressure of 0.1 MPa) and (2) silicon oil does not seem to have any detrimental effect on the membranes. Moreover, for our purposes silicon oil is non-conductive which makes it possible to use commercially available electrical devices such as the load cell, proximity sensors, thermistor, etc, inside the cell chamber. The silicone oil used (Dow Corning, 200 Fluid 20 cst) has the advantage of being fully compatible with the rubber membranes used. Indeed, no observable detrimental effect has been observed on the membranes.

The choice of the membranes themselves is certainly an important factor in controlling the rate of flow. The common latex membranes used in this study are probably not of the highest quality in terms of impermeability (Poulos, 1964). Ramses prophylactic membranes tested by Poulos (1964) and Leroueil et al. (1986) seem to give much lower rate of leakage. While such membranes would appear preferable, they are available only in one size, suitable for tests on 3.75 cm (1.5 in) diameter specimens.

Leakage through membranes is not measured, but a very simple experiment has been carried out to see if a large amount of silicone oil (Dow Corning, 200 Fluid 20 cst) permeates through the membranes. An aluminum dummy 84 mm in diameter and 170 mm high completely surrounded by filter paper is installed in the cell. The top cap and the bottom cap are coated with a thin layer of silicon grease before the first membrane and the O-rings are installed. The outside of the first membrane is also
coated with a thin layer of silicon grease before installation of the second membrane and the O-rings. The confining cell pressure applied to the dummy was 10.0 MPa, and the experiment lasted one week. After one week, the specimen is dismantled very carefully. No trace of silicon oil was visible between the membranes, and silicon oil was not present on the filter paper which directly surrounded the dummy. For all practical purposes, the membranes were considered to be impervious to silicon oil.

An application of the formulae based on thermodynamics considerations (Poulos, 1964) has shown that the volume of silicone oil permeating through the membranes is negligible. With a pressure difference of 10 MPa across the membranes, it is found that the volume permeating is less than 5 mm³/month.

D.2.2.3 Conclusions

Both experimental observation and computations show that the amount of silicone oil permeating through the two membranes is negligible and can be ignored for all practical purposes.

D.2.3 INFLUENCE OF CONFINING PRESSURE ON THE ZERO OF THE LOAD CELL

D.2.3.1 Description of the experiment

A load cell (Hydronics, model TH-LP1B, capacity 25000 lbs, serial #029387) is placed inside the chamber of the triaxial cell in order to measure the actual vertical load applied by the axial piston on the rock specimen. An experiment is carried out to see if the load cell is sensitive to the confining pressure; i.e. to see if its "zero" is shifting when the confining pressure is increased or decreased.

D.2.3.2 Discussion of the results

The results for the three cycles are shown in Figure D-27. The following comments can be made:

- Submersion of the load cell in the oil does not affect the "zero" reading (start of loading cycle #1).
- There is a slight continuous shift of the zero towards tension side with increasing confining pressure. The average shift observed is on the order of -55 N for an increase of 10 MPa in confining pressure.

- The correction to the vertical stress applied to the rock specimen to account for the observed shift in the zero reading of the load cell due to confining pressure is:

\[ \sigma_{ac} = \left( \frac{\Delta L}{\Delta CP_{ave}} \right) \times CP \times \frac{1}{A_3} \]  

(D-25)

where: \( \sigma_{ac} \) : correction to the vertical stress,

\( \left( \frac{\Delta L}{\Delta CP_{ave}} \right) \) : average shift observed for 10 MPa = 10 N/mm²

confining pressure change,

\( A_3 \) : cross-sectional area of the specimen, \( A_3=5518 \text{ mm}^2 \),

\( \text{diameter}=84 \text{ mm} \),

\( CP \) : change in confining pressure (MPa).

Thus, since \( \left( \frac{\Delta L}{\Delta CP_{ave}} \right)_{ave} = (-55/10) = -5.5 \text{ mm}^2 \) (Figure D-27), Equation (D-25) yields:

\[ \sigma_{ac} = (-5.5) \times CP / 5518 \]

\[ \sigma_{ac} = -9.967 \times 10^{-4} \times CP \]

or, approximately,

\[ \sigma_{ac} = -10^{-3} \times CP \]  

(D-26)

Equation D-26 shows that an increase of 10 MPa in the confining pressure produces a decrease in the reading of the stress applied on the rock specimen of 0.01 MPa. By assuming that the load cell is insensitive to confining pressure changes, the error introduced in the vertical stress is equal to:

\[ \left| \frac{\sigma_{ac}}{CP} \right| = \left| 10^{-3} \right| = 0.1\% \]  

(D-27)
The order of magnitude of the error introduced is so small that it is considered to be negligible.

D.2.3.3 Conclusions

*For all practical purposes, the load cell is assumed to be insensitive to the confining pressure.* The error introduced in the vertical stress is negligible (Equation D-27). Therefore, no correction will be made for the very slight shift observed in the zero of the load cell due to changes in the confining pressure.

D.2.4 Apparatus Compressibility

In the literature, attention has been drawn for many years to the difficulty of obtaining true undrained pore pressure response $\Delta u/\Delta \sigma_{oct}$ in triaxial testing due to the compressibility of the apparatus measuring the pore pressure (Wissa, 1969; Bishop, 1973; Bishop, 1976). This is particularly important in rock under high effective stresses, where great care is required in order to get meaningful values of $\Delta u/\Delta \sigma_{oct}$ in the laboratory.

In this section the theoretical derivation of $\Delta u/\Delta \sigma_{oct}$ for a closed apparatus is briefly presented. The procedure followed here is similar to that presented by Bishop (1976) in his Technical Note entitled: "The Influence of System Compressibility on the Observed Pore-Pressure Response to an Undrained Change in Stress in Saturated Rock". The experimental procedure used to measure the apparatus compressibility is described, and the obtained data are compared with values found in the literature. Finally, the expected accuracy in the measurement of the undrained pore pressure response $\Delta u/\Delta \sigma_{oct}$ is discussed.

D.2.4.1 Derivation of $\Delta u/\Delta \sigma_{oct}$ with correction for apparatus compressibility

The derivation of $\Delta u/\Delta \sigma_{oct}$ is made for a closed pore pressure apparatus. The analysis in its simplest form requires the following assumptions (Bishop, 1976):
- the pores of the porous medium are interconnected,
- the solid material forming the skeleton of the porous medium is elastic and isotropic,
- the bulk behavior of an element of the skeleton when subjected to a change in boundary stress with zero change in pore pressure is that of an elastic isotropic material,
- the distribution of the pore space within the skeleton is statistically random,
- the fluid (water) filling the pore space is linearly compressible.

When the pore pressure response $\Delta u$ to a change in octahedral stress, $\Delta \sigma_{\text{oct}}$, is measured, the measured $\Delta u$ is also a function of the apparatus compressibility. Bishop (1976) has derived the expression for the pore pressure response for a closed apparatus (i.e. zero fluid flow across the boundaries of the rock specimen except into the measuring apparatus and no cavitation (if the change is negative)) to include terms of apparatus compressibility:

$$\frac{\Delta u}{\Delta \sigma_{\text{oct}}} = \frac{1}{1 + n \left[ \frac{C_v - C_s}{C - C_s} + \frac{V_1}{V} \frac{C_v}{C - C_s} + \frac{C_1 + C_m}{V(C - C_s)} \right]}$$ (D-28)

where: 
- $n$: porosity of the rock specimen,
- $C_v$: compressibility of the water,
- $C_s$: compressibility of the solid material of the skeleton,
- $C$: bulk compressibility of the rock skeleton (measured under drained conditions, with constant pore pressure, $u$)
- $C_1$: compressibility of the drainage lines,
- $C_m$: compressibility of the pore pressure measuring element,
- $V_1$: volume of the fluid in the water lines,
- $V$: volume of the rock specimen.

It will be noted that $C_1$ and $C_m$ are defined as the change in volume per unit change in pressure. Their units are $m^3/MPa$. 
Equation D-28 reduces to the expression obtained by Wissa (1969) when
C₈ is negligible compared to D.

If the apparatus compressibility were equal to zero, the observed
value of $\Delta u/\Delta \sigma_{oc}$ would be equal to the pore pressure parameter termed
B by Skempton (1954). Its theoretical value is, for the assumptions
stated above (Bishop, 1973 and Equation D-28):

$$B = \frac{1}{1 + n \left( \frac{C_v - C_s}{C - C_s} \right)} \quad (D-29)$$

One can therefore re-arrange Equation D-29 to provide an expression
for B for the case of an apparatus of finite compressibility:

$$B = B_{corr} = \frac{1}{\left[ \frac{1}{B_{obs}} \frac{V_1}{V} \left( \frac{C_v}{C - C_s} \right) \frac{C_v + C_s}{V(C - C_s)} \right]} \quad (D-30)$$

where:

- $B_{corr}$: "true" B-value of the rock specimen, corrected
  for the apparatus compressibility,
- $B_{obs} = \frac{\Delta u/\Delta \sigma}{\sigma}$: observed B-value given by the laboratory test.

The terms involving the apparatus compressibility in Equation D-30
are defined as $\Omega$ and can be arranged in the form:

$$\Omega = \left[ \frac{-C_v}{C - C_s} \right] \left[ \frac{V_1}{V} + \frac{C_v + C_s}{VC_v} \right]$$

or:

$$\Omega = \left[ \frac{-C_v}{C - C_s} \right] \frac{1}{V} \left[ V_1 + \frac{C_v + C_s}{C_v} \right] \quad (D-31)$$

The left-hand side of Equation D-31 can be interpreted as the product
of two terms in two different manners:
First interpretation

\[
\left\{ \begin{array}{c}
\frac{-C_v}{C-C_s} \frac{1}{V} \\
C-C_s
\end{array} \right\} \times \left\{ \begin{array}{c}
\frac{V_1 + \frac{C_t+C_s}{C_v}}{V}
\end{array} \right\}
\]

(D-32)

term 1  term 2

In Equation D-32 term 1 depends only on the characteristics of the rock specimen (\(C_v\) is known). The more compressible and the larger the rock specimen, the smaller term 1 of Equation D-32 is.

In Equation D-32 term 2 is only dependent upon the configuration of the apparatus (\(C_v\) is known). For a given configuration, term 2 of Equation D-32 is determined and becomes a characteristic of the apparatus.

Conclusion 1

In order to keep the effect of the apparatus compressibility as small as possible, one should keep term 2 of Equation D-32 as small as possible; i.e. (a) reduce the volume of water of the apparatus and (b) build an apparatus as stiff as possible.

Second interpretation

\[
\left\{ \begin{array}{c}
\frac{-C_v}{C-C_s} \\
C-C_s
\end{array} \right\} \times \left\{ \begin{array}{c}
\frac{V_1 + \frac{C_t+C_s}{C_v}}{V}
\end{array} \right\}
\]

(D-33)

term 1  term 2

In Equation D-33 term 1 depends only on the mechanical characteristics of the rock specimen (\(C_v\) is known). As \(C_v\) can be considered constant for our practical purposes, the more compressible the rock specimen, the smaller term 1 of Equation D-33 becomes.

In Equation D-33 term 2 depends on the configuration of the apparatus and on the volume of the rock specimen (\(C_v\) is known). For a given configuration of the apparatus, the larger the rock specimen, the smaller term 2 of Equation D-33.
Conclusion 2

In order to keep the effect of the apparatus compressibility as small as possible, one should test rock specimens which are as large as possible.

The considerations made previously have led to a design configuration for the pore pressure apparatus which will minimize term 2 of Equation D-33. The adopted solution is illustrated in Figure D-16.

D.2.4.2 Measurement procedure and estimation of the apparatus compressibility

The compressibility of the pore pressure apparatus was measured experimentally. This was done by creating an isolated and closed system. First, both extremities of the system were closed: closing off the ball valve (BV) at the upstream side, and by placing a closure-nut at the outlet of the micrometer valve (MMV) on the downstream side. Second, the two water lines of the top cap and the bottom cap were connected, in two different manners in order to create: (1) a rigid-confined apparatus and (2) a flexible-confined apparatus.

In the rigid-confined apparatus, the two drainage lines of the top were interconnected by a small piece of stainless steel tubing of 1/8" O.D., which is silver-soldered on the top cap as shown in Figure D-17. The same was done for the bottom cap. In the flexible-confined apparatus, the set-up was identical to that used with rock specimens (three layers of stainless steel foil, two drainage layers at both ends of the specimen, two rubber membranes), but the rock specimen was replaced by a dummy in aluminum of the same size. As the rigidity of the dummy was about six times greater than the rigidity of any rock specimen tested in this study (K_{aluminum}=5x10^4 MPa; K_{rock}=8x10^3 MPa), it was assumed that the apparatus compressibility was not affected by the compressibility of the dummy.

For both the rigid- and the flexible-confined apparatus, compressibility measurements were carried out by measuring the increase or decrease of the water pressure in the closed apparatus when the small
pin of the micrometer valve was driven entirely in or out. The volume inserted was known to a great accuracy \( V_{\text{pin}} = 1.96 \, \text{mm}^3 \). The following procedure was used for both rigid- and flexible-confined apparatus:

1/ Fill the drainage lines with desired water. Then wait until full saturation of the apparatus is achieved (typically saturation is reached overnight),

2/ Close the outlet of the micrometer valve with a closure-nut. The apparatus is closed on the downstream side,

3/ Open the micrometer valve (MMV),

4/ Open both needle valves (NV1 and NV2),

5/ Close the ball valve (BV). The apparatus is closed on the upstream side,

6/ Record the initial water pressure,

7/ Drive the pin of the micrometer valve into the confined apparatus (7 turns),

8/ Record the increase in water pressure,

9/ Drive the pin of the micrometer valve out of the confined apparatus (7 turns),

10/ Record the decrease in water pressure,

11/ Repeat steps 7/ to 10/ five times in order to obtain an average value of the water pressure change,

12/ Open the ball valve (BV),

13/ Increase or decrease the initial water pressure,

14/ Proceed to the next set of measurements by repeating steps 5/ to 13/.

For both rigid- and flexible-confined apparatus, two different compressibility tests were conducted from step 1/ to step 14/ in order to check the repeatability of the data obtained for each test. The data of each test are reported in Figure D-28. It will be noted that for all the compressibility tests performed, the volume of the pin driven into or out the closed rigid- or flexible-confined system is the same \( V_{\text{pin}} = 1.96 \, \text{mm}^3 \) so that the curves obtained in Figure D-28 are directly comparable.
From Figure D-28 it is observed that:

- The larger changes in water pressure are obtained for the rigid-confined apparatus (as expected, the curves of the rigid-confined system lie above the curves of the flexible-confined system). The curves of the rigid-confined apparatus can be taken as an upper bound for the change in water pressure. It will be noted that the change in water pressure for the flexible-confined apparatus, for a confining pressure of 4.0 MPa, is 30% lower than the change in the water pressure for the rigid-confined apparatus.

- The changes in water pressure in the rigid-confined apparatus are slightly dependent on the initial water pressure, whereas they are not for the flexible-confined apparatus. The dependence on the initial water pressure for the rigid-confined apparatus may be attributed to the compressibility of the O-rings and of the teflon seatings of the valves. For the flexible-confined apparatus, the measured compressibility depends primarily on the two drainage layers of geotextile placed at each end of the specimen, on the rubber membranes, and to a much lesser extent on the compressibility of the O-rings of the teflon seatings of the valves. In this case, the compressibility of the O-rings and of the valve seatings becomes insignificant and cannot be measured.

- Repeatability tests with the rigid-confined and flexible-confined apparatus show that results may differ by 10%.

- The recorded changes in water pressure are dependent on the confining pressure for the flexible-confined apparatus. It is believed that, as the confining pressure increases, the rubber membranes and the stainless steel foils, which are placed around the contacts between the aluminum dummy and both the top and bottom caps to prevent protrusion of the rubber membranes, become less deformable. In other words, the flexible-confined apparatus becomes stiffer as the confining pressure increases.
These results show that the apparatus compressibility is dependent on the confining pressure. The correction for the $B$-value should be made accordingly. This has nevertheless little effect on the final value of $B$ (see Tables D-7 and D-8). On the other hand, half of the apparatus compressibility (column (7) in Table D-6) is due to the compressibility of the water in the lines (column (3) in Table D-6) and the other half to the compressibility of the drainage lines and the measuring device (column (5) in Table D-6).

For all practical purposes, we have assumed, for the flexible-confined apparatus, that:

- The change in water pressure is independent of the initial water pressure in the apparatus.

- The change in water pressure is taken to be equal to 0.10 MPa for all values of the confining pressure. This value has been observed for a confining pressure of 6.0 MPa. For higher values of the confining pressure, the change in water pressure is slightly underestimated; so is the $B$-value. At lower confining pressures, the stiffness of the apparatus is slightly overestimated, and so is the $B$-value.

The apparatus compressibility can now be estimated by:

$$ V_{pin} = \Delta u \left( C_1 + C_2 V_1 C_w \right) \quad \text{(D-34)} $$

where: $V_{pin}$: volume of the pin of the micrometer valve (MMV)
driven into and out of the closed apparatus,
$V_{pin}=1.96 \text{ mm}^3$,  
$\Delta u = 0.100 \text{ MPa} = 0.100 \text{ N/mm}^2$ for the flexible-confined apparatus,
$\Delta u = 0.125 \text{ MPa} = 0.125 \text{ N/mm}^2$ for the rigid-confined apparatus,  
(see Figure D-28 and discussion above),
$V_1$: volume of the drainage lines, $V_1=18390 \text{ mm}^3$ for the rigid-confined apparatus, $V_1=20900 \text{ mm}^3$ for the flexible-confined apparatus,
$C_w$: compressibility of the water, $C_w=48 \times 10^{-5} \text{ MPa}$,
$C_1 + C_m$: compressibility of the drainage lines and of the measuring device.

With these values, the compressibilities for the drainage lines and the measuring devices are:

- for the flexible-confined apparatus:
  \[ C_1 + C_m = 9.99 \times 10^{-9} \text{ m}^2/\text{MPa} \]  \hspace{1cm} (D-35)

- for the rigid-confined apparatus:
  \[ C_1 + C_m = 7.22 \times 10^{-9} \text{ m}^2/\text{MPa} \]  \hspace{1cm} (D-36)

All the information necessary to proceed further in the investigation of the performance of the pore pressure apparatus is thus available, and the apparatus compressibility (term 2 of Equation D-33) can be computed. All the relevant information concerning the performance of the pore pressure apparatus can be found in Table D-6.

D.2.4.3 Comparison with other apparatus

The apparatus compressibility (term 2 of Equation D-33) has been computed and compared with typical values found in the literature. Table D-6 provides all the relevant information. One half of the compressibility of the apparatus is due to the water (column (3) in Table D-6), the other half is due to the compressibility of the drainage lines and of the measuring device (column (5) in Table D-6). These proportions differ significantly from those of Wissa's or Mesri's apparatus, for which the compressibility of the water is the most important factor. Although the term $V_1/V$ (column (3) in Table D-6) is slightly higher in comparison to other apparatus, the main difference is observed in the compliance of the drainage lines and of the measuring device (column (5) in Table D-6), which is 11 to 25 times higher than some reported values (Wissa, 1969; Mesri et al., 1976). These differences
can be explained as follows:

- The apparatus involves a large number of valves (4 valves). Each valve has very deformable teflon seatings which increase the compressibility of the apparatus.

- Intermediate connections of the stainless steel tubings are necessary. Each connection adds to the compliance of the apparatus.

- The connections between the loading cap and the top cap, and between the bottom cap and the pedestal, are achieved through faced O-ring seals, which are very deformable and therefore increase the compressibility of the apparatus.

- The data presented for the pore pressure apparatus are measured. This is not the case with others apparatus, for which the presented values are either estimated or backcalculated; they should therefore be interpreted with caution.

D.2.4.4 Discussion and conclusions

The performance of the pore pressure apparatus is satisfactory. Tables D-7 and D-8 demonstrate this and show the influence of the apparatus compressibility on the apparent magnitude of the B-value. They compare the different apparatus already presented in Table D-6 for two different values of the drained bulk modulus of the rock specimen, K. The bulk modulus is assumed to be 4000 MPa in Table D-7 and 8000 MPa in Table D-8. Even with the stiffer rock specimen, the pore pressure apparatus should measure about 90% of the "true" B-value (assumed to be equal to 0.60 for this example). This is still considered to be a good observational technique (Bishop, 1976). As can be seen from Tables D-7 and D-8, the error in the observation of B depends not only on the apparatus compressibility, but also on the value of K (the drained bulk modulus of the rock skeleton, K) and on the actual value of B, itself a function of both K and n (the porosity) as indicated by Equation D-28 (Bishop, 1976).
Given Equation D-30, one can find the domain in which the pore pressure apparatus will give results judged acceptable, those for which the ratio \( B_{\text{obs}}/B_{\text{corr}} \) is larger than 90%. Equation D-30 can be rewritten as:

\[
B_{\text{corr}} = \frac{1}{\frac{1}{B_{\text{obs}}} - \Omega}
\]

\[\Omega = \left[ \frac{-C_{v}}{C-C_{s}} \right] \frac{V_{1} + C_{1} + C_{s}}{V C_{v}} = 2.16480 \times 10^{-5}
\]

\[
\left[ (1/K) - 2.7 \times 10^{-5} \right]
\]

Thus, with \( B_{\text{obs}}/B_{\text{corr}} > 90\% \), we obtain:

\[
B_{\text{obs}} \leq 4.6194 \times 10^{3} ((1/K)-2.7 \times 10^{-5})
\]

If the relation given in Equation D-37 holds, then the error involved in the measurements is less than 10%, and the observational technique is considered to be good. Equation D-37 is drawn in Figure D-18 in a plot of \( B_{\text{obs}} \) versus \( C \) (the drained compressibility of the rock skeleton). Once \( C \) and \( B_{\text{obs}} \) are known, it is easy to see if the point representing the couple \( (C, B_{\text{obs}}) \) falls below the curve described by Equation D-37. If this is the case, the corrected \( B \)-value obtained by Equation D-30', \( B_{\text{corr}} \), gives a reasonably accurate value, within 10%, of the response of the water pore pressure \( \Delta u/\Delta \sigma_{\text{oct}} \).

The reader should nevertheless keep in mind that, even corrected, the value of the pore pressure response \( \Delta u/\Delta \sigma_{\text{oct}} \) is subject to the set of assumptions invoked in the derivation of \( \Delta u/\Delta \sigma_{\text{oct}} \) with correction for the apparatus compressibility (Section D.2.4.1).
<table>
<thead>
<tr>
<th>Geological formation</th>
<th>Specimen name</th>
<th>Pure shear in</th>
<th>Simulation in field</th>
<th>$\sigma_c'$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oxfordian shale</td>
<td>TR#H3†</td>
<td>Compression</td>
<td>Crown/invert</td>
<td>1.99</td>
</tr>
<tr>
<td></td>
<td>TR#H4*</td>
<td>Extension</td>
<td>Springlines</td>
<td>1.98</td>
</tr>
<tr>
<td></td>
<td>TR#H5</td>
<td>Compression</td>
<td>Crown/invert</td>
<td>3.50</td>
</tr>
<tr>
<td>Opalinus sh.</td>
<td>RB 23-1</td>
<td>Compression</td>
<td>Springlines</td>
<td>4.99</td>
</tr>
<tr>
<td>Lias alpha shale</td>
<td>RB 23-9†</td>
<td>Compression</td>
<td>Springlines</td>
<td>2.53</td>
</tr>
<tr>
<td></td>
<td>RB 23-10</td>
<td>Compression</td>
<td>Springlines</td>
<td>2.51</td>
</tr>
</tbody>
</table>

*Symbols used in Table:*  
$\sigma_c'$ consolidation pressure  
* test stopped at 0.15% shear strain  
† significant drying before testing

**Table D-1**  
List of Tests Run in the New Triaxial System
<table>
<thead>
<tr>
<th>Confining pressure required by the operator (MPa)</th>
<th>Confining pressure monitored by the pressure transducer (mV)</th>
<th>(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.400</td>
<td>6.47</td>
<td>0.407</td>
</tr>
<tr>
<td>0.800</td>
<td>12.63</td>
<td>0.795</td>
</tr>
<tr>
<td>1.200</td>
<td>19.03</td>
<td>1.198</td>
</tr>
<tr>
<td>1.600</td>
<td>25.40</td>
<td>1.599</td>
</tr>
<tr>
<td>2.000</td>
<td>31.77</td>
<td>1.999</td>
</tr>
<tr>
<td>3.000</td>
<td>47.57</td>
<td>2.994</td>
</tr>
<tr>
<td>4.000</td>
<td>63.65</td>
<td>4.006</td>
</tr>
<tr>
<td>5.000</td>
<td>79.55</td>
<td>5.006</td>
</tr>
</tbody>
</table>

**mV–MPa relationship:**  \[ CP(\text{mV}) = 0.06293135 \times CP(\text{MPa}) \]

(from calibration of pressure transducer #47257)

**Table D-2**  
Comparison between Required and Monitored Confining Pressure
<table>
<thead>
<tr>
<th>Axial stress required by the operator (MPa)</th>
<th>Monitored axial stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.300</td>
<td>0.304</td>
</tr>
<tr>
<td>0.500</td>
<td>0.495</td>
</tr>
<tr>
<td>0.700</td>
<td>0.693</td>
</tr>
<tr>
<td>0.900</td>
<td>0.899</td>
</tr>
<tr>
<td>1.500</td>
<td>1.496</td>
</tr>
<tr>
<td>2.800</td>
<td>2.795</td>
</tr>
<tr>
<td>4.500</td>
<td>4.502</td>
</tr>
<tr>
<td>6.000</td>
<td>6.003</td>
</tr>
</tbody>
</table>

*Note:* The axial stress is computed with the formula displayed in Figure C-14, where the plugged value of CP is the confining pressure required by the operator.

**Table D-3** Comparison between Required and Monitored Axial Stress
<table>
<thead>
<tr>
<th>Control board</th>
<th>Adress</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indexer card</td>
<td>300</td>
<td>controls position of proximity probes by rotational motion of shaft of microstepping motor.</td>
</tr>
<tr>
<td>(Compumotor, model PC-21)</td>
<td>768</td>
<td></td>
</tr>
<tr>
<td>Control board</td>
<td>340</td>
<td>channel #0: confining pressure apparatus, channel,#1: axial loading apparatus.</td>
</tr>
<tr>
<td>Metrabyte, model DAC-02)</td>
<td>832</td>
<td></td>
</tr>
<tr>
<td>Printer Adapter Card</td>
<td>378</td>
<td>switches mounted on leadscrew for overrun protection of moving support.</td>
</tr>
<tr>
<td>(IBM, model PAC)</td>
<td>888</td>
<td></td>
</tr>
<tr>
<td>Asynchronous</td>
<td>COM 1</td>
<td>connection from data acquisition system to personal computer.</td>
</tr>
<tr>
<td>Communications Adapter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(IBM, model ACA)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Asynchronous</td>
<td>COM 2</td>
<td>connection from personal computer to printer.</td>
</tr>
<tr>
<td>Communications Adapter</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(IBM, model ACA)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table D-4**  Configuration of the Personal Computer
<table>
<thead>
<tr>
<th>Channel #</th>
<th>Description</th>
<th>Measurement of...</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>DCDT of volume change device</td>
<td>Volume change of water, $\Delta V_w$</td>
</tr>
<tr>
<td>5</td>
<td>DCDT for axial displacement</td>
<td>Axial deformation of the rock specimen, $\delta_a$</td>
</tr>
<tr>
<td>6</td>
<td>Proximity sensor #1</td>
<td>Diameter change of rock specimen, $\delta_d$</td>
</tr>
<tr>
<td>7</td>
<td>Proximity sensor #2</td>
<td>Diameter change of rock specimen, $\delta_d$</td>
</tr>
<tr>
<td>8</td>
<td>Pressure transducer of confining pressure</td>
<td>Confining pressure, CP</td>
</tr>
<tr>
<td>9</td>
<td>Not used</td>
<td>--</td>
</tr>
<tr>
<td>10</td>
<td>Pressure transducer of pore water</td>
<td>Pore water pressure, PP</td>
</tr>
<tr>
<td>11</td>
<td>Thermocouple</td>
<td>Temperature, $T$</td>
</tr>
<tr>
<td>12</td>
<td>Load cell</td>
<td>Axial force, $AF$</td>
</tr>
<tr>
<td>13</td>
<td>Control of input voltage for</td>
<td>Input voltage</td>
</tr>
<tr>
<td></td>
<td>proximity sensors</td>
<td>$\pm15$ VDC, VPS</td>
</tr>
<tr>
<td>14</td>
<td>Control of input voltage for</td>
<td>Input voltage</td>
</tr>
<tr>
<td></td>
<td>load cell</td>
<td>$+12$ VDC, VLC</td>
</tr>
<tr>
<td>15</td>
<td>Control of input voltage for</td>
<td>Input voltage</td>
</tr>
<tr>
<td></td>
<td>DCDT's and pressure transducers</td>
<td>$+5.5$ VDC, V</td>
</tr>
</tbody>
</table>

Table D-5 Channel Allocation of the Data Acquisition System
<table>
<thead>
<tr>
<th>Apparatus reference</th>
<th>(1) $V_1$ [mm$^3$]</th>
<th>(2) $V$ [mm$^3$]</th>
<th>(3) $V_1/V$</th>
<th>(4) $C_1+C_m$ [mm$^3$/N]</th>
<th>(5) $\exists$ [/]</th>
<th>(6) $V_1/V\exists$ [/]</th>
<th>(7) $\forall$ [/]</th>
<th>(8) Compressibility ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vissa (1969), Standard base</td>
<td>12000</td>
<td>80000</td>
<td>0.1500</td>
<td>0.160</td>
<td>0.00417</td>
<td>35.971</td>
<td>0.15417</td>
<td>10.574</td>
</tr>
<tr>
<td>Vissa (1969), Modified base (a)</td>
<td>1000</td>
<td>80000</td>
<td>0.0125</td>
<td>0.160</td>
<td>0.00417</td>
<td>2.998</td>
<td>0.01667</td>
<td>1.143</td>
</tr>
<tr>
<td>Vissa (1969), Modified base (b)</td>
<td>1000</td>
<td>80000</td>
<td>0.0125</td>
<td>0.080</td>
<td>0.00208</td>
<td>6.010</td>
<td>0.01458</td>
<td>1.000</td>
</tr>
<tr>
<td>Bruhn (1972)</td>
<td>38700</td>
<td>292000</td>
<td>0.1325</td>
<td>0.308</td>
<td>0.00219</td>
<td>60.502</td>
<td>0.13469</td>
<td>9.238</td>
</tr>
<tr>
<td>Mesri et al. (1976)</td>
<td>4420</td>
<td>247000</td>
<td>0.0179</td>
<td>*0.112</td>
<td>0.00995</td>
<td>18.842</td>
<td>0.01885</td>
<td>1.293</td>
</tr>
<tr>
<td>Bellwald (1989), Flexible-confined</td>
<td>420900</td>
<td>925000</td>
<td>0.0226</td>
<td>9.990</td>
<td>0.02250</td>
<td>1.004</td>
<td>0.04510</td>
<td>3.093</td>
</tr>
<tr>
<td>Bellwald (1989), Rigid-confined</td>
<td>18390</td>
<td>925000</td>
<td>0.0199</td>
<td>7.220</td>
<td>0.01426</td>
<td>1.224</td>
<td>0.03616</td>
<td>2.480</td>
</tr>
</tbody>
</table>

**Abbreviations:**
\[ \exists = \frac{C_1+C_m}{V_{cw}} \]
\[ \forall = \frac{V_1}{V} \]
\[ \exists \text{ (term 2 of Equation (C-33))} \]

**Hypothesis:**
\[ C_w = 48.0 \times 10^{-5} \text{ MPa}^{-1} \]

**Symbols used in Table:**
(a) from his Table 1
(b) with transducer as used for experiment described
* backcalculated values
+ with 2 layers of filter geotextil at ends of specimen
in small: hypothetical theoretical values
<table>
<thead>
<tr>
<th>Apparatus reference</th>
<th>V</th>
<th>Ω</th>
<th>$B_{obs}$</th>
<th>$B_{obs}/B_{corr}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vissa (1969), Modified base (b)</td>
<td>0.01458</td>
<td>0.0314</td>
<td>0.59</td>
<td>98.2</td>
</tr>
<tr>
<td>Bruhn (1972)</td>
<td>0.13469</td>
<td>0.2899</td>
<td>0.51</td>
<td>85.2</td>
</tr>
<tr>
<td>Mesri et al. (1976)</td>
<td>0.01885</td>
<td>0.0406</td>
<td>0.59</td>
<td>97.6</td>
</tr>
<tr>
<td>Bellvald (1989), Flexible-confined</td>
<td>0.04510</td>
<td>0.0992</td>
<td>0.57</td>
<td>94.4</td>
</tr>
<tr>
<td>Bellvald (1989), Rigid-confined</td>
<td>0.03616</td>
<td>0.0794</td>
<td>0.57</td>
<td>95.5</td>
</tr>
</tbody>
</table>

**Abbreviations:**

\[
V = \frac{V_l + C_1 + C_m}{V \cdot V_C_W} \\
\Omega = \frac{N \cdot C_w}{C - C_s}
\]

**Hypotheses:**

- \( C = 25.0 \times 10^{-5} \text{ MPa}^{-1} \)
- \( C_s = 2.7 \times 10^{-5} \text{ MPa}^{-1} \)
- \( C_w = 48.0 \times 10^{-5} \text{ MPa}^{-1} \)
- \( B_{corr} = 0.60 \)
### Table D-8

Influence of the System Compressibility on the Apparent Magnitude of the B-Value. Case Study No. 2: Bulk Modulus

<table>
<thead>
<tr>
<th>Apparatus reference</th>
<th>$V$</th>
<th>$\Omega$</th>
<th>$B_{obs}$</th>
<th>$B_{obs}/B_{corr} \ [%]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vissa (1969), Modified base (b)</td>
<td>0.01458</td>
<td>0.0719</td>
<td>0.58</td>
<td>95.9</td>
</tr>
<tr>
<td>Bruhn (1972)</td>
<td>0.13469</td>
<td>0.6597</td>
<td>0.43</td>
<td>71.6</td>
</tr>
<tr>
<td>Mesri et al. (1976)</td>
<td>0.01885</td>
<td>0.0925</td>
<td>0.57</td>
<td>94.7</td>
</tr>
<tr>
<td>Bellwald (1989), Flexible-confined</td>
<td>0.04510</td>
<td>0.2299</td>
<td>0.53</td>
<td>88.3</td>
</tr>
<tr>
<td>Bellwald (1989), Rigid-confined</td>
<td>0.03616</td>
<td>0.1771</td>
<td>0.54</td>
<td>90.4</td>
</tr>
</tbody>
</table>

**Abbreviations:**

\[
V = \frac{V_1 + C_1 + C_m}{V} \quad \Omega = V - \frac{C_m}{C - C_s}
\]

**Hypotheses:**

- $C = 12.5 \times 10^{-5}$ MPa$^{-1}$ \(\equiv K = 8000\) MPa
- $C_s = 2.7 \times 10^{-5}$ MPa$^{-1}$
- $C_m = 48.0 \times 10^{-5}$ MPa$^{-1}$
- $B_{corr} = 0.60$
**List of symbols**

- **AP**: Air pressure
- **BV**: Ball valve
- **CP**: Cell pressure
- **DC**: Direct current
- **DP**: Driver of proximity probe
- **LC**: Load cell
- **MM**: Microstepping motor
- **NV**: Needle valve
- **PP**: Pore pressure
- **TR**: Thermistor
- **VD**: Vertical displacement
- **VW**: Volume change of water

**Figure D-1** Schematic View of the New Triaxial System
Figure D-2  General View of the Large Cell
Figure D-3  Detailed View of the Inside of the Large Cell with all Measuring Devices
Figure D-4  Set-up for Measurement of Vertical Displacement with a DCDT
Figure D-5 Schematic View of the Diametric Deformations Measuring Device
Figure D-6  Cross Section and Detailed View of the Rock Specimen at the Level of the Proximity Probes

(a) Top View

(b) Detailed View
Figure D-7  Eddy-Currents Induced in Steel Targets by Alternating Magnetic Field of Probe Tip

(a) Proximity Probe Operation

(b) Functional Diagram
Figure D-8 Scan Record by Proximity Probe
(a) Full Height of the Specimen, H=170 mm
(b) Detailed View
Proximity sensor #1
"Loading" & "Unloading"

Voltage response (volt) \( V(\text{input}) = 30.0V \)

Distance "target-probe" (x1000 microns)

Proximity sensor #2
"Loading" & "Unloading"

Voltage response (volt) \( V(\text{input}) = 30.0V \)

Distance "target-probe" (x1000 microns)

**Figure D-9** Calibration Curves of the Proximity Sensors

(a) Proximity Sensor #1
(b) Proximity Sensor #2
Figure D-10 Temperature Variations Inside the Cell Chamber during Testing
This pressure transducer monitors the pressure in the piston chamber during the refilling operations.

List of Symbols

CP : Cell pressure
NV : Needle valve

Figure D-11 Schematic View of the Confining Pressure Device
Figure D-12 View of the Loading Frame
List of Symbols

LC : Load cell
NV : Needle valve
AP : Air Pressure

Figure D-13 Schematic View of the Axial Loading Device
Vertical equilibrium: \[ \sigma_a A_s + CP (A_1-A_s) - CP A_1 - F_{1c} = 0 \]

\[ \sigma_a = \frac{F_{1c} + CP}{A_s} \]

where: \( \sigma_a \) is in MPa, \( CP \) is in MPa, \( F_{1c} \) is in N and \( A_s = 5518 \text{ mm}^2 \).

**Figure D-14** Axial Stress Acting on Specimen as a Function of the Confining Pressure and the Force Monitored by the Load Cell

(a) Set-up
(b) Force Diagram
List of Symbols:

BV  Ball Valve
PT  Pressure Transducer
NV1, NV2 Needle Valves 1 & 2, respectively
MMV Micrometer Valve

Figure D-15 Schematic View of the Pore Pressure Measuring Apparatus
Symbols

BV  Ball valve
NV1,NV2  Needle valves 1 & 2, respectively
MMV  Micrometer valve
PT  Pressure transducer

NB: All parts are in stainless steel

Figure D-16 Dimensions of the Elements Composing the Pore Pressure Measuring Apparatus
Figure D-17 Rigid-Confined Apparatus with Stainless Steel Tubing
Silver-Soldered on the Top Cap
If the point defined by the measured values \((B_{\text{obs}}, K)\) falls within the bounded area, the correction to the observed \(B\)-value, \(B_{\text{obs}}\), which is made in order to get the "true" \(B\)-value, is less than 10%.

**Figure D-18** \(B_{\text{obs}}-K\) Diagram. Experimental Check-Diagram of the Observed \(B\)-value, \(B_{\text{obs}}\), versus the Drained Bulk Modulus of the Rock, \(K\), for a Ratio of \(B_{\text{obs}}/B_{\text{corr}} \leq 90\%\).
Figure D-19 Diagrams of Change in the Distance "Probe-Target" versus Confining Pressure. Observed Data with Dummy Concrete Specimen
Figure D-20 Diagrams of Change in the Distance "Probe-Target" versus Confining Pressure. Resulting Experimental Calibration Curve with Dummy Concrete Specimen
Figure D-21 Calibration of the Proximity Probes. Accuracy in the Measurement in the Change in the Distance "Probe-Target" as a Function of the Confining Pressure
Figure D-22 Set-up for the Measurement of the Extension Modulus of the Rubber Membranes
Figure D-23 Experimental Results of the Extension Modulus of the Rubber Membranes
Abbreviations used in Figure:

- H&G,1952 : Henckel and Gilbert (1952)
- LR et al.,1986 : LaRochelle et al. (1986)
- SM@1%, 20% : Secant modulus at 1% and 20% strain, respectively

**Figure D-24** Comparison of the Extension Moduli of Rubber Membranes
a) Bulging failure

\[ a_c = a_o \frac{1 + \Delta V/V_o}{1 - \epsilon} \]
\[ \epsilon = -\Delta h/h_o \]

\[ a_o = \frac{\pi d_o^2}{4} \]

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\[ a_c = \frac{d_o^2}{4} \left[ \frac{\pi \theta}{180} - \sin \theta \right] \]

where \( \theta = 2 \arccos \left( \frac{-2 \delta}{\tan \alpha} \right) \)

\( \delta = \frac{-\Delta h_p}{h_p} \)

\( h_p \) = height of sample at the appearance of the shear plane (\( h_p = 2d \))

\( -\Delta h_p \) = decrease of height after the appearance of the shear plane

\( \alpha \) = \( \cot \alpha \)

b) Shear plane failure

(RIGID DUMMY)

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**Figure D-25** Bulging and Shear Plane Failures

(from LaRochelle et al., 1986)
Figure D-26 Shear Plane Failure. Set-up for Measurement of Membrane Strength with Rigid Dummy (from LaRochelle et al., 1986)
Figure D-27 Influence of the Confining Pressure on the Zero of the Load Cell
Figure D-28 Pore Pressure Increase due Pin Insertion ($V_{pin}=1.96 \text{ mm}^3$) as a Function of the Initial Pore Pressure for Both the Rigid-confined and the Flexible-confined Systems
APPENDIX E

COMPUTER CONTROL SOFTWARE

E.1 INTRODUCTION

Software developed to control the operations of dry loading, saturation, B-check, and shearing are listed in this appendix. The programs, which are user-friendly and interactive, are written in Basic A and can be run on a PC-XT or PC-AT. Two of the most important programs are:

- <LOAD&SAT> : dry, isotropic loading, saturation, and B-check

- <CID> : drained loading/unloading.

These two programs are listed in Section E.3, and their structure is briefly described in Section E.2.

E.2 STRUCTURE OF PROGRAMS

E.2.1 <LOAD&SAT>

The program <LOAD&SAT> consists of:

- a main program,

- a subroutine <SCAN> which controls the movements of the microstepping motor and thus of the proximity probes as well,

- a subroutine <PRESCTRL> which controls the cell pressure and axial stress applied to the rock specimen,

- a subroutine <DATAREC> which controls the data acquisition system reading sequence and reading interval, and
- a subroutine <B-CHECK> which controls the operations to perform a B-check according to Wissa’s procedure (Wissa, 1969).

Once the B-check is satisfactory, i.e. once the B-values are constant, one can ask to leave the program <LOAD&SAT> and enter automatically a shearing program, such as <CID>.

F.2.2 <CID>

The program <CID> consists of:

- a main program,

- a subroutine <SCAN> which controls the movements of the microstepping motor and thus of the proximity probes as well,

- a subroutine <PRESCTRL> which controls the cell pressure and axial stress applied to the rock specimen, and

- a subroutine <DATAREC> which controls the data acquisition system reading sequence and reading interval.

Note that the program <CIU>, which controls undrained loading/unloading, has exactly the same structure as <CID>.

E.3 LISTING OF PROGRAMS

E.3.1 <LOAD&SAT>

This program is listed on pages 399 through 414.

E.3.2 <CID>

This program is listed on pages 415 through 427.
**** PROGRAM LOAD&SAT **** / This program controls the dry isotropic
loading, the saturation process and the B-check operation of the
rock specimen. At the very beginning you are asked to initialize
the measuring devices, to set the maximum cell-pressure
that defines the maximum isotropic state of stress at which the
saturation process will start, and then to define particular cell-
pressure levels at which the loading operation will be stopped in
order to proceede to some measures of the non-instantaneous defor-
manations of the rock specimen. At the end of each deformation mea-
surements you are asked to give a new cell-pressure level.
The following operations are performed during the loading procedure
1/ scan of the initial cylindrical boundary, 2/ increase of the
isotropic stress applied to the rock specimen by small increments
to a given state of stress defined by the cell-pressure level (iso-
tropic loading), 3/ as 1/, 4/ keep the isotropic state of stress
constant for a few minutes, 5/ as 1/, 6/ repeat steps 2/ to 5/ as
many times as needed until the maximum cell-pressure that defines
the isotropic state of stress at which the saturation process will
start, is reached. At the very beginning of the saturation process
you are asked to open manually the backpressure line. During the
saturation process all the operations are computer-controlled.
Pressures and deformations are recorded throughout the saturation
process. When the full saturation is thought to be reached, the
operator can proceed further with a B-check test. Depending on
the B-value, one can either pursue the saturation process or
proceed with a shear test.

*****

******************************************************************************
******************************************************************************
******************************************************************************

***** MAIN PROGRAM <LOAD&SAT> -- MP <LOAD&SAT> *****
******************************************************************************
******************************************************************************

****

LPRINT: *****************************************************
LPRINT: *****************************************************
LPRINT: *****************************************************

**** INITIALIZATION OF VARIABLES ****

CLEAR ,,2000
2BCP=2045
2BLC=360
CPLEVO=0!: NN=0
DIM G8(194)
DIM L(10),Z(10),BP(110),CP(110),AS(110),OCT(110),BV(110),TEMP(110),DTEMP(110)

**** INPUT OF ZEROS OF MEASURING DEVICES ****

CLS: LOCATE 5,1,0: DEEP: PRINT "IN MP <LOAD&SAT>:": PRINT: PRINT: PRINT
440 PRINT "Input zeros of measuring devices": PRINT: PRINT: PRINT
450 INPUT " " ZERO OF CHANNEL 4, in V":ZCH4: PRINT
460 INPUT " " ZERO OF CHANNEL 5, in V":ZCH5: PRINT
470 INPUT " " ZERO OF CHANNEL 6, in V":ZCH6: PRINT
480 INPUT " " ZERO OF CHANNEL 7, in V":ZCH7: PRINT
490 INPUT " " ZERO OF CHANNEL 8, in mV":ZCH8: PRINT
500 INPUT " " ZERO OF CHANNEL 10, in mV":ZCH10: PRINT
510 INPUT " " ZERO OF CHANNEL 12, in V":ZCH12: PRINT
520 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>": PRINT: PRINT: PRINT
530 PRINT "Check of the zero inputs": PRINT: PRINT: PRINT
540 PRINT " " ZERO OF CHANNEL 4 =";ZCH4,"V": PRINT
550 PRINT " " ZERO OF CHANNEL 5 =";ZCH5,"V": PRINT
560 PRINT " " ZERO OF CHANNEL 6 =";ZCH6,"V": PRINT
570 PRINT " " ZERO OF CHANNEL 7 =";ZCH7,"V": PRINT
580 PRINT " " ZERO OF CHANNEL 8 =";ZCH8,"mV": PRINT
590 PRINT " " ZERO OF CHANNEL 10 =";ZCH10,"mV": PRINT
600 PRINT " " ZERO OF CHANNEL 12 =";ZCH12,"V": PRINT: PRINT: PRINT
610 INPUT "Do you want to make changes? (Y/N)"",BY
620 IF INSTR(BS,Y)<>0 OR INSTR(BS,N)<>0 THEN GOTO 430
630 LPRINT "CHECK OF THE ZERO INPUTS": LPRINT
640 LPRINT "ZERO OF CHANNEL 4 =";ZCH4,"V"
650 LPRINT "ZERO OF CHANNEL 5 =";ZCH5,"V"
660 LPRINT "ZERO OF CHANNEL 6 =";ZCH6,"V"
670 LPRINT "ZERO OF CHANNEL 7 =";ZCH7,"V"
680 LPRINT "ZERO OF CHANNEL 8 =";ZCH8,"mV"
690 LPRINT "ZERO OF CHANNEL 10 =";ZCH10,"mV"
700 LPRINT "ZERO OF CHANNEL 12 =";ZCH12,"V": LPRINT
710 ' **** INPUT MAXIMUM CELL-PRESSURE ****
720 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>": PRINT: PRINT: PRINT
730 INPUT " " Final maximum cell-pressure, in MN/m2":CPMAX
740 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>": PRINT: PRINT: PRINT
750 PRINT " " FINAL MAXIMUM CELL-PRESSURE=";CPMAX,"MN/m2"
760 PRINT: PRINT: PRINT
770 INPUT "Do you want to make changes? (Y/N)"",BY
780 IF INSTR(BS,Y)<>0 OR INSTR(BS,N)<>0 THEN GOTO 720
790 LPRINT "FINAL MAXIMUM CELL-PRESSURE=";CPMAX,"MN/m2": LPRINT
800 ' **** GOSUB <DATAREC> ****
810 GOSUB 5140 ' Initialisation data reading
820 ' **** GOSUB <SCAN> ****
830 LPRINT "GOSUB <SCAN>": LPRINT
831 ' GOSUB
840 ' **** INPUT NEXT CELL-PRESSURE LEVEL AND THE NUMBER OF INCREMENTS TO REACH THAT CELL-PRESSURE LEVEL ****
850 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>": PRINT: PRINT: PRINT
860 ON KEY(1) GOSUB 3910 ' <F1> controls refilling of piston chamber
870 ON KEY(2) GOSUB 1090 ' <F2> controls the next inputs of CPLEV and NINCR
880 KEY (1) ON:KEY (2) ON
890 CLS: LOCATE 5,1,0: BEEP: BEEP
900 PRINT "Press <F1> to refill the piston chamber, after having:"
910 PRINT " (1) record the pressure in the piston chamber with "
920 PRINT " the voltmeter reading"
930 PRINT " (2) closed the valve to the cell chamber,"
940 PRINT " (3) opened the valve to the to the oil tank."; BEEP : BEEP
950 PRINT: PRINT
960 PRINT " Press <F2> to continue "
970 FOR I=1 TO 20000: NEXT
980 GOTO 890
990 STOP
1000 KEY(4) OFF 'Confining pressure is adjusted after refilling of the piston
1010 XO=28CP+INT(162.504062*CPINCR)
1020 X0H%=INT(XO/16): X0%=XO%+(XO-X0H%*16)
1030 OUT 832,X0H%: OUT 833,X0%
1040 GOTO 1130
1050 STOP
1060 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>"; PRINT: PRINT: PRINT
1070 PRINT" CPLEV IS LARGER THAN CPMAX, PLEASE RE-ENTER CPLEV."; PRINT: PRINT
1080 GOTO 1150
1090 IFLAG=O:KEY(2) OFF: KEY(1) OFF
1100 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>"; PRINT: PRINT: PRINT
1110 INPUT "Do you want to adjust the cell pressure reading and/or the axial stress reading? (Y/N)",BS
1120 IF INSTR(BS,"Y")<>O OR INSTR(BS,"y")<>O THEN GOTO 4330
1130 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>"; PRINT: PRINT: PRINT
1140 PRINT " ACTUAL CELL PRESSURE LEVEL =";CPINCR;"MN/m2"; PRINT: PRINT
1150 INPUT " Next cell-pressure level, in MN/m2 "; CPLEV: PRINT: PRINT: PRINT
1160 INPUT " Number of increments to reach the next cell-pressure level";
1170 IF CPLEV>CPMAX THEN GOTO 1060
1180 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>"; PRINT: PRINT: PRINT
1190 PRINT" NEXT CELL-PRESSURE LEVEL ="; CPLEV;"MN/m2"; PRINT: PRINT
1200 PRINT" NUMBER OF INCREMENTS TO REACH THE NEXT CELL-PRESSURE LEVEL = ";INCR
1210 INPUT "Do you want to make changes? (Y/N)",BS
1220 IF INSTR(BS,"Y")<>O OR INSTR(BS,"y")<>O THEN GOTO 1130
1230 LPRINT"NEXT CELL-PRESSURE LEVEL ="; CPLEV;"MN/m2"
1240 LPRINT" NUMBER OF INCREMENTS TO REACH THE NEXT CELL-PRESSURE LEVEL = ";INCR
1250 ' The basic loop used to control the loading procedure between two cell
1260 ' pressure levels involves the following subroutines: <PRESCTRL> and
1270 ' <SCAN>. In subroutine <SCAN>, the pointer goes to subroutine
1280 ' <DATAREC> in order to record the vertical deformation too, so
1290 ' that the tensor of deformations can be entirely measured.
1300 ' *** GOTO SUBROUTINE <PRESCTRL> ***
1310 GOTO 3580
1320 STOP
1330 ' *** GOSUB <SCAN> ***
1340 LPRINT "GOSUB <SCAN> " GOSUB 'for CP=CPMAX
1350 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>"; PRINT: PRINT: PRINT
1360 PRINT " The maximum isotropic state of stress is reached, the saturation
1370 PRINT " process can start. Don't forget to open the backpressure !!!": LP
1380 LPRINT "THE MAXIMUM ISOTROPIC STATE OF STRESS IS REACHED, THE SATURATION "
1390 LPRINT "PROCESS CAN START. DON'T FORGET TO OPEN THE BACKPRESSURE !!!!": LPRINT
1400 FOR I=1 TO 5000: NEXT
1410 CLS: LOCATE 5,1,0: BEEP: PRINT "IN HP <LOAD&SAT>" : PRINT: PRINT: PRINT
1420 PRINT " The saturation process is under progress."
1430 PRINT " Everything is computer-controlled." : PRINT
1440 PRINT " If you want to adjust the cell pressure or the axial stress,"
1450 PRINT " press the key <F10>."
1460 PRINT " When you are ready for a B-check, press key <F5> to continue."
1470 ON KEY(5) GOSUB 1600
1472 ON KEY(8) GOSUB 7170
1480 ON KEY(10) GOSUB 4325
1490 KEY(5) ON
1492 KEY(8) ON
1500 KEY(10) ON
1501 I1=1
1505 IFLAG=0
1510 IF I1=300 GOTO 1540
1511 SAT=11
1512 GOTO 5140 'GOSUB <DATAREC> for printing data on screen only
1513 I1=II+1
1514 GOTO 1510
1515 STOP
1530 ' **** GOSUB <DATAREC> ****
1540 KEY(5) OFF: KEY(8) OFF: KEY(10) OFF
1541 SAT=01
1550 GOSUB 5140
1560 ' **** GOSUB <SCAN> ****
1570 LPRINT "GOSUB <SCAN>" : LPRINT
1571 'gosub
1580 GOTO 1410
1585 STOP
1590 ' **** GOSUB <B-CHECK> ****
1600 IFLAG=0: KEY(5) OFF: KEY(8) OFF: KEY(10) OFF
1605 SAT=01
1610 GOSUB 6140
1620 END
1630 ' ******************************************************
1640 ' **** SUBROUTINE <SCAN> ****
1650 ' ******************************************************
1660 ADDRESS%=768
1670 GOSUB 3050: JS=""
1680 FOR I=1 TO 2000: NEXT
1690 P=0
1700 ' **** INPUT DATA FOR CONTINUOUS AND SCAN DISTANCES ****
1710 Z(1)=1080001
1720 L(1)=20
1730 Z(2)=1500001
1740 L(2)=20
1750 Z(3)=1500001
1760 L(3)=20
1770 Z(4)=150000!
1780 L(4)=20
1790 Z(5)=145000!
1800 L(5)=20
1810 ON KEY(6) GOSUB 1970
1820 PRINT "This are the input data for traveling, Z(1), and"
1830 PRINT "scanning, L(1), distances (in microsteps) from the"
1840 PRINT "bottom (1) to the top (5)."
1850 PRINT: PRINT:
1860 PRINT "Z(1)=",Z(1)," L(1)=",L(1)
1870 PRINT "Z(2)=",Z(2)," L(2)=",L(2)
1880 PRINT "Z(3)=",Z(3)," L(3)=",L(3)
1890 PRINT "Z(4)=",Z(4)," L(4)=",L(4)
1900 PRINT "Z(5)=",Z(5)," L(5)=",L(5): PRINT: PRINT
1910 PRINT "Do you want to change the traveling and scanning lengths?"
1920 PRINT "Which are above? You have 10 seconds to press the key <F6>."
1930 KEY(6) ON
1940 FOR I=1 TO 5000: NEXT I
1950 KEY(6) OFF
1960 GOTO 2200
1970 KEY(6) OFF
1980 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>:": PRINT: PRINT: PRINT
1990 PRINT "INPUT THE NEW DATA": PRINT: PRINT: PRINT
2000 INPUT "Z(1)=",Z(1)
2010 INPUT "Z(2)=",Z(2)
2020 INPUT "Z(3)=",Z(3)
2030 INPUT "Z(4)=",Z(4)
2040 INPUT "Z(5)=",Z(5)
2050 INPUT "L(1)=",L(1)
2060 INPUT "L(2)=",L(2)
2070 INPUT "L(3)=",L(3)
2080 INPUT "L(4)=",L(4)
2090 INPUT "L(5)=",L(5): PRINT: PRINT
2100 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>:": PRINT: PRINT: PRINT
2110 PRINT "CHECK NEW INPUT DATA": PRINT: PRINT: PRINT
2120 PRINT "Z(1)=",Z(1)," L(1)=",L(1)
2130 PRINT "Z(2)=",Z(2)," L(2)=",L(2)
2140 PRINT "Z(3)=",Z(3)," L(3)=",L(3)
2150 PRINT "Z(4)=",Z(4)," L(4)=",L(4)
2160 PRINT "Z(5)=",Z(5)," L(5)=",L(5): PRINT: PRINT
2170 INPUT "Do you want to make changes? <Y/N> ",BS
2180 IF INSTR(BS,″Y″)=0 OR INSTR(BS,″y″)=0 THEN GOTO 1980 ELSE 2200
2190 ' **** ENTER FIXED DATA FOR UPWARDS TRAVEL ****
2200 VS=″2.0″: ' VS is the speed during scan move
2210 VCS=″0.5″: ' VCS is the speed during continuous move
2220 DS=″2500″: ' DS is the distance between two scan-positions (microsteps)
2230 AS=″2.0″: ' AS is the acceleration for all moves (continuous & scan)
2240 CMDAS=″ STI X1 MN A=+AS+″
2250 CMDBS=""
2260 FOR I=1 TO 5
2270 Z=Z(I):ZS=STRS(Z):ZS=RIGHTS(ZS,6)
2280 L=L(I):LS=STRS(L):LS=RIGHTS(LS,2)
2290 CMD$=CMD$ + "$H+VCS$+" D"+ZS$+" X1 G L"+LS$+" V"+VS$+" D"+DS$+" G TRIXX X1 N +
2300 NEXT I
2310 CMD$= CMD$ + CMD$
2320 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
2330 PRINT "COMMAND SEND TO MOTOR FOR UPWARDS TRAVEL, CMD$": PRINT: PRINT: PRINT
2340 LPRINT " COMMAND SEND TO MOTOR FOR UPWARDS TRAVEL, CMD$"
2350 PRINT CMD$;
2360 LPRINT CMD$;
2370 R1=R1 ' Variable attached to CMD$ during upwards move
2380 GOTO 2760
2390 CMD$=""'
2400 ' **** GOTO SUBROUTINE <DATAREC> ****
2410 PRINT " GOTO SUBROUTINE <DATAREC>": PRINT
2420 GOSUB 5140
2430 PRINT " BACK FROM SUBROUTINE <DATAREC>": PRINT
2440 ' **** ENTER FIXED DATA FOR DOWNWARDS TRAVEL ****
2450 DS$="-2500" ' DS$ is the distance between two scan-positions (microsteps)
2460 VS$="-2.0" ' VS$ is the speed during scan move
2470 VC$="0.5" ' VC$ is the speed during continuous move
2480 AS$="2.0" ' AS$ is the acceleration for all moves (continuous & scan)
2490 CMD$="MN A"+AS$+""
2500 CMD$=""
2510 FOR I=1 TO 5
2520 L(I)=L(I-1)
2530 Z(I)=-Z(I-1)
2540 DATA 20,-145000,20,-150000,20,-150000,20,-108000
2550 L=L(I):LS=STRS(L):LS=RIGHTS(LS,2)
2560 Z=Z(I):ZS=STRS(Z):ZS=RIGHTS(ZS,7)
2570 PRINT LS,ZS
2580 PRINT #2,LS,ZS
2590 CMD$=CMD$ + "$H+LS$+" V"+VS$+" D"+DS$+" G TRIXX X1 N V"+VC$+" D"+DS$+" X1 G +
2600 NEXT I
2610 CMD$= CMD$ + CMD$
2620 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
2630 PRINT "COMMAND SEND TO MOTOR FOR DOWNWARDS TRAVEL, CMD$": PRINT: PRINT: PRINT
2640 LPRINT " COMMAND SEND TO MOTOR FOR DOWNWARDS TRAVEL, CMD$"
2650 PRINT CMD$;
2660 LPRINT CMD$;
2670 R1=R1 ' Variable attached to CMD$ during downwards move
2680 GOTO 2760
2690 CMD$=""
2700 ' Stop microstepping motor and send signal of end of travel. "CR"
2710 PRINT "COMMAND SEND TO MOTOR TO STOP AND DISCONNECT"
2720 LPRINT " COMMAND SEND TO MOTOR TO STOP AND DISCONNECT"
2730 CMD$=" STO CR ="
2740 PRINT CMD$;
2750 LPRINT CMD$;
2760 GOSUB 2770:GOSUB 3120:GOTO 1690
2770 AD=AS=VS=WS=HS=DL=DLS=D$=
2780 IF RIGHTS(CMD$)=1 THEN CMD$=CMD$+CHR$(13)
2790 FOR K=1 TO LEN(CMD$) ' Enter routine here with PC21 command=CMD$
2800 X$=MID$(CMD$,K,1)
2810 GOSUB 2970
2820 NEXT K
2830 XM=CHR$(13):GOSUB 2970;J$="":GOSUB 2850
2840 IF RIGHTS(CMD$)=1 THEN CMD$=CMD$+CHR$(13)
2850 B=INP(ATRRESS%+1)
2860 IF B AND &H8 THEN GOTO 2870 ELSE RETURN
2870 J=INP(ATRRESS%)
2880 OUT ADDRESS%+1,&HE0
2890 B=INP(ATRRESS%+1)
2900 IF B AND &H8 THEN GOTO 2890 ELSE GOTO 2910
2910 OUT ADDRESS%+1,&H60
2920 X$=CHR$(J)
2930 JS=JS+X$
2940 IF J=&H5 THEN GOTO 2950 ELSE GOTO 2850
2950 JS=LEFT$(JS,9)
2960 IF JS="" THEN GOTO 2850
2970 B=INP(ATRRESS%+1)
2980 IF B AND &H10 GOTO 2990 ELSE GOTO 2970
2990 OUT ADDRESS%,ASC(X$)
3000 OUT ADDRESS%+1,&H70
3010 B=INP(ATRRESS%+1)
3020 IF B AND &H10 GOTO 3010 ELSE GOTO 3030
3030 OUT ADDRESS%+1,&H60
3040 RETURN
3050 OUT ADDRESS%+1,&H64
3060 OUT ADDRESS%+1,&H60
3070 FOR I=1 TO 100:NEXT
3080 OUT ADDRESS%+1,&H40
3090 OUT ADDRESS%+1,&H60
3100 FOR I=1 TO 100:NEXT
3110 RETURN
3120 CMD$="R":GOSUB 2780:IF INSTR(J$,""S")=0+(INSTR(J$,""S")=0) THEN RETURN
3130 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
3140 ' **** READ FROM DATA ACQUISITION SYSTEM ****
3150 CLOSE #1
3160 OPEN "COM1:2400,N,8,1,CS,DS,CD" AS #1
3170 ON ERROR GOTO 3500
3180 IF LOC(1)<199 THEN GOTO 3180
3190 GS=INPUT$(LOC(1),#1)
3200 FOR I=1 TO 198
3210 FS=MID$(GS,1,2)
3220 IF (FS=CHR$(13)+CHR$(10)) THEN GOTO 3250
3230 IF I=198 GOTO 3150
3240 NEXT
3250 H=199-1;E=I+2
3260 GS=MID$(GS,E,H)+LEFT$(GS,1)
3270 GGS=MIDS(GS,2,13)+MIDS(GS,48,8)+MIDS(GS,63,8)+MIDS(GS,153,8)
3280 AAS=MIDS(GS,2,13)
3290 BB$=MIDS(GS,48,8);BB$=VAL(BB$)
3300 CC$=MIDS(GS,63,8);CC$=VAL(CC$)
3310 CP$=MIDS(GS,93,8);CP$=VAL(CP$);CP$=ABS(CP$)
3320 DD$=MIDS(GS,153,8);DD$=VAL(DD$)
3330 SS=CHR$(13)+CHR$(10)
3340 JJ=VAL(J$)
3350 FILE$="A:TARGET.PRN"
3360 OPEN FILE$ FOR APPEND AS 2
3370 WRITE #2,AAS,CP,BB,CC,DD,JJ
3380 PRINT GGS,J$,S$;
3390 FOR I=1 TO 12
3400 K$=MIDS(J$,1,2)
3410 IF(K$=CHR$(42)+CHR$(82)) THEN GOTO 3510 'Looks for "ER"
3420 IF(K$=CHR$(67)+CHR$(82)) THEN GOTO 3530 'Looks for "CR"
3430 NEXT I
3440 J$=""
3450 FOR I=1 TO 10:OUT 890,14
3460 NEXT
3470 FOR I=1 TO 10:OUT 890,0
3480 NEXT
3490 GOTO 3120
3500 RESUME : GOTO 3160
3510 IF R1=1 THEN GOTO 2390 'Gets the data for downwards move
3520 IF R1=2 THEN GOTO 2690 'Gets the data for stopping motor
3530 CLOSE #2:PRINT "file TARGET.PRN closed"
3540 ON ERROR GOTO 0:RETURN
3550 i:*****************************************************************************
3560 i:**** SUBROUTINE <PRESCTRL> ****
3570 i:*****************************************************************************
3580 L=1 'Variable controlling the data reading of the DAC-system:
3590 ' if L=1, data are trashed; if L=2, data are recorded.
3600 FOR INCR=1 TO MINCR
3610 CIPNCR=(CPLEV-CPLEVO)/MINC
3620 CIPNCR=CPLEVO+INCR*CIPNCR
3630 i:**** COMMANDS SEND TO DAC-02, CHANNEL #0, Cell pressure ****
3640 XO=ZBCP+INT(162.504062#*CIPNCR)
3650 XOXY=INT(XO/16);XOLX=16*(XO-XOXY*16)
3660 OUT 832,XOLX:OUT 833,XOXY
3670 i:**** COMPUTE NEEDED AXIAL FORCE TO RE-ESTABLISH ISOTROPIC STATE OF STRESS
3680 ONTO ROCK SPECIMEN ****
3690 AFCOMP=2.02683*CIPNCR 'Area of piston=20.2683 cm2
3700 i:**** WANTED AXIAL STRESS ONTO ROCK SPECIMEN ****
3710 AINCPR=CIPNCR 'for non hydrostatic state of stress, use:ASCOMP=AFCOMP/5 .518044+.6326*P2*CIPNCR
3720 i:**** COMMANDS SEND TO DAC-02, CHANNEL #1, Axial force ****
3730 X1=ZBLC+INT(146.983313#*AFCOMP)
3740 X1XY=INT(X1/16);X1LX=16*(X1-X1XY*16)
3750 OUT 834,X1LX:OUT 835,X1XY
3760 PRINT:PRINT
3760 PRINT "REQUIRED CONFINING PRESSURE=";CIPNCR;"MN/m2"
3770 PRINT "REQUIRED AXIAL STRESS=";ASINCR;"MN/m2"
3780 FOR I=1 TO 5000:NEXT I
3790 IF NN=0 GOTO 6640
3800 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>"; PRINT: PRINT: PRINT: PRINT
3810 PRINT " Please wait ! The required state of stress is being adjusted. ": PR
3820 INT: PRINT
3830 PRINT "REQUIRED CONFINING PRESSURE=";CPINCR;"MN/m2"
3840 H=0
3850 ' **** GOTO SUBROUTINE "<DATAREC>"; with partial results and screen printing
3860 GOTO 4490
3870 FOR I=1 TO 10000: NEXT I
3880 IF CPINCR<CPMAX GOTO 830 ELSE 1340
3890 STOP
3900 ' **** REFILLING THE PISTON CHAMBER ****
3910 IPFLAG=0: KEY (1) OFF: KEY (2) OFF
3920 XO= 1000
3930 XO%=INT(XO/16): XO%=16*(XO-XO%)*16
3940 OUT 832,XO%: OUT 833,XO%
3950 CLS: LOCATE 5,1,0: BEEP: BEEP
3960 ON KEY(3) GOSUB 4120  ' <F3> controls refilling operations
3970 KEY (3) ON
3980 PRINT " Refilling the piston chamber.: PRINT
3990 PRINT " When ready, perform the following steps:" PRINT
4000 PRINT " (1) unplug the motor from the controller"
4010 PRINT " (2) close the valve to the oil tank"
4020 PRINT " (3) unplug the pressure transducer of the triaxial cell"
4030 PRINT " (4) plug-in by pass wire to the controller"
4040 PRINT " (5) press key <F3>""
4050 PRINT " (6) plug-in the motor to the controller"
4060 PRINT " (7) bring the pressure in the piston chamber to the pressure"
4070 PRINT " previously recorded by the voltmeter reading"
4080 PRINT: PRINT: PRINT: BEEP: BEEP
4090 FOR I=1 TO 20000: NEXT I
4100 GOTO 3980
4110 STOP
4120 KEY (3) OFF
4130 XO= 2070 'Incr.press.in piston which corresponds to CP
4140 XO%=INT(XO/16): XO%=16*(XO-XO%)*16
4150 OUT 832,XO%: OUT 833,XO%
4160 ON KEY (4) GOSUB 1000  ' <F4>, controls the inputs of CLEVEL and new NINCR
4170 KEY (4) ON
4180 PRINT: PRINT
4190 PRINT " When the pressure in the piston has reached the cell pressure,"
4200 PRINT " perform the following steps:";
4210 PRINT " (1) unplug the motor from the controller"
4220 PRINT " (2) unplug the by-pass wire cable from the controller"
4230 PRINT " (3) plug-in the pressure transducer of the triaxial cell"
4240 PRINT " to the controller"
4250 PRINT " (4) open the valve of the cell chamber"
4260 PRINT " (5) press key <F4>"
4270 PRINT " (6) plug-in the motor to the controller"
4280 FOR I=1 TO 10000: NEXT I
4290 PRINT " Increasing the pressure in the piston chamber ": PRINT
4300 GOTO 4190
4310 STOP
4320 ' **** CHANGE ZERO-BYTES OF THE PRESSURE CONTROLLER AND OF THE AXIAL STRES
4325 S CONTROLLER ****
4330 IFLAG=0: SAT=0!
4335 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <LOAD&SAT>": PRINT: PRINT: PRINT
4340 PRINT " THE DEFAULT VALUE OF THE ZERO-BYTE OF THE CELL PRESSURE CONTROLLER
4350 PRINT: PRINT
4360 INPUT " New value of the zero-byte of the cell pressure controller":;ZBCP: P
4370 PRINT " THE NEW DEFAULT VALUE OF THE ZERO-BYTE OF THE CELL PRESSURE CONTROL
4380 PRINT "LER =":;ZBCP
4390 PRINT: PRINT
4390 PRINT " THE DEFAULT VALUE OF THE ZERO-BYTE OF THE AXIAL STRESS CONTROLLER =
4400 PRINT: PRINT
4410 INPUT " New value of the zero-byte of the axial stress controller":;ZBLC: PR
4420 PRINT " THE NEW DEFAULT VALUE OF THE ZERO-BYTE OF THE AXIAL STRESS CONTROLL
4430 PRINT: PRINT
4440 INPUT "Do you want to make changes? (Y/N)"; BS
4450 IF INSTR(BS,"Y")<>O OR INSTR(BS,"N")<>O THEN GOTO 4340
4460 GOTO 1130
4470 STOP
4480 ' **** READ DATA FROM DATA-ACQUISITION SYSTEM ****
4490 FOR I=1 TO 20000: NEXT I
4500 RESET
4510 OPEN "COM1:2400,N,8,1,CS,DS,CD" AS #1
4520 ON ERROR GOTO 5050
4530 IF LOC(1)<194 THEN GOTO 4530
4540 GS=INPUT$(LOC(1),1)
4550 IF L<>2 GOTO 5090
4560 ' **** LOOK FOR BEGINNING OF STRING ****
4570 FOR J=1 TO 193
4580 FS=H$(GS,J,2)
4590 IF (FS=CHR$(13)>CHR$(10)) THEN 4630
4600 IF J=193 GOTO 5080
4610 NEXT J
4620 GOTO 4570
4630 H=194-J:E=J+2
4640 ' **** WRITE STRING IN RIGHT ORDER ****
4650 GS=H$(GS,E,H)>LEFT$(GS,J)
4660 ' **** READ ALL INPUT VOLTAGES ****
4670 'VLCS=Reading of input voltage of load cell, 10.0 VDC
4680 VLCS=H$(GS,168,B):VLC=VAL(VLCS):VLC=ABS(VLC)
4690 'VS=Reading of input voltage (pressure transducers, DCDT), 5.5 VDC
4700 VS=H$(GS,183,B):V=VAL(VS):V=ABS(V)
4710  ' **** CHECK IF ALL INPUT VOLTAGES ARE OK ****
4720  IF (V<5.45 OR V>5.55) THEN GOTO 5060
4730  IF (VLC<9.95 OR VLC>10.05) THEN GOTO 5070
4740  ' **** TIME READING ****
4750  ' TS=Reading time / day:hours:minutes:seconds
4760  TS=HID5(GS,2,12)
4770  ' TS=Time reading in seconds
4780  TS=864001*VAL(HID5(GS,2,3))+3600*VAL(HID5(GS,6,2))+60*VAL(HID5(GS,9,2))+VAL(HID5(GS,11,2))
4790  ' **** CELL PRESSURE ****
4800  ' CH8$=Reading Channel #8, PT #C47257, CF=0.0629315 ((kN/m2)/mV)
4810  CH8$=HID5(GS,78,8):CH8=VAL(CH8$):CH8=ABS(CH8)
4820  CP=(.3641224*(CH8-20CH8))/V-.0026
4830  ' **** AXIAL FORCE IN LOADING PISTON ****
4840  ' CH12$=Reading Channel #12, CF=3.71470 (kN/mV)
4850  CH12$=HID5(GS,138,8):CH12=VAL(CH12$)
4860  AF=37.147*(CH12+20CH12)/VLC
4870  ' **** AXIAL STRESS ON ROCK SPECIMEN ****
4880  AS=AF/5.518044+.991789*CP
4890  ' **** PRINTING OF DATA ****
4900  CLS: LOCATE 5,1,0: BEEP: PRINT "IN <PRESCTRL>": PRINT: PRINT: PRINT
4910  PRINT "*************": PRINT
4920  PRINT "Time":TS: PRINT
4930  PRINT "Increment # ................. ",,INCR
4940  PRINT "Maximum number of Increments .... ",,NINCR
4950  PRINT "Cell pressure (wanted/measured) .... \",CPINCR:CP:MN/m2"
4960  PRINT "Axial stress (wanted/measured) .... \",ASINCR:AS:MN/m2"
4970  PRINT "Input voltage -- 5.5 VDC ...... \",V,"VDC"
4980  PRINT "Input voltage -- 10.0 VDC ...... \",VLC,"VDC": PRINT
4990  PRINT "*************": PRINT
5000  CLOSE #1
5010  NEXT INCR
5020  CPLEVO=CPINCR
5030  GOTO 3870
5040  ' If ERROR occurs then reboot program automatically
5050  RESUME: GOTO 4510
5060  BEEP: PRINT "Check input voltage 5.5 VDC": GOTO 5080
5070  BEEP: PRINT "Check input voltage 10.0 VDC"
5080  CLOSE #1: GOTO 4510
5090  CLOSE #1: L=2: GOTO 4510
5100  STOP
5110  ' ********************************************
5120  ' **** SUBROUTINE <DATAREC> ****
5130  ' ********************************************
5140  L=1 'Variable controlling the data reading of the DAC-system:
5150  ' if L=1, data are trashed; if L=2, data are recorded.
5160  RESET
5170  OPEN "COM1:2400,N,8,1,CS,DS,CD" AS #1
5180  ON ERROR GOTO 6040
5190  IF LOC(1)<194 GOTO 5190
5200  GS=IN-UTS(LOC(1),#1)
5210  IF L<2 GOTO 6090
5220 FOR J=1 TO 193
5230 FS=MID$(G$,J,2)
5240 IF (FS=CHR$(13)+CHR$(10)) THEN 5280
5250 IF J=193 GOTO 6080
5260 NEXT J
5270 GOTO 5220
5280 H=194-J;E=J+2
5290 ' **** WRITE STRING IN RIGHT ORDER ****
5300 GS=MID$(GS,E,H)+LEFT$(GS,J)
5310 ' **** READ ALL INPUT VOLTAGES ****
5320 ' VPS$=Reading of input voltage of proximity sensors, 15.0 VDC
5330 VPS$=MID$(GS,153,8);VPS=VAL(VPS$);VPS=ABS(VPS)
5340 ' VLCS=Reading of input voltage of load cell, 12.0 VDC
5350 VLC$=MID$(GS,168,8);VLC=VAL(VLC$);VLC=ABS(VLC)
5360 ' VS=Reading of input voltage (pressure transducers, DCDT, 5.5VDC
5370 VS=MID$(GS,183,8);V=VAL(VS$);V=ABS(V)
5380 ' **** CHECK IF ALL INPUT VOLTAGES ARE OK ****
5390 IF (V<5.45 OR V>5.55) THEN GOTO 6050
5400 IF (VLC<9.95 OR VLC>10.05) THEN GOTO 6060
5410 'IF (VPS<29.95 OR VPS>30.05) THEN GOTO 6070
5420 ' **** TIME READING ****
5430 'TS=Reading time / day:hours:minutes:seconds
5440 TS=MID$(GS,2,12)
5450 'TS=Time reading in seconds
5460 TS$=86400*VAL(MID$(GS,2,3))=3600*VAL(MID$(GS,6,2))=60*VAL(MID$(GS,9,2))=VAL(MID$(GS,11,2))
5470 ' **** VOLUME CHANGE OF WATER ****
5480 'CH4=Reading Channel #4, CF for Volume Change =3.246488 (CH3/V*Vinp)
5490 CH4$=MID$(GS,18,8);CH4=VAL(CH4$)
5500 VOL.CH=3.246488*V*(CH4-ZCH4)
5510 ' **** VERTICAL DISPLACEMENT ****
5520 'CH5$=Reading Channel #5, CF for vertical displ.= 0.0686531 (CM/V*Vinp)
5530 CH5$=MID$(GS,35,8);CH5=VAL(CH5$);CH5=ABS(CH5)
5540 VERT.DISPL.1=0.0686531*V*(CH5-ZCH5)
5550 ' **** HORIZONTAL DISPLACEMENTS (TWO) ****
5560 'CH6=Reading Channel #6, CF for PS(1) = 514.04785 (uH/V)
5570 CH6$=MID$(GS,48,8);CH6=VAL(CH6$);CH6=ABS(CH6)
5580 HORIZ.DISPL.1=514.04785*(CH6-ZCH6)
5590 'CH7=Reading Channel #7, CF for PS(2) = 514.74450 (uH/V)
5600 CH7$=MID$(GS,63,8);CH7=VAL(CH7$);CH7=ABS(CH7)
5610 HORIZ.DISPL.2=514.74450*(CH7-ZCH7)
5620 SUNDISPL=HORIZ.DISPL.1+HORIZ.DISPL.2
5630 ' **** CELL PRESSURE ****
5640 'CH8=Reading Channel #8, PT #C47257, CF=0.06293135 ((NH/N2)/mV)
5650 CH8$=MID$(GS,78,8);CH8=VAL(CH8$);CH8=ABS(CH8)
5660 CP=0.3461224*(CH8-ZCH8)/V).0026
5670 ' **** PORE PRESSURE ****
5680 'CH10=Reading Channel #10, PT #CO3154, CF=0.012724 ((NH/N2)/mV)
5690 CH10$=MID$(GS,108,8);CH10=VAL(CH10$);CH10=ABS(CH10)
5700 PP=0.069982*(CH10-ZCH10)/V).0047
5710 ' **** TEMPERATURE ****
5720 'CH11=Reading Channel #11, Equation: Temp.=51.30565*log(mV)+108.846
5730 CH11=VAL(CH11$):CH11=ABS(CH11)*5.5/V
5740 CH11=LOG(CH11)/2.302585
5750 TEMP=(-51.30565*CH11)+108.846
5760 * **** AXIAL FORCE IN LOADING PISTON ****
5770 CH12$=Reading Channel #12, CF=3.71470 (KN/m3)
5780 CH12=VAL(CH12$)
5790 AF=37.147*(CH12+ZCH12)/VLC
5800 * **** MEASURED AXIAL STRESS ON ROCK SPECIMEN ****
5810 AS=AF/5.518044+.991789*CP
5815 ON KEY(1) GOSUB 3910 '<F1> to refill piston
5816 ON KEY(2) GOSUB 1090 '<F2> to increase CP
5817 ON KEY(5) GOSUB 1600 '<F5> to check B-Value
5818 ON KEY(10) GOSUB 4325 '<F10> to adjust CP
5819 KEY (1) ON:KEY (2) ON:KEY (5) ON:KEY (10) ON
5820 IF IFLAG=1 GOTO 5854
5822 IFLAG=1:CLS:KEY OFF: LOCATE 1,1,0: PRINT "IN <DATAREC>"
5825 KEY 1,"REFILL":KEY 2,"CP-INC":KEY 3,"":KEY 4,"":KEY 5,"B-VALUE":KEY 6,"":KEY 7,"":KEY 8,"":KEY 9,"":KEY 10,"RE-ZERO":KEY ON
5826 FOR IPX=1 TO 80:LOCATE 2,IPX,0:PRINT CHR$(220):NEXT IPX
5828 LOCATE 3,1,0:PRINT " Time ..................... "
5830 LOCATE 5,1,0:PRINT " Vertical displacement ........ "
5832 LOCATE 6,1,0:PRINT " Horizontal displacement no.1 .. "
5834 LOCATE 7,1,0:PRINT " Horizontal displacement no.2 .. "
5836 LOCATE 8,1,0:PRINT " Sum of horizontal displacements . "
5838 LOCATE 9,1,0:PRINT " Cell pressure (wanted/measured) . "
5840 LOCATE 10,1,0:PRINT " Pore Pressure .................. "
5842 LOCATE 11,1,0:PRINT " Temperature .................. "
5844 LOCATE 12,1,0:PRINT " Axial stress (wanted/measured) . "
5846 LOCATE 14,1,0:PRINT " Input voltage -- 5.5 VDC .................. "
5848 LOCATE 15,1,0:PRINT " Input voltage -- 10.0 VDC .................. "
5850 LOCATE 16,1,0:PRINT " Input voltage -- +/-15.0 VDC ............. "
5852 FOR IPX=1 TO 80:LOCATE 17,IPX,0:PRINT CHR$(220):NEXT IPX
5854 LOCATE 3,3,0:PRINT "$"
5856 LOCATE 4,3,0:PRINT VOL.CH,"cm3"
5860 LOCATE 5,3,0:PRINT VERT.DISPL,"cm"
5870 LOCATE 6,3,0:PRINT HORIZ.DISPL.1,CHR$(230);"m"
5880 LOCATE 7,3,0:PRINT HORIZ.DISPL.2,CHR$(230);"m"
5890 LOCATE 8,3,0:PRINT SUMDISPL,"um"
5900 LOCATE 9,3,0:PRINT CPINCR;"/";CP,"MN/m2"
5910 LOCATE 10,3,0:PRINT PP,"MN/m2"
5920 LOCATE 11,3,0:PRINT TEMP,CHR$(248);"C"
5930 LOCATE 12,3,0:PRINT ASINCR;"/";AS,"MN/m2"
5940 LOCATE 14,3,0:PRINT V,"VDC"
5950 LOCATE 15,3,0:PRINT VLC,"VDC"
5960 LOCATE 16,3,0:PRINT VPS,"VDC":LOCATE 18,1,0
5971 IF SAT=11 GOTO 1513
5980 FILES = "A:\L\DATA.PRM"
5990 OPEN FILES FOR APPEND AS 3
6000 WRITE #3,TS,VOL.CH,VERT.DISPL,HORIZ.DISPL.1,HORIZ.DISPL.2,SUMDISPL,CP,PP,TEMP,AS
6010 CLOSE #1,#3
6020 ON ERROR GOTO 0: RETURN
6030 'If ERROR occurs then reboot program automatically
6040 RESUME: GOTO 5170
6050 BEEP:PRINT "Check input voltage 5.5 VDC":GOTO 6080
6060 BEEP:PRINT "Check input voltage 10.0 VDC":GOTO 6080
6070 BEEP:PRINT "Check input voltage 15.0 VDC"
6080 IFLAG=0:CLOSE #1: GOTO 5160
6090 CLOSE #1: L=2: GOTO 5170
6100 END
6110 ; ****************************
6120 ; **** SUBROUTINE <B-CHECK> ****
6130 ; ******************************
6135 SAT=0
6140 NN=1 'Attached to the first B-check increment
6150 INCN=NN
6160 SP$=""
6170 ; **** INPUT CELL-PRESSURE AND NUMBER OF INCREMENTS TO REACH THAT CELL-PRESSURE ****
6180 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <B-CHECK>": PRINT: PRINT: PRINT
6190 PRINT " "Give the next cell pressure level you want to reach:"
6200 PRINT " i.e. actual(CP)+delta(CP)=CLEV, where delta(CP) is the"
6210 PRINT "total increment in cell pressure to be produced."
6220 PRINT "Note: Delta(CP)=<1.0 MN/m2 !!!!": PRINT: PRINT
6230 PRINT "ACTUAL CELL PRESSURE LEVEL ="; CPLEV; "MN/m2"
6240 INPUT " "Next cell pressure level "; CPLEV: PRINT: PRINT
6250 INPUT " Number of increments to reach the next cell pressure level"
6260 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <B-CHECK>": PRINT: PRINT: PRINT
6270 GOTO 6300
6280 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <B-CHECK>": PRINT: PRINT: PRINT
6290 PRINT " CPLEV IS LARGER THAN CPLEV+1.0, PLEASE RE-ENTER CPLEV"
6300 PRINT " NEXT CELL PRESSURE LEVEL = "; CPLEV; "MN/m2": PRINT: PRINT
6310 PRINT " NUMBER OF INCREMENTS TO REACH THE NEXT CELL PRESSURE LEVEL"
6320 PRINT "CELL-PRESSURE INCREMENTAL NUMBER = " INCN: PRINT
6330 IF CPLEV<CLEVE0+1.0 THEN GOTO 6280
6340 IF INCN>1 GOTO 6600 'After increment, increments are added automatically, the ball-valve is kept closed
6350 INPUT " Do you want to make changes? <Y/N>:"; BS: PRINT: PRINT
6360 IF INSTR(BS, "Y")<0 OR INSTR(BS, "N")<0 THEN GOTO 6180
6370 LPRINT
6380 LPRINT " **** IN SUBROUTINE <B-CHECK> ****": LPRINT
6390 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <B-CHECK>": PRINT: PRINT: PRINT
6400 PRINT " Please, CLOSE THE BALL VALVE !!!"
6410 FOR I=1 TO 10: BEEP: NEXT I
6420 FOR I=1 TO 2000: I=I+1: NEXT I
6430 LPRINT "THE BALL-VALVE IS CLOSED": LPRINT
6440 FOR I=1 TO 10: BEEP: NEXT I
6450 LPRINT "ACTUAL CELL PRESSURE LEVEL = "; CPLEV; "MN/m2"
6450 LPRINT "NEXT CELL PRESSURE LEVEL = "; CPLEV; "MN/m2"
6470 LPRINT "NUMBER OF INCREMENTS TO REACH THE NEXT CELL PRESSURE LEVEL = "; NINC
6480 LPRINT "IN SUBROUTINE <B-CHECK>"; PRINT; PRINT; PRINT; PRINT; PRINT
6490 ' **** GOSUB <DATAREC> ****
6500 GOSUB 5140
6510 BPO=PP
6520 CPO=CP
6530 ASO=AS
6531 TEMPO=TEMPO
6532 OCTO=(1/3)*(ASO+2*CPO)
6540 FILE$ = "A:B-CHECK.PRN"
6550 OPEN FILE$ FOR APPEND AS 3
6560 WRITE #3,0,OCTO,BPO,CPO,ASO,TEMPO
6570 LPRINT "INITIAL MEASURED BACKPRESSURE = "; BPO; "MN/m2"
6580 LPRINT "INITIAL MEASURED CELL PRESSURE = "; CPO; "MN/m2"
6590 LPRINT "INITIAL MEASURED AXIAL STRESS = "; ASO; "MN/m2"; LPRINT
6600 CLS; LOCATE 5,1,0; BEEP; PRINT "IN SUBROUTINE <B-CHECK>"; PRINT; PRINT; PRINT
6610 PRINT "AN INCREMENT IS ADDED BY KEEPING THE BALL-VALVE CLOSED"; PRINT; PRINT
6620 ' **** GOSUB <PRESCTRL> ****
6630 GOSUB 3610 ' Increase cell pressure by an increment (CPLEV-CPLEV)/NINC
6640 LPRINT "B-CHECK: CELL PRESSURE = "; CPLEV; "MN/m2"
6650 LPRINT "B-CHECK: ACTUAL NUMBER OF INCREMENTS = "; INCR
6660 FOR I=1 TO 8000; NEXT I
6670 JJ=1
6680 ' **** GOSUB <DATAREC> ****
6690 GOSUB 5140
6700 BP(JJ)=PP
6710 CP(JJ)=CP
6720 AS(JJ)=AS
6721 TEMP(JJ)=TEMPO
6730 OCT(JJ)=(1/3)*(AS(JJ)+2*CP(JJ))
6750 BV(JJ)=(BP(JJ)-BPO)/(OCT(JJ)-OCTO)
6751 DTEMPO(JJ)=TEMPO(JJ)-TEMPO
6760 LPRINT JJ; BV(JJ); DTEMPO(JJ); SPS;
6770 FILE$ = "A:B-CHECK.PRN"
6780 OPEN FILE$ FOR APPEND AS 3
6790 WRITE #3, JJ, OCT(JJ), BP(JJ), BV(JJ), TEMP(JJ)
6800 CLOSE #3
6810 JJ=JJ+1
6820 IF JJ=75 GOTO 6820 ELSE GOTO 6690
6830 IF INCR=INCR+1
6840 LPRINT; LPRINT
6850 GOTO 6600
6860 STOP
6870 IF CPLEV=CPMAX GOTO 6960
6880 CPLEV=CPLEV
6890 CPLEV=CPMAX
6900 LPRINT: LPRINT
6910 INCR=1
6920 ZBCP=ZBCP-2 'Adjust the cell pressure from loading to unloading operations
6930 ZBLC=ZBLC-2 'Adjust the zero-byte of the load cell from loading to unloading operations
6940 GOTO 6630
6950 STOP
6960 ON KEY(5) GOSUB 7160 'If swelling can start, press key <F5>
6970 ON KEY(7) GOSUB 7120 'If saturation should be continued, press key <F7>
6980 FOR I=1 TO 10: BEEP: NEXT I
6990 FOR I=1 TO 20: NEXT I
7000 FOR I=1 TO 10: BEEP: NEXT I
7010 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <B-CHECK>": PRINT: PRINT: PRINT
7020 PRINT " You have 20 seconds to start a new program": PRINT: PRINT
7030 PRINT " If specimen is saturated, press key <F5>": PRINT
7040 PRINT " If specimen is not saturated, press key <F7>": PRINT
7050 KEY(5) ON
7060 KEY(7) ON
7070 FOR I=1 TO 10000: NEXT I
7080 KEY(5) OFF
7090 KEY(7) OFF
7100 GOTO 6960
7110 STOP
7120 KEY(7) OFF
7130 NN=0
7140 GOTO 1350 ' Restart saturation from beginning
7150 STOP
7160 KEY(5) OFF
7161 CHAIN "TEST.BAS",10,ALL
7165 STOP
7170 KEY(5) OFF: KEY(8) OFF: KEY (10) OFF
7200 CHAIN "TEST.BAS",10,ALL
7210 END
10 **** PROGRAM <CID> **** / This program controls the drained
20 unloading/loading of a fully saturated rock specimen. At the
30 very beginning you are asked to initialize the measuring devices.
40 The following operations are performed during the testing procedure
50 1/ change of the state of stress applied on the rock specimen,
60 2/ keep this state of stress constant during the consolidation/
70 swelling process, 3/repeat steps 1/ to 2/ as many times as needed.
80 Pressures and deformations are recorded throughout the saturation
90 process.
100 ****
110*******************************************************************************
120*******************************************************************************
130 ****
140*******************************************************************************
150 **** MAIN PROGRAM <CID> -- MP <CID> ****
160*******************************************************************************
170 ****
180 ****
190 LPRINT"*******************************************************************************
200 LPRINT" **** MAIN PROGRAM <CID> -- MP <CID> ****
210 LPRINT"*******************************************************************************
220 ****
240 FOR I=1 TO 20: BEEP: NEXT I
241 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>": PRINT: PRINT: PRINT
250 PRINT "The ball valve IS CLOSED.": PRINT
260 PRINT "OPEN it to perform a drained test.";
270 LPRINT "The ball valve IS CLOSED."
280 LPRINT "OPEN it to perform a drained test."
290 FOR I=1 TO 10000: NEXT I
300 **** INITIALIZATION OF VARIABLES ****
310 ASLEVO=CPMAX 'Initialisation of the axial stress (=cell pressure)
320 ASINC=ASLEVO
330 **** INPUT THE ZEROS OF THE MEASURING DEVICES ****
340 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>": PRINT: PRINT: PRINT
350 PRINT "Input zeros of measuring devices": PRINT: PRINT: PRINT
360 INPUT " ZERO OF CHANNEL 4, in V":ZCH4: PRINT
370 INPUT " ZERO OF CHANNEL 5, in V":ZCH5: PRINT
380 INPUT " ZERO OF CHANNEL 6, in V":ZCH6: PRINT
390 INPUT " ZERO OF CHANNEL 7, in V":ZCH7: PRINT
400 INPUT " ZERO OF CHANNEL 10, in mV":ZCH10: PRINT
410 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>": PRINT: PRINT: PRINT
420 PRINT "Check of the zero-inputs": PRINT: PRINT: PRINT
430 PRINT " ZERO OF CHANNEL 4 =":;ZCH4;"V": PRINT
440 PRINT " ZERO OF CHANNEL 5 =":;ZCH5;"V": PRINT
450 PRINT " ZERO OF CHANNEL 6 =":;ZCH6;"V": PRINT
460 PRINT " ZERO OF CHANNEL 7 =":;ZCH7;"V": PRINT
470 PRINT " ZERO OF CHANNEL 10 =":;ZCH10;"mV": PRINT
480 INPUT "Do you want to make changes? (Y/N)",B$  
490 IF INSTR(B$,"Y")<>0 OR INSTR(B$,"Y")<>0 THEN GOTO 340
500 LPRINT "Check of the zero-inputs"
510 LPRINT "ZERO OF CHANNEL 4 =";ZOFF4;"v"
520 LPRINT "ZERO OF CHANNEL 5 =";ZOCH5;"v"
530 LPRINT "ZERO OF CHANNEL 6 =";ZOCH6;"v"
540 LPRINT "ZERO OF CHANNEL 7 =";ZOCH7;"v"
550 LPRINT "ZERO OF CHANNEL 10 =";ZOCH10;"v"
560 : **** GOTO SUBROUTINE <DATAREC> ****
570 GOSUB 4900 'initialisation data reading
580 : **** GOSUB <SCAN> ****
590 LPRINT "GOSUB <SCAN>="; LPRINT
600 'GOSUB
610 : **** INPUT THE NEXT CELL-PRESSURE LEVEL, THE NEXT AXIAL STRESS LEVEL AND
THE NUMBER OF INCREMENTS TO REACH THEM ****
620 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>="; PRINT: PRINT: PRINT
630 ON KEY(1) GOSUB 3640 ' <F1> controls refilling of piston chamber
640 ON KEY(2) GOSUB 860 ' <F2> controls the next inputs of CPLEV and NINCR
650 KEY (1) ON:KEY (2) ON
660 CLS:LOCATE 5,1,0: BEEP:BEEP
670 PRINT " Press <F1> to refill the piston chamber, after having:"
680 PRINT "(1) record the pressure in the piston chamber with "
690 PRINT " the voltmeter reading"
700 PRINT "(2) closed the valve to the cell chamber,"
710 PRINT "(3) opened the valve to the to the oil tank": BEEP : BEEP
720 PRINT: PRINT
730 PRINT " Press <F2> to continue "
740 FOR I=1 TO 20000: NEXT
750 GOTO 660
760 STOP
770 KEY(4) OFF 'Confining pressure is adjusted after refilling of the piston
780 XO=ZBCP+INT(162.5040628*CPINCR)
790 XO-X=INT(XO/16): XO%=16*(XO-XO%X*16)
800 OUT 832,XO%: OUT 833,XO%
810 GOTO 900
820 STOP
830 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>="; PRINT: PRINT: PRINT
840 PRINT" CPLEV IS SMALLER THAN PP, PLEASE RE-ENTER CPLEV."; PRINT: PRI
850 GOTO 920
860 KEY(2) OFF: KEY(1) OFF
870 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>="; PRINT: PRINT: PRINT
880 INPUT "Do you want to adjust the cell pressure reading and/or the axial stre
ss reading? (Y/N)"; BS
890 IF INSTR(BS,"y")>0 OR INSTR(BS,"y")>0 THEN GOSUB 4080
900 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CIU>="; PRINT: PRINT: PRINT
910 PRINT " ACTUAL CELL PRESSURE LEVEL =";CPINCR;"MN/m2": PRINT: PRINT
920 INPUT " Next cell-pressure level, in MN/m2 ";CPLEV; PRINT
930 PRINT " ACTUAL AXIAL STRESS LEVEL =";ASINCR;"MN/m2": PRINT: PRINT
940 INPUT " Next axial stress level, in MN/m2 ";ASLEV; PRINT
950 INPUT " Number of increments to reach the next cell-pressure/axial st
ess level":NINCR: PRINT
960 IF CPLEV<PP+.01 THEN GOTO 830
970 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>="; PRINT: PRINT: PRINT
980 PRINT" NEXT CELL-PRESSURE LEVEL = ";CPLEV;"MN/m2": PRINT: PRINT
990 PRINT" NEXT AXIAL STRESS LEVEL = ";ASLEV;"MN/m2": PRINT: PRINT
1600 PRINT "NUMBER OF INCREMENTS TO REACH THE NEXT CELL-PRESSURE LEVEL = ":;INCNR: PRINT: PRINT
1610 INPUT "Do you want to make changes? (Y/N)",:$B$
1620 IF INSTR($B$,$Y$)=0 OR INSTR($B$,$y$)=0 THEN GOTO 900
1630 PRINT: PRINT
1640 LPRINT "NEXT CELL-PRESSURE LEVEL = ";CPELV:"MN/m2"
1650 LPRINT "NEXT AXIAL STRESS LEVEL = ";ASLEV:"MN/m2"
1660 LPRINT "NUMBER OF INCREMENTS TO REACH THE NEXT CELL-PRESSURE LEVEL = ";INCNR
1670 ' The basic loop used to control the testing procedure between two cell
1680 ' pressure levels involves the following subroutines: <PRESCTRL> and
1690 ' <SCAN>. In subroutine <SCAN>, the pointer goes to subroutine
1700 ' <DATAREC> in order to record the vertical deformation too, so
1710 ' that the tensor of deformations can be entirely measured.
1720 ' **** GOTO SUBROUTINE <PRESCTRL> ****
1730 1120 GOTO 3330
1740 1130 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CIU>": PRINT: PRINT: PRINT
1750 1140 PRINT " The consolidation is under process.": LPRINT
1760 1150 PRINT " Everything is computer-controlled.": PRINT
1770 1160 PRINT " If you want to adjust the cell pressure or the axial stress,"
1780 1170 PRINT " press the key <F10>."
1790 1180 PRINT " When you want to proceed to a next stress state level, "
1800 1190 PRINT " press the key <F2>."
1810 1200 ON KEY(2) GOSUB 1340
1820 1210 ON KEY(10) GOSUB 4060
1830 1220 KEY(2) ON
1840 1230 KEY(10) ON
1850 1239 I=1
1860 1240 IF I=3 GOTO 1280
1870 1250 CONS=II
1880 1260 GOTO 4900 'GOSUB <DATAREC> for printing data on screen only
1890 1270 I=II+1
1900 1271 GOTO 1240
1910 1272 STOP
1920 1280 KEY(2) OFF: KEY(10) OFF
1930 1285 CONS=01
1940 1290 ' **** GOSUB <DATAREC> ****
1950 1300 GOSUB 4900
1960 1310 ' **** GOSUB <SCAN> ****
1970 1320 LPRINT "GOSUB <SCAN>"
1980 'GOSUB
1990 1330 GOTO 1130
2000 1331 STOP
2010 1340 KEY(2) OFF: KEY(10) OFF
2020 1350 CONS=01
2030 1360 GOTO 620
2040 1370 END
2050 1380 ' ********************************************
2060 1390 ' **** SUBROUTINE <SCAN> ****
2070 ' ********************************************
2080 1410 ADDRESS$=768
2090 1420 GOSUB 2800:J$="**
2100 1430 FOR I=1 TO 2000: NEXT
1440 P=0
1450  **** INPUT DATA FOR CONTINUOUS AND SCAN DISTANCES ****
1460 Z(1)=1080000
1470 L(1)=20
1480 Z(2)=1500000
1490 L(2)=20
1500 Z(3)=1500000
1510 L(3)=20
1520 Z(4)=1500000
1530 L(4)=20
1540 Z(5)=1450000
1550 L(5)=20
1560 ON KEY(6) GOSUB 1720
1570 PRINT "This are the input data for traveling, Z(i), and"
1580 PRINT "scanning, L(i), distances (in microsteps) from the"
1590 PRINT "bottom (1) to the top (5)."
1600 PRINT: PRINT:
1610 PRINT "Z(1)=",Z(1)," L(1)=",L(1)
1620 PRINT "Z(2)=",Z(2)," L(2)=",L(2)
1630 PRINT "Z(3)=",Z(3)," L(3)=",L(3)
1640 PRINT "Z(4)=",Z(4)," L(4)=",L(4)
1650 PRINT "Z(5)=",Z(5)," L(5)=",L(5): PRINT: PRINT
1660 PRINT "Do you want to change the traveling and scanning lengths?"
1670 PRINT "Which are above? You have 10 seconds to press the key <F6>."
1680 KEY(6) ON
1690 FOR I=1 TO 5000: NEXT I
1700 KEY(6) OFF
1710 GOTO 1950
1720 KEY(6) OFF
1730 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
1740 PRINT "INPUT THE NEW DATA": PRINT: PRINT: PRINT
1750 INPUT "Z(1)=",Z(1)
1760 INPUT "L(1)=",L(1)
1770 INPUT "Z(2)=",Z(2)
1780 INPUT "L(2)=",L(2)
1790 INPUT "Z(3)=",Z(3)
1800 INPUT "L(3)=",L(3)
1810 INPUT "Z(4)=",Z(4)
1820 INPUT "L(4)=",L(4)
1830 INPUT "Z(5)=",Z(5)
1840 INPUT "L(5)=",L(5): PRINT: PRINT
1850 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
1860 PRINT "CHECK NEW INPUT DATA": PRINT: PRINT: PRINT: PRINT
1870 PRINT "Z(1)=",Z(1)," L(1)=",L(1)
1880 PRINT "Z(2)=",Z(2)," L(2)=",L(2)
1890 PRINT "Z(3)=",Z(3)," L(3)=",L(3)
1900 PRINT "Z(4)=",Z(4)," L(4)=",L(4)
1910 PRINT "Z(5)=",Z(5)," L(5)=",L(5): PRINT: PRINT
1920 INPUT "Do you want to make changes? <Y/N> ":,BS
1930 IF INSTR(BS, "Y")<>0 OR INSTR(BS, "y")<>0 THEN GOTO 1730 ELSE 1950
1940  **** ENTER FIXED DATA FOR UPWARDS TRAVEL ****
1950 VS=2.0  ' VS is the speed during scan move
1960 VCS="0.5"  ' VCS is the speed during continuous move
1970 DS="2500"  ' DS is the distance between two scan-positions (microsteps)
1980 AS="2.0"  ' AS is the acceleration for all moves (continuous & scan)
1990 CMDAS=" ST1 X1 MN A"+AS+" "
2000 CMDBS=""
2010 FOR I=1 TO 5
2020 Z=Z(I):ZS=STRS(Z):ZS=RIGHTS(ZS,6)
2030 L=L(I):LS=STRS(L):LS=RIGHTS(LS,2)
2040 CMDBS= CMDBS + "M"+VCS+" D"+ZS+" X1 G L"+LS+" V"+VS+" D"+DS+" G TR1XX X1 N "
2050 NEXT I
2060 CMD= CMDAS + CMDBS
2070 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
2080 PRINT " COMMAND SEND TO MOTOR FOR UPWARDS TRAVEL, CMD$ "
2090 LPRINT " COMMAND SEND TO MOTOR FOR UPWARDS TRAVEL, CMD$ 
2100 PRINT CMD$;
2110 LPRINT CMD$;
2120 R1=1  ' Variable attached to CMD$ during upwards move
2130 GOTO 2510
2140 CMD="
2150 ' **** GOTO SUBROUTINE <DATAREC> ****
2160 PRINT " GOTO SUBROUTINE <DATAREC>": PRINT
2170 GOSUB 4900
2180 PRINT " BACK FROM SUBROUTINE <DATAREC>": PRINT
2190 ' **** ENTER FIXED DATA FOR DOWNWARDS TRAVEL ****
2200 DS="-2500"  ' DS is the distance between two scan-positions (microsteps)
2210 VS="2.0"  ' VS is the speed during scan move
2220 VCS="0.5"  ' VCS is the speed during continuous move
2230 AS="2.0"  ' AS is the acceleration for all moves (continuous & scan)
2240 CMDAS=" MN A"+AS+" "
2250 CMDBS=""
2260 FOR I=1 TO 5
2270 L(I)=L(6-I)
2280 Z(I)=Z(6-I)
2290 DATA 20,-145000,20,-150000,20,-150000,20,-150000,20,-108000
2300 L=L(I):LS=STRS(L):LS=RIGHTS(LS,2)
2310 Z=Z(I):ZS=STRS(Z):ZS=RIGHTS(ZS,7)
2320 PRINT LS,ZS
2330 PRINT 92,LS,ZS
2340 CMDBS= CMDBS + "M"+L$+" V"+VS+" D"+DS+" G TR1XX X1 N V"+VCS+"D"+ZS+" X1 G "
2350 NEXT I
2360 CMD= CMDAS + CMDBS
2370 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>": PRINT: PRINT: PRINT
2380 PRINT " COMMAND SEND TO MOTOR FOR DOWNWARDS TRAVEL, CMD$": PRINT: PRINT: PRINT
2390 LPRINT " COMMAND SEND TO MOTOR FOR DOWNWARDS TRAVEL, CMD$ 
2400 PRINT CMD$;
2410 LPRINT CMD$;
2420 R1=2  ' Variable attached to CMD$ during downwards move
2430 GOTO 2510
2440 CMD$=""n
2450 ' Stop microstepping motor and send signal of end of travel "CR"
2460 PRINT " COMMAND SEND TO MOTOR TO STOP AND DISCONNECT"
2470 LPRINT " COMMAND SEND TO MOTOR TO STOP AND DISCONNECT"
2480 CMD$=""n STO CR "
2490 PRINT CMD$;
2500 LPRINT CMD$;
2510 GOSUB 2520:GOSUB 2870:GOTO 1440
2520 AL$=AS$;VL$=VS$;HL$=HS$;DL$=DS$
2530 IF RIGHTS(CMD$,1)<>CHR$(13) THEN CMD$=CMD$+CHR$(13)
2540 FOR K=1 TO LEN(CMD$) ' Enter routine here with PC21 command=CMD$ X$=KIDX(CMD$,K,1)
2550 NEXT K
2560 GOSUB 2720
2570 NEXT K
2580 X$=CHR$(13):GOSUB 2720:JS="":GOSUB 2600
2590 IF RIGHTS(CMD$,1)<>CHR$(13) THEN CMD$=CMD$+CHR$(13)
2600 B=INP(ADDRESS$+1)
2610 IF B AND &H8 THEN GOTO 2620 ELSE RETURN
2620 J=INP(ADDRESS$)
2630 OUT ADDRESS$+1,&H0
2640 B=INP(ADDRESS$+1)
2650 IF B AND &H8 THEN GOTO 2640 ELSE GOTO 2660
2660 OUT ADDRESS$+1,&H60
2670 X$=CHR$(J)
2680 JS=JS+X$  
2690 IF J=&H6 THEN GOTO 2700 ELSE GOTO 2600
2700 JS=LEFT$(JS,9)
2710 IF JS=""m" THEN GOTO 2600
2720 B=INP(ADDRESS$+1)
2730 IF B AND &H10 GOTO 2740 ELSE GOTO 2720
2740 OUT ADDRESS$,ASC(X$)
2750 OUT ADDRESS$+1,&H70
2760 B=INP(ADDRESS$+1)
2770 IF B AND &H10 GOTO 2760 ELSE GOTO 2780
2780 OUT ADDRESS$+1,&H60
2790 RETURN
2800 OUT ADDRESS$+1,&H64
2810 OUT ADDRESS$+1,&H60
2820 FOR I=1 TO 100:NEXT
2830 OUT ADDRESS$+1,&H40
2840 OUT ADDRESS$+1,&H60
2850 FOR I=1 TO 100:NEXT
2860 RETURN
2870 CMD$=""n";GOSUB 2530:IF(INSTR(J$,"**")<>0)+(INSTR(J$,"***")<>0) THEN RETURN
2880 CLS: LOCATE 5,1,0: BEEP: PRINT "IN SUBROUTINE <SCAN>_": PRINT: PRINT: PRINT
2890 ' **** READ FROM DATA ACQUISITION SYSTEM ****
2900 CLOSE #1
2910 OPEN "COM1:2400,W,8,1,CS,DS,CD* AS #1
2920 ON ERROR GOTO 3250
2930 IF LOC(1)<199 THEN GOTO 2930
2940 GS=INPUT$(LOC(1),#1)
2950 FOR I=1 TO 198
2960 FS=MIDS(GS,1,2)
2970 IF (FS=CHR$(13)+CHR$(10)) THEN GOTO 3000
2980 IF I=198 GOTO 2900
2990 NEXT
3000 H=199-I:E=I+2
3010 GS=MIDS(GS,E,H)+LEFTS(GS,1)
3020 GGS=MIDS(GS,2,13)+MIDS(GS,48,8)+MIDS(GS,63,8)+MIDS(GS,153,8)
3030 AAS=MIDS(GS,2,13)
3040 BBS=MIDS(GS,48,8):BB=VAL(BBS)
3050 CCS=MIDS(GS,63,8):CC=VAL(CCS)
3060 CPS=MIDS(GS,78,8):CP=VAL(CPS):CP=ABS(CP)
3070 DDS=MIDS(GS,153,8):DD=VAL(DDS)
3080 SS=CHR$(13)+CHR$(10)
3090 JJ=VAL(JS)
3100 FILES = "A:TARGET.PRN"
3110 OPEN FILES FOR APPEND AS 2
3120 WRITE #2,AAS,CP,BB,CC,DD,JJ
3130 PRINT GGS,JS,SS;
3140 FOR I=1 TO 12
3150 KS=MIDS(JS,1,2)
3160 IF(KS=CHR$(42)+CHR$(82)) THEN GOTO 3260 ' Looks for "RN"
3170 IF(KS=CHR$(67)+CHR$(82)) THEN GOTO 3280 ' Looks for "CR"
3180 NEXT I
3190 JS=""
3200 FOR I=1 TO 10:OUT 890,14
3210 NEXT
3220 FOR I=1 TO 10:OUT 890,0
3230 NEXT
3240 GOTO 2870
3250 RESUME : GOTO 2910
3260 IF R1=1 THEN GOTO 2140 ' Gets the data for downwards move
3270 IF R1=2 THEN GOTO 2440 ' Gets the data for stopping motor
3280 CLOSE #2: PRINT "file TARGET.PRN cloned"
3290 ON ERROR GOTO 0: RETURN
3300 '******************************************************************************
3310 ' **** SUBROUTINE <PRESCTRL> ****
3320 '******************************************************************************
3330 L=1 'Variable controlling the data reading of the DAC-system:
3340 ' if L=1, data are trashed; if L=2, data are recorded.
3350 FOR INCR=1 TO NINC
3360 CPINC=(CPEV-CPEVO)/NINC
3370 CPINC=CPEVO+INCR*CPINC
3380 ASINCR=(ASLEV-ASLEVO)/NINC
3390 ASINCR=ASLEVO+INCR*ASINCR
3400 ' **** COMMANDS SEND TO DAC-02, CHANNEL #0, Cell pressure ****
3410 X0=2BCP+INT(162.5040628*MCPINC)
3420 XO%=INT(X0/16): XO%=16*(X0-XO%)%16
3430 OUT 852,XO%: OUT 833,XO%
3440 ' **** COMPUTE NEEDED AXIAL FORCE TO BE APPLIED BY THE PISTON ****
3450 AFCOMP=5.51804*ASINCR-.991789*CPINC
3460 ' **** COMMANDS SEND TO DAC-02, CHANNEL #1, Axial stress ****
3470 X1=ZBLC+INT(146.9833158*MFCOMP)
3480 X1%=INT(X1/16): X1%=16*(X1-X1%)%16
PRINT "CONFINING PRESSURE=";CPINCR;"MN/m2"
3510 PRINT "A XIAL STRESS=";ASINCR;"MN/m2"
3520 FOR I=1 TO 5000:NEXT I
3530 CLS: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>"": PRINT: PRINT: PRINT
3540 PRINT "Please wait I The required state of stress is being adjusted.": PRI
3550 NT: PRINT
3550 PRINT "CONFINING PRESSURE=";CPINCR;"MN/m2"
3560 PRINT "A XIAL STRESS=";ASINCR;"MN/m2"
3570 H=0
3580 FOR 1=1 TO 2: H=H+1: NEXT 1
3580 GOTO SubxOL: E=""<DATAREC>"", with partial results and screen printing only
3590 GOTO 4240
3600 FOR I=1 TO 5000: NEXT I
3610 GOTO 1130
3620 STOP
3630 FOR 1=1 TO 3: GOSUB 3850: NEXT I
3630 F3 controls filling operations
3700 KEY (3) ON
3710 PRINT "Refilling the piston chamber."": PRINT
3720 PRINT "When ready, perform the following steps:"": PRINT
3730 PRINT "(1) unplug the motor from the controller"
3740 PRINT "(2) close the valve to the oil tank"
3750 PRINT "(3) unplug the pressure transducer of the triaxial cell"
3760 PRINT "(4) plug-in by pass wire to the controller"
3770 PRINT "(5) press key <F3>
3780 PRINT "(6) plug-in the motor to the controller"
3790 PRINT "(7) bring the pressure in the piston chamber up to the pressure"
3800 PRINT "previously recorded by the voltmeter reading"
3810 PRINT: PRINT: BEEP: BEEP
3820 FOR I=1 TO 20000: NEXT I
3830 GOTO 3710
3840 STOP
3850 KEY (3) OFF
3860 XO= 2070 'Incr. press. in piston which corresponds to CP
3870 XOH%=INT(XO/16); XOL%=16*(XO-XOH%)*16
3880 OUT 832,XOLX: OUT 833,XOHA
3890 ON KEY (4) GOSUB 770 'F4>, controls the inputs of CPLEV and new NINCR
3900 KEY (4) ON
3910 PRINT: PRINT
3920 PRINT "When the pressure in the piston has reached the cell pressure,"
3930 PRINT "perform the following steps:""
3940 PRINT "(1) unplug the motor from the controller"
3950 PRINT "(2) unplug the by-pass wire cable from the controller"
3960 PRINT "(3) plug-in the pressure transducer of the triaxial cell"
3970 PRINT "to the controller"
3980 PRINT "(4) open the valve of the cell chamber"
3990 PRINT " (5) press key <F4>"
4000 PRINT " (6) plug-in the motor to the controller"
4010 FOR I=1 TO 10000: NEXT I
4020 PRINT " Increasing the pressure in the piston chamber ": PRINT
4030 GOTO 3920
4040 STOP
4050 ' **** CHANGE ZERO-BYTES OF THE PRESSURE CONTROLLER AND OF THE AXIAL STRES
S CONTROLLER ****
4060 CONS=01
4070 KEY(10) OFF
4080 CLS:IFFLAG=0: LOCATE 5,1,0: BEEP: PRINT "IN MP <CID>"; PRINT: PRINT: PRINT
4090 PRINT " THE DEFAULT VALUE OF THE ZERO-BYTE OF THE CELL PRESSURE CONTROLLER
=":ZBCP
4100 PRINT: PRINT
4110 INPUT " New value of the zero-byte of the cell pressure controller";ZBCP: P
RINT
4120 PRINT " THE NEW DEFAULT VALUE OF THE ZERO-BYTE OF THE CELL PRESSURE CONTROL
LER =":ZBCP
4130 PRINT: PRINT
4140 PRINT " THE DEFAULT VALUE OF THE ZERO-BYTE OF THE AXIAL STRESS CONTROLLER = 
";ZBLC
4150 PRINT: PRINT
4160 INPUT " New value of the zero-byte of the axial stress controller";ZBLC: PR
INT
4170 PRINT " THE NEW DEFAULT VALUE OF THE ZERO-BYTE OF THE AXIAL STRESS CONTROLL
ER =":ZBLC
4180 PRINT: PRINT: PRINT
4190 INPUT " Do you want to make changes? (Y/N)";B$ 
4200 IF INSTR(B$,"Y")<>0 OR INSTR(B$,"y")<>0 THEN GOTO 4090
4210 RETURN
4220 STOP
4230 ' **** READ DATA FROM DATA-ACQUISITION SYSTEM ****
4240 FOR I=1 TO 20000: NEXT I
4250 RESET
4260 OPEN "COM1:2400,N,8,1,CS,DS,CD" AS #1
4270 ON ERROR GOTO 4810
4280 IF LOC(1)<194 THEN GOTO 4280
4290 GS=INPUT$(LOC(1),#1)
4300 IF L<>2 GOTO 4850
4310 ' **** LOOK FOR BEGINNING OF STRING ****
4320 FOR J=1 TO 193
4330 FS=MID$(GS,J,2)
4340 IF (FS=CHR$(13)+CHR$(10)) THEN 4380
4350 IF J=193 GOTO 4840
4360 NEXT J
4370 GOTO 4320
4380 H=194-J: E=J+2
4390 ' **** WRITE STRING IN RIGHT ORDER ****
4400 GS=MID$(GS,E,H)+LEFT$(GS,J)
4410 ' **** READ ALL INPUT VOLTAGES ****
4420 VLC$=Reading of input voltage of load cell, 12.0 VDC
4430 VLC$=MID$(GS,168,8): VLC=VAL(VLC$): VLC=ABS(VLC)
4440 ' V$=Reading of input voltage (pressure transducers, DCDT), 5.5 VDC
VS=MIDS(GS,183,B);V=VAL(VB);V=ABS(V)
4460 ' **** CHECK IF ALL INPUT VOLTAGES ARE OK ****
4470 IF (V<5.45 OR V>5.55) THEN GOTO 4820
4480 IF (VLC<9.95 OR VLC>10.05) THEN GOTO 4830
4490 ' **** TIME READING ****
4500 T$=Reading time / day:hours:minutes:seconds
4510 TS=MIDS(GS,2,12)
4520 'Time reading in seconds
4530 TS=86400*VAL(MIDS(TS,1,2))+3600*VAL(MIDS(TS,4,2))+60*VAL(MIDS(TS,7,2))+VAL
   (MIDS(TS,10,2))
4540 ' **** CELL PRESSURE ****
4550 'CHB$=Reading Channel #8, PT #CA7257, CF=0.06293135 (KN/m2)/eV
4560 CHB$=MIDS(GS,78,B);CHB=VAL(CHB$);CHB=ABS(CHB)
4570 CP=(.3461224*(CHB-ZOCBH))/V-.0026
4580 ' **** AXIAL FORCE IN LOADING PISTON ****
4590 'CH12S=Reading Channel #12, CF=3.71470 (KN/m2)
4600 CH12S=MIDS(GS,138,B);CH12=VAL(CH12S)
4610 AF=37.147*(CH12-ZOC12)/VLC
4620 ' **** AXIAL STRESS ON ROCK SPECIMEN ****
4630 AS=AF/5.51804+.99178*CP
4640 ' **** PRINTING OF DATA ****
4650 CLS: LOCATE 5,1,0: BEEP: PRINT "IN <PRESCTRL>": PRINT: PRINT: PRINT
4660 PRINT "***********": PRINT
4670 PRINT "Time":TS: PRINT
4680 PRINT "Increment # *******
4690 PRINT "Maximum number of increments *******
4700 PRINT "Cell pressure (wanted/measured) *******
4710 PRINT "Axial stress (wanted/measured) *******
4720 PRINT "Input voltage -- 5.5 VDC *******
4730 PRINT "Input voltage -- 12.0 VDC *******
4740 PRINT "***********": PRINT
4750 CLOSE #1
4760 NEXT INCR
4770 CPLEVO=CPINC
4780 ASEVO=ASINC
4790 GOTO 3600
4800 'If ERROR occurs then reboot program automatically
4810 RESUME: GOTO 4260
4820 BEEP:PRINT "Check input voltage 5.5 VDC": GOTO 4840
4830 BEEP:PRINT "Check input voltage 12.0 VDC"
4840 CLOSE #1: GOTO 4260
4850 CLOSE #1: L=2: GOTO 4260
4860 STOP
4870 ' ****************************
4880 ' **** SUBROUTINE <DATAREC> ****
4890 ' ****************************
4900 L=1 'Variable controlling the data reading of the DAC-system:
4910 ' if L=1, data are trashed; if L=2, data are recorded.
4920 RESET
4930 OPEN ^COM1:2400,N,B,1,CS,DS,CD# AS #1
4940 ON ERROR GOTO 5810
4950 IF LOC(1)<194 THEN GOTO 4950
4960 GS=INPUTS(LOC(1),#1)
4970 IF L<>2 GOTO 5860
4980 FOR J=1 TO 193
4990 FS=MIDS(GS,J,2)
5000 IF (FS=CHR$(13)+CHR$(10)) THEN 5040
5010 IF J=193 GOTO 5850
5020 NEXT J
5030 GOTO 4980
5040 H=194-J:J=E+J+2
5050 1 **** WRITE STRING IN RIGHT ORDER ****
5060 GS=MIDS(GS,E,H)+LEFT$(GS,J)
5070 1 **** READ ALL INPUT VOLTAGES ****
5080 'VPS=Reading of input voltage of proximity sensors, 15.0 VDC
5090 VPS=NIDS(GS,153,8):VPS=VAL(VPS):VPS=ABS(VPS)
5100 'VLCS=Reading of input voltage of load cell, 12.0 VDC
5110 VLCS=NIDS(GS,168,8):VLCS=VAL(VLCS):VLCS=ABS(VLCS)
5120 'VS=Reading of input voltage (pressure transducers, DCDT), 5.5 VDC
5130 VS=NIDS(GS,183,8):VS=VAL(VS):VS=ABS(VS)
5140 1 **** CHECK IF ALL INPUT VOLTAGES ARE OK ****
5150 IF (VS<5.45 OR VS>5.55) THEN GOTO 5820
5160 IF (VLCS<9.95 OR VLCS>10.05) THEN GOTO 5830
5170 IF (VPS<29.95 OR VPS>30.05) THEN GOTO 5840
5180 1 **** TIME READING ****
5190 'TS=Reading time / day:hours:minutes:seconds
5200 TS=MIDS(GS,2,12)
5210 1 'TS=Time reading in seconds
5220 TS=86400*VAL(MIDS(TS,1,2))+3600*VAL(MIDS(TS,4,2))+60*VAL(MIDS(TS,7,2))+VAL(MIDS(TS,10,2))
5230 1 **** VOLUME CHANGE OF WATER ****
5240 'CH4S=Reading Channel #4, CF for Volume Change = 3.246488 (CH5/V*Vinp)
5250 CH4S=NIDS(GS,18,8):CH4S=VAL(CH4S)
5260 VOL.CH=3.246488*V*CH4-2*CH4)
5270 1 **** VERTICAL DISPLACEMENT ****
5280 'CH5S=Reading Channel #5, CF for vertical displ. = 0.0686531 (CH4/V*Vinp)
5290 CH5S=NIDS(GS,33,8):CH5S=VAL(CH5S):CH5S=ABS(CH5S)
5300 VERT.DISPL.=-0.0686531*V*(CH5-2*CH5S)
5310 1 **** HORIZONTAL DISPLACEMENTS (TWO) ****
5320 'CH6S=Reading Channel #6, CF for PS(1) = 514.04785 (um/V)
5330 CH6S=NIDS(GS,48,8):CH6S=VAL(CH6S):CH6S=ABS(CH6S)
5340 HORIZ.DISPL.1=514.04799*CH6-2*CH6S)
5350 'CH7S=Reading Channel #7, CF for PS(2) = 514.74450 (um/V)
5360 CH7S=NIDS(GS,63,8):CH7S=VAL(CH7S):CH7S=ABS(CH7S)
5370 HORIZ.DISPL.2=514.7445*CH7-2*CH7S)
5380 SUMDISPL=HORIZ.DISPL.1+HORIZ.DISPL.2
5390 1 **** CELL PRESSURE ****
5400 'CH8S=Reading Channel #8, PT #C47257, CF=0.06293135 ((mH/m2)/mV)
5410 CH8S=NIDS(GS,78,8):CH8S=VAL(CH8S):CH8S=ABS(CH8S)
5420 CP=0.3461224*(CH8-2*CH8S)/V)-.0026
5430 1 **** PORE PRESSURE ****
5440 'CH10S=Reading Channel #10, PT #C03154, CF=.012724 ((mH/m2)/mV)
5450 CH10S=NIDS(GS,108,8):CH10S=VAL(CH10S):CH10S=ABS(CH10S)
5460 PP=0.069982*(CH10-2*CH10S)/V)-.0047
5470  * **** TEMPERATURE ****
5480  *Ch11=Reading Channel #11, Equation: Temp.=51.30565*log(mV)+108.846
5490  CH11=NID$(GS,123,8):CH11=VAL(CH11):CH11=ABS(CH11)*5.5/V
5500  CH11=LOG(CH11)^2.305285
5510  TEMP=(-51.30565*CH11)+108.846
5520  * **** AXIAL FORCE IN LOADING PISTON ****
5530  CH12=Reading Channel #12, CF=3.51747 (KN/mV)
5540  CH12=NID$(GS,135,8):CH12=VAL(CH12)
5550  AF=37.147*(CH12+20*CH12)/VLC
5560  * **** AXIAL STRESS ON ROCK SPECIMEN ****
5570  AS=AF/5.518044+.991789*CP
5574  ON KEY(1) GOSUB 3640  '<F1> to refill piston
5575  ON KEY (10) GOSUB 4090  '<F10> to re-zero CP & AS
5580  IF IFLAG=1 GOTO 5732
5585  IFLAG=1:CLS:KEY OFF:LOCATE 1,1,0;PRINT "IN <DATAENC>"
5587  KEY 1,"REFILL":KEY 2,"=":KEY 3,"=":KEY 4,"=":KEY 5,"=":KEY 6,"=":KEY 7,"=":KEY 8,
5588  "=":KEY 9,"=":KEY 10,"RE-ZERO":KEY ON
5590  FOR IPX=1 TO 80:LOCATE 2,IPX,0;PRINT CHR$(220):NEXT IPX
5600  LOCATE 5,1,0;PRINT " Time ............................ "
5610  LOCATE 4,1,0;PRINT " Volume change .........................."
5620  LOCATE 5,1,0;PRINT " Vertical displacement ............."
5630  LOCATE 6,1,0;PRINT " Horizontal displacement no.1 .."
5640  LOCATE 7,1,0;PRINT " Horizontal displacement no.2 .."
5650  LOCATE 8,1,0;PRINT " Sum of horizontal displacements ."
5660  LOCATE 9,1,0;PRINT " Cell pressure (wanted/measured) ."
5670  LOCATE 10,1,0;PRINT " Core Pressure ....... ........."
5680  LOCATE 11,1,0;PRINT " Temperature ................. ......
5690  LOCATE 12,1,0;PRINT " Axial stress (wanted/measured) ."
5699  LOCATE 14,1,0;PRINT " Input voltage -- 5.5 VDC ........."
5700  LOCATE 15,1,0;PRINT " Input voltage -- 12.0 VDC ........
5720  LOCATE 16,1,0;PRINT " Input voltage -- +/-15.0 VDC ....
5730  FOR IPX=1 TO 80:LOCATE 17,IPX,0;PRINT CHR$(220):NEXT IPX
5732  LOCATE 3,38,0;PRINT T8,":days:hrs:min:sec"
5733  LOCATE 4,38,0;PRINT VOL.CH,"cm3"
5734  LOCATE 5,38,0;PRINT VERT.DISPL,",cm"
5735  LOCATE 6,38,0;PRINT HORIZ.DISPL.1,"microns"
5736  LOCATE 7,38,0;PRINT HORIZ.DISPL.2,"microns"
5737  LOCATE 8,38,0;PRINT SUMDISPL,"microns"
5738  LOCATE 9,38,0;PRINT CPINCR,"/":CP,"Mpa"
5739  LOCATE 10,38,0;PRINT PP,"Mpa"
5740  LOCATE 11,38,0;PRINT TEMP,CHR$(248),"C"
5741  LOCATE 12,38,0;PRINT ASINCR,"/":AS,"Mpa"
5742  LOCATE 14,38,0;PRINT V,"VDC"
5743  LOCATE 13,38,0;PRINT VLC,"VDC"
5744  LOCATE 16,38,0;PRINT VPS,"VDC":LOCATE 18,1,0
5748  IF CONS=11 GOTO 1270
5750  FILES = "A:CIDDATA.PRN"
5760  OPEN FILES FOR APPEND AS 3
5770  WRITE #3,TS,VOL.CH,VERT.DISPL,HORIZ.DISPL.1,HORIZ.DISPL.2,SUMDISPL,CP,PP,TE
5780  MP,AS
5780  CLOSE #1,#3
5790  ON ERROR GOTO 0: RETURN
5800 'If ERROR occurs then reboot program automatically
5810 RESUME: GOTO 4930
5820 BEEP:LOCATE 17,1,0:PRINT "Check input voltage 5.5 VDC":GOTO 5850
5830 BEEP:LOCATE 17,1,0:PRINT "Check input voltage 12.0 VDC":GOTO 5850
5840 BEEP:LOCATE 17,1,0:PRINT "Check input voltage 15.0 VDC"
5850 IPFLAG=0:CLOSE #1: GOTO 4930
5860 CLOSE #1: L=2: GOTO 4930
5870 END