Workshop: Creep Behaviour of Soils – Focus on Practical Applications

Norwegian Geotechnical Institute
Oslo, Norway
8th January 2015

Preliminary program

Session 1: Theories and consequences in applications

9.00 – 9.30	Welcome and Introduction Hans Petter Jostad & Gustav Grimstad
9.30 – 10.10	Evaluation of creep hypotheses A and B Samson Degago
10.10 – 10.35	One-dimensional creep behaviour Hans-Petter Jostad
10.35 – 10.50	**Coffee Break**
10.50 – 11.30	Parameter selection for creep models Gustav Grimstad
11.30 – 12.15	**Lunch**

Session 2: Practical applications

12.15 – 12.45	From 1D Creep models to the current 3D creep models Minna Karstunen
12.45 – 13.00	Identifying parameter of creep by GA optimization Zhen-Yu Yin

13.00 – 13.15	Optimization procedure for determining internal model parameters Jon Ronningen
13.15 – 13.30	A new GUI software for assessing (creep) model parameters Jean-Philippe Gras
13.30 – 14.00	Onsøy test fill Toralv Berre
14.00 – 14.15	**Tea Break**
14.15 – 14.45	Back calculation of Onsøy test fill Magne Mehli
14.45 – 15.15	Bjørvika – case study Kjell Karlsrud
15.15 – 15.45	Creep behaviour of peat Cor Zwanenburg
15.45 – 16.15	Creep behaviour of frozen soil Fan Yu and Seyed Ali Ghoreishian
16.15 – 17.00	**Panel Discussion**

CREEP Workshop: Focus on Practical Application

Hans Petter Jostad

Discipline leader in numerical modeling at NGI

Adjunct Professor at NTNU

NGI, Oslo, Norway, 8. January 2015

NGI

Key questions:

What is the definition of creep?

Do we have creep deformations at the same time as we have deformations due to effective stress changes?

When does creep start?

When does creep stop?

What controls the (volumetric) creep deformation?

How to extrapolate from laboratory tests to long term field condition?

How to expand from 1D to a general 3D stress state?

How to determine soil properties?

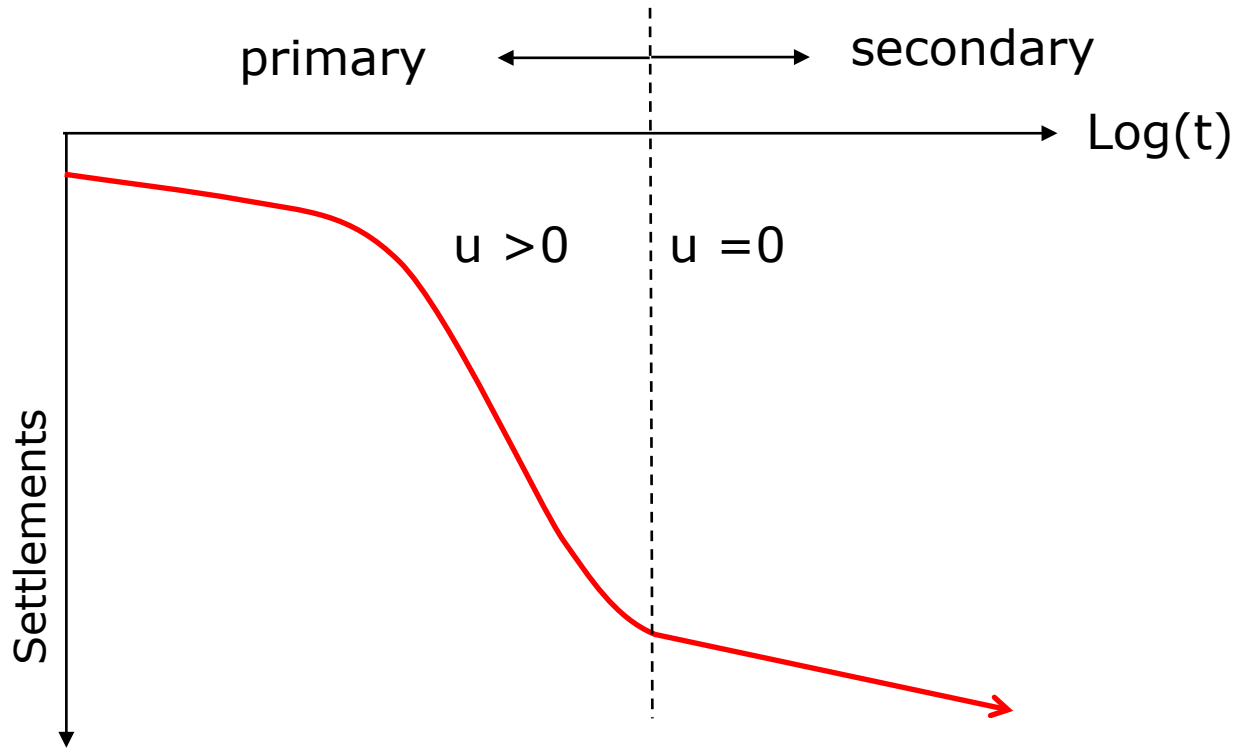
Motivation

How to calculate long term settlements in soft clay?

A) Primary and secondary compression phases?
   - standard practice in Norway

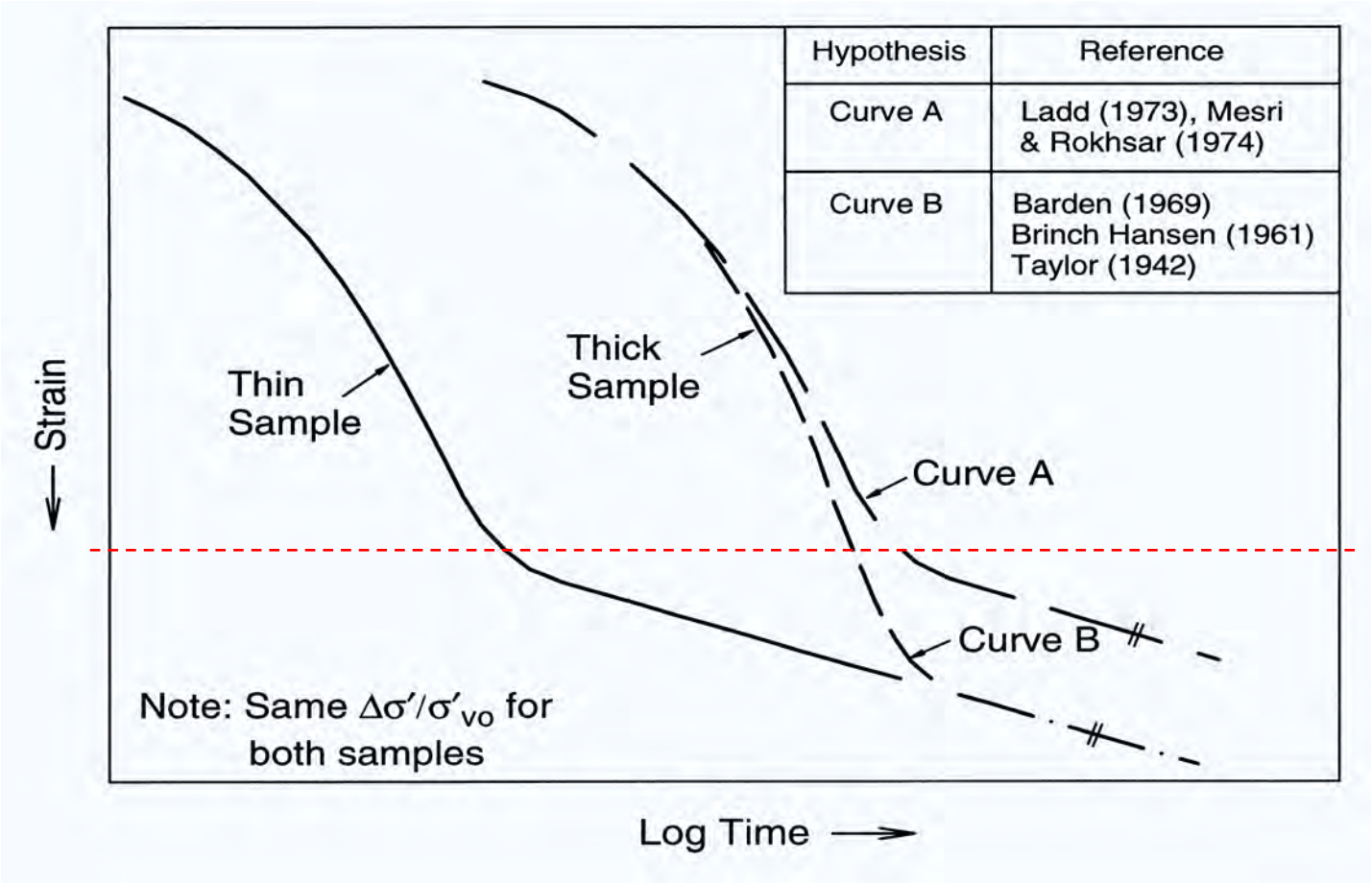
B) Coupled consolidation and creep?
   - creep models

Secondary consolidation (creep)



The classical approach in Norway for creep settlements!

Unique end-of-primary (EOP) void ratio



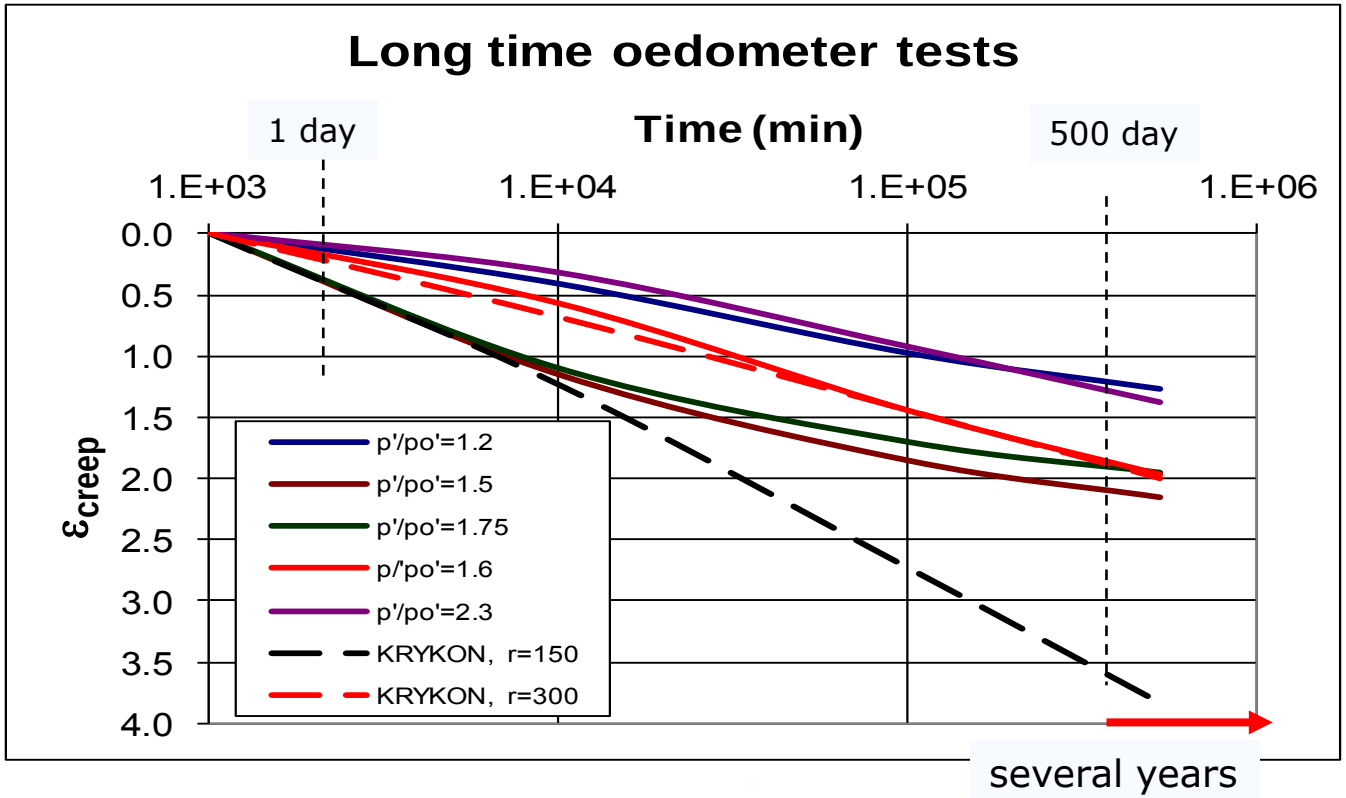
t_p

Need this assumption in order to divide into primary- and secondary phases

A or B most correct?

Main challenge

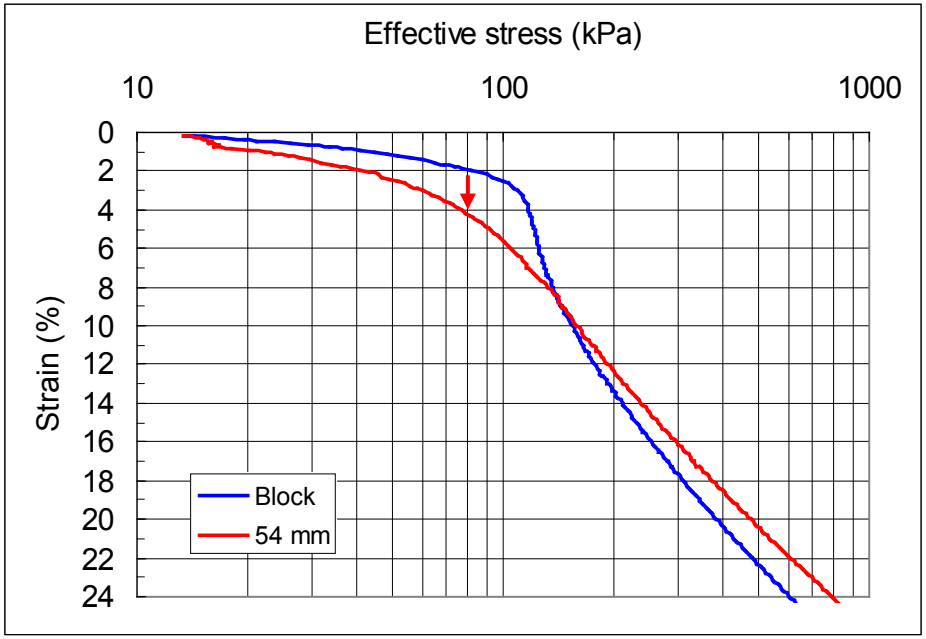
Due to significantly different time scale in field and laboratory conditions, the deformation in the field must be described (extrapolated) by a creep model based on input from laboratory tests.

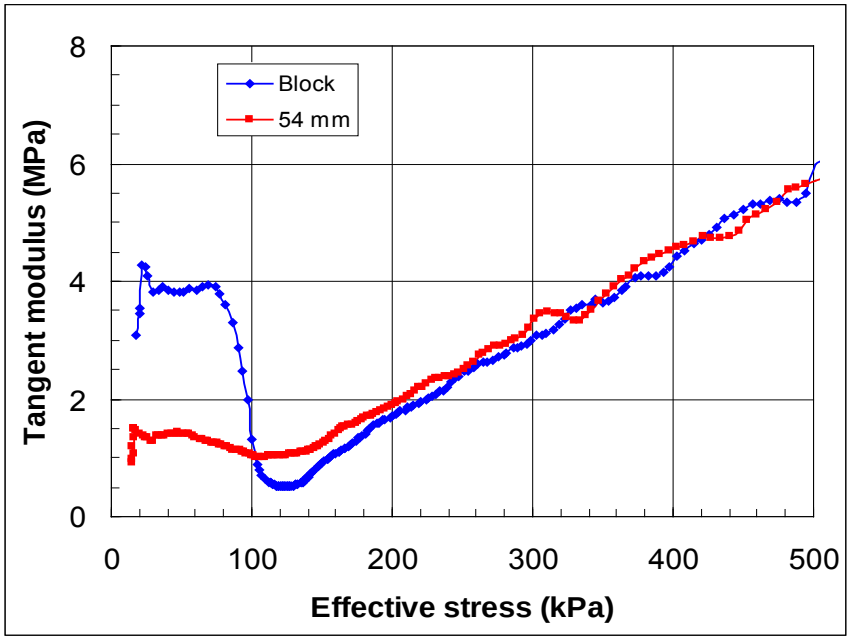


Verification/calibration of creep models

- Back-analyses of measured field data
  - Generally large number of uncertainties
- Back-analyses of idealised model tests (e.g. oedometer tests with different specimen heights)
  - Extrapolation is still necessary
- Long term laboratory tests
  - Extrapolation is still necessary

Sample disturbance (apparent creep)





Is soil disturbance the reason for good agreements between standard consolidation analyses (without creep) and field observation?

$$M_{oc} = a \cdot M_{pc}$$

$$a = 5 \rightarrow 15$$

Creep of Geomaterials

FP7 IAPP – CREEP of Geomatrials

- Support for training and career development of researchers (Marie Curie)

Deltares
Enabling Delta Life

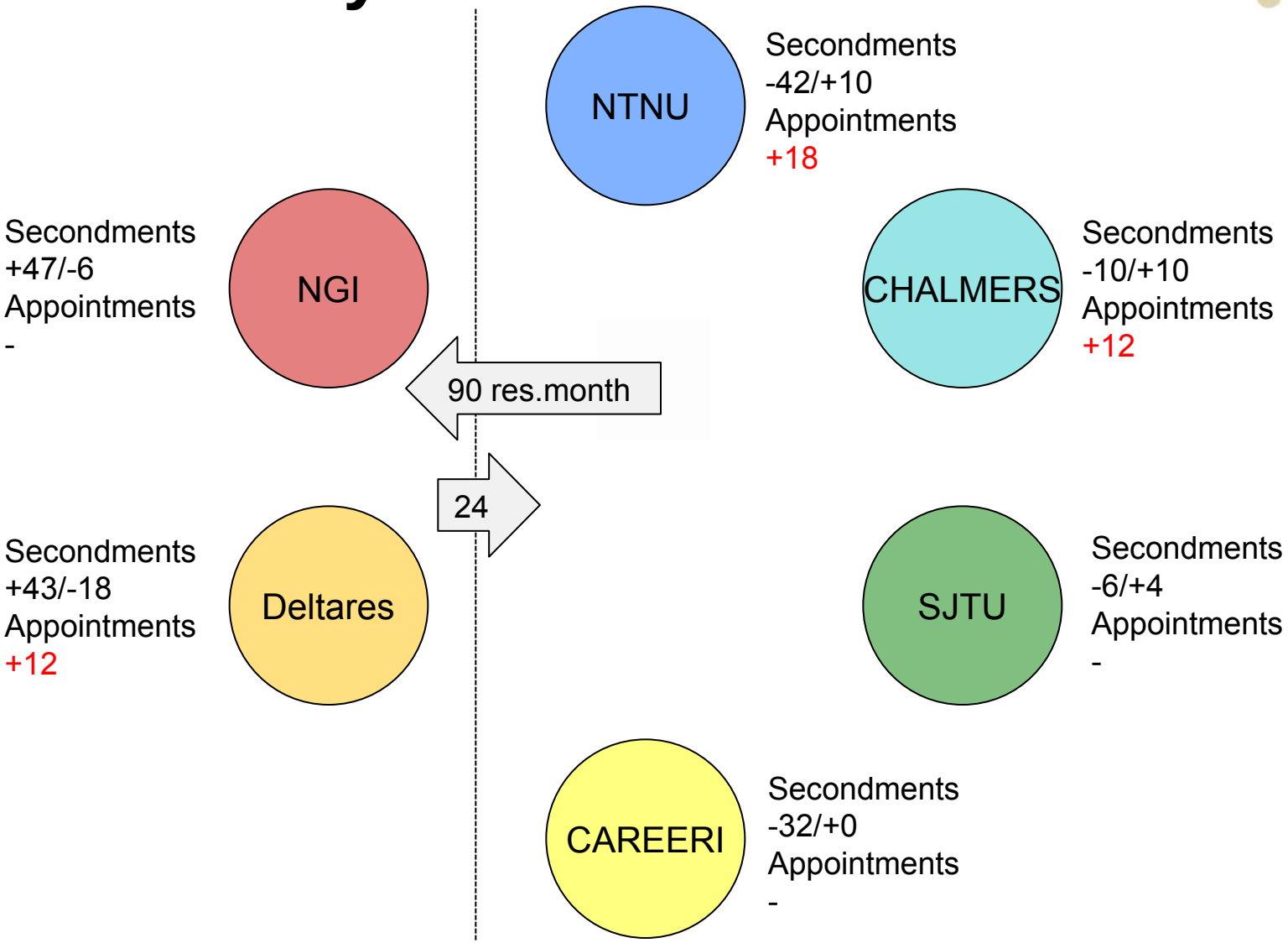
NTNU
Fakultet for ingeniørvitenskap
og teknologi
Institutt for bygg, anlegg og transport

Scientific objectives

Overall Science and Technology (S&T) Objectives

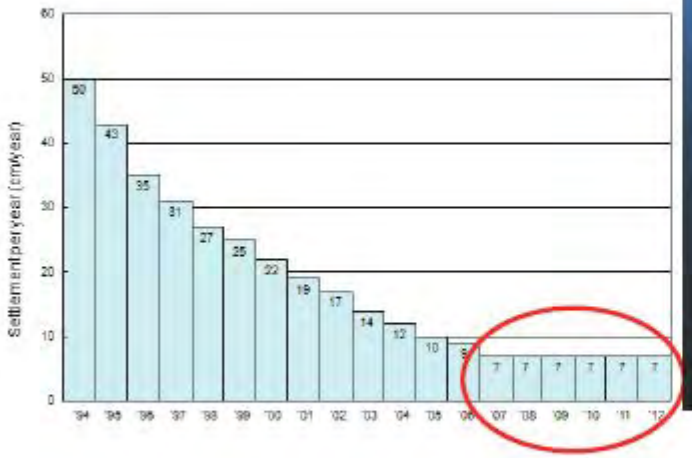
The project's overall S&T objectives is to formulate, implement, and validate a set of novel time dependent material (creep) models for clay, peat, *sand*, and frozen soil, which allow for enhanced creep predictions. As model formulation prerequisites experimental quantification of creep behavior in the respective materials, the latter is an overall S&T objective as well.

IAPP - Industry-Academia Partnerships and Pathways



Background - motivation

e.g. Kansai Airport



On-going settlement 7 cm/yr

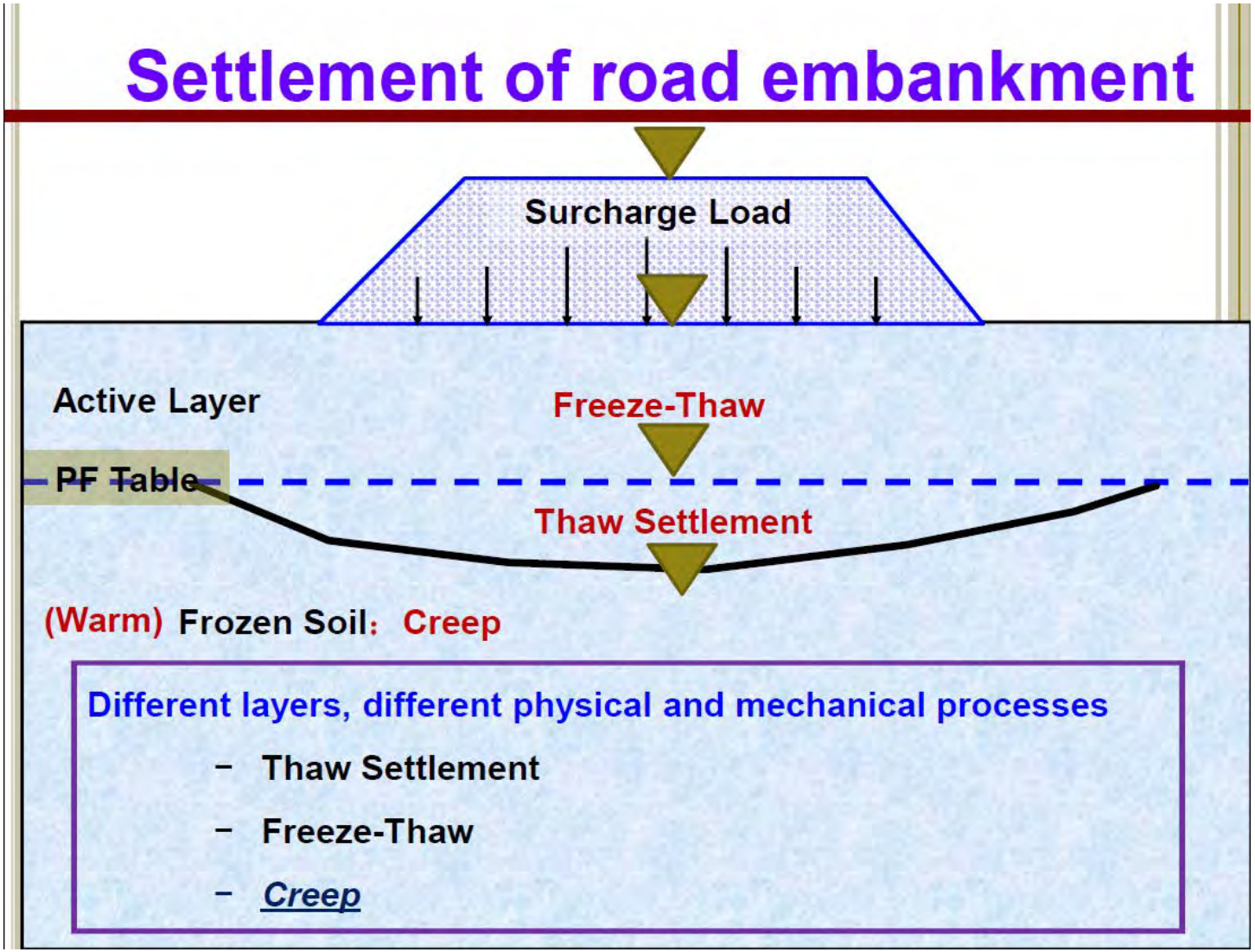
http://www.nkiac.co.jp/en/tech/sink/index.html

and daily life

Deltares

Field tests

Warm frozen soil

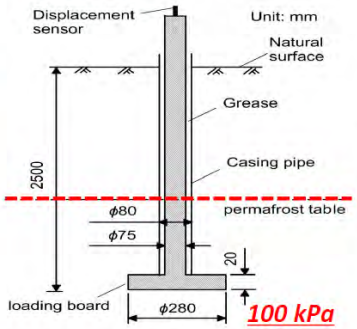


Creep in frozen soil - CAREERI

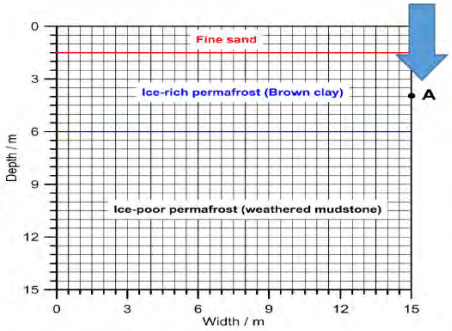
➢ ***Long-term load test***

Field load test

Loading pile

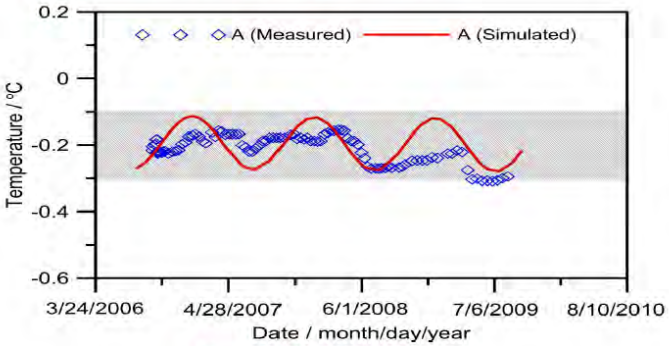


Numerical model

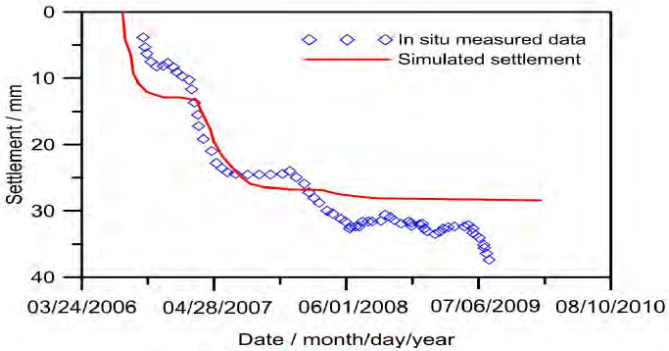


Only *creep of underlying permafrost* was considered

After implementation of this model,



Thermal state analysis



Deformation analysis

List of work packages

Work package No	Work package title	Lead Beneficiary short name	Start month	End month
1	**Soil characterisation**	CAREERI & NGI	1	36
2	**Comparison and unification of soft soil creep concepts**	NGI & CHALMERS	1	36
3	**Adoption of existing creep formulations to new materials**	NTNU & DELTARES	13	48
4	**Model application and recommendations**	DELTARES & CHALMERS	25	48
5	**Management, knowledge-transfer, dissemination and publicity**	NTNU	1	48
6	**Outreach activities**	NTNU	13	48

WP 1

WP1 is concerned with characterization and quantification of creep properties and mechanisms in clay, peat, sand, and permafrost. The characterization is based on available in situ and laboratory tests data. For what not already covered by literature data, it will be necessary to perform specific laboratory tests, e.g. to quantify the creep behaviour of peat and permafrost or to determine the influence of soft clay sampling methods.

S&T objectives WP1:

1-1 To characterise creep in clay, peat, sand, and permafrost through laboratory testing – Milestones 1 to 4.

1-2 To compile a database containing time dependent behaviour of clay, peat, sand, and permafrost from literature and own testing – Milestone 5; Deliverable 1.

Database available on internet

WP 2

WP2 innovates modelling of creep in clayey soils. Classical creep concepts are compared by defining simple benchmark examples with the purpose of assessing the capabilities of the most common creep formulations through finite element analysis. Assessment of the outcome will constitute the basis for developing a clay model that unifies other concepts in their response.

S&T objectives WP2:

2-1 To identify the most relevant model mechanisms in soft soil creep modelling through FE benchmark exercises involving commonly available soft soil modelling frameworks – Milestone M5, Deliverable D2.

2-2 To formulate, implement, and validate a user-friendly time dependent soft clay model relevant to engineering practise…

2-1 Benchmarks are identified – For clay: Onsøy was selected – Report and data are available to public.

2-2 Several models for clay have been implemented

WP 3

WP3 generates industry relevant creep formulations for peat, frozen soils, and sand. Existing creep concepts shall be enhanced/ adopted to the new geomaterials considered in this WP.

S&T objectives WP3:

3-1 To formulate, implement, and validate a novel creep model for peat – Milestone M7, Deliverable D4.

3-2 To formulate, implement, and validate a novel creep model for sand – Milestone M8, Deliverable D5.

3-3 To formulate, implement, and validate a novel creep model for warm permafrost – Milestone M9, Deliverable D6.

3-1 Deltares has had an recruitment for 12 months and a secondment to Chalmers working on this task. NTNU has sent a PhD student (M.A.H. Ashrafi) to Deltares and NTNU has also recruited one Post Doc. (D. Boumezerane)

3-3 CAREERI have Seconded personnel to NGI, in addition NTNU have recruited one Post Doc. (S.A.G. Amiri) and one PhD. student (M. Kadivar) on this topic

WP 4

WP4 relates to model application and recommendations so as to reach the project aim to formulate creep models for engineering practise. Finite element models will be defined assuming materials within the scope of CREEP. The sensitivity of constitutive parameters on predictions at boundary value problem level is looked at in detail. This will enable the publication of good recommendations for the usage of proposed models, which is a necessary prerequisite for their use in practical geotechnical engineering.

Started month 25 and is ongoing

WP 5

WP5 is devoted to knowledge-transfer (ToK) and dissemination of research results to the scientific community. A project website will be created, both for internal and external use. It will be a useful tool for internal data exchange and for making available important outcome as soon as it is ready for publication. On the other hand, the results will be published using conventional channels (technical and scientific journals, conferences, workshops), too. Scientific workshops will be organized within the network at the purpose of keeping a close contact amongst the partners and involving external experts or possible beneficiaries of the results obtained.

ToK and dissemination objectives WP5:

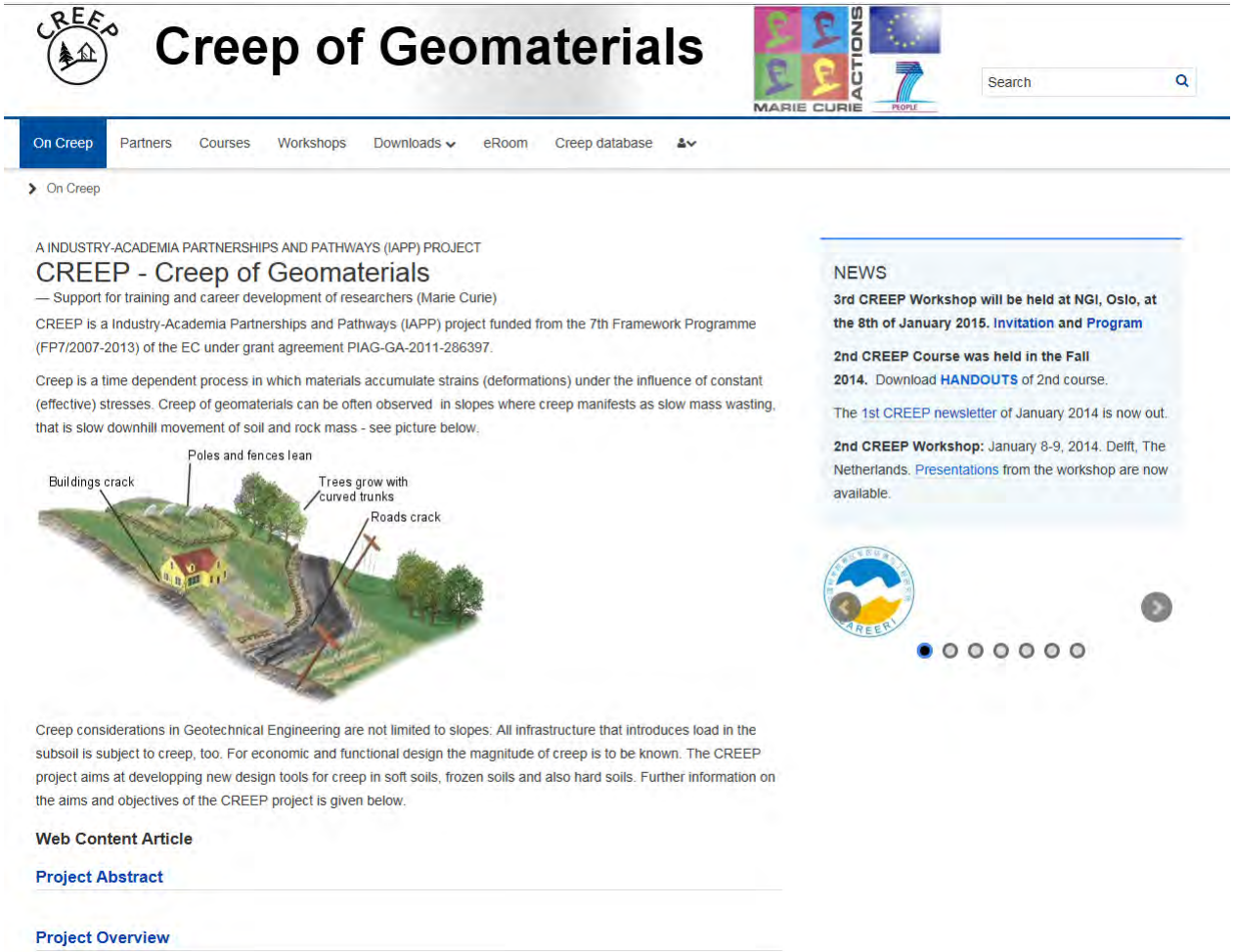
5-1 To create a CREEP web site – Deliverable D11

5-2 To disseminate research results to scientists and geotechnical professionals via publications and workshops – Deliverables D12/13.

5-3To train geotechnical engineers and scientists in creep modelling and numerical analysis. This will be done through secondments and training courses – Deliverables D14 to D16.

The Creep webpage

- http://www.ntnu.edu/creep



NGI - Workshop
Creep Behaviour of Soils

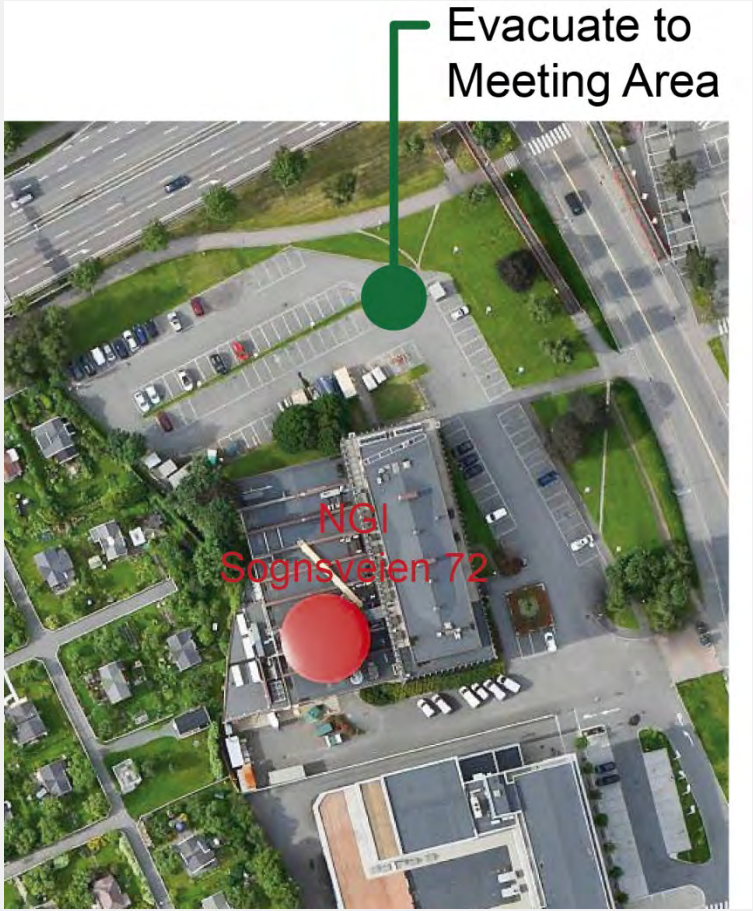
About NGI and Offshore Energy

Karl Henrik Mokkelbost
Director Offshore Energy

Emergency

Emergency telephone numbers in Norway:

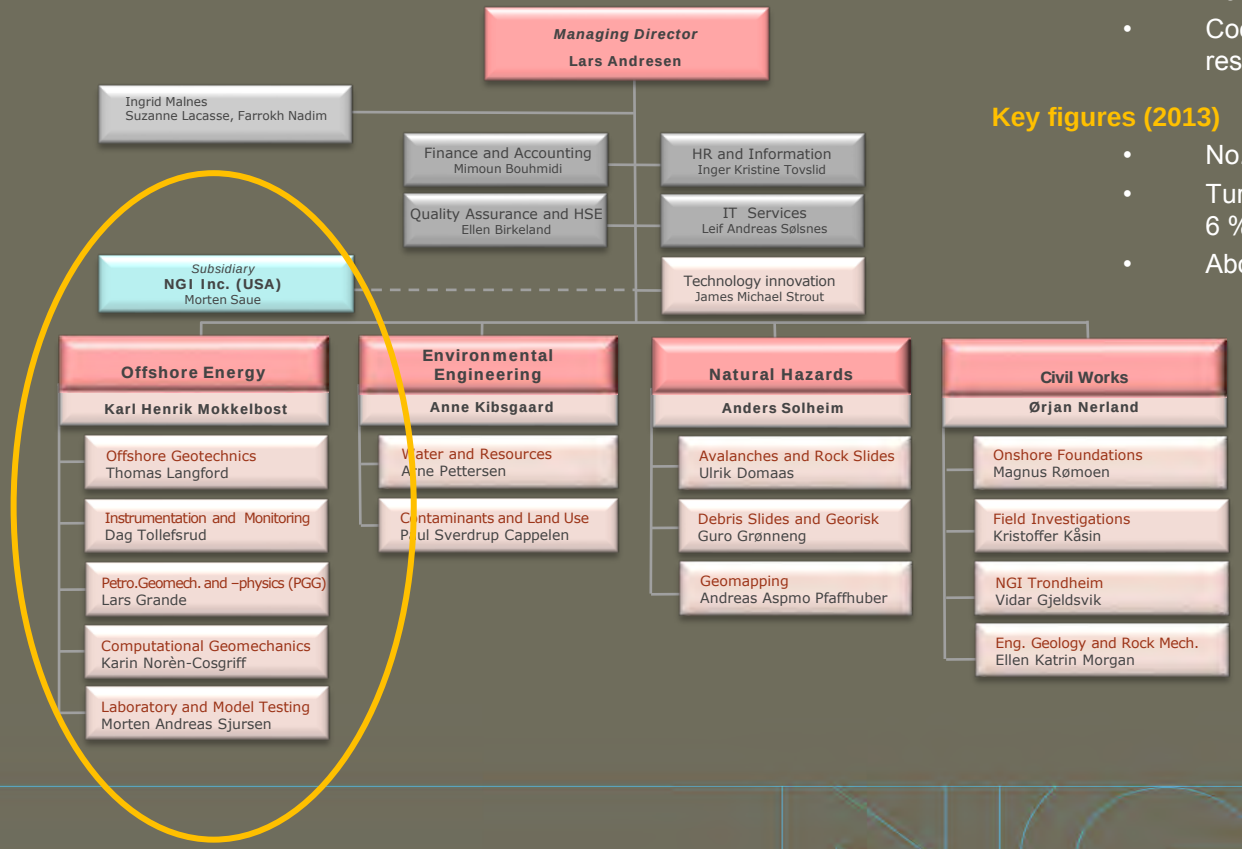
110 - Fire
112 – Police
113 - Ambulance



NGI

NGI organization

+
NGI Perth
Q4/2014



NGI - Private independent foundation

- Established 1953
- R&D and consultancy work
- National and international clients
- Cooperation with universities and research organizations

Key figures (2013)

- No. of employees = 220
- Turnover = 370 million NOK
  6 % from Norwegian Research C
- About 1/3 Offshore Energy

Civil works

- NGI's services

- Field and laboratory investigations
- Geotechnical engineering
- Engineering geology and rock mechanics
- Foundation design
- Geology for soil, rock and permafrost
- Evaluation of ground-borne vibration

NGI

Natural hazards

- NGI's services

- Mapping
- Hazard and risk assessment
- Avalanche warnings
- Tsunami analysis
- Risk mitigation measures
- Monitoring systems
- Geophysical surveys
- Remote sensing
- Assistance in acute situations

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Environmental technology

Consulting | Research & Development | Laboratory

Contaminated land

- Site investigation
- Groundwater monitoring
- Risk assessment
- Remediation
- Sustainable land-use

Marine sediments

- Coastal and offshore monitoring (IMiRO)
- Safe disposal of dredged material
- Capping of sediments
- Impact assessment

Waste & Disposal

- Waste and leachate characterisation
- Site closure and capping
- Beneficial reuse
- Tailings/mining waste

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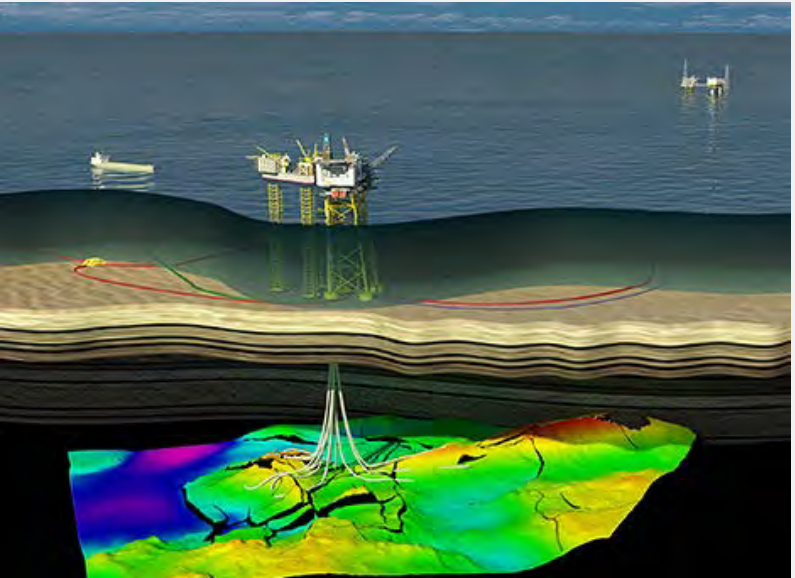
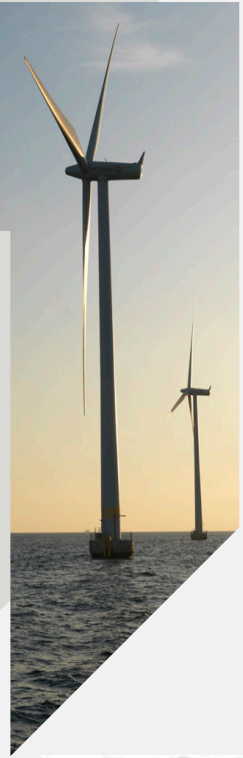
Offshore Energy





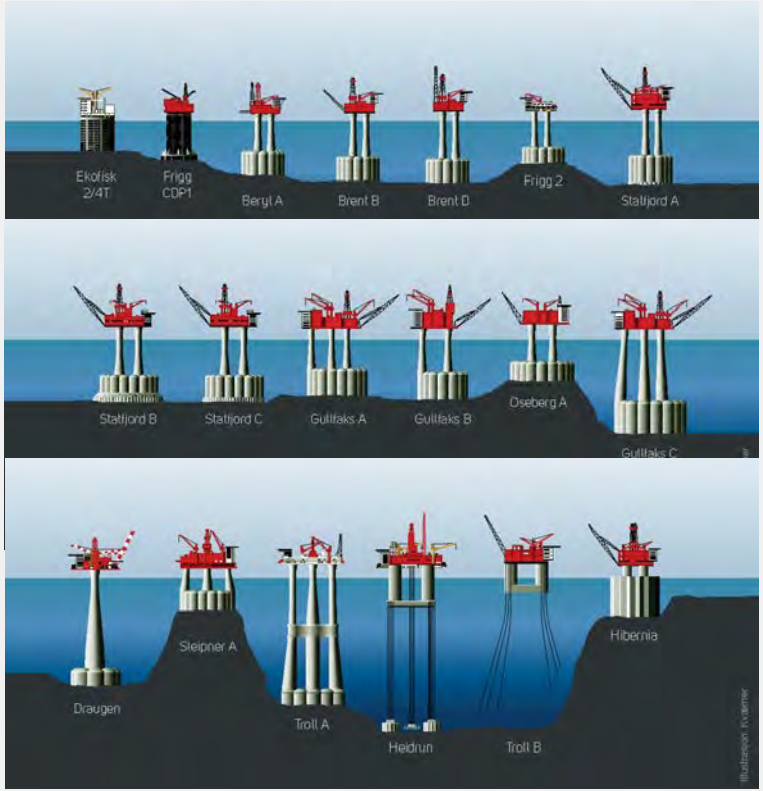
Offshore Energy = 5 Divisions at NGI
+ 2 Daughter companies

OG - Offshore Geotechnics
I&O - Instrumentation & Monitoring
CGM - Computational Geomechanics
PGG - Petroleum geophysics
Lab - Geotechnical and Rock lab
+ NGI Houston + NGI Perth

NGI

Offshore oil and gas over 40 years

Ekofisk tank
First Offshore CPT
in 1972 with NGI



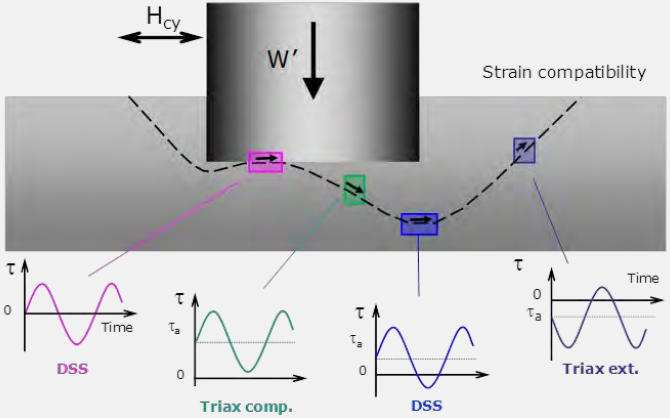
Troll Platform
Installed in 1994
305 m water depth

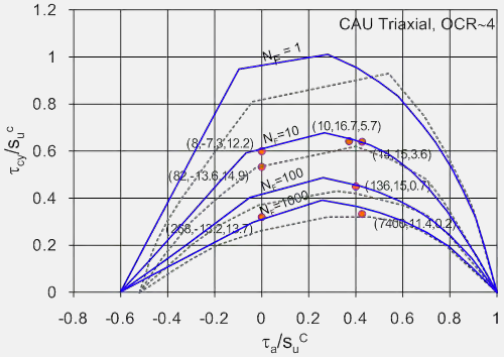
16 Projects for NGI

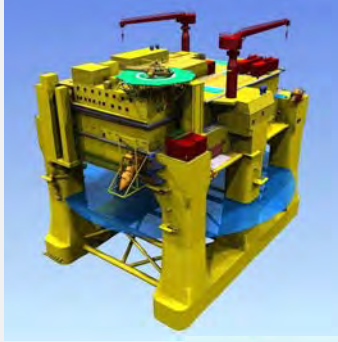
- ✓Soil Investigation
- ✓Lab testing
- ✓Soil properties
- ✓Foundation design
- ✓Instrumentation

NGI

Advanced laboratory testing







Cyclic testing for foundation design:

- Offshore structures are subjected to cyclic loading from wind and waves
- Cyclic testing in triaxial and DSS
- Tests performed in stress and strain control
- Tests interpreted using 'NGI' framework, and results applied directly in design

NGI

SP2 – Strategic R&D project on offshore wind funded by NGI

Troll plattform, Statoil

- Single structure (platform)
- Cyclic loads from waves dominate
- Big and robust structure
- Large return on investment

Alpha Ventus (www.alpha-ventus.de Photo: Matthias Ibeler)

- Multiple structures (OWTs)
- Cyclic loads from wind and wave
- Slender structure (serviceability issues)
- Lesser return on investment

TAILOR MADE → MASS PRODUCTION

(alters basis for site characterisation, design, fabrication & installation)

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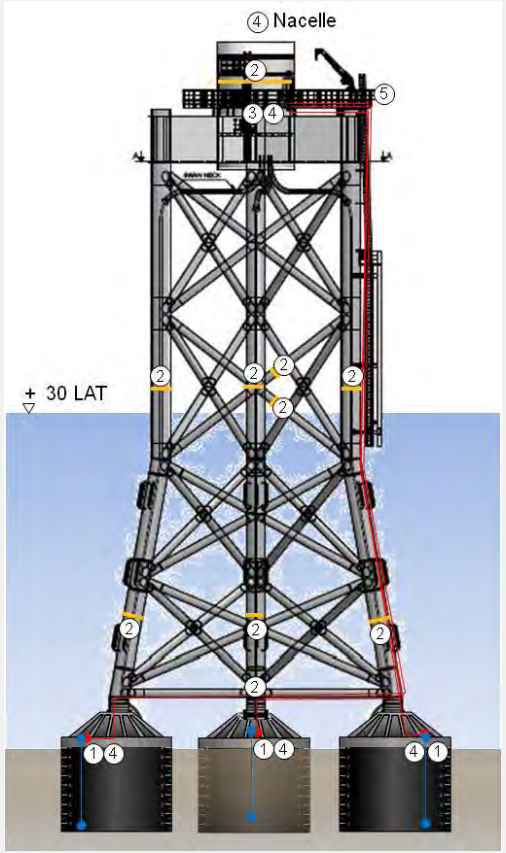
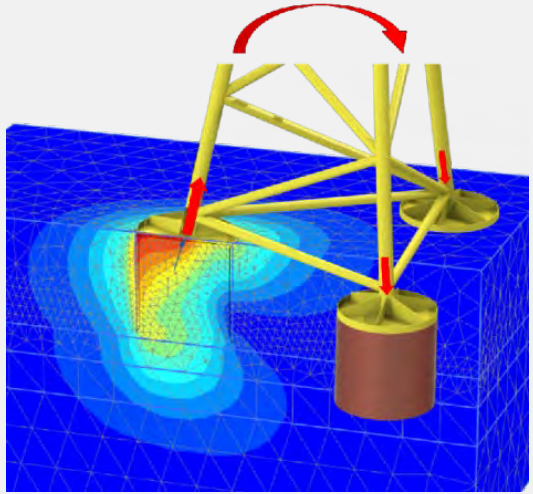
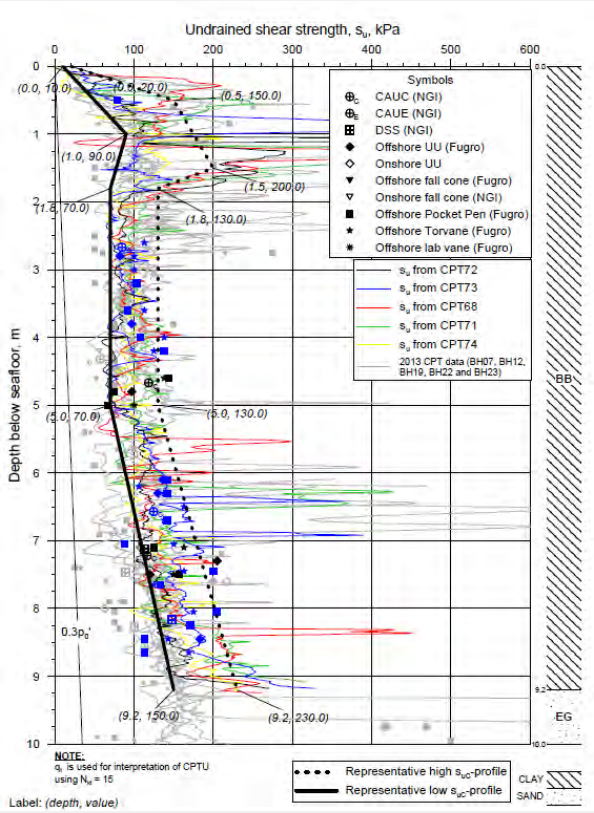
Jacket for Offshore Wind – full scale in situ field test



1. Pore pressure
2. Strain
3. Inclination
4. Dynamic motion
5. Wave radar (air gap)





I&O

CGM

OG & Lab

NGI

3rd International Symposium on Frontiers in Offshore Geotechnics

Oslo, Norway 10-12 June 2015

ISFOG
OSLO 2015

NGI

isfog2015@ngi.no
www.isfog2015.no

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3rd International Symposium on Frontiers in Offshore Geotechnics

Oslo, Norway 10-12 June 2015

NGI www.isfog2015.no

ISFOG OSLO 2015

Sponsoring and exhibition opportunities

About the Symposium

The 3rd International Symposium on Frontiers in Offshore Geotechnics (ISFOG) will be hosted by NGI on 10-12 June 2015 in Oslo, Norway. ISFOG provides a specialist international forum to address geotechnical engineering challenges for those working in offshore construction, design and research.

This symposium will continue the success of ISFOG 2005 and 2010 hosted by the Centre for Offshore Foundation Systems in Perth, Australia. Based on attendance at the previous symposia, we anticipate between 300 and 400 delegates at ISFOG 2015.

ISFOG 2015 will be held at the scenic Holmenkollen Park Hotel Rica with a panoramic view of the city and the Oslo fjord. The venue offers a unique setting with excellent corporate exposure and various opportunities for exhibitions and displays.

For further information on the symposium, please see:
www.isfog2015.no

About NGI

Norwegian Geotechnical Institute (NGI) is a leading international centre for research and consulting within the geosciences. NGI develops optimum solutions for society, and offers expertise on the behaviour of soil, rock and snow and their interaction with the natural and built environment. NGI works within the following sectors: Offshore energy; Building, Construction and transportation; Natural hazards; and Environmental technology.

NGI is a private foundation established in 1953 with the head office and laboratory in Oslo, a branch office in Trondheim and a daughter company in Houston, Texas, USA. NGI was awarded Centre of Excellence status in 2002.

For more information on NGI, please see:
www.ngi.no/en

Why become a sponsor?

As a sponsor of the 3rd International Symposium on Frontiers in Offshore Geotechnics, you get high visibility in a quality environment with direct access to the international offshore geotechnical community.

Sponsoring ISFOG 2015 will enable you to:

- Increase your visibility before an international audience
- Enhance your corporate image
- Reach an influential, exclusive market
- Associate your company with ISFOG 2015

NGI's aim is for the symposium to go break-even economically, whilst maintaining a reasonable fee level for delegates. Sponsorship assists in offsetting the costs for hosting the symposium. Note that sponsorship in excess of the target level will be used to establish a scholarship for student(s) to undertake research in offshore geotechnics at NGI.

Contact information

For further information regarding sponsorship at ISFOG 2015 please contact:

Tom Lunne
e-mail: tom.lunne@ngi.no
Telephone: +47 900 29 267

NGI
PO Box 3930 Ullevål Stadion
N-0806 Oslo
NORWAY

Sponsorship packages

Package	Price (NOK)	Benefits
Gold	200 000	• Company name printed on programme announcements, all flyers and website • Company name printed on flags / banners at the symposium • Company acknowledged as a "principal" sponsor of the symposium • Company acknowledged in Proceedings • One-page company advertisement at rear of Proceedings (black & white) • Exhibitor space included (if desired) • Wallet / bag item (if desired) • 4 No. complementary admissions to symposium
Silver	100 000	• Company name printed on programme announcements, all flyers and website • Company acknowledged as a "session" sponsor of the symposium • Company acknowledged in Proceedings • Half-page company advertisement at rear of Proceedings (black and white) • Exhibitor space available at reduced price of 10.000 NOK • Wallet / bag item (if desired) • 2 No. complementary admissions to symposium
Bronze	50 000	• Company name printed on programme announcements, all flyers and website • Company acknowledged in Proceedings • Quarter-page company advertisement at rear of Proceedings (black and white) • Exhibitor space available at reduced price of 15.000 NOK • Wallet / bag item included • 1 No. complementary admissions to symposium

Additional Item	Price (NOK)
Indoor exhibitor space (ca. 3x3m)	20 000
Wallet / Bag item (one A4 item or equivalent of publicity material to be included with each delegate pack - all items to be supplied by exhibitor)	2 000

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NORGES GEOTEKNISKE INSTITUTT
NGI.NO

Statens vegvesen

Evaluation of creep hypotheses A and B

Samson Abate Degago
Norwegian Public Roads Administrations (SVV)
(*Formerly, Norwegian University of Science and Technology (NTNU)*)

3rd Creep Workshop, A Industry-Academia Partnerships and Pathways (IAPP) project
CREEP – Creep of Geomaterials

Norwegian Geotechnical Institute (NGI), Oslo, January 08, 2015

Background

- This study was motivated by the core theme of 1st CREBS workshop held in Oslo in 2006.

- In CREBS II (Pisa, 2007) a need for in-depth study, e.g. in form of a PhD study, was stressed by Adjunct Professor Hans Petter Jostad.

- This study was then initiated and conducted at Norwegian University of Science and Technology (NTNU) (2007–2011) in collaboration with Norwegian Geotechnical Institute (NGI) and Chalmers University of Technology.

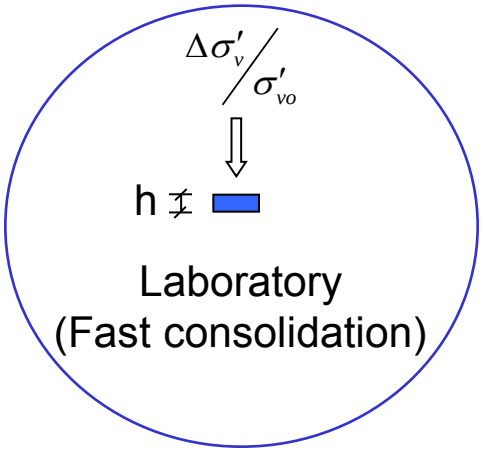
- Researchers who are directly involved in this work are acknowledged as
  - ✓ Hans Petter Jostad (NGI)
  - ✓ Gustav Grimstad (NTNU)
  - ✓ Steinar Nordal (NTNU)
  - ✓ Mats Olsson (Chalmers and NCC)
  - ✓ Peter Hedborg (Chalmers)

- The work has also benefited from valuable feedbacks, discussions and review critics by several other researchers.

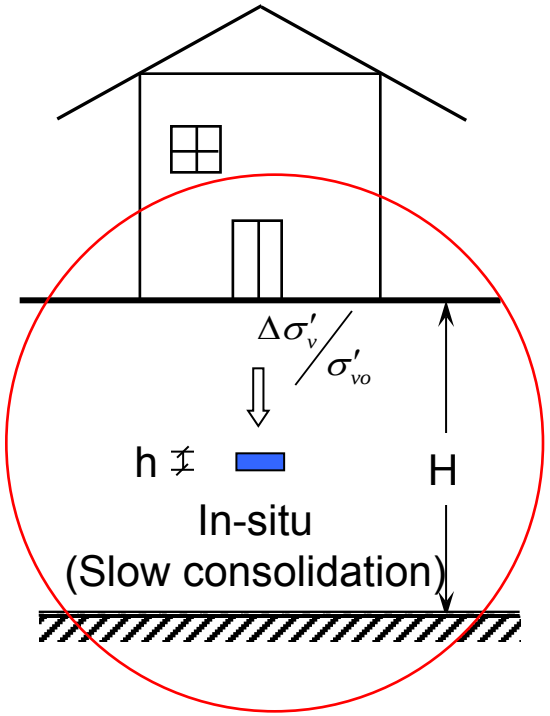
General

Measurements

e.g. minutes, hours



Significantly different consolidation times

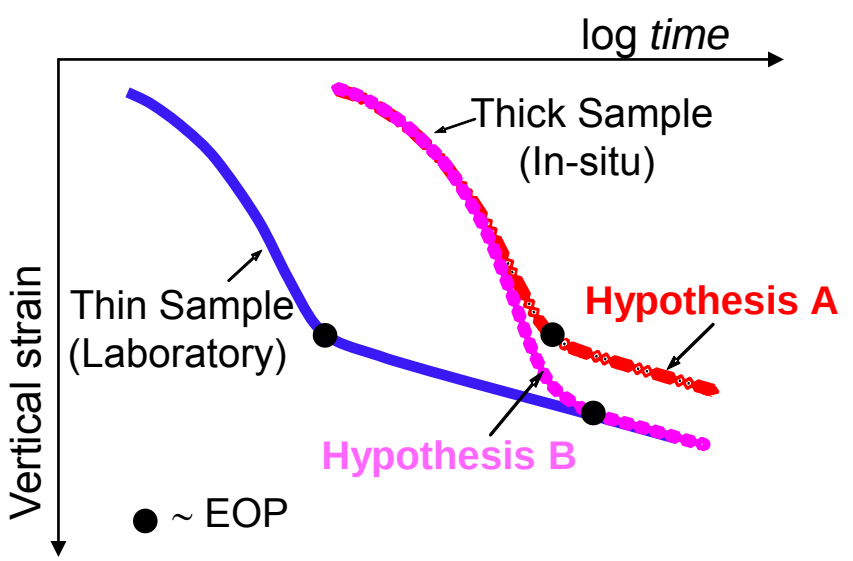


Prediction

e.g. months, years, decades

Two hypotheses on role of creep during primary consolidation

- Proposed in 9th ICSMFE by Ladd et al. (1977).



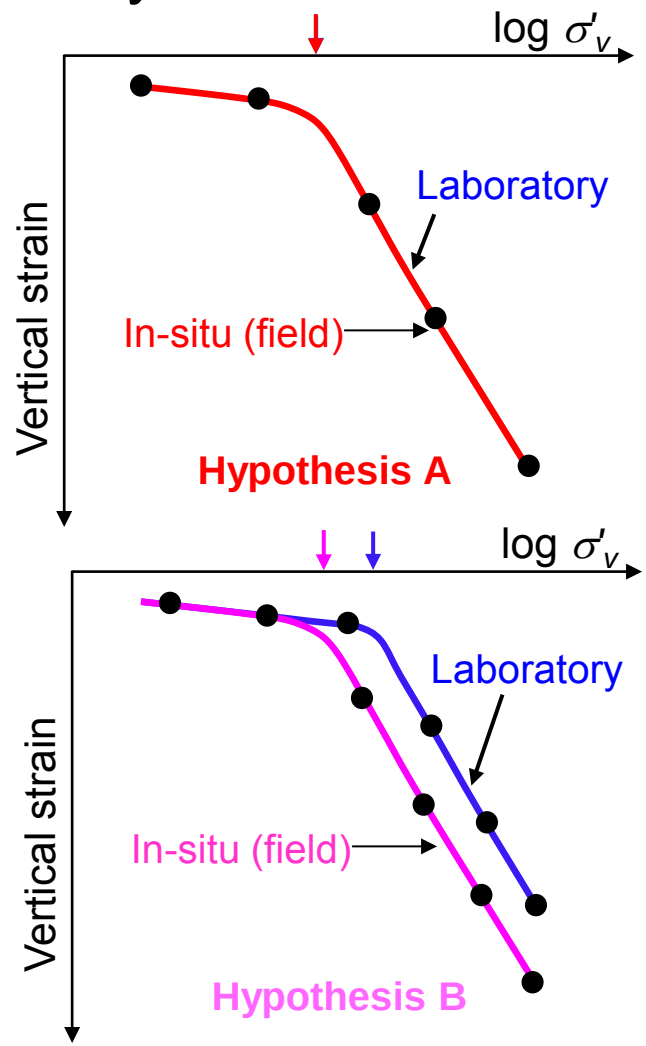


Fig. Hypothesis **A** and **B** (after Ladd *et al.*, 1977)

- Advocates of the two different creep hypotheses have *independently* presented voluminous laboratory and field data to substantiate their opinions.

Experimental substantiation of the two hypotheses, e.g.

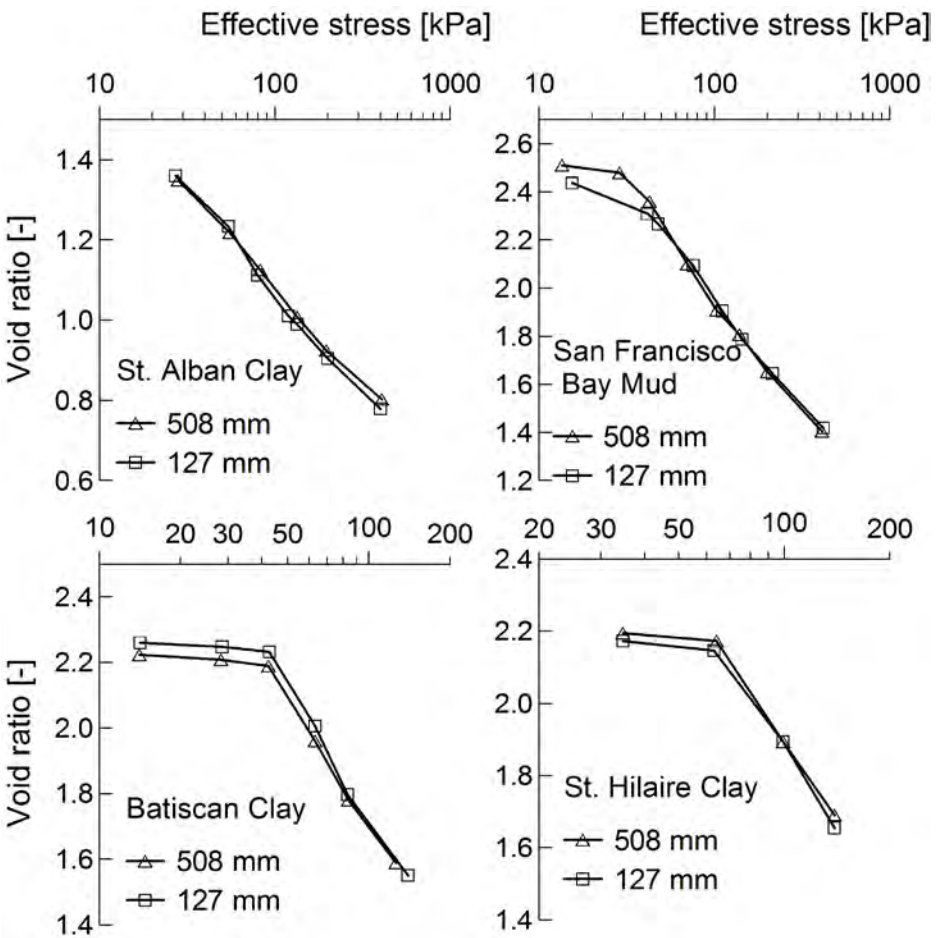


Fig.: EOP laboratory tests supporting **hypothesis A** (after Choi, 1982; Feng, 1991)

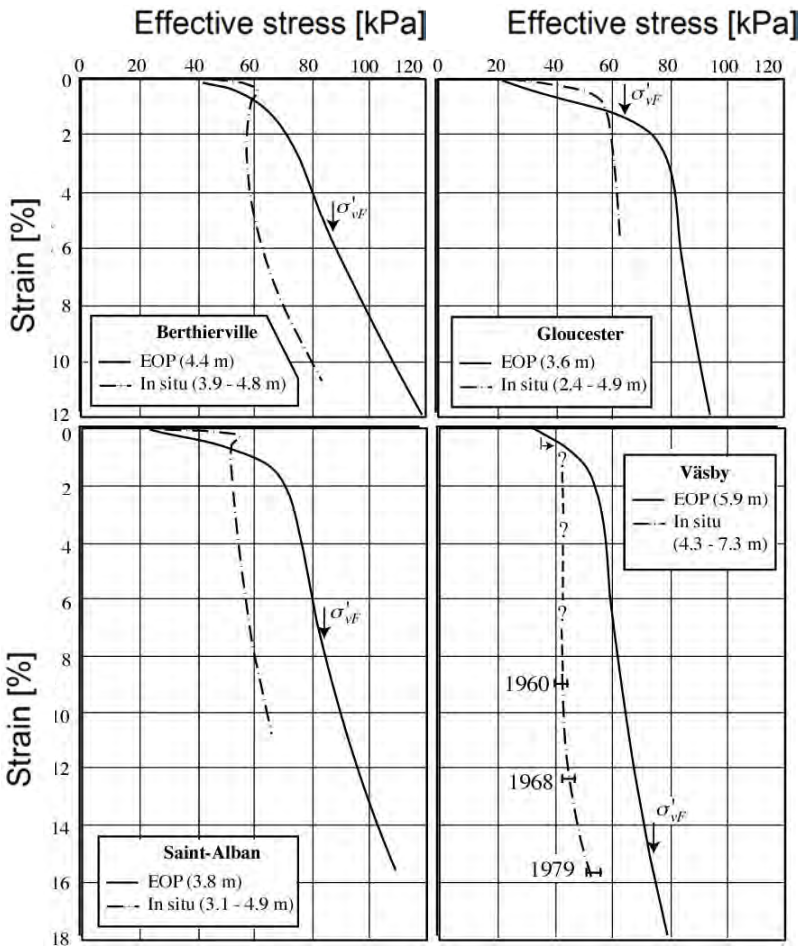


Fig.: In-situ and EOP laboratory tests that support **hypothesis B** (after Kabbaj *et al.*, 1988)

Numerical substantiation of the two hypotheses, e.g.

- Analysis of field cases using constitutive models based on the two hypotheses

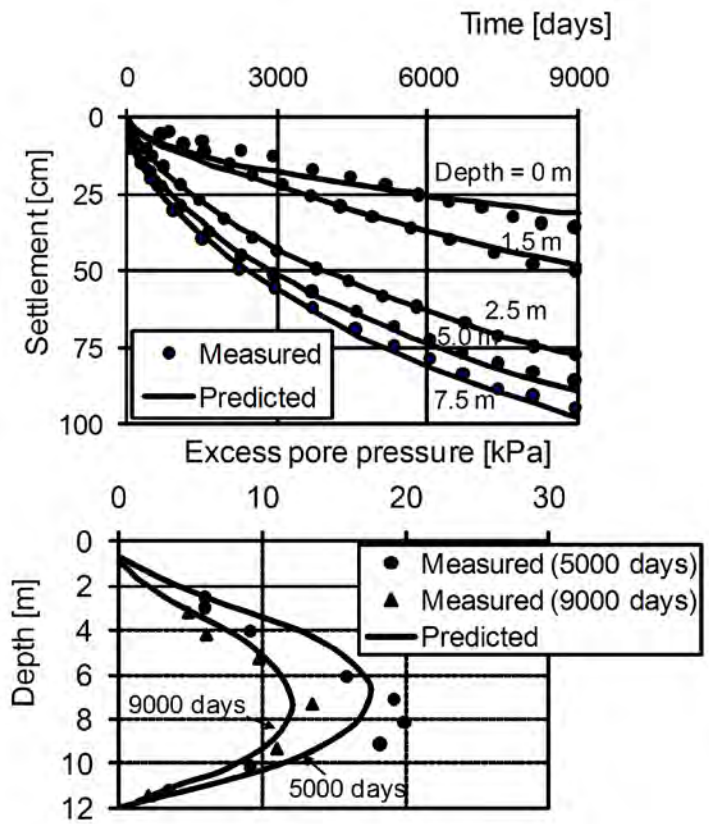


Fig. : Measurement Vs. predictions at Skå-Edeby test fill using **hypothesis A** model (after Mesri and Lo, 1989)

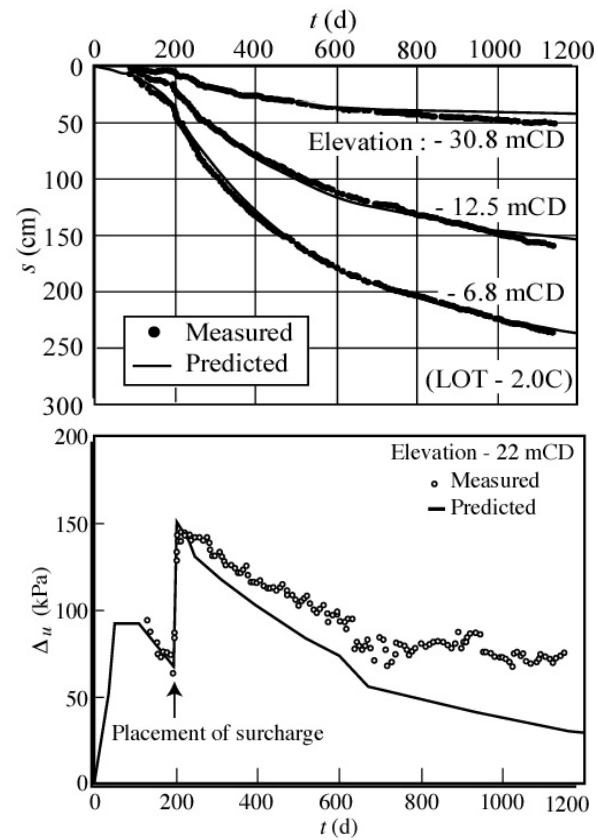


Fig. : Measurements Vs. predictions at Changi Airport using **hypothesis B** model (Cao *et al.*, 2001)

More on the two creep hypotheses

- With an inclination to hypothesis A, Ladd *et al.* in 1977 concluded that "*little definitive data exists to show which of the two hypotheses is more nearly correct for the majority of cohesive soils*".
- Ever since, this becomes a topic of active debate and discussion and remained an issue that needed to be resolved.
- This discussion was re-started by NGI in 2006 at 1st CREBS workshop (Oslo), where advocates from both sides as well as others have attended.
- In 2007, this study was initiated at NTNU in collaboration with NGI and ICG (International Center for Geohazards).

Main motivation and objectives – CREBS I

- How to extrapolate creep from short time observation to long term predictions ?
- The two conflicting hypotheses are well substantiated with laboratory and field data. Why ?
- Constitutive models based on the two hypotheses are seen to produce acceptable field predictions. Study and evaluate the models based on field cases.
- To increase understanding on time- and stress-compressibility of clays during primary consolidation.
- To produce the most convincing creep hypothesis and a numerical tool that can consistently explain laboratory and field observations.

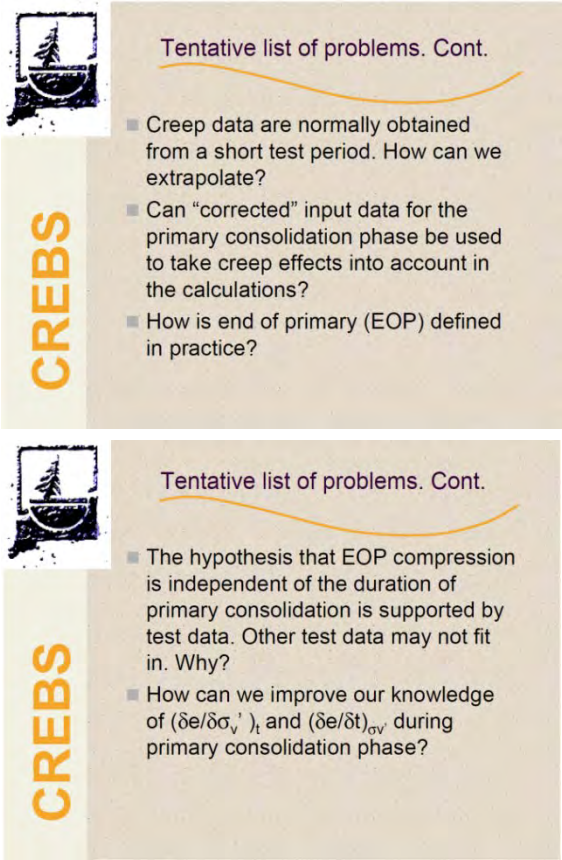


Fig. : Tentative list of problems as presented in the 1st CREBS workshop (Jostad, 2006)

Outline of the presentation, hence the evaluation

- Laboratory studies
  - Part I: Specimens of varying thicknesses
  - Part II: Soil element compressibility (varying consolidation duration)
- Field studies
- Present implication of the hypotheses for a specific case
  - A look at the relevant laboratory tests
  - Numerical studies

Laboratory studies I: Creep hypotheses for varying soil layer thicknesses

- EOP strain-effective stress relationships: the creep hypotheses

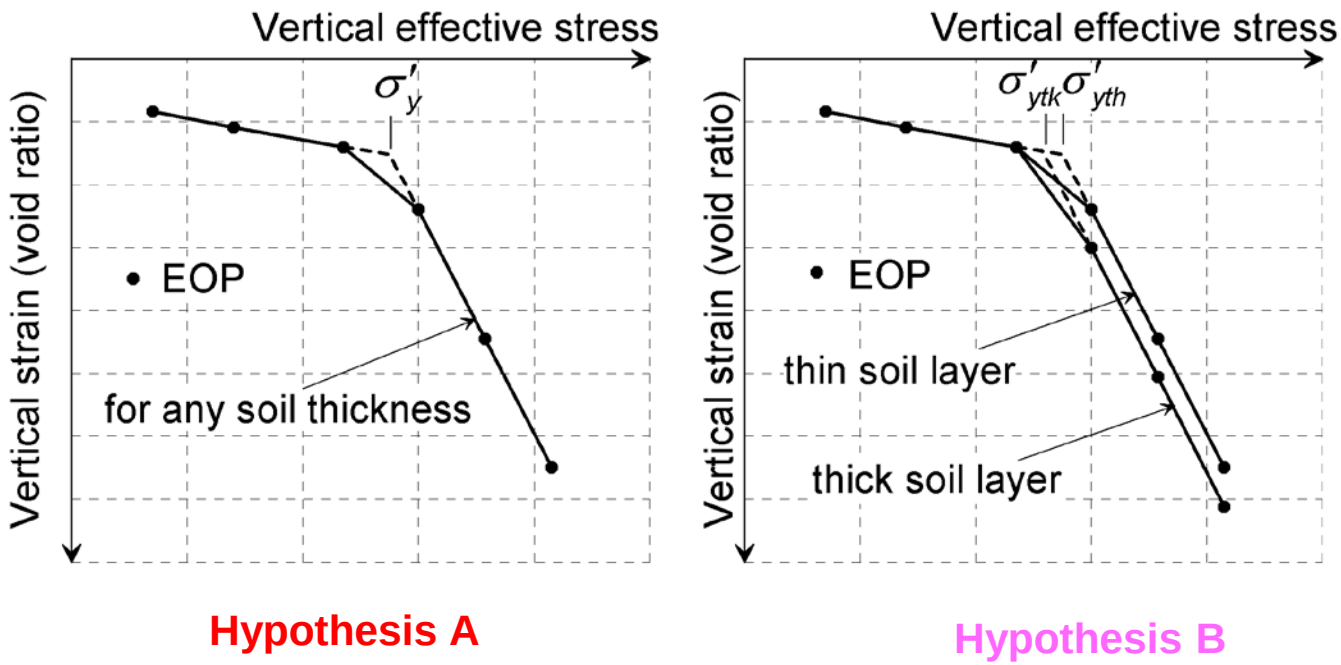


Fig.: Principle sketch of the two creep hypotheses for varying soil layer thicknesses

- EOP strain-effective stress relationships: laboratory tests
- 14 laboratory tests conducted by advocates of hypothesis A….
- End effects (testing problem) on 9 of them ?
- Strain rate effects ?

✓ Evaluate the two 508 mm thick specimen (the action and the reaction)

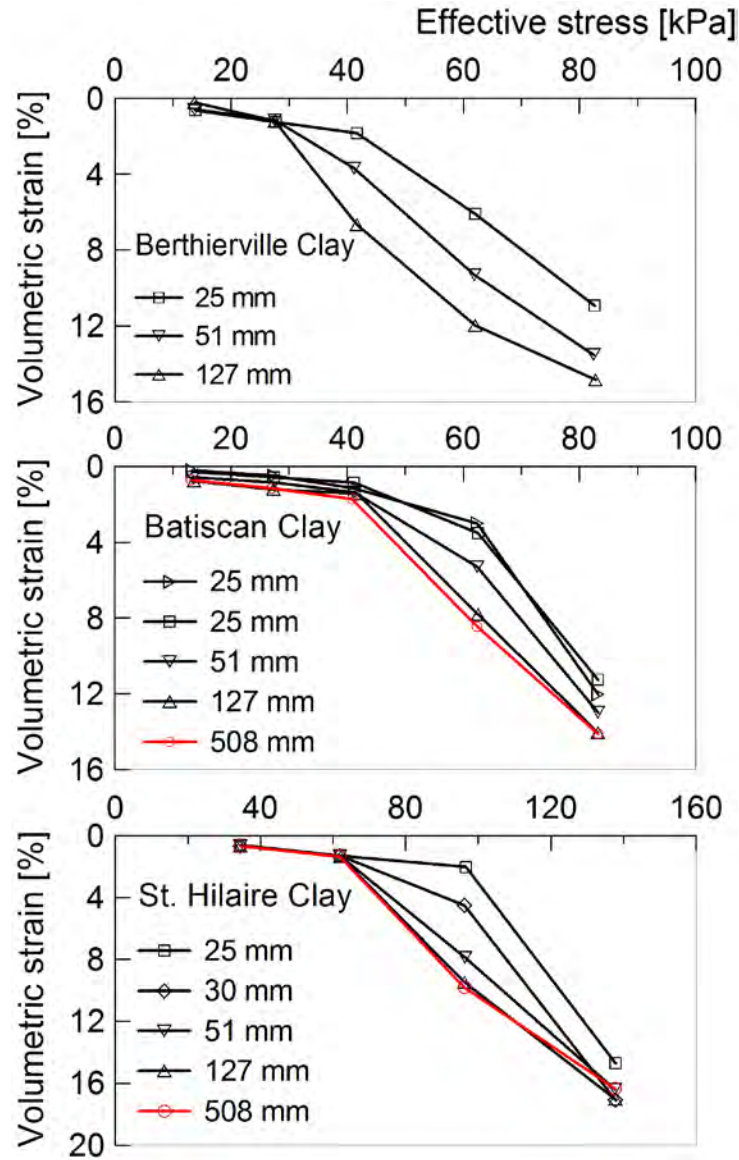


Fig. : EOP $\Delta V/Vo-\sigma'$ relationships for various thicknesses (after Feng, 1991)

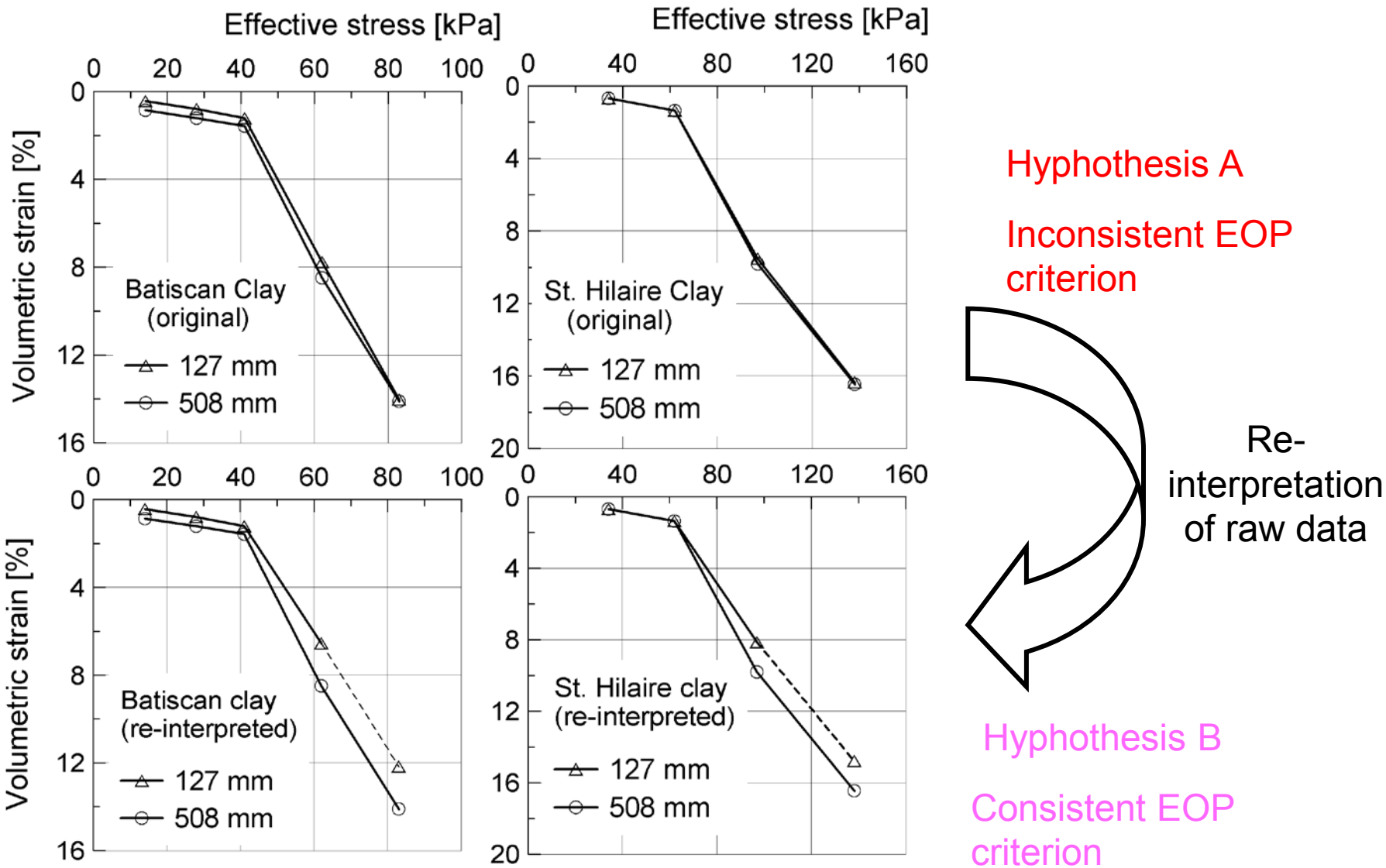


Fig.: Original and re-interpreted volumetric strain–effective stress relationships

Numerical study of raw experimental data with hypothesis B model

- Similar load sequence and duration adopted from the actual test.
- Identical set of soil parameters for the thin and thick specimen
- Three load increments with respect to p'_c

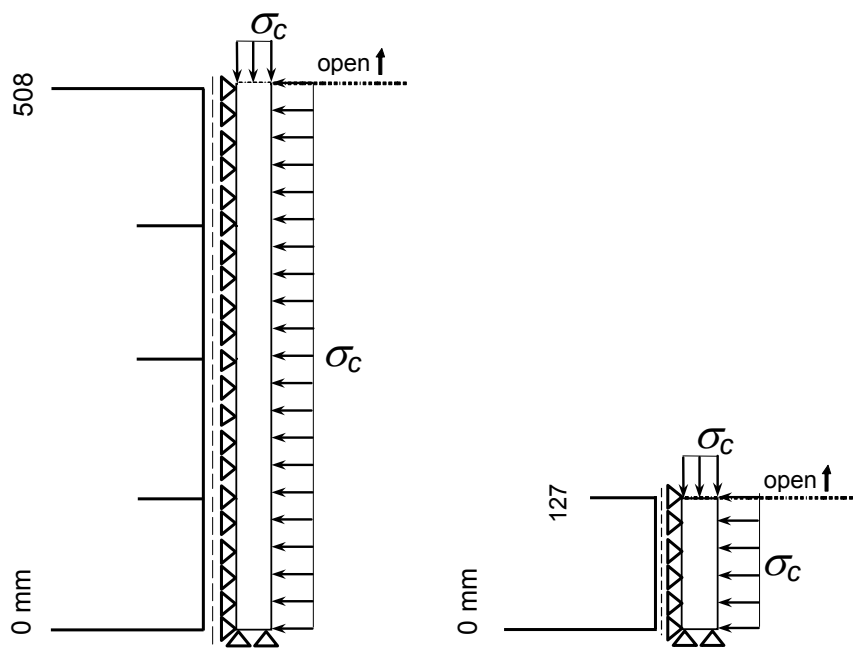


Fig.: Axisymmetric FE-model of the triaxial specimens

- Numerical study of raw experimental data with hypothesis B model

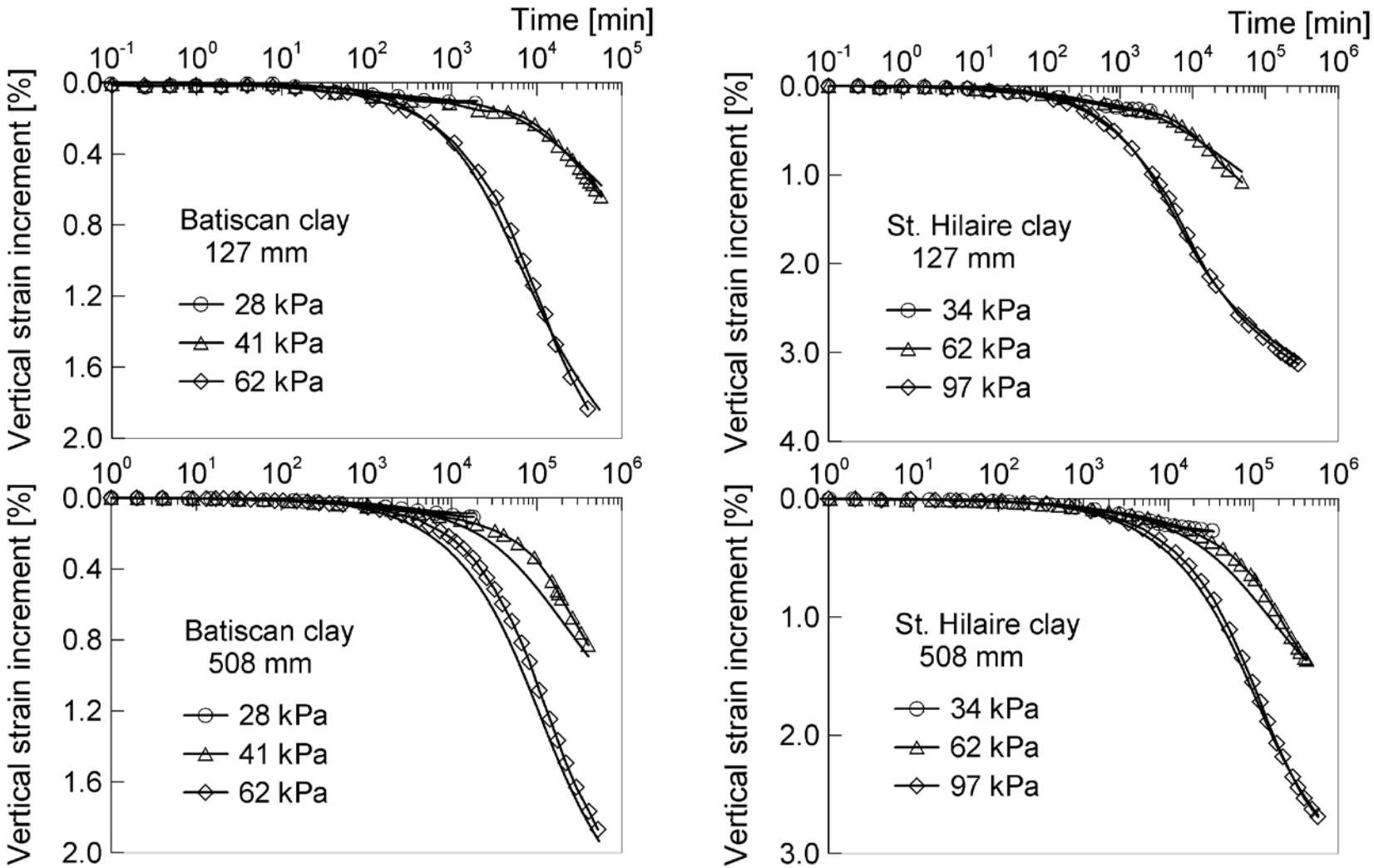
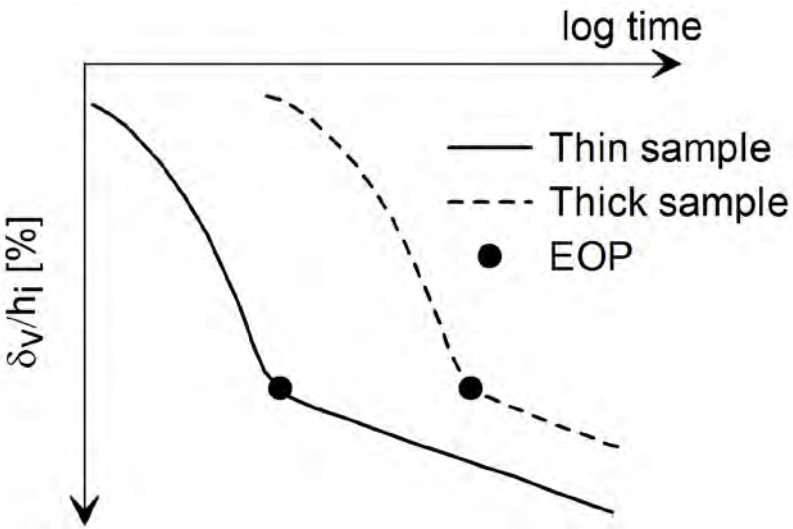


Fig.: Numerical simulation (smooth lines) vs. measurements (lines with symbols)

Strain-time relationships: the creep hypotheses

- Hypothesis A



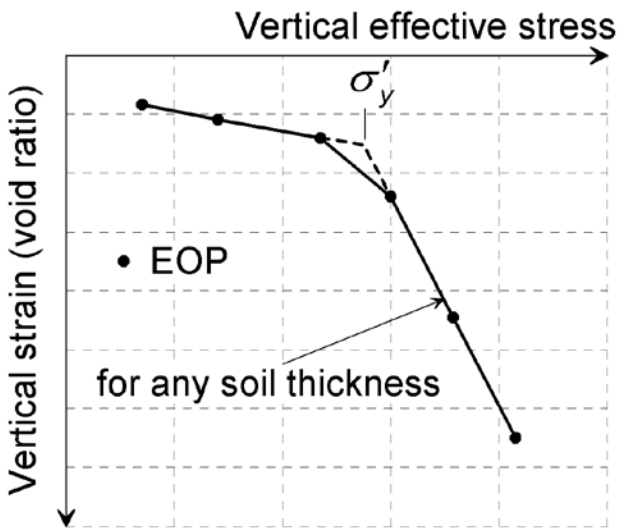


Fig. : Principle sketches of action–response relationships according to hypothesis A

- Hypothesis B

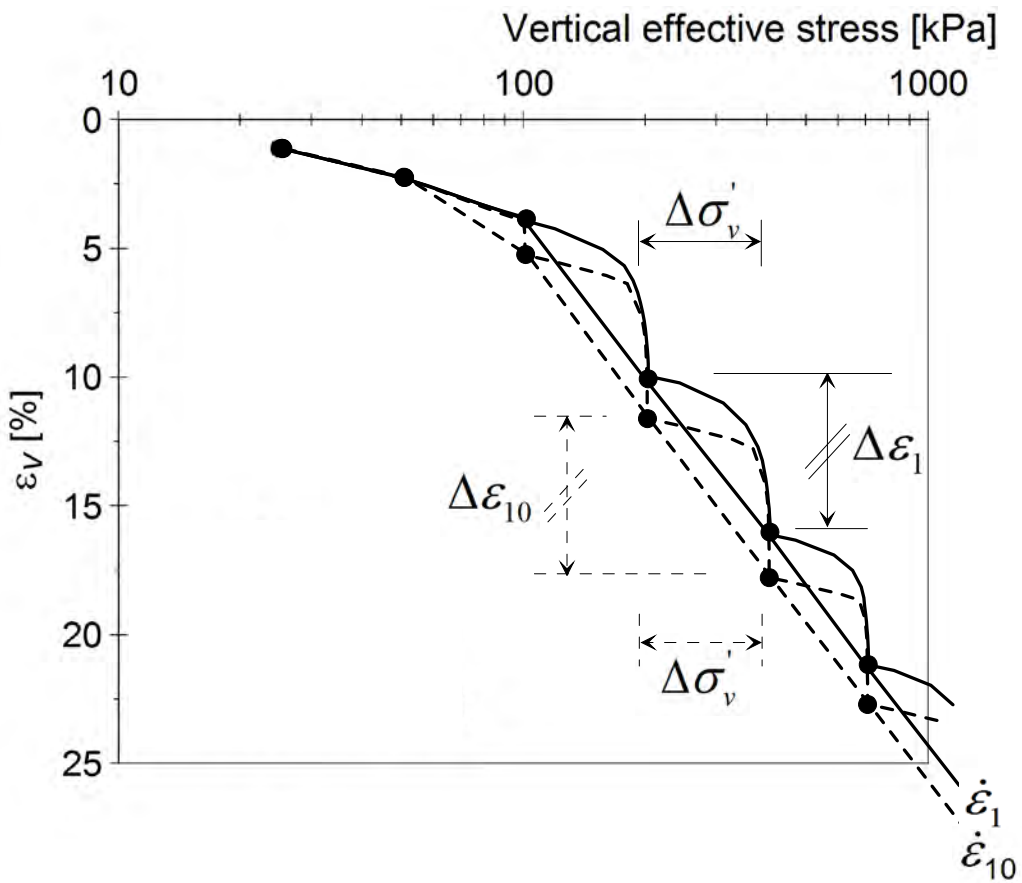


Fig.: Effective stress-strain relationship followed by a soil element close to the top of 1 and 10 cm thick specimens according to hypothesis B

- Hypothesis B – stress increment *exceeding* the initial p'_c

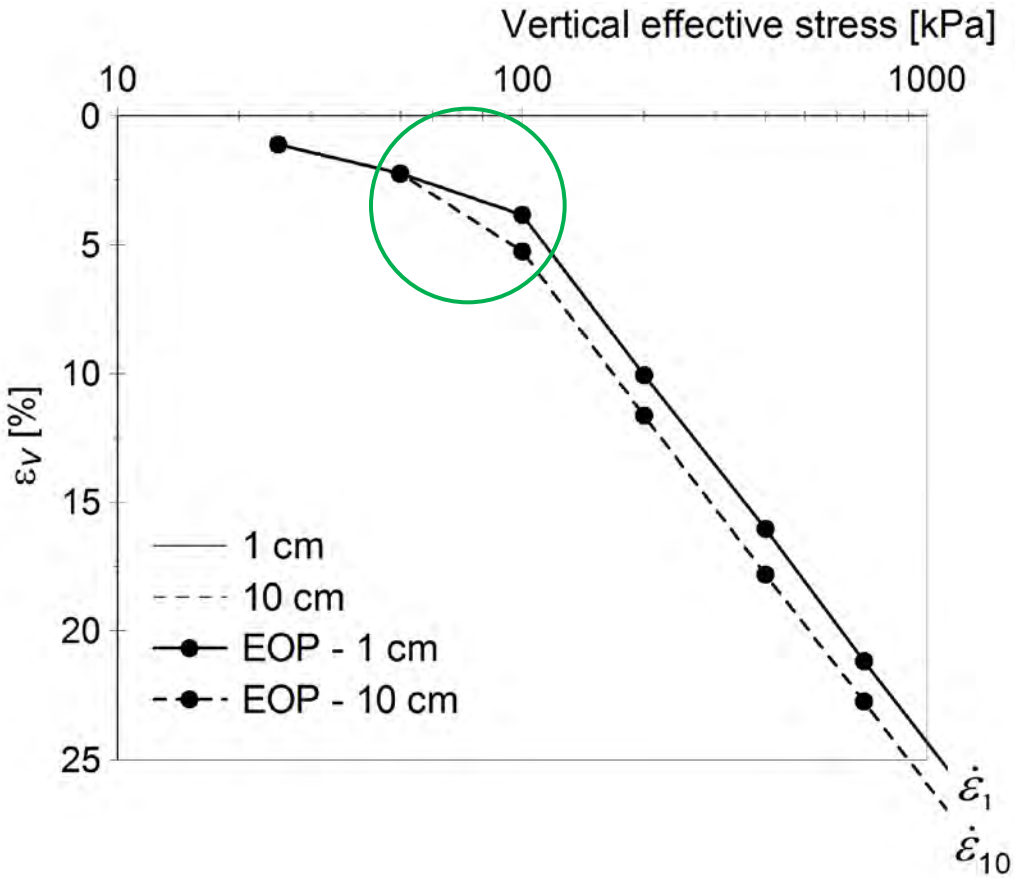


Fig.: Effective stress–Strain and Strain-Time relationships according to hypothesis B

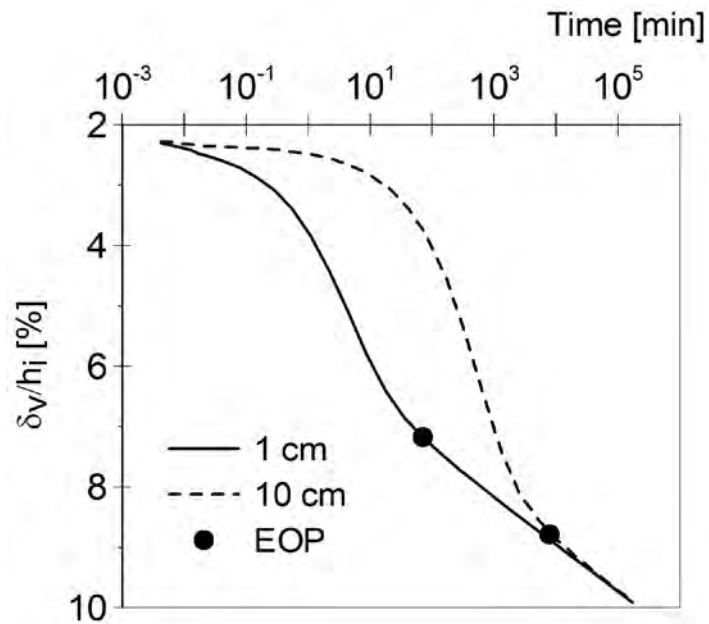
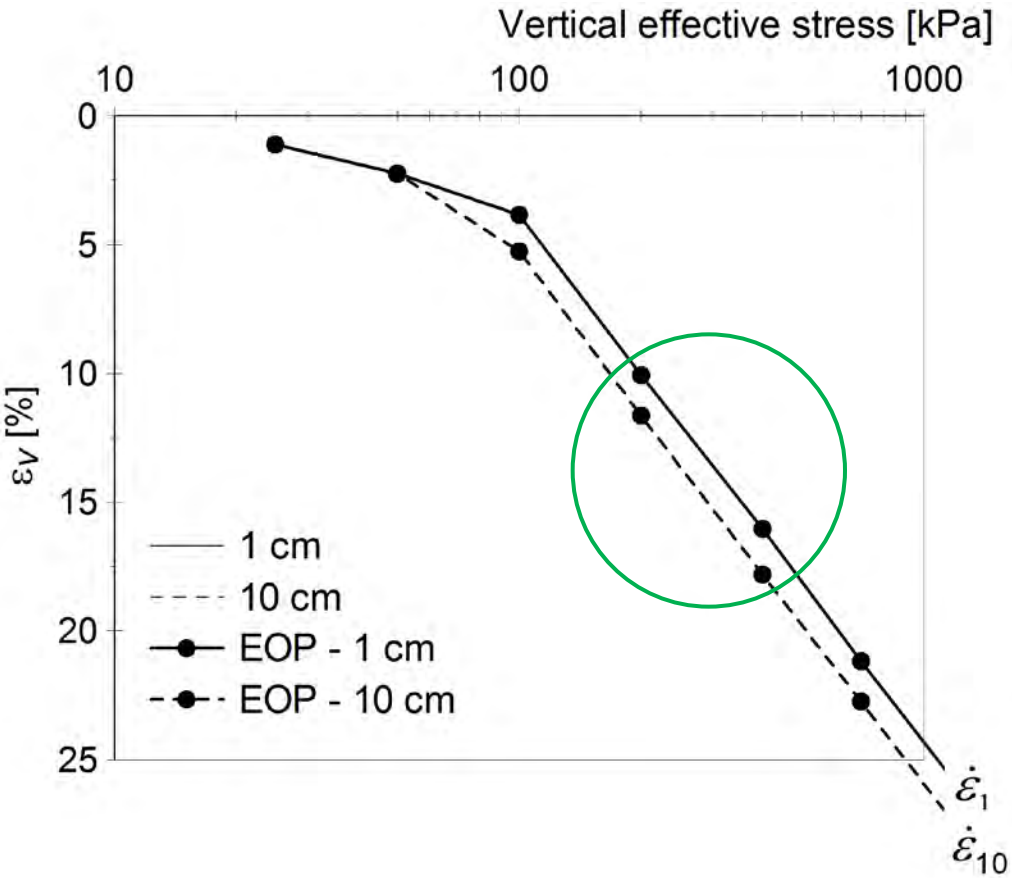


Fig. : $\Delta\sigma'$ = 50–150 kPa, p'_{c_24hr} = 100 kPa (allowed to creep at 150 kPa for 100 days)

- Hypothesis B – stress increment *after exceeding* the initial p'_c



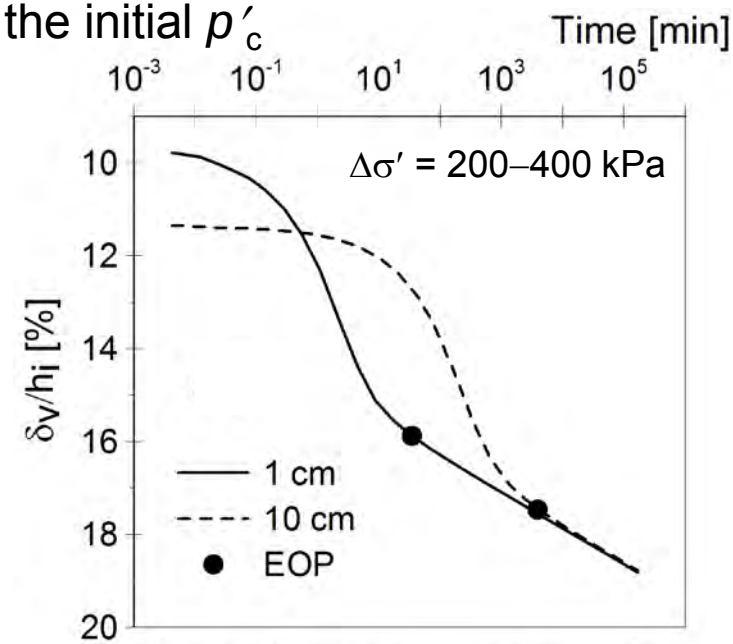
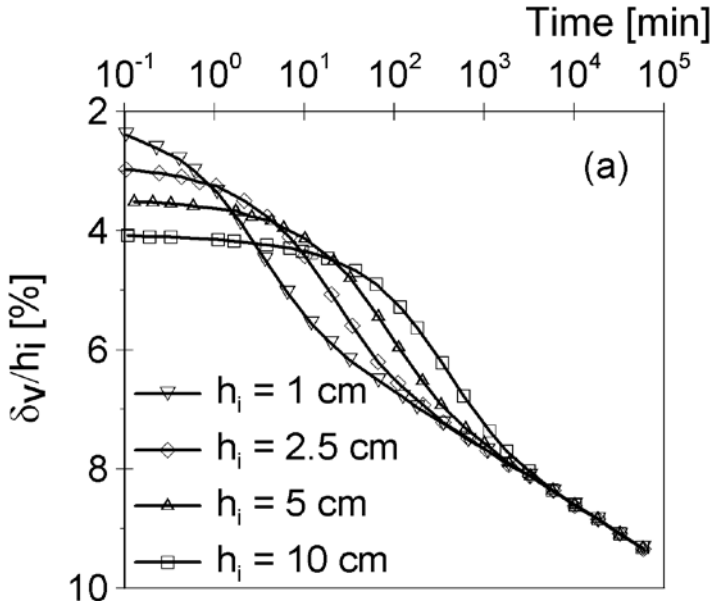


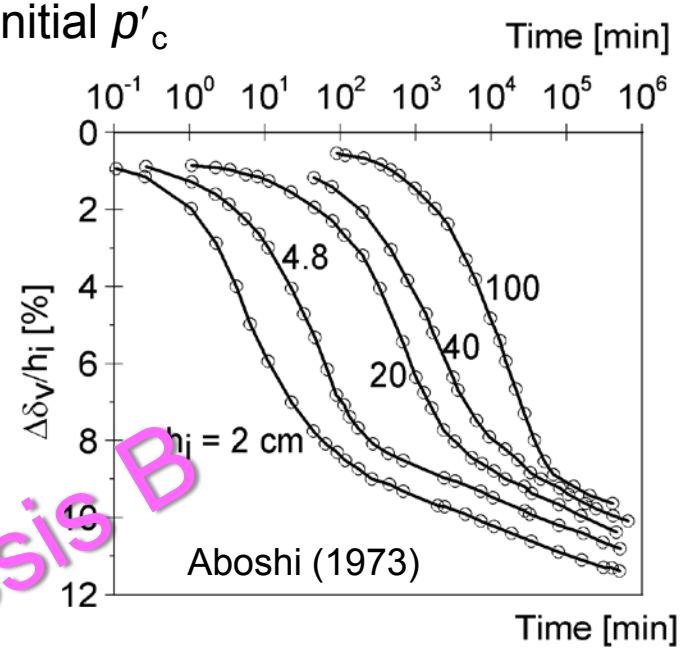
Fig.: Effective stress–Strain and Strain-Time relationships according to hypothesis B

Some typical experimental observations

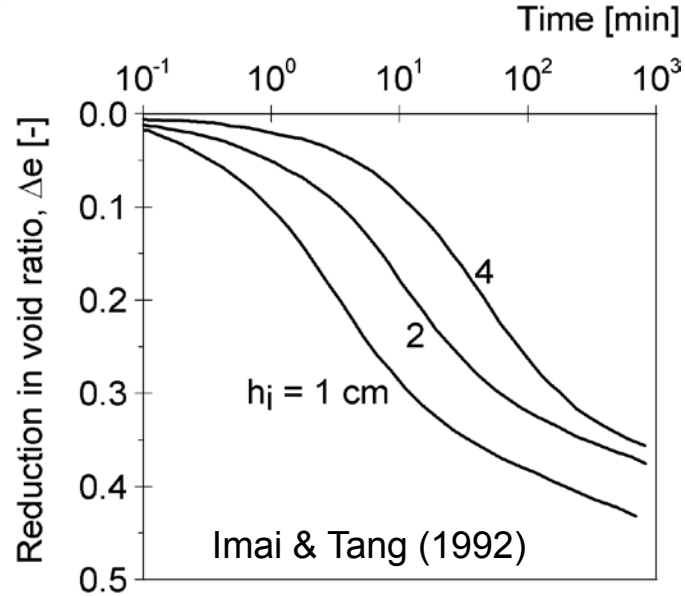
❖ Single load increment tests *after exceeding* initial p'_c



Konovalov & Bezvolev (2005)



Hypothesis B



(Degago et al. (2011), *Géotechnique* 61(10))

Laboratory studies I: Creep hypotheses for varying soil layer thicknesses

Final remarks

- ✓ Laboratory tests on specimens of varying thicknesses imply hypothesis B.
  - EOP strain-effective stress relationship is not unique.
  - EOP strain increases with increasing consolidation duration
- ✓ Numerical simulation results using hypothesis B model can explain experimental measurements.

Laboratory studies II: Creep hypotheses for soil element compressibility

- The two hypotheses are best differentiated by consolidation duration of soil layers than soil layer thickness

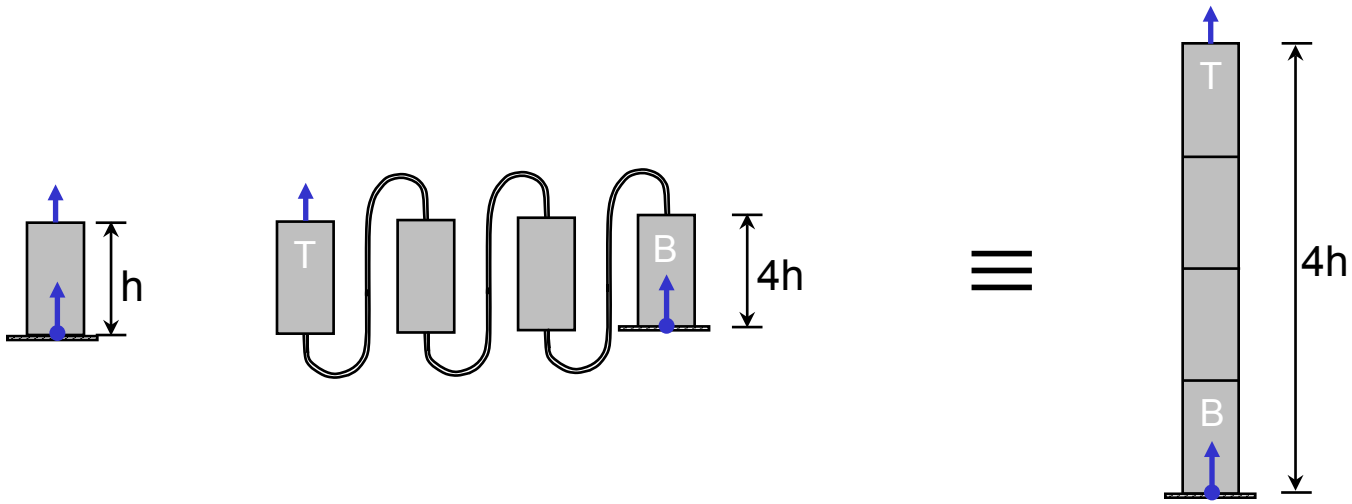


Fig.: Interconnected tests

Creep hypotheses for soil element compressibility

EOP strain-effective stress relationships: the creep hypotheses

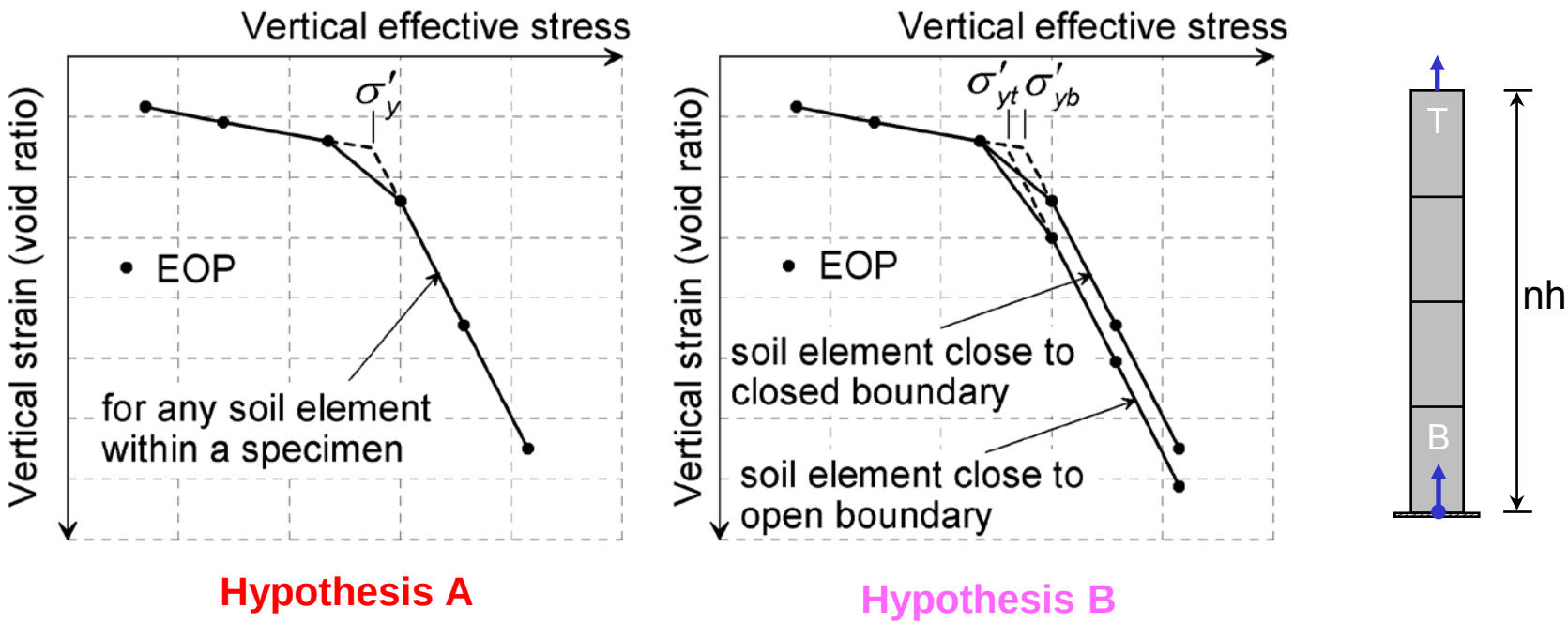


Fig.: Principle sketch of the two creep hypotheses for compressibility of soil elements within a specimen

EOP strain-effective stress relationships: laboratory test results

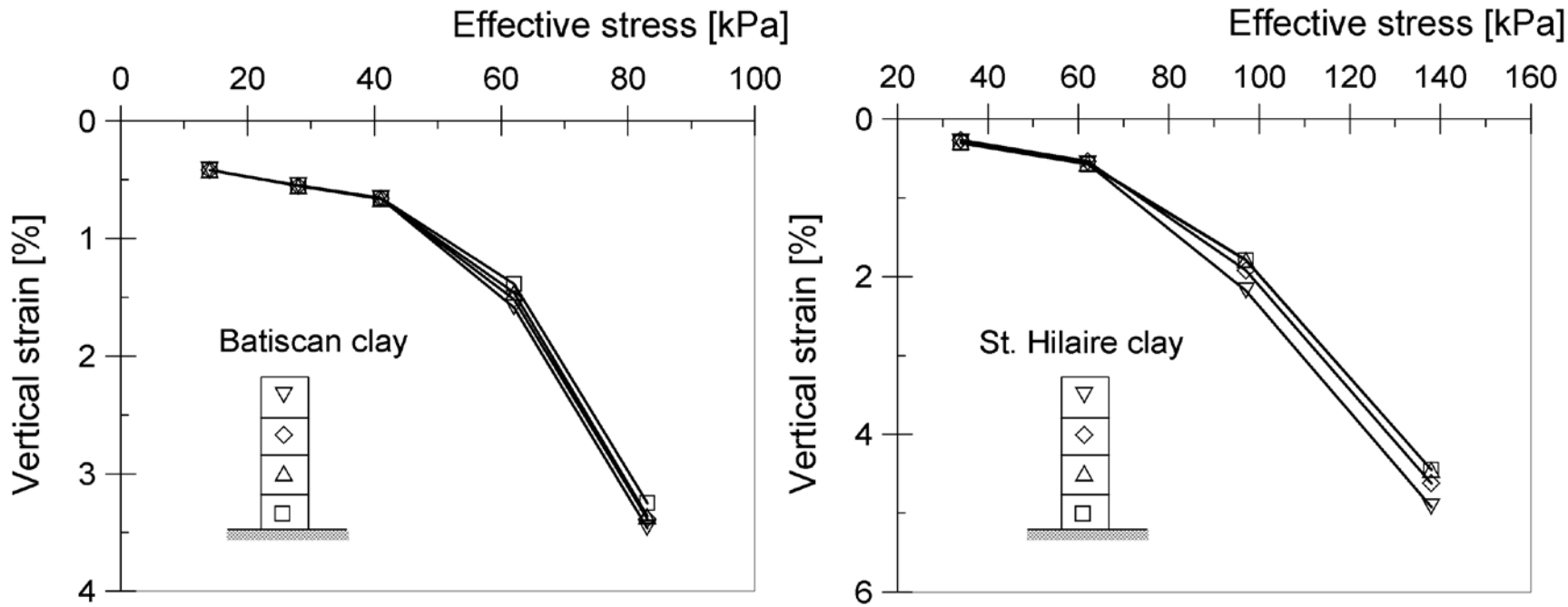


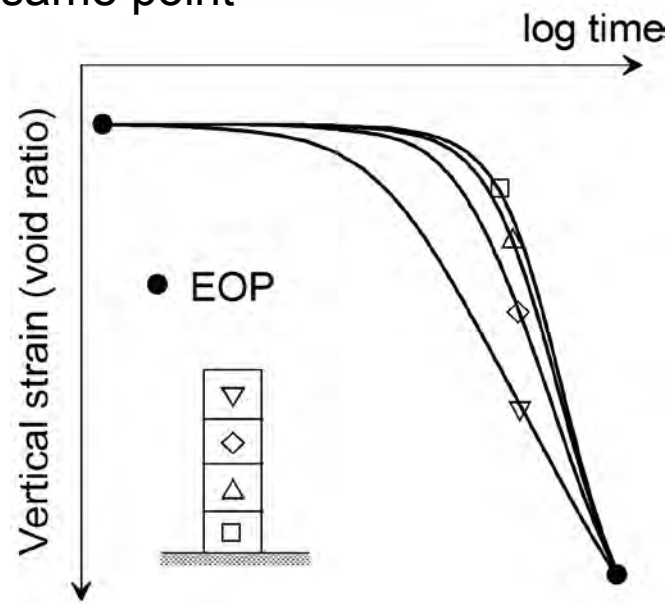
Fig.: EOP vertical strain–effective stress of sub-specimens (interpreted from Feng, 1991)

- Hypothesis B

Strain-time relationships: the creep hypotheses

Hypothesis A

- At EOP, the strain-time relationships of all sub-specimens converge to the same point



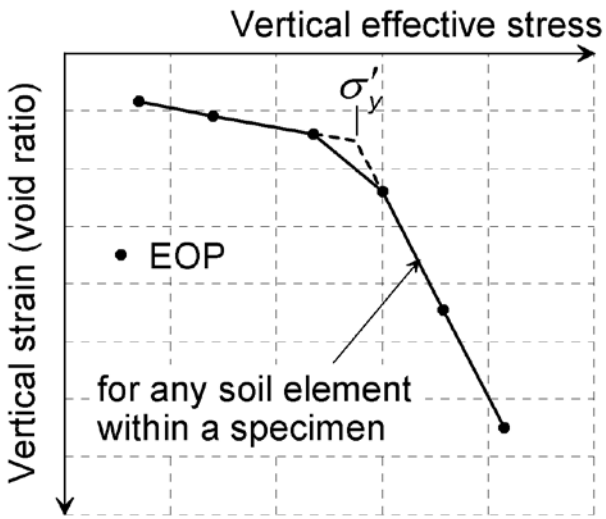
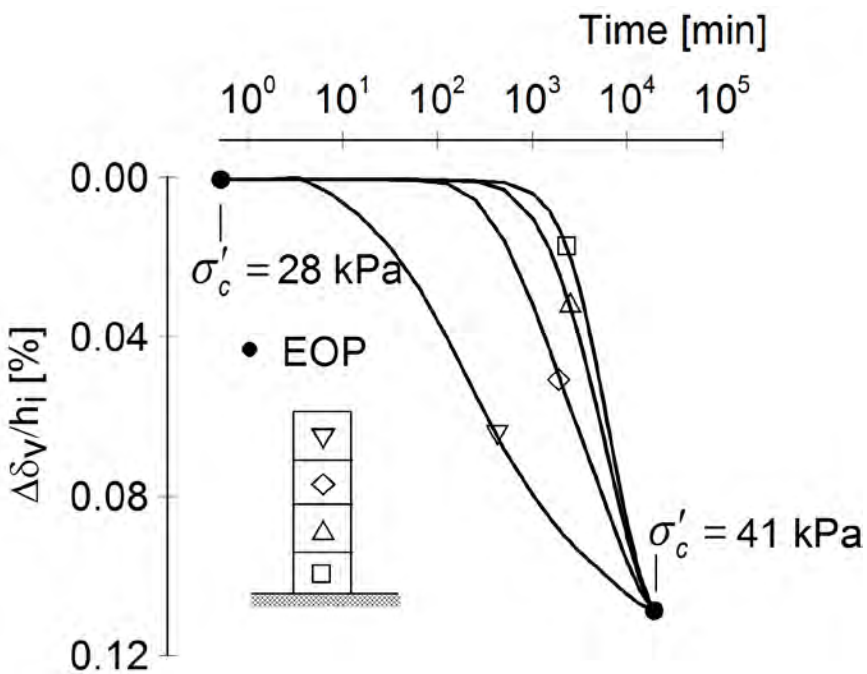


Fig.: Principle sketches of Strain-Time and Effective stress–Strain relationships according to hypothesis A

Hypothesis B – stress increment *before exceeding* the initial p'_c



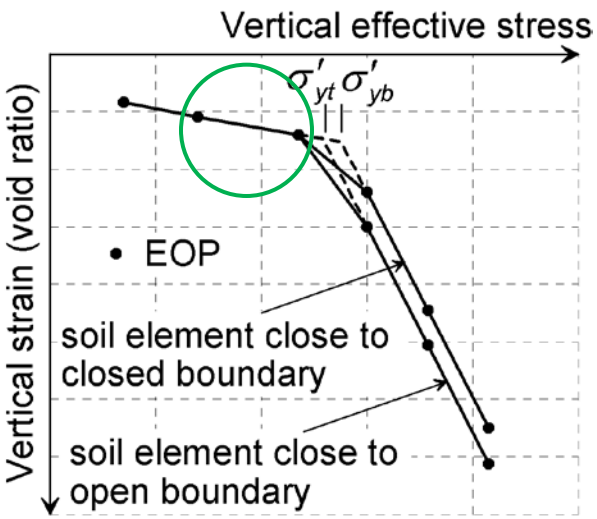
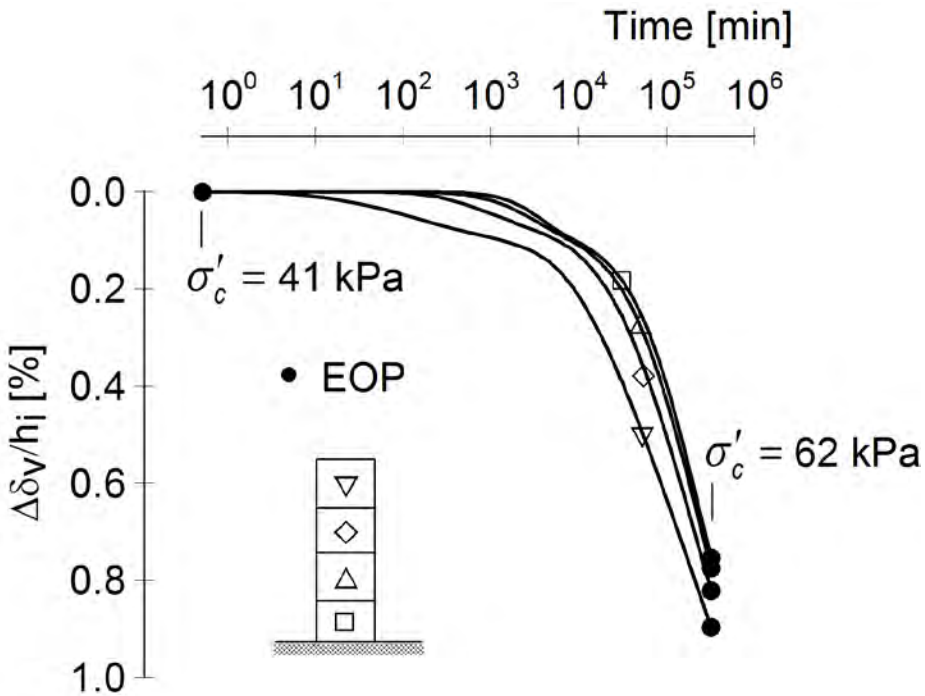


Fig.: Strain-Time and Effective stress–Strain relationships according to hypothesis B

Hypothesis B – stress increment *exceeding* the initial p'_c



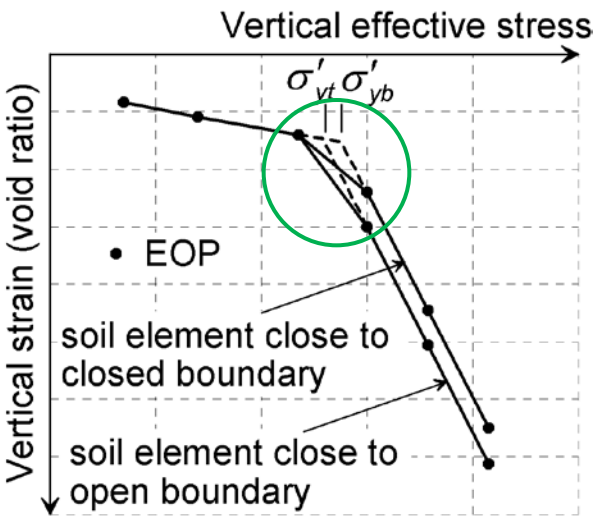
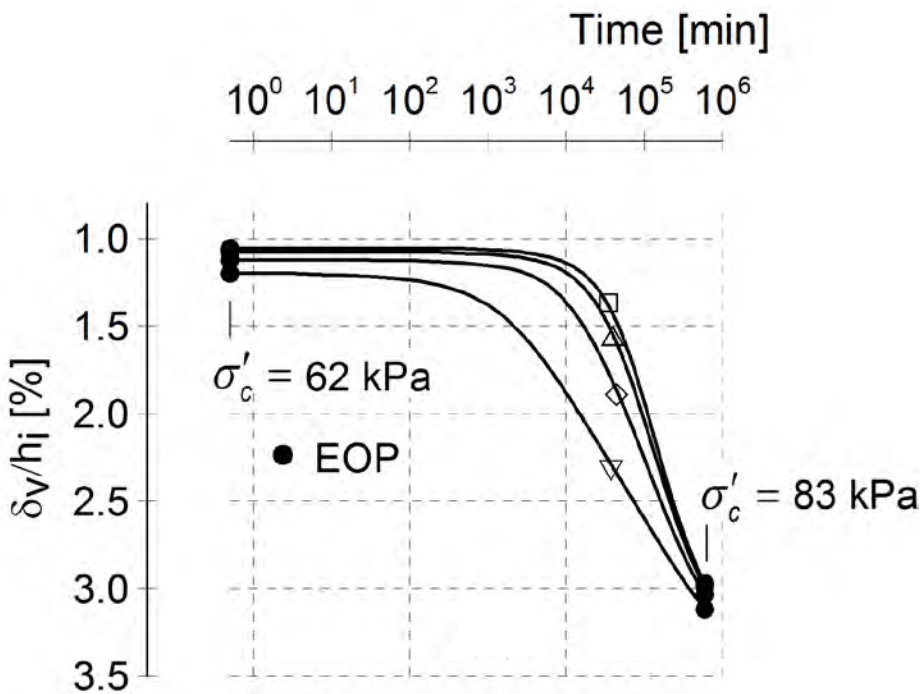


Fig.: Strain-Time and Effective stress–Strain relationships according to hypothesis B

Hypothesis B – stress increment *after exceeding* the initial p'_c



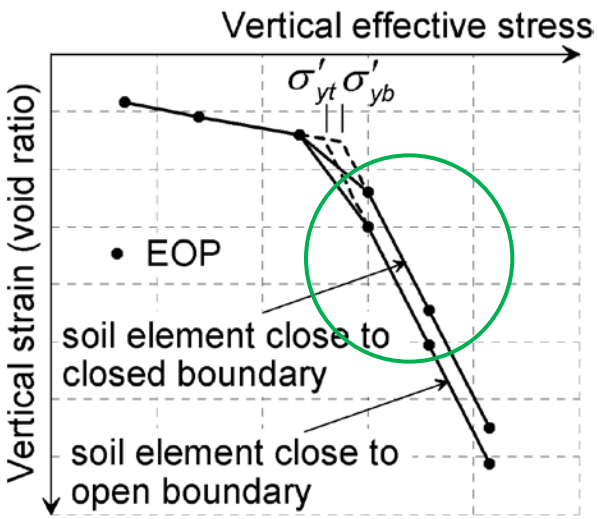


Figure : Strain-Time and Effective stress–Strain relationships according to hypothesis B

Strain-time relationships: laboratory test results

- Hypothesis B

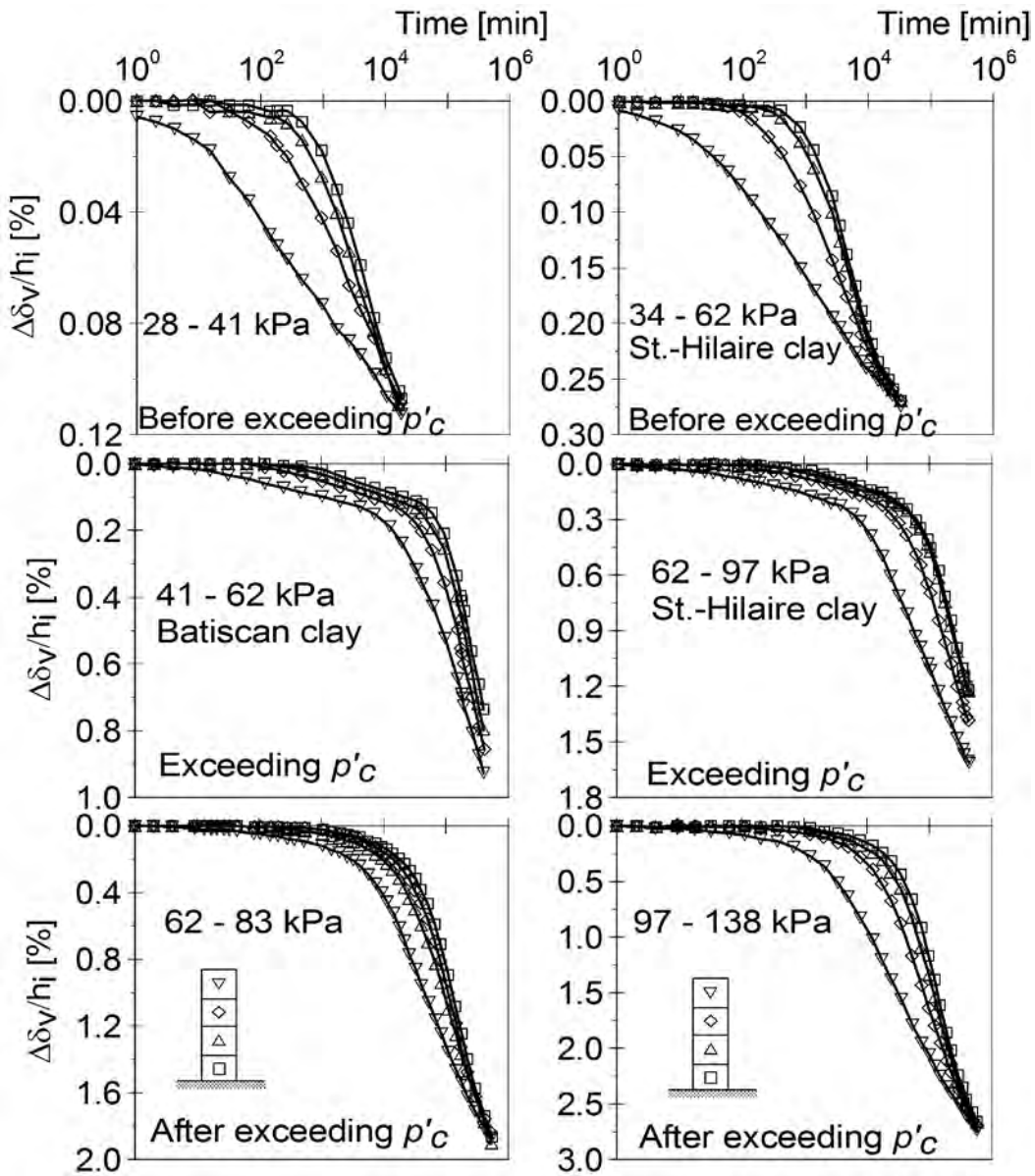


Fig: Experimental results on Batiscan and St. Hilaire clay (Feng, 1991)

- Strain-time relationships: numerical study
- Simulation using hypothesis B (SSC) model
- FE-code PLAXIS

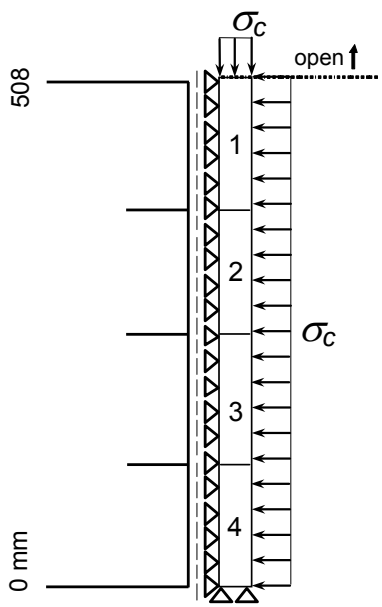


Fig.: Geometry adopted in FE simulation

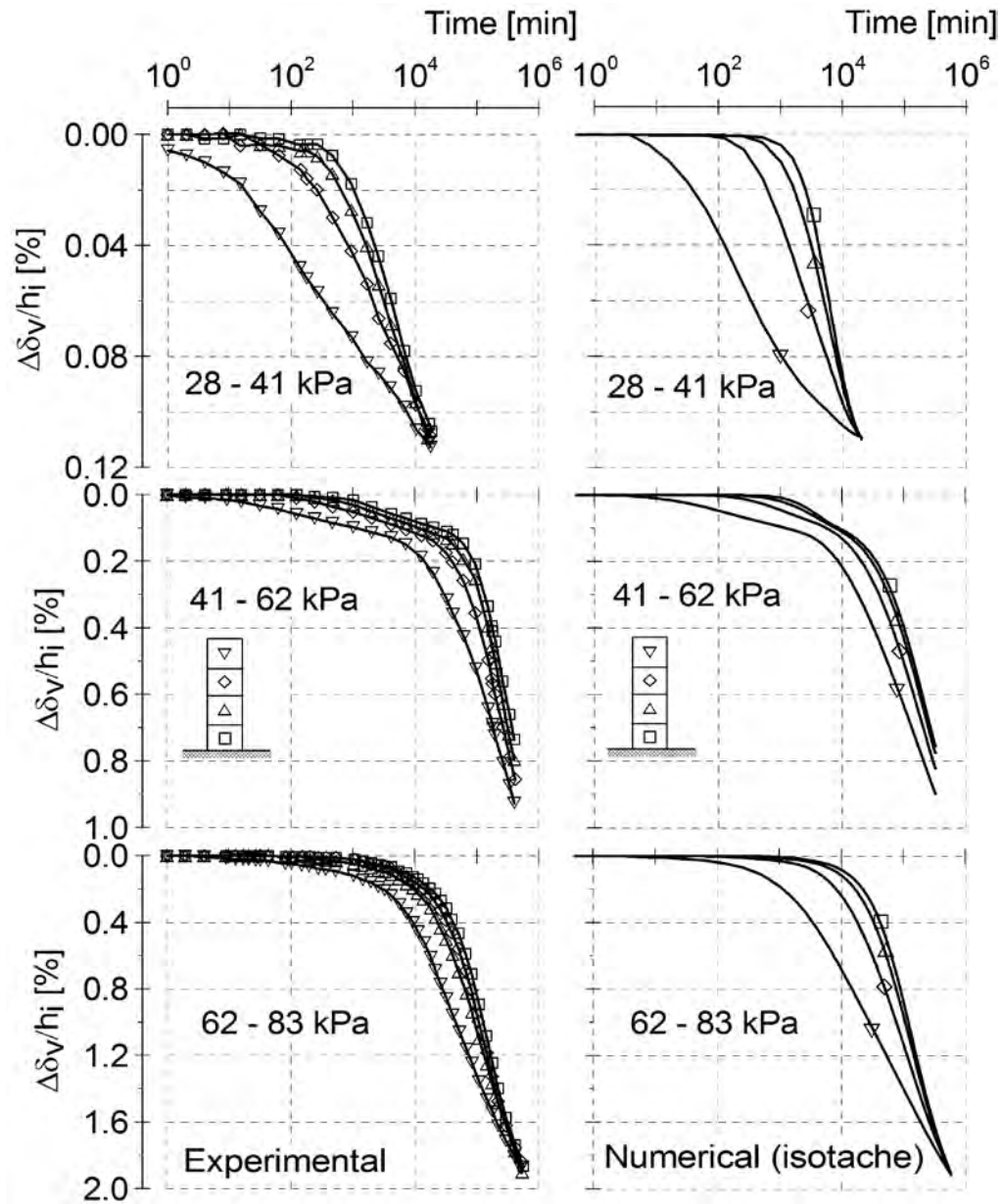


Fig.: Experimental measurements (Feng, 1991) Vs Simulation results of Batiscan clay

Tests conducted during this study
(@Chalmers University of Technology)

Hypothesis A :-

- The sub-layer at the drainage face does not experience any secondary consolidation until EOP state of the bottom sub-layer (Mesri & Vardhanabhuti, 2006).

Motivation

- Will a soil element at the drainage face really 'wait' for the EOP state of the bottom sub-layer to start its secondary consolidation? (Jostad, 2006 @CREBS I)

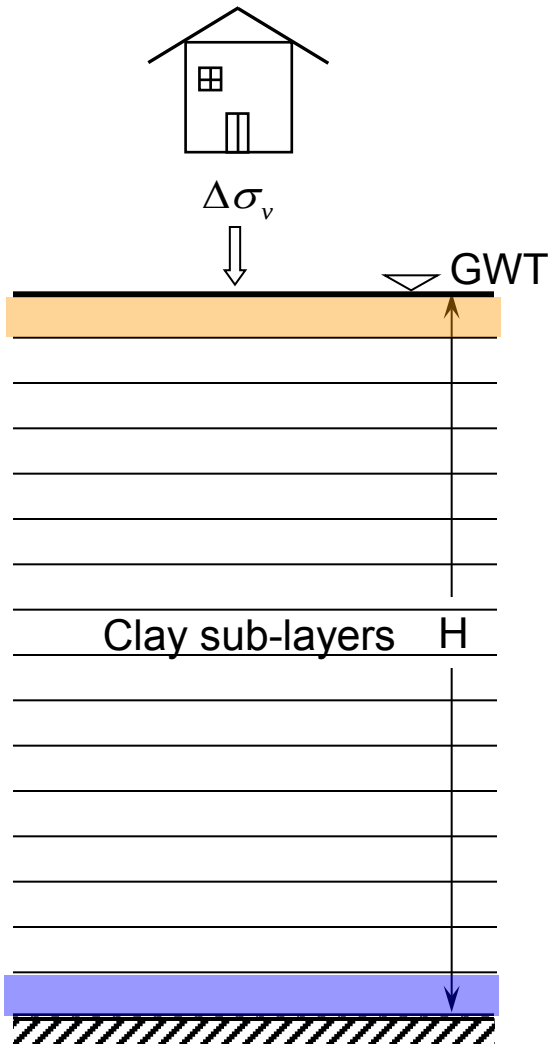


Fig.: A soil layer consisting of several soil sub-layers

An idealized case

- A clay layer placed on top of similar clay as compared to a clay layer placed on top of a soil material with different coefficient of consolidation.

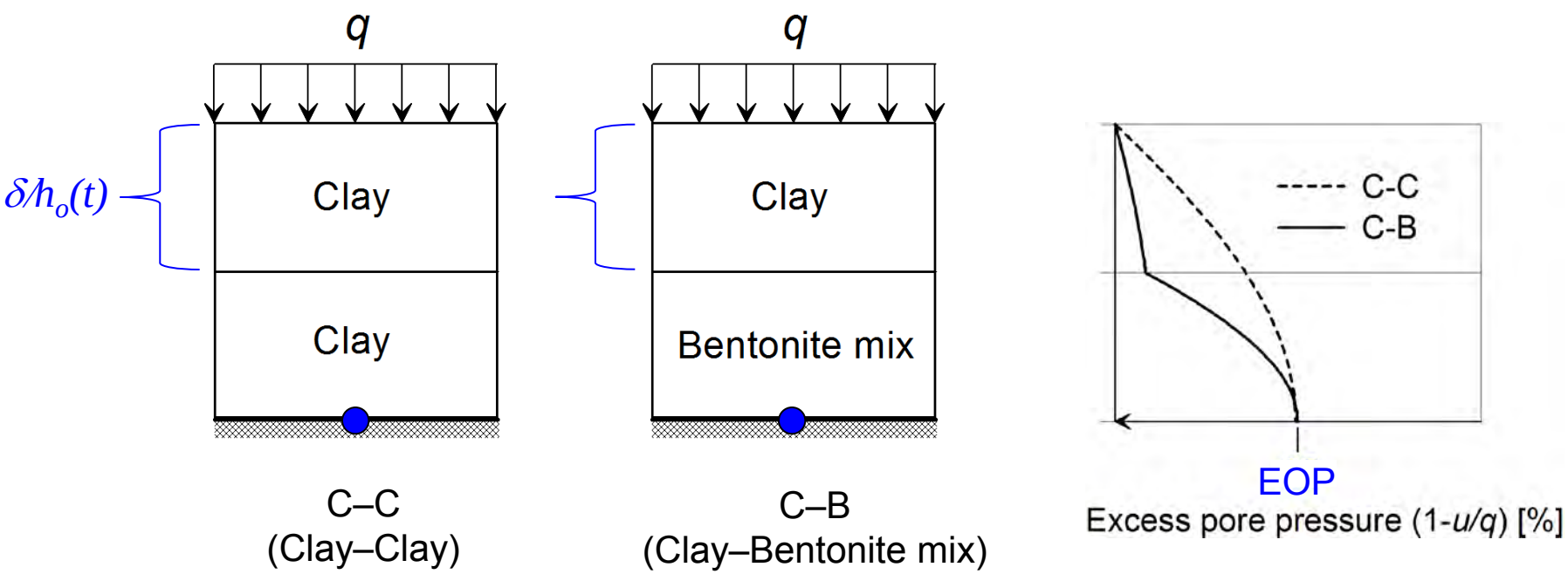
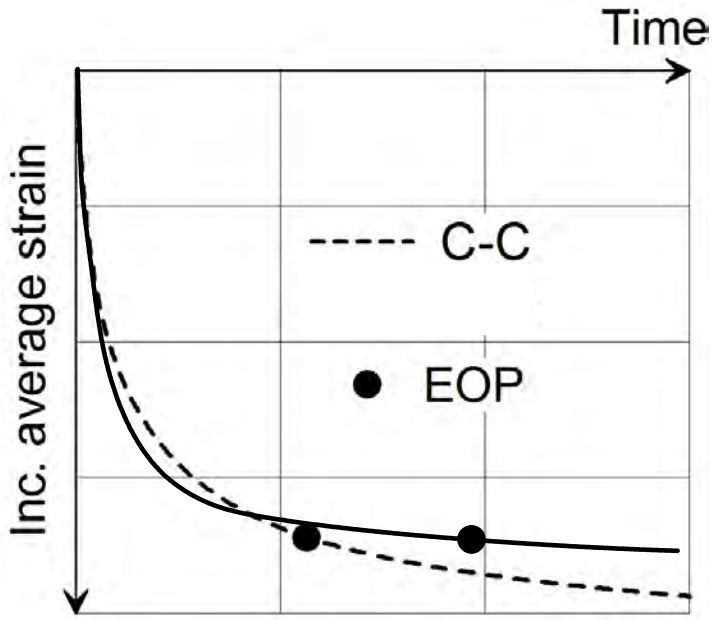
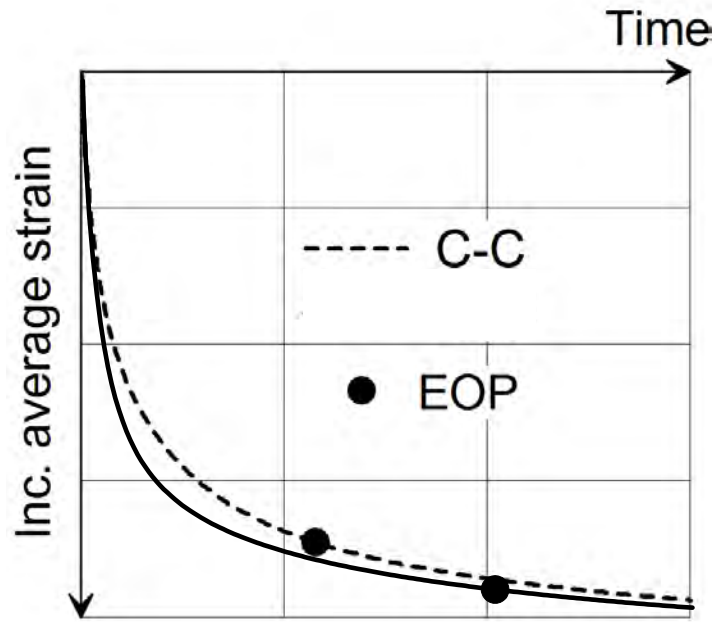


Fig.: Idealized cases

Expected strain-time relationship of the top clay: the creep hypotheses



Hypothesis A



Hypothesis B

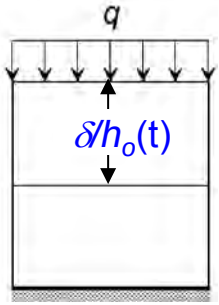


Fig.: Predicted incremental nominal strain-time relationship of the top clay

Test set up and measurements

- Conducted in Chalmers University of Technology
- Incremental load sequence of 10, 20, 30 and 80 kPa (EOP = 95 % EPP dissp.)
- Two sets of tests

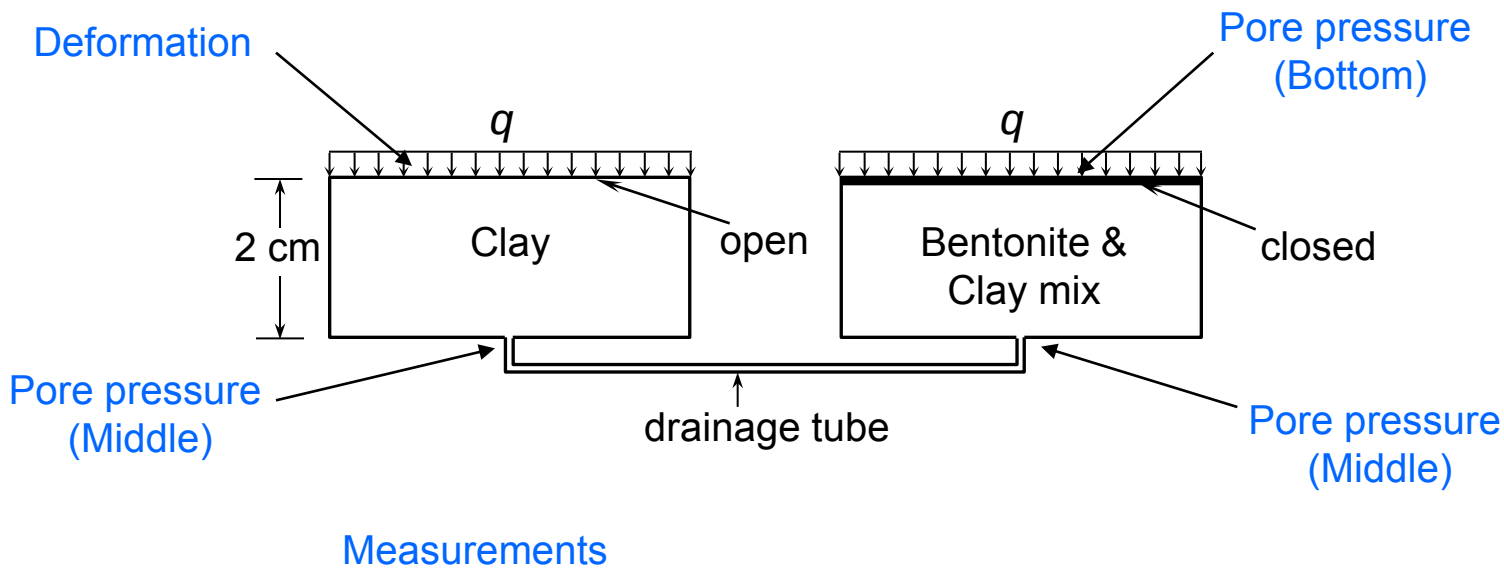
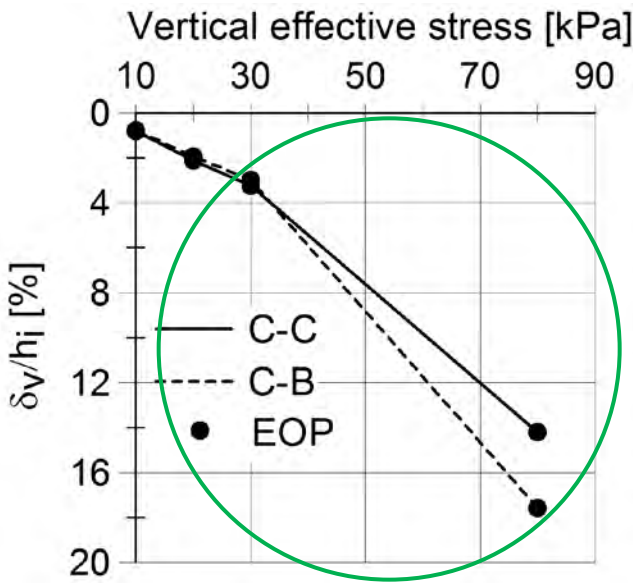
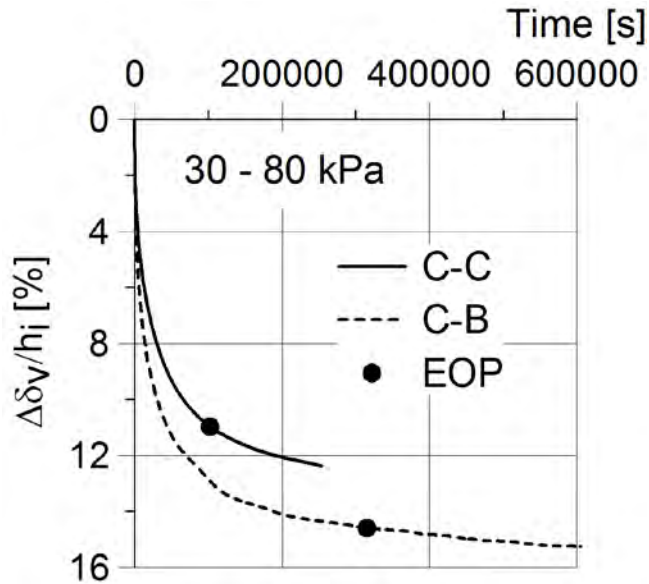


Fig.: Test set up and measurements

Fig.: Running the interconnected tests at Chalmers University of Technology

- Experimental results





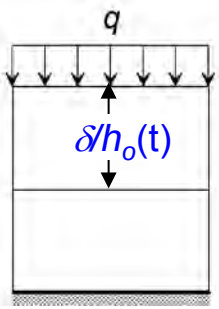


Fig.: Test measurements

- EOP not unique !
- EOP difference is slightly more than expected for hypothesis B

Numerical study

- Simulation using hypothesis B (SSC) model
- FE-code PLAXIS

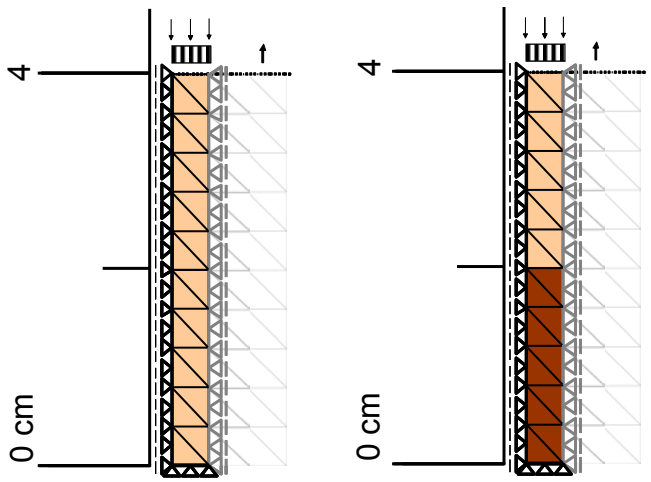


Fig.: Geometry adopted in FE simulation

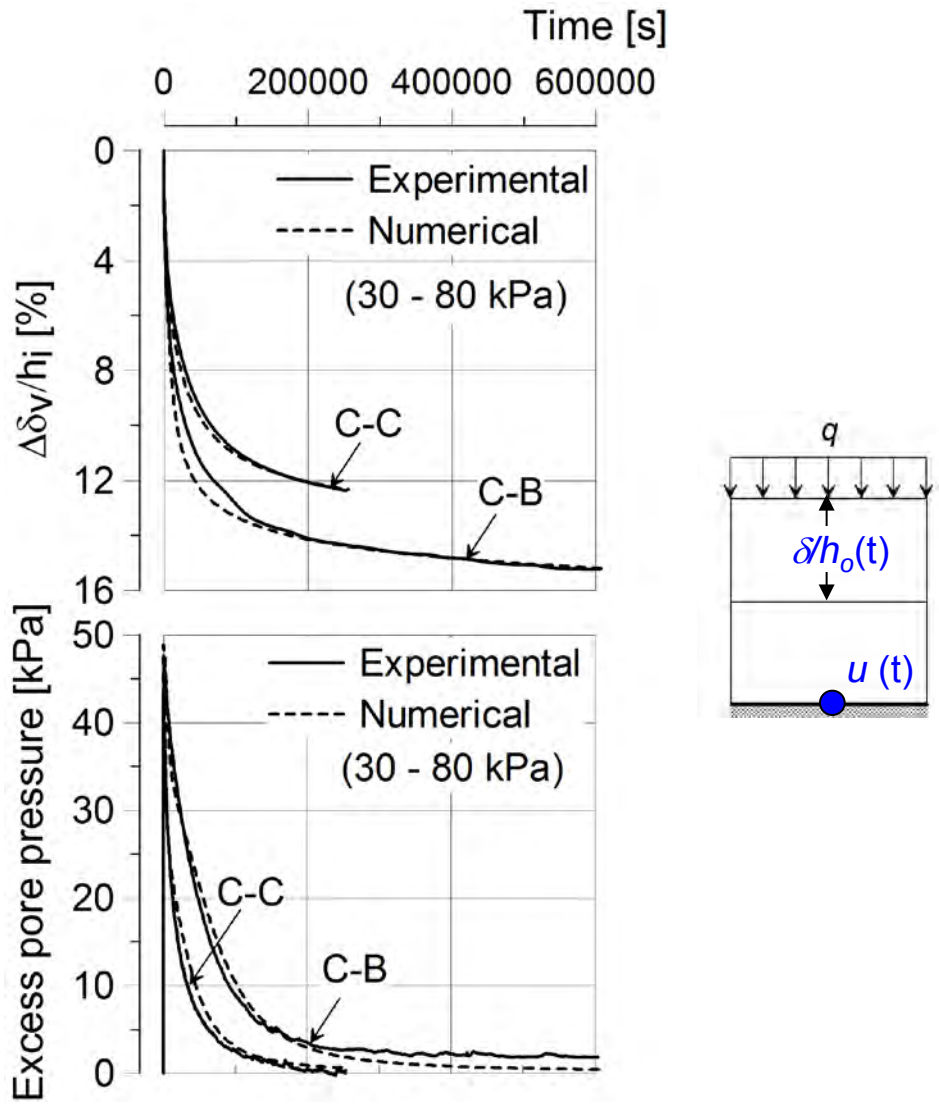


Fig.: Measurements vs. simulation (30-80 kPa)

Laboratory studies II: Creep hypotheses for soil element compressibility

Final remarks

- ✓ Laboratory studies on soil element compressibility imply hypothesis B.
  - Local compressibility of a soil element is governed by its prevailing effective stress-strain-strain rate on that particular soil element rather than what is happening elsewhere in the soil layer.
  - This means that a soil element creeps during primary consolidation and starts its secondary consolidation phase right after its primary consolidation phase rather than 'wait' until the completion of the primary consolidation of all the other soil elements
- ✓ Numerical simulation results using hypothesis B model can explain experimental measurements.

Field studies

- The two hypotheses could give significant practical differences when predicting settlements of in-situ soil layers
- However, on several occasions, the advocates of the two hypotheses have *independently* presented acceptable predictions of in-situ settlements to support the hypotheses.
- In this study, the *constitutive models* for the two hypotheses are evaluated based on the performance of a *common* and well-documented test fill.
- This is mainly motivated by the hypothetical case exercises given to CREBS II participants by Jostad in 2007.
- Constitutive models for hypothesis A (ILLICON), hypothesis B (SSC) and a rate-independent elasto-plastic model (SS) are considered.

Model comparisons – Strain formulations

- ILLICON strain decomposition

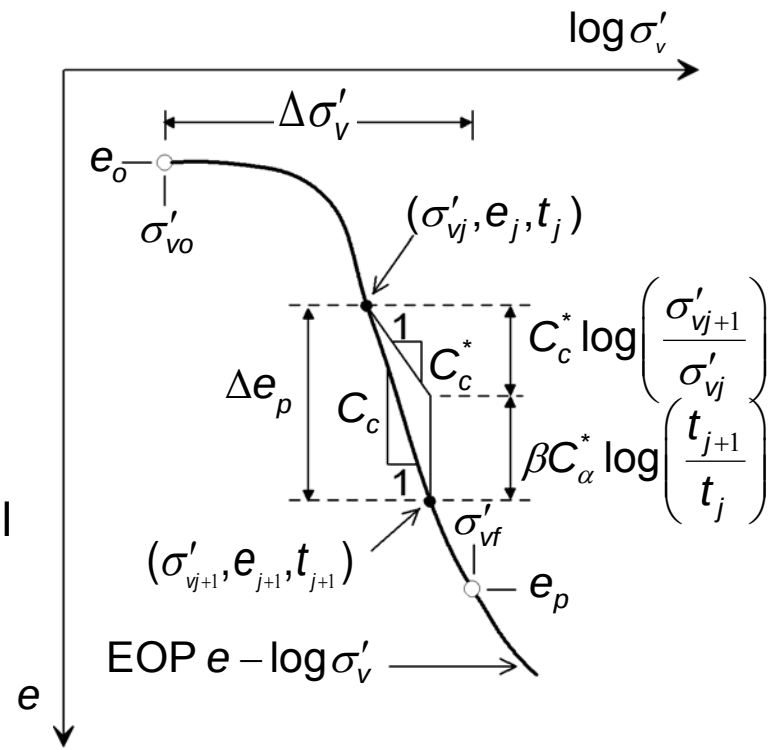
$$\Delta e_p = C_c^* \Delta \log \sigma_v' + \beta C_\alpha^* \Delta \log t$$

where C_α^* merely decomposes the input and out put Δe_p into two 'arbitrary' parts.

- SS is a rate-independent elasto-plastic model
- SSC is a rate-dependent elasto-viscoplastic model

- ILLICON is equivalent to SS model.

- The SSC would give larger EOP strain than both ILLICON and SS models.



$$C_\alpha^* / C_c^* = C_\alpha / C_c$$

Fig.: ILLICON strain formulations (after Choi, 1982)

Model comparisons – Excess pore pressure formulations

- Continuity equation as used in ILLICON assumes that the excess pore pressure dissipation is only affected by the so-called stress-compressibility.

$$\frac{(1+e_o)^2}{\gamma_w}\frac{\partial}{\partial z}\left(\frac{k_v}{1+e}\frac{\partial u}{\partial z}\right)=\frac{de_\sigma}{dt}\neq\frac{de}{dt}\left(=\frac{de_\sigma}{dt}+\frac{de_t}{dt}\right)$$

- In SSC and SS model the continuity equation is controlled by total strain rate.
- ILLICON would give faster EPP dissipation than SS model.
- SSC would give significantly slower EPP dissipation than both ILLICON and SS model.

Comparison of the models based on analysis of Väsby test fill

- ILLICON vs. SS
- SSC vs. SS

- ILLICON, SSC and SS models are indirectly compared based on analysis of the test fills.
- For a given set of soil data, the SS model is used in order to provide reference predictions with respect to disregarding the effect of creep.

Comparison of ILLICON vs. SS model

- Similar boundary and loading conditions, i.e. 1D condition, Boussinesq stress distribution and no buoyancy effect
- "ILLICON-Equivalent" parameters were adopted for SS model.

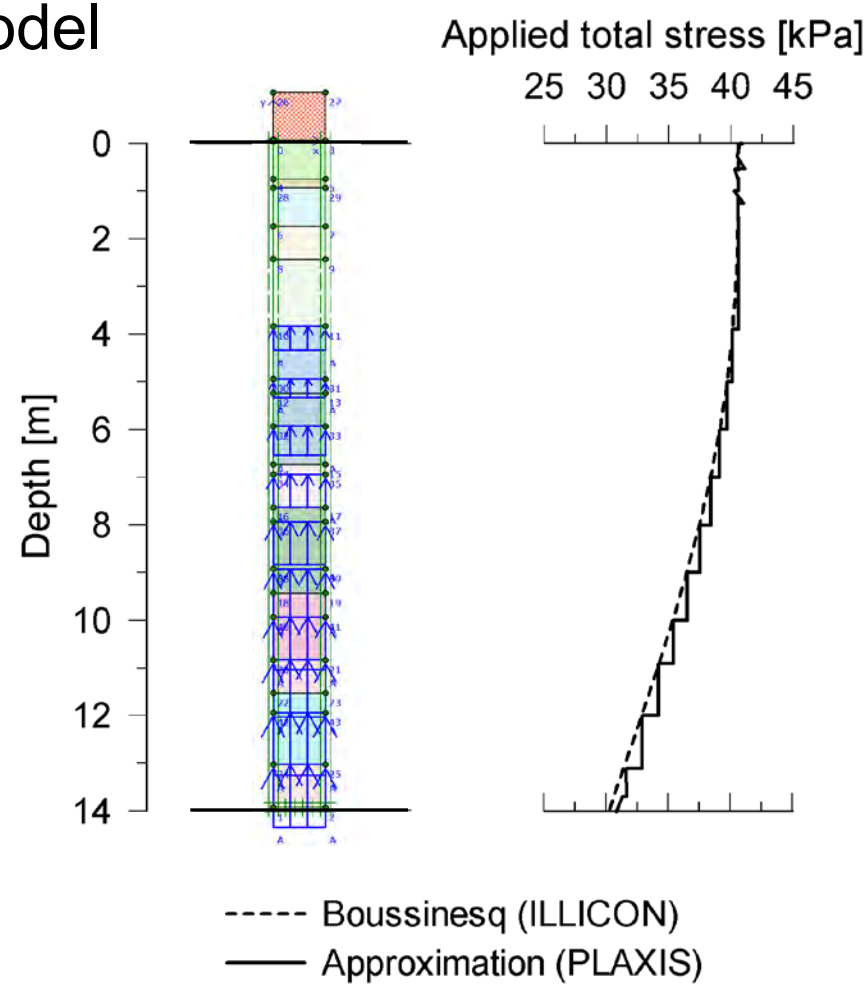


Fig.: Equivalent load distribution as adopted for ILLICON and SS (PLAXIS).

Analyses results ILLICON & SS – Väsby test fill

- "ILLICON-Equivalent" parameters were adopted for SS model.

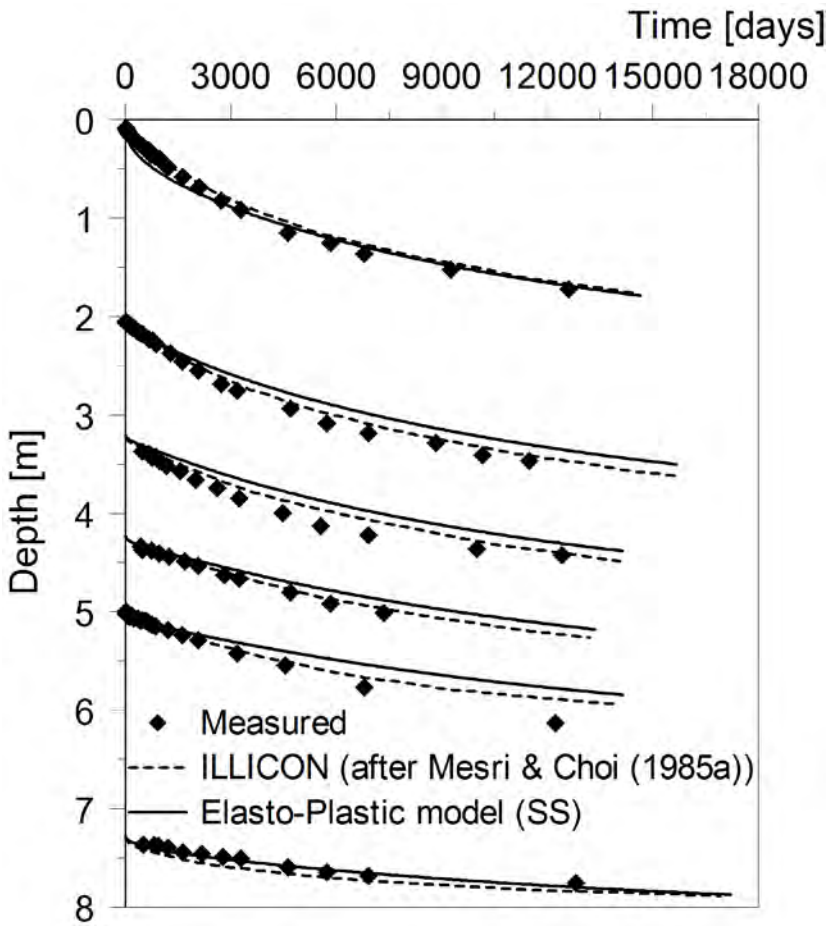


Fig.: Settlement history predictions (ILLICON vs. SS)

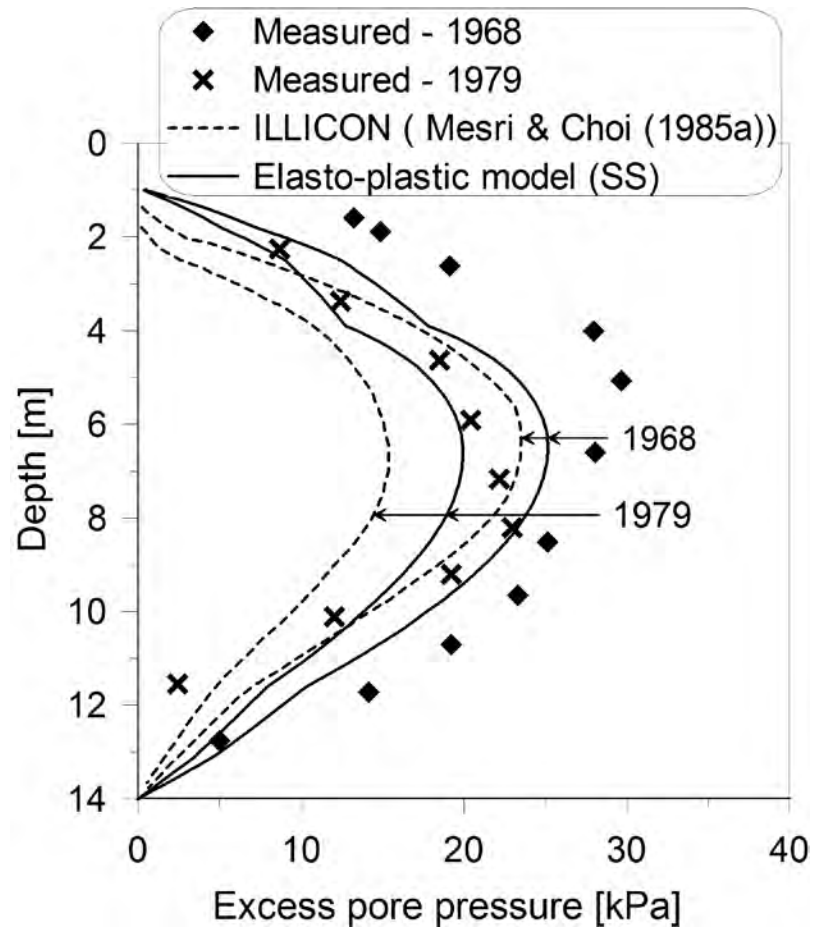


Fig.: Excess pore pressure profile predictions (ILLICON vs. SS)

ILLICON and SS model predictions vs. Measurements

- As expected, ILLICON and the SS models are practically similar and this could imply that effect of creep in the ILLICON model is negligible.
- Still, while disregarding creep, both ILLICON and SS model gave an overall acceptable predictions.
- This should not imply that the soft clays considered do not undergo creep deformation.
- The acceptable predictions were mainly due to two factors, i.e. use of soil data from disturbed samples and disregarding effect of large deformations.

(1) Sample disturbance

- Generally the OCR values used in ILLICON and SS analysis were low and are believed to be affected by sample disturbance.

- For instance,
  - Väsby test fill, EOP OCR = 1.31 or 1.82 ? (Leroueil and Kabbaj (1987))

  - In Skå-Edeby test fill, OCR = 1.0 ? (field tests by SGI)

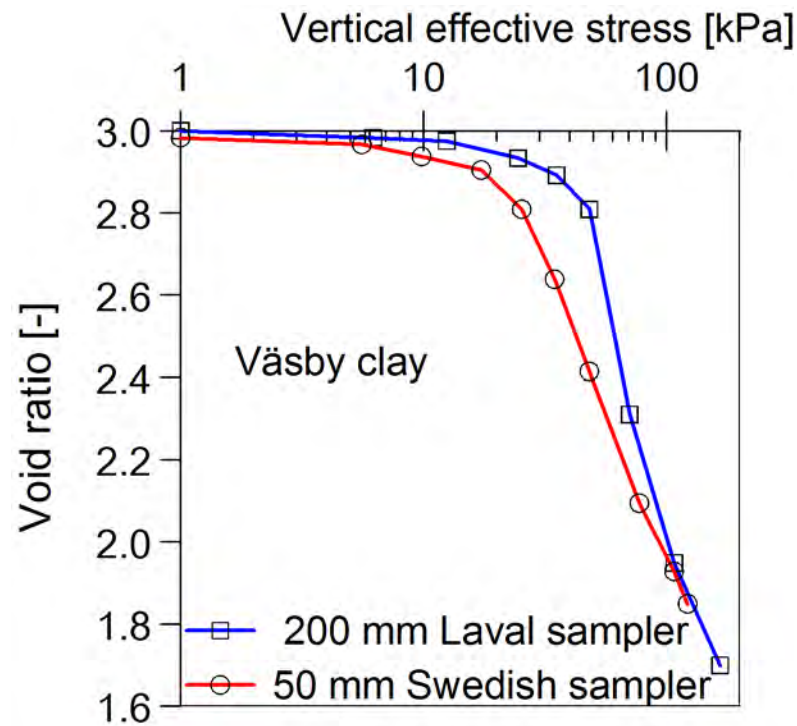


Fig. : Sample disturbance at Väsby test fill (after Leroueil & Kabbaj, 1987)

(2) Effect of large deformations (buoyancy)

- ILLICON and SS model analyses disregarded load reduction due to buoyancy forces.

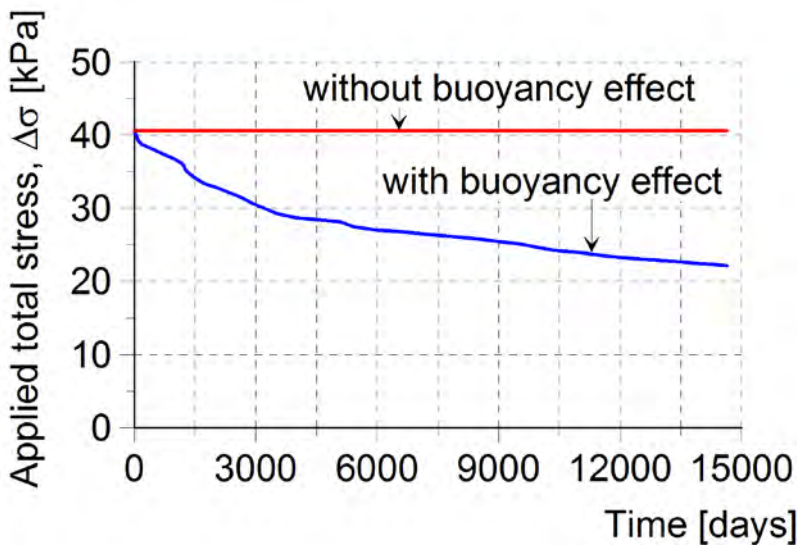


Fig. : Applied load with and without consideration of buoyancy effect (Väsby)

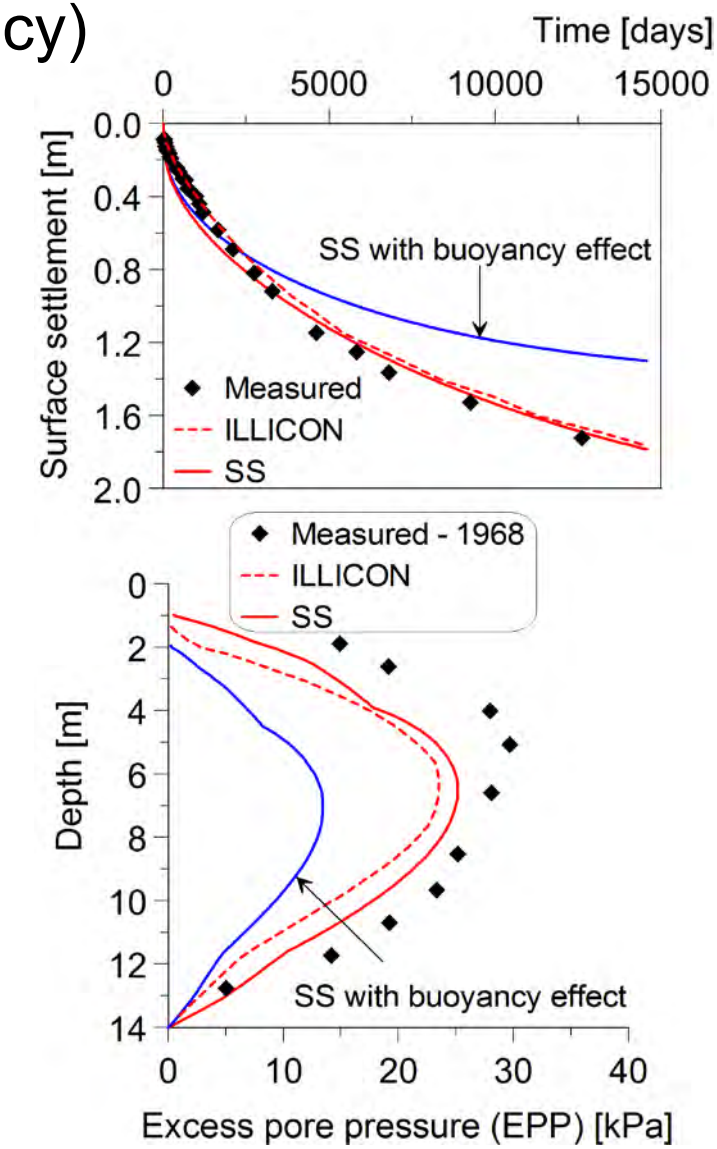


Fig.: Effect of buoyancy on predictions

Comparison of SSC vs. SS model

- ✓ Use of OCR values from high quality sample data or clay age considerations
- ✓ Effect of large deformation (buoyancy) taken into account

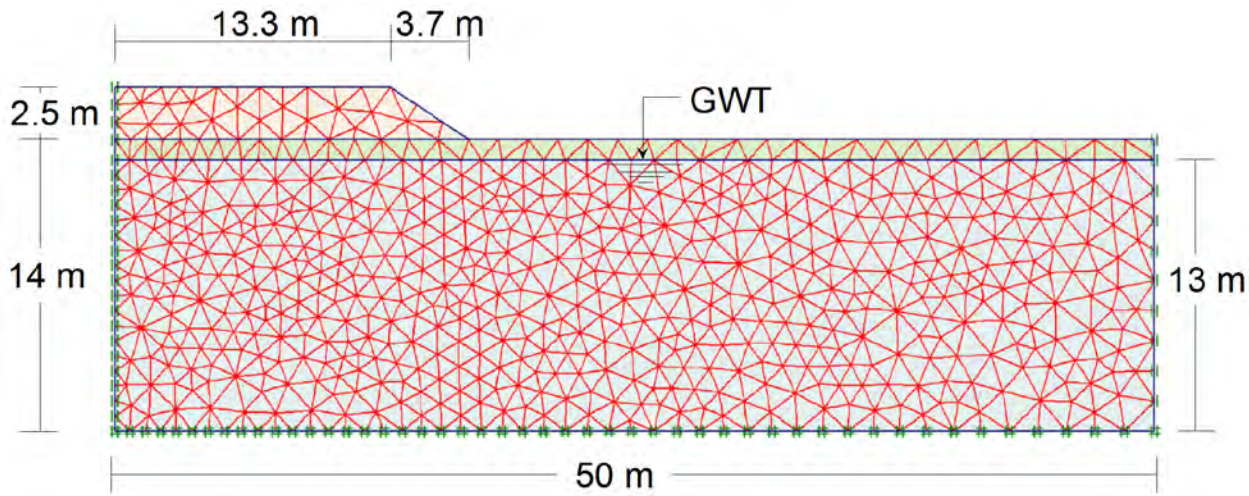


Fig.: Axisymmetric FE geometry adopted for Väsby test fill analysis

Analyses results SSC & SS – Väsby test fill

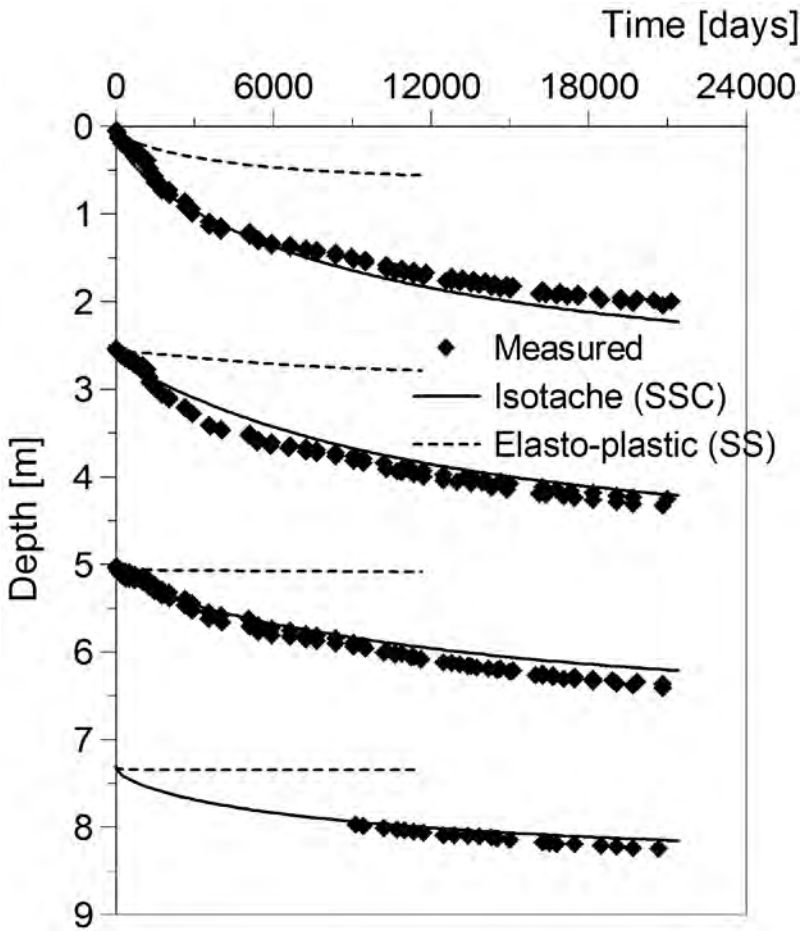


Fig.: Settlement history predictions (SSC vs. SS)

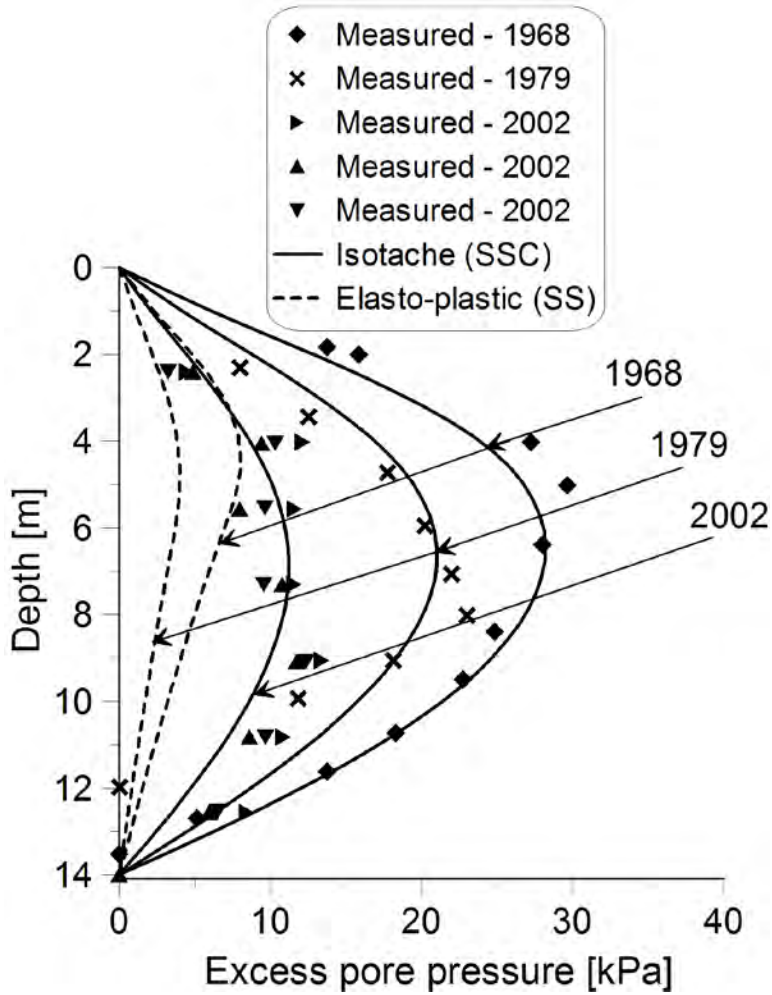


Fig.: Excess pore pressure profile predictions (SSC vs. SS)

Field studies

Final remarks (based on Väsby, Skå-Edeby & Ellingsrud test fills)

- ✓ When soil data are interpreted from tests on disturbed samples are used for settlement analysis then some effect of creep is already 'incorporated'.
  - A rate-independent elasto-plastic model, along with some simplifying assumption, could give acceptable settlement and reasonable but somehow low excess pore pressure responses.
  - An isotache model would significantly overestimate settlement and could give unrealistically large excess pore pressure responses.
- ✓ When soil data are interpreted from tests on high quality samples and used for settlement analysis,
  - A rate-independent elasto-plastic model significantly underestimates settlement and excess pore pressure responses
  - An isotache model would yield excellent prediction of settlements and excess pore pressure.

Conclusions

- In response to the important question raised by Ladd *et al*. in 1977, this study has shown that there exist definitive data to demonstrate that hypothesis B agrees very well with the measured behaviour of cohesive soils.

- Several EOP laboratory tests considered in this study demonstrated the validity of hypothesis B. In fact, this study disclosed that the empirical data that were previously used to support substantiate hypothesis A actually imply hypothesis B.

- The experienced p'_c as well as EOP strain are rate dependent even for EOP loading conditions and this fact has been experimentally supported by several EOP tests and field observations.

- The isotache theory (hypothesis B (SSC)) can explain and convincingly capture important feature of various types of laboratory tests considered in this study.

Conclusions

- Great care needs to be exercised during interpretation and use of relevant soil parameters in settlement analyses.
- With this aspect, sample quality deserves extra attention. This (sample quality) is as important as modelling an entire problem at hand, if not more.
- Awareness regarding the significance of p'_c (OCR due to creep) on settlement analysis needs to be stressed by the profession.
- The isotache models are well suited to predict settlements of water saturated soft clay deposits when the input data are deduced from laboratory tests of good quality soil samples.
- Future developments related to the compressibility of natural clays such as anisotropy and destructuration should be focused on enhancing models that are based on the isotache framework or similar.

Outlook

- It is high time that the practice starts to benefit from research and the level of understanding achieved so far in creep behaviour of soils.
- Thus, *creeping creep into the practice* should simultaneously be emphasised along with the ongoing R&D activities.
- The Norwegian Public Roads Administration (Statens vegvesen) is currently revising its guideline (Handbook V220 (Hb 016)).The settlement calculation chapter will be subjected to a major update/upgrade.
- In this revision, important aspects of creep and main underlying concepts will be introduced with a room for future improvements.
- With this aspect, results of these creep workshops and the ongoing activities/study will be crucial.

Thank you for your attention !

Statens vegvesen

Norwegian Public Roads Administration

1D creep behaviour

Hans Petter Jostad

Discipline leader in numerical modeling at NGI

Adjunct Professor at NTNU

CREEP Workshop: Focus on Pratical Applications

NGI, Oslo, Norway, 8. January 2015

NGI

Creep strains

Stress and time dependent strains

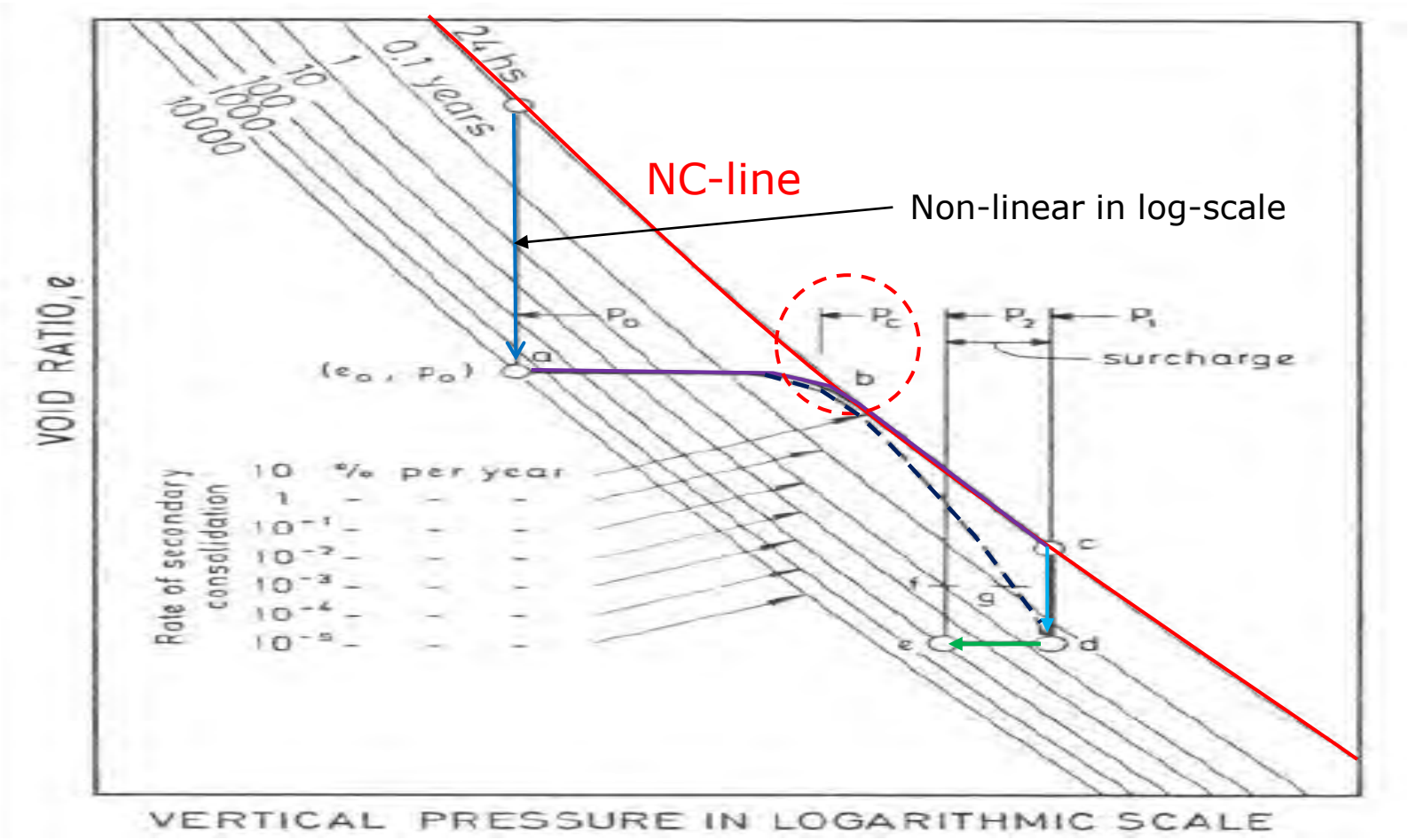
$$\frac{d\varepsilon}{dt} = \left(\frac{\partial \varepsilon}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial \varepsilon}{\partial t}\right)_{\sigma'}$$

stress induced creep

$$\Delta\varepsilon = \int_0^{t_p} \left[\left(\frac{\partial \varepsilon}{\partial \sigma'}\right)_t \frac{d\sigma'}{dt} + \left(\frac{\partial \varepsilon}{\partial t}\right)_{\sigma'}\right] dt + \int_{t_p}^{t} \left[\left(\frac{\partial \varepsilon}{\partial t}\right)_{\sigma'}\right] dt$$

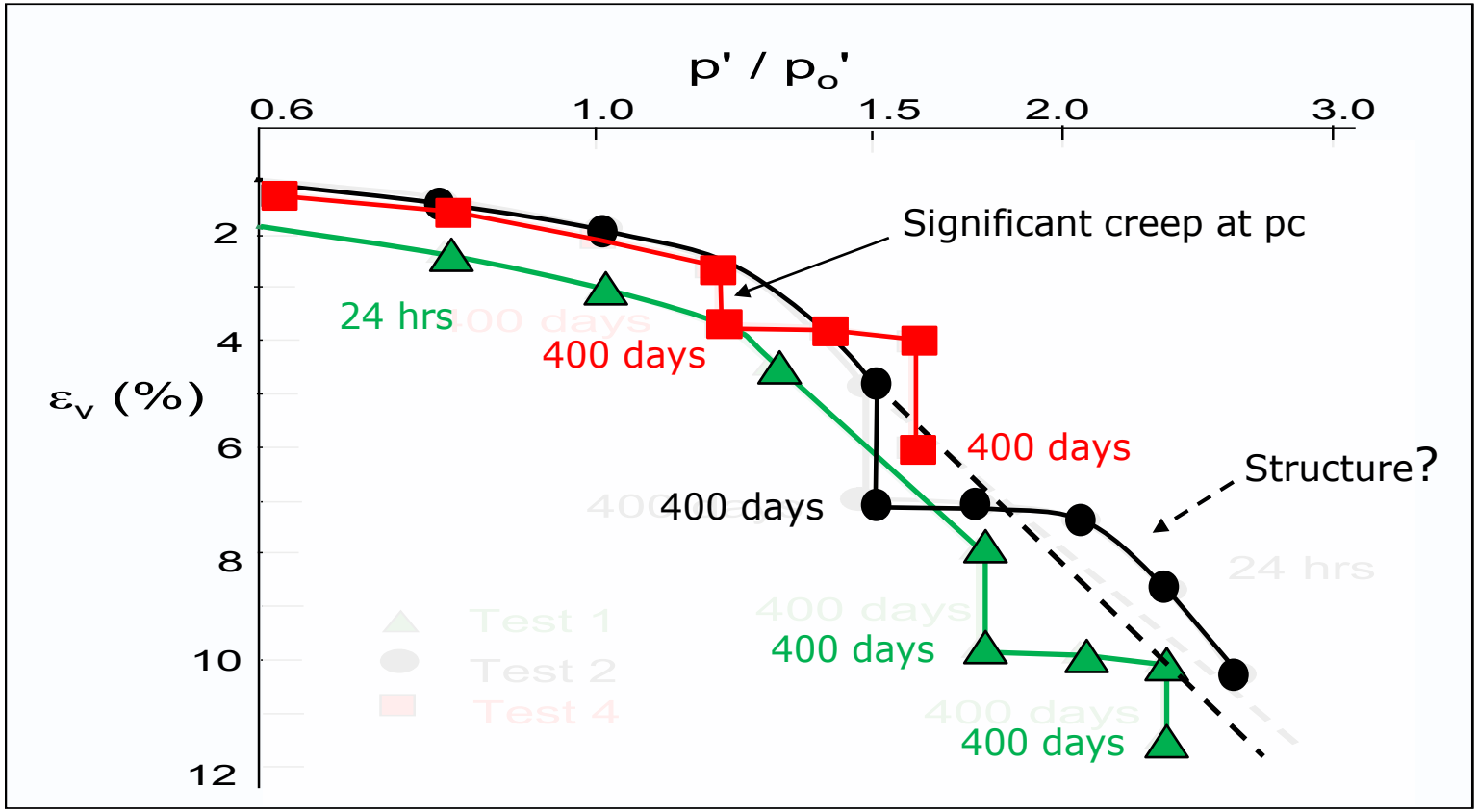
Primary (consolidation) Secondary (creep)

Bjerrum's delayed compression concept

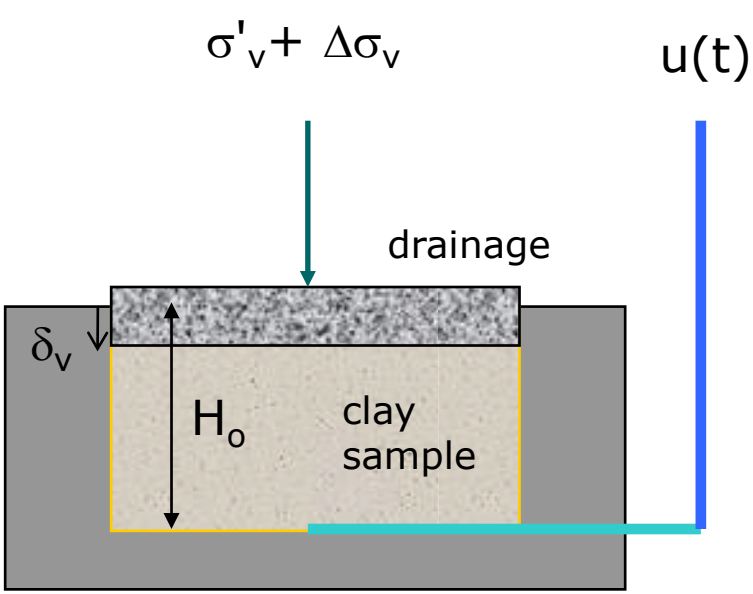


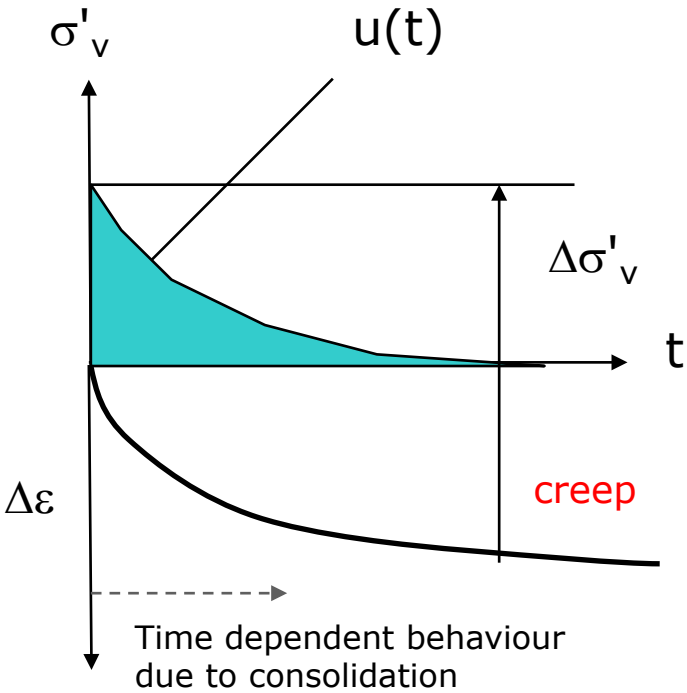
Unique relationship between creep rate, effective (vertical) stress and void ratio

Validation of the concept



Oedometer test (Incremental Loading Tests)

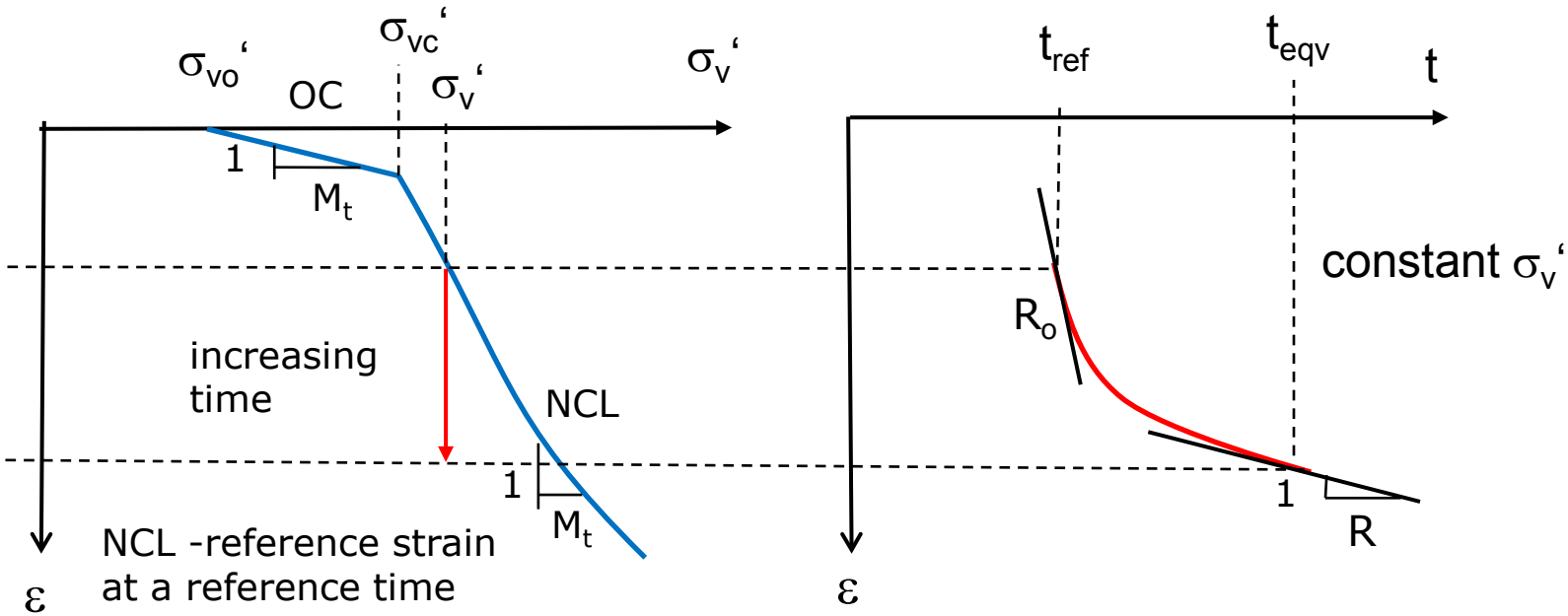




Need to separate into contributions
from effective stress changes and creep:

- Creep "starts" when u ~ 0 or after 1 day (as a reference)?
- Or, all plastic strains are time dependent (Soft Soil Creep Model)

Janbu's resistance concept (EP+VP)



$$\frac{d\varepsilon_v}{dt} = \frac{1}{M_t}\frac{d\sigma_v'}{dt} + \frac{1}{R}$$

$M_t = f(\sigma_v')$

$R = f(\sigma_v', \varepsilon)$

Need these relationships!

Creep is added to the elasto-plastic strains

Janbu's time resistance

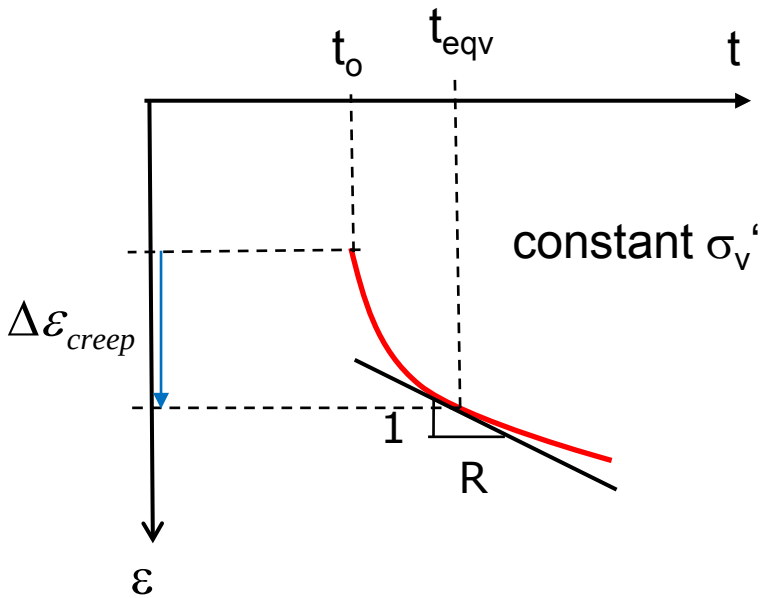
$$R = R_o + r\,(t - t_o)$$

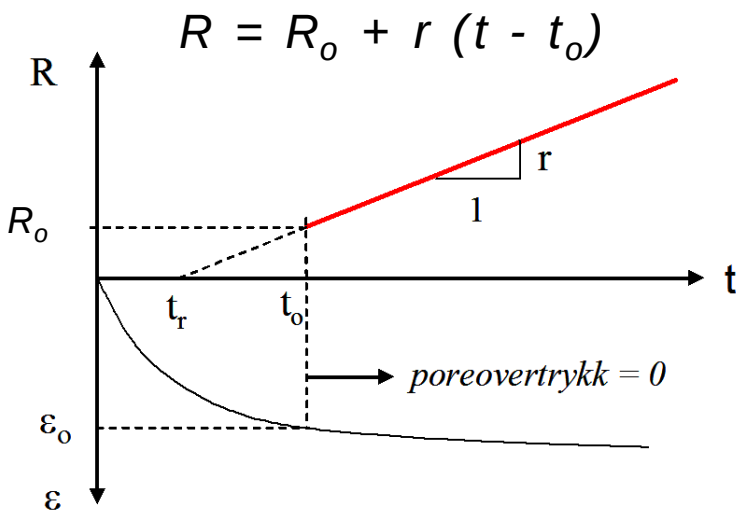
$$\Delta\varepsilon_{creep} = \frac{1}{r}\ln\left(\frac{t}{t_o}\right) = \frac{1}{r}\ln\left(\frac{R}{R_o}\right) \longrightarrow t_{eqv}$$

$$\Delta\varepsilon_{creep} = \varepsilon_{tot} - \varepsilon_o(\sigma_v')$$

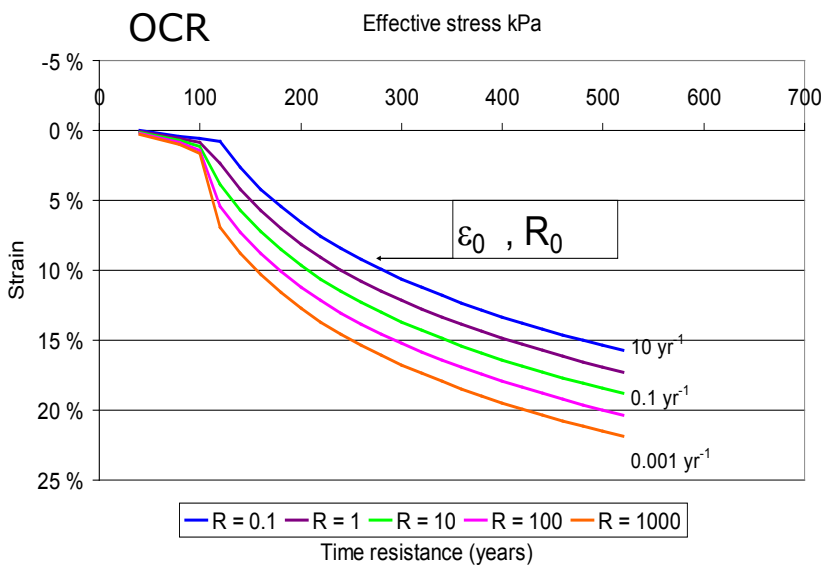
$$R = R_o e^{\left(r\,\Delta\varepsilon_{creep}\right)}$$

The accumulate creep strain is the state parameter for creep rate



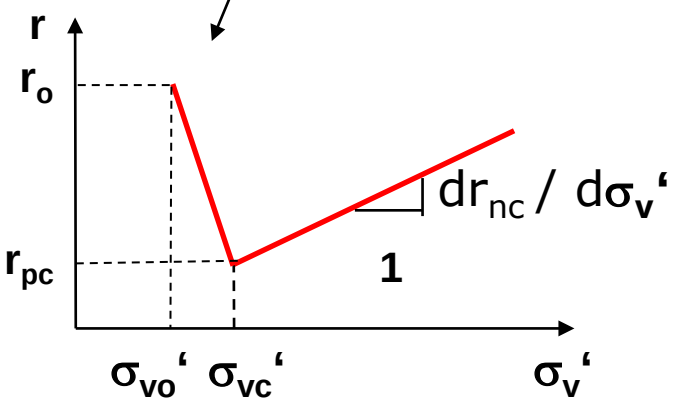


Effective stress dependency



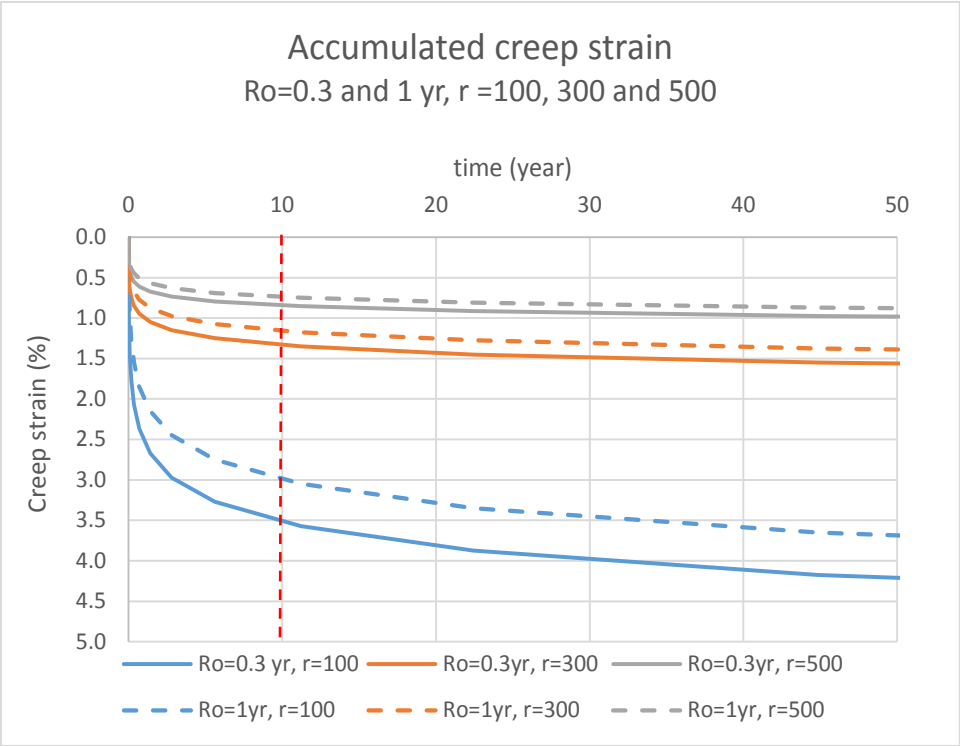
Stress or OCR dependent?

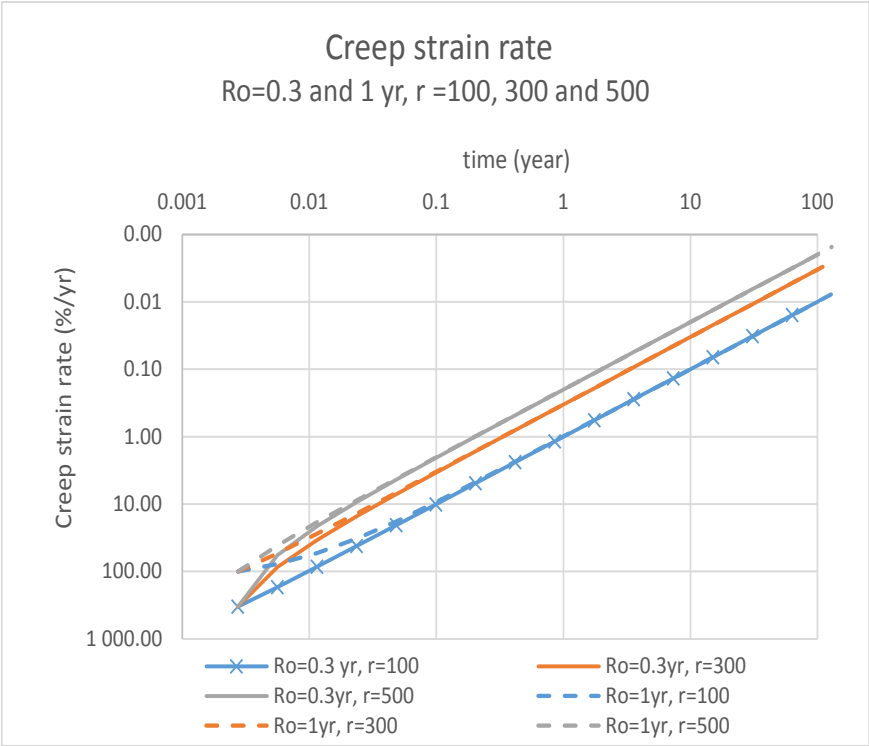
$$R = R_o + r\,(t - t_o)$$



Example

Ro = 0.3 and 1 year, r = 100, 300 and 500

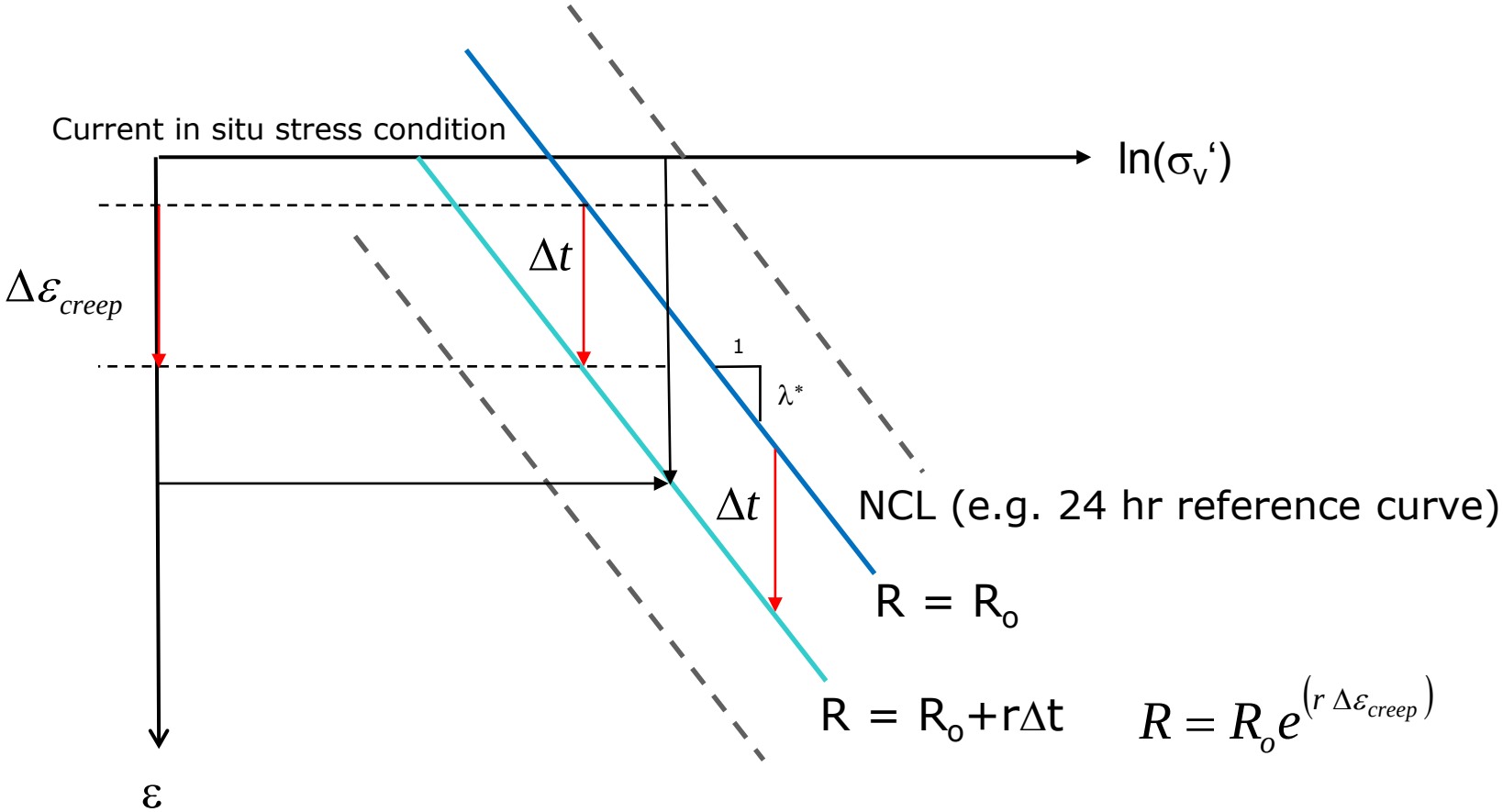




A large contribution of creep may occur during primary consolidation

Isotaches – lines of constant (creep) strain rate

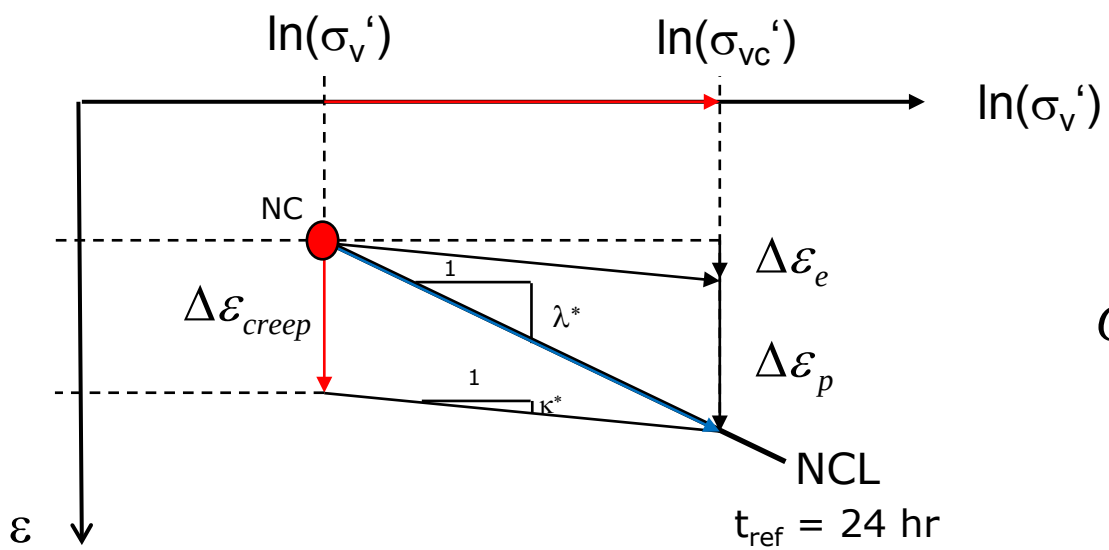
unique relationship between effective stress – strain (void ratio) – strain rate



These curves may be non-linear (curved)!

“Apparent” pre-consolidation pressure

Plaxis - Soft Soil Creep Model (E+EVP)



$$OCR = \left(\frac{\sigma_{vc}'}{\sigma_v'}\right)$$

$$r = \frac{1}{\mu^*}$$

$$\Delta\varepsilon_{creep} = \frac{1}{r}\ln\left(\frac{R}{R_o}\right) = \mu^*\ln\left(\frac{R}{R_o}\right) = (\lambda^* - \kappa^*)\ln\left(\frac{\sigma_{vc}'}{\sigma_v'}\right) = (\lambda^* - \kappa^*)\ln(OCR)$$

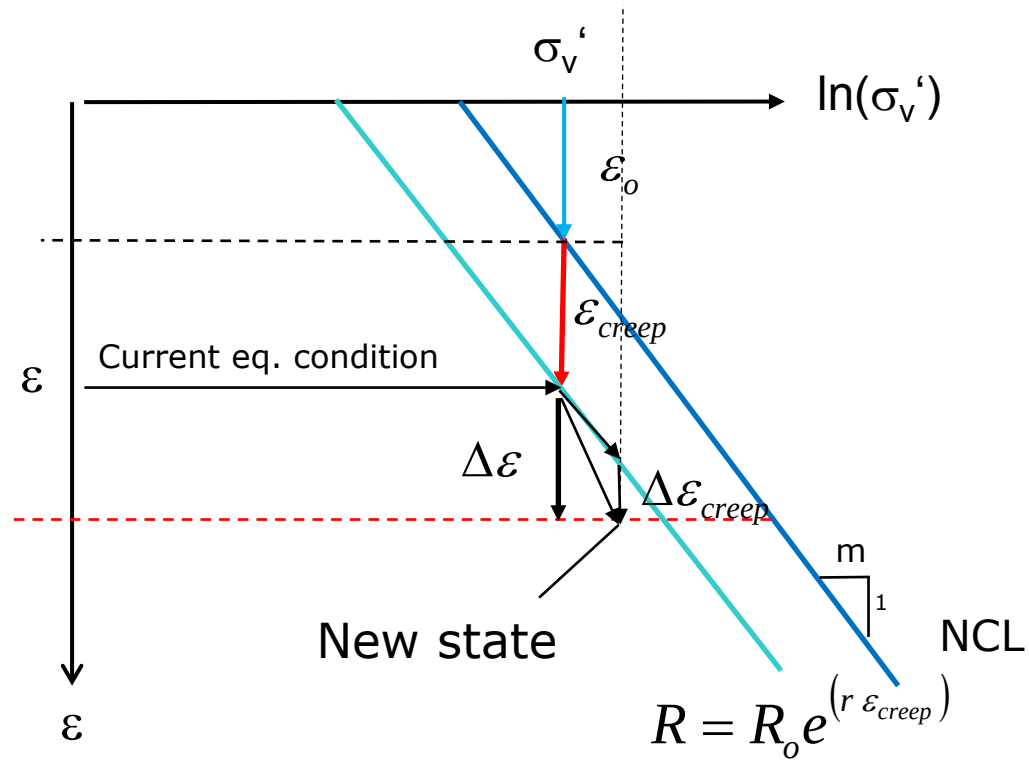
$$R = R_o e^{\left(r\,\Delta\varepsilon_{creep}\right)} = R_o OCR^{\frac{\lambda^* - \kappa^*}{\mu^*}} = \frac{t_{ref}}{\mu^*} OCR^{\frac{\lambda^* - \kappa^*}{\mu^*}}$$

Solution algorithm – FE program

Input:
- Current equilibrium state:
  - σ_v' and ε
- New increment:
  - $\Delta\varepsilon$ (predictor)
  - Δt

Output:
- New stress state:
  - σ_v'

Calculations:
- $\varepsilon_o = f(\sigma_v')$
  - $\varepsilon_{creep} = \varepsilon - \varepsilon_o$
- $R = f(\varepsilon_{creep})$ Account for non-linear behaviour of R
  - $\Delta\varepsilon_{creep} = \Delta t / R$
  - $\Delta\sigma_v' = M_t(\sigma_v') \cdot (\Delta\varepsilon - \Delta\varepsilon_{creep})$



Global equilibrium iteration!

Soil Investigation

Soil profile from e.g. CPTU and location of depth to bedrock (or a stiff layer)

In-situ pore pressure measurements (piezometers)

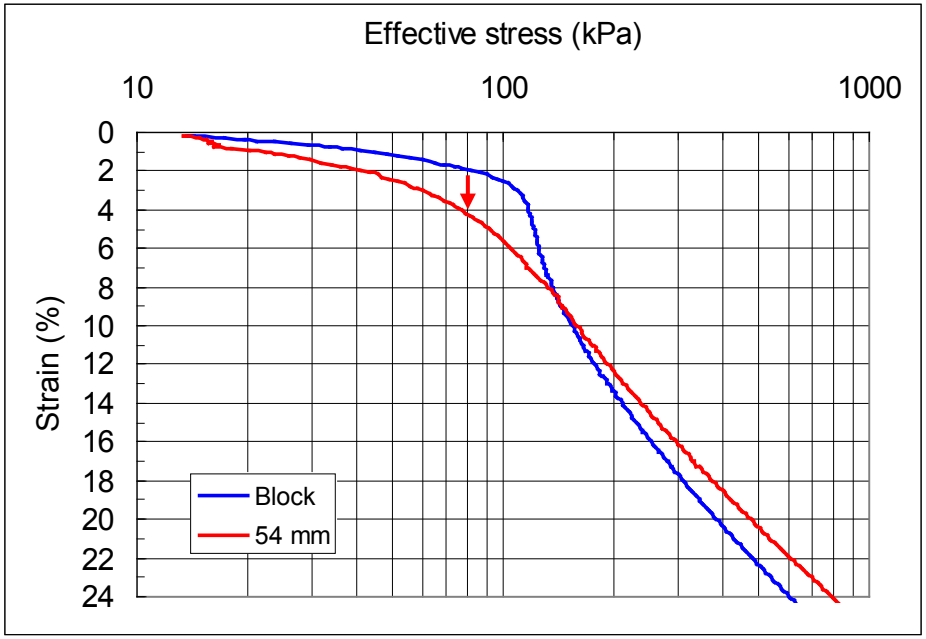
Soil samples from different depths/soil layers

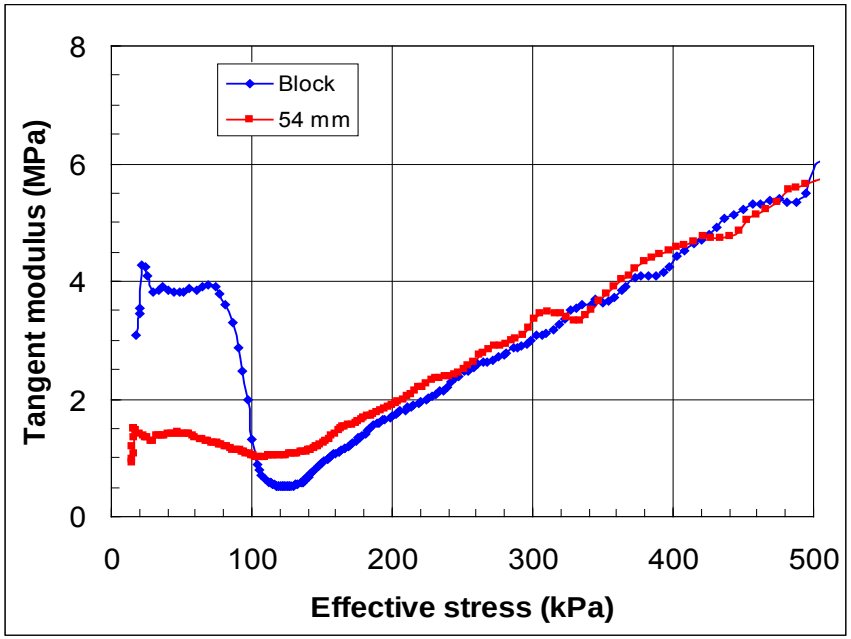
Standard index tests

Oedometer tests

- constant strain rate (CRS) tests with unloading/reloading loops.
- x days creep test (and/or CRS tests with different rates)
- additional permeability tests?
- incremental loading (IL) tests (specification: $\Delta q/q=1$?, duration=24 hrs, pore pressure measurements, long term creep phases, etc)

Sample disturbance (apparent creep)





Is soil disturbance the reason for good agreements between standard consolidation analyses (without creep) and field observation?

$$M_{oc} = a \cdot M_{pc}$$

$$a = 5 \rightarrow 15$$

Recommendations of laboratory tests

IL tests are well suited to provide data on creep parameters and the location of the RTL

CRS tests is recommended to define the shape of one isotache specially around the yield stress

How should we define creep behaviour before pc?

- problem of sample disturbance

Parameter selection for creep models

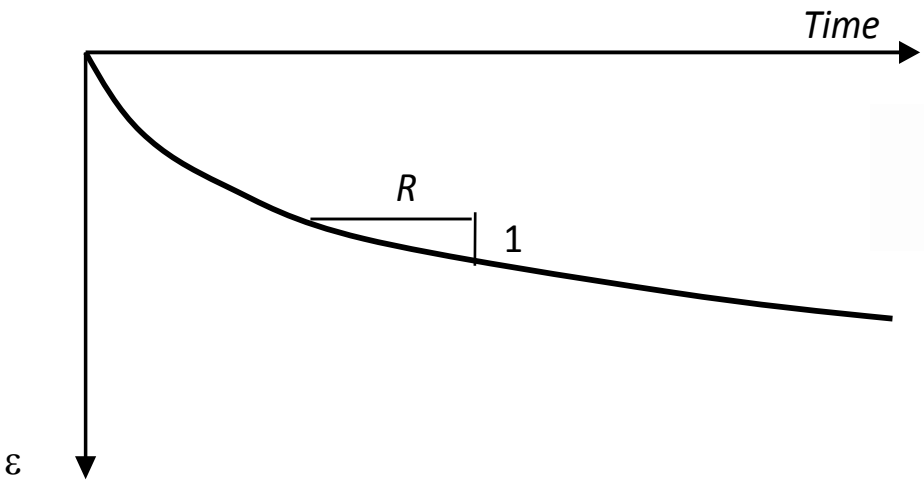
Gustav Grimstad

Modelling of creep deformations

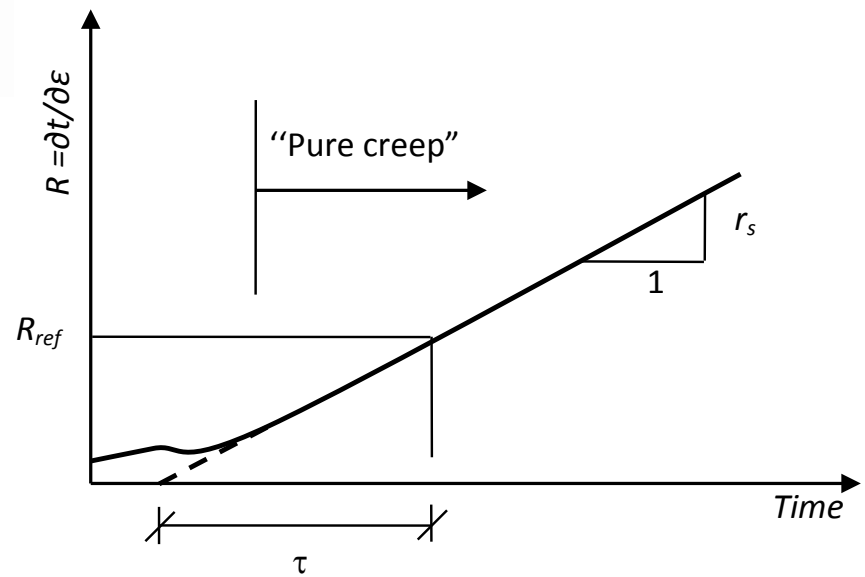
- Norwegian practice today
  1. Neglect creep
  2. Divide into primary- and secondary deformations (Hyp. A)
  3. GeoSuite Settlement with KRYKON
  4. Soft Soil Creep (SSC) in PLAXIS (or using an UDSM)

Common for “all” creep models: The Time Resistance Concept

- Time Resistance: $R = \frac{increment\ in\ time}{increment\ in\ strain} = \frac{\Delta t}{\Delta \varepsilon} = \frac{\mathrm{d}t}{\mathrm{d}\varepsilon}$
- = Inverse of the strain rate, $\dot{\varepsilon}$

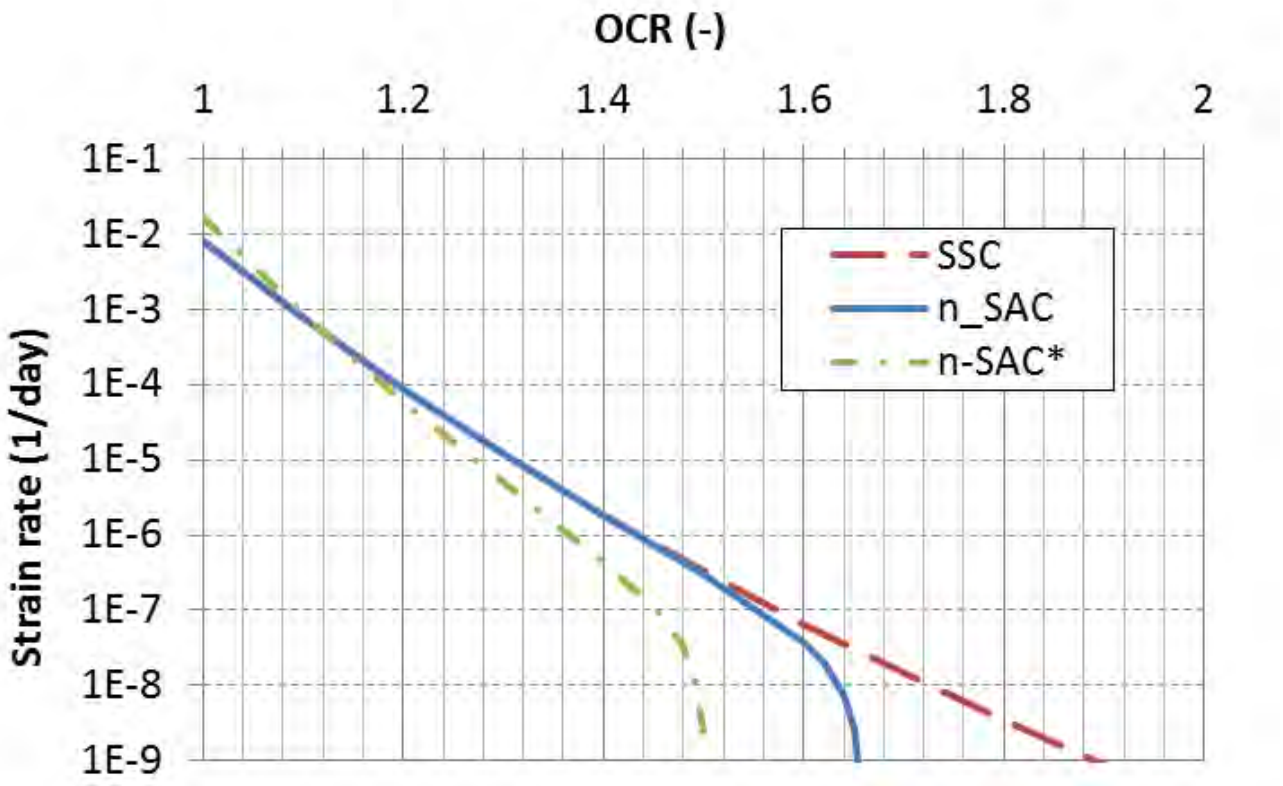


- Time Resistance number: r_s



PLAXIS (SSC and some UDSM)

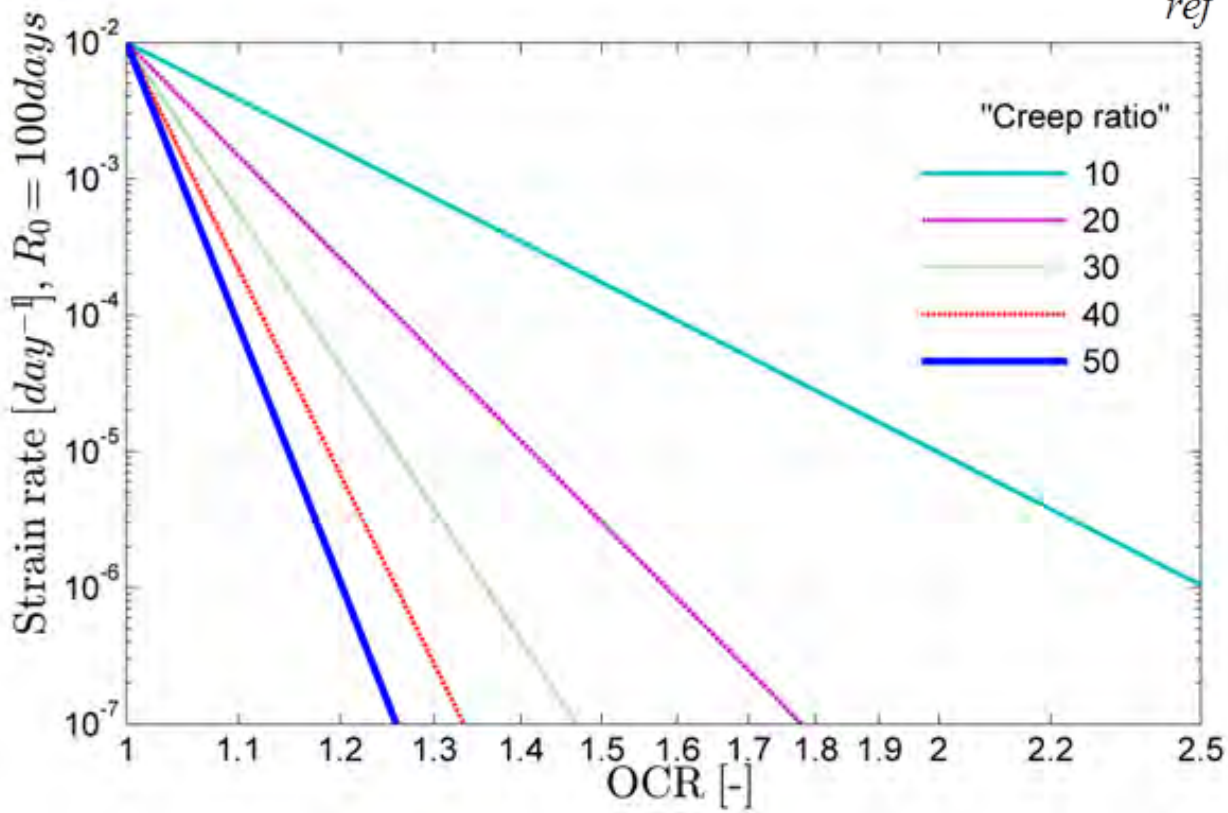
$$\dot{\varepsilon}_a^{vp} = \frac{1}{R_{ref}} \cdot OCR_\tau^{-r_s \cdot \left(\frac{1}{m_{\sec}} - \frac{1}{m_{OC}} \right)}$$



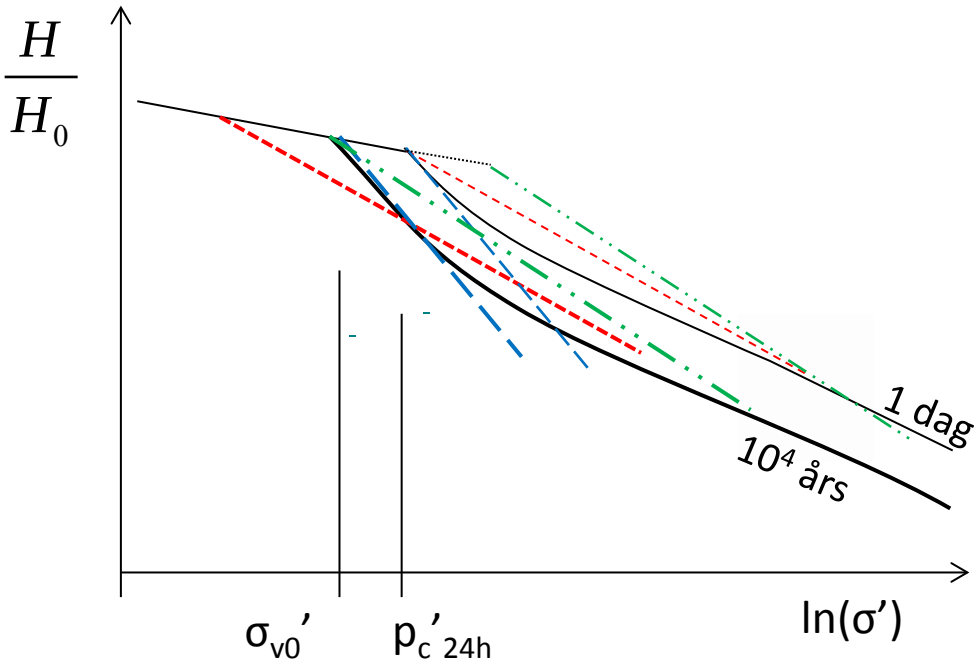
OCR is important for SSC

- OCR gives the initial strain rate

$$\dot{\varepsilon}_a^{vp} = \frac{1}{R_{ref}} \cdot OCR_\tau^{-r_s \cdot \left(\frac{1}{m_{\sec}} - \frac{1}{m_{OC}} \right)}$$

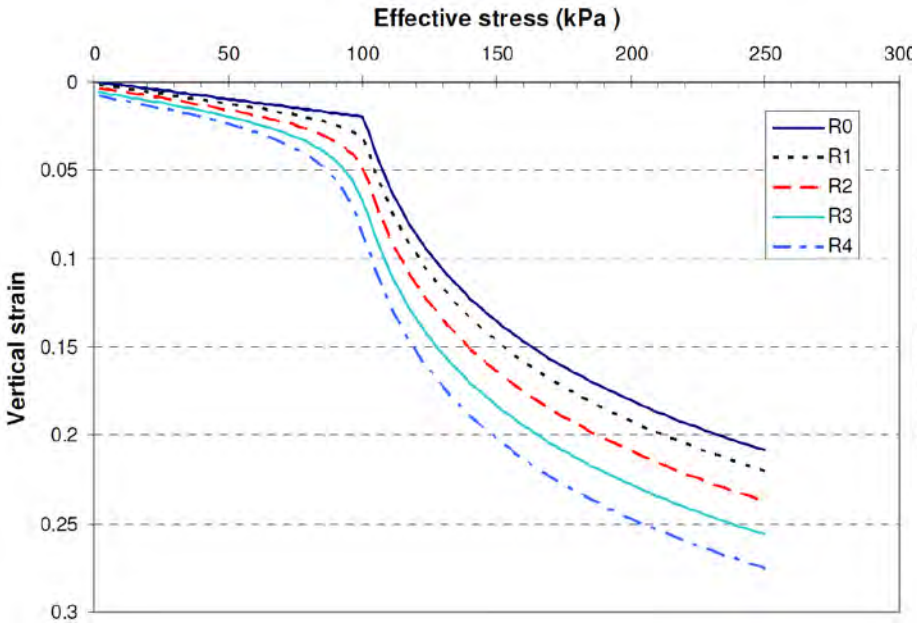


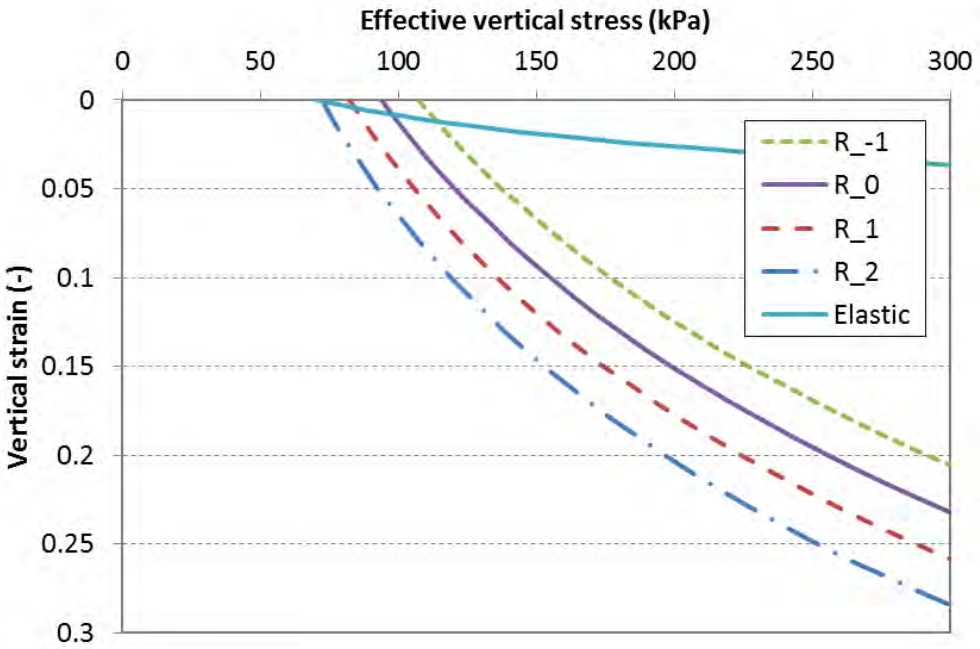
How to choose the parameters?



Always adjust the model parameters for the relevant stress range

SSC vs KRYKON

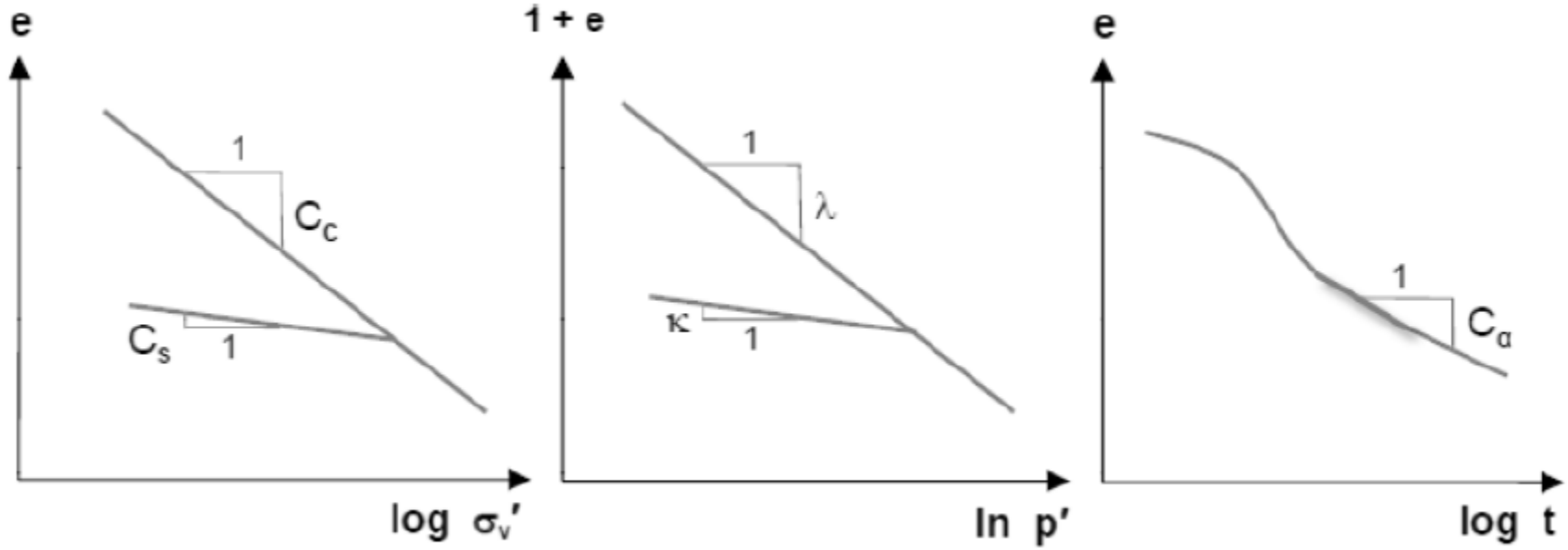




KRYKON

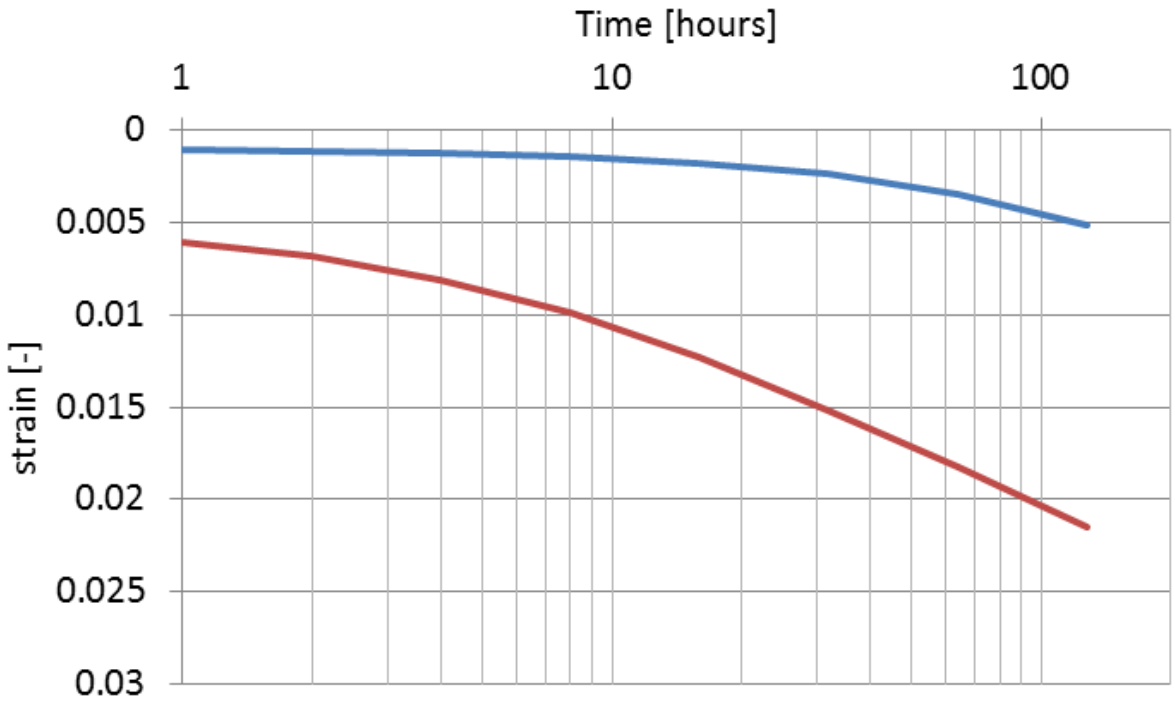
SSC

Different parameters for different models



Origination	Compression index	Recompression index or swelling index	Secondary compression index or creep index
International	C_c	C_r or C_s	C_α or $C_{\alpha e}$
Cam-Clay	$\lambda = \frac{C_c}{\ln 10}$	$\kappa \approx \frac{3}{\ln 10}\frac{(1-\nu_{ur})}{\nu_{ur}}C_s$	$\mu = \frac{C_\alpha}{\ln 10}$
Plaxis	$\lambda^* = \frac{\lambda}{1+e}$	$\kappa^* = \frac{\kappa}{1+e}$	$\mu^* = \frac{\mu}{1+e}$
Norway	$m_{nc} = \frac{1}{\lambda^*}$	$m_{oc} = \frac{\ln 10(1+e)}{C_s}$	$r = \frac{1}{\mu^*}$

The “problematic” log t



$\mu^* = 0.0024$

$\mu^* = 0.0047$

The “problematic” log t

Janbu:

$$R = \frac{1}{\dot{\varepsilon}} = r_s \cdot (t + t_0)$$

$$\dot{\varepsilon} = \frac{1}{r_s} \cdot \frac{1}{t + t_0} = \mu^* \cdot \frac{1}{t + t_0}$$

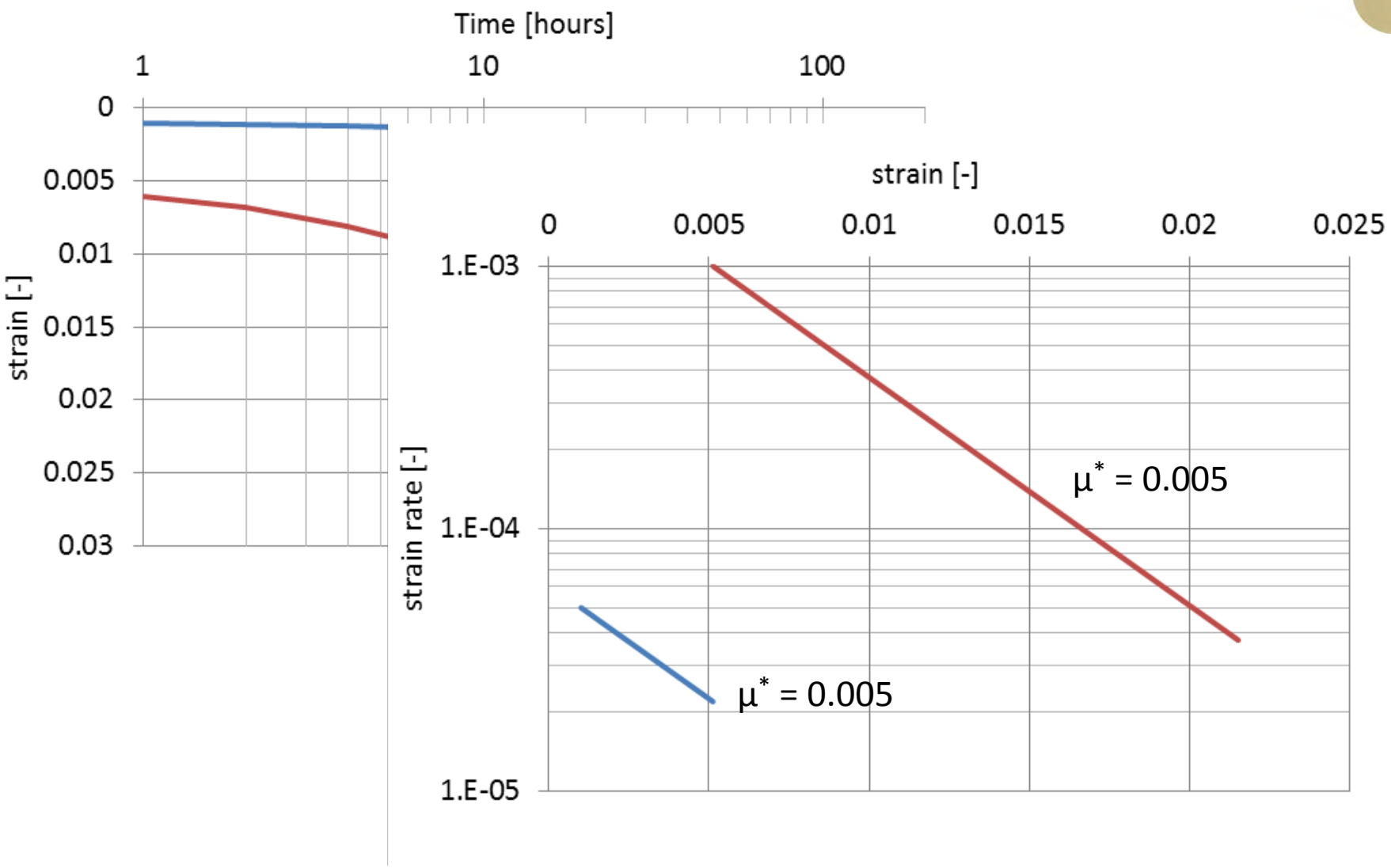
$$\varepsilon = \frac{1}{r_s} \cdot [\ln(t + t_0) - \ln t_0] = \frac{1}{r_s} \cdot \ln \frac{t + t_0}{t_0}$$

$$\varepsilon = -\frac{1}{r_s} \cdot \ln(\dot{\varepsilon} \cdot r_s \cdot t_0) = -\frac{1}{r_s} \cdot [\ln \dot{\varepsilon} + \ln(r_s \cdot t_0)]$$

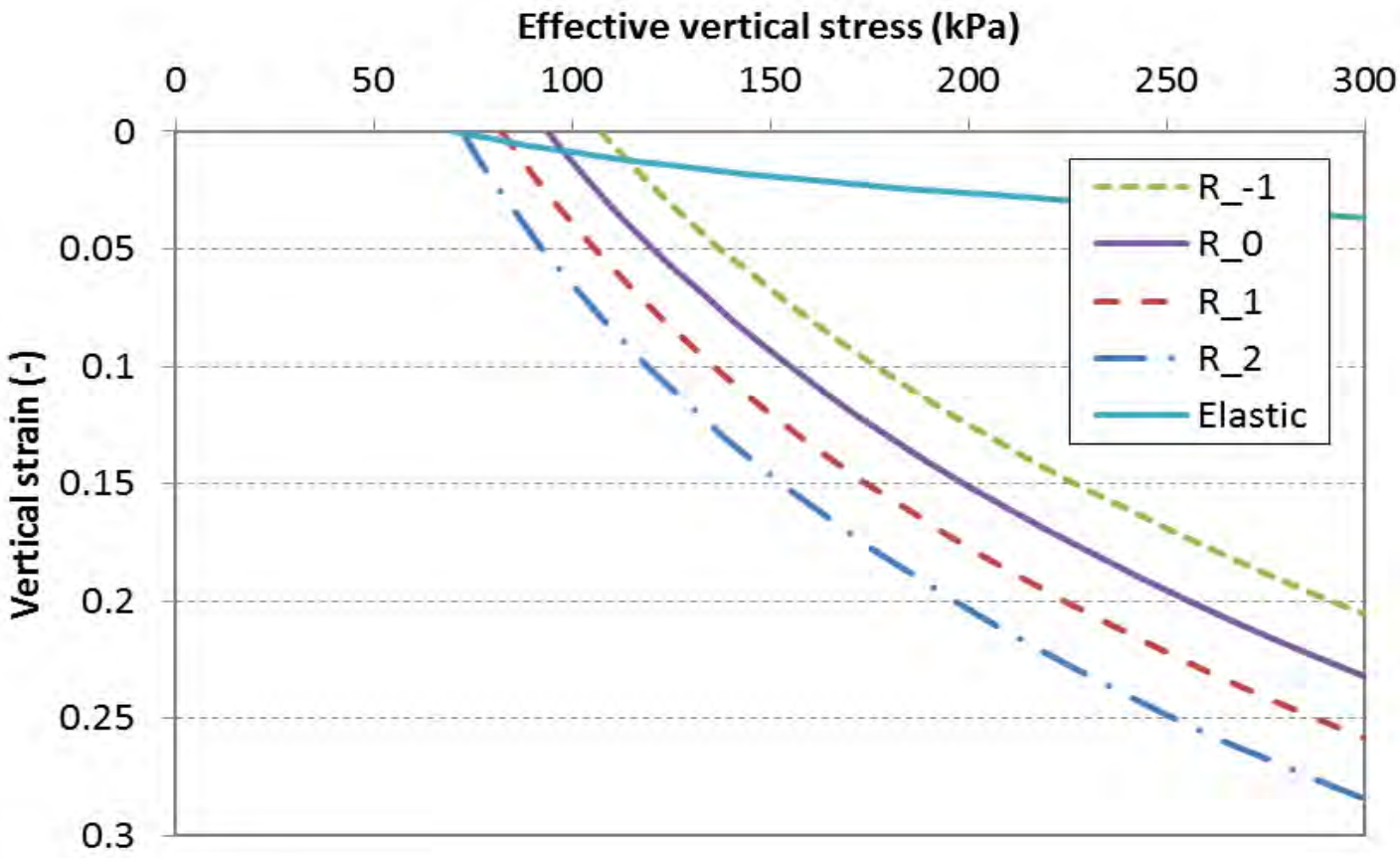
Inspired by David Nash:

$$\varepsilon = -\mu^* \cdot \left[\ln \dot{\varepsilon} - \ln\left(\frac{\mu^*}{t_0}\right)\right]$$

The “problematic” log t



Why and when?


Effective vertical stress (kPa)
0
50
100
150
200
250
300
Vertical strain (-)
0
0.05
0.1
0.15
0.2
0.25
0.3
R_-1
R_0
R_1
R_2
Elastic

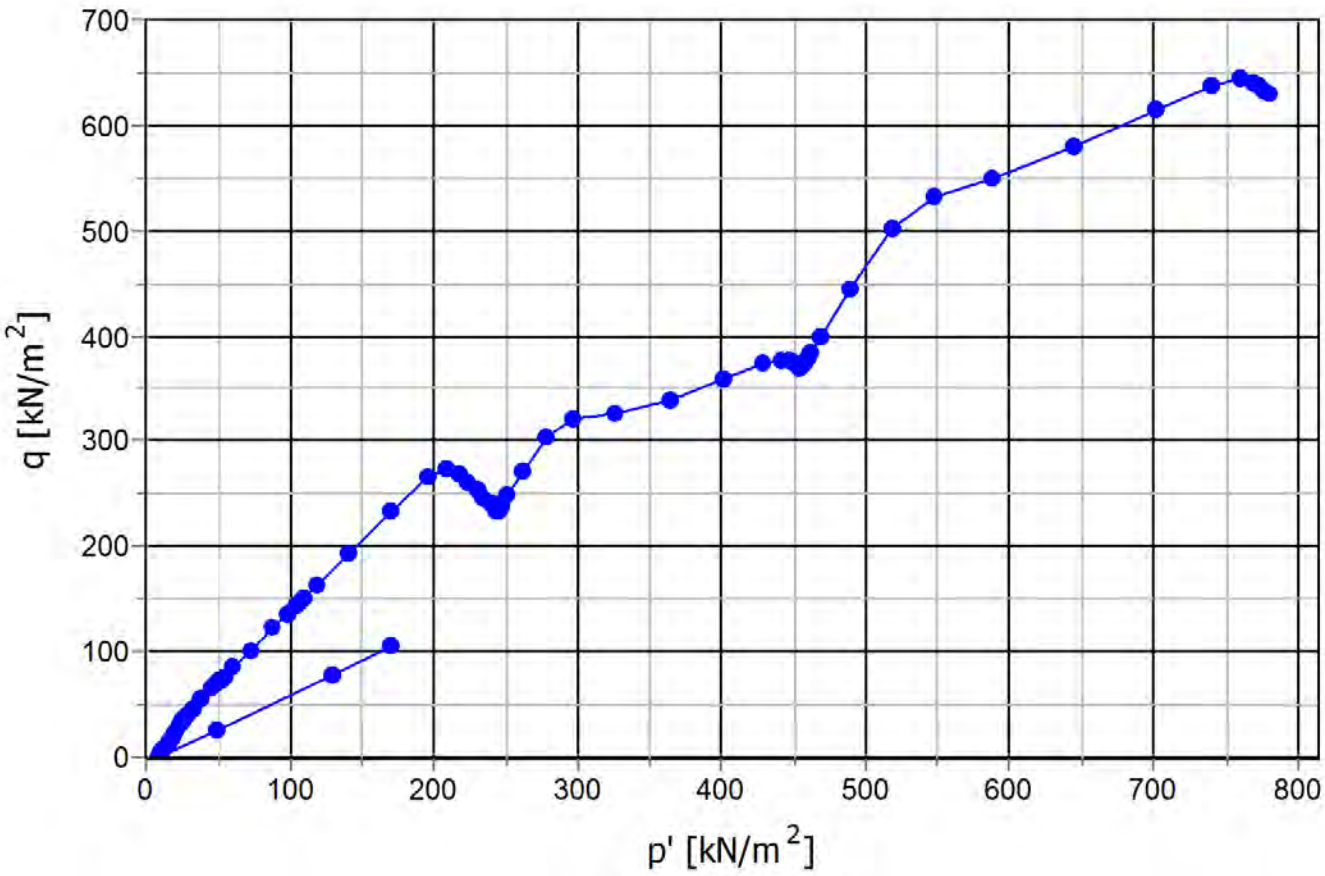
The oedometer test…

- Can we rely on OCR from IL oedometer tests?
  1) Sample disturbance? (we all know this)
  2) Stress condition?
     - Start at some unknown isotropic stress condition and consolidates to 12.5 kPa of vertical stress
     - Loads further along a line different from K_0^{NC} line (i.e. stress path hits the reference pre-consolidation at different place than it would in-situ!)

     → Do we need to simulate the oedometer test rather then interpret OCR from it?

     → Should we measure horizontal stress in the oedometer?
  3) Consolidation (is the effective stress constant for most of the 24h?, e.g. clays with low permeability)
  4) Extrapolation… (should model OCR and reality OCR be the same?)

Simulated oedometer with SSC

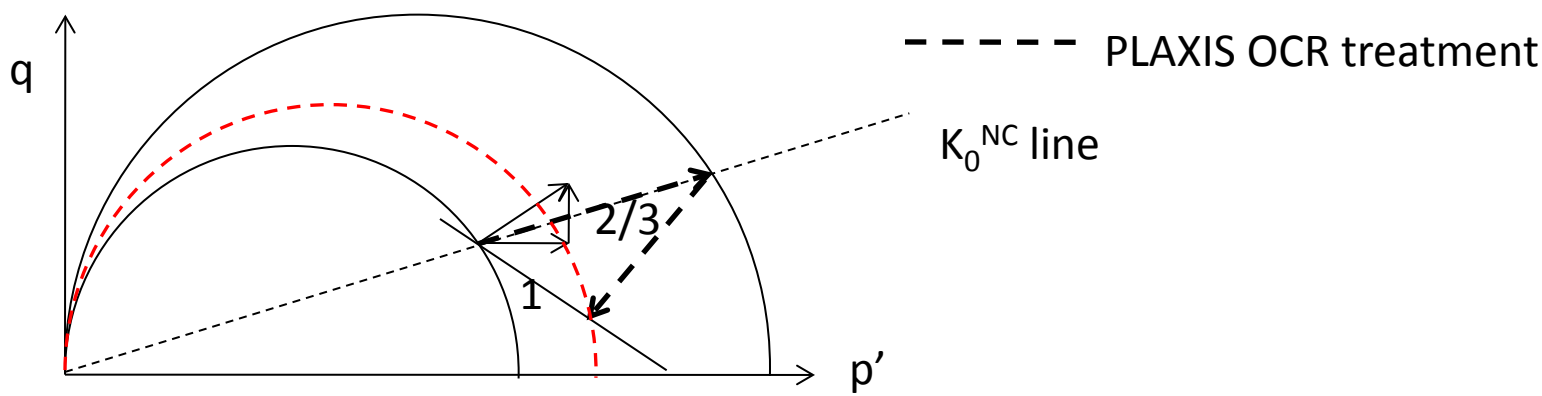


What about K_0?

- Is the in-situ K_0 affected by creep (NC clay)?
  - Model says: very limited influence, i.e. $K_0 \approx K_0^{NC}$
- Has the material been unloaded (OC clay)?
  - Model says: yes, but creep will try to make $K_0 \approx K_0^{NC}$ if the model is not changing its plastic potential, since the volumetric strain should be equal to the vertical strain
- Should we then set $K_0 \approx K_0^{NC}$ for models like SSC?

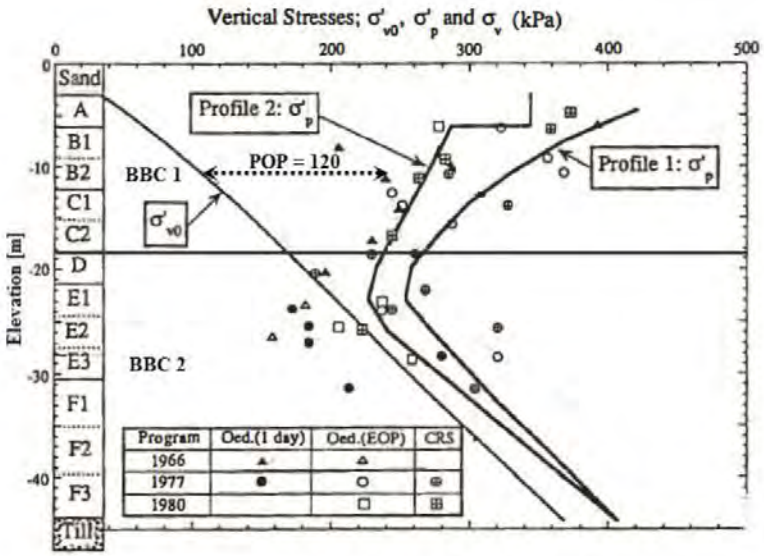
OCR and K_0

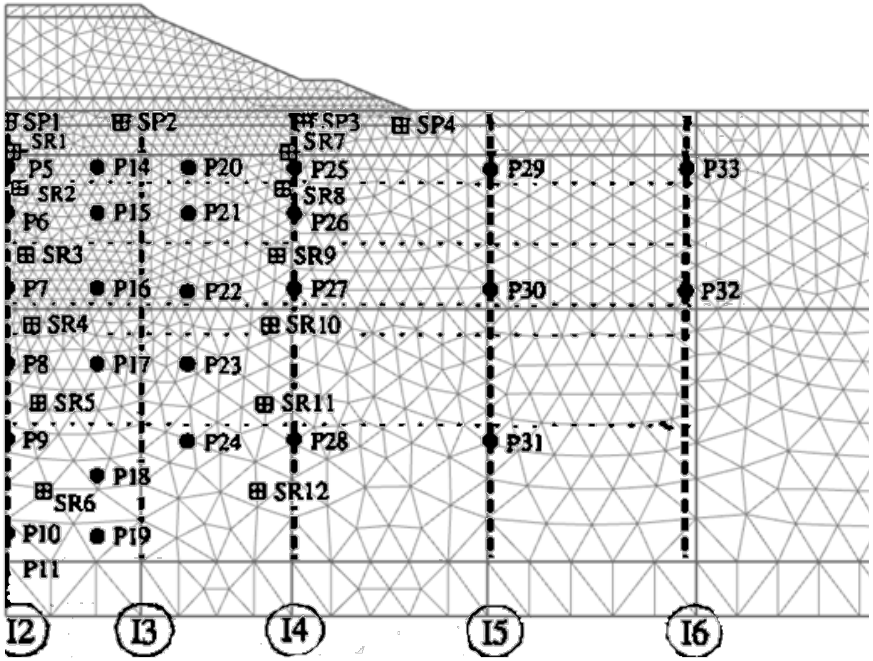
- The K_0 value does not change significantly in a 1D creep case due to the increase in OCR. Since 1D creep requires $d\varepsilon_1^{vp} = d\varepsilon_v^{vp}$, then the stress state is fixed to one point at the potential surface.
- In PLAXIS if one specify a OCR (due to creep alone), the suggested initial horizontal stress generated (suggested K_0) is based on the assumption of unloading. Remember to change this back to a value close to the real K_0^{NC}

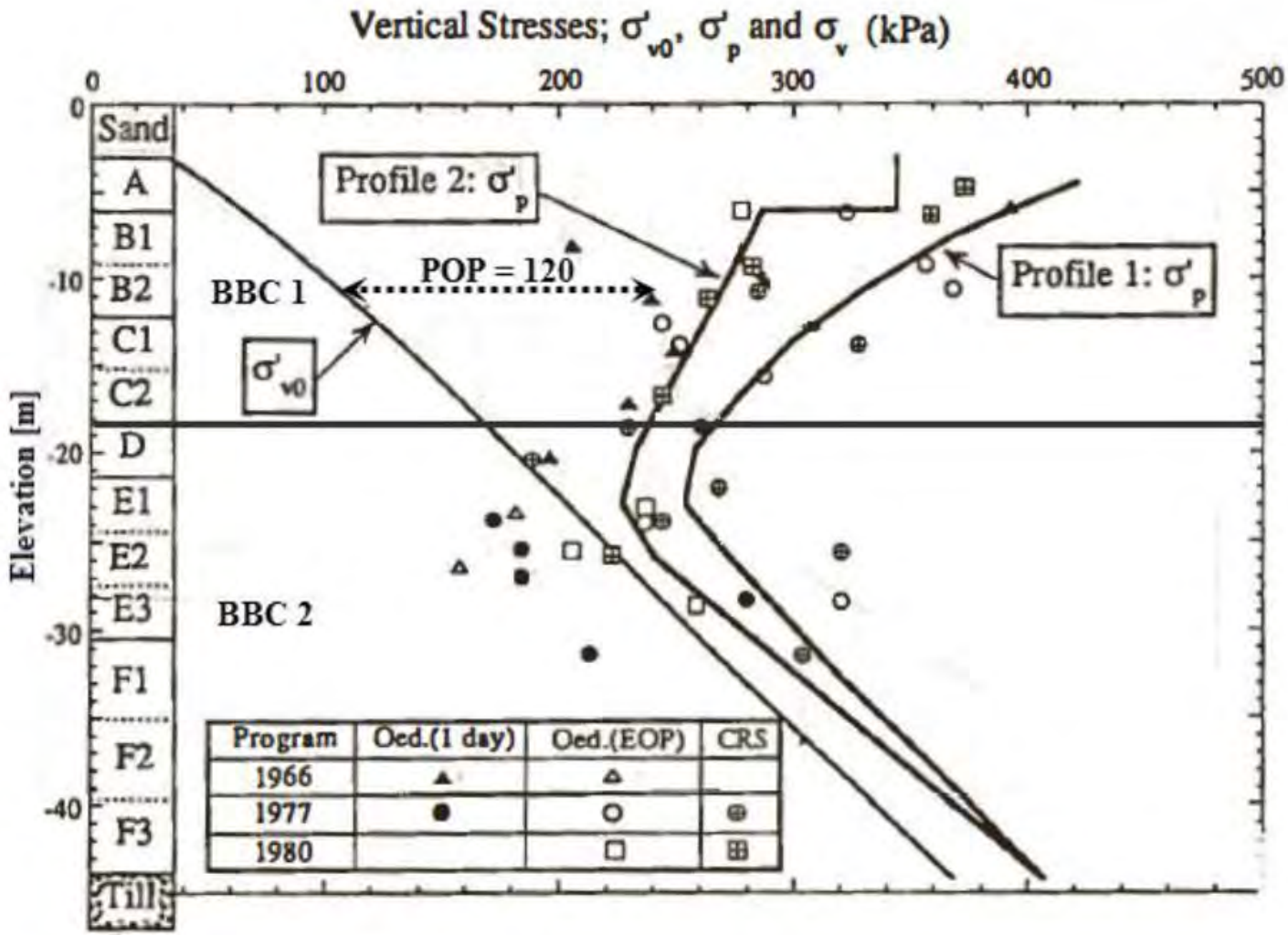


The MIT–MDPW embankment

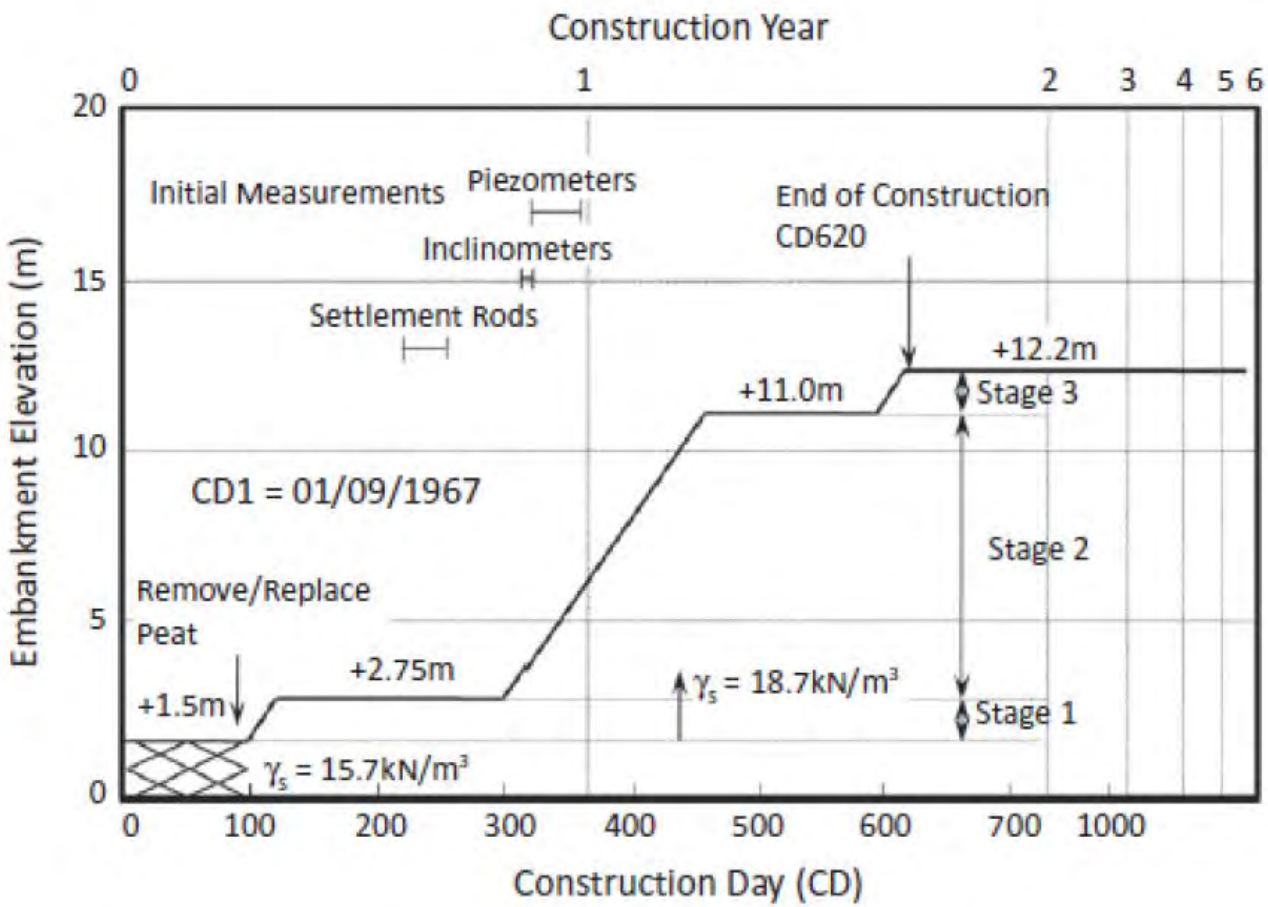
- Latest paper looking at back calculating this is from 2012 (Fatahi et al.)



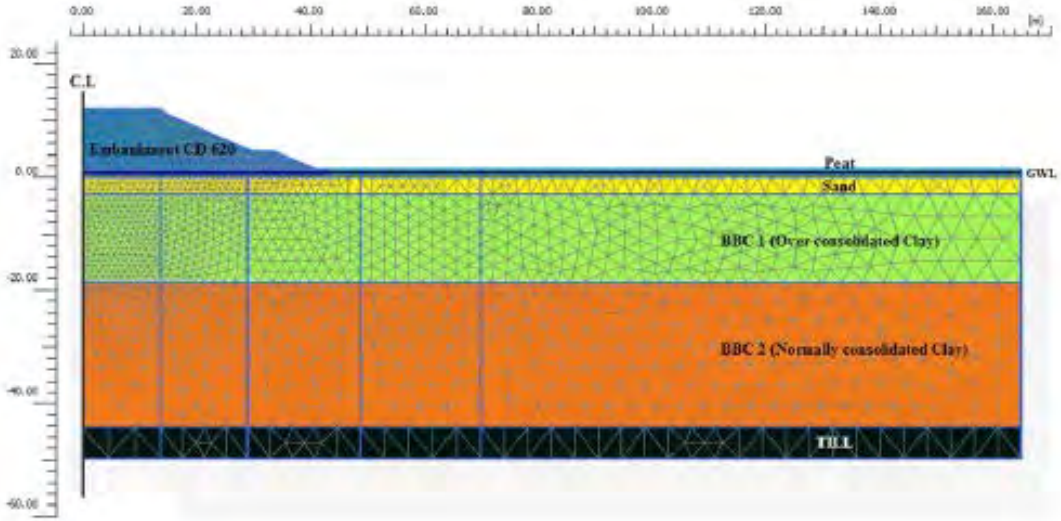


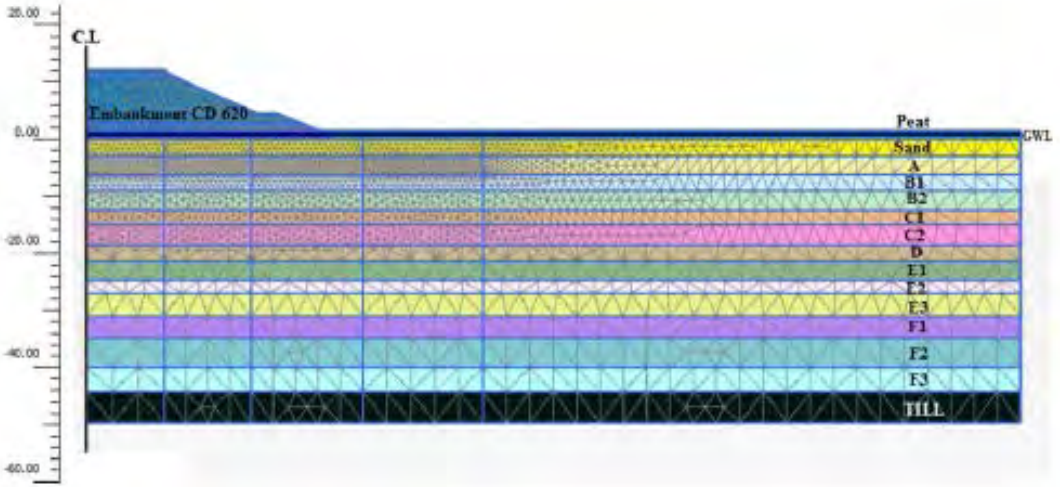

Vertical Stresses; σ'v0, σ'p and σv (kPa)
0
100
200
300
400
500
Elevation [m]
0
-10
-20
-30
-40
Sand
A
B1
B2
C1
C2
D
E1
E2
E3
F1
F2
F3
Till
Profile 2: σ'p
POP = 120
BBC 1
σ'v0
Profile 1: σ'p
BBC 2
Program
Oed.(1 day)
Oed.(EOP)
CRS
1966
1977
1980

The trial embankment

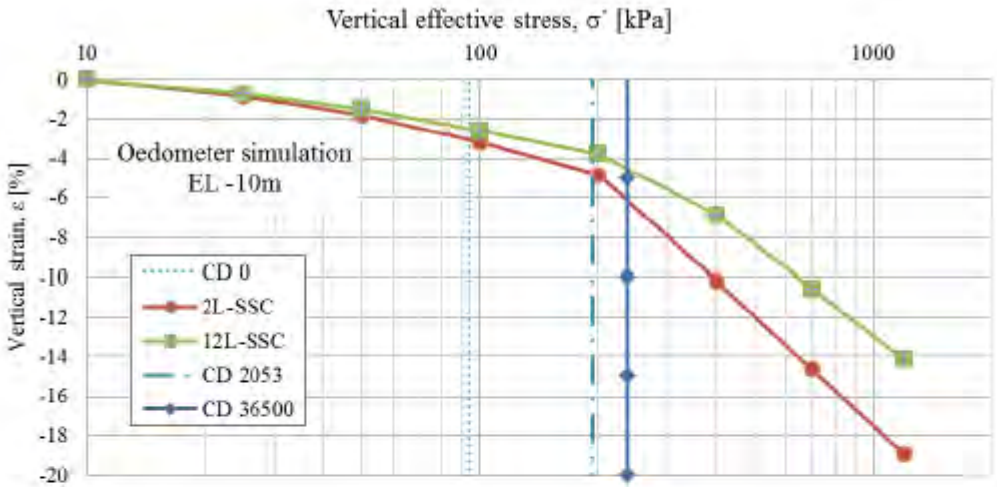


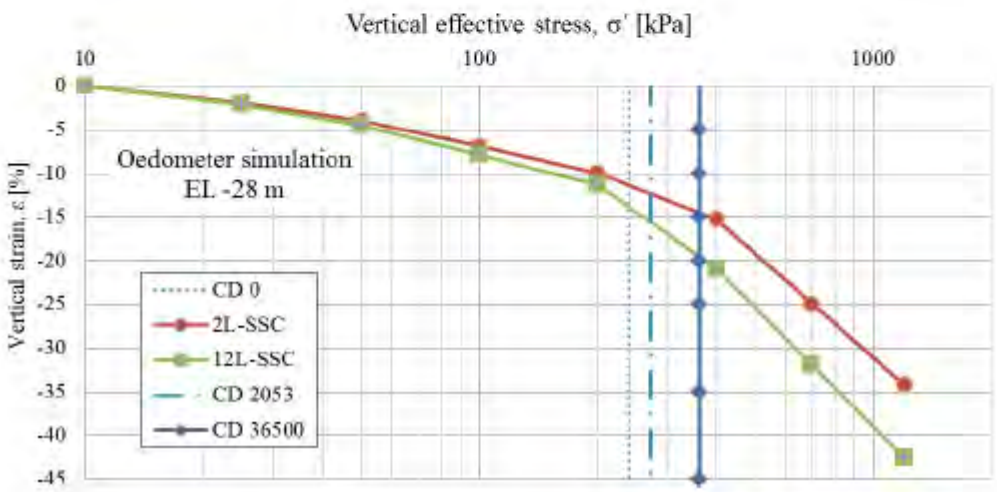
Alternative models



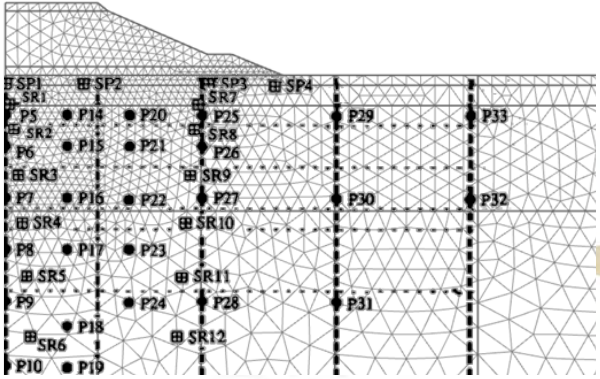


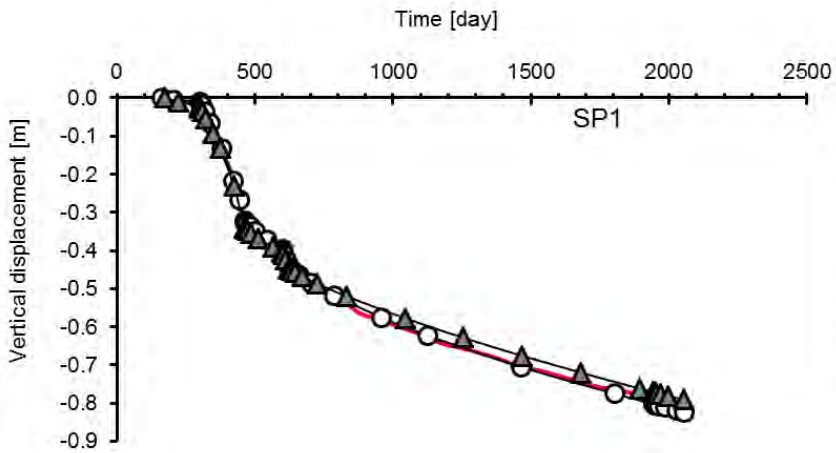
Oedometer simulations

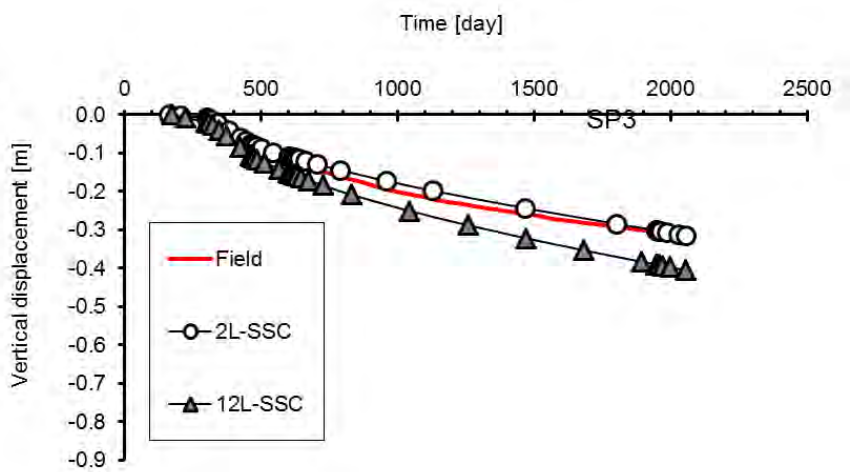


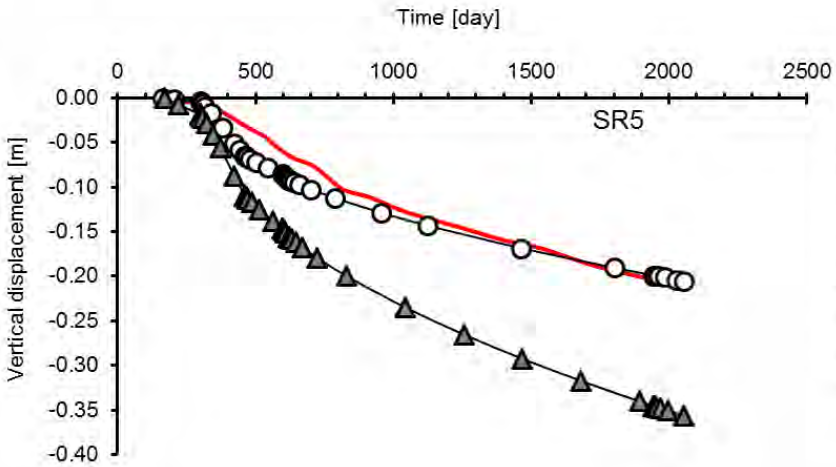


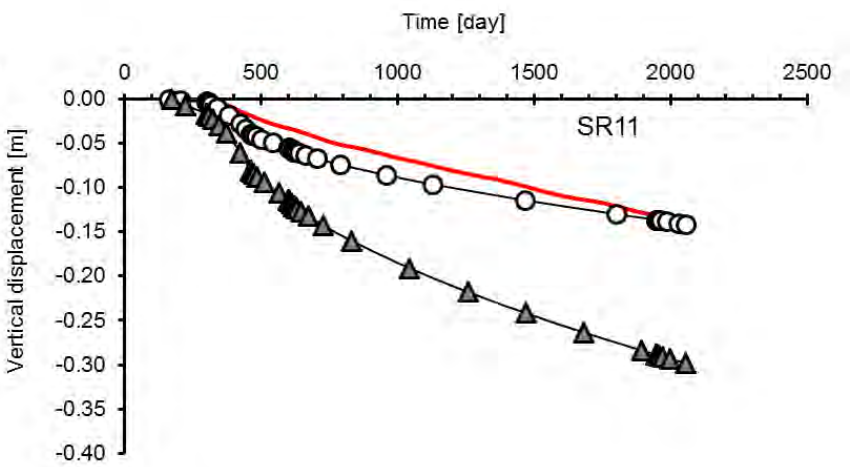
Results

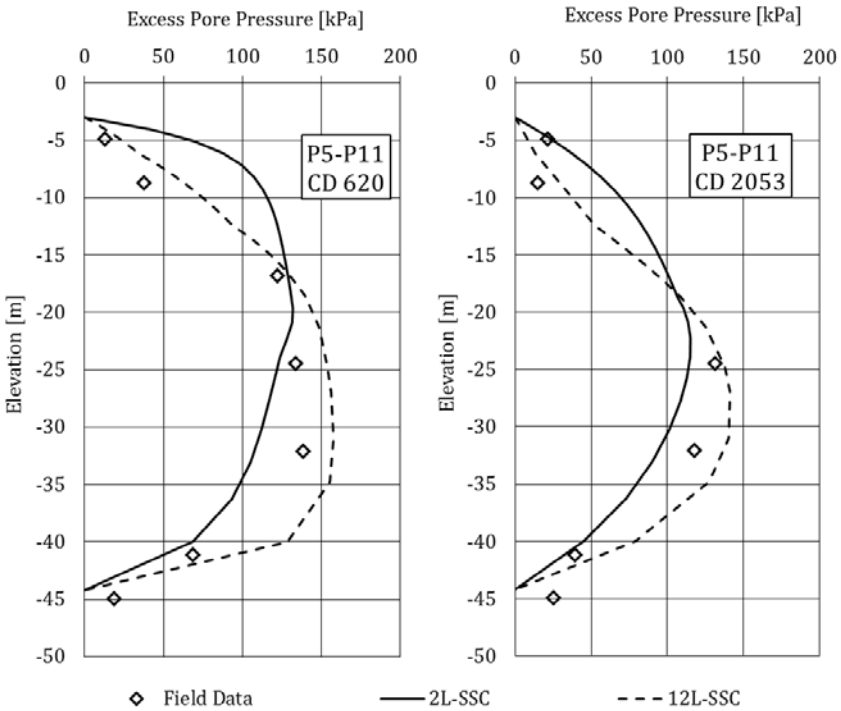

SP1
SP2
SP3
SP4
SR1
SR2
SR3
SR4
SR5
SR6
SR7
SR8
SR9
SR10
SR11
SR12
P5
P6
P7
P8
P9
P10
P14
P15
P16
P17
P18
P19
P20
P21
P22
P23
P24
P25
P26
P27
P28
P29
P30
P31
P32
P33

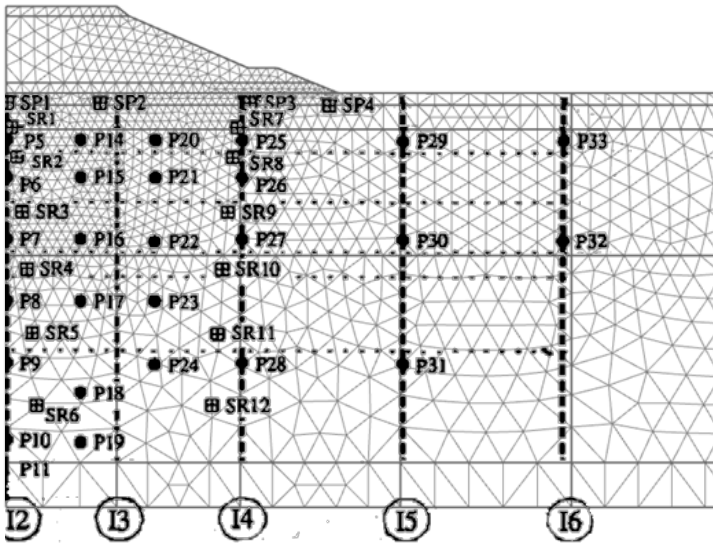

Time [day]
Vertical displacement [m]
SP1

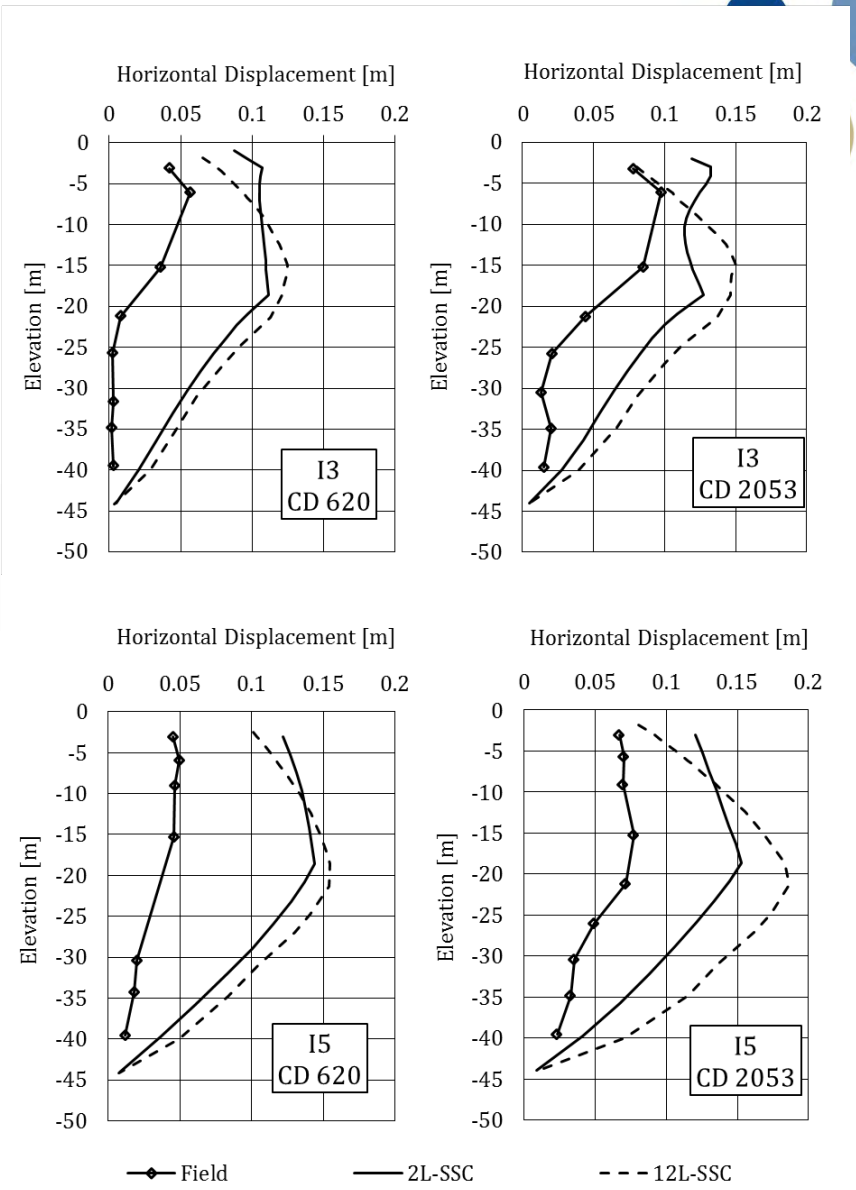

Time [day]
Vertical displacement [m]
SP3
Field
2L-SSC
12L-SSC


Time [day]
Vertical displacement [m]
SR5

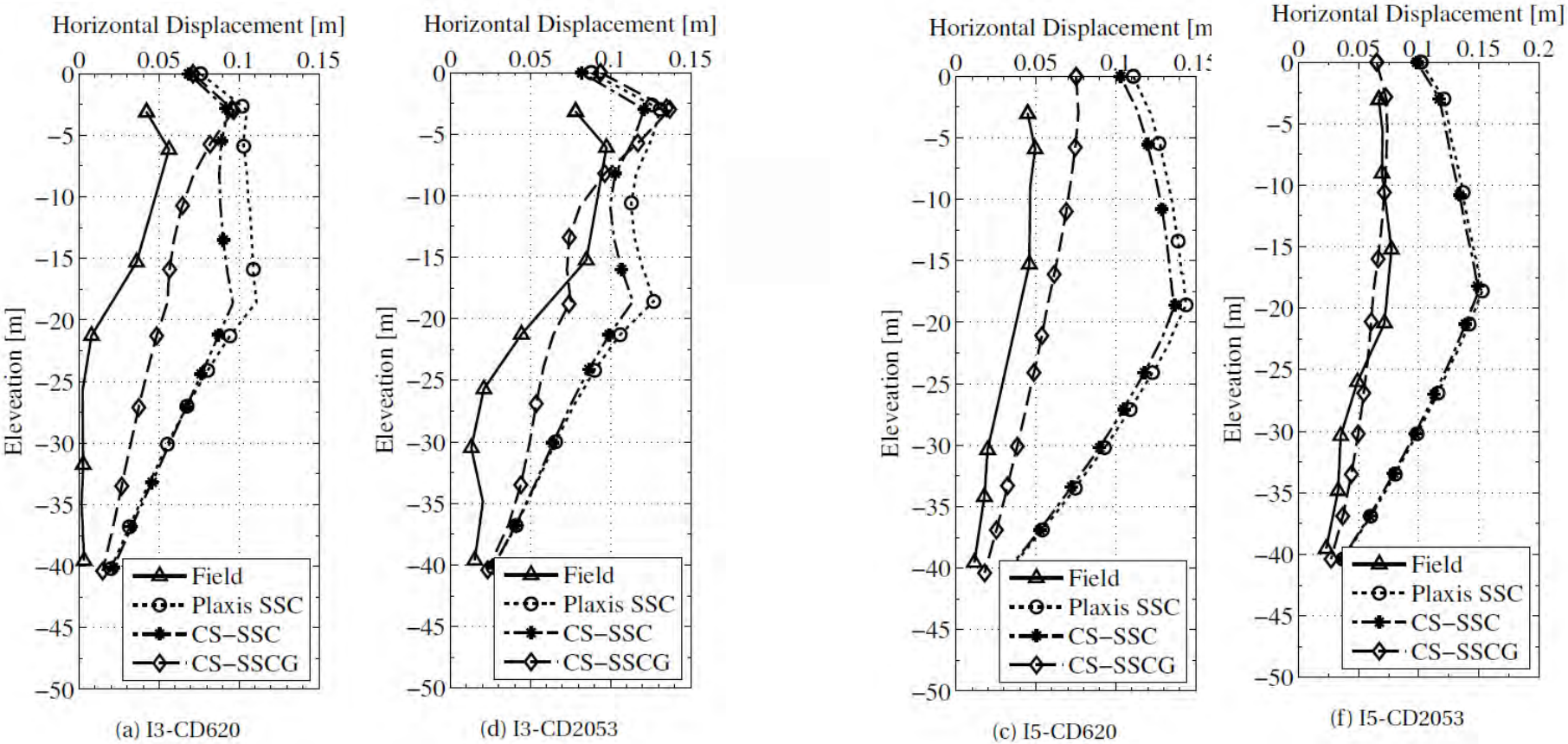

Time [day]
Vertical displacement [m]
SR11


Excess Pore Pressure [kPa]
0 50 100 150 200
Elevation [m]
0 -5 -10 -15 -20 -25 -30 -35 -40 -45 -50
P5-P11
CD 620
P5-P11
CD 2053
Field Data
2L-SSC
12L-SSC


SP1
SP2
SP3
SP4
SR1
SR2
SR3
SR4
SR5
SR6
SR7
SR8
SR9
SR10
SR11
SR12
P5
P6
P7
P8
P9
P10
P11
P14
P15
P16
P17
P18
P19
P20
P21
P22
P23
P24
P25
P26
P27
P28
P29
P30
P31
P32
P33
I2
I3
I4
I5
I6


Horizontal Displacement [m]
0 0.05 0.1 0.15 0.2
Elevation [m]
0 -5 -10 -15 -20 -25 -30 -35 -40 -45 -50
I3
CD 620
I3
CD 2053
I5
CD 620
I5
CD 2053
Field
2L-SSC
12L-SSC

The horizontal deformations with improved description (Ashrafi 2014)



Conclusion

- Select all parameters for the relevant stress interval
- Assumption on the initial strain rate (i.e. OCR in a model) is very important
- OCR is not an index property of a clay that can be given to the engineer, but a parameter for a model and must be given with a reference to some rate or time and work for the relevant stress range
- Small strain stiffness matters for horizontal deformation, i.e. size of FE model also matters.

From 1D creep models to the current creep models

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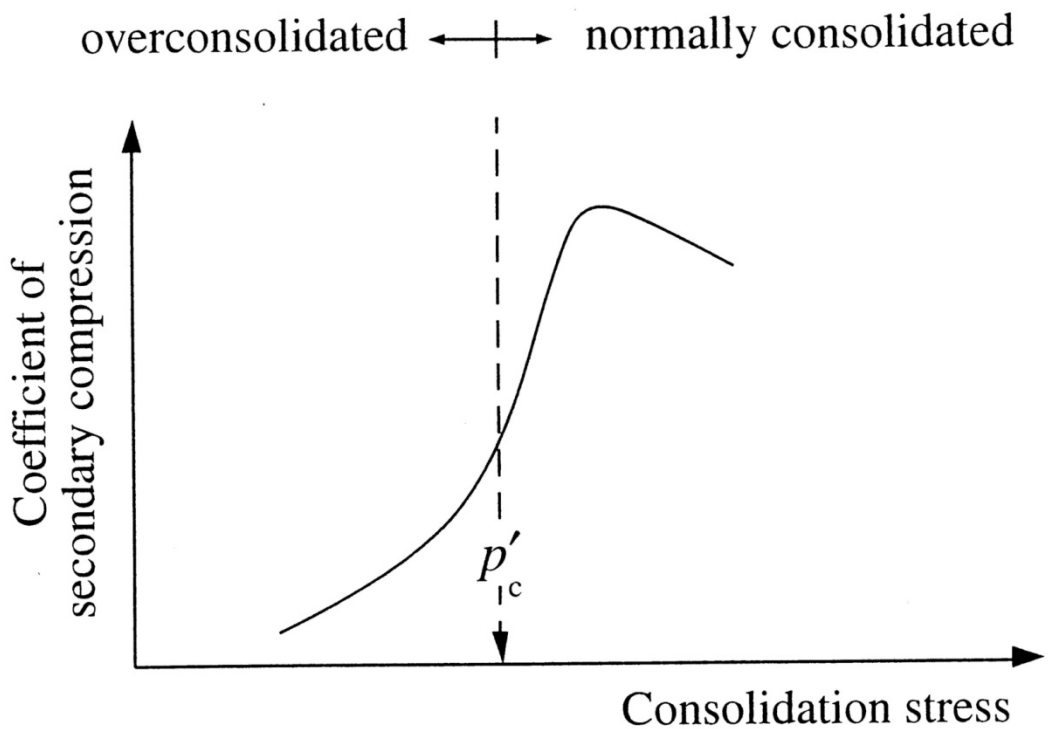
Outline

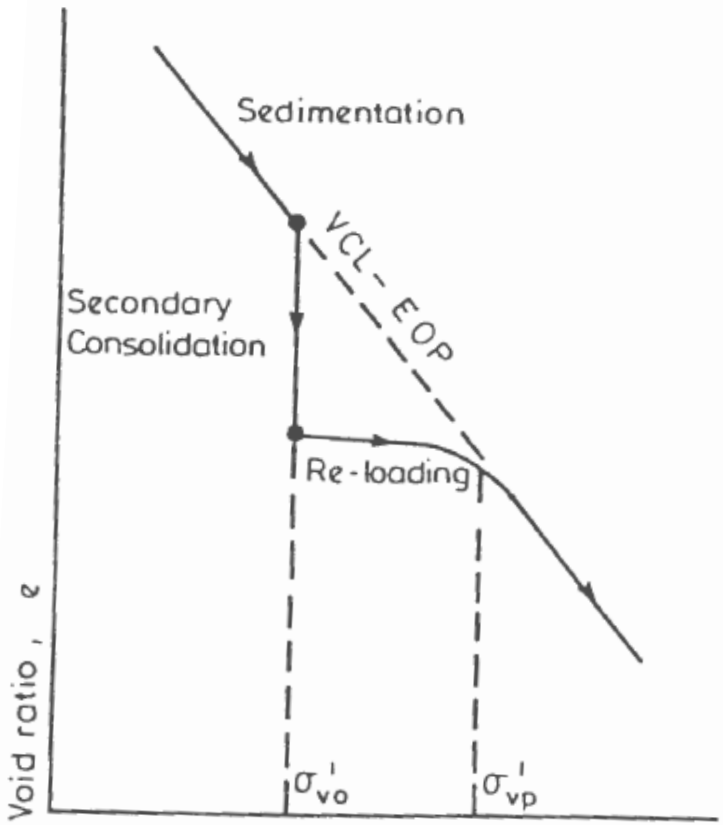
- Introduction
  - 1D creep model
- Constitutive modelling of creep
- Conclusions and future work

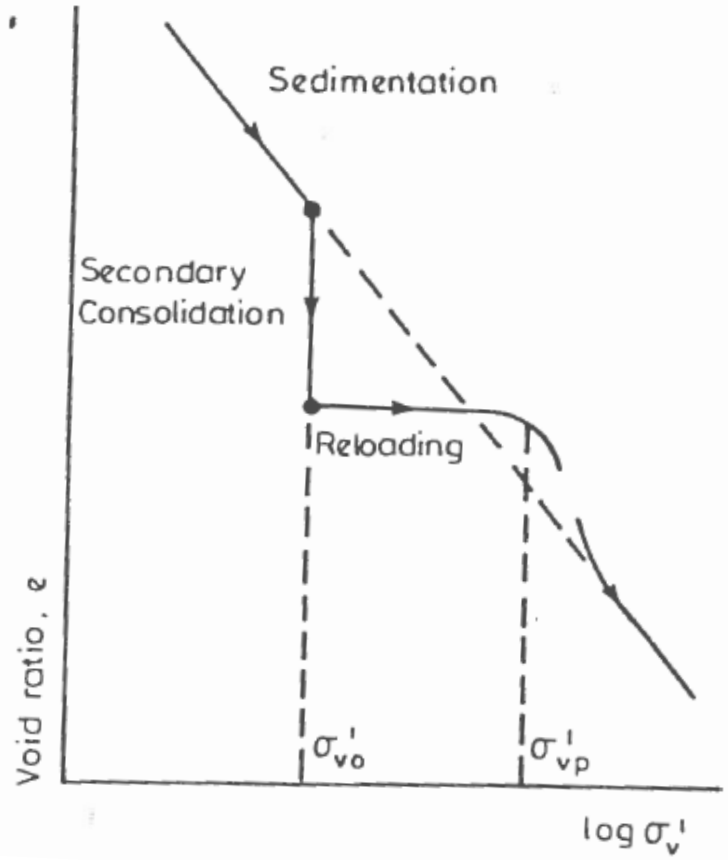
Secondary consolidation is insignificant for stress levels <u>below</u> the preconsolidation pressure but can be large when plastic straining occurs.



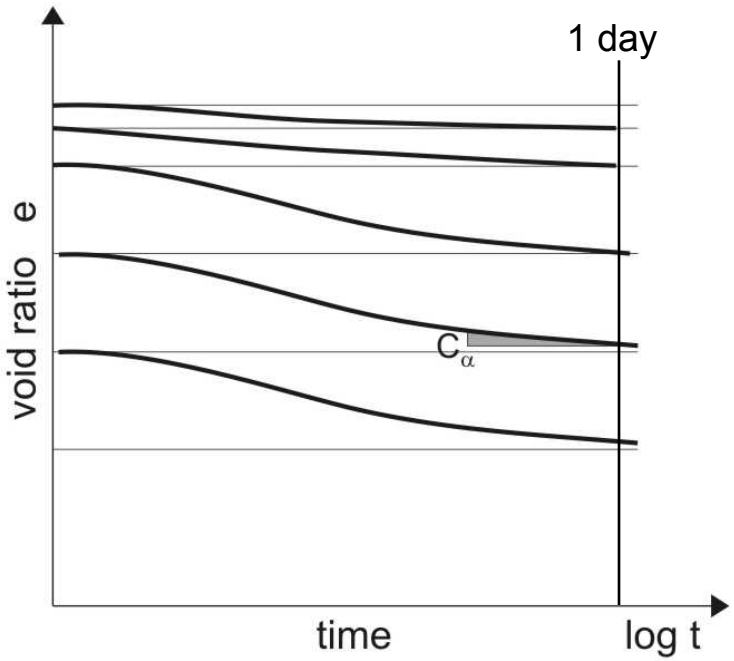
$$C_\alpha = \frac{\Delta e}{\log_{10} \frac{t_2}{t_1}}$$

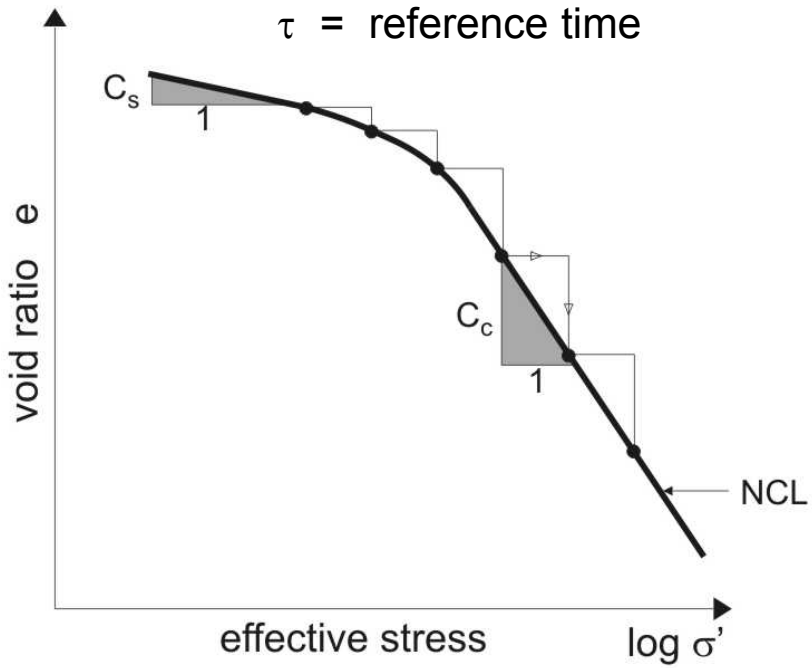
Effect of aging and cementation





1D creep model





Each day: $\Delta\sigma = \text{constant} \cdot \sigma$

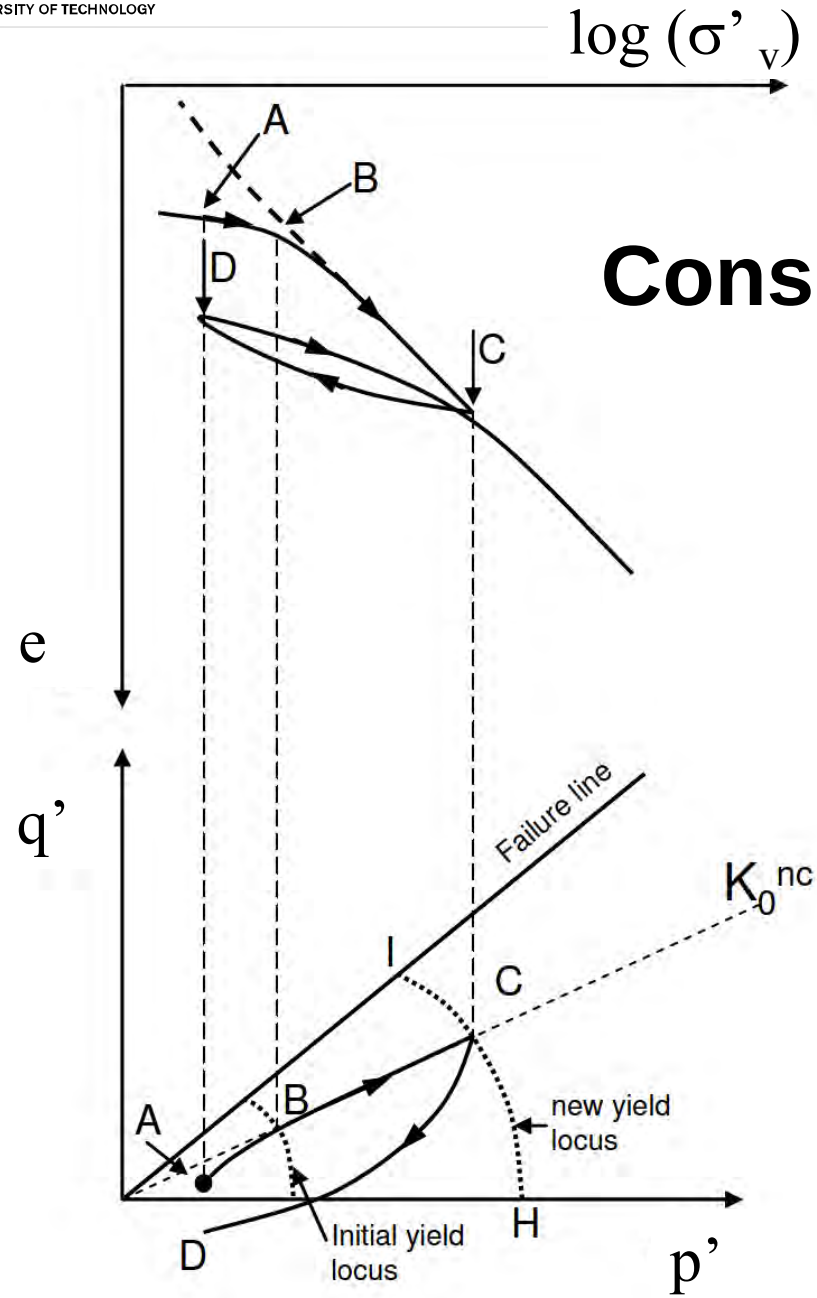
$$\beta = \frac{C_c - C_s}{C_\alpha}$$

$$\dot{e} = \frac{\dot{e}_{nc}}{OCR^{\beta}} \qquad \dot{e}_{nc} = -\frac{C_\alpha}{\ln 10} \cdot \frac{1}{\tau}$$

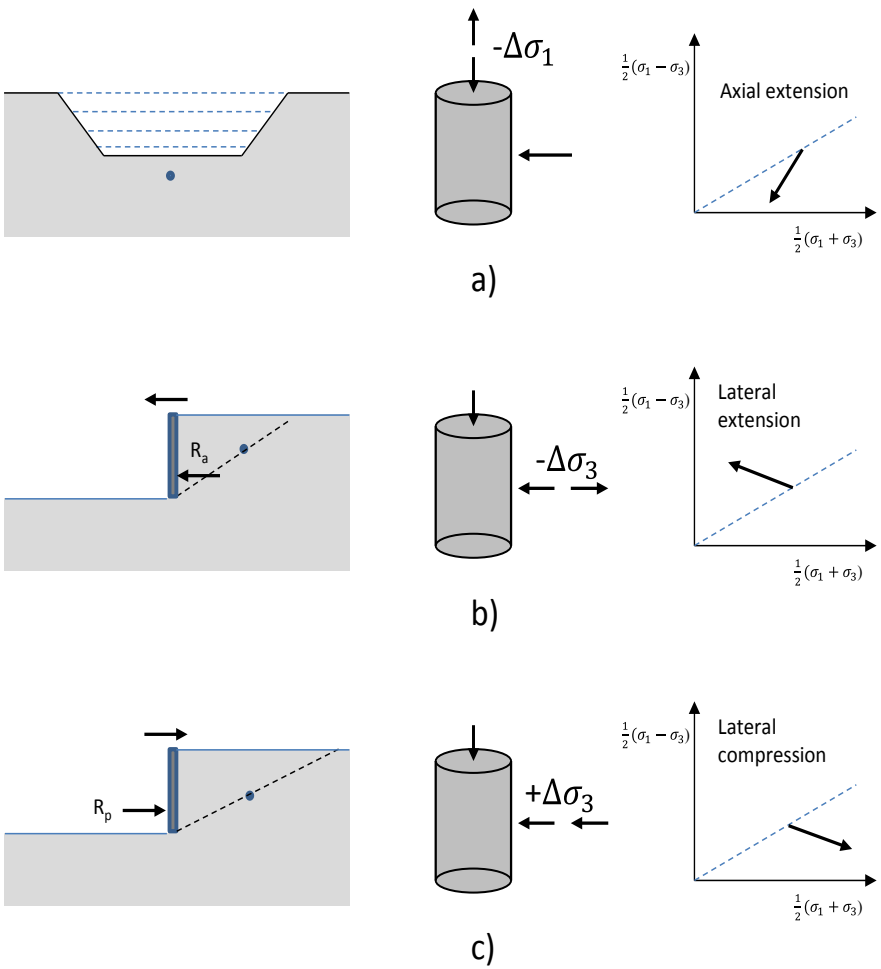
C_C = compression index $\qquad$ C_S = swelling index $\qquad$ C_α = creep index

Consolidation is never 1D

in terms of stresses



Geotechnics is not 1D



after Ortigao (1995)

3D constitutive modelling of creep

- Creep models:
  - **Soft Soil Creep** (Vermeer et al. 1998, 1999)
  - **ACM** (Leoni et al. 2008) and ACM-S (Kamrat-Pietraszewska 2011)
- Overstress models:
  - **EVP-SCLAY1S** (Karstunen & Yin 2010), AniCreep (Yin et al. 2011)
- Enhanced creep models:
  - Time-resistance model **nSAC** (Grimstad & al. 2010)
  - **Creep-SCLAY1S** (Sivasithamparam et al. 2013, Karstunen et al. 2013, Sivasithamparam et al. *in press*)

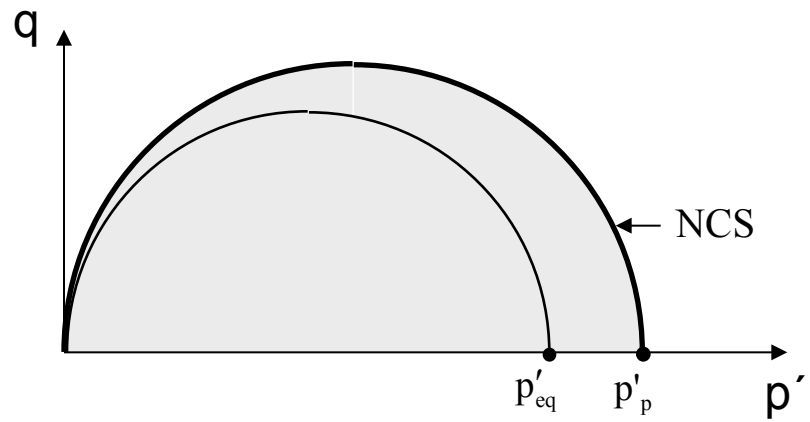
3D Soft Soil Creep model

$$\dot{\varepsilon}_v = \dot{\varepsilon}_v^e + \dot{\varepsilon}_v^c = \kappa^* \frac{\dot{p}'}{p'} + \frac{\mu^*}{\tau}\left(\frac{p_{eq}'}{p_p'}\right)^{\beta} \qquad \beta = \frac{\lambda^* - \kappa^*}{\mu^*}$$

Modified compression index: $\lambda^* = \dfrac{C_c}{\ln 10}$

Modified swelling index: $\kappa^* \approx \dfrac{2C_s}{\ln 10}$

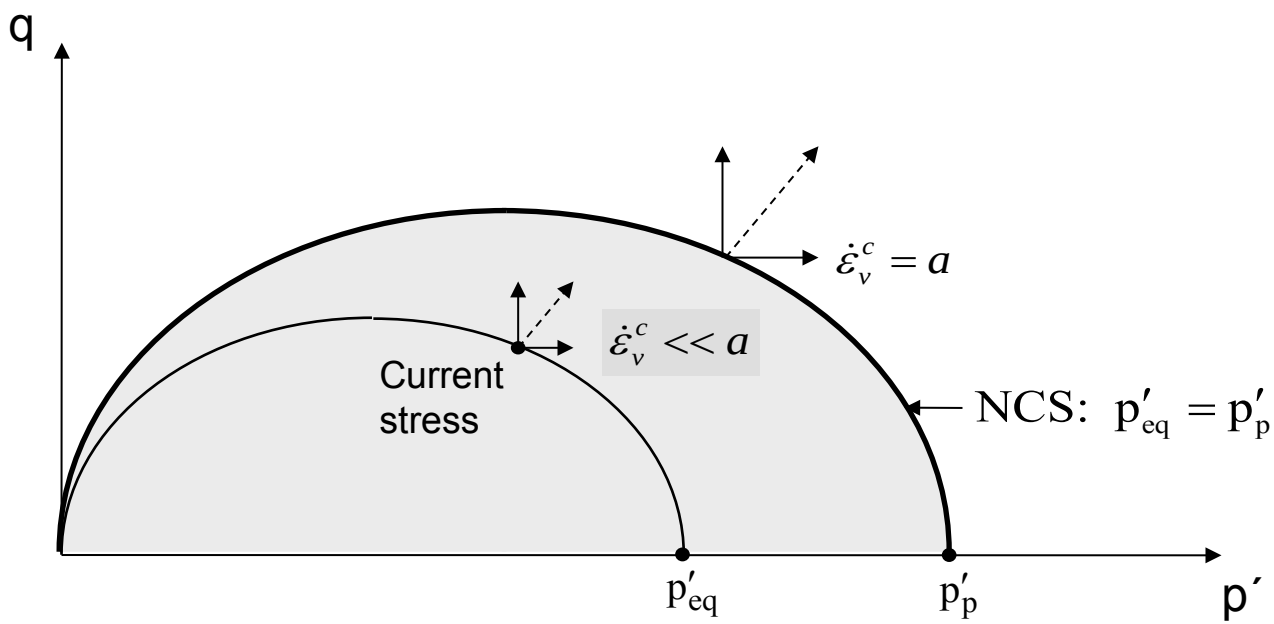
Modified creep index: $\mu^* = \dfrac{C_\alpha}{\ln 10}$



Isotropic preconsolidation pressure $p'_p = p' \exp\left(\dfrac{\dot{\varepsilon}_v^c}{\lambda^* - \kappa^*}\right)$

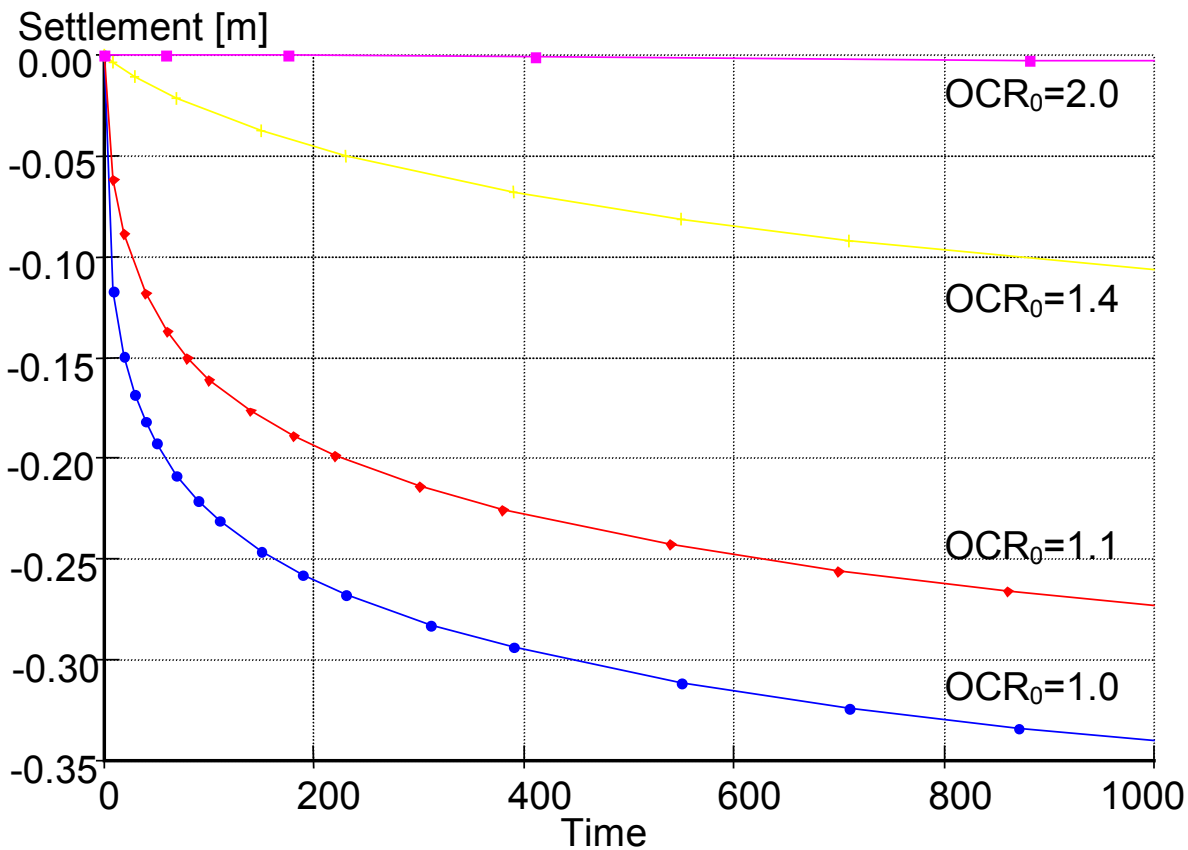
3D Soft Soil Creep model

Ellipses of Modified Cam Clay are taken as contours of volumetric strain rate



SSC model

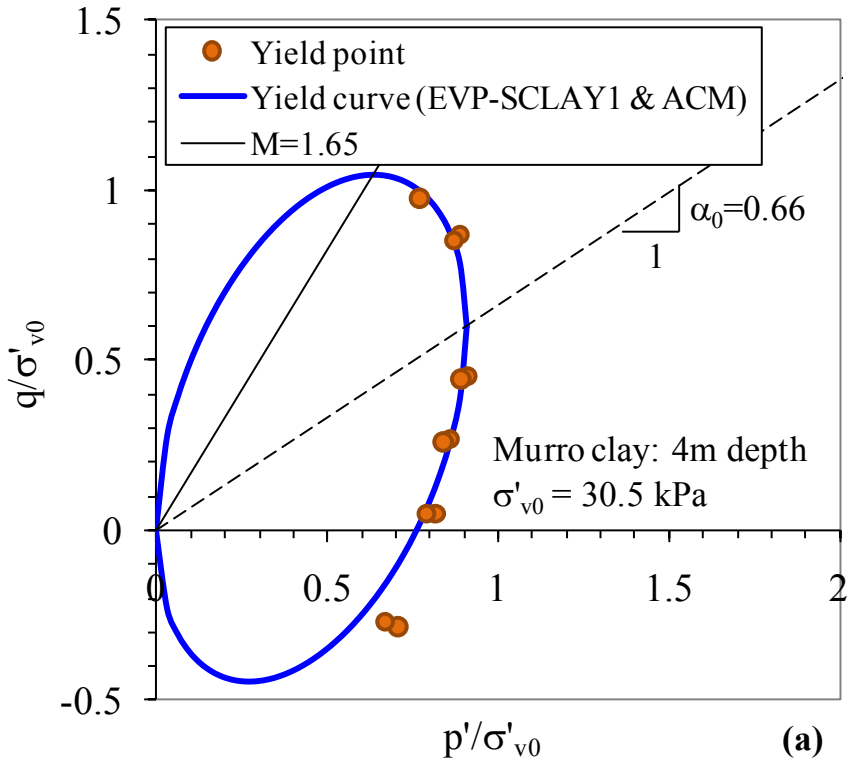
The role of OCR in self-weight loading and creep:

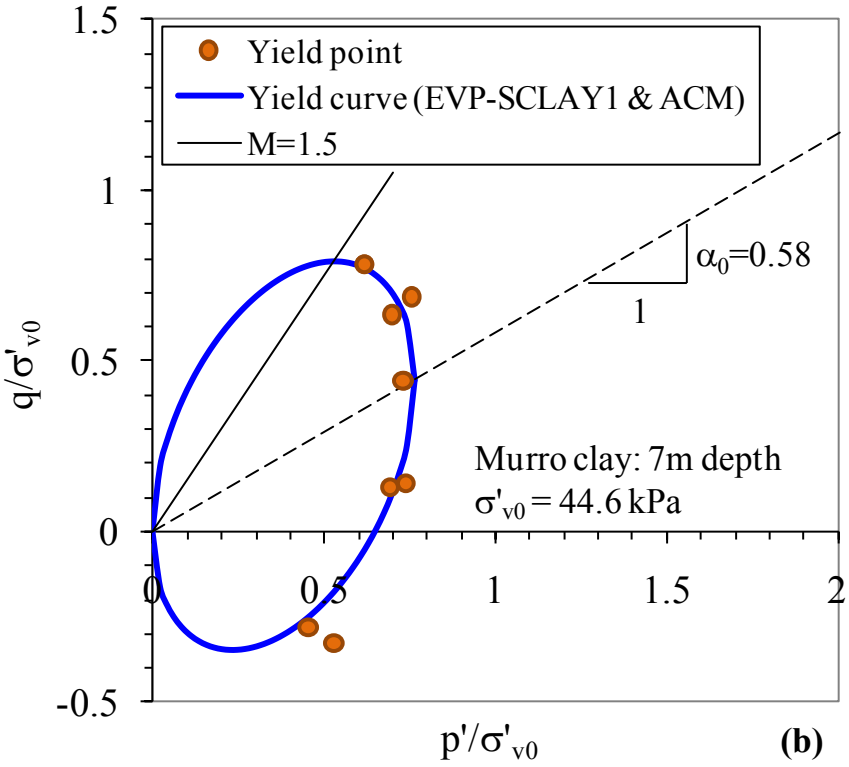


Settlement of 10m thick layer

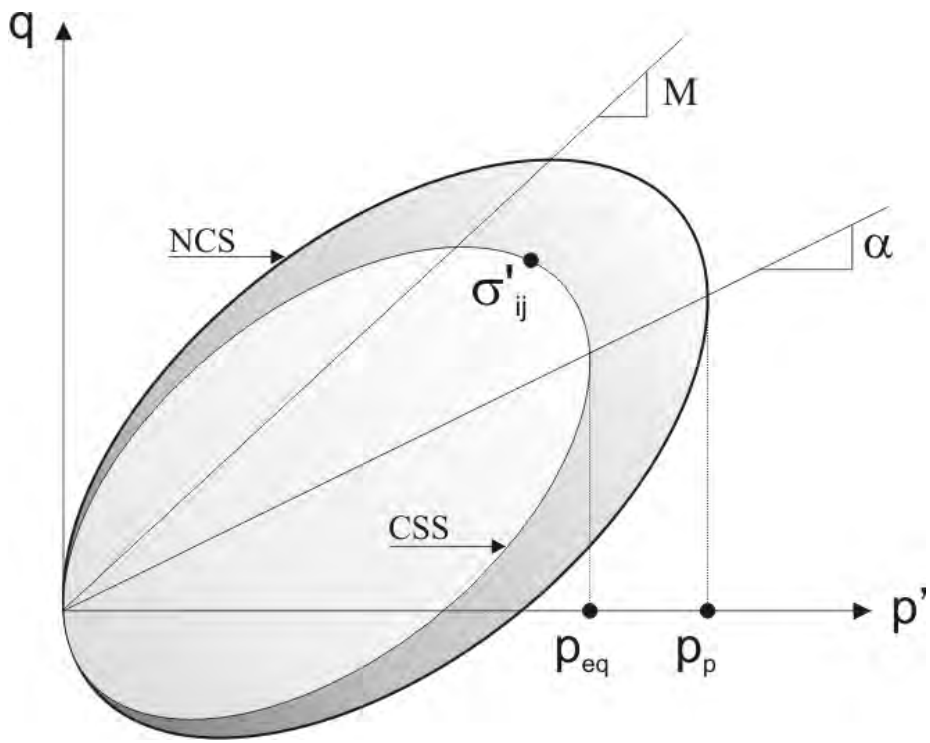
λ^*	0.10
κ^*	0.02
μ^*	0.005
ν_{ur}	0.15
c'	0.0 kPa
φ'	25°
ψ	0°
K_0^{nc}	0.677 $(1-\sin\varphi')$

Anisotropy of Murro Clay





Anisotropic Creep Model



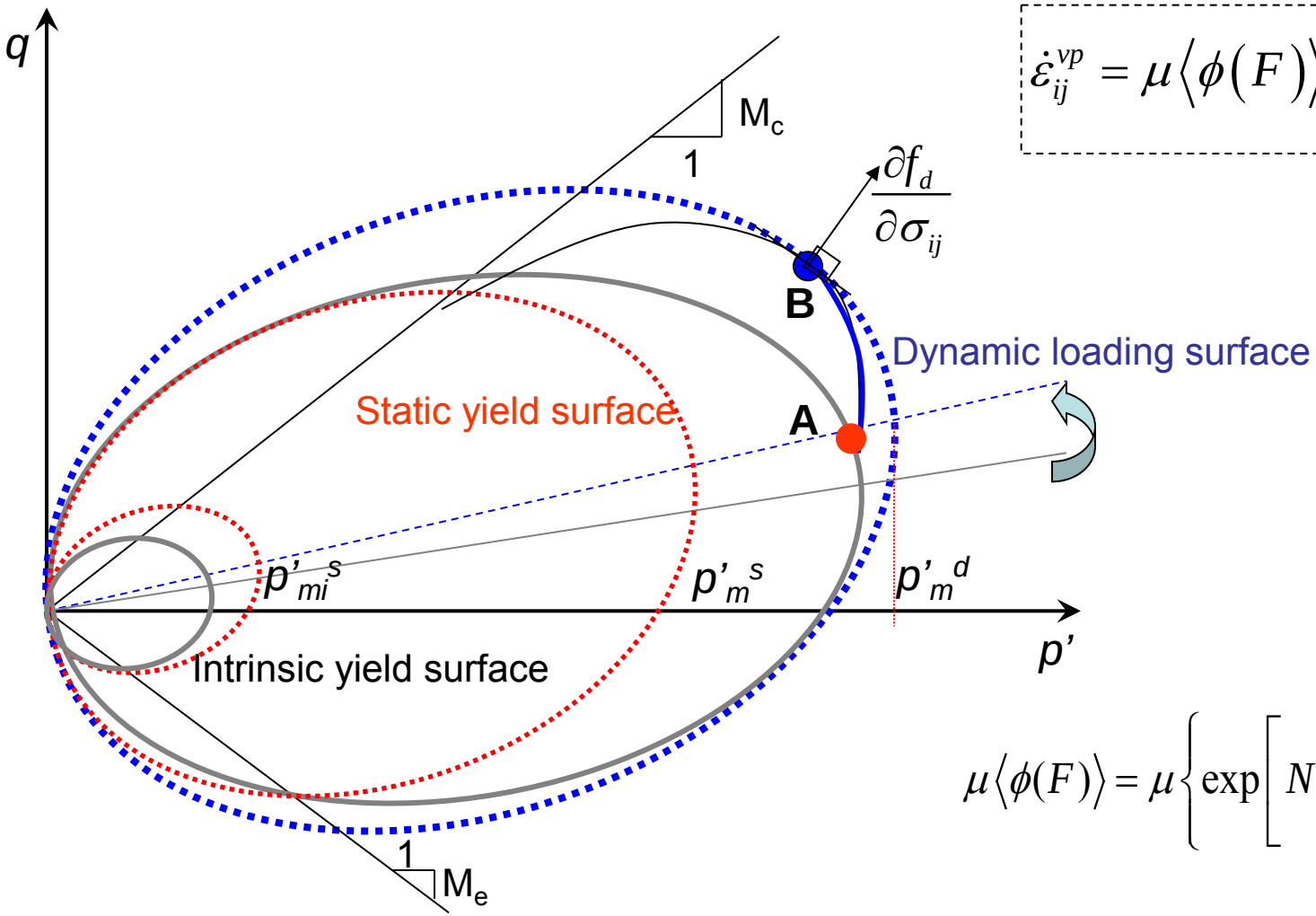
$$p'_{eq} = p' + \frac{(q - \alpha p')^2}{M^2 - \alpha^2} \frac{1}{p'}$$

(S-CLAY1)

$\alpha = 0$ gives isotropic model

The ellipse rotates with creep strains, therefore a so-called “rotational hardening law” is included in the formulation.

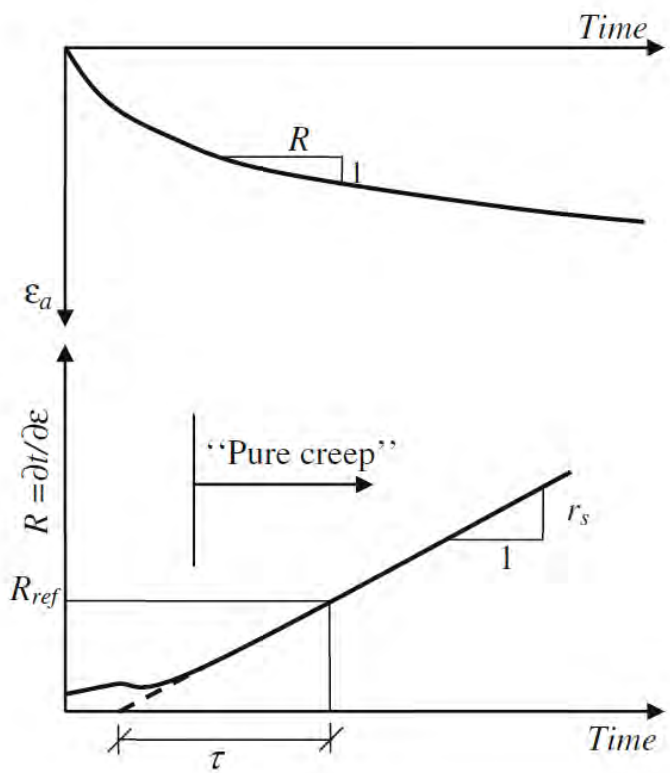
EVP-SCLAY1S

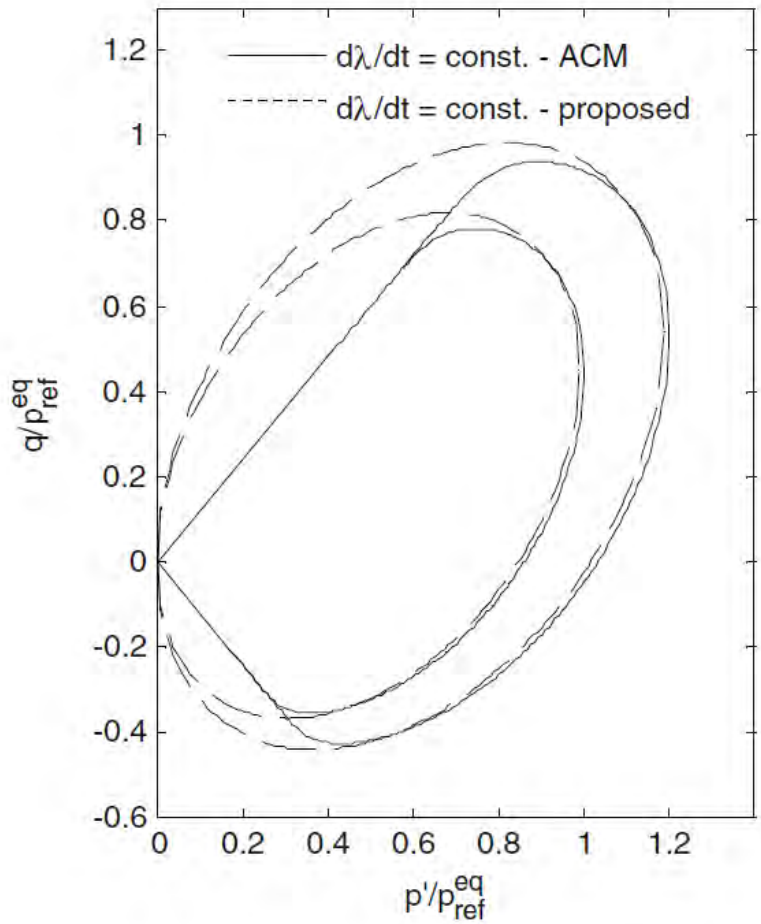


$$\dot{\varepsilon}_{ij}^{vp} = \mu\left\langle \phi(F) \right\rangle \frac{\partial f_d}{\partial \sigma_{ij}}$$

$$\mu\left\langle \phi(F) \right\rangle = \mu\left\{ \exp\left[N \cdot \left(\frac{p_m^d}{p_m^s} - 1 \right) \right] - 1 \right\}$$

nSAC model by Grimstad

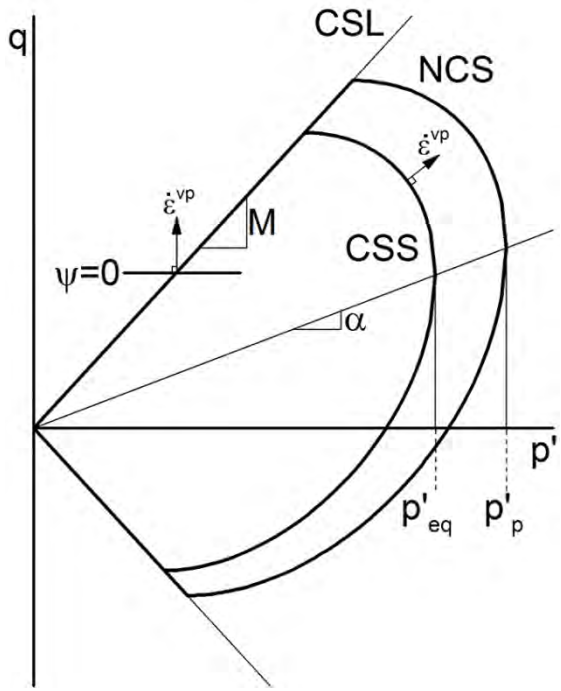




$$\frac{d\lambda}{dt} = \dot{\lambda} = \frac{1}{r_{si} \cdot \tau} \cdot \left(\frac{p^{eq}}{(1+x) \cdot p'_{mi}} \right)^{r_{si} \cdot \zeta_i} \cdot \frac{M_f^2 - \alpha_{K0NC}^2}{M_f^2 - \eta_{K0NC}^2}$$

ACM & Creep-SCLAY1

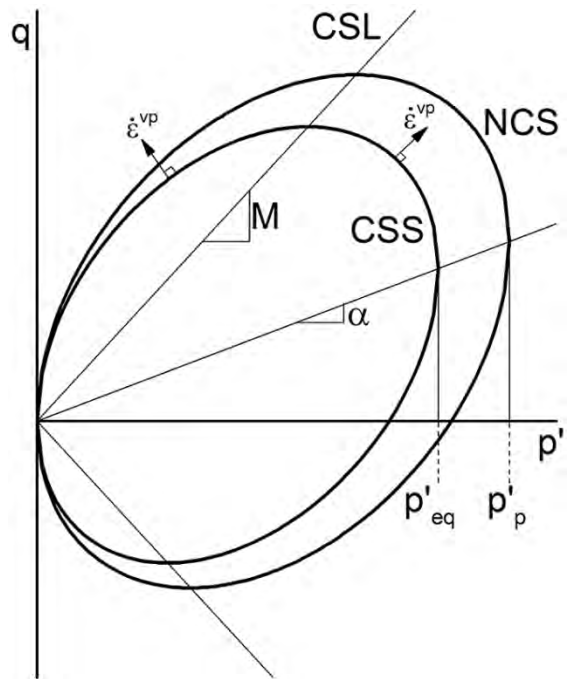
ACM



$$\dot{\varepsilon}^{c}_{vol} = \frac{\mu^*}{\tau}\left(\frac{1}{OCR^*}\right)^{\beta}$$

Leoni et al. (2008)

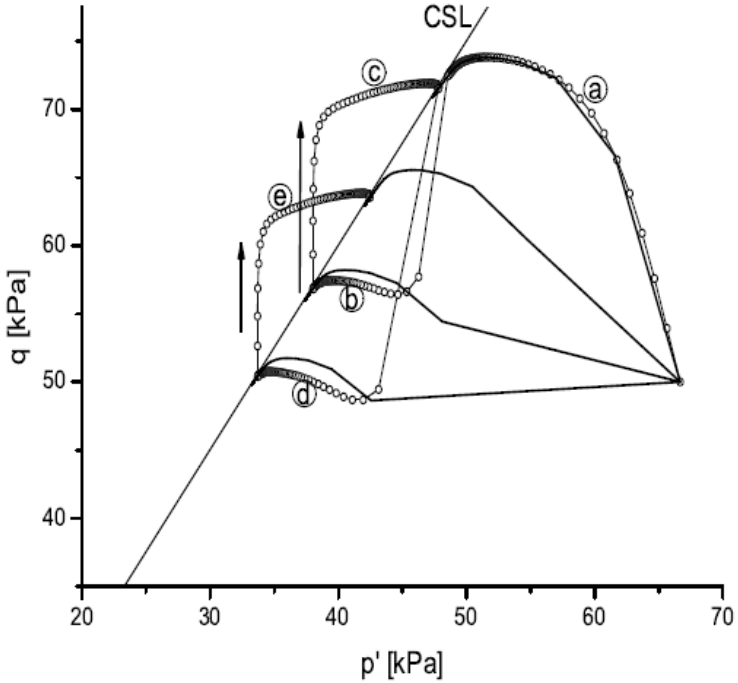
Creep-SCLAY1

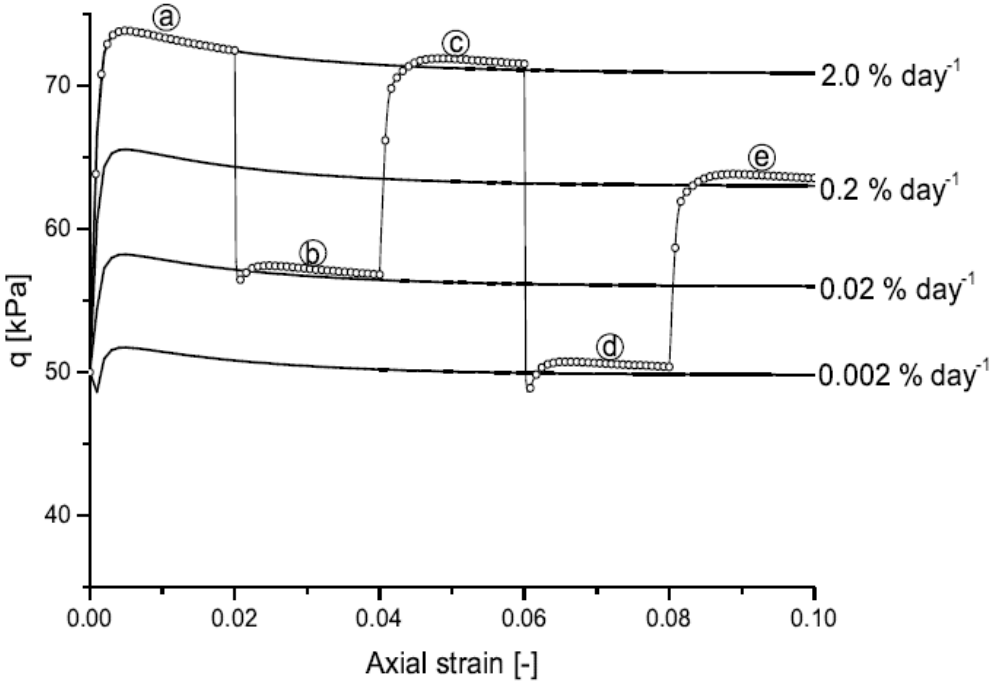


$$\dot{\varepsilon}^{c}_{v} = \frac{\mu^*}{\tau}\left(\frac{1}{OCR^*}\right)^{\beta}\left(\frac{M^2-\alpha^2_{K_0^{NC}}}{M^2-\eta^2_{K_0^{NC}}}\right)$$

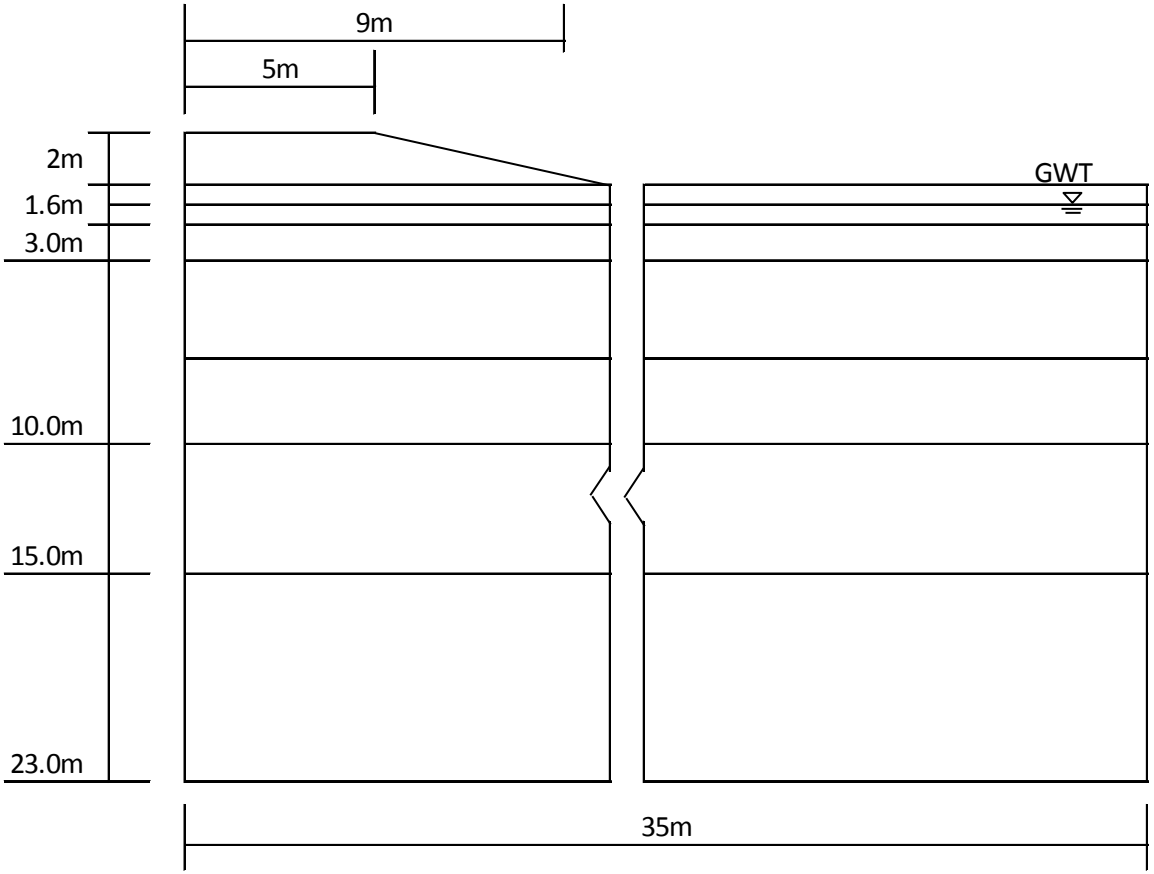
Sivasithamparam et al. (2013)

CREEP-SCLAY1

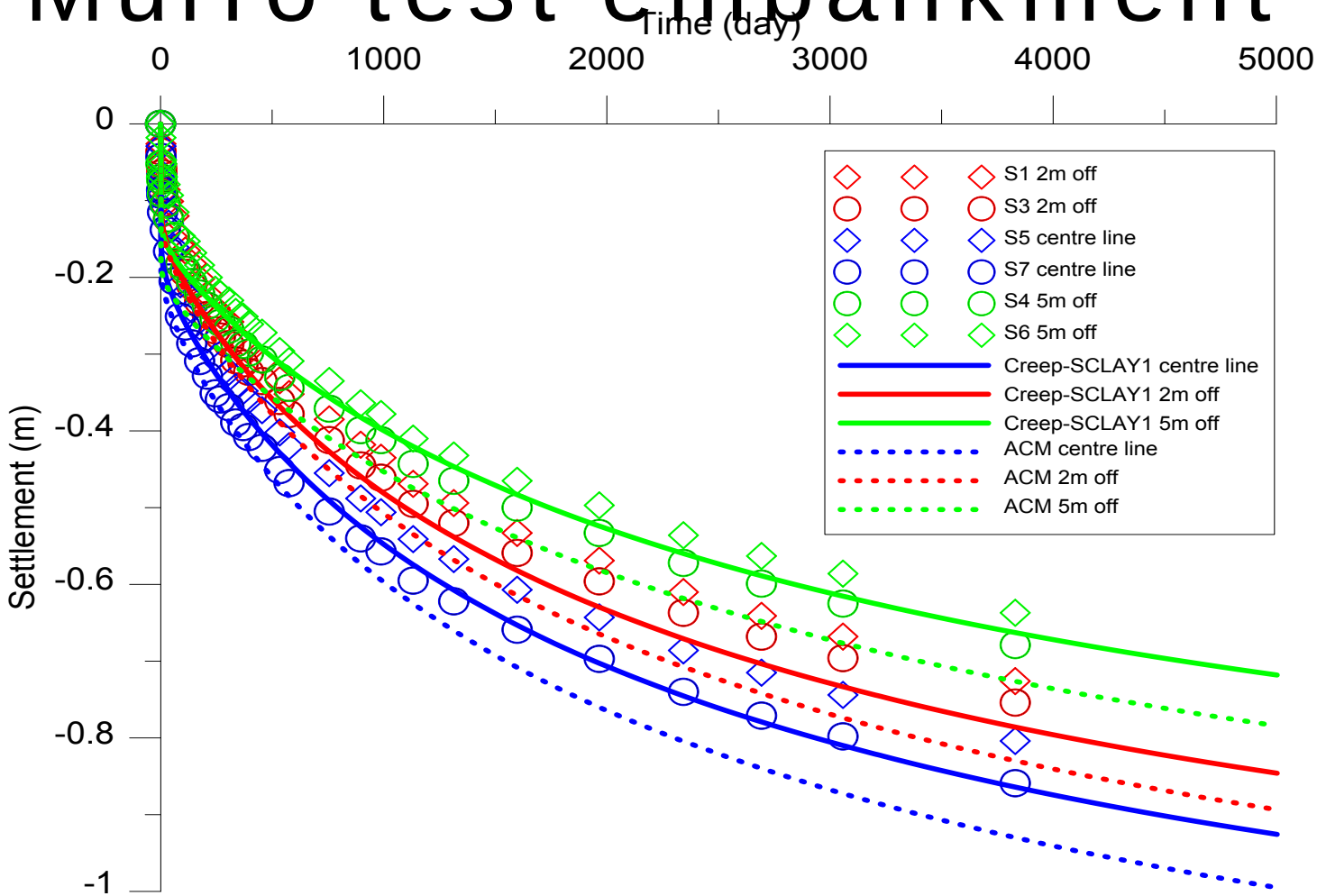




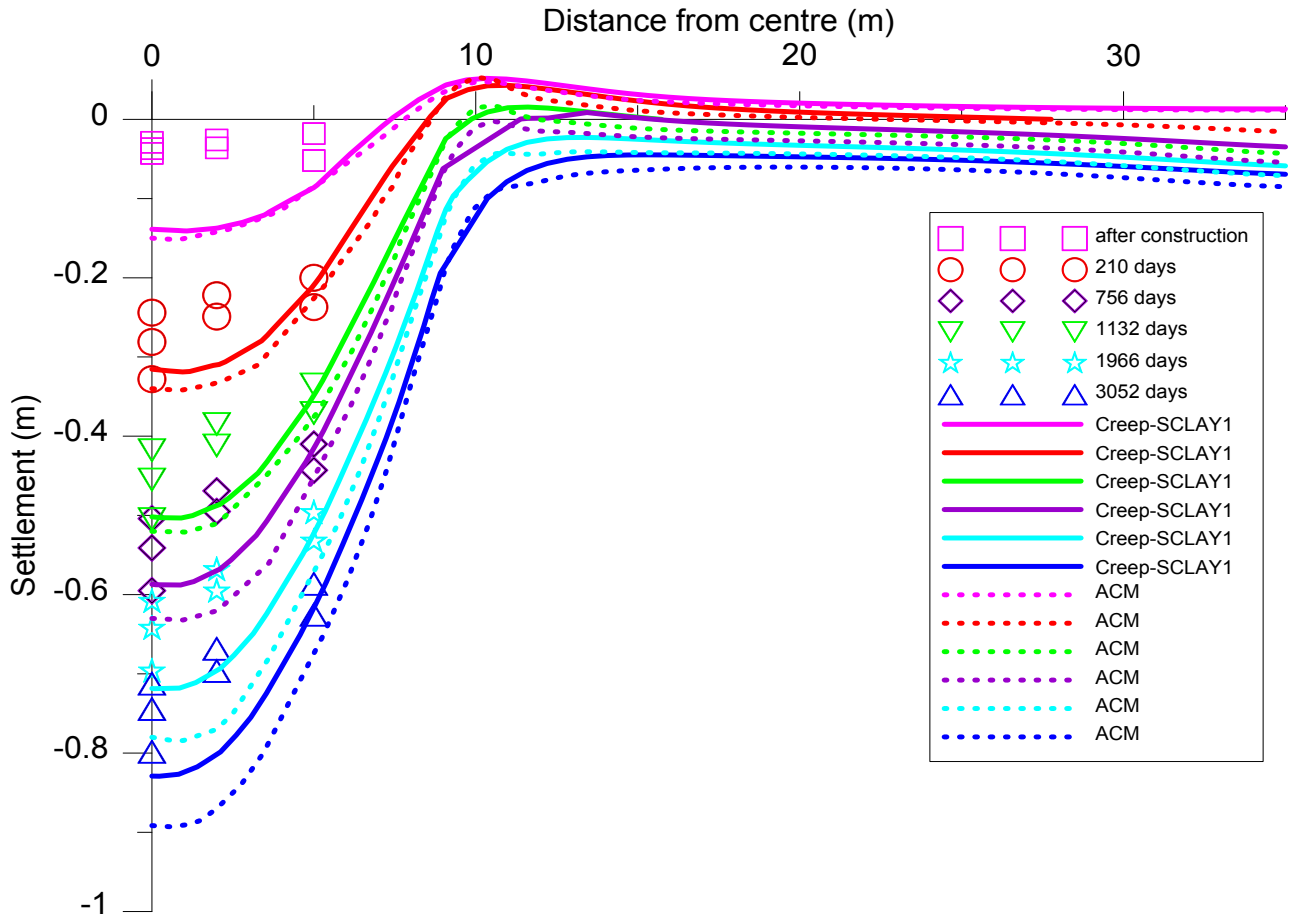
Murro test embankment



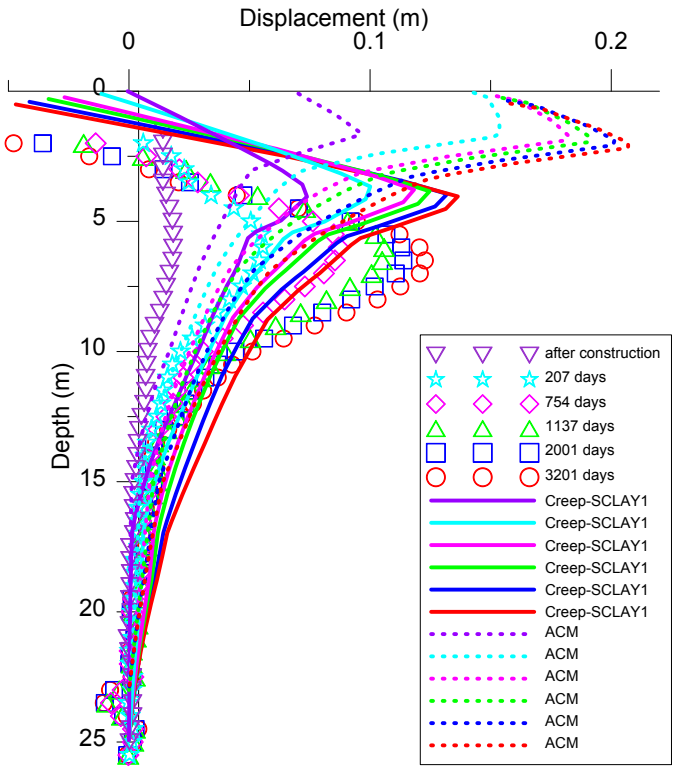
Murro test embankment

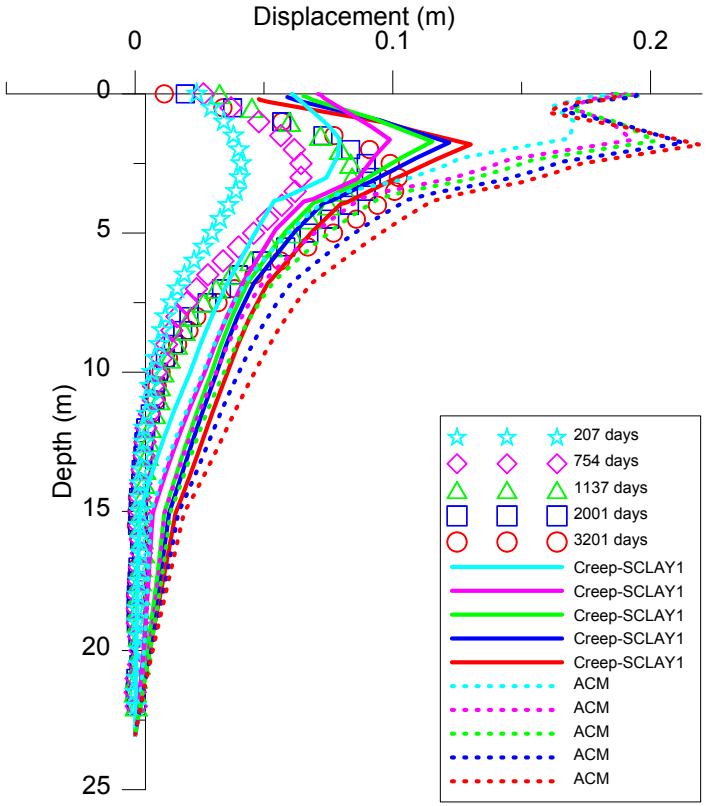


Murro test embankment



Murro test embankment





Conclusions

- Natural soft clays are complex materials (structured and rate-dependent)
- With recently developed constitutive models it is possible to model in a simple manner some of these features, such as
  - Initial anisotropy & evolution of anisotropy
  - Destructuration: strain softening, progressive failure
  - Rate-effects
- Validation of the models is on-going
- In parallel, we work on objective parameter determination, in order to ease the adaptation of these models in industry

Future work

- Need to improve sampling procedures and testing accuracy
- Micromechanical understanding necessary to explain complex phenomena
- None of the current models can predict unloading/reloading and cyclic behaviour in a satisfactory way, so further developments necessary
- Further validation of the model needed at boundary value level against model tests and instrumented test structures
  - Appeal for long-term measurements & extensive soil characterization

Identifying parameter of creep by GA optimization

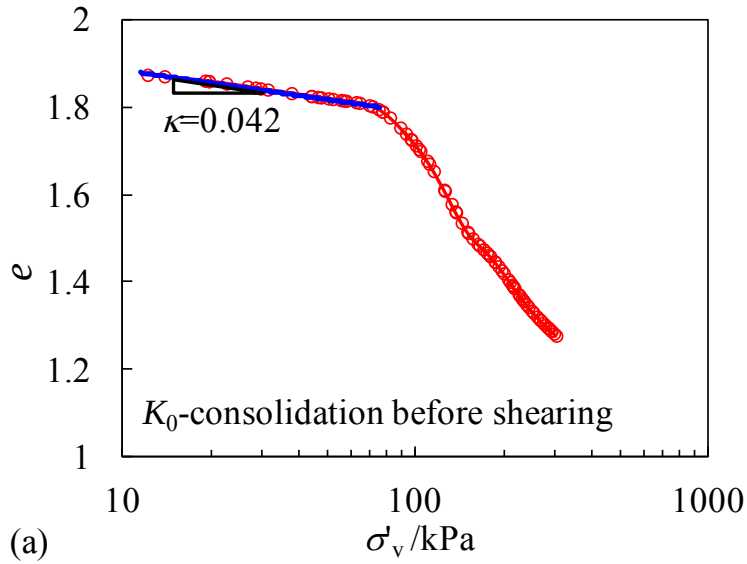
Zhenyu YIN and Yinfu JIN

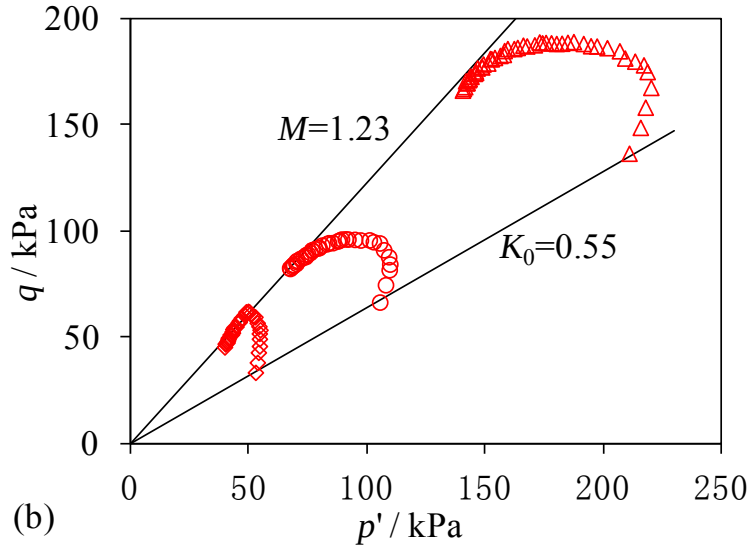
Department of Civil Engineering, Shanghai Jiao Tong University

8 Jan. 2015, NGI

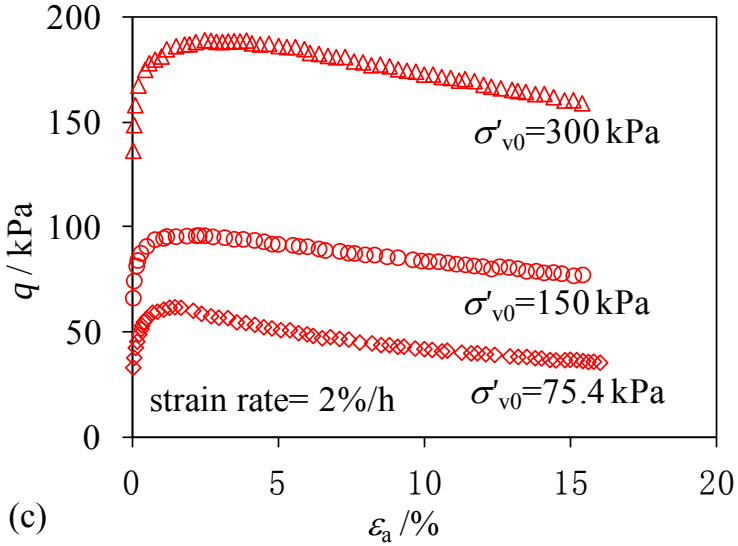
Difficulties of measing parameters

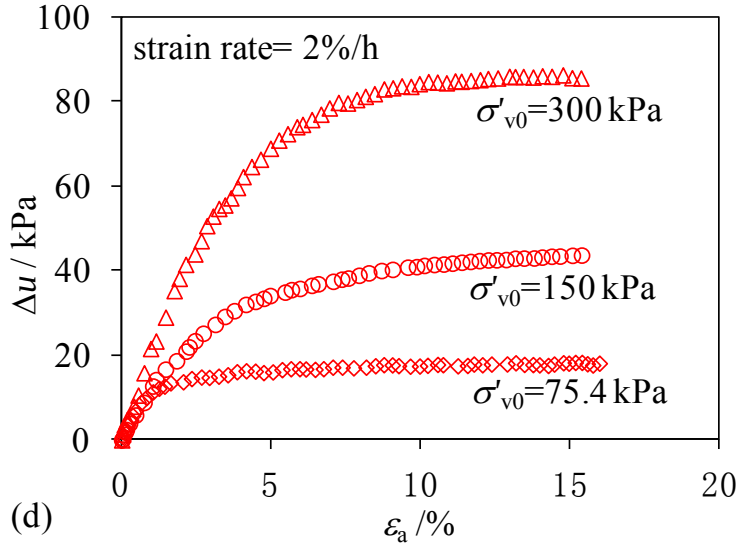
Typical laboratory tests of clay





e_0, κ
λ, M
…





C_{ae}?
S_t?
…

2 Related parameters

2.1 Parameters

$$\dot{\varepsilon}_v^{vp} = \frac{C_{\alpha e}}{(1+e_0)\tau}\left(\frac{\sigma'_v}{\sigma'_p}\right)^{\frac{\lambda-\kappa}{C_{\alpha e}}}$$

Parameters

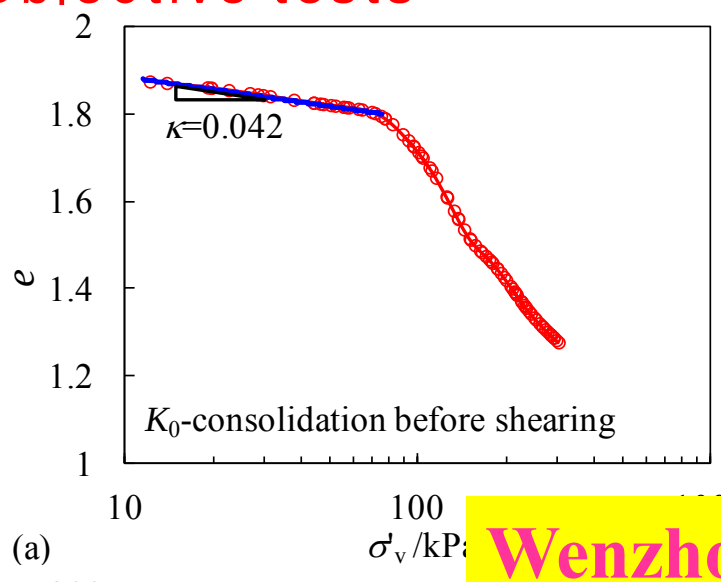
- **Creep related**
  - C_{ae}
- **Destructuration related**
  - χ_0
  - ξ, ξ_d

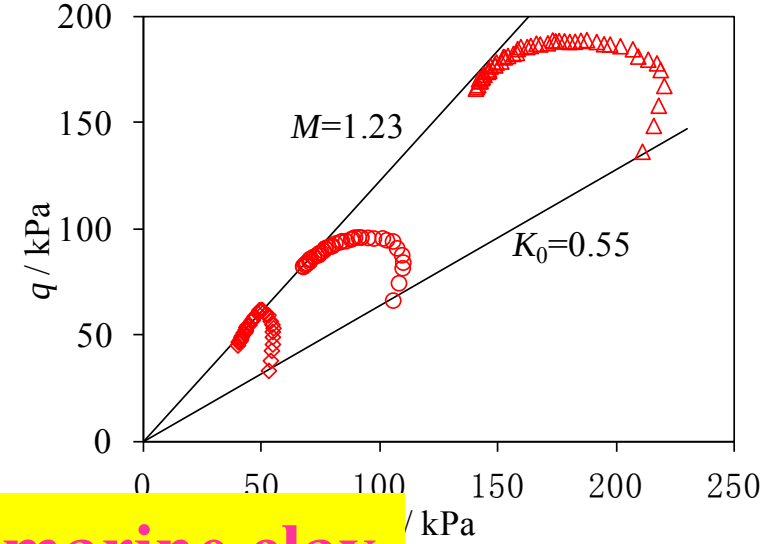
Bonding: $\sigma_p^r = (1+\chi)\sigma_{pi}^r$

Debonding: $\chi = \chi_0 \exp\left[-\left(\xi\varepsilon_v^{vp} + \xi_d\varepsilon_d^{vp}\right)\right]$

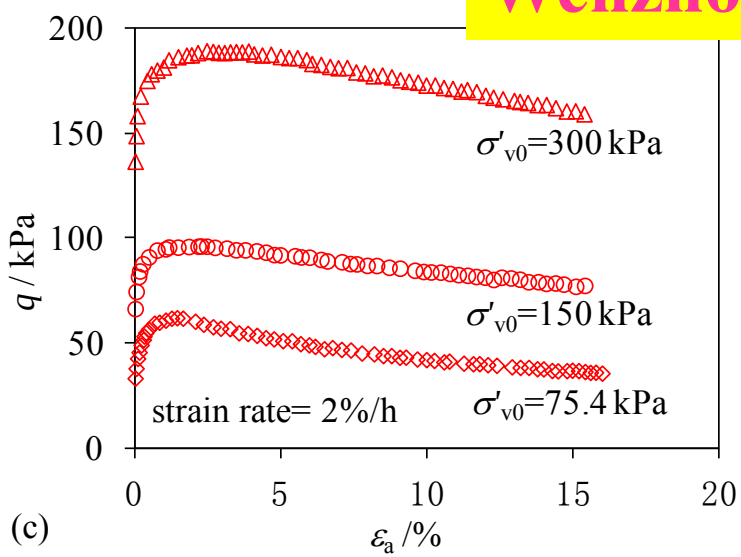
2 Related Parameters

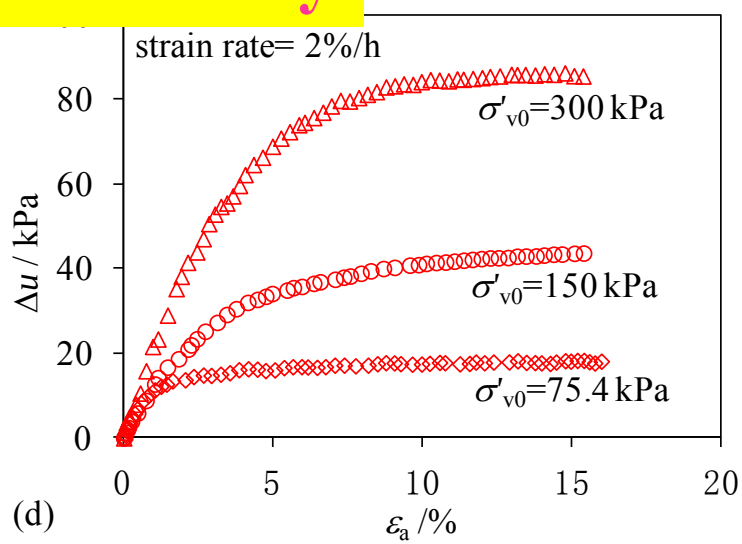
2.2 Objective tests





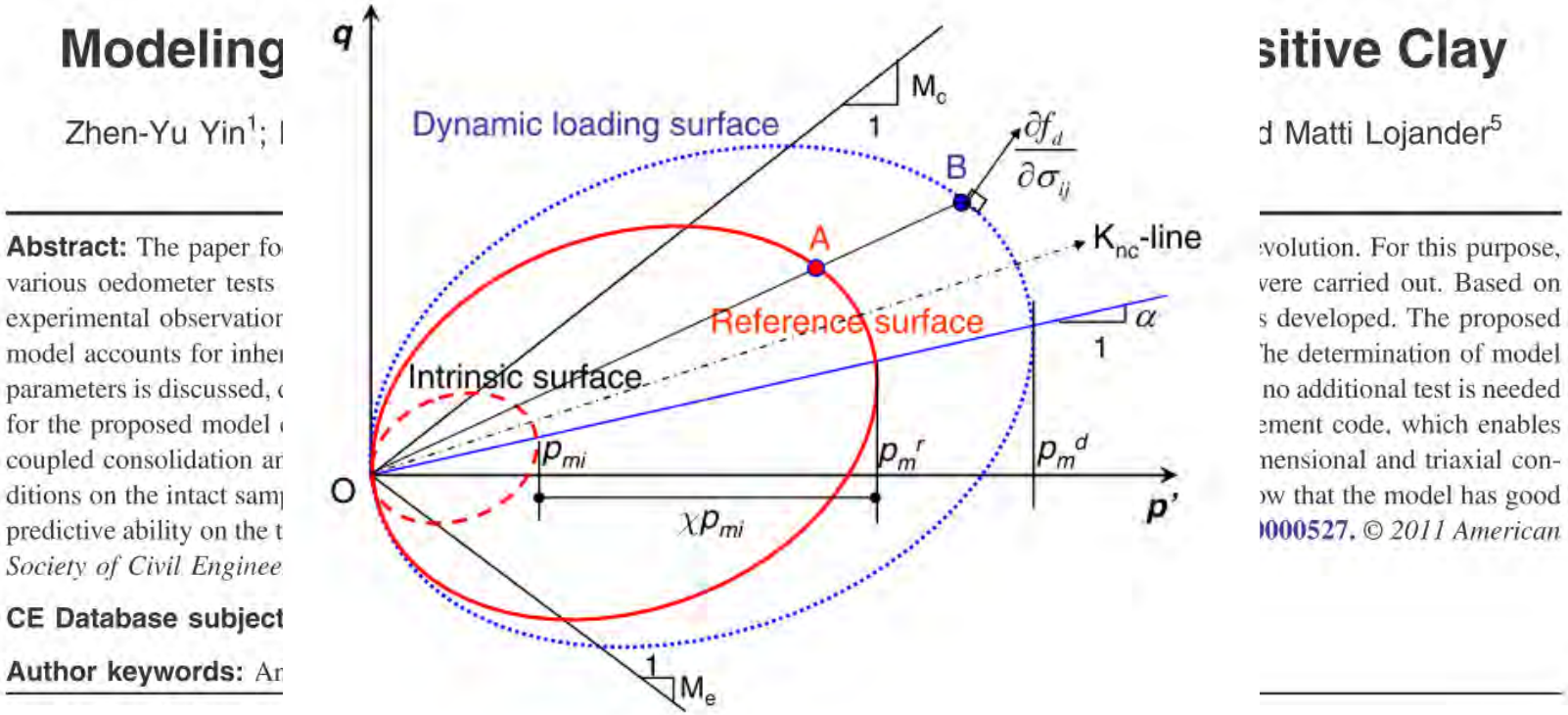
Wenzhou marine clay





2 Related Parameters

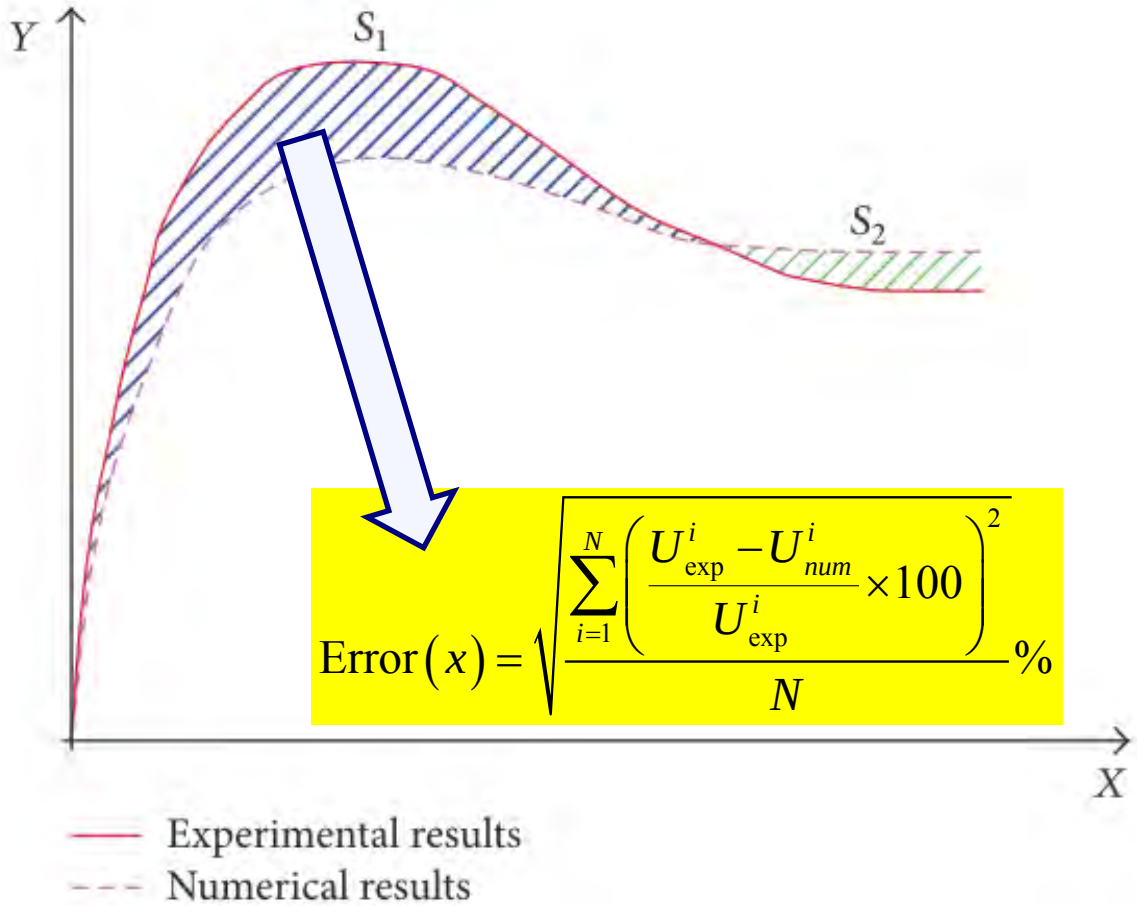
2.3 Adopted constitutive model



ANICREEP Model developed by Yin et al. 2011

3 Optimization method

3.1 Error function

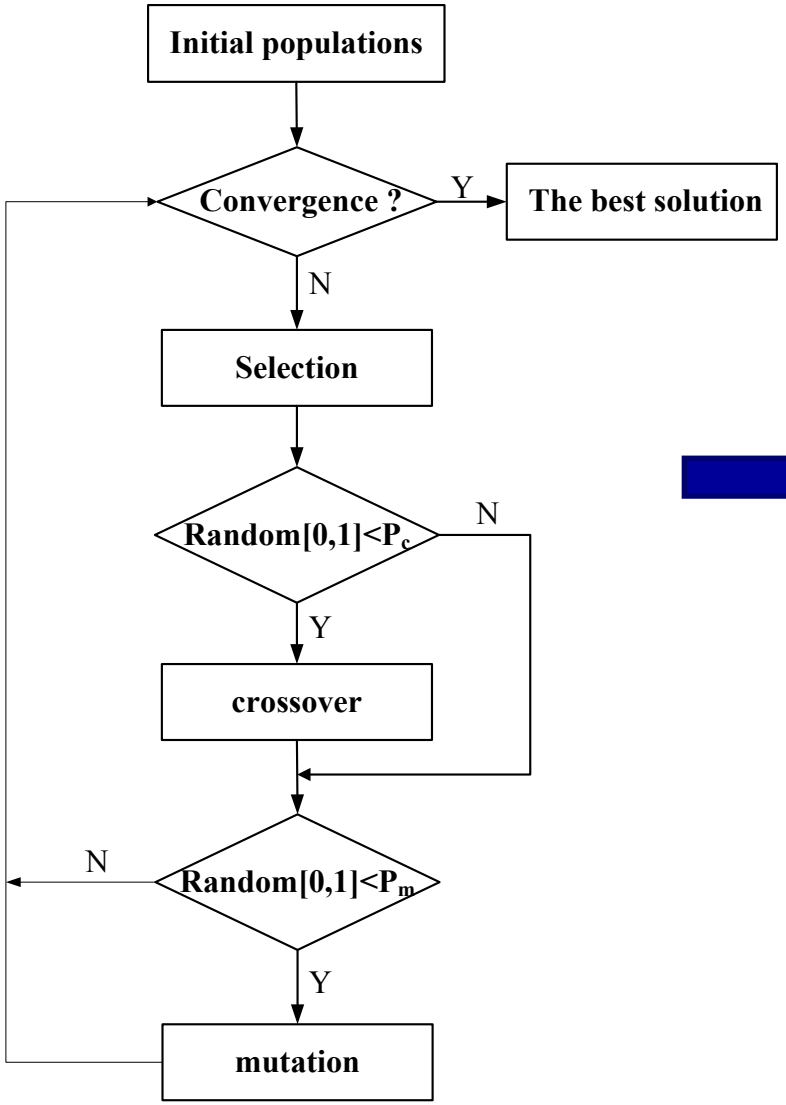


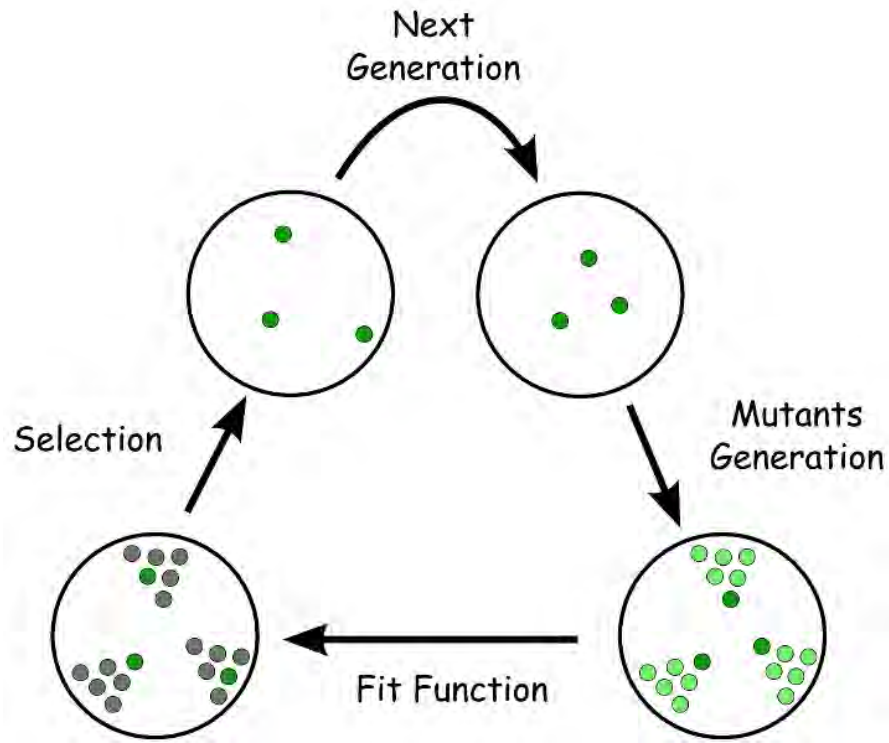
$$\mathrm{Error}(x) = \sqrt{\frac{\sum_{i=1}^{N}\left(\frac{U_{\exp}^{i} - U_{num}^{i}}{U_{\exp}^{i}} \times 100\right)^2}{N}}\,\%$$

Levasseur et.al [2008]

3 Optimization method

3.2 Genetic algorithm



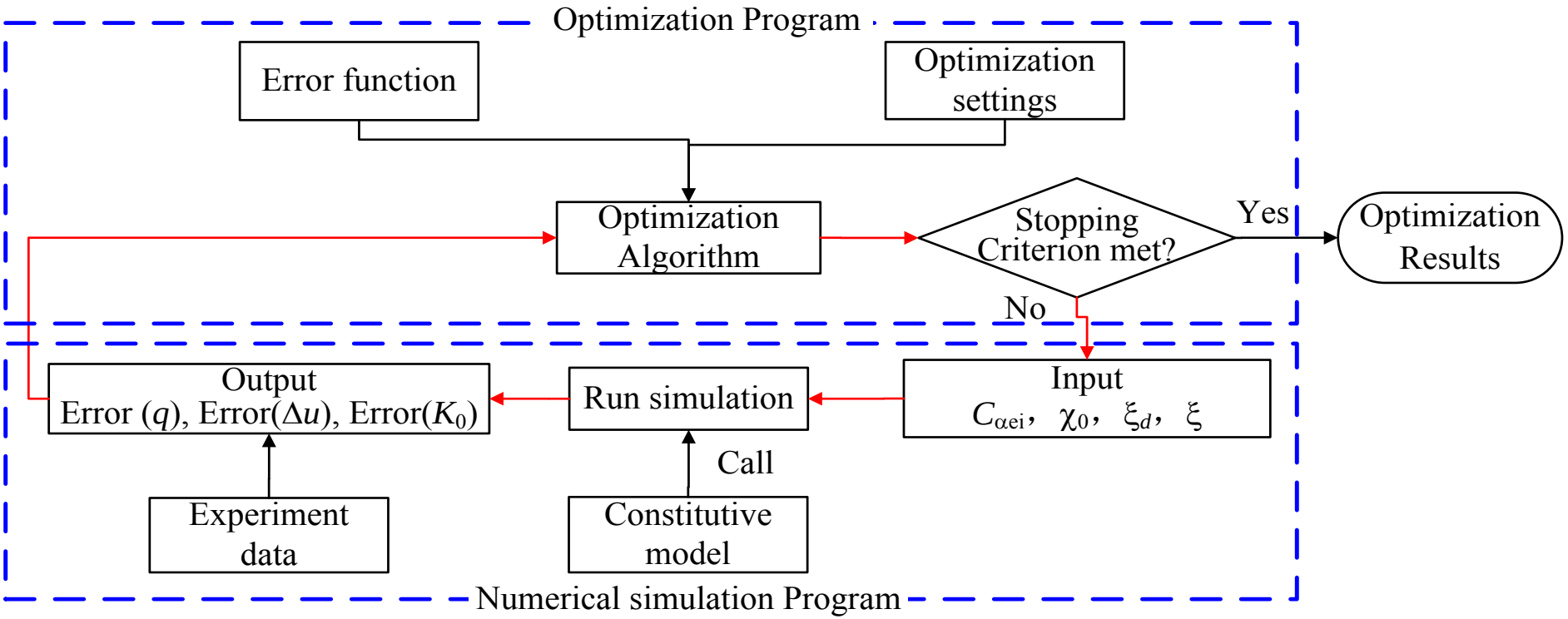


4 Optimization Procedure

4.1 Procedure

Mono-objective

$$\min\left[\mathrm{Error}(x)\right]=\min\left[\frac{\mathrm{Error}(K_0)+\mathrm{Error}(q)+\mathrm{Error}(\Delta u)}{3}\right]$$



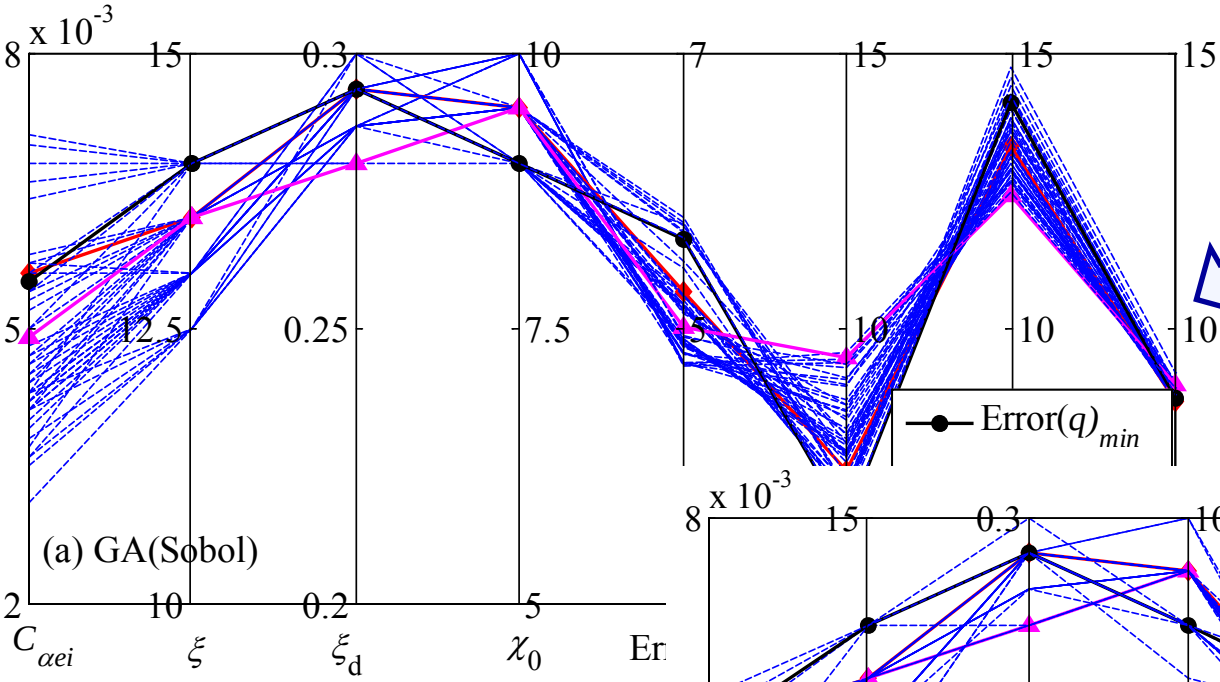
4 Optimization Procedure

4.2 Range of optimization parameters

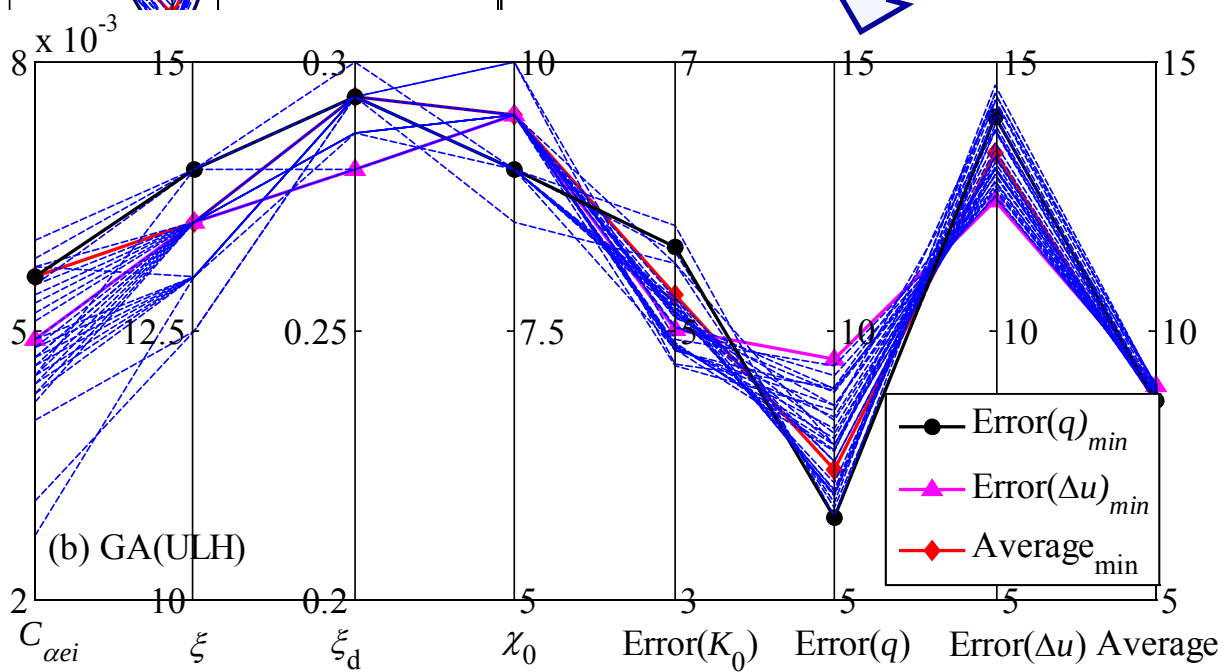
Parameters	Lower bound	Upper bound	Step
$C_{\alpha\mathrm{ei}}$	0.0001	0.1	0.0001
χ_0	0	50	0.5
ξ	0	20	0.5
ξ_d	0	0.5	0.02

5 Optimization Results

5.1 Results of optimization



Error<=9%



5 Optimization Results

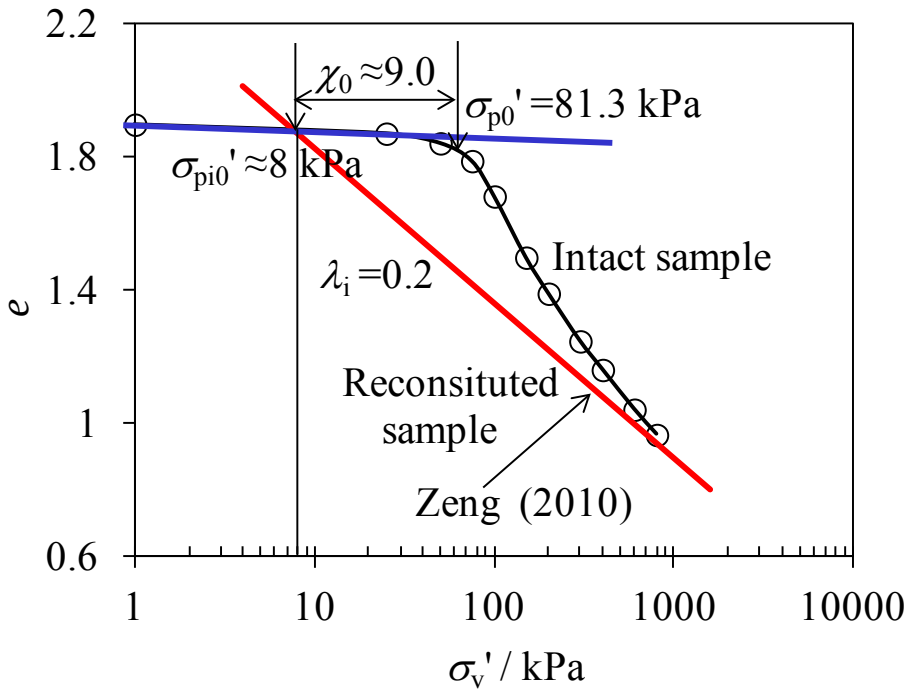
5.1 Results of optimization

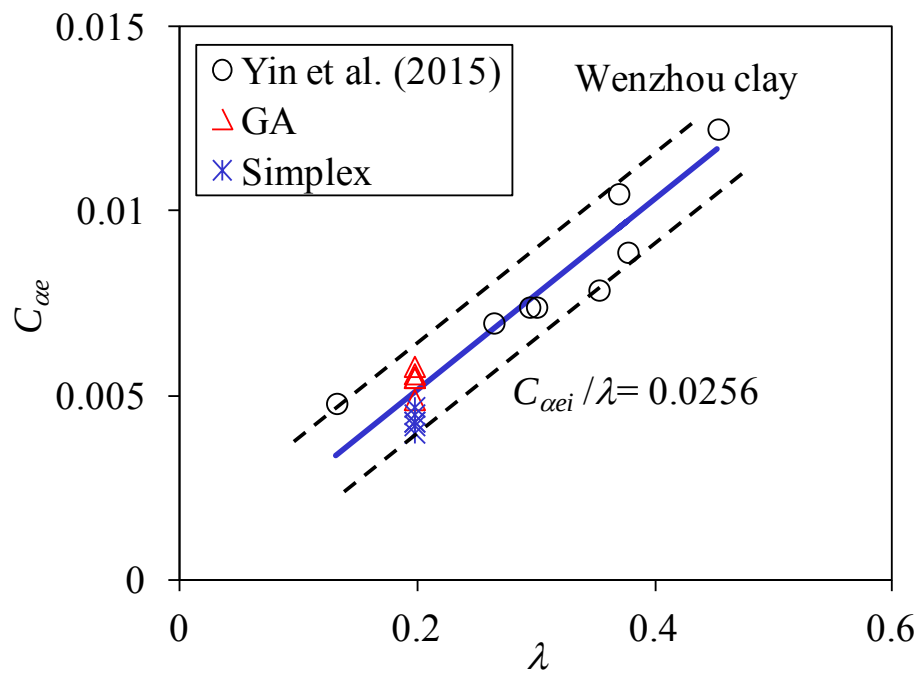
Initialization method	Optimal parameters				Average error /%
	$C_{\alpha ei}$	χ_0	ξ	ξ_d	
Sobol	0.0056	8.5	13.5	0.28	8.66
ULH	0.0056	8.5	13.5	0.28	8.66

Minimum average Error

6 Validation

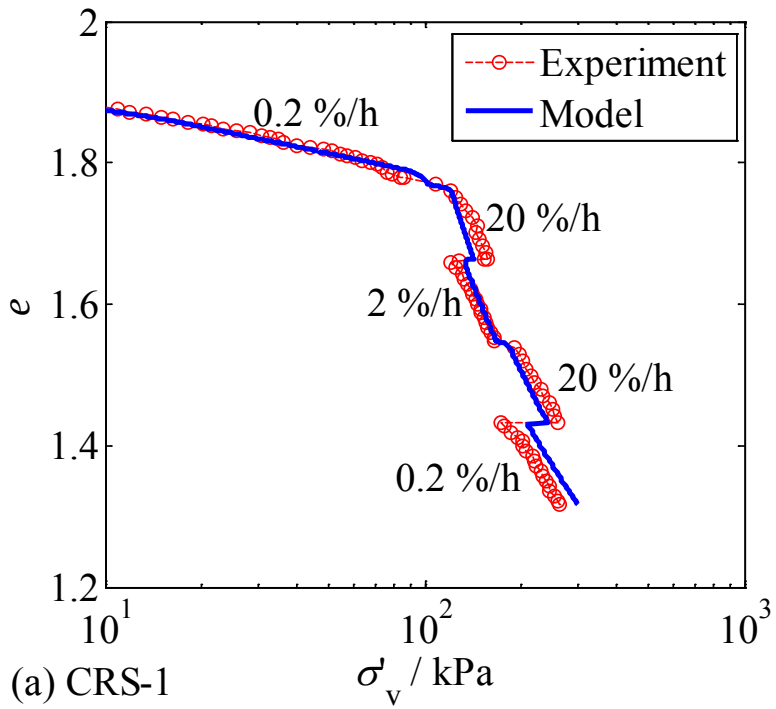
6.1 Validation based on experimental measurements



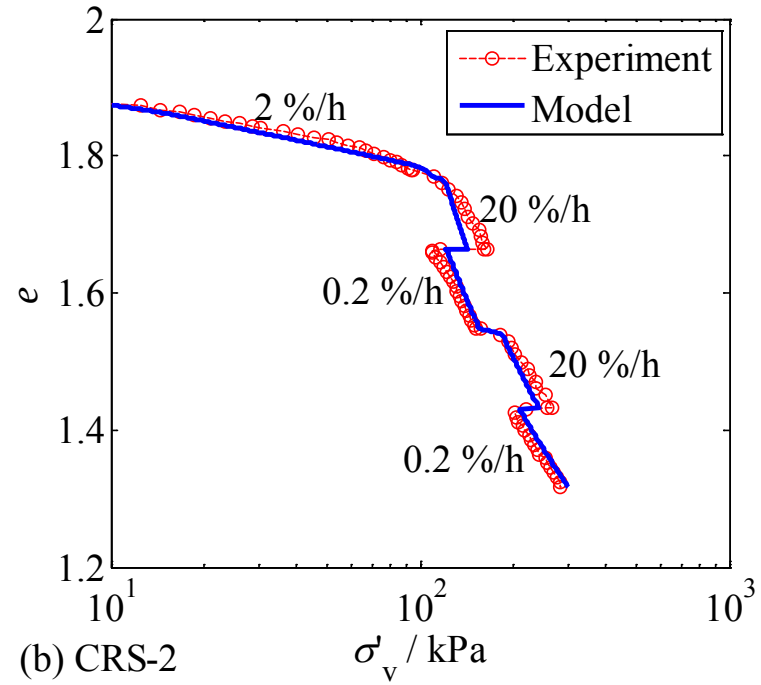


6 Validation

6.2 Validation based on simulations of other tests



(a) CRS-1

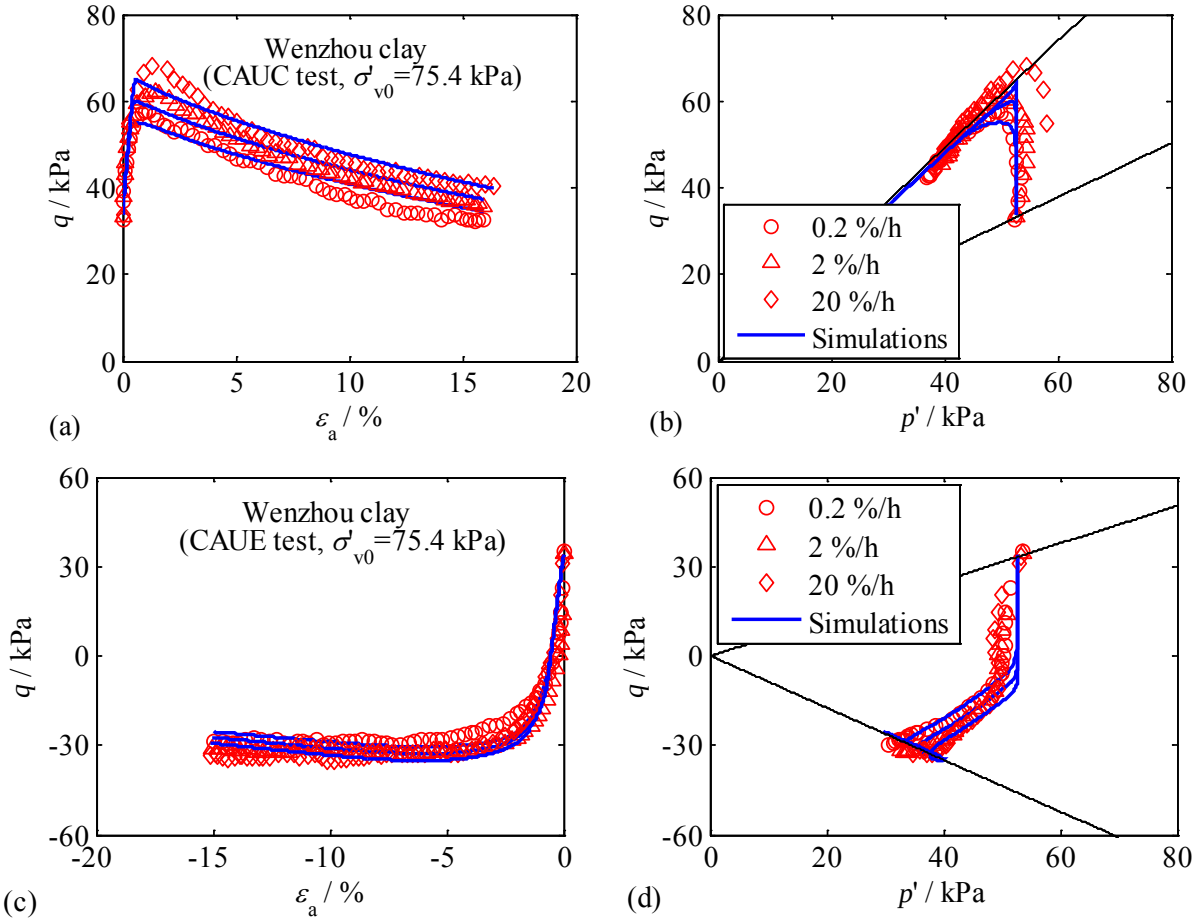


(b) CRS-2

1D CRS tests

6 Validation

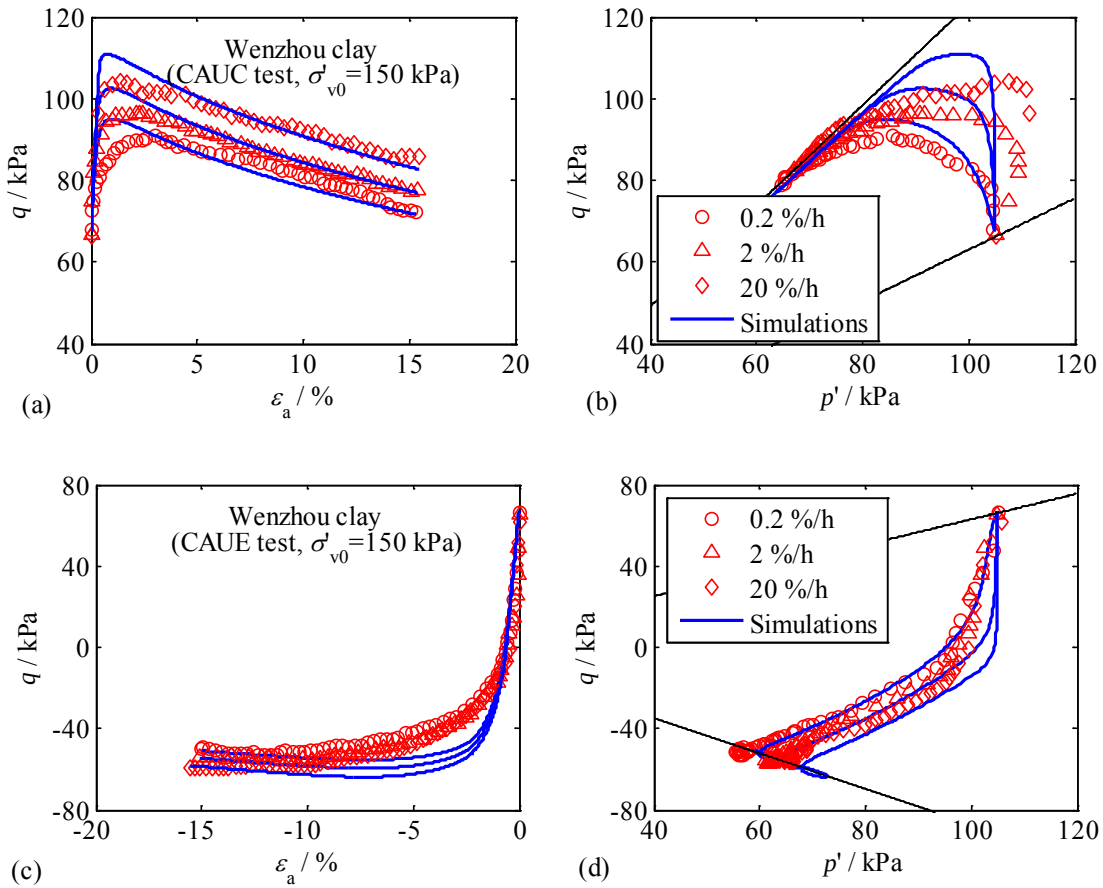
6.2 Validation based on simulations of other tests



3D CRS tests: Compression and extension (σ'_{v0}=75.4 kPa)

6 Validation

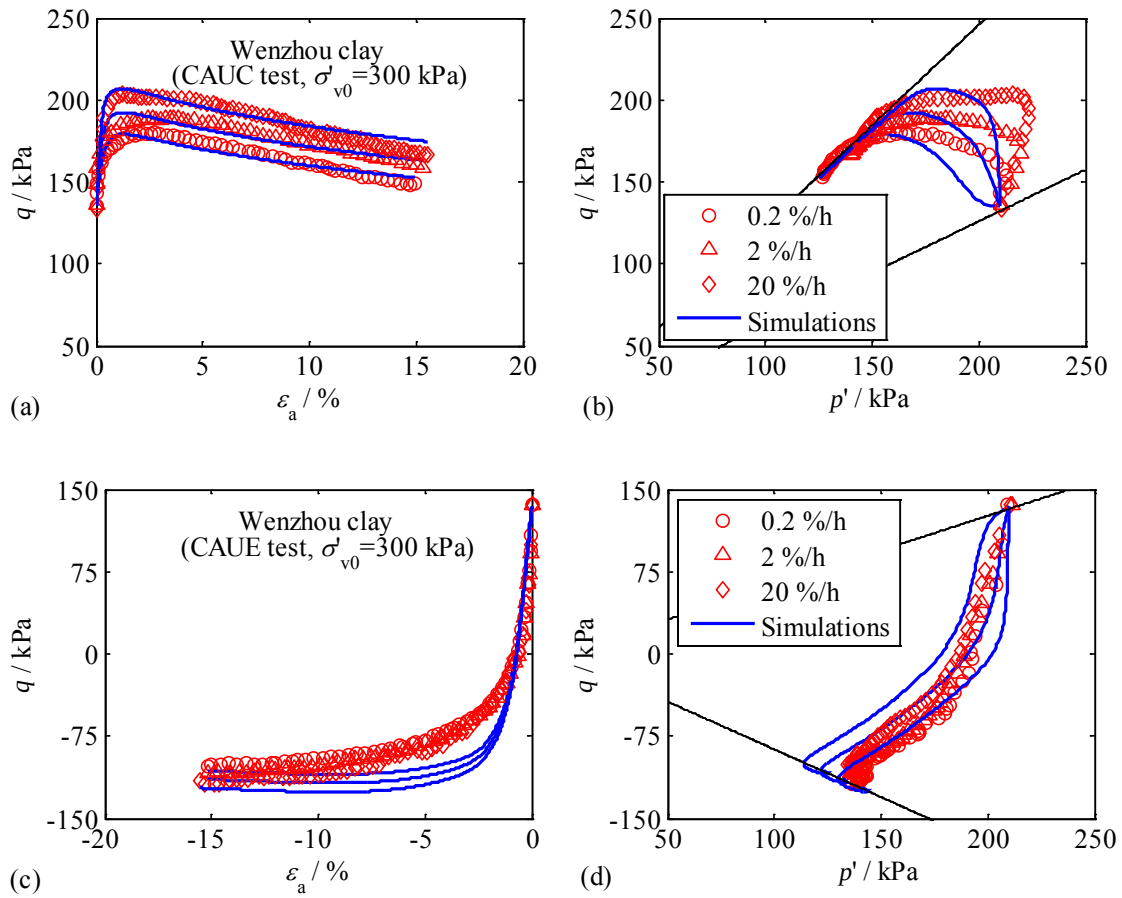
6.2 Validation based on simulations of other tests



3D CRS tests: Compression and extension (σ'_{v0}=150 kPa)

6 Validation

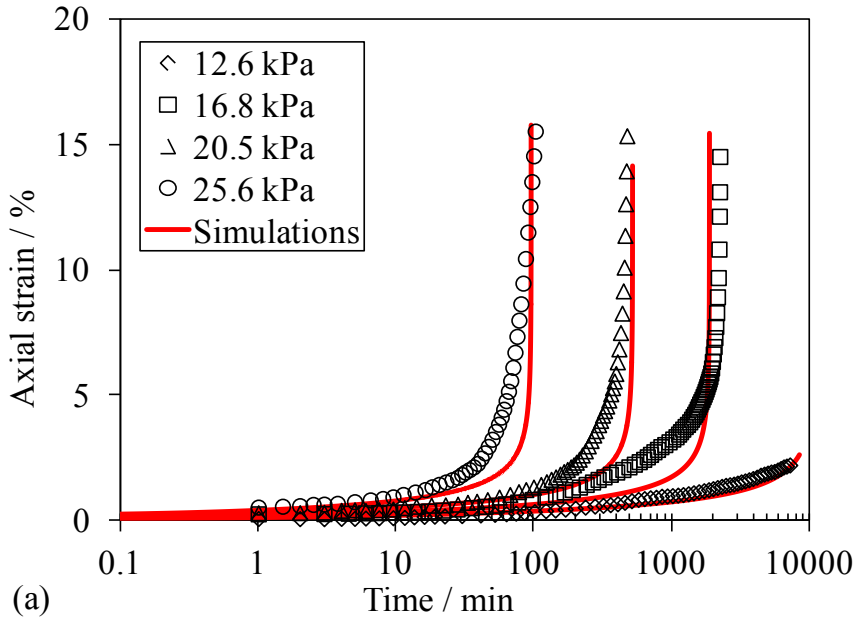
6.2 Validation based on simulations of other tests



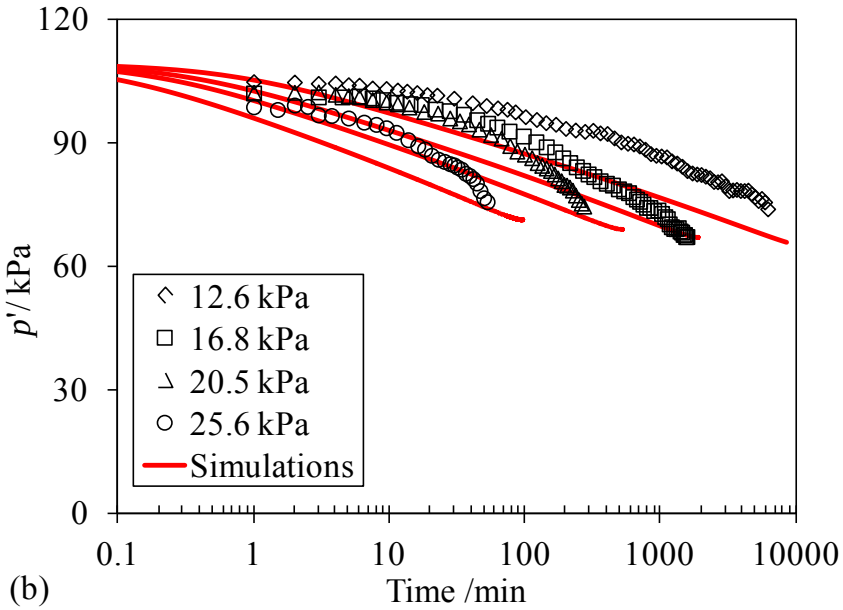
3D CRS tests: Compression and extension (σ'_{v0}=300 kPa)

6 Validation

6.2 Validation based on simulations of other tests



(a)



(b)

3D Creep tests

7 Conclusions

The genetic optimization provides us an efficient and reliable way to identify the creep and destructuration related parameters based on only the standard laboratory tests

Thank you very much
for your attention !

NTNU
Kunnskap for en bedre verden

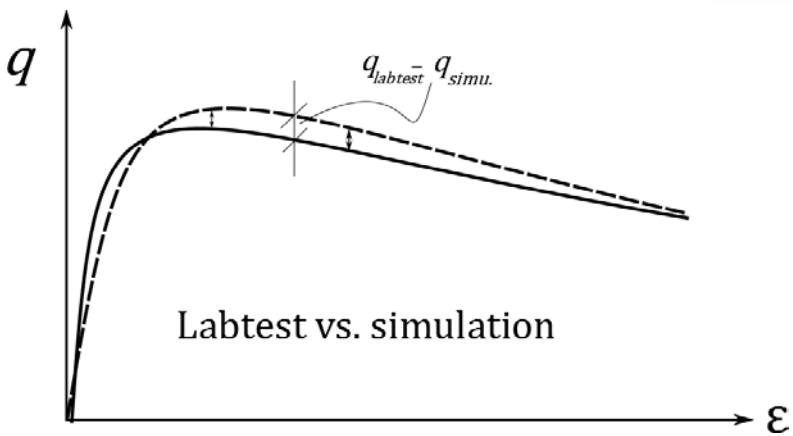
Optimization procedure for determining internal model parameters

J.A. Rønningen

Background

The objective is to create a tool in the programming language Fortran for determining soil model input parameters. The goal is to use this code to replace lesser known input parameters with well known quantities within the industry.

It is used together with a “framework” for implementing constitutive models for use in PLAXIS using both Fortran and MATLAB in combination.



$$\min_{x \in \mathbb{R}} f(\boldsymbol{x}) = \sum_{i=1}^{N} \left(\frac{q_{i,lab} - q_{i,sim}}{q_{i,lab}} \right)^2$$

The optimization tries to find a minimum of one function $f(\boldsymbol{x})$ with lower bounds $\boldsymbol{x}_{\min}$ and higher bounds $\boldsymbol{x}_{\max}$. It is necessary to provide an initial guess $\boldsymbol{x}_0$.

The $\boldsymbol{x}$ will be the input parameters, and $f(\boldsymbol{x})$ will typically be the overall difference (least squares) between simulations and lab test curves.

NLOpt – Nonlinear optimization library (S.G. Johnson, MIT)

A library compatible with many programming languages (C, C++, Fortran, MATLAB/GNU Octave, Python and several others)

Provides many different algorithms, some of which offer only local convergence, others global with or without the need for derivates.

Can switch between the algorithms by changing only one constant in the code.

Global optimization
DIRECT and DIRECT-L
Controlled Random Search (CRS) with local mutation
MLSL (Multi-Level Single-Linkage)
StoGO
ISRES (Improved Stochastic Ranking Evolution Strategy)
ESCH (evolutionary algorithm)

Local derivative-free optimization
COBYLA (Constrained Optimization by Linear Approximations)
BOBYQA
NEWUOA
PRAXIS (Principal Axis)
Nelder-Mead Simplex
Sbplx (based on Subplex)

Local gradient-based optimization
MMA (Method of Moving Asymptotes) and CCSA
SLSQP
Low-storage BFGS
Preconditioned truncated Newton
Shifted limited-memory variable-metric

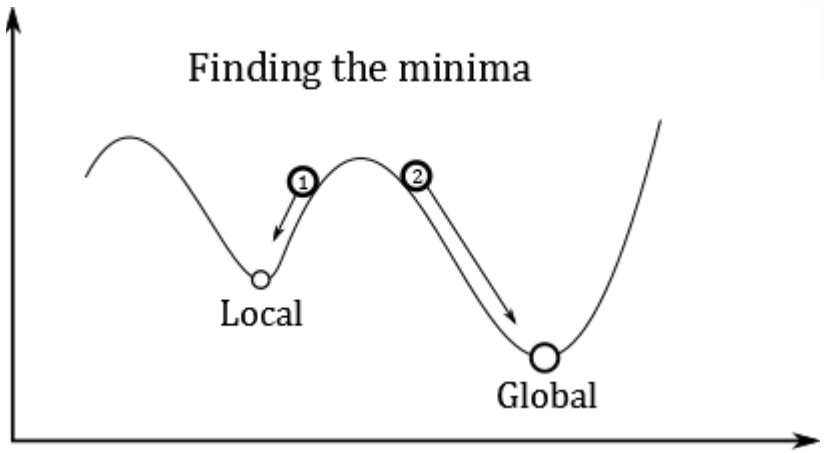
Website: ab-initio.mit.edu/nlopt

Basic procedure

The optimization procedure could be run when the soil model is given the initial state from the F.E. application, «hidden» from the user.

The model will then call itself and run simulations internally => undrained triaxial, CRS oedometer and undrained direct simple shear tests.

Several lab test curves can be added to find the overall best fit.

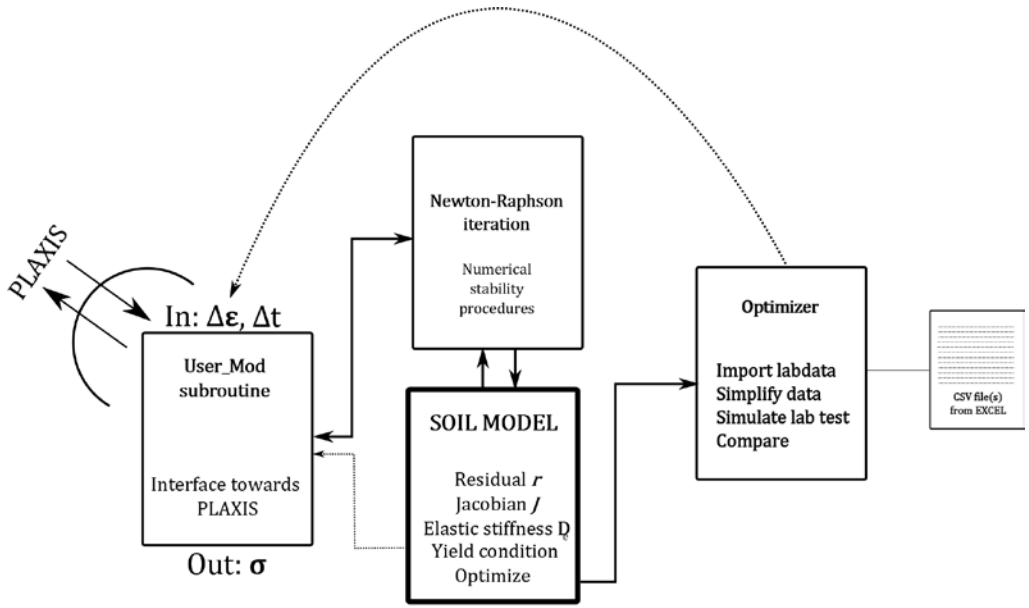


$$\boldsymbol{x} = \begin{Bmatrix} \kappa^* \\ \lambda^* \\ G \\ M \\ K_0 \\ x_0 \\ OCR \\ a_p \\ a_q \\ \omega \\ \omega_d \end{Bmatrix}$$

Can some lesser known parameters be replaced by others which are well known within the geotechincal field, e.g. undrained shear strength $c_{u,a}/p'_0$ *for a given strain rate?*

Numerical «framework»

Concept: Separate the computer code that is dependent on the soil model (constitutive formulations) and what is independent (iterations, optimization, interface towards PLAXIS etc.)



Soil model calls itself initially to obtain the optimized material parameters.

One advantage is to be able to validate the soil model against lab data, and modify the constitutive equations if necessary.

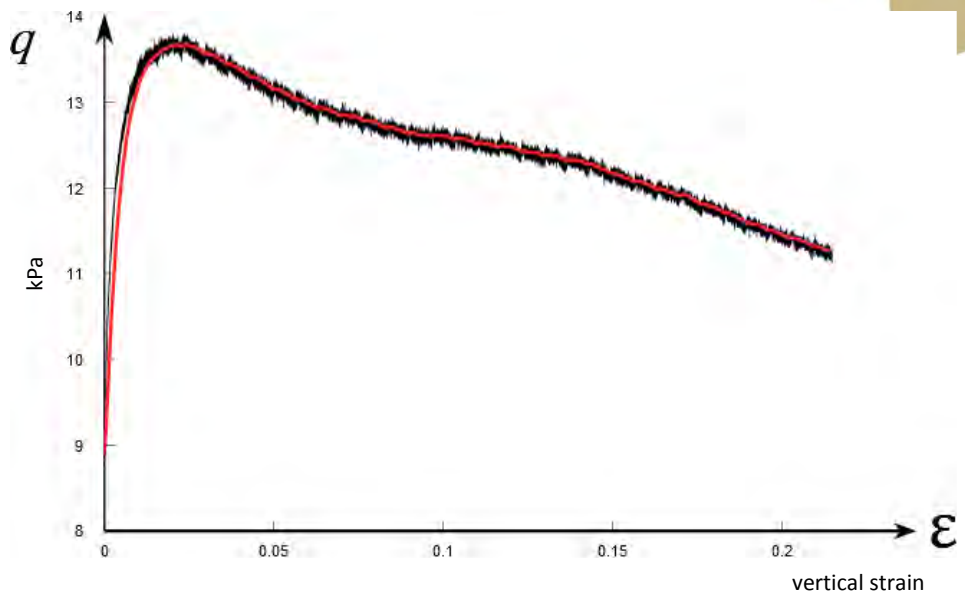
1. Validate, compare simulations against lab data using the optimisation procedures.
2. Reformatulate the constitutive equations in MATLAB.
3. Generate new FORTRAN code.
4. Re-validate.

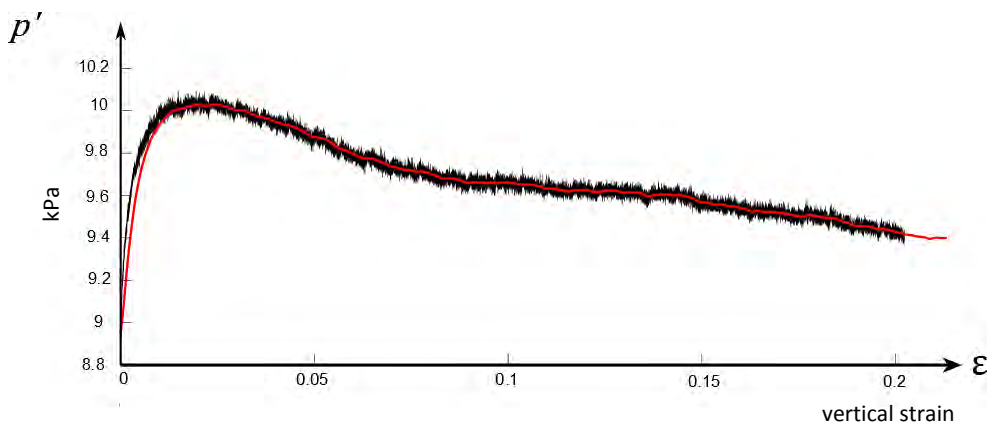
A first test: OVP clay

<u>Undrained triaxial compression test</u>

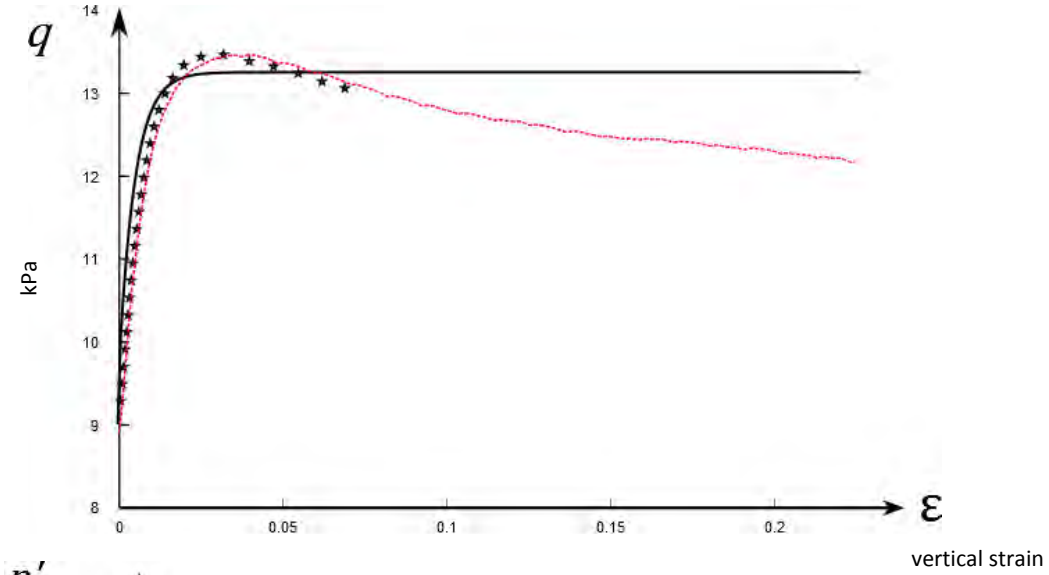
1. Import data from CSV file.
2. Filter with exponential moving average.
3. Pick a representative selection of points to use.

$$\boldsymbol{x} = \begin{Bmatrix} \varphi \\ K_0{}^{NC} \\ m_{oed} \\ M_{oed}/\sigma_y \\ OCR \end{Bmatrix} \qquad \boldsymbol{x} \in \begin{Bmatrix} 25° - 40° \\ 0.35 - 0.85 \\ 3 - 150 \\ 10 - 500 \\ 1 - 3 \end{Bmatrix}$$

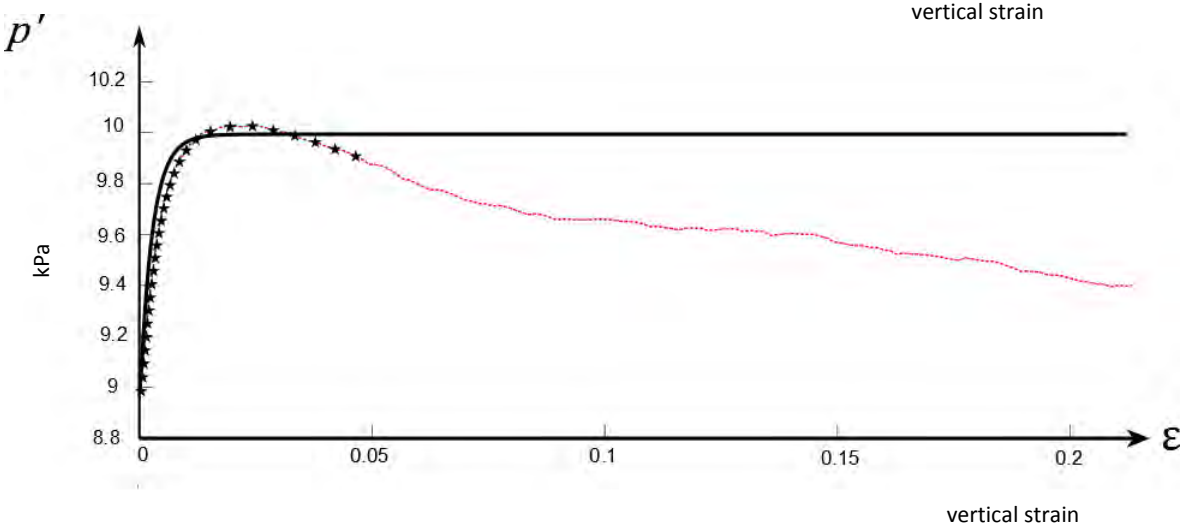


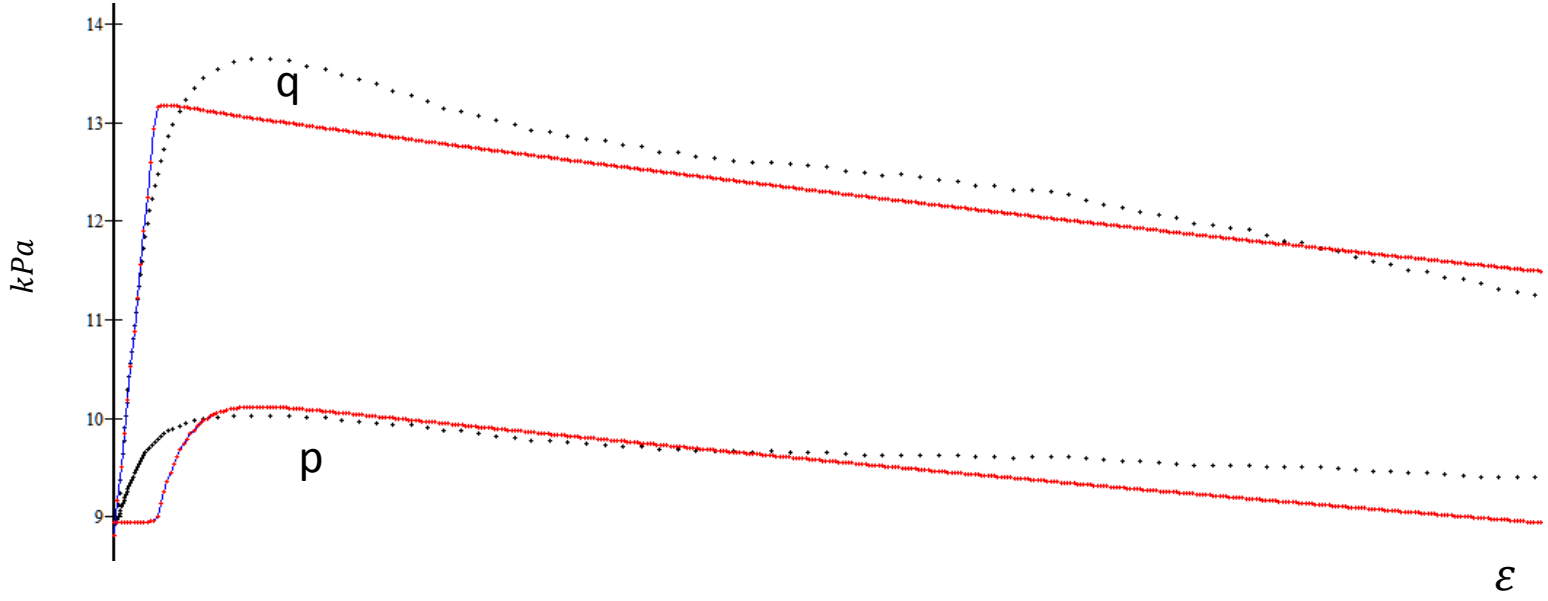


A first test: OVP clay



$$\boldsymbol{x} = \begin{Bmatrix} \varphi \\ K_0{}^{NC} \\ m_{oed} \\ M_{oed}/\sigma_y \\ OCR \end{Bmatrix} = \begin{Bmatrix} 35° \\ 0.8 \\ 123 \\ 239.5 \\ 1.02 \end{Bmatrix}$$




14
13
12
11
10
9
kPa
q
p
ε

Experiences so far

- Data from several lab tests should be used in combination in order to provide enough information to fit all input parameters. In addition the lower and upper bounds of the parameters can be set based on experience (or exact value of parameter if known).

- It should be noted that some input parameters influence certain stress paths much more than others (e.g. triaxial test: friction angle φ, oedometer: m_{oed}).

- Still need to gain experience using this tool.

vertical strain

Literature

Grimstad G. 2009. ***Development of effective stress based anisotropic models for soft clays,*** PhD diss., Norwegian University of Science and Technology, NTNU, Trondheim.

Grimstad G. and Degago S. 2010. ***A non-associated creep model for structured anisotropic clay (n-SAC)***, European Conference on Numerical Methods in Geotechnical Engineering 7, Trondheim 2010. Proceedings, pp. 3.14.

Grimstad G., Degago S., Nordal S., and Karstunen M. 2010. ***Modeling creep and rate effects in structured anisotropic soft clays,*** Acta Geotechnica, April 2010, Volume 5, Issue 1, pp 69-81.

Olsson, M. 2013. ***On Rate-Dependency of Gothenburg Clay,*** PhD diss., Chalmers University of Technology, Göteborg.

Olsson, M. 2010. ***Calculating long-term settlement in soft clays,*** Licentiate thesis, Chalmers University of Technology, Göteborg.

A new GUI software for assessing (creep) model parameters

Jean-Philippe Gras, Chalmers University of technology

Introduction

- Creep constitutive soil models have a lot of parameters in general.
- Most of these parameters are measured experimentally based on different load tests (triaxial, Oedometer…)with some uncertanties on the measures. These measures are often done manually.
- Some parameters are very hard to derive from experimental results

Example of the Creep_SClay1S model

Parameters of the model

Soil constants

κ Elastic swelling index

λ_i Intrinsic Compression index

M_e Slope of critical state line in extension

M_c Slope of critical state line in compression

ν' Poisson´s ratio

Parameters for rotational hardening

μ Absolute effectiveness of rotational hardening

β Relative effectiveness of rotational hardening

Parameter for destructuration

a Absolute rate of destructuration

b Relative rate of destructuration

Initial State

e_0 Initial void ratio

p'_{m0} preconsolidation pressure

K_0^{nc} K_0 for normal consolidation

χ_0 Initial bonding

α_0Initial inclination of yield surface

Creep parameters

Modified creep index

Time of reference

Example of the CreepSClay1S model

17 parameters:

Type of load test	Oedometer test	Triaxial test	Fall cone test
Parameters	Elastic swelling index Intrinsic Compression index Preconsolidation pressure Modified creep index Time of reference	Slope of critical state line in extension Slope of critical state line in compression K_0 for normal consolidation Initial inclination of yield surface Relative effectiveness of rotational hardening	Initial bonding

For some parameters, experimental determination may require a lot of tests:

Absolute effectiveness of rotational hardening
Absolute rate of destructuration
Relative rate of destructuration

} **We use a default value**

For the Poisson´s ratio, a value of 0.2 is often assumed an the initial void ratio is determined by standart procedures.

Objectives

Make a tool allowing:

- An automatic measure of the parameters from experimental datas (gain of time, objectivity).

- A multi-objective optimization of parameters (optimization of parameters for different load paths). The initial set of parameters is taken from the measure and default values

- The single element testing of model and comparison with experimental datas

Design of a GUI for INCREMENTAL DRIVER

Single-element testing of models:

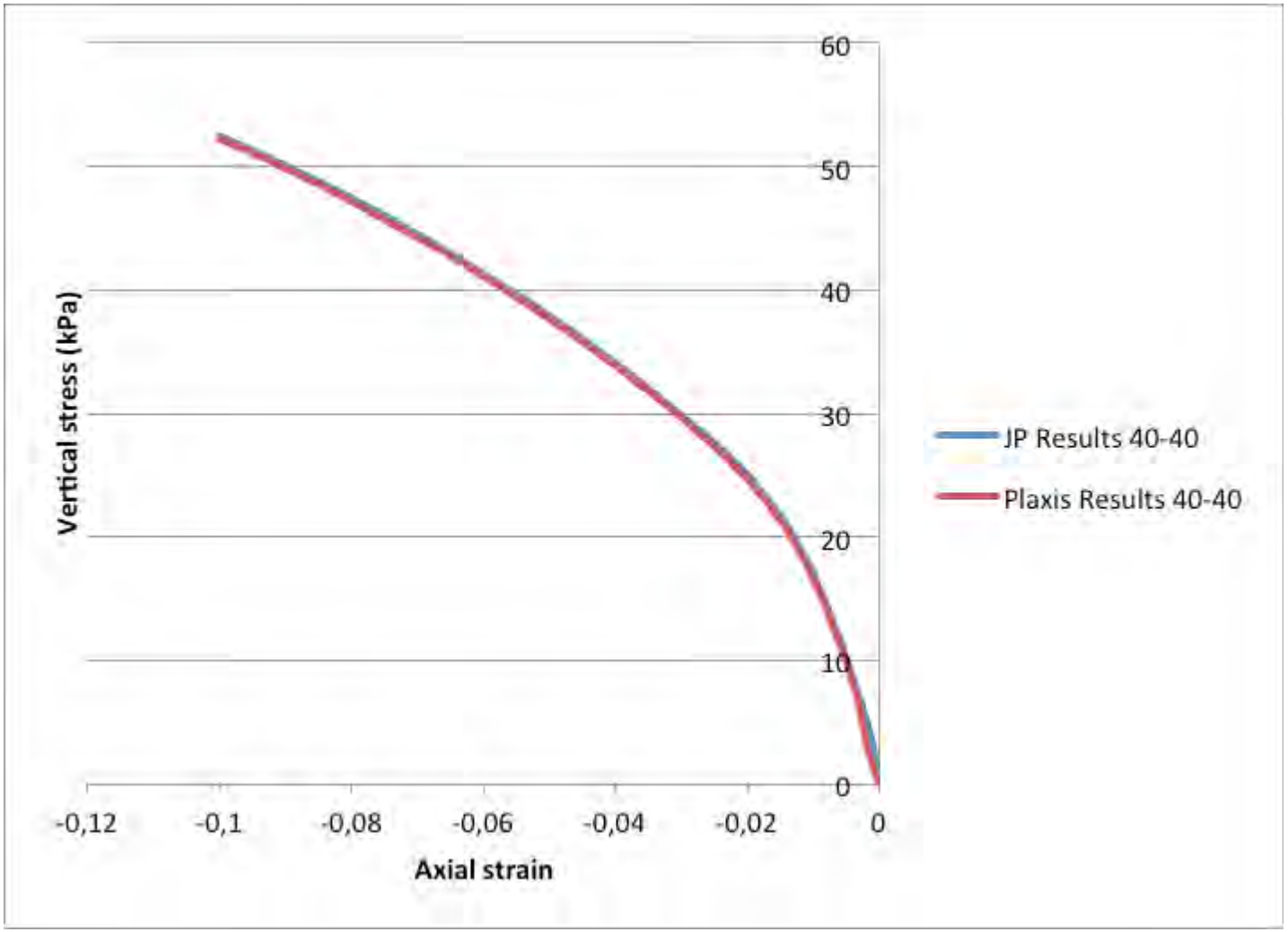
Design of a graphical user interface for single element testing of models based on **INCREMENTAL DRIVER** which is an open-source program for testing constitutive models. It calls a material routine (constitutive relations) with the syntax of the user material subroutine **umat** of **ABAQUS.**

It allows loading program to follow a prescribed combined stress/strain path:

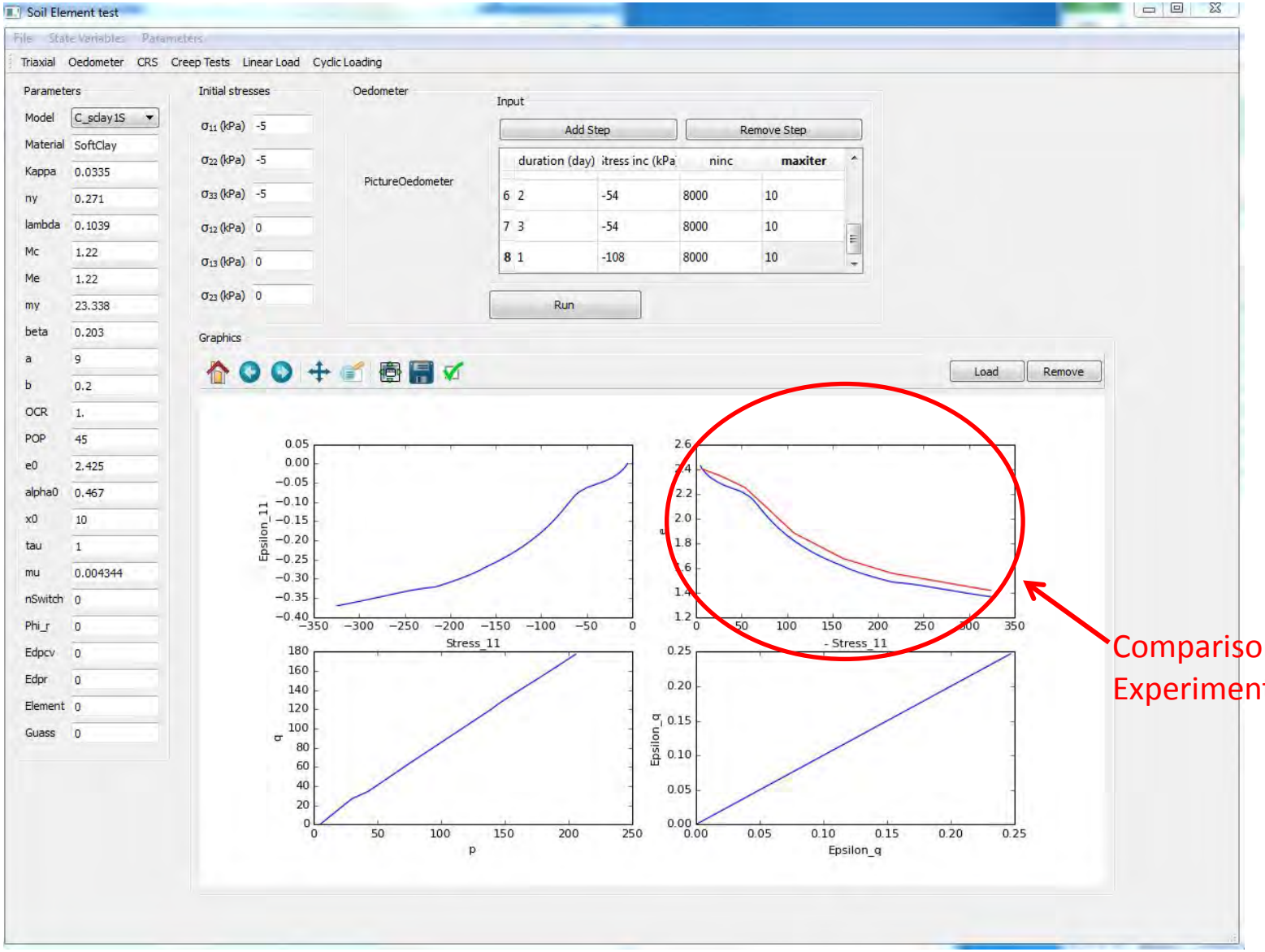
- Popular paths: Oedometer, CRS, Drained and Undrained Triaxial, Pure Creep…
- Proportional stress/strain paths in all directions
- Harmonic load
- Repetition of a group of load paths

GUI for INCREMENTAL DRIVER

Comparison with Plaxis

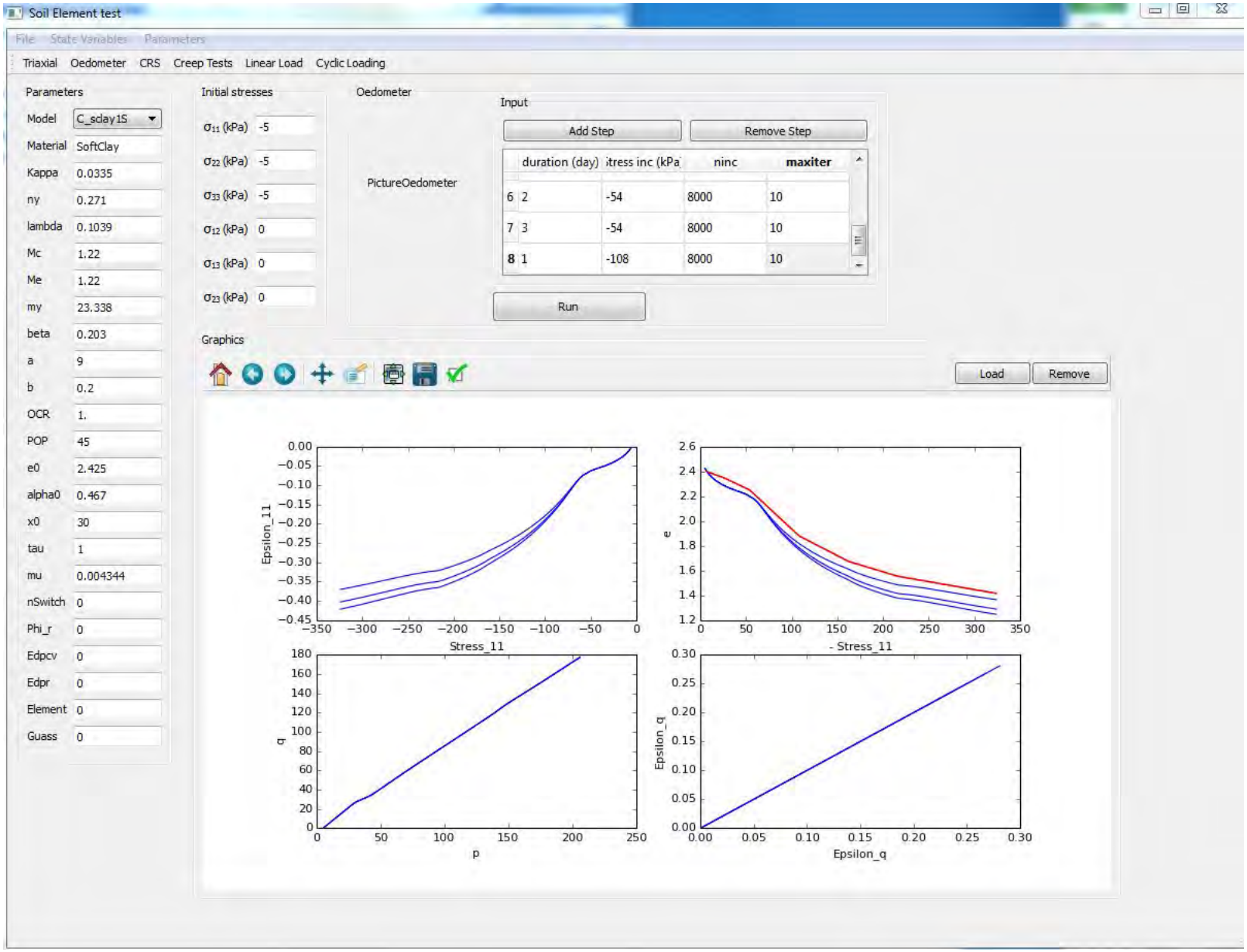


Comparison with experimental datas



Comparison with Experimental datas

Effect of variation of parameters



Further work: optimization process

The optimization problem consist in finding the $\boldsymbol{n}$ values $\boldsymbol{X_n}$ of parameters minimizing the following function

$$\boldsymbol{Y_m} - \boldsymbol{M}(\boldsymbol{X_n})$$

Where $\boldsymbol{Y_m}$ is the $\boldsymbol{m}$ experimental datas and $\boldsymbol{M}$ is the model

Single objective optimization (only one type of load path) could be inadequate for the parameters calibration of soil models. So we will optimize the parameters **simultaneously on different load paths**.

Optimization process

- Start with an initial set of parameters (from experimental values)
- Run the optimization process on different load path
- Test the new set of parameters

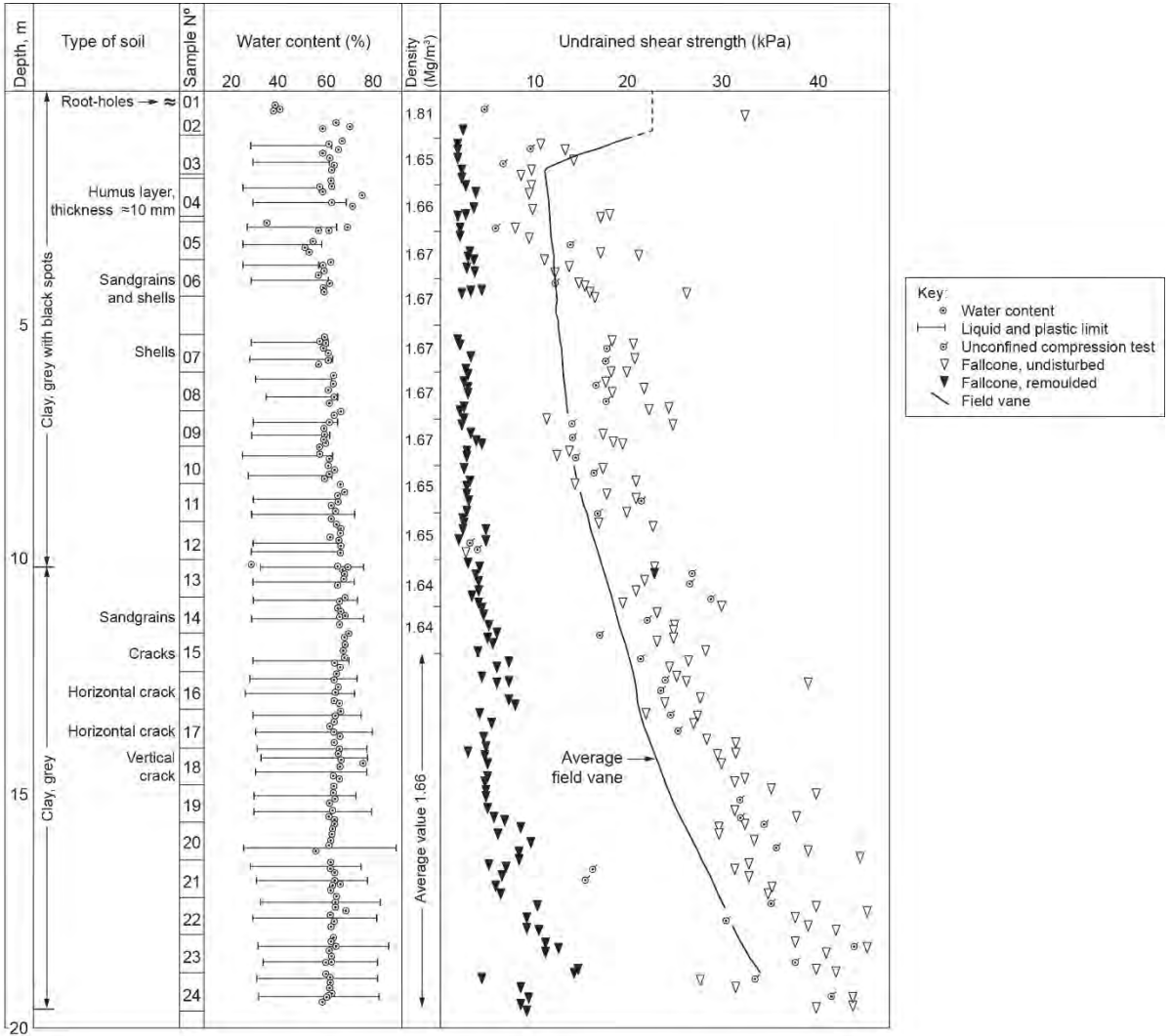
Conclusion

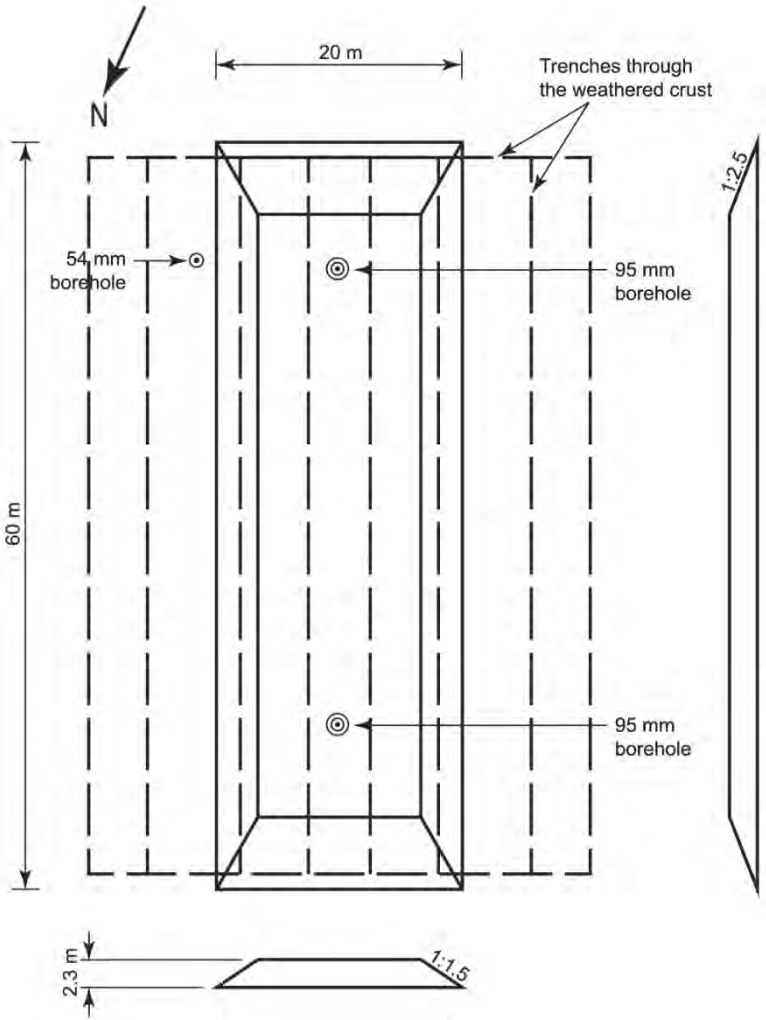
- Multiple load paths testing
- Easy comparison with experimantal datas
- Big gain of time by combining:
  - Measure of the parameters from experimental datas
  - Optimization of parameters
  - Soil element testing
- Objective set of parameter in comparison of calibration made by manually ajusting the parameters

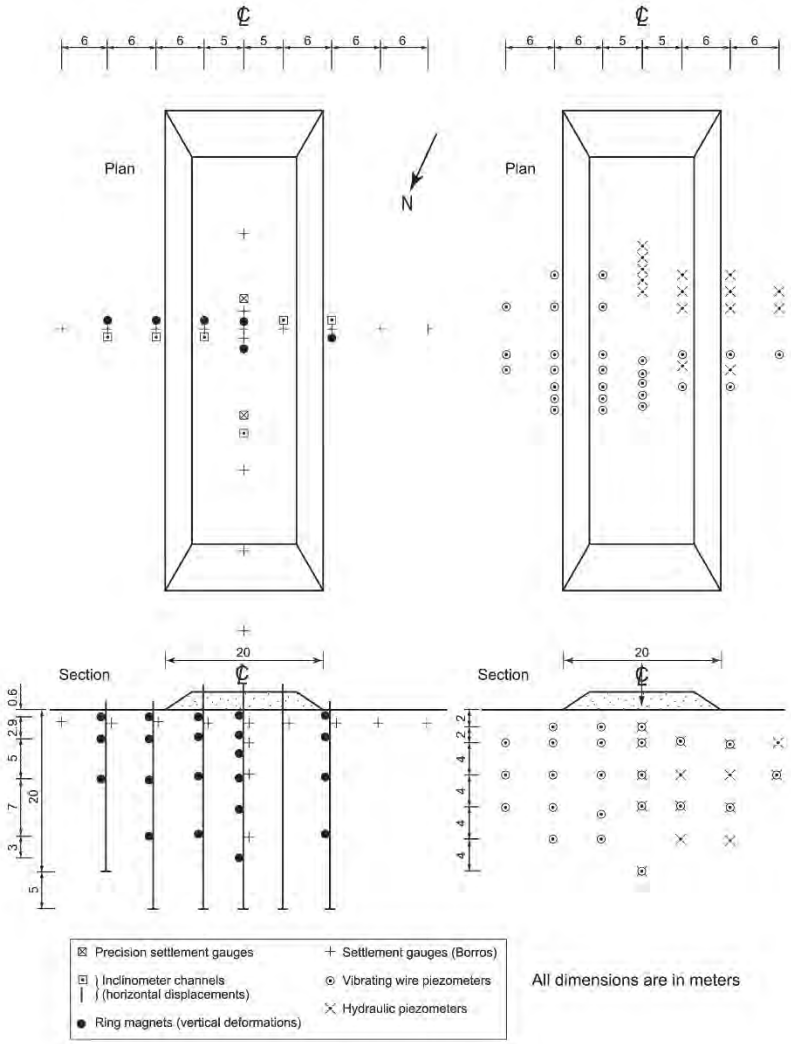
Onsøy Test Field

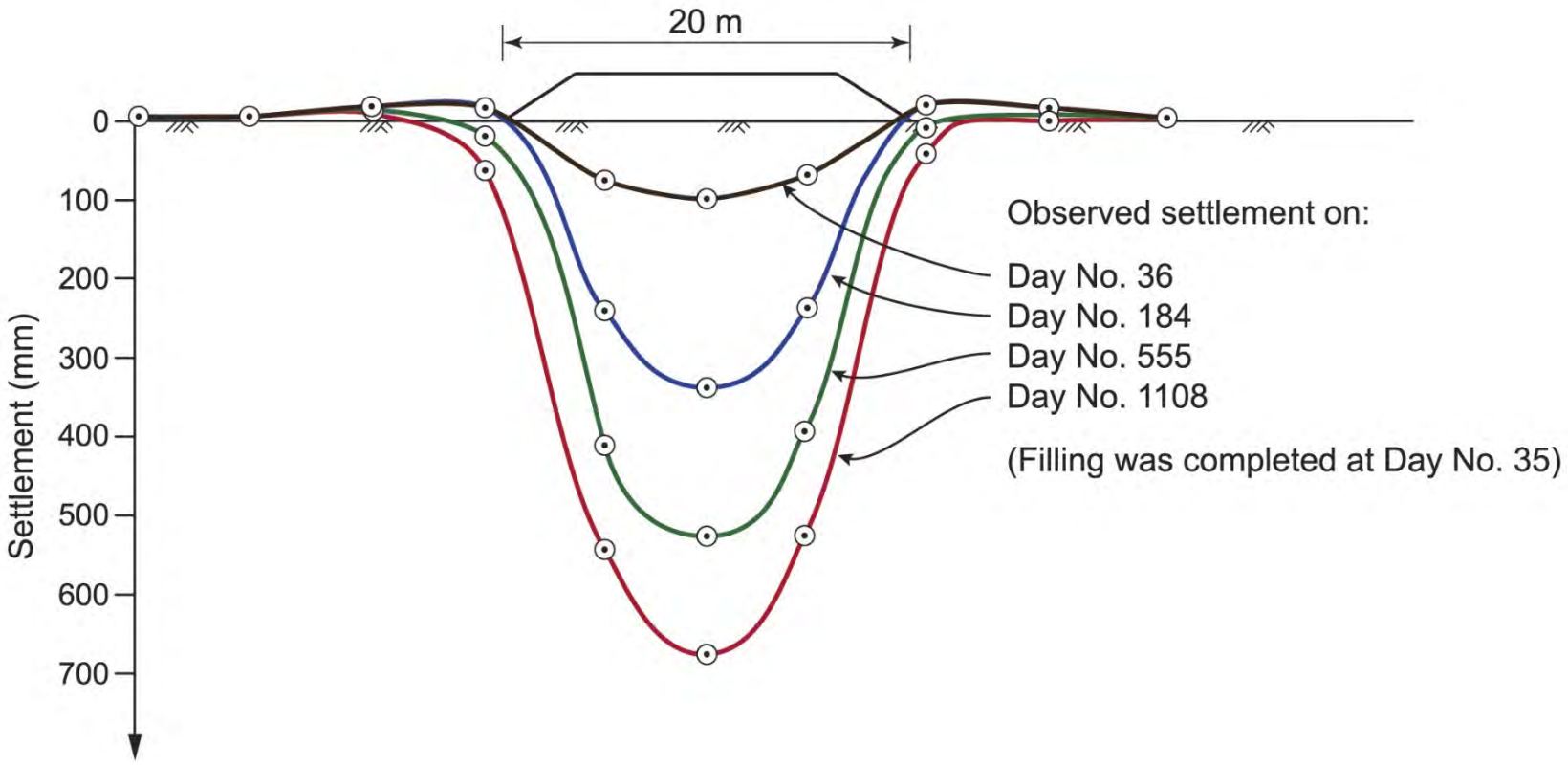
Toralv Berre

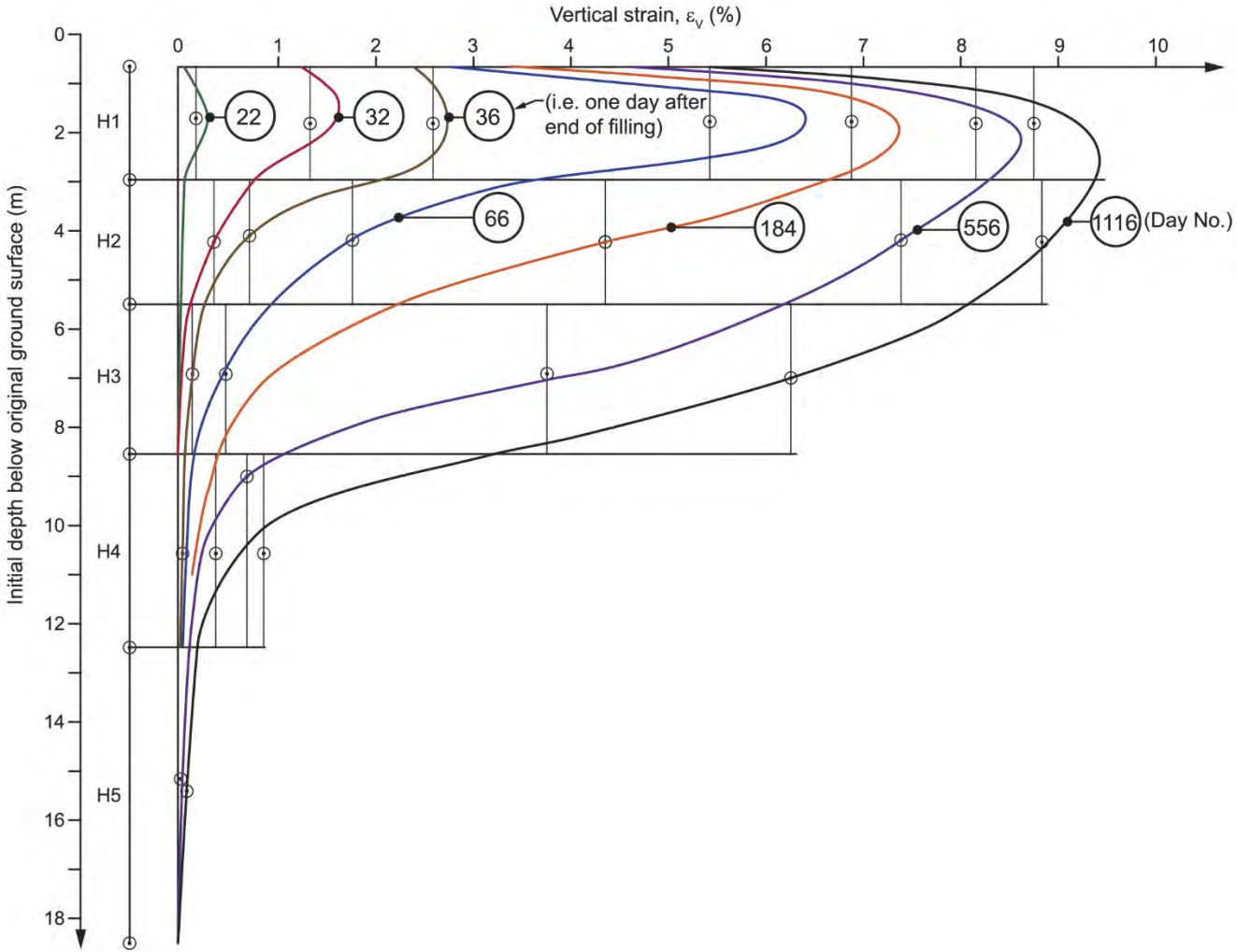
NGI

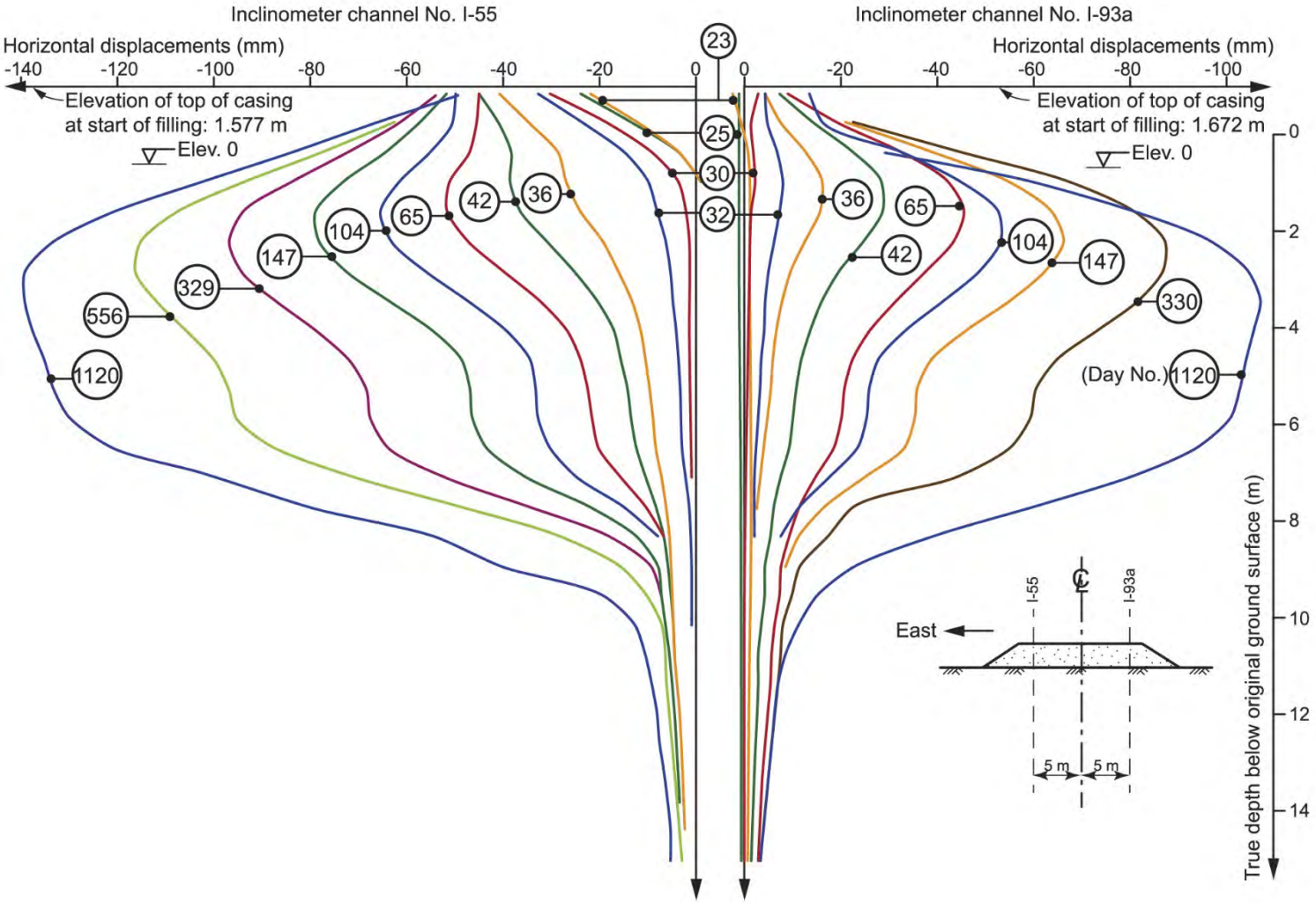

Depth, m
Type of soil
Sample Nº
Water content (%)
Density (Mg/m³)
Undrained shear strength (kPa)
Root-holes
Humus layer, thickness ≈10 mm
Sandgrains and shells
Shells
Sandgrains
Cracks
Horizontal crack
Horizontal crack
Vertical crack
Clay, grey with black spots
Clay, grey
Average value 1.66
Average field vane
Key:
Water content
Liquid and plastic limit
Unconfined compression test
Fallcone, undisturbed
Fallcone, remoulded
Field vane

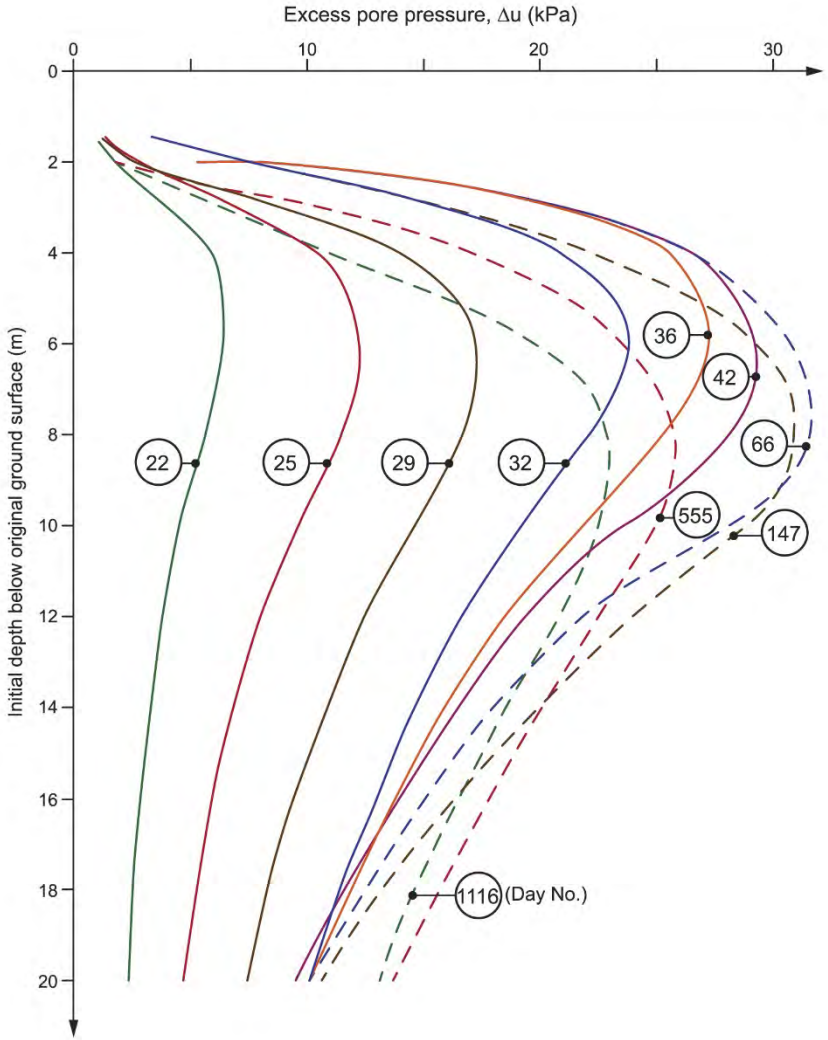

N
20 m
Trenches through
the weathered crust
1:2.5
54 mm
borehole
95 mm
borehole
95 mm
borehole
60 m
2.3 m
1:1.5

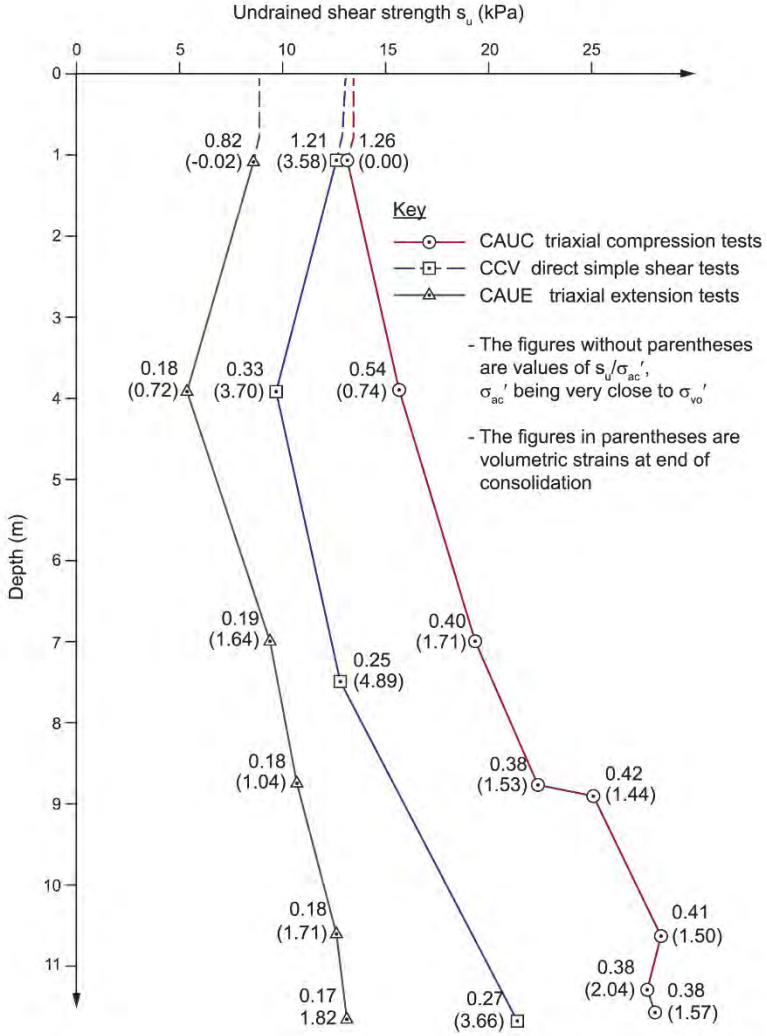

Plan
N
Section
20
Precision settlement gauges
Inclinometer channels
(horizontal displacements)
Ring magnets (vertical deformations)
Settlement gauges (Borros)
Vibrating wire piezometers
Hydraulic piezometers
All dimensions are in meters

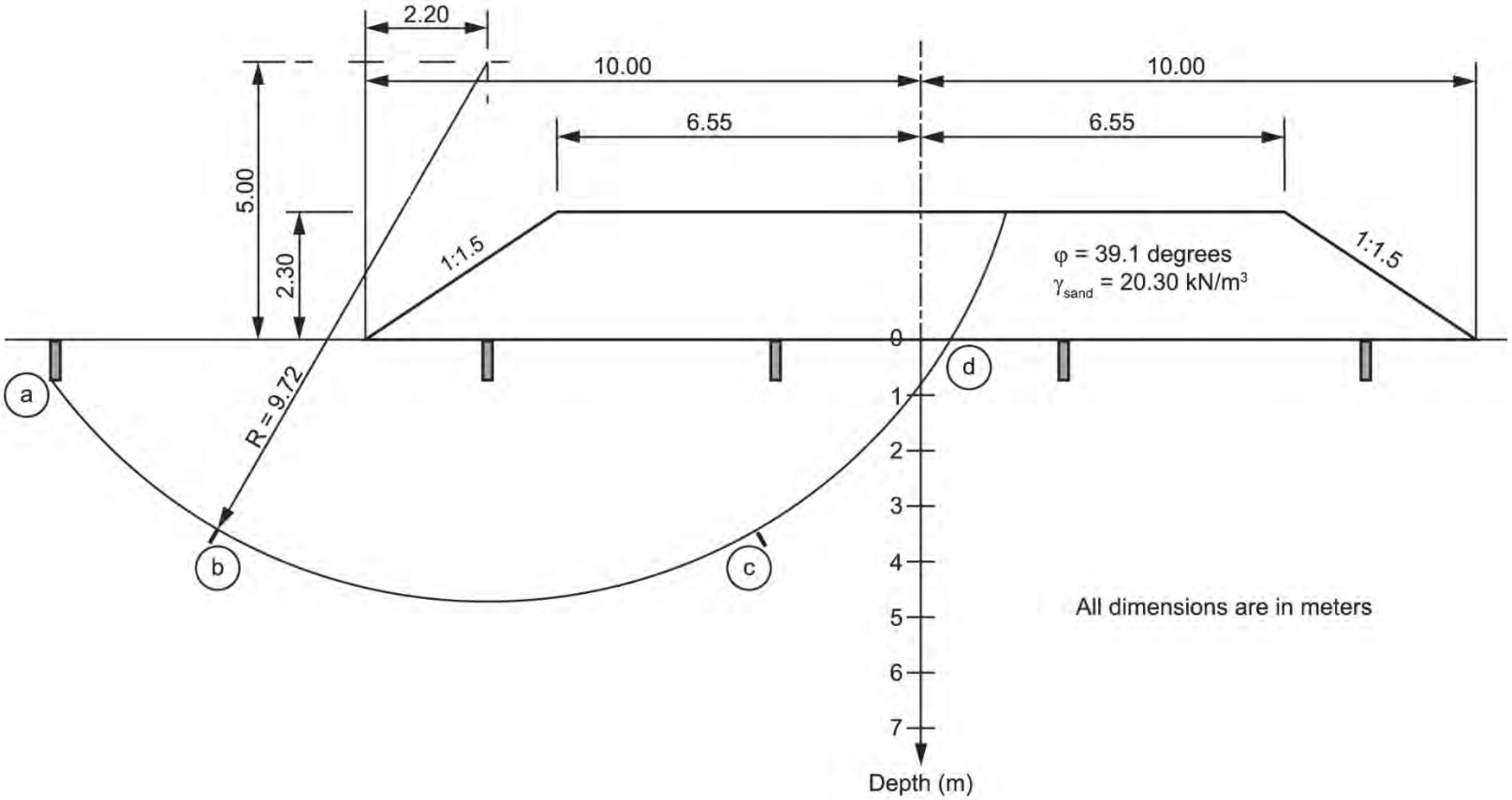

20 m
0
100
200
300
400
500
600
700
Settlement (mm)
Observed settlement on:
Day No. 36
Day No. 184
Day No. 555
Day No. 1108
(Filling was completed at Day No. 35)


Vertical strain, ε_v (%)
0
1
2
3
4
5
6
7
8
9
10
Initial depth below original ground surface (m)
0
2
4
6
8
10
12
14
16
18
H1
H2
H3
H4
H5
22
32
36
(i.e. one day after end of filling)
66
184
556
1116
(Day No.)


Inclinometer channel No. I-55
Horizontal displacements (mm)
-140
-120
-100
-80
-60
-40
-20
0
Elevation of top of casing
at start of filling: 1.577 m
Elev. 0
556
329
147
104
65
42
36
1120
23
25
30
32
Inclinometer channel No. I-93a
Horizontal displacements (mm)
0
-20
-40
-60
-80
-100
Elevation of top of casing
at start of filling: 1.672 m
Elev. 0
36
65
42
104
147
330
(Day No.)
1120
0
2
4
6
8
10
12
14
True depth below original ground surface (m)
East
I-55
I-93a
5 m
5 m


Excess pore pressure, Δu (kPa)
0
10
20
30
Initial depth below original ground surface (m)
0
2
4
6
8
10
12
14
16
18
20
22
25
29
32
36
42
66
555
147
1116
(Day No.)


Undrained shear strength s_u (kPa)
0
5
10
15
20
25
Depth (m)
0
1
2
3
4
5
6
7
8
9
10
11
0.82
(-0.02)
1.21
(3.58)
1.26
(0.00)
0.18
(0.72)
0.33
(3.70)
0.54
(0.74)
0.19
(1.64)
0.25
(4.89)
0.40
(1.71)
0.18
(1.04)
0.38
(1.53)
0.42
(1.44)
0.18
(1.71)
0.41
(1.50)
0.38
(2.04)
0.38
(1.57)
0.17
1.82
0.27
(3.66)
Key
CAUC triaxial compression tests
CCV direct simple shear tests
CAUE triaxial extension tests
- The figures without parentheses are values of s_u/σ_{ac}', σ_{ac}' being very close to σ_{vo}'
- The figures in parentheses are volumetric strains at end of consolidation


2.20
10.00
10.00
6.55
6.55
5.00
2.30
1:1.5
1:1.5
φ = 39.1 degrees
γsand = 20.30 kN/m3
R = 9.72
a
b
c
d
0
1
2
3
4
5
6
7
Depth (m)
All dimensions are in meters

Computed factors of safety, F

- From in situ vane test: F = 1.35
- In situ vane strength corrected as suggested by Bjerrum (1973): F = 1.20
- From standard undrained triaxial and direct simple shear tests: F = 1.14
- Triaxial and simple shear tests at the same shear strain: F = 1.06
- Plane strain triaxial and direct simple shear tests: F = 1.10
- Plane strain triaxial and direct simple shear tests corrected to in situ rate of loading: F = 1.10 x 0.85 = 0.94
- Plane strain triaxial and direct simple shear tests corrected both to in situ rate of loading and to in situ temperature: F = 0.94 x 1.10 = 1.03

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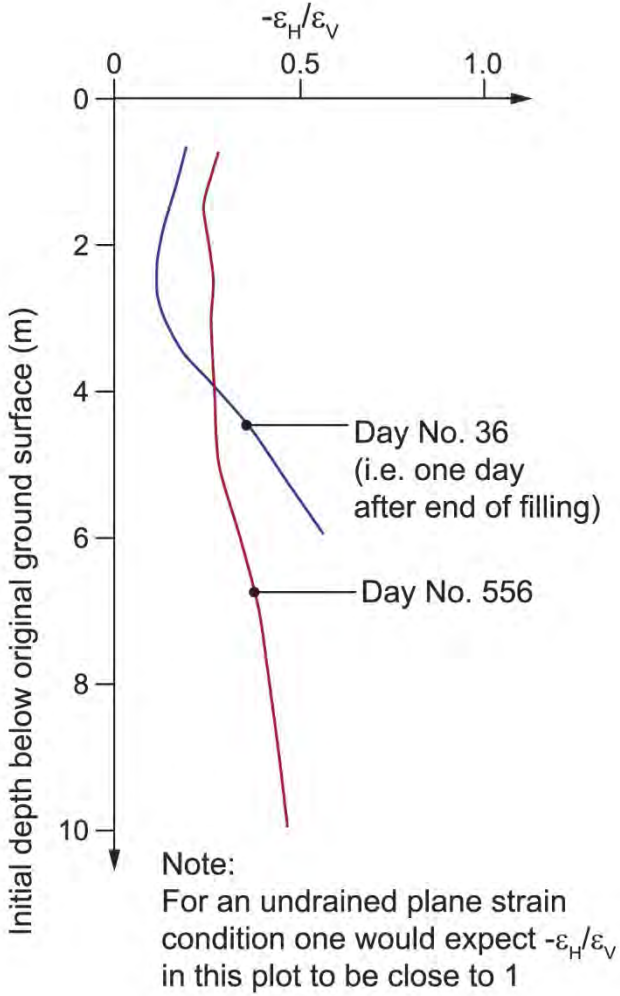
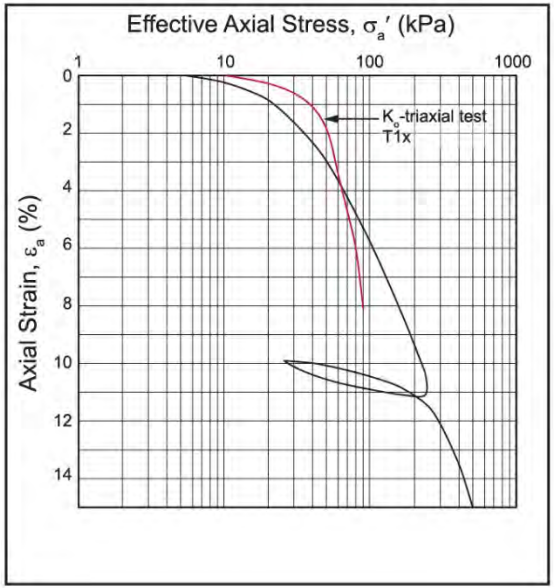
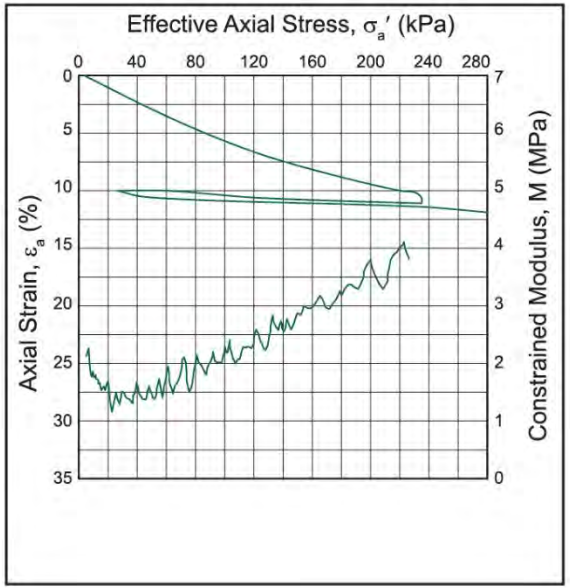
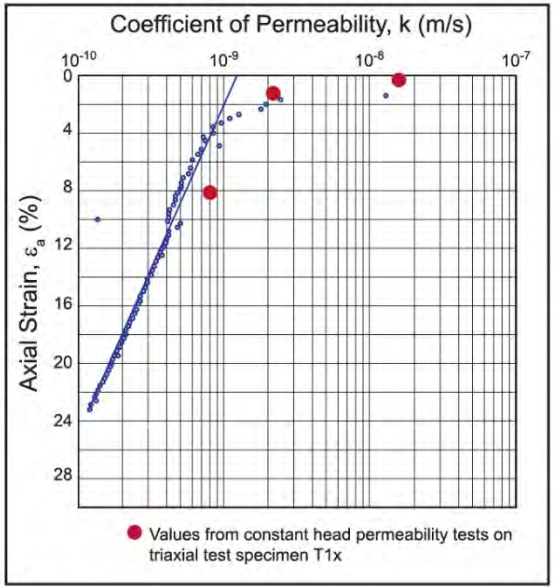

$-\varepsilon_H/\varepsilon_V$
0
0.5
1.0
0
2
4
6
8
10
Initial depth below original ground surface (m)
Day No. 36
(i.e. one day
after end of filling)
Day No. 556
Note:
For an undrained plane strain
condition one would expect $-\varepsilon_H/\varepsilon_V$
in this plot to be close to 1

NGI

Depth = 1.01 m

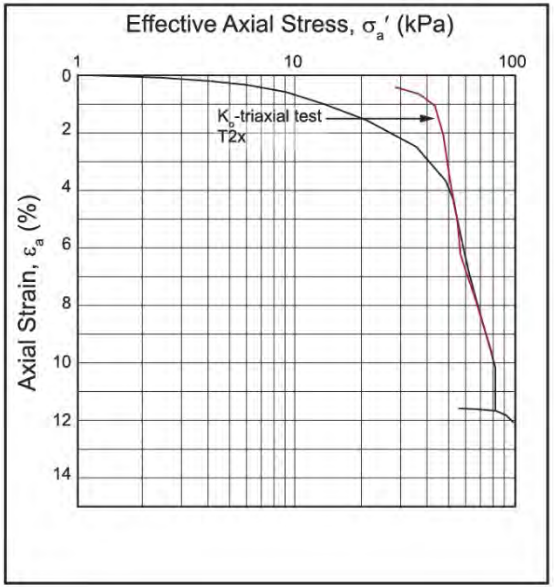


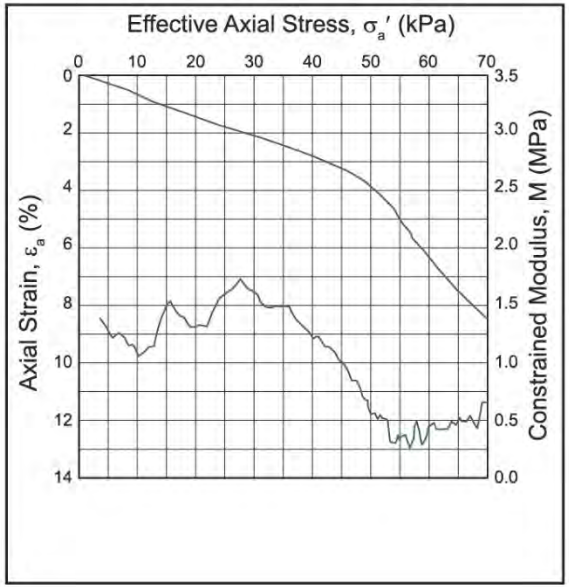


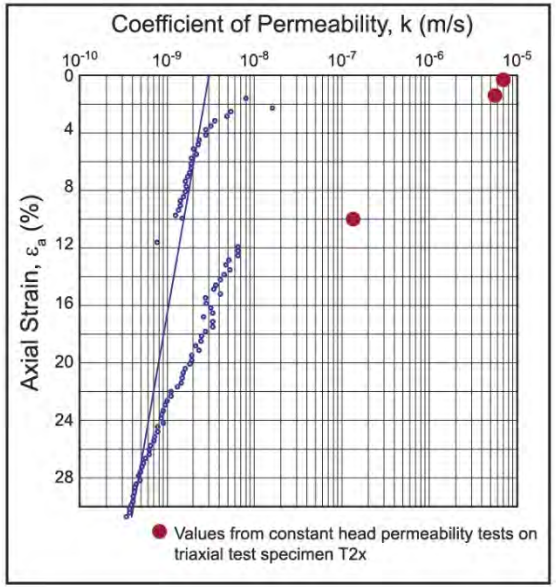


σ'_{v0} = 10.1 kPa, $(\sigma_v')_{final}$ = 57 kPa

Depth = 3.87 m

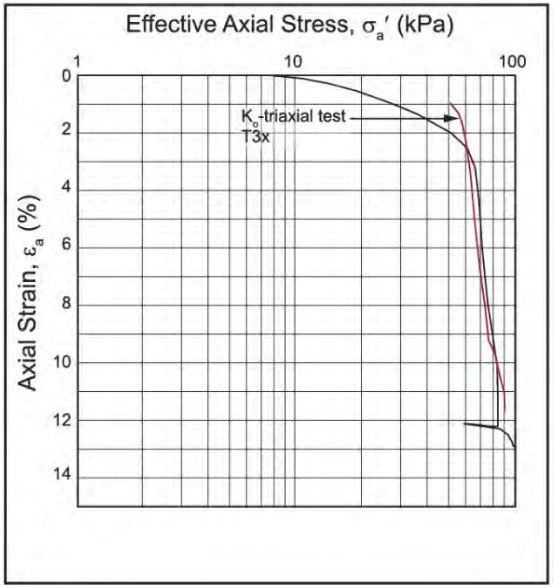


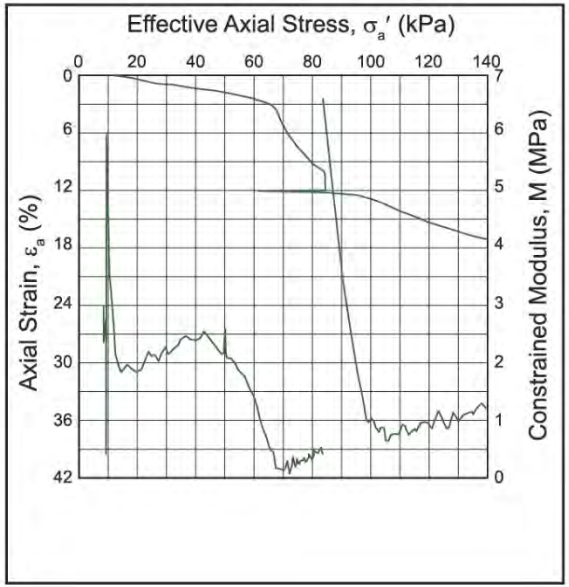


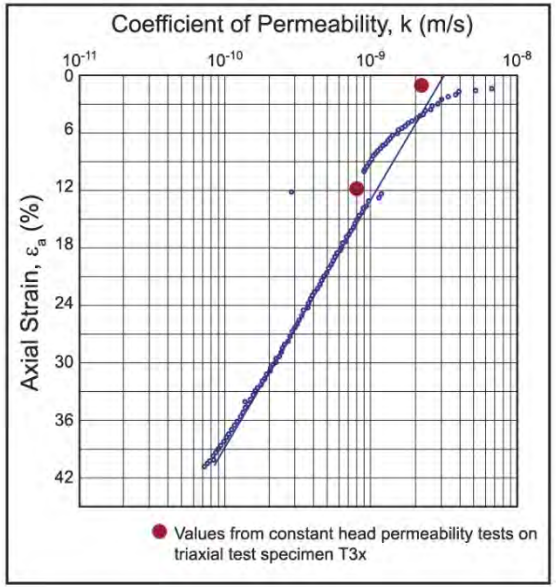


σ'_{v0} = 28.6 kPa, $(\sigma_v')_{final}$ = 74 kPa

Depth = 7.45 m

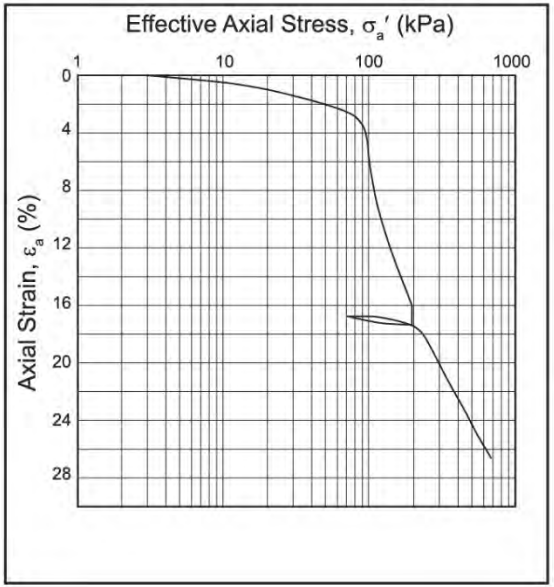


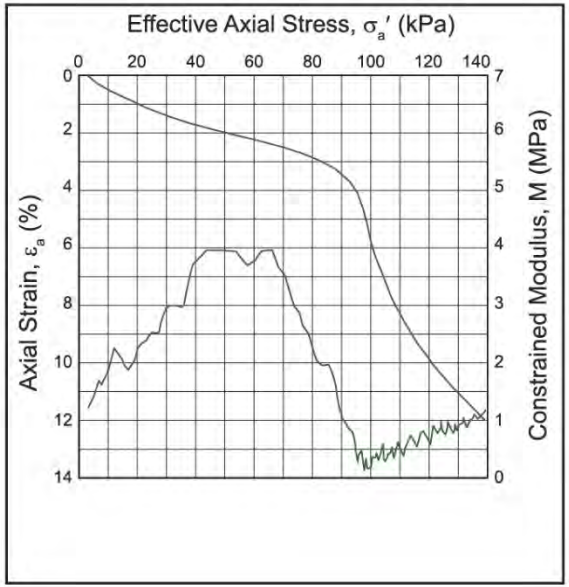


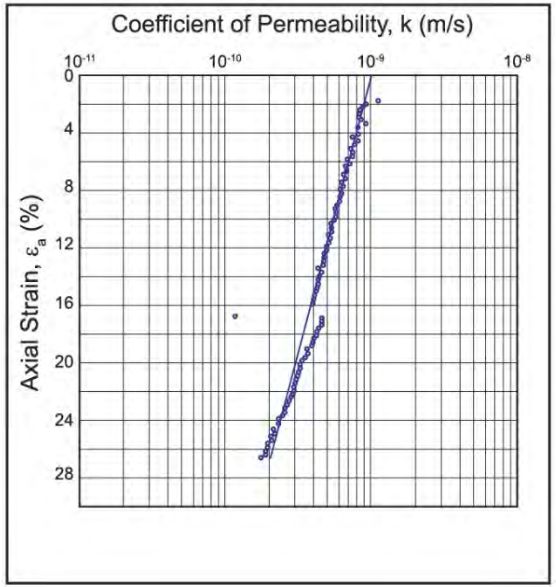


σ'_{v0} = 50.6 kPa, $(\sigma_v')_{final}$ = 94 kPa

Depth = 10.82 m

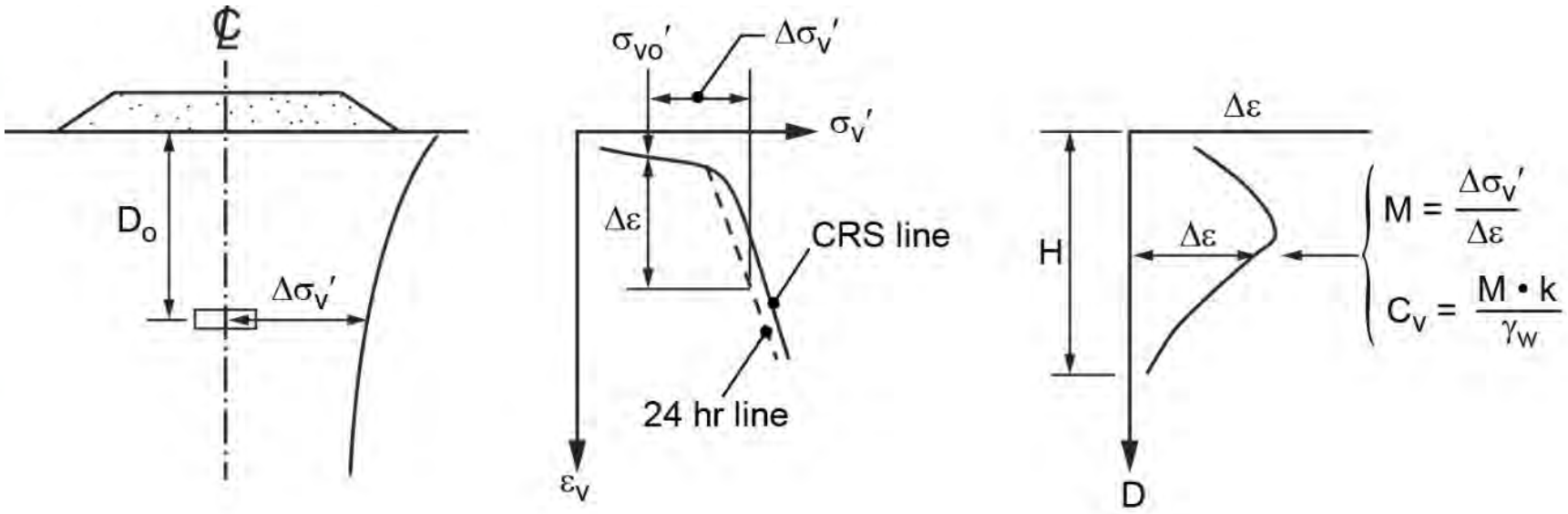


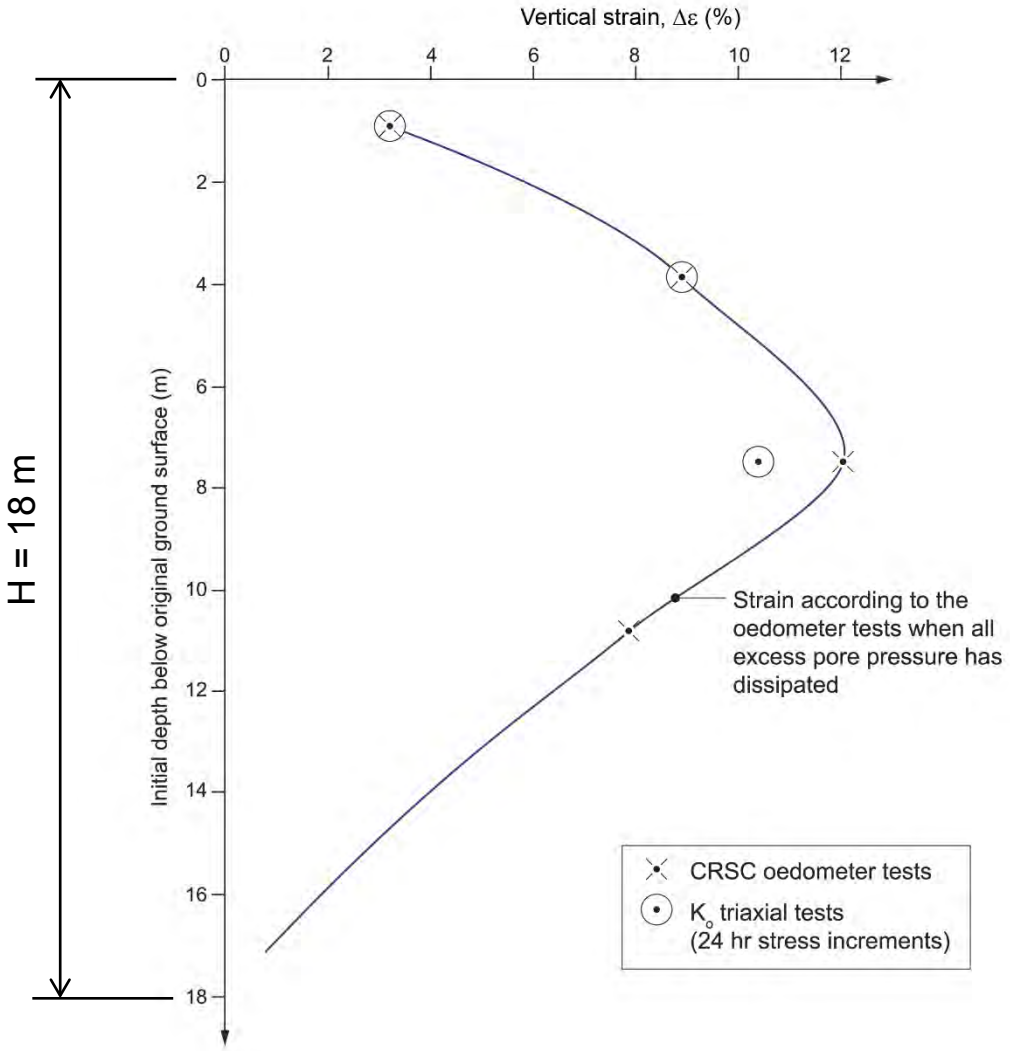




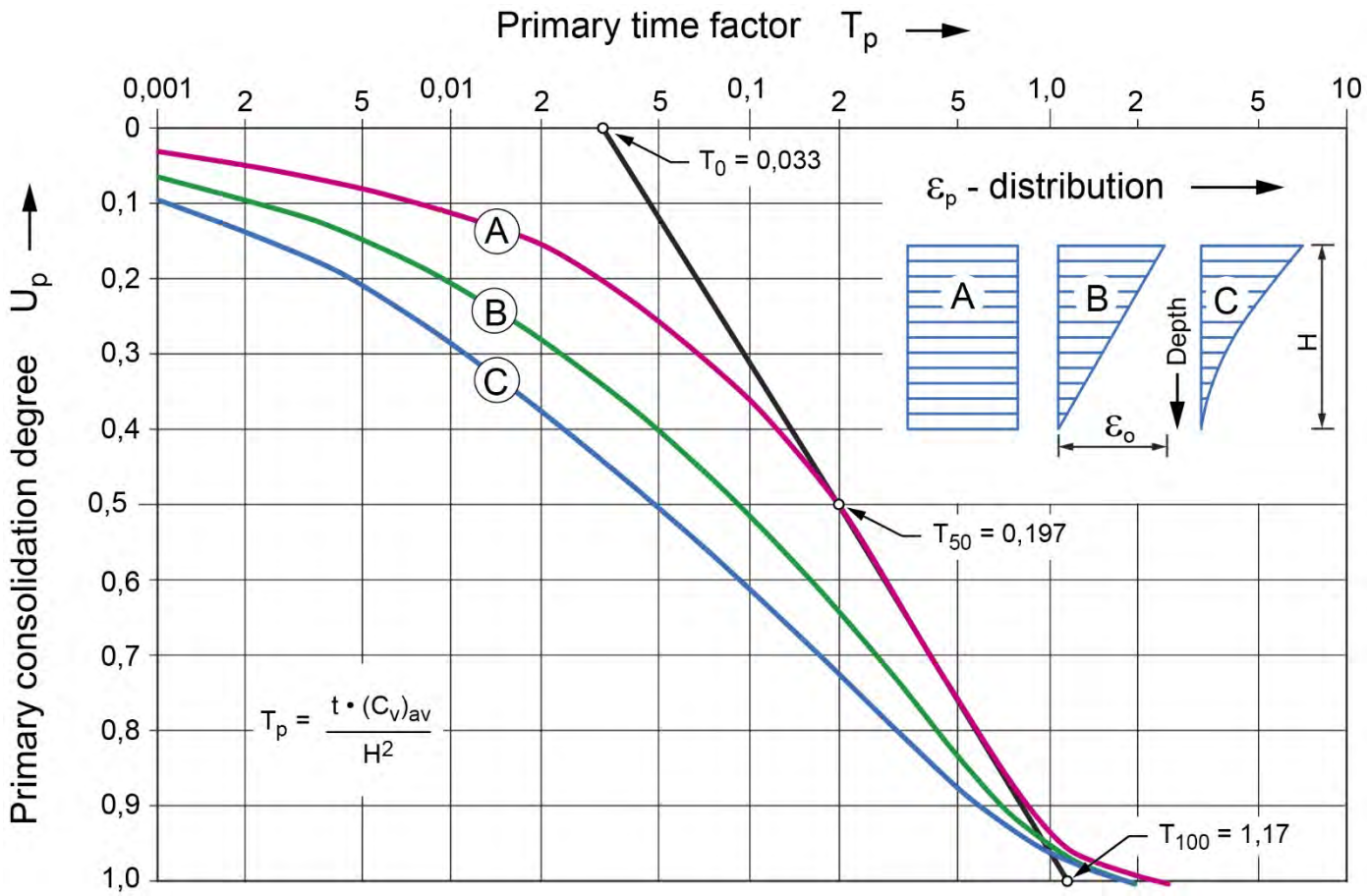
σ'_{v0} = 69.9 kPa, $(\sigma_v')_{final}$ = 108 kPa

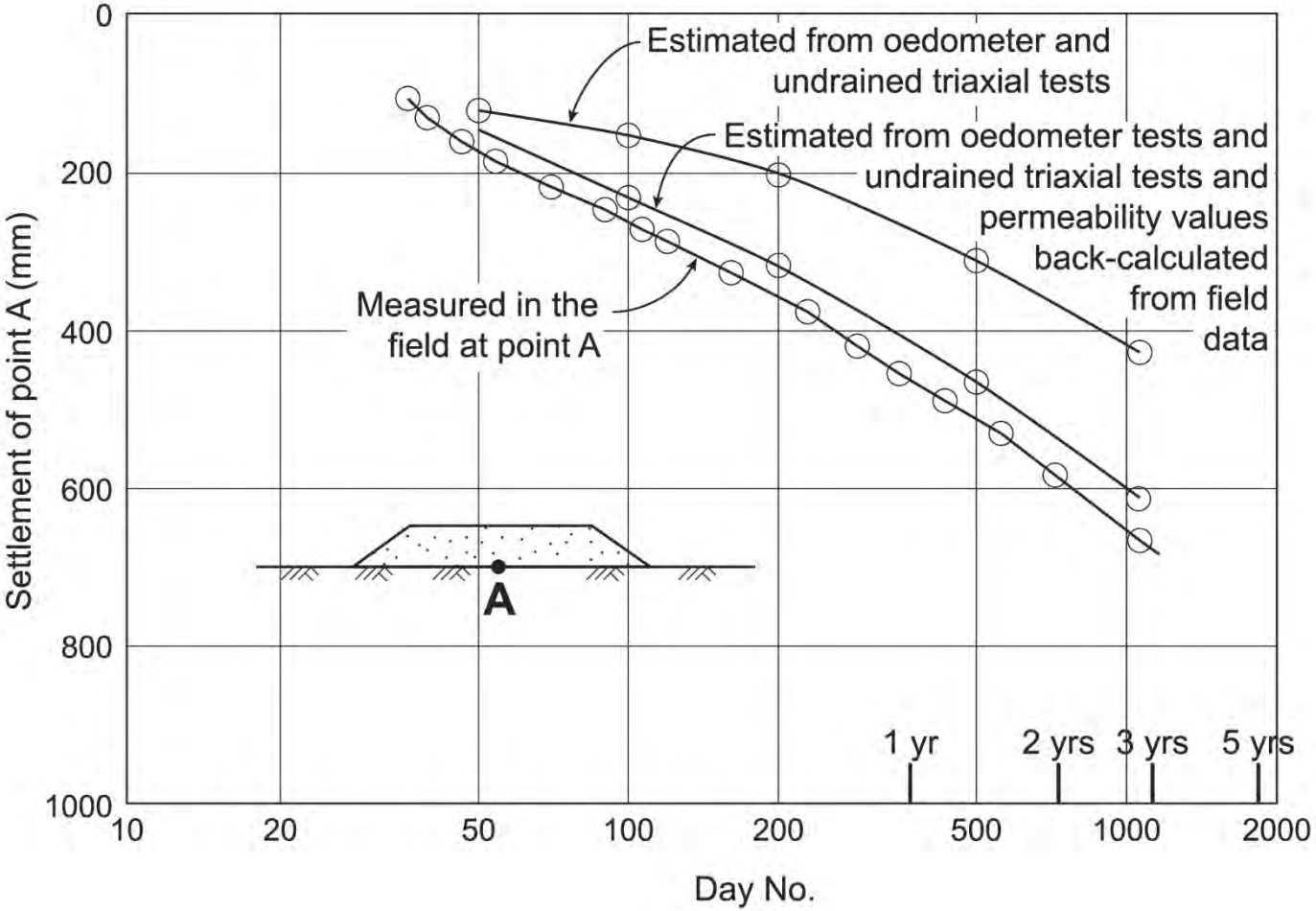
Method of settlement computation, principal sketch

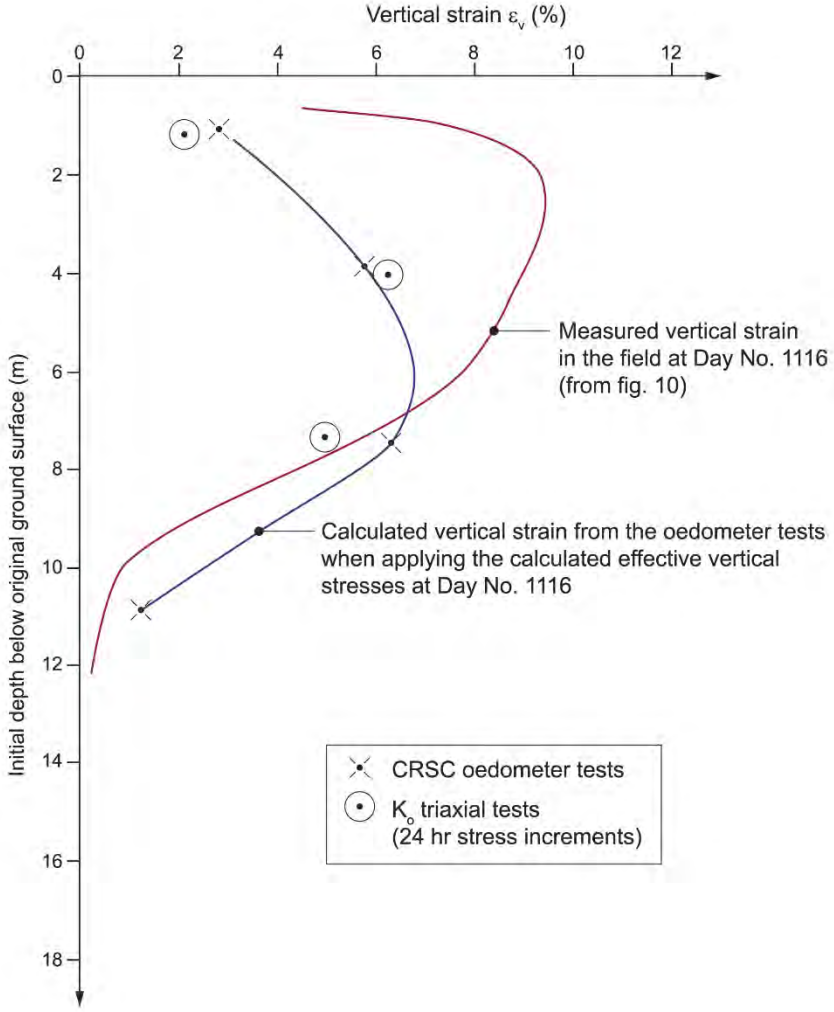


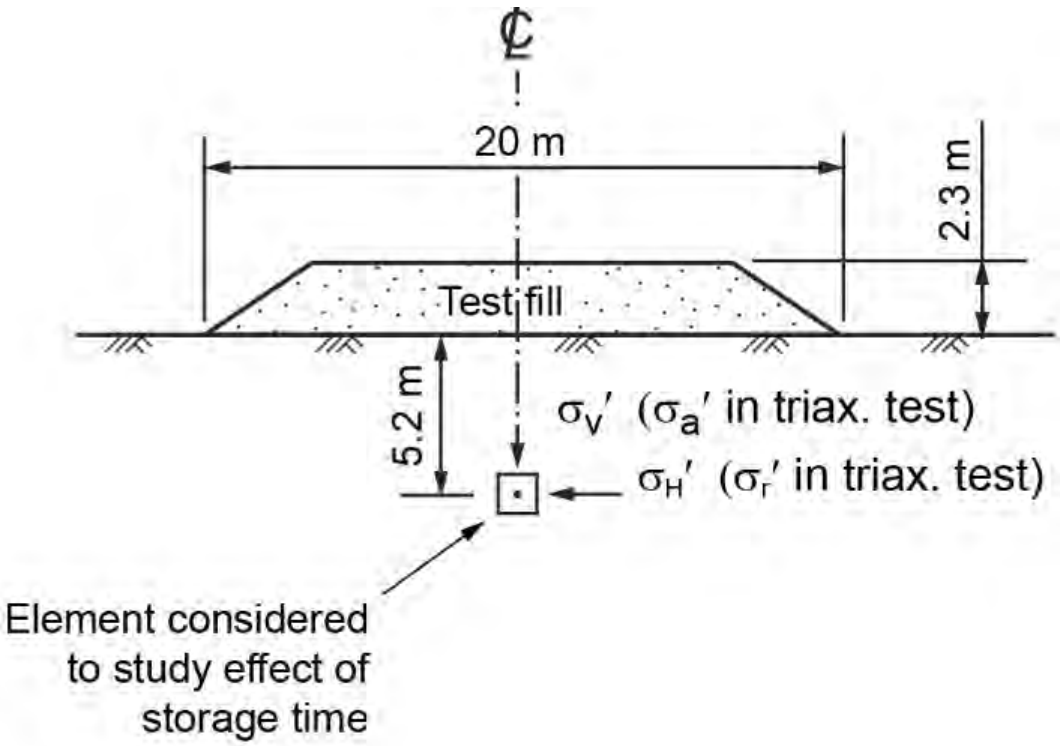

Vertical strain, Δε (%)
0
2
4
6
8
10
12
Initial depth below original ground surface (m)
0
2
4
6
8
10
12
14
16
18
H = 18 m
Strain according to the oedometer tests when all excess pore pressure has dissipated
CRSC oedometer tests
K$_o$ triaxial tests (24 hr stress increments)

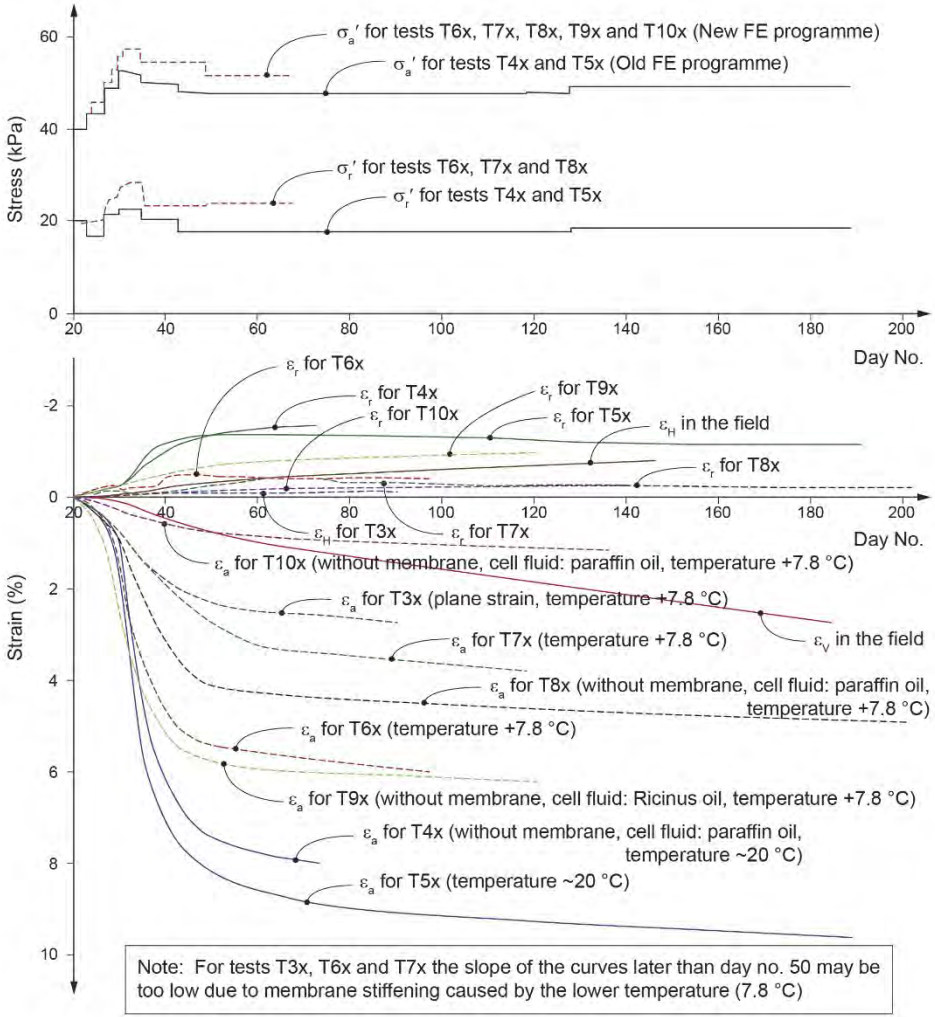
Degree of primary consolidation versus time factor (after Janbu 1970)

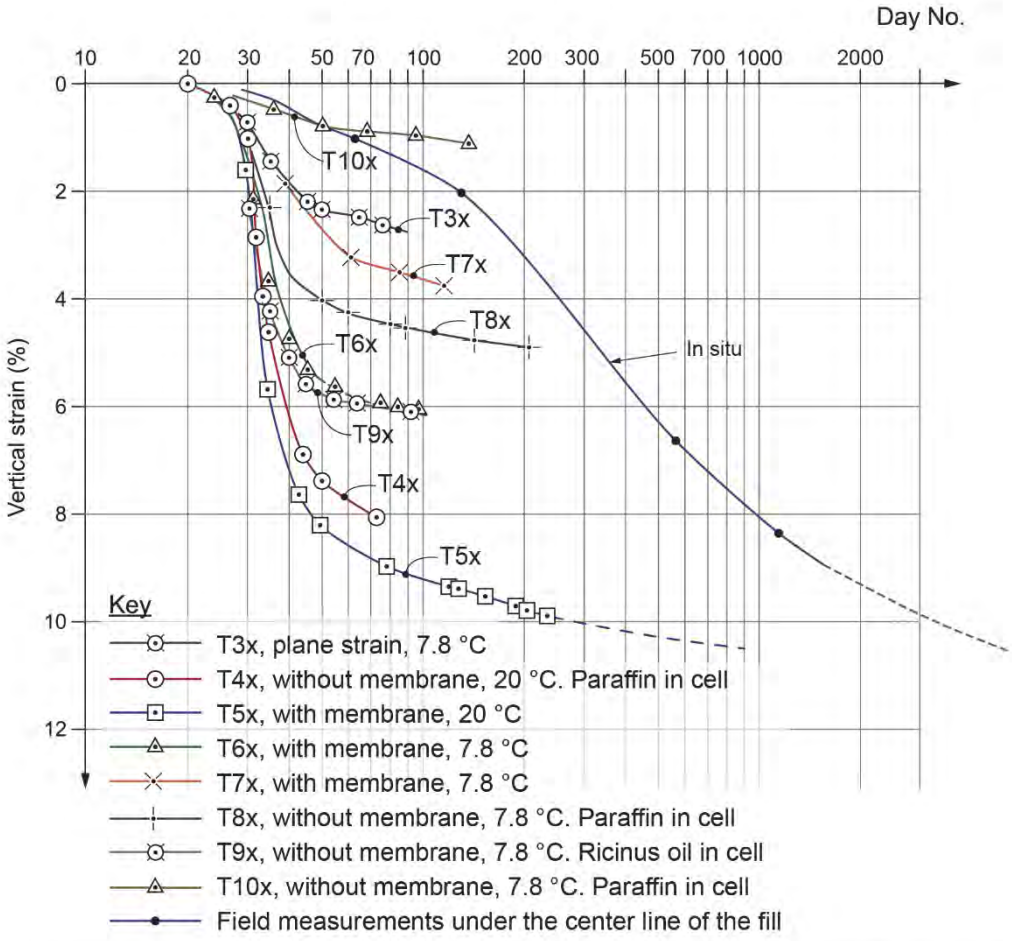



Estimated from oedometer and undrained triaxial tests
Estimated from oedometer tests and undrained triaxial tests and permeability values back-calculated from field data
Measured in the field at point A
A
Settlement of point A (mm)
0
200
400
600
800
1000
10
20
50
100
200
500
1000
2000
Day No.
1 yr
2 yrs
3 yrs
5 yrs

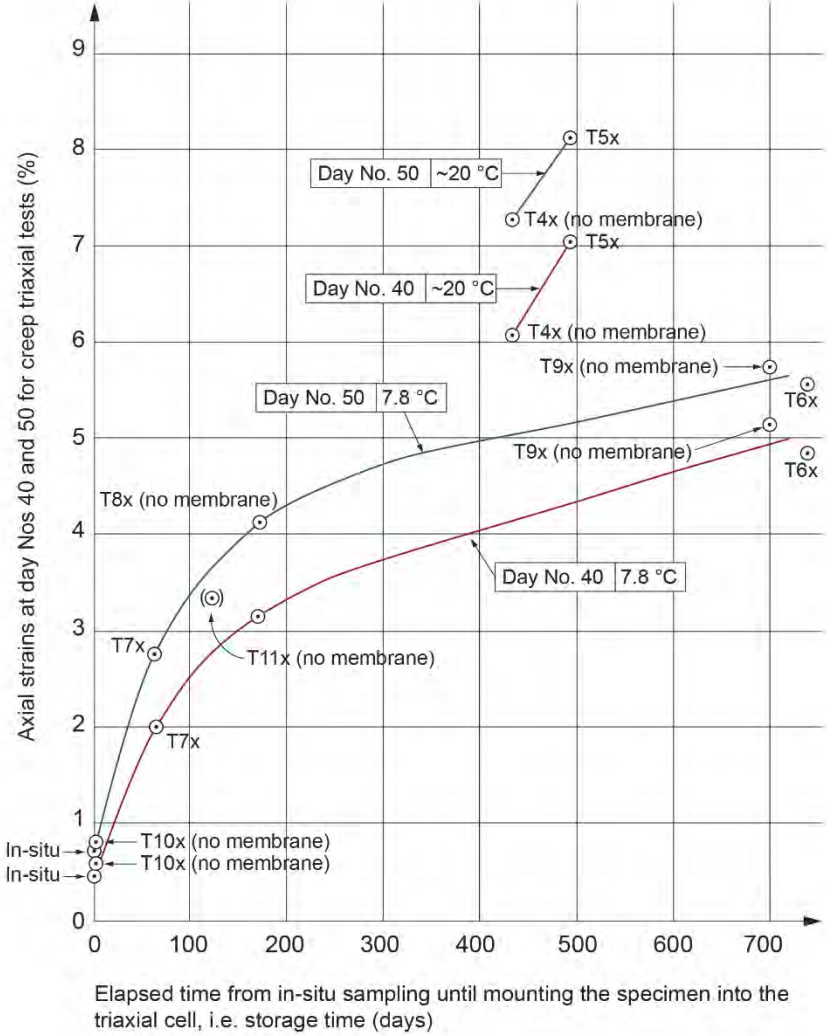

Vertical strain ε_v (%)
0
2
4
6
8
10
12
Initial depth below original ground surface (m)
0
2
4
6
8
10
12
14
16
18
Measured vertical strain in the field at Day No. 1116 (from fig. 10)
Calculated vertical strain from the oedometer tests when applying the calculated effective vertical stresses at Day No. 1116
CRSC oedometer tests
K_o triaxial tests (24 hr stress increments)


℄
20 m
2.3 m
Test fill
5.2 m
σv′ (σa′ in triax. test)
σH′ (σr′ in triax. test)
Element considered
to study effect of
storage time


Stress (kPa)
σa' for tests T6x, T7x, T8x, T9x and T10x (New FE programme)
σa' for tests T4x and T5x (Old FE programme)
σr' for tests T6x, T7x and T8x
σr' for tests T4x and T5x
Day No.
Strain (%)
εr for T6x
εr for T4x
εr for T10x
εr for T9x
εr for T5x
εH in the field
εr for T8x
εH for T3x
εr for T7x
εa for T10x (without membrane, cell fluid: paraffin oil, temperature +7.8 °C)
εa for T3x (plane strain, temperature +7.8 °C)
εa for T7x (temperature +7.8 °C)
εV in the field
εa for T8x (without membrane, cell fluid: paraffin oil, temperature +7.8 °C)
εa for T6x (temperature +7.8 °C)
εa for T9x (without membrane, cell fluid: Ricinus oil, temperature +7.8 °C)
εa for T4x (without membrane, cell fluid: paraffin oil, temperature ~20 °C)
εa for T5x (temperature ~20 °C)
Note: For tests T3x, T6x and T7x the slope of the curves later than day no. 50 may be too low due to membrane stiffening caused by the lower temperature (7.8 °C)



Note: For tests T3x, T6x and T7x the slope of the curves later than day no. 50 may be too low due to membrane stiffening caused by the lower temperature (7.8 °C)


Axial strains at day Nos 40 and 50 for creep triaxial tests (%)
0
1
2
3
4
5
6
7
8
9
0
100
200
300
400
500
600
700
Elapsed time from in-situ sampling until mounting the specimen into the triaxial cell, i.e. storage time (days)
T5x
Day No. 50
~20 °C
T4x (no membrane)
T5x
Day No. 40
~20 °C
T4x (no membrane)
T9x (no membrane)
T6x
Day No. 50
7.8 °C
T9x (no membrane)
T6x
T8x (no membrane)
Day No. 40
7.8 °C
T11x (no membrane)
T7x
T7x
In-situ
T10x (no membrane)
In-situ
T10x (no membrane)

NGI

Settlement analysis of Onsøy test fill

Magne Mehli

NGI Trondheim

Settlement analysis of Onsøy test fill

- Scope of work
  - Interpret soil investigations from Onsøy by modelling oedometer and triaxial tests in PLAXIS.
  - Prediction with Soft Soil Creep (SSC).
  - Evaluate data and adjust input data to find best fit for field measurements with SSC.
  - Test different creep models.
- Project objective
  - To validate existing creep models in FE benchmark problems and identify most relevant mechanisms in soft soil creep modelling.

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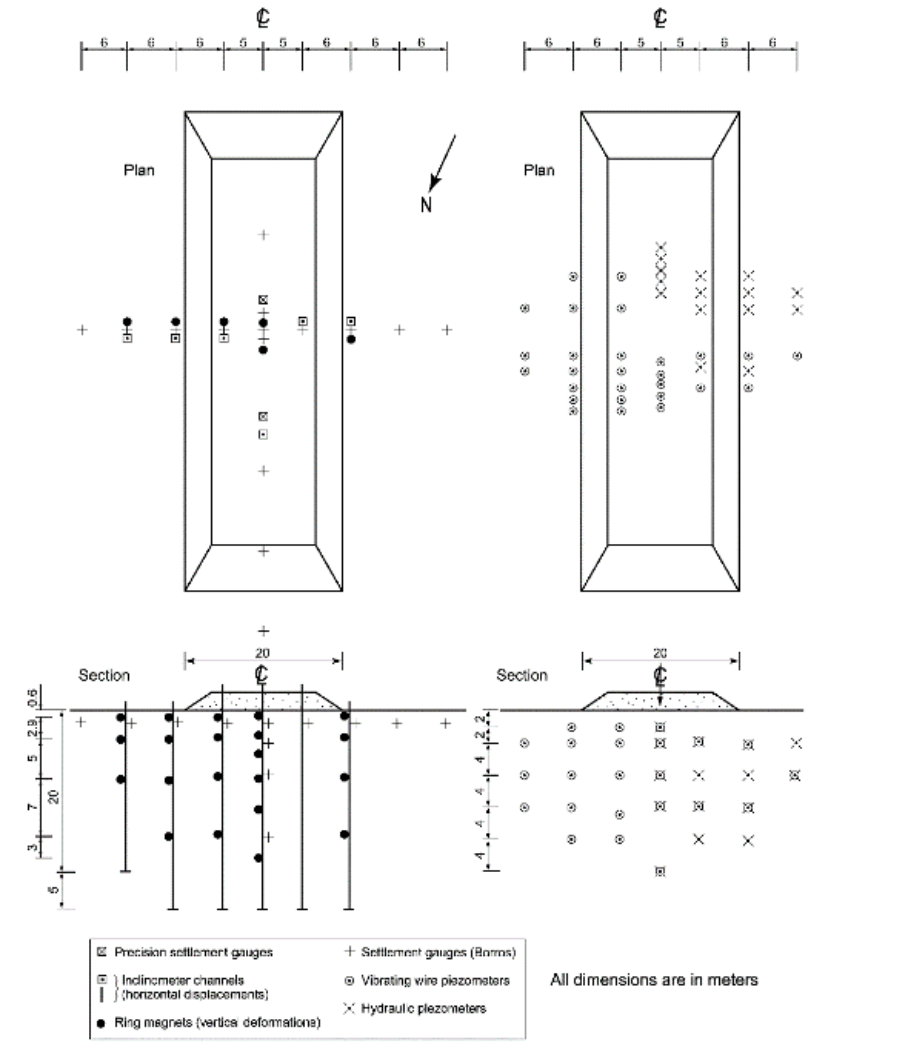
Onsøy tes

- 2,3 m high te 072
- The fill was a o failure
- Main purpos ormation characteristic
- The test fill w nted

Interesting *Toralv Berre*



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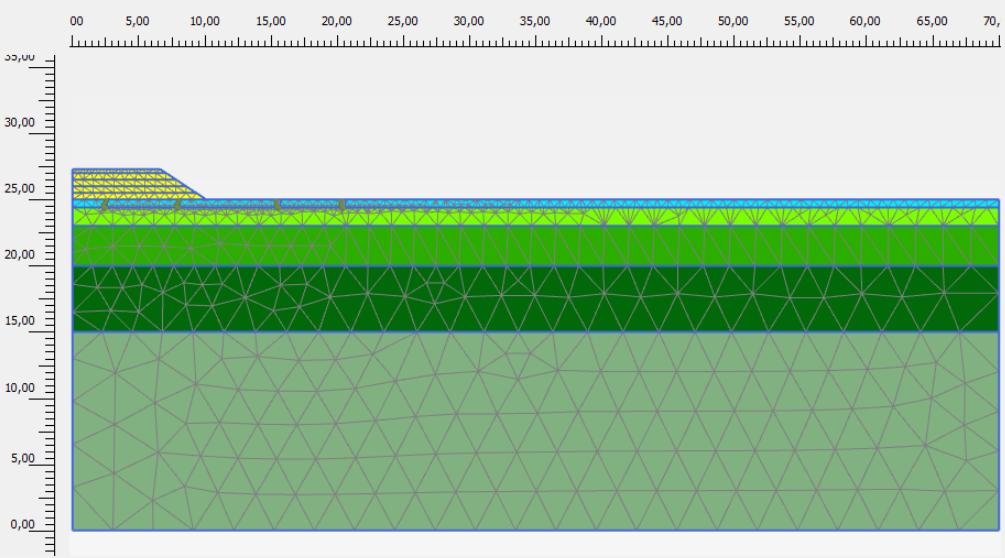
Finite Element Model



Phase	Description	Phase duration [days]	Accumulated time [days]	"Real time" [days]
0	Initial phase – K_0-procedure	-	-	22
1	Fill 0,5m	1	1	23
2	Wait	1	2	24
3	Fill 0,5m	1	3	25
4	Wait	3	6	28
5	Fill 0,5m	1	7	29
6	Wait	3	10	32
7	Fill 0,5m	1	11	33
8	Wait	2	13	35
9	Fill 0,3m	1	14	36
10	Wait	6	20	42
11	Wait	24	44	66
12	Wait	38	82	104
13	Wait	43	125	147
14	Wait	37	162	184
15	Wait	145	307	329
16	Wait	227	534	556
17	Wait	235	769	791
18	Wait	274	1043	1065
19	Wait	43	1086	1108
20	Wait	12	1098	1120

Creep models

Model	Plasticity					Elasticity
	Creep		Anisotropy	Destructuration	Lode angle	Small strain shear stiffness
	Vol. creep	Pl. multiplier				
SSC	✓	-	-	-	-	-
CS-SSCG	-	✓	-	-	✓	✓
Sekiguchi-Ohta	✓	-	✓	-	-	-
n-SAC	-	✓	✓	✓	✓	-
KRYKON	“✓”	-	-	“✓”	-	-

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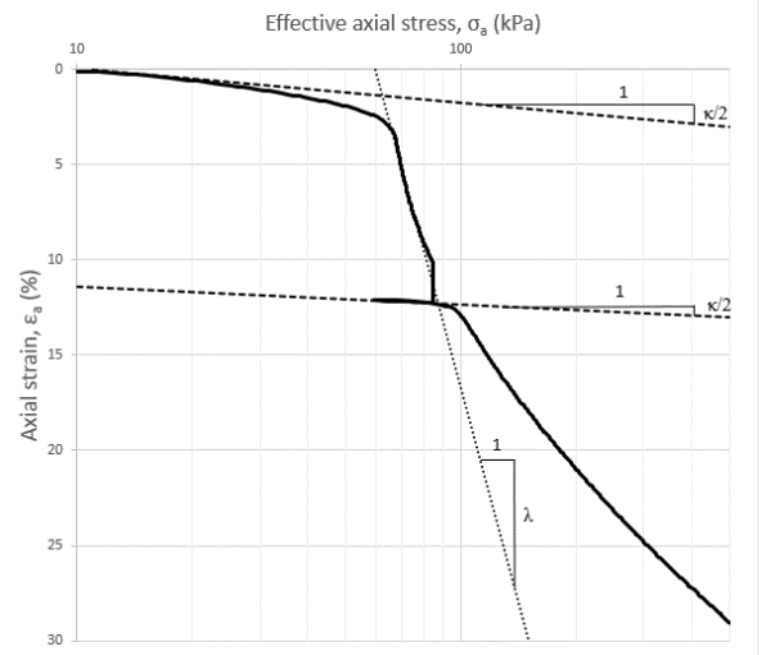
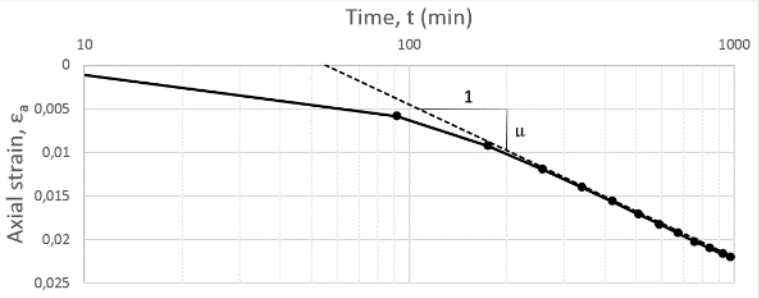
Soft Soil Creep (SSC)

Parameter	Description
c	Cohesion
φ	Friction angle
ψ	Dilatancy angle
κ*	Modified swelling index
λ*	Modified compression index
μ*	Modified creep index
υ_{ur}	Poisson's ratio for unloading-reloading
K_0^{NC}	Stress ratio in a state of normal consolidation
OCR	Overconsolidation ratio
POP	Preoverburden pressure
M	K_0^{NC}-related parameter

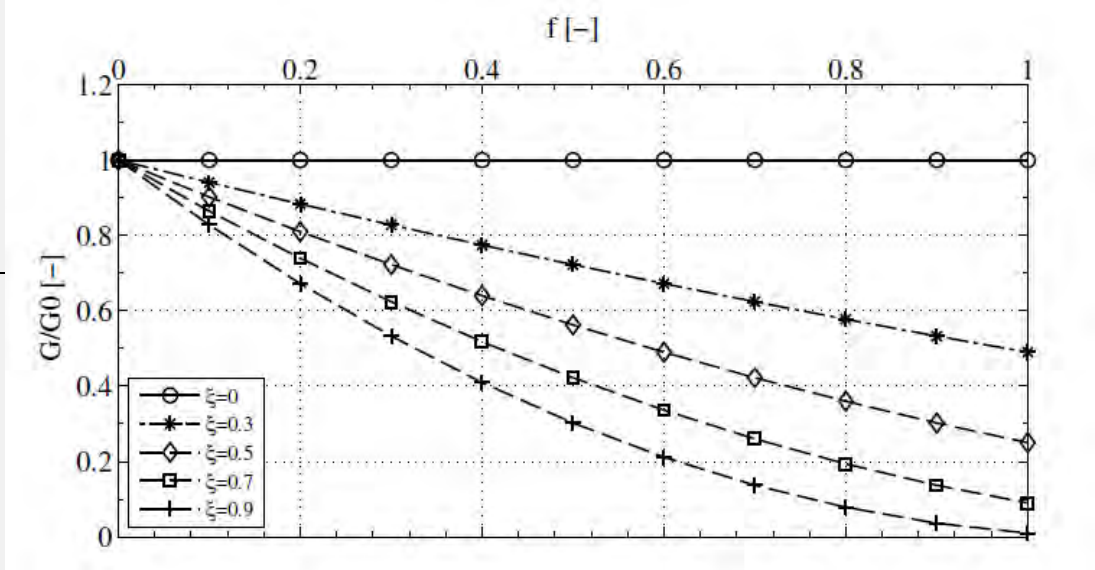
Stolle et al. (1999a)
Stolle et al. (1999b)





Critical State Soft Soil Creep with non-linear Shear stiffnes (CS-SSCG)

Parameter	Description
κ^*	Modified swelling index
λ^*	Modified compression index
μ^*	Modified creep index
η_{K0}	Stress ratio at K_0'. $\eta_{K0} = q/p'$ at rest $\rightarrow \eta_{K0} = 3(1-K_0')/(1+2K_0')$
ζ	Shear stiffness degradation factor, $G_M = G_0\,(1 - \xi \cdot f)^2$, $f = (\eta - \eta_{K0})/(M_0 - \eta_{K0})$
K_0^{NC}	Stress ratio in a state of normal consolidation
OCR_τ	Overconsolidation ratio at reference time
POP_τ	Preoverburden pressure at reference time
M_c	Slope of the critical state line
τ	Reference time
y_{ref}	Reference depth
G_{ref}	Shear stiffness at reference depth
G_{inc}	Shear stiffness increase per meter depth



Ashrafi (2014)

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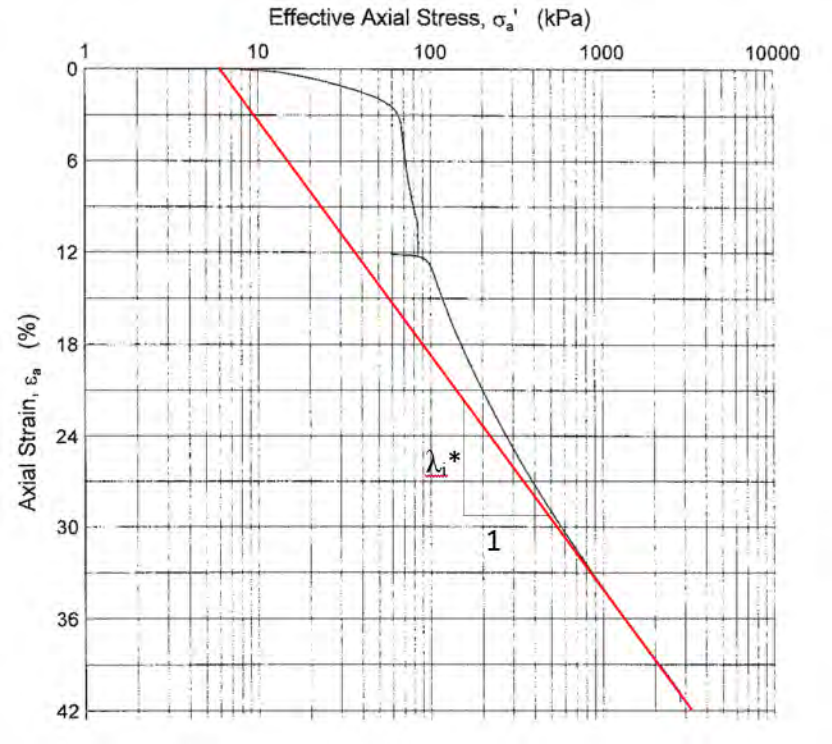
Non-associated creep model for Structured Anisotropic Clay (n-SAC)

Parameter	Description
υ	Poisson's ratio for unloading-reloading
K_0^{NC}	Earth pressure coefficient at rest in normally consolidated stress state (for remoulded material) This value would typically be a bit smaller than what is normally measured for natural clay
E_{ref}	Elastic Young's modulus at p_{ref}
E_{oed}^{ref}	Intrinsic oedometer modulus at p_{ref}
p_{ref}	Reference stress. Typically 100 kPa
$r_{s,min}$	The minimum time resistance number
$r_{s,i}$	The intrinsic time resistance number
ω	Gives the contribution of viscoplastic shear strain to destructuration
φ_T	Friction angle at peak of undrained stress path
φ_{CS}	Critical state friction angle
OCR_τ	Together with the reference time this parameter defines the position of the reference surface to the initial stress condition. POP can also be used.
t_{max}	Is a time for which significant reduction in creep rate occurs.

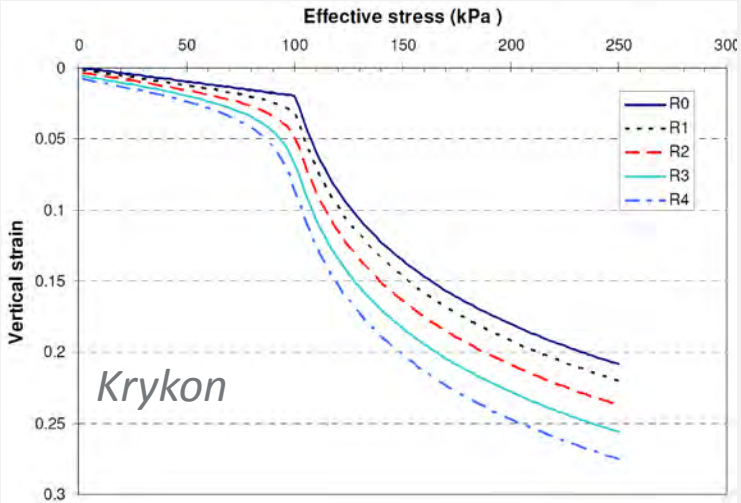
Grimstad and Degago (2010)

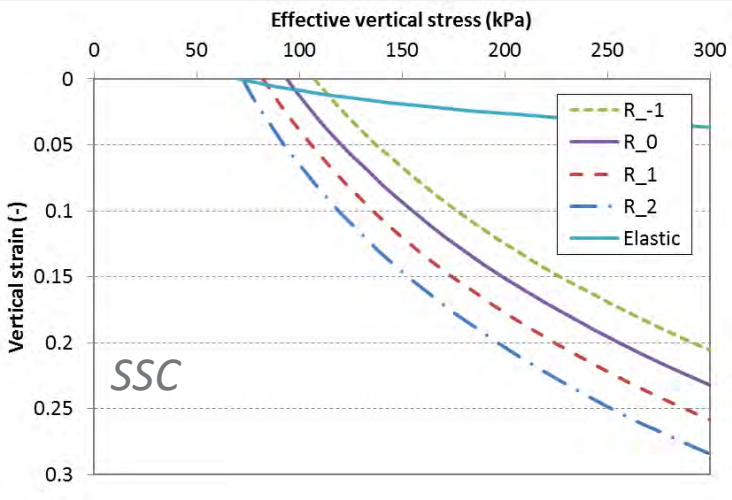


Krykon

Parameter	Description
M_{OC}	Oedometer modulus in OC state (from σ_0' to p_c')
m	Oedometer modulus number in NC state
p_c'	Pre-consolidation stress
p_r'	Intersection stress
Rc	Time resistance for the reference curve
r_0	The time resistance number at σ_0'
r_{pc}	The time resistance number at p_c'
m_r	Increase in time resistance with stress above p_c'

Svanø (1986)





Calculations

- Prediction with SSC
- “Best-fit” with SSC
- CS-SSCG, n-SAC and Krykon with comparable soil parameter input
- Calculation with Soft Soil (SS) to study what is the effect of not including creep when the soil parameters are obtained from high quality samples

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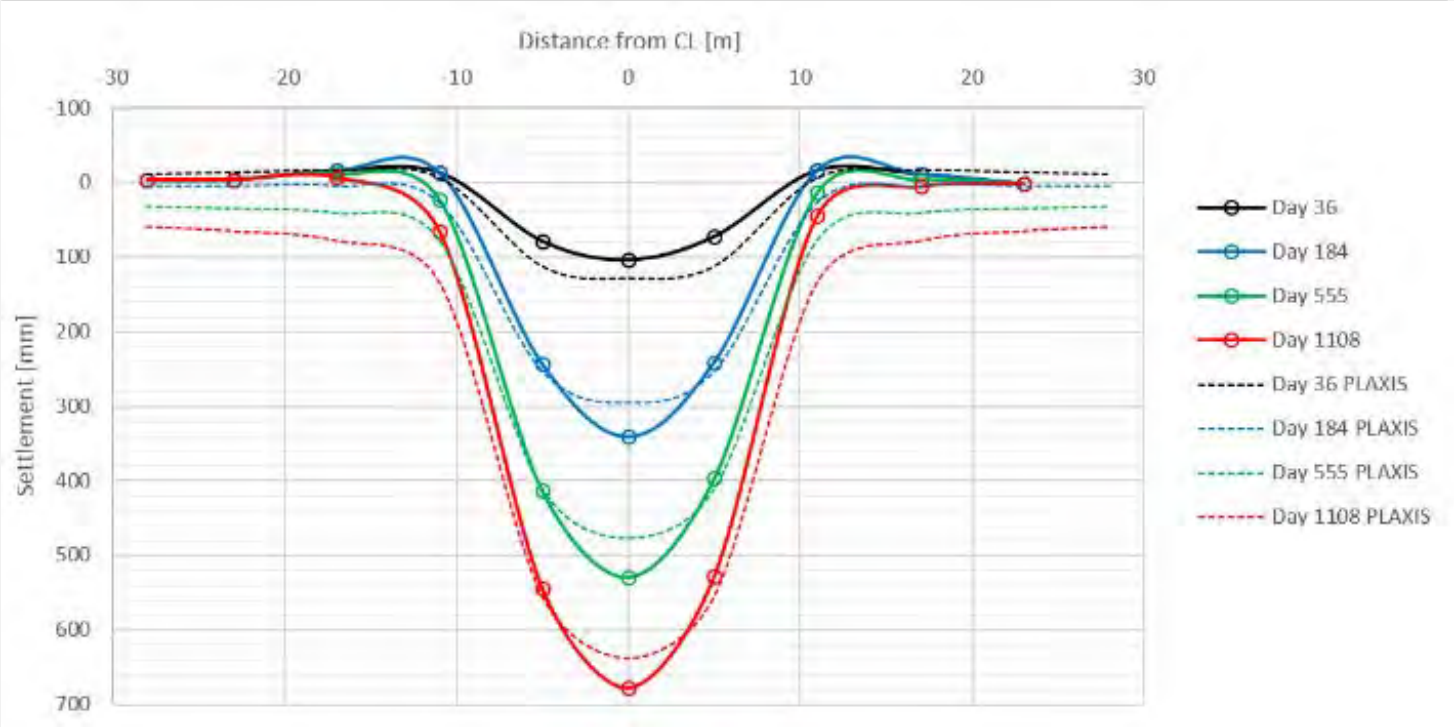
Results – Vertical displacement Prediction with SSC



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Results
Prediction with SSC

$$\dot{\varepsilon}_c = \dot{\varepsilon}_{c0} \cdot OCR^{-\left(\frac{\lambda^* - \kappa^*}{\mu^*}\right)}$$

$$\dot{\varepsilon}_{c0} = \frac{\mu^*}{\tau_0}$$

OCR \ $\frac{\lambda^* - \kappa^*}{\mu^*}$	30 [% / year]	25 [% / year]	20 [% / year]	15 [% / year]	10 [% / year]
1,1	21	34	54	87	140
1,2	1,5	3,8	9,5	24	59
1,4	0,02	0,08	0,4	2,3	13
1,6	0,0003	0,003	0,03	0,3	3,3
1,8	0,00001	0,0002	0,003	0,05	1,0

$\mu^* = 0{,}01$ and $\tau_0 = 1$ day

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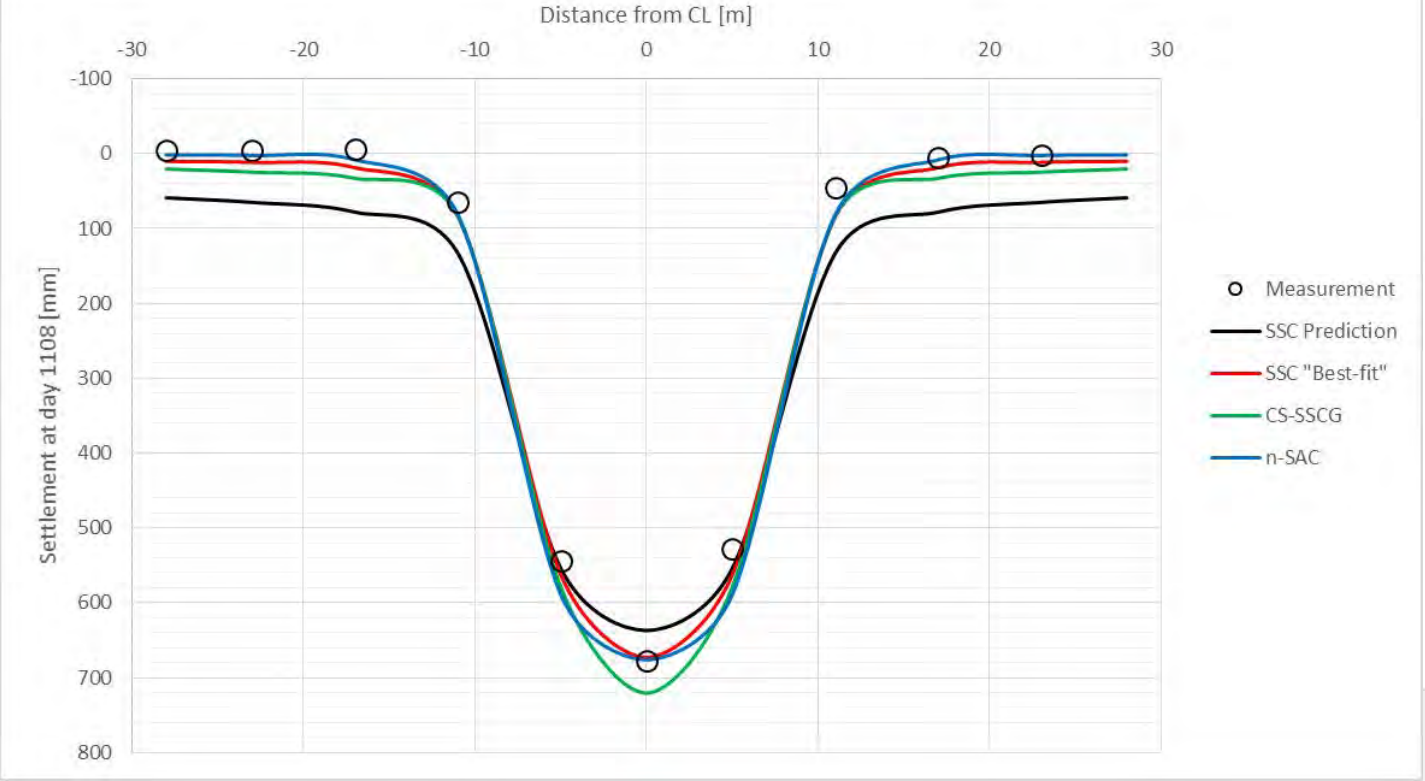
Results – Vertical displacement and strains
All calculations



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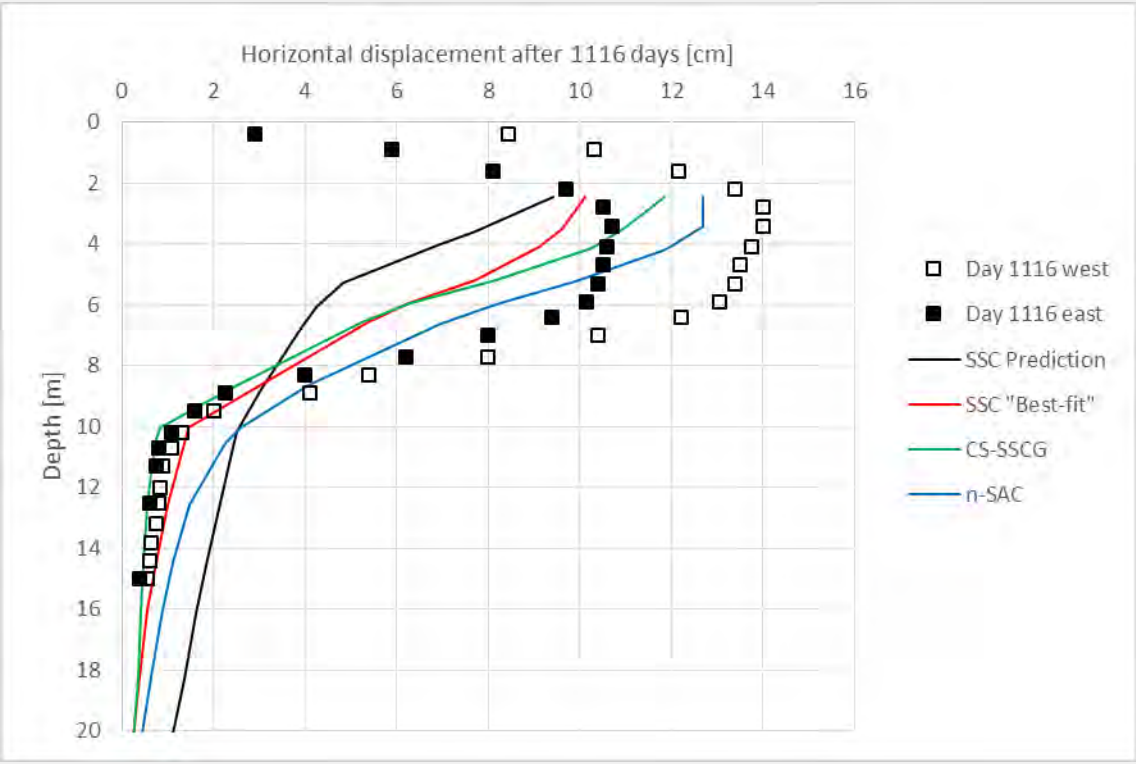
Results – Horizontal displacement
All calculations



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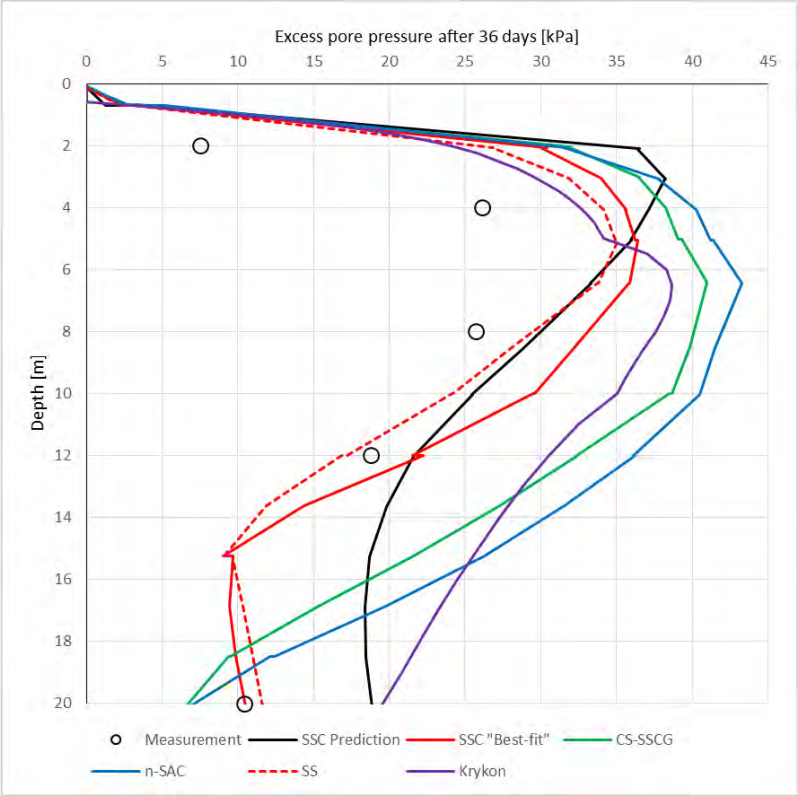
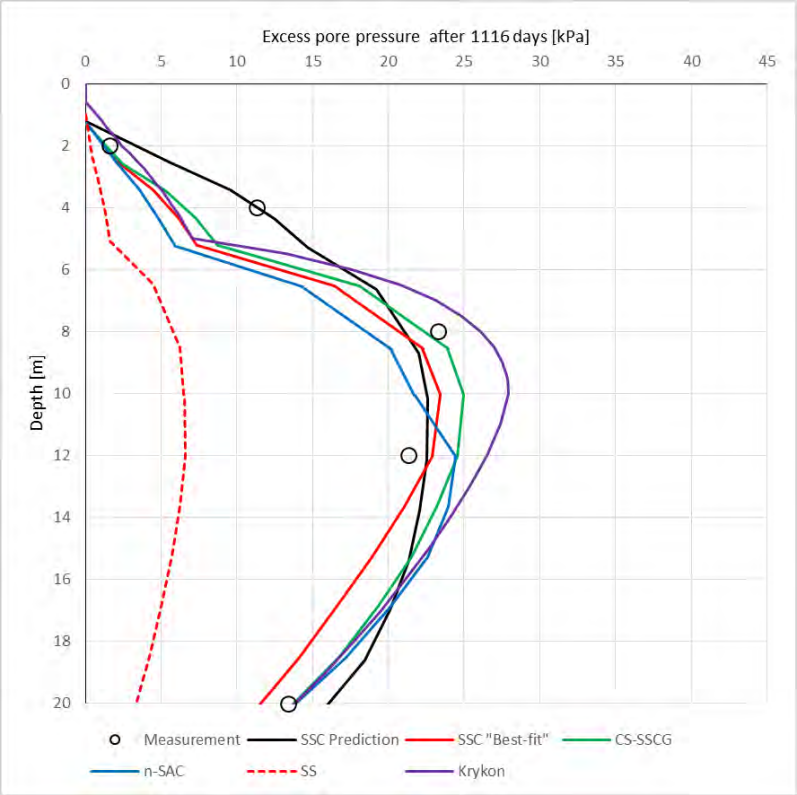
Results – Excess Pore Pressure under centerline
All calculations





Summary

- The Soft Soil Creep model is able to give good estimates of vertical displacements, excess pore pressure and long term horizontal displacement to some degree when good quality laboratory test results are available
- The Soft Soil Creep model is not able to capture the horizontal displacement during undrained conditions due to the low shear stiffness. This also affects the initial vertical displacement
- The CS-SSCG model produces good results for horizontal displacement during undrained conditions
- The long term horizontal displacement is best captured with the n-SAC model
- The Krykon model gives good results for vertical displacements and strains and pore pressure after 1116 days, but its more complicated to handle the input data than for the more advanced models. This is mainly due to the R_c parameter giving at to high initial strain rate.
- The calculation with the Soft Soil model shows that creep can be an important factor when doing settlement calculations with soil parameters based on quality laboratory data.

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Some recommendations

- The choice of OCR is very important when using soil models with creep formulation. The degree of sample disturbance must be evaluated when interpreting laboratory data. Check the initial strain rate in the calculation model.
- It’s useful to back-calculate oedometer test (CRS or IL) to determine a parameter set. Fit the stress range relevant for the problem.
- If quality laboratory data is not available, then one should consider to neglect creep when evaluating final settlement.

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Questions ?

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Some experiences with practical use of creep programs

Dr. Ph. Kjell Karlsrud

Expert Adviser, NGI

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Content

- Some comments to creep models presently in use in Norway
- Some experiences from observed and calculated settlements due to landfill and building construction around Oslo Central Station
- Revisting some observations by Bjerrum in relation to settement of buildings in Drammen
- Way forward

Programs part of Geosuite Settlement package

- KRYKON model based on Janbu/Svanø time resistance concept
- CHALMERS model (Claesson, Sällfors,…) more founded on the Bjerrum model seems more versatile and better guidance is given by e.g. Claesson (2003) wrt determination of relevant parameters
- The major difference lies in how creep parameters below the apparent pre-consolidation pressure is defined

The Geosuite user manuals should provide more guidance for how to determine relevant creep parameters

- What can we determine from conventional incremental or CRS oedometer tests?
- Do we have sufficient empirically based data and guidance for selecting all relevant parameters that are needed for the different models?

Definition of parameters

<u>Compressibility parameters</u>

$C_c/(1+e_0) = \lambda = \ln 10/m$ = Virgin compression index

m= Janbu's modulus number

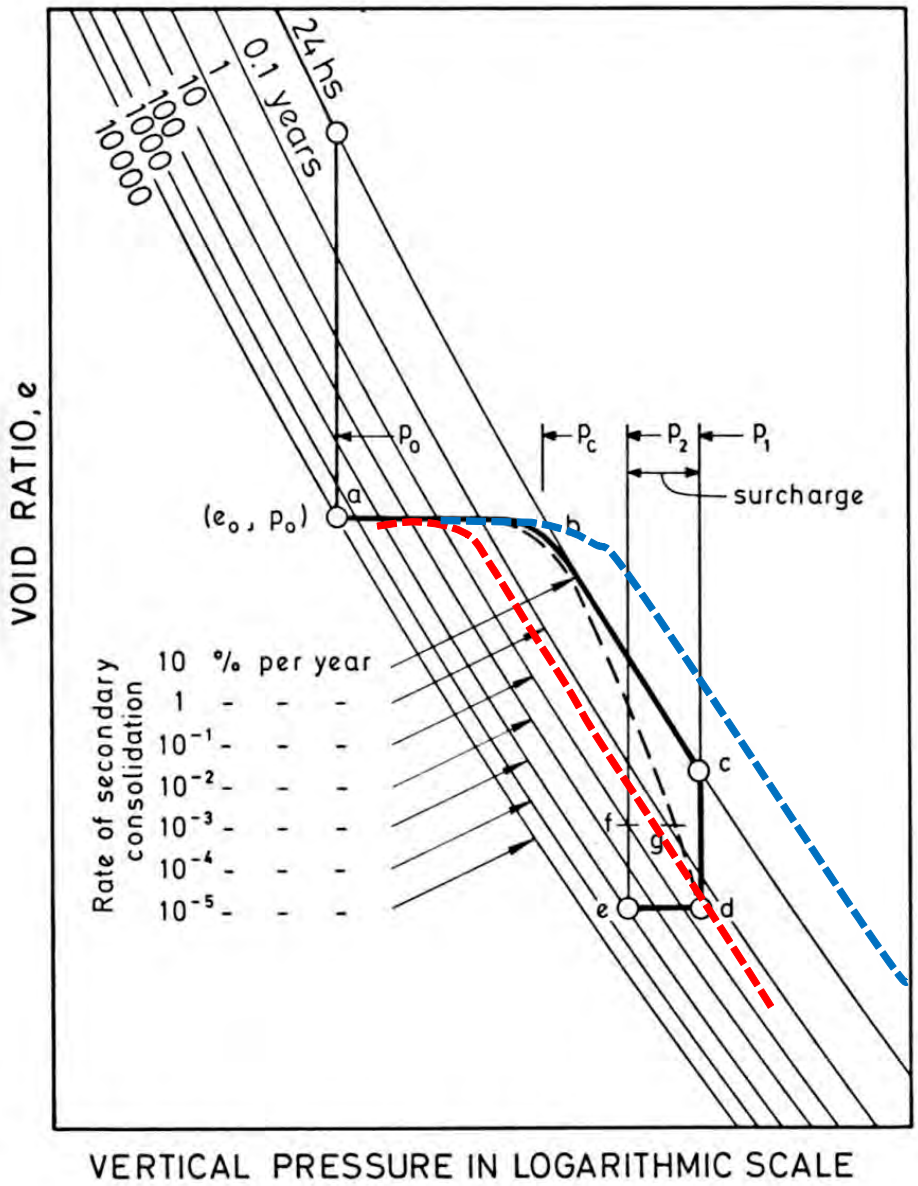
$C_r/(1+e_0) = \kappa$ = Recompression index

<u>Creep parameters</u>

$C_\alpha/(1+e_0) = \alpha_s = \ln 10/r$ = coefficient of secondary compression

r= time resistance number (Janbu)

Creep behavior and Isotach curves proposed by Bjerrum , 1967



Rate dependant p'_c and OCR must be considered to properly anchor creep curves

Figure after Bjerrum, 1967

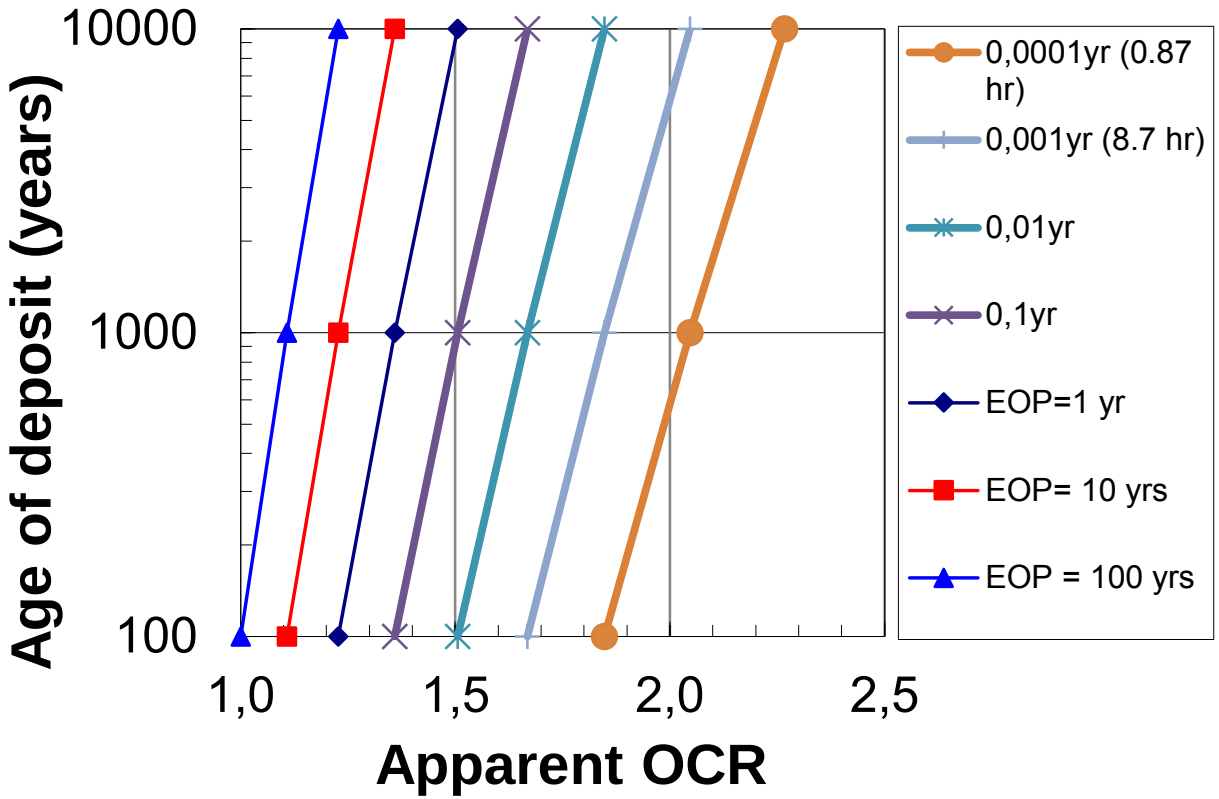
Mesri and Castro (1987) showed close correlation C_α and Cc.

Typical value: C_α/C_c =0,04

- An implication of this is that apparent OCR due to creep is independent of clay type

Apparent OCR due to creep

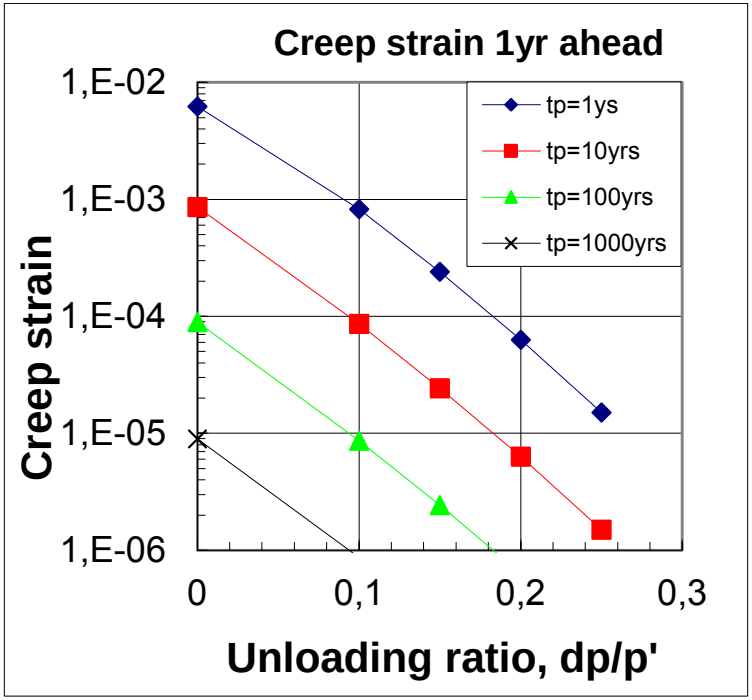
for C_α/C_c =0.04 and C_r/C_c = 0.1

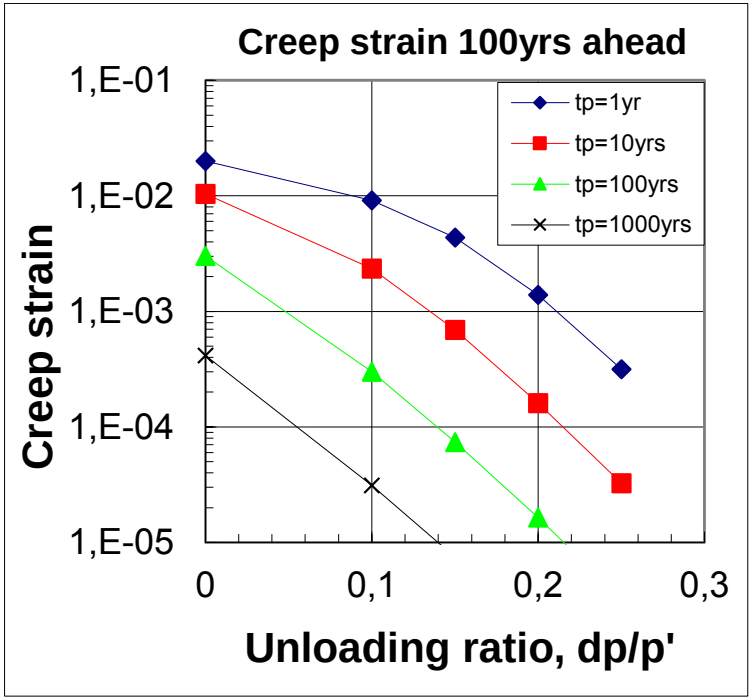


OCR values are unreasonably large, suggesting C_α decreases with time

Use of Isotach curves to directly assess impact of pre-loading on future creep

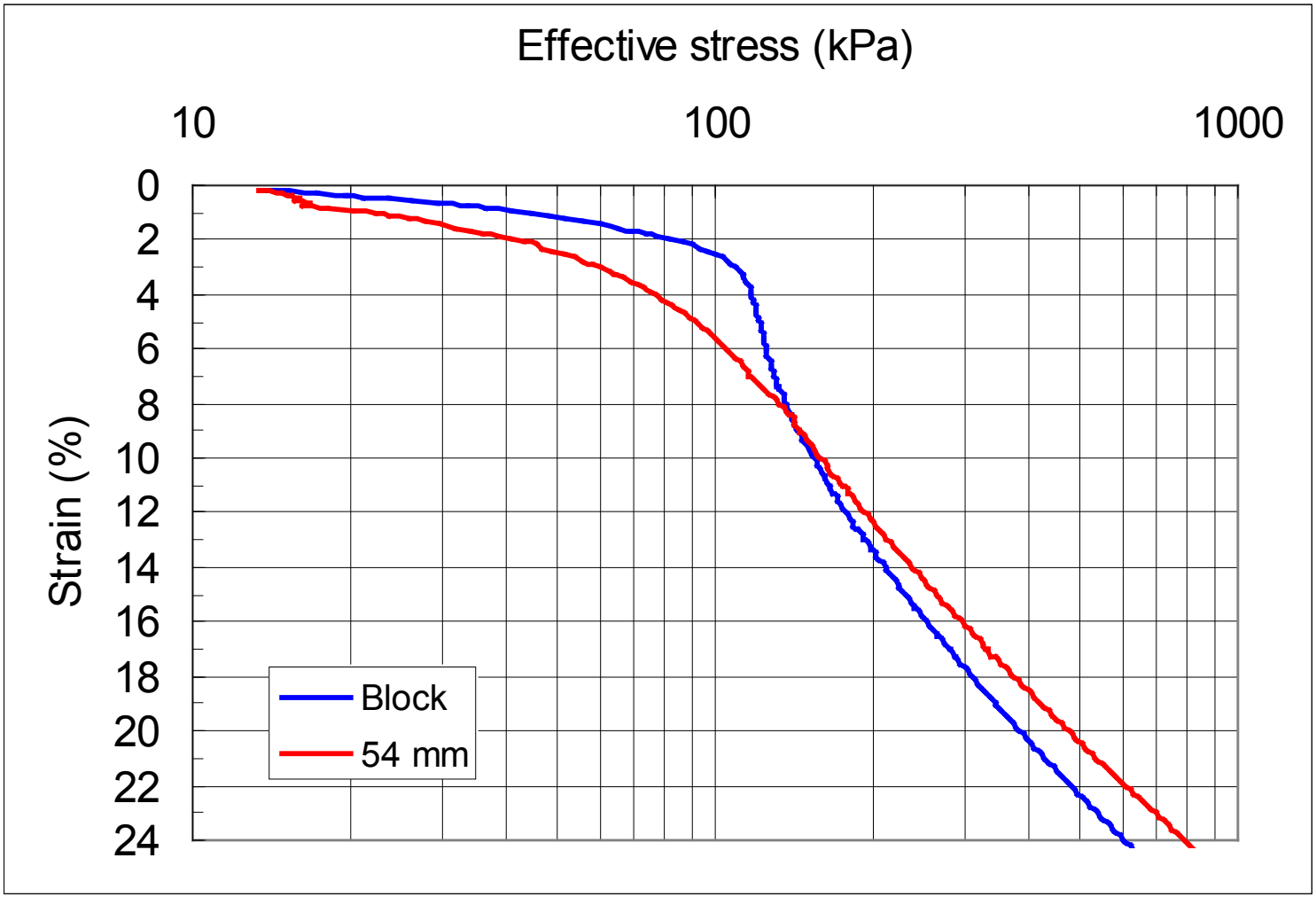
(for clay with λ= 0.25 or m=9,2)





Strain of 10^{-4} corresponds to 1 mm settlement for 10 m thick clay layer

Illustration of sample disturbance effect, Onsøy clay, z=14.2 m



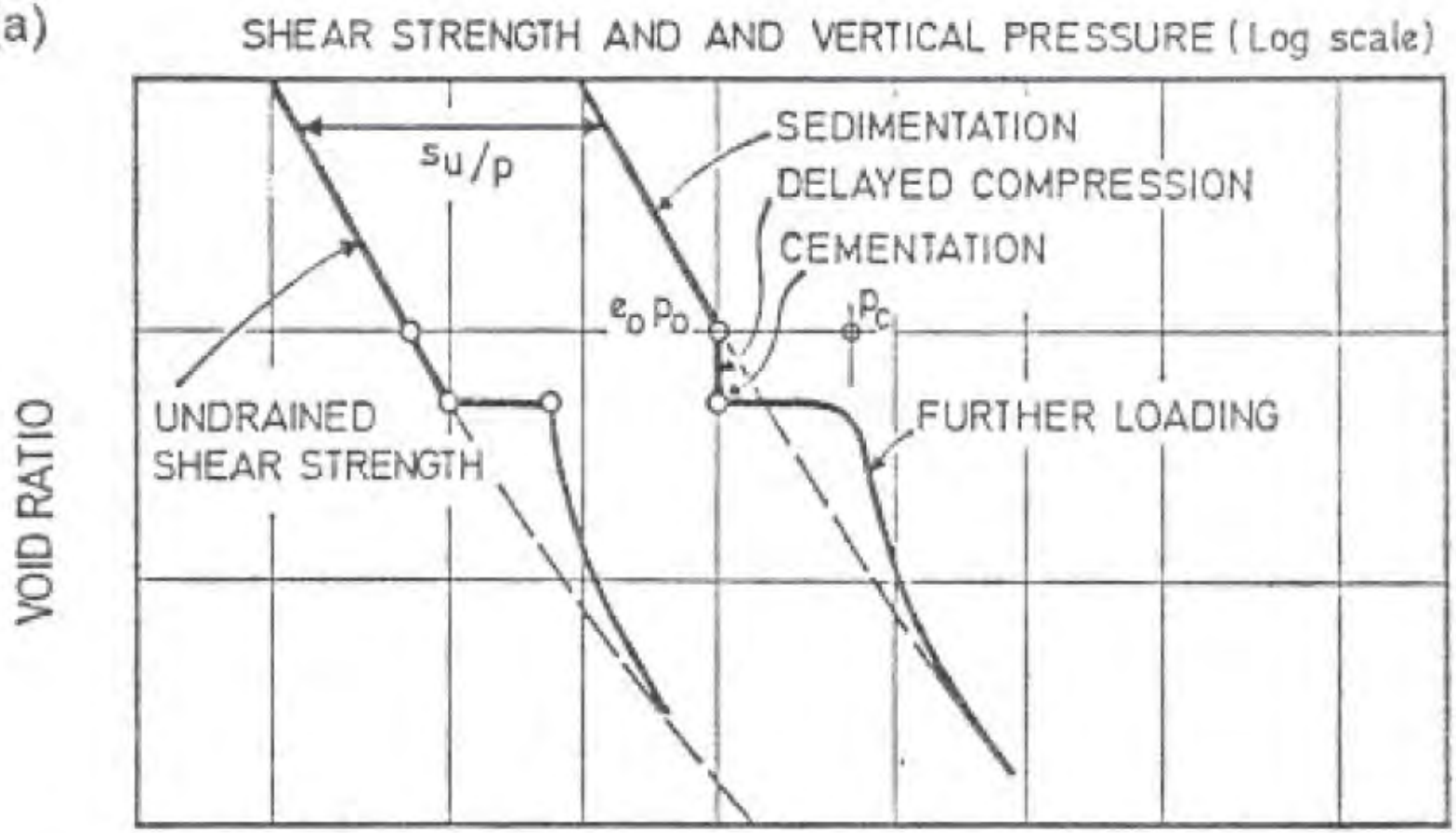
Sample quality has a pronounced effect on the behavior and must be accounted for when selecting parameters

Cementation or chemical bonding may have an effect on basic creep behavior that diffesr from pure secondary creep effects

Causes may be:

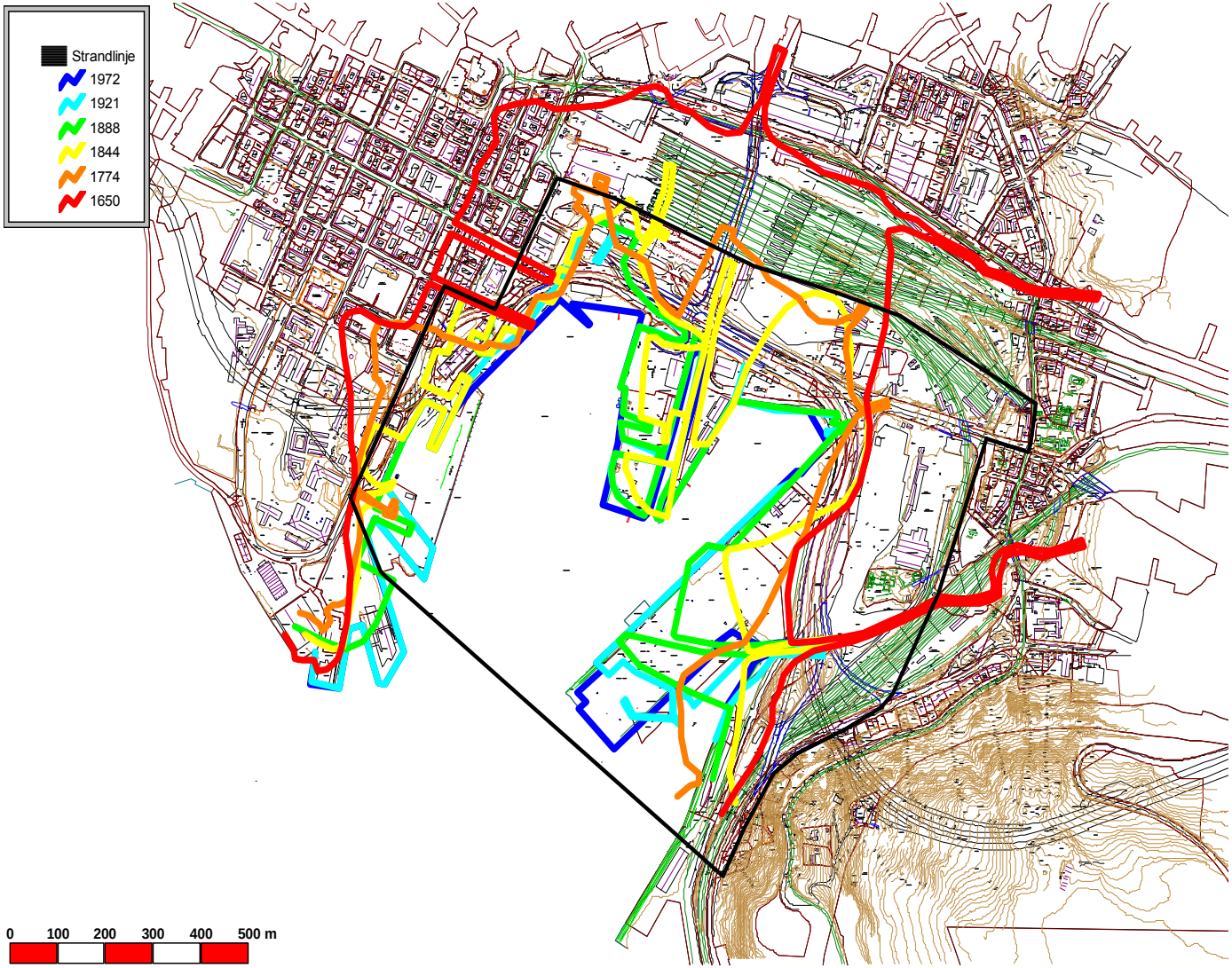
- Cold welding of contact points
- Echange of cations
- Preciptation of cementing agents

(a)

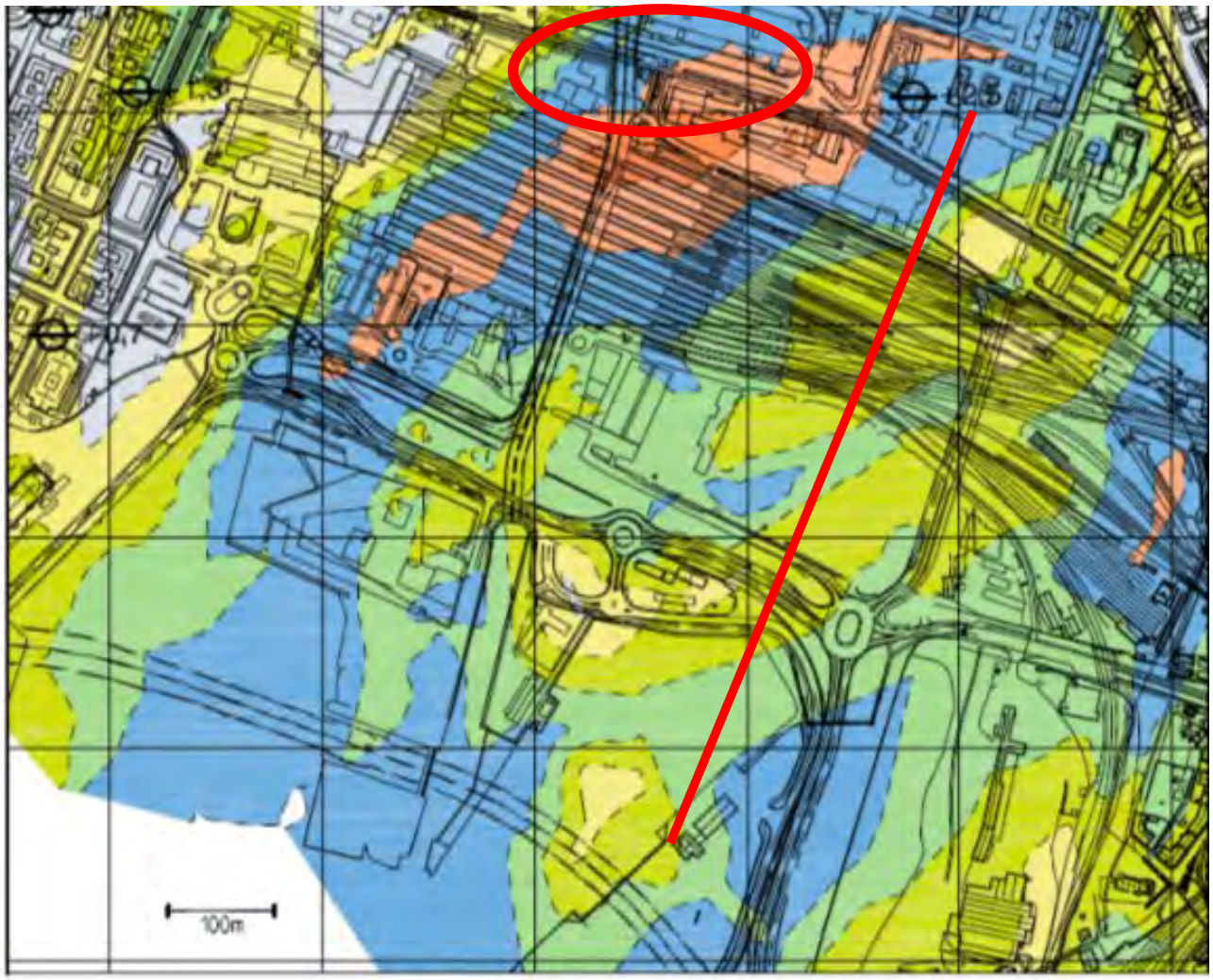

SHEAR STRENGTH AND AND VERTICAL PRESSURE (Log scale)
VOID RATIO
s_u/p
SEDIMENTATION
DELAYED COMPRESSION
CEMENTATION
e_0 p_0
p_c
UNDRAINED
SHEAR STRENGTH
FURTHER LOADING

Example of measured and calculated settlements Oslo Central Station area

Land reclamation after 1650



Depth to bedrock



Dybdekoter til berg :

- 0 til -10
- -10 til -20
- -20 til -30
- -30 til -40
- -40 til -60
- -60 til -80
- > -80

Eksisterende poretrykksmåler ved berg på 1970-tallet

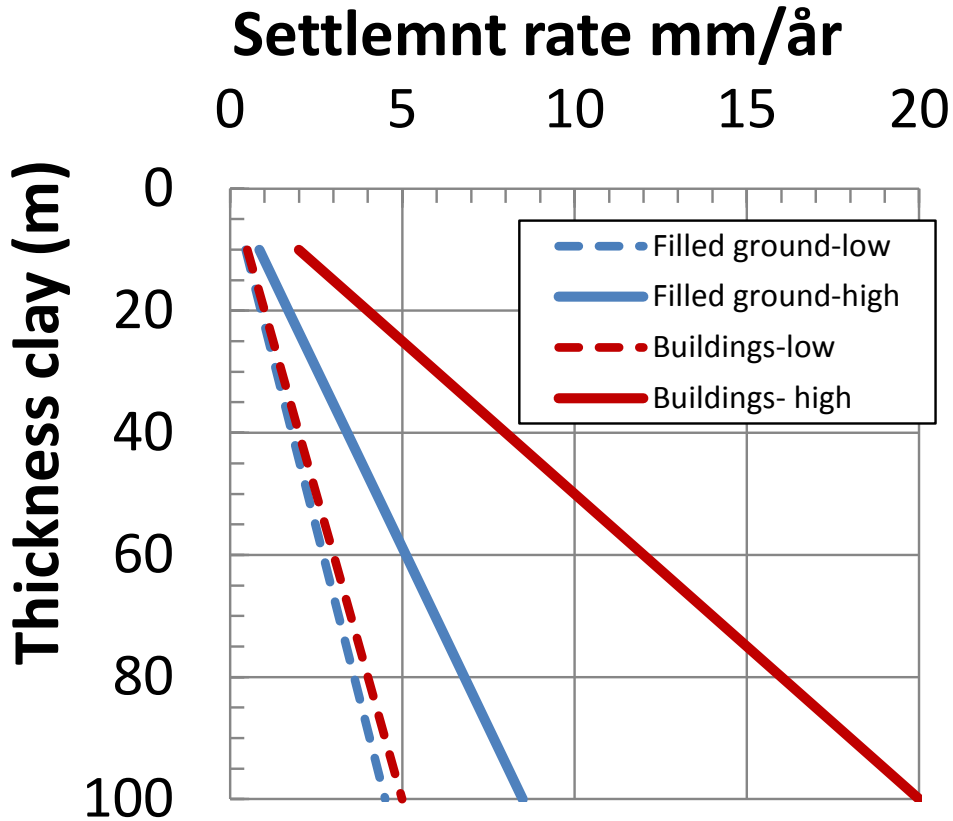
1,0 Normal stigehøyde

Natural settement rates in Oslo period 1950-70

(Røste og Sander, 1971)

Causes:

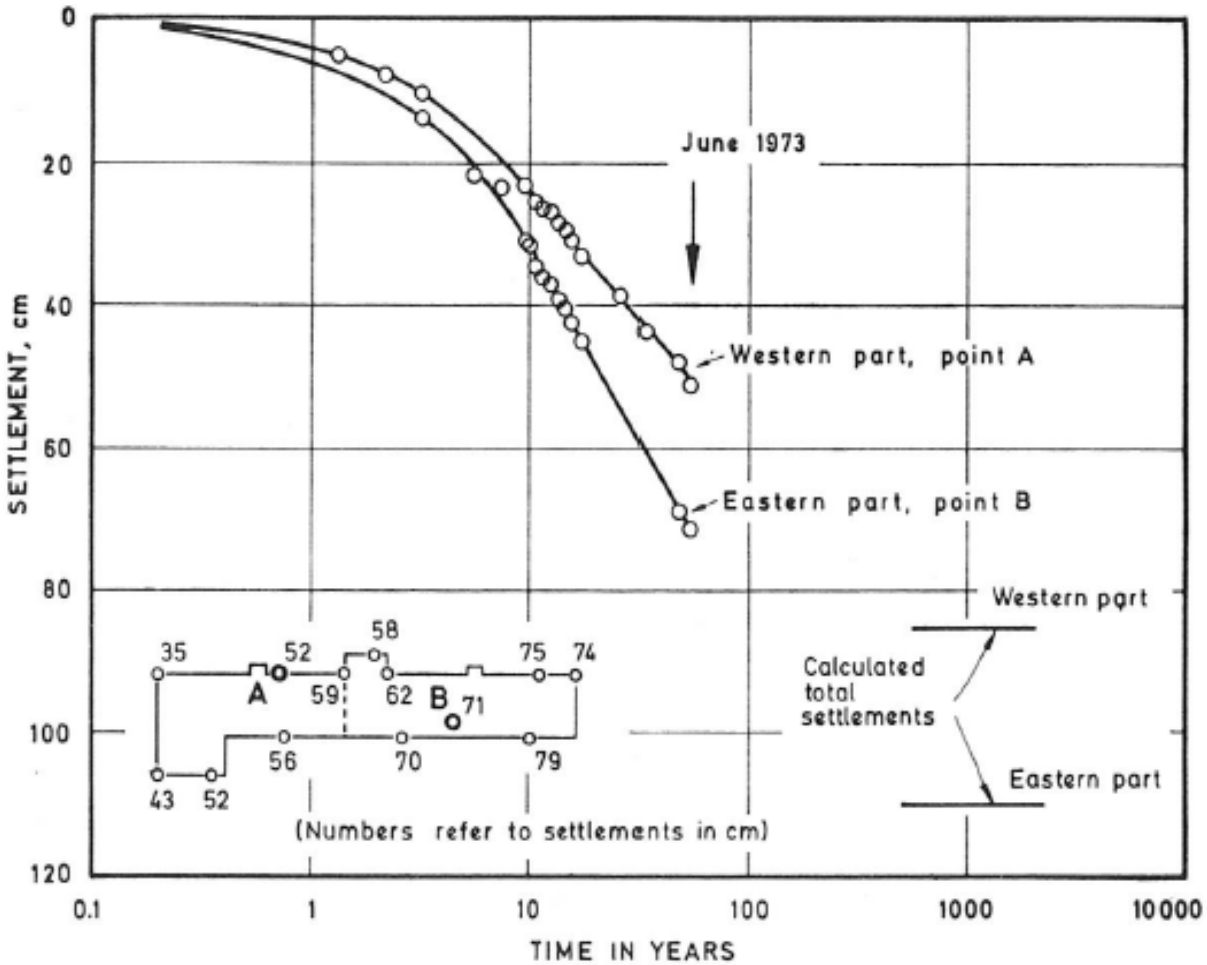
- Landfill
- Building loads
- Isostatic uplift



InSAR data NGU (2004) for period 1992-2001 generally agrees with these observations

Measured settlement Oslo Jernbanetollsted 1919-73

(Andersen og Clausen, 1974)



A, q= 50 kPa

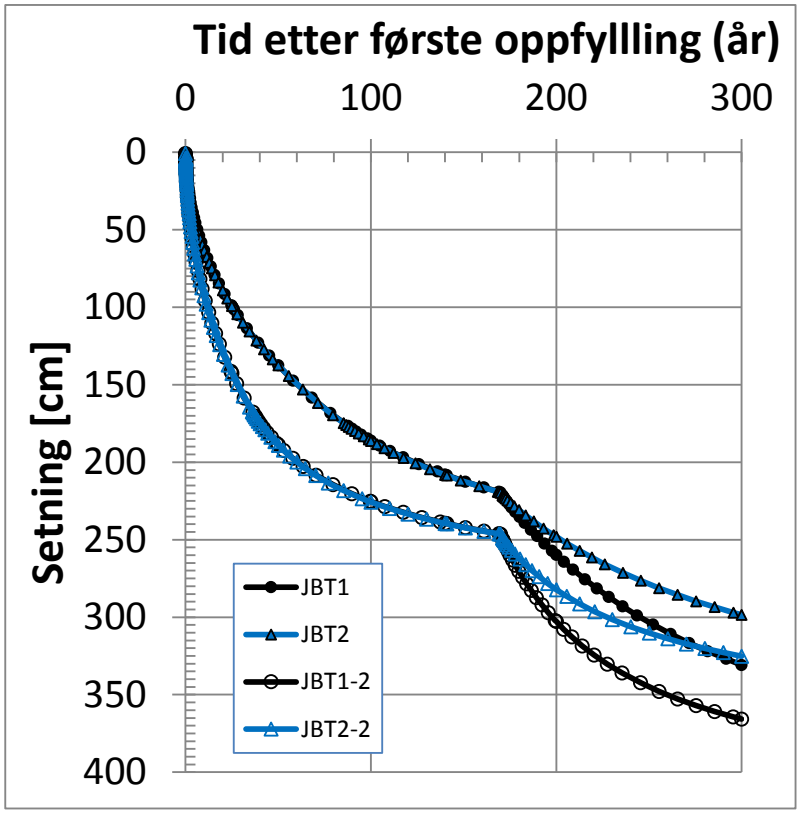
B, q= 90 kPa

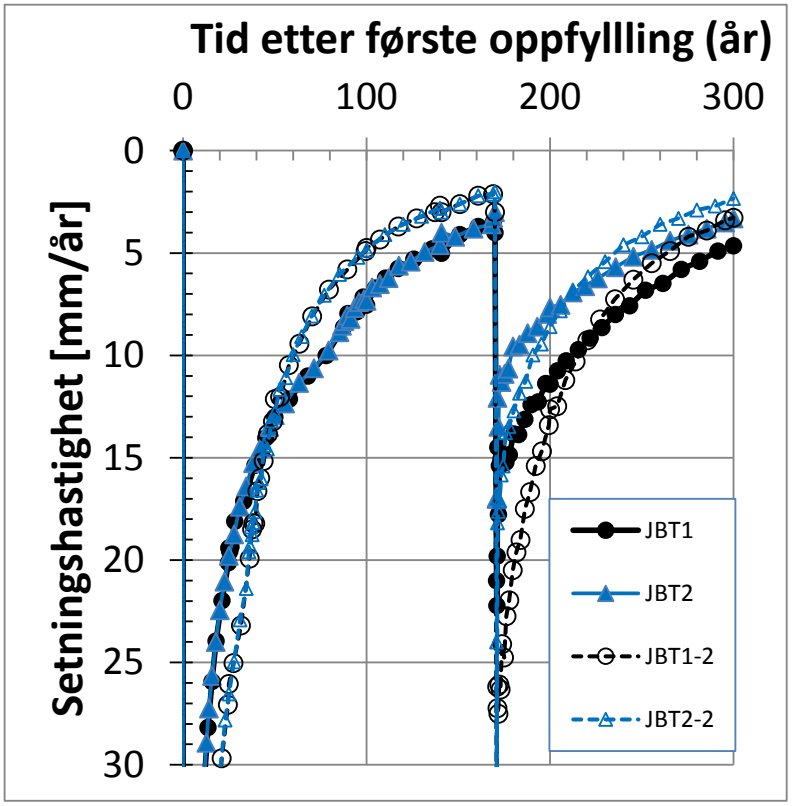
Settlement parameters used in calculations with Geosuite-KRYKON model

Elev.	**OCR**	**m**	**p'_r (kPa)**	**Perm. k_0 (m/s)**	**Perm. factor β_k**	**Creep param. r_{pc}**
0 til -20	1,3	13	$0{,}6p'_c$	$2 \cdot 10^{-9}$	4	290
>-20	1,3	16	$0{,}6p'_c$	$0{,}8 \cdot 10^{-9}$	4	356

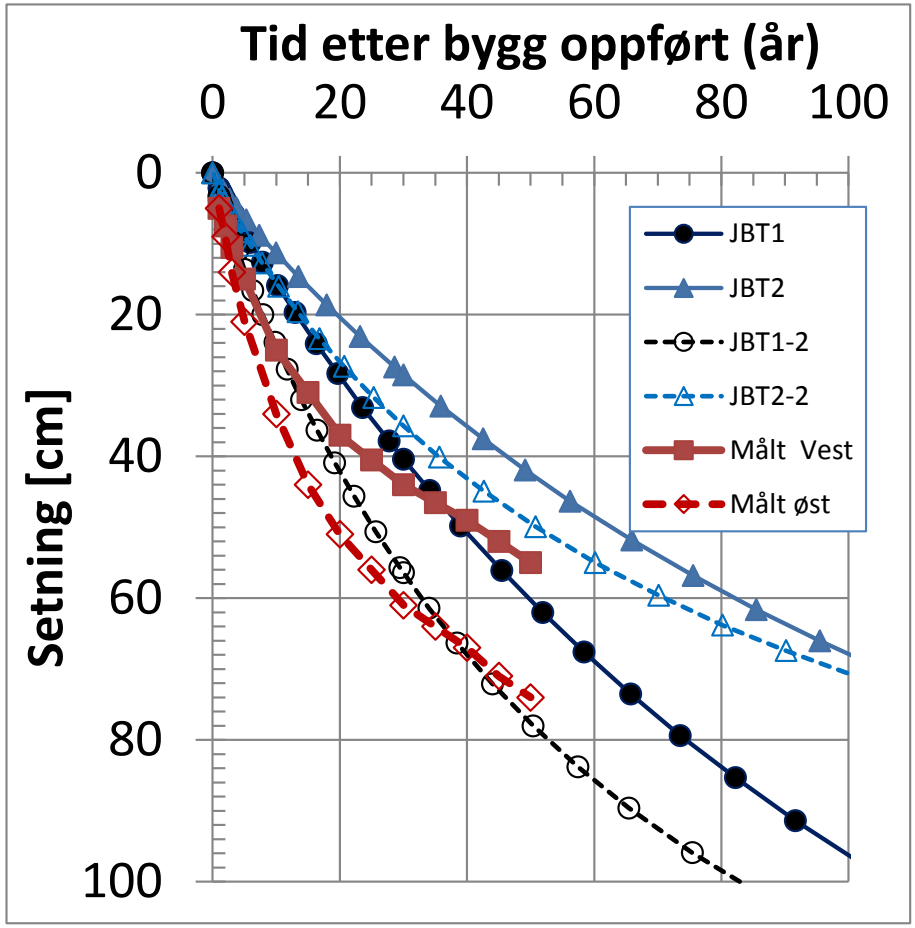
Calculated settelemnts- Oslo Jernbanetollsted

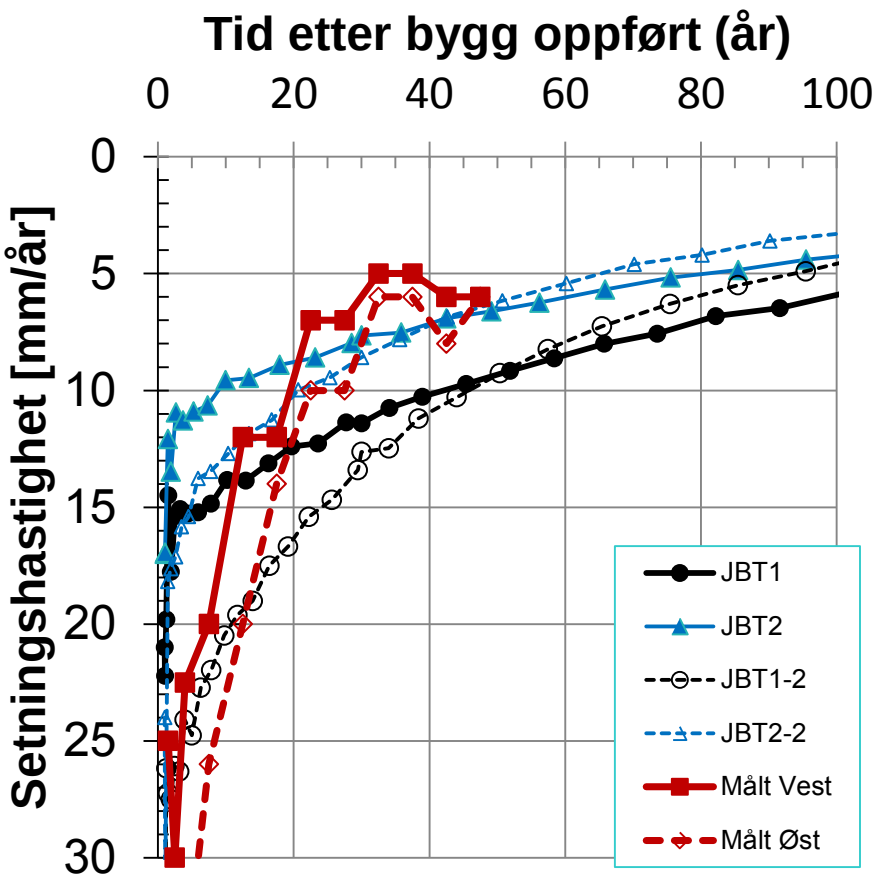
Fill: q= 45 kPa, Building: Δq = 50/90 kPa



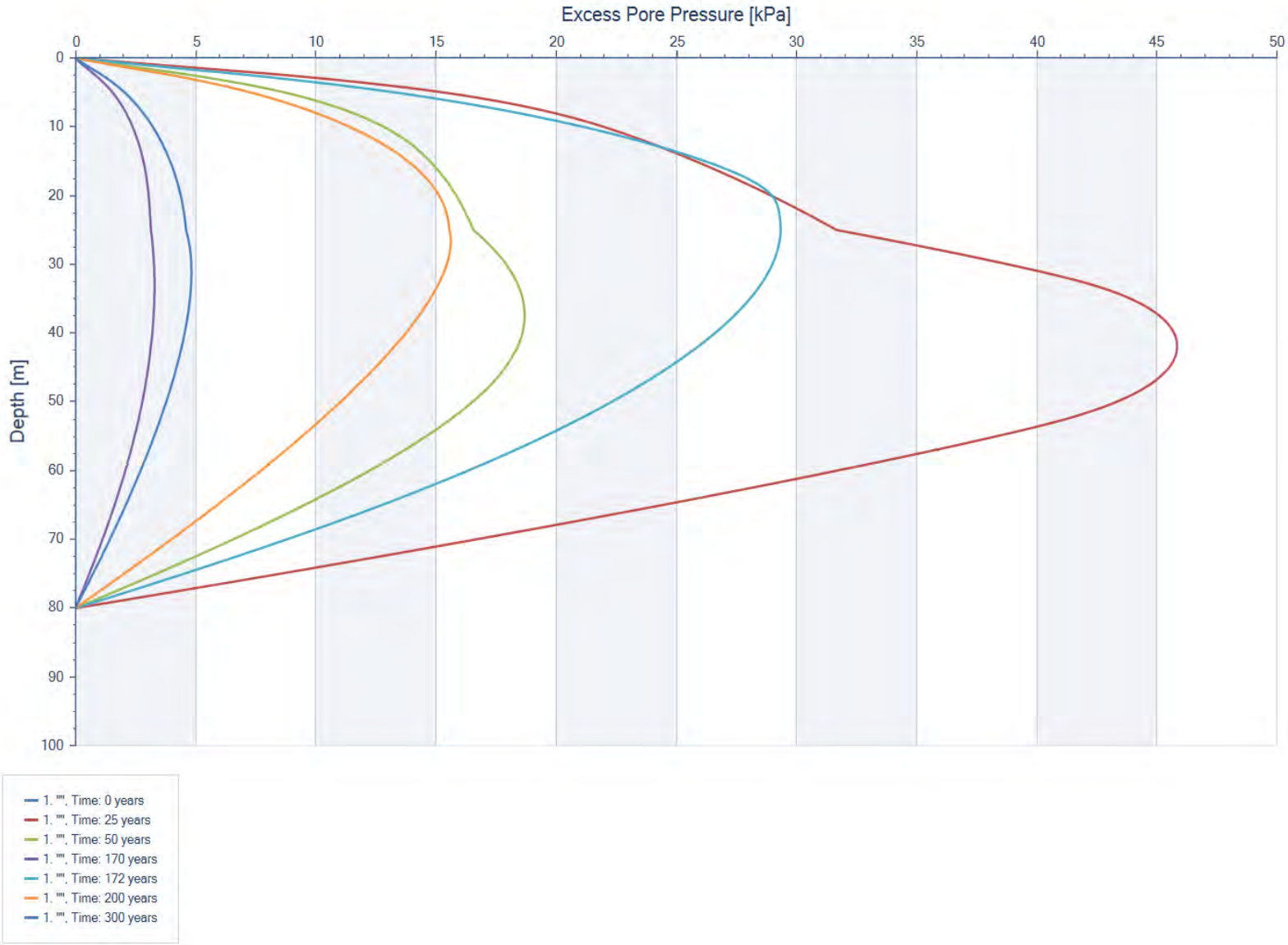


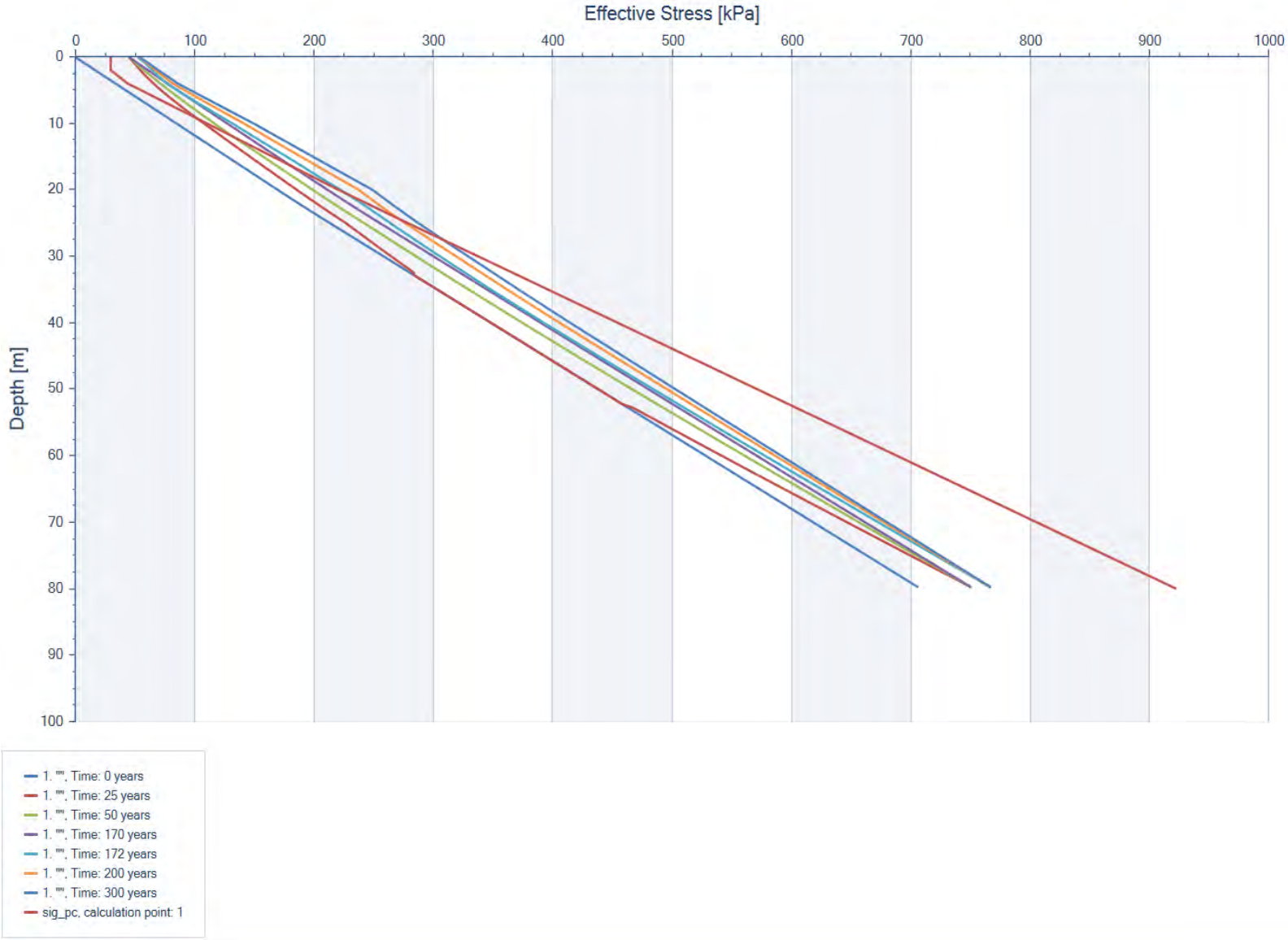
Predicted settlement rate after 170 yrs landfill (q= 45kPa) of 2,5-3,5 mm/yr agrees reasonably well with measured surface settlements in the area

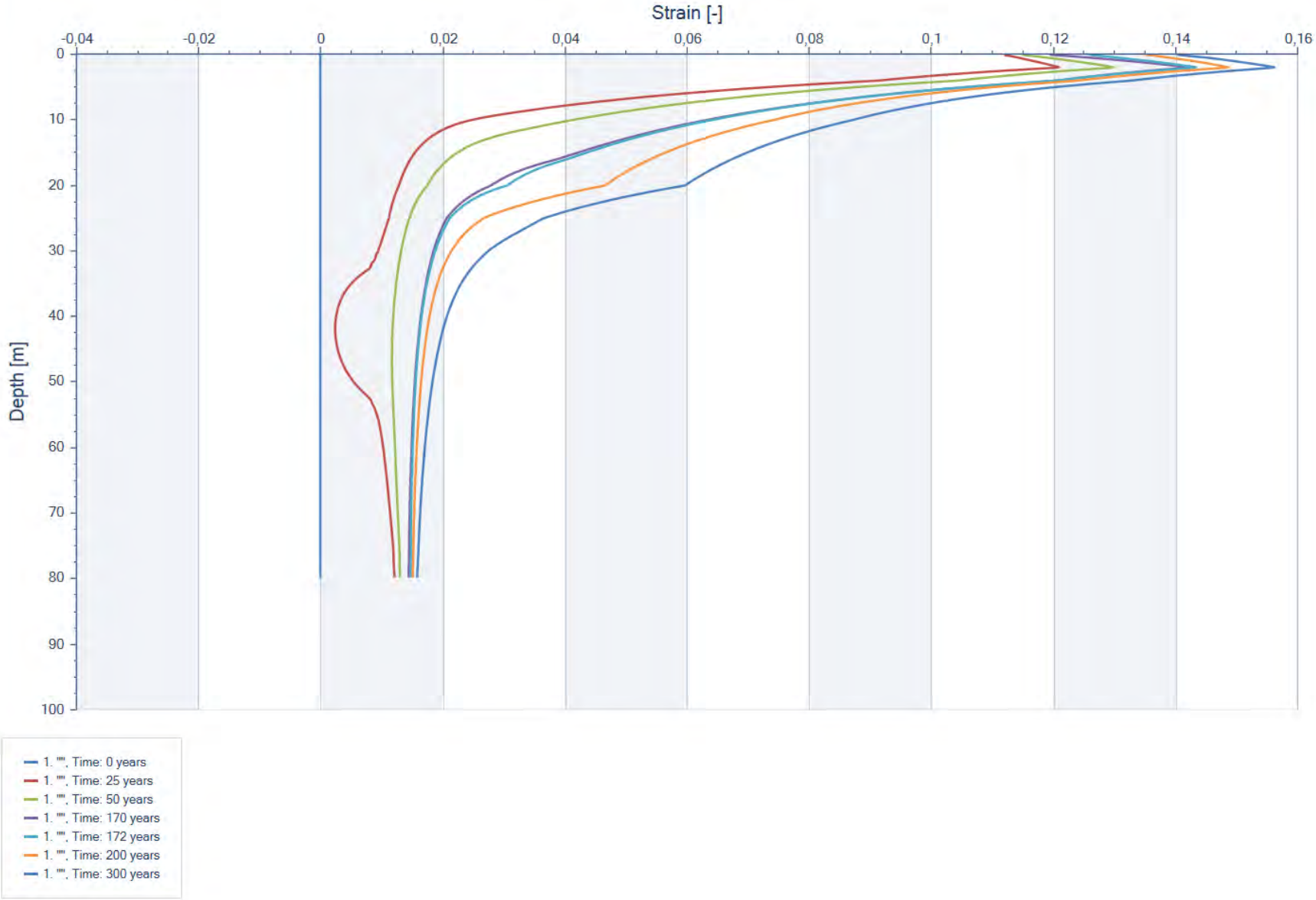




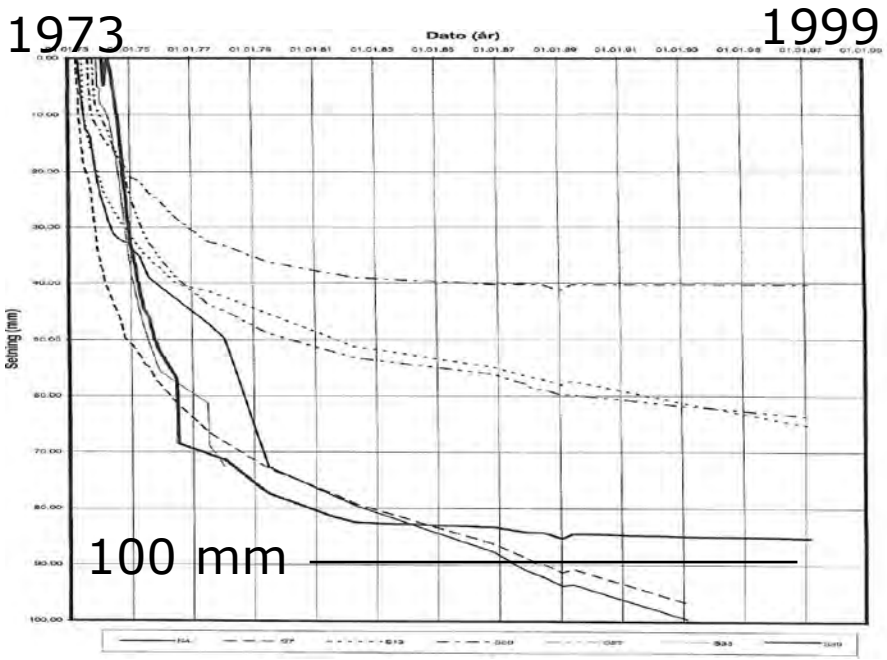
Measured settlement rates for the buliding are initially larger than predicted but decreases more rapidly and are smaller than predicted after 50 years

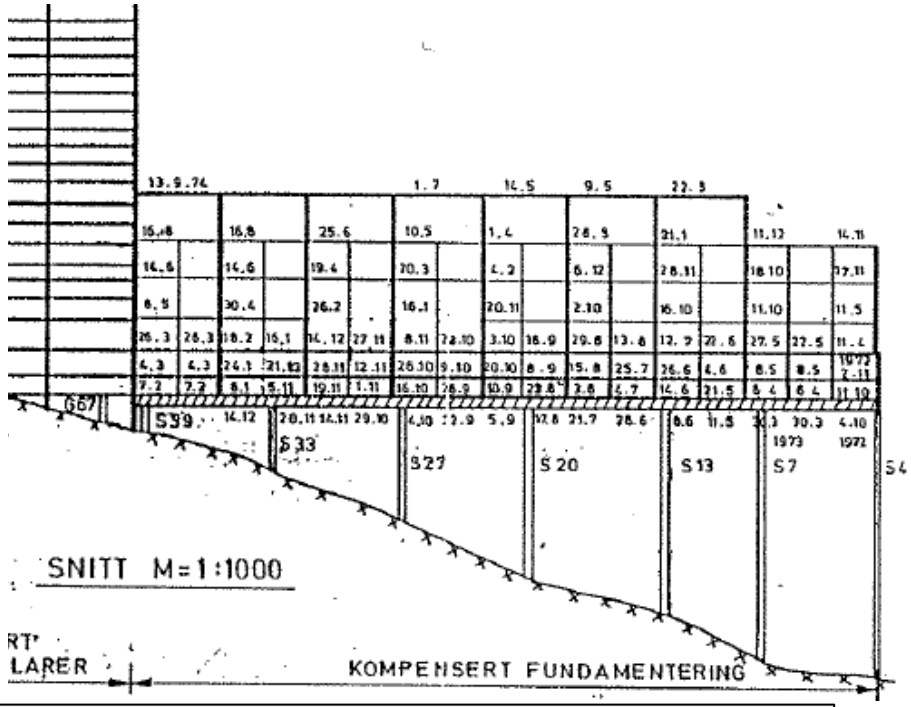

Excess Pore Pressure [kPa]
0
5
10
15
20
25
30
35
40
45
50
Depth [m]
0
10
20
30
40
50
60
70
80
90
100
1. "", Time: 0 years
1. "", Time: 25 years
1. "", Time: 50 years
1. "", Time: 170 years
1. "", Time: 172 years
1. "", Time: 200 years
1. "", Time: 300 years


Effective Stress [kPa]
0
100
200
300
400
500
600
700
800
900
1000
Depth [m]
0
10
20
30
40
50
60
70
80
90
100
1. "", Time: 0 years
1. "", Time: 25 years
1. "", Time: 50 years
1. "", Time: 170 years
1. "", Time: 172 years
1. "", Time: 200 years
1. "", Time: 300 years
sig_pc, calculation point: 1


Strain [-]
-0,04
-0,02
0
0,02
0,04
0,06
0,08
0,1
0,12
0,14
0,16
Depth [m]
0
10
20
30
40
50
60
70
80
90
100
1. "", Time: 0 years
1. "", Time: 25 years
1. "", Time: 50 years
1. "", Time: 170 years
1. "", Time: 172 years
1. "", Time: 200 years
1. "", Time: 300 years

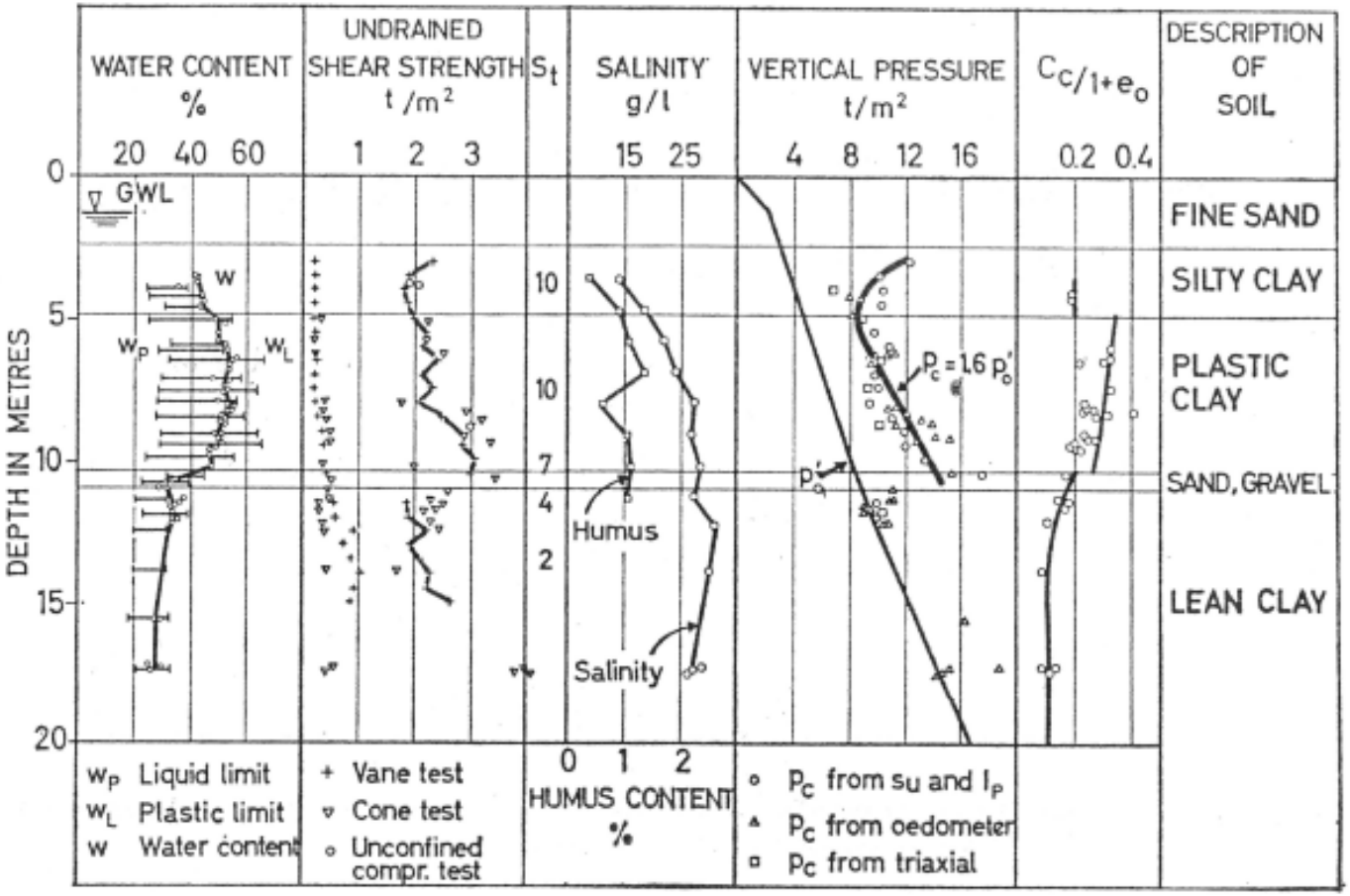
Settlement Oslo Post Terminal 1973-99



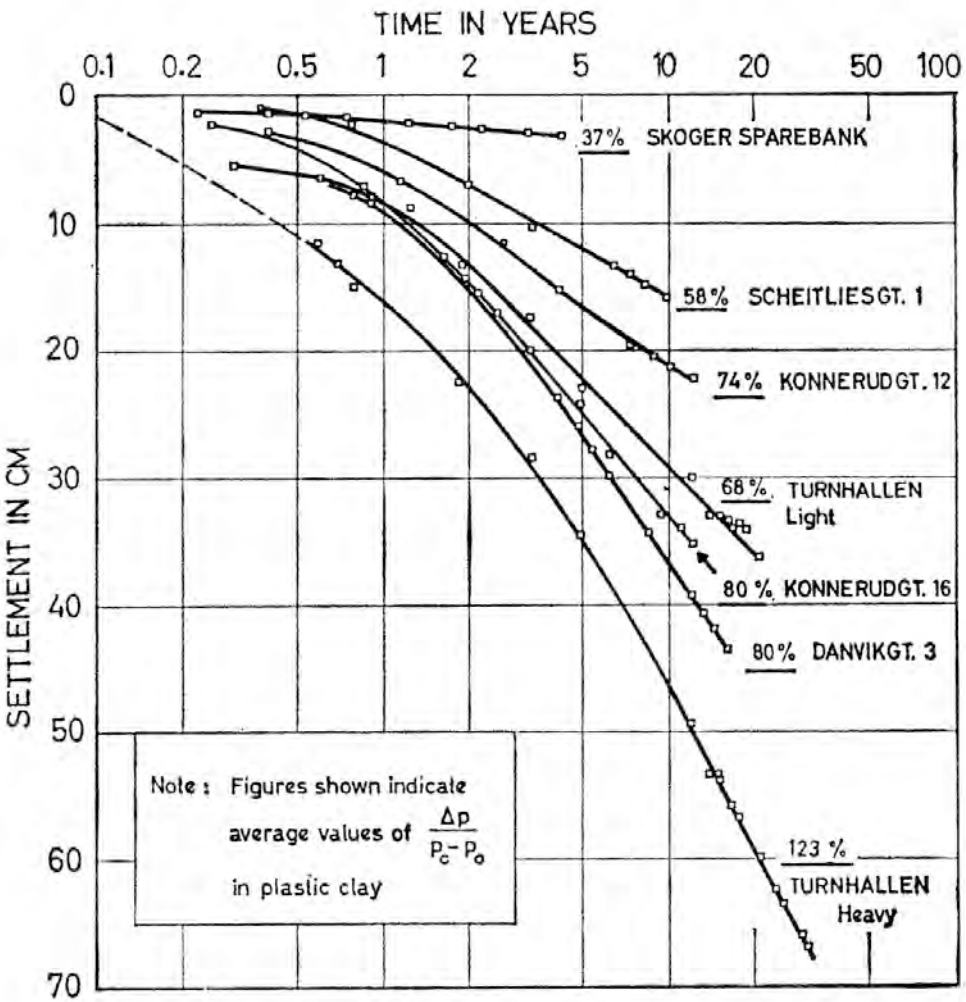


- Present settlement rate is up to 1,6 mm/år
- The building implies net unloading of 30-35 kPa, but the load is still about 10 kPa larger than before reclamation. Is this reasonable?

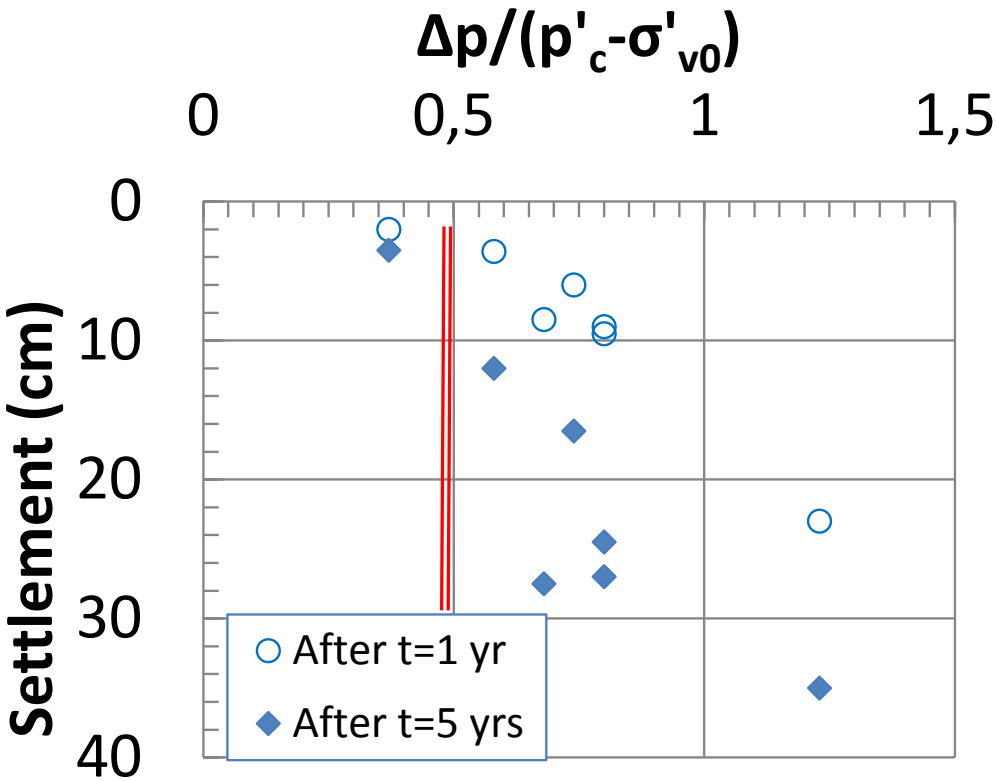
Revisiting some observations from settlements of buildings in Drammen (Bjerrum 1967)



Settlement og buildings in Drammen (After Bjerrum 1967)



Settlements in relation to normalized load for buildings in Drammen (Based Bjerrum, 1967)



- To achieve small settlements limit load to 50 % of $(p'_c-\sigma'_{v0})$
- With assumed OCR=1,6 this implies a mobilised stress level corresponding to OCR=1,3.

Way forward

- We need a numerical model that makes it easier for users to understand and set correct parameters anchored to conventional oedometer tests and empirical correlations
- We need more creep studies in lab on high quality samples, especially to better capture creep settlements for loads under and around p’c, effects of preloading and clay structure

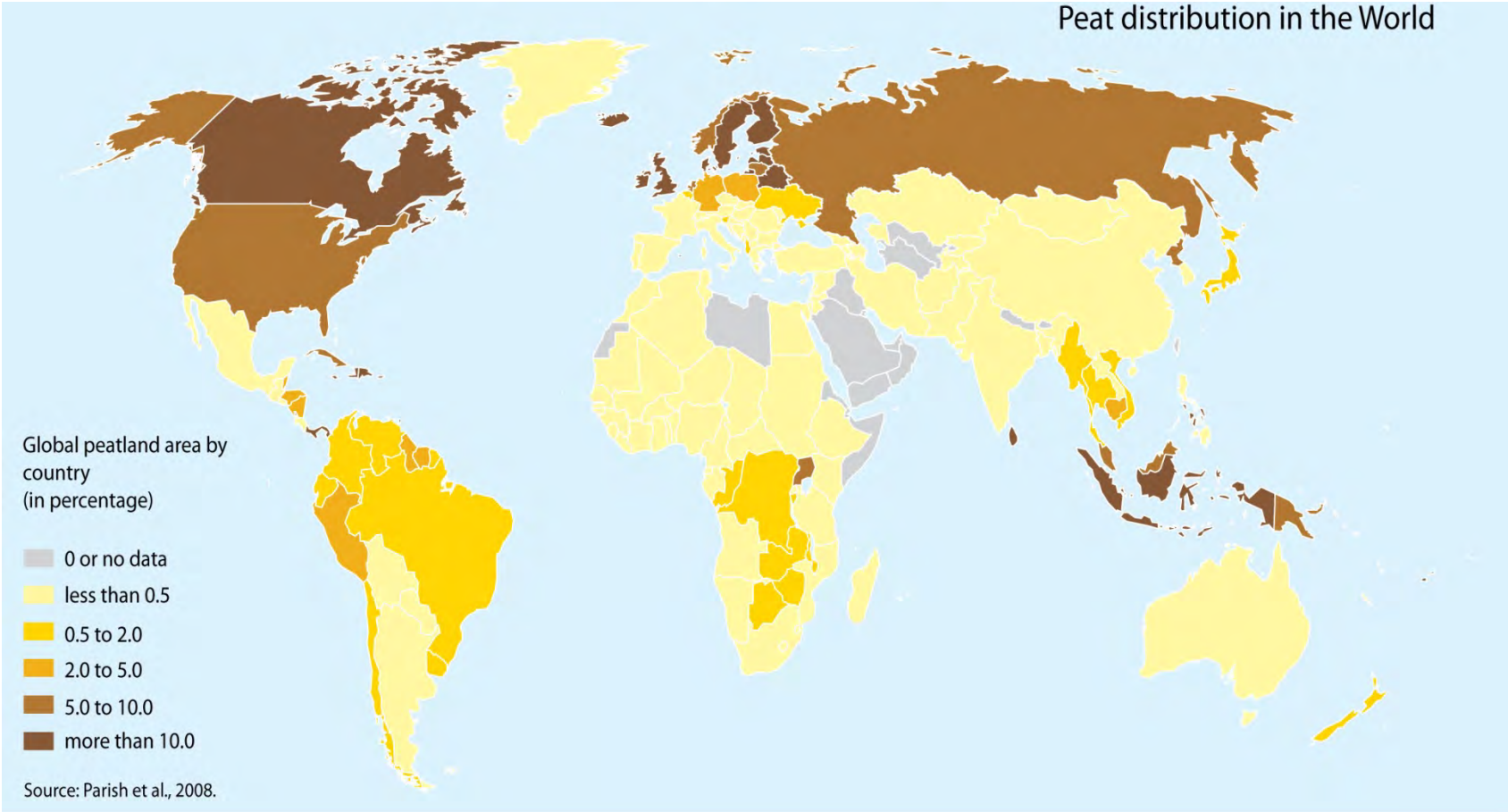
Deltares

Enabling Delta Life

Creep behaviour of peat

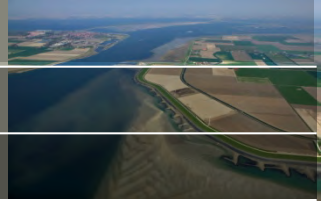
Cor Zwanenburg

13 januari 2015



(http://www.grida.no/graphicslib/detail/peat-distribution-in-the-world_8660)

Organic content



Organic content $P = M_{org} / M_s$

Ash content = 1-P

How to determine *P:*

- Determine dry solid mass (M_s), by drying sample for 24 h at 105˚C
- Determine remaining mass (m_1) after drying for 4 h at 550 ˚C
- Loss on ignition $N = (M_s - m_1)/M_s$.
- Ash content = 1.04(1 - N)
- $P = 1 - 1.04(1 - N)$

The correction of 4% follows from loss of fixed water from clay minerals.

(Skempton & Petley, 1970; Hobbs, 1986)

Definition

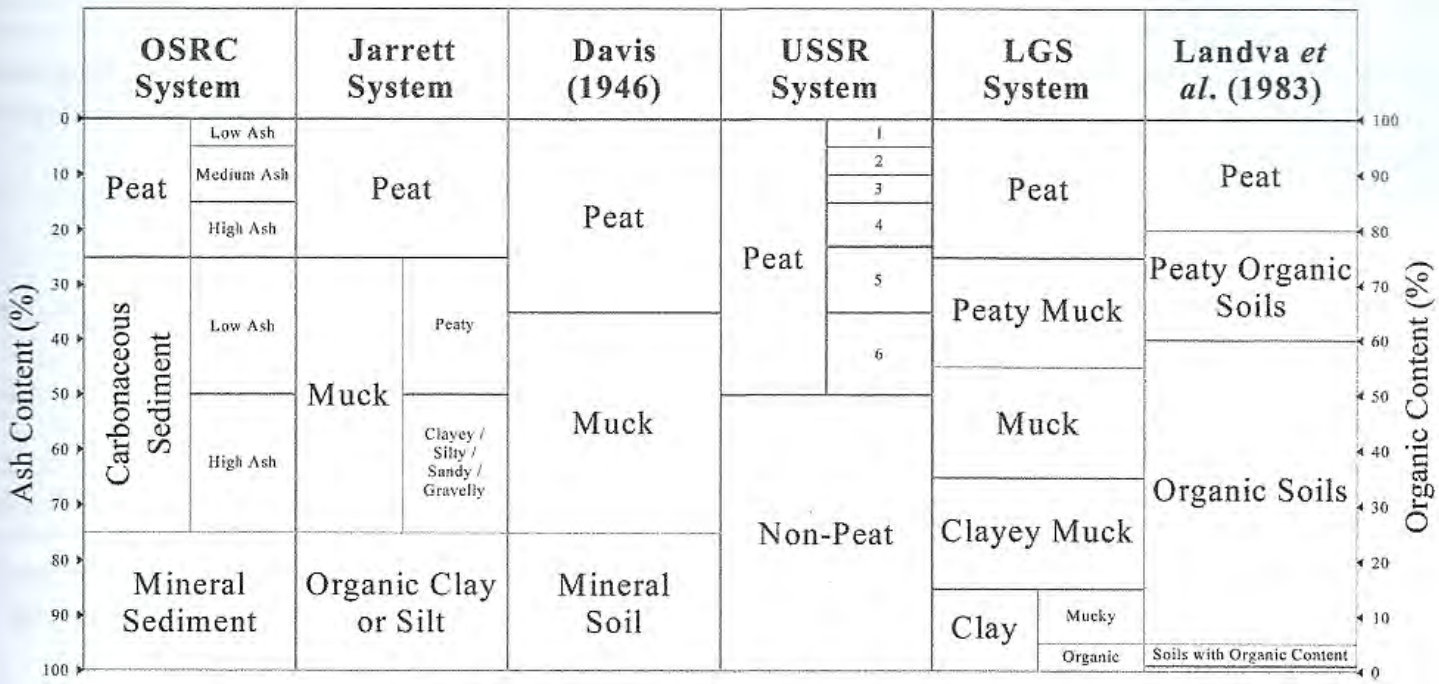
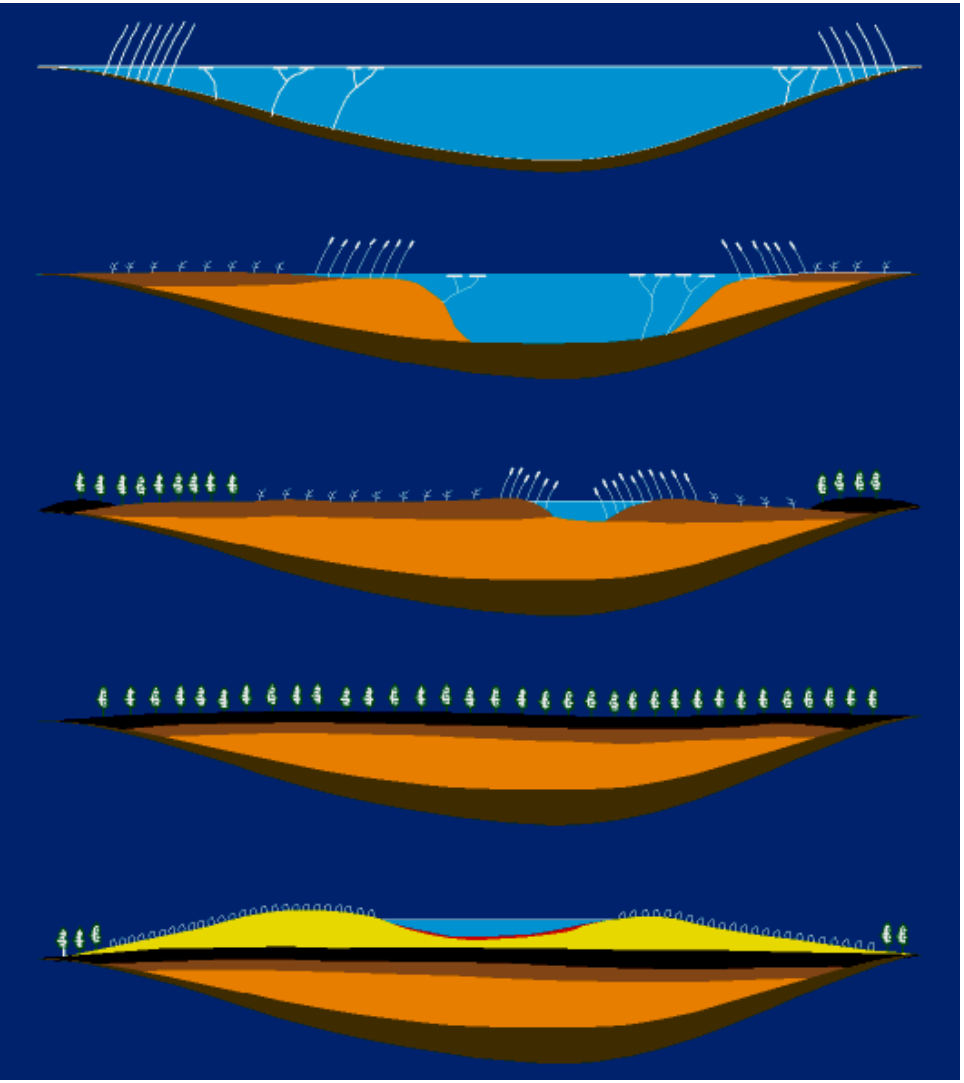


Figure 2.5 Comparison of classification systems used for peat and organic soils (*after* Andrejko *et al.*, 1983). (B.B.K. Huat et al 2014)

Conditions during deposition



Lake

fen

Raised bog

(after Visscher 1949, Lowe & Walker 1997)

Deltares

Example, eutrophic lake

Examples of Marshes and Swamps

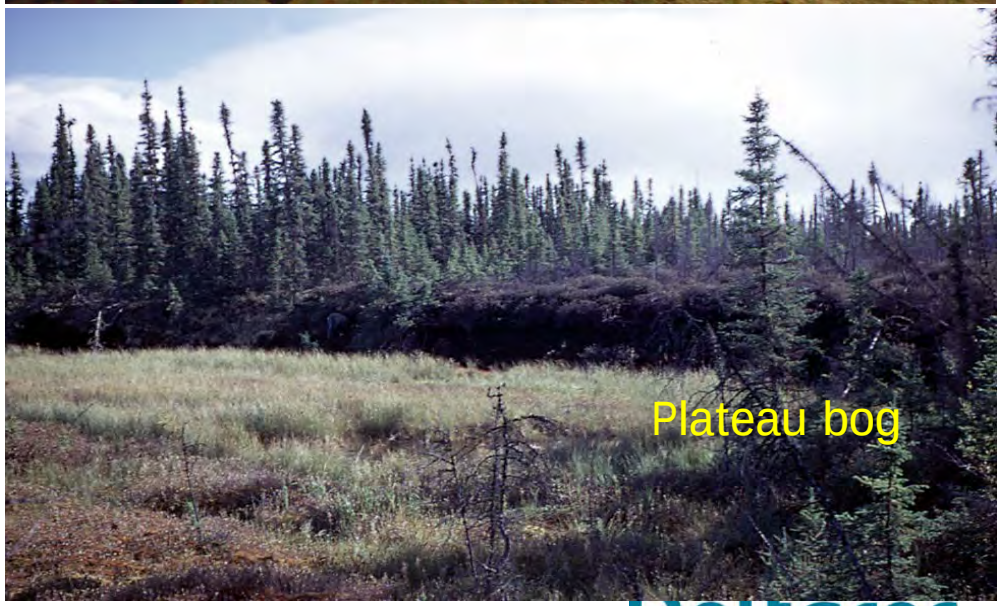
Marsh (= low wetland area, covered by reed, grass etc.)
Swamp (= low wetland area, covered by forest)

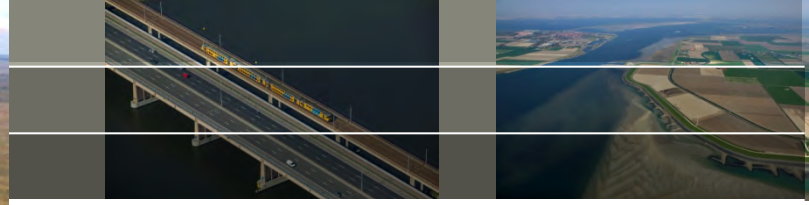
http://sts.gsc.nrcan.gc.ca/

Deltares

Fen (= low area covered by grass and reed)

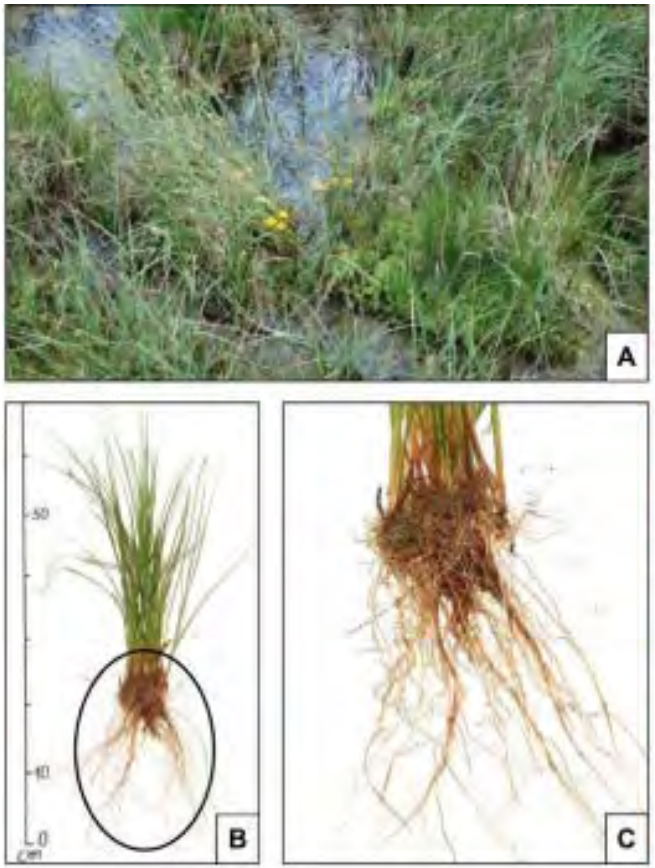
Examples of raised bogs





Raised bog in Siberia

Botanical background, *Carex* (sedge)



(Meier-Uhlherr et al, 2011)

Photo G. Erkens

Sphagnum *(sphagnum Palustre)*

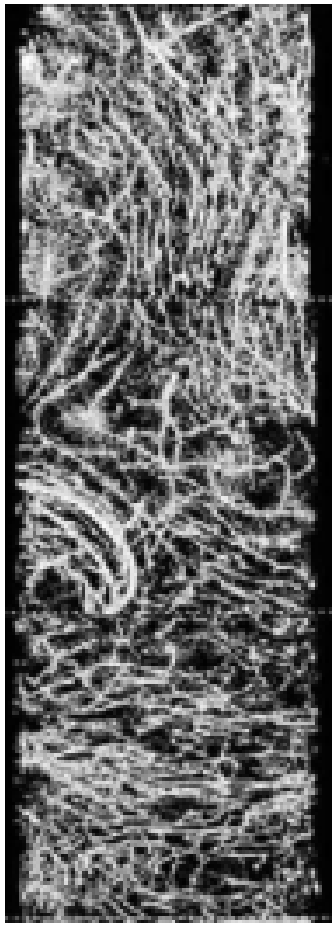
Photo G. Erkens 2012

(Meier-Uhlherr et al, 2011)

Structure

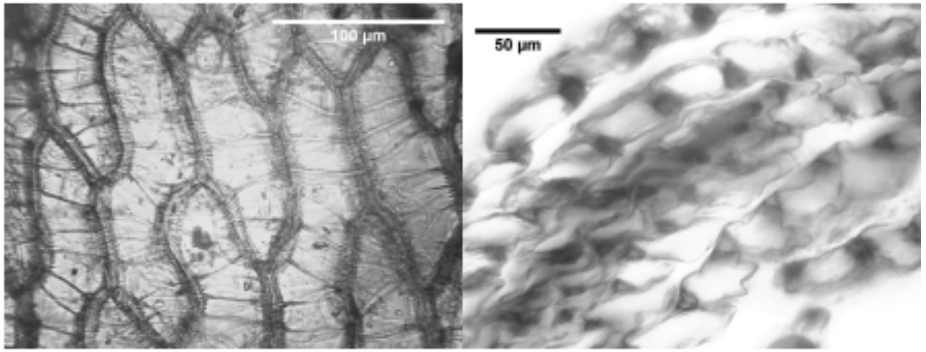
Structure consist of large fibres, depending on botanical background and humification and fill which consists of clay particles small fibres, humified peat etc.

Pore water can be divided in free water in large pores, capillary water in smaller pores and bounded water.
(Hobbs 1986 and others)



Eriophorum (Cotton-grass)

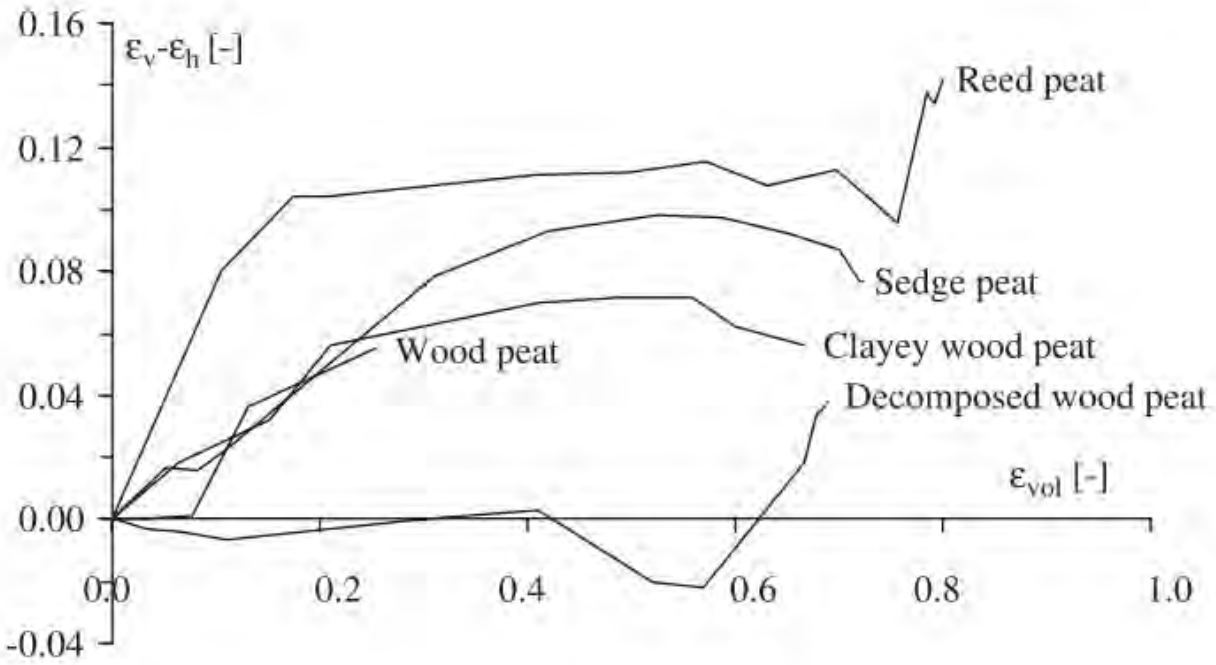
Microscopic photo of *Sphagnum austinii* (Sphagnum)
photo G. van Wierdum



More photo's (a.o.):
Landva, 2007
Mesri & Aljouni, 2007
Hendry et al, 2012

X-ray CT images of sphagnum peat
(Kettridge & Binley 2011, See also: https://www.youtube.com/watch?v=NreGpZLhSrI)

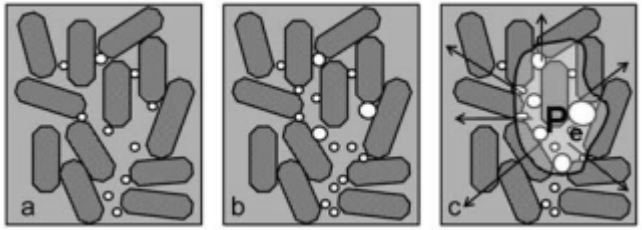
Structure revealed by shrinkage



(Den Haan & Kruse, 2007)

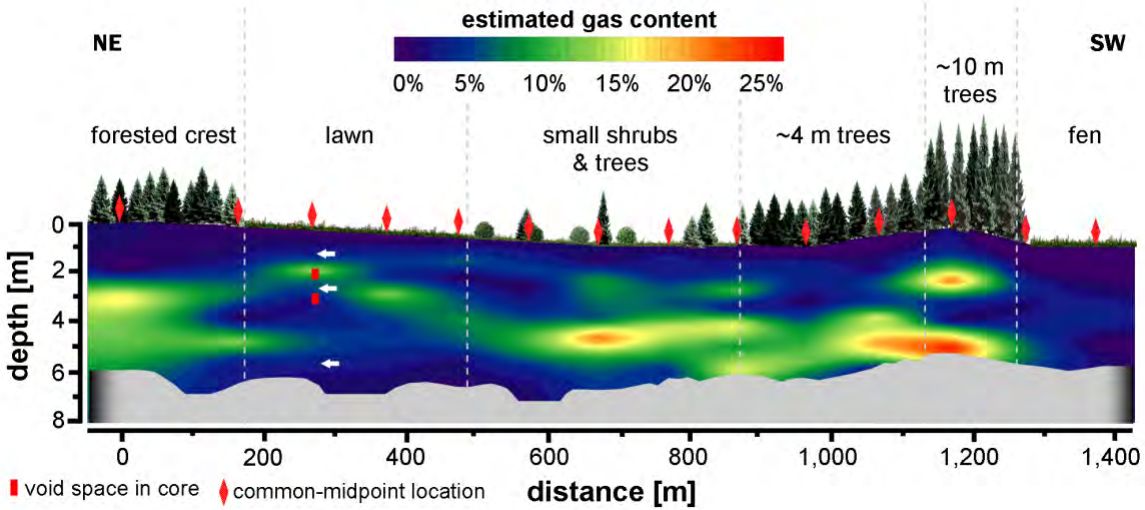
Deltares

Presence of gas



Kellner & Waddington (2005)

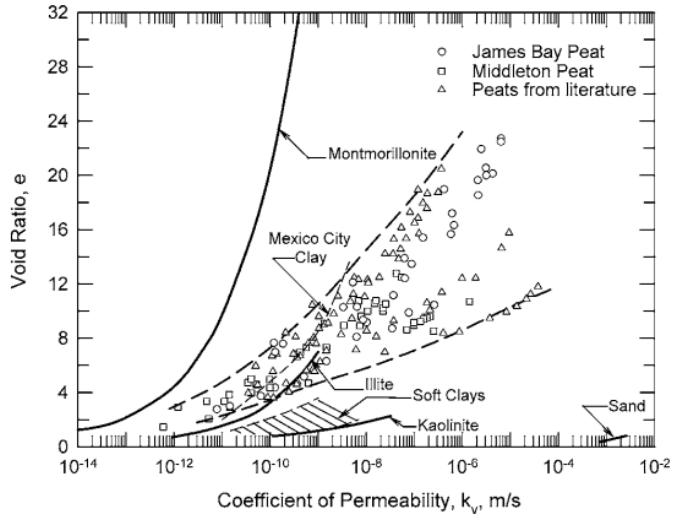
5 mm



(Persekian, 2011)

Deltares

Permeability



(Mesri & Aljouni, 2007)

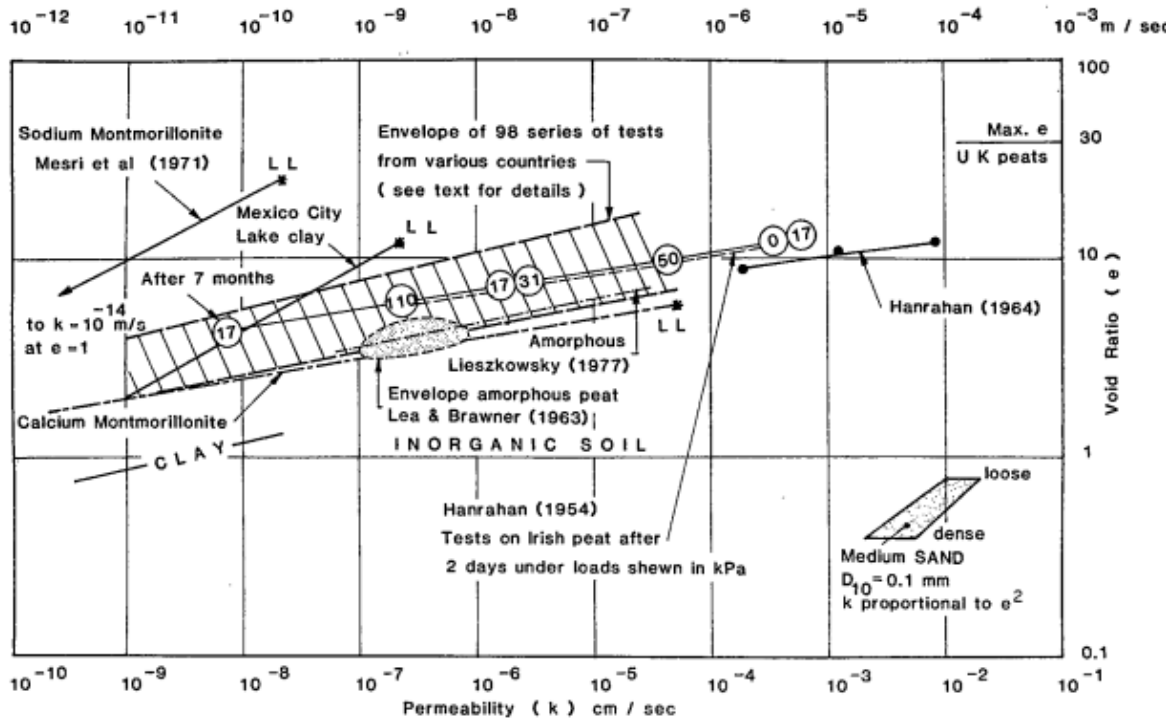
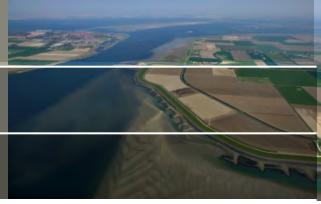


FIG. 19. Vertical permeability during pauses in consolidation tests on undisturbed peat.

(Hobbs, 1986)

Due to large pores and gas bubbles the permeability determined in the field might differ one or two orders in magnitude compared to lab test data. Field values can be larger or smaller than lab values

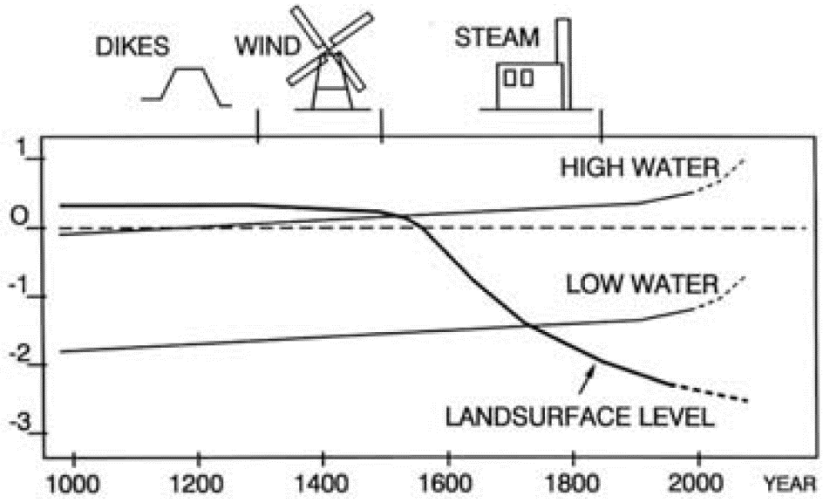
Deltares

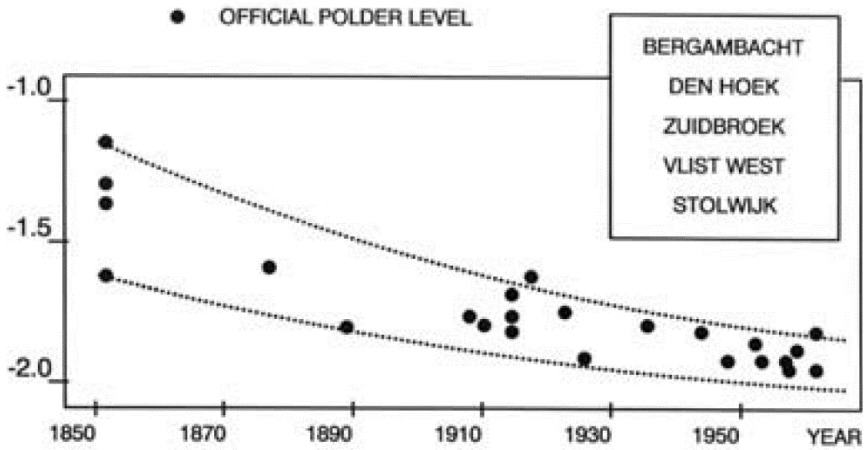
CREEP IN PRACTICE

Deltares

Long term settlement



POLDER LAND SUBSIDENCE IN HOLLAND

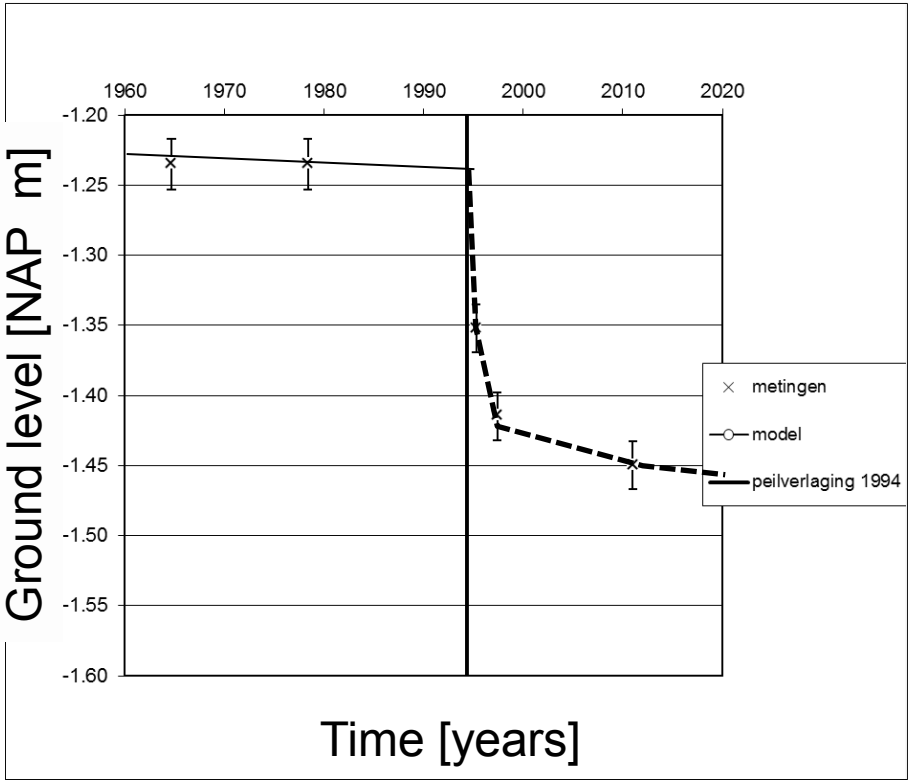


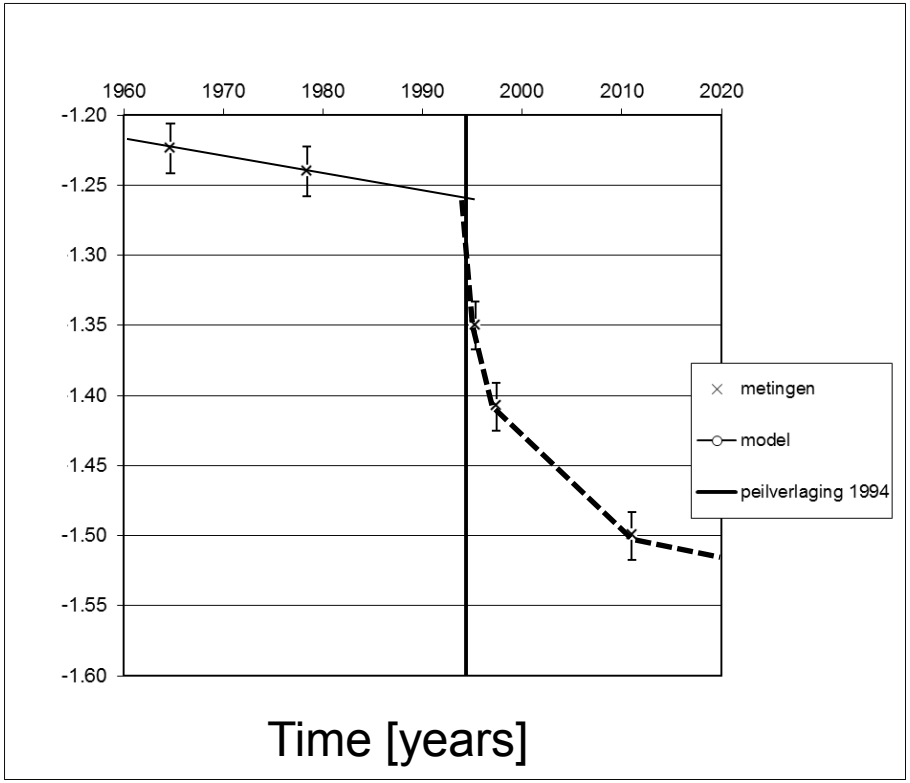
POLDER WATER LEVEL IN HOLLAND

(vertical scales give elevation in m relative to Dutch datum N.A.P.)

Den Haan & Kruse 2007

Consequences of adjusting water table;





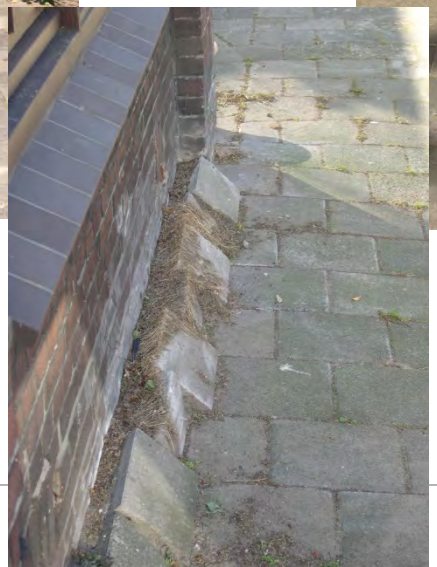
	Year	Level adjustment
1	1976	NAP-1.43m - NAP-1.49m
2	20 May 1994	NAP-1.49m - NAP-1.96m

Residual settlement

Deltares

Skin friction on piled foundations

Deltares

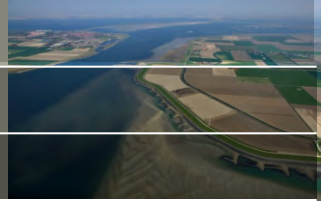


Deltares

differential settlement

Goudse Houtsingel, near Gouda

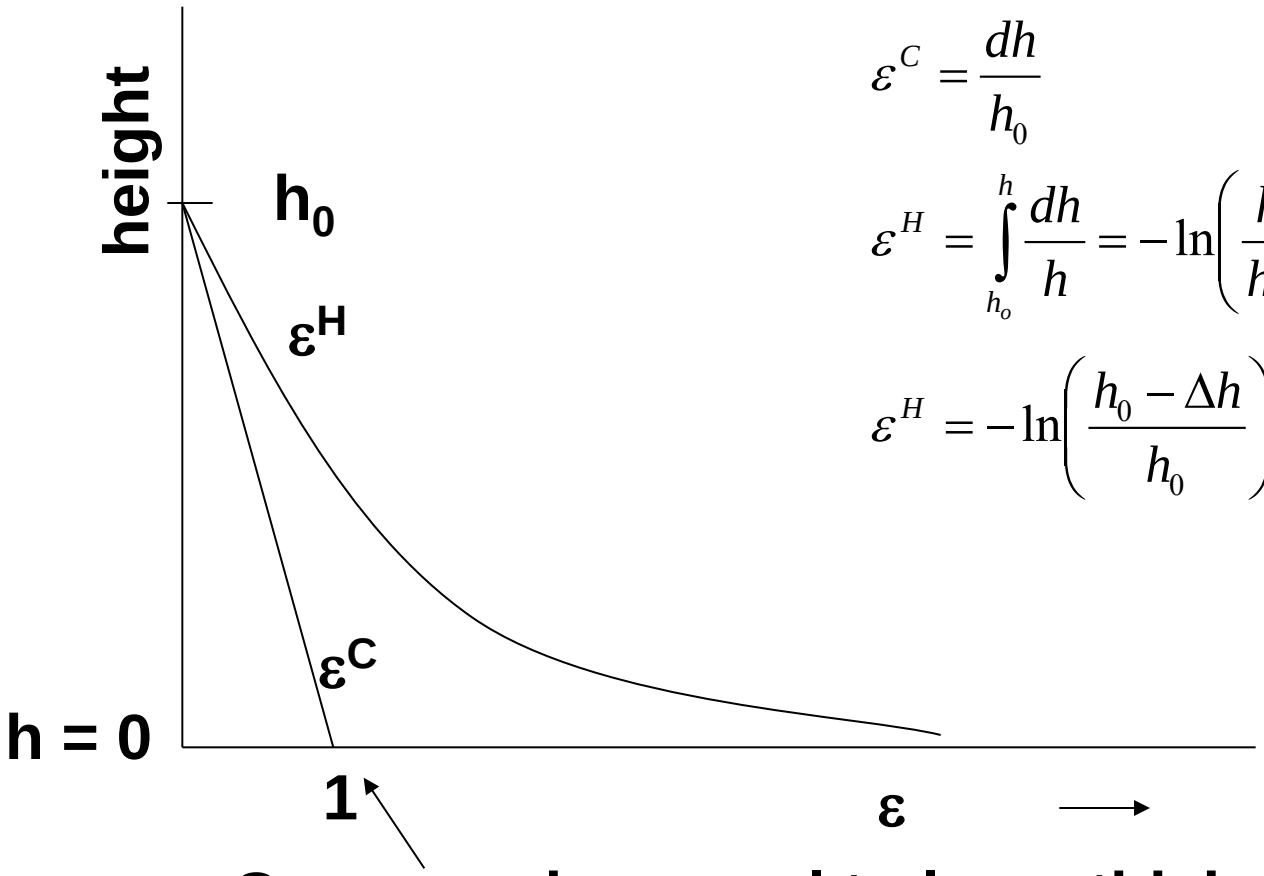
Deltares

1D SETTLEMENT CALCULATION

Deltares

Linear strain - Natural strain



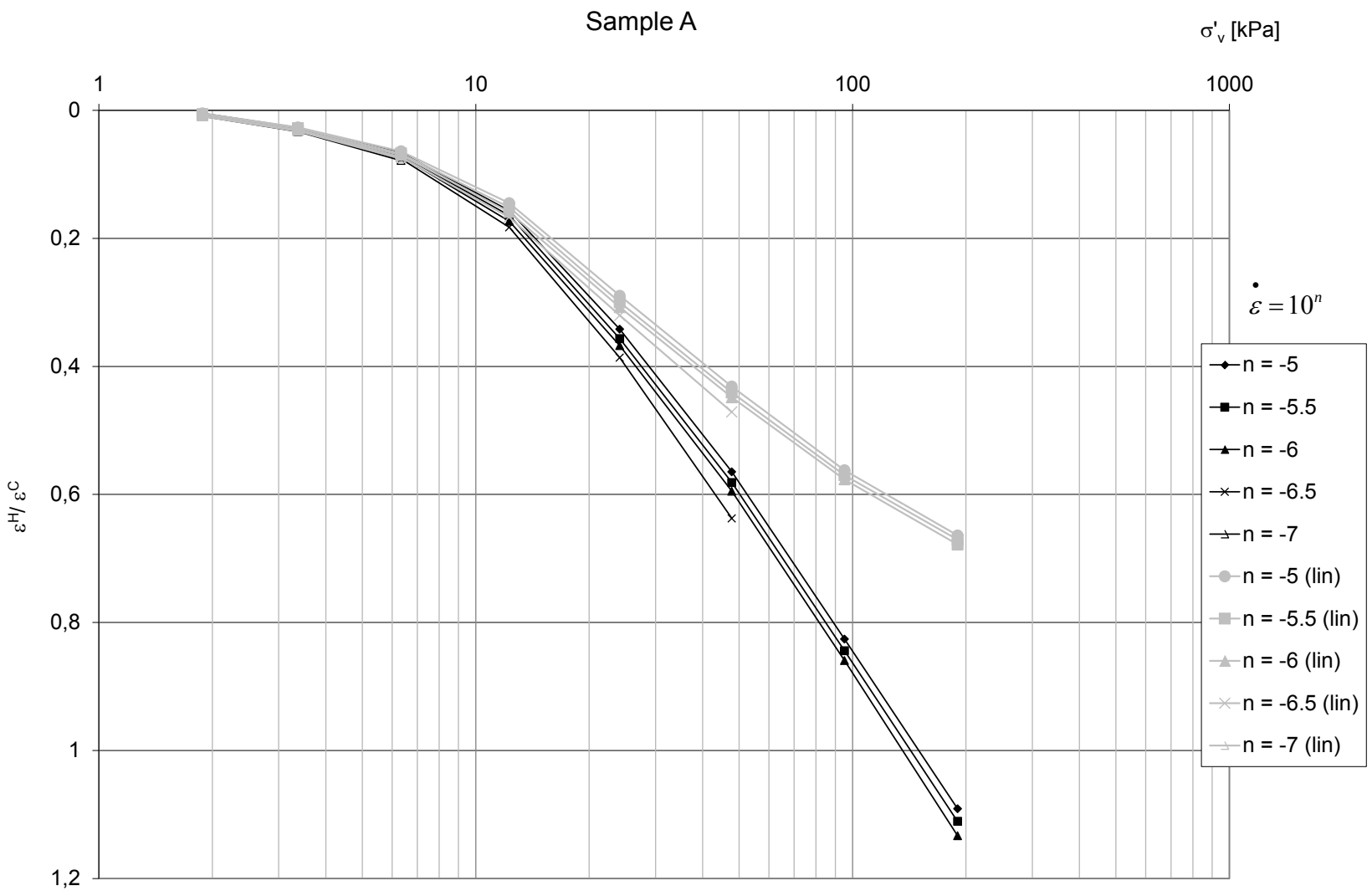
$$\varepsilon^C = \frac{dh}{h_0}$$

$$\varepsilon^H = \int_{h_o}^{h} \frac{dh}{h} = -\ln\left(\frac{h}{h_0}\right)$$

$$\varepsilon^H = -\ln\left(\frac{h_0 - \Delta h}{h_0}\right) = -\ln\left(1 - \varepsilon^C\right)$$

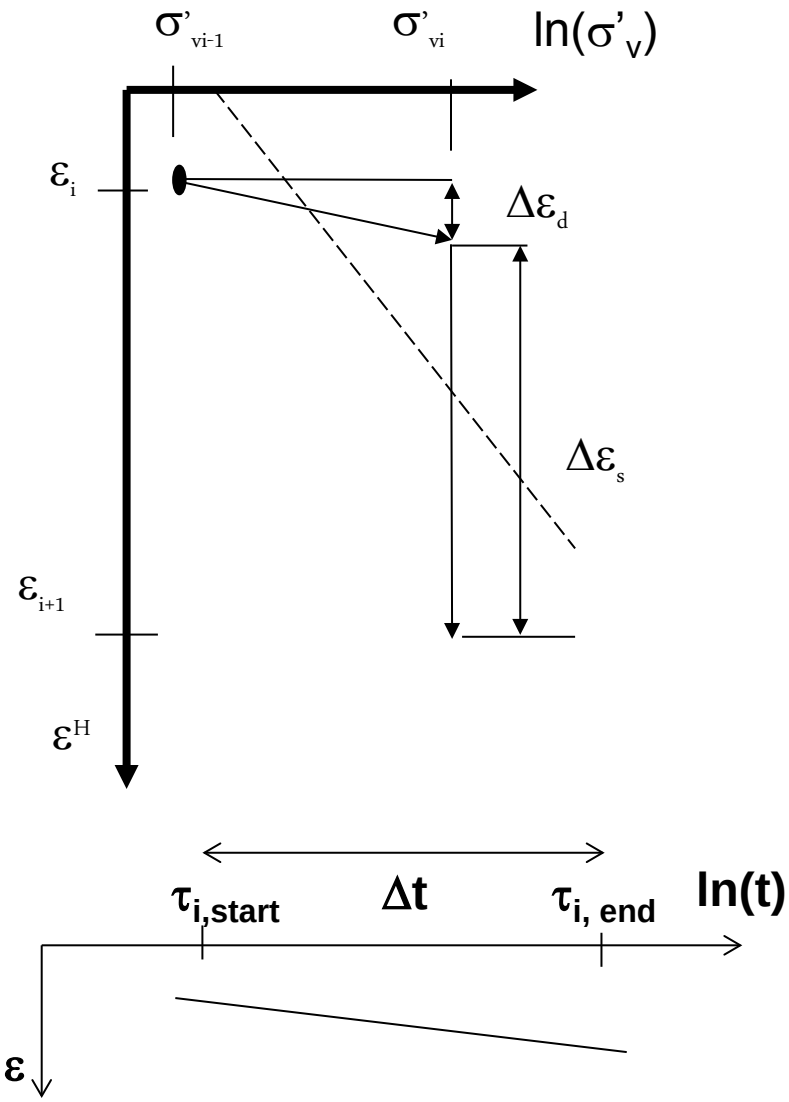
Compression equal to layer thickness

Oedometer test results in linear and natural strain



Deltares

Isotach model based on natural strain, Den Haan (1994) in incremental form



1. Direct strain:

$$\varepsilon_d^H = a \ln\left(\frac{\sigma_i}{\sigma_{i-1}}\right)$$

2. Equivalent time τ directly after loading

$$\tau_i = \tau_{i-1}\left(\frac{\sigma'_{i-1}}{\sigma'_i}\right)^{\frac{b-a}{c}}$$

3. Equivalent time τ end load step

$$\tau_{i,end} = \tau_i + \Delta t$$

4. Viscous strain

$$\varepsilon_s^H = c \ln\left(\frac{\tau_{i,eind}}{\tau_{i,begin}}\right)$$

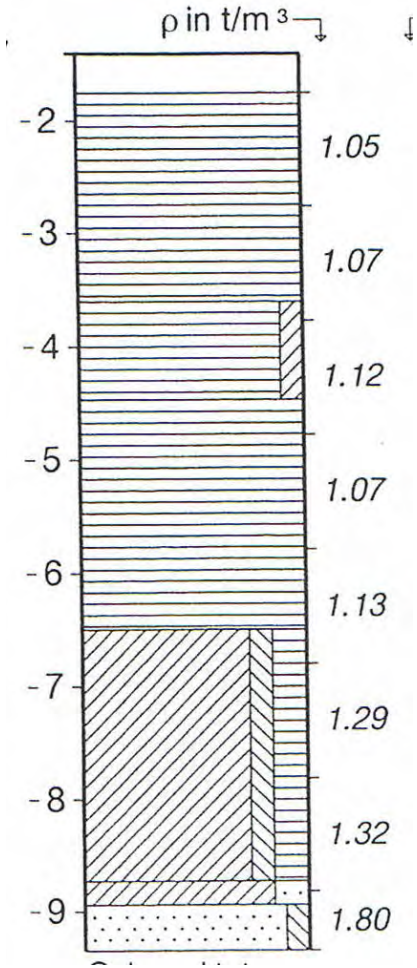
5. Total strain and settlement

$$\varepsilon^H = \varepsilon_d^H + \varepsilon_s^H, \quad \Delta h = h_0\left(1 - \exp(-\varepsilon^H)\right)$$

Deltares

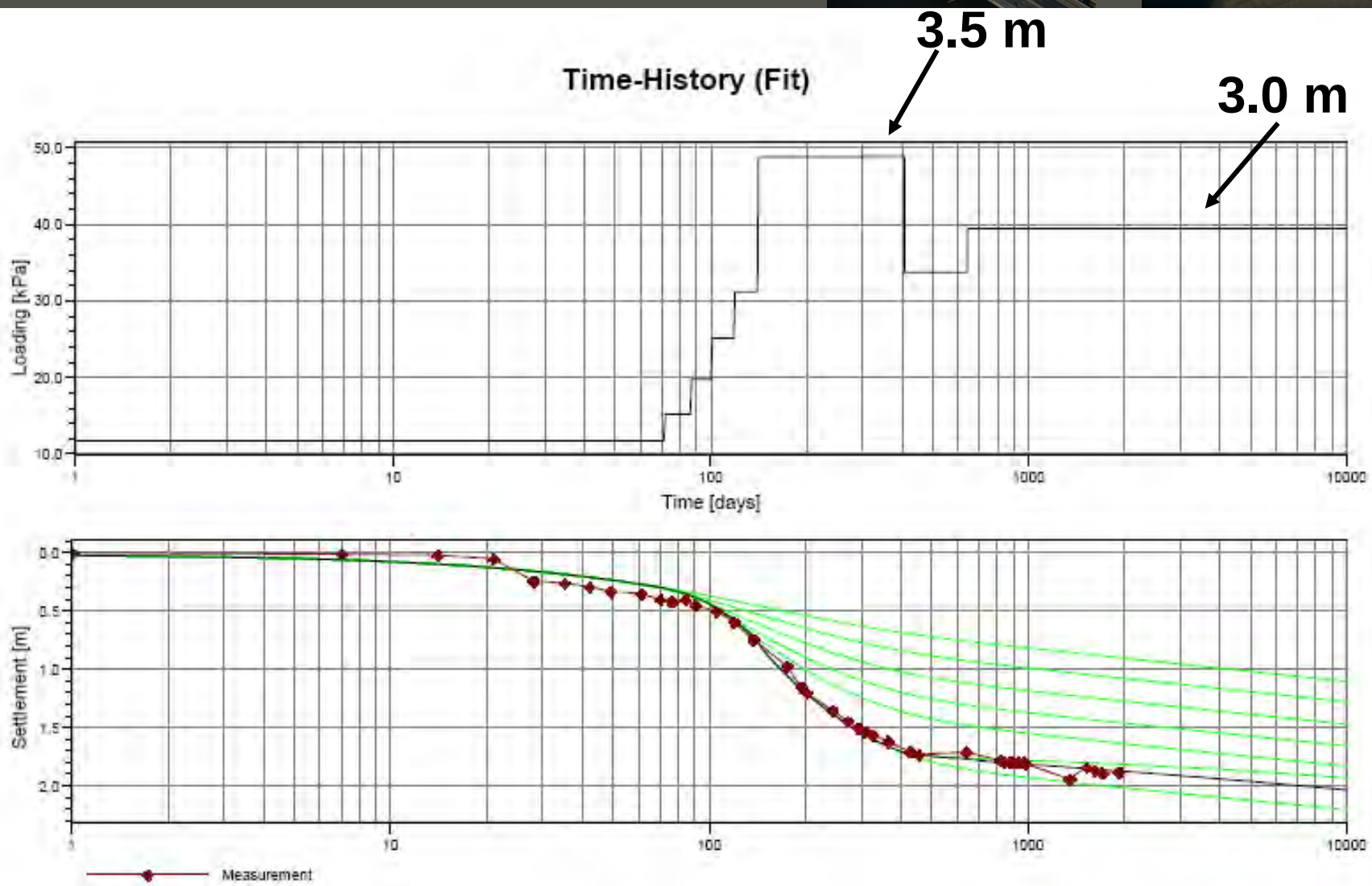
Case: Railway line Rotterdam – Germany

Soil profile



Deltares

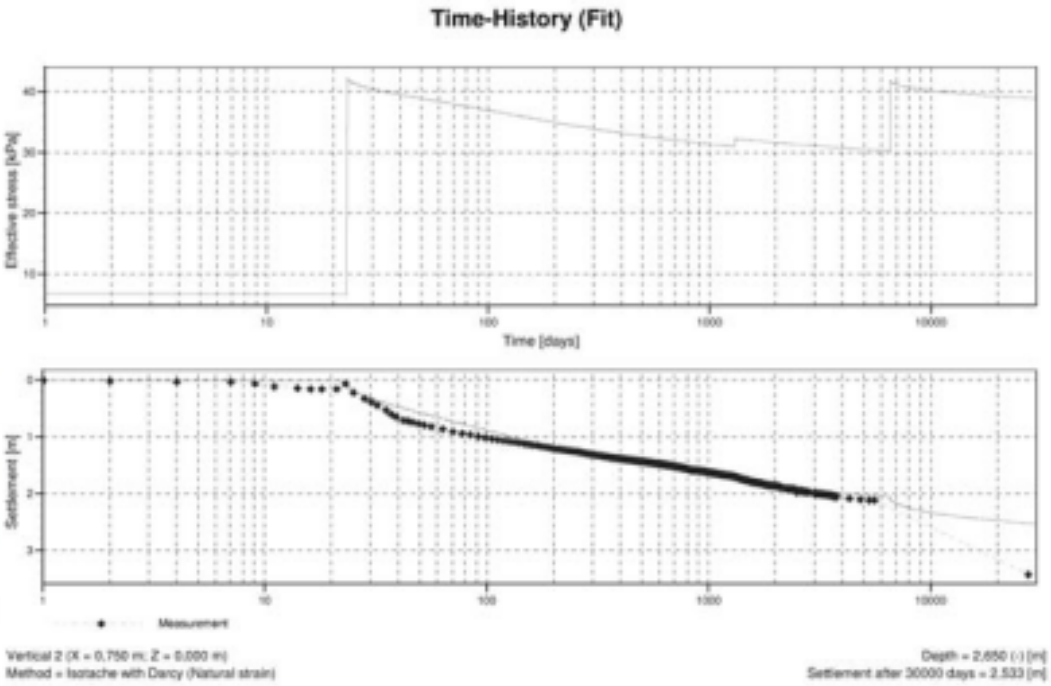
Calculation vs measurements



Long term settlement

Foto 4 *Aanvullen rijzenbed met zand, Stolwijk 1950.*

Figuur 4 – *Aanprikken van het 75 jaar oude rijzen bed ter plaatse van het proefvak Stolwijk In 2009. Deze weg is ten opzichte van de oude weg iets naar links opgeschoven en ligt nu gedeeltelijk op de vroegere trambaan.* FOTO DELTARES

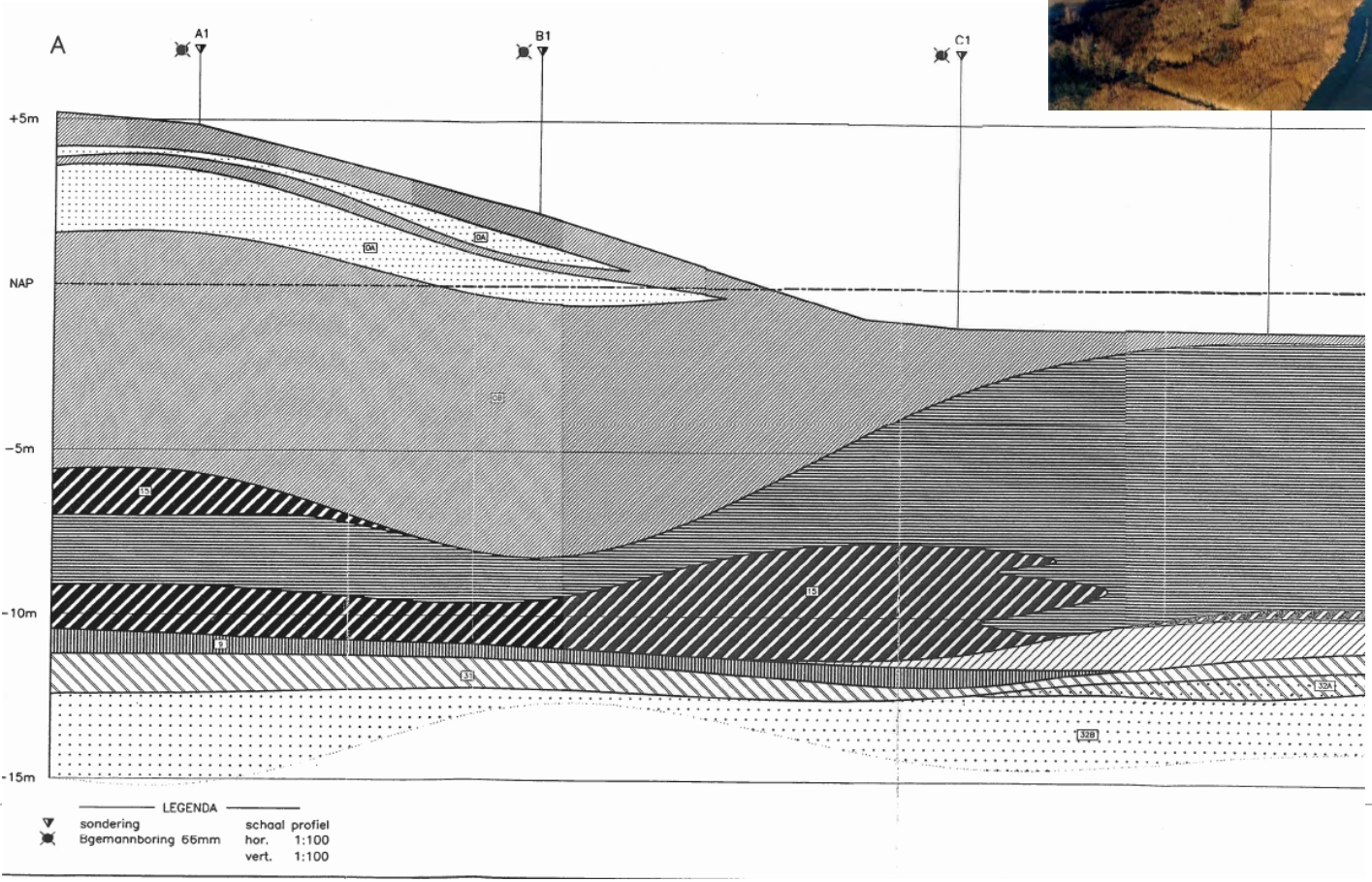


Heemstra 2008 en 2012

Deltares

Long term settlement

Dike with medieval origin



Deltares

Elastoplastic behavior of frozen soil

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Mehdi kadivar,

Gustav Grimstad

NTNU

1910

Overview

- Major factors influencing frozen soils behavior
- Existing models
- Framework of proposed model
- Results
- Conclusion

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Major Factors

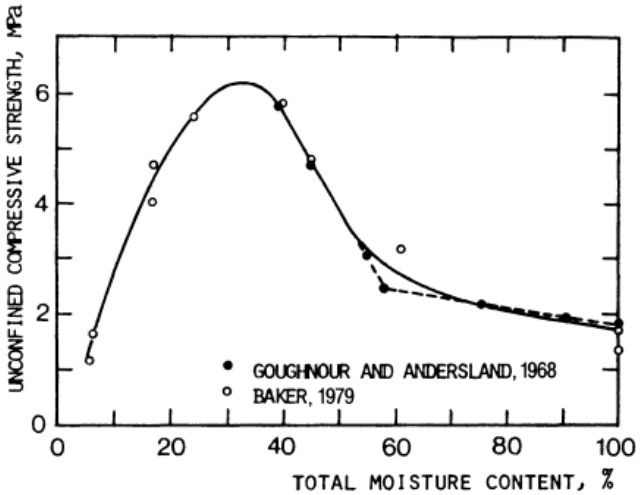
➢ Ice content:

1) Poor ice soils:
   - ✓ Binding effects on grains
   - ✓ Ice cementation

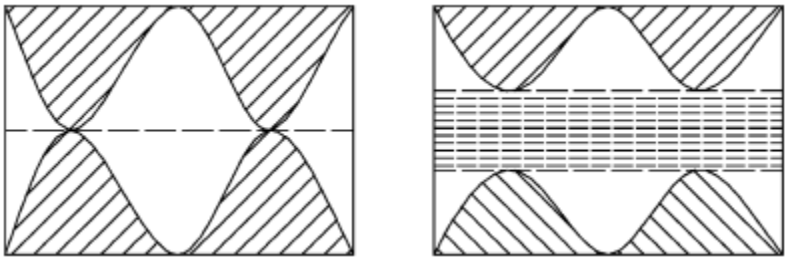
→ **An increase in ice content results in an increase in strength**

2) Ice rich soils:
   - ✓ Decreases grains contact

→ **An increase in ice content results in a decrease in strength**



Effect of total moisture on strength of frozen soil (adopted from Baker, 1979)



Schematic of ice increasing in an ice rich soil body (Li et al., 2002)

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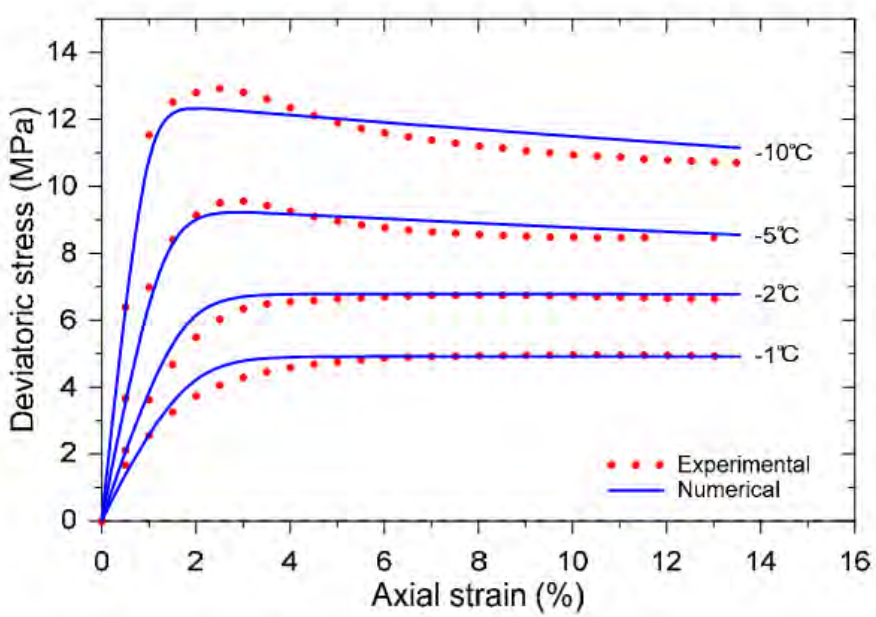
Major Factors

- Temperature:

  - ✓ Decreasing temperature results in:
    a. An increase in E modulus
    b. An increase in strength
    c. A decrease in strain at yield

In other word: Change of behavior from plastic type to a brittle type



Stress-strain curves at different temperature (Xu 2014)

NTNU

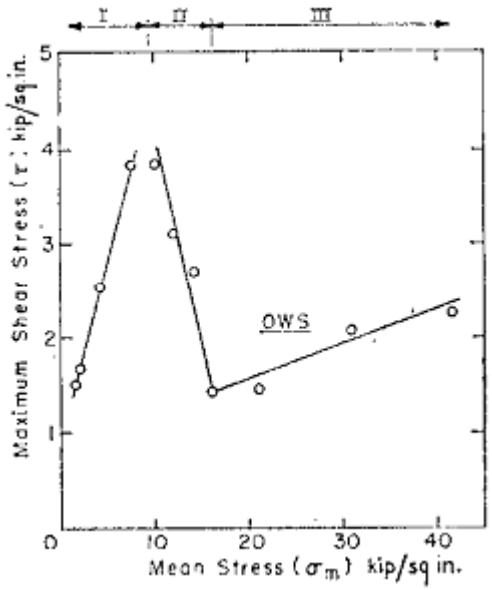
Major Factors

➢ Confining pressure:

1) Low pressure: (Region I)
   - ✓ Confining pressure makes the solid phase (soil and ice) more compact
   - ✓ Strength increases with confining pressure

2) High pressure: (Region 2)
   - ✓ Ice in the sample begins to be crushed
   - ✓ Pressure melting occurs
   - ✓ Strength decreases with confining pressure

3) Higher pressure: (Region 3)
   - ✓ Ice content tends to zero
   - ✓ Strength increases with confining pressure

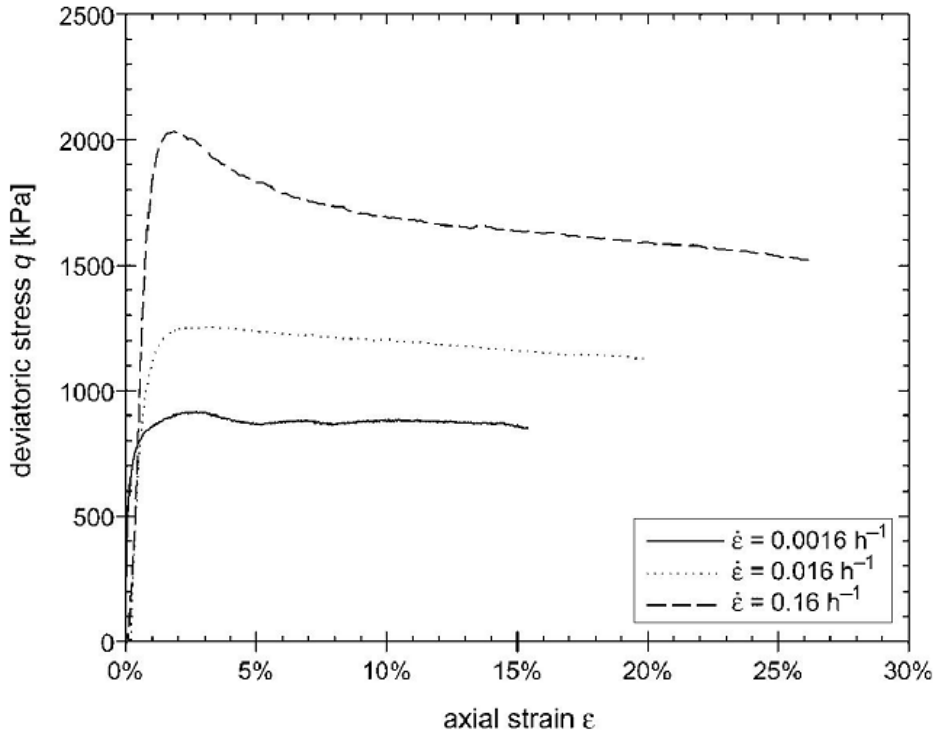


Relation between strength and confining pressure (Chamberlin et al., 1972)

Major Factors

- Strain rate:

  - ✓ Increasing strain rate results in:
    a. An increase in strength due to stiffening effect of ice
    b. Softening behavior due to bands cracking between ice and grains
    c. More brittle behavior



Stress-strain curves at different strain rate (Arenson et al., 2004)

Existing Models

Total stress approach

Two stress-state variables approach

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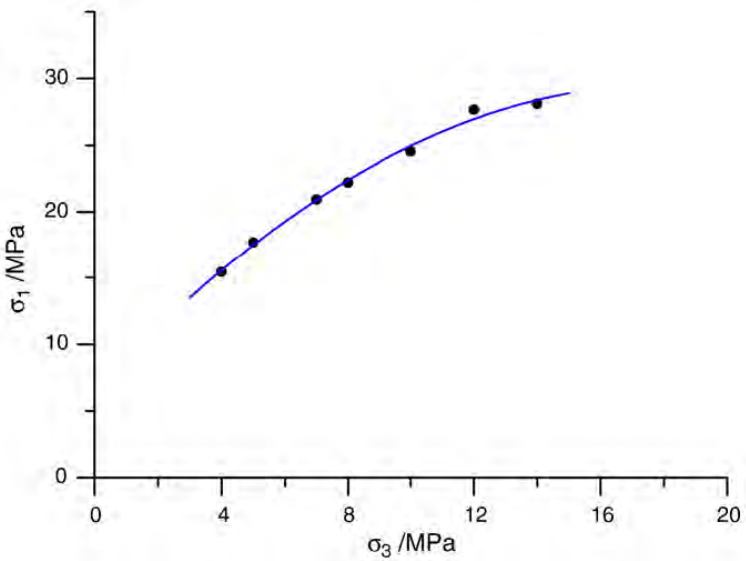
Existing Models

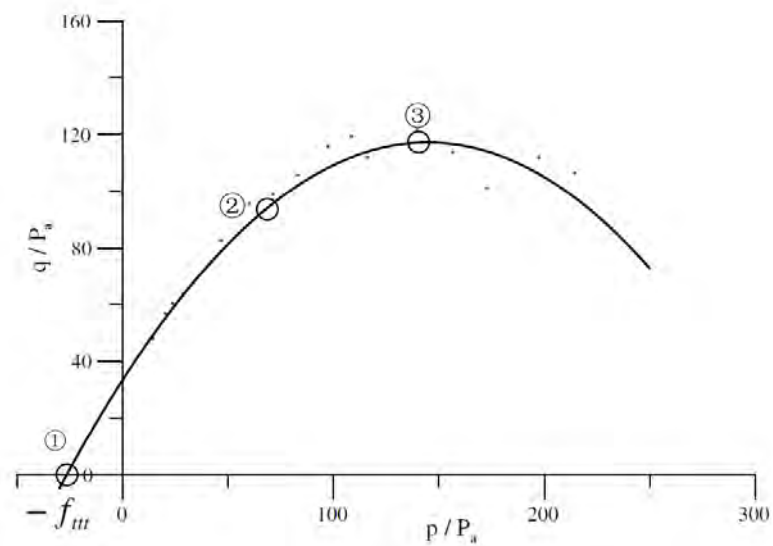
➢ Total stress approach:

(e.g. Lai et al., 2010; Yang et al., 2010; Lai et al., 2009; Lai et al., 2014)

✓ Due to variation of behavior with confining pressure, some new strength criterion and critical state line were defined:



Strength criteria (Yang et al., 2010)



Critical state line (Lai et al., 2009)

Existing Models

- Total stress approach:

  - Well predictions in constant temperature during mechanical loading
  - How they can simulate the behavior when there is unfrozen water?
  - How they can predict deformation during freezing and thawing without any additional loading?

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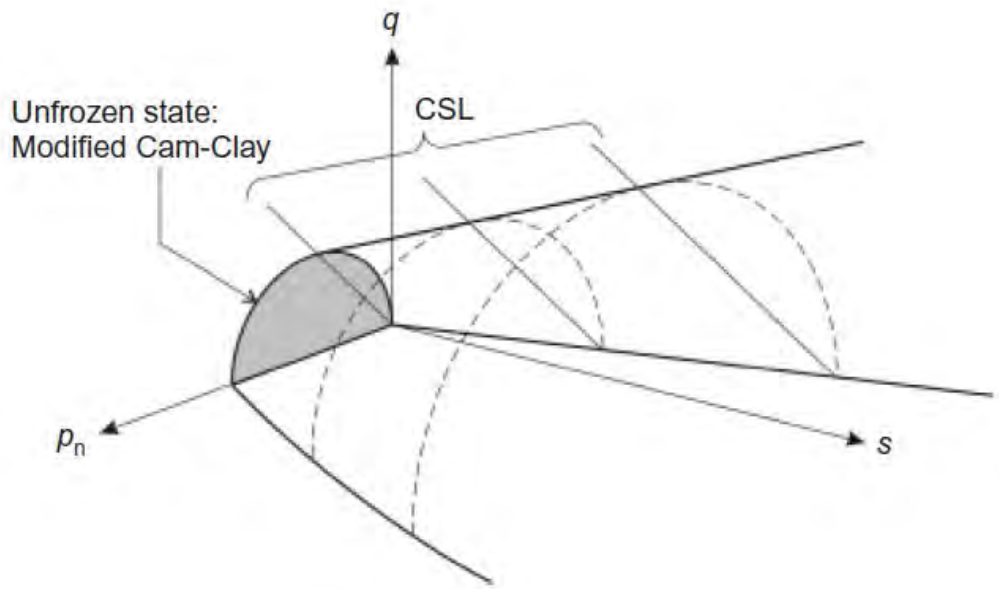
Existing Models

➢ Two stress-state variables approach:
(e.g. Nishimura et al., 2009; Shastri and Sanchez, 2012)

❖ Stress variables: Net stress: $\boldsymbol{\sigma}_n = \boldsymbol{\sigma} - p_i \mathbf{I}$

Suction: $S = p_i - p_w$



Complete yield surfaces of Nishimura et al. 2009

Existing Models

- Two stress-state variables approach:

  - Definition of net stress?
  - Frost heave?

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The Proposed Framework

Proposed Framework

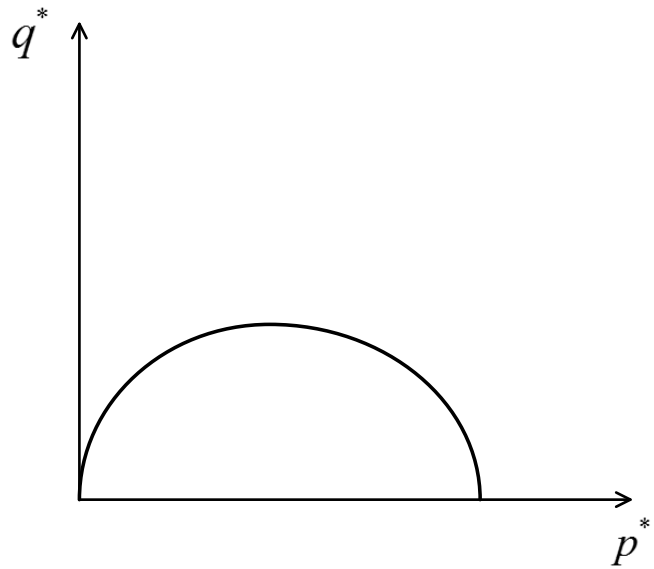
- Neglecting the air phase:
  1. Soil particles
  2. Ice
  3. Unfrozen water

There is only one suction: Cryogenic suction

- Frozen soil could be explained by
  - Solid phase (consists of soil particles and ice)
  - Water phase

$$\boldsymbol{\sigma}^* = \boldsymbol{\sigma} - s_w p_w \mathbf{I}$$

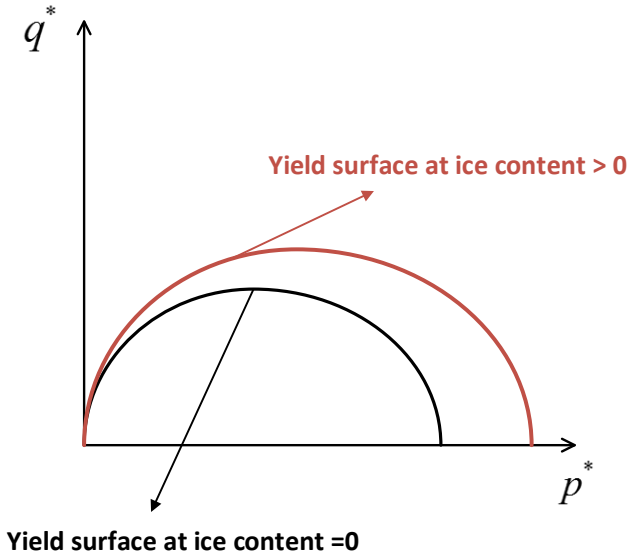
- Yield surface in $p^* - q^*$ space:



Proposed Framework

- Ice content effects (for lower ice content)

Hardening behavior with increasing ice content:

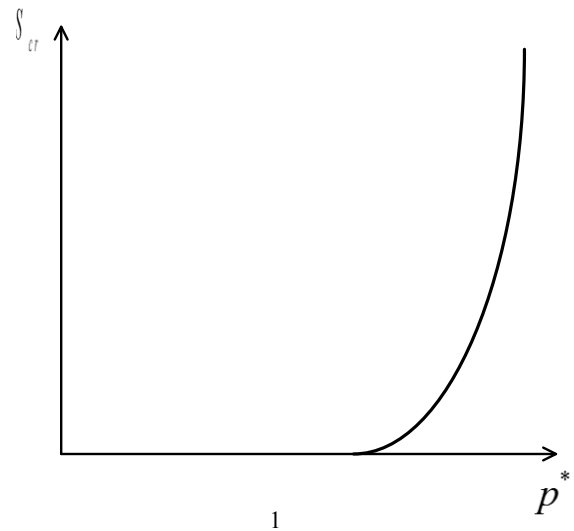


Also we should considered the strain due to ice content variation:

There is a relation between suction and ice content variations: (Nishimura et al. 2009)

$$s_i = 1 - \left\{ 1 + \left[\frac{-(1-\frac{\rho_i}{\rho_l})\rho_l - \rho_i l \ln(\frac{T}{T_0})}{p} \right]^{\frac{1}{1-\lambda_R}} \right\}^{-\lambda_R}$$

T_0 = freezing point under the existing pressure



$$T_0 = 273.16\left(\frac{p}{-395}+1\right)^{\frac{1}{9}}$$

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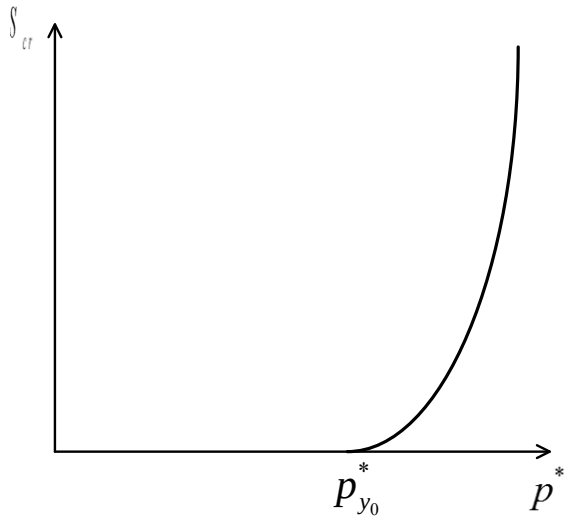
Proposed Framework

- Ice content effects (for lower ice content)

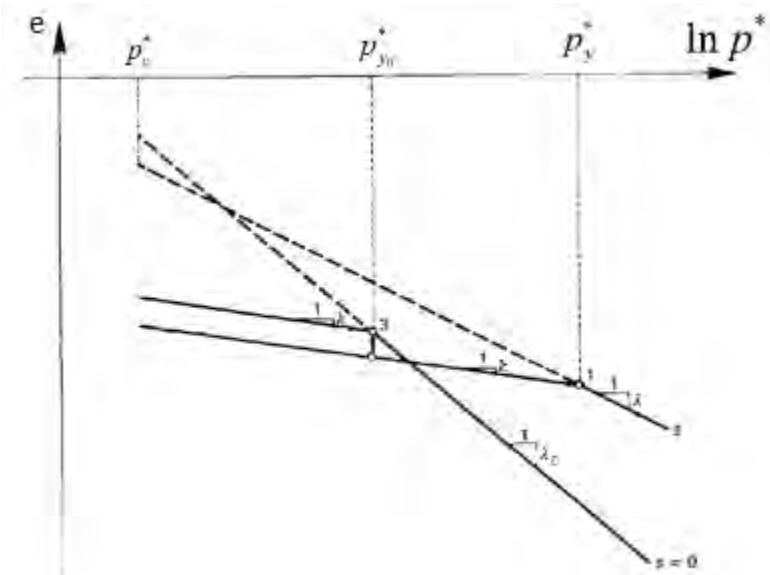


From Barcelona Basic Model (BBM):

$$p^*_y = p^*_c \left(\frac{p^*_{y_0}}{p^*_c} \right)^{\frac{\lambda_0 - \kappa}{\lambda - \kappa}}$$

$$\lambda = \lambda_0 \left[(1-r)\exp(-\beta S_{cry}) + \mathrm{r} \right]$$

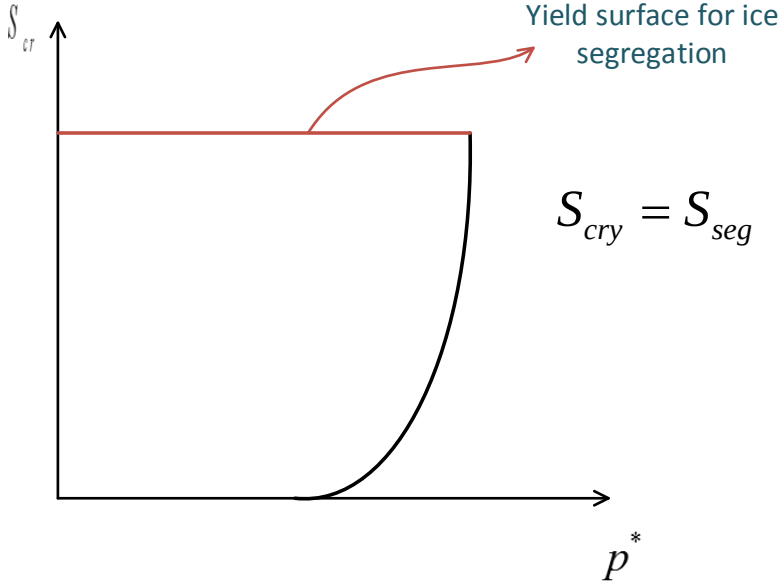
Definition of parameters:



Proposed Framework

- Ice content effects (for higher ice content)

If ice content becomes larger than a certain value, ice segregation occurs and we need another yield due to increase in suction:



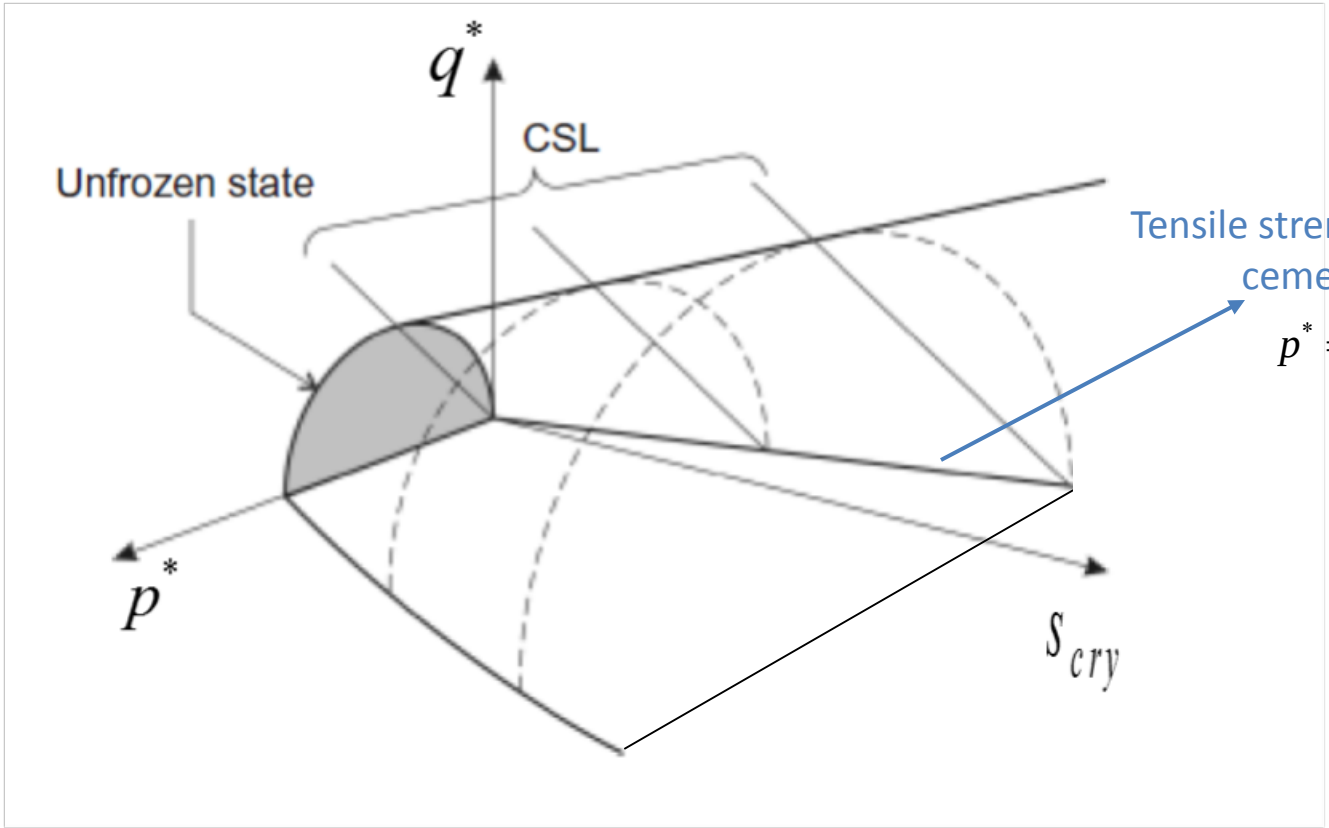
Definition of new yield because of ice segregation

we should consider the effect on volumetric strain: $\varepsilon_v^p = \varepsilon_v^{mp} + \varepsilon_v^{sp}$

In this way it is possible to simulate the frost heave

Proposed Framework

- Complete yield surfaces



Tensile strength due to ice cementation:

$$p^* = -K_t S_{cry}$$

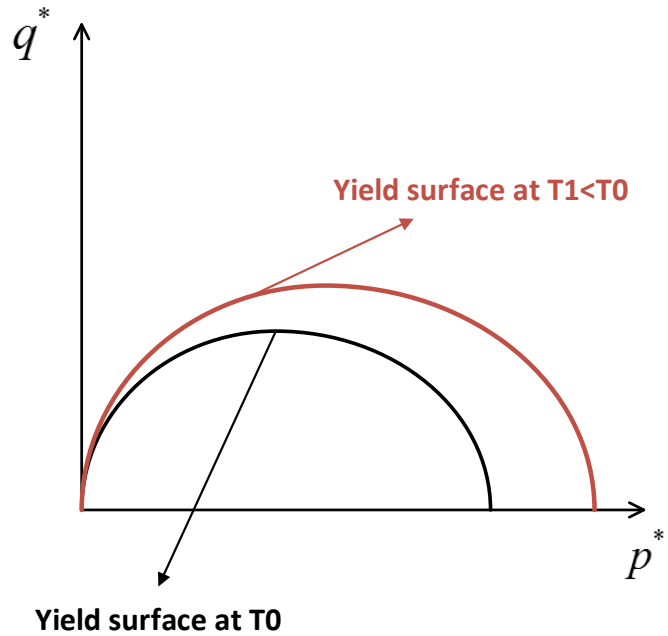
Proposed Framework

- Temperature effects

Temperature variation results in change in suction: (Clausius-Clapeyron)

$$S_{cry} = p_i - p_l = p_l\left(\frac{\rho_i}{\rho_l} - 1\right) - \rho_i l \ln\frac{T}{T_0}$$

$$l = \text{ specific latent heat of fusion}$$



Also we should have E as a function of temperature (Zhu et al. 2010):

$$E_{composit} = \frac{\left[\theta_s E_s(1-2\mu_i) + \theta_i E_i(1-2\mu_s)\right]\left[\theta_s E_s(1+\mu_i) + \theta_i E_i(1+\mu_s)\right]}{\theta_s E_s(1+\mu_i)(1-2\mu_i) + \theta_i E_i(1+\mu_s)(1-2\mu_s)}$$

$$\mu_{composit} = \frac{\theta_s E_s \mu_s(1+\mu_i)(1-2\mu_i) + \theta_i E_i \nu_i(1+\mu_s)(1-2\mu_s)}{\theta_s E_s(1+\mu_i)(1-2\mu_i) + \theta_i E_i(1+\mu_s)(1-2\mu_s)}$$

$$E_i = E_{i_{ref}} - \Delta E_i\left(T - T_{ref}\right)$$

In this way:

brittle behavior with decreasing temperature is considered

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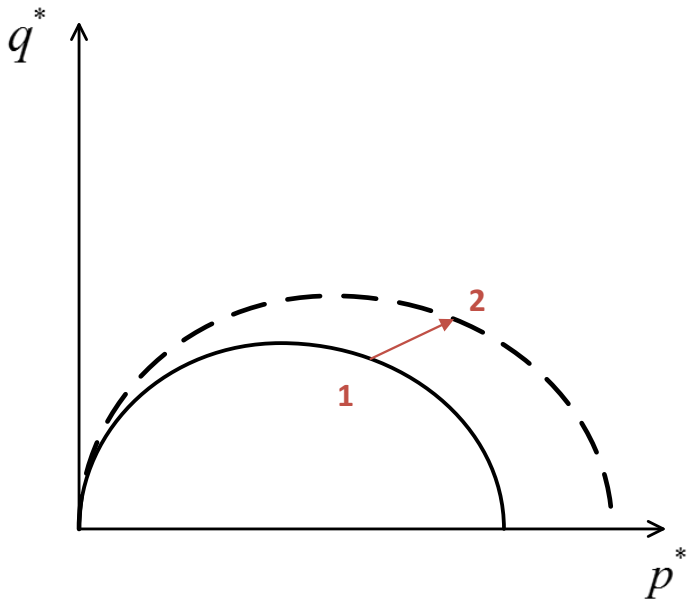
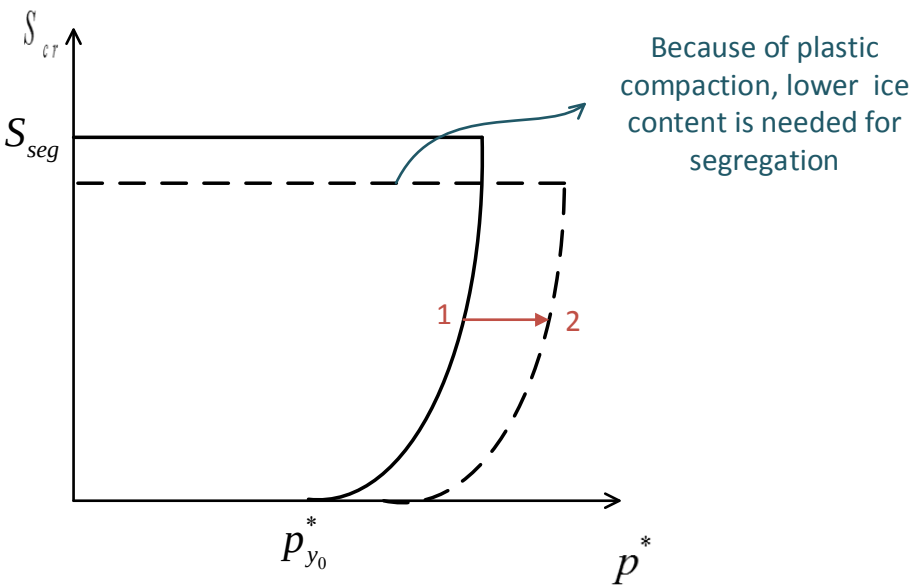
Proposed Framework





Because of plastic compaction, lower ice content is needed for segregation

Hardening laws:

$$\frac{dp^*_{y_0}}{p^*_{y_0}} = \frac{1+e}{\lambda_0 - \kappa} d\varepsilon_v^{mp} + \frac{1+e}{\lambda_s - \kappa_s} d\varepsilon_v^{sp}$$

$$\frac{dS_{seg}}{S_{seg} + p_{at}} = -\frac{1+e}{\lambda_s - \kappa_s} d\varepsilon_v^{sp} - \frac{1+e}{\lambda_0 - \kappa} d\varepsilon_v^{mp}$$

κ_s : compressibility coefficinet of soil for change in suction in elastic region

λ_s : compressibility coefficinet of soil for change in suction after ice segregation

λ_s has a negative value

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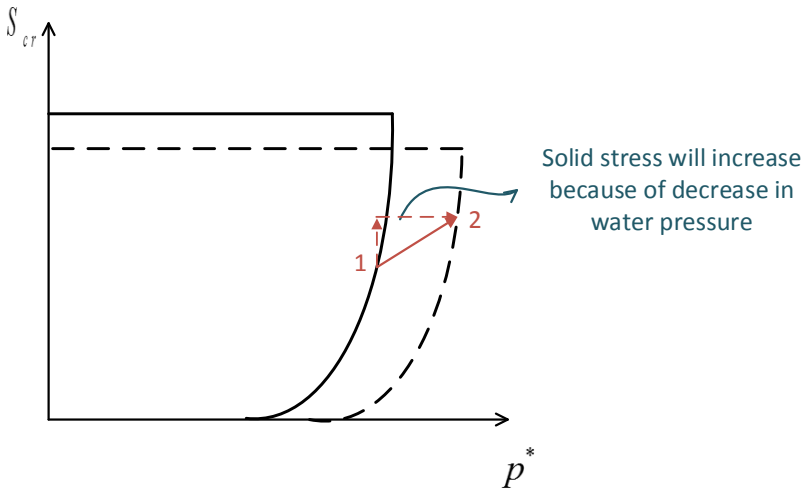
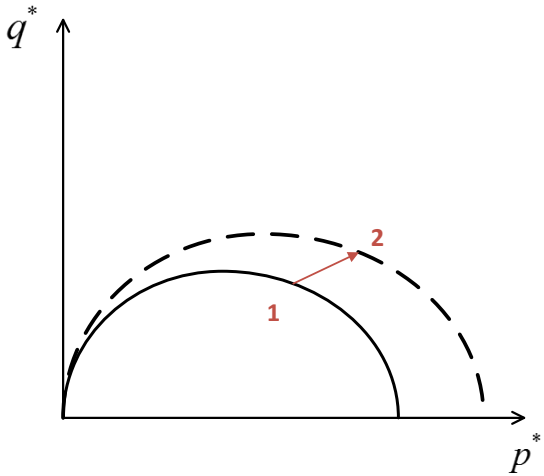
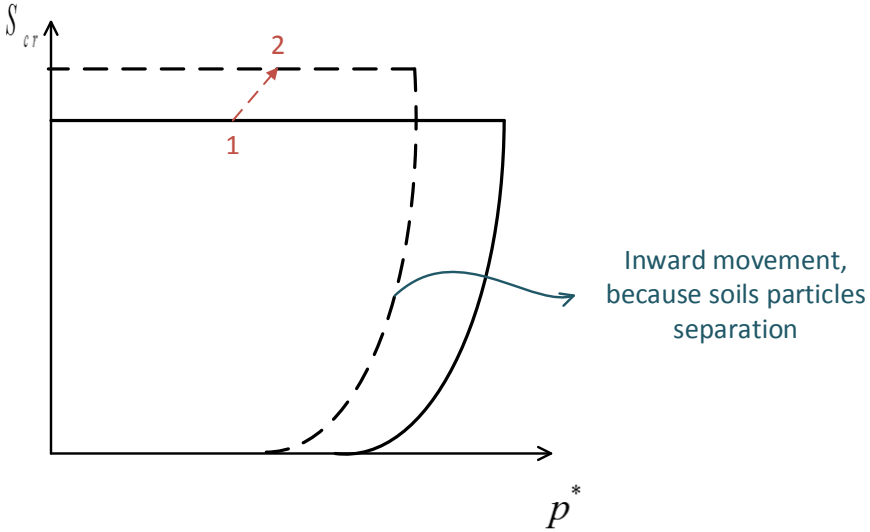
Proposed Framework

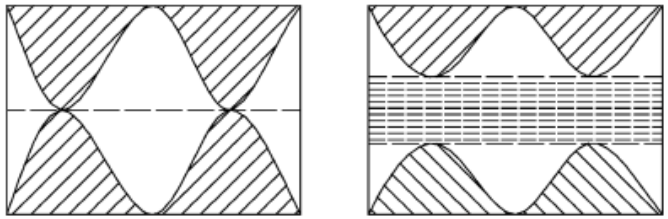
- Freezing in poor ice soils





- Freezing in ice rich soils



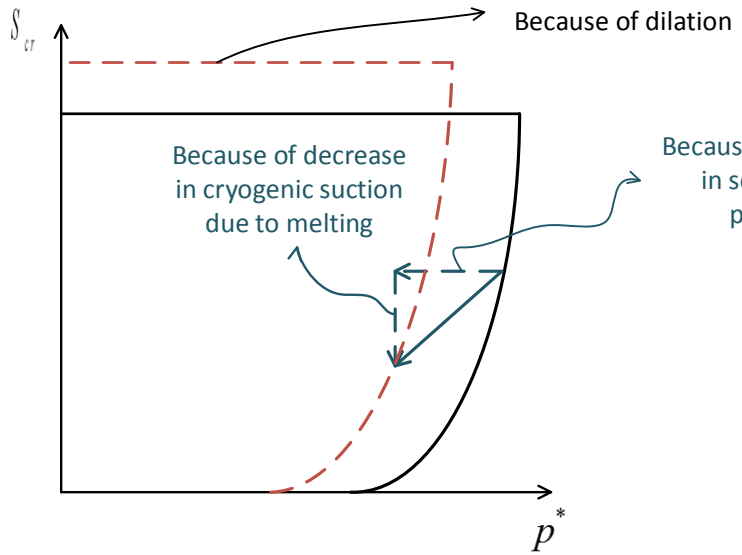


Proposed Framework

- Pressure melting

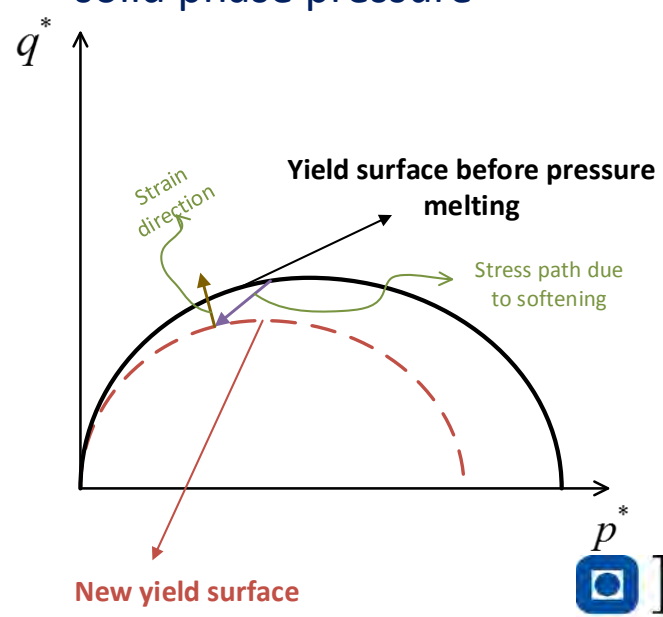
High pressure will change the thawing temperature: $T_0 = 273.16\left(\frac{p}{-395}+1\right)^{\frac{1}{9}}$

So, liquid pressure will increase and cryogenic suction will decrease: $S_{cry} = p_l - p_i = p_l\left(\frac{\rho_i}{\rho_l}-1\right) - \rho_i l \ln\frac{T}{T_0}$



During pressure melting there are decreases in suction and solid phase pressure

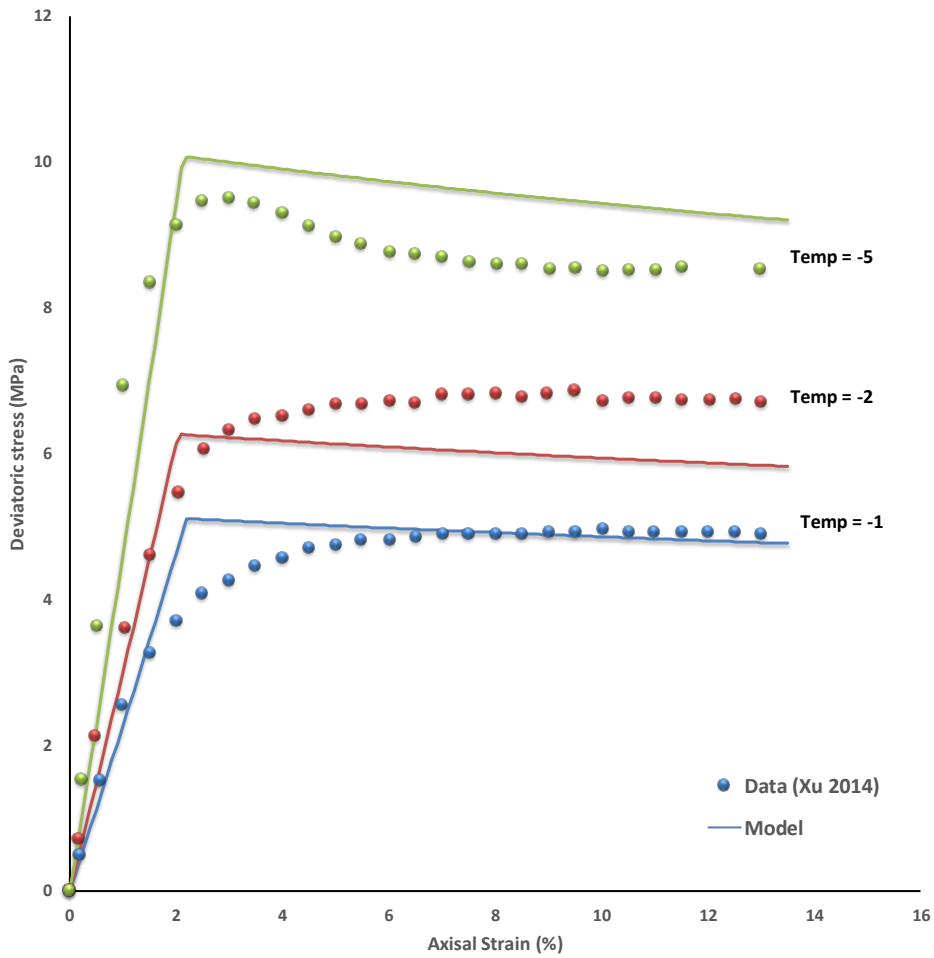
This results in softening behavior during pressure melting

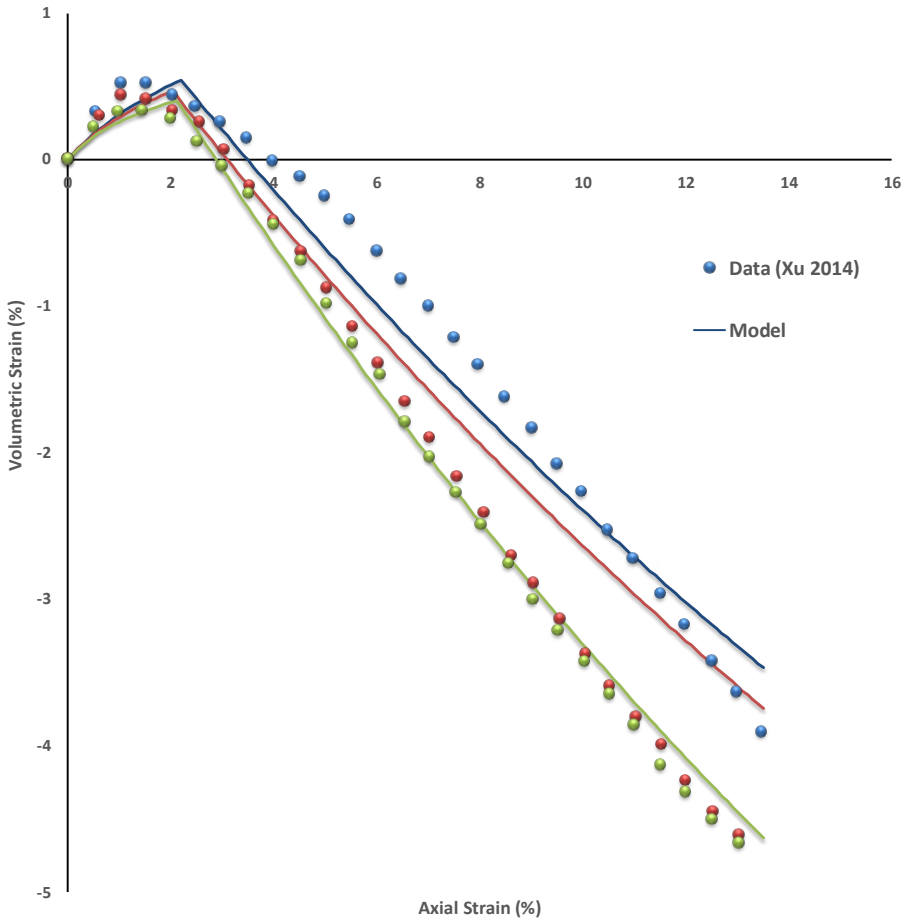


Model Results

Effect of temperature

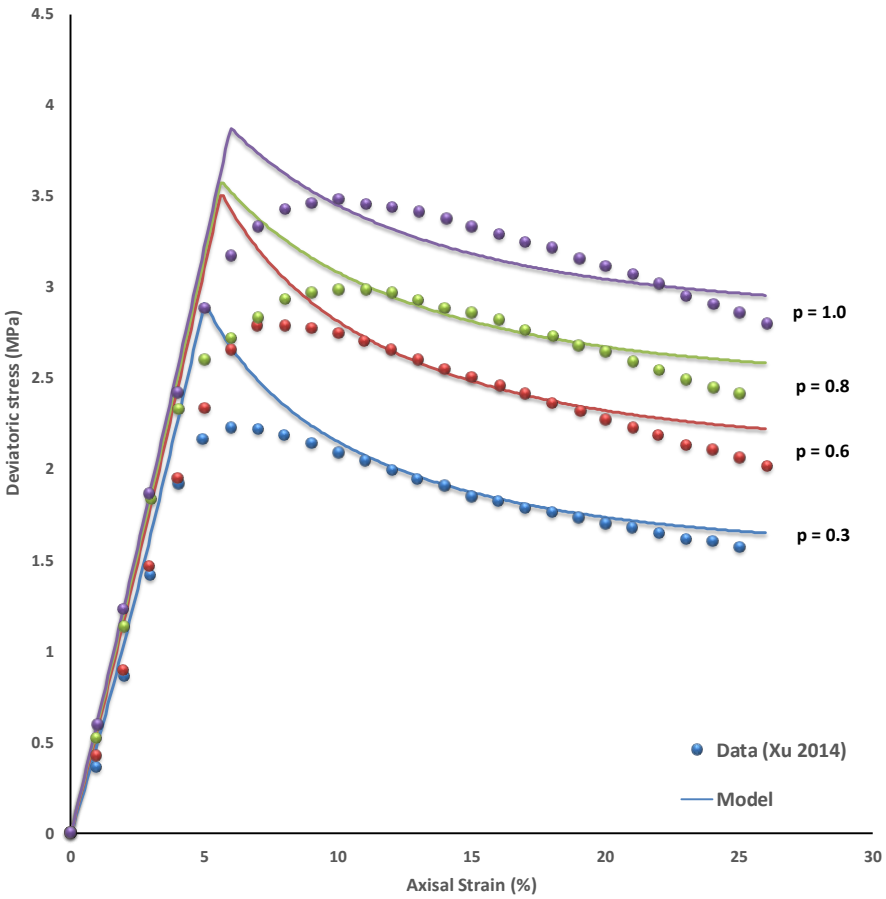
- Triaxial tests under 1 MPa confining pressure and **different temperature**:

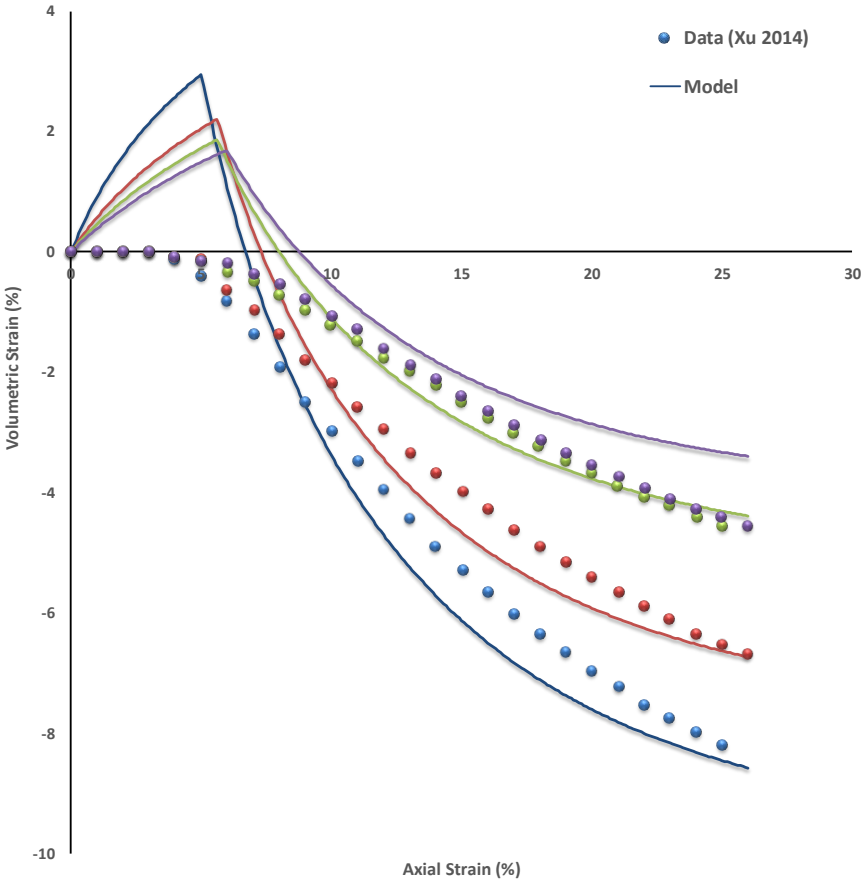




Effect of confining pressure

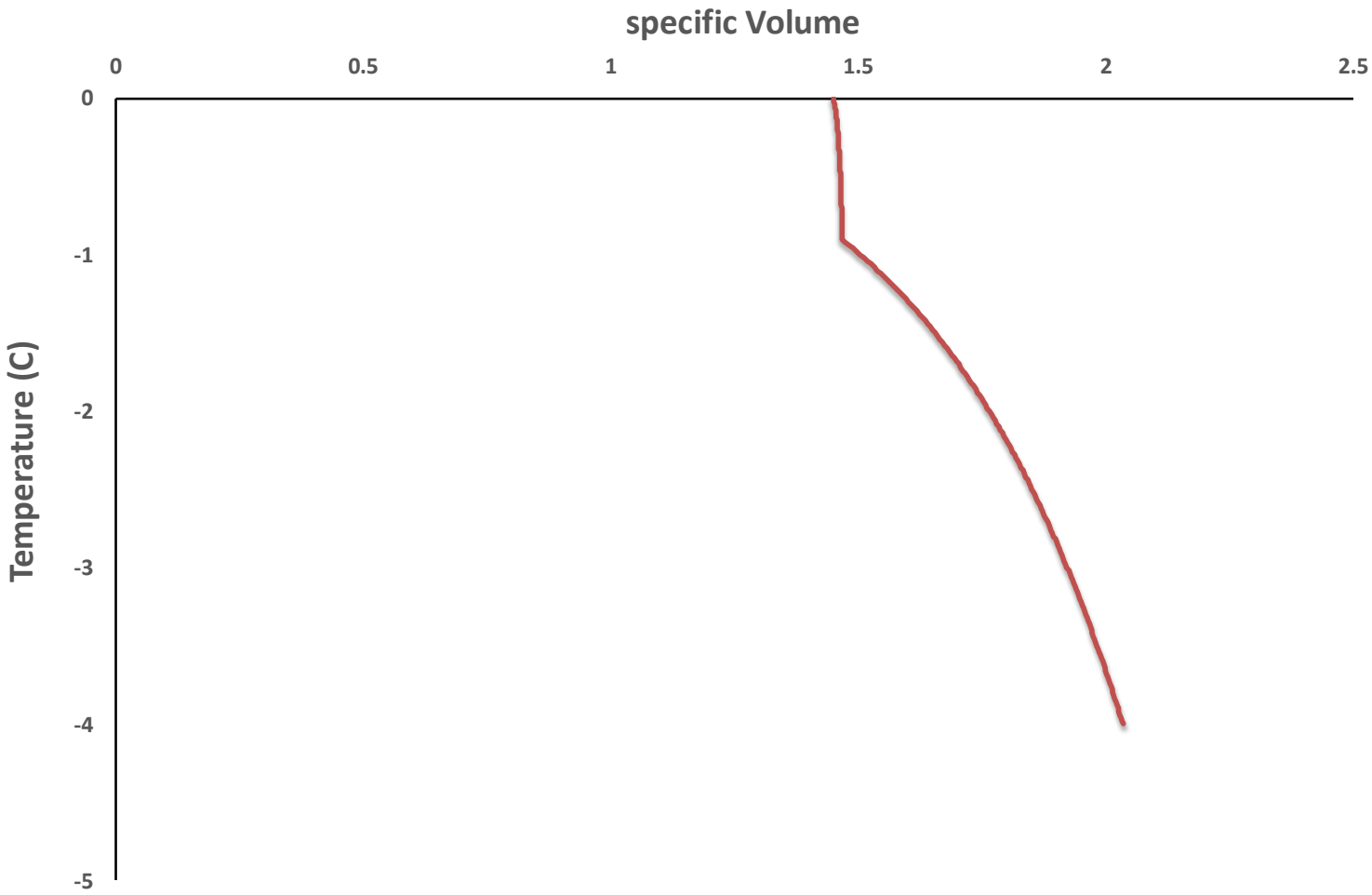
- Triaxial tests under **different confining pressure** and temperature = -4





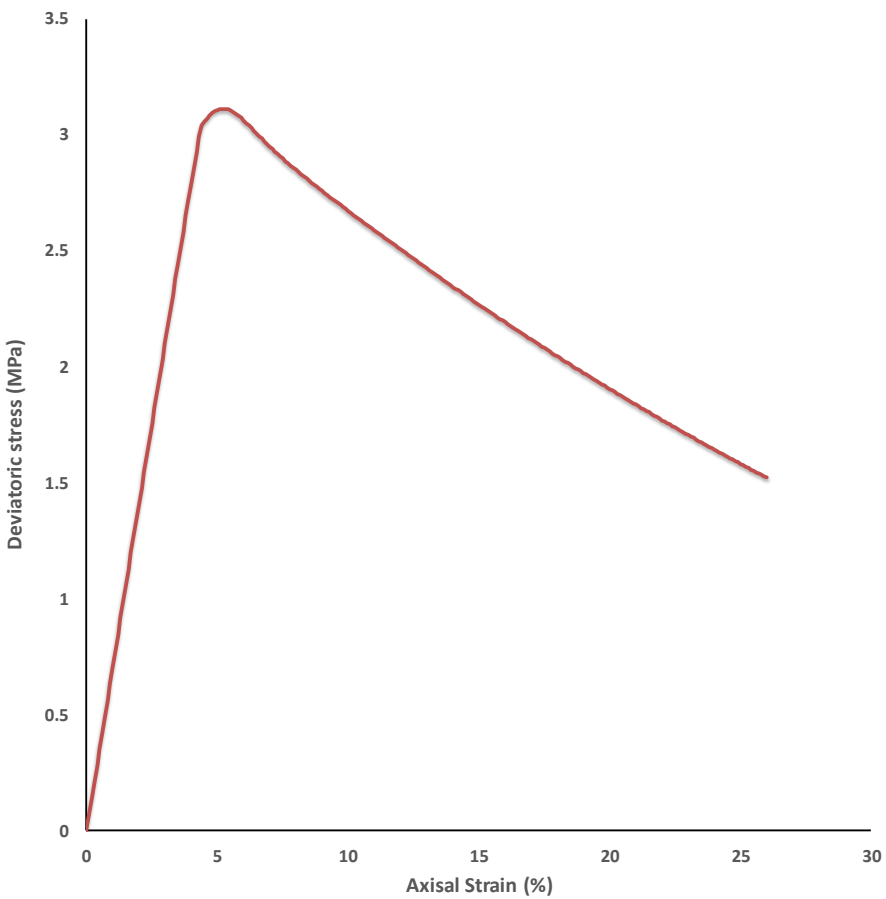
Freezing a sample

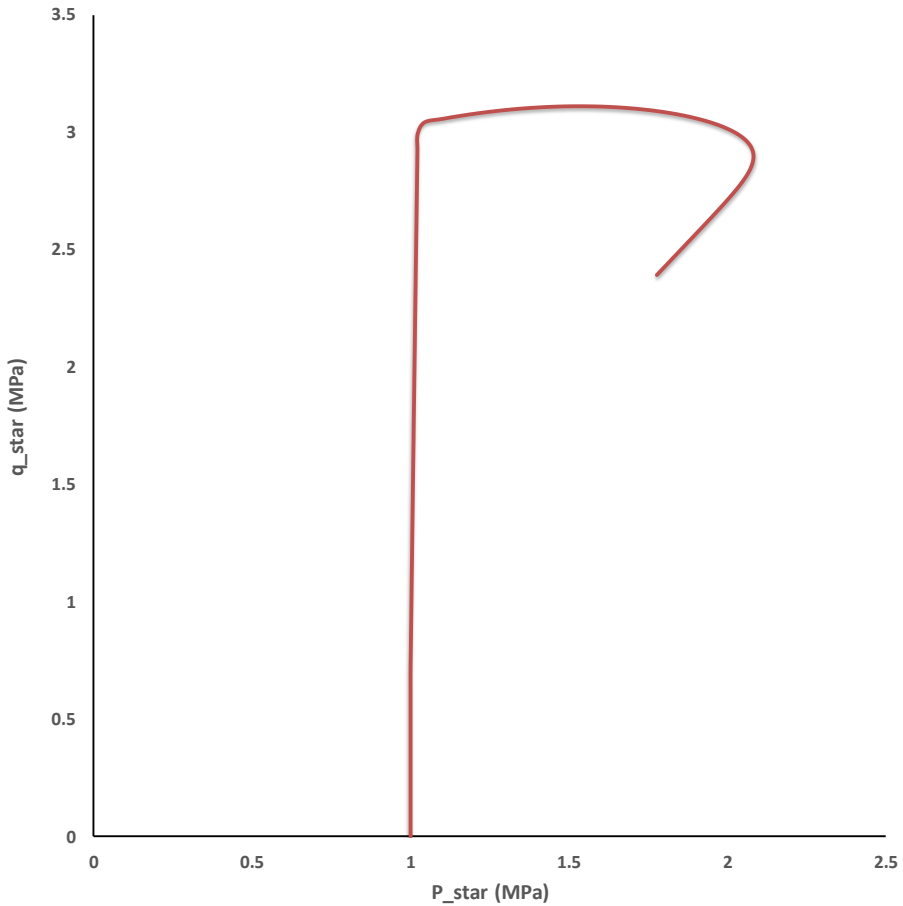
- Decreasing the temperature from 0 to -4 :



Pressure Melting

- Triaxial test in temperature = -1





Conclusion

- By introducing the solid phase stress, it is possible to work with the part of total stress which is responsible for soil deformation

- By considering the cryogenic suction as the second stress state variable, it is possible to simulate the influence of temperature and ice content on the behavior

- Ice segregation phenomenon is considered in the model by introducing a yield surface in the suction-mean solid phase stress plane

- Decrease of strength by confining pressure in ice rich soils due to the pressure melting is simulated by changing the thawing temperature of ice

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THANKS FOR YOUR ATTENTION!

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