Analysis of surface subsidence in crystalline rocks above the Gotthard highway tunnel, Switzerland

Author(s):
Zangerl, Christian Josef

Publication Date:
2003

Permanent Link:
https://doi.org/10.3929/ethz-a-004613973

Rights / License:
In Copyright - Non-Commercial Use Permitted
Analysis of Surface Subsidence in Crystalline Rocks above the Gotthard Highway Tunnel, Switzerland

A dissertation submitted to the
SWISS FEDERAL INSTITUTE OF TECHNOLOGY (ETH)
ZÜRICH
for the degree of
DOCTOR OF NATURAL SCIENCE

presented by

CHRISTIAN JOSEF ZANGERL

Magister der Naturwissenschaften
University of Innsbruck
born 11. August 1969
Austrian citizen

accepted on the recommendation of
Prof. Dr. Simon Löw, examiner
Dr. Erik Eberhardt, co-examiner
Dr. Giovanni Lombardi, co-examiner
2003
Acknowledgements

I would like to thank my supervisor Prof. Simon Löw from the ETH Zürich for giving me the opportunity to develop such a diverse, challenging and interesting thesis project. His continuous support and encouragement from the beginning and the numerous discussions helped to guide this thesis to its successful completion.

I would also like to express my gratitude to my co-supervisor Dr. Erik Eberhardt from ETH Zürich for his support and guidance. Especially his great encouragement for the numerical modelling portions and great support in writing this thesis are acknowledged.

I am also thankful to my thesis external examiner Dr. Giovanni Lombardi (Lombardi SA, Ingegneri Consulenti) for his input and the numerous discussions to further clarify the work in this thesis.

Many additional people were instrumental in the development and completion of this thesis: Special thanks to Dr. Keith Evans (ETH Zürich) for our many insightful discussions on poroelastic rock mechanics. I would also like to acknowledge Dr. Martin Brändli (WSL Birmensdorf) who helped me with the GIS system and his accessibility to answer emergency questions.

Dr. Richard Tessardi (University of Innsbruck) performed mineralogical analyses and Dr. Achim Kamelger (University of Innsbruck) supported me with several PC’s to permit several numerical simulations in parallel. Prof. Wulf Schubert (TU Graz) provided me access to the triaxial testing equipment in his laboratory, where a collaboration was formed with Dr. Manfred Blämel who performed these tests. Prof. Helmut Rott and Dr. Thomas Nagler (University of Innsbruck) analysed the subsidence problem by application of satellite radar interferometry. All are thanked for their kind help.

Furthermore, I am thankful to Auke Barnhoorn (ETH Zürich) who instructed me on using the SEM and to Dr. Delphine Fitzenz (USGS, formerly ETH Zürich) for her insightful discussions about rock mechanics. Both were also my flatmates and good friends who helped to make my stay during these years here in Zürich more enjoyable. Similarly, many people from the Institute supported me during my time at ETH and I would like to acknowledge them: Bettina Roth-Galamb, Rita Gysin, Dr. Fanny Leuenberger, Dr. Werner Balderer, Dr. Kurosch Thuro and Dr. Corrado Fidelibus.

Special thanks go to my fellow doctoral students and friends in the Engineering Geology group for the good times we had together. In particular Dr. Susanne Laws and Dr. Volker Lützenkirchen for their company during the field seasons and Heike Willenberg, David Estoppey, Christian Klose for their endless discussions about geology and world events.
Above all, special thanks go to my family and friends in Austria, who supported and motivated me during this time. This work is dedicated to my wife Viktoria and my daughter Jana for their constant strength, support and love.
# Contents

**Summary**

**Zusammenfassung**

1. Introduction
   1.1 Study Motivation
   1.2 Study Objectives
   1.3 Gotthard Highway Tunnel
   1.4 Geodetic Data
   1.5 Geological and Hydrogeological Framework
   1.6 Field Investigations
   1.7 Laboratory Investigations
   1.8 Structure and Contents of this Thesis

2. Brittle Fault Zones and Fractures in Anisotropic Crystalline Rocks of the Central Gotthard Massif
   2.1 Introduction
   2.2 Regional Geological Setting
   2.3 Structure of Brittle Fault Zones
   2.3.1 Brittle Fault Zones in Central Gotthard Massif
   2.4 Anisotropy Control on Brittle Fault Zone Formation
   2.4.1 Geological Boundaries (e.g. Igneous Dykes, Compositional Layering in Meta-Sedimentary Rocks)
   2.4.2 Ductile Structures (e.g. Schistosity, Mylonitic Foliation)
   2.4.3 Brittle Structures (e.g. Meso-Scale Fractures)
   2.5 Discussion
   2.6 Conclusion
   2.7 Acknowledgements

3. Laboratory Measurements of Biot’s coefficient for Low-Porosity Granitic Rocks
   3.1 Introduction
   3.2 Theoretical Background
   3.3 Experimental Procedure
   3.4 Results
   3.5 Discussion
   3.6 Conclusion
   3.7 Acknowledgements

4. Generic 2-D Studies in Vertical Tunnel Cross Sections
   4.1 Introduction
   4.2 Flow and Deformation Models for the Underlying Subsidence Mechanisms
   4.2.1 Fluid Flow and Fluid Pressure
5. Analysis of Surface Subsidence in Crystalline Rocks Above the Gotthard Highway Tunnel

5.1 Introduction

5.2 Theoretical Background

5.2.1 Hydro-Mechanically Coupled Behaviour of Discontinuities

5.2.2 Hydro-Mechanical Behaviour of the Intact Rock Matrix

5.3 Site Description

5.3.1 Deformation Measurements

5.3.2 Geological and Structural Background

5.3.3 Hydrogeology

5.4 Analytical Assessment of Pore Pressure Diffusion

5.4.1 Time Required to Attain Steady State Pore Pressure Drawdown Conditions in a Fractured Rock Mass

5.4.1.1 Time for Pore Pressure Drawdown within the Discontinuity Network

5.4.1.2 Time for Pore Pressure Drawdown within Intact Blocks

5.5 Discontinuum Modelling of Consolidation Subsidence

5.5.1 Controlling Input Parameters

5.5.1.1 Effective Stress Coefficient ($\alpha_f$)

5.5.1.2 Normal Stiffness of Meso-Scale Fractures and Brittle Faults

5.5.1.3 Shear Stiffness of Fractures and Brittle Faults

5.5.1.4 Shear Strength of Fractures and Brittle Faults

5.5.1.5 Dilation of Fractures and Brittle Faults

5.5.2 Parametric Study and Sensitivity Analysis

5.5.2.1 Impacts of Fracture Network Geometry

5.5.2.2 Impacts of Mechanical Properties and Initial Conditions

5.5.2.3 Effects of Hydraulic Properties and Boundary Conditions

5.5.2.4 Summary of Parametric Study Results

5.5.3 The Gotthard Tunnel Case Study

5.5.3.1 Underlying Conceptual Model

5.5.3.2 Model Geometry, Boundary Conditions and Initial Conditions

5.5.3.3 Results

5.6 Continuum Modelling of Consolidation Subsidence
List of figures

1.1 Geographical and geological setting of the Gotthard massif (after Labhardt 1999), location of Gotthard highway and railway tunnel, Gotthard base tunnel and hydroelectric dams. 3

1.2 Geological map of the study area. The location of the Gotthard highway and railway tunnel, the surface levelling profile and the triangulation points (magnitude and mean error in mm) are shown. 5

1.3 Levelling profile along the Gotthard pass road showing surface subsidence in the time interval 1970 to 1993/98 and alpine uplift. 6

1.4 Early time inflow rates into the Gotthard highway tunnel (i.e. safety tunnel). 8

2.1 (a) Location map with study area; (b) Geological overview of the study area. 28

2.2 Cross section along the Gotthard highway tunnel showing geological and tectonic units. Within the Gotthard massif a schematised pattern of brittle fault zones (i.e. the fan structure) is shown. 29

2.3 Fault zones mapped in the area of the Gotthard pass. Black lines represent mapped brittle fault zones, dashed lines represent inferred fault zones based on aerial photos and geomorphological mapping. 30

2.4 (a) Brittle fault zones mapped within the Gotthard safety tunnel; (b) Brittle fault zones mapped on surface (plotted as poles); (c) Brittle fault zones mapped on surface with measured slickenside striations. 31

2.5 Outcrop view of conjugate fault system observed in the Gamsboden granitic gneiss. 32

2.6 Aerial view of a heavily faulted region in the Fibbia granite gneiss showing the individual fault sets (i.e. BF1, BF2 and BF3) marked by dashed black lines. 32

2.7 North-South profile along the Gotthard highway tunnel showing the dip angle of the brittle fault zones and the location of the fan structure axis. 33

2.8 Schematic of the relative age of anisotropic structures. 33

2.9 Orientation of intrusion contacts for the lamprophyric dykes. 34

2.10 Microscopic view of a foliated lamprophyric dyke, showing en-echelon micro-fractures filled with zeolites. 34

2.11 Large micro-fracture within lamprophyric dykes, aligned parallel to foliation and suggesting tensile opening. 34

2.12 Photo and plan view of a brittle fault zone initiated on a pre-existing lamprophyric dyke. 35

2.13 Photo of clayey-sandy fault gouge within a sheared heavily foliated lamprophyric dyke. 36

2.14 Structural map of the main foliation as mapped at individual outcrops. 36

2.15 Photo and front view of a brittle fault zone initiated on a pre-existing ductile shear zone observed in the Gamsboden granitic gneiss. 38

2.16 Photo of a small-scale brittle fault (fault gouge < 2 cm) observed in the Gamsboden granitic gneiss. 39

2.17 Full view of a complete thinsection that shows layers of breccia and gouge
oriented parallel to foliation observed within the host rock (Gamsboden granitic gneiss). A right-lateral sense of shear could be observed based on Riedel shears and offset markers.

2.18 (a) Displaced gouge layer through Riedel shears indicating a dextral shear sense. Offset of the foliation within the breccia and filled with zeolite (zeo) can be observed. (b) Angular fragments of alkali-feldspar, plagioclase, quartz (qtz), clinozoisite/epidote (czo), zircon and fishes of muscovite (ms), biotite (bio) and chlorite (chl) as embedded in the gouge matrix. (c) Cohesive cataclasite fragment embedded in fine-grained fault gouge matrix.

2.19 Scanning electron microscopy (SEM) views showing fault gouge clasts and grains at varying magnifications (qz=quartz, kfs=alkali-feldspar, bio=biotite).

2.20 Outcrop trace maps of meso-scale fractures, mapped within the Gamsboden- and Fibbia granitic gneiss showing secondary fractures (F1, F2, F3...meso-scale fracture sets).

2.21 Location and orientation of meso-scale fractures that are sampled on surface and within the Gotthard highway tunnel through outcrop and scanline measurements.

2.22 (a) Normal set spacing distribution of F1 meso-scale fractures measured on surface within the Gamsboden granitic gneiss; (b) Normal set spacing distribution of F1 meso-scale fractures measured in the Gotthard highway tunnel within the Gamsboden granitic gneiss.

2.23 Fracture frequency near a 0.5 m wide brittle fault zone.

2.24 (a) Schematic drawing of the stress direction required for extensional jointing and hybrid shear fracturing (after Hancock, 1985); (b) Stress model explaining stress rotation between meso-scale fracturing and brittle faulting.

3.1 Schematic diagram of equipment set-up.

3.2 Experimental set up showing the cylindrical granitic test specimen covered by a transparent teflon jacket and the sensors used to measure circumferential and axial strain.

3.3 History of axial and circumferential strains resulting from the series of ramp-like increases of hydrostatic stress. Each increase of 10 MPa took 3 min to attain.

3.4 Variation of the cumulative volume of water expelled and cumulative change in rock volume with applied hydrostatic stress. The plateaus in the rock volume curve represent deformation under constant stress conditions.

3.5 Blow-ups of the strain response to the loading steps shown in Figure 3.3 demonstrating that equilibration was essentially achieved before further loading. For the calculations of the hydraulic diffusivity, the times for the implied volume strains to reach 90% of their final value were used.

3.6 Cumulative volumetric strain measured for the saturated and dry rock samples as a function of applied hydrostatic stress. The black circles indicate the equilibrium strain for the saturated samples. The grey curves represent the horizontally shifted response of the dry rock sample so as to pass through the saturated curve after equilibrium at the 10 MPa level.
3.7 History of strain in the dry rock samples during the loading cycle. Expanded views of the first loading steps are shown in Figure 3.8.

3.8 Magnification of the first loading step for the dry Aar granitic gneiss No. 1 and No. 2. Small creep strains are evident.

3.9 Comparison of axial to lateral rock strains for hydrostatic loading tests on saturated samples.

3.10 Uniaxial compression tests showing axial and radial strains. The loading and unloading curves from where the Young's modulus and Poisson's ratio were determined are shown.

3.11 Compilation of Biot's coefficient estimates from this study. The continuous curves represents $\alpha$ values calculated from $\alpha=1-K/K_s$ where $K$ was measured (Figure 3.6) and $K_s$ estimated from the grain compressibility ($K_s=40$ and $50$ GPa). Coloured bars indicate directly measured Biot's coefficients based on the drained hydrostatic compression test. The bar represents a mean $\alpha$ coefficient for the stress range of 10 MPa.

3.12 Comparison of Biot's coefficients obtained from this study with published estimates for crystalline rocks. Error bars were not included. See text for discussion.

4.1 Location map with study area.

4.2 Geological map of Gotthard region.

4.3 Surface subsidence in the time interval 1970 to 1993/98 and Alpine uplift (upper diagram). Early time water inflow rates into A2 road tunnel (lower diagram).

4.4 Conceptual models showing mechanical response to fluid drainage of: (a) horizontal joints, (b) vertical joints, (c) vertical faults, and (d) intact rock.

4.5 Distinct-element model geometry and boundary conditions.

4.6 Contoured pole plots showing brittle fault zone orientations (left) and corresponding total spacing histograms (right).

4.7 Scaling of multiple joint properties to those representative of the bulk rock mass.

4.8 Normal deformation law for one single joint.

4.9 Finite-element model geometry and boundary conditions.

4.10 Pore pressure distribution (units in Pa) before (upper) and after (lower) tunnel drainage for groundwater drawdown close to tunnel level (boundary condition A). The 0.0 Pa pressure line represents the boundary between the saturated and unsaturated zone in the model. The maximum pore pressure calculated directly above the tunnel reaches only 2e-5 Pa.

4.11 Vertical subsidence (in metres) for horizontal joint-controlled model (conceptual model 1).

4.12 Horizontal strains for vertical brittle fault zone-controlled model (conceptual model 3).

4.13 Vertical strains for vertical brittle fault zone-controlled model (conceptual model 3).

4.14 Shear deformation showing zones of right- or left-lateral displacement for conceptual model 3 (upper). Block models showing shear and normal fracture displacements at location numbers 1, 2, 3 and 4 (lower).

4.15 Vertical subsidence (in metres) for vertical brittle fault zone-controlled
model (conceptual model 3).

4.16 Pore pressure distribution (units in Pa) for fixed water table condition (boundary condition B).

4.17 Vertical subsidence (in metres) for fixed water table condition (boundary condition B).

4.18 Pore pressure distribution (units in Pa) calculated for the continuum intact rock matrix case.

4.19 Vertical subsidence (in metres) calculated for the continuum intact rock matrix case (conceptual model 4).

4.20 Transient response of vertical displacements with time for the continuum intact rock matrix case.

5.1 (a) Conceptual model of full hydro-mechanical coupling mechanism along a discontinuity. The hydraulic aperture is controlled by the mechanical aperture, which is influenced by normal closure and shear dilation. (b) Conceptual model of partial hydro-mechanical coupling mechanism along a discontinuity. The hydraulic aperture remains constant and does not vary with effective normal stress changes.

5.2 Surface subsidence between the time interval 1970 to 1993/98 and alpine uplift in the time interval 1918 to 1970 (upper diagram). Initial water inflow rates into Gotthard highway tunnel (i.e. safety tunnel) during excavation (lower diagram).

5.3 Calculated vertical strain rates (dashed line) and surface topography along the levelling profile (i.e. Gotthard pass road, gray line).

5.4 Location map with study area (upper diagram). Geological map of the Gotthard region including levelling and triangulation measurements (lower diagram).

5.5 Detailed map of the Gamsboden granitic gneiss showing traces of mapped and inferred fault zones. As well, the location of levelling points and the major inflow zone are shown.

5.6 In situ block size distribution near surface and at tunnel elevation.

5.7 Conceptual diffusion model.

5.8 (a) Normal closure laboratory experiments on fractures in granitic rock, including all loading cycles. Standard normal closure law implemented into UDEC models. (b) Parameters for the semi-logarithmic normal closure law determined from laboratory and in situ tests.

5.9 (a) Compilation of laboratory E-moduli tests determined on fault core samples within the Swiss Alps. (b) Distribution of fault zone (i.e. core) thickness measured by Schneider (1979) and Wanner (1982) in the Gotthard (A2) safety tunnel.

5.10 Model geometries used for the parametric study: (a) Continuous 50 m spaced vertical fault zones and 10 m spaced horizontal fractures; (b) Continuous 50 m spaced vertical fault zones, 50 m spaced vertical fractures and 10 m spaced horizontal fractures; (c) Continuous 50 m spaced inclined fault zones and non-continuous 10 m spaced inclined fractures; (d) Fault zone pattern implemented from measurement in the Gotthard safety tunnel. Fault zones 2000 m north and south from the major inflow zone were included. Continuous horizontal fractures with a fracture spacing of 25 m.
Hydraulic and mechanical boundary conditions were identical for all four geometry types.

5.11 (a) Initial pore pressure distribution before tunnel drainage. (b) Pore pressure distribution after tunnel drainage for model geometry types A, B, C. (c) Pore pressure distribution after tunnel drainage for model geometry type D.

5.12 Surface subsidence determined for model geometry types A, B, C, and D. Note large boundary effects on model type C due shear slip near the upper left and lower right model boundary.

5.13 Shear displacement along brittle fault zones. Negative values indicate left-handed shear and positive values indicate right-handed shear displacement.

5.14 Parametric study results for model geometry A: (a) Variation of maximum subsidence with intact rock bulk modulus; (b) Variation of maximum subsidence with normal stiffness of horizontal fractures; (c) Variation of maximum subsidence with normal stiffness of vertical fault zones; (d) Variation of maximum subsidence with shear stiffness of vertical fault zones; (e) Variation of maximum subsidence with in situ horizontal to vertical stress ratio.

5.15 Normal closure law for 1 m spaced horizontal fractures implemented for the parametric study (see Figure 5.14 b).

5.16 Parametric study results for model geometry C: (a) Variation of maximum subsidence with in situ horizontal to vertical stress ratio; (b) Variation of maximum subsidence with shear stiffness of vertical faults.

5.17 Pore pressure distribution for hydraulic conductivity study on model geometry A after the tunnel drainage. (a) Horizontal to vertical conductivity equals 1.1; (b) Horizontal to vertical conductivity equals 0.1; (c) Horizontal to vertical conductivity equals 0.01; (d) Horizontal to vertical conductivity equals 0.001.

5.18 Vertical displacement contours of hydraulic conductivity study using model geometry A.

5.19 Surface subsidence of hydraulic conductivity study using model geometry A.

5.20 Variation of the maximum subsidence with the hydraulic conductivity ratio.

5.21 Shear displacement along vertical fault zones for the hydraulic conductivity study using model geometry A: (a) Horizontal to vertical conductivity equals 1.1; (b) Horizontal to vertical conductivity equals 0.001; Negative values indicate left-handed shear and positive values indicate right-handed shear displacement.

5.22 Variation of hydraulic apertures for the fully coupled model (scenario 1). Variation of the hydraulic apertures (a) of the vertical brittle fault zones; (b) of the horizontal fractures.

5.23 (a) Pore pressure distribution after the tunnel drainage for the fully coupled model (scenario 1). (b) Vertical displacement contours.

5.24 (a) Pore pressure distribution after tunnel drainage for the fully coupled model (scenario 2). (b) Vertical displacement contours.

5.25 Variation of hydraulic apertures along vertical fault zones for the fully
coupled model (scenario 2).

5.26 Pore pressure drawdown for model geometry D: (a) Free water table with no recharge; (b) Fixed pore pressure boundary condition (recharge). 161

5.27 Water flow rates into the model from surface model boundary. 162

5.28 Subsidence for model geometry D: (a) Surface subsidence for free water table and fixed pore pressure boundary. (b) Vertical displacement contours for free water table boundary model. (c) Vertical displacement contours for fixed pore pressure boundary model. 163

5.29 Conceptual and boundary condition model along a N-S section for the Gotthard case study. 164

5.30 Conceptual hydrogeological model (i.e. water table drawdown). 164

5.31 Discontinuity pattern and applied hydraulic transmissivities for the Gotthard case study (given in Table 5.9). 165

5.32 Pore pressure distributions before and after tunnel drainage, respectively, for: (a, b) Hydraulic boundary type I; (c, d) Hydraulic boundary type II; (e, f) Hydraulic boundary type III. 166

5.33 Vertical displacement contours for the Gotthard case study. 167

5.34 Surface subsidence trough: (a) Numerically simulated; (b) Measured by leveling technique along the Gotthard pass road. 167

5.35 Shear displacement along discontinuities and surface subsidence profile. 168

5.36 Normal closure along brittle fault zones. 169

5.37 Principal stress direction within intact blocks (green) and discontinuities (red lines) near the large inflow zone. 169

5.38 Pore pressure distribution: (a) Before tunnel drainage; (b) After tunnel drainage for the hydraulically isotropic model; (c) After tunnel drainage for the hydraulically anisotropic model. 170

5.39 Surface subsidence for intact rock matrix models. 171

5.40 Distribution of: (a) Vertical strain; (b) Horizontal strain for the intact rock matrix models. 171

5.41 Pore pressure difference between initial and drained condition. Inside the gray colored region pore pressure drawdown reaches magnitudes exceeding 0.5 MPa. 172

5.42 Surface subsidence for the equivalent rock mass models. 172
List of tables

3.1 Change in volume of fluid expelled, change in volume of the specimen, determinations for the Biot’s coefficient, $\alpha$, from saturated and dry tests, and errors for each loading step (i.e. stress interval of the saturated specimen).  

3.2 Estimation of the hydraulic diffusivity from the time required to achieve 90% consolidation for each loading step for Aar granitic gneiss No. 2.  

3.3 Drained intact rock parameters determined from the uniaxial compression test.  

4.1 Intact rock properties for discontinuum models.  

4.2 Joint and brittle fault zone properties.  

4.3 Intact rock properties for continuum models.  

4.4 Summary of results showing maximum settlements modelled and point of maximum settlement relative to surface point directly above the tunnel.  

5.1 Orientation and spacing data of mapped meso-scale fractures.  

5.2 Time for pore pressure drawdown within the discontinuity network.  

5.3 Dependency of specific storage and diffusion coefficient from Biot’s and Skempton’s coefficient.  

5.4 Time for pore pressure drawdown within intact blocks.  

5.5 Compilation of laboratory and in situ normal closure experiments on meso-scale fractures in granitic rock (Parameters for the semi-logarithmic closure law).  

5.6 Intact rock and discontinuity properties for parametric study.  

5.7 Discontinuity properties of fully coupled models.  

5.8 Normal stiffness and spacing of horizontal meso-scale fractures for the Gotthard case study.  

5.9 Hydraulic properties of brittle fault zones for the Gotthard case study.  

5.10 Hydraulic properties of horizontal meso-scale fractures for the Gotthard case study.  

5.11 Analytical solutions of surface subsidence.  

5.12 Input parameters for the isotropic intact rock model.  

5.13 Input parameters for the layered intact rock model.  

5.14 Input parameters for the layered equivalent rock mass model.  

5.15 Summary of results showing maximum subsidence generated from discrete discontinuity (discontinuum) and intact rock matrix deformation (continuum).  

5.16 Summary of results showing maximum subsidence originated from equivalent rock mass models (continuum approach).
Summary

Substantial surface subsidence due to deep tunnelling in a crystalline rock mass was unexpected before such a phenomena was observed above the Gotthard highway tunnel in central Switzerland. Analyses of a N-S orientated levelling profile along the Gotthard pass road showed considerable magnitudes of surface subsidence, where the maximum of 12 cm coincides with a stiff granitic gneiss unit. This subsidence trough was also confirmed by triangulation measurements, which in addition provided information about the shape of the trough outside the pass road. The close spatial relationship between maximum subsidence and maximum water inflow rates into the tunnel and the temporal correlation between tunnel construction and surface subsidence suggested that hydro-mechanically coupled processes may have induced such deformation processes.

Starting from this working hypothesis, the objectives of this study focus on: (1) the investigation of underlying subsidence mechanisms and the quantification of drainage-induced surface subsidence in a crystalline rock mass, and (2) whether surface subsidence is mainly driven by consolidation of discontinuities (i.e. tensile or shear fractures and brittle fault zones) or by consolidation induced through poroelastic strains within the intact rock matrix. As such, this study incorporates detailed field mapping, laboratory testing and 2D-numerical simulation with regards to the hydro-mechanical coupled behaviour of discontinuities and intact rock.

Extensive field mapping carried out in the central Gotthard massif showed that the nucleation and propagation of brittle fault zones was likely influenced by pre-existing rock anisotropy, formed through geological boundaries (e.g. igneous dykes), ductile structures (e.g. schistosity, mylonitic foliation) and brittle structures (e.g. meso scale fractures). Three sets of brittle fault zones, a NE-SW, NNE-SSW and E-W striking could be defined on the basis of fault plane measurements and geomorphic lineaments. Based on measurements of the pitch of slickenside striations, most of the brittle fault zones were activated in a strike-slip mode. Observations from meso- to micro-scale indicate that right-handed shear sense was the dominating movement process. For each, the main foliation, sub-parallel meso-scale fractures and sub-parallel brittle faults all form a fan-like structure showing the same orientation (i.e. NE-SW strike) and location of the symmetry plane.

The laboratory testing campaign focussed on the poroelastic behaviour of the intact rock matrix to ascertain the contribution of the intact rock matrix on consolidation subsidence processes. The key parameter, the Biot's coefficient ($\alpha$), represents the deformation behaviour through coupling between the pore pressure and the rock stress within the intact rock matrix. Results clearly showed that coupling between pore pressure and intact rock response cannot be neglected for such low-porosity crystalline rocks. Values determined for the Biot's coefficient on rock samples from the Aar and Gamsboden granitic gneiss in Central Switzerland were found to range between 0.27±0.13 and 0.80±0.08, depending on the applied hydrostatic stress condition.

Before simulations involving the actual case study of the Gotthard highway tunnel were performed, generic analyses were performed to investigate the underlying subsidence mechanisms and sensitivity of individual parameters. Studied in detail were the effects of (1) sub-horizontal fractures; (2) sub-vertical discontinuities (i.e. fractures and brittle fault zones) and (3) the intact rock matrix, on poroelastic deformations.
resulting in surface subsidence. Results showed that the frequency and the normal stiffness of sub-horizontal fractures have a large impact on the magnitude of vertical displacements. In addition, numerical simulations showed that fracture closure alone is not the only key subsidence producing mechanism, but that intact rock matrix can considerably contribute to rock mass deformation through poroelastic strains. Vertical discontinuities contribute to surface subsidence through intact block strains induced through a Poisson’s ratio effect.

Building on results from the generic study, the attained knowledge was extended to the actual case study of the Gotthard pass region and highway tunnel. To study the magnitude of intact rock matrix- and discontinuity-induced subsidence on the Gotthard pass region, two separate modelling approaches were applied, adopting first a discontinuum approach (UDEC) followed by a continuum approach (VISAGE). Although the direct summation of magnitudes in subsidence generated from the continuum (i.e. poroelastic strains of the intact rock matrix) and the discontinuum (i.e. closure and shear on discrete discontinuities and Poisson’s ratio induced strains in the intact rock matrix) analyses is physically not valid, relative comparisons between the two simulation techniques can be done. Results emphasized the importance in quantifying the pore pressure drawdown, which is influenced by the hydraulic connectivity of the rock mass around the tunnel, the hydraulic anisotropy of the rock mass, the hydraulic boundary conditions (i.e. no flow boundary or fixed pore pressure boundary) and the magnitude of the surface recharge. Discrete-element discontinuum modelling provided insight into the consolidation mechanisms involving discontinuity deformation and Poisson’s ratio effect in the intact rock. Results from the Gotthard case study produced maximum subsidence magnitudes ranging between 0.032 to 0.080 m, or 0.04 to 0.05 m when the most reasonable input parameters were used. Finite-element continuum modelling provided insights into the poroelastic response of the intact rock matrix. The calculated maximum subsidence for the Gotthard case study ranged between 0.016 to 0.068 m, or 0.05 to 0.06 m when the more likely input parameter values were used. Of key importance, it was found that shear and normal displacement along steeply inclined brittle fault zones can have a large impact on the shape of the subsidence trough. Accordingly, the fan structure observed along the Gotthard pass and the spatial distribution of these brittle fault zones (i.e. clustering) likely generates such shear and normal deformations, and thus, the shape of the subsidence trough in the Gotthard pass region.
Zusammenfassung


Die Untersuchung des Einflusses, den die Porenwasserdruckabsenkung in der intakten Gesteinsmatrix auf das Verformungsverhalten hat, bildet einen weiteren zentralen Bestandteil dieser Dissertation. In diesem Zusammenhang wurden Laborversuche an ungeklüfteten Granitgneisproben durchgeführt. Der Schlüsselparameter, der die hydromechanische Koppelung zwischen Gesteinsspannung und Porenwasserdruck kontrolliert, wird durch den „Biot’s Koeffizienten“ beschrieben. An den Gesteinen, die von den Gesteineinheiten des Aar- und Gamsbodengranitgneises aus dem Gotthardstrassentunnel stammen, konnten Werte für die Biot’s Koeffizienten zwischen 0.27±0.13 und 0.80±0.08 gemessen werden. Diese starke Variation wurde durch die Abhängigkeit des Koeffizienten von der beim Versuch angelegten Spannung verursacht.

Die anschliessenden Modellrechnungen mit Berücksichtigung der vorhandenen geologische und hydrogeologischen Situation am Gotthardstrassentunnel bauen auf diesen Resultaten der Parameterstudien auf. Hierbei wurden die zwei wesentlichen Verformungsmechanismen, d.h. Verformung an Diskontinuitäten und Verformung der Gesteinsmatrix separat untersucht. Da sich beide Verformungsprozesse in der Natur beeinflussen und überlagern, können die errechneten Senkungsbeträge nicht direkt aufsummiert werden. Dennoch kann ein relater Vergleich zwischen beiden Mechanismen durchaus durchgeführt werden und zeigt folgende Ergebnisse: 1) Maximale Senkungsbeträge über dem Gotthardstrassentunnel, die aufgrund von Verformungen an Sprödstrukturen errechnet wurden, liegen im Bereich zwischen 0.032 and 0.080 m. Die wahrscheinlichsten Beträge liegen zwischen 0.04 und 0.05 m. 2) Maximale Senkungsbeträge, die aus Verformungen der intakten Gesteinsmatrix resultieren, liegen im Bereich von 0.016 zu 0.068 m. Hier liegen die wahrscheinlichsten Beträge zwischen 0.04 und 0.05 m. Scher- und Normalverformungen an steilstehenden Störungszon en können die Form des Setzungstrichters beeinflussen. Für den Fall des Gotthardstrassentunnel konnte speziell aufgezeigt werden, dass die Fächerstruktur aus spröden Störungszon en und der Abstand dieser Störungszon en untereinander die Scher- und Normalverformungsbeträge beeinflussen und in weiterer Folge zu Unregelmässigkeiten des Senkungstrichters führen kann.
1. Introduction
1.1 Study Motivation

Problems based on pore pressure drawdown resulting in surface subsidence are generally encountered in petroleum extraction, groundwater pumping, shallow tunneling and geothermal projects. Geertsma (1973), Segall (1985), Jones and Mathiesen (1993), Hettema et al. (2000) and Cook et al. (2001) studied the effects of oil extraction on land subsidence. Extensive long-term groundwater extraction from soil-aquifers in the Las Vegas Valley have led to subsidence induced damage of structures and well casings (Hoffmann et al. 2001, Burbey 2001). The same phenomenon has been experienced in Mexico City, where land subsidence of 8 m was measured between 1984 and 1991 due to consolidation of a lacustrine aquitard caused by aquifer exploitation (Ortega-Guerrero et al. 1999). Mossop and Segall (1997, 1999) investigated surface displacements of up to 1 m in relationship to geothermal power production within 'The Geysers geothermal field', which is located in a highly fractured greywacke and felsite rock mass. On other geothermal sites, for example the Wairakei geothermal field in New Zealand, surface subsidence of about 14 m was induced through fluid extraction between 1950 and 1997 (Allis 2000). A general conclusion from these case studies was that pore pressure drawdown due to fluid extraction (i.e. water, oil, gas) within a porous and/or fractured rock mass can produce substantial surface displacements.

In contrast, such effects are not considered or have rarely been the focus of detailed investigations and/or studies, in relationship to deep tunnels in crystalline rock. The surface settlements measured along the Gotthard-pass road in fractured crystalline rock above the Gotthard highway tunnel located in central Switzerland were unexpected. There, a maximum subsidence of approximately 12 cm was measured 800 m above the elevation of the tunnel in a fractured granitic-gneiss rock mass unit. The differences between observations at the Gotthard-pass to those from the past can be realized when looking closer at the underlying rock mass properties (i.e. porosity, degree of fracturing, Young's modulus and Poisson's ratio). In general, subsidence problems related to fluid extraction occur in highly fractured and commonly unconsolidated, porous sedimentary rock masses. In contrast, the Gotthard highway tunnel was driven through low-porosity (≤ 1% intact matrix porosity) fractured crystalline rock, where the maximum subsidence displacements coincide with a stiff granitic gneiss unit and exceptionally high tunnel water inflows.

The reason for initiating a study on surface subsidence in crystalline rocks can be more clearly discerned when looking at the impact such deformations may have on structures, most notably concrete dams, bridge piers and abutments. Only minor differential displacements are necessary to induce damage to such structures. For example, technical problems were experienced with a retaining dam at Zeuzier in Switzerland, where groundwater drainage through an investigation adit located 1.5 km from the dam resulted in settlements of approximately 13 cm (Lombardi 1988, 1992, 1994). The resulting subsidence-induced cracking affected the integrity of the dam in such a way that it had to be emptied and repaired over a period of several years. The temporal relationship between the observed dam deformation and the adit construction clearly showed the causality between the water flow of a confined fractured limestone aquifer into the adit and the subsequent subsidence below the dam.

In light of the new 57 km Gotthard base tunnel (AlpTransit), which is currently under construction, prevention and attenuation of surface subsidence are of major concern
given the nearby location of several dams under which the tunnel passes (Figure 1.1). The close spatial relationship to these dams requires methodologies to estimate the influence that tunnel construction may have on surface subsidence. To avoid/prevent large-scale subsidence in crystalline rock masses through applying advanced tunnelling methods (i.e. controlling rock mass drainage and pore pressure drawdown), the underlying hydro-mechanical coupled mechanisms must be understood and investigative methods developed. As such, this study was initiated to develop conceptual and numerical models to study, understand and predict subsidence in fractured crystalline rock.

Figure 1.1: Geographical and geological setting of the Gotthard massif (after Labhardt 1999), location of Gotthard highway and railway tunnel, Gotthard base tunnel and hydroelectric dams.

1.2 Study Objectives

The primary objective of this thesis is to study processes related to large-scale surface subsidence above deep tunnels in crystalline rock masses. Exceptional new data sets suggesting surface subsidence in a crystalline rock mass in the Swiss Alps present the basis to improve our knowledge about these processes, taking into consideration the complex geology of an actual site. Previous studies (for example the Zeuzier case study) have demonstrated that pore pressure drawdown due to fluid extraction within a fractured rock mass can produce substantial surface displacements. Starting from this working hypothesis, the objectives of this study focus on: (1) the investigation of
underlying subsidence mechanisms and the quantification of drainage-induced surface subsidence in a crystalline rock mass, and (2) whether surface subsidence is mainly driven by consolidation of discontinuities (i.e. tensile or shear fractures and brittle fault zones) or by consolidation induced through poroelastic strains within the intact rock matrix. As such, this study incorporates detailed field mapping, laboratory testing and 2D-numerical simulation with regards to the hydro-mechanical coupled behaviour of discontinuities and intact rock.

Field mapping, primarily focussed on the formation and structural architecture of brittle fault zones and their interrelationship to meso-scale fractures, and statistical data on discontinuity orientation, spacing and length provided useful input parameters to generate a 2D fracture network model. In addition, the information pertaining to the structural architecture of discontinuities helped to constrain their hydrogeological and mechanical properties (i.e. permeability, shear and normal stiffness, Mohr-coulomb strength parameters). The Laboratory testing campaign focussed on the poroelastic behaviour of the intact rock matrix. Determination of rarely measured poroelastic parameters, for example the Biot's coefficient (Biot 1941; Detournay & Cheng 1993) for granitic rocks, was an important objective of this investigation. In addition, these tests provided the linear elastic constants (i.e. bulk modulus, Young's modulus and Poisson's ratio). Results derived through the field investigations and laboratory testing were subsequently used to provide the conceptual framework input parameters and constraints for the 2D numerical analysis. This analysis first focussed on discrete-element modelling to study the effect and deformation mechanisms of fractures and brittle fault zones and their contribution to surface subsidence. Secondly, finite-element continuum simulations were performed to study deformation mechanisms induced through consolidation of the assumed isotropic linear elastic poroelastic intact rock matrix. Finally, the combination and integration of results based on field mapping, laboratory testing and numerical simulations were used to resolve the underlying mechanisms and quantify magnitudes of deep tunnelling induced subsidence above the Gotthard highway tunnel.

1.3 Gotthard Highway Tunnel

The 16918 m long two-lane Gotthard highway tunnel (A2) was built between 1969 and 1977 and ensures that a safe N-S connection between the northern and southern part of Switzerland is assured all year around. During tunnel construction, a safety tunnel was excavated 12 to 18 months and several hundred meters in advance of the primary road tunnel. In doing so, the safety tunnel served as an investigation and drainage adit. The cross section of the main tunnel varied between 69 and 96 m². The safety tunnel is located 30 m east from the main tunnel and shows a cross section between 7 and 10 m² (Keller et al. 1987). A maximum overburden of 1500 m was reached near the Gotthard pass for the highway tunnel. In addition, four ventilation shafts were drilled at locations having small overburdens to ensure that adequate air circulation is provided for the tunnel.

Several kilometres east from the highway tunnel the 14900 m long Gotthard railway tunnel (SBB) was built in a time span between 1872 and 1881, with a maximum overburden of 1700 m. Nearby, surface tunnels showing maximum overburdens of
150m were constructed to connect the Lucendo- and Sella dams with the power station in Airolo (1947). In addition, extensive underground construction was conducted in the region of the Gotthard pass by the Swiss army. However, information about the exact location and depth of these excavations are not accessible to the public. In Figure 1.2 the location and geological setting of the Gotthard highway and railway tunnel and the Lucendo- and Sella dams is shown.

Figure 1.2: Geological map of the study area. The location of the Gotthard highway and railway tunnel, the surface levelling profile and the triangulation points (magnitude and mean error in mm) are shown.
1.4 Geodetic Data

The Swiss Federal Office of Topography has carried out high precision levelling measurements since the beginning of the last century (starting between 1903-1927) to establish an official elevation and reference system for Switzerland. Measuring campaigns initiated afterwards (i.e. between 1943-1991) were used to study significant recent crustal movements (Kahle et al. 1997). In the region of the Gotthard pass, the Swiss Federal Office of Topography carried out precise levelling measurements (in 1993/98) using a closed loop over the old Gotthard pass road and through the highway and railway tunnels (unpublished reports: TB 97-40 and TB 98-27). Two earlier measurement campaigns were likewise made along this N-S profile over the old Gotthard pass road in 1918 and 1970 (Figure 1.2 and 1.3). Between this time interval (i.e. before tunnel construction), an undisturbed uplift was measured with a rate of 1 mm/year, corresponding to uplift processes related to the alpine orogeny. In contrast, the surveys made between 1970 and 1993/98 (i.e. after tunnel construction), showed significant downward displacements along a 10 km N-S line above the tunnel (Figure 1.3).

![Levelling profile along the Gotthard pass road showing surface subsidence in the time interval 1970 to 1993/98 and alpine uplift.](image)

The maximum subsidence occurs at Sustenegg and reaches 12 cm (characterised by a mean error of 4.8 mm at this point). The geological assessment of the reliability of the levelling points with respect to their regional significance, compared to local surface displacements (i.e. landslides, flexural toppling, unstable rock blocks and other forms of mass movement), was done by a team of specialists assigned by the Swiss Federal Office of Topography.
As part of this study, levelling points placed on the road between Hospental and Airolo were similarly inspected. Along the road, most levelling points are located on rock outcrops or concrete structures, which are directly in contact with the *in situ* rock mass. Only a few levelling points are placed on soil foundations. Other sources of uncertainty and error in the levelling profile can result from the measurement technique itself. Levelling of the closed loop, as performed above the Gotthard pass and returned through the Gotthard railway/highway tunnels, always produced final non-zero displacements. These errors were related to the earth curvature (i.e. the shape of the geoid). In addition, atmospheric refraction due to variations in the density of the earth’s atmosphere may affect the levelling results through atmospheric pressure and temperature fluctuations. The error induced from the shape of the geoid (i.e. levelling error of closure) can be explained by the non-parallelism of equipotential surfaces affecting the alignment of the level and are an important factor in mountainous regions. As such, the levelling data presented by the Swiss Federal Office of Topography and used within this study are corrected for the shape of the geoid.

More recently, surface triangulation measurements have confirmed the existence of the subsidence trough (Salvini 2002). These measurements are based on classical triangulation techniques (i.e. theodolite, tachymeter) and supplemented by data from a global positioning system (GPS). Triangulation points measured in 1920 were compared with those measured recently in the year 2000 (Figure 1.2). The technique also enabled measurements of surface deformations on survey points outside the levelling profile along the road, thereby providing extra data to define the spatial dimension of the surface subsidence trough. Results clearly show that the extension of the trough in the E-W direction is noticeably smaller than in the N-S direction. It should be noted that two triangulation points located at exposed spots southeast from the pass do show large magnitudes of subsidence (i.e. 33 mm, 114 mm). However the source of these deformation anomalies can be related to surface mass movements. Overall, the accuracy of the triangulation method was found to be considerably lower than that of the levelling technique. For example, a mean error of 29 mm was calculated for the largest magnitude of subsidence (i.e. 82 mm near Sustenegg as measured through triangulation).

### 1.5 Geological and Hydrogeological Framework

The Gotthard massif is situated in the central Swiss Alps (Figure 1.1 and 1.2) and covers an area of 580 km². It outcrops in the form of an 80 km long and 12 km wide N-E striking mountain range. The study area is located in the central part of the massif through which the Gotthard railway and highway tunnels are driven. The Gotthard massif belongs to the ‘External Crystalline Massifs’ of the Central Alps and consists of a pre-Variscan, polyorogenic and polymetamorphic basement (primarily gneisses, schists, migmatites and amphibolites), which are intruded by Variscan magmatic rocks (Labhart 1999). A more detailed description of the geological situation within the Gotthard massif can be found in Chapter 2. In addition, Chapter 2 focusses on the structural evolution of the study area, especially with respect to brittle faulting and fracturing processes in this region. Given that fractures and brittle fault zones have the highest permeability and provide preferential pathways for fluid flow in crystalline rock
masses, studying these features was required to obtain some insight into the hydrogeological situation. The initial inflows into the safety tunnel are shown in Figure 1.4. From this figure it can be seen that a sharp increase in the initial inflow rate, reaching 300 l/s per 100 m tunnel interval, was encountered in the Gamsboden granitic gneiss. This 300 l/s per 100 m interval inflow rate predominantly flowed from two brittle fault zones situated 23 m apart. Luetzenkirchen (2003), derived hydraulic transmissivities based on these two highly permeable fault zones obtaining yields of 3e-4 m²/s (for the 150 l/s fault) and 2e-4 m²/s (for the 110 l/s fault). Inflow rates of 8 l/s are still measured today for these fault zones within this tunnel section.

![Figure 1.4: Early time inflow rates into the Gotthard highway tunnel (i.e. safety tunnel).](image)

Spring line mapping on surface above the central section of the highway tunnel shows a relatively constant altitude between 2300 and 2500 m a.s.l. for the existing springs, including a relatively clear spring line directly above the two high permeability brittle fault zones. Measurements of low tritium content and high water temperatures sampled from the major fault zones within the tunnel, further suggest that the water is old and mostly flows upwards into the tunnel from the fault zone beneath it (Luetzenkirchen 2003). These findings support the hypothesis that the regional ground water table was not significantly influenced by subsurface drainage of the Gotthard highway tunnel. Outside these two major fault zones, highly permeable structures have only been observed in competent granitoid rock, whereas in schistous paragneisses the transmissivities of the respective permeable structures is several orders of magnitude lower (Luetzenkirchen 2003). As observed by the geodectic triangulation measurements, no substantial subsidence was observed in regions characterised by the
schistous paragneisses. Excluded from this trend was the paragneissic rock unit located directly above the Gotthard highway tunnel and bounded by the Gamsboden- (in the north) and by the Fibbia (in the south) granitic gneiss. There, significant magnitudes in surface subsidence were measured (Figure 1.2). The interrelationship between pore pressure drawdown due to drainage and surface subsidence is discussed further in detail in chapters 4 and 5.

1.6 Field Investigations

The first stage of the field investigation campaign involved the mapping of existing meso-scale fractures (i.e. joints and shear fractures) and brittle fault zones on a regional scale in the area between Hospental and Airola (Figure 1.2). The diverse spatial properties exhibited by brittle fault zones relative to single fractures, especially with respect to their size and properties, required that these structures be treated and evaluated independently. Mapping of brittle fault zones included the sampling of orientation data, the structural architecture and their interrelationship to foliation and meso-scale structures and geological boundaries. Outcrop mapping of meso-scale fractures focussed on orientation data, surface structure, the termination relationship between individual fracture sets, and fracture infilling. Geological, structural and topographical data were collected and managed through a GIS database, which was subsequently programmed to resolve the orientations and spatial relationships between dominant joint sets.

The second stage of this field mapping involved the detailed scanline mapping of meso-scale fractures along the Gotthard pass road and within unlined sections of the Gotthard safety tunnel. Whereas outcrop measurements are suited to provide discontinuity orientation data, the more systematic scanline technique allows for measures of fracture size, spacing and frequency. These geometrical parameters, when represented by different probability distributions, formed the input for the 2-D fracture network generation required for the discrete-element modelling. Aerial photos were also analysed to map lineaments, which were then verified in the field. At selected fully exposed outcrops, the structural architecture of brittle fault zones was mapped and studied in detail.

To examine brittle fault zone structure at the micro-scale, a thin section analysis was performed on collected samples. Thin section analysis was either performed by classical light microscopy or by scanning electron microscopy (SEM). Pre-existing structural data, primarily brittle fault zones collected during the construction of the Gotthard safety tunnel by Schneider (1979) and Wanner (1982), were newly re-evaluated and applied to this study.

1.7 Laboratory Investigations

A laboratory testing campaign was conducted to test intact granitic rock samples. The test program included the evaluation of the Biot’s coefficient and the drained bulk modulus, both tested under hydrostatic stress conditions. In addition, uniaxial compression tests were performed to determine the Young’s modulus and Poisson’s ratio of these samples. These tests were performed on a MTS Rock Testing System-815
in collaboration with the Institute of Rock Mechanics at the Technical University in Graz (Austria). Samples were taken from cored boreholes drilled in the Gotthard highway tunnel (i.e. safety tunnel) at locations corresponding to the rock units of the Gamsboden granitic gneiss (9183 m from the north portal; Luetzenkirchen 2003) and Aar granitic gneiss (2014 m from the north portal; Laws 2001). Cores were drilled with triple-tubes or double-tubes to minimize sampling disturbance and later re-cored and cut into right-angled cylinders with lengths of 107 mm and diameters of 54 mm. With respect to texture, a slight foliation can be observed in the Aar-granite samples, but for the Gamsboden granitic gneiss, no clear macroscopic foliation indicators are visible (although the hosting geological unit is classified as a granitic gneiss). The range of hydrostatic stresses tested was 0 to 70 MPa and were chosen to cover stress conditions applicable for most engineering problems. In addition to performing the tests under saturated conditions, the same samples were re-tested at dry conditions applying the same loading procedure. Findings from these tests were related to the drained bulk modulus. Subsequent to the testing directed towards determining the poroelastic properties, the samples were used again in a series of uniaxial compression tests in order to determine their Young's modulus and Poisson's ratio.

1.8 Structure and Contents of this Thesis

This thesis consists of four papers, which form the main chapters of this document.

Chapter 2 presents the geological investigations, focussing on the occurrence, structural architecture and genesis of brittle fault zones and meso-scale fractures in the highly anisotropic rock masses of the central Gotthard massif. The resulting paper is based on an extensive field mapping campaign, complemented by optical microscopy and scanning electron microscopy. Chapter 2 includes a deterministic and statistical analysis of geometrical properties of faults and meso-scale fractures and attempts to elucidate the evolutionary history of faults and fractures. These temporal changes are then used to develop conceptual models with respect to the nucleation and propagation of brittle faults and fractures, and accordingly, their superposition in forming complex rock mass structures and temporal changes in the regional in situ stress field.

Chapter 3 presents the results from the laboratory testing campaign as performed on intact granitic rock samples. The determination of the Biot’s coefficient through the application of different testing approaches is shown. In addition, the dependency of the Biot’s coefficient on the applied stress regime is discussed. Results from tests performed to determine the elastic constants for the intact rock matrix are finally given and conclude this chapter.

Chapter 4 focuses on general mechanisms involved in the development of surface displacements above deep tunnels and presents the results of 2-D discontinuum (i.e. distinct-element; UDEC) and 2-D continuum (i.e. finite-element; VISAGE) modelling. Results show that settlements are most sensitive to horizontal joints, as would be expected, but that vertical fractures also contribute to the settlement profile through a 'Poisson ratio' effect. However, this study also suggests that fracture deformation alone
cannot explain the total subsidence measured in the Gotthard area. As such, preliminary 2-D poroelastic finite-element models are presented to demonstrate the contributing effect of consolidation of the intact rock matrix.

In chapter 5, the hypothesis formulated in chapter 4 is studied in more detail using the same modelling tools (UDEC and VISAGE). The chapter includes an expanded parametric sensitivity analysis, and numerical simulations of the Gotthard case study. As such, this chapter integrates the input provided by the previous chapters (i.e. model geometry and discontinuity parameters from Chapter 2, intact rock properties from Chapter 3 and findings relating to the underlying deformation mechanisms from Chapter 4), with the input parameters for the Gotthard subsidence case study. Finally, findings from this chapter are discussed with respect to recommendations regarding the testing and monitoring programs that would be necessary for more accurate predictions for future deep underground (i.e. tunnelling) projects in crystalline rocks.

This thesis then concludes with a 6th chapter summarizing and highlighting the major findings from this work.
2. Brittle Fault Zones and Fractures in Anisotropic Crystalline Rocks of the Central Gotthard Massif

C. Zangerl, S. Loew and E. Eberhardt
Abstract:

The spatial relationship between large water inflow rates into the Gotthard highway tunnel, most notably flowing from two distinct brittle fault zones intersecting the tunnel, and the maximum surface subsidence measured along the Gotthard pass road, located above and adjacent to the tunnel, can be directly correlated. Extrapolation along the strike of the main brittle fault structure (NE-SW), from which the maximum water inflows into the tunnel were measured, to the region of maximum subsidence show excellent agreement. A hypothesis was thus developed that these brittle fault zones may act as high permeable conduits, which can undergo large normal- and shear strains through rock mass consolidation processes as the stress state changes due to pore pressure drawdown. To investigate this hypothesis and understand the underlying mechanisms, numerical simulations examining surface subsidence processes would be required, for which data pertaining to structural architecture, orientation and spacing of the brittle fault structures would be essential.

This paper presents the results from an extensive field mapping campaign that focuses on the formation of brittle fault zones and meso-scale fractures in a highly anisotropic crystalline rock mass (e.g. granitic- and para-gneisses and schists), using the central Gotthard massif in Switzerland as an example. In addition, the spatial occurrence and structural architecture of brittle fault zones and their interrelationship to meso-scale fractures are investigated based on outcrop observations, scanline mapping, and light- and scanning electron microscopy. The analysis presented utilizes field mapping data to illustrate that several pre-fault anisotropic features (i.e. lithological boundaries, ductile and brittle structures), coupled together with temporal changes in the regional in situ stress field, control the nucleation and propagation of brittle faults. In addition, a preliminary attempt was initiated to derive a stress history model for several brittle deformation phases in the central Gotthard region based on brittle fault- and meso-scale fracture patterns. Results show that three sets of brittle fault zones, a NE-SW, a NNE-SSW and E-W could be distinguished, activated predominately in a strike-slip regime. A fan structure, encompassing the main foliation, formed the meso-scale fractures and the brittle fault zones, each of which show the same orientation and location of the symmetry plane (NE-SW orientated). Secondary fractures propagating near the tip of those fractures (NW-SE) aligned sub-parallel to the brittle fault zones and enclosing an angle of 20 to 50° were also observed. It is suggested that these secondary fractures may be formed under tensile stress conditions through strike-slip reactivation along the main fractures.

2.1 Introduction

Anisotropic rock masses possess planes of mechanical weakness caused by brittle or ductile deformation structures (e.g. fractures, secondary foliations, faults), sedimentary structures (e.g. bedding planes) or geological boundaries (e.g. compositional layering of sedimentary or metamorphic rocks or intrusion dykes). These features influence the mechanical properties of the rock mass, including compressive and tensile strength, the elastic moduli (Poisson ratio, Young’s modulus) etc., in such a way as to produce a strong dependency on the stress field and loading direction.
Whereas brittle faults are characterised by slip parallel to a single discrete fracture plane, fault zones are formed through subparallel or anastomising interconnected closely spaced faults. Laboratory compression tests suggest that faults rarely originate as shear fractures in isotropic rock masses (Petit and Barquins 1988) and that pre-existing dilatant fractures and rock anisotropy would strongly influence fault growth. In laboratory compression tests on anisotropic rocks (i.e. schists or gneisses), rock strength significantly decreases when applying axial loads inclined 30° to 60° to the foliation (Donath 1961; Gottschalk et al. 1990; Kwasniewski 1993; Brosch et al. 2000).

To understand faulting processes several scale-dependent mechanisms have been proposed. For example, a large-scale planar fault is likely to have developed through the coalescence of propagating micro-cracks, joints and/or veins. Field observations (Hancock 1972; Willemse et al. 1997; Mollema and Antonellini 1999) and laboratory experiments (Scholz 1968; Lockner et al. 1994) confirm this hypothesis. Similarly, larger fault zones have been observed as forming through the propagation and coalescence of smaller fault segments (e.g. Peacock 1991; Peacock and Sanderson 1991; Cartwright et al. 1995; Willemse 1997). Most of these faulting mechanisms described above developed in sedimentary rocks. Another type of faulting mechanism, developed primarily from crystalline rock sites, is related to shear traction on a meso-scale planar discontinuity that generates fracture parallel slip (Segall and Pollard 1983; Granier 1985; Martel et al. 1991; Martel 1990; Cruikshank et al. 1991; Martel and Boger 1998; Peacock 2001; Wilkins et al. 2001). In other words, pre-existing discontinuities formed in tension (mode I), but were followed by in-plane shear (mode II) to create faults. Increasing movement along the plane generally results in the fracturing and breaking off of wall rock fragments, as a result of the process of cataclasis. Martel and Peterson (1991) described lamprophyre dykes and ductile shear zones in granitic host rocks that act as a nucleus for brittle faulting. Furthermore, pre-existing shear zones (i.e. mylonitic rocks) or foliation structures favour the faulting process sub-parallel to these structures. The coexistence of cataclastic and mylonitic fabrics within a single fault zone is frequently described in the literature (Sibson 1977; Gibson and Gray 1985; Simpson 1986) and generally interpreted as indicative of a brittle-to-plastic transition shear zone (Rutter 1986) and attributed to a progressive phase of deformation within a single phase of faulting (Sibson 1977; Simpson 1986; Stel 1986). More rarely, it is attributed to different tectonic events acting along relict fabrics within a reactivated fault zone (Flinn 1977; Obee and White 1986; Gaudemer and Taponnier 1987; Tremblay and Malo 1991). However, it is frequently difficult to distinguish between fabric characteristics of progressive fault-related deformations and those associated with reactivated faults.

This paper focusses on the formation of brittle fault zones and meso-scale fractures in a highly anisotropic crystalline rock mass (e.g. granitic- and para-gneisses and schists) using the central Gotthard massif in Switzerland as an example. The analysis presented utilizes field mapping data to illustrate that several pre-fault anisotropic features (i.e. lithological boundaries, ductile and brittle structures), coupled together with temporal changes in the regional in situ stress field, control the nucleation and propagation of brittle faults, and accordingly, their superposition in forming complex rock mass structures. In addition, a preliminary attempt was initiated, to derive a stress history model for brittle deformation phases in the central Gotthard region based on brittle fault- and meso-scale fracture patterns.
2.2 Regional Geological Setting

The Gotthard massif is situated in the central Swiss Alps (Figure 2.1a) and covers an area of 580 km². It outcrops in the form of a 80 km long and 12 km wide N-E striking mountain range. The study area is located in the central part of the massif through which the Gotthard SBB- and A2-road tunnels are driven. The Gotthard massif belongs to the ‘External Crystalline Massifs’ of the Central Alps and consists of a pre-Variscan, polyorogenic and polymetamorphic basement (primarily gneisses, schists, migmatites and amphibolites), which are intruded by Variscan magmatic rocks (Labhart 1999) (Figures 2.1b and 2.2). The variscan intrusives in the Gotthard-pass region, are mostly granitoids and were intruded in two different phases separated by several million years. During the older phase (303-301 Ma) the Fibbia- and Gamsboden-granites were intruded as shown in Figure 2.1b. The younger intrusion phase took place between 295-293 Ma and involved the crystallization of the Rotondo, Mt. Prosa and Winterhorn-aplite granites (Oberli et al. 1981; Sergeev et al. 1995 a,b). The Fibbia-granite, located in the southern part of the study area, is constrained along its southern margin by a 100 to 300 m thick layer of Rotondo-granite and eastwards by the Mt. Prosa granite. Going north to the northern boundary of the Gamsboden-granite, a several 100 m thick layer of Winterhorn-aplite granite separates the Gamsboden-granite from the pre-Variscan basement rocks. The northern boundary of the Gotthard massif is marked by an alpine-tectonic contact to the Permo-carboniferous and Mesozoic sediments (Urseren-Gavera-zone; Wyss 1986) and the Tavetsch massif along the Rhine-Rhone valley. This heavily tectonized zone separates the Gotthard- from the Aar-massif. At the southern border of the Gotthard massif another steeply dipping zone of parautochtonous mesozoic metasediments separates the first from the units of the Pennine domain. This tectonic unit is separated into the so called ‘Piora zone’ east and ‘Nufenen zone’ west of the Gotthard pass, which are characterised by schists and a sequence of carbonates, gypsum/anhydrite of Triassic to Jurassic age (Herwegh and Pfiffner 1999).

During alpine metamorphism, greenschist facies conditions were reached throughout the Gotthard massif, with an increase in peak pressure and temperature from north to south. Along the southern boundary, amphibolite facies conditions were achieved (Frey et al. 1980; Labhart 1999). The main alpine deformation phase in the Gotthard massif starts in the lower Oligocene around 35 and 30 Ma (Schmid et al. 1996), corresponding with the peak metamorphic overprint characterised through a ductile deformation regime. Formation of ductile deformation structures (i.e. foliation and shear zones) predominately occurred in a NW-SE orientated compressional stress regime (Steck 1968; Merz 1989; Marquer 1990; Pettke and Klaiper 1992). A higher degree of ductile overprint, represented by penetrative foliation textures and shear zones is clearly observed in the Fibbia- and Gamsboden-granite. The younger intrusives, the Winterhorn-, Rotondo-granites etc., also show ductile structures but these are much more limited to shear zones. As such, Guerrot et al. (1991) postulate a variscan deformation phase between the older and younger intrusion events. Conversely, Marquer (1990) argues that deformation in the region is only alpine related (i.e. significantly younger), although it should be noted that his study focussed primarily on the Fibbia-granite. Additionally, Merz (1989) attributed the foliation of the Medel granite exclusively to the alpine deformation phase. In the central Gotthard Massif alpine shear zones and foliation strike NE-SW or E-W and dip southwards in the northern part and
northwards in the southern part, forming a fan like structure (Labhart 1999). Ongoing deformation changed gradually from a ductile to a brittle deformation regime characterised by brittle faulting. Little work was done on the formation of brittle structures within the Gotthard massif (Kvale 1966; Arnold 1970), even though they are of major importance to understanding the tectonic evolution of the region. The complexity, which may be involved during the late alpine deformation phase was shown by Wyder and Mullis (1998) within the Tavetsch massif adjacent to the Gotthard massif. They concluded on the basis of microstructural, mineralogical and microthermometric studies on brittle fault rocks from the Tavetsch massif, five different alpine deformation phases. Ductile shear zones were formed 20 Ma ago, relating to a depth of 18 km and 435°C. Whereas 11 to 9 Ma ago, brittle faulting started by cataclastic flow conditions at a depth of 6-9 km and a temperature of 190°C. However, given that these observations were made inside the Tavetsch massif, conclusions derived from these studies cannot necessarily be attributed to brittle deformation processes within the Gotthard massif.

2.3 Structure of Brittle Fault Zones

Following the terminology suggested by Engelder (1987), fractures may be classified as either joints or faults. The term ‘joint’ is used where it is clear that appreciable shear displacement did not occur (i.e. primarily tensile) and the term ‘fault’ is used where there is evidence of appreciable shear. The more general term ‘fracture’ is used where there is no clear evidence as to whether the discontinuity formed under tensile or shear loading conditions. The term meso-scale is used to embrace fractures that range in size from less than a centimetre to a few metres, and that are usually observable in a single continuous exposure (Hancock 1985). The term ‘secondary foliation’ is used after Passchier and Trouw (1996) and includes cleavage, schistosity, differentiated compositional layering, mylonitic foliation, etc. Primary foliations are structures related to the original rock-forming process, where the most common examples are bedding planes in sedimentary rock and magmatic layering in igneous rocks. Brittle fault rocks result from the process of cataclasis and are classified according to Ramsay and Huber (1987). Whereas the terms ‘fault breccia’ and ‘gouge’ apply to initially cohesionless fault rock, the term ‘cataclasite’ is used for fault rock that possesses primary internal cohesion. Although both breccia and gouge are cohesionless materials, they can become impregnated and sealed by crystal growth in the voids to produce cemented breccia or cemented gouge.

2.3.1 Brittle Fault Zones in Central Gotthard Massif

Figure 2.3 shows the trace pattern of mapped and inferred brittle fault zones on surface, and the strike and dip of faults measured along the Gotthard highway-safety tunnel between Hospental and Airolo. Two major sets striking NE-SW (set F1) and NNE-SSW (set F2), and one minor W-E set (set F3) can be distinguished. Brittle fault zone orientation data from surface and highway tunnel mapping are plotted in Figures 2.4a,b using equal-area Kamb-contour pole plots projected on the lower-hemisphere. In these figures, it can be seen that data from both the tunnel and surface show similar pole
distribution patterns, although it is not possible to resolve clearly the three different fault sets. The stereoplot in Figure 2.4c includes only surface fault planes in which striations could be mapped, and through which the inferred sets are more distinctly discerned. The NE-SW and NNE-SSW striking fault zones intersect each other at a relatively low angle of approximately 25° (Figures 2.3 and 2.6). In general, the location where both sets intersect each other is covered with debris and therefore clear observations regarding the manner of intersection cannot be discerned. Nevertheless it could be observed how the two brittle fault sets intersect each other with an angle of 25° to form a conjugate fault system (Figure 2.5). E-W striking fault zones are statistically minor but can be clearly seen in aerial photos of fibbia-granitic-gneiss (Figure 2.6) and through field mapping observations. Figure 2.6 also shows that the major NE-SW striking fault zones fork into different branches within a tight 20° arc. The NNE-SSW striking faults terminate at the major NE-SW structures. The pitch of slickenside striations on fault planes are mostly flat plunging with 87% having measured plunge values between 0 and 35° (Figure 2.4c). The remaining 13% have measured striations plunging steeply in a range from 57 to 75°. All striations were observed and measured on smooth, polished, mirror-like slickenside planes representing the contact shear plane or along the contact between the fault gouge layer with the intact host rock. Based on these observations, most of the mapped fault zones can be classified as pure strike-slip faults following the classification scheme by Angelier (1994). The rest can be grouped as oblique-slip faults. All of these observations relate to the youngest faulting events.

Shear movement indicators (i.e. slickensides, offset markers, Riedel shears) from NE-SW and NNE-SSW striking fault zones generally show a right-lateral sense of slip. Regardless, some left-handed strike slip faults, sometimes in relation with conjugate fault systems, were observed (Figure 2.5). Offset values ranging from a few cm to a maximum of 50 m were mapped through the help of displacement markers, most notably NW-SE striking lamprophyric dykes. Large-scale offset values, i.e. greater than 100 m, are not present as can be demonstrated by discordant lithological boundaries (relative to the orientation of brittle fault zones) for which no noticeable displacements occur. Such displacements would be expected across alpine fault zones activated in a strike-slip regime which in turn would dislocate the east N-S striking intrusion contact of the Gamsboden- and the Fibbia-granitic-gneiss (Figures 2.3). Only east of Mätteli does a clearly buckled intrusion contact allow for an interpretation of possible right-handed strike-slip displacements on the order of magnitude of several 100 m. Given the convoluted nature of these intrusion contacts, however, it is not possible to define this structure as primary or fault-related.

Due to the lack of fully exposed outcrops, determination of the shear sense for E-W striking fault zones (set BF3) becomes more complicated. Limited data from slickensides and offset markers suggest left-handed shear sense. Arnold (1970) also observed left-handed shear for E-W striking brittle fault zones 12 km east of the Gotthard pass in the pre-Variscan basement unit. Shear sense indicators further suggest that these faults could have developed through conjugate faulting processes together with the NE-SW or NWW-SSW sets. Brittle faults zones in the central Gotthard massif form a ‘fan’-like structure characterised in the northern part by southeast dipping faults and in the southern part by northwest dipping faults. A N-S profile along the Gotthard highway tunnel illustrates the fan structure of faults and shows the point of dip overturn (Swiss coordinates: X=686840 m, Y=158765 m; Figure 2.7). The geological cross
section along the Gotthard highway tunnel shown in Figure 2.2 further clarifies schematically the nature of the fan structure. The orientation of the sub-vertical dipping 'axial plane' of the fan structure drawn on Figure 2.3 is based on surface and tunnel measurements and strikes 60° from NE to SW.

2.4 Anisotropy Control on Brittle Fault Zone Formation

Three modes of brittle fault generation based on different forms of anisotropy were established. Geological boundaries, ductile structures and brittle structures are the basis for the grouping (Figure 2.8), which also follows a relative chronological order beginning with geological boundaries as the oldest features, ductile structures at an intermediate age and finishing with brittle structures as the youngest source for pre-faulting anisotropy. Superposition of these anisotropic features was frequently observed. For example, geological boundaries (e.g. dykes) can deform under ductile regimes (e.g. shear zones) and sub-sequentially act as nuclei for brittle fault zone propagation. Given the obvious interaction between these processes, grouping was done primarily through outcrop observations based on the oldest anistropic structure that could be distinguished and verified by microscopic studies.

2.4.1 Geological Boundaries (e.g. Igneous Dykes, Compositional Layering in Meta-Sedimentary Rocks)

Within the granitic gneisses, the frequent occurrence of brittle fault zones at contacts to igneous dykes demonstrates that they can serve as nuclei for fault zones. Igneous dykes within the Gotthard region were intensively studied by Oberhänsli (1986) and classified as lamprophyres, kersantites, speserartites. Within the study area numerous dykes were mapped with widths varying from several centimetres to several metres. Orientation measurements of intrusion contacts to the granitic host rock show two main sets, one striking NE-SW (set L1), the other striking NW-SE (set L2). A minor set can also be discerned with contacts that strike E-W (set L3; Figure 2.9). Occasionally, the dykes (especially set L1) show a 'biotite-schist'-like texture characterised through a high biotite content and a densely spaced schistosity (Figures 2.10 and 2.11). In many cases these L1-dykes acted as pre-formed weakness zones for ductile shearing and brittle faulting. The mineralogical composition includes mainly biotite and quartz with small amounts of plagioclase and muscovite. Dykes, which are only minimally deformed (i.e. foliation textures) and typically belong to set L2, are composed of amphiboles, feldspar, muscovite, epidote/zoisite, chlorite, titanite and biotite. These dykes were not activated as ductile shear or brittle fault zones. Lamprophyric dykes re-activated as brittle faults zones typically show a sharp boundary (mirror-like fault plane with slickensides) to the intact host-rock (Figures 2.12 and 2.13). Adjacent to this fault plane, a layer of fine-grained, greenish, sandy-clayey fault gouge is formed. Fault gouge layers range in thickness from a few mm to 30 cm (Figure 2.13). Faulting processes can also incorporate adjacent granitic-gneisses, observed as zones of brecciation (Figure 2.12). Rarely were these fault zones observed as involving a central gouge layer bounded by damage or fracture zones (Figure 2.13). Faulted lamprophyre dykes show
tight asymmetrical Z-shaped drag folds with a vertical dipping fold-axis, an indicator for right-handed shear (Figure 2.12).

Microscopic observations of lamprophyre dyke samples taken from brittle fault zones also provide evidence for brittle deformation. Filled micro-fractures with dimensions of several 100 μm and aligned as en-echelon fractures are shown in Figure 2.10. The fracture in Figure 2.11 with a trace length of several mm and observed in the same rock sample, was orientated parallel to the schistosity and showed isolated biotite fishes that were embedded in the fracture infilling. Partly detached biotite grains bend into the fracture opening. The infill in these fractures is composed of low-temperature zeolites, most likely stilbite, which is characterised by a radial growing texture. The occurrence of zeolites in fault related fractures and fault zones agree well with observations in other rock samples deformed by cataclasis, as well as those by Luetzenkirchen (2003) in the Rotondo granite west of the study area.

The structural architecture of brittle fault zones in the pre-Variscan basement rock cannot be observed in detail due to the unsatisfactory outcrop situation. Nevertheless, observations on a few outcrops showed that the orientation of brittle fault zones in the pre-Variscan Basement rock (i.e. para-gneisses and migmatitic gneisses) often is driven through their compositional layering. Compositional layering is created through alternating layers of cm to m wide intervals of mica-feldspar-gneiss, quartzite, mica rich schists and amphibolites. Less competent mica rich layers (i.e. schists) of this meta-sedimentary series are predominately sheared, showing abrupt contacts between faulted rock (gouge and fault breccia) and undeformed host rock. Within most of these fault zones, the pre-existing compositional layering is reflected by spatial distributions of fault breccia and gouge that form along layers of low strength.

2.4.2 Ductile Structures (e.g. Schistosity, Mylonitic Foliation)

Foliation in the granitic gneisses within the study area (e.g. Gamsboden, Fibbia) is defined by aligned mica (muscovite and biotite) grains and shear zone bands (mylonites). Feldspar, quartz grains and mafic xenoliths within the foliation planes are flattened parallel to the foliation strike direction. When ductile shear zones are present the structure of the foliation is characterised by shear zones surrounding lenses of more weakly deformed material (Marquer 1990; Gapais et al. 1987). These zones generally form anastomosing arrays, enclosing lens-shaped domains that underwent smaller and more homogeneous strain. Thus, the overall shear zone pattern consists of mylonite zones surrounding lenses of lower strain. Non-deformed or weakly foliated domains were rarely observed in the study area, except within the younger variscan intrusives of Winterhorn-, Mt.Prosa- and Rotondo-granites. The pre-Variscan basement is characterized by compositional layering and foliation formed by preferred orientations of mica grains, and grain boundaries of quartz and feldspar. In general, the foliation is aligned sub-parallel to the compositional layering except from locations where intensive folding occurred. Alternating layers of schists and gneisses, quartzites, migmatites or amphibolites and the generally higher mica content contribute to an increased anisotropic structure as seen in the granitic gneisses.

Figure 2.14 shows the regional distribution of the main foliation in the granitic gneisses and pre-Variscan basement rocks. Local variations in the number of foliation sets and strike azimuth of the foliation are present, but in general a systematic pattern
indicating a mean strike of NE-SW can be seen. Foliation dip angles in the northern part of the mapping region are to the SE, but experience a change in dip to the NW towards the south. Therefore the main foliation forms the same fan structure as previously described for the brittle fault zones. The exact same structural pattern was found in the highway tunnel at depths of up to 1500 m below surface. The axis of the fan structure strikes NE-SW and is congruent with the axis observed for the brittle fault zones (see Figure 2.14).

Figure 2.15 provides a photo and schematic representation of a typical fault zone in the Gamsboden-granitic-gneiss that is aligned sub-parallel to the main foliation and a ductile shear zone. Brittle faulting occurred on pre-existing ductile shear zones that are characterised by alternating layers of elongated quartz-feldspar and mica rich bands. Again, these faults are characterised by a sharp contact with the undisturbed host rock where the boundary is often marked by a mirror like fault plane. Adjacent to the fault plane, a several mm to cm thick, grey-greenish coloured, clayey to sandy layer of fault gouge can be found. At the contact fault surface, and also within the gouge, striations are present. Adjacent to the gouge zone, mylonitic rock overprinted by fracturing and local brecciation occur. Lenses composed of quartz and feldspar grains are aligned parallel to the shear zone and brecciated. As such, the clayey-sandy fault gouge forms anastomosing arrays around the lenses of pre-existing, partly fractured shear zones (Figure 2.15). Within the gouge layers, which obtain thicknesses of several mm to cm, flat dipping slickensides were found. Occasionally shear zone fragments are internally ductile folded and truncated by fault gouge layers, which forms discordant structures between the foliated shear zone fragments and foliated host rock. Inside the brittle fault zone, foliation- and fault parallel fractures with increasing frequency can be seen as shown in Figure 2.15. For the fault zone shown in Figure 2.15, its width decreases from 1.5 m to 0.5 m over a distance of less then 15 m sub-vertically. In general, fault zones are smaller than one meter in width but in some cases reach widths of up to 3 m. Similarly, fault gouge layers change in width and frequency along the fault zone. Figure 2.16 show an example of a small-scale fault with only a single layer of fault gouge (1-3 cm). Notable are the fault sub-parallel fractures (F1). Adjacent to the fault no increase in fracture frequency can be observed.

Microstructure of Fault Breccia

The protolith granitic gneisses (Fibbia, Gamsboden) contain approximately 30-50% quartz, 20-36% plagioclase, 14-31% alkali-feldspar (microcline, orthoclase), 2-10% muscovite, 2-10% biotite/chlorite and minor traces of epidote, calcite, garnet, apatite, zircon and opaque minerals (Hofmänner 1964; Schneider 1979; Wanner 1982). Grain sizes range from 0.01 mm to 2 cm with alkali-feldspars usually forming the larger grains. Densely spaced micro fracturing wasn’t observed in the intact host-rock.

Figure 2.17 shows a complete thinsection from a fault zone, which incorporates cemented breccia and gouge and is orientated perpendicular to the foliation. Three different types of cataclastic foliation structures can be observed: (a) layering of fine grained fault gouge and fault breccia; (b) foliation within the fault gouge layers characterised by varying colours, grain sizes and seams of secondary and opaque minerals; and occasionally (c) foliation within gouge or breccia, defined by sub-parallel alignment of small grains of muscovite, biotite and chlorite, inclined 0-30° to the fault
boundary (Figure 2.18a). The cataclastic foliation of type (a) observed within the brittle fault zone is aligned parallel to the main foliation of the host rock.

Breccia is characterized by densely fractured fragments encompassed by zones of gouge. Kinked and folded grains of biotite, muscovite and chlorite deform within shear fractures and date as pre-fault mineral grains. A later stage of crystallisation of sericites and zeolites within open voids and open fractures can be observed. Clear identifiable shear fractures (Riedels) offset these mica layers (foliation type ‘c’ described above) and are partly filled with zeolites (Figure 2.18a). More specifically, Ca-Zeolites (stilbite) were identified as the fracture infill by applying optical microscopy, scanning electron microscopy (SEM) with EDS and X-ray diffraction (XRD) techniques. Some minor deformed clasts show a micro-fracture pattern characterised by 2-3 sets of fractures. Traces of micro-fracture set 1 form an acute angle of 10-30° to the fault layering, set 2 traces an angle of 60-80° and set 3 traces an angle of 70-80° but in reverse direction (set1 and 2 in shear direction and set 3 opposite direction). These fractures do not show any observable interrelationships to the meso-scale fracture pattern of the host rock and thus they may be defined as Riedel shears. Gouge layers are composed of angular fragments of alkali-feldspar, plagioclase, quartz, clinozoisite/epidote, zircon and fishes of muscovite, biotite and chlorite (Figures 2.18a-c and 2.19) embedded in a fine grained matrix (<10 µm) of quartz, alkali-feldspar, plagioclase, sericite, zeolites (stilbite) and clay-minerals (illite/montmorillonite).

In addition, angular fragments of pre-existing cohesive cataclasites were observed, which is found to be in agreement with observations in the Rotondo granite from Luetzenkirchen 2003 (Figure 2.18c). Survival of isolated cohesive cataclastic fragments in a gouge layer would indicate at least two deformation phases. In addition, Wyder and Mullis (1998) also found two deformation stages (V and VI) in the Tavetsch massif, where cataclasis was observed as the dominant deformation process. These cataclasite fragments are composed of angular fragments of quartz, plagioclase, alkali-feldspar and mica embedded in a very fine grained matrix. Angular fragments in general can reach grain sizes of up to 1 mm. Fault gouge layers were displaced by Riedel shears (dextral), enclosing an angle between the main cataclastic foliation and Riedels of 20 to 40° (Figure 2.18a). A right-lateral sense of shear could be determined using microscopic shear indicators like Riedel shears and offset markers in fractured grains (Figures 2.17 and 2.18a).

2.4.3 Brittle Structures (e.g. Meso-Scale Fractures)

Anisotropy formed through meso-scale fractures, generally tension joints, can activate faulting processes when shear tractions acting along their surfaces cause fracture parallel slip. Subsequent deformation then acts to produce a gouge or other structures related to mechanical wear that are so common of typical faults. Shear deformation on pre-existing tensional joints, especially near fracture tips, create secondary fractures (syn-fault fractures). These secondary fractures are usually small joints (mode I) that tend to propagate oblique to the associated pre-existing slipped tension joint, enclosing an angle of 20 to 50° (less frequently up to 70°) and extending from only one side of the fault (Granier 1985; Martel 1997). Furthermore, these fractures often occur in clusters, but also sometimes as single features propagating from the fault tip. Depending on the angle, frequency or regional preference several different
terms for secondary fractures have been used including: splay fractures (Martel 1990), pinnate or feather joints (Hancock 1985), tip cracks (Kim et al. 2001), kinks (Cruikshank et al. 1991), and horsetail fractures (Kim et al. 2001; Cruikshank et al. 1991; Granier 1985). Accordingly, the more general term ‘secondary fractures’ has been adopted here and the interrelationship between brittle fault zones and these fractures are explained based on field observations and fracture mapping in the following section. Fractures were mapped through surface outcrops and by applying scanline sampling techniques. These data are supplemented with data measured in the A2-Gotthard safety tunnel passing parallel to the highway tunnel.

Meso-Scale Fracture Pattern

Four to five meso-scale fracture sets were mapped and characterized (Figure 2.21). The most dominant fracture set (F1) can be found in all rock types of the massif and is orientated sub-parallel to the main foliation structures (Figure 2.14). Definition of fracture set F1 is based on statistical clustering of orientation measurements, and in addition, on the relationship to the main foliation (Figure 2.21). Accordingly, these F1 fractures also form the fan structure with an identical NE-SW striking axis. On a regional scale, these fractures show a relatively constant average NE-SW strike. However on a more localized scale, the strike of the fractures vary following the foliation or anastomosing pattern of the shear zones. This variation in strike ranges from 0 to 20° from the mean. Within basement rocks, the strike of the F1 fractures is continuous and parallel to that of the foliation. Within granitic rock bodies, the same general trend was observed but in exceptional cases a variation in strike between foliation and F1 fractures of up to 30° was measured (Figure 2.20b). The length of F1 fracture traces are in general within the range of cm to dm and their surfaces are generally planar to curviplanar with very rough and undulating faces. Distinct plumose structures weren’t observed on F1 fracture faces either in the granitic or basement rocks. Occasionally, flat plunging striations on F1 surfaces (totally or partly) indicate shear deformation. It would appear that open mode joints parallel to the foliation planes don’t evolve clear identifiable plumose ornaments as can be observed for joints steeply crosscutting the foliation. Whereas some fractures of set F1 are filled with biotite, muscovite, quartz, feldspar, calcite and Fe-hydroxides, others from the same set are totally unfilled.

In surface outcrops, fracture set F1 was mapped as having a mean strike of 49° in the northern sector of the Gamsboden granitic gneiss, but further south was observed to rotate by 14° to a mean strike of 63° (Figure 2.21a-c). Scanline data analysis (shown in Figure 2.21j-k) produce a trend that is characterised by a mean strike of 40° along profile I and a mean strike of 52° along profile II (rotation of 12° from north to south). Orientation data within the Gamsboden granitic gneiss measured in the safety tunnel show the same F1 cluster with striking NE-SW and dipping steeply to SE (Figure 2.21l).

Within the Fibbia granitic gneiss, located further south, fracture set F1 strikes 51° (mean) and 56° when mapped at outcrops or along scanlines, respectively, and dips steeply to the NW (Figure 2.21d,i). The surface morphology, spacing and trace length characteristics of these fractures is similar to those of Gamsboden-granitic-gneiss.
Adjacent to the Fibbia granitic gneiss, F1 fractures were measured within the thin layer of Rotondo- and Mt. Prosa-granite, which underwent only minor ductile overprinting during alpine deformation and macroscopically shows a granitic texture. These F1 fractures strike 26° along measured scanlines and 20° at individual outcrops (Figure 2.21g,h). As such, F1 fractures were seen to rotate with respect to the same F1 set within the Fibbia granitic gneiss by approximately 30°. A second set called F6 striking almost E-W (mean strike 80°), forms a mean angle of 55° with the mean strike of F1. The bisecting line of the angle between the two strikes closely matches that of the F1 fractures within the Fibbia granitic gneiss and that of the southern pre-Variscan basement rocks. Based on field observations of mutual abutting/cutting relationships, F1 and F6 fractures were interpreted as belonging to a conjugate fracture system (i.e. hybrid shear fractures as referred to according to Hancock 1985).

Surface mapped F1 fractures within the southern pre-Variscan basement rocks (i.e. amphibolite, paragneiss and migmatites layers) show again the same mean strike of 58° and 48°, but dipping only 50 to 60° NW. Data collected from the tunnel indicate steeper dips (60 to 75°) but with little variation in the mean strike (i.e. 48 and 57°).

Figure 2.22a shows the normal-set spacing distribution (Priest 1983) of F1-fractures defined by a mean spacing of 0.47 m measured on surface along scanline profiles I and II. The mean spacing measured within the Gotthard safety tunnel along a segment which is located at depths of 550 to 1250 m below surface reaches 1.68 m (Figure 2.22b). On the basis of the ‘maximum likelihood’ algorithm, parameters for the negative exponential- and Weibull-distributions (see Mathab et al. 1995) were estimated from the experimental spacing data. These two distributions show a good fit to the normal-set spacing histogram for F1 fractures measured at surface. However, for F1 fractures sampled within the tunnel, only the Weibull-distribution adequately fits to the spacing data. The Weibull-distribution as a best fit was also determined for other fracture sets within the Gamsboden-granitic gneiss (Zangerl et al. 2001). The increase in fracture spacing with depth suggests that additional fracturing episodes may have occurred, most likely during alpine uplift due to unloading effects (release joints or postglacial unloading and weathering). The fracture frequency measurements based on scanline method across a 0.5 m wide fault zone in the Fibbia granitic-gneiss is shown in Figure 2.23. Field observations (Figures 2.12, 2.15 and 2.16) and scanline spacing data (Figure 2.23) across brittle fault zones suggest no increase in fracture frequency towards the fault zone in granitic gneisses.

A less frequently observed fracture set (F2) measured within the Gamsboden- and the Fibbia-granitic-gneisses strikes roughly E-W, and dips steeply to the north and/or south (Figure 2.21). The mean of the F2 set cluster is not well defined, and partially overlaps with the F1 cluster. As such, distinction between the F1 and F2 clusters becomes impossible if it is done on a statistical basis. But when assigning fractures to sets in the field on individual outcrops, the distinction becomes more easy, as can be shown on sub-horizontally orientated trace maps (Figure 2.20). In general, F2 fractures propagate from the tip of F1 fractures (exception Figure 2.20b) and extend from only one side of F1 fractures. The angle between the strike of F1 and F2 fractures is within the range of 20 to 50°. Through observations of termination, angle and propagation relationships, F2 fractures were interpreted as secondary fractures and therefore syntectonic to shearing of F1 fractures. F2 fractures propagate in such a way that a right-lateral shear for F1 fractures can be deduced. Fractures belonging to set F2 were
also measured in the Gotthard highway tunnel along a section of Gamsboden-granitic-gneiss (Figure 2.21). F2 fractures do not appear in the pre-Variscan basement rocks.

In contrast, joint set F3 strikes NW-SE and is characterised by a typical greenish coloured hydrothermal infilling of chlorite. The surfaces of these fracture faces are much more planar and smooth. Plumose structures are common. The large dispersion of pole points seen in stereonet plots is related to conjugate shear, hybrid and additional opening mode fractures (e.g. Hancock 1985) recognised in the field by mutual abutting/cutting relationships within the set.

The medium to flat dipping fracture set F4 and F5 (Figure 2.21) are interpreted as gravitational or post-glacial unloading joints, since they follow the smoothed topography of the Gotthard massif mountain ridge. Such unloading joints form near surface during uplift, glacial relaxation and erosion. As such, in the northern pre-Variscan basement rocks and Gamsboden-granitic-gneisses, these joints dip either west (F5) or north (F4). In contrast, F4 joints measured within the southern pre-Variscan basement rocks and Fibbia-granitic-gneisses predominately dip to the south. Clearly recognizable plumose structures were found on faces of F4 and F5 fractures within medium grained lamprophyric dykes.

2.5 Discussion

Data presented above clearly show a structural relationship between brittle fault zones and pre-existing anisotropy, i.e. geological boundaries (e.g. igneous dykes, compositional layering in meta-sedimentary rocks), ductile structures (e.g. schistosity, mylonitic foliation) and brittle structures (e.g. meso-scale fractures). Analyses performed at both large- to small-scale confirm this: Aerial photos and outcrop observations show that most mapped fault zones are orientated sub-parallel to the dip direction and dip angle of geological boundaries, ductile and brittle structures. The main foliation, brittle meso-scale fractures (F1 fractures) and brittle fault zones form the same fan-like structure. The axis of this fan structure, represented by an overturning dip, is congruent for all three. Micro-scale observations on samples of brittle fault zones that are aligned parallel to the overall foliation show alternating layers (mm width) of gouge and breccia. The alternating character and abrupt transition from intensely deformed gouge layers to much less deformed breccia layers suggest an influence of pre-existing anisotropy during faulting processes. The presence of pre-fault foliation structures and/or meso-scale fractures thus work to control fracture propagation, mechanical fragmentation of the rock and subsequent sliding and rotation of the fragments. In other words, anisotropy influences micro cracking, and subsequently microfracture coalescence during the faulting process, in such a way as to prevent fracture penetration into the adjacent wall rock (i.e. strains are localized).

That faulting can nucleate along zones of mechanical weakness has long been established through laboratory experiments on intact anisotropic rock samples. Donath (1961) showed that strength of slates were lowest when loaded at angles of 30° to the plane of foliation. The same 30° angle was found by Brosch et al. (2000) through uniaxial compressive tests on mylonitic gneisses. Laboratory tests conducted by Handin (1969) on fractures confirm that the critical stress required to cause reactivation along
pre-existing fracture surfaces was less than that required to break an unfractured specimen of the same lithology.

Based on these mechanical principles and our field observations we suggest therefore a reactivation of pre-existing F1 fractures to form ‘faulted joints’, and later when strain increases, to form brittle faults zones. Precipitation of biotite, chlorite, muscovite, quartz or feldspar within F1 fractures indicate that the F1 fractures were formed during greenschist facies conditions at temperatures above 300°C after the peak of the alpine metamorphism. Thus F1 fractures are asserted as being older than the brittle fault zones, which in turn formed through cataclasis at temperatures below 300°C according to their microstructures. At very low-grade conditions (below 300°C) quartz and feldspars deform by brittle fracturing and cataclastic flow (Passchier and Trouw 1996). Several pieces of evidence to support the hypothesis of fault nucleation along pre-existing F1 fractures in the Gotthard massif include:

(a) occurrence of secondary fractures;
(b) shear sense derived from termination and orientation relationships of secondary fractures correspond with shear sense observed on strike-slip faults;
(c) there is no geometric or kinematic relationship between faults and F1 fractures, suggesting that they formed under the same stress regime;
(d) meso-scale fracture frequency does not increase towards the fault zones;
(e) traces of slickensides observed on very rough, undulated or stepped F1 fracture surfaces (originally mode I) reveal subsequent shearing episodes;
(f) parallelism of fractures showing on surface slickensides to F1 fractures (mode I) with rough undulated or stepped surface (interpreted as mode I fractures).

These observations on brittle fault zone and fracture genesis can be further applied to construct a preliminary stress history model. Figure 2.24 shows a schematic model of the stress history within the central Gotthard region.

During the alpine orogeny most of the ductile structures, e.g. ductile shear zones, probably formed under low-grade metamorphic conditions 35-50 Ma ago. Schmid et al. (1996) postulate a NW-SE compressional stress regime. Marquer (1990) concluded from studies within the Fibbia-granitic-gneiss that the main foliation as well as the ductile shear zones were formed contemporaneously. Exhumation of the Alps would then generate a decrease in pressure and temperature conditions which would cause a gradual transition from ductile to brittle deformation mechanisms. Previous thrusting conditions would subsequently alter to strike-slip or extensional stress regimes (Luetzenkirchen 2003).

During this stage (Figure 2.24a), the NE-SW striking F1 meso-scale fractures (i.e. pre-fault) formed within the granitic gneisses and ‘Altkristallin’-basement rocks, most likely under tensile conditions (i.e. mode I). The strike direction of F1 fractures had been consistent with the overall strike, when locally, a deviation between the main foliation and these fractures occurred (Figure 2.20b). Within the isotropic Rotondo- and Mt. Prosa granites, conjugate hybrid shear fractures (F1 and F6) were generated. These hybrid shear fractures formed under extensional shear failure at a dihedral angle (2θ) in the range of 1 to 60° (Hancock 1985). The stress conditions responsible for the fracturing process requires a sub-horizontal compressive effective maximum principal stress (σ1) direction that strikes 50-60° NE. Accordingly, the effective intermediate principal stress (σ2) would be compressive and orientated sub-vertical. The intersection lineation between fracture planes from the F1- and F6 sets, measured within the
Rotondo- and Mt. Prosa granites, coincides with the axis of $\sigma_2$ and shows a mean dip angle of $65^\circ$ NW. The effective minimum principal stress ($\sigma_3$) strikes again sub-horizontally SE-NW but in extension.

During a later stage (Figure 2.24b) within the brittle deformation phase, which is accompanied by decreasing temperature and pressure conditions, reactivation of pre-existing ductile structures (i.e. shear zones) and slip on F1 fractures begins. Faulting of these fractures would in turn create along F1 fractures syn-fault secondary fractures (F2), propagating oblique to the associated pre-existing sheared fracture enclosing an angle of 20 to 50°. Laboratory experiments and numerical models predict that the orientation of secondary fractures emanating from fault tips will vary according to: (a) the ratio of shear stress to effective normal stress responsible for kinking (Cruikshank et al. 1991); (b) in response to variations in fault-parallel normal stress (Willemse and Pollard 1998); and (c) as a function of frictional strength along the fault (Cooke 1997). The strike of the secondary fractures is consistent with the shear sense observed on the fault zones indicating right-handed displacement. Within this phase, clockwise rotation of the effective maximum and minimum principal stress ($\sigma_1$, $\sigma_3$) relative to the rock mass would have occurred, forming individual brittle fault zones maybe as conjugate faults. The maximum effective principal stress is orientated sub-horizontal and strikes between $60^\circ$ (= strike of BF1 fault zones) and $100^\circ$ (= strike of BF3 fault zones and F2 secondary fractures). Again, $\sigma_2$ is orientated sub-vertical and all effective principal stress vectors are within a compressive regime.

Continuous stress or block rotation can be assumed leading to stress conditions characterised by a maximum or intermediate principal stress orientated NW-SE ($\sigma_1$ or $\sigma_2$) and a NE-SW minimum principal stress ($\sigma_3$) sub-parallel to the mean strike of the faults. Recent stress data derived from fault plane solutions of seismic active regions surrounding the Gotthard massif indicate a strike-slip or extensional regime (Maurer et al. 1997; Deichmann et al. 2000). In addition, Kastrup (2002) derived for surrounding tectonically active regions that strike-slip to thrust faulting conditions are dominant. Data from the world stress map also favours a currently compressional regime in the NW-SE direction. It should be noted though that given that these principal stress data sets were derived from surrounding regions, a direct comparison to the central Gotthard massif may not be applicable.

2.6 Conclusion

Pre-faulting anisotropy can control the nucleation and propagation of brittle fault zones in the Central Gotthard massif. Such anisotropy is formed through geological boundaries (e.g. igneous dykes, compositional layering in meta-sedimentary rocks), ductile structures (e.g. schistosity, mylonitic foliation) and brittle structures (e.g. meso-scale fractures). Analyses of the interrelationships between brittle fault zones and meso-scale fractures suggest that during a late stage of faulting a change in stress conditions from a compressional to a strike-slip regime occurred. Comparison of the stress regime derived for brittle faulting processes (i.e. strike-slip) with that required for meso-scale fracturing suggests a principal stress rotation. Further work is needed to test conclusions derived from this study with observation from adjacent regions within the Gotthard massif. In particular, the minor ductile deformed granitic rock masses (i.e. Rotondo
granite) represent excellent study sites to investigate the highly complex spatial and temporal interrelationship between fractures and brittle faults.

2.7 Acknowledgements

The authors would like to thank the maintenance team of the Gotthard A2 highway tunnel for their permission to enter the safety tunnel. Thanks also to Dr. Richard Tessardi from University Innsbruck for the X-ray diffractometry analysis and to Auke Barnhoorn for his kind support on the scanning electron microscope. In addition, we would like to thank Dr. Susanne Laws and Dr. Volker Luetzenkirchen for numerous discussions during the field mapping campaign and to Dr. Martin Brändli for his great support on the GIS system.
Figure 2.1: (a) Location map with study area; (b) Geological overview of the study area.
Figure 2.2: Cross section along the Gotthard highway tunnel showing geological and tectonic units. Within the Gotthard massif a schematised pattern of brittle fault zones (i.e. the fan structure) is shown.
Figure 2.3: Fault zones mapped in the area of the Gotthard pass. Black lines represent mapped brittle fault zones, dashed lines represent inferred fault zones based on aerial photos and geomorphological mapping.
Figure 2.4: (a) Brittle fault zones mapped within the Gotthard safety tunnel; (b) Brittle fault zones mapped on surface (plotted as poles); (c) Brittle fault zones mapped on surface with measured slickenside striations.
Figure 2.5: Outcrop view of conjugate fault system observed in the Gamsboden granitic gneiss.

Figure 2.6: Aerial view of a heavily faulted region in the Fibbia granitic gneiss showing the individual fault sets (i.e. BF1, BF2 and BF3) marked by dashed black lines.
Figure 2.7: North-South profile along the Gotthard highway tunnel showing the dip angle of the brittle fault zones and the location of the fan structure axis.

Figure 2.8: Schematic of the relative age of anisotropic structures.
Figure 2.9: Orientation of intrusion contacts for the lamprophyric dykes.

Figure 2.10: Microscopic view of a foliated lamprophyric dyke, showing en-echelon micro-fractures filled with zeolites.

Figure 2.11: Large micro-fracture within lamprophyric dykes, aligned parallel to foliation and suggesting tensile opening.
Figure 2.12: Photo and plan view of a brittle fault zone initiated on a pre-existing lamprophyric dyke.
Figure 2.13: Photo of clayey-sandy fault gouge within a sheared heavily foliated lamprophyric dyke.
Figure 2.14: Structural map of the main foliation as mapped at individual outcrops.
increased F1 fracture frequency

fault contact

brittle fault zone

Figure 2.15: Photo and front view of a brittle fault zone initiated on a pre-existing ductile shear zone observed in the Gamsboden granitic gneiss.
Figure 2.16: Photo of a small-scale brittle fault (fault gouge < 2 cm) observed in the Gamsboden granitic gneiss.
Figure 2.17: Full view of a complete thinsection that shows layers of breccia and gouge oriented parallel to foliation observed within the host rock (Gamsboden granitic gneiss). A right-lateral sense of shear could be observed based on Riedel shears and offset markers.
Figure 2.18: (a) Displaced gouge layer through Riedel shears indicating a dextral shear sense. Offset of the foliation within the breccia and filled with zeolite (zeo) can be observed. (b) Angular fragments of alkali-feldspar, plagioclase, quartz (qtz), clinozoisite/epidote (czo), zircon and flakes of muscovite (ms), biotite (bio) and chlorite (chl) as embedded in the gouge matrix. (c) Cohesive cataclasite fragment embedded in fine-grained fault gouge matrix.
Figure 2.19: Scanning electron microscopy (SEM) views showing fault gouge clasts and grains at varying magnifications (qz=quartz, kfs=alkali-feldspar, bio=biotite).
Figure 2.20: Outcrop trace maps of meso-scale fractures, mapped within the Gamsboden- and Fibbia granitic gneiss showing secondary fractures (F1, F2, F3…meso-scale fracture sets).
Figure 2.21: Location and orientation of meso-scale fractures that are sampled on surface and within the Gotthard highway tunnel (l) through outcrop (a-g) and scanline (h-k) measurements.
Figure 2.22: (a) Normal set spacing distribution of F1 meso-scale fractures measured on surface within the Gamsboden granitic gneiss; (b) Normal set spacing distribution of F1 meso-scale fractures measured in the Gotthard highway tunnel within the Gamsboden granitic gneiss.

Figure 2.23: Fracture frequency near a 0.5 m wide brittle fault zone.
Figure 2.24: (a) Schematic drawing of the stress direction required for extensional jointing and hybrid shear fracturing (after Hancock, 1985); (b) Stress model explaining stress rotation between meso-scale fracturing and brittle faulting.
3. Laboratory Measurements of Biot's Coefficient for Low-Porosity Granitic Rocks

C. Zangerl, M. Bluemel, K. F. Evans, E. Eberhardt and S. Loew
Abstract:

The initial hypothesis investigated in this dissertation with respect to induced surface subsidence in fractured crystalline rock masses was that the consolidation of the rock mass was primarily accommodated through normal and shear strains along the discontinuities rather than the intact blocks. This presumption was based on the low porosity and low permeability typically observed in intact crystalline rocks relative to that of the discontinuity network. To test this hypothesis, a laboratory testing campaign was initiated to measure a key parameter, Biot’s coefficient, $\alpha$, that represents the pore pressure-stress coupling factor and controls effective stresses within the intact rock matrix. It is commonly assumed that the Biot’s coefficients for soils and high-porosity sedimentary rocks are approximately equal to one ($\alpha=1$), whereas those for low-porosity (<1%) intact granitic rocks are significantly lower. However, very few direct measurements of Biot’s constant have been made for crystalline rocks.

This chapter presents the results of laboratory tests to measure the Biot’s coefficient of several granitic rock samples obtained from the Gotthard highway tunnel at locations with different overburden loads. The resulting estimates provide essential poroelastic input parameters for the numerical simulations presented in later chapters for evaluating the contribution of consolidation of the intact blocks (rock matrix) to the surface subsidence. Two independent methods of estimating Biot’s constant were applied and the resulting values compared. A drained hydrostatic compression test was performed on a fully water-saturated rock specimen, where the volumetric rock strain and water volume expelled from the sample were measured. From the ratio of the changes in rock and expelled-water volumes, the Biot’s coefficient can be directly estimated. The second type of testing procedure requires the determination of the drained- and intrinsic bulk modulus. The former was directly measured through a jacketed hydrostatic test on a dry sample, and the intrinsic bulk modulus was estimated from the mineralogical composition of the samples. Results of tests at hydrostatic stress levels of 10-70 MPa yielded estimates of Biot’s coefficient ranging between 0.27±0.13 and 0.80±0.08. Our best estimates, taken at hydrostatic stress levels comparable to the overburden at the sample source locations, ranged between 0.6 and 0.8.

3.1 Introduction

Measurement of Biot’s coefficient, $\alpha$, for low-permeable crystalline rocks is important for modelling many geotechnical processes, such as subsidence due to deep tunnel drainage of crystalline rock masses (Zangerl et al. 2003), drainage-related disturbance of stresses about tunnels and its impact on stress measurements (Evans et al. 2003), and the study of hydraulic fracturing processes (Ito and Hayashi 1991, Berchenko and Detournay 1997). Similarly, investigations of fully coupled thermo-hydro-mechanical (T-H-M) processes in low-permeable rock mass for the long-term storage of nuclear waste and hot dry rock (HDR) geothermal systems require knowledge of the Biot’s coefficient of the rock as input for numerical simulations (Nguyen and Selvadurai 1995, Noorishad et al. 1992, Jing et al. 1995, Kohl and Hopkirk 1995, Kohl et al. 1995, Willis-Richards and Wallroth 1995). Biot’s coefficient represents the pore pressure-stress coupling factor and controls effective stresses that govern the
deformation of the intact rock matrix (Robin 1973, Boitnott and Scholz 1990). Specifically,

\[ \sigma' = \sigma - \alpha \cdot p \cdot \delta \]

where \( \sigma' \) and \( \sigma \) are the macroscopic effective and total stresses respectively, \( p \) is the pore fluid pressure and \( \delta \) = Kroenecker's delta. Biot's coefficient is a rock property, and takes values between 0 and 1. This exact effective stress law for 3-D elastic deformation was originally derived by Biot (1941) for linearly elastic, isotropic and homogeneous rock. A similar formula with unity replacing \( \alpha \) had earlier been proposed by Terzaghi (1925) and remains valid for soils.

Although there are numerous determinations of Biot's constant for sedimentary rocks, very few direct determinations have been made for low-permeable crystalline rocks (Hart and Wang 2001). Thus, to provide values needed for modelling the consolidation resulting from drainage in the vicinity of the Gottard tunnel, a series of laboratory tests were conducted in collaboration with the Technical University Graz. The tests were conducted on granitic specimens sampled from boreholes drilled from the Gotthard highway tunnel in central Switzerland and are reported in this Chapter. The testing campaign was motivated by the need to determine the importance of drainage-driven consolidation of intact rock blocks in explaining the surface subsidence observed following excavation of the tunnel (Zangerl et al. 2001; 2003).

### 3.2 Theoretical Background

Consolidation and related poroelastic properties of crystalline rock derive from the presence, geometry and the orientation of pore space, either in form of grain voids or brittle micro-cracks as described by Robin (1973) and Nur and Byerlee (1971). For most crystalline rocks, micro-cracks exert the predominant influence on poroelastic behaviour (Walsh 1965). Biot's coefficient can be measured in several different ways. We have used two methods. The first is based upon the formula for Biot's constant derived intuitively by Nur and Byerlee (1971) and given by,

\[ \alpha = 1 - \frac{K}{K_s} \]

where \( K \) is the drained bulk modulus measured under drained (i.e. constant pore pressure) conditions, and \( K_s \) is the intrinsic bulk modulus, (i.e. the reciprocal of grain compressibility). Both modulii are most easily measured with hydrostatic compression tests. The bulk modulus is determined on a jacketed sample that is either dry (i.e. pores are filled with air) or saturated, although in the latter case the fluid pressure must be kept constant throughout the test. The determination of \( K_s \) requires an unjacketed specimen saturated throughout by the confining fluid. Because of practical difficulties in saturating the rock samples with the fluid used in the pressure vessel, \( K_s \) was not measured but rather estimated from consideration of the mineralogical composition of the rocks.

The second method also uses a hydrostatic compression test of a jacketed sample
under drained conditions. However it requires that the sample be saturated and the volume of fluid expelled during the loading cycle, $\Delta V_n$, be measured. This quantity provides a direct measure of the pore volume change. Net volume change of the sample, $\Delta V$, must also be measured. Biot's coefficient can then be determined from the relation (e.g. Detournay and Cheng 1993):

$$\alpha = \frac{\Delta V_n}{\Delta V}$$

Before testing, the sample must undergo a careful saturation process and during the test procedure the pore pressure must be held constant and equal to atmospheric pressure (i.e. drained test).

### 3.3 Experimental Procedure

Core samples of Aar granitic gneiss were taken at a rock exposure within the Gotthard highway tunnel at a location where the overburden reaches 500 m. A second sampling/drill site was located further along the tunnel in the Gamsboden granitic gneiss more than 1200 m below surface. Specimens were double-tube drilled to 90 mm diameter to minimize sampling disturbance and later re-cored and cut into right-angled cylinders with lengths of 107 mm and diameters of 54 mm. All cores were taken normal to the foliation. However the foliation was slight for the Aar-granite samples, and not visually perceptible for the Gamsboden granitic gneiss, even though the host unit is classified as a granitic gneiss. The porosity of the test samples was inferred from the difference between dry and saturated weights and ranged between 0.007 to 0.009. To reduce test errors, the samples were dried and then carefully saturated within a vacuum chamber. The procedure involved first drying a sample for one week at 105 °C in a desiccation chamber and then transferring it to an empty beaker in a vacuum chamber where it was left for 2 days at 74 cm of vacuum. The beaker was then partly filled with deionized and degassed water so that the sample was approximately one third submerged in water, and the sample placed under the vacuum for two days. The procedure was then repeated until the sample was completely submerged whence it was left under vacuum for two weeks. The samples in their de-gassed water were then packed into waterproof bottles and sent to the Technical University Graz where they remained in storage until a window of opportunity for testing presented itself. The total period of time the samples spent in the bottles ranged between 1 and 2 months.

Just prior to testing, a sample was removed from the water in the bottles, placed into a grinding machine and the ends ground flat and orthogonal to its axis. Then a Teflon jacket was shrunk around it and the end caps inserted and bound to the jacket ends. The assembly was moved to a servo-controlled, stiff testing machine which was an MTS Rock Testing System 815. A schematic diagram of the testing configuration is shown in Figure 3.1 and a photo of the sample showing the strain sensors is presented in Figure 3.2. The machine was equipped with two axial strain measurement sensors and one circumferential strain measurement system. The MTS axial and circumferential extensometers conform to the standard ISO 9613 class 0.5 and ASTM E83 class B-1 and guarantee a maximum hysteresis of 0.15%, and a non-linearity of 0.10%. Holes
through each end-cap allowed fluid to escape the sample at both ends. The holes connected to a vertical transparent glass stand-pipe with an internal diameter of 5.0 mm so that the volume of fluid expelled or drawn into the sample could be visually estimated to about ±0.5 mm which is equivalent to a volume of ±0.01 ml.

Once assembled, the system was saturated with water at a pressure of 0.03 MPa for 4 to 5 days to eliminate any air pockets from the tubes and the space between jacket and sample. This was achieved by allowing water to flow through the sample in direction from the lower end-cap to the upper one. Lastly, immediately before testing, two hydrostatic load cycles (i.e. seasoning) to 10 MPa were applied to the sample to ensure the jacket fitted snugly.

Hydrostatic loading was applied in ramp-like steps of 10 MPa to a peak load of 70 MPa. The range was chosen to cover stress conditions representative of those applicable for most engineering problems. Records of applied stresses and strains during the loading cycle of Aar granite sample No. 2 are shown in Figure 3.3. After each ramp-like increased had been accomplished (3 minutes), hydrostatic stress was then held constant until the recorded strain equilibrated and remained constant. The water volume squeezed out from the sample by each load increment was then determined by measuring the level change in the standpipe (error of ±0.01 ml). When the peak stress had been reached, the sample was unloaded at around 0.02 MPa/s. No further cycles were conducted with the sample saturated.

After one cycle of saturated testing, the sample was removed from the machine, its jacket removed and the sample dried at 105°C for several days. It was then re-jacketed and subjected to the same loading procedure as before. The test determines the unsaturated and hence 'drained' bulk modulus, $K$, which can be compared with the value obtained from the saturated-but-drained test conducted previously as a consistency check.

To conclude the testing of each dry sample, a series of uniaxial compression tests were performed immediately after the hydrostatic loading tests to determine the drained Young’s modulus, $E$, and drained Poisson’s ratio, $v$. Loading was increased continuously up to a peak of 55 MPa and then returned to 18 MPa. This was followed by two complete loading and unloading cycles between the stress range of 18 to 55 MPa.

### 3.4 Results

The volume change of the rock specimen, $\Delta V$, and the volume of water expelled, $\Delta V_a$, after equilibrium had been achieved at each stage of hydrostatic loading of the three saturated samples are shown in Figure 3.4. The values of $\Delta V$ and $\Delta V_a$ defined at equilibrium for each 10 MPa loading increment are listed in Table 3.1 together with the resulting estimate of Biot’s coefficient derived using Equation (3). In most cases, during the first loading step between 0 to 10 MPa, the volume of water expelled was greater than the volumetric contraction of the rock sample itself (Figure 3.4). This resulted in the calculation of Biot’s coefficient estimates greater than 1.0, which are unrealistic. The error is probably due to the extrusion of excess water stored between the specimen and the jacket, an effect that would be greatest at low confining stress conditions when the storage volume is greatest. For this reason we believe the estimates at higher
confining stresses are more accurate since then the jacket is compressed increasingly tightly on the sample. However, we cannot rule out the possibility that some small contribution persists at higher stress levels. To quantitatively evaluate the effect, it is planned to conduct further tests on Ailsa Craig microgranite which has negligible connected porosity. In any case, Figure 3.4 shows that the net volume of water expelled from the sample during each load increment declines with higher stress. Evidently, at still higher stress conditions than those applied on these tests, the quantities of water drained from the sample will become extremely small.

During the tests on saturated samples, the times required for the strains that followed each loading step to totally stabilise ranged between 1000 and 2000 seconds (Figure 3.5). Immediately after each 10 MPa increase in hydrostatic stress, the fluid pressure within the sample will have become elevated above the ambient pressure of the fluid system (essentially atmospheric) reflecting the undrained response. The transient dissipation of this excess pore pressure within the sample is controlled by its hydraulic diffusivity, $D_s$. Thus, knowledge of the time taken for the pressure to dissipate, and consolidation to be complete, places constraints on $D_s$. Terzaghi (1925) gives an expression for calculating $D_s$ from the time required to achieve 90% consolidation, $t_{90\%}$, in a sample of length $L$ drained symmetrically from both ends:

$$D_s = \frac{0.85(L/2)^2}{t_{90\%}}$$

(4)

This equation assumes an instantaneous step-change in pressure within the sample, symmetrical, one-dimensional pore pressure diffusion to both end faces, constant axial stress and zero radial strain in the specimen during equilibration. The times to 90% consolidation were estimated from the volume-strain response curves to the loading, as shown in Figure 3.5 and are listed in Table 3.2 together with the estimates of diffusivity derived from Equation (4). The values estimated for $D_s$ range between 8.4e-6 and 3.1e-6 m²/s and decrease with increasing hydrostatic loading. Values published for $D_s$ by Detournay (1993) are 7.0e-6 for the Charcoal granite (porosity 0.02) and 2.2e-5 for the Westerly granite (porosity 0.01). Analysis of a pulse decay test by Hart and Wang (2001) for one Barre granite sample (porosity 0.0085) results in a hydraulic diffusivity of 1.5e-6m²/s. It should be recognized that the estimates of diffusivity are only approximate. The primary limitation arises from the fact that each increment in applied load took 3 minutes to reach 10 MPa, rather than being instantaneously increased as assumed by the relation.

As noted earlier, following the saturated tests, the samples were dried and re-tested under identical hydrostatic loading. If the saturated tests were truly drained, then the slope of the stress-volumetric strain curve (i.e. the bulk modulus curve) should be the same as obtained in the dry tests. The stress-volumetric strain curves for both saturated and dry tests of the three samples are shown in Figure 3.6. We also show the dry curves translated so as to pass through the saturated curve after equilibration at the 10 MPa level. It can be seen that the translated curves pass through the equilibration points of the saturated curves at all pressures above 10 MPa, implying that these points do indeed reflect the cumulative strains under fully-drained conditions. That the dry and saturated-drained curves (defined by the points) are not consistent for the 0-10 MPa loading step probably reflects the effects of sample seating at the lowermost stress step.
It is evident from Figure 3.6 and 3.7 that the transient strains of the dry samples during periods when the load was held constant, although much smaller than those of the saturated samples, are not zero, indicating creep was occurring. This is seen more clearly in the enlargement of the first loading steps of the Aar granitic gneiss No. 1 and No. 2 in Figure 3.7 which is presented in Figure 3.8.

The stress-volumetric strain curves in Figure 3.6 are non-linear and show non-reversible permanent deformations. It is suspected that the permanent deformations are largely due to seating and jacketing rather than true permanent deformation of the rock. This is suggested by the fact that the loading curves for the drained-saturated and unsaturated cases are almost identical at 10 MPa and above (i.e. can be regarded as essentially the first and second cycles of a single test). The difference between loading and unloading cycles is probably largely due to hysteresis, which is controlled by micro-cracks. In addition, also the non-linear behaviour of the stress-volumetric strain curves is most likely related to closure of micro-cracks present in the rock (Walsh 1965). As such, the approximation of a linear elastic relationship seems not to be applicable or is locally applicable over a limited range of stresses.

Since the loading state is hydrostatic, the axial and lateral strains should be the same if the rock is elastically-isotropic. However, Figure 3.9 shows that the lateral strains are invariably smaller than those measured in the axial direction, implying elastic anisotropy. The difference is greater for Aar granite samples which also showed the stronger visible foliation (all samples were cored normal to the perceived foliation plane). However, it is uncertain whether the elastic anisotropy is mineralogical (i.e. foliation) or whether it is due to the presence of oriented micro-cracks that either exist in situ, due perhaps to contemporary tectonic stress (e.g. Crampin 1987), or were induced through the sampling process.

The hydrostatic loading tests on the dry samples, were followed immediately by a uniaxial loading test to determine Young's modulus and Poisson's ratio. The curves of stress and strain are shown for the three samples in Figure 3.10. Young's modulus and Poisson's ratio were derived from these load-unload cycles and not from the entire loading path (Table 3.3). Since, Biot's coefficient is more naturally associated with hydrostatic loading than uniaxial loading, determination of Biot's coefficient from uniaxial rock parameters (i.e. Young's modulus and Poisson's ratio) seems to be not meaningful. Thus, the drained bulk modulus estimates used in computing Biot's coefficient from Equation (2) were only taken from the hydrostatic loading tests on dry samples. Equation (2) requires an estimate of the bulk modulus of the solid constituent, K_s. As noted earlier, this was not measured owing to the difficulty in conducting unjacketed hydrostatic compression tests. Thus, the range of possible values was estimated from consideration of the mineralogical composition of the samples. Strict lower and upper bounds are given by intrinsic bulk modulus of quartz crystal (K_s=36 GPa) and plagioclase/alkali-feldspar (K_s=60 GPa) respectively (Hearmon 1979, 1984). The appropriate value of K_s for the granitic samples is intermediate between these extremes and is set to 40 GPa as the lower and 50 GPa as the upper bound. Assuming values for K_s above 50 GPa would increase the magnitude of the Biot's coefficient (Figure 3.11).

The estimates of Biot's coefficient derived from the hydrostatic loading tests using the two methods are shown in Figure 3.11 and listed in Table 3.1. In general, the estimates derived from the relationship 1-K/K_s are in accord with those derived from the
alternative relationship $\Delta V_n/\Delta V$, although at higher hydrostatic stresses the difference begins to increase (Figure 3.11). Standard error bars were obtained from Gaussian error propagation rule and include measurement inaccuracies related to the sample volume, water volume and rock strain.

3.5 Discussion

The results show that the Biot's coefficient of low-porosity granitic rocks is sufficiently high to contribute to intact rock matrix deformation if pore pressure changes occur. In addition, tests indicate a tendency for the Biot's coefficient to decline with increased hydrostatic stress level. This effect can be explained through the non-linear stress-strain behaviour often observed in granitic samples as well as numerous other rock types (Fabre and Gustiewicz 1997). The most likely source for this overall non-linearity is the presence and closure of micro-cracks (Walsh 1965). The origin of such cracks in our samples cannot be clearly identified or attributed to just one process. For example, brittle micro-fracturing may be induced in situ through tectonic stress (Crampin 1987), or through unloading when the in situ stress acting on the sample is relaxed, such as occurred in our samples when they were exposed by tunnel construction, and then removed by coring (e.g. Kowallis and Wang 1983; Eberhardt et al. 1999; Martin 1994). In this regard, Schild et al. (2001) measured the in situ interconnected matrix porosity at the Grimsel Test Site in the Central Swiss Alps by injecting resin into a borehole and then overcoring it. They compared the porosity of the resin-impregnated overcores with that of the core from the original hole and found the latter was 2-2.5 times greater. They concluded that this discrepancy was a result of sample preparation, stress relaxation and drilling. If the porosity of the samples we tested has been significantly enhanced by tunneling and extraction through the inducement of micro-cracks, then the estimates of Biot’s coefficient shown in Figure 3.7 would be greater than in situ values. Since newly-induced micro-cracks tend to close when external stress is applied, the Biot's coefficient estimates obtained in our tests at applied hydrostatic stresses between 13 and 32 MPa (i.e. $\alpha$ between 0.6 and 0.8) are possibly more representative of in situ values. Because of the uncertainty as the presence of a significant population of micro-cracks in situ, the Biot's coefficient estimates obtained at hydrostatic stress levels above 50 MPa represent lower bounds on the in situ values (Figure 3.11). Nonetheless, it remains true to say that the higher values of Biot's coefficient obtained at hydrostatic loads in the range above 50 MPa apply to low porosity rocks since the porosity of the samples was measured at atmospheric pressure and found to lie in the range 0.007 to 0.009.

Figure 3.12 compares the results of this study with published estimates taken from the literature. Given the differences in the rock types, the results are in very reasonable accord, both in terms of the absolute values and the trend with increasing hydrostatic stress. This is particularly true of Brace's (1965) estimates for Westerly and Stone Mountain granites, although his values are not direct measurements but rather are similar to our 'dry' estimates inasmuch as the intrinsic bulk modulus was estimated from mineralogy. Hart and Wang (2001) determined the poroelastic constants and flow parameters of one Barre-granite specimens (porosity 0.0085) from a single transient pulse decay test and found $\alpha$ to be 0.88 at an effective confining stress of 8 MPa. Ito
and Hayashi (1991) measured the drained bulk moduli, $K_s$ (jacketed test) and the bulk moduli of the solid constituent, $K_s$ (un jacketed test) of two andesites over the stress range 0-30 MPa. They inferred Biot’s coefficients of 0.53 for the Kofi andesite (porosity, 4.6%) and 0.82 for the Honkomatsu-andesite (porosity, 5.4%), but did not find any change of these values with stress level.

It is relevant to mention some determinations of Biot's coefficient for sedimentary rocks. Fabre and Gustkiewicz (1998) performed detailed experiments on several different limestones and sandstones and found that the magnitude of the Biot’s coefficient was controlled by the interconnected rock porosity. For sedimentary rock types characterized by porosities lower than 5%, Biot’s coefficient values fell below 0.4 and were stress independent as their values were determined from the linear elastic range of the drained bulk modulus curve. Warpinski and Teufel (1993) measured the effective stress law for deformation on carbonate rocks and found that most values of Biot’s coefficients were in the range between 0.7 and 1.0. The only exceptions were samples of Austin Chalk which yielded Biot's coefficient estimates in the range 0.25-0.35. Hart and Wang (1995) report an extensive testing campaign to determine the poroelastic parameters of highly porous sandstones with porosities between 13% and 19%. They report values of Biot’s coefficient in the range between 0.64 and 0.81.

### 3.6 Conclusion

Although Biot's coefficient is an important poroelastic parameter for many geotechnical problems related to tunnelling, geological waste disposal and deep heat mining, very few measurements are reported in the current literature for crystalline rocks. In this study, two crystalline rock types penetrated by the Gottard road tunnel were subjected to hydrostatic loading tests to determine Biot's coefficient to 70 MPa. Two approaches to estimating the parameter were employed: one measured the ratio of the fluid volume expelled from the sample to the total sample volume strain; the other measured the ratio of the drained bulk modulus of the sample to the bulk modulus of the solid constituent (estimated from the mineralogical composition of the samples). The estimates of Biot's coefficient obtained from both methods were in accord to within the error estimates. Values for the Aar and Gamsboden granitic gneiss of Central Switzerland were found to range between 0.55 ±0.12 and 0.80 ±0.08 for the stress range of 10-50 MPa. Such values demonstrate that crystalline rocks with measured porosities as low as 0.007-0.009 can have high values of Biot's coefficient. Thus, coupling between pore pressure and stress is strong even for intact, low-porosity crystalline rocks.

The estimates of Biot's coefficient generally declined with increasing hydrostatic stress level, consistent with expectations based on the closure of micro-cracks. We consider our best estimates to be those obtained at stress levels comparable to those prevailing under pre-disturbance in situ conditions since this will tend to close that population of micro-cracks that have been induced by relaxation of these stresses.

Unrealistically high values for Biot's coefficient were obtained at the lowest stress levels, most likely because of storage of residual water between the sample and the jacket. Future work will examine this possibility by testing samples of a near-zero connected porosity rock (i.e. Ailsa craig granite from Great Britain). In addition,
samples of rocks collected from the ground surface above the Gottard tunnel will be tested, and an instantaneous water volume measuring system will be used.

3.7 Acknowledgements

We would like to thank Ulrich Schärli for the loan of the vacuum chamber and Thomas Jaggi for assistance in drilling and cutting the core samples. In addition, we thank Dr. Wulf Schubert of the Technical University in Graz for his kind support on the tests.
Table 3.1: Change in volume of fluid expelled, change in volume of the specimen, determinations for the Biot’s coefficient, $\alpha$, from saturated and dry tests, and errors for each loading step (i.e. stress interval of the saturated specimen).

### Aar granitic gneiss No. 1

<table>
<thead>
<tr>
<th>Hydrostatic stress interval (MPa)</th>
<th>$\Delta V_n$ (mm$^3$)</th>
<th>$\Delta V$ (mm$^3$)</th>
<th>$\alpha = \Delta V_n / \Delta V$</th>
<th>Error</th>
<th>$\alpha = 1 - K / K_s$ K=50 GPa</th>
<th>$\alpha = 1 - K / K_s$ K=40 GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 to 10.6</td>
<td>420</td>
<td>421</td>
<td>1.00</td>
<td>±0.04</td>
<td>0.94 to 0.81</td>
<td>0.91 to 0.76</td>
</tr>
<tr>
<td>10.6 to 20.7</td>
<td>180</td>
<td>253</td>
<td>0.71</td>
<td>±0.06</td>
<td>0.81 to 0.75</td>
<td>0.76 to 0.67</td>
</tr>
<tr>
<td>20.7 to 31.0</td>
<td>140</td>
<td>196</td>
<td>0.71</td>
<td>±0.08</td>
<td>0.75 to 0.69</td>
<td>0.67 to 0.60</td>
</tr>
<tr>
<td>31.0 to 40.6</td>
<td>90</td>
<td>159</td>
<td>0.57</td>
<td>±0.10</td>
<td>0.69 to 0.65</td>
<td>0.60 to 0.55</td>
</tr>
<tr>
<td>40.6 to 50.4</td>
<td>70</td>
<td>126</td>
<td>0.55</td>
<td>±0.12</td>
<td>0.65 to 0.61</td>
<td>0.55 to 0.50</td>
</tr>
<tr>
<td>50.4 to 60.4</td>
<td>40</td>
<td>128</td>
<td>0.31</td>
<td>±0.12</td>
<td>0.61 to 0.57</td>
<td>0.50 to 0.45</td>
</tr>
<tr>
<td>60.4 to 69.6</td>
<td>30</td>
<td>111</td>
<td>0.27</td>
<td>±0.13</td>
<td>0.57 to 0.54</td>
<td>0.45 to 0.43</td>
</tr>
</tbody>
</table>

### Aar granitic gneiss No. 2

<table>
<thead>
<tr>
<th>Hydrostatic stress interval (MPa)</th>
<th>$\Delta V_n$ (mm$^3$)</th>
<th>$\Delta V$ (mm$^3$)</th>
<th>$\alpha = \Delta V_n / \Delta V$</th>
<th>Error</th>
<th>$\alpha = 1 - K / K_s$ K=50 GPa</th>
<th>$\alpha = 1 - K / K_s$ K=40 GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 to 10.4</td>
<td>440</td>
<td>408</td>
<td>1.08</td>
<td>±0.04</td>
<td>0.97 to 0.83</td>
<td>0.95 to 0.77</td>
</tr>
<tr>
<td>10.4 to 20.5</td>
<td>210</td>
<td>264</td>
<td>0.80</td>
<td>±0.06</td>
<td>0.83 to 0.76</td>
<td>0.77 to 0.69</td>
</tr>
<tr>
<td>20.5 to 30.3</td>
<td>110</td>
<td>183</td>
<td>0.60</td>
<td>±0.08</td>
<td>0.76 to 0.71</td>
<td>0.69 to 0.62</td>
</tr>
<tr>
<td>30.3 to 40.6</td>
<td>100</td>
<td>145</td>
<td>0.69</td>
<td>±0.11</td>
<td>0.71 to 0.66</td>
<td>0.62 to 0.57</td>
</tr>
</tbody>
</table>

### Gamsboden granitic gneiss No. 1

<table>
<thead>
<tr>
<th>Hydrostatic stress interval (MPa)</th>
<th>$\Delta V_n$ (mm$^3$)</th>
<th>$\Delta V$ (mm$^3$)</th>
<th>$\alpha = \Delta V_n / \Delta V$</th>
<th>Error</th>
<th>$\alpha = 1 - K / K_s$ K=50 GPa</th>
<th>$\alpha = 1 - K / K_s$ K=40 GPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 to 11.5</td>
<td>250</td>
<td>240</td>
<td>1.04</td>
<td>±0.07</td>
<td>0.89 to 0.79</td>
<td>0.85 to 0.72</td>
</tr>
<tr>
<td>11.5 to 22.0</td>
<td>150</td>
<td>187</td>
<td>0.80</td>
<td>±0.08</td>
<td>0.79 to 0.72</td>
<td>0.72 to 0.64</td>
</tr>
<tr>
<td>22.0 to 32.4</td>
<td>110</td>
<td>155</td>
<td>0.71</td>
<td>±0.10</td>
<td>0.72 to 0.67</td>
<td>0.64 to 0.58</td>
</tr>
<tr>
<td>32.3 to 42.0</td>
<td>80</td>
<td>130</td>
<td>0.61</td>
<td>±0.11</td>
<td>0.67 to 0.63</td>
<td>0.58 to 0.53</td>
</tr>
<tr>
<td>42.0 to 52.5</td>
<td>80</td>
<td>133</td>
<td>0.60</td>
<td>±0.11</td>
<td>0.63 to 0.59</td>
<td>0.53 to 0.48</td>
</tr>
<tr>
<td>52.5 to 61.3</td>
<td>30</td>
<td>99</td>
<td>0.30</td>
<td>±0.15</td>
<td>0.59 to 0.55</td>
<td>0.48 to 0.43</td>
</tr>
</tbody>
</table>
Table 3.2: Estimation of the hydraulic diffusivity from the time required to achieve 90% consolidation for each loading step for Aar granitic gneiss No. 2.

<table>
<thead>
<tr>
<th>Aar granitic gneiss No. 2</th>
<th>Loading step</th>
<th>Hydrostatic stress interval (MPa)</th>
<th>Time 90% strain (s)</th>
<th>Hydraulic diffusivity (m$^2$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>0.0 to 10.4</td>
<td>291</td>
<td>8.4e-6</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>10.4 to 20.5</td>
<td>646</td>
<td>3.8e-6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>20.5 to 30.3</td>
<td>792</td>
<td>3.1e-6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>30.3 to 40.6</td>
<td>792</td>
<td>3.1e-6</td>
</tr>
</tbody>
</table>

Table 3.3: Drained intact rock parameters determined from the uniaxial compression test.

<table>
<thead>
<tr>
<th>Rock units</th>
<th>Young's modulus, E (GPa)</th>
<th>Poisson's ratio, $v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aar granitic gneiss No. 1</td>
<td>42.9</td>
<td>0.095</td>
</tr>
<tr>
<td>Aar granitic gneiss No. 2</td>
<td>47.2</td>
<td>0.125</td>
</tr>
<tr>
<td>Gamsboden granitic gneiss No. 1</td>
<td>48.9</td>
<td>0.150</td>
</tr>
</tbody>
</table>
Figure 3.1: Schematic diagram of equipment set-up.

Figure 3.2: Experimental set up showing the cylindrical granitic test specimen covered by a transparent teflon jacket and the sensors used to measure circumferential and axial strain.
Figure 3.3: History of axial and circumferential strains resulting from the series of ramp-like increases of hydrostatic stress. Each increase of 10 MPa took 3 min to attain.
Figure 3.4: Variation of the cumulative volume of water expelled and cumulative change in rock volume with applied hydrostatic stress. The plateaus in the rock volume curve represent deformation under constant stress conditions.
Figure 3.5: Blow-ups of the strain response to the loading steps shown in Figure 3.3 demonstrating that equilibration was essentially achieved before further loading. For the calculations of the hydraulic diffusivity, the times for the implied volume strains to reach 90% of their final value were used.
Figure 3.6: Cumulative volumetric strain measured for the saturated and dry rock samples as a function of applied hydrostatic stress. The black circles indicate the equilibrium strain for the saturated samples. The grey curves represent the horizontally shifted response of the dry rock sample so as to pass through the saturated curve after equilibrium at the 10 MPa level.
Figure 3.7: History of strain in the dry rock samples during the loading cycle. Expanded views of the first loading steps are shown in Figure 3.8.
Figure 3.8: Magnification of the first loading step for the dry Aar granitic gneiss No. 1 and No. 2. Small creep strains are evident.
Figure 3.9: Comparison of axial to lateral rock strains for hydrostatic loading tests on saturated samples.
Figure 3.10: Uniaxial compression tests showing axial and radial strains. The loading and unloading curves from where the Young's modulus and Poisson's ratio were determined are shown.
Figure 3.11: Compilation of Biot's coefficient estimates from this study. The continuous curves represent $\alpha$ values calculated from $\alpha=1-K/K_s$ where $K$ was measured (Figure 3.6) and $K_s$ estimated from the grain compressibility ($K_s=40$ and 50 GPa). Coloured bars indicate directly measured Biot's coefficients based on the drained hydrostatic compression test. The bar represents a mean $\alpha$ coefficient for the stress range of 10 MPa.
Figure 3.12: Comparison of Biot's coefficients obtained from this study with published estimates for crystalline rocks. Error bars were not included. See text for discussion.
4. Generic 2-D Studies in Vertical Tunnel Cross Sections

C. Zangerl, E. Eberhardt and S. Loew

This Chapter was published in Hydrogeology Journal, 2003, vol. 11, pp 162-173

Ground Settlements above Tunnels in Fractured Crystalline Rock: Numerical Analysis of Coupled Hydro-Mechanical Mechanisms
Abstract:

Vertical settlements with magnitudes reaching 12 cm were measured in fractured crystalline rock several hundred meters above the Gotthard highway tunnel in central Switzerland. Such magnitudes of surface subsidence were unexpected, especially in granitic gneisses and appear to be related to large-scale consolidation of fractures resulting from fluid drainage and pore pressure changes following tunnel construction. This paper focuses on the mechanisms involved in the development of such surface displacements and presents the preliminary results of 2-D discontinuum (i.e. distinct-element) and 2-D continuum modelling (i.e. finite-element). Results show that settlements are most sensitive to horizontal joints, as would be expected, but that vertical fractures also contribute to the settlement profile through a 'Poisson ratio' effect. However, these models also suggest that fracture deformation alone cannot explain the total subsidence measured. As such, 2-D poroelastic finite-element models are presented to demonstrate the contributing effect of consolidation of the intact rock matrix.

4.1 Introduction

Settlements in fractured crystalline rocks are rarely observed and in the past engineers would not expect substantial subsidence to be generated above a deep seated tunnel. Large-scale displacements in such projects can have a negative influence on surface structures especially with respect to concrete dams, bridge piers and abutments, and other subsidence sensitive structures. For example, subsidence-induced cracks if generated may threaten the integrity of these structures leading to costly repairs and/or possible failure (e.g. see the Zeuzier Dam, Lombardi 1988).

Recent high precision levelling measurements of surface displacements along the Gotthard pass road in central Switzerland, have revealed up to 12 cm of subsidence along sections that pass several hundred metres above the Gotthard A2 highway tunnel (Figure 4.1 and 4.2). The Swiss Federal Office of Topography carried out the levelling measurements in 1993/98 as a closed loop over the old Gotthard-pass road and through the A2 road tunnel. Two earlier measurement campaigns were made along this N-S profile over the old Gotthard pass road in 1918 and 1970. During the time interval between 1918 and 1970 (i.e. before tunnel construction), an undisturbed alpine uplift with a rate of 1 mm/year was detectable (Figure 4.3). This uplift rate concurs with estimated rates of 0.6 mm/year as determined using fission-track techniques (Kohl et al. 2000). In contrast, the time interval between 1970 and 1993/98 (i.e. after tunnel construction) shows significant downward displacements along a 10 km region above the tunnel (Figure 4.3).

The close spatial proximity between maximum tunnel inflow rate and maximum settlement (Figure 4.3) and the temporal relationship between tunnel construction and settlement clearly shows causality between water drainage into the tunnel and surface deformation. Localized surface processes, e.g. creeping landslides or flexural toppling, could be excluded as alternative explanations given the absence of local indicators and the extent over which the settlements were measured (10 km along a N-S line, roughly parallel to the tunnel axis; see Figure 4.2).
Maximum settlements and high initial inflow rates into the tunnel (Figure 4.3) were measured along sections involving heavily fractured granitic-gneiss (Gamsboden granitic-gneiss). Surprising was the relatively small tunnel interval over which the high initial inflow rates occurred (3 km) relative to the measured settlement trough (10 km). Subsidence of this magnitude in a fractured crystalline rock mass would not generally be expected and appears to be related to large-scale consolidation resulting from fluid drainage and pore pressure changes in the rock mass (i.e. fractures and intact rock).

A more detailed description of the problem can be found in Zangerl et al. (2001). The hydrogeological situation in the Gotthard area is described in Loew (2001) and Luetzenkirchen (2002). This paper presents the first results of a comprehensive modeling exercise and focuses on the mechanisms involved in the development of such surface displacements through the application of 2-D coupled hydro-mechanical discontinuum modelling using the distinct-element code UDEC (Itasca 1999). In addition, 2-D poroelastic continuum modelling of the intact rock material and smaller scale fractures is included using the finite-element program VISAGE (VIPS 2001). All models presented in this paper are generic and conceptual. Realistic site specific models, which honour the local topographical, geological and hydrological conditions, and which are calibrated with the site specific hydro-mechanical observations, are to be presented in subsequent papers.

4.2 Flow and Deformation Models for the Underlying Subsidence Mechanisms

4.2.1 Fluid Flow and Fluid Pressure

The excavation of a tunnel in a water-saturated fractured crystalline rock mass enables inter-connected brittle fault zones and joints to drain. Initially this quick fluid drainage would cause a drop in water pressure along fractures adjacent to the tunnel. In a later stage, the pressure change would penetrate more deeply into the rock mass, lowering the phreatic water table until a new equilibrium between water inflow into the tunnel and far-field water recharge is reached. In addition to the drainage of the more permeable fracture network, pore pressures in the low-permeability intact rock matrix will slowly adjust to the new boundary conditions. The time taken to reach equilibrium between the fluid pressure in the fractures and the fluid pressure in the intact rock matrix is strongly dependent on the matrix block size and the hydraulic diffusivity.

These pore pressure changes will affect the effective stress conditions, which in turn influence the hydraulic flow field either through variations in the fracture aperture or the porosity within the intact rock matrix. A decrease in the mechanical aperture (i.e. fracture width) as a result of pore pressure change also means a decrease in the hydraulic aperture, and thus a decrease in the permeability of the fractures. This aperture-flow coupling relationship is often represented by the cubic law equation for laminar flow between two parallel plates with smooth surfaces:

\[ Q = \frac{ga^3 dp}{12v dl} \tag{1} \]
where: \( Q \) = flow rate; \( g \) = gravitational acceleration constant; \( a \) = mean fracture aperture; \( v \) = kinematic viscosity of the fluid; \( dp/dl \) = hydraulic gradient; \( w \) = fracture width in the direction of flow.

Given the simplifying assumptions in its formulation, it must be questioned whether the cubic law is valid for fluid flow in fractures with large aperture and hydraulic gradient around the tunnel. In addition, Pyrak-Nolte and Morris (2000) suggest that stress alone is not the link between hydraulic and mechanical fracture properties. Instead, they suggest that the fracture geometry and how it deforms under stress provides this link. Regardless, the cubic law provides a simple and numerically efficient approximation to laminar fluid flow in fractures. As such the cubic law was used in this study, in part, to model deformation mechanisms relating to pore pressure changes. Furthermore, as the quantification of inflow rates into the tunnel was not an objective of this study, limitations relating to the use of the cubic law relationship are reduced. In UDEC, the fractures are viewed as defining a network of interconnected voids and channels that will be referred to as ‘domains’ (Itasca, 1999). Changes of domain pore pressures were calculated, taking into account the net flow into the domain, and possible changes in domain volume due to the incremental motion of the surrounding blocks. The pore pressure change \( \Delta p \) is represented by:

\[
\Delta p = \frac{K_w}{V} \left( \sum Q \cdot \Delta t - \Delta V \right)
\]  

(2)

where: \( \sum Q \) is the flow into the node, \( \Delta V \) is the mechanical volume change, \( K_w \) is the bulk modulus of the fluid, and \( \Delta t \) is the timestep. Fracture deformation and hydraulic apertures were calculated as a function of the effective stresses and normal stiffness of the joints. The hydraulic aperture, \( a \), is given by:

\[
a = a_0 + u_n
\]  

(3)

where: \( a_0 \) is the aperture at zero normal effective stress, and \( u_n \) is the contact normal displacement (convention: fracture closure represented by negative number). A minimum value, \( a_{res} \), is assumed for an aperture beyond which mechanical closure does not affect the contact permeability.

4.2.2 Deformation Models

Three different fracture-based hydro-mechanical deformation models were developed for this study. These models, and their respective roles in generating the settlements measured above the Gotthard tunnel were analysed using discontinuum-modelling techniques (i.e. the distinct-element method). A fourth deformation model based on microfracture permeability and poroelastic consolidation of the intact rock matrix was also developed and analysed using continuum finite-element techniques (note that the distinct-element formulation employed by the code UDEC does not allow for fluid flow within the intact blocks, treating them as impermeable).

Each of the deformation models involves either drainage of water filled discontinuities and/or drainage of the intact rock. Water filled discontinuities change
their mechanical aperture during pore pressure drops as can be shown through the effective stress law for fracture closure:

\[ \Delta \sigma_n' = \Delta \sigma_n - \alpha_f \cdot \Delta p_w \]  

(4)

where \( \sigma_n' \) = effective normal stress; \( \sigma_n \) = total normal stress; \( \alpha_f \) = effective stress coefficient; and \( \Delta p_w \) pore pressure (Robin 1973).

The four deformation models can be described as follows:

1. **Horizontal Joint-Controlled**: Intuitively horizontal joint closure through water pressure decrease would contribute the most towards vertical settlements. In this deformation model the total normal vertical stress is assumed to stay constant (or varies only a little during this process); only a change in the normal effective stress across the fracture takes place (Figure 4.4a). As such, the deformation model does not rely on deformation of the intact rock blocks.

2. **Vertical Joint-Controlled**: In contrast, vertical joint closure during the drainage process will affect both the total and effective normal stress acting horizontal to the fracture plane. This change in the horizontal total normal stress with the vertical stress remaining constant would subsequently generate strains within the intact rock blocks (Figure 4.4b). As such, they should experience shortening in the vertical direction and expansion in the horizontal direction (i.e. Poisson’s ratio effect).

3. **Vertical Brittle Fault Zone-Controlled**: Deformation model 3 is similar to model 2 but substitutes the hydro-mechanical properties of the vertically aligned joints for those of vertical brittle fault zones (based on field mapping observations; Zangerl et al. 2001). This differentiation is important given that shear and normal stiffness values for the brittle fault zones are much lower than those for unfilled joints (Figure 4.4c).

4. **Intact Rock Matrix-Controlled**: Deformation model 4 explains the surface deformation by applying the theory of linear poroelasticity to intact low-permeability rock blocks. The effective stress law for a homogeneous poroelastic continuum can be described as:

\[ \sigma_{ij}' = \sigma_{ij} - \alpha \cdot p \cdot \delta_{ij} \]  

(5)

where: \( \sigma_{ij}' \) = effective stress; \( \sigma_{ij} \) = total stress; \( \alpha \) = Biot’s constant; \( p \) = pore pressure; and \( \delta_{ij} \) = Kroenecker’s delta (Nur and Byerlee 1971). In this case, pore space and permeability are attributed to microfractures in the crystalline rock matrix (Figure 4.4d). Values reported for Westerly- and Charcoal granite by Detournay and Cheng (1993), together with recent laboratory tests performed on granitic gneiss from the Gotthard study area, would seem to indicate that some poroelastic deformation could be expected during pore pressure changes within the intact rock matrix. However, these studies are currently ongoing and are not discussed in detail in this paper.
4.3 Discontinuum Modelling

4.3.1 Model Geometry

Based on the first three deformation models outlined above, two sets of continuous, orthogonal (i.e. horizontal and vertical), fully persistent discontinuities were used to form the fracture network. The model is two-dimensional and assumes half-symmetry with a width of 2000 m and a height of 1500 m (Figure 4.5). This model geometry was selected after studying the influence of boundary-effects using a model with an extended width of 4000 m. A 20 m diameter tunnel was placed at 800 m depth, with 8 radial joints added to promote deeper hydraulic interaction. Joint spacing in the orthogonal fracture pattern relates to surface scan-line data from the Gamsboden granitic-gneiss (Zangerl et al. 2001). The mean joint normal-set spacing measured on surface ranges from 0.5 to 1.5 m for both horizontal and vertical joints. Joint data sets from the Gotthard tunnel show that joint spacing increases with depth. Brittle fault zones mapped from the tunnel show a strong preferred orientation (strike direction perpendicular to tunnel axis) and a mean spacing of 35 m (Figure 4.6).

The direct application of field discontinuity data, however, can result in extremely complex models that are unmanageable in terms of computer memory requirements and solution run times. A balance must therefore be found between reproducing the important geological elements (and their effect on the modelled mechanisms), and developing a model that is numerically efficient. As such, the mean joint spacing used in the UDEC models was set to 10 m for horizontal joints and 50 m for vertical joints and faults. By adopting a horizontal joint spacing where 10 joints mapped in the field are replaced by one joint, the properties of the one modelled joint are scaled to those representative of the bulk rock mass (Figure 4.7 – as described in the next section).

4.3.2 Material Properties

All models were solved assuming an elastic constitutive model for the intact block material. Plastic strains would only be expected, at most, along the tunnel periphery <0.5 m into the rock mass and, given the 800 m overburden height between the tunnel boundary and surface, were assumed to be insignificant. The selection of an elastic constitutive material model effectively restricts the subsidence generating deformation mechanisms to those produced through elastic strain of the intact blocks and shear and normal displacements along fractures. Material properties for the intact block parameters were selected based on typical values for granite (Table 4.1).

Intuitively, it is expected that the normal stiffness of joints and/or brittle fault zones would have the largest influence on the calculated subsidence. A detailed literature study was therefore performed to review joint normal deformation behaviour in granitic rocks. Most published studies report laboratory derived normal stiffness values for natural or artificially generated granitic joints (e.g. Sun et al. 1985; Makurat et al. 1990; Bart et al. 2000). Only a few tests report values measured in situ (e.g. Pratt et al. 1977; Makurat et al. 1990). These tests show a strong non-linearity in the normal stress-normal displacement relationship, with a wide range of maximum joint closure values.

The semi-logarithmic law, proposed by Bandis et al. (1983) for unmated joints, was chosen in this study to represent the behaviour of modelled joint closure.
\[ \log_{10}(\sigma'_n) = q \cdot \Delta v + p \]  

where: \( \sigma'_n \) = joint effective normal stress in (MPa); \( q \) and \( p \) = constants (for this study \( q=10; p=-1 \)); and \( \Delta v \) = joint normal deformation (i.e. closure) in (mm). The initial reference normal stress characterizing a joint normal deformation of \( \Delta v = 0 \) reaches a value of \( \sigma'_n = 0.1 \) MPa. This law was fitted to several tests from the literature and their parameters established. Implementation into UDEC required that the semi-logarithmic law be represented as four points along the normal stiffness curve. Figure 4.8 shows the joint displacement curve derived from laboratory and in-situ measurements for a single joint and additionally the four points implemented into UDEC to represent the curve.

Published studies pertaining to normal deformation behaviour of brittle fault zones are more limited than those for joints. Only a few data sets were found in the literature. Martin et al. (1990) report values for a brittle fault zone consisting of fractures, fault breccias and clay-gouge in the Lac du Bonnet granite batholith at the URL-test site in Canada. The test was conducted in a 96 mm-diameter borehole with a specially developed packer system (PAC-ex-system). Results from this test report a stress independent normal stiffness with relatively low values between 2 to 6 MPa/mm. Given the small stress interval applied in these tests (0 to 2 MPa), such values must be used with caution. Similar values, however, were also reported by Infanti and Kanji (1978) for clay-filled joints with normal stiffness values ranging between 0.1 to 5 MPa/mm depending on the thickness of the clay filling. They too concluded a stress independent normal stiffness. In contrast, higher values (approximately an order of magnitude stiffer) were measured \textit{in situ} by Majer et al. (1990) at the Grimsel Rock Laboratory in central Switzerland, for two granite-hosted ductile shear zones overprinted by minor brittle deformation.

An additional consideration with respect to joint normal stiffness is its relationship with mean fracture spacing as deformation controlling parameters. As such, the extent of deformation may be controlled either by varying the mean spacing or normal stiffness. This proves valuable for simplifying large models where realistic field measured spacing values would lead to an unmanageable number of blocks. It’s important to keep in mind however that this relationship is only valid for constant normal stress. An example of its use in the UDEC models is demonstrated in the selection of a 10 m spacing for the horizontal joints in contrast to the 1 m mean spacing measured in the field. Thus one modelled joint represents ten joints in terms of their bulk normal stiffness and deformation characteristics (Figure 4.7). The following equation shows this relationship:

\[ \Delta l = \frac{L}{X} \cdot \frac{\sigma_n}{k_n} \]  

which yields: \( k_{n1} \cdot X_1 = k_{n2} \cdot X_2 \)  

where: \( \Delta l \) = normal deformation; \( L \) = block length; \( X \) = mean spacing; \( \sigma_n \) = constant normal stress; and \( k_n \) = normal stiffness. Normal stiffness values derived through this procedure are given in Table 4.2. Shear stiffness values were assumed to be constant (i.e. non stress dependent) with values for joints and brittle fault zones based on those given by Bandis et al. (1983). A rock mass hydraulic conductivity of 2e-6 m/s is
calculated from residual fracture apertures in vertical and horizontal direction and the cubic law (Equation 1). Such values are indicative for medium permeable fault zones in the Gotthard area. Conductivity values observed in an intact fractured granitic rock mass are typically two magnitudes lower (Loew 2001).

4.3.3 Boundary and Initial Conditions

Initial total stress conditions were set to $K=1$ (horizontal to vertical stress ratio equals one) with vertical stresses determined by gravitational loading. Initial hydrostatic pore pressures were set by assuming a groundwater table at surface (Figure 4.5). The effective stresses in the fractures were initialized as the difference between the normal component of the initial total stress acting across the discontinuity plane and the initial pore pressures within the discontinuity.

Two different models in terms of hydraulic boundary conditions were established. All models were solved assuming steady state conditions and the pore pressures were calculated as relative pressures (i.e. 0 Pa in UDEC models were equivalent to atmospheric pressure). The first condition represents a free water surface with no recharge, that allows drawdown due to tunnel drainage (boundary condition A). Depending on the frequency and hydraulic apertures of the fractures (i.e. the equivalent rock mass hydraulic conductivity), the phreatic-surface could drop down as far as the tunnel elevation. This condition, therefore, represents the maximum pore pressure drop that could be achieved in the rock mass. Whether probable or not, this condition was chosen so that each discontinuum model could be directly compared to one another (e.g. deformation model 1 vs 2, 2 vs 3, etc.). By modelling a similar redistribution of pore pressures in each case, a more direct comparison could be made between the resulting subsidence profiles.

The second hydraulic boundary condition involves a fixed water level that allows drainage of the rock mass around the tunnel but without changing the water table (boundary condition B). The reason for establishing this condition was to check if for a more limiting drainage condition, i.e. one which would not alter surface groundwater conditions (e.g. springs), considerable surface subsidence could still result. Based on field observations, it is believed that condition A is the more likely situation for high-permeable structures (i.e. a network of brittle fault zones or fracture zones) that allow locally large values of drawdown. In contrast, boundary condition B can be applied to models, where on regional scale negligible changes in the water surface occur mainly due to direct infiltration from surface water bodies and strong recharge. Both types of boundary conditions are discussed in more detail in Loew (2002).

4.4 Continuum Modelling

4.4.1 Model Development

For the fourth deformation model, that of poroelastic consolidation of the intact rock matrix (Figure 4.4 d), a 2-D finite-element model was generated. The same model geometry/dimensions were used as those for the discontinuum models, but calculated as full model (Figure 4.9). Similarly, the intact rock properties, in situ stresses and
boundary conditions were kept the same. Values for the Biot’s and Skempton coefficients, α and β, were chosen to allow maximum consolidation for the prescribed permeability. The water table level along the top of the model was fixed in the same manner as ‘boundary condition B’ described for the discontinuum models. This meant that drainage would be restricted to the rock mass immediately surrounding the tunnel, and no drawdown of the water surface occurs. The permeability values used in the continuum models were based on those for an intact fractured granitic rock mass (i.e. without fault zones). Given the aim of the continuum modelling was to study the poroelasticity effects of the intact rock matrix, mechanical and hydraulic input parameters (i.e. Young’s modulus, Poisson’s ratio and hydraulic conductivity) were defined according to intact granitic rock. These values are summarized in Table 4.3.

4.5 Results and Discussion

4.5.1 Boundary Condition A: Groundwater Drawdown to Tunnel Level

Each UDEC model presented in this paper includes both vertical and horizontal fractures. Depending on the deformation models tested, the normal stiffness of the vertical and horizontal fractures was varied and its effect on surface displacements calculated. For the first model series (boundary condition A), a free water surface was assumed enabling drawdown of the water table close to the tunnel level. After setting the initial stress and fluid pressure conditions and cycling them to equilibrium, the tunnel was opened allowing fluid flow (i.e. drainage). The resulting pore pressure drop throughout the fracture network (Figure 4.10) in turn generated vertical and horizontal block displacements.

Horizontal joints, as was expected, were found to have the biggest influence on subsidence. Depending on the values of normal stiffness or mean joint spacing used, surface settlements ranging between 0.005-0.22 m were modelled. For example, implementing an exceptionally low normal stiffness ($k_n = 0.3$ MPa/mm between 0 and 5 MPa normal stress) resulted in a maximum settlement of 0.22 m. If a more realistic normal stiffness curve was used, and scaled to represent the stiffness of 10 joints within a 10 m interval ($k_n = 3$ MPa/mm between 0 and 5 MPa normal stress), the settlements only reach 0.03 m (Figure 4.11). The shape of the settlement trough in the model plane (i.e. perpendicular to the levelling profile) was generally flat, extending towards the left model boundary (i.e. away from the tunnel), with vertical displacements still reaching up to 20 % of the maximum value 1500 m from its centre. The point of maximum settlement in these models was observed directly above the tunnel.

Similar results were obtained for models focusing on vertical joint and vertical brittle fault zone mechanisms contributing to surface settlements (i.e. deformation models 2 and 3). Maximum settlements of only 0.036 m were calculated. The corresponding decrease in total horizontal and vertical stress throughout the model was found to be less than 2 MPa. Only in the immediate area surrounding the tunnel was a larger stress change observed. As hypothesized and described in reference to the deformation models, the modelled strains depict closure of the vertical fractures and horizontal expansion of the intact blocks – i.e. the Poisson Ratio effect (Figure 4.12).
In comparison, modelled intact block strains in the vertical direction clearly show expansion below and partly above the tunnel and contraction throughout the rest of the model (Figure 4.13). The maximum amount of vertical shrinkage is located several hundred metres away from the tunnel. Positive and small negative vertical intact block strains above and below the tunnel (i.e. showing mainly expansion) are mostly due to the tunnel excavation and the pore pressure decrease near the tunnel. Horizontal joint closure due to changes in the vertical effective stress continuously increases from the left to right model boundary and generates vertical shear deformation between adjacent columns of blocks. Directly above and below the tunnel the amount of shear displacement along vertical fractures is small in comparison to horizontal joint closure. Fractures located further away from the tunnel show the opposite behaviour. They are characterized by large shear deformation combined with small horizontal joint closure (Figure 4.14). Low values of vertical shear deformation together with large values of horizontal joint closure results in vertical expansion of the intact blocks. In contrast, large shear displacements combined with minor closure of the horizontal joints creates vertical block shrinkage and horizontal expansion. The amount of shear offset shown in Figure 4.14 results from the summation of horizontal joint closure values along a column of blocks and doesn’t originate from deformation on one single fracture. Within the model shear occurs at elastic deformation conditions (i.e. no shear failure occurred), attaining maximum values of 1.4 mm. As such, results for deformation models 3 suggest that the point of maximum subsidence may not occur directly above the tunnel (Figure 4.15). Vertical fractures therefore can affect the shape and location of the settlement trough if vertical displacements are dominated by vertical fracture closure and intact block deformation as opposed to horizontal joint closure (as shown for deformation model 1).

Relatively high inflow rates into the tunnel of 12 l/s/m were modelled at steady state conditions. This is due to the elevated equivalent rock mass hydraulic conductivity, the large tunnel diameter and the ‘drainage fractures’ implemented in the model. The modelled hydraulic parameters (i.e. inflow rates, water table drawdown) are typical for conductive brittle fault zones in the Gotthard region.

4.5.2 Boundary Condition B: Fixed Water Table

Models in which pore pressures were fixed along the top boundary (i.e. fixed water table) roughly show a 20-50% reduction in subsidence magnitudes. Figure 4.16 shows the pore pressure distribution for these models subsequent to tunnel drainage. The extent and shape of the reduced pore pressure zone is strongly controlled by the hydraulic apertures of the horizontal and vertical fractures, as well as the aperture-ratio between them. Thus surface deformation depends largely on these hydraulic parameters for the case of a fixed water table. Subsidence results from these models also show a completely different distribution of vertical displacements from that of the ‘water table drawdown’ series of models (i.e. boundary condition A). Instead of an extensive settlement trough along most of the model’s surface, the subsidence profile calculated for the fixed water table models is generally more restricted to the area directly overlying the tunnel (Figure 4.17).

Adopting the same fixed pore pressure boundary condition, the finite-element solution for the continuum case, deformation model 4, showed a similar drained pore
pressure redistribution pattern (Figure 4.18). Likewise, the distribution of vertical displacements showed similar trends with respect to predicting a more narrow subsidence trough largely confined to the region directly above the tunnel. The maximum settlement calculated for the continuum case, for the given material properties and permeability, was 2.5 cm (Figure 4.19). An added advantage afforded by means of using the finite-element consolidation solution is the ability to model the time-dependent response. The model was solved assuming a 10-day time step, with results showing that steady state conditions were reached after approximately 200 days (Figure 4.20). Further investigations with respect to the 2-D and 3-D evolution of the surface settlements generated through the poroelastic response above the Gotthard tunnel are part of an on-going study.

4.6 Summary and Conclusion

Several deformation models have been presented to explain possible mechanisms acting to promote surface settlements in crystalline rock masses through deep tunnel drainage. Models involving elements of fracture drainage and consolidation were studied using distinct-element modelling techniques. Similar models concentrating on the poroelastic response of the intact rock matrix were solved using finite-element continuum modelling.

Preliminary distinct-element results suggest that the measured settlements of 12 cm in a fractured granitic rock mass above the Gotthard tunnel cannot be explained easily by fracture deformation alone. Only by decreasing the joint spacing or normal stiffness to values below those observed in the field or in the lab is it possible to model such large magnitudes of subsidence. However, continuum models suggest that the contributing influence of consolidation of the intact rock mass must also be considered. Table 4.4 summarizes the modelled magnitudes of vertical displacement for the four different conceptual models presented. These results show that in terms of the underlying coupled hydro-mechanical mechanisms, it is the combined influence of vertical and horizontal fractures and the intact rock matrix that can act to generate surface settlements as a consequence of deep tunnel drainage. Further investigations within the framework of this study are now focussing on the influence of inclined joints and fault zones, as well as the sensitivity of the different deformation models to other key input parameters (e.g. horizontal to vertical stress ratio, permeability, etc.). Additionally, numerical modelling will be applied to reproduce the measured settlement trough in its shape and quantity and also the distribution of water inflow rates into the tunnel at the particular site in the Gotthard region.

4.7 Acknowledgements

The authors would like to thank the maintenance team of the Gotthard A2 highway tunnel for their kind support of this work and the AlpTransit Gotthard AG for the permission to publish the settlement data. Thanks are also extended to Dr. Keith Evans, ETH Zürich, and Dr. Giovanni Lombardi, Lombardi Consulting, for their input during numerous discussions.
Table 4.1. Intact rock properties for discontinuum models.

<table>
<thead>
<tr>
<th>Material</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact Granite</td>
<td>Young’s modulus, $E$</td>
<td>50 GPa</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio, $\nu$</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Density, $\rho$</td>
<td>2700 kg/m$^3$</td>
</tr>
</tbody>
</table>

Table 4.2. Joint and brittle fault zone properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Joint</th>
<th>Brittle Fault Zone</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness</td>
<td>Stress-dependent (see Fig. 4.8)</td>
<td>Stress-independent (varied from 0.1 to 100 GPa/m)</td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>10 GPa/m</td>
<td>1 GPa/m</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0 Mpa</td>
<td>0 Mpa</td>
</tr>
<tr>
<td>Friction angle</td>
<td>30°</td>
<td>20°</td>
</tr>
<tr>
<td>Zero stress aperture</td>
<td>0.6 mm (represents 10 joints)</td>
<td>1.5 mm (represents one fault zone)</td>
</tr>
<tr>
<td>Residual aperture</td>
<td>0.3 mm (represents 10 joints)</td>
<td>0.5 mm (represents one fault zone)</td>
</tr>
<tr>
<td>Rock mass equivalent</td>
<td>$2 \times 10^{-6}$ m/s (parallel to joints)</td>
<td>$2 \times 10^{-6}$ m/s (parallel to faults)</td>
</tr>
<tr>
<td>Hydraulic conductivity</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.3. Intact rock properties for continuum models.

<table>
<thead>
<tr>
<th>Material</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intact Granite</td>
<td>Young’s modulus, $E$</td>
<td>50 GPa</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio, $\nu$</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>Density, $\rho$</td>
<td>2700 kg/m$^3$</td>
</tr>
<tr>
<td></td>
<td>Biot’s coefficient, $\alpha$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Skempton’s coefficient, $\beta$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Hydraulic conductivity, $K_{xx} = K_{yy}$</td>
<td>$1 \times 10^{-8}$ m/s</td>
</tr>
</tbody>
</table>
Table 4.4. Summary of results showing maximum settlements modelled and point of maximum settlement relative to surface point directly above the tunnel.

<table>
<thead>
<tr>
<th>Deformation models</th>
<th>Condition A Maximum settlement (cm)</th>
<th>Condition B Maximum settlement (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 – Horizontal joints dominate</td>
<td>2.5</td>
<td>1.3</td>
</tr>
<tr>
<td>2 – Vertical joints dominate</td>
<td>3</td>
<td>2.4</td>
</tr>
<tr>
<td>3 – Vertical faults dominate</td>
<td>3.6*</td>
<td>2.4*</td>
</tr>
<tr>
<td>4 – Intact rock matrix dominates</td>
<td>No data</td>
<td>2.5</td>
</tr>
</tbody>
</table>

*Maximum settlements not located directly above tunnel, but 650m and 450m displaced from point directly above the tunnel.
Figure 4.1: Location map with study area.

Quaternary sediments
Variscan intrusives
AGG...Aar-granitic-gneiss
GGG...Gamsboden-granitic-gneiss
FGG...Fibbia-granitic-gneiss
Amphibolites
"Altkristallin"-basement
Late-ordovician-granitoids
Permo-carboniferous and Mesozoic sediments
Dams, Lakes
Levelling profile
689 Swiss coordinates (km)

Figure 4.2: Geological map of Gotthard region.
Figure 4.3: Surface subsidence in the time interval 1970 to 1993/98 and Alpine uplift (upper diagram). Early time water inflow rates into A2 road tunnel (lower diagram).
Figure 4.4: Conceptual models showing mechanical response to fluid drainage of: (a) horizontal joints, (b) vertical joints, (c) vertical faults, and (d) intact rock.
Figure 4.5: Distinct-element model geometry and boundary conditions.

Figure 4.6: Contoured pole plots showing brittle fault zone orientations (left) and corresponding total spacing histograms (right).
Figure 4.7: Scaling of multiple joint properties to those representative of the bulk rock mass.

Figure 4.8: Normal deformation law for one single joint.
Figure 4.9: Finite-element model geometry and boundary conditions.
Figure 4.10: Pore pressure distribution (units in Pa) before (upper) and after (lower) tunnel drainage for groundwater drawdown close to tunnel level (boundary condition A). The 0.0 Pa pressure line represents the boundary between the saturated and unsaturated zone in the model. The maximum pore pressure calculated directly above the tunnel reaches only 2e-5 Pa.
Figure 4.11: Vertical subsidence (in metres) for horizontal joint-controlled model (conceptual model 1).
Figure 4.12: Horizontal strains for vertical brittle fault zone-controlled model (conceptual model 3).

Figure 4.13: Vertical strains for vertical brittle fault zone-controlled model (conceptual model 3).
Figure 4.14: Shear deformation showing zones of right- or left-lateral displacement for conceptual model 3 (upper). Block models showing shear and normal fracture displacements at location numbers 1, 2, 3 and 4 (lower).
Figure 4.15: Vertical subsidence (in metres) for vertical brittle fault zone-controlled model (conceptual model 3).

Figure 4.16: Pore pressure distribution (units in Pa) for fixed water table condition (boundary condition B).
Figure 4.17: Vertical subsidence (in metres) for fixed water table condition (boundary condition B).

Figure 4.18: Pore pressure distribution (units in Pa) calculated for the continuum intact rock matrix case.
Figure 4.19: Vertical subsidence (in metres) calculated for the continuum intact rock matrix case (conceptual model 4).

Figure 4.20: Transient response of vertical displacements with time for the continuum intact rock matrix case.
5. Analysis of Surface Subsidence in Crystalline Rocks above the Gotthard Highway Tunnel

C. Zangerl, E. Eberhardt, S. Loew and K. F. Evans
Abstract:

Building on the results and conclusions previously observed in the generic study (presented in Chapter 4), the parametric study is extended in this chapter to include other system variables and boundary conditions. In doing so, the sensitivity of the fracture geometry network, the mechanical and hydraulic input parameters, the in situ stress conditions and the hydraulic boundary conditions are studied for their contributing effects on surface subsidence. Results from the parametric study showed that the driving force for consolidation processes is the decay of the pore pressure field in the rock mass during drainage. The magnitude in pore pressure or watertable drawdown is controlled by several hydraulic parameters and boundary conditions.

Based on these results, the subsidence problems observed above the Gotthard highway tunnel are then studied and discussed in detail through analytical and numerical investigations. Structural mapping data and results from laboratory testing on intact samples are implemented into the numerical discontinuum (discrete element) and continuum (finite element) models and are directly compared with geodetic measurements. Again, by utilizing both discontinuum and continuum techniques both the contribution of the discontinuity network and that of the intact rock matrix could be studied. Experience regarding the geological and hydrogeological framework was further extended through numerical simulations, to reproduce the key features of the measured subsidence trough i.e. irregularities in its shape. Simulations performed on the Gotthard case study showed the importance of considering both consolidation deformation mechanisms, i.e. the intact rock deformation and the discontinuity network deformation for subsidence problems in a crystalline rock mass.

Accordingly, this chapter gives a complete picture of tunnel induced surface subsidence mechanisms in a crystalline rock mass based on the work presented in the proceeding chapters: structural mapping (chapter 2), laboratory poroelastic testing (chapter 3), and generic 2-D discontinuum and continuum mechanistic modeling studies (chapter 4). In doing so, this work concludes with a comprehensive insight into the understanding and prediction of consolidation settlements in crystalline rock masses such as those above the Gotthard highway tunnel.

5.1 Introduction

Surface subsidence reaching magnitudes of 12 cm was measured in a fractured crystalline rock mass following the construction of the Gotthard highway tunnel in Central Switzerland (Zangerl et al. 2001). Overburdens above the tunnel reach up to 1500 m including 800 m at the point of maximum measured subsidence. Differential displacements on this scale can have a negative influence on surface structures especially with respect to concrete dams, bridge piers and abutments. For example, subsidence-induced damage in the form of substantial cracking of the Zeuzier dam in Switzerland resulted when nearby tunnel inflows in a carbonate rock mass produced vertical displacements of up to 13 cm below the dam (Lombardi 1988, 1992, 1994).

Surface subsidence problems are generally encountered in petroleum extraction, groundwater pumping, tunnelling in/below soil deposits or geothermal projects. Geertsma (1973), Segall (1985), Jones and Mathiesen (1993), Pattillo et al. (1996),
Hettema et al. (2000) and Cook et al. 2001 studied the effects of oil extraction on land subsidence. Extensive long-term groundwater extraction from soil-aquifers in the Las Vegas Valley have led to subsidence induced damage of structures and well casings (Hoffmann et al. 2001; Burbey 2002). The same phenomenon has been experienced in Mexico City, where land subsidence of 8 m developed between 1984 and 1991 due to consolidation of a lacustrine aquitard caused by aquifer over-exploitation (Ortega-Guerrero et al. 1999). Mossop and Segall (1997, 1999) investigated surface displacements of up to 1 m in relationship to geothermal power production within ‘The Geysers geothermal field’, which is located in a highly fractured greywacke and felsite rock mass. On other geothermal sites, for example the Wairakei geothermal field in New Zealand, surface subsidence of about 14 m was induced through fluid extraction between 1950 and 1997 (Allis 2000). Tunnelling induced ground subsidence in rock was observed by Karlsrud and Sanders (1978), Karlsrud and Kveldsvik (2002) and Olofsson (1990). In these cases, subsidence was triggered through water inflows into tunnels located in fractured rock, which drained soil deposits directly above. A general conclusion from these case studies was that pore pressure drops due to fluid extraction or drainage (i.e. water, oil, gas) within a porous and/or fractured rock mass can produce substantial surface displacements. In addition, for geothermal cases where injection of cold water into the rock mass is cooling down the reservoir, thermoelastic strains result.

The difference between observations from these previous cases and the study herein can be discerned when looking closer at the underlying rock mass properties. In general, subsidence problems related to fluid extraction occurs in soils or highly fractured and/or weakly consolidated, sedimentary rock masses. Soils and porous sedimentary rocks are characterised by rock mass stiffnesses and deformation moduli that are several orders of magnitude lower than those for fractured crystalline rocks. In contrast the Gotthard highway tunnel was driven through low-porosity (≤ 1% intact matrix porosity) fractured crystalline rock, and maximum subsidence displacements coincide with a stiff granitic gneiss unit. As such, subsidence related to deep tunneling in crystalline rock is generally not expected and has not really been properly examined in detail.

This paper focuses and presents results from an extensive 2-D numerical modelling investigation aimed at determining and quantifying those processes and mechanisms that may produce tunnel-drainage induced surface subsidence (i.e. through pore pressure drawdown) within a crystalline (i.e. mostly granitic) rock mass above a deep-seated tunnel. To do so, both discontinuum (distinct-element) and continuum (finite-element) techniques have been applied to focus on the hydro-mechanical response of the discontinuities and the intact rock matrix, respectively. In particular this work tries to ascertain to what degree the poroelasticity of the low-porosity intact rock matrix and/or the deformation of the discontinuity network (brittle fault and fractures) contributes to the induced surface subsidence. In addition, the paper also focuses on a direct comparison between the measured (i.e. surface levelling profile) and the numerically simulated shape and magnitude of the subsidence trough along a N-S orientated section above the Gotthard highway tunnel.
5.2 Theoretical Background

Pore pressure drawdown in water saturated crystalline rock masses due to drainage effects of a tunnel act to cause a redistribution of the effective stress conditions. Firstly, the pore pressure change occurs along the more permeable structures, such as brittle fault zones and fractures (i.e. tensile joints and shear fractures) and later on penetrates into the adjacent intact rock matrix through pore pressure diffusion. The diverse mechanical and hydraulic characteristics of brittle fault zones, meso-scale fractures and the intact rock matrix requires different modelling approaches to study their influence on surface subsidence. For example, fault zones and fractures can be treated as individual discrete structures, whereas the low-permeable intact rock matrix can be best described through the theory of poroelasticity. Two separate commercial codes were applied within this study. UDEC (Itasca 2001) was used to analyze the hydro-mechanical coupled processes along fractures within the rock mass. The intact rock matrix was analyzed through a poroelastic consolidation solution implemented through the continuum code VISAGE (VIPS 2002). It should be noted that although the rock mass (i.e. discontinuities and intact rock matrix) could be treated as an equivalent continuum using poroelastic solutions, this study instead focused on individual components to better understand the underlying mechanisms and the individual contributions of the discontinuities and intact rock matrix in generating drainage-induced surface subsidence.

5.2.1 Hydro-Mechanically Coupled Behaviour of Discontinuities

The term meso-scale is used to describe fractures that range in size from less than a centimetre to a few metres, and that are usually observable in a single continuous exposure (Hancock 1985). Meso-scale fractures include both tensile fractures (mode I), usually called ‘joints’, and shear fractures (mode II or III). Filled and unfilled fractures are substantially different in their mechanical and hydraulic behaviour, although most laboratory and in-situ studies have been conducted on unfilled fractures. Despite this, the complex hydro-mechanically coupled behaviour of a single unfilled fracture is not yet fully understood. Still, some fundamental but simplified laws have been derived that enable the quantification of fluid flow, stresses, pore pressures and strains in hydro-mechanically coupled discontinuities.

The stress state acting on a fracture can be resolved into normal ($\sigma_n$) and shear ($\tau$) components. Water pressurised fractures change the effective normal stress conditions acting on the fracture during pore pressure changes:

$$\Delta\sigma_n' = \Delta\sigma_n - \alpha_f \Delta p$$

where $\sigma_n'$ = effective normal stress; $\sigma_n$ = total normal stress; $\alpha_f$ = effective stress coefficient; and $\Delta p$ pore pressure change (Robin 1973). The effective stress coefficient controls the ‘effective’ pore pressure change ($\alpha_f \Delta p$). Whether pore pressure variations in a fracture influence only the effective stresses or also the total stresses is strongly dependent on the boundary conditions and the orientation of the discontinuities of the system. This is discussed in detail in Zangerl et al. (2003).
The change in the normal effective stress acting on fractures causes normal deformation (i.e. fracture closure or opening). The change in normal deformation $\Delta u$ can be calculated by:

$$\Delta u_n = \frac{\Delta \sigma_n'}{k_n}$$

where $\Delta \sigma_n' = \text{change in effective normal stress}$ and $k_n = \text{normal stiffness}$, which controls the magnitude of the normal deformation, which in turn generally varies with effective stress.

Shear deformation along a discontinuity can occur under two different boundary conditions involving either the normal stress acting on the discontinuity or when the normal stiffness remains constant. The former represents the pure behaviour of an unconfined discontinuity and the latter represents a discontinuity confined by surrounding rock (Stephansson and Jing 1995). Under both these boundary conditions, shear displacements may be initiated in a purely elastic manner (during the initial stages of deformation) and then migrate through a transition phase into plastic deformation behaviour. The magnitude in shear displacement within the elastic range is controlled by the value of the shear stiffness, $k_s$:

$$\Delta s_c = \frac{\Delta \tau}{k_s}$$

where $\Delta s_c = \text{elastic shear displacement}$ and $\Delta \tau = \text{change in shear stress}$.

For the constant normal stiffness model $k_s$ would remain approximately constant, as opposed to the constant normal stress model where $k_s$ is influenced by the applied normal stress (Bandis et al. 1983; Sun et al. 1985) In general, shear deformation parameters are influenced by scale-effects and surface conditions along the shearing planes. The transition from purely elastic to plastic shear deformation can be described in its simplest form through the Coulomb failure criterion:

$$\tau_f = c + \sigma_n' \cdot \tan(\phi)$$

where the peak shear stress, $\tau_p$, is controlled by a combination of cohesion, $c$, and friction, $\phi$. A pore pressure drop increases the normal effective stress which results in a higher shear strength of the drained fracture. In other words, drained fractures experience a shear strengthening. It has to be mentioned that the deformation behaviour within the range of plastic shear is characterized through increased complexity due to processes of shear weakening, strain hardening, and/or the evolution of the asperities; i.e. surface damage and variation of the normal stiffness during deformation (Plesha 1987).

In the distinct-element code UDEC, the Coulomb-slip model as shown in Equation (4) is used. Thus, if $|\tau| \leq \tau_f$ then the elastic shear displacement component, $\Delta s_c$, may be calculated by Equation (3). If $|\tau| \geq \tau_f$ then plastic slip occurs as long as the system
geometry and/or the normal and shear stress conditions enable shear displacement. Thus the shear stress is calculated by:

$$\tau = \text{sign}(\Delta s)\tau_f$$


$$\text{sign}(\Delta s) = \begin{cases} 
+1 & \Delta s > 0 \\
0 & \Delta s = 0 \\
-1 & \Delta s < 0 
\end{cases}$$

(5)

where $\Delta s$ is the total incremental shear displacement and ‘sign’ determines the sign of the shear displacement, (i.e. equals 1 if $\Delta s$ is positive, 0 if $\Delta s$ is 0, and -1 if $\Delta s$ is negative).

Fractures deformed in shear dilate due to the effect of asperities overriding one another (i.e. dilation). The process of dilation itself can lead to an increase in fracture aperture, or when the adjacent rock material is confined within the rock mass (i.e. stiff system boundaries), to an increase in normal stress. Similarly, dilation can influence the mechanical and hydraulic fracture apertures. In other words, perfectly mated fracture surfaces should show smaller apertures than slipped fractures (Esaki et al. 1999; Chen et al. 2000). A strongly simplified relationship to quantify these effects (Barton and Choubey 1977; Barton et al. 1985; Chen et al. 2000) may be written as:

$$\Delta d = \Delta s \cdot \tan(\psi)$$

(6)

where $\Delta d$=fracture opening. The dilation angle $\psi$ is controlled by the roughness and the surface strength of the fracture and the normal stress acting across it. According to findings from Barton and Choubey (1977), $\psi$ can be related to the fracture roughness index (JRC), the joint wall compressive strength (JCS), the normal stress and the fracture size.

Pore pressure changes will affect the effective stress conditions, which in turn influence the hydraulic flow field through variations in the fracture aperture. A decrease in the mechanical aperture (i.e. fracture width) as a result of pore pressure change also means a decrease in the hydraulic aperture, and thus a decrease in the permeability of the fractures. This aperture-flow coupling relationship is often represented by the cubic law equation for laminar flow between two parallel plates with smooth surfaces:

$$Q = \frac{ga^3 dp}{12v \frac{dp}{dl}} w$$

(7)

where: $Q$ = volumetric flow rate; $g$ = gravitational acceleration constant; $a$ = equivalent hydraulic aperture; $v$ = kinematic viscosity of the fluid; $dp/dl$ = hydraulic gradient; $w$ = fracture width in the direction of flow. It should be noted that Equation (7) was derived for the case of open fractures with planar, parallel surfaces that are not in contact at any point. In nature though, fractures under in situ conditions are neither smooth nor parallel. Instead, they are in contact with each other, defined by a normal stress controlled contact area. The equivalent hydraulic aperture, $a_e$, is defined as the effective parallel plate aperture for a rough-walled fracture, and can be measured and back-calculated through flow tests. The mechanical aperture, $a_m$, is defined as the arithmetic
average aperture, which is only equal to the hydraulic aperture when the two plates are separated and perfectly smooth (Renshaw 1995). Laboratory data suggest that this approximation may not be valid for fractures loaded under normal stresses, where fracture apertures approach the scale of the surface roughness (Tsang and Witherspoon 1981; Raven and Gale 1985; Schrauf and Evans 1986; Brown 1987). Theoretical considerations conducted by Renshaw (1995) regarding the relationship between mechanical and hydraulic apertures suggest that these apertures are not equal and do not show a linear dependency to each other. In contrast, a reinterpretation of published experiments by Alvarez et al. (1995) showed a linear relationship between hydraulic and mechanical apertures at low effective normal stresses (<25 MPa), and in some studies, a divergence from linear behaviour as an irreducible flow rate was approached under higher stress states. Apart from fracture normal deformation, shear displacement on a fracture can affect the mechanical aperture (i.e. dilation) and subsequently the hydraulic aperture (Chen et al. 2000). Nevertheless the cubic law provides a simple and numerically efficient approximation to laminar fluid flow in open fractures.

From the equations mentioned above a conceptual model for a fully coupled hydro-mechanical discontinuity system can be derived (Figure 5.1). In this model, hydraulic apertures can adjust to changing mechanical apertures as a consequence of changing effective stress conditions. In cases where there is a decoupling of the mechanical aperture from the hydraulic aperture (i.e. constant hydraulic apertures) the system is referred to as 'partly coupled'.

Within this study, UDEC models characterized by low values of normal stiffness (<1 MPa/mm), are simulated as a partly coupled model. The justification for such a simplification can be found in the way UDEC calculates the hydraulic apertures:

\[ a = a_0 + u_a \geq a_{res} \]  
(8)

where \( a_0 \) is the hydraulic fracture aperture at zero normal stress, and \( u_a \) is the discontinuity normal displacement (positive denoting discontinuity opening). A minimum value, \( a_{res} \), is assumed for the aperture, below which mechanical fracture closure, \( u_n \), does not affect the hydraulic aperture, \( a \). Equation (8) shows that UDEC couples the mechanical normal displacement (i.e. closure or opening) linearly to the hydraulic aperture. In addition, Equation (8) shows that a large magnitude of normal closure easily results in a hydraulic aperture range, where only the residual aperture remains. For example, a \( k_n \)=1 MPa/mm and pore pressure change of 10 MPa would require an unrealistic zero stress aperture of more than 10 mm. A further consideration with the application of UDEC is its limitation with respect to intact block non-permeability and the non-communication of effective stress changes along fractures with those within the intact rock blocks.

5.2.2 Hydro-Mechanical Behaviour of the Intact Rock Matrix

Low-permeability, saturated, intact rock blocks that are bounded by discontinuities (meso-scale fractures, brittle fault zones, etc.) adjust their internal pore pressure distribution due to changes in the pressure conditions along their boundaries mainly through pore pressure diffusion. The pressure change within the intact rock matrix would induce strains according to the theory of linear elasticity. As such, effective
stresses within an assumed homogeneous porous medium can be defined through the effective stress law:

\[
\sigma_{ij}^{e} = \sigma_{ij} - \alpha p \delta_{ij}
\]

Here \(\sigma_{ij}^{e}\) and \(\sigma_{ij}\) are the macroscopic effective and total stress matrices, respectively, \(p\) is the fluid pore pressure and \(\delta_{ij} = \text{Kronecker's delta}\). The constant \(\alpha\) is known as the Biot's constant and controls the degree of pore pressure-stress coupling, ranging from 0 for uncoupled to 1 for fully coupled. Biot's constant is directly influenced by the bulk modulus, \(K\), of the intact rock measured under drained conditions and the intrinsic bulk modulus, \(K_s\), of the solid constituent (Nur and Byerlee 1971):

\[
\alpha = 1 - \frac{K}{K_s}
\]

The rock blocks between the fractures and faults are taken to be elastically isotropic as characterised by the Young's modulus, \(E\), and Poisson's ratio, \(\nu\), both measured under drained conditions. Shear modulus, \(G\), and bulk modulus, \(K\), are directly related to the Young's modulus, \(E\), and Poisson's ratio, \(\nu\), by:

\[
G = \frac{E}{2(1 + \nu)}
\]

\[
K = \frac{E}{3(1 - 2\nu)}
\]

The stress-strain relationship for a poro-elastic medium (Rice and Cleary 1976) may be written as

\[
\varepsilon_{ij} = \frac{1}{2G} \left[ \sigma_{ij} - \frac{\nu}{1 + \nu} \sigma_{kk} \delta_{ij} \right] + \frac{\alpha}{3K} p \delta_{ij}
\]

Here \(\varepsilon_{ij}\) and \(\sigma_{ij}\) are the macroscopic strain and stress matrices and \(\sigma_{kk} = \sigma_{xx} + \sigma_{yy} + \sigma_{zz}\) (i.e. the first stress invariant).

Fluid flow, i.e. the vector of specific discharge, \(v\), within a porous medium is represented by Darcy's law:

\[
v = -K_d \cdot \nabla (h)
\]

\[
\nabla (h) = \begin{bmatrix}
\frac{\partial h}{\partial x} \\
\frac{\partial h}{\partial y} \\
\frac{\partial h}{\partial z}
\end{bmatrix}
\]

where \(K_d\) = tensor of hydraulic conductivity and \(h = \) hydraulic head.
Biot (1941) derived from the basic equations given above the 3-dimensional consolidation theory, which describes the hydraulic and mechanical transient response of a linear elastic porous medium. Analysis of time dependent consolidation requires the solution of Biot's consolidation equations coupled with the material constitutive model and the equilibrium equations.

The displacement formulations in the x-, y-, and z-directions, derived from equilibrium equations and the material constitutive model (i.e. linear elasticity), may be written as:

\[ GV^2 u + \frac{G}{1-2\nu} \frac{\partial \varepsilon_r}{\partial x} = \alpha \frac{\partial p}{\partial x} - F_x \]  
(16a)

\[ GV^2 v + \frac{G}{1-2\nu} \frac{\partial \varepsilon_r}{\partial y} = \alpha \frac{\partial p}{\partial y} - F_y \]  
(16b)

\[ GV^2 w + \frac{G}{1-2\nu} \frac{\partial \varepsilon_r}{\partial z} = \alpha \frac{\partial p}{\partial z} - F_z \]  
(16c)

where \( u, v, w \) = displacement, \( \varepsilon_r \) = volumetric strain, \( F_x, F_y, F_z \) = body forces.

The transient hydro-mechanical coupling between pore pressure and volumetric strain for a linear elastic, mechanically isotropic porous medium and fully saturated with a single fluid phase, i.e. water, is given by the fluid continuity equation:

\[ \alpha \frac{\partial \varepsilon_r}{\partial t} = -S_e \frac{\partial p}{\partial t} + \frac{k}{\mu} \nabla^2 p + Q \]  
(17)

where \( Q \) = explicit fluid source (which for this study was not considered), \( k \) = permeability matrix, \( \mu \) = dynamic fluid viscosity, and \( t \) = time. The parameter \( S_e \) refers to the constrained specific storage (Wang 2000) and by definition is the volume of fluid released from storage per unit control volume per unit pressure decline while holding the control volume constant (i.e. constant strain). \( S_e \) can also be referred to with respect to its reciprocal, termed the Biot's modulus, \( M \) (Detournay and Cheng 1993).

\[ S_e = \frac{\alpha(1-\alpha B)}{KB} = \frac{1}{M} \]  
(18)

Here \( B \) is known as Skempton's coefficient, which is a function of the Biot's coefficient, \( \alpha \), the porosity, \( n \), the bulk modulus, \( K \), the intrinsic bulk modulus, \( K_s \), and the fluid bulk modulus, \( K_f \) (Beavan et al. 1991).

\[ B = \left[ 1 + \frac{nK}{\alpha} \left( \frac{1}{K_f} - \frac{1}{K_s} \right) \right]^{-1} \]  
(19)
The four differential Equations (16 a, b, c) and (17) are the basic equations satisfied by the four unknowns \( u, v, w \) (i.e. displacement) and \( p \) (pore pressure). Solving the equilibrium and fluid continuity equations using a finite element solution (e.g. Lewis and Schrefler 1987) enables calculation of the transient response of displacement rates, stress conditions and pore pressures within an isotropic, homogenous rock mass. Based on the equations explained above, numerical simulations performed by a continuum finite-element code requires as input seven independent parameters: water density, rock density, Young’s modulus, Poisson’s ratio, Biot’s and Skempton’s coefficient and the hydraulic permeability.

5.3 Site Description

5.3.1 Deformation Measurements

The Swiss Federal Office of Topography carried out geodetic levelling measurements in 1993/98 as a closed loop over the old Gotthard-pass road and through the Gotthard highway tunnel (Figure 5.4). Two earlier measurement campaigns were made along this N-S profile over the old Gotthard pass road in 1918 and 1970 (Figure 5.2). During the time interval between 1918 and 1970 (i.e. before tunnel construction), a natural uplift with a rate of 1 mm/year was measured corresponding to uplift forces of the alpine orogeny.

This uplift rate concurs with estimated rates of 0.6 mm/year as determined using fission-track techniques (Kohl et al. 2000). In contrast, the surveys made between 1970 and 1993/98 (i.e. after tunnel construction) show significant downward displacements along a 10 km region above the tunnel (Figure 5.2). The maximum settlement is located at Sustenegg and reaches about 12 cm.

A spatial relationship between the rock mass overburden and the amount of surface subsidence is not evident. Figure 5.3 shows ‘vertical strains’ calculated from measured vertical displacements normalized by the distance between surface and the tunnel elevation at specific levelling points. A maximum vertical strain of -0.015% is found between Mätteli and Sustenegg in the Gamsboden-granitic-gneiss (Figure 5.3). At the Gotthard-pass, where the highway-tunnel overburden reaches its maximum, the vertical strain is less and remains constant (0.0055%) down to Airolo. The region north of Mätteli is characterized by a continuous decrease in the vertical strain to 0.004% in Hospental. The lowest strain values are calculated for the area around Andermatt, which is furthest from the tunnel and also far from the high water inflow zone. The weak spatial relationship between vertical strain and overburden, as demonstrated in Figure 5.3, indicates that surface subsidence is not influenced through topographical effects.

Recently conducted surface triangulation measurements confirmed the existence of the subsidence trough (Figure 5.4; Salvini 2002). In addition, these measurements included the surveying of surface deformations on points outside the zone of levelling measurements made alongside the road, thereby providing extra data to define the spatial dimension of the subsidence trough. Results clearly show that the extension of the trough in the E-W direction is noticeably smaller than that in the N-S direction. Direct comparison between both measuring techniques shows that the surface subsidence measured through triangulation does not extend further south than the
Gotthard pass. Two triangulation points located at exposed spots southeast from the pass show large magnitudes in subsidence (i.e. 33 mm, 114 mm). However these vertical deformations may be related to surface related mass movements.

5.3.2 Geological and Structural Background

The Gotthard highway and train tunnels pass through the tectonic units of the Aar massif, Urseren zone, Gotthard massif and Mesozoic cover sediments (Figure 5.4). The Aar and Gotthard massifs are an uplifted part of the European crust situated in the central part of the Swiss Alps. The area of investigation is directly above the Gotthard A2-Highway tunnel in the central Gotthard massif. Labhart (1999) describes these massifs as representing a polyorogenic and poly-metamorphic crystalline basement (‘Altkristallin’) made up of paragneisses, amphibolites, migmatites, migmatic gneisses, late-Ordovician granites and middle-Palaeozoic metasediments, intruded by late-Variscian plutons (Aar, Gamsboden, Fibbia granite). These rock bodies were overprinted by alpine metamorphism, mostly in greenschist facies. The alpine deformation is characterised by the development of a NE-SW striking main foliation. Throughout these massifs, brittle and ductile alpine fault zones showing a NE-SW, NNE-SSW or E-W strike can be found. The structural architecture of ductile and brittle fault zones is discussed in detail in Laws (2001), Luetzenkirchen (2003) and Chapter 2.

A 2-D trace map of mapped and inferred brittle fault zones can be seen in Figure 5.5. Primarily, the orientation of brittle fault traces mapped within the Gamsboden granitic gneiss are shown. In addition, the spatial relationship between orientation of the major fault zones to maximum subsidence and inflow rates is clearly observable. Apart from large-scale fault zones, meso-scale fractures (i.e. extensional joints and shear fractures) complement the discontinuity network. The statistical characterisation of mapped meso-scale fractures from the Gamsboden granitic gneiss rock mass is described in more detail in Zangerl et al. (2001). Meso-scale fractures measured on surface and along unlined sections of the Gotthard highway safety tunnel are provided in Table 5.1. The table shows that both on surface and at depths of at least 500 m, the same fracture sets can be found. However, their mean normal-set spacing (Priest 1993) varies by a factor of 1 to 3.6. The knowledge of the probability distribution and the mean normal set spacing of three representative fracture sets allows the calculation of the in situ block size distribution (ISBD) through the application of ‘Wang’s Equation method’ (Wang 1992). This method, modified by Lu and Latham (1999), is based on empirical equations to estimate the ISBD. To generate an upper bound of the ISBD, two real fracture sets (138/74 and 236/80) and a synthesized one derived from sets #3 and #4 (1/45), with mean normal set spacings of 2 m, 4 m, and 3 m respectively, were used for the calculation. The ISBD of the Gamsboden-granitic-gneiss shows that the largest block volume reaches approximately 1000 m³ based on the tunnel spacing data (Figure 5.6) and is significantly lower at surface. For the calculation of the cumulative block size distribution, a negative exponential and fractal probability distribution for the fracture set spacing is assumed based on previously performed scan-line analysis shown in Zangerl et al. (2001). Assuming a cubic shape for the blocks, longest block lengths of 4.4 m on surface and 10.3 m at depth are obtained.
5.3.3 Hydrogeology

During construction of the Gotthard highway tunnel, a safety tunnel was excavated 12 to 18 months in advance several hundred meters ahead of the primary road tunnel. In doing so, the safety tunnel served as a pilot and drainage adit. The initial inflows to the safety tunnel are shown in Figure 5.2. From this figure it can be seen that a sharp increase in the initial inflow rate, reaching 300 l/s per 100 m tunnel interval, was encountered in the Gamsboden granitic gneiss. This very high inflow rate predominantly occurred at the location of two brittle fault zones situated 23 m apart and showed a strong geothermal anomaly. Luetzenkirchen (2003), derived hydraulic transmissivities of these two highly permeable fault zones with yields of 2.6e-4 m²/s (for the 150 l/s fault) and 2.2e-4 m²/s (for the 110 l/s fault). Today inflow rates of 8 l/s are measured within this tunnel sector.

Spring line mapping at the surface above the central section of the highway tunnel show a relatively constant altitude between 2300 and 2500 m a.s.l. for the existing springs. Such observations are typical for low to medium permeability crystalline rock masses without highly conductive fault zones (e.g. Ofterdinger 2001). A relatively clearly defined spring line is also found above the two high permeably brittle fault zones (110 and 150 l/s), which is not consistent with simple model calculations (e.g. Löw et al. 1996).

Luetzenkirchen (2003), concluded that the regional groundwater flow in the vicinity of the Gotthard highway tunnel before tunnel construction can be characterised by two superimposed flow systems of different importance: (1) Flow parallel to the mean orientation of the NE-SW striking permeable fault zones; and (2) Flow from regions of higher elevation northwards across the major permeable fault zones (i.e. along NW-SE striking fracture set #2 and flat dipping fracture set #4). As such, the permeable fault zones with their pronounced contrast in conductivity with respect to the matrix rock would act as a regional sink with considerable lateral extent. Therefore, very old waters north of the fault zone, which were already ascending before the tunnel was built, would flow towards the fault zone. Through the opening of the tunnel, the head gradient towards the major fault zone would increase thus intensifying the flow in the direction of this structure.

5.4 Analytical Assessment of Pore Pressure Diffusion

Before focussing on the numerical discontinuum and continuum simulations, fundamental analytical calculations were performed to estimate whether today the subsidence process has converged to steady state conditions. In other words, how many years after the construction of the tunnel does the system require to equilibrate. If it can be assumed that the system has reached steady state conditions, then time consuming numerical simulations performed using the discrete element technique (i.e. UDEC) can be quickened by solving the problem in steady state mode as opposed to transient mode.
5.4.1 Time Required to Attain Steady State Pore Pressure Drawdown Conditions in a Fractured Rock Mass

5.4.1.1 Time for Pore Pressure Drawdown within the Discontinuity Network

The time, \( t \), (i.e. short-term response) where the effect of water drainage into a tunnel reaches the surface through pore pressure diffusion along the granitic discontinuity network (Figure 5.7), i.e. faults and interconnected fractures, can be approximated by the equation (Jacob 1946):

\[
    t = \frac{r^2}{2.25D_s}
\]

where \( r \) is the radius of the affected area around the tunnel and \( D_s \) the diffusion constant for the connected discontinuity network. The calculation is based on pore pressure diffusion starting from the drained tunnel (i.e. representing a zero pore pressure \( p=0 \) MPa) and penetrating into the adjacent rock mass along discontinuities. Specific storage coefficients of \( 1\times10^{-8} \) and \( 1\times10^{-6} \) m\(^{-1}\) are assumed, where the latter is considered as the most appropriate for fault zones in granitic rocks (Löw et al. 1996). Rutqvist et al. (1998) also back-calculated storativity values from coupled numerical modelling of several multiple-pressure tests on individual filled and unfilled sub-horizontal fractures in granitic rock. Results showed that the storativity varied from \( 1\times10^{-8} \) to \( 3.3\times10^{-7} \) m\(^{-1}\).

As such, the maximum time span to affect the surface 800 m above the tunnel within the discontinuity network was calculated to be 0.9 years assuming a hydraulic diffusivity of \( 1\times10^{-2} \) m\(^2\)/s (Table 5.2). A more reliable estimate of the pore pressure drawdown was found to be in the range between several days to weeks.

5.4.1.2 Time for Pore Pressure Drawdown within Intact Rock Blocks

Following short-term drainage where the pore pressure is reduced within the brittle fault zones and inter-connected fractures, pore pressure drawdown starts to penetrate into the adjacent intact rock blocks (Figure 5.7). This long-term response results from pressure diffusion into the rock matrix and can be approximated through a linear diffusion model (Carslaw and Jaeger 1973). The time, \( t \), required for the pressure drawdown to reach 90% of the fracture water pressure at a depth \( L/2 \) (i.e. the centre of the intact block) may be written as:
$t = t_{90\%} = \frac{1.03(L/2)^2}{D_s}$  \hspace{1cm} (21)

$D_s$ is the diffusion constant for the intact rock matrix, which is defined by:

$$D_s = \frac{K_i}{S_s}$$  \hspace{1cm} (22)

where $K_i$ is the isotropic hydraulic conductivity of the intact blocks and $S_s$ the coefficient of specific storage. The definition of the specific storage is the volume of water released per unit decline of head per unit bulk volume while the representative elementary volume (REV) remains in a state of zero lateral strain, i.e. $\varepsilon_{11}=\varepsilon_{22}=0$, and constant vertical total stress, $\sigma_{33}=0$ (Green and Wang 1990). The dependency of specific storage as a function of the linear poroelasticity coefficients is given by:

$$S_s = \rho_w g \frac{\alpha}{K_B} \left( 1 - \frac{2(1-2\nu)\alpha B}{3(1-\nu)} \right)$$  \hspace{1cm} (23)

where: $\rho_w$ = water density, $g$ = gravitational acceleration constant, $\alpha$ = Biot's coefficient, $B$ = Skempton's coefficient, $\nu$ = Poisson's ratio and $K$ = bulk modulus.

The low sensitivity of the Biot's- and Skempton's coefficients on the magnitudes of the specific storage- and diffusion-constants are shown in Table 5.3. To calculate the intact block diffusion, the intact rock matrix porosity was based on the measured value of 0.01 (assumed to be constant), the drained bulk modulus was measured as 20 GPa, the fluid bulk modulus, $K_f$, was assumed to be 2.3 GPa, the hydraulic conductivity, $K_i$, was 1e-11 m/s and the intrinsic bulk modulus, $K_s$, was 50 GPa. This last value can be compared to the intrinsic bulk modulus for Barre granite, found by Hart and Wang (2001), and given as 66 GPa. Overall, the intrinsic bulk modulus would be expected to lie somewhere between the grain bulk modulus of quartz ($K_s=36$ GPa as a lower bound) and plagioclase/alkali-feldspar ($K_s=60$ GPa as an upper bound), the most and the least compliant minerals (Hearmon 1979, 1984). Hart and Wang (2001) found a hydraulic diffusivity of 2.4e-6 m$^2$/s for laboratory granite samples according to a Biot's coefficient of 0.86. Published values from Detournay and Cheng (1993) also for granitic rocks show hydraulic diffusivities of 7e-6 and 2e-5 m$^2$/s. Estimations for the hydraulic diffusivity from laboratory testing on a sample collected from the Aar granitic gneiss show values for $D_s$ in the range between 3.1e-6 and 8.4e-6 m$^2$/s (see Chapter 3).

It is assumed that under in situ conditions, the hydraulic diffusivity increases due to scale effects (Keith et al. 2003). Adopting a maximum edge length for the intact blocks within the rock mass as was previously derived through Wang's Equation method (Figure 5.6), the calculated 90% drawdown times are given in Table 5.4. For example, an edge length of 10 m ($L/2=5$ m) and a pore pressure diffusion front penetrating from a drained fracture, would require a period of several days to months to reach a near-complete drawdown in the centre of the block. For a very conservative estimate involving a block length of 100 m ($L/2=50$ m) and a particularly low diffusivity of $D=3e-6$ m$^2$/s, the drawdown time is about 27 years. This suggests that when the tunnel...
breached the rock mass fracture system, resulting in their short-term drainage, 27 years ago, the much slower response within the intact rock matrix should have by now also achieved steady state equilibrium.

Strongly simplified estimations on the time dependent response of the pore pressure drawdown within the rock mass suggest that several (i.e. 1 to 8) years after the tunnel construction the pore pressure drawdown and rock mass deformation should have reached equilibrium. Given that the Gotthard highway tunnel was excavated approximately 20 years prior to the deformation (i.e. levelling) measuring campaign, the steady state assumption for the discontinuum and continuum modelling should be valid.

5.5 Discontinuum Modelling of Consolidation Subsidence

The discontinuum study presented can be separated into two self-contained parts: a first part focussing on the evaluation of subsidence-sensitive model parameters (i.e. parametric study) but only including some geological, structural and hydrogeological characteristics derived from field measurements (e.g. fracture and fault spacing and fault orientations); and a second part focussing on the subsidence measured above the Gotthard highway tunnel incorporating the essential geological, structural and hydrogeological features of the region into the model. By doing so, the numerical simulation of the shape and extent of the subsidence trough based on discontinuity-consolidation deformation mechanisms will be focussed on, followed by a direct comparison between the measured N-S orientated levelling profiles and the numerically simulated surface subsidence.

5.5.1 Controlling Input Parameters

The numerical simulations performed require key input pertaining to the mechanical properties of the discontinuities. Given that the determination of such discontinuity parameters were outside the scope of this study, neither in situ nor laboratory measurements were made but instead a detailed literature survey was performed to obtain fracture and fault zone related input parameters.

5.5.1.1 Effective Stress Coefficient ($\alpha_f$)

It is commonly assumed that the value of $\alpha_f$ for meso-scale fractures at total stresses of interest to subsidence problems is unity. This presumption is based on contact theory (Brown and Scholz 1985), that the real area of asperity contact for fractures is extremely small. In contrast, Walsh (1981) found a decrease of $\alpha_f$ from unity to 0.56 on an artificial tension fracture for confining stress conditions in the range between 30 and 170 MPa. Kranz et al. (1979) determined a value of 0.9 for granitic joints with a polished surface. A more recent study from Boitnott and Scholz (1990) conducted on ground glass surfaces at stress levels in the range of 0 to 25 MPa show a decrease of $\alpha_f$ from 1 to 0.9. In the same study, tests performed on Diabase samples produced no change of $\alpha_f$ over the entire range of stresses (0-25 MPa). Bart et al. (2000) found a best fit value of 0.77 for tests where the total normal stress was kept constant at 7.5 MPa and pore pressures were varied from 0 to 7.5 MPa. However, they also concluded that it was
impossible to find a constant value of $\alpha_f$ for the full stress range. For both brittle fault zones and meso-scale fractures the effective stress coefficient implemented into UDEC is set by default to unity.

5.5.1.2 Normal Stiffness of Meso-Scale Fractures and Brittle Faults

Based on a preliminary study using UDEC (Zangerl et al. 2003), it was found that the normal stiffness of fractures and/or brittle fault zones have a large influence on the calculated subsidence. A detailed literature study was therefore performed to review fracture/joint normal deformation behaviour in granitic rocks. Most published studies report laboratory derived normal stiffness values for natural or artificially generated granitic joints tested under several loading and unloading paths (Table 5.5). It should be noted that the stiffness values measured in situ were conducted on shallow test sites under low normal stress conditions (Table 5.5). These tests show a strong non-linearity in the normal stress-normal displacement relationship, with a high proportion of plastic deformation during the first loading cycle (Figure 5.8a). Subsequent loading cycles tend to result in more elastic behaviour. The semi-logarithmic empirical model, proposed by Bandis et al. (1983) for unmade joints, was chosen in this study to represent the modelled fracture closure behaviour:

$$\log(\sigma'_n) = q \cdot u_n + p$$  \hspace{1cm} (24)

where: $\sigma'_n$ = joint effective normal stress (MPa), $q$ and $p$ = constants, and $u_n$ = joint normal deformation (i.e. closure) in mm. The initial reference normal stress value characterizing a joint normal deformation of $u_n = 0$ was $\sigma'_0 = 0.1$ MPa. The derivative of the stress-closure equation provides the normal stiffness formulation, which is linearly dependent on the effective normal stress:

$$k_n = \frac{\partial \sigma'_n}{\partial u_n} = 2.3q\sigma'_n$$  \hspace{1cm} (25)

Equation (24) was fitted to each stress versus normal closure data set compiled from the literature, by transforming Equation (24) into the linear relation:

$$Y = qX + p$$  \hspace{1cm} (26)

where $Y=\log(\sigma'_n)$ and $X=u_n$. A regression analysis was then performed to obtain $q$, $p$ and the coefficient of determination $R^2$ for each case. The results from several test data sets obtained from the literature are listed in Table 5.5 and are graphically shown in Figure 5.8b. The wide variation in values of $q$ and $p$ also suggest a large variation in the normal stiffness of fractures. Possible explanations for the large range of these values are the different loading cycles used in the different studies, variations in testing equipment and conditions, and/or fracture surfaces.

Published studies pertaining to normal deformation behaviour of brittle fault zones are even more limited than those for joints. Only a few data sets were found in the literature. Martin et al. (1990) reported values for a brittle fault zone consisting of
fractures, fault breccias and clay-gouge in the Lac du Bonnet granite batholith at the URL-test site in Canada. The test was performed in a 96 mm-diameter borehole with a specially developed packer system (PAC-ex-system). Results from this test report a stress independent normal stiffness with relatively low values between 2 to 6 MPa/mm. Given the small load levels applied in these tests (0 to 2 MPa), such values must be used with caution. In contrast, higher values (approximately an order of magnitude stiffer) were measured in situ by Majer et al. (1990) at the Grimsel Rock Laboratory in central Switzerland, for two granite-hosted ductile shear zones overprinted by minor brittle deformation. They also applied a packer system equipped with a displacement transducer (BOFEX-system) and measured a fracture pore pressure of 1.9 MPa.

Finally, an estimation of the normal stiffness may also be obtained from laboratory tests performed on fault rock material, although the results of these test may be questionable because of sample disturbance and scale effect problems. Still, if the mean thickness, X, and the Young's modulus, E, of the fault zone is known, then the stiffness can be calculated by:

$$k_n = \frac{E}{X}$$

Figure 5.9a shows values for the Young's modulus obtained in different studies performed on crystalline rock mass hosted fault zones in the Swiss Alps. Depending on the fault thickness and confining stress, the stiffness values vary from 0.1 (at very low stresses and a 1m fault width) to 100 MPa/mm (at high stresses 50 MPa and 0.1m fault width). Figure 5.9b shows the brittle fault (i.e. fault core) thickness distribution measured in the safety tunnel (Scheider 1979; Wanner 1982). Because of the large variation in thickness along the fault plane, these values are only rough estimates. Published results of laboratory and in situ testing on brittle fault zones did not show unambiguous evidence as to whether the normal stiffness is stress dependent or not. Thus, values for brittle fault zones were implemented into UDEC as constant and for meso-scale fractures as stress-dependent.

5.5.1.3 Shear Stiffness of Fractures and Brittle Faults

The shear stiffness of discontinuities is strongly dependent on the effective normal stress, the size of the fracture sheared, the fracture surface roughness and the type of discontinuity (i.e. infilling). Whereas large and weak fault zones tend to result in very low shear stiffness values (as low as 0.01 MPa/mm), small fractures are much stiffer in shear with values up to 100 MPa/mm (Bandis et al. 1983). Yoshinaka et al. (1993) determined the shear stiffness in situ and in laboratory tests to study scale effects. In this study it was found that the shear stiffness decreases with scale from 0.01 (in situ) to 40 MPa/mm (laboratory).

5.5.1.4 Shear Strength of Fractures and Brittle Faults

Material parameters for the Coulomb-slip model defined through Equation (3) are given by Barton (1976) and Byerlee (1978). Whereas meso-scale fracture cohesion, c_f, was generally found to be close to zero, the fracture friction angle $\phi_f$ varies between 31
113

and 42 degrees. Ehrbar and Pfenninger (1999) published values for the friction angle, \( \phi_f \), for brittle fault zones between 24 and 34\(^\circ\). These tests were conducted on samples from a fault zone several 100 m's thick located in the Tavetsch massif in central Switzerland. Based on these values, fracture cohesion for the UDEC discontinuum models was set to zero, a friction angle of 30\(^\circ\) was implemented for the brittle fault zones and an angle of 40\(^\circ\) was used for the meso-scale fractures.

### 5.5.1.5 Dilation of Fractures and Brittle Faults

The dilation angle of most models was set to zero, but to determine the influence on the system it was also varied between 10 and 20 degrees during the parametric study. Reported values from Barton and Choubey (1977) for granitic fractures range between 0 and 20 degrees.

### 5.5.2 Parametric Study and Sensitivity Analysis

#### 5.5.2.1 Impacts of Fracture Network Geometry

Four different model geometries (A-D) were developed for this study and their influence on subsidence generating mechanisms was tested (Figure 5.10). All models studied were two-dimensional with a width of 4000 m and a height of 1500 m. In each model (A-D), a 16 m diameter tunnel intersection was placed at 800 m depth. With this configuration, the models do not represent a section perpendicular to the Gotthard tunnel axis, but rather a simplified N-S one orientated normal to key geological structures. Thus the 2-D representation focuses on the intersection of the tunnel with the main inflow features represented as an excavated drainage point (labelled as 'tunnel' in the following figures). This opening was given a zero pore pressure boundary condition surrounded by a radius of 50 m of increased permeability (i.e. fault and fracture apertures within this zone were increased to 1 mm to permit a near full watertable drawdown). Whether probable or not, this pore pressure distribution was chosen to permit direct comparisons between the different geometry configurations studied as part of the parametric study.

Geometry model A (Figure 5.10a) involved two sets of continuous, fully persistent discontinuities aligned horizontal and vertical to one another. The discontinuity spacing was uniformly distributed with a value of 50 m for the vertical discontinuities (i.e. representing either brittle fault zones or in other models vertical joints) and 10 m for horizontal joints. To reduce excessive simulation runtimes, spacing values for the horizontal discontinuities were upscaled, so that a horizontal spacing of 1 m measured in the field was represented in the model as a spacing of 10 m. To compensate for the lower deformability of the rock mass, the normal stiffness was reduced to maintain the same *in situ* rock mass deformability (for more detail, see Zangerl et al. 2003).

The fracture network for geometry model B (Figure 5.10b) was similar to that for model series A but with a vertical fracture spacing of 25 m. In addition, the properties for vertical discontinuities were varied every 25 m alternating between those representing vertical fault zones (i.e. constant normal stiffness) and those representing vertical joints (i.e. stress dependent normal stiffness).
In geometry model C (Figure 5.10c), spacing of brittle fault zones and fractures are equal to geometry model A, but both types of discontinuities are inclined. The dip angle for the joints was fixed at 30° and they abut onto the persistent brittle fault zones, dipping at 60° in the opposite direction.

The final geometry scenario, model D (Figure 5.10d), involved a set of continuous horizontal joints (i.e. spacing 25 m), a set of continuous 60° inclined fractures (i.e. spacing 50 m), and a series of inclined brittle fault zones as measured from within the tunnel along an interval spanning 2000 m north and south of the major inflow zone (Schneider 1979; Wanner 1982). Fault zones were assumed to be planar and continuous, and accordingly, extrapolated from the tunnel elevation upwards to the surface and downwards to the lower model boundary. Comparisons between surface and tunnel data suggest no steepening or flattening of the fault planes with depth from a statistical point of view. Interpolation between individual fault zones that are mapped on surface with those measured in the Gotthard safety tunnel was not possible due to the fact that reliable markers (i.e. lamprophyric dykes) were not found. In addition, it is suggested that the individual fault planes are most probably curved, and that some small-scale fault zones do not extend to the surface and terminate somewhere in between.

Intact block deformation was modelled adopting a linear elastic isotropic constitutive relationship, requiring as input a bulk and shear modulus (calculated from Young’s modulus and Poisson’s ratio determined through laboratory testing; Chapter 3). In UDEC, intact blocks deform under plane strain conditions and are treated as being impermeable (no flow). Although effective stresses can be calculated for the intact blocks, these are treated separately and independently from fluid pressure acting along the discontinuities (where fluid flow is permitted). All models were solved assuming steady state conditions and the pore pressure within the discontinuities were calculated as relative pressures (i.e. 0 Pa in the UDEC models is equivalent to atmospheric pressure). The upper hydraulic boundary represents a free watertable with no recharge that allows watertable drawdown due to tunnel drainage. Depending on the frequency and hydraulic apertures of the fractures (i.e. the equivalent rock mass hydraulic conductivity), the phreatic-surface could drop down as far as the tunnel elevation. This condition, therefore, represents the maximum pore pressure drop that would be achieved in the rock mass following the excavation of the tunnel. Whether probable or not, this condition was chosen so that each discontinuum model could be directly compared to one another inside the parameter study. By modelling a similar redistribution of pore pressures in each case a more direct comparison could be made with respect to the resulting subsidence profiles. Mechanical boundary conditions were defined as no-displacement boundaries normal to the bottom, left and right model boundaries (depicted as rollers in Figure 5.10).

Initial total stress conditions were set assuming a horizontal to vertical stress ratio of 0.5, with vertical stresses determined by gravitational loading. Repeated trials to simulate geometry model D, assuming a σh/σv ratio equal to 0.5, caused numerical errors and did not generate meaningful results. Thus, the stress ratio was increased to a value of 1.0 for simulations for this type of geometry model. Initial hydrostatic pore pressures were set assuming a groundwater table at surface. The effective stresses in the fractures were initialized as the difference between the normal component of the initial total stress acting across the discontinuity plane and the initial pore pressures within the discontinuity. In Table 5.6 input parameters for geometry model series A, B, C and D
are summarized. For clarity, it should again be noted that the normal stiffness assigned to the fractures follows the previously discussed semi-logarithmic law. The normal stiffness of the brittle fault zones was kept constant regardless of the stress conditions developed within the model. The stiffness values assigned to the meso-scale fractures were stress dependent and upscaled according to the implemented spacing (Figure 5.8a and 5.15; for more detail see Chapter 4).

In Figure 5.11 the pore pressure distributions for fracture geometry models A, B, C and D before and after the tunnel construction are shown. Whereas the pore pressure distributions for models A, B and C after drainage are identical (Figure 5.11b), the pore pressure distribution for series D (Figure 5.11c) shows a slight variation in distribution and magnitude reflecting the non-symmetrical pattern of the sub-vertical fault zones. Nevertheless, these pore pressure variations are relatively small allowing comparisons to be made with the other three model series.

Shapes and magnitudes of the subsidence trough for all four model types are given in Figure 5.12. Model A and B are almost identical in their shape and maximum vertical displacement, with the exception that the horizontal extent of the subsidence trough in geometry model B is approximately 800 m smaller (even if it is characterized by a denser vertical fracture spacing of 25 m in comparison to 50 m). The subsidence trough calculated from parallel inclined faults and joints (fracture geometry C) shows an irregular pattern, especially distorted near the left boundary of the model, but with magnitudes larger than the others. The point of maximum subsidence generally occurred directly above the tunnel (A and B), but in model C the maximum subsidence (approximately 0.08 m) is shifted to the left model boundary as a consequence of increased fracture shear slip along the inclined faults. Vertical displacement measured directly above the tunnel in geometry model C reaches about 0.057 m and only decreased to 0.032 m at the right model boundary. These large displacements on the right and left model boundaries suggest that boundary effects may be somewhat biasing the results. Further studies based on this model series (i.e. geometry type C) showed that such boundary effects reduce when the horizontal to vertical stress ratio was increased to 1 or 2. Boundary effects on the other models are significantly lower and do not so strongly influence the results. Model type D characterized by a fault zone pattern derived from tunnel measurements showed twice the maximum subsidence as observed for model geometries A and B. Subsidence results derived from geometry model D cannot directly be compared to the others because of differences in the stress ratio (i.e. \( \sigma_{v}/\sigma_{t} = 1.0 \) instead of 0.5) and the horizontal spacing (i.e. 25 m instead of 10 m). Nevertheless, the lower deformability of the increased spacing was balanced through a lower fracture normal stiffness to simulate a one meter spacing (i.e. upscaling). An irregularly shaped subsidence trough results as a consequence of the structure of the fault pattern. In addition, the irregular nature of vertical displacement contours suggests considerable shear displacement along several distinct fault zones (Figure 5.28b). Large elastic shear displacements of up to 2.5 mm were seen to occur along the inclined fault zones (Figure 5.13). Towards the left and right model boundaries, considerable vertical displacements may occur due to block shearing on inclined fault zones (i.e. boundary effects). In general, most of the shear deformation corresponds to elastic conditions, so that shear slip occurred only close to the model surface. Several 100 m below the tunnel elevation vertical uplift was calculated for all four models.
5.5.2.2 Impacts of Mechanical Properties and Initial Conditions

Model geometry A was used to study the influence of the normal and shear stiffness of meso-scale fractures and faults, the frictional angle and dilation angle of faults, the bulk and shear modulus of the intact blocks and the horizontal to vertical stress ratio on the system behaviour, especially with respect to discontinuity consolidation and surface subsidence. In addition, the influence of the shear stiffness and horizontal to vertical stress ratio on inclined fractures was investigated using model geometry C. These results are outlined in the following sections.

Elastic constants

The bulk modulus of the intact rock blocks was varied between 20, 40 and 60 GPa while keeping the shear modulus constant at 20 GPa. Figure 5.14a shows the variation of the bulk modulus, \( K \), versus the maximum subsidence directly above the tunnel. It clearly shows that as \( K \) increases from 20 to 60 GPa (i.e. corresponding to a change in Poisson’s ratio from 0.12 to 0.35 and Young’s modulus from 45 to 54 GPa), an increase in the maximum vertical displacement occurs (from 0.021 m to 0.033 m). The positive slope in the consolidation subsidence versus bulk modulus curve relates to the Poisson’s ratio effect, i.e. to an increased vertical block contraction and horizontal block expansion (see also chapter 4 for more details about this effect).

Fracture and brittle fault normal stiffness

Four model runs were produced with varying stress-dependent normal stiffnesses of horizontal fractures (Figure 5.15), e.g. \( k_n1 \) was varied between 294.1, 29.4, 3.8 and 2.9 MPa/mm for an effective stress interval of 0 to 5 MPa. Results show a sharp non-linear drop in subsidence magnitude when the normal stiffness of the fractures was increased (Figure 5.14b). For this model series, vertical displacements primarily develop through horizontal fracture closure following fracture drainage by the tunnel. The maximum subsidence observed for the lowest stiffness curve (where \( k_n1 = 2.9 \) MPa/mm for \( \sigma_n = 0 \) to 5 MPa) reaches more than 22 cm exceeding that observed above the Gotthard tunnel. However, the lowest stiffness value for unfilled fractures in granitic-rocks given in the literature for in situ or laboratory testing conditions was reported as 3.8 MPa/mm for an effective stress range of 0 to 5 MPa (Jung 1989). Thus, an upper bound for the maximum consolidation subsidence modelled corresponds to 0.155 m and is based on a horizontal fracture spacing of 1 m and a full watertable drawdown from surface to the tunnel elevation. Applying a more appropriate value of normal stiffness i.e. \( k_n1 = 29.4 \) MPa/mm, results in a maximum subsidence of 2.2 cm.

For model geometry A, a normal stiffness for the vertical brittle fault zones was set to 0.1, 1 and 100 MPa/mm (Figure 5.14c). In general, the sensitivity of fault stiffness to surface deformation is a magnitude lower than that of the normal stiffness of horizontal fractures. Based on the Poisson’s-ratio effect, a low stiffness value of 0.1 MPa/mm generates almost twice as much deformation as the highest value of 100 MPa/mm. In addition, a low fault stiffness value was also observed to produce a bimodal settlement trough (i.e. the point of maximum subsidence does not occur directly above the tunnel). These effects are reported and discussed in detail in Zangerl et al. (2003).
Fracture and brittle fault shear stiffness

The impact of shear stiffness on the system behaviour was only studied for vertical faults, as horizontally orientated fractures undergo very little shearing (e.g. $1\times10^{-5}$ to $5\times10^{-4}$ m). Furthermore, horizontal shear does not contribute to vertical deformation. In contrast, the shear stiffness of vertically inclined brittle fault zones does contribute to subsidence deformations, although its impact is relatively small (Figure 5.14d). Model runs where shear stiffness values were set to 0.1, 0.01, 1 and 10 MPa/mm show a range in vertical displacements varying by only 0.007 m. Regions, indicating shear failure are rare within the model domain. Close examinations of these results show that a decrease of the shear stiffness permits larger elastic shear displacements along the vertical fault zones. The amount of shear deformation is controlled by the magnitude of horizontal fracture closure and to a minor degree by the shear stiffness on the sub-vertical faults itself. As such, low shear stiffness values permit larger shear displacements, and thus subsequent larger normal displacements along the horizontal fractures results in an increase in surface subsidence. With respect to the other parameters, it can be noted that the influence of the shear stiffness on surface displacements increases for fracture geometry type C (i.e. when the fault zones are inclined; compare Figure 5.14d and Figure 5.16b).

Dilation and friction angle

Shear deformations can influence normal deformations due to dilation effects. A model where the dilation angle was set to $10^\circ$ for both the horizontal fractures and vertical faults, showed reduced magnitudes in surface subsidence. Dilation on sheared horizontal fractures lowers the amount of fracture closure. In addition, it was observed that dilation along the vertical faults acted to diminish normal discontinuity deformations and the induced vertical deformations related to the Poisson’s ratio effect. Lowering the friction angle from 30° to 5° along the vertical fault zones did not significantly change the subsidence results. Even the extremely low friction angle of 5° did not result in considerable shear slips, as shear strengths for these faults did not exceed the range where elastic shear displacement dominates. Minor shear slip only occurs close to the left and right model boundaries, independently of the applied friction angle, suggesting the influence of boundary effects.

In situ stress ratio

The variation of the initial total in situ stress ratio (i.e. $\sigma_n/\sigma_v$) between 0.5, 1 and 2 showed no effect on the magnitude of vertical displacement of model geometry A (Figure 5.14e). This insensitivity to stress conditions can be explained as a consequence of the constant normal stiffness of the faults. A drainage induced pore pressure change within a vertical fault generates magnitudes of normal closure according to the applied normal stiffness. If the normal stiffness is stress independent, then the same pore pressure drawdown induces the exact same magnitude of fault normal closure. As such, this fault deformation also controls horizontal intact block deformation, and thus the vertical shrinkage of the intact blocks due to the Poisson’s ratio effect. A stress-dependent fault normal stiffness on the other hand, would not generate the same normal closure rates along the faults and would therefore produce different magnitudes in subsidence with varying in situ stress ratios.
Based on a different geometrical configuration such as that for model geometry type C, where the discontinuities were inclined, variations in the in situ stress ratio were seen to affect the magnitude of surface subsidence, even when a constant fault normal stiffness was applied. Figure 5.16a shows the results of varying the in situ stress ratio (e.g. \( \sigma_h/\sigma_v = 0.5, 1, \) and 2) on the magnitude of subsidence induced directly above the tunnel. A reduction of 35% with respect to vertical displacement was found when increasing the stress ratio from 0.5 to 2.

5.5.2.3 Effects of Hydraulic Properties and Boundary Conditions

Horizontal to vertical hydraulic conductivity ratio: model geometry A

Hydraulic apertures for model fracture geometry A were varied to study the influence of hydraulic anisotropy (= ratio between horizontal and vertical hydraulic conductivity) on pore pressure drawdown and the resulting surface subsidence. Depending on the horizontal and vertical hydraulic conductivities (i.e. hydraulic conductivity ratio=\( K_h/K_v \)), the watertable would adjust during drainage to form different pore pressure distributions. Values for the horizontal and vertical hydraulic conductivities were each kept constant for the model (i.e. partly coupled) and were calculated based on Equation (28):

\[
K_c = \frac{ga^3}{12vX}
\]  

(28)

where \( a \) = equivalent hydraulic aperture, \( g \) = gravitational acceleration constant, \( v \) = kinematic viscosity of the fluid and \( X \)=the mean discontinuity spacing. As previously described, the hydraulic conductivities were increased within a zone of 50 m around the drainage source (i.e. increased apertures of 1 mm). All models however, did allow a near full drawdown of the watertable to the tunnel elevation, marked by a 1 MPa pore pressure isobar just below the tunnel. Results in Figure 5.17 show the dependency of the pore pressure distributions on the hydraulic conductivity ratio. Values of \( K_h/K_v \) near unity (i.e. isotropic hydraulic conditions) generate a flat and wide watertable drawdown, reaching out towards the left and right model boundary (Figure 5.17a). In contrast, low values of \( K_h/K_v \) generate pore pressure distributions that are more directly centred above the tunnel (Figure 5.17d). The hydraulic conductivity ratio \( K_h/K_v \) and the resulting pore pressure distribution, in turn, were found to also control the shape, the horizontal extension and the vertical magnitude of the subsidence trough (Figures 5.18 and 5.19). Maximum subsidence values vary from 0.025 m for near isotropic permeability conditions (\( K_h/K_v = 1.1 \)) to 0.014 m where \( K_h/K_v = 0.001 \), representing highly permeable vertical fault zones intersected by low permeable horizontal fractures (Figure 5.17). Figure 5.18 shows that the dependency of maximum vertical subsidence on the ratio of \( K_h/K_v \) for model geometry A is approximately semi-logarithmic. Decreases in the amount of subsidence modelled were found to be partly controlled by the shear stiffness of the vertical faults (Figure 5.21). With a narrow subsidence trough, the relative shear displacement between adjacent columns of blocks has to increase to absorb the total potential shear displacement. Hence, large magnitudes in shear displacement were generated near surface. In contrast, a wide subsidence trough, as observed in the
hydraulically isotropic models, can redistribute the total potential shear displacement
over a larger number of vertical discontinuities and thus lower the relative shear
displacement on a single vertical fault zone. As such, subsidence becomes dependent on
the shear stiffness of the faults, although shear slip (i.e. plastic shear deformation) does
not occur within the models. The extension of the deformation zone around the tunnel
characterized by fault normal closure was, for the isotropic $K_h/K_v$ models, a factor of 24
times larger than that for the highly anisotropic model.

**Fully-coupled models: model geometry A**

Given that the models presented above were only partly hydro-mechanically coupled
(Figure 5.1), two model scenarios were performed to investigate the effect of employing
a fully coupled solution on the pore pressure drawdown and resulting consolidation
subsidence. Using the fully coupled solution, discontinuities can adjust their hydraulic
apertures as a function of the effective stress changes occurring after tunnel drainage.
The input parameters for the two model scenarios are given in Table 5.7. To solve the
fully coupled models, the normal stiffness of the faults was increased from 1 MPa/mm
to 10 MPa/mm to avoid unrealistically large variations in hydraulic aperture. For
example, a fault defined through a normal stiffness of 1 MPa/mm would change its
aperture by 10 mm for a stress change of 10 MPa. The first model scenario considered
full coupling of both the vertical and horizontal discontinuities. The hydraulic apertures
of the vertical brittle faults before tunnel drainage decrease with depth from 2.5 mm (i.e.
zero stress aperture; $K_v=2.6e-4$ m/s) at surface to 1.9 mm (i.e. $K_v=1.1e-4$ m/s) at the
bottom of the model. For the horizontal joints, hydraulic apertures varied from 2.4 mm
(i.e. zero stress aperture; $K_h=1.1e-3$ m/s) at surface to 0.06 mm (i.e. residual aperture;
$K_h=1.8e-8$ m/s) at the bottom. Thus the ratio of $K_h/K_v$ was set to vary from surface to
bottom from 4.2 to 0.0002. This variation was linearly controlled by the magnitude of
discontinuity closure as described in Equation (8).

Results show that the fully coupled solution produced smaller hydraulic apertures for
the vertical fault zones and horizontal fractures in the region close to the tunnel. At the
elevation of the tunnel (i.e. far field), initial fault zone apertures of 0.0022 m decreased
to 0.00015 m (Figure 5.22a). Hydraulic apertures for the horizontal fractures reduce
from 0.35 mm to 0.15 mm near the tunnel. Again, it has to be mentioned that the highly
permeable zone around the tunnel (i.e. within a radius of 50 m where highly permeable
structures are given apertures of 1 mm) was required for this model to generate
significant watertable drawdowns for comparison purposes. As a consequence, the
reduction in hydraulic apertures hampers water flow within the model and influenced
the steady state pore pressure distribution. As such, the watertable drawdown is not as
large as calculated using the partly coupled solution (compare Figure 5.17a and 5.23a)
and thus the resulting maximum subsidence reduces to only 0.013 m (Figure 23b).

The second model scenario differed by adopting near-constant apertures of 0.06 mm
for the horizontal joints ($K_h=1.8e-8$ m/s) throughout the entire model. Only between the
surface and depths of 40 m were the apertures varied (between 0.3 to 0.06 mm). In
contrast, vertical faults were fully coupled with $K_v=5.6e-6$ m/s ($a=0.75$ mm) at surface,
decreasing to $K_v=2.0$ e-7 m/s ($a=0.23$ mm) along the model bottom. This resulted in the
ratio $K_h/K_v$ varying from 0.39 in the upper part to 0.09 in the lower part, respectively, a
smaller variation in the ratio as modelled in the previous case (i.e. scenario 1). As such,
the water table dropped to the tunnel level as indicated by the location of the 1 MPa pore pressure isobar just below the tunnel (Figure 5.24a). In contrast to the previous case, because the aperture of the horizontal joints remains nearly constant instead of decreasing with increasing effective stress (as would be the case if they were fully coupled), the relative conductivity ratios are higher, thus producing a larger watertable drawdown (Figure 5.25). As such, the subsidence magnitudes (Figure 5.24b) are higher than those observed in the previous case (Figure 5.23b). It should be noted that under steady state conditions, a difference between the fully and partly coupled solutions will only result, if the pore pressure distribution is different between the two cases. Of course for transient processes, there should always be some difference in the evolution of pore pressures.

Comparison of free watertable and fixed pore pressure at surface: model geometry D

A comparison was made between two contrasting surface boundary conditions: 1) a free watertable without recharge and 2) fixed pore pressure boundary simulating recharge. For the latter, a fixed pore pressure of 0.01 MPa was applied at surface to simulate surface recharge. In Zangerl et al. (2003) a similar comparison was done, but for a model geometry similar to type A (i.e. vertical faults). As already mentioned, for model type D the initial in situ stress ratio \( \sigma_h/\sigma_v \) was set to unity and the horizontal fracture spacing was set to 25 m.

Results show that for the model with a free watertable and no recharge, the 1 MPa pore pressure isobar dropped below the tunnel (i.e. lowering of the groundwater table by about 800 m; Figure 5.26a), and for the recharge model the drop was limited to approximately 100 m below the surface (Figure 5.26b). Recharge rates at surface for the latter model series averaged 5.2e-2 m/d, which is about an order of magnitude higher than mean values of 1.2e-3 m/d estimated by Ofterdinger 2001 for an alpine region close to this study area. Water inflow rates into the model from surface, concentrated along individual faults, are shown in Figure 5.27. Directly above the tunnel, inflow rates (i.e. recharge) increase as a consequence of the applied hydraulic boundary conditions, i.e. a fixed pore pressure (UDEC does not support constant flow rate hydraulic conditions). The reduced pore pressure drawdown simulated by the model through the adoption of a fixed pore pressure boundary at the surface generated a maximum subsidence of 0.031 m in contrast to the full drawdown model showing 0.042 m (Figure 5.28). Vertical displacements at the left and right model boundaries suggest some boundary effects, even with boundaries 2000 m from the drainage tunnel.

5.5.2.4 Summary of Parametric Study Results

In summary, the geometry of the discontinuity pattern controls the shape and the magnitude of surface subsidence within a drained rock mass. Maximum vertical displacements between 0.021 m and 0.055 m were observed directly above the tunnel. Whereas for symmetric model geometries (i.e. horizontal and vertical discontinuities) the shape of the resulting subsidence trough is also symmetric; for models showing inclined and irregularly orientated discontinuities asymmetric surface subsidence patterns were generated.
Normal stiffness and the frequency of sub-horizontal fractures were found to be highly sensitive parameters. Variations in the normal stiffness by three orders of magnitudes (i.e. $k_n = 2.9, 29, 294$ MPa/mm for an effective stress range between 0 and 5 MPa) generated maximum surface displacements in the range between 2.75 mm and 22 cm. Published laboratory and in situ tests indicate a large range in values of normal stiffness from 3.9 as a lower bound and >1000 MPa/mm as an upper bound (for an effective stress range between 0 and 5 MPa). The effect of the normal stiffness along vertical fault zones, showing values in the range between 1.6 and 3.7 cm (i.e. $k_n = 0.1, 1, 100$ MPa/mm), was considerably lower. The contribution of vertical fault zones on vertical deformation occurs indirectly through the Poisson’s ratio effect. As such, normal stiffness of brittle faults has an impact on the magnitude and shape of the subsidence trough, and in addition, on the location of the point of maximum subsidence (e.g. bimodal subsidence trough). A direct control on the Poisson’s ratio effect can be achieved through varying the elastic constants of the intact rock blocks (i.e. Poisson’s ratio and Young’s modulus). It was observed, that by varying the Poisson’s ratio from 0.12 to 0.35 (and E from 45.5 to 54 GPa) the maximum vertical displacement increased from 2.2 to 3.3 cm. Considerably less influence than that found from variations in the normal stiffness was observed for the shear stiffness of brittle faults (i.e. varying $k_s$ between 0.01 to 10 MPa/mm result in a maximum subsidence between 1.7 to 2.5 cm). However, inclined fault zones were more sensitive to the influence of the shear stiffness. A similar difference between geometry models, showing vertical or inclined fault zones, was observed from sensitivity analyses studying the influence of the horizontal to vertical stress ratio. Whereas model geometry A was found to be absolutely insensitive to the stress ratio, geometry type C showed a minor but explicit dependency between subsidence and applied stress ratio (i.e. $\sigma_h/\sigma_v = 0.5, 1, 2$ result in 5.5, 4.3 and 3.6 cm maximum subsidence). The insensitivity observed for model geometry A occurs due to the structural geometry (vertical fault zones and horizontal fractures) and the adoption of a stress independent normal stiffness for the brittle fault zones.

Results observed from studies on the influence of hydraulic properties and hydraulic boundary conditions showed that subsidence mainly is driven through the underlying pore pressure distribution resulting from water drainage into the tunnel. Especially, anisotropy in the form of different horizontal and vertical hydraulic conductivities affect the pore pressure distribution within the rock mass and induced consolidation strains. Hydraulic anisotropy characterized by highly permeable vertical fault zones and low permeable horizontal fractures (i.e. $K_h/K_v = 0.001$), generates shallower and narrower subsidence troughs than those assuming isotropy.

Hydraulic boundary conditions with a fixed pore pressure applied to the upper model boundary were used to simulate surface recharge. In comparison with the ‘no recharge model’, the pore pressure drawdown within the rock mass after drainage and the magnitude of vertical displacements were reduced. The maximum surface subsidence decreased by 25% for the fixed pore pressure boundary conditions.

Finally, studies on fully hydraulic coupled models were performed to investigate their effect on surface subsidence in comparison to the partly coupled models. The comparison shows that the difference in surface displacement magnitudes between the two simulation approaches only increased when the flow field and the pore pressure distribution was affected. As such, in regions close to the tunnel the increase of the
effective normal stress, induced through water drainage, reduced the apertures of the discontinuities and hence influenced the fluid flow field. On the other hand, if the steady state pore pressure distribution was not affected, i.e. if full watertable drawdown to the tunnel occurred, deformations resulting from both solution modes were equal. Of course, for simulations performed in a transient mode the effect of time dependent response on surface subsidence would become relevant.

5.5.3 The Gotthard Tunnel Case Study

5.5.3.1 Underlying Conceptual Model

The spatial conformity of measured maximum subsidence and maximum water inflow rates into the Gotthard safety tunnel, indicates that a steeply inclined brittle fault zone may have acted as a primary drainage conduit. From this feature initial water inflow rates of 260 l/s were recorded along a tunnel interval of only 27 m width driven through a fault zone (Figure 5.29). Within the interval of 1.5 km north and south from the major inflow zone, initial inflow rates of 1 to 20 l/s were recorded (Figure 5.2). Inside the primary drainage conduit, it can be assumed that the pore pressure was significantly reduced given the magnitude of water inflow into the tunnel. Moving 1 to 1.5 km eastwards from the tunnel (i.e. parallel to the fault plane), the pore pressure drawdown within the fault zone is expected to diminish at its contact with the lower permeability paragneissic rock series of the crystalline basement (Figure 5.4 and Figure 5.30). Minor water inflow rates along the Gotthard highway and railway tunnel showed that mica-rich paragneisses are several magnitudes less permeable than the adjacent granitic rock bodies (Luetzenkirchen 2003) and the Gotthard railway tunnel did not encounter substantial inflows within the paragneiss hosted portion of this fault zone. A second boundary dividing large and minor drawdown potential within the major fault zone is located to the west within the Gamsboden-granitic-gneiss 1.5 to 2 km from the tunnel (Figure 5.4).

These assumptions regarding the watertable drawdown along the major draining fault zone is compatible with interpretations of surface geodetic triangulation measurements (Figure 5.4; Salvini, 2002). Small magnitudes of subsidence or even positive displacements indicative of alpine uplift suggest that the region with pore pressure drawdown along the strike of the major fault zone is limited to the east and west. During and after drainage of the major fault zone, pore pressure drawdown would then diffuse first along strike (within the highly conductive fault zone section) and then through the fracture network into the adjacent rock mass. In the following discontinuum based numerical analyses (i.e. UDEC), full watertable drawdown within the major fault zones was assumed to drop down to the tunnel level. However, it should also be noted that the assumption of a full watertable drawdown does not completely agree with hydrogeological observations made in the region by Luetzenkirchen 2003, as previously discussed. As discussed in the previous section, this hydrogeological boundary condition generates the largest magnitudes of surface subsidence and accordingly represents the worst case scenario.
5.5.3.2 Model Geometry, Boundary Conditions and Initial Conditions

The model geometry used for the case study analyses corresponds to a N-S topographic profile along the Gotthard highway tunnel and includes the steeply inclined fault zones mapped from within the tunnel (see Chapter 2). The actual topography has been smoothed from high mountain peaks in such a manner that the model topography represents the topography along the surface levelling profile. Continuous horizontal fractures, also based on mapped data, were added to provide connectivity for fluid flow and fracture deformation (Figure 5.31). Normal set spacing for the horizontal fractures were set to decrease with depth (Table 5.8), but to avoid an unmanageable number of intact blocks (i.e. to maintain numerical efficiency), horizontal fracture spacing was upscaled by reducing the fracture frequency and the corresponding normal stiffness (Zangerl et al. 2003). Through this procedure, a mean measured fracture spacing of 1 m and a normal stiffness of 15.4 MPa/mm (for effective normal stress between 0 and 2 MPa) derived from testing was transformed in the model into an increased spacing of 40 m with a reduced normal stiffness of 0.4 MPa/mm. Table 5.8 provides the spacing and normal stiffness values that were used in the UDEC case study analyses, together with the corresponding measured ‘field’ values. According to the semi-logarithmic closure law, the normal stiffness values were varied with depth (i.e. stress). In addition, a constant shear stiffness of $k_s = 1$ MPa/mm and Mohr-Coloumb strength parameters of $c = 0$ and $\phi = 40^\circ$ were applied to the horizontal fractures. For the subvertical brittle fault zones, constant normal and shear stiffness values of $k_n = 1$ MPa/mm and $k_s = 0.1$ MPa/mm was used together with a cohesion of $c = 0$ and a friction angle of $\phi = 30^\circ$. Elastic constants for the impermeable intact matrix blocks were set to $K = 20.0$ GPa, $G = 20.3$ GPa (equivalent: $E = 45.5$ GPa, $v = 0.12$). The rock density for the granitic rock mass was set to 2700 kg/m$^3$.

Integration of the conceptual hydrogeological model into UDEC was achieved by varying the sub-vertical hydraulic conductivities of the representative fault zones based on their transmissivities as estimated by Luetzenkichen (2003). Calculation of these transmissivities was based on water inflow measurements into the Gotthard safety tunnel and the analytical solution of Jacob and Lohmann (1952). Based on these estimates, intervals with fault zones having similar transmissivities were defined and the respective mean hydraulic apertures were calculated. The two-dimensional fracture network and hydraulic conductivity values (i.e. apertures) of faults (Table 5.9) and fractures (Table 5.10) are shown in Figure 5.31. Zero displacement boundaries were applied normal to the sides and at the bottom of the model (Figure 5.29). Regarding the hydraulic boundary conditions the side boundaries were set as impermeable (i.e. no flow boundaries). The lower boundary was fixed to a pore pressure of 26.5 MPa, which represents a mean water table of 500 m above the tunnel elevation (pore pressure boundary type I, Figure 5.29). Simulations were also performed where a fixed pore pressure of 30.9 MPa (i.e. calculated from the maximum overburden) or a no flow boundary at the bottom was applied (pore pressure boundary types II and III; respectively Figure 5.29). To consider surface recharge a constant pore pressure of 0.001 MPa or 0.01 MPa is applied to the upper boundary (i.e. UDEC does not enable a constant flow boundary). The motivation to study several different hydraulic boundary conditions was based on findings learned from the parametric study (i.e. major impact of pore pressure distribution on subsidence) and the limited information about the
regional watertable and pore pressure distribution before and after water drainage to the tunnel at the Gotthard pass region. During initialisation the water table follows the topography of the model surface. Again, numerical simulations were run in the steady state calculation mode. Initially, the horizontal to vertical stress ratio (i.e. $\sigma_h/\sigma_v$) was set to unity. After cycling the models in an undrained state to an initial hydraulic and mechanical equilibrium, a hydraulic sink (i.e. pore pressure=0 MPa) representing the opening of the tunnel excavation was placed at its intersection with the major fault zone (Figure 5.29).

5.5.3.3 Results

For pore pressure boundary type I (see Figure 5.29) the initial and final pore pressure distributions are given in Figure 5.32a and b. Note that given the high permeable nature of the fault zone even in its initial state (i.e. prior to drainage) pore pressures within the fault zone are reduced. However, attenuation of the pore pressures in the fault zone in comparison to the lower permeable adjacent rock mass does not exceed 2 MPa. Once drainage is permitted, by simulating the opening of the tunnel, the watertable drops within the permeable fault structure to the elevation of the tunnel (Figure 5.32b). In addition, the pore pressure disturbance abates deeply into the fault zone below the tunnel. The subsequent effective stress changes within the discontinuities and induced strains within the intact rock blocks through the Poisson’s ratio effect, result in a maximum consolidation subsidence of 0.042 m (Figure 5.33 and 5.34a). Topography and hydraulic conductivity effects shift the point of maximum subsidence from directly above the tunnel (or sink) several hundred metres to the south. The subsidence trough extends about 9000 m in total with subsidence values greater than 0.01 m occurring over an area that is more than 4500 m wide. Vertical uplift movements calculated on surface 2000 m north of the major fault zone may be related to the orientation and the permeability of the brittle fault zones within this region. Shear deformations on faults and fractures are shown in Figure 5.35. The relative large magnitudes in shear displacements, which were observed close to surface on the left (up to 2.9 cm) and right model boundary (up to 1.3 cm) might be related to boundary effects. Within the central subsidence trough, shear displacements of up to 4 mm were calculated under elastic conditions. Normal closure of the sub-vertical brittle fault zones were characterised by large magnitudes (i.e. up to 5 mm) near the tunnel (Figure 5.36).

Further numerical simulations based on pore pressure boundary type I were performed. The normal stiffness was reduced from 1 MPa/mm to 0.5 MPa/mm for the sub-vertical fault zones. In addition, the normal stiffness for the horizontal fractures were set to those measured by Jung (1989; test number SB5; $q$=1.2 $p$=-0.86; Figure 5.34a).

The simulation based on these input values produced a maximum subsidence of 0.080 m. Applying a higher surface pore pressure of 0.01 instead of 0.001 MPa to the upper boundary of the model (i.e. higher recharge) reduces the maximum subsidence by 0.01 m, as a result of less extensive pore pressure drawdown within the rock mass (Figure 5.34a). In both cases, i.e. initial surface pore pressure 0.001 or 0.01 MPa, the watertable drawdown (i.e. 1 MPa pore pressure contour line) within the major fault zone drops to the elevation of the tunnel.
Simulations performed on a model characterised by a bottom boundary pore pressure of 30.9 MPa (pore pressure boundary type II) showed almost the same maximum subsidence of approximately 0.045 m (Figure 5.34). In comparison to boundary case I, the pre-drainage pore pressures within the major fault zone were slightly higher and upon drainage reduced significantly (Figure 5.32c and d).

For pore pressure boundary type III where the bottom model boundary was defined as a no flow boundary, the initial and drained pore pressure distributions are shown in Figure 5.32e and f. Under initial equilibrium conditions, the pore pressure distribution evolves uniformly across the model and the major fault zone. However, once tunnel drainage was permitted, large pressure drops of up to 5 MPa resulted within the major fault zone producing a maximum vertical displacement of up to 0.049 m (Figure 5.34a). Although this maximum value is close to those generated for the two alternative boundary conditions, in this case, a wider subsidence trough forms as a result of larger vertical displacements south of the fault zone. In addition, north of the major fault zone on the north edge of the subsidence trough, more than one centimetre of vertical surface uplift developed (Figure 5.34a).

Water inflow rates into the tunnel of 0.1 l/s per unit metre model thickness were modelled with 76% of the total inflow originating along fault zones from below the tunnel. The remaining 24% of total water inflow modelled as originating from the brittle fault zone above the tunnel may still be viewed as high in terms of flow rate and are a function of the applied hydraulic boundary condition along the top of the model (i.e. the fixed pore pressure boundary condition produces considerably larger flow rates than would be generated through that provided by the measured surface recharge; Figure 5.27). Assuming an inflow zone of 1000 to 2000 m in the out of plane direction, the modelled inflow rate would correspond to 100 to 200 l/s, which is approximately an order of magnitude higher than the actual measured 'steady state' inflow rates of about 8 l/s (Luetzenkirchen 2003). Pore pressure drawdown changes the effective stress conditions along the discontinuities by a maximum of 8 MPa. These stress changes and resulting discontinuity and block strains affect the total stress conditions of the rock mass. As such, horizontal total stresses near the tunnel decreased in relationship to the initial far-field stress conditions by 1 MPa, although localized concentrations could vary by ± 2MPa. In general, vertical total stresses decreased by 1.25 MPa including localized drops of up to 4.6 MPa. However, vertical total stresses could also increase by up to 1.3 MPa depending on the fracture network geometry. As such, due to the geometrical complexity of the discontinuity network, principal stresses rotated or deflected near brittle structures thereby forming heterogeneous total stress changes (Figure 5.37).

**Shape of the subsidence trough**

The shape of the simulated subsidence trough reveals several inflections that were also observed in the measured subsidence profile (Figure 5.34a and b). The agreement between measured and modelled subsidence curves were reached only through the synthesis of a geometrical model having the capacity to consider all observed dominant rock structures (i.e. all observed and measured brittle faults within the Gotthard safety tunnel during the tunnel mapping campaign and a ‘modified’ horizontal dipping joint set that represents the sub-horizontally joints in this region). The steeply inclined joints, sub-parallel to the brittle fault zones were not considered in the model set-up. The reason for the simplification can clearly be discerned when looking closer to the
limitations in available computer capacity and simulation run time. Already, the numerical models discussed above required a simulation run time between 5 and 6 weeks to reach equilibrium.

The following distinctive curve features were observed:

a) The steeply inclined central part of the trough 2000 m in width may be a result of the fan structure of brittle fault zones, where the point of dip overturn corresponds exactly with the location of maximum subsidence. Clusters of densely spaced parallel faults form the boundary between the central and outer trough through increased shear displacement along these fault planes (Figure 5.35).

b) The inflection in the subsidence curve 2.5 to 1.5 km north of the peak subsidence may be explained through the orientation of the fault zone pattern. Different steeply inclined clusters of densely spaced fault zones affect the trough through variations in the elastic shear and normal closure displacement.

c) Again, south of the central trough increased shearing on north dipping fault zone clusters occurs forming an inflection in the subsidence trough. Outside this zone, the slope flattens and the magnitude of subsidence reduces gradually. The trough slope flattening south of the maximum subsidence point are a result of the reduced pore pressure changes in the rock mass.

d) The fluctuations of the trough south from the peak subsidence are an effect of non-uniform shear deformation along the fault zones caused by few south dipping faults in a generally north dipping fault domain.

e) The uplift or low subsidence north of the peak subsidence represents the major difference between measured and simulated deformations and appears 2000 m north of the major fault zone. Effective stress changes due to rock mass drainage further south within the major fault zone and the occurrence of unfavourably dipping faults can induce surface uplift or low magnitudes in subsidence.

Thus, the fault pattern and elastic shear and normal displacements on these fault zones can be shown to significantly influence the shape of the subsidence trough.

5.6 Continuum Modelling of Consolidation Subsidence

5.6.1 Analytical Estimations of Surface Subsidence of the Intact Rock Matrix

If the mean pore pressure drawdown can be estimated, it then becomes possible to determine the change in volumetric strain, $\varepsilon_v$, within the drained rock mass. This quasi-static approach, derived from Equation (13), assumes constant total stresses and estimates the volumetric strain change from the Biot’s coefficient ($\alpha$), the bulk modulus (K), the intrinsic bulk modulus ($K_s$) and the mean pore pressure drawdown, $\Delta p$:

$$
\Delta \varepsilon_v = \frac{\Delta p}{\frac{1}{K} - \frac{1}{K_s}} = \frac{1}{K} \cdot \alpha \cdot \Delta p
$$ (29)
Table 5.11 shows the isotropic shrinkage (i.e. vertical displacement) calculated from Equation (29). Depending on the change in the mean pore pressure (in this study varied between 2 and 4 MPa), the bulk modulus of the intact rock matrix (i.e. 10 or 20 GPa) and the de-pressurized rock interval (800, 1000 or 1200m), subsidence values between 0.019 to 0.112 m were calculated. Calculations are based on intact rock properties and demonstrate the lower bound due to the fact that isotropic shrinkage is assumed. The bulk modulus of 20 GPa was derived from several loading and unloading loops of a uniaxial compressive test, although direct laboratory measurements of K show a considerably lower value (K≈10 GPa) for stress conditions applicable to the Gotthard subsidence problem (Chapter 3). In general, small-scale laboratory tests tend to overestimate the bulk modulus as they do not account for fractures greater than that of the sample size and smaller in length than those individually measured in situ. Thus, an initial estimate of the maximum vertical displacement based solely on the poroelastic response of the intact rock matrix would seem to be in the range of 0.019 to 0.112 m. Given that the magnitude of pore pressure drawdown and the affected rock volume was difficult to quantify, Table 5.11 only represents a possible range for surface displacement. In addition, it is assumed that subsidence values calculated from a bulk modulus of 10 GPa are more reliable.

5.6.2 Poroelastic Finite-Element Modelling

5.6.2.1 Model Geometry, Boundary Conditions and Initial Conditions

Based on the hydrogeological conceptual model already presented for the discontinuum Gotthard case study modelling investigation (UDEC), several 2D finite-element continuum models were derived using the same N-S section (Figure 5.29). Solutions were based on Biot’s consolidation theory using the commercial finite-element program VISAGE (VIPS 2002). By default full pore pressure drawdown along the fault zone was simulated by introducing, upon tunnel drainage, zero pore pressure conditions to the respective nodes along the fault above the tunnel. As such, the full drawdown within the fault zone in both the UDEC and VISAGE models allowed for direct comparison of their results. Again, the applied mechanical and hydraulic boundary conditions together with the other initial conditions were identical to those implemented in the Gotthard case study discontinuum models. For example, Figure 5.38a shows the initial pore pressure distributions used where the pore pressure condition at the surface was set to 0.001 MPa and the pore pressure along the bottom was equal to 30.9 MPa (compare with boundary type II, Figure 5.29).

5.6.2.2 Material Properties

The first series of continuum models were characterised by the assumption of an isotropic and homogeneous hydraulic conductivity field. Similarly, the Young’s modulus, Poisson’s ratio, Biot’s- and Skempton’s coefficients were assumed to be isotropic and constant throughout the full model domain. Parameter values are given in Table 5.12. The second series adopted an anisotropic model in terms of the hydraulic conductivity contrast generated by the more permeable nature of the subvertical fault zones (characterised by a horizontal hydraulic conductivity of $K_h=1e^{-8} m/s$ and a
vertical one of \( K_v = 1 \times 10^{-6} \) m/s). These values can be compared to those for a low-permeable intact granitic rock (approximately \( 1 \times 10^{-10} \) to \( 1 \times 10^{-12} \) m/s) and as such were used to simulate water drainage of a permeable fractured rock mass; i.e. fluid flow was modelled assuming a fractured rock mass but the deformation was modelled based on the response of intact rock matrix. These two model series were further extended to consider variations in the poroelastic coefficients with depth. Based on laboratory testing results (Chapter 3) and published studies by Brace (1965), the values of the elastic and poroelastic constants were varied with depth through the subdivision of the model into five individual but isotropic layers (i.e. from 0 to 200 m, from 200 to 400 m, from 400 to 800 m, from 800 to 1400 m and deeper than 1400 m; see Table 5.13). The first of these numerical simulations assumed a varying Biot’s and Skempton’s coefficient, while the Young’s modulus was kept constant (with a value of \( E = 45.5 \) GPa). Additional model runs were then executed where all three elastic constants varied with depth (Table 5.13).

5.6.2.3 Results

The first model series as described above, assumed fully isotropic conditions (i.e. isotropic permeability and mechanical parameters). Pore pressure distributions for the undrained (i.e. initial condition before the tunnel was opened) and fully drained conditions are shown in Figure 5.38a and b. A maximum vertical displacement of 0.060 m was generated. In addition, the pore pressure distribution following tunnel drainage conditions for the hydraulic anisotropic case (i.e. \( K_h = 1 \times 10^{-8} \) and \( K_v = 1 \times 10^{-6} \), \( K_h / K_v = 0.01 \)) is given in Figure 5.38c. Whereas horizontally, the pore pressure drawdown reduces considerably an increase was observed in the vertical direction penetrating below the tunnel. Surface subsidence rates calculated for both the isotropic and anisotropic models are shown in Figure 5.39. Comparing the two cases, models introducing the hydraulic anisotropy experience a reduction in the maximum subsidence resulting in a value of 0.016 m. This comparison was repeated for a Poisson’s ratio of \( v = 0.12 \) (representing the lower bound) and \( v = 0.21 \) (representing the upper bound). Results showed that the maximum subsidence decreased from 0.06 to 0.05 m. Increasing \( v \), while keeping \( E \) constant means a change in the bulk modulus from 20 to 26 GPa (shear modulus \( G = 19 \) GPa). Thus, deformation behaviour does not exclusively rely on intact rock bulk modulus. Model results where \( E = 30 \) GPa and \( v = 0.25 \), which equate to \( K = 20 \) GPa and \( G = 12 \) GPa, showed much larger vertical displacements (0.070 m) than those observed for the other model runs. Accordingly, deformation mechanisms observed from finite-element continuum models indicate changes in total stresses during drainage, which stand in contrast to theoretical assumptions (i.e. a constant total stress during the subsidence process assumed by Equation 29) used in the analytical estimations.

Layered models, where only the Biot’s- and Skempton’s coefficient were varied with depth, produced vertical displacements of 0.043 m (Figure 5.39). Given that the applied boundary conditions and hydraulic conductivities were identical to those models where homogeneous isotropy was assumed, the pore pressure distributions were similar in shape and extent of drawdown (Figure 5.38a and b). Variation of both the Biot’s- and Skempton’s coefficient together with the Young’s modulus for the intact rock matrix with depth, showed increased maximum vertical displacements to 0.068 m (Figure 5.39). Conspicuous was a bulge at the subsidence trough hinge that forms as a result of
upward bending of the individual layers (manifesting itself as a buckle at the surface). The positive vertical strain increase was calculated for three surface nodes (Figure 5.39). Such a significant anomaly in the trough may originate from the model setup (i.e. layering) and does not necessarily represent the true in situ situation (numerical simulation effect). In other words, the contrast of elastic and poroelastic parameters between the layers induce strain concentrations followed by vertical uplift in the hinge point. The maximum subsidence value of 0.068 m for the layered model agrees well with estimated values derived through the analytical approach. Drainage induced strains reduce the magnitude of the vertical and horizontal total stresses by a maximum of 2 MPa. Vertical and horizontal nodal strains, illustrated in Figure 5.40 show that close to the drainage trough surrounding the conductive vertical fault zone, horizontal and vertical contraction results. As such, it can be concluded that the layering influences vertical strain contours to form flat lower contour boundaries.

5.6.3 Equivalent Rock Mass Continuum Finite-Element Analysis

5.6.3.1 Material Properties

The FE-program VISAGE was also applied to model subsidence of the fractured rock mass above the Gotthard highway tunnel assuming an equivalent continuum (i.e. including both discontinuities and the intact rock matrix). The observed high structural anisotropy of the rock mass and limitations of continuum techniques to implement poroelastic anisotropy, required considerable structural simplification (i.e. isotropic equivalent properties for each layer; see Table 5.14). Based on the fracture spacing and the normal stiffness of the sub-horizontal fracture set as the most representative one for subsidence, an upper bound for the equivalent deformation modulus was calculated. The equivalent deformation modulus, $E_e$, was calculated using the relationship:

$$E_e = \left( \frac{1}{E} + \frac{1}{Xk_n} \right)^{-1}$$

where $X$ represents the mean fracture spacing. The calculation was based on a semi-logarithmic fracture closure law ($q=10$ and $p=-1$; Bandis et al. 1983) and a mean fracture spacing of 1 m. The Biot’s and Skempton’s coefficients and the Poisson’s ratio were adopted from the intact rock models. From a theoretical point of view, the Poisson’s ratio should decrease in a fractured rock mass. Equivalent deformation moduli derived from the RMR-rock mass rating system (i.e. RMR=65) after Bieniawski (1978) were found to be 24 GPa near surface and 41 GPa at depth (i.e. highly stressed regions).

5.6.3.2 Results

The extent and reduction in pore pressures following tunnel drainage, formed elliptically shaped contours encompassing a unit volume of approximately $7 \times 10^6$ m$^3$ (Figure 5.41). This unit volume extends 3600 m in the horizontal direction and 2500 m in the vertical direction (with a 2-D plane strain thickness of 1 m). Within this region, a
mean pore pressure drawdown of 1.6 MPa can be calculated. Assuming constant total stresses and a Biot's coefficient of 0.7, an equivalent rock mass bulk modulus of only $K=7.6$ GPa was back-calculated from Equation (29) in order to get 0.12 m of maximum subsidence.

Pore pressure drawdowns, observed in these model series are identical to those calculated from previously simulated models (Figure 5.38a and b). Depending on the rock mass deformation modulus used (denoted with the subscript a, b, and c in Table 5.14), maximum vertical surface displacements of 0.054 m, 0.081 m and 0.110 m were modelled, respectively (Figure 5.42). The model where the rock mass modulus was based on a constant Young's modulus for the intact rock matrix and a depth dependent fracture normal stiffness produced less than 50% of the actual measured subsidence. Simulations that adopt both a depth dependent intact rock Young's modulus and depth dependent fracture normal stiffness generated consolidation subsidence magnitudes closer to those measured. However, it should be noted that an exceptionally low Young's modulus combined with a full watertable drawdown is required to approach subsidence magnitudes of 12 cm.

5.7 Discussion

Subsidence in a fractured crystalline rock masses resulting from deep tunnel drainage, such as that which occurred above the Gotthard highway tunnel, is mainly driven by a pore pressure reduction and consolidation within the rock mass. Sensitivity analysis of hydraulic conductivity confirms the strong dependency of the resulting subsidence profile shape, extension and magnitude on the pore pressure distribution. Even if only minor watertable drawdown is observed at surface, modelling results show that the pore pressure reduction around the tunnel can act to generate surface subsidence. In the present case study of the Gotthard highway tunnel, pore pressure drawdown due to tunnel drainage is not well constrained. Prior to construction, the prospect of generating surface displacements several hundred metres above the tunnel (in crystalline rocks) was not considered and therefore data relating to the pore pressure evolution were not recorded. This is in contrast to cases relating to reservoir compaction and surface subsidence due to oil or water extraction where such effects are expected from beginning (Hettema et al. 2000; Jones and Mathiesen 1993; Mossop and Segall 1999). Ideally, in the case of the Gotthard tunnel, it would have been preferable to have monitored rock mass pore water pressures measured before tunnel construction began until several years after the tunnel had been finished. Instead, monitoring only included tunnel water inflow rates measured during and shortly after tunnel construction (Schneider 1979; Wanner 1982) and then more recently by Luetzenkirchen (2003). Surface observations made during the mapping campaign performed for this study showed the presence of surface springs in the vicinity of the major inflow zone. However, it should also be noted that a similar mapping survey of springs was not performed before the tunnel was constructed and therefore conclusions regarding the presence or drying out of springs is difficult to confidently assert. Interestingly, measurements of low tritium content and high water temperatures sampled from the major fault zone within the tunnel suggest that mostly old water is flowing upwards into the tunnel through the fault zone from beneath it (Luetzenkirchen 2003). Thus, it is
suggested that the tunnel did not significantly influence the regional ground water table near surface. Given the high permeability of the major fault zone at its intersection with the tunnel (Luetzenkirchen 2003) but showing no clear signs of ground water table drawdown at surface, it could be inferred that the permeability structure of the fault structure varies with depth. Although further investigations would be required, it could be hypothesized that the upper section of the fault zone is plugged with fine-grained material and the permeability increases with depth.

A second hypothesis that could help to understand the spring observations on surface (i.e. unaffected water table) is related to the viscosity variation of the water at different temperatures. The dynamic viscosity of water significantly reduces from 1.3e-3 Pa·s at 10°C to 0.3e-3 Pa·s at 60°C and hence, influences the fluid flow through the fracture network. At greater depth (i.e. higher water temperature), increasing flow rates due to viscosity effects would allow a more extensive rock mass drainage than in regions located near surface.

Another hypothesis to explain this ground water situation could be found in very locally defined water table drawdowns. The permeability of the major fault zone is also believed to vary laterally, as triangulation results indicate that eastwards from the tunnel at the geological boundary between the Gamsboden-granitic-gneiss and the less permeable paragneiss the pore pressure drawdown and thus the rock deformation is considerably lower eastwards. Westwards a similar situation can be observed. There the boundary between minor surface deformation and large displacements (i.e. minor and large pore pressure drawdown) is found inside the granitic rock mass of the Gamsboden. Still, whether several lakes located over a heavily fractured granitic rock mass confirms a fixed head boundary condition or the major fault zone terminates or has significantly reduced its permeability, cannot be clearly resolved. It was not possible to track the major fault zone on surface outside the subsidence region by studying geomorphical signs and lineaments on aerial photos (Figure 5.5). In addition, fault zone permeability is controlled by the interplay between fracture dilatancy, cementation, shear-enhanced compaction and clay formation (e.g. Evans et al. 1997; Wibberley and Shimamoto 2002).

If near surface, only minor pore pressure drawdown had occurred, the contribution of fracture based consolidation on subsidence becomes significantly reduced given that the lowest normal stiffnesses would be found near the surface. Parametric studies postulate a reduction in vertical displacement between 25 to 50 % (see this study and that presented in Zangerl et al. (2003), respectively) for models characterised by a fixed water table relative to those involving a free water table boundary condition. As such, the summation of results from both the continuum and discontinuum models barely reach 12 cm of surface subsidence, thereby suggesting that considerable pore pressure drawdown had to be induced through the tunnel drainage.

The findings from this study indicate that the subsidence process can be subdivided into two separate deformation mechanisms. During the initial stage of rock mass drainage, discontinuity deformation is the predominant mechanism and can be characterised by the closure of horizontal fractures (involving minor shear), shear and closure along sub-vertical fractures, and induced strains of the intact blocks as a consequence of discontinuity closure (i.e. Poisson ratio effect). Results from the parametric study show that the normal stiffness and frequency of sub-horizontal fractures contributes the most to generating higher magnitudes of subsidence. However,
the normal stiffness of steeply inclined brittle fault zones can also affect the magnitude of subsidence as well as the shape of the trough through the Poisson's ratio effects. Simultaneous to the first mechanism, a second mechanism becomes more relevant as increased drainage of the fracture network transpires. This mechanism involves pore pressure decay penetrating into the low-permeability intact rock matrix and generating poroelastic strains (i.e. contraction or shrinkage). Parametric analyses performed for the poroelastic effect show that the most important mechanical parameters are the Young’s modulus, Poisson’s ratio (drained bulk modulus, shear modulus) and the pore pressure-stress coupling (i.e. Biot’s coefficient, \( \alpha \)).

Together, superposition of intact block and discontinuity strains work to induce changes in the total stress field, which in turn causes deformations due to Poisson ratio effects. For this reason, it is physically not acceptable to just sum the contributing effects of the discontinuum and the intact blocks to produce the total amount of consolidation subsidence observed above the Gotthard tunnel. On the other hand, it can be assumed that the overlapping influence, which is hard to quantify, was not significantly large. In addition, poroelastic intact rock strains affect shear and normal displacements. Nevertheless, given the unexpected nature and lack of previous investigation into consolidation processes in crystalline rocks, it is essential that the underlying processes and mechanisms relating to the intact rock matrix and the discontinuities be initially considered and treated separately. Due to limitations in available numerical codes, the complete problem was treated as an isotropic equivalent rock mass using the continuum finite-element poroelastic formulation, whereby both the intact rock and the discontinuity network were included through adjustments to the input material parameters.

The complex structural anisotropy observed for Gotthard case study (i.e. sub-vertical fault zones and densely spaced fractures and widely spaced flat to medium dipping fractures; see Table 5.1), could not be explicitly considered with this approach. Anomalies and fluctuations in the measured subsidence profile were reproduced through the discontinuum case study models. Results showed that the shape of the subsidence trough is strongly affected through the fan structure and spacing of steeply inclined fault zones which control the location and magnitude of shear deformation. In addition, clusters of steeply inclined fault zones can generate additional vertical displacements through their normal closure. Model results for the discontinuum representation of the Gotthard tunnel case history show that since full fracture closure may be reached with effective stresses of approximately 30 MPa below 1800 m (i.e. in the case of a fully saturated medium), fracture deformation provides only a minor contribution to surface subsidence although large pore pressure drawdowns do occur. In addition, the observed decrease in fracture frequency reduces the deformability of the rock mass through fracture closure mechanisms. Continuum analysis showed that below such depths, linear poroelasticity of the intact rock matrix becomes the dominant process, even if the poroelastic coefficients are treated as being depth dependent.

One-dimensional calculations of the intact rock matrix and the fracture network suggest that pore pressure diffusion reaches steady state conditions after several months to a maximum of 1-8 years. In addition, FEM transient simulations performed with VISAGE suggest that for an isotropic rock mass permeability of \( 1 \times 10^{-9} \) m/s, but with depth dependent poroelastic properties (\( \alpha, B \) and \( E \)), consolidation reaches equilibrium after 250 days. Zangerl et al. (2003) found for the fully isotropic case (i.e. constant
poroelastic parameters; \( K=1\times10^{-8} \text{ m/s}, \alpha=1, B=1, E=50 \text{ GPa} \) that steady state conditions were reached after 200 days.

Considerations lacking from this analysis include anisotropic poroelasticity and three-dimensional effects relating to the geologic structure relative to the tunnel alignment, the hydraulic flow field and the topographic situation. As such, two-dimensional numerical results represent an upper bound of the subsidence, as shear stresses acting in the out of plane direction (i.e. east and west from the 2D cross-section) are not taken into account. Such shear stresses acting parallel to NW-SE striking fractures and intact rock bridges were initiated through lower magnitudes in pore pressure drawdown sidewise from the model section and would act to reduce the magnitudes of the subsidence.

Overall results from the numerical discrete fracture modelling and poroelasticity continuum modelling clearly suggest that the measured surface subsidence of 12 cm above the Gotthard tunnel cannot be explained through discontinuity deformation alone, nor through the intact rock matrix deformation alone. Instead, subsidence within the granitic rock mass is seen to be driven by a combination of both deformation mechanisms as summarized in Table 5.15 and 5.16.

### 5.8 Conclusion

Results from this study show that consolidation processes driven by the disturbance and reductions in the pore pressure field was the driving force for subsidence related to a deep tunnel project in a crystalline rock mass (the Gotthard highway tunnel). The magnitude of pore pressure, or watertable, drawdown was controlled by the interconnectivity of discontinuities intersecting the tunnel, the hydraulic conductivity of the discontinuities (i.e. ratio of vertical and hydraulic aperture), the hydraulic boundary condition and surface recharge. In addition, the type of coupling between the mechanical and hydraulic apertures was seen to affect the pore pressure distribution and therefore the total subsidence modelled.

It was found that the influence of the intact rock matrix in generating surface subsidence was as important as the contribution of the discontinuity network. Simulations directly based on the Gotthard case study showed that subsidence may be explained by mechanisms relating to the consolidation of the discontinuity network, and in addition, subsidence may be explained through poroelastic mechanisms relating to the intact rock matrix.

Normal stiffness and spacing of flat dipping fractures also had a high structural impact on the modelled consolidation subsidence. Especially when close to the surface, where normal stiffness and fracture spacing values are low, high deformation rates can be achieved through the closure of sub-horizontal fractures. At greater depth, this deformation mechanism diminishes due to increasing normal stiffnesses, and as such, the contribution of the intact matrix on the deformation becomes more relevant. Shear slip on steeply inclined brittle fault zones and the orientation of these fault zones can considerably affect the shape and localized variations in the subsidence profile.

The contribution of the intact rock matrix on steady state subsidence magnitudes were primary controlled by the values of the drained bulk modulus and Biot's
Analytical and numerical investigations suggest that steady state conditions were likely reached approximately several years after tunnel excavation.

Given the lack of prior treatment with respect to consolidation effects in crystalline rock masses in relation to large tunnelling projects, the lessons learned from this study may have implications for similar future tunnelling projects (i.e. the new AlpTransit high speed tunnelling project in the Swiss Alps; Loew et al. 2000). Findings from this study demonstrate the importance for continuous spatial and temporal deformation and rock mass pore pressure measurements over the course of the tunnelling project. The technical difficulty in installing such a pore pressure monitoring system and the resulting high costs for installation and maintenance, also in future will limit the availability of in situ pore pressure and rock mass deformation data. Nevertheless, the knowledge of the pore pressure drawdown from in situ measurements would be helpful to accurately predict the magnitude and extent of potential vertical displacements. Furthermore, in situ and laboratory testing of the poroelastic constants (i.e. Biot’s coefficients, bulk and shear modulus), and discontinuity deformation parameters (most notably normal stiffness) are necessary to adequately estimate deep tunnel induced surface deformations. To provide this data, current investigations are recommended to ascertain these parameters both in the lab and through in situ measurements.

### 5.9 Nomenclature

**Stresses and pressures**

- $\sigma_n$: total normal stress on a discontinuity (Pa)
- $\sigma'_n$: effective normal stress on a discontinuity (Pa)
- $\sigma_h/\sigma_n$: ratio of total horizontal to total vertical stress (Pa)
- $\sigma_{ij}$: macroscopic total stress (Pa)
- $\sigma'$: macroscopic effective stress (Pa)
- $\sigma_{kk}$: mean stress (Pa)
- $\tau$: shear stress on a discontinuity (Pa)
- $\tau_p$: shear strength of a fracture (Pa)
- $p$: water pore pressure (Pa)
- $F_x, F_y, F_z$: body forces

**Displacements and strains**

- $u_n$: normal displacement (m)
- $s_e$: elastic shear displacement (m)
- $s$: total shear displacement (m)
- $\Delta d$: fracture dilation (m)
- $\varepsilon_{ij}$: macroscopic strain ( )
- $\varepsilon_v$: macroscopic volumetric strain ( )
- $u, v, w$: displacements in x-, y- and z-direction (m)

**Material properties**

- $k_n$: normal stiffness (MPa/mm)
- $k_s$: shear stiffness (MPa/mm)
\( \alpha_f \) effective stress coefficient for fracture normal deformation ( )

\( \phi_i \) friction angle of meso-scale fractures (degree)

\( \phi_f \) friction angle of brittle fault zones (degree)

\( c_i \) cohesion of meso-scale fractures (Pa)

\( c_f \) cohesion of brittle fault zones (Pa)

\( \psi \) dilation angle (degree)

\( \alpha \) Biot’s coefficient of a porous medium ( )

\( B \) Skempton’s coefficient of a porous medium ( )

\( K \) drained intact rock bulk modulus (Pa)

\( K_s \) intrinsic bulk modulus of the solid constituent (Pa)

\( K_f \) fluid bulk modulus (Pa)

\( G \) drained intact rock shear modulus (Pa)

\( E \) intact rock Young’s modulus (Pa)

\( E_o \) deformation modulus of a rock mass including both the intact rock and discontinuities (Pa)

\( \nu \) Poisson’s ratio ( )

\( S_c \) constrained specific storage (1/Pa)

\( S_s \) coefficient of specific storage for constant vertical stress and zero lateral strain (1/m)

\( M \) Biot’s modulus (Pa)

\( D_s \) coefficient of hydraulic diffusivity (m²/s)

\( n \) rock porosity ( )

\( q, p \) constants for the empirical semi-logarithmic closure law of Bandis (1983), where \( \sigma_n \) in MPa and \( u_n \) in mm

**Fluid flow**

\( Q_c \) flow rate of a fracture (cubic law) (m³/s)

\( g \) gravitational acceleration constant (9.81 m/s²)

\( \rho_f \) density of fluid (kg/m³)

\( a \) equivalent hydraulic aperture (m)

\( a_m \) mechanical aperture

\( \nu \) kinematic viscosity of the fluid (?)

\( \mu \) dynamic viscosity of the fluid (Pa s)

\( dp/dl \) hydraulic gradient inside a fracture (Pa/m)

\( w \) fracture width in the direction of flow (m)

\( K_d \) tensor of hydraulic conductivity (Darcy’s law)

\( K_i \) isotropic hydraulic conductivity (m/s)

\( K_c \) equivalent hydraulic conductivity for fracture fluid flow (cubic law) in (m/s)

\( K_h \) equivalent horizontal hydraulic conductivity for fracture fluid flow (m/s)

\( K_v \) equivalent vertical hydraulic conductivity for fracture fluid flow (m/s)

\( V(h) \) hydraulic gradient within a porous medium ( )

\( Q \) explicit fluid source (m³/s)

\( k \) permeability matrix of a porous medium (m²)
Other variables
\(X\) fracture normal spacing (m)
\(t\) time (s)
\(\delta_{ij}\) Kronecker’s delta ( )

5.10 Acknowledgements

The authors would like to thank the AlpTransit Gotthard AG for their permission to publish the deformation analysis. Thanks are extended to Dr. Achim Kamelger for his PC support to shorten the time consuming numerical simulations. In addition, we would like to thank Dr. Giovanni Lombardi, Dr. Mohamed El Tani, Lombardi Consulting, and Dr. Corrado Fidelibus, ETH Zürich for their input during numerous discussions. The authors thank to Dr. Volker Luetzenkirchen and Dante Salvini, both ETH Zürich, for insightful discussions.
Table 5.1: Orientation and spacing data of mapped meso-scale fractures.

<table>
<thead>
<tr>
<th>Location</th>
<th>Number of set</th>
<th>Mean orientation dip direction/dip angle</th>
<th>$k^*$</th>
<th>$\theta^*$</th>
<th>Number of discontinuities</th>
<th>Mean spacing (m)</th>
<th>Standard deviation (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface data</td>
<td>1</td>
<td>137/71</td>
<td>21.7</td>
<td>1.5</td>
<td>461</td>
<td>0.47</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>222/87</td>
<td>1.3</td>
<td>18.3</td>
<td>217</td>
<td>1.31</td>
<td>2.84</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>352/61</td>
<td>13.8</td>
<td>2.4</td>
<td>280</td>
<td>0.91</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>264/40</td>
<td>9.7</td>
<td>5.9</td>
<td>67</td>
<td>0.90</td>
<td>1.32</td>
</tr>
<tr>
<td>Tunnel data</td>
<td>1</td>
<td>138/74</td>
<td>32.1</td>
<td>2.9</td>
<td>169</td>
<td>1.69</td>
<td>3.84</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>236/80</td>
<td>37.7</td>
<td>5.4</td>
<td>20</td>
<td>no data</td>
<td>no data</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1/60</td>
<td>14.0</td>
<td>5.5</td>
<td>159</td>
<td>0.79</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>257/43</td>
<td>24.8</td>
<td>4.3</td>
<td>46</td>
<td>&gt;3 estimated</td>
<td>no data</td>
</tr>
</tbody>
</table>

*Precision, $k$, and apical half-angle, $\theta$, of 95%-confidence cone from Fisher-analysis.

Table 5.2: Time for pore pressure drawdown within the discontinuity network.

<table>
<thead>
<tr>
<th>Dimension of fracture network (m)</th>
<th>$S_s$ (1/m)</th>
<th>$K$ (m/s)</th>
<th>$D_s$ (m²/s)</th>
<th>$t$ (d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L=800 m</td>
<td>1e-7</td>
<td>1e-6</td>
<td>1e+1</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>1e-7</td>
<td>1e-8</td>
<td>1e-1</td>
<td>32.9</td>
</tr>
<tr>
<td></td>
<td>1e-6</td>
<td>1e-6</td>
<td>1e+0</td>
<td>3.3</td>
</tr>
<tr>
<td></td>
<td>1e-6</td>
<td>1e-8</td>
<td>1e-2</td>
<td>329.3</td>
</tr>
</tbody>
</table>

Table 5.3: Dependency of specific storage and diffusion coefficient from Biot's and Skempton's coefficient.

<table>
<thead>
<tr>
<th>Biot's coefficient</th>
<th>Skempton's coefficient</th>
<th>$S_s$ (1/m)</th>
<th>$D_s$ (m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.92</td>
<td>2.5e-7</td>
<td>4.0e-5</td>
</tr>
<tr>
<td>0.75</td>
<td>0.90</td>
<td>2.5e-7</td>
<td>4.0e-5</td>
</tr>
<tr>
<td>0.5</td>
<td>0.86</td>
<td>2.2e-7</td>
<td>4.6e-5</td>
</tr>
<tr>
<td>0.25</td>
<td>0.75</td>
<td>1.5e-7</td>
<td>6.9e-5</td>
</tr>
</tbody>
</table>

Table 5.4: Time for pore pressure drawdown within intact blocks.

<table>
<thead>
<tr>
<th>Diffusion distance (m)</th>
<th>$D_s=1e-4$ (m²/s)</th>
<th>$D_s=1e-5$ (m²/s)</th>
<th>$D_s=3e-6$ (m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L/2=5</td>
<td>3 d</td>
<td>30 d</td>
<td>99 d</td>
</tr>
<tr>
<td>L/2=10</td>
<td>12 d</td>
<td>119 d</td>
<td>397 d</td>
</tr>
<tr>
<td>L/2=20</td>
<td>48 d</td>
<td>477 d</td>
<td>1590 d</td>
</tr>
<tr>
<td>L/2=50</td>
<td>298 d</td>
<td>2980 d</td>
<td>9934 d</td>
</tr>
</tbody>
</table>
Table 5.5: Compilation of laboratory and in situ normal closure experiments on meso-scale fractures in granitic rock (Parameters for the semi-logarithmic closure law).

<table>
<thead>
<tr>
<th>Reference and description</th>
<th>Rock type / fracture size</th>
<th>Sample number</th>
<th>Loading cycle [Mpa]</th>
<th>Parameter q</th>
<th>p</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laboratory tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Detournay (1980):</td>
<td>Charcoal Granite; 11x25cm</td>
<td>1</td>
<td>7.0</td>
<td>9.25</td>
<td>-1.48</td>
<td>0.98</td>
</tr>
<tr>
<td>Elliott et al. (1985):</td>
<td>Carnmcnllis Granite; 5x10cm</td>
<td>2</td>
<td>4.0</td>
<td>12.24</td>
<td>-0.97</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>6.0</td>
<td>16.77</td>
<td>-0.44</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>2.0</td>
<td>10.76</td>
<td>-0.55</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>4.0</td>
<td>26.36</td>
<td>-1.02</td>
<td>0.99</td>
</tr>
<tr>
<td>Raven &amp; Gale (1985):</td>
<td>Charcoal Black Granite; 610, 15, 19, 29 cm</td>
<td>1</td>
<td>30.0</td>
<td>12.24</td>
<td>-1.07</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>35.20</td>
<td>-0.55</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>64.85</td>
<td>-0.39</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>30.0</td>
<td>12.01</td>
<td>-2.45</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>30.0</td>
<td>32.75</td>
<td>-1.63</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>30.0</td>
<td>35.49</td>
<td>-1.42</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>30.0</td>
<td>10.51</td>
<td>-0.59</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>30.0</td>
<td>22.99</td>
<td>-1.54</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>30.0</td>
<td>24.40</td>
<td>-1.57</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>24.0</td>
<td>15.38</td>
<td>-1.12</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11</td>
<td>24.0</td>
<td>20.11</td>
<td>-1.03</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>22.0</td>
<td>22.02</td>
<td>-1.25</td>
<td>0.96</td>
</tr>
<tr>
<td>Schrauf &amp; Evans (1986):</td>
<td>Granodiorite; 28x28cm</td>
<td>Uniform flow</td>
<td>14.0</td>
<td>11.74</td>
<td>-2.63</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Radial flow</td>
<td>16.0</td>
<td>7.97</td>
<td>-3.45</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sample cycle 1</td>
<td>1</td>
<td>20.0</td>
<td>-1.76</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sample cycle 2</td>
<td>2</td>
<td>17.0</td>
<td>-6.49</td>
<td>0.98</td>
</tr>
<tr>
<td>Gale (1987):</td>
<td>Pinawa Granite; 46cm</td>
<td>H1</td>
<td>30.0</td>
<td>24.59</td>
<td>-0.42</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>H1</td>
<td>30.0</td>
<td>24.80</td>
<td>-0.72</td>
<td>0.87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>STR2</td>
<td>30.0</td>
<td>2.78</td>
<td>-1.94</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>STR2</td>
<td>30.0</td>
<td>12.21</td>
<td>-1.29</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>STR2</td>
<td>30.0</td>
<td>18.83</td>
<td>-1.44</td>
<td>0.98</td>
</tr>
<tr>
<td>Gentier (1987):</td>
<td>Granite; 412cm</td>
<td>4-1</td>
<td>80.0</td>
<td>50.00</td>
<td>-1.57</td>
<td>0.83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4-2</td>
<td>80.0</td>
<td>36.50</td>
<td>-1.55</td>
<td>0.94</td>
</tr>
<tr>
<td>Pyrak-Nolle et al (1987):</td>
<td>Strips granite; 45.2cm</td>
<td>E35</td>
<td>85.0</td>
<td>56.31</td>
<td>0.61</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E30</td>
<td>85.0</td>
<td>187.23</td>
<td>0.38</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>E32</td>
<td>85.0</td>
<td>312.58</td>
<td>0.42</td>
<td>0.94</td>
</tr>
<tr>
<td>Sundaram et al. (1987):</td>
<td>Charcoal Black Granite; 690cm</td>
<td>1</td>
<td>20.0</td>
<td>49.08</td>
<td>-2.05</td>
<td>0.81</td>
</tr>
<tr>
<td>Makurat et al (1990):</td>
<td>Strips granite; 60cm and in-situ block 100x140cm</td>
<td>1</td>
<td>25.0</td>
<td>8.66</td>
<td>-0.83</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>25.0</td>
<td>19.60</td>
<td>0.15</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>25.0</td>
<td>20.72</td>
<td>0.25</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>27.0</td>
<td>5.90</td>
<td>-1.32</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>27.0</td>
<td>28.46</td>
<td>-1.09</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3 (in situ)</td>
<td>10.0</td>
<td>19.20</td>
<td>-0.69</td>
<td>0.99</td>
</tr>
<tr>
<td>Witherspoon et al. (1980)</td>
<td>Granite; 415cm</td>
<td>1</td>
<td>20.0</td>
<td>34.13</td>
<td>-2.66</td>
<td>0.97</td>
</tr>
<tr>
<td>Zhao et al. (1992):</td>
<td>Carnmcnllis Granite; 5.1x10.2cm</td>
<td>EF1</td>
<td>8.0</td>
<td>26.25</td>
<td>-0.93</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>EF3</td>
<td>2.5</td>
<td>24.50</td>
<td>-0.50</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NJ1</td>
<td>7.0</td>
<td>42.77</td>
<td>-1.63</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NJ6</td>
<td>3.8</td>
<td>11.50</td>
<td>-0.85</td>
<td>0.98</td>
</tr>
<tr>
<td>Durham &amp; Bonner (1995)</td>
<td>Westerly Granite; 15x14cm,</td>
<td>Sample 1</td>
<td>160.0</td>
<td>2666.60</td>
<td>192.16</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sample 2</td>
<td>160.0</td>
<td>33.76</td>
<td>-1.60</td>
<td>0.95</td>
</tr>
</tbody>
</table>
Table 5.5: continued.

<table>
<thead>
<tr>
<th>Reference and description</th>
<th>Rock type / fracture size</th>
<th>Sample number</th>
<th>Loading cycle</th>
<th>( \sigma_n' ) [Mpa]</th>
<th>Parameter q</th>
<th>Parameter p</th>
<th>R²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laboratory tests</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sun et al (1985):</td>
<td>Grey Granite; 1009cm², Red Granite; 731cm²</td>
<td>red granite</td>
<td>1</td>
<td>2.7</td>
<td>11.83</td>
<td>-0.83</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.15</td>
<td>1</td>
<td>10.9</td>
<td>7.79</td>
<td>-0.72</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.14</td>
<td>1</td>
<td>8.2</td>
<td>7.49</td>
<td>-0.58</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.13</td>
<td>1</td>
<td>5.5</td>
<td>9.16</td>
<td>-0.63</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.12</td>
<td>1</td>
<td>2.7</td>
<td>11.13</td>
<td>-0.94</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.69</td>
<td>2</td>
<td>10.9</td>
<td>9.67</td>
<td>-0.86</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.60</td>
<td>2</td>
<td>8.2</td>
<td>5.08</td>
<td>-0.46</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.63</td>
<td>2</td>
<td>5.5</td>
<td>9.08</td>
<td>-0.90</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.66</td>
<td>2</td>
<td>2.7</td>
<td>11.99</td>
<td>-0.97</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.25</td>
<td>1</td>
<td>5.9</td>
<td>12.16</td>
<td>-1.16</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.24</td>
<td>1</td>
<td>4.0</td>
<td>22.62</td>
<td>-1.26</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.23</td>
<td>1</td>
<td>2.0</td>
<td>38.23</td>
<td>-0.52</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.05</td>
<td>2</td>
<td>5.9</td>
<td>23.65</td>
<td>-1.04</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.03</td>
<td>2</td>
<td>4.0</td>
<td>7.75</td>
<td>-0.63</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No.04</td>
<td>2</td>
<td>2.0</td>
<td>46.90</td>
<td>-1.51</td>
<td>0.97</td>
</tr>
<tr>
<td>Bandis (1983)</td>
<td>Dolerite; 4-6 x 8-10cm</td>
<td>1</td>
<td>1</td>
<td>50.0</td>
<td>29.72</td>
<td>-0.93</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>50.0</td>
<td>53.31</td>
<td>-0.59</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>50.0</td>
<td>61.28</td>
<td>-0.52</td>
<td>0.97</td>
</tr>
<tr>
<td>Bart et al. (2000)</td>
<td>Granite; unknown</td>
<td>1</td>
<td>1</td>
<td>25.0</td>
<td>24.52</td>
<td>-1.15</td>
<td>0.96</td>
</tr>
<tr>
<td>Gale (1982)</td>
<td>Granitic Gneiss; 15x30cm</td>
<td>sample 9</td>
<td>1</td>
<td>30.0</td>
<td>21.03</td>
<td>-7.67</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>28.15</td>
<td>-3.18</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>48.88</td>
<td>-3.54</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sample 10</td>
<td>1</td>
<td>30.0</td>
<td>12.38</td>
<td>-5.06</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>24.72</td>
<td>-3.35</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>42.50</td>
<td>-3.17</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sample 33</td>
<td>1</td>
<td>30.0</td>
<td>10.91</td>
<td>-2.26</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>24.82</td>
<td>-3.77</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>22.05</td>
<td>-2.73</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sample 34</td>
<td>1</td>
<td>30.0</td>
<td>22.66</td>
<td>-4.64</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>17.40</td>
<td>-2.57</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>17.67</td>
<td>-2.66</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sample 40</td>
<td>1</td>
<td>30.0</td>
<td>11.94</td>
<td>-0.90</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>35.79</td>
<td>-3.03</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sample 41</td>
<td>1</td>
<td>30.0</td>
<td>13.89</td>
<td>-1.36</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>14.64</td>
<td>-1.18</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>sample 42</td>
<td>1</td>
<td>30.0</td>
<td>15.70</td>
<td>-1.24</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>30.0</td>
<td>27.23</td>
<td>-1.94</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>30.0</td>
<td>28.12</td>
<td>-2.38</td>
<td>0.99</td>
</tr>
<tr>
<td>Lamas (1995)</td>
<td>Granite; 5.1 x 10.2cm</td>
<td>A01</td>
<td>1</td>
<td>12.0</td>
<td>65.74</td>
<td>1.25</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A03</td>
<td>1</td>
<td>4.0</td>
<td>20.79</td>
<td>-1.00</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A07</td>
<td>1</td>
<td>10.0</td>
<td>49.38</td>
<td>-1.05</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A08</td>
<td>1</td>
<td>4.0</td>
<td>200.54</td>
<td>-1.12</td>
<td>1.00</td>
</tr>
<tr>
<td>Iwano &amp; Einstein (1995)</td>
<td>Kikuma Granodiorite; 50x10cm</td>
<td>natural</td>
<td>1</td>
<td>20.0</td>
<td>10.19</td>
<td>-0.43</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td>tension</td>
<td>1</td>
<td>20.0</td>
<td>29.60</td>
<td>-0.40</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>20.0</td>
<td>27.98</td>
<td>-0.38</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>20.0</td>
<td>7.46</td>
<td>-0.14</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>20.0</td>
<td>9.04</td>
<td>0.10</td>
<td>0.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>20.0</td>
<td>9.29</td>
<td>0.07</td>
<td>0.97</td>
</tr>
</tbody>
</table>
### Table 5.5: continued.

<table>
<thead>
<tr>
<th>Reference and description</th>
<th>Rock type / fracture size</th>
<th>Sample number</th>
<th>Loading cycle</th>
<th>$\sigma_n^+$ [Mpa]</th>
<th>Parameter value</th>
<th>$q$</th>
<th>$p$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>In situ test</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Makurat et al (1990):</td>
<td></td>
<td>3 (in situ)</td>
<td>1</td>
<td>10.0</td>
<td>19.20</td>
<td>-0.69</td>
<td>0.99</td>
<td></td>
</tr>
<tr>
<td>Jung (1989)</td>
<td>Falkenberg Granite; in-situ</td>
<td>HB4a</td>
<td>1</td>
<td>1.9</td>
<td>1.78</td>
<td>-0.85</td>
<td>0.93</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>SB5</td>
<td>1</td>
<td>1.9</td>
<td>1.20</td>
<td>-0.86</td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td>Pratt et al. (1977)</td>
<td>Sherman Granite; in-situ 280x260cm</td>
<td>D5</td>
<td>1</td>
<td>4.0</td>
<td>38.76</td>
<td>-1.06</td>
<td>0.92</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>D4</td>
<td>1</td>
<td>4.0</td>
<td>10.39</td>
<td>-1.08</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>D6</td>
<td>1</td>
<td>4.0</td>
<td>8.46</td>
<td>-1.14</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>D2 (2 fractures)</td>
<td>1</td>
<td>4.0</td>
<td>2.12</td>
<td>-0.97</td>
<td>0.86</td>
<td></td>
</tr>
<tr>
<td>Evans &amp; Wyatt (1984)</td>
<td>Granite</td>
<td></td>
<td>1.5</td>
<td></td>
<td>$K_n&lt;20$ MPa/mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rutqvist (1995)</td>
<td>Granite</td>
<td></td>
<td></td>
<td></td>
<td>$K_n=10$ MPa/mm</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Back-calculated parameters by numerical modelling of well tests</td>
<td>Granite</td>
<td>* in situ total normal stress to the fracture</td>
<td>2.2*</td>
<td>$K_n=100$ MPa/mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.5*</td>
<td>$K_n=150$ MPa/mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6.5*</td>
<td>$K_n=200$ MPa/mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.4*</td>
<td>$K_n=250$ MPa/mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9.5*</td>
<td>$K_n=1000$ MPa/mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>11.5*</td>
<td>$K_n=1100$ MPa/mm</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 5.6: Intact rock and discontinuity properties for parametric study.

<table>
<thead>
<tr>
<th>Model Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rock Properties</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td>2700</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>Bulk modulus</td>
<td>20.0</td>
<td>GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>20.3</td>
<td>GPa</td>
</tr>
<tr>
<td><strong>Discontinuity Properties</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal stiffness</td>
<td>stress-and fracture spacing dependent (see Fig.5.15)</td>
<td>MPa/mm</td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>1</td>
<td>MPa/mm</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0</td>
<td>MPa</td>
</tr>
<tr>
<td>Friction angle</td>
<td>40</td>
<td>degree</td>
</tr>
<tr>
<td>Dilation angle</td>
<td>0</td>
<td>degree</td>
</tr>
<tr>
<td>Residual aperture</td>
<td>0.3</td>
<td>mm</td>
</tr>
<tr>
<td>Zero stress aperture</td>
<td>0.6</td>
<td>mm</td>
</tr>
</tbody>
</table>
Table 5.7: Discontinuity properties of fully coupled models.

<table>
<thead>
<tr>
<th>Discontinuity Properties</th>
<th>Fractures</th>
<th>Brittle fault zones</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal stiffness</td>
<td></td>
<td></td>
<td>10</td>
</tr>
<tr>
<td>Stress range 0-5 MPa; $k_n=2.9^\circ$</td>
<td></td>
<td></td>
<td>MPa/mm</td>
</tr>
<tr>
<td>Stress range 5-20 MPa; $k_n=25^\circ$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress range 20-50 MPa; $k_n=75^\circ$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear stiffness</td>
<td>1</td>
<td>0.1</td>
<td>MPa/mm</td>
</tr>
<tr>
<td>Cohesion</td>
<td>0</td>
<td>0</td>
<td>MPa</td>
</tr>
<tr>
<td>Friction angle</td>
<td>40</td>
<td>30</td>
<td>degree</td>
</tr>
<tr>
<td>Dilation angle</td>
<td>0</td>
<td>0</td>
<td>degree</td>
</tr>
<tr>
<td>Residual aperture</td>
<td>0.06/0.06</td>
<td>0.15/0.15</td>
<td>mm</td>
</tr>
<tr>
<td>Zero stress aperture</td>
<td>2.4/0.3</td>
<td>2.5/0.75</td>
<td>mm</td>
</tr>
</tbody>
</table>

Upscaled for 10 m fracture spacing

Table 5.8: Normal stiffness and spacing of horizontal meso-scale fractures for the Gotthard case study.

<table>
<thead>
<tr>
<th>Depth range (m)</th>
<th>Normal spacing (m)</th>
<th>Range of effective normal stress; Normal stiffness (MPa); (MPa/mm)</th>
<th>Actual spacing (m)</th>
<th>Range of effective normal stress; normal stiffness of one single fracture (MPa/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-600</td>
<td>40</td>
<td>0-2; $k_n=0.4$</td>
<td>1</td>
<td>0-2; $k_n=15.4$</td>
</tr>
<tr>
<td>600-1600</td>
<td>80</td>
<td>2-6.3; $k_n=2.2$</td>
<td>2</td>
<td>2-6.3; $k_n=86.2$</td>
</tr>
<tr>
<td>1600-2600</td>
<td>160</td>
<td>6.3-15.9; $k_n=6.0$</td>
<td>4</td>
<td>6.3-15.9; $k_n=238.5$</td>
</tr>
<tr>
<td>2600-3150</td>
<td>320</td>
<td>15.9-50; $k_n=17.1$</td>
<td>8</td>
<td>15.9-50; $k_n=683.0$</td>
</tr>
</tbody>
</table>

Table 5.9: Hydraulic properties of brittle fault zones for the Gotthard case study.

<table>
<thead>
<tr>
<th>Region, see Figure 5.31</th>
<th>Transmissivity (m$^3$/s)</th>
<th>Hydraulic conductivity (m/s)</th>
<th>Hydraulic aperture (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (BF5N, BF5S)</td>
<td>3.0e-6</td>
<td>1.2e-9</td>
<td>0.039</td>
</tr>
<tr>
<td>2 (BF2)</td>
<td>1.8e-5</td>
<td>1.1e-8</td>
<td>0.074</td>
</tr>
<tr>
<td>3 (BF1)</td>
<td>2.4e-4</td>
<td>9.1e-6</td>
<td>0.459</td>
</tr>
<tr>
<td>4 (BF3)</td>
<td>9.6e-6</td>
<td>6.3e-9</td>
<td>0.070</td>
</tr>
<tr>
<td>5 (BF4)</td>
<td>5.0e-6</td>
<td>2.4e-9</td>
<td>0.046</td>
</tr>
</tbody>
</table>
### Table 5.10: Hydraulic properties of horizontal meso-scale fractures for the Gotthard case study.

<table>
<thead>
<tr>
<th>Depth range (m)</th>
<th>Spacing (m)</th>
<th>Hydraulic conductivity (m/s)</th>
<th>Hydraulic aperture (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1 0-600</td>
<td>40</td>
<td>1e-7</td>
<td>0.170</td>
</tr>
<tr>
<td>F2 600-1600</td>
<td>80</td>
<td>1e-8</td>
<td>0.099</td>
</tr>
<tr>
<td>F3 1600-2600</td>
<td>160</td>
<td>5e-9</td>
<td>0.099</td>
</tr>
<tr>
<td>F4 2600-3150</td>
<td>320</td>
<td>1e-9</td>
<td>0.011</td>
</tr>
</tbody>
</table>

### Table 5.11: Analytical solutions of surface subsidence.

<table>
<thead>
<tr>
<th>Drained intact rock (m)</th>
<th>Δp (MPa)</th>
<th>α (°)</th>
<th>K (GPa)</th>
<th>ε11 = ε22 = ε33</th>
<th>Vertical subsidence (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>800</td>
<td>2</td>
<td>0.7</td>
<td>20</td>
<td>7.0e-5</td>
<td>2.33e-5</td>
</tr>
<tr>
<td>800</td>
<td>4</td>
<td>0.7</td>
<td>20</td>
<td>1.4e-4</td>
<td>4.67e-5</td>
</tr>
<tr>
<td>800</td>
<td>2</td>
<td>0.7</td>
<td>10</td>
<td>1.4e-4</td>
<td>4.67e-5</td>
</tr>
<tr>
<td>800</td>
<td>4</td>
<td>0.7</td>
<td>10</td>
<td>2.8e-4</td>
<td>9.33e-5</td>
</tr>
<tr>
<td>1000</td>
<td>2</td>
<td>0.7</td>
<td>10</td>
<td>1.4e-4</td>
<td>4.67e-5</td>
</tr>
<tr>
<td>1000</td>
<td>4</td>
<td>0.7</td>
<td>10</td>
<td>2.8e-4</td>
<td>9.33e-5</td>
</tr>
<tr>
<td>1200</td>
<td>2</td>
<td>0.7</td>
<td>10</td>
<td>1.4e-4</td>
<td>4.67e-5</td>
</tr>
<tr>
<td>1200</td>
<td>4</td>
<td>0.7</td>
<td>10</td>
<td>2.8e-4</td>
<td>9.33e-5</td>
</tr>
</tbody>
</table>

### Table 5.12: Input parameters for the isotropic intact rock model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>2700</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Young’s modulus</td>
<td>45.5</td>
<td>GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.12</td>
<td>(</td>
</tr>
<tr>
<td>Bulk modulus</td>
<td>20</td>
<td>GPa</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>20</td>
<td>GPa</td>
</tr>
<tr>
<td>Hydraulic conductivity</td>
<td>1e-8</td>
<td>m/s</td>
</tr>
<tr>
<td>Biot’s coefficient</td>
<td>0.70</td>
<td>(</td>
</tr>
<tr>
<td>Skempton’s coefficient</td>
<td>0.92</td>
<td>(</td>
</tr>
</tbody>
</table>
Table 5.13: Input parameters for the layered intact rock model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Layer measured from surface in (m)</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-200</td>
<td>200-400</td>
</tr>
<tr>
<td>Density</td>
<td>2700</td>
<td>2700</td>
</tr>
<tr>
<td>Young’s modulus(^a)</td>
<td>45.5</td>
<td>45.5</td>
</tr>
<tr>
<td>Young’s modulus(^b)</td>
<td>20</td>
<td>23.5</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.12</td>
<td>0.12</td>
</tr>
<tr>
<td>Hydraulic conductivity</td>
<td>1e-8</td>
<td>1e-8</td>
</tr>
<tr>
<td>Biot’s coefficient</td>
<td>0.70</td>
<td>0.65</td>
</tr>
<tr>
<td>Skempton’s coefficient</td>
<td>0.92</td>
<td>0.90</td>
</tr>
</tbody>
</table>

\(^a\) constant intact rock Young’s modulus
\(^b\) stress dependency of the intact rock Young’s modulus

---

Table 5.14: Input parameters for the layered equivalent rock mass model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Layers measured from surface in (m)</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-200</td>
<td>200-400</td>
</tr>
<tr>
<td>Density</td>
<td>2700</td>
<td>2700</td>
</tr>
<tr>
<td>Equivalent E-modulus(^a)</td>
<td>13.2</td>
<td>19.5</td>
</tr>
<tr>
<td>Equivalent E-modulus(^b)</td>
<td>20.8</td>
<td>32.6</td>
</tr>
<tr>
<td>Equivalent E-modulus(^c)</td>
<td>8.7</td>
<td>14.1</td>
</tr>
<tr>
<td>Equivalent Poisson’s ratio</td>
<td>0.12</td>
<td>0.12</td>
</tr>
<tr>
<td>Equivalent hydraulic conductivity</td>
<td>1e-8</td>
<td>1e-8</td>
</tr>
<tr>
<td>Biot’s coefficient</td>
<td>0.70</td>
<td>0.65</td>
</tr>
<tr>
<td>Skempton’s coefficient</td>
<td>0.92</td>
<td>0.90</td>
</tr>
</tbody>
</table>

\(^a\) stress dependency of the intact rock Young’s modulus
\(^b\) constant intact rock Young’s modulus (E=45.5 GPa)
\(^c\) stress dependency of the intact rock Young’s modulus (lower boundary)
Table 5.15: Summary of results showing maximum subsidence generated from discrete discontinuity (discontinuum) and intact rock matrix deformation (continuum)

<table>
<thead>
<tr>
<th>Underlying deformation mechanisms</th>
<th>Maximum subsidence (m)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discontinuum (Discontinuity network)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal and shear deformation on discontinuities; Poisson’s ratio effects on intact matrix blocks</td>
<td>0.042</td>
<td>LHB$^1$ $p_0$=26.5 MPa, UHB$^2$ $p_0$=0.001 MPa</td>
</tr>
<tr>
<td></td>
<td>0.045</td>
<td>LHB $p_0$=30.9 MPa, UHB $p_0$=0.001 MPa</td>
</tr>
<tr>
<td></td>
<td>0.049</td>
<td>LHB impermeable, UHB $p_0$=0.001 MPa</td>
</tr>
<tr>
<td></td>
<td>0.080</td>
<td>Exceptional low $k_0$ of faults and fractures</td>
</tr>
<tr>
<td></td>
<td>0.032</td>
<td>LHB $p_0$=26.5 MPa, UHB $p_0$=0.01 MPa</td>
</tr>
<tr>
<td>Range</td>
<td><strong>0.032-0.080</strong></td>
<td></td>
</tr>
<tr>
<td>Continuum (Intact rock matrix)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poroelasticity of the intact rock matrix (fully isotropic)</td>
<td>0.060</td>
<td>$E$, $v$, $\alpha$, $B$, $K$ constant</td>
</tr>
<tr>
<td>Poroelasticity of the intact rock matrix (anisotropic permeability)</td>
<td>0.016</td>
<td>$E$, $v$, $\alpha$, $B$ constant; $K_3=1e-8$m/s, $K_1=1e-6$m/s; small pore pressure drawdown</td>
</tr>
<tr>
<td>Poroelasticity of the intact rock matrix (layered model)</td>
<td>0.043</td>
<td>$E$, $v$, $K$ constant; $\alpha$ and $B$ stress dependent</td>
</tr>
<tr>
<td>Poroelasticity of the intact rock matrix (layered model)</td>
<td>0.068</td>
<td>$v$, $K$ constant; $\alpha$, $B$ and $E$ stress dependent</td>
</tr>
<tr>
<td>Range</td>
<td><strong>0.016-0.068</strong></td>
<td></td>
</tr>
</tbody>
</table>

$^1$ Lower hydraulic boundary, $^2$ Upper hydraulic boundary (surface), $^3$ $p$=pore pressure

Table 5.16: Summary of results showing maximum subsidence originated from equivalent rock mass models (continuum approach).

<table>
<thead>
<tr>
<th>Underlying deformation mechanisms</th>
<th>Maximum subsidence (m)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuum (Equivalent medium)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Poroelasticity of the rock mass (layered model)</td>
<td>0.054 m</td>
<td>$E_o$, $\alpha$, $B$, stress dependent; $v$, $K$, $E$ constant</td>
</tr>
<tr>
<td>Poroelasticity of the rock mass (layered model)</td>
<td>0.081 m</td>
<td>$E_o$, $E$, $\alpha$, $B$ stress dependent; $v$, $K$ constant</td>
</tr>
<tr>
<td>Poroelasticity of the rock mass (layered model)</td>
<td>0.110 m</td>
<td>$E_o$, $E$, $\alpha$, $B$ stress dependent; $v$, $K$ constant</td>
</tr>
<tr>
<td>Range</td>
<td><strong>0.054-0.110</strong></td>
<td></td>
</tr>
</tbody>
</table>
Figure 5.1: (a) Conceptual model of full hydro-mechanical coupling mechanism along a discontinuity. The hydraulic aperture is controlled by the mechanical aperture, which is influenced by normal closure and shear dilation. (b) Conceptual model of partial hydro-mechanical coupling mechanism along a discontinuity. The hydraulic aperture remains constant and does not vary with effective normal stress changes.
Figure 5.2: Surface subsidence between the time interval 1970 to 1993/98 and alpine uplift in the time interval 1918 to 1970 (upper diagram). Initial water inflow rates into Gotthard highway tunnel (i.e. safety tunnel) during excavation (lower diagram).

Figure 5.3: Calculated vertical strain rates (dashed line) and surface topography along the levelling profile (i.e. Gotthard pass road, gray line).
Figure 5.4: Location map with study area (upper diagram). Geological map of the Gotthard region including levelling and triangulation measurements (lower diagram).
Figure 5.5: Detailed map of the Gamsboden granitic gneiss showing traces of mapped and inferred fault zones. As well, the location of levelling points and the major inflow zone are shown.

Figure 5.6: *In situ* block size distribution near surface and at tunnel elevation.
Figure 5.7: Conceptual diffusion model.

Figure 5.8: (a) Normal closure laboratory experiments on fractures in granitic rock, including all loading cycles. Standard normal closure law implemented into UDEC models. (b) Parameters for the semi-logarithmic normal closure law determined from laboratory and in situ tests.
Figure 5.9: (a) Compilation of laboratory E-moduli tests determined on fault core samples within the Swiss Alps. (b) Distribution of fault zone (i.e. core) thickness measured by Schneider (1979) and Wanner (1982) in the Gotthard (A2) safety tunnel.
Figure 5.10: Model geometries used for the parametric study: (a) Continuous 50 m spaced vertical fault zones and 10 m spaced horizontal fractures; (b) Continuous 50 m spaced vertical fault zones, 50 m spaced vertical fractures and 10 m spaced horizontal fractures; (c) Continuous 50 m spaced inclined fault zones and non-continuous 10 m spaced inclined fractures; (d) Fault zone pattern implemented from measurement in the Gotthard safety tunnel. Fault zones 2000 m north and south from the major inflow zone were included. Continuous horizontal fractures with a fracture spacing of 25 m. Hydraulic and mechanical boundary conditions were identical for all four geometry types.
Figure 5.11: (a) Initial pore pressure distribution before tunnel drainage. (b) Pore pressure distribution after tunnel drainage for model geometry types A, B, C. (c) Pore pressure distribution after tunnel drainage for model geometry type D.
Figure 5.12: Surface subsidence determined for model geometry types A, B, C, and D. Note large boundary effects on model type C due shear slip near the upper left and lower right model boundary.

Figure 5.13: Shear displacement along brittle fault zones. Negative values indicate left-handed shear and positive values indicate right-handed shear displacement.
Figure 5.14: Parametric study results for model geometry A: (a) Variation of maximum subsidence with intact rock bulk modulus; (b) Variation of maximum subsidence with normal stiffness of horizontal fractures; (c) Variation of maximum subsidence with normal stiffness of vertical fault zones; (d) Variation of maximum subsidence with shear stiffness of vertical fault zones; (e) Variation of maximum subsidence with in situ horizontal to vertical stress ratio.
Figure 5.15: Normal closure law for 1 m spaced horizontal fractures implemented for the parametric study (see Figure 5.14 b).

Figure 5.16: Parametric study results for model geometry C: (a) Variation of maximum subsidence with \textit{in situ} horizontal to vertical stress ratio; (b) Variation of maximum subsidence with shear stiffness of vertical faults.
Figure 5.17: Pore pressure distribution for hydraulic conductivity study on model geometry A after the tunnel drainage. (a) Horizontal to vertical conductivity equals 1.1; (b) Horizontal to vertical conductivity equals 0.1; (c) Horizontal to vertical conductivity equals 0.01; (d) Horizontal to vertical conductivity equals 0.001.
Figure 5.18: Vertical displacement contours of hydraulic conductivity study using model geometry A.
Figure 5.19: Surface subsidence of hydraulic conductivity study using model geometry A.

Figure 5.20: Variation of the maximum subsidence with the hydraulic conductivity ratio.
Figure 5.21: Shear displacement along vertical fault zones for the hydraulic conductivity study using model geometry A: (a) Horizontal to vertical conductivity equals 1.1; (b) Horizontal to vertical conductivity equals 0.001; Negative values indicate left-handed shear and positive values indicate right-handed shear displacement.
Figure 5.22: Variation of hydraulic apertures for the fully coupled model (scenario 1). Variation of the hydraulic apertures (a) of the vertical brittle fault zones; (b) of the horizontal fractures.

Figure 5.23: (a) Pore pressure distribution after the tunnel drainage for the fully coupled model (scenario 1). (b) Vertical displacement contours.
Figure 5.24: (a) Pore pressure distribution after tunnel drainage for the fully coupled model (scenario 2). (b) Vertical displacement contours.

Figure 5.25: Variation of hydraulic apertures along vertical fault zones for the fully coupled model (scenario 2).
Figure 5.26: Pore pressure drawdown for model geometry D: (a) Free watertable with no recharge; (b) Fixed pore pressure boundary condition (recharge).

Figure 5.27: Water flow rates into the model from surface model boundary.
Figure 5.28: Subsidence for model geometry D: (a) Surface subsidence for free watertable and fixed pore pressure boundary. (b) Vertical displacement contours for free watertable boundary model. (c) Vertical displacement contours for fixed pore pressure boundary model.
Figure 5.29: Conceptual and boundary condition model along a N-S section for the Gotthard case study.

Figure 5.30: Conceptual hydrogeological model (i.e. watertable drawdown).
Figure 5.31: Discontinuity pattern and applied hydraulic transmissivities for the Gotthard case study (given in Table 5.9).
Figure 5.32: Pore pressure distributions before and after tunnel drainage, respectively, for: (a, b) Hydraulic boundary type I; (c, d) Hydraulic boundary type II; (e, f) Hydraulic boundary type III.
Figure 5.33: Vertical displacement contours for the Gotthard case study.

Figure 5.34: Surface subsidence trough: (a) Numerically simulated; (b) Measured by leveling technique along the Gotthard pass road.
Vertical displacement in (m)
Regions of increased shear displacement
Discontinuity pattern

North-south section (m)
0 2000 4000 6000 8000 10000

Elevation (m)
0 -500 -1000 -1500 -2000 0 2000

0.02 0.00 -0.02 -0.04

Figure 5.35: Shear displacement along discontinuities and surface subsidence profile.
Figure 5.36: Normal closure along brittle fault zones.

Figure 5.37: Principal stress direction within intact blocks (green) and discontinuities (red lines) near the large inflow zone.
Figure 5.38: Pore pressure distribution: (a) Before tunnel drainage; (b) After tunnel drainage for the hydraulically isotropic model; (c) After tunnel drainage for the hydraulically anisotropic model.
171

Figure 5.39: Surface subsidence for intact rock matrix models.

Figure 5.40: Distribution of: (a) Vertical strain; (b) Horizontal strain for the intact rock matrix models.
Figure 5.41: Pore pressure difference between initial and drained condition. Inside the gray colored region pore pressure drawdown reaches magnitudes exceeding 0.5 MPa.

Figure 5.42: Surface subsidence for the equivalent rock mass models.
6. Final Conclusions and Perspectives
6.1 Conclusions

Surface subsidence induced by deep tunnelling in heavily fractured crystalline rock masses has been shown through this work to be possible and to be controlled by the magnitude and extension of the pore pressure drawdown within the rock mass. This involves a short-term phase where the pore pressure drawdown first affects the discontinuity network altering their effective and total stress state. Simultaneous to this phase, a longer-term process begins involving pore pressure diffusion starting from along the discontinuities and penetrating into the intact rock matrix. Stress changes due to the pore pressure decrease induce in the fractured rock mass contraction strains and accordingly surface subsidence. Whereas the magnitude of the pore pressure drawdown is predominately controlled by the hydraulic permeability of the discontinuity network and their connectivity with the tunnel, deformations within the rock mass are generated by both the hydro-mechanical coupling properties of the discontinuity network and the poroelastic response of the low-porosity intact rock matrix to the pore pressure drawdown.

The following subsections summarize the key findings and conclusions made through the different components of this thesis work.

6.1.1 Brittle Fault Zones and Fractures in Anisotropic Crystalline Rocks of the Central Gotthard Massif

To study the influence of the discontinuity network on drainage induced surface subsidence, knowledge about the geometrical structure of the fracture network and their structural architecture and characteristics is essential. Orientation measurements and spacing data were used to ascertain the geometry of the discontinuity network, which in turn was implemented into a two-dimensional distinct-element analysis. In addition, information about the formation and the stress history of brittle structures were provided.

- It was found that the nucleation and propagation of brittle fault zones was likely influenced by pre-existing rock anisotropy, formed through geological boundaries (e.g. igneous dykes), ductile structures (e.g. schistosity, mylonitic foliation) and brittle structures (e.g. meso scale fractures).

- Three sets of brittle fault zones, a NE-SW, NNE-SSW and E-W could be defined on the basis of fault plane measurements and geomorphic lineaments. Based on measurements of the pitch of slickenside striations, most of the brittle fault zones were activated in a strike-slip mode. Observations from meso- to micro-scale indicate that right-handed shear sense was the dominating movement process.

- For each, the main foliation, sub-parallel meso-scale fractures and sub-parallel brittle faults all form a fan-like structure showing the same orientation (i.e. NE-SW strike) and location of the symmetry plane.
• Approximately four different orientated meso-scale fracture sets were mapped through outcrop and scanline sampling techniques: 1) The main fracture set predominately strikes NE-SW, and steeply dips sub-parallel to the main foliation. The fracture normal spacing of this set increases with depth by a factor of 3.5 and the spacing probability distribution changes from that of a negative exponential (determined on surface) to a Weibull distribution (found in the highway tunnel); 2) Occasionally, at the tip of these fractures, a second fracture set can be discerned enclosing an angle of 20 to 50°. It is suggested that these secondary fractures may be formed through reactivation of the main fracture set in a shear mode, syn-tectonically with the strike-slip faulting process; 3) Perpendicular to the main fracture set, a steeply dipping NW-SE orientated set was mapped, often developed as a conjugate fracture system, and 4) A flat to medium dipping fracture set parallel to the large-scale topography occurs with a reduced frequency at the tunnel level.

6.1.2 Laboratory Measurements of Biot's Coefficient for Low-Porosity Granitic Rocks

This component of the study was initiated to test whether poroelastic strains induced within the intact rock matrix could be sufficiently high enough to contribute towards surface deformation. The key parameter, the Biot's coefficient ($\alpha$), represents the coupling factor between pore pressure changes and the rock stress response within the intact rock matrix. Due to the lack of available and reliable values for the Biot’s coefficient for low-porosity (< 1%), low-permeable (< 1e-10 m/s) granitic rocks, a laboratory testing campaign was initiated to investigate this coefficient. Results clearly showed that the coupling between the pore pressure and the intact rock response cannot be neglected for such crystalline rocks:

• Values determined for the Biot’s coefficient on rock samples from the Aar and Gamsboden granitic gneiss in Central Switzerland were found to range between 0.27±0.13 and 0.80±0.08, depending on the applied hydrostatic stress condition.

• Biot’s coefficient measured for granitic samples from the Aar and Gamsboden granitic gneiss was found to be stress dependent.

6.1.3 Generic 2-D Studies in Vertical Tunnel Cross Sections

Before simulations involving the actual case study of the Gotthard highway tunnel were performed, generic analyses were performed to investigate underlying subsidence mechanisms and sensitivity of individual parameters. Studied in detail were the effects of (1) sub-horizontal fractures, of (2) sub-vertical discontinuities (i.e. fractures and brittle fault zones) and (3) the intact rock matrix on poroelastic deformations resulting in surface subsidence.

• Frequency and the normal stiffness of sub-horizontal fractures have a large impact on the magnitude of vertical displacements.
Results involving simple model geometries (i.e. horizontal and vertical orientated discontinuities), showed that fracture closure alone is not the only key subsidence producing mechanism, but that the intact rock matrix also considerably contributes towards rock mass deformation through poroelastic strains.

Vertical discontinuities contribute to surface subsidence through intact block strains induced through the Poisson’s ratio effect. As such, depending on the applied values of normal and shear stiffness they can influence strongly the shape of the trough, forming in extreme cases a bimodal subsidence profile.

6.1.4 Analysis of Surface Subsidence in Crystalline Rocks above the Gotthard Highway Tunnel

Building on results from the generic study, the attained knowledge was extended to the actual case study of the Gotthard pass region and highway tunnel. To study the magnitude of discontinuity and intact rock matrix induced subsidence on the Gotthard pass region, two separate modelling approaches were applied, adopting first a continuum approach followed by a discontinuum approach. Although, the direct summation of magnitudes in subsidence generated from the continuum (i.e. poroelastic strains of the intact rock matrix) and the discontinuum (i.e. closure and shear on discrete discontinuities and Poisson’s ratio induced strains in the intact rock matrix) analyses incorporates some physical overlap, relative comparisons between the two simulation techniques can be made.

Results emphasized the importance of quantifying the pore pressure drawdown, which is influenced by the hydraulic connectivity of the rock mass around the tunnel, the hydraulic anisotropy of the rock mass, the hydraulic boundary conditions (i.e. no flow boundary or fixed pore pressure boundary) and the magnitude of surface recharge.

Discrete-element discontinuum modelling (UDEC) provided insight into the consolidation mechanisms involving discontinuity deformation and Poisson’s ratio effect in the intact rock. Results from the Gotthard case study produced maximum subsidence magnitudes ranging between 0.032 to 0.080 m, narrowing to 0.04 to 0.05 m when the most reasonable input parameters were used.

Finite-element continuum modelling (VISAGE) provided insights into the poroelastic response of the intact rock matrix. The calculated maximum subsidence for the Gotthard case study ranged between 0.016 to 0.068 m, narrowing to 0.05 to 0.06 m when the more realistic input parameter values were used.

Of key importance, it was found that shear and normal displacement along steeply inclined brittle fault zones can have a large impact on the shape of the
subsidence trough. In doing so, the fan structure observed at the Gotthard pass and the spatial distribution of these brittle fault zones (i.e. clustering) influences the degree of shear and normal deformation and subsequently, the shape of subsidence trough in the Gotthard pass region.

6.2 Perspectives for Further Investigations

Based on the results from this thesis work, further investigations may be extended with respect to this project (i.e. Gotthard highway tunnel), to the new deep alpine tunnels currently under construction (i.e. AlpTransit) or in conjunction with other deep tunneling projects in crystalline rock being undertaken in the European Alps or elsewhere around the world.

6.2.1 In situ Determination of Poroelastic Parameters

For rock masses showing minor structural anisotropy, poroelastic rock mass parameters can be estimated through applying barometric and tidal loading based measurements collected through in situ experiments. Based on time series analyses (sampling duration in the range from several months to years) estimations of poroelastic parameters (i.e. shear and bulk modulus, Skempton’s and Biot’s coefficient) can be obtained. Such a study has been recently initiated in the Bedretto Tunnel located nearby to the Gotthard highway tunnel to investigate the poroelastic response of the Rotondo granite.

6.2.2 In situ Determination of Mechanical Fracture Parameters

Pore pressure induced subsidence can be used as a large-scale field experiment that enables the determination of the rock mass deformability of a large volume of rock. In combination with measuring the in situ pore pressure, direct measurements of rock strain along boreholes can be conducted. Kovari and Peter (1983) developed a method for continuous monitoring of strains along vertical boreholes by using a “Sliding Micrometer” (ISETH) with a high accuracy of \( \pm 0.003 \) mm. Application of this or similar techniques would accordingly provide useful information about the deformability of sub-horizontal fractures and the intact rock matrix when applied before, during and after the consolidation subsidence process.

6.2.3 3-D Numerical Simulations

It is recommended that further numerical simulations be extended into 3D to investigate the effect of the surface topography through applying three-dimensional numerical codes. It was found that the most favorable numerical approach would rest upon an approach having the capacity to simulate 3D hydro-mechanical coupled processes of discrete fractures and the intact rock matrix (see Guvanasen and Chan 2000). Important is the capability to couple the pore pressure along the fractures with the pore pressure of the intact rock matrix. Due to the observed pronounced structural anisotropy in the Gotthard case study (i.e. fan structure and varying orientation and
spacing of brittle faults and fractures), it is recommended to apply a modelling approach based on isotropic poroelasticity formulations of an equivalent porous medium (e.g. Barla et al. 2000) only to sites that involve minor structural complexity.
7. References


Jacob, C.E. and Lohman, S.W., 1952. Nonsteady flow to a well of constant drawdown in an extensive aquifer. Trans. AGU, 33: 559-569.


CURRICULUM VITAE

Personal details

Name: Zangerl
First name: Christian Josef
Date of birth: 11.08.1969
Place of birth: Innsbruck
Nationality: Austria

Education

1976-1980: Elementary school, Stams
1980-1982: Secondary school, Stams
1982-1984: Bundesrealgymnasium in Imst
1984-1989: Höhere Technische Lehrenstalt für Maschinenbau, Innsbruck
Final degree: Matura

University studies

10/1990-12/1997: Studies in Petrology and Mineralogy at the Leopold-Franzens University Innsbruck
Diploma thesis: „Kristallingeologische und petrologische Untersuchungen im vorderen Pitz- und Kaunertal“
12/1997: Final Degree: Diploma in Earth science