Tunnels
Underground Excavations

Design Analyses
Case Studies and Observed Performance

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Lecture 4: Tunnelling under Swelling Conditions

Lecture Outline

- Introduction
- Models of behaviour
- Example of geotechnical characterisation
- Case studies

SWELLING

SWELLING in tunnels usually exhibits invert heave and associated abutment movements. This can be sudden, i.e. during construction, or it can be long lasting.
The consequences of such movements range from problems associated with broken drainage channels to destruction of initial and final supports as well as deformation of the railway or highway roadbeds impeding traffic.

SWELLING

The **SWELLING** mechanism is a combination of physico-chemical reaction involving water and stress-relief. Stress changes “usually” have a significant effect. One can distinguish three typical mechanisms:

“**Mechanical**” swelling
“**Osmotic**” swelling
“**Intercrystalline**” swelling.

MECHANICAL SWELLING

“**Mechanical swelling**” occurs in most clays, silty clays, clayey silts and corresponding rocks. It is inverse of consolidation or, otherwise expressed, it is caused by the dissipation of negative excess pore pressures.

OSMOTIC SWELLING

"Osmotic swelling" occurs in clays or clayey (arigllaceous) rocks. It is related to the double layer effect, i.e. the large difference in concentration between ions electrostatically held close to the clay particle surfaces and the ions in the pore water further away.

INTERCRYSTALLINE SWELLING

"Intercrystalline swelling" occurs in smectite and mixed layer clays, in anhydrite and in pirite and marcasite. The specifics of the mechanism differ depending on the material (ground) involved. In smectite and mixed layer clays, intercrystalline swelling is caused by the hydration of the exchangeable cations.

SWELLING AND SQUEEZING

Factor	Type of behaviour	
	Swelling	Squeezing
Cause	Volume increase due to water adsorption in soils with expandable minerals	Yielding of the rock mass with time-dependent deformations
Convergence: - Rate - Period	Initially low, until water does not re-equilibrate the system	Initially high and decreasing with time
	May last or resume with reshaping of the tunnel contour	May last for years
Interested zone	Few metres around the tunnel	Several diameters
Stress state	Relevant	Triggering
Minerals	Determinant	Relatively relevant
Water	Necessary	Relevant
Strains	Volumetric	Shear
Material properties	Relatively relevant	Determinant

Geotechnical Characterisation

From the physical point of view, swelling materials are characterised in the same manner as ordinary geomaterials. Tests are performed in order to determine: mineralogical composition, petrographic characteristics [I.S.R.M., 1978], total and dry density, natural water content, grain specific volume [I.S.R.M., 1979].
Other important parameters are Atterberg limits, grain size distribution, carbonate content and mineralogical contents by X-ray diffraction.

Geotechnical Characterisation

According to the I.S.R.M. recommendations, the procedure for testing argillaceous swelling rocks containing clay and anhydrite is divided into four steps:

• Sampling, storage and preparation of the samples
• Determining the axial swelling stress
• Determining the axial and radial free swelling strain
• Determining the axial swelling stress versus the axial swelling strain (modified Huder-Amberg test)

Modelling and Design Analyses

Predicting the performance of tunnel in swelling (and squeezing) conditions requires the knowledge of:

- Natural stress state
- Stress changes
- Ground water conditions
- Material properties (as in any geotechnical problem).

Experiments in which the time dependent and near failure behaviour can be accurately determined are necessary.

Modelling and Design Analyses

Tunnels in swelling ground can be designed by preventing access of groundwater to the swelling ground, an unralistic option in most instances, and by either allowing the swell deformation to take place or by preventing it; combinations which allow some deformation to take place as well as providing some resistance are possible, as already discussed in lecture 3.

Modelling and Design Analyses

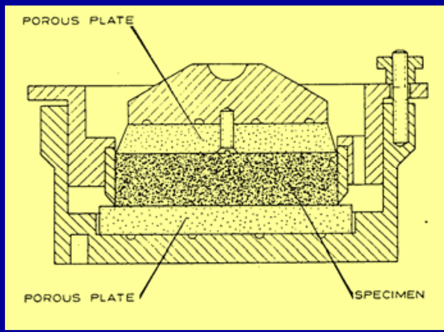
Modelling and Design of Tunnels in swelling ground conditions lead to a number of models which have been proposed by different Authors to describe swelling and squeezing (i.e. creep) behaviour. Some of these models are listed below. Only the simplified models by Grob and Wittke will be discussed in this lecture.

Modelling and Design Analyses

Grob (1972)

The swelling strain law was written as follows, based on oedometer tests:

$$\varepsilon_{z\infty}^{q} = K_q \cdot \left(1 - \frac{\log \sigma_z}{\log \sigma_0}\right)$$



The swelling strain is a function of the vertical stress. K_q is the swelling strain parameter and describes the slope of a straight line (linear regression of the experimental data obtained from the Huder-Amberg tests in an oedometer) on the plane. It is the stress at which the swelling strain is equal to zero.

Modelling and Design Analyses

Wittke (1976-2000)

A three-dimensional generalisation of Grob’s swelling law was done under certain assumptions and simplifications by Wittke and Rißler (1976) by introducing the first invariant of the strain tensor:

$$I_{1,\varepsilon q} = K_q \left[\frac{\log\left(I_{1,\sigma} \dfrac{1-\nu}{1+\nu} \right)}{\log\left(I_{1,\sigma_0} \dfrac{1-\nu}{1+\nu} \right)} \right]$$

where: $I_{1,\sigma}$ = first stress invariant in situ

$I_{1,\sigma o}$ = first stress invariant following excavation

ν = Poisson’s ratio

Modelling and Design Analyses

Additional models have been developed by the following Authors:

Gysel (1987-2001)

Einstein, Aristorenas, Bellwald (1988-2000)

Anagnostou (1991-1995)

Gens and Alonso (1991-2008)

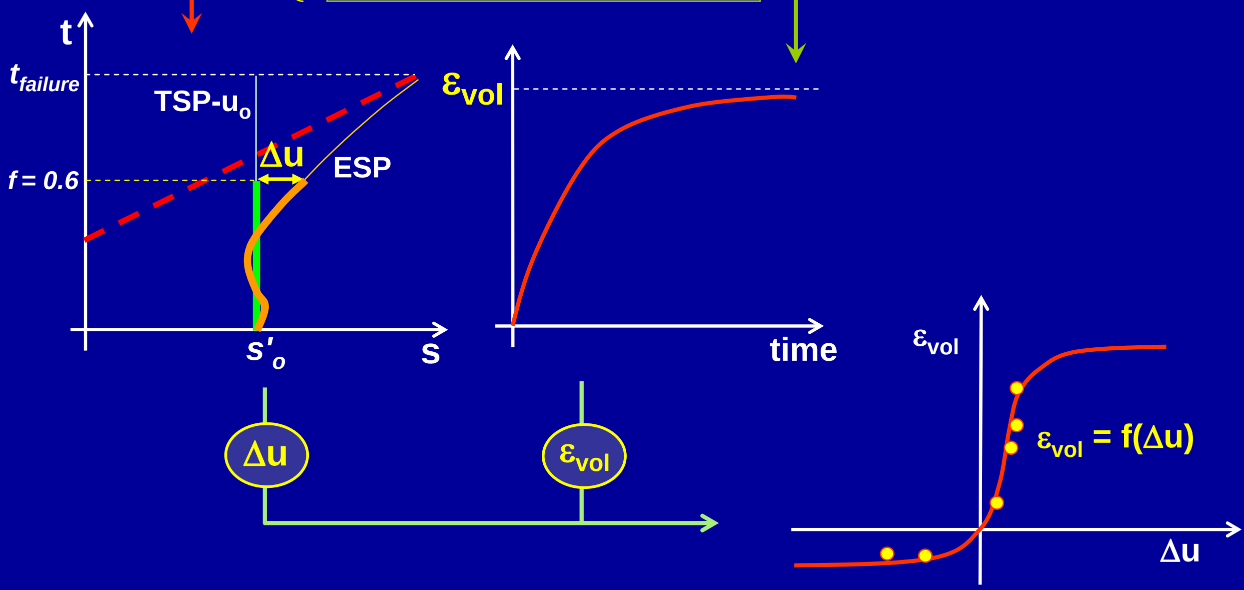
Following the studies by Bellwald (1990) and Aristorenas (1992) a new approach was proposed. Two main differences are put forward by this method (Barla M., 2008):

1. **the experimental evaluation of swelling parameters is due to triaxial test results in order to overcome the limitations of the oedometer test**

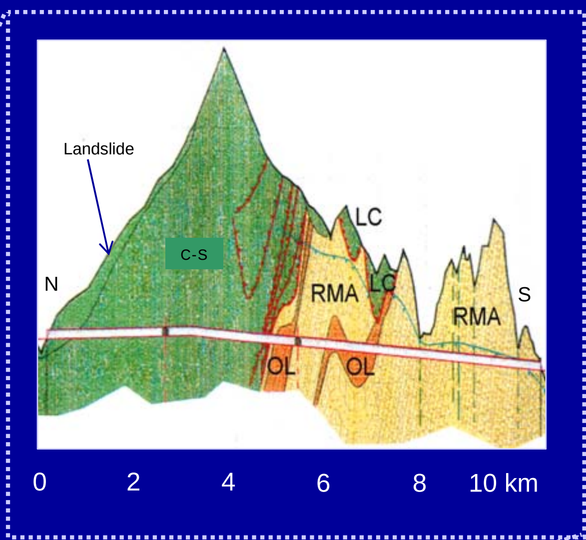
2. **the condition whether swelling occurs or not is related to the excess pore pressure experienced in the ground around the tunnel during excavation**

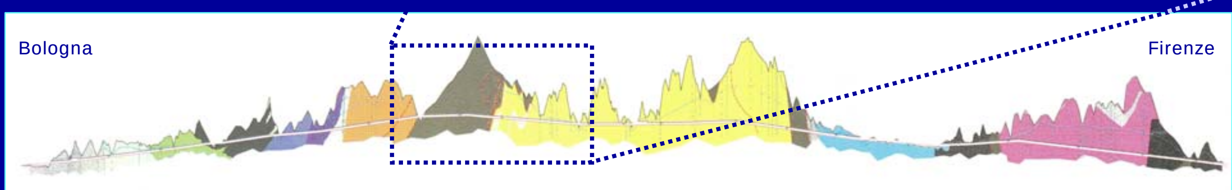
The triaxial swelling test

- Set up of the specimen (*dry setting*)
- Flushing, Saturation & Consolidation
- Undrained shearing
- Drained swelling/consolidation



Raticosa Tunnel

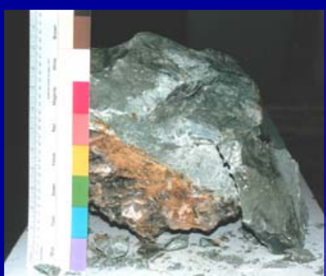




High Speed Railway Line Bologna-Firenze

TESTING PROGRAMME

Cubic samples → Laboratory → Samples



1. Physical properties
2. Mineralogical contents
3. Oedometer tests (conventional and Huder-Amberg tests on natural and reconstituted materials)
4. Triaxial tests in closely controlled stress-path conditions

CHARACTERISATION OF CLAY-SHALES

PHYSICAL PROPERTIES

According to the Plasticity Chart, the C-S can be classified as "inorganic clays of low to average plasticity".
Index properties vary in a wide range, which underlines the great heterogeneity of the material, both at the sample and at the rock mass scale.

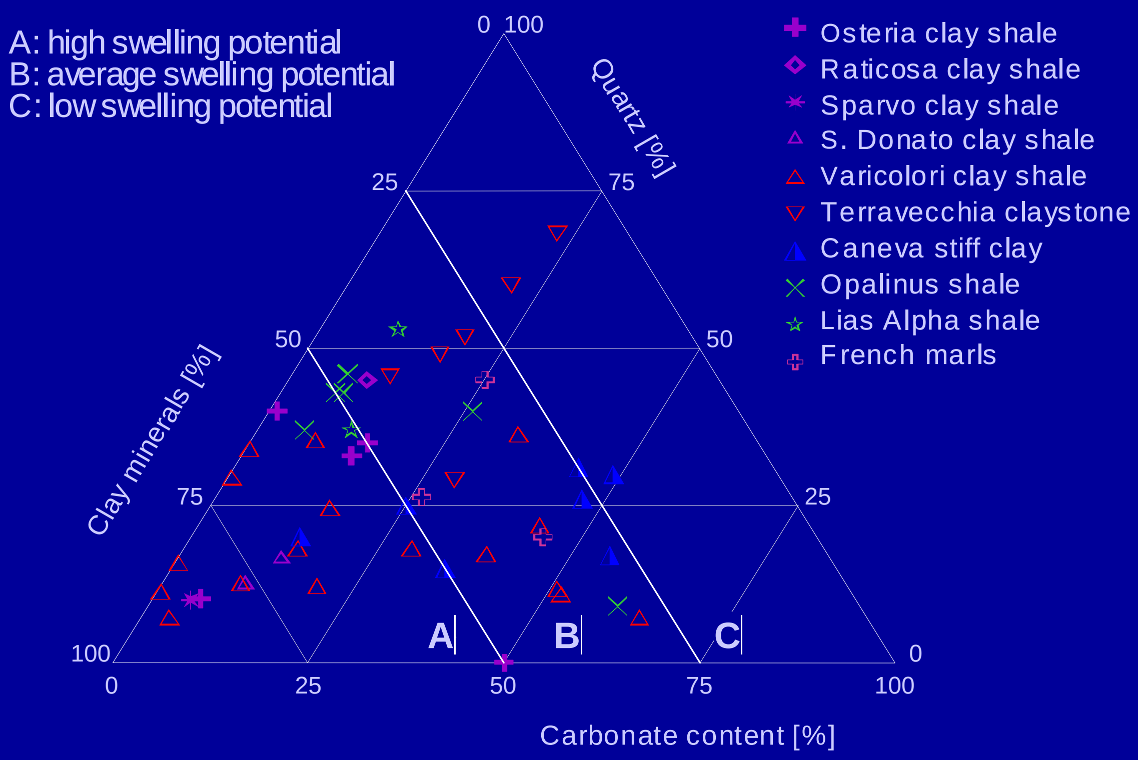
Site	Cubic sample	Depth [m]	w_n [%]	γ_u [kN/m^3]	G_s [-]	e [-]	LL [%]	LP [%]	PI [%]	$CaCO_3$ [%]
Raticosa	4	22	11.5	22.9	2.72	0.301	40	22	18	10

Soil	Calcite [%]	Quartz [%]	Clay minerals [%]	Albite [%]
Raticosa (cubic sample 4)	10	35	45	10

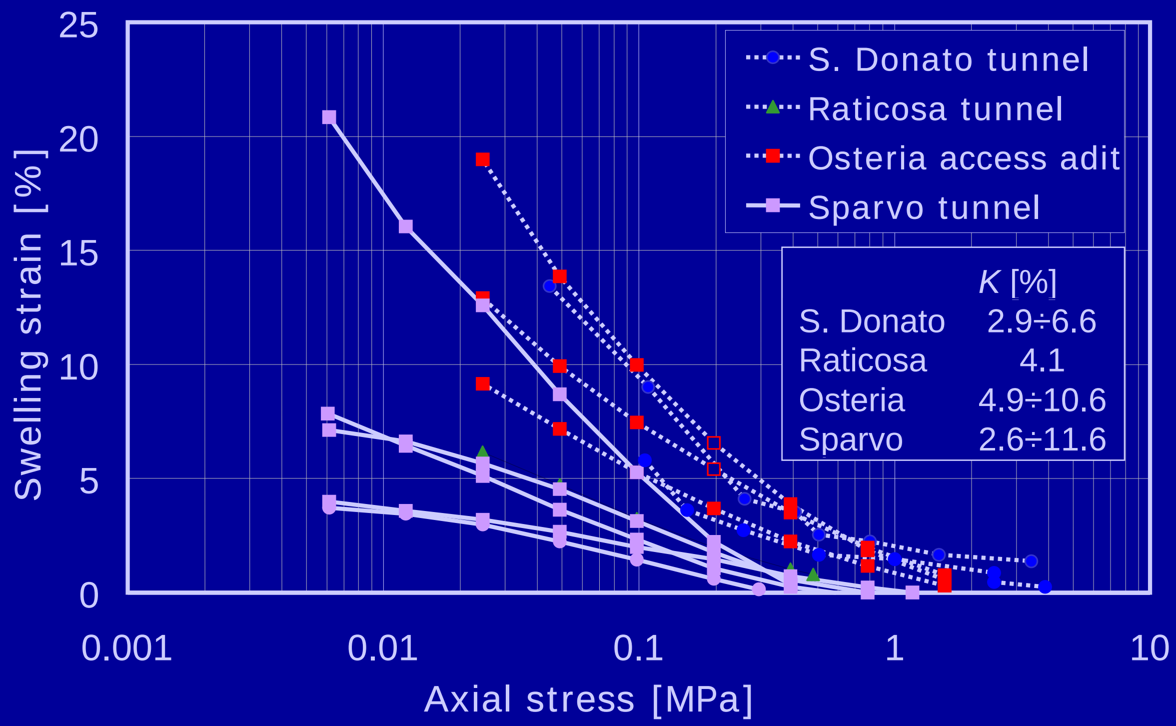
Soil	Smectite [%]	Illite [%]	Illite-Smectite [%]	Chlorite [%]	Kaolinite [%]
Raticosa (cubic sample 4)	5	25÷50	10÷20	40÷50	-

Low water content, Low Atterberg limits, Large amount of clay minerals, Presence of swelling minerals

X-RAY DIFFRACTION ANALYSES



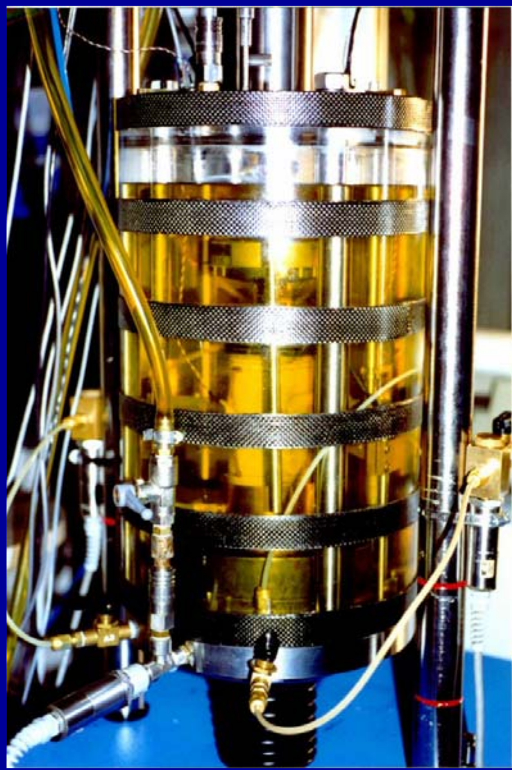
HUDER-AMBERG SWELLING TESTS



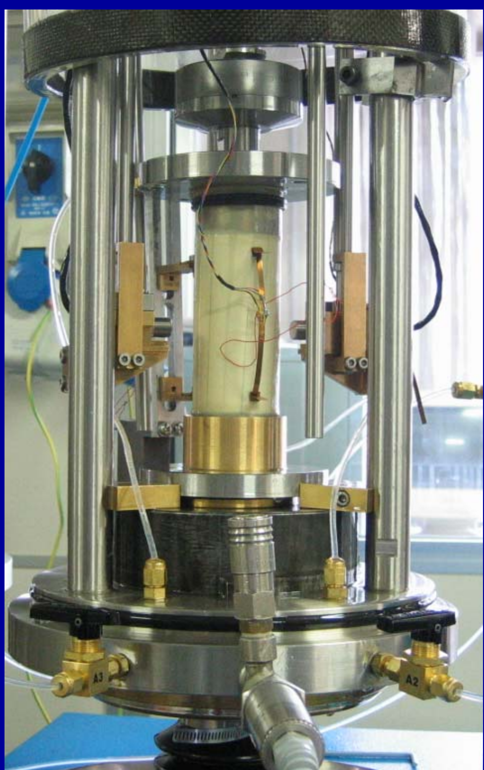
The SRTA (Soft Rock Triaxial Apparatus)

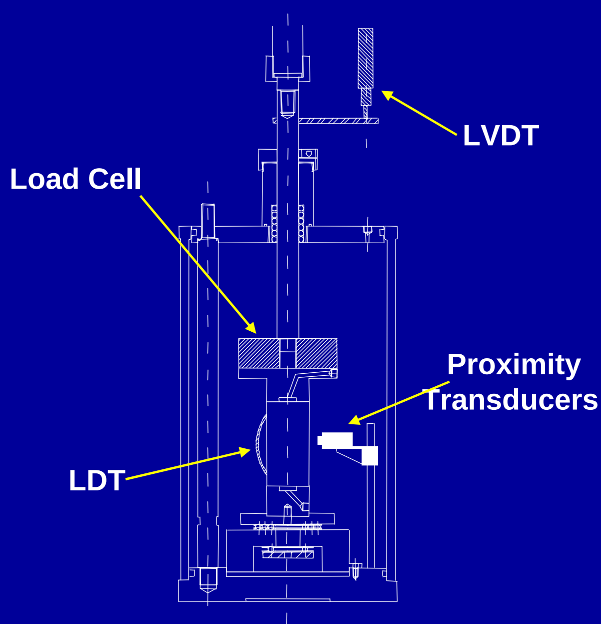
- Axial strains: external (LVDTs) and local (LDTs) measurement
- Radial strains: local measurement (inductive proximity transducers)
- Cell pressure: 2 MPa
- Pore pressure: 1 MPa
- Load cell:50 kN, inside the confinement cell
- Volume gauge
- Multi-point conditioning system is used for data acquisition
- Complete remote control system

SRTA
Soft Rock Triaxial Apparatus

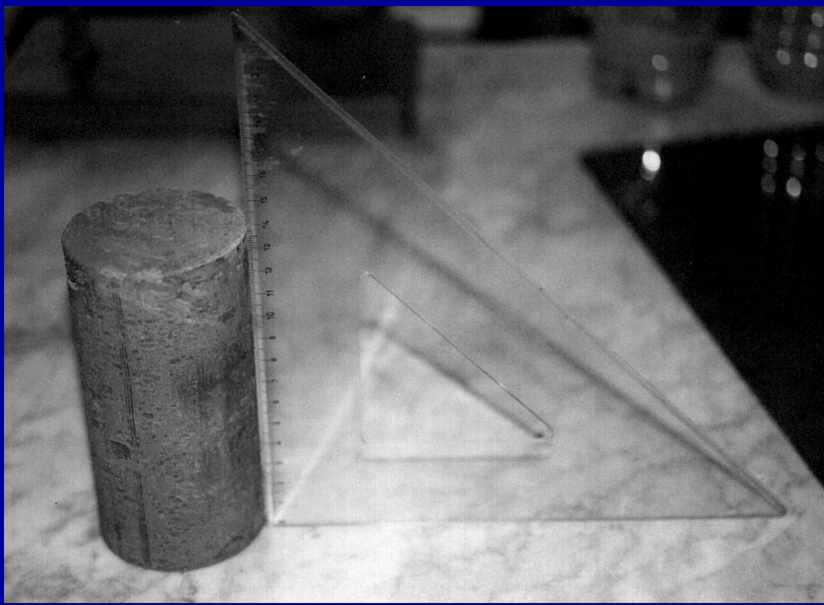


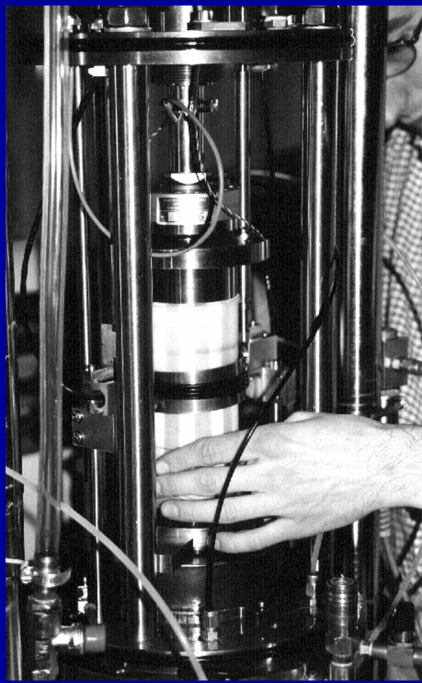
The SRTA (Soft Rock Triaxial Apparatus)





The SRTA (Soft Rock Triaxial Apparatus)





The SRTA (Soft Rock Triaxial Apparatus)

TRIAXIAL TESTS

OST3 RTC1 RTC2 RTC3 RTC4 RTC5

PHASES:

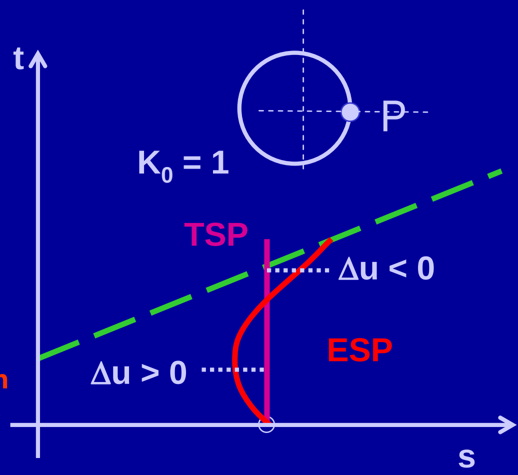
1. Flushing
2. Saturation
3. Consolidation

⇨ Initial conditions

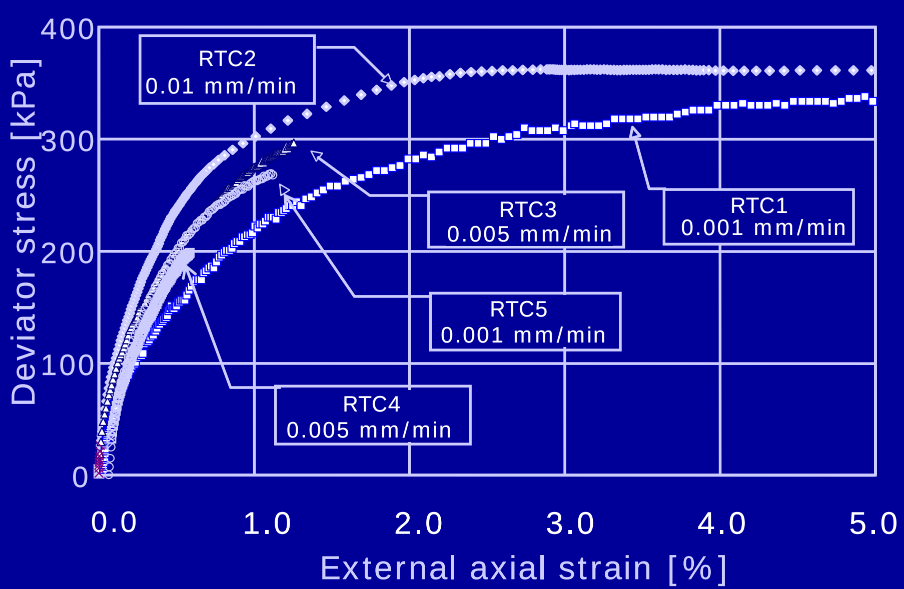
4. Stress-path ⇨ Excavation

5. Creep
6. Consolidation / Swelling

⇨ Time dependence in undrained / drained conditions

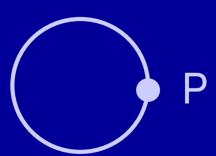


DEVIATOR – AXIAL STRAIN



PHASES:

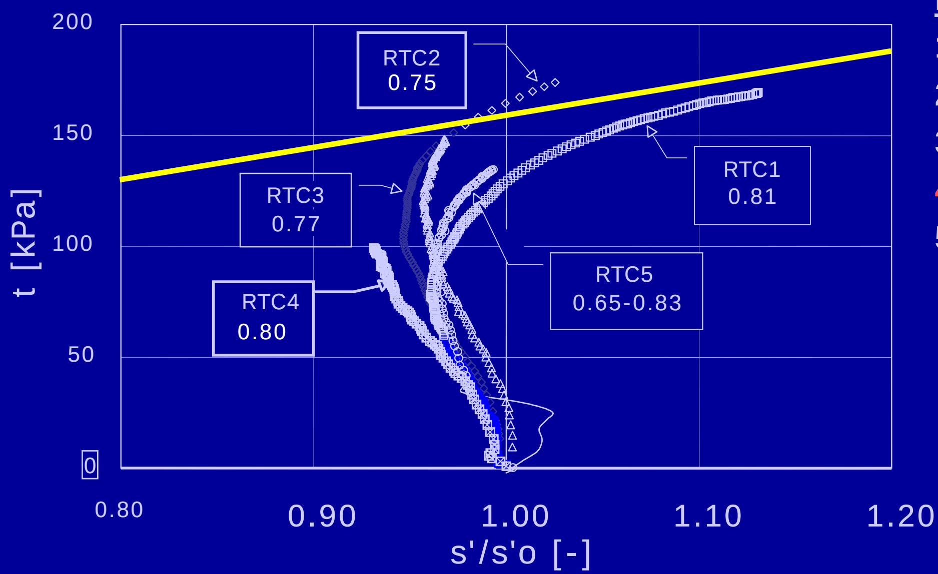
1. Flushing
2. Saturation
3. Consolidation
4. Stress-path
5. Swelling/ Consolidation



Vertical strain rate: 0.01÷0.001 %/min
Saturation degree: 0.75÷0.88 %

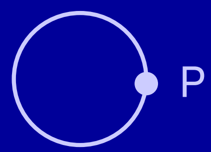
The CCTCS exhibit a ductile stress-strain behaviour, with axial failure strain reaching about 5%

TRIAXIAL TESTS: STRESS-PATH



PHASES:

1. **Flushing**
2. **Saturation**
3. **Consolidation**
4. **Stress-path**
5. **Swelling/ Consolidation**

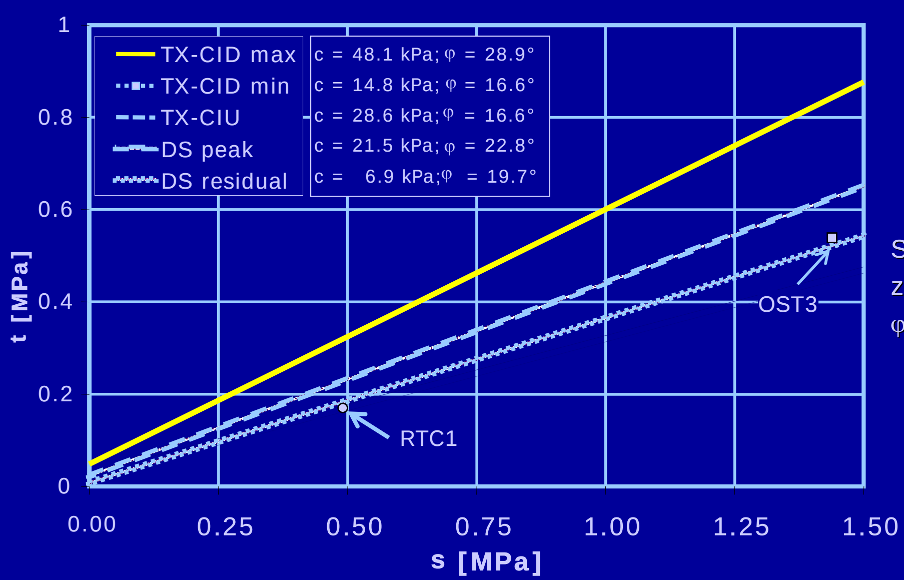


$$s = \frac{\sigma_v + \sigma_h}{2}$$

$$t = \frac{\sigma_v - \sigma_h}{2}$$

The ESP initially bends to the left (excess pore pressure > 0). Then a negative excess pore pressure is produced when approaching failure (development of mechanical swelling).

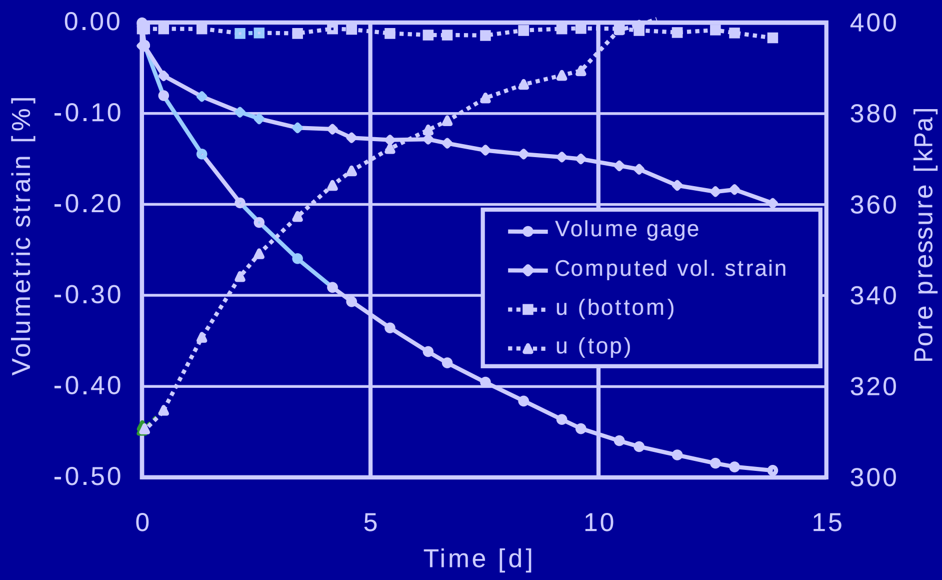
SHEAR STRENGTH PROPERTIES

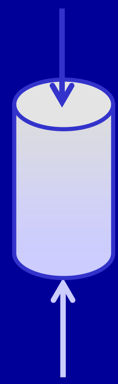


Samples from the landslide zone: c' = 20.3 kPa φ' = 16.6°.

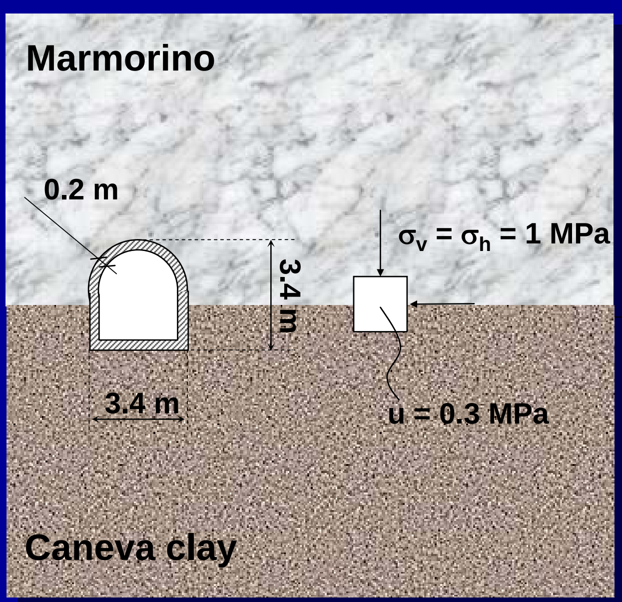
The peak and residual strength envelopes from direct shear tests lie well within the range of shear strength values resulting from triaxial tests

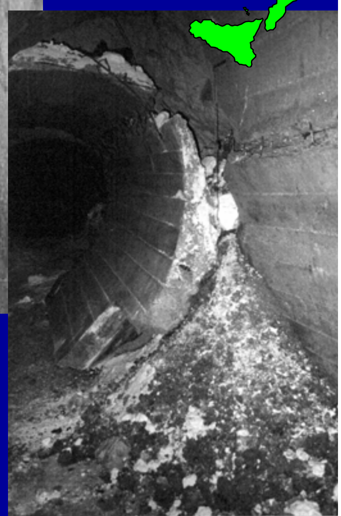
TIME DEPENDENT BEHAVIOUR
DRAINED CONDITIONS



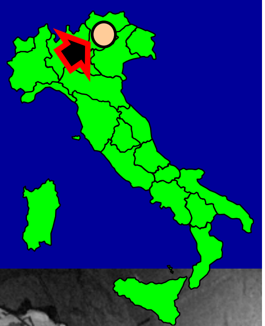


Caneva Stevenà quarry

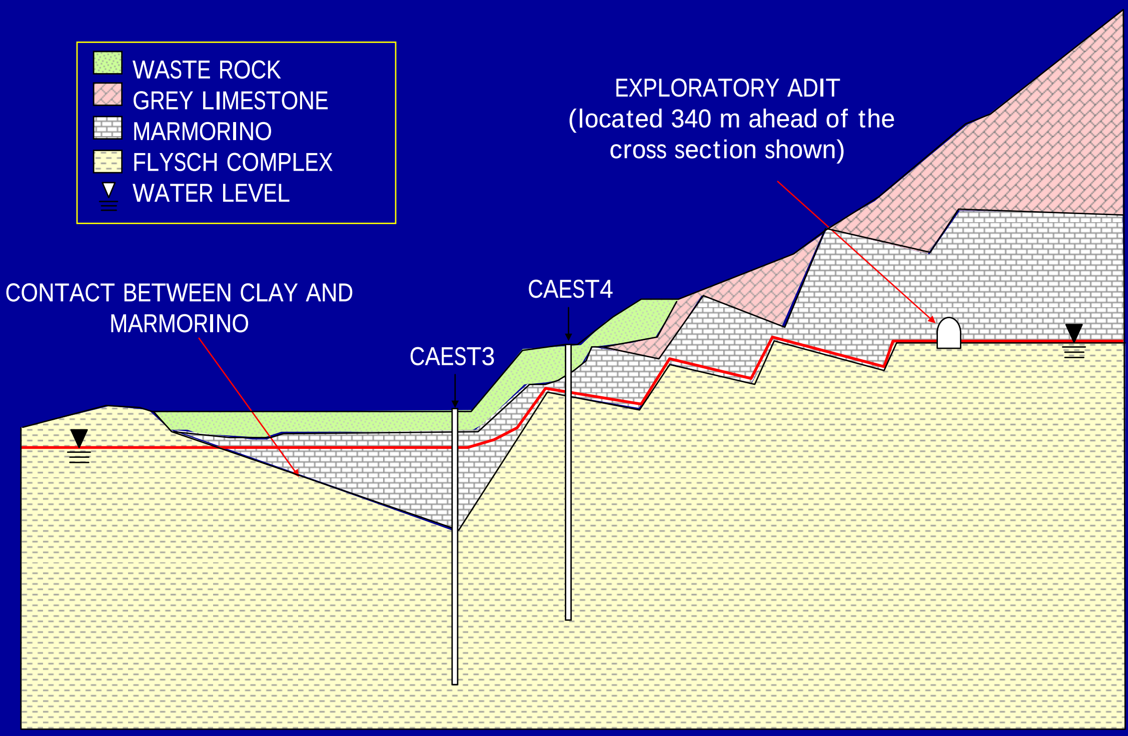




Typical conditions of the exploratory adit after swelling

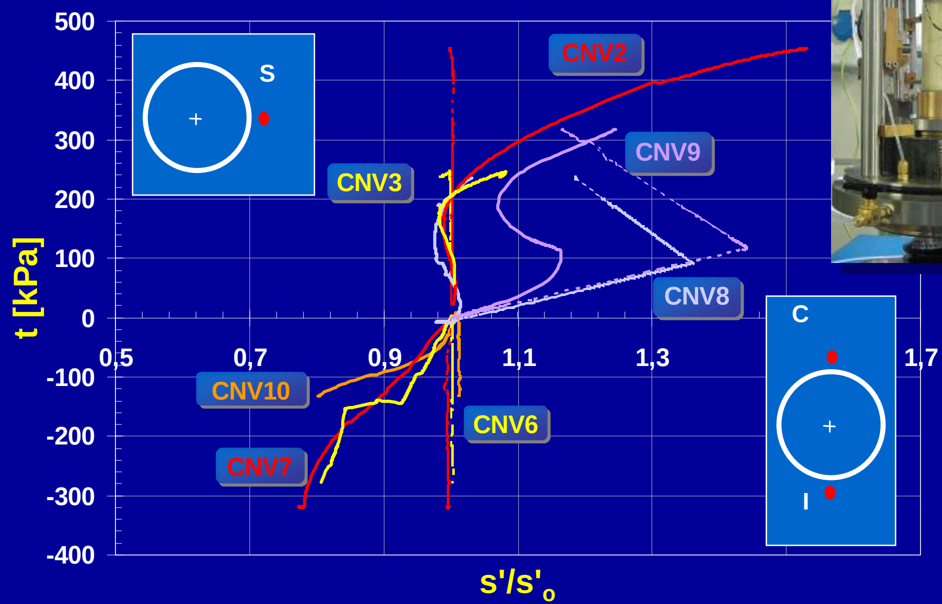


Geological section



Laboratory tests

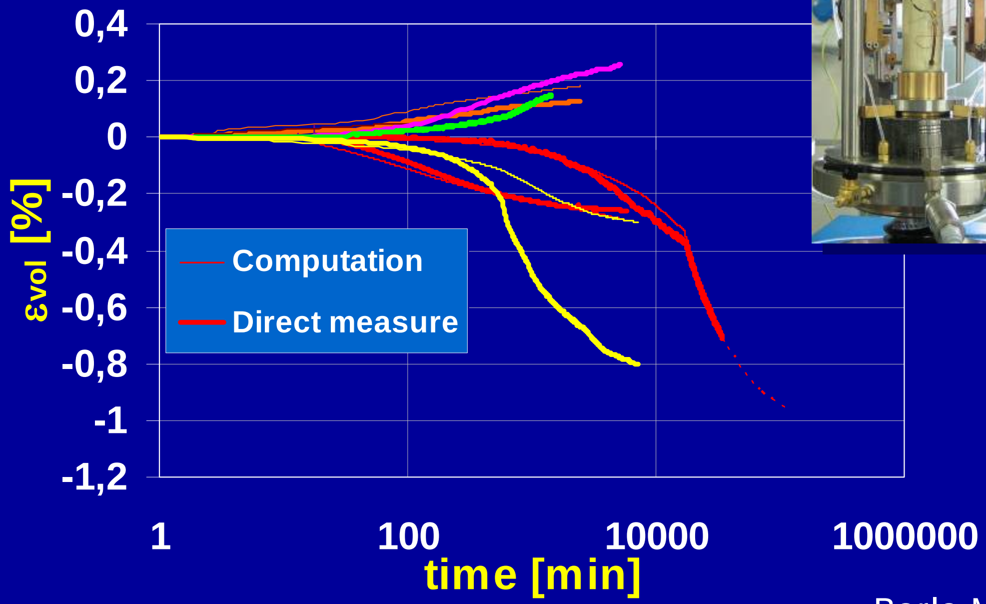
Swelling triaxial tests



Barla M. 2008

Laboratory tests

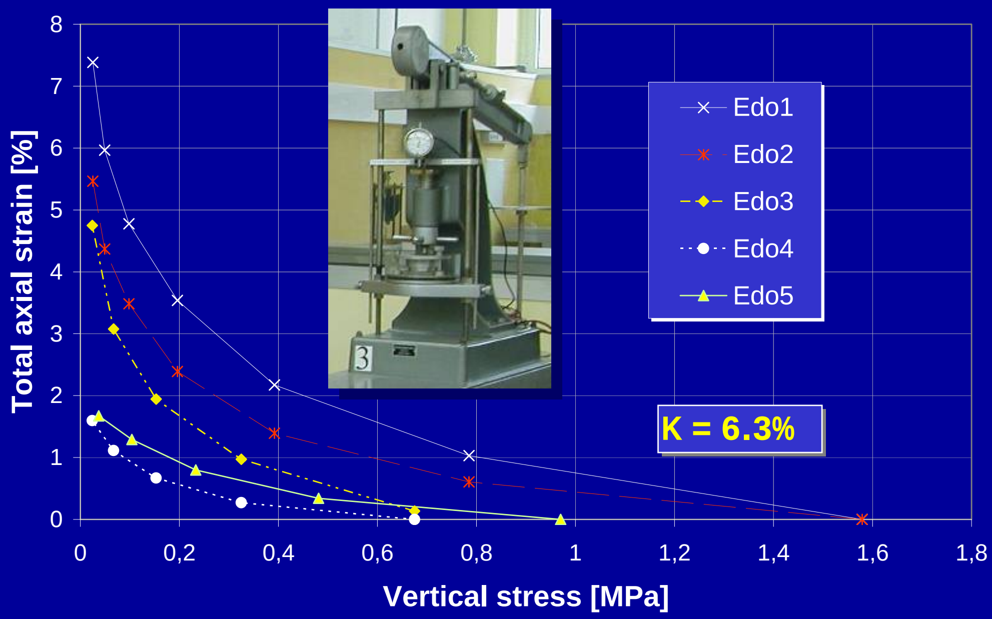
Swelling triaxial tests



Barla M. 2008

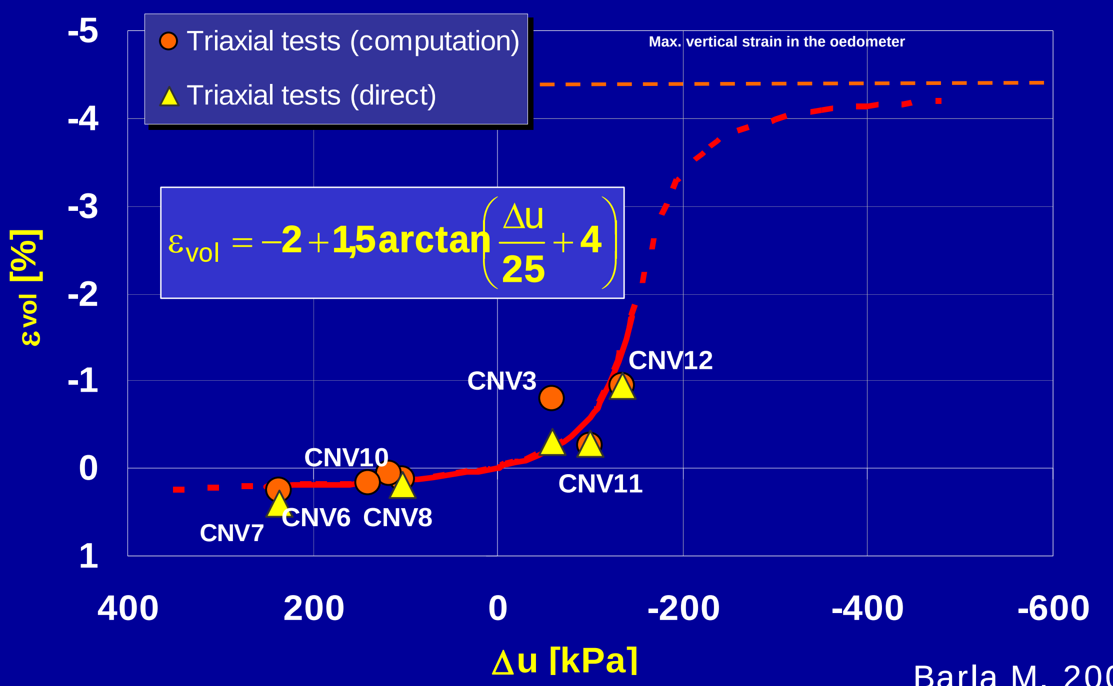
Laboratory tests

Huder-Amberg Oedometric tests



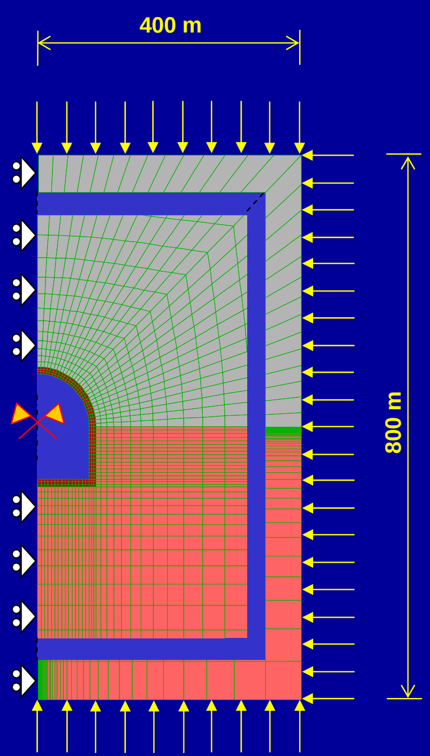
Barla M. 2008

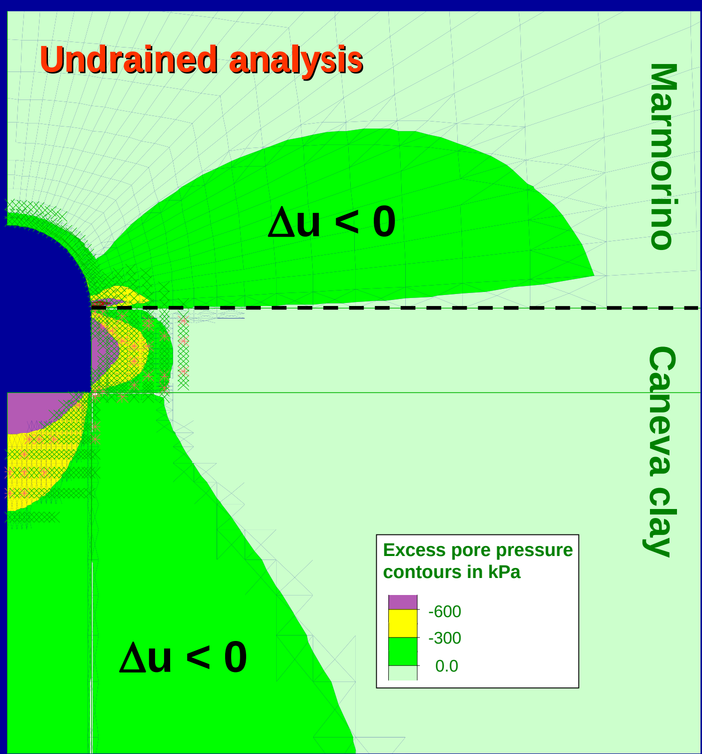
Laboratory tests



Barla M. 2008

Numerical Modelling

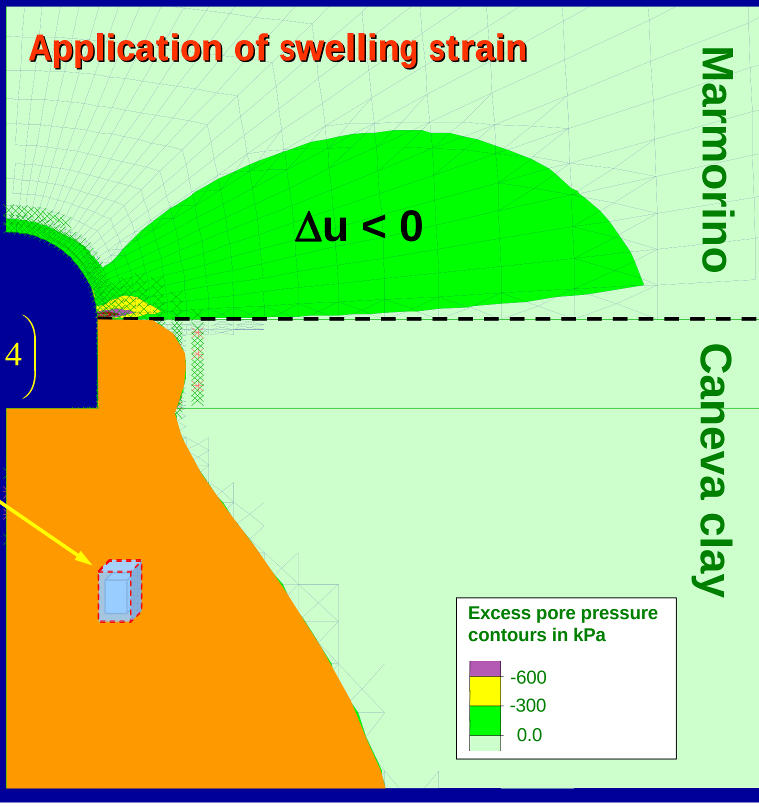




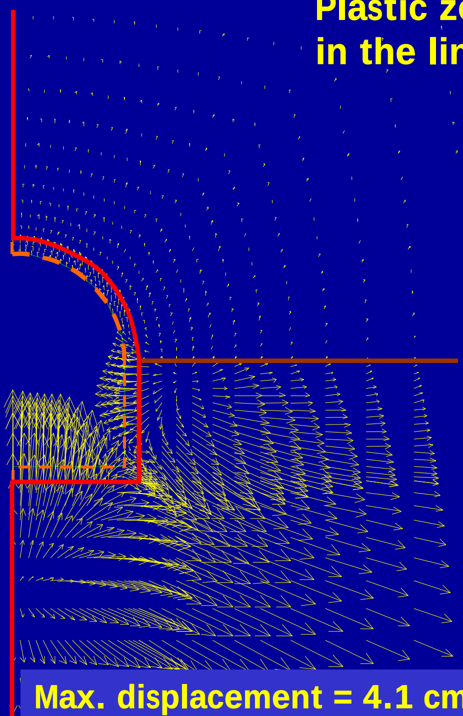
Numerical Modelling

Swelling strain of the clay element in the case of the Barla method

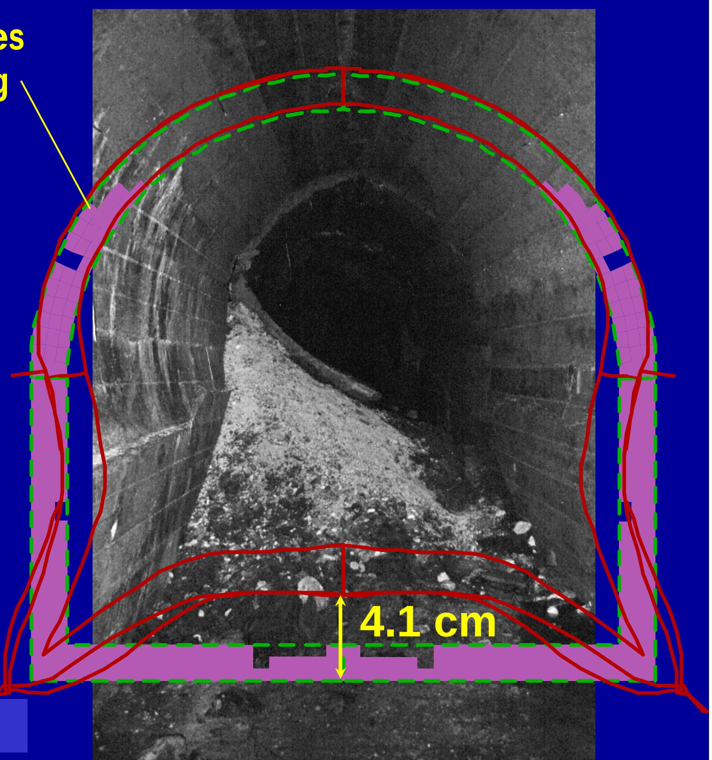
$$\varepsilon_{vol} = -2 + 1{,}5\,\arctan\left(\frac{\Delta u}{25} + 4\right)$$



Numerical Modelling



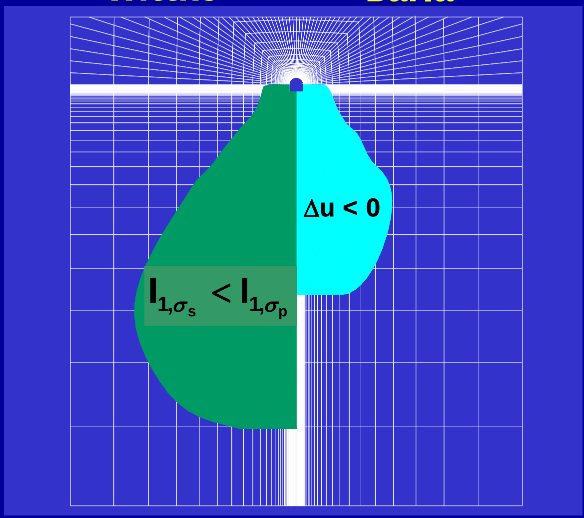
Plastic zones in the lining



Max. displacement = 4.1 cm

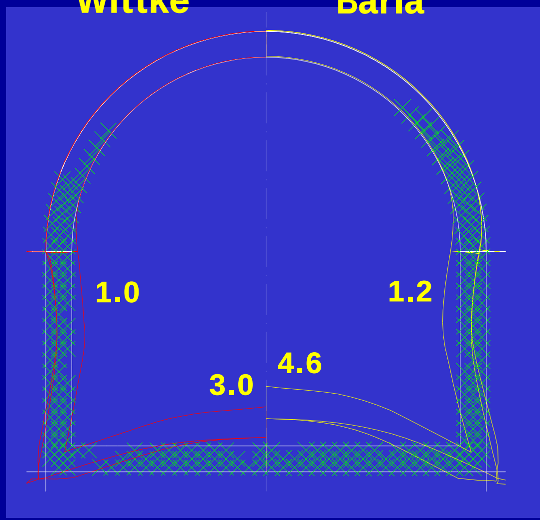
Numerical Modelling

Wittke Barla



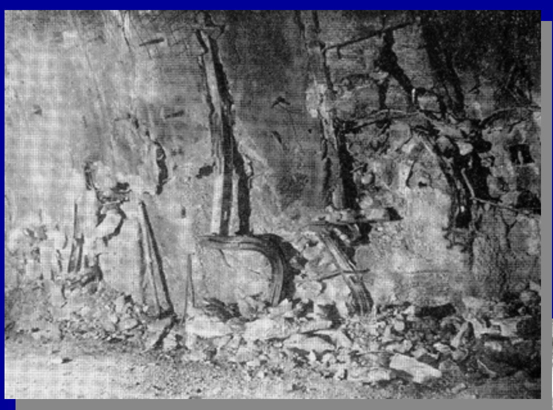
Swelling zones

Wittke Barla



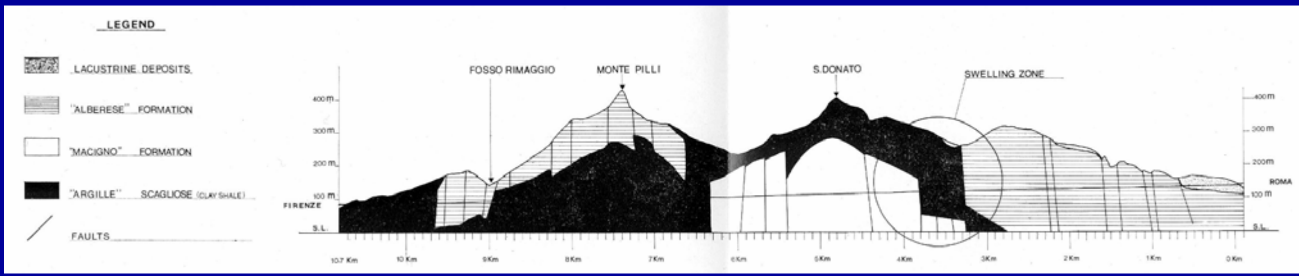
Deformed lining [cm]

S. Donato Tunnel (1990)

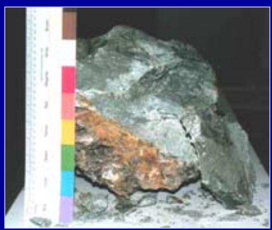


New railway line
Florence-Rome

Geological section



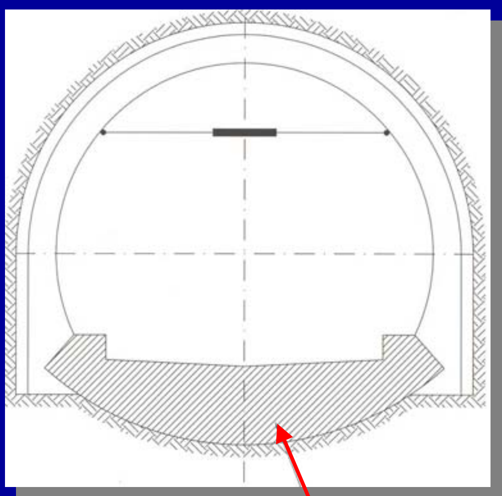
Cubic samples



Specimens

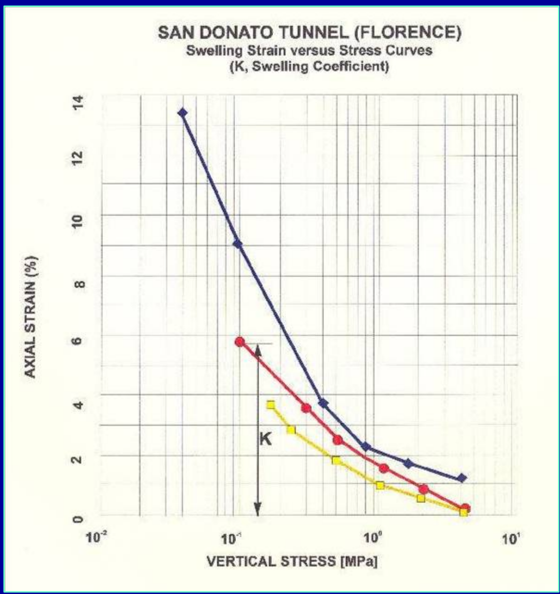
S. Donato tunnel (1990)

Construction steps



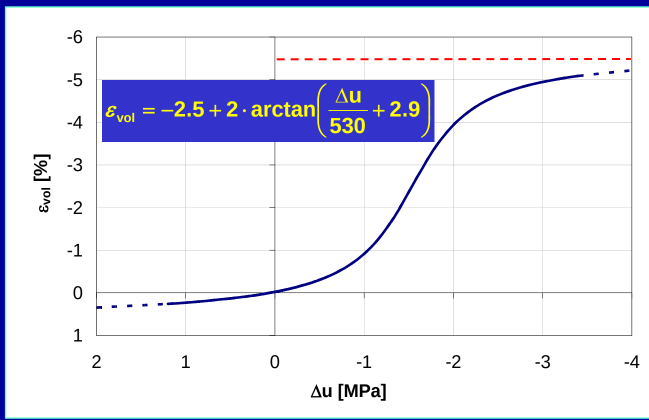
A concrete arch was added after swelling took place at the invert years after opening

Laboratory investigation



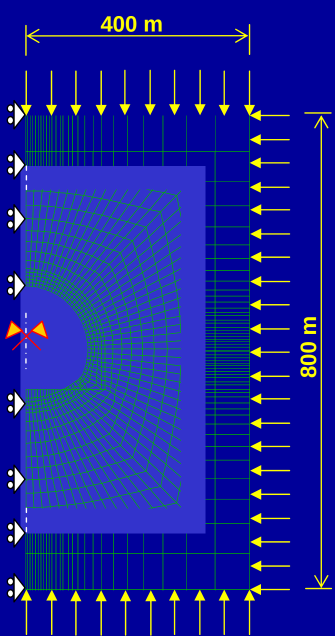
Huder & Amberg
oedometer test results

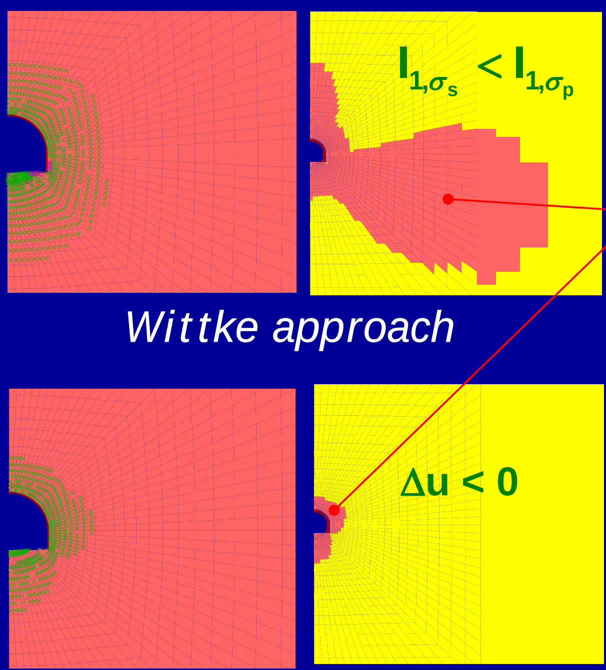
K = 5.8%



Inferred ε_{vol}-Δu curve

Numerical modelling



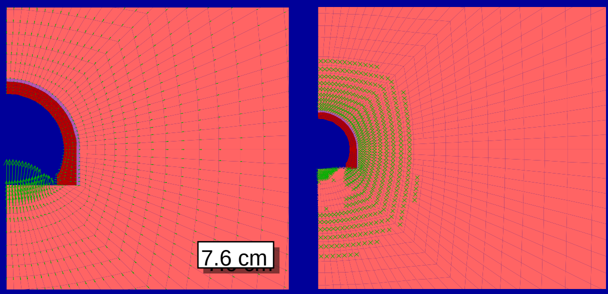


Wittke approach

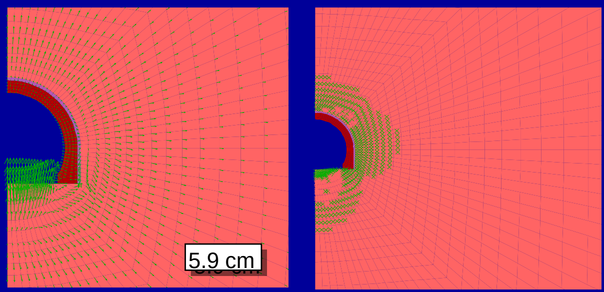
Barla approach

Area where swelling strains are applied

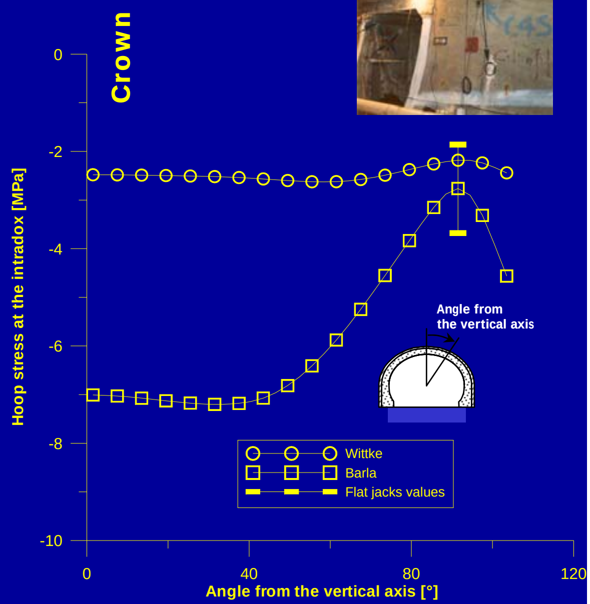
Numerical modelling



Wittke approach



Barla approach



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