LAPORAN KEGIATAN PKM PRODI SIPIL

Judul Kegiatan : **Utusan Teknik Sipil dalam Keikutsertaan pada KONGRES HATHI 2020**

Terselenggara pada : **20 November 2020 secara daring, bukti undangan dan kehadiran terlampir**

Utusan dari Prodi Teknik Sipil Itenas :

1. DR Yati Muliati, Ir., MT
2. Fransiska Yustiana, ST., MT

Kami berdua ditunjuk oleh Ketua HATHI cabang Jawa Barat sebagai utusan HATHI jabar pada KONGRES HATHI 2020 dan mewakili dalam komisi C

Hasil Kegiatan terlampir sebagai berikut ini :

HIMPUNAN AHLI TEKNIK HIDRAULIK INDONESIA
Indonesian Association of Hydraulic Engineers
Pengurus Pusat

Nomor : 01/Set-Pus/XI/2020 Jakarta, 26 November 2020

Yth,
Ketua Cabang HATHI
(daftar terlampir)
di –
Tempat

Perihal : Penyampaian Hasil Kongres ke-13 Himpunan Ahli Teknik Hidraulik Indonesia (HATHI)

Sehubungan dengan selesainya Kongres Ke-13 HATHI pada tanggal 21 November 2020, kami sampaikan hasil keputusan kongres antara lain :

a. Keputusan nomor 05/KPTS-KONGRES/HATHI/XIII/2020 tentang Pengesahan Anggaran Dasar (AD) dan Anggaran Rumah Tangga (ART) HATHI 2020.
b. Keputusan nomor 10/KPTS-KONGRES/HATHI/XIII/2020 tentang Komposisi dan Personalia Pengurus Pusat HATHI Periode 2020 – 2023 yaitu :

   Ketua Umum : Ir. Jarot Widyoko, Sp1.
   Sekretaris Umum : Dr. Ismail Widadi, ST. MSc.
   Bendahara Umum : Ir. Gunawan Lukito, PU-SDA

Keputusan kongres selengkapnya, akan kami kirimkan via email.

Menindaklanjuti hasil keputusan kongres tersebut diatas, dengan hormat kami harapkan hal-hal sebagai berikut:

1. Semua Cabang HATHI melaksanakan hasil kongres tersebut dengan penuh tanggung jawab dengan tetap melaksanakan tugasnya sesuai Program Kerja Cabang masing-masing.
2. Semua Cabang HATHI segera mengadakan Rapat Koordinasi Cabang atau Musyawarah Cabang dalam rangka sosialisasi hasil kongres tersebut di atas dan sekaligus memperbaharui organisasinya (anggota baru, penggantian yang meninggal dan mengundurkan diri, maupun menyusun kembali kepengurusan Cabang yang telah berakhir masa baktinya).
3. Semua Cabang HATHI segera menyampaikan program kerjanya ditahun 2021 kepada Ketua Umum, terutama kegiatan yang melibatkan Pengurus Pusat.

Demikian disampaikan, atas perhatian dan kerja samanya yang baik, kami sampaikan terima kasih.

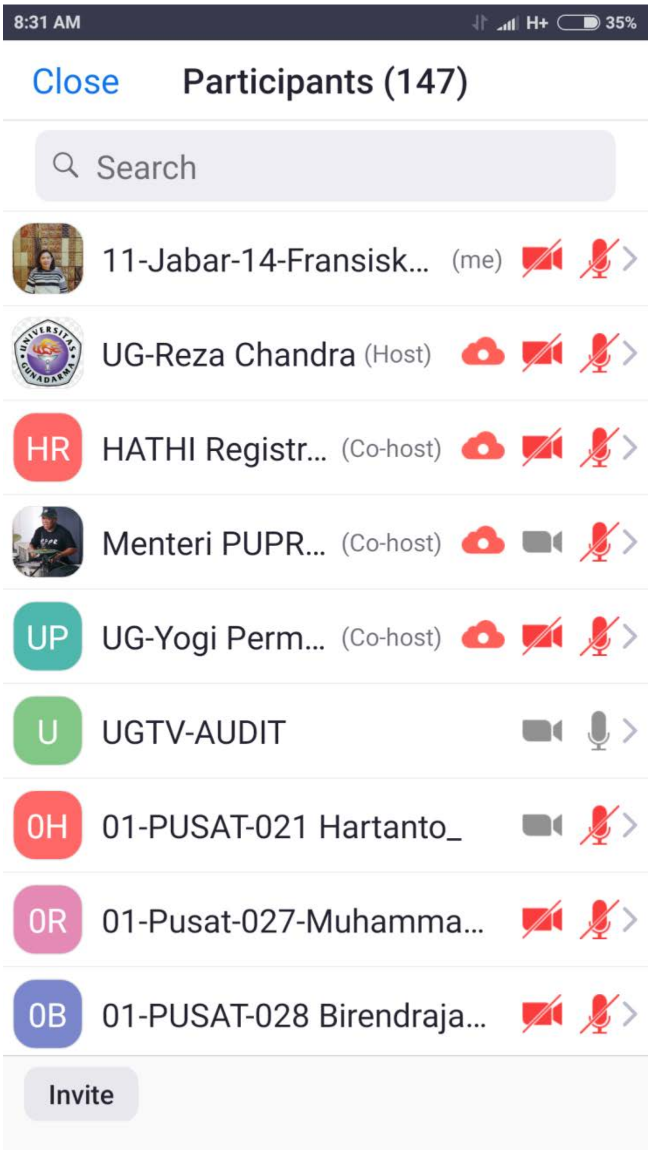
Pengurus Pusat HATHI,

HATHI PUSAT

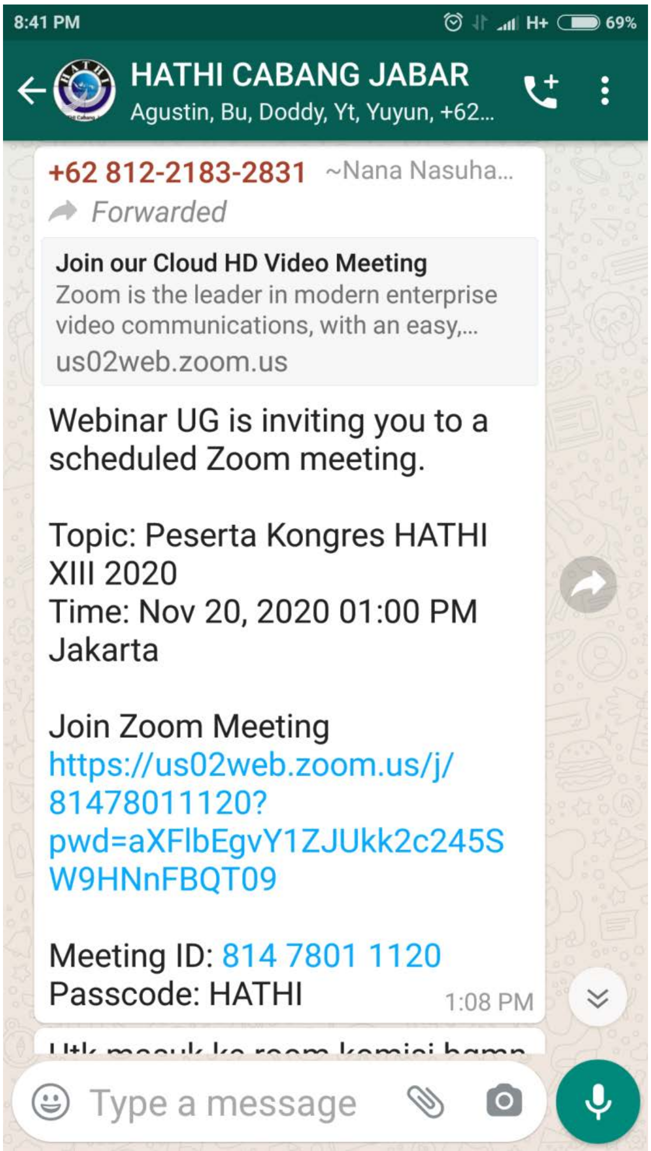
<u>Ir. Jarot Widyoko, Sp.1.</u>
Ketua Umum

<u>Tembusan:</u>
1. Sekretaris Umum HATHI
2. Bendahara Umum HATHI

Bukti Kehadiran



Bukti undangan



CG01
INTRODUCTION TO GEOTECHNICAL FINITE ELEMENT MODELLING

Helmut F. Schweiger [1)] dan Indra Noer Hamdhan [2)]

[1)] Computational Geotechnics Group
Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology
[2)] Civil Engineering Department
National Institute of Technology (Itenas) Bandung

NUMERICAL VS CONVENTIONAL ANALYSIS

method of analysis	Equil.	Comp.	Const. Behav.	B.C. Force	B.C. Displ.
Closed form	yes	yes	linear elastic	yes	yes
Limit equilibrium	yes	no	rigid with failure criterion	yes	no
Lower bound	yes	no	ideal plasticity with associated flow	yes	no
Upper bound	no	yes	ideal plasticity with associated flow	no	yes
Beam-spring	yes	yes	springs	yes	yes
Numerical analysis	yes	yes	any	yes	yes

THE FINITE ELEMENT METHOD

- **is a versatile numerical method to obtain (approximate) solutions to mathematical problems (**the governing mathematical equations are approximated by a series of algebraic equations involving quantities that are evaluated at discrete points within the region of interest)

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} = b_x \quad \Longrightarrow \quad a_{11}x_1 + a_{12}x_2 + \cdots + a_{1n}x_n = c_1$$

- **has been extensively used in many areas of engineering (and other disciplines)**

- **has a long tradition in geotechnical research**

- **has been developed to a stage where user-friendly codes at a high technical level are commercially available, some of them designed specifically for geotechnics**

FEM has become a standard (design) tool in practical geotechnical engineering

THE FINITE ELEMENT METHOD

What kind of problems?

Apply load → **obtain displacements**

stiffness matrix

Apply head → **obtain flow**

permeability matrix

Though we would like to know our solution at any coordinates in our project, we will only calculate them in a certain amount of discrete points (nodes) and estimate our solution anywhere else

> **> discretization:** This is the process of modelling the geometry of the problem under investigation by an assemblage of small regions, termed finite elements. These elements have nodes defined on the element boundaries, or within the elements.

Primary variable approximation: A primary variable must be selected (e.g. displacements) and rules as how it should vary over a finite element established. This variation is expressed in terms of nodal values.

- A polynomial form is assumed, where the order of the polynomia depends on the number of nodes in the element.
- The higher the number of nodes (the order of the polynomial), the more accurate are the results.

BASICS STEPS OF FINITE ELEMENT ANALYSIS

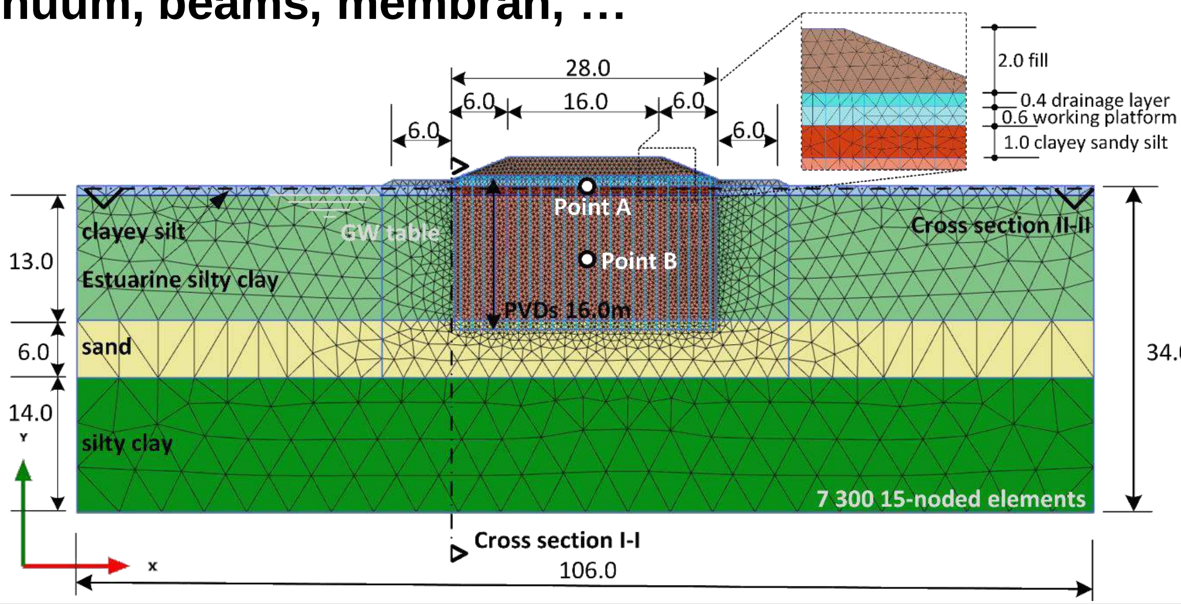
Define geotechnical problem and purpose of analysis

⬇

Define model for analysis
boundary conditions, domain to be analysed,…

⬇

Mesh generation, choose type of elements
continuum, beams, membran, …



BASICS STEPS OF FINITE ELEMENT ANALYSIS

define and assign material parameters

⬇

define loading / construction stages

⬇

solve equation system > displacements as primary variables

(not necessarily required but most popular formulation)

⬇

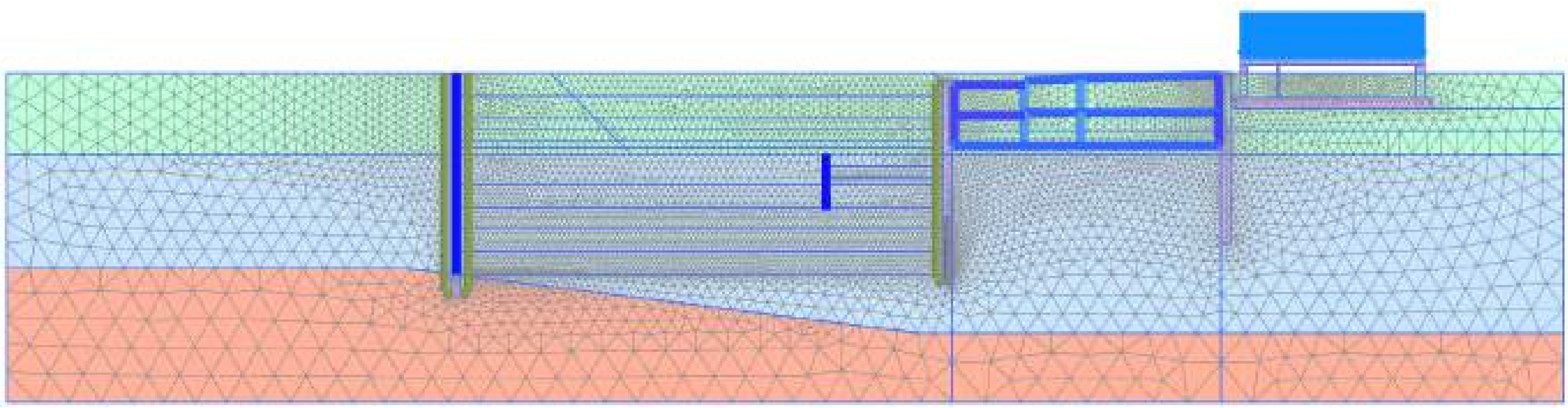
strains > stresses

⬇

critical assessment of results

of paramount importance in geotechnics

BASICS OF FINITE ELEMENT METHOD

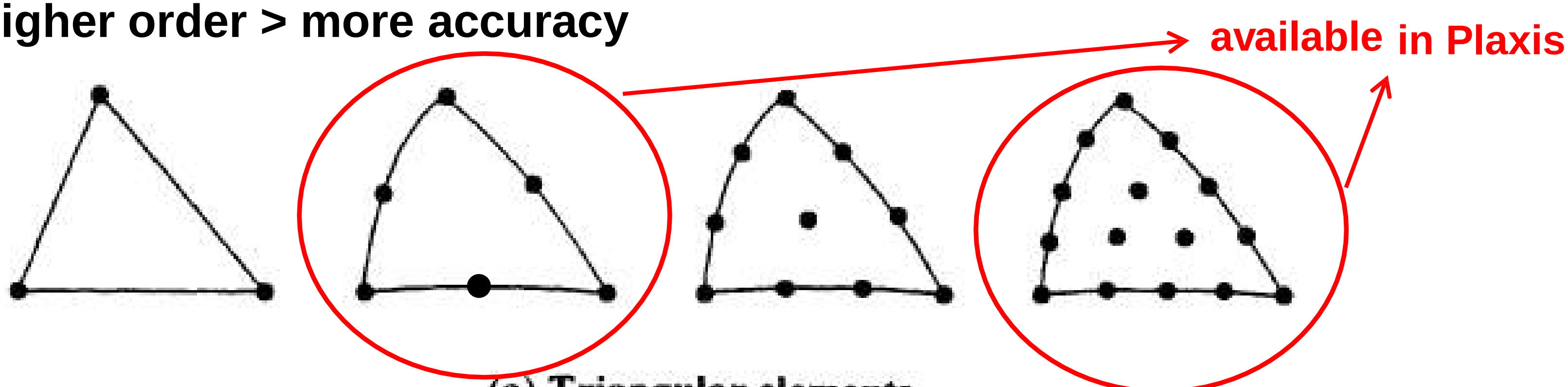


Typical 2D mesh for deep excavation

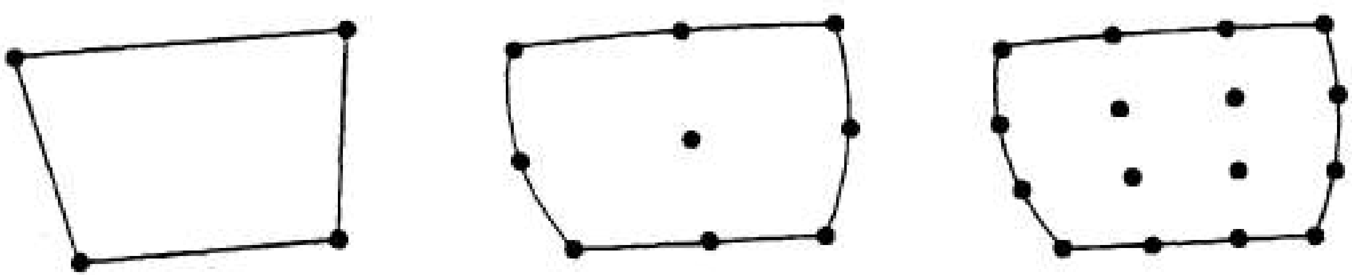
Hints for modelling: **- finer where high gradients are expected**
- aspect ratio (careful with highly distorted elements)

BASICS OF FINITE ELEMENT METHOD

- **isoparametric elements**
- **polynomial interpolation of displacements within element**
- **higher order > more accuracy**



(a) Triangular elements



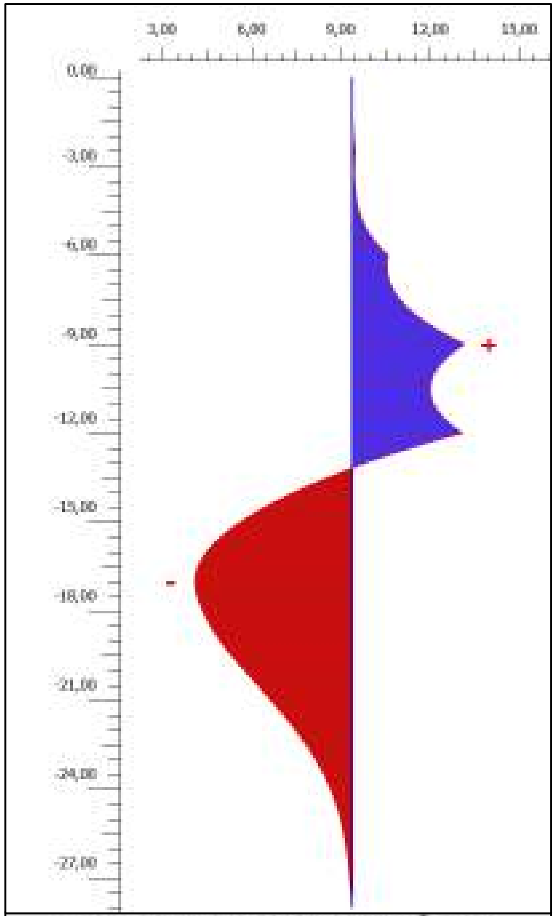
(b) Lagrange elements

BASICS OF FINITE ELEMENT METHOD - RESULTS

- **Primary unknowns (displacements in stress analysis) at *nodes***
- **Stresses and strains are most accurate in *integration points***
- **Internal forces (e.g. bending moments) in structural elements (e.g. beams and plates)**
- **Information which stress points have reached ultimate strength**

most codes provide comprehensive graphical output of contourlines for displacement and stress components, (excess) pore pressures, strength mobilisation etc.

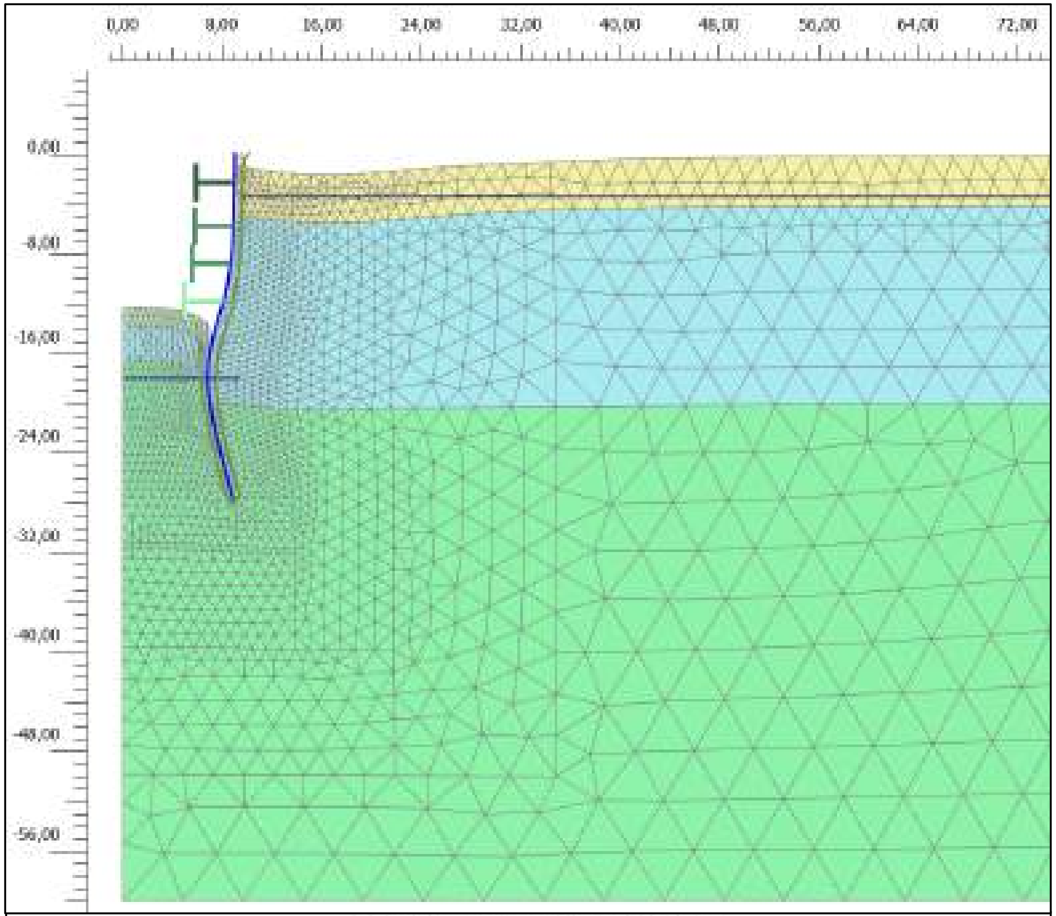
BASICS OF FINITE ELEMENT METHOD



Bending moments M (scaled up 5,00*10^{-3} times)

Maximum value = 758,4 kN m/m (Element 23 at Node 3761)

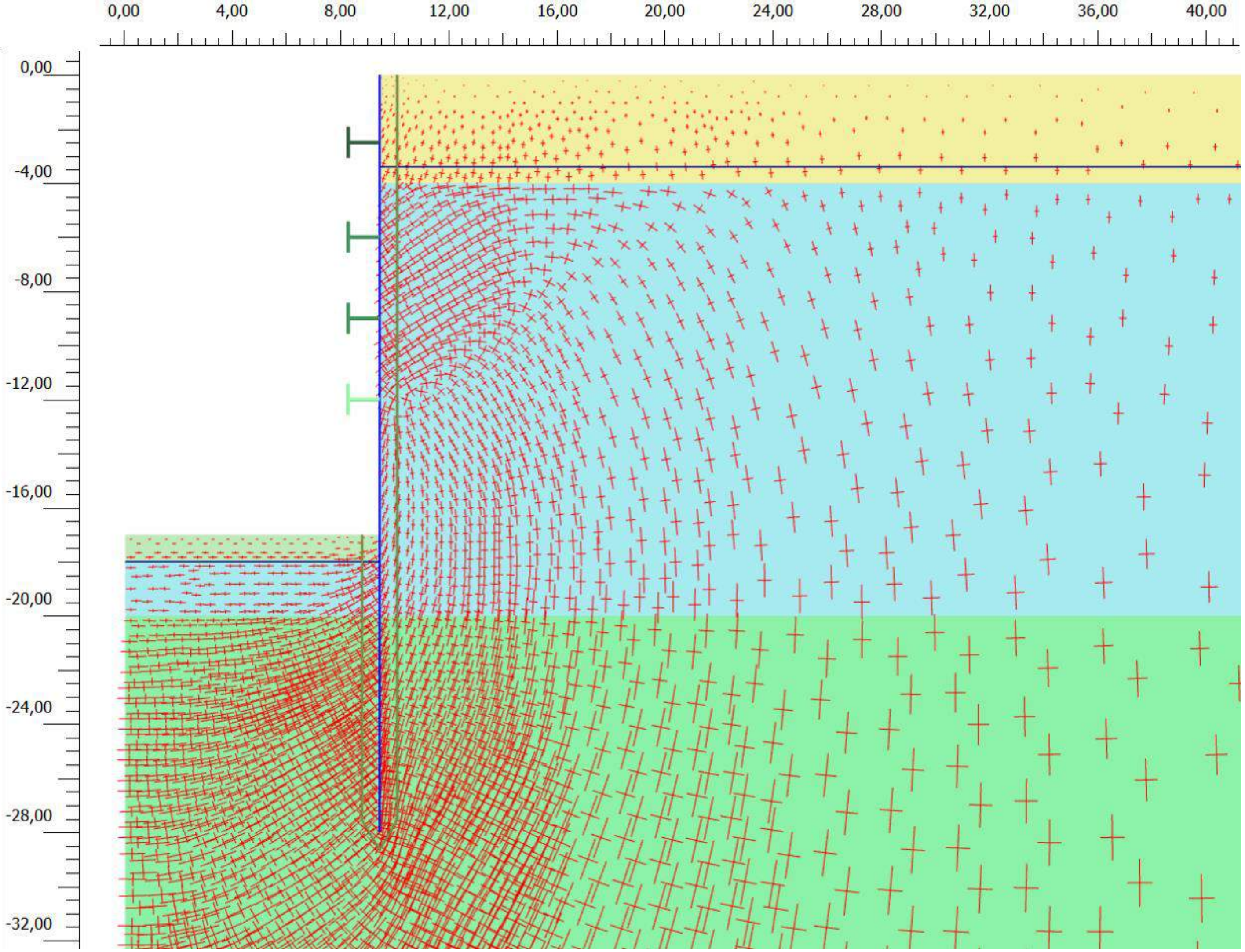
Minimum value = -1062 kN m/m (Element 39 at Node 4102)



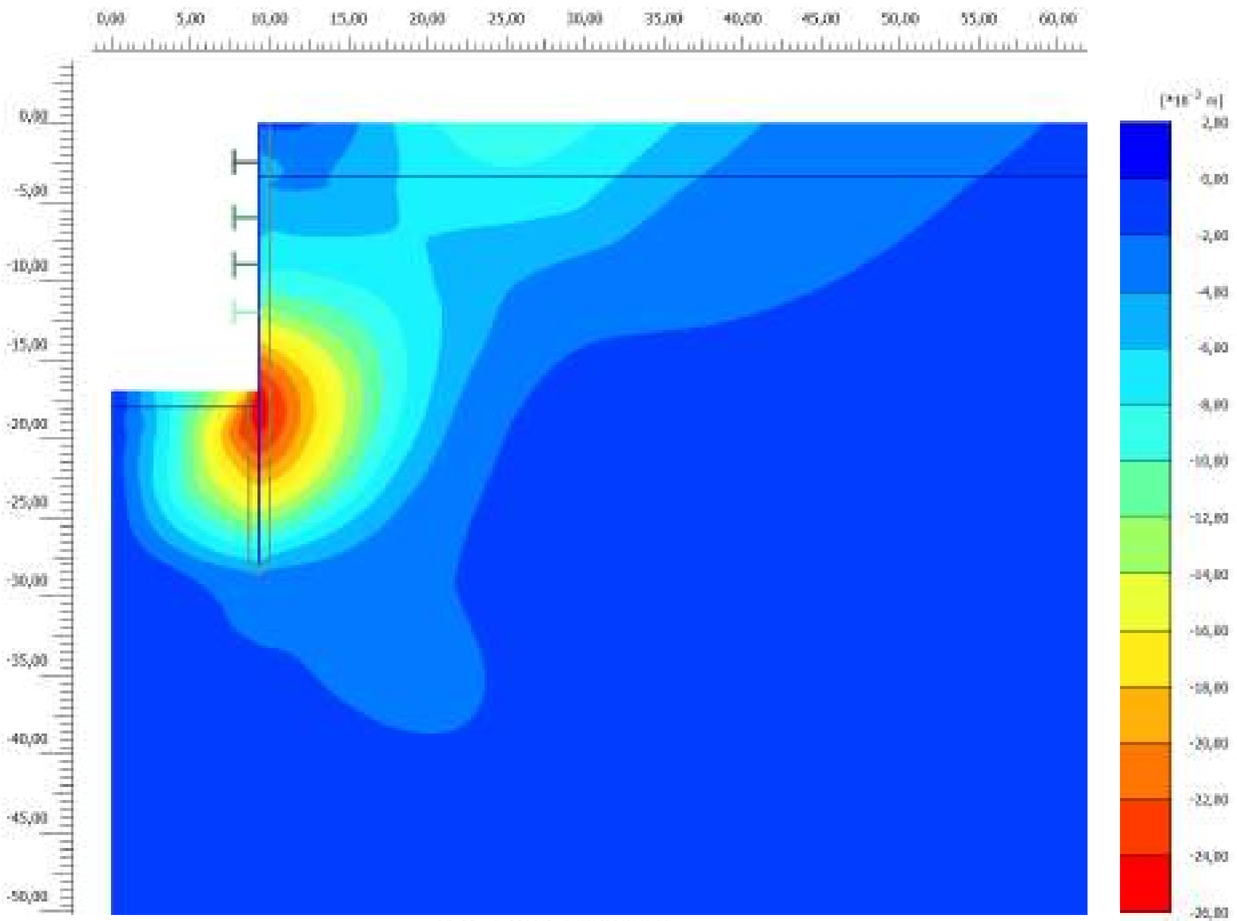
Deformed mesh |u| (scaled up 100 times)

Maximum value = 0,04746 m (Element 2908 at Node 474)

BASICS OF FINITE ELEMENT METHOD



BASICS OF FINITE ELEMENT METHOD



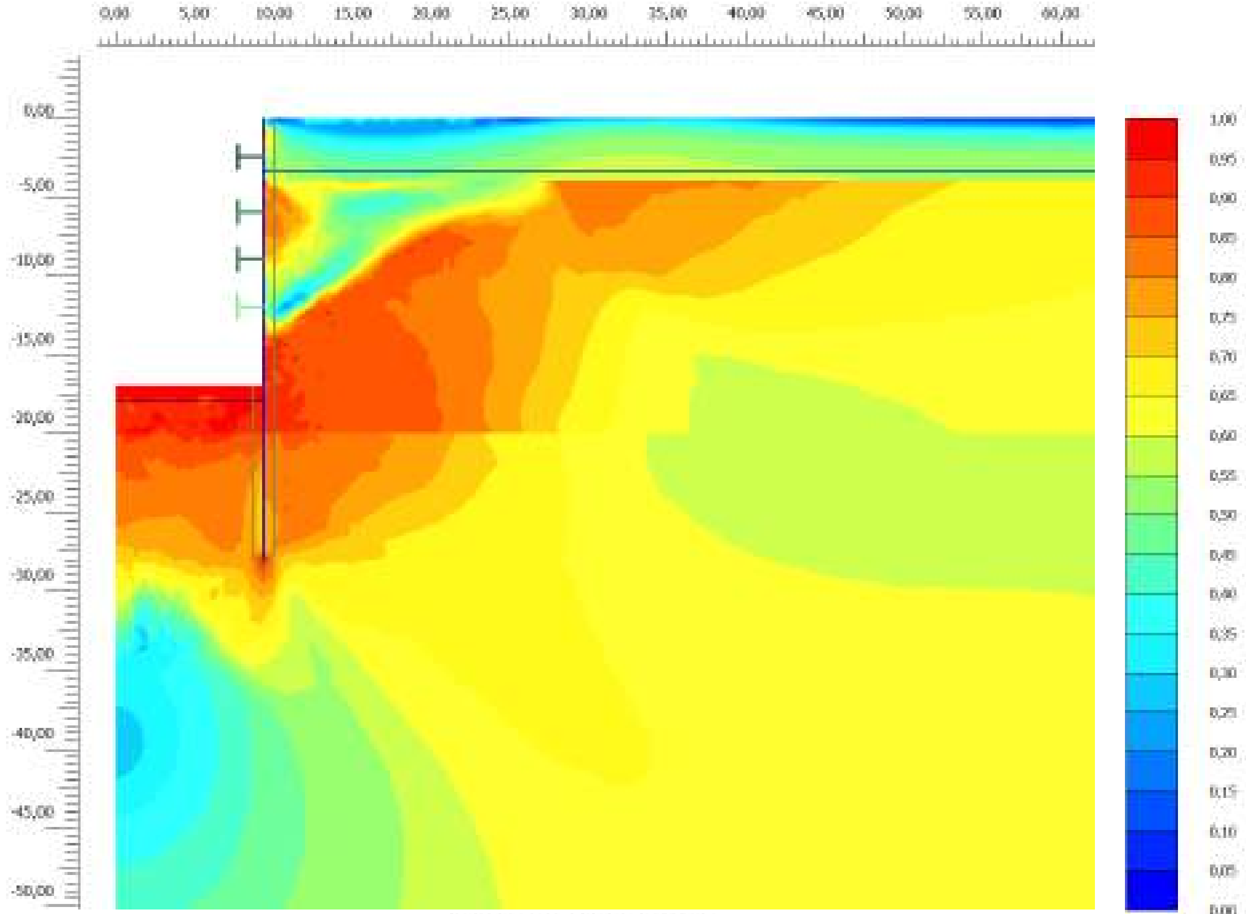
Total displacements u_x

Maximum value = $1,893*10^{-3}$ m (Element 20 at Node 2838)

Minimum value = -0,02540 m (Element 2961 at Node 4030)

Institute for SOIL MECHANICS and FOUNDATION ENGINEERING
COMPUTATIONAL GEOTECHNICS GROUP

BASICS OF FINITE ELEMENT METHOD

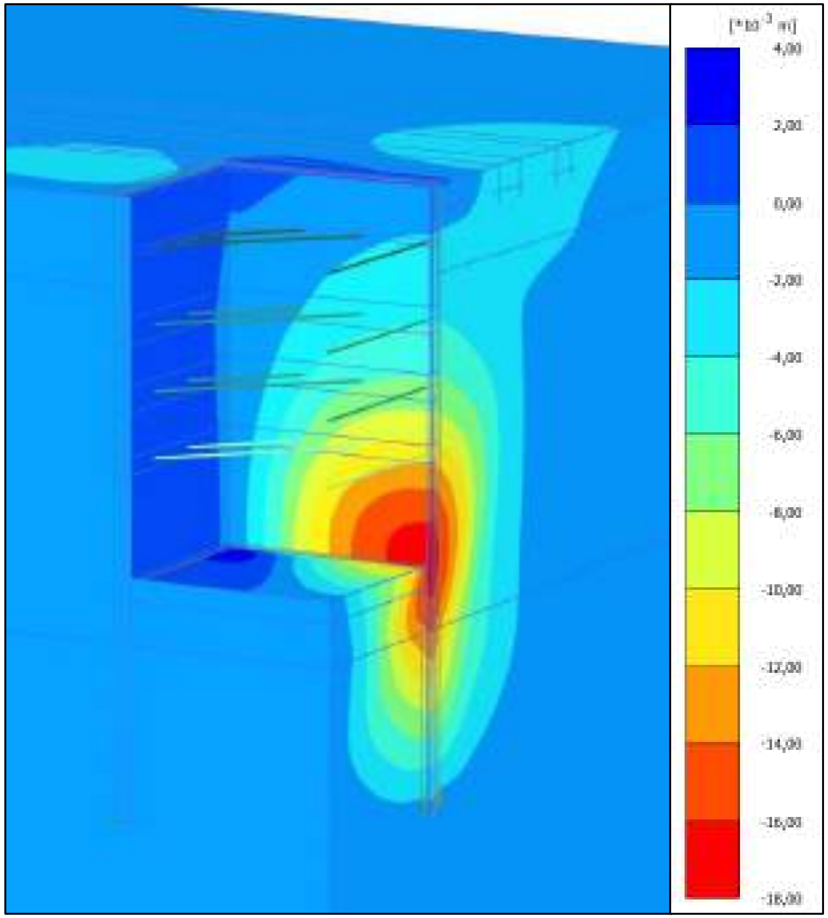


Relative shear stress τ_{rel}

Maximum value = 1,000 (Element 1845 at Node 3492)

Minimum value = 0,01677 (Element 420 at Node 10786)

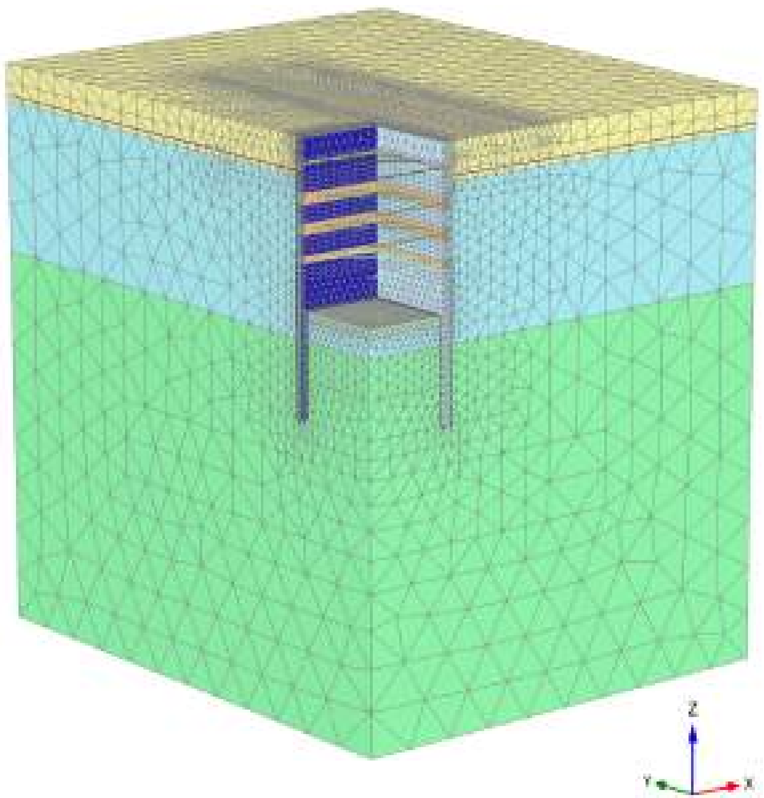
BASICS OF FINITE ELEMENT METHOD



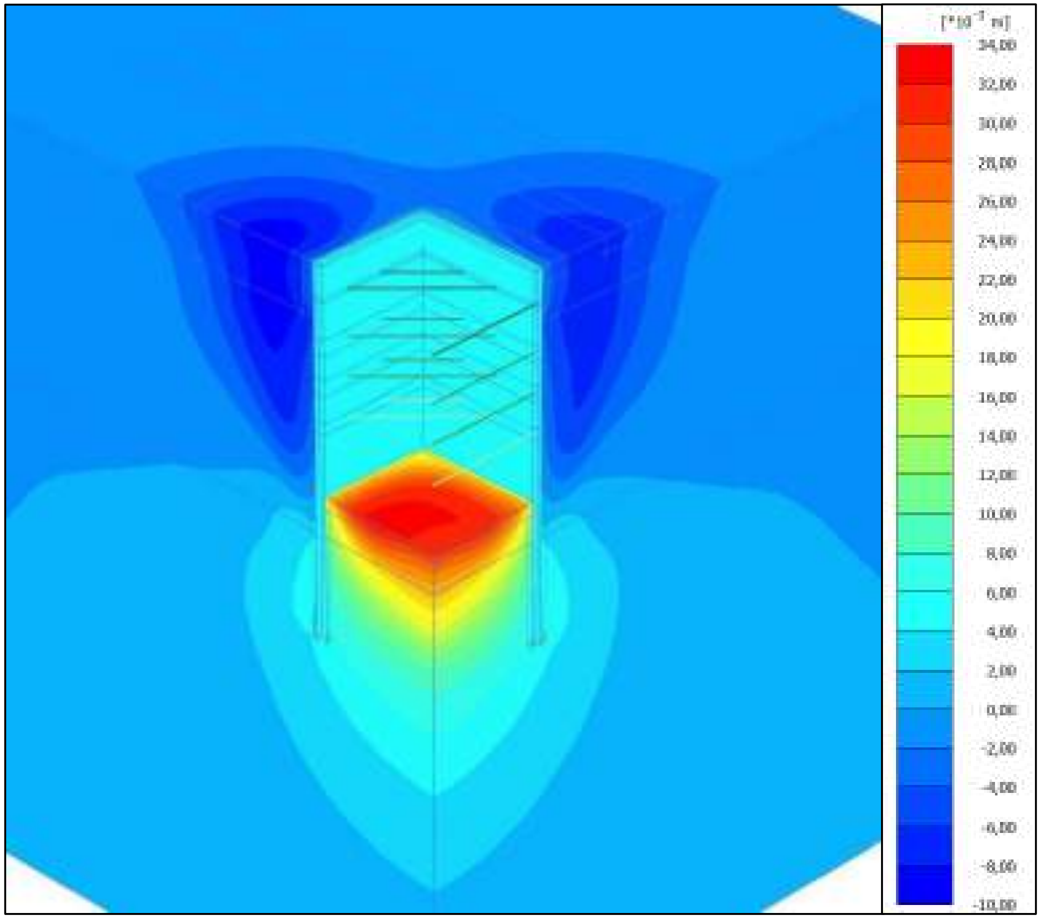
Total displacements u_z

Maximum value = $3{,}016 \cdot 10^{-3}$ m (Element 69229 at Node 190928)

Minimum value = -0,01686 m (Element 15314 at Node 181960)



BASICS OF FINITE ELEMENT METHOD

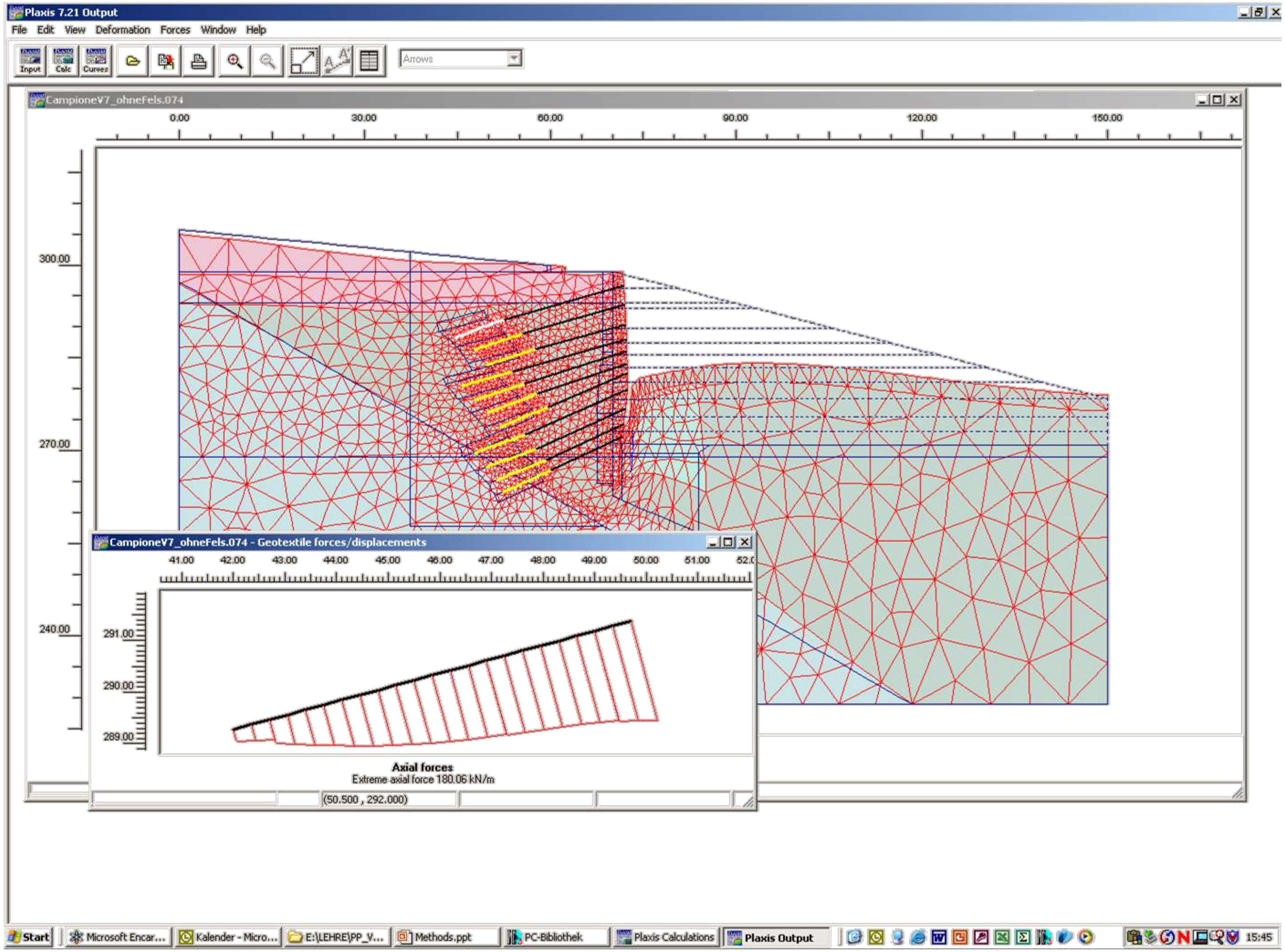


Total displacements u_z

Maximum value = 0,03251 m (Element 69148 at Node 216879)

Minimum value = $-9,067 \cdot 10^{-3}$ m (Element 116485 at Node 86409)

BASICS OF FINITE ELEMENT METHOD



CONCLUDING REMARKS

Objectives of the numerical (finite element) analysis

- Selection of design alternatives
- Quantitative predictions
- Backcalculations
- **Understanding!**
  - **Identification of critical mechanisms**
  - **Identification of key parameters**

CONCLUDING REMARKS

Advantages of numerical (finite element) analysis

- Simulation of complete construction history
- Interaction with water can be considered rigorously
- Complex geometries (2D-3D) can be modelled
- Structural elements can be introduced
- No failure mechanism needs to be postulated (it is an outcome of the analysis)

(Nearly) unavoidable uncertainties

- Ground profile
- Initial conditions (initial stresses, pore water pressure…)
- Boundary conditions (mechanical, hydraulic)
- Appropriate model for soil behaviour
- Model parameters

CONCLUDING REMARKS

Some requirements for successful numerical modelling

- Construction of an adequate conceptual model that includes the basic features of the model. *The model should be as simple as possible but not simpler.*
- Selection of an appropriate constitutive model. It depends on:
  - type of soil or rock
  - goal of the analysis
  - quality and quantity of available information
- Pay attention to patterns of behaviour and mechanisms rather than just to quantitative predictions.
- Perform sensitivity analyses. Check robustness of solution.
- Model calibration (using field results) should be a priority, especially when quantitative predictions are sought.
- Check against alternative computations if available (even if simplified).

FINAL REMARKS

1. **Geotechnical engineering is complex. It is not because you are using the FEM that it becomes simpler.**

2. **The quality of a tool is important, yet the quality of a result also (mainly) depends on the user's understanding of both the problem and the tool.**

3. **The design process involves considerably more than analysis.**

Borrowed from C. Viggiani, with thanks

S C I E N C E ▪ P A S S I O N ▪ T E C H N O L O G Y

CG02

CONCEPTS OF PLASTICITY MOHR COULOMB MODEL

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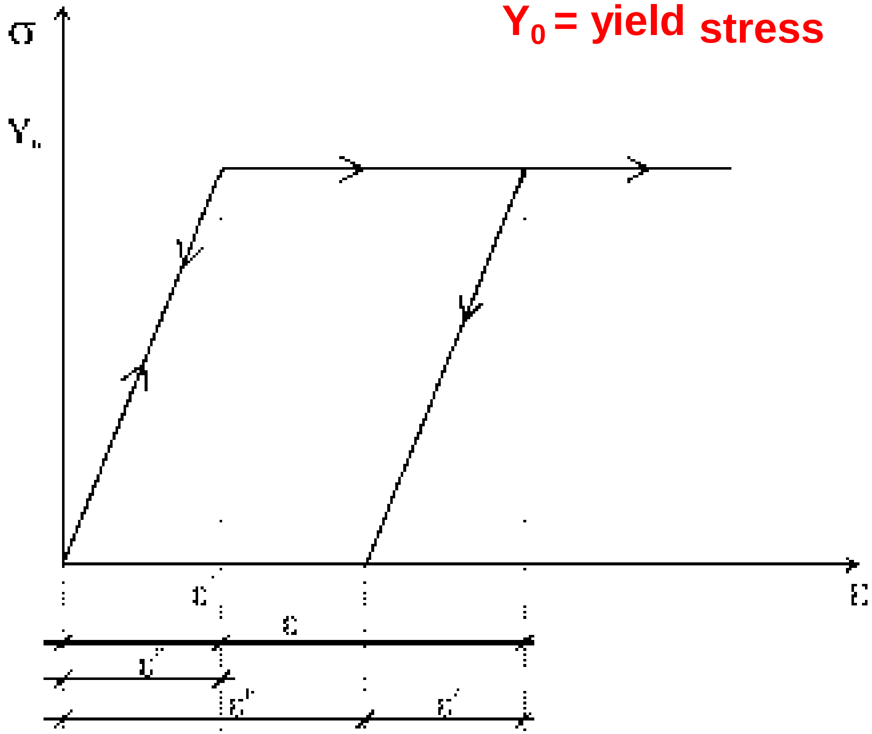
Graz University of Technology
[2)] Civil Engineering Department
National Institute of Technology (Itenas) Bandung

LINEAR ELASTIC - PERFECTLY PLASTIC

One-dimensional



Y_0 = yield stress

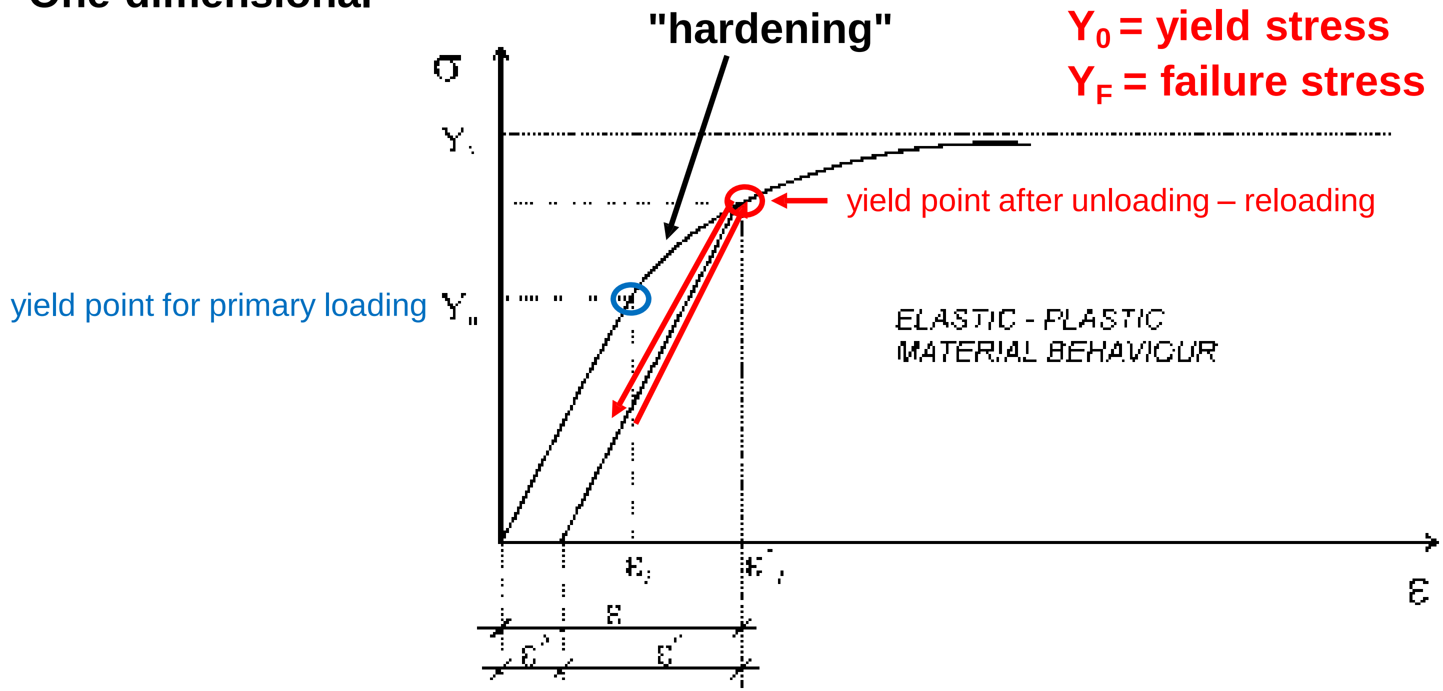
IMPORTANT: yield stress = failure stress for perfect plasticity

$$\varepsilon = \varepsilon^e + \varepsilon^p$$

General three-dimensional stress state $\{\varepsilon\} = \{\varepsilon\}^e + \{\varepsilon\}^p$

LINEAR ELASTIC - PLASTIC

One-dimensional



IMPORTANT: yield stress $\neq$ failure stress

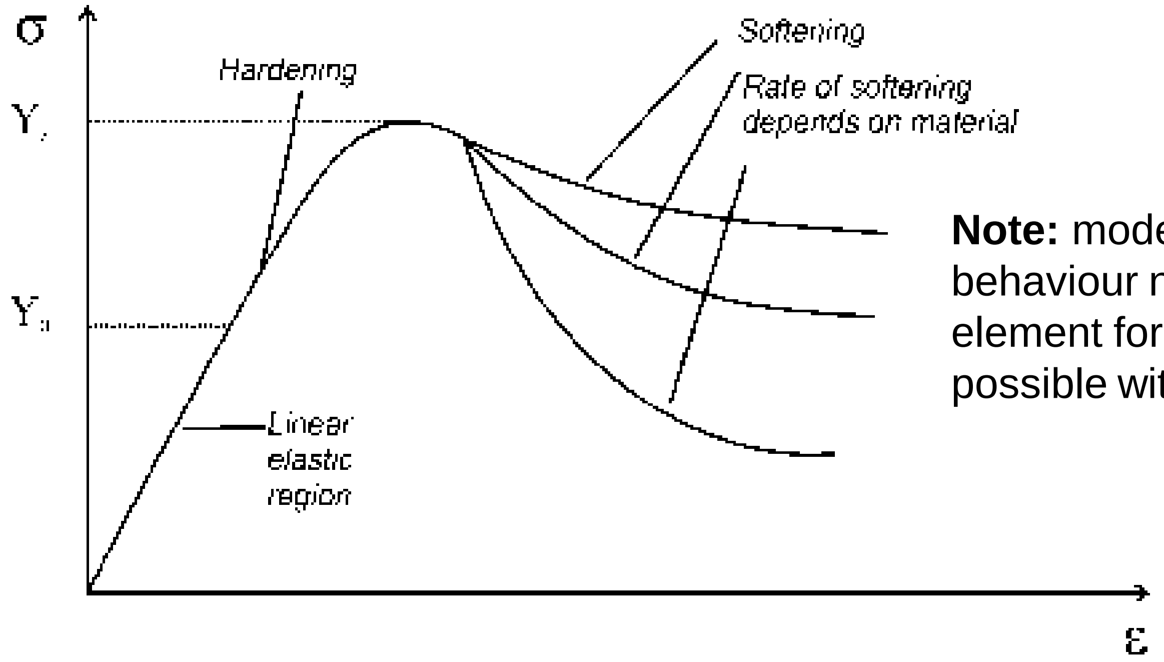
$$\varepsilon = \varepsilon^{e} + \varepsilon^{p}$$

General three-dimensional stress state

$$\{\varepsilon\} = \{\varepsilon\}^{e} + \{\varepsilon\}^{p}$$

LINEAR ELASTIC - PLASTIC WITH SOFTENING

One-dimensional



Note: modelling strain softening behaviour needs enhancement of finite element formulation > at present not possible with Plaxis

Y_0 = yield stress
Y_F = failure stress

THEORY OF PLASTICITY

For describing linear elastic - plastic material behaviour we need (for general stress states)**:**

1. Stress-strain behaviour in elastic range

2. **Yield function or failure function**
   (defines onset of plastic deformation)

3. **Flow rule**
   (defines direction of plastic strain increment)

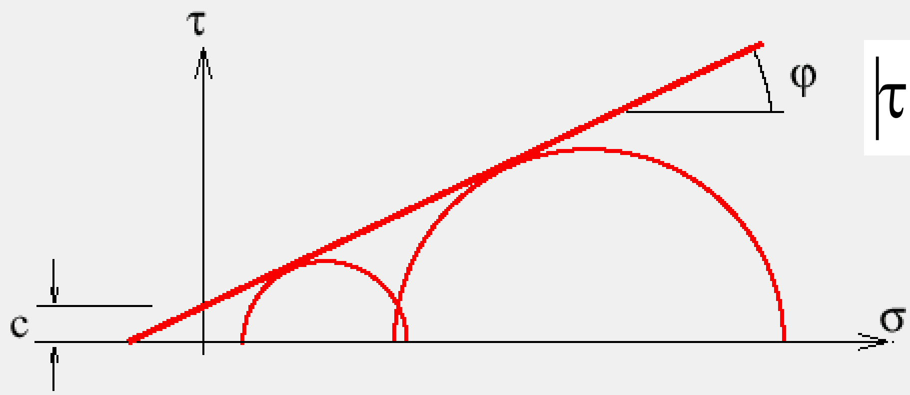
4. **Definition of strain hardening (softening)**
   (defines change of yield function with stress and/or strain)

For standard MC-model:

- Linear elasticity in elastic range
- No strain hardening/softening > perfect plasticity

MOHR COULOMB CRITERION

Mohr-Coulomb yield function



on any plane:

$$|\tau| = \sigma_n' \cdot \tan\varphi' + c'$$

yield function: $f = \frac{1}{2}(\sigma'_1 - \sigma'_3) + \frac{1}{2}(\sigma'_1 + \sigma'_3)\sin\varphi' - c'\cos\varphi'$

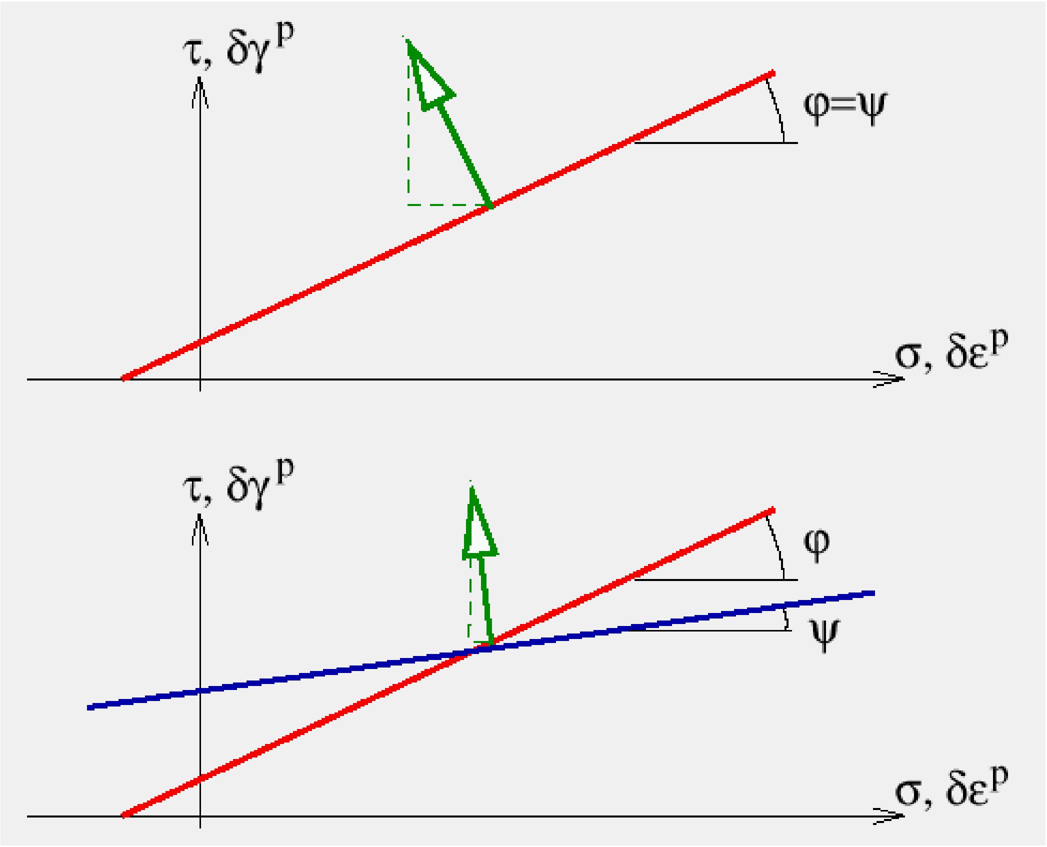
σ_1' and σ_3' : major and minor principal stresses

sign convention

positive: tensile stress, elongation, volume increase

negative: compressive stress, compression, volume decrease

MOHR COULOMB MODEL - PLASTIC POTENTIAL



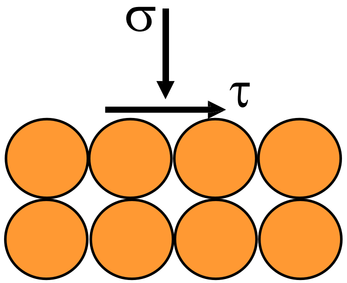
$$f = \tfrac{1}{2}(\sigma'_1 - \sigma'_3) \quad \tfrac{1}{2}(\sigma'_1 + \sigma'_3)\sin\varphi' - c'\cos\varphi$$

dilatancy angle

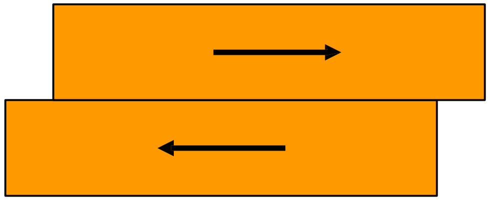
$$g = \tfrac{1}{2}(\sigma'_1 - \sigma'_3) + \tfrac{1}{2}(\sigma'_1 + \sigma'_3)\sin\psi' + \text{const.}$$

DILATANCY

Model | **Mechanism**

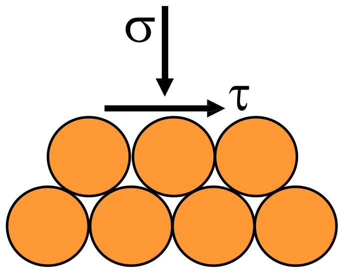


loose soils are non-dilatant or contractant

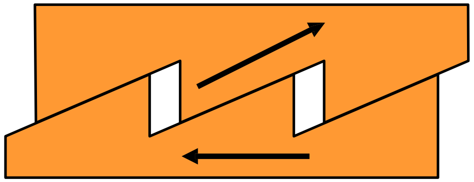


loose

ψ **= 0**



dense soils are dilatant



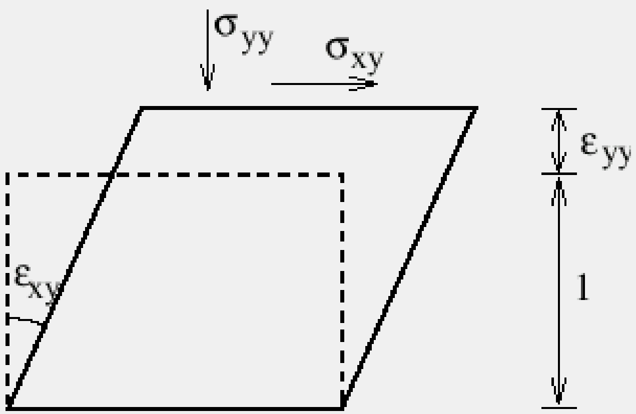
dense

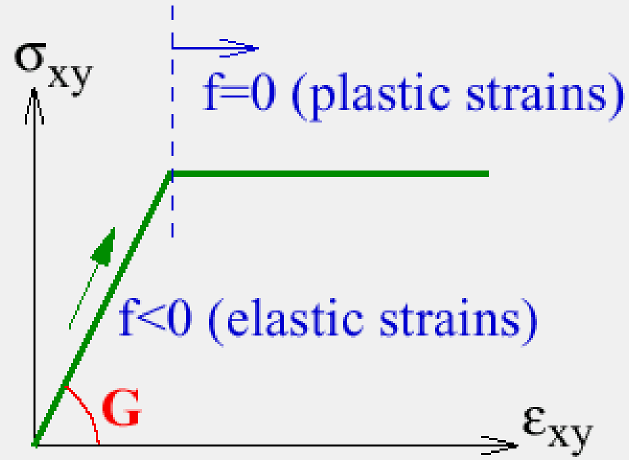
$\psi \neq$ **0, but <** φ‘

strength = friction + dilatancy

DILATANCY

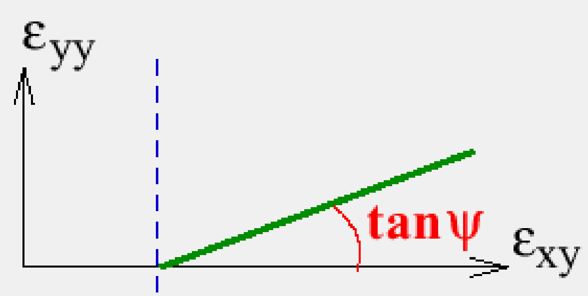
Simple shear test (drained)





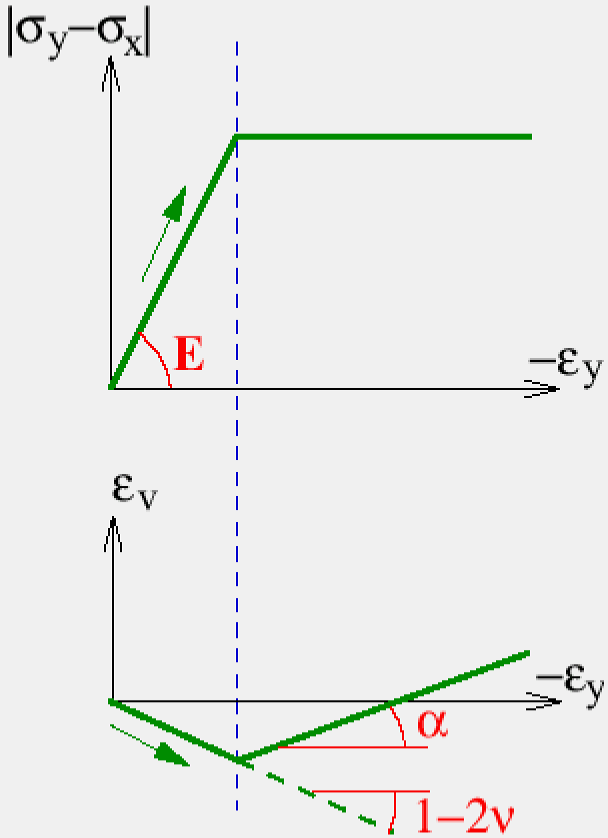
$$G = \frac{E}{2(1+\nu)}$$

$$\tan\psi = \frac{\Delta\varepsilon_{xy}}{\Delta\varepsilon_{yy}}$$



DILATANCY

Triaxial test (drained)



$$E = \frac{\Delta|\sigma_y - \sigma_x|}{\Delta|\varepsilon_y|} = 2G(1+\nu)$$

$$\varepsilon_v = 2\varepsilon_x + \varepsilon_y$$

$$\tan\alpha = \frac{2\sin\psi}{1-\sin\psi}$$

MOHR COULOMB MODEL - PARAMETERS

E	Young's modulus	[kN/m^2]
ν	Poisson's ratio	[-]
c'	(effective) cohesion	[kN/m^2]
φ'	(effective) friction angle	[^{o}]
ψ	dilatancy angle	[^{o}]

MOHR COULOMB MODEL - SUMMARY

- **Simple elastic perfectly-plastic model**
- **Suitable for some practical applications**
  (not for deep excavations and tunnels)
- **Limited number and clear parameters**
- **Good representation of failure behaviour (drained)**
- **Dilatancy can be considered**

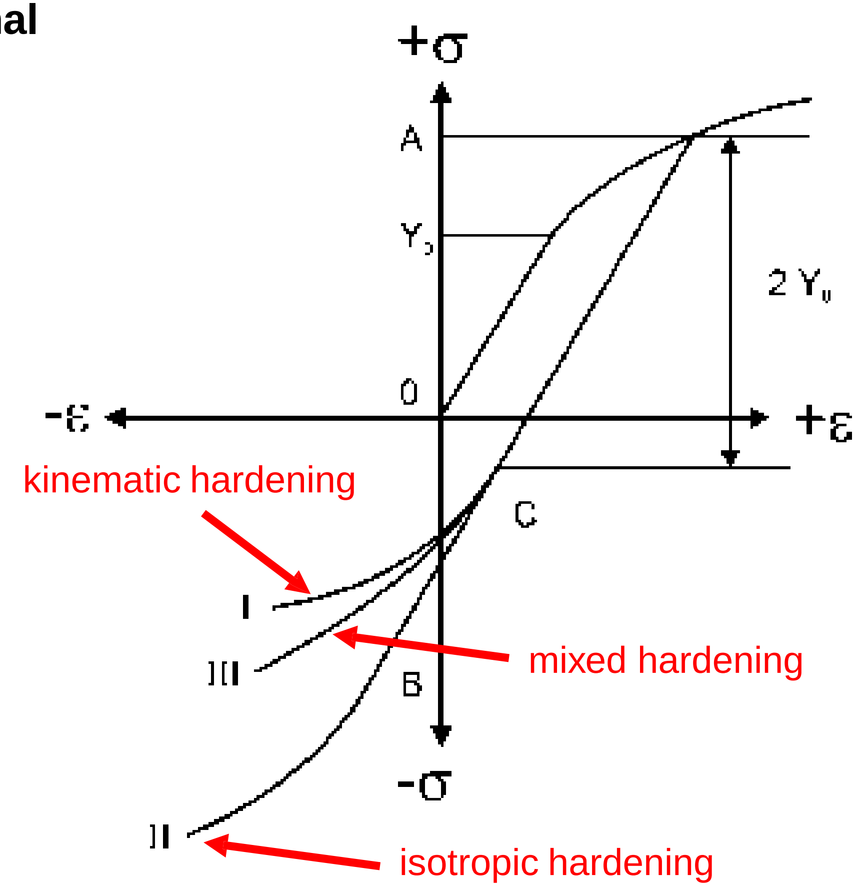
MOHR COULOMB MODEL - SUMMARY

- **Isotropic behaviour**
- **Linear elastic behaviour until failure**
- **No stress-dependent stiffness**
- **No distinction between primary loading and unloading / reloading**
- **Constant dilatancy (non-associated flow)**

  (for associated flow dilatancy would be significantly overpredicted)
- **Undrained behaviour not realistic (in particular for soft soils)**
- **No anisotropy, no time-dependency (creep)**

STRAIN HARDENING

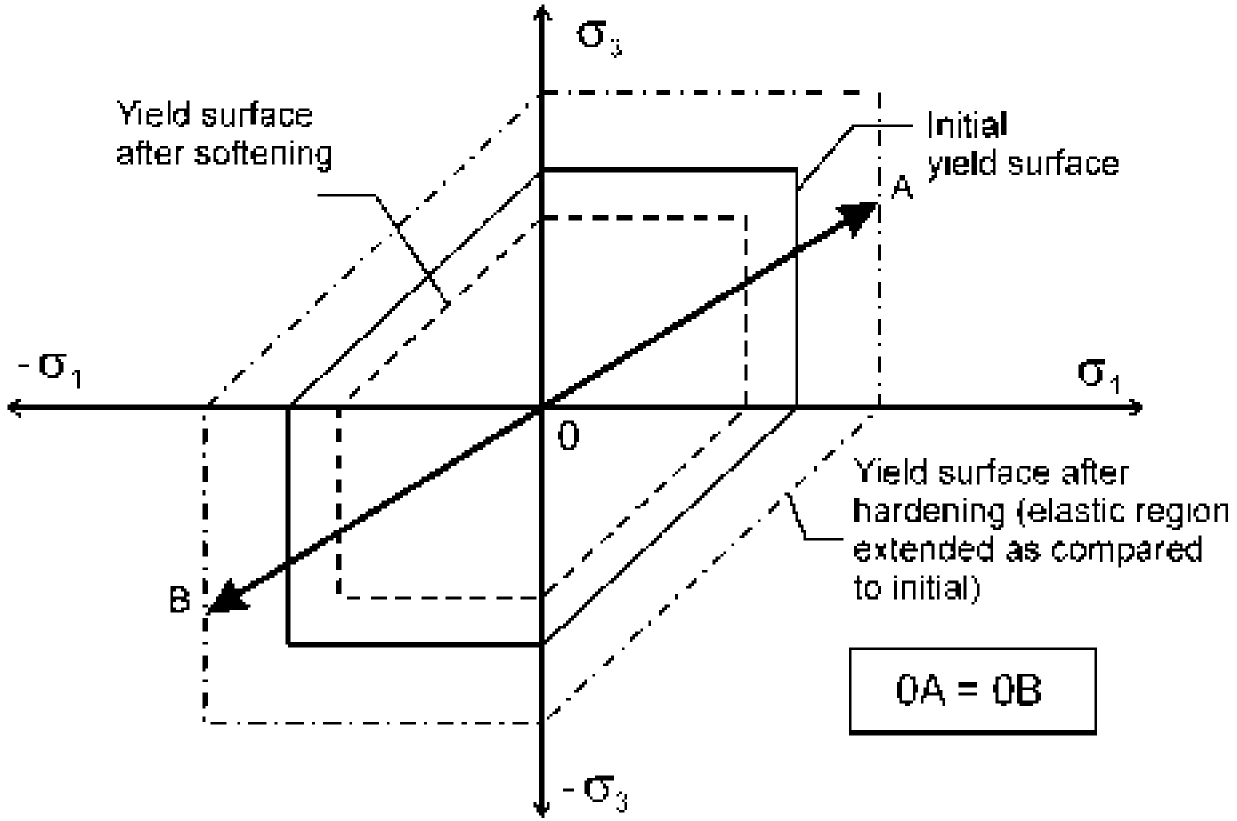
One-dimensional



$f_{(\{\sigma\},\{\varepsilon\}^p)} = 0$ more general $f_{(\{\sigma\},h)} = 0$ with $h = f_{(\{\varepsilon\}^P)}$

STRAIN HARDENING (FOR ADVANCED MODELS)

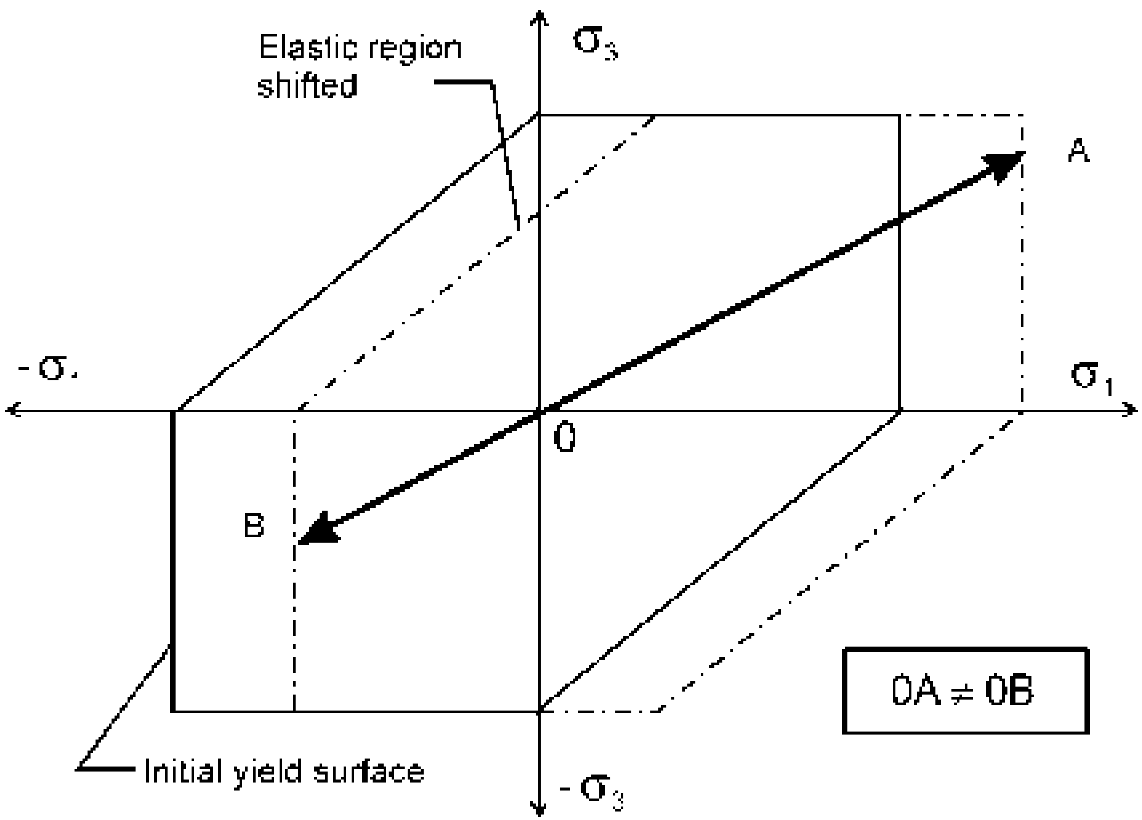
Two-dimensional



Schematic representation of isotropic hardening (all models in Plaxis at present)

STRAIN HARDENING (FOR ADVANCED MODELS)

Two-dimensional



Schematic representation of kinematic hardening

CG03 Exercise 1-Modelling of a Strip Foundation

William W.L CHEANG PhD MSc PGDip BEng (Hons)

Contents

This is CG3 (Exercise 1)

1. This exercise is related to CG 1 (Concepts of Plasticity and Mohr Coulomb Model)
2. We will use an Elasto Perfectly Plastic Model with Mohr-Coulomb failure criterion.
3. We will model a simple strip foundation (reason being, we are familiar with the calcaulation of Ultimate Bearing Capacity). This boundary value problem is related to ground loading and we are basically doing an FEM validating exercise.
4. We will look at the Long Term (Drained) condition in this first part of the exercise (1A).
5. Additional exercise (1B), we will simulate Short Term condition (Undrained).

The problem

One of the simplest forms of a foundation is the shallow foundation. In this exercise we will model such a shallow foundation with a width of 2 meters and a length that is sufficiently long in order to assume the model to be a plane strain model. The foundation is put on top of a 4m thick clay layer. The clay layer has a saturated weight of 18 kN/m^3 and an angle of internal friction of 20^0.

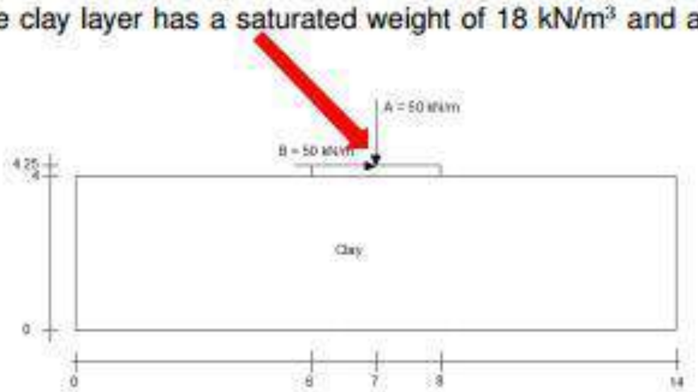


Figure 1: Geometry of the shallow foundation.

The problem (continuation..)

The foundation carries a small building that is being modelled with a vertical point force. Additionally a horizontal point force is introduced in order to simulate any horizontal loads acting on the building, for instance wind loads. Taking into account that in future additional floors may be added to the building the maximum vertical load (failure load) is assessed. For the determination of the failure load of a strip footing analytical solutions are available from for instance Vesic, Brinch Hansen and Meyerhof:

$$\frac{Q_f}{B} = c * N_c + \tfrac{1}{2}\gamma' B * N_\gamma$$
$$N_q = e^{\pi \tan\varphi'} \tan^2(45 + \tfrac{1}{2}\varphi')$$
$$N_c = (N_q - 1)\cot\varphi'$$
$$N_\gamma = \begin{cases} 2(N_q+1)\tan\varphi' & (Vesic) \\ 1.5(N_q-1)\tan\varphi' & (Brinch\ Hansen) \\ (N_q-1)\tan(1.4\varphi') & (Meyerhof) \end{cases}$$

This leads to a failure load of 117 kN/m^2 (Vesic), 98 kN/m^2 (Brinch Hansen) or 97 kN/m^2 (Meyerhof) respectively.

The problem (in the Appendix)

Bearing capacity calculation based on

Vesic, Meyehof & Brinch Hansen

1. These are the values we used to compare with our FEM model
2. Important to see the close agreement (Build confidence & understading)
3. Also, you can use (re-use) this model to check FEM code (s) and future release/updates in PLAXIS.

APPENDIX A: BEARING CAPACITY CALCULATION

Given the formula for bearing capacity of a strip footing:

$$\frac{Q_f}{B} = c \cdot N_c + \tfrac{1}{2}\gamma' B \cdot N_\gamma$$
$$N_q = e^{\pi \tan\varphi'} \tan^2(45 + \tfrac{1}{2}\varphi')$$
$$N_c = (N_q - 1)\cot\varphi'$$
$$N_\gamma = \begin{cases} 2(N_q+1)\tan\varphi' & (Vesic) \\ 1.5(N_q-1)\tan\varphi' & (Brinch\ Hansen) \\ (N_q-1)\tan(1.4\varphi') & (Meyerhof) \end{cases}$$

Filling in given soil data:

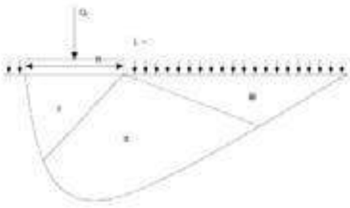
$$N_q = e^{\pi \tan(20)} \tan^2(55) = 6.4$$
$$N_c = (6.4 - 1)\cot(20) = 14.84$$
$$N_\gamma = \begin{cases} 2(6.4+1)\tan(20) = 5.39 & (Vesic) \\ 1.5(6.4-1)\tan(20) = 2.95 & (Brinch\ Hansen) \\ (6.4-1)\tan(28) = 2.97 & (Meyerhof) \end{cases}$$

The effective weight of the soil:

$$\gamma' = \gamma_w - 10\,kN/m^3 = 18 - 10 = 8\,kN/m^3$$

For a strip foundation this gives:

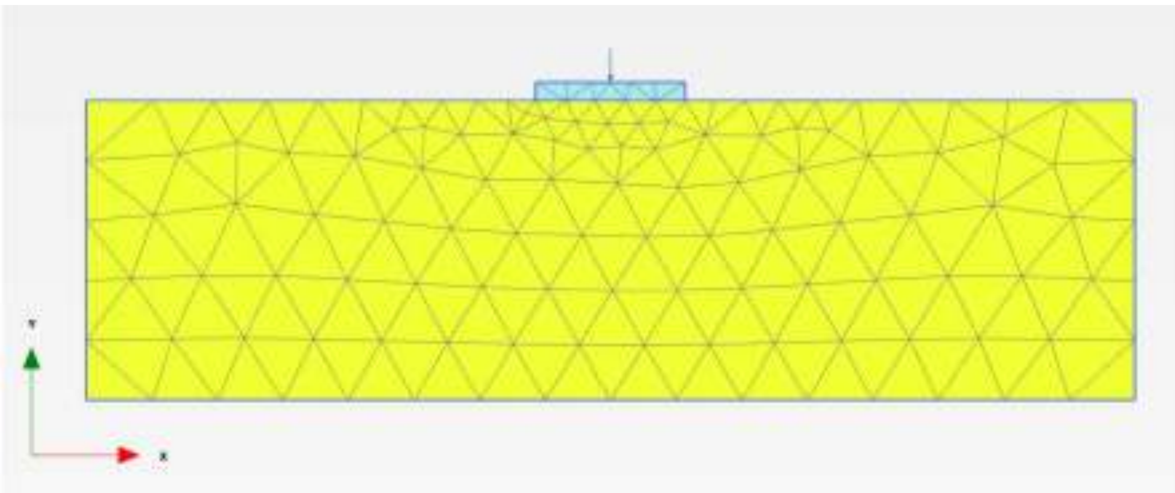
$$\frac{Q_f}{B} = c \cdot N_c + \tfrac{1}{2}\gamma' B \cdot N_\gamma = \begin{cases} 5 * 14.83 + \tfrac{1}{2} * 8 * 2 * 5.39 \approx \mathbf{117\,kN/m^2} & (Vesic) \\ 5 * 14.83 + \tfrac{1}{2} * 8 * 2 * 2.95 \approx \mathbf{98\,kN/m^2} & (Brinch\ Hansen) \\ 5 * 14.83 + \tfrac{1}{2} * 8 * 2 * 2.87 \approx \mathbf{97\,kN/m^2} & (Meyerhof) \end{cases}$$

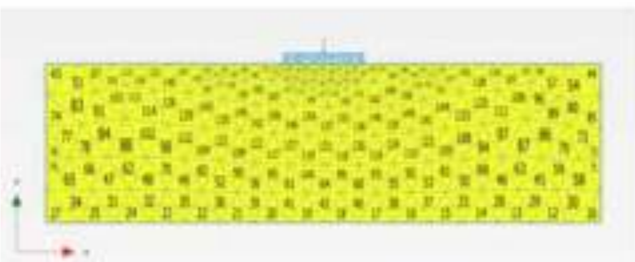


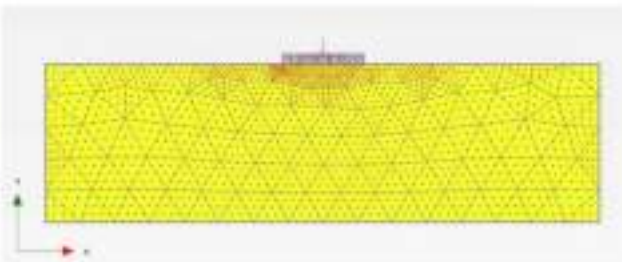
PLAXIS FEM MODEL AND SIMULATION

1. First, we model the problem in “Drained” condition (Long-term).
    a) We look at the movement of the strip foundation
    b) We’ll look at the failure mechanism
    c) We will plot calculate/find the ultimate bearing capacity
2. Additional exercise.
    a) In continuation with “Drained” model we will proceed with simulating the problem for short-term condition
    b) In Plaxis we use the “Undrained” drainage type.
    c) Can you calculate the Short-trem bearing capacity value?

The complete Model







STEPS IN PLAXIS AE

MODES: SOIL > STRUCTURES > MESH > WATER > STAGE

- Input
  - Start new project
  - Soil mode
    - Create soil layers
    - Create and assign soil material sets
  - Structures mode
    - Create footing
    - Create load
  - Mesh mode
    - Generate mesh
  - Staged construction mode
    - Determine initial situation
    - Calculation of vertical load representing the building weight
    - Calculation of vertical and horizontal load representing building weight and wind force
    - Calculation of vertical failure load.

The geometry and loading conditions

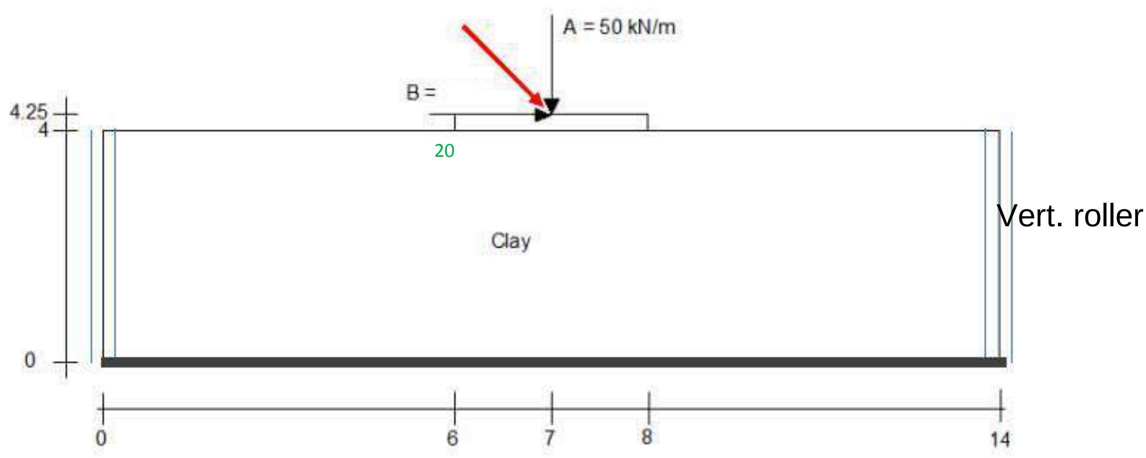
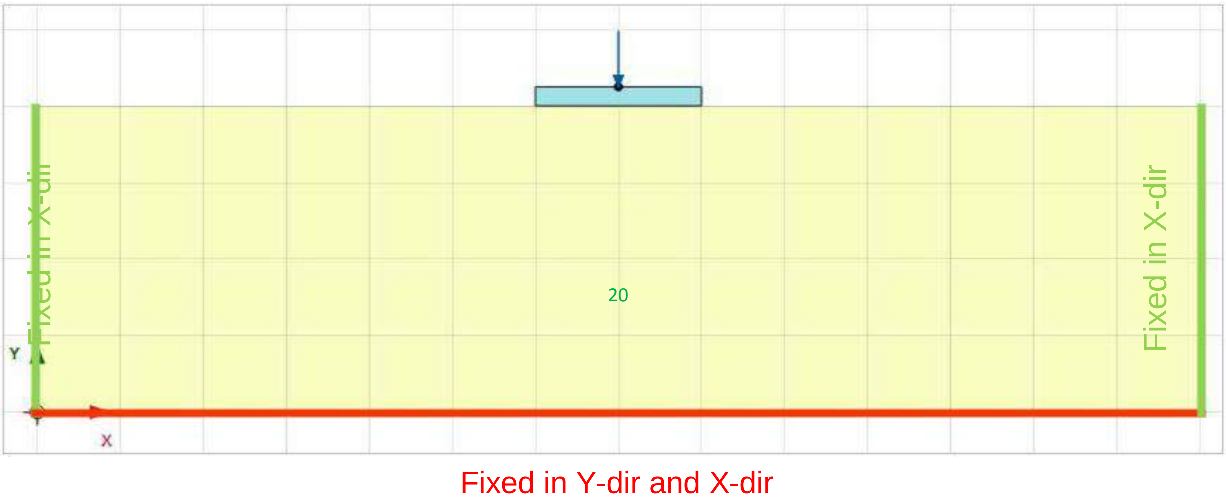
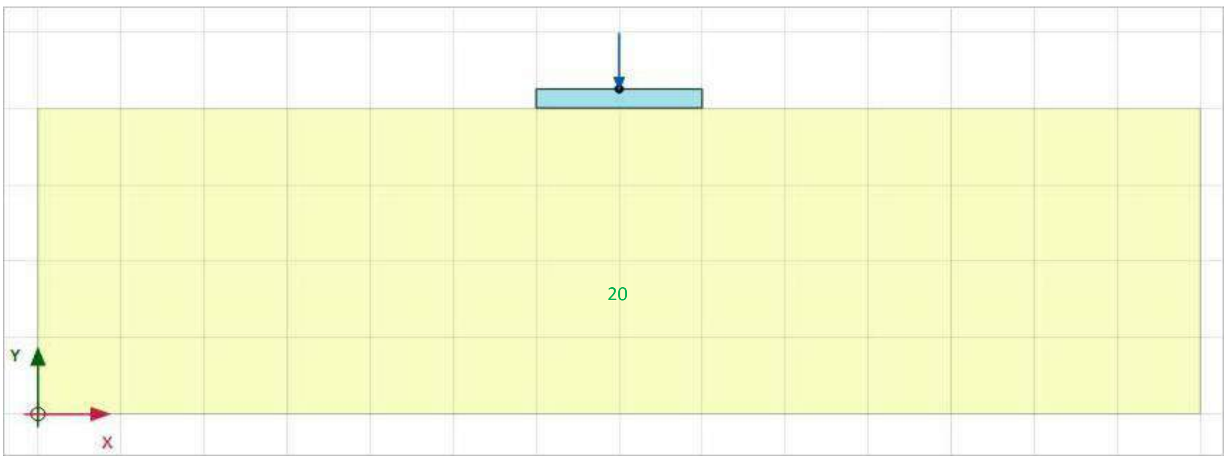


Figure 1: Geometry of the shallow foundation.

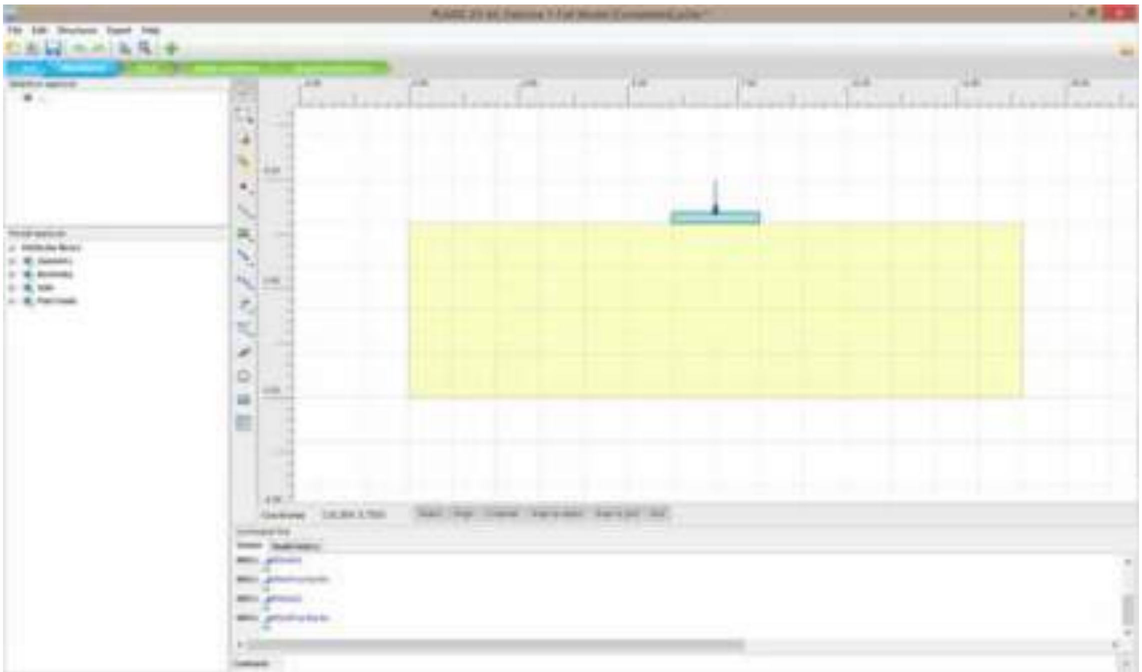
The geometry and loading conditions



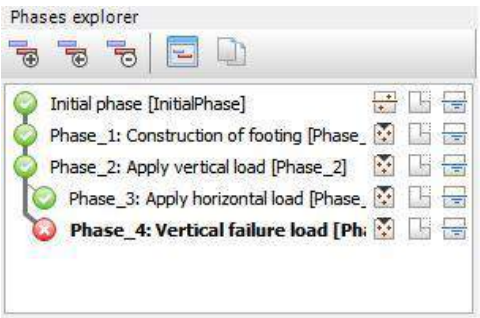
The geometry and loading conditions

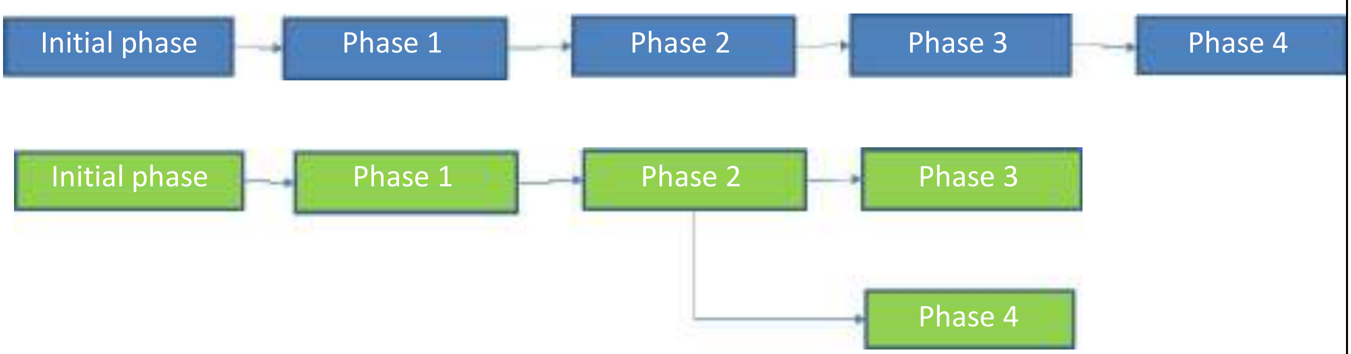


Plaxis Model: Graphical User Interface



Some information on calculation stages

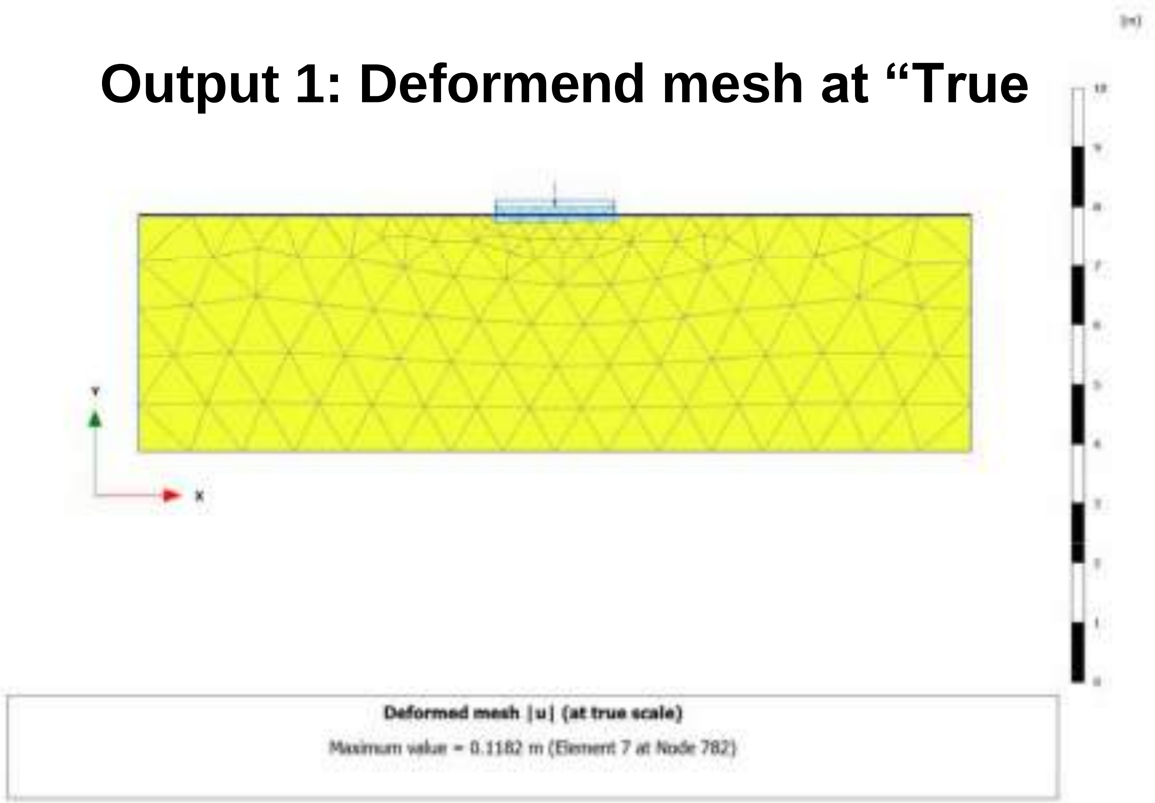




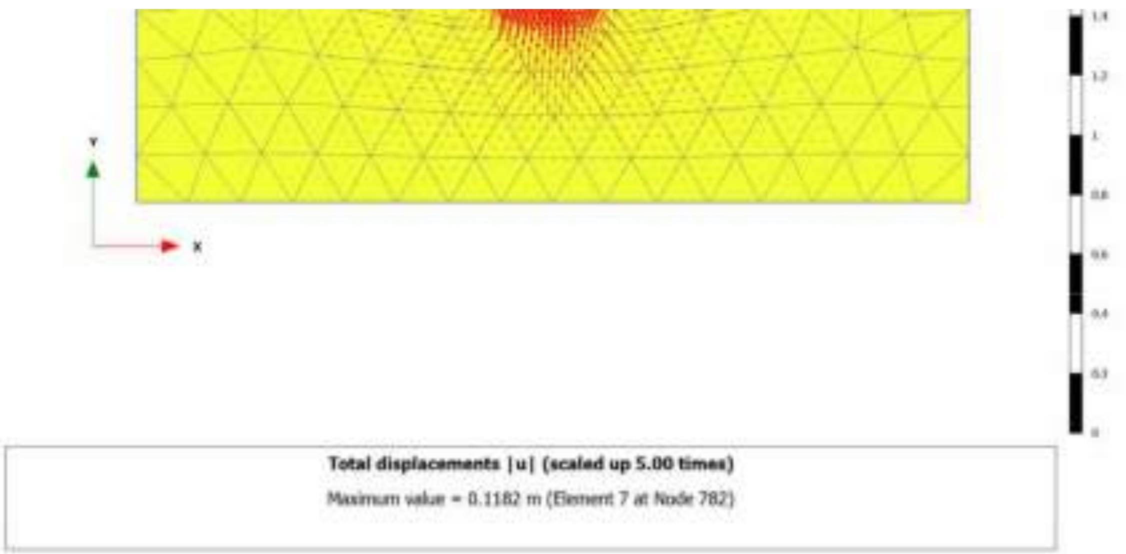
RESULTS

DRAINED MODEL

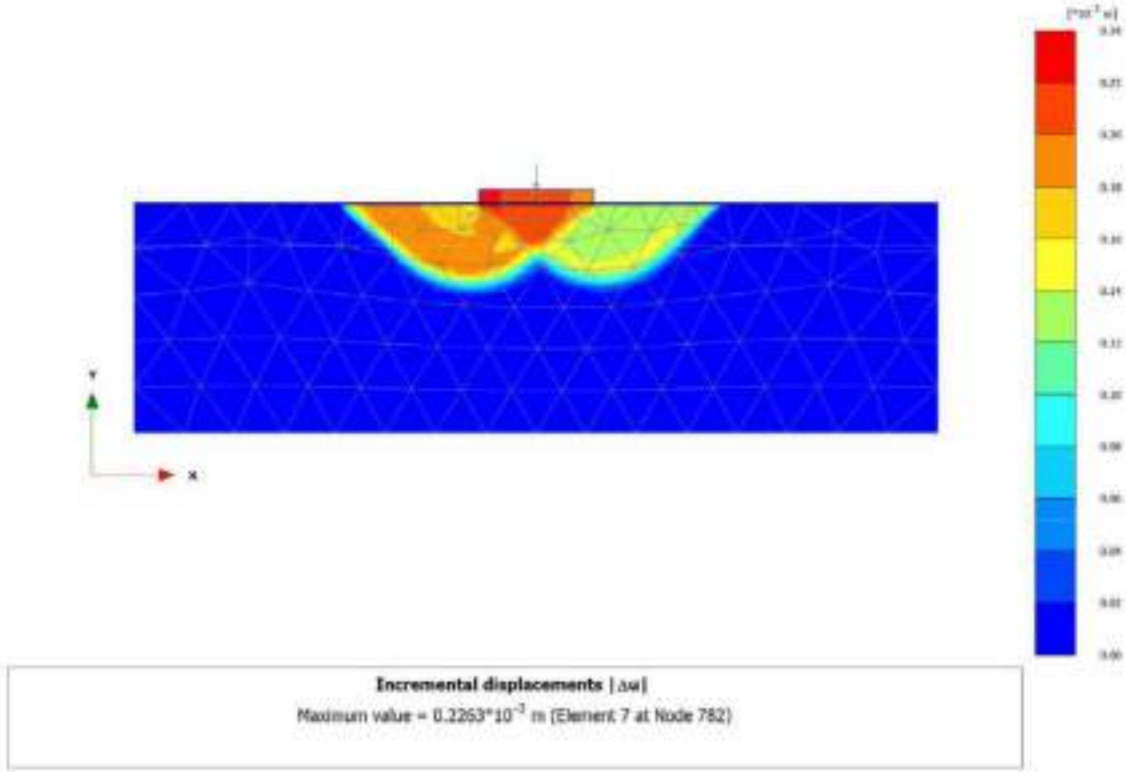
Output 1: Deformend mesh at “True



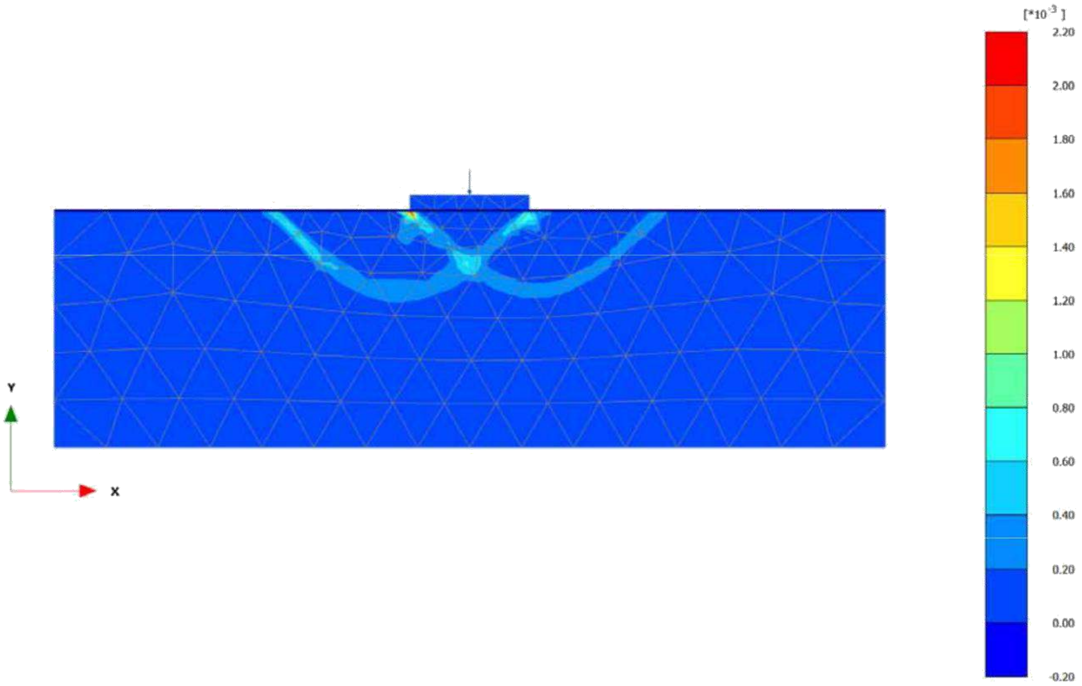
Output 2: Vectors



Output 4: Incremental Displacements



Output 4: Incremental shear strains



Incremental deviatoric strain $\Delta\gamma_s$

Maximum value = $2.045*10^{-3}$ (Element 208 at Node 931)

Minimum value = $0.6678*10^{-9}$ (Element 7 at Node 782)

Solution of Vesic, Brinch-Hansen & Meyerhof

Given the formula for bearing capacity of a strip footing:

$$\frac{Q}{B} = c \cdot N_c + \tfrac{1}{2}\gamma' B \cdot N_\gamma$$
$$N_q = e^{\pi \tan \varphi'} \tan^2(45 + \tfrac{1}{2}\varphi')$$
$$N_c = (N_q - 1)\cot \varphi'$$
$$N_\gamma = \begin{cases} 2(N_q + 1)\tan \varphi' & (Vesic) \\ 1.5(N_q - 1)\tan \varphi' & (Brinch\ Hansen) \\ (N_q - 1)\tan(1.4\varphi') & (Meyerhof) \end{cases}$$

Filling in given soil data:

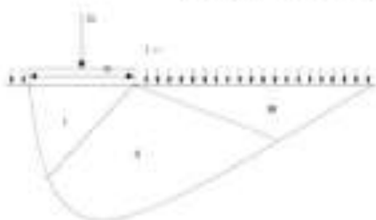
$$N_q = e^{\pi \tan(20)} \tan^2(55) = 6.4$$
$$N_c = (6.4 - 1)\cot(20) = 14.84$$
$$N_\gamma = \begin{cases} 2(6.4 + 1)\tan(20) = 5.39 & (Vesic) \\ 1.5(6.4 - 1)\tan(20) = 2.95 & (Brinch\ Hansen) \\ (6.4 - 1)\tan(28) = 2.87 & (Meyerhof) \end{cases}$$

The effective weight of the soil:

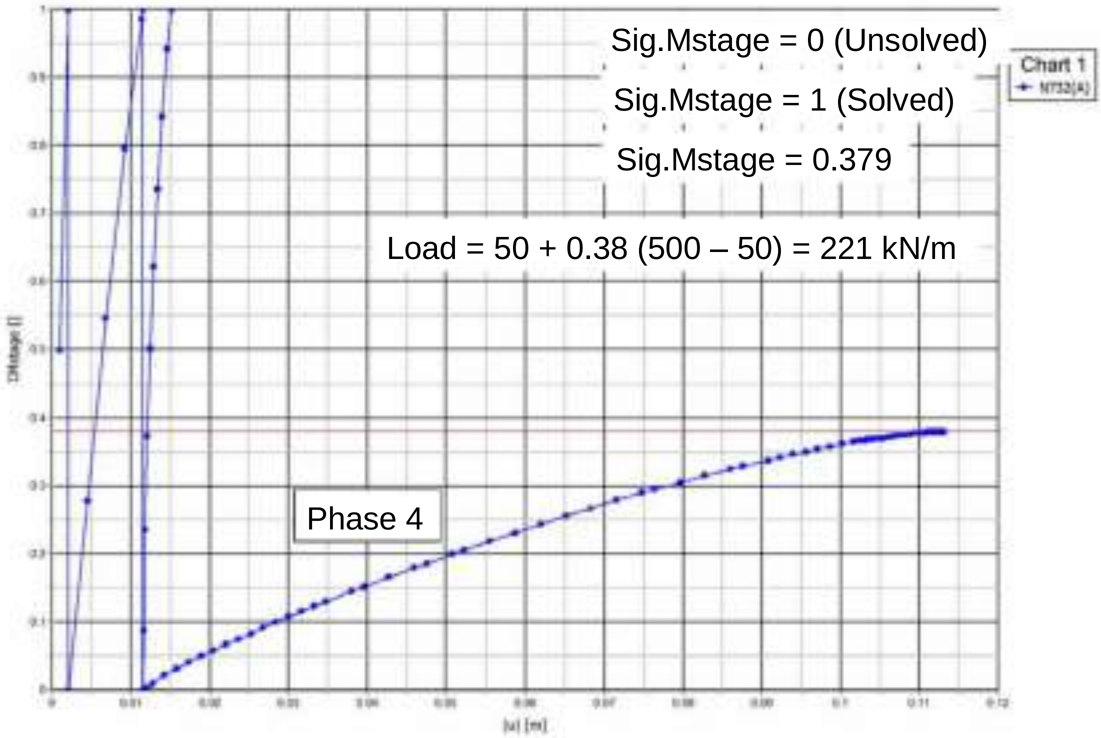
$$\gamma' = \gamma_{sat} - 10\,kN/m^3 = 18 - 10 = 8\,kN/m^3$$

For a strip foundation this gives:

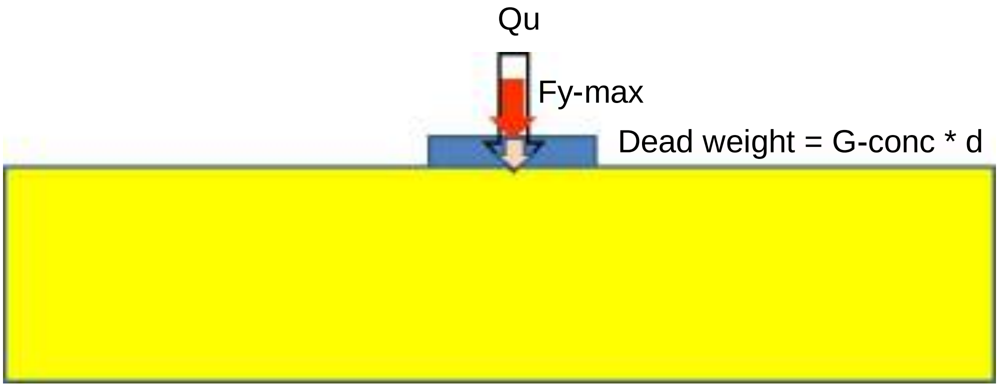
$$\frac{Q}{B} = c \cdot N_c + \tfrac{1}{2}\gamma' B \cdot N_\gamma = \begin{cases} 5 * 14.83 + \tfrac{1}{2} * 8 * 2 * 5.39 \approx 117\ \mathrm{kN/m^2} & (Vesic) \\ 5 * 14.83 + \tfrac{1}{2} * 8 * 2 * 2.95 \approx 98\ \mathrm{kN/m^2} & (Brinch\ Hansen) \\ 5 * 14.83 + \tfrac{1}{2} * 8 * 2 * 2.87 \approx 97\ \mathrm{kN/m^2} & (Meyerhof) \end{cases}$$



Output 5: Load-settlement curve



Ultimate Bearing Pressure



Qu / B = (Maximumn Load / B) + (gam.conc * d)

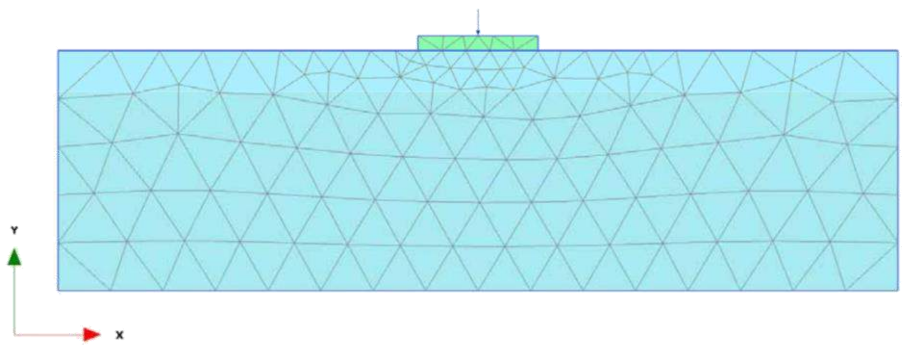
Qu / B = (221/ 2) + (24 * 0.25)

Extra: FEM, Descretisation & Element Type

Table 2: Results for the maximum load reached on a strip footing on the drained sub-soil for different 2D meshes

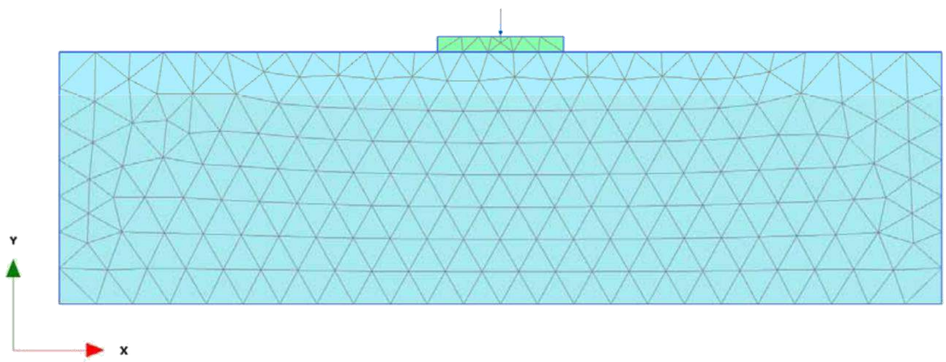
Mesh size	Element type	Nr. of elements	Max. load [kN/m]	Failure load [kN/m^2]
Medium mesh	15-noded	212	221	117
Very coarse mesh	6-noded	84	281	147
Medium mesh	6-noded	212	246	129
Very fine mesh	6-noded	626	245	129
Very coarse mesh	15-noded	84	224	118
Very fine mesh	15-noded	626	221	117
Analytical solutions of:				
- Vesic				117
- Brinch Hansen				98
- Meyerhof				97

Mesh ‘Medium’ (212 Elements)



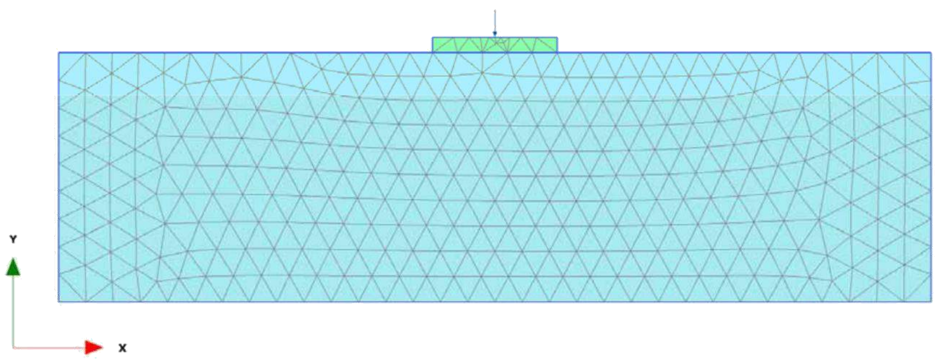
Connectivity plot

Mesh ‘Fine’ (212 Elements)



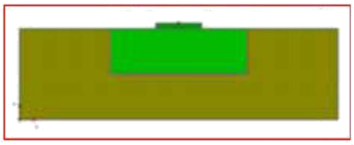
Connectivity plot

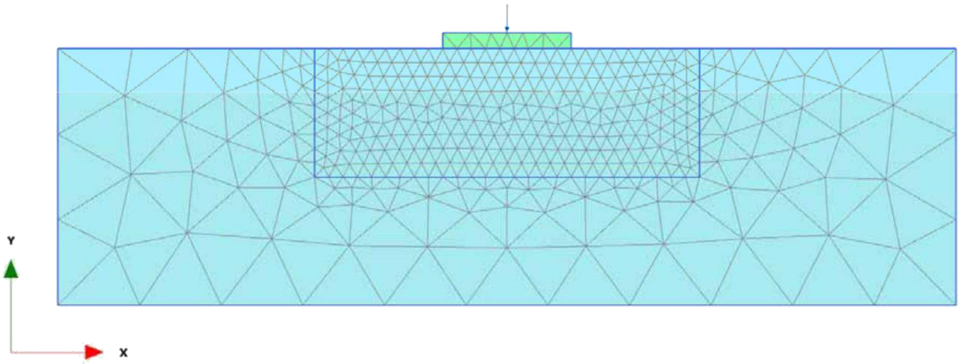
Mesh ‘Very Fine’ (626 Elements)



Connectivity plot

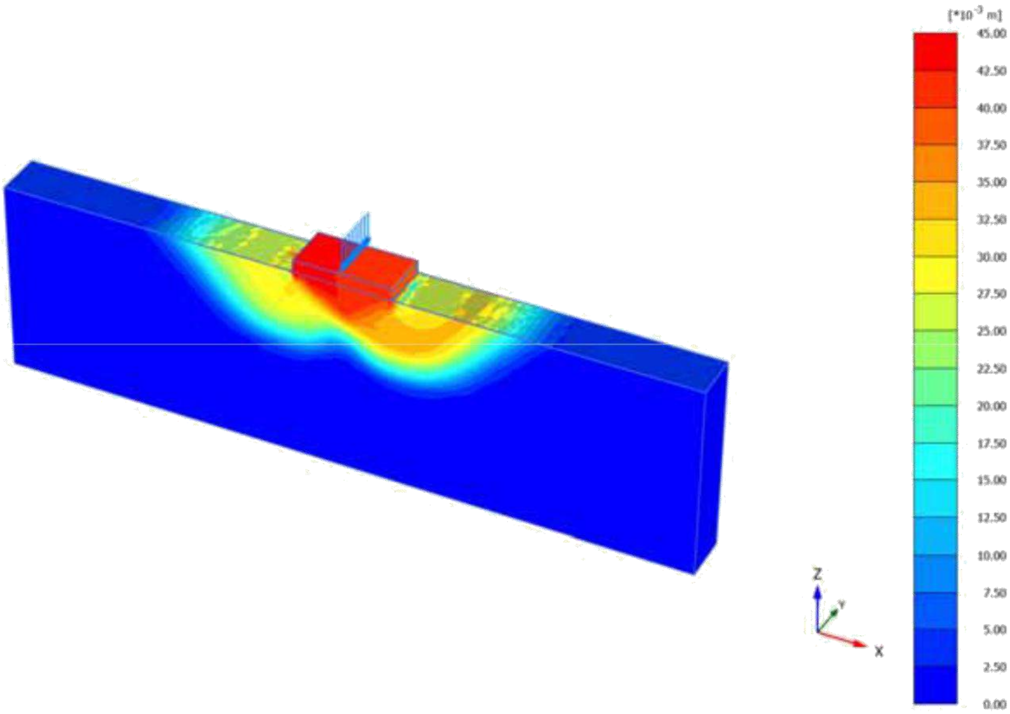
Mesh ‘Local Refinement’ (725 Elements)





Connectivity plot

Exercise 1A (3D Model)



Model: Exercise 1B – Undrained Model

When saturated soils are loaded rapidly, the soil body will behave in an undrained manner, i.e. excess pore pressures are being generated. In this exercise the special PLAXIS feature for the treatment of undrained soils is demonstrated.

SCIENCE ▪ PASSION ▪ TECHNOLOGY

CG04
CRITICAL STATE SOIL MECHANICS

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Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology
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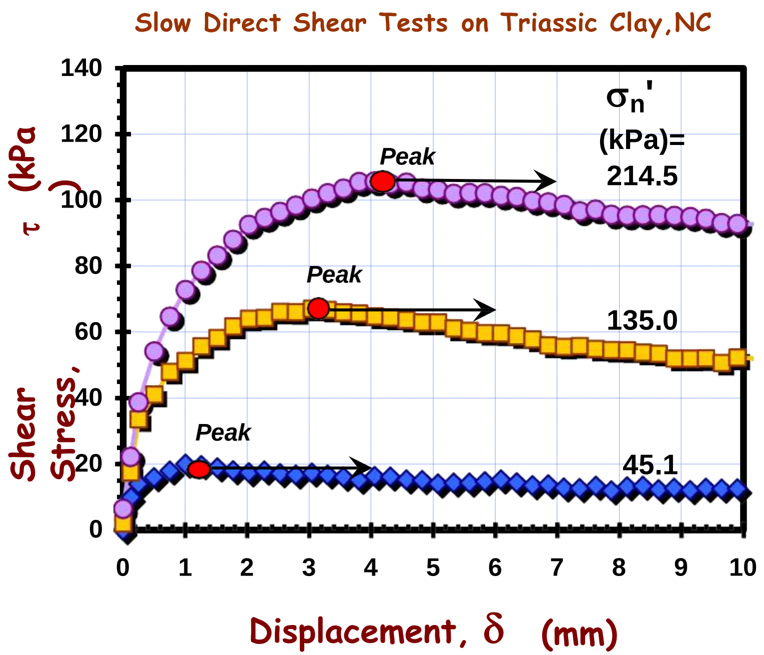
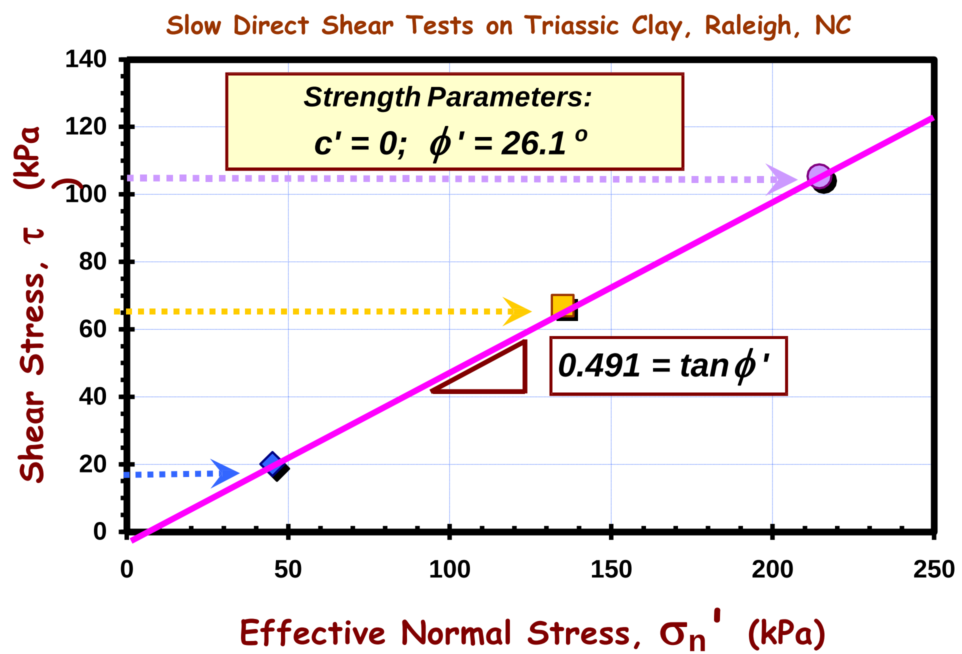
National Institute of Technology (Itenas) Bandung

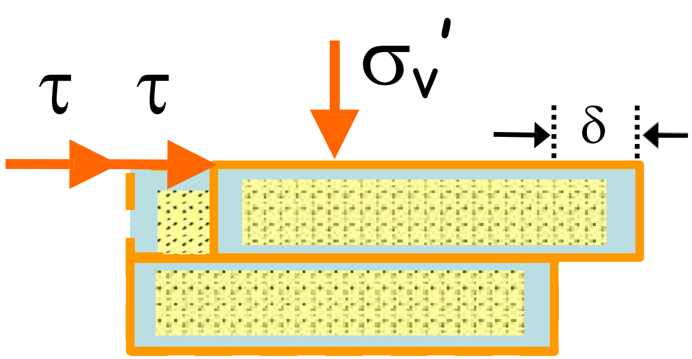
CONTENTS

- **Direct shear test**
- **Triaxial tests**
- **Drained and undrained triaxial stress paths (NC / OC)**
- **Plaxis Soft Soil model**

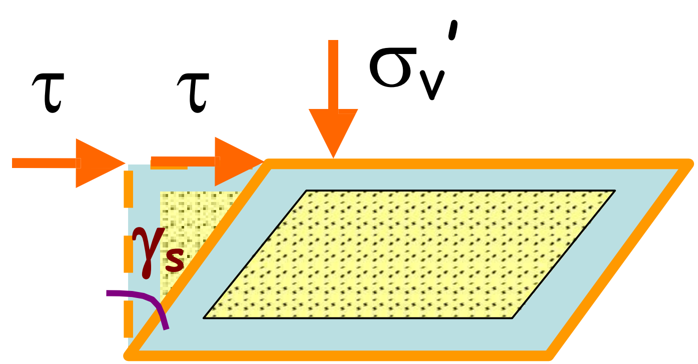
Mayne, 2006





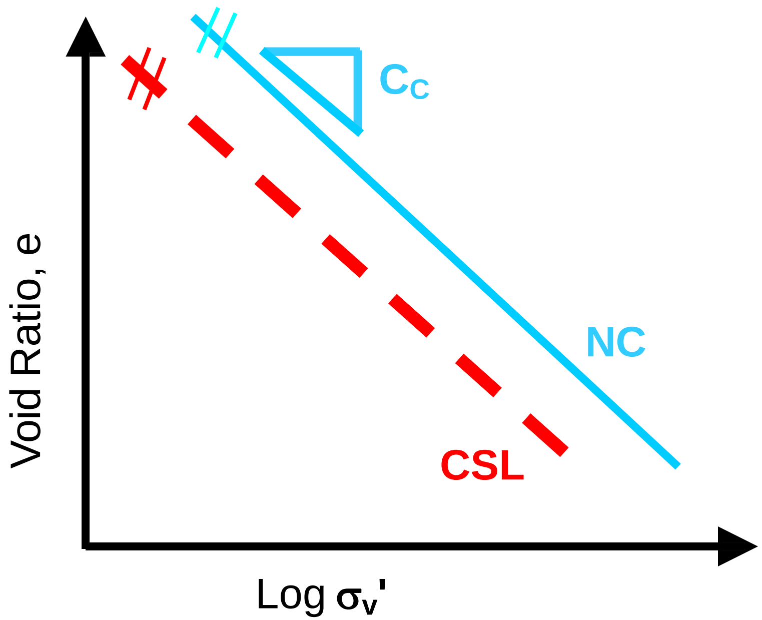


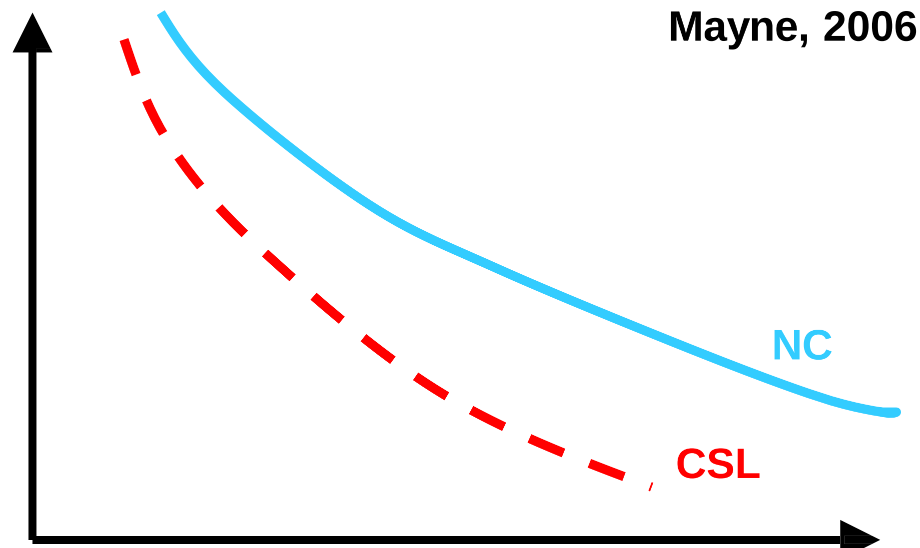
Direct Shear Box (DSB)

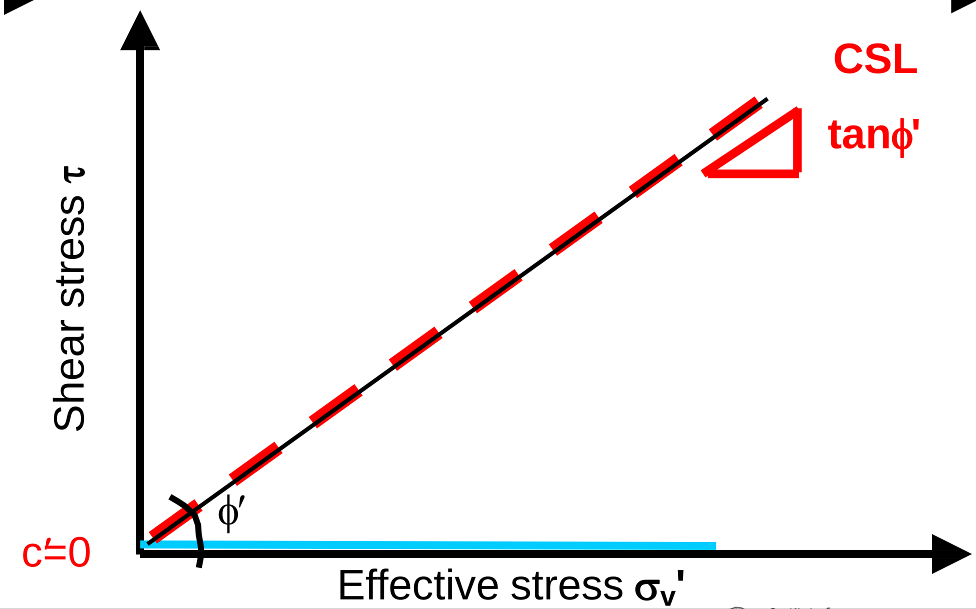


Direct Simple Shear (DSS)

Mayne, 2006



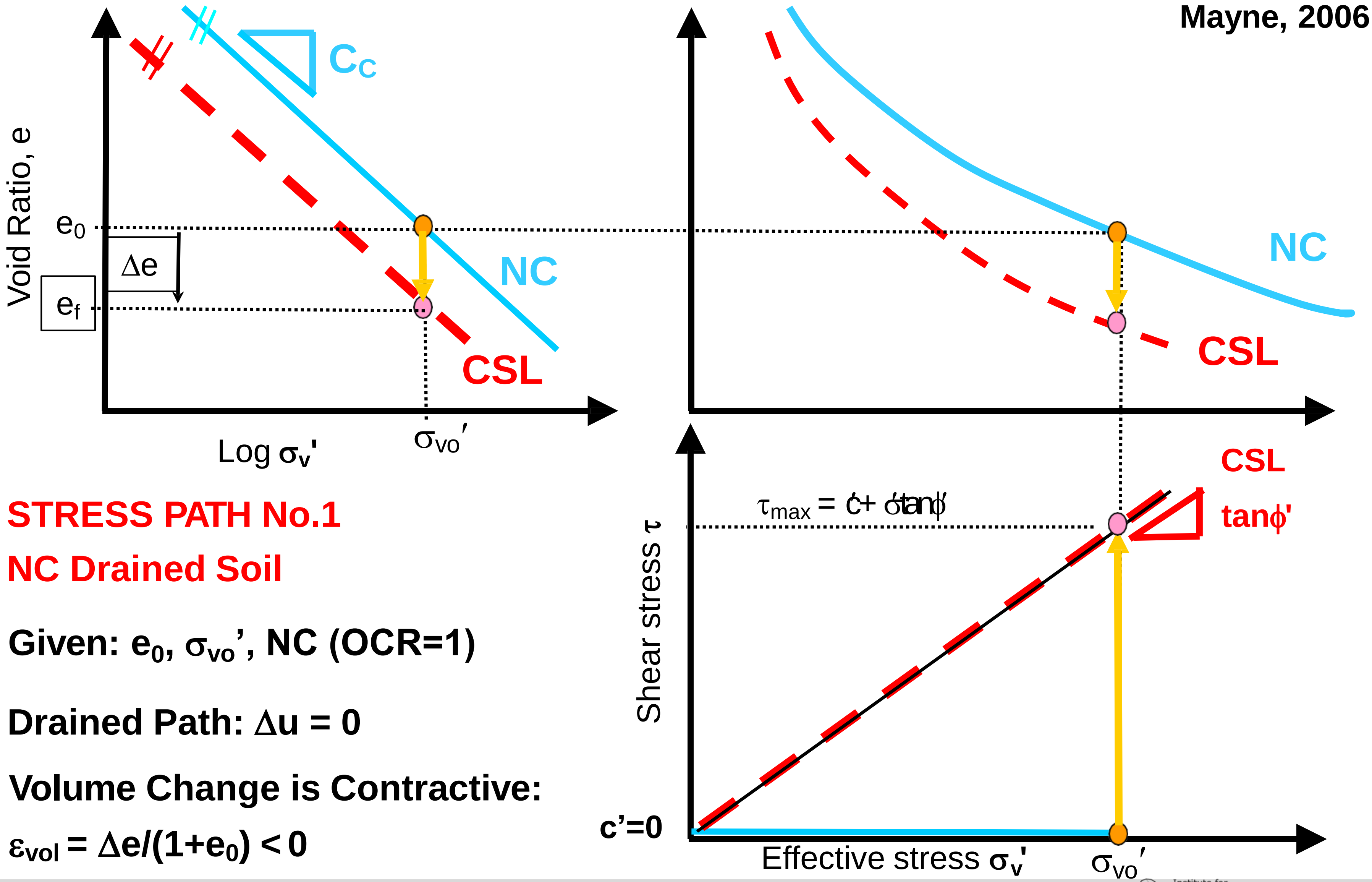




CSSM Premise:

"All stress paths fail on the critical state line (CSL)"

Mayne, 2006



STRESS PATH No.1

NC Drained Soil

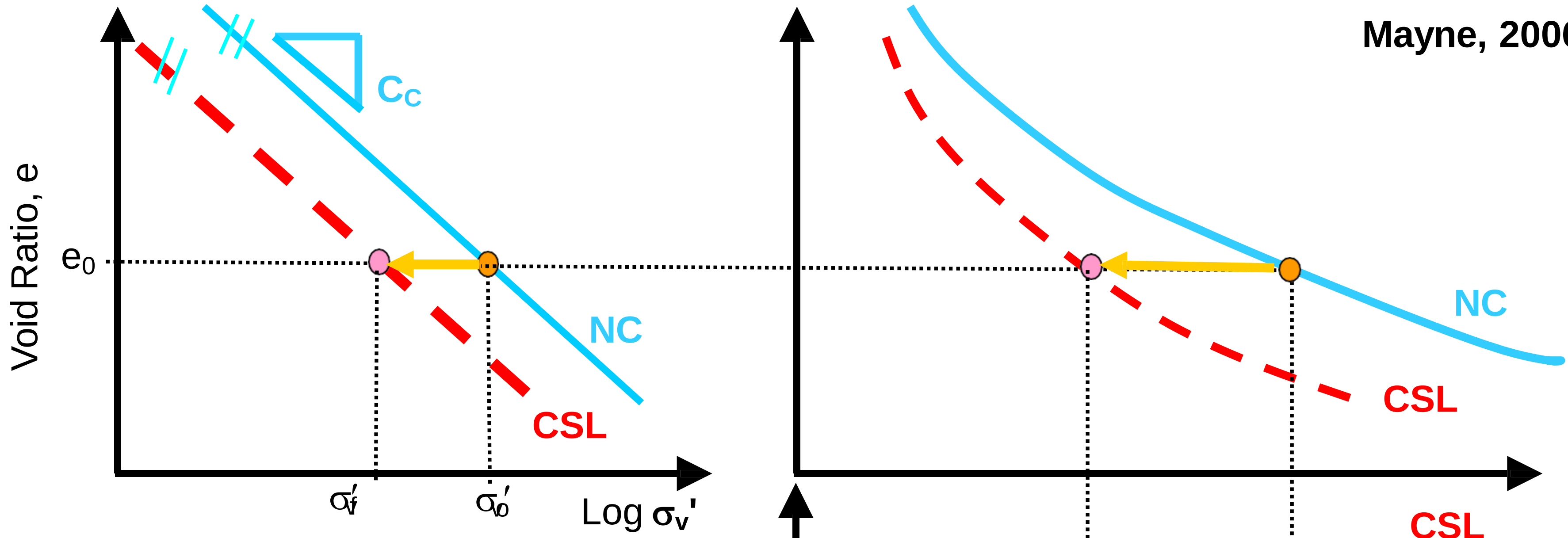
Given: e_0, σ_{vo}', NC (OCR=1)

Drained Path: $\Delta u = 0$

Volume Change is Contractive:

$\varepsilon_{vol} = \Delta e/(1+e_0) < 0$

Mayne, 2006



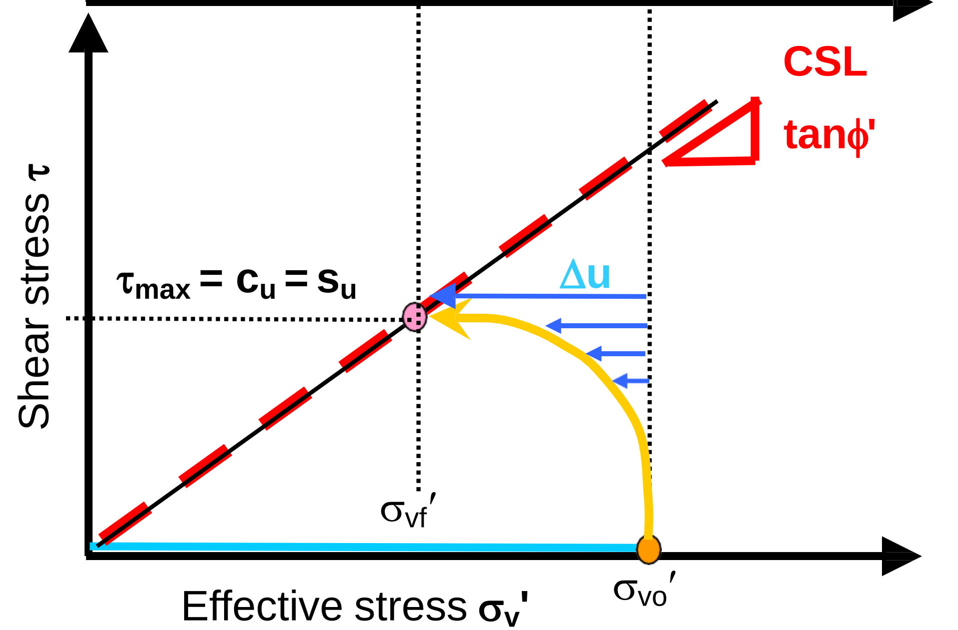
STRESS PATH No.2

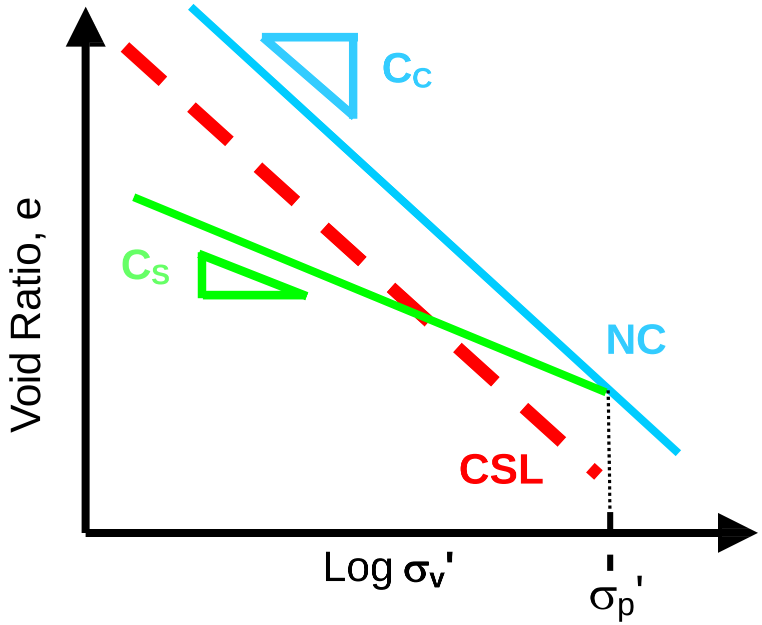
NC Undrained Soil

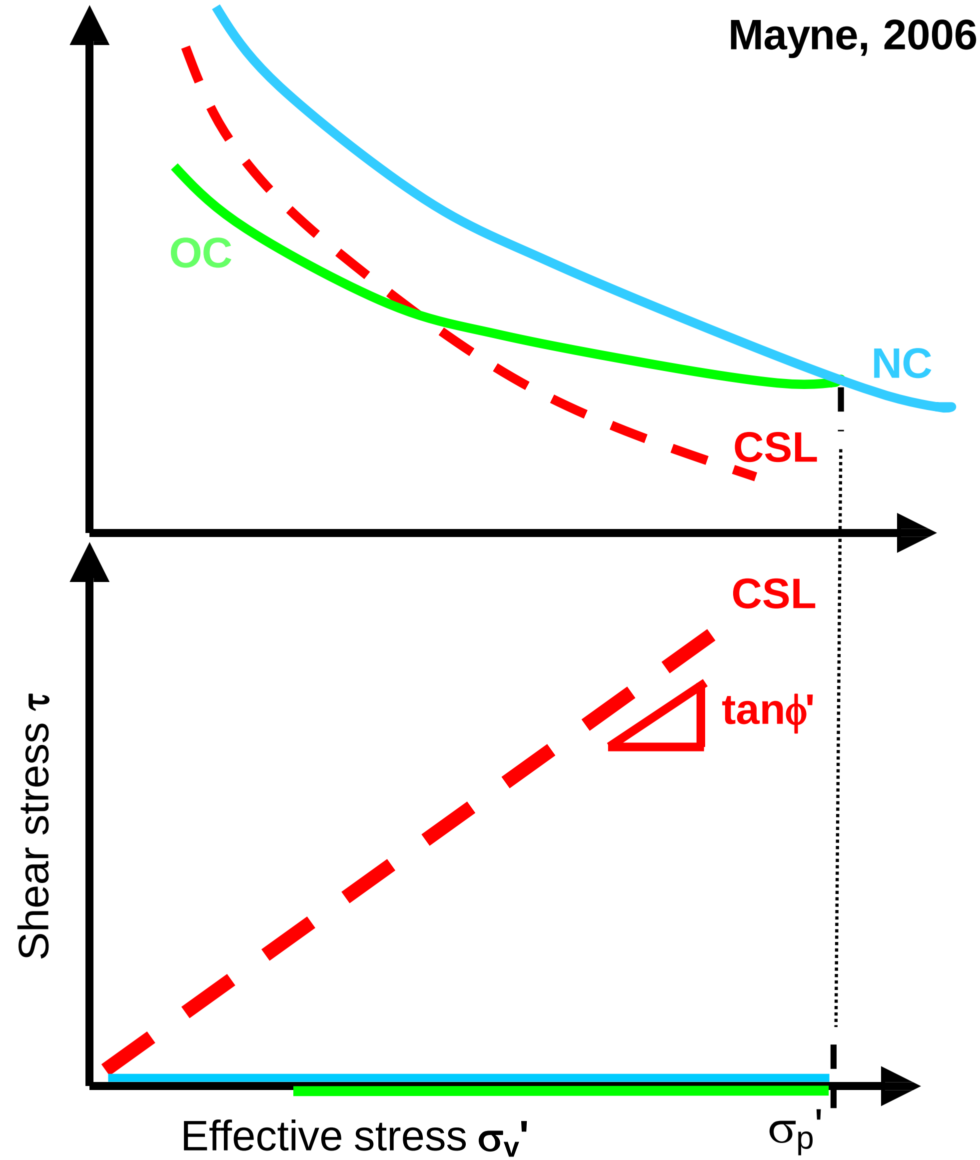
Given: e_0, σ_{vo}’, NC (OCR=1)

Undrained Path: $\Delta V/V_0 = 0$

+Δu = Positive Excess Porewater Pressures







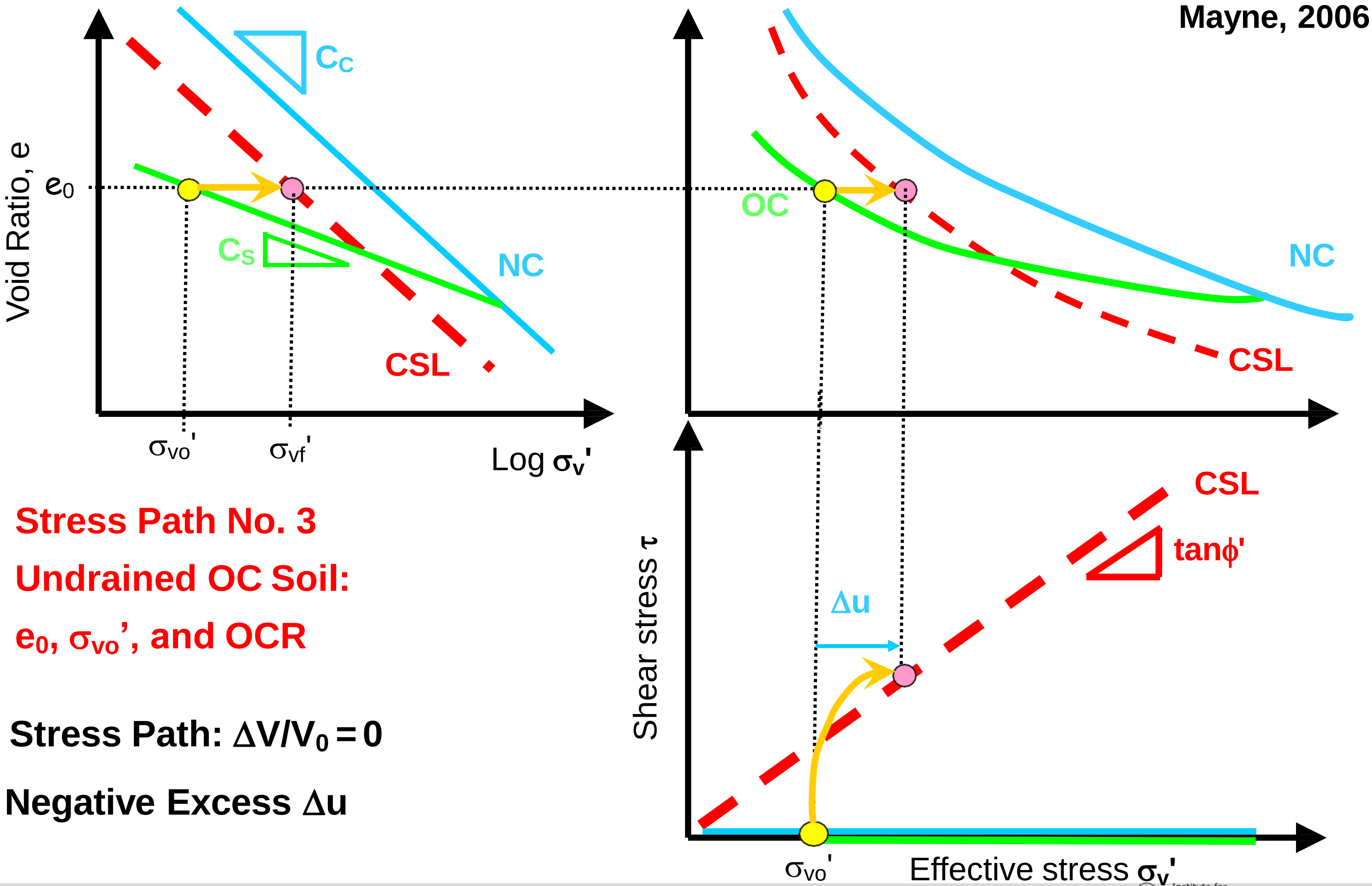
Overconsolidated States:

e_0, σ_{vo}', and OCR = σ_p'/σ_{vo}'

where $\sigma_p' = \sigma_{vmax}' = P_c'$ =

preconsolidation stress;

OCR = overconsolidation ratio



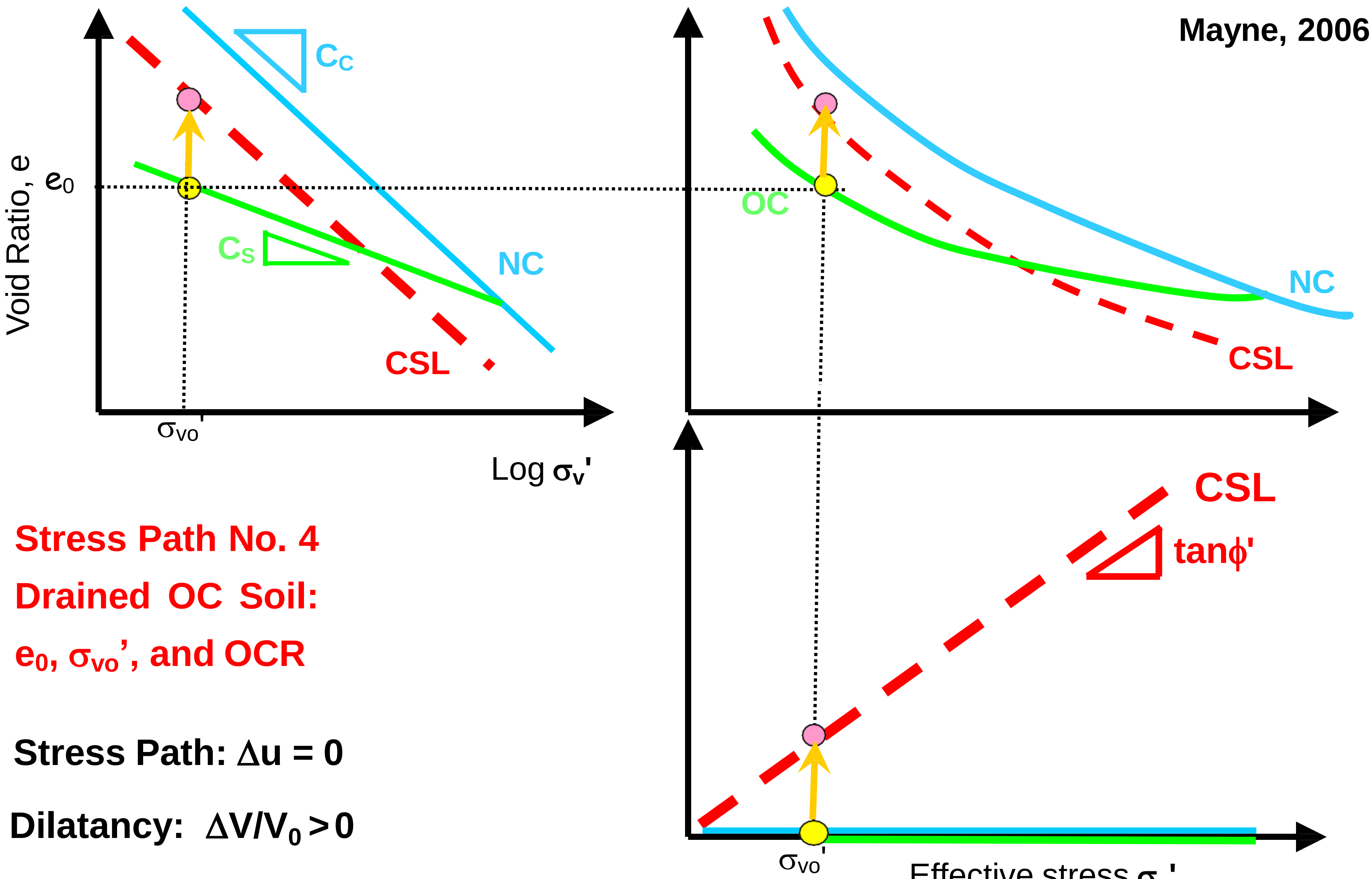
Stress Path No. 3

Undrained OC Soil:

e_0, σ_{vo}', and OCR

Stress Path: $\Delta V/V_0 = 0$

Negative Excess Δu



Stress Path No. 4

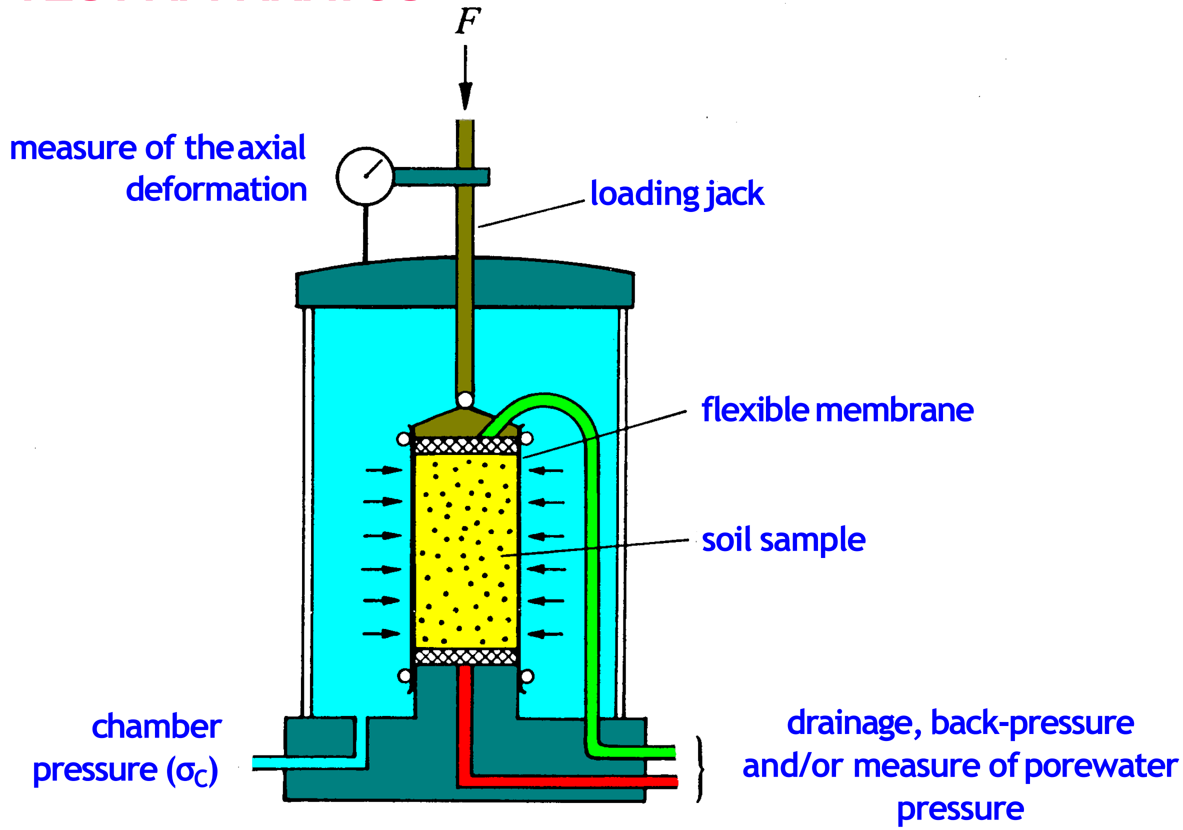
Drained OC Soil:

e_0, σ_{vo}', and OCR

Stress Path: $\Delta u = 0$

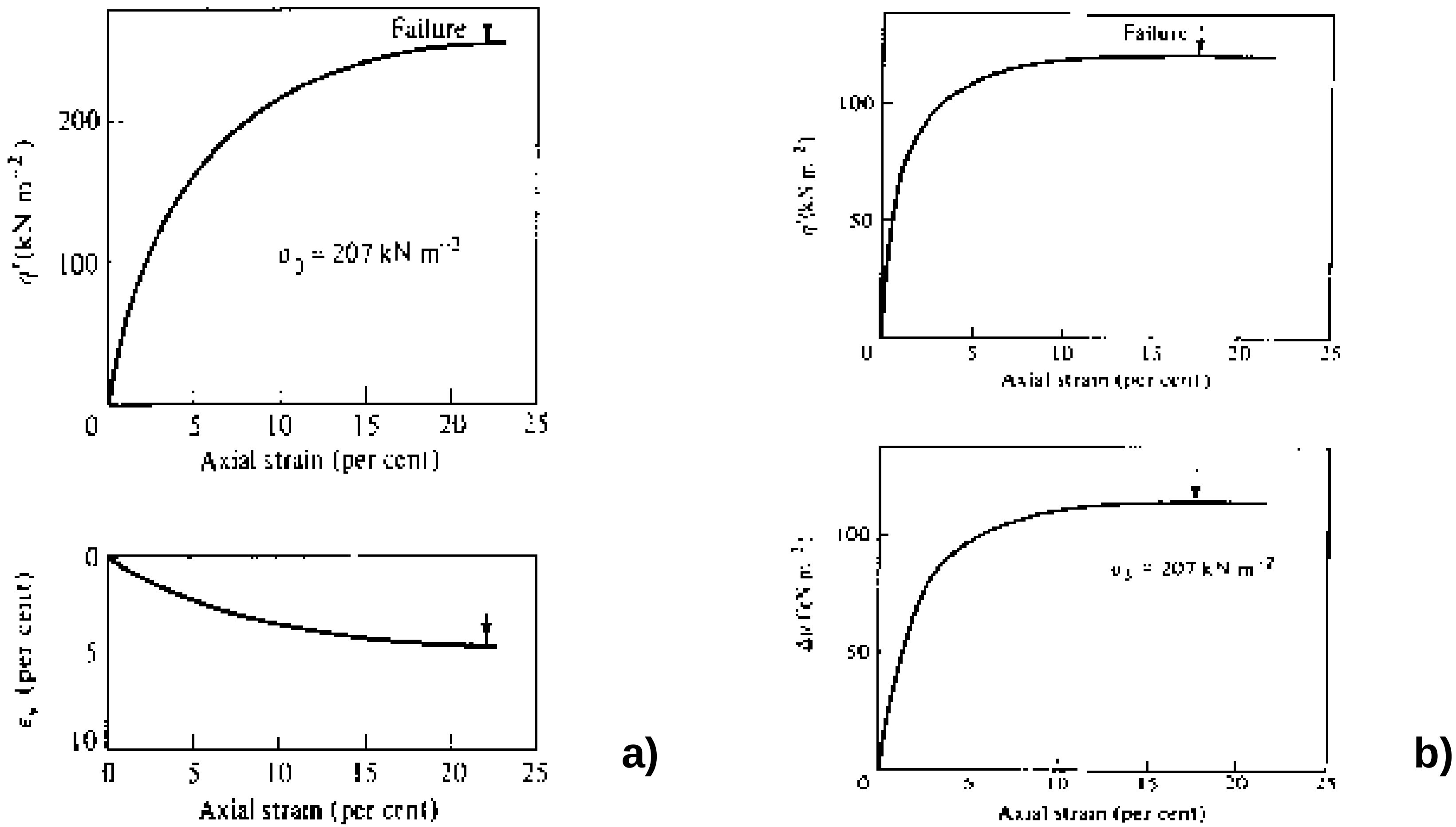
Dilatancy: $\Delta V/V_0 > 0$

TRIAXIAL TEST APPARATUS

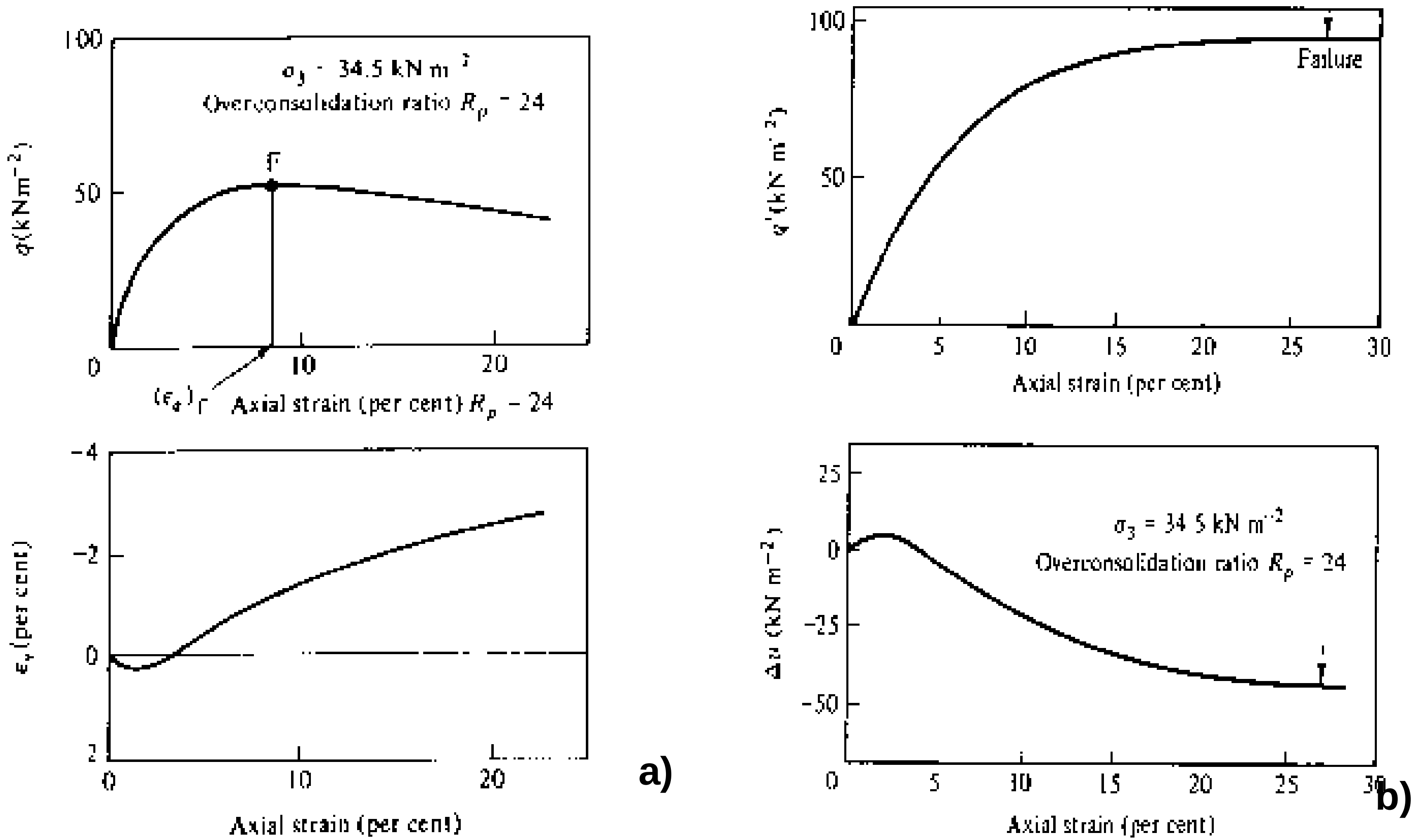


$$p' = \frac{1}{3}(\sigma_1' + 2\sigma_3'), \qquad q = \sigma_1' - \sigma_3'$$

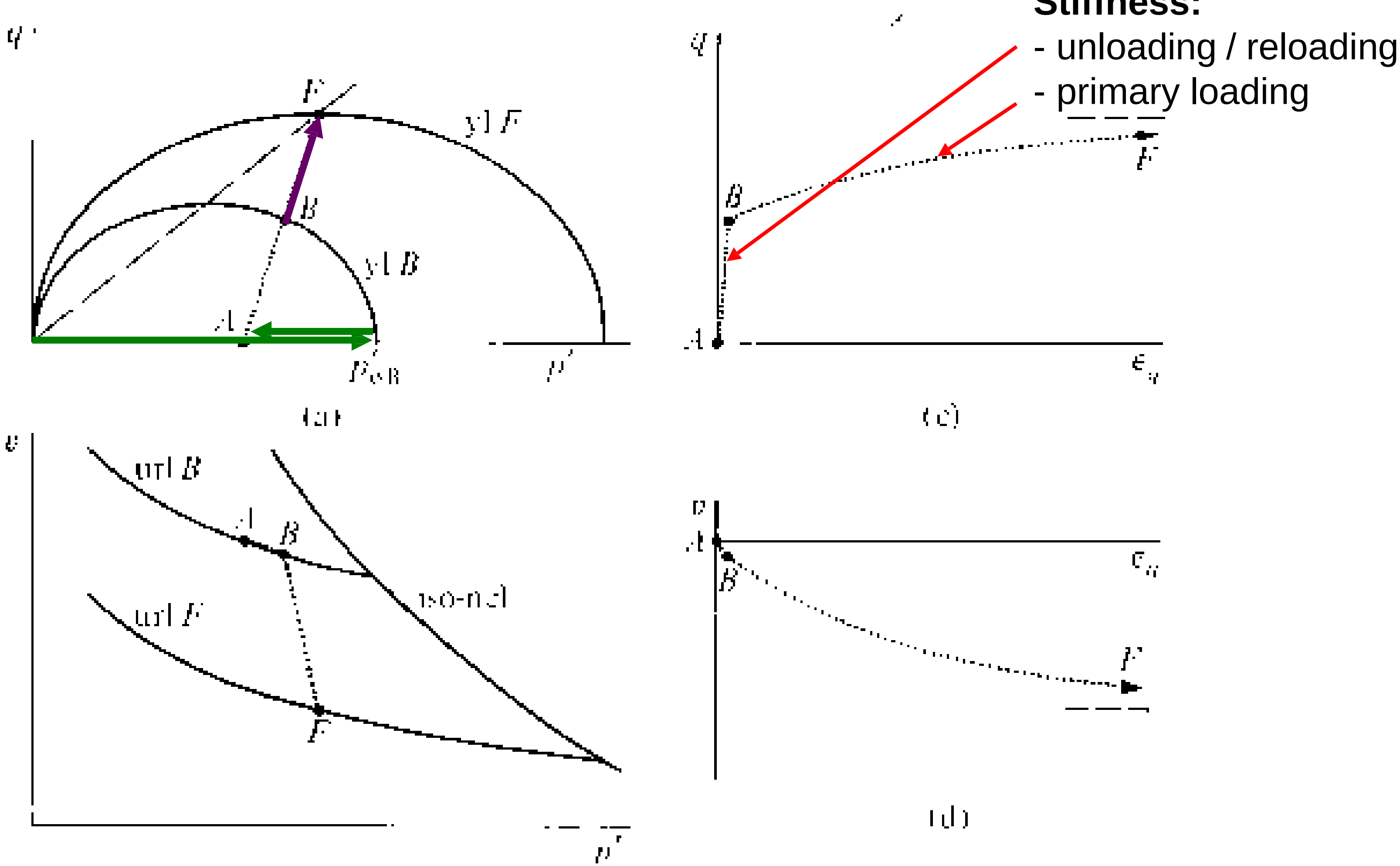
$$\varepsilon_v = \varepsilon_p = \varepsilon_1 + 2\varepsilon_3, \qquad \varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3)$$



Typical results from drained (a) and undrained (b) triaxial tests on normally consolidated soils (from Atkinson & Bransby, 1978)



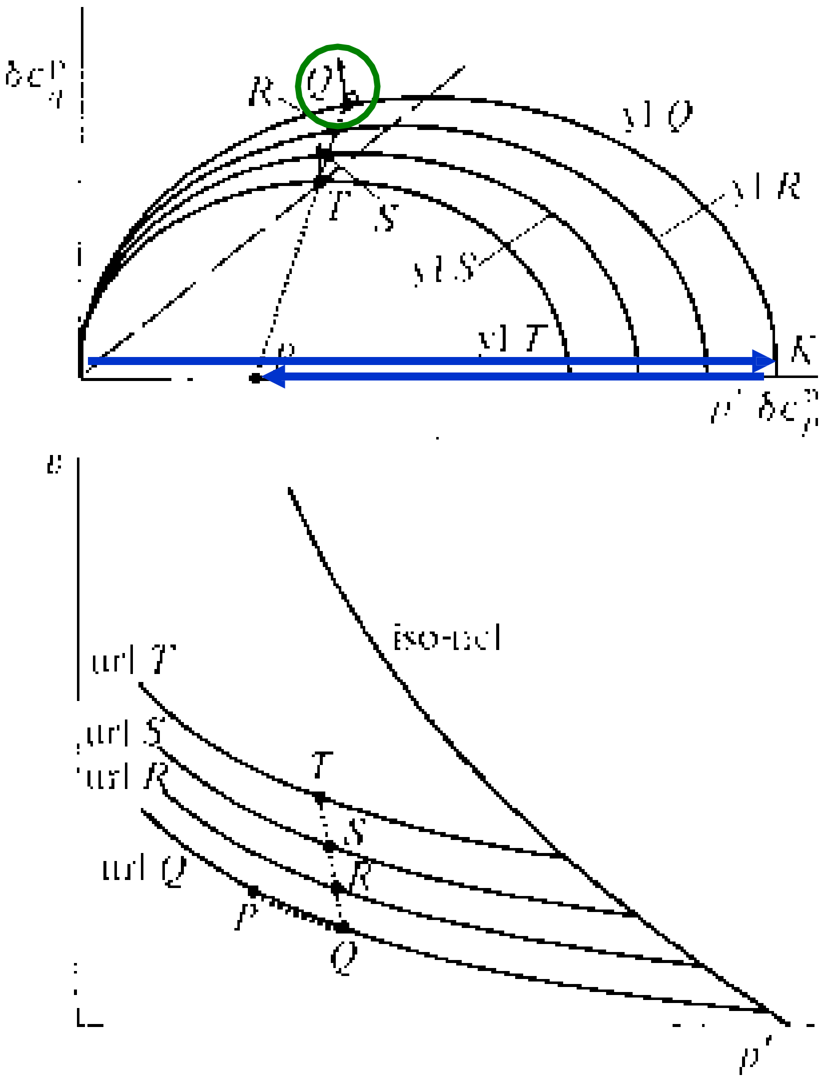
Typical results from drained (a) and undrained (b) triaxial tests on overconsolidated soils (from Atkinson & Bransby, 1978)

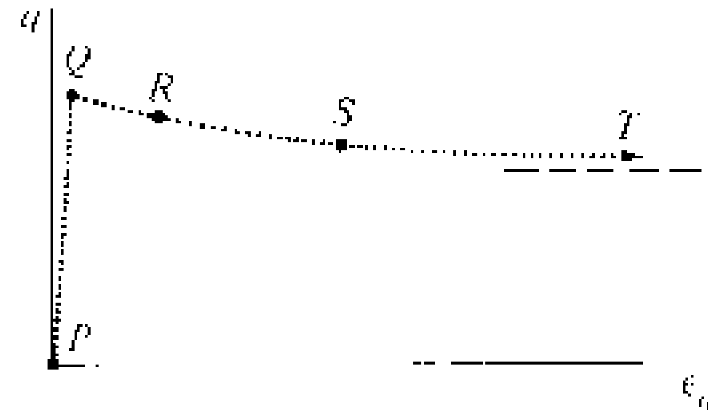


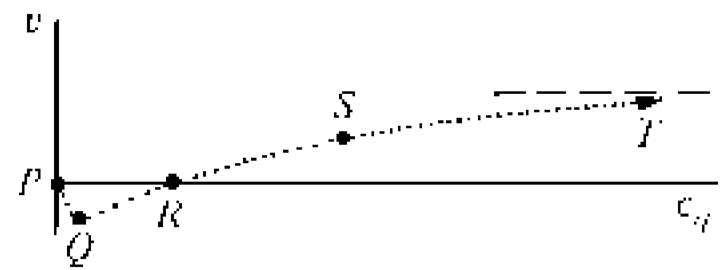
lightly overconsolidated

drained compression

D. Muir Wood, 1990



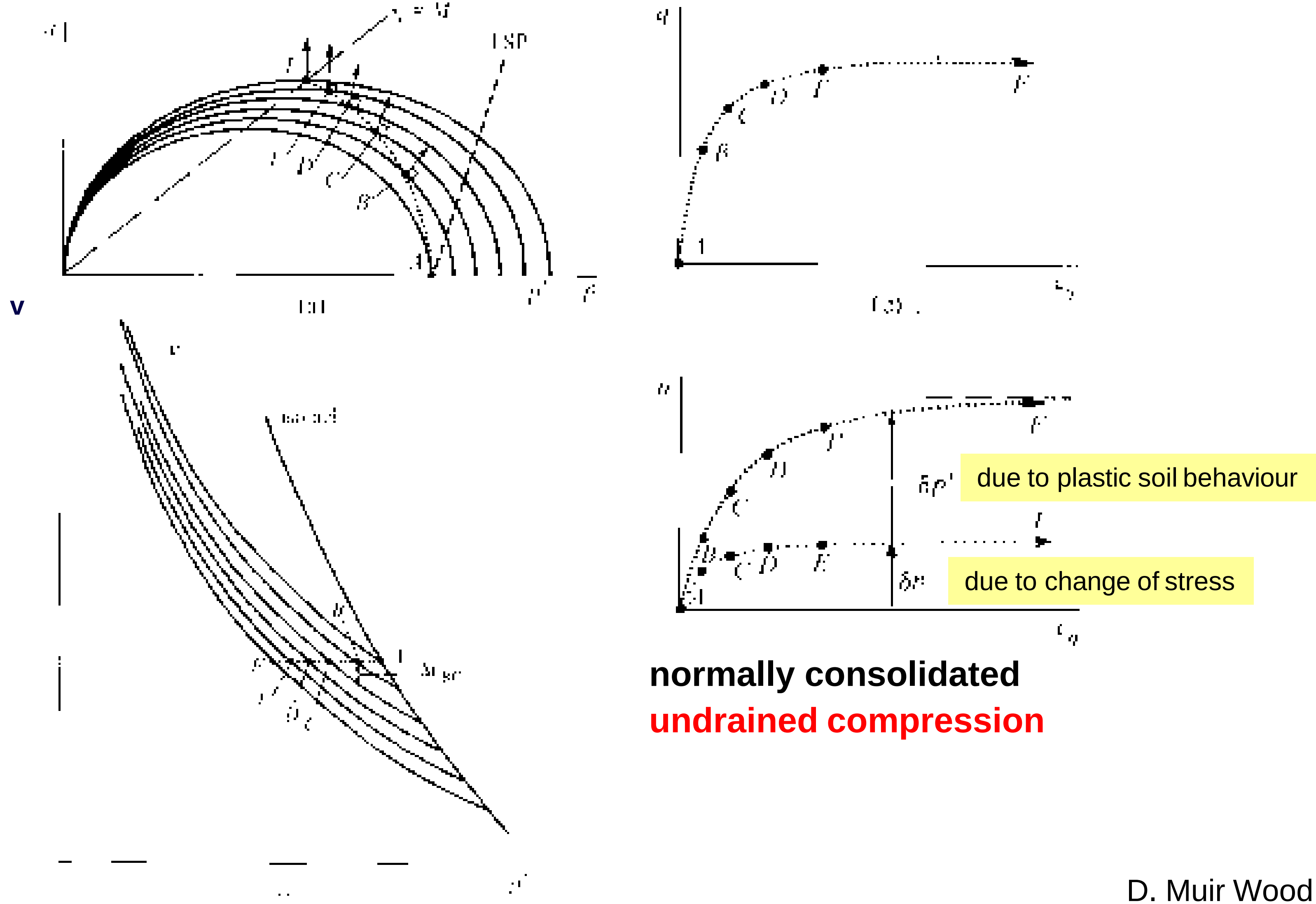




heavily overconsolidated

drained compression

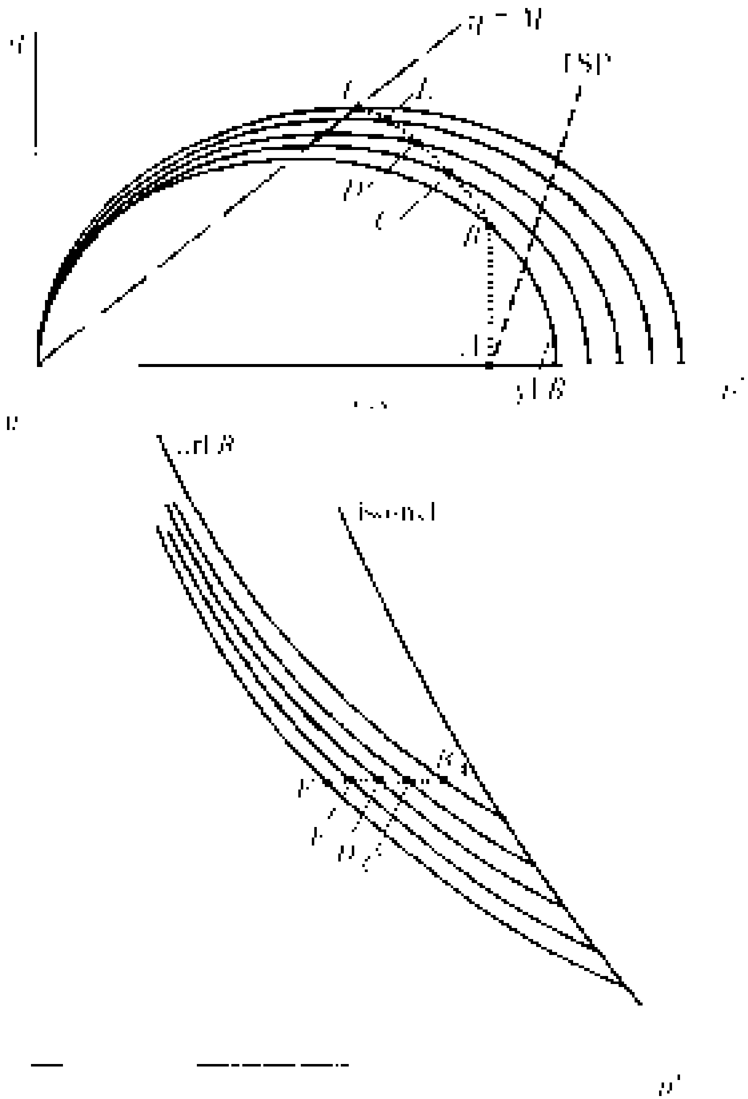
D. Muir Wood, 1990

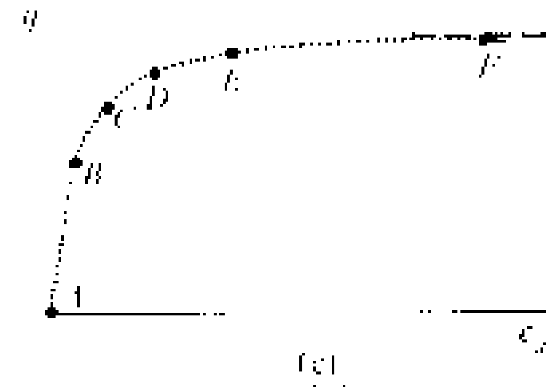


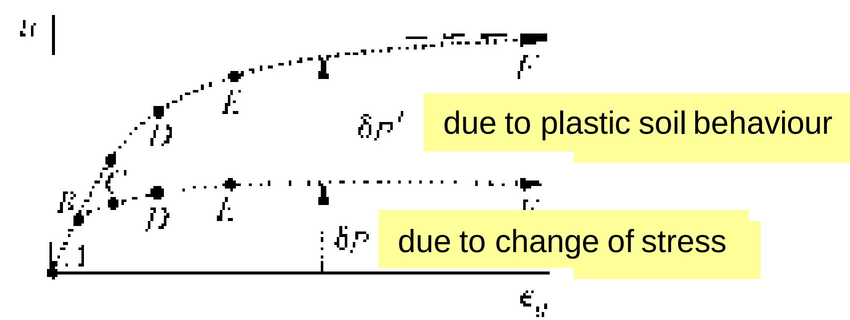
normally consolidated

undrained compression

D. Muir Wood, 1990



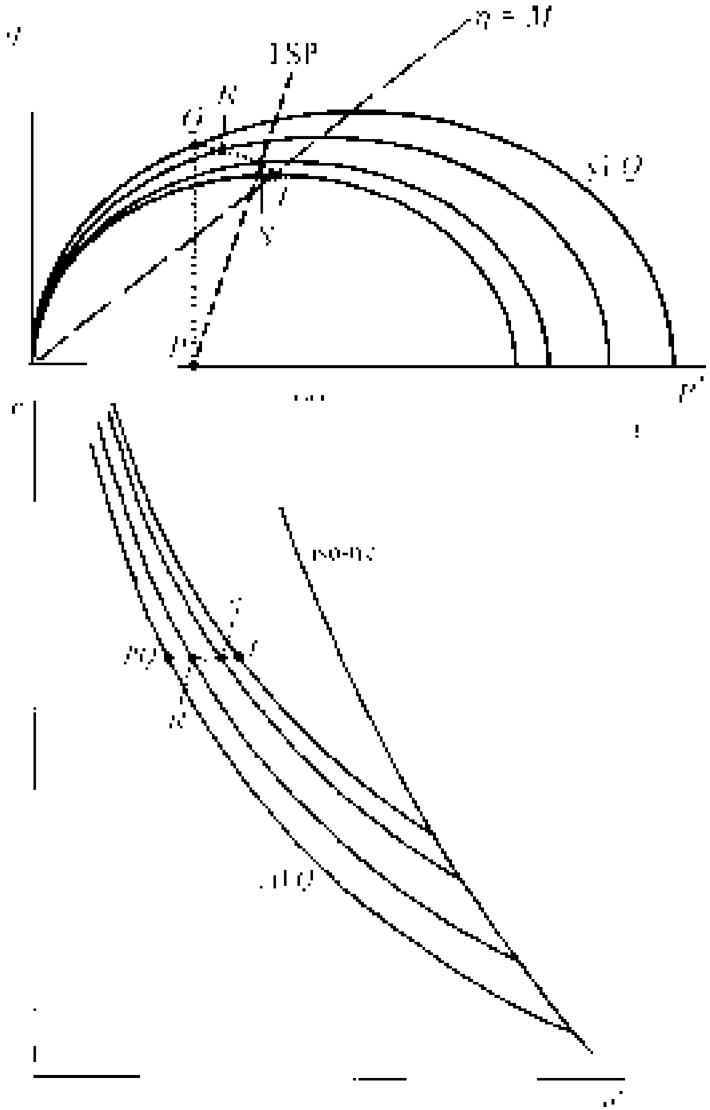


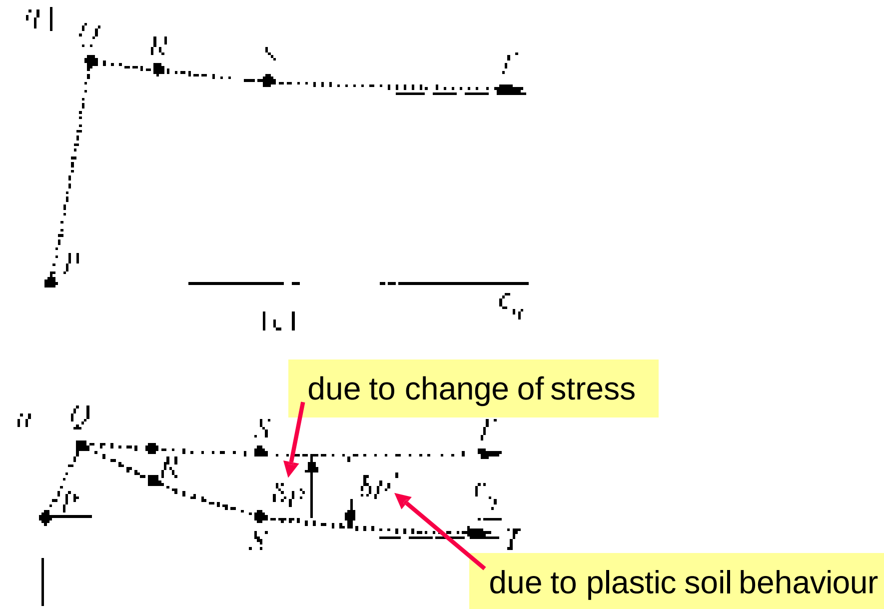


lightly overconsolidated

undrained compression

D. Muir Wood, 1990





heavily overconsolidated

undrained compression

D. Muir Wood, 1990

SCIENCE ▪ PASSION ▪ TECHNOLOGY

CG05

HARDENING SOIL SMALL MODEL

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Pelatihan software Plaxis di Tembagapura (In-house training) PT. Freeport Indonesia/ Timika, Papua, 22-30 September 2018

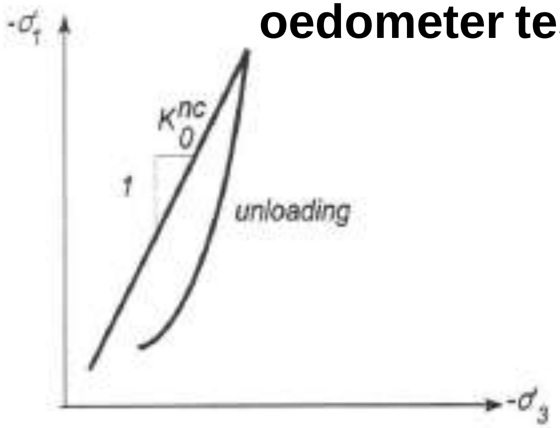
CONTENTS

Soil behaviour includes:

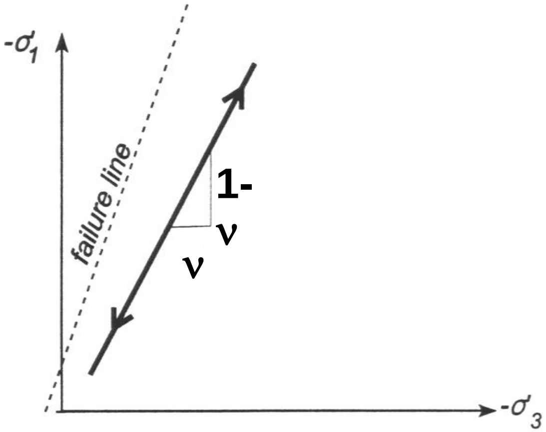
- difference in behaviour for primary loading – reloading/unloading
- nonlinear behaviour well below failure conditions
- stress dependent stiffness
- plastic deformation for isotropic or K_0-stress paths
- dilatancy is not constant
- small strain stiffness (at very low strains and upon stress reversal)
- influence of density on strength and stiffness

cannot be accounted for with simple elastic-perfectly plastic constitutive models

oedometer test

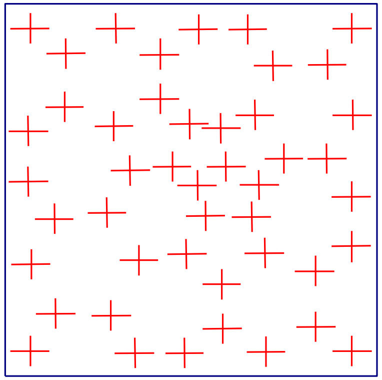


Real test

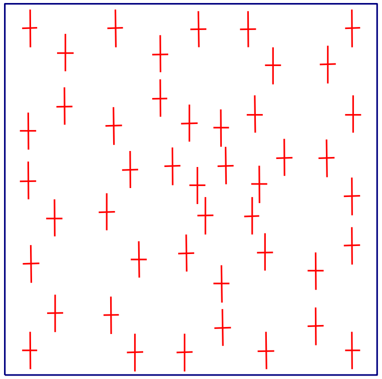


Mohr-Coulomb model

HS

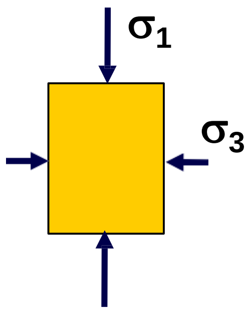


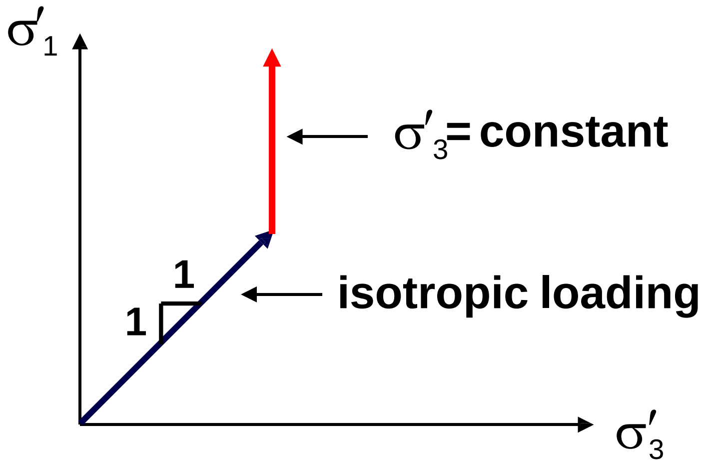
MC

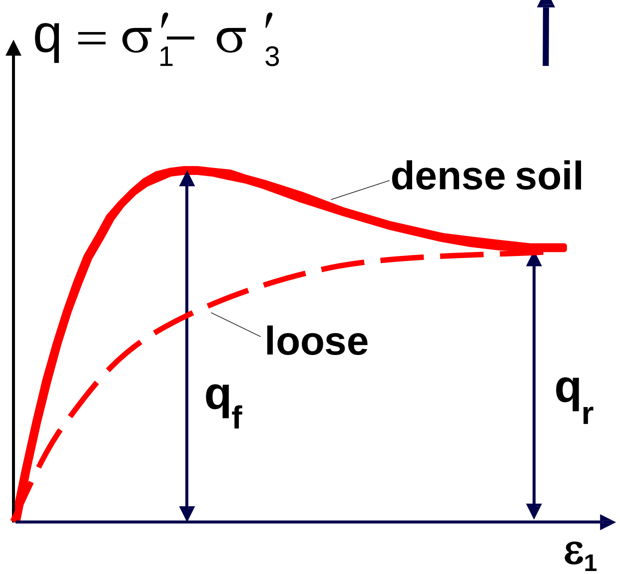


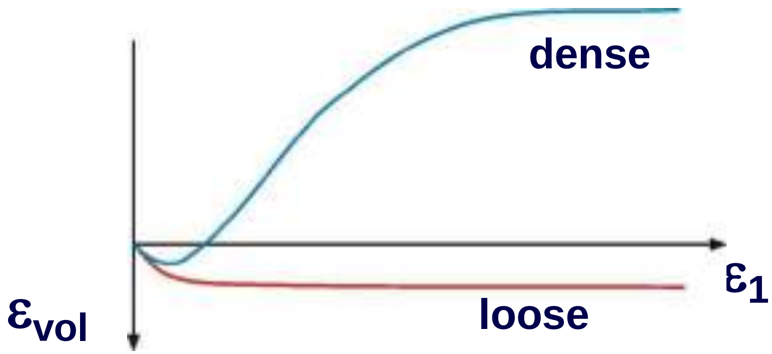
TRIAXIAL TEST

Applied stress path and results for standard drained triaxial test







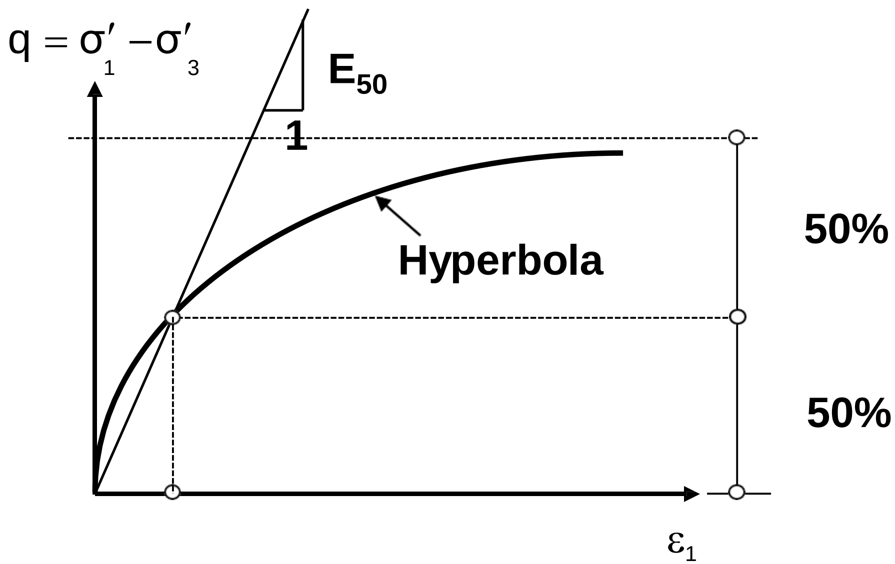


q_f = failure value

q_r = residual value

$$\varepsilon_{vol} = \varepsilon_{volumetric} = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$$

HYPERBOLIC APPROXIMATION OF STANDARD DRAINED TEST



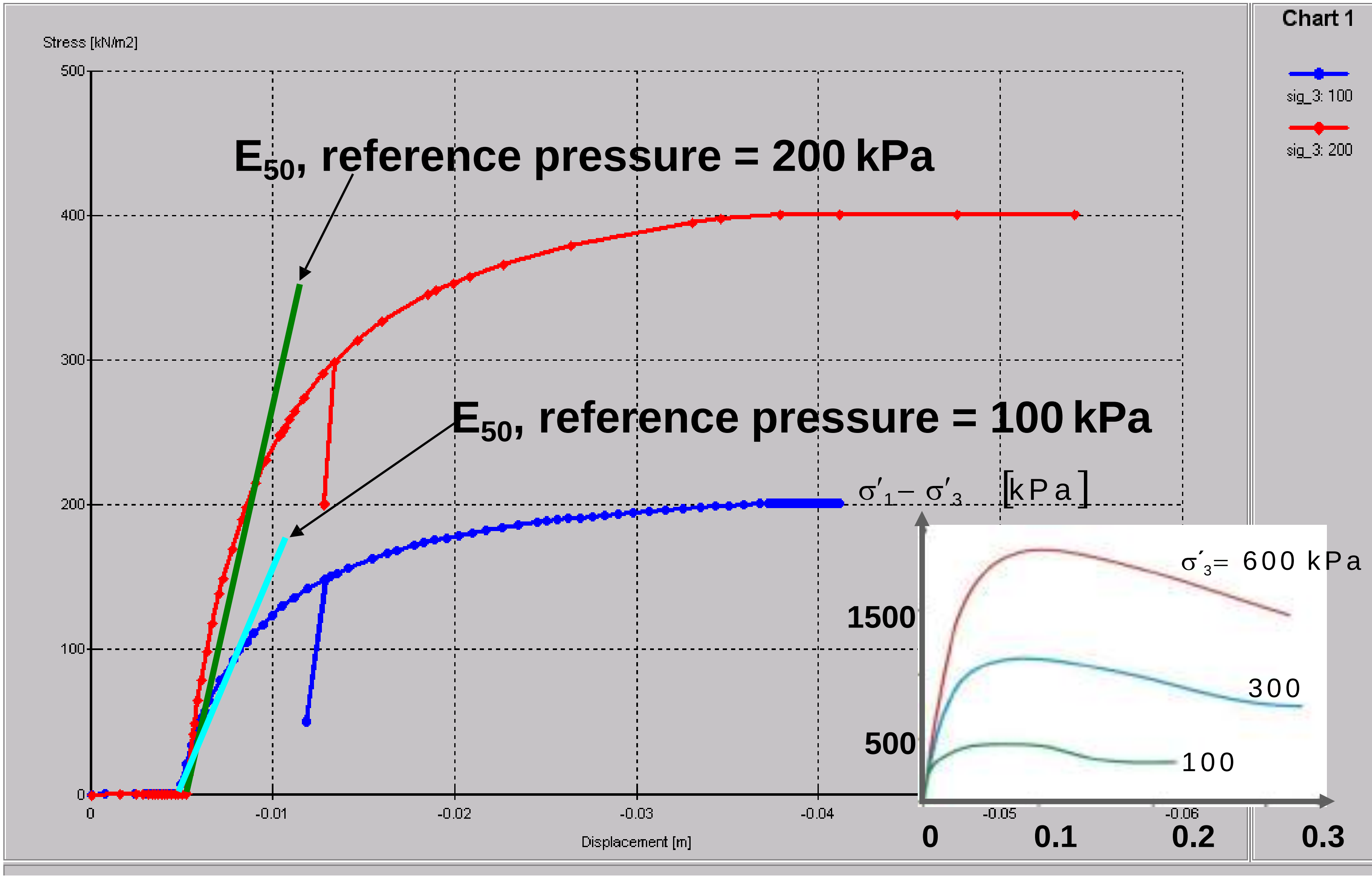
$$\varepsilon_1 = \frac{q_a}{2 \cdot E_{50}} \cdot \frac{q}{q_a - q}$$

$$E_{50} = E_{50}^{ref} \cdot \left(\frac{\sigma'_3 + a}{p_{ref} + a} \right)^m$$

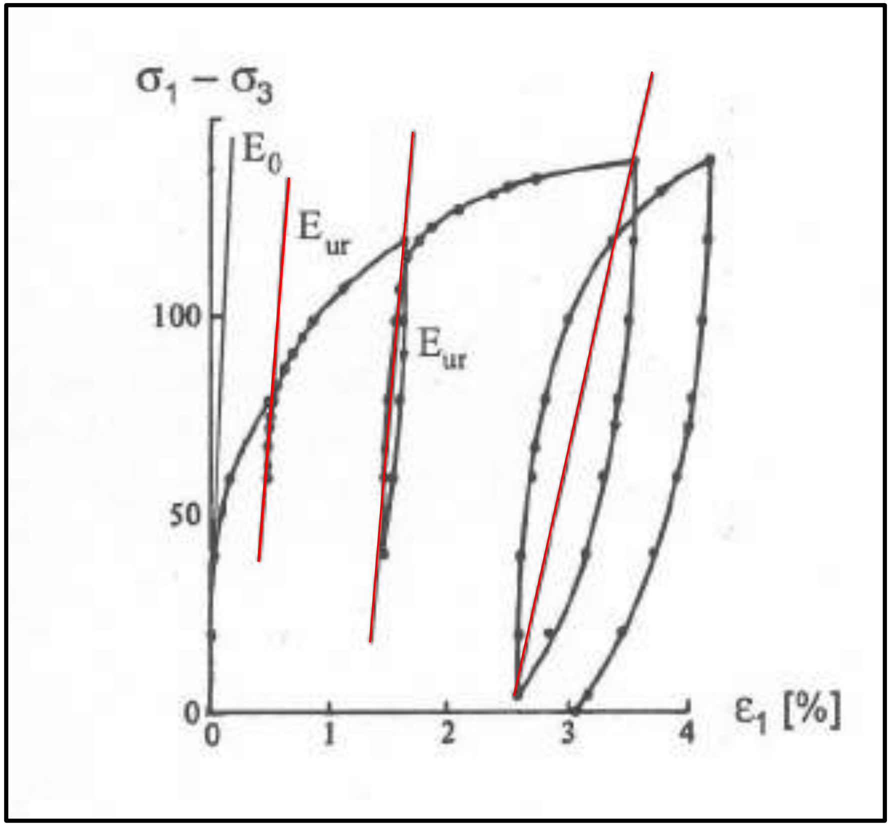
E_{50}^{ref} = **reference modulus for primary loading at 50% of strength**

$\mathbf{m_{sand} \approx 0.5}$; $\mathbf{m_{clay} \approx 1}$

DEFINITION OF E_{50}

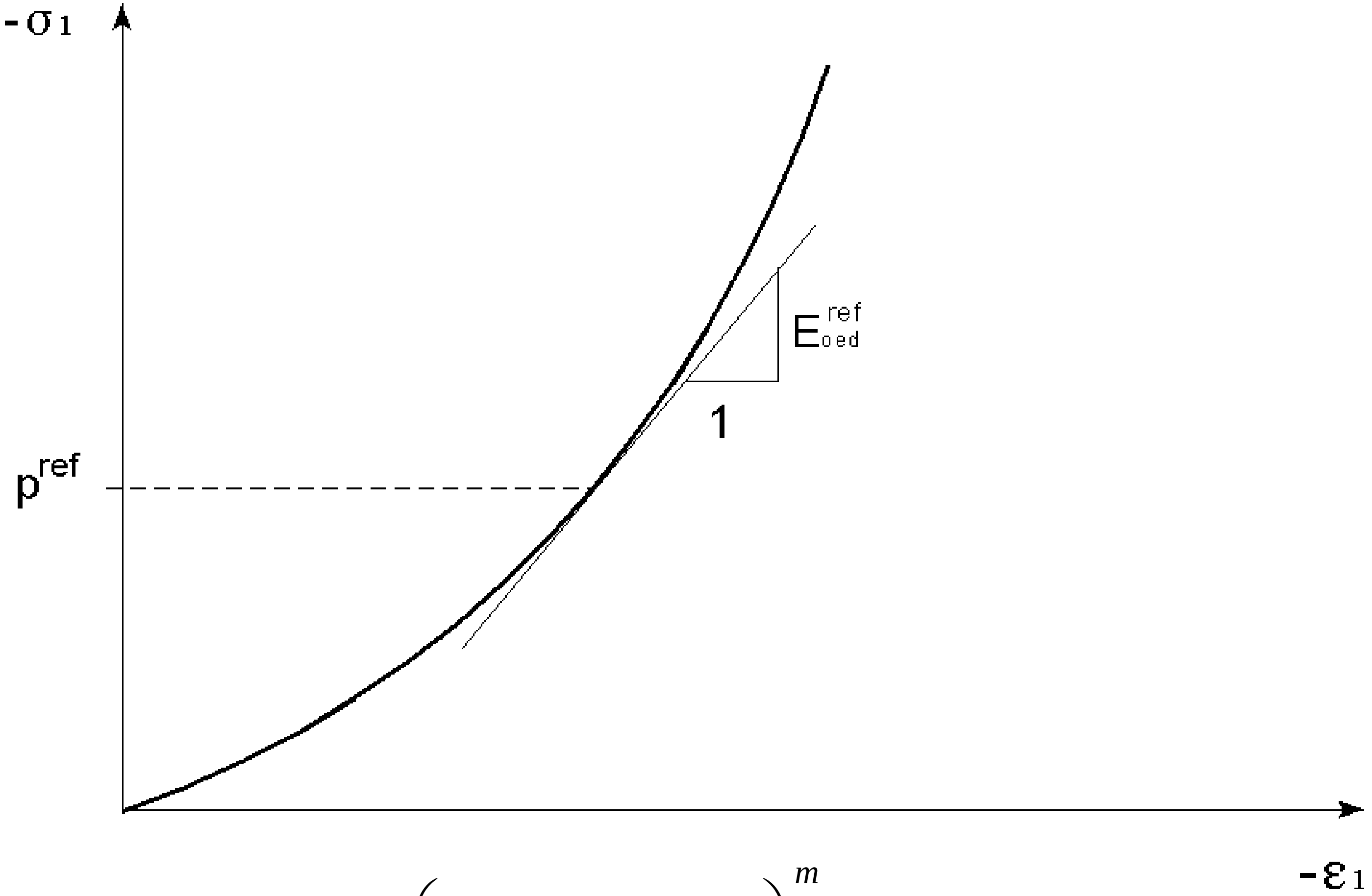


STIFFNESS IN UNLOADING-RELOADING



Triaxial tests:
Unloading is purely elastic in HS model

DEFINITION OF E_{oed}



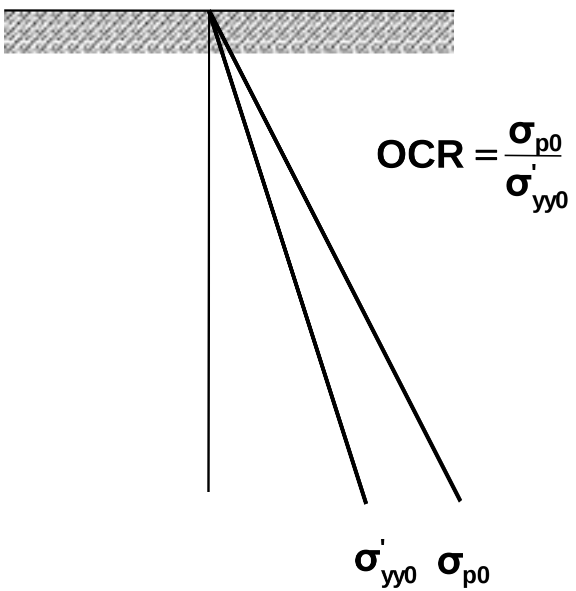
$$E_{oed} = E_{oed}^{ref} \cdot \left(\frac{c \cdot \cot \varphi + \sigma_1'}{c \cdot \cot \varphi + p_{ref}} \right)^m$$

holds strictly for K_0-stress paths only

OVERCONSOLIDATION

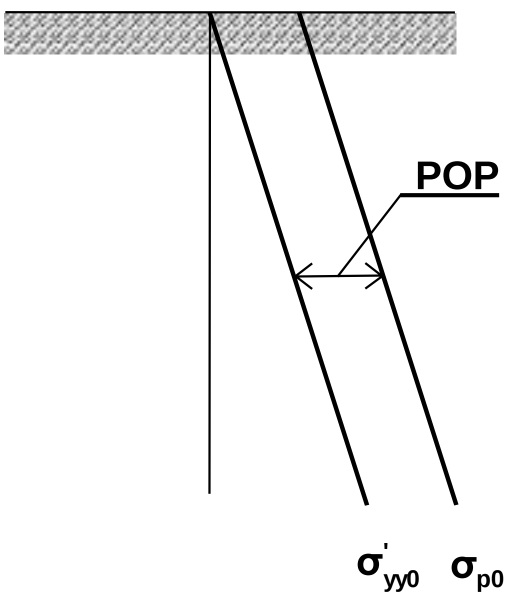
Calculation of σ_{p0} based on OCR:

$$\sigma_{p0} = OCR\,\sigma'_{yy0}$$

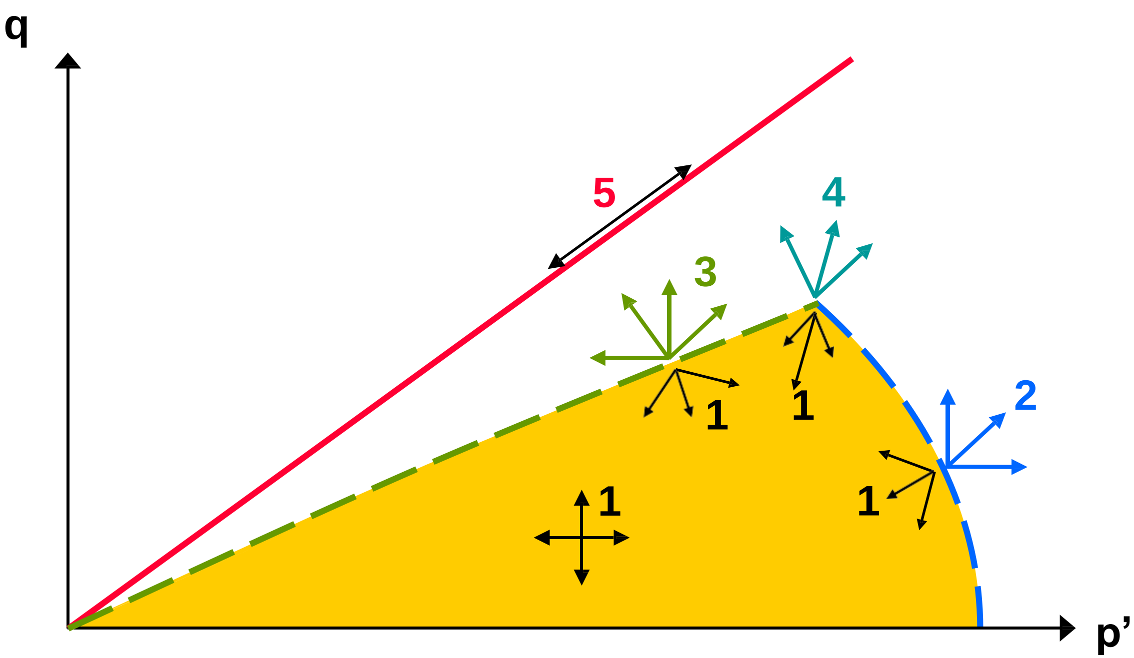


Calculation of σ_{p0} based on POP:

$$\sigma_{p0} = \sigma'_{yy0} - POP$$



PLASTICITY IN HS MODEL



1. **Elasticity** (unloading reloading)
2. **Plasticity** (compression)
3. **Plasticity** (shear)
4. **Plasticity** (compression + shear)
5. **Plasticity** (failure criterion)

PARAMETERS OF HARDENING SOIL MODEL

φ' friction angle
c' cohesion
ψ' dilatancy angle

E_{50}^{ref} **secant modulus from triaxial test** (controls deviatoric hardening)

E_{oed}^{ref} **tangential modulus from oedometer test**
(controls volumetric hardening)

E_{ur}^{ref} **unloading / reloading modulus**

default: $E_{ur}^{ref} = 3\ E_{50}^{ref}$

m **power for stress dependency of stiffness**

PARAMETERS OF HARDENING SOIL MODEL

ν_{ur} **Poisson ratio for unloading / reloading** (default $\nu_{ur} = 0.2$)

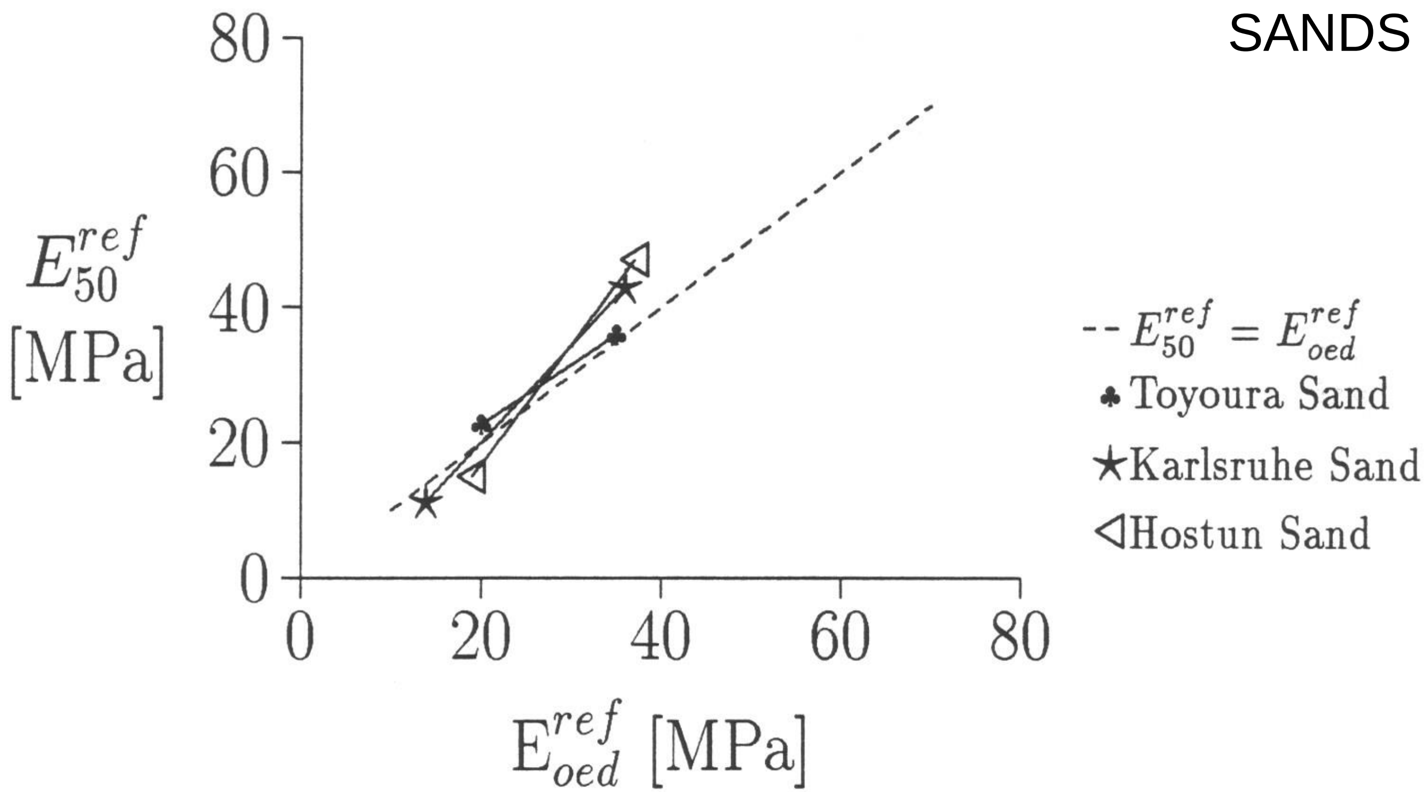
p^{ref} **reference stress**
(default $p^{ref} = 100$ stress units)

K_0^{nc} **K_0-value for normal consolidation** (default $= 1\text{-}\sin\varphi$)
(controls volumetric hardening)

$\mathbf{R_f = q_f / q_a}$ (default $R_f = 0.9$)

PARAMETERS OF HARDENING SOIL MODEL

SANDS



$$E_{oed}^{ref} \approx I_D \bullet 60MPa$$

Correllation for p^{ref}=100 kPa (Lengkeek)
I_D = relative density

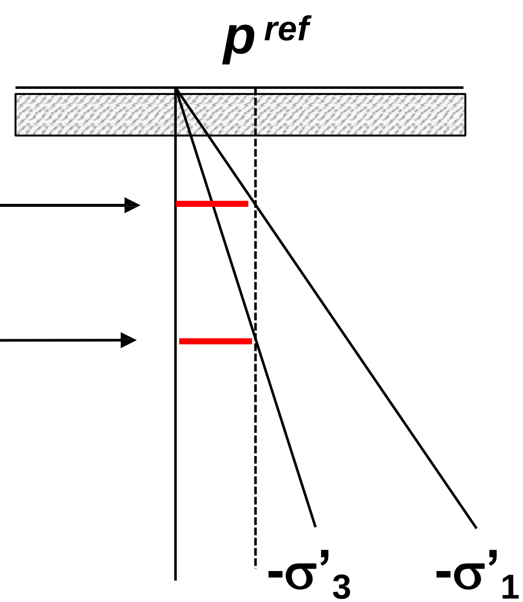
Stiffness of sand

$$E_{50}^{ref} \approx E_{oed}^{ref}$$

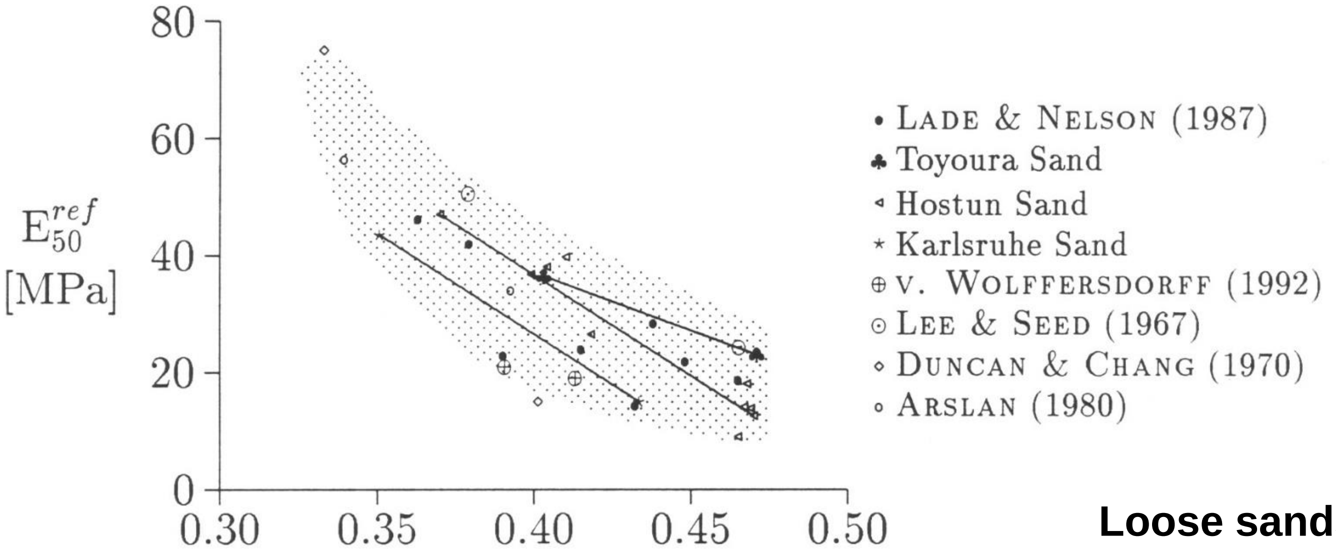
How can this be true?

$$-\sigma'_1 = p^{ref}$$

$$-\sigma'_3 = p^{ref}$$

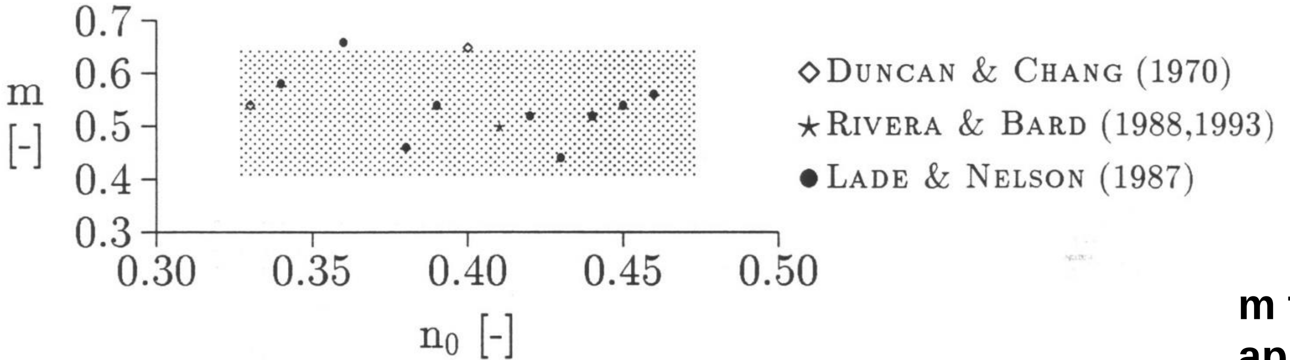


Note: always plot E_{oed}, E_{ur} distribution for initial stress state when using HS-model



Loose sands: $E_{50}^{ref} \approx 15$ MPa

Dense sands: $E_{50_{ref}} \approx$ 50 MPa



m for clays:
approx. 0.9 –1.0

For normally consolidated clays (m=1):

$$E_{oed}^{ref} \approx \tfrac{1}{2} E_{50}^{ref}$$

Order of magnitude (very rough)

$$E_{oed}^{ref} \approx \frac{50000\,kPa}{I_p}$$

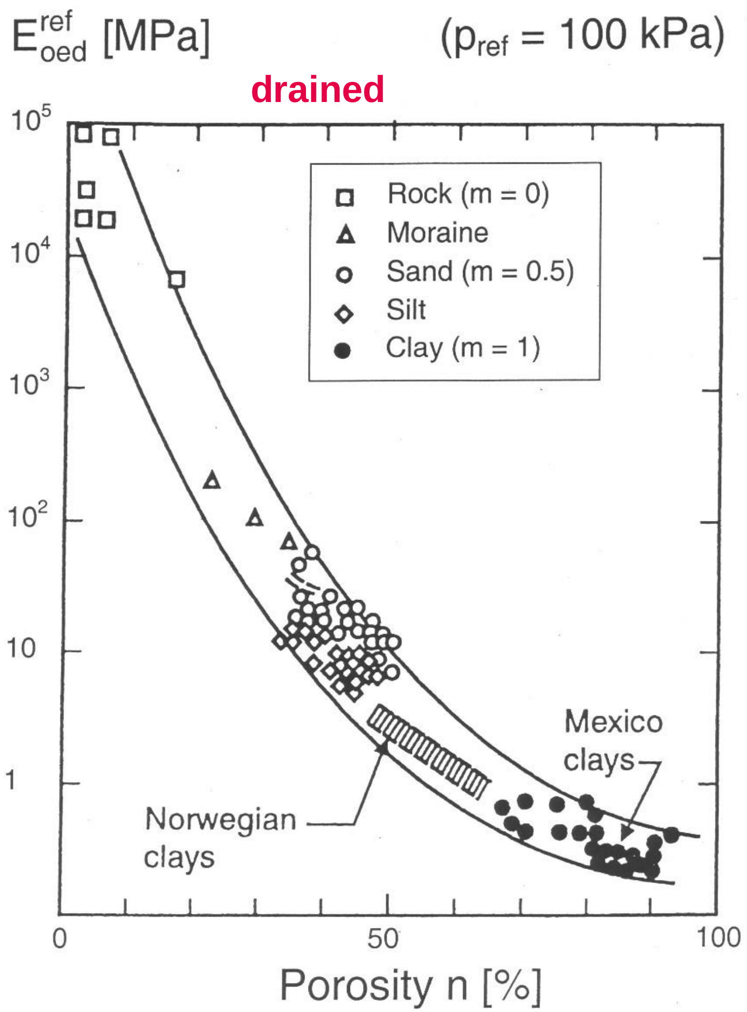
Correlation with I_p for p^{ref} =100 kPa

$$E_{oed}^{ref} \approx \frac{500\,kPa}{w_L - 0.1}$$

Correlation by Vermeer

$$E_{oed}^{ref} \approx p^{ref} / \lambda^{*}$$

Relation with Soft Soil model

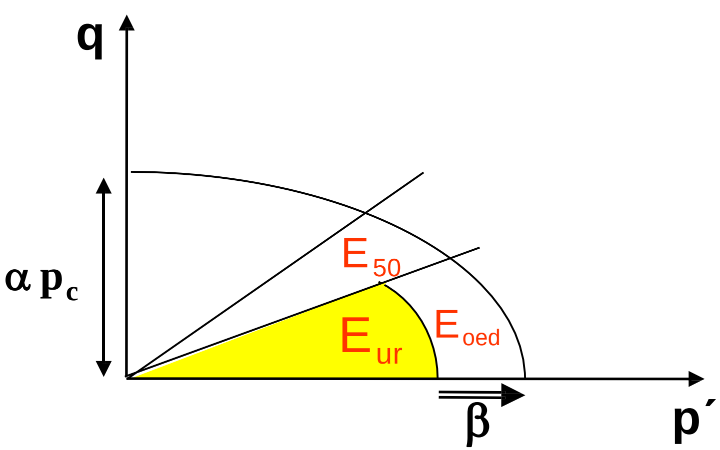


Ohde / Janbu:

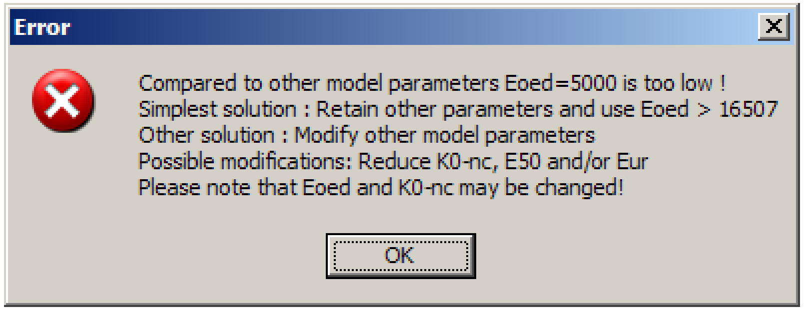
$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma_1'}{p^{ref}}\right)^m$$

Parameter limitations

HS model has internal parameters that are computed from our ”engineering” input parameters > not all combinations of input parameters can be used. For very soft soils this could be a problem in certain cases.

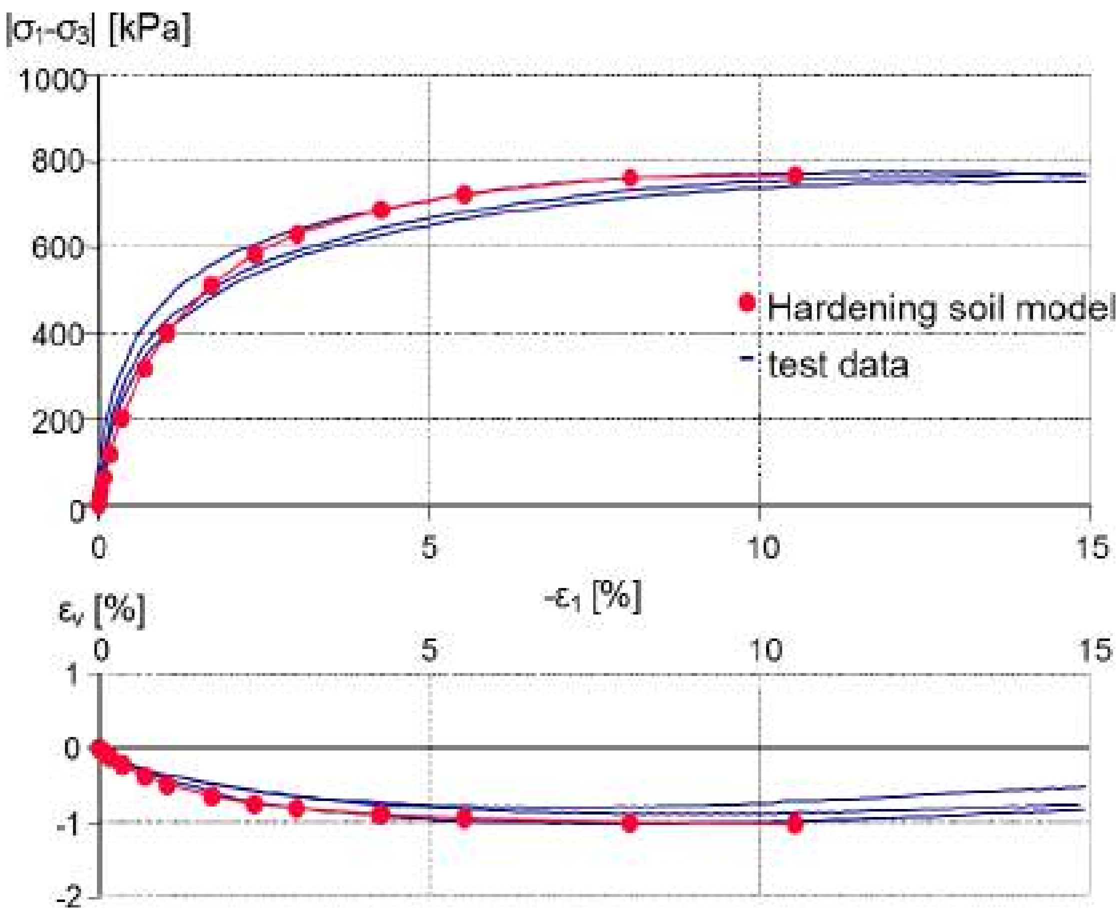


double hardening

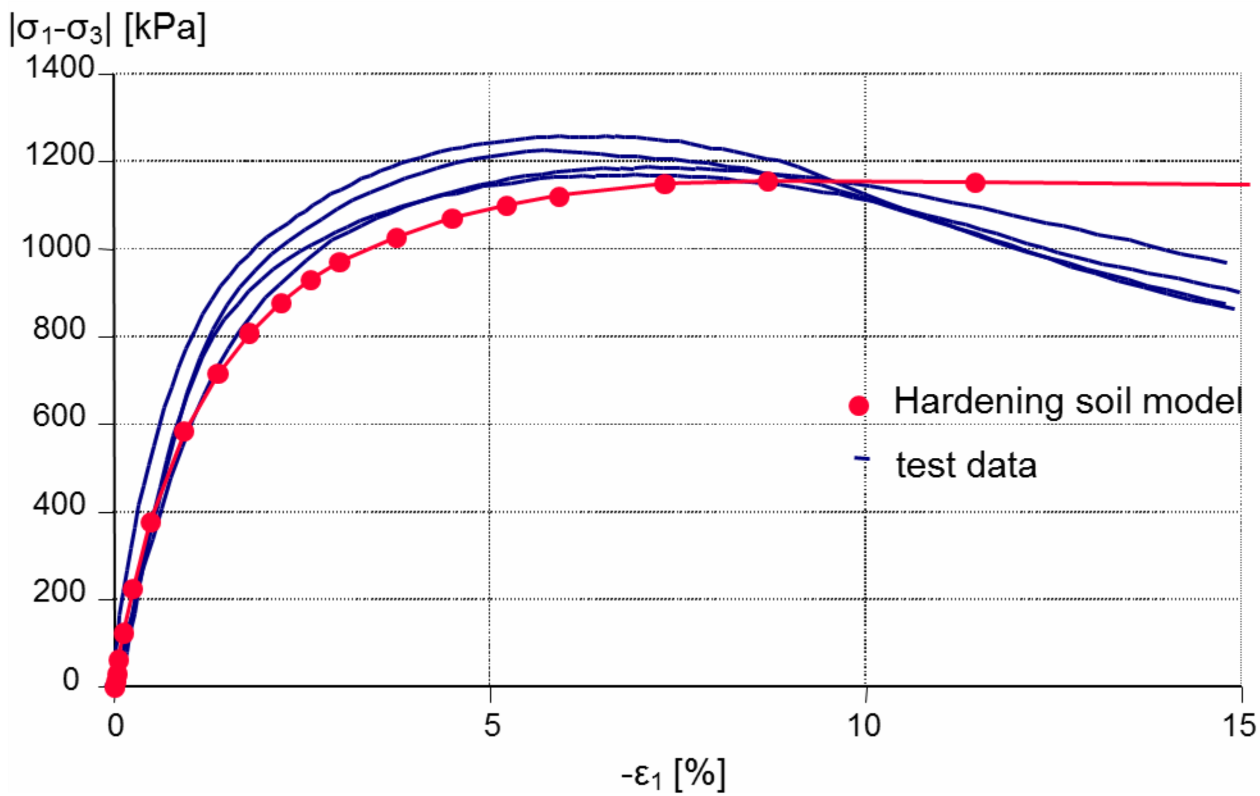


e.g. E_{50} / E_{oed} > 2 difficult to input

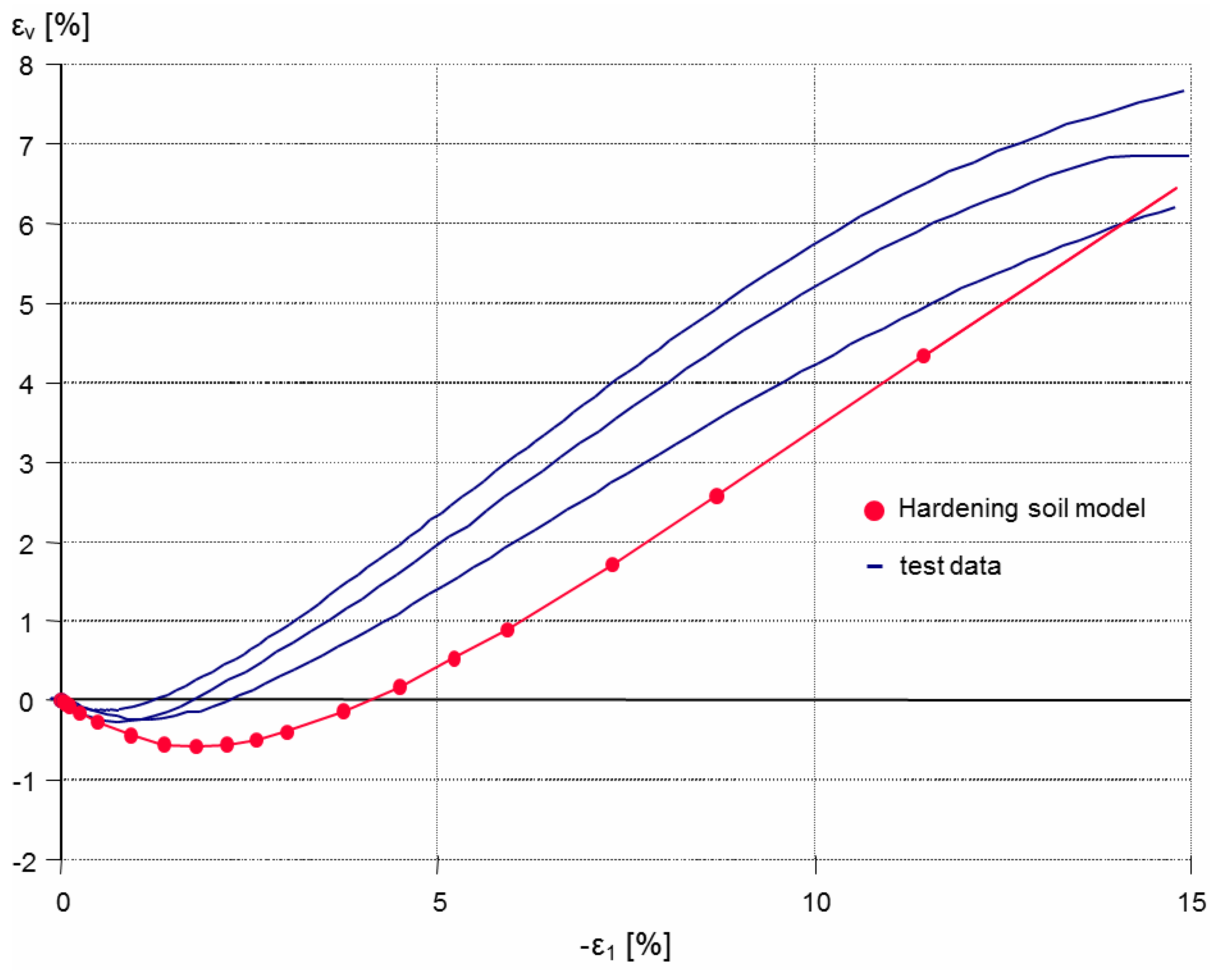
TRIAXIAL TESTS ON LOOSE SAND



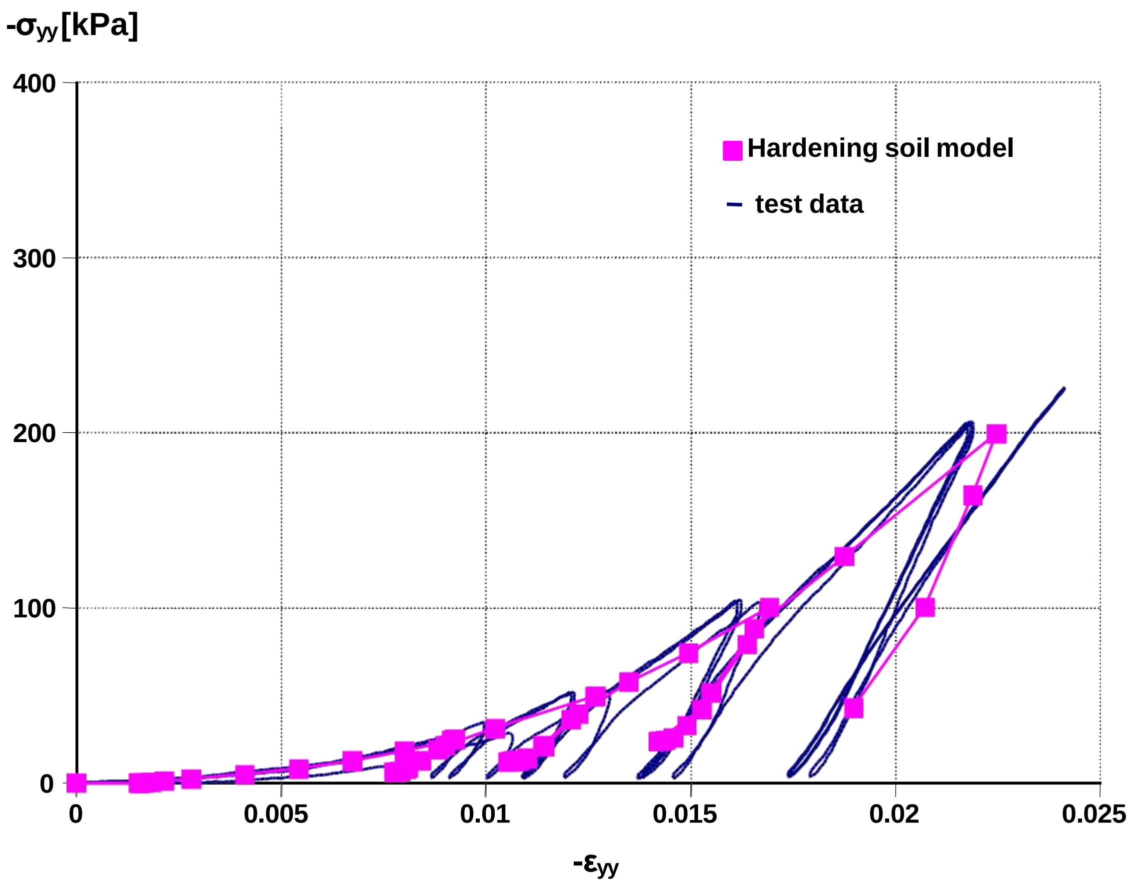
TRIAXIAL TESTS ON DENSE SAND



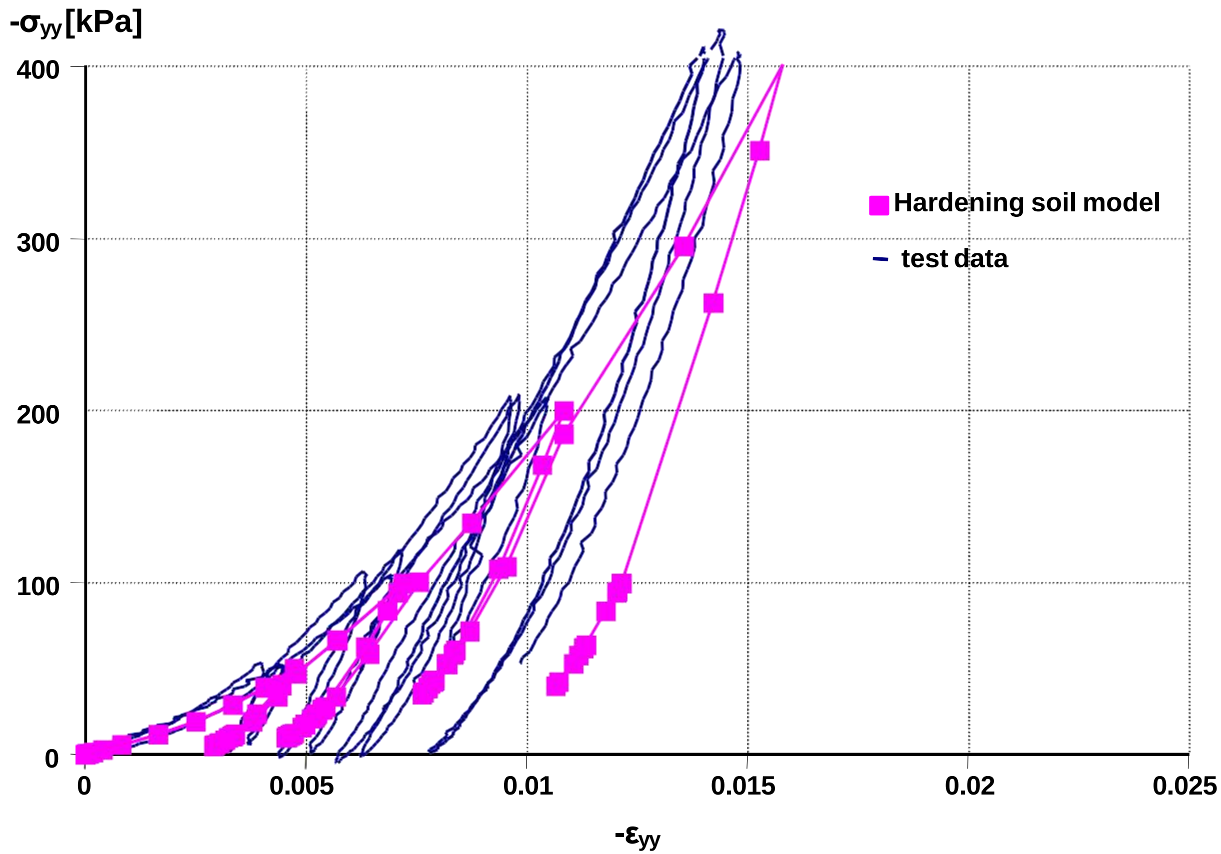
TRIAXIAL TESTS ON DENSE SAND



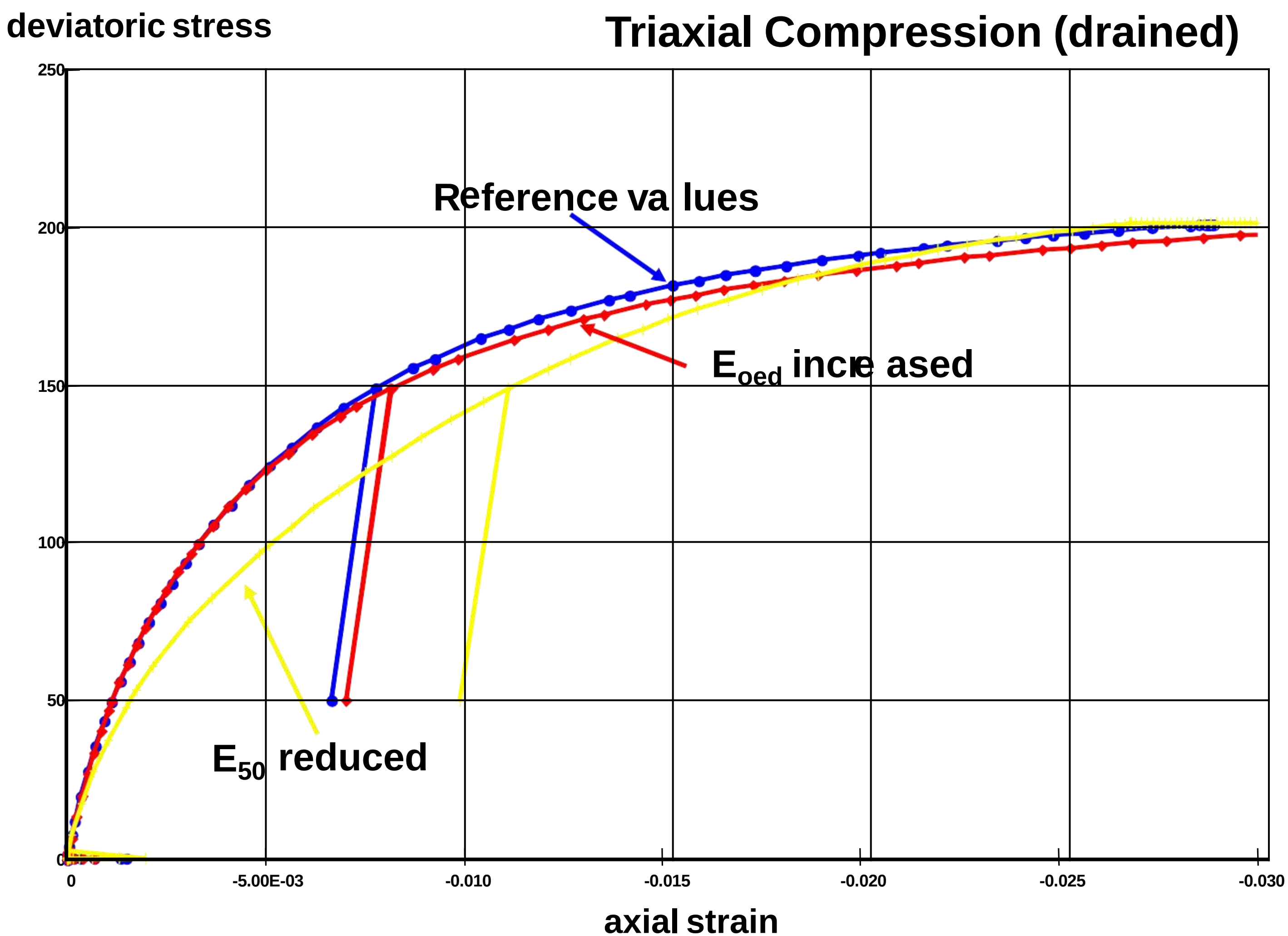
OEDOMETER TESTS ON LOOSE SAND



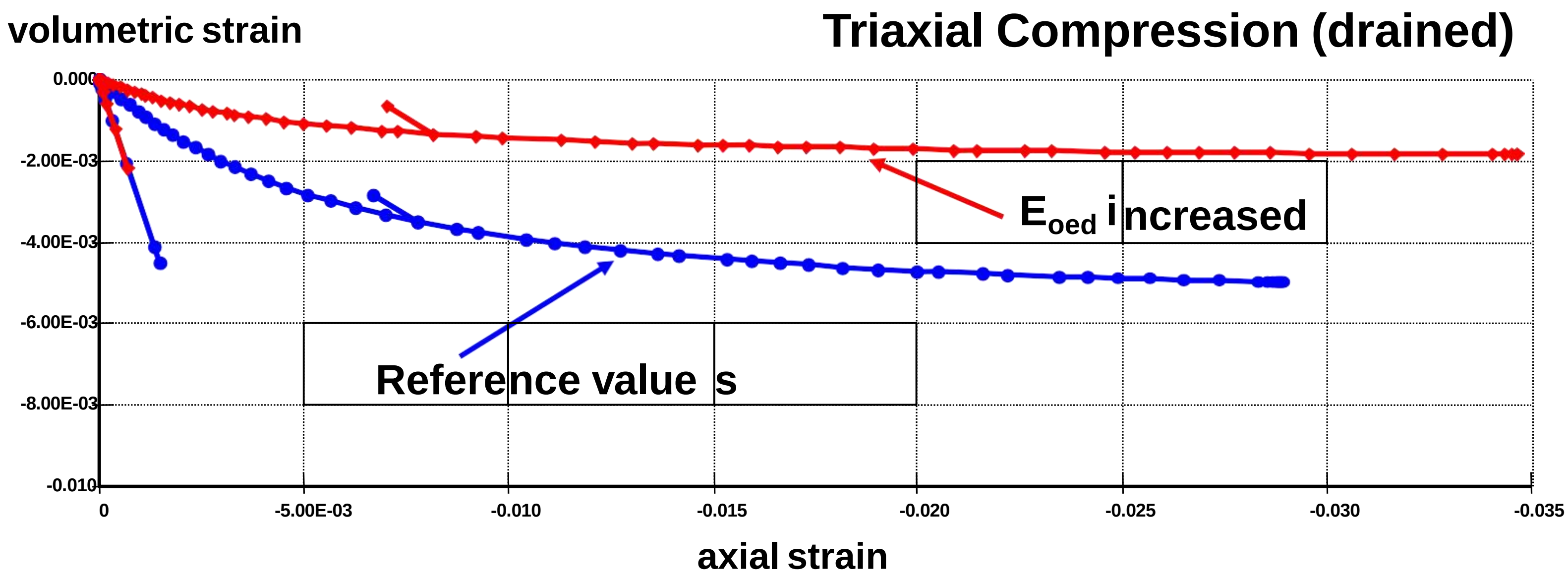
OEDOMETER TESTS ON DENSE SAND



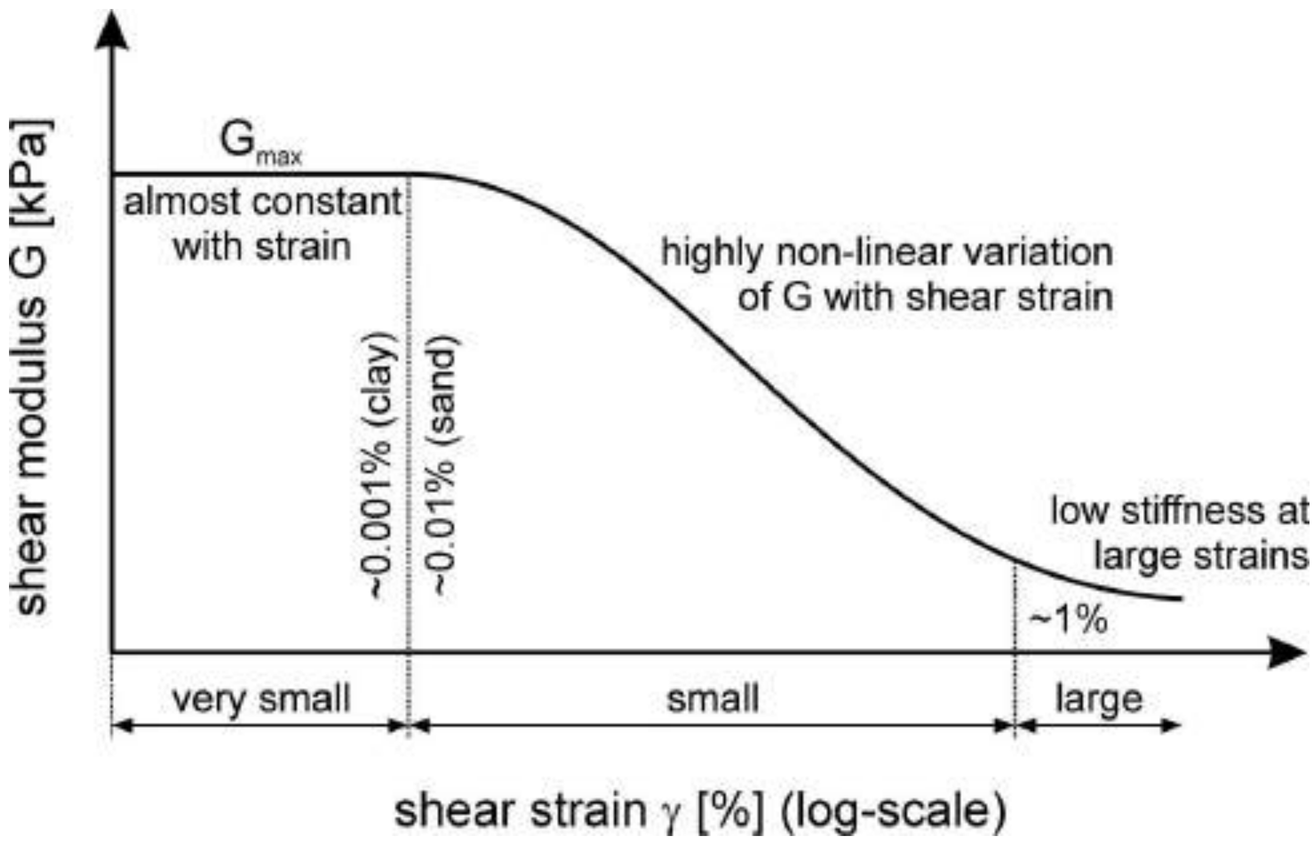
INFLUENCE E_{50} / E_{oed}



INFLUENCE E_{50} / E_{oed}



SMALL STRAIN STIFFNESS

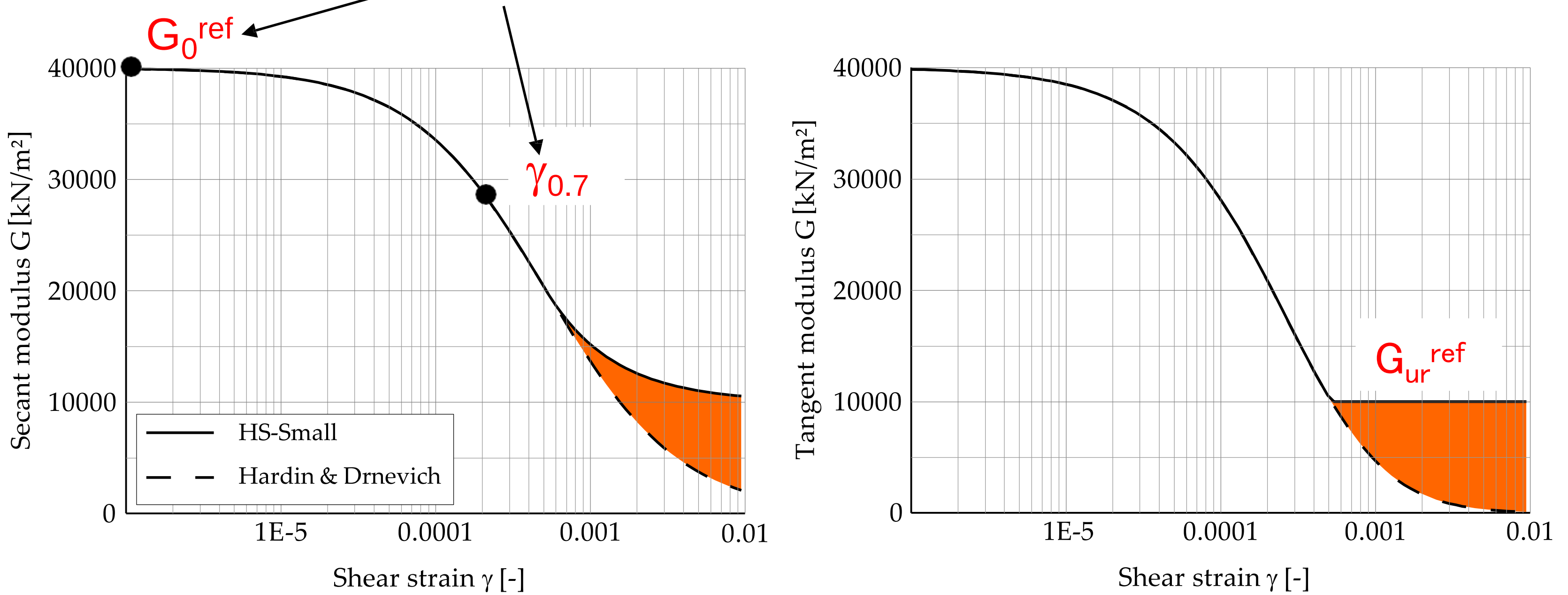


Typical curve of degradation of stiffness with strain

If the small strain stiffness model after Hardin & Drnevich predicts a stiffness lower than G_{ur}^{ref} (E_{ur}^{ref} respectively) the model switches to hardening plasticity of the standard Hardening Soil model.

IMPORTANT NOTE: flow rule for deviatoric yield surface (volumetric behaviour) is different in HSS model.

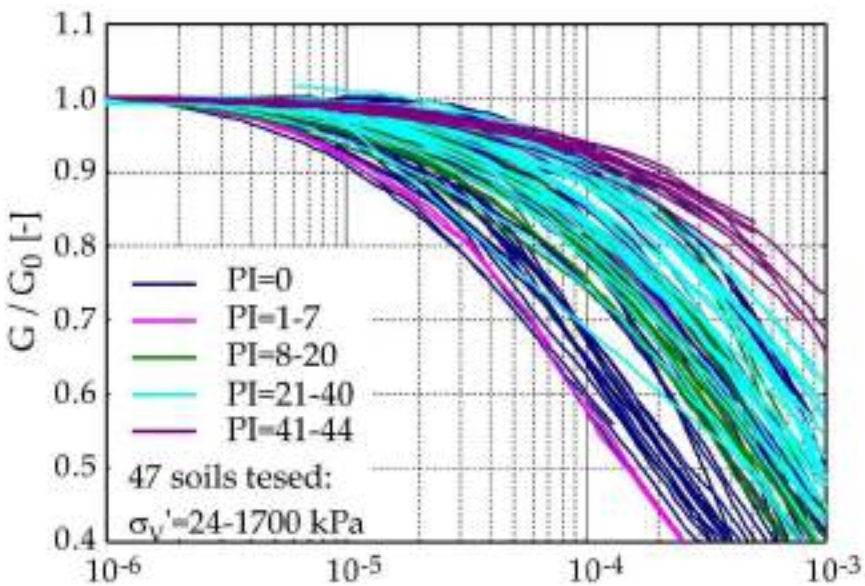
additional input parameters for HS-small



see also:
Thomas Benz, *Small-Strain Stiffness of Soils and its Numerical Consequences*, Mitteilung 55 des Instituts für Geotechnik, Universität Stuttgart, 2007.

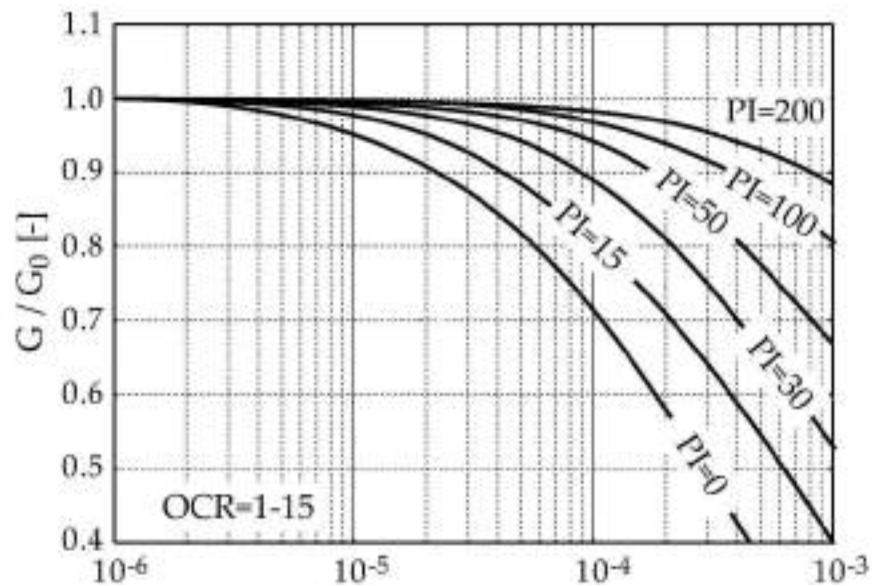
DATA FOR SMALL STRAIN STIFFNESS

Hsu & Vucetic



Shear strain γ

Vucetic & Dobry



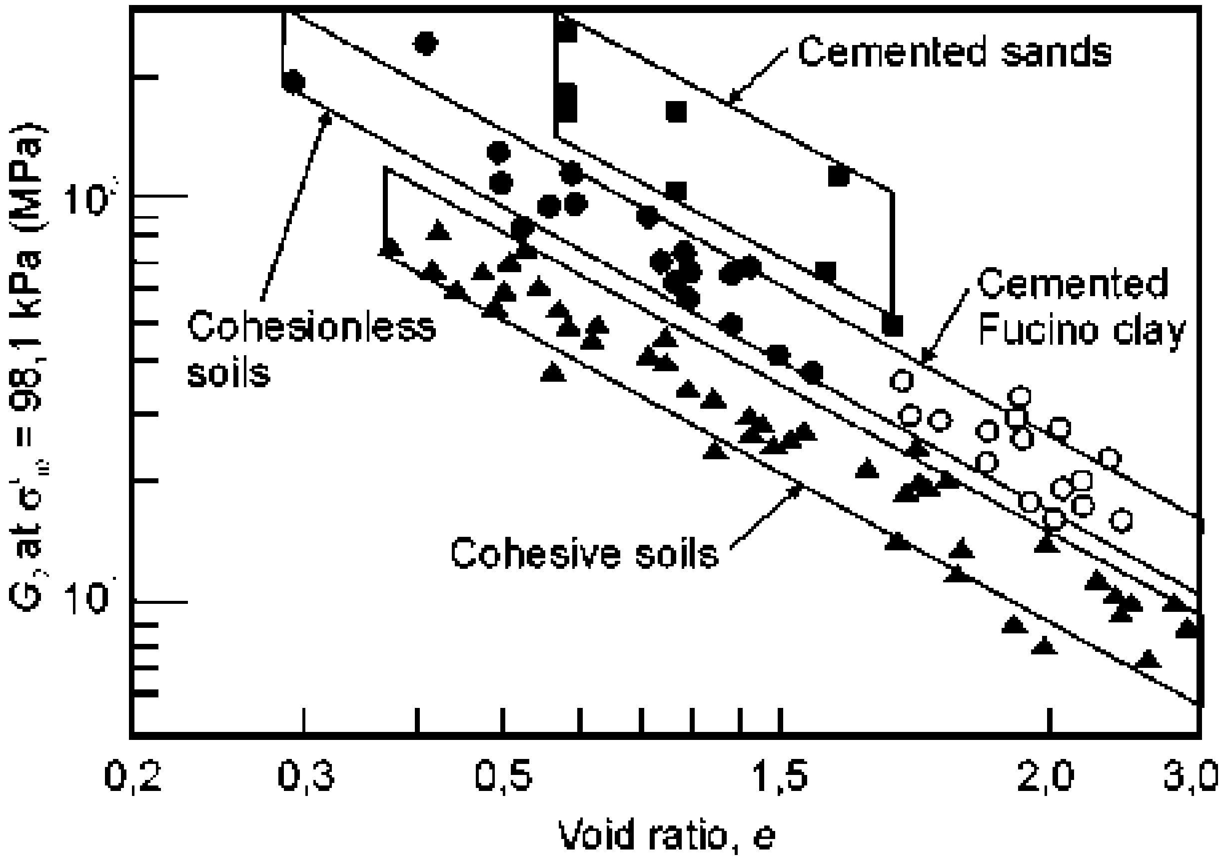
Shear strain γ

C-C. Hsu, M. Vucetic. Dynamic and cyclic behavior of soils over a wide range of shear strains in NGI-type simple shear testing device, UCLA Report ENG-02-228, 2002.

M. Vucetic, R. Dobry. Effect of soil plasticity on cyclic response, Journal of Geotechnical Engineering, ASCE 117 (1991), No. 1, 89-107.

DATA FOR SMALL STRAIN STIFFNESS

Typical values for G_0 (in MPa) for reference stress of 100 kPa



Jamiolkowski et al. 1991

DATA FOR SMALL STRAIN STIFFNESS

$$G_0^{ref} = \frac{(2.97 - e)^2}{1 + e} 33\ [MPa]$$ Hardin & Black (1969)

$$G_0^{ref} \approx RD \bullet 70MPa + 60MPa$$ Lengkeek

$$\gamma_{0.7} = \frac{0.385}{4G_0}\left[2c(1 + \cos(2\varphi)) - \sigma_1(1 + K_0)\sin(2\varphi)\right]$$ Benz (2007)

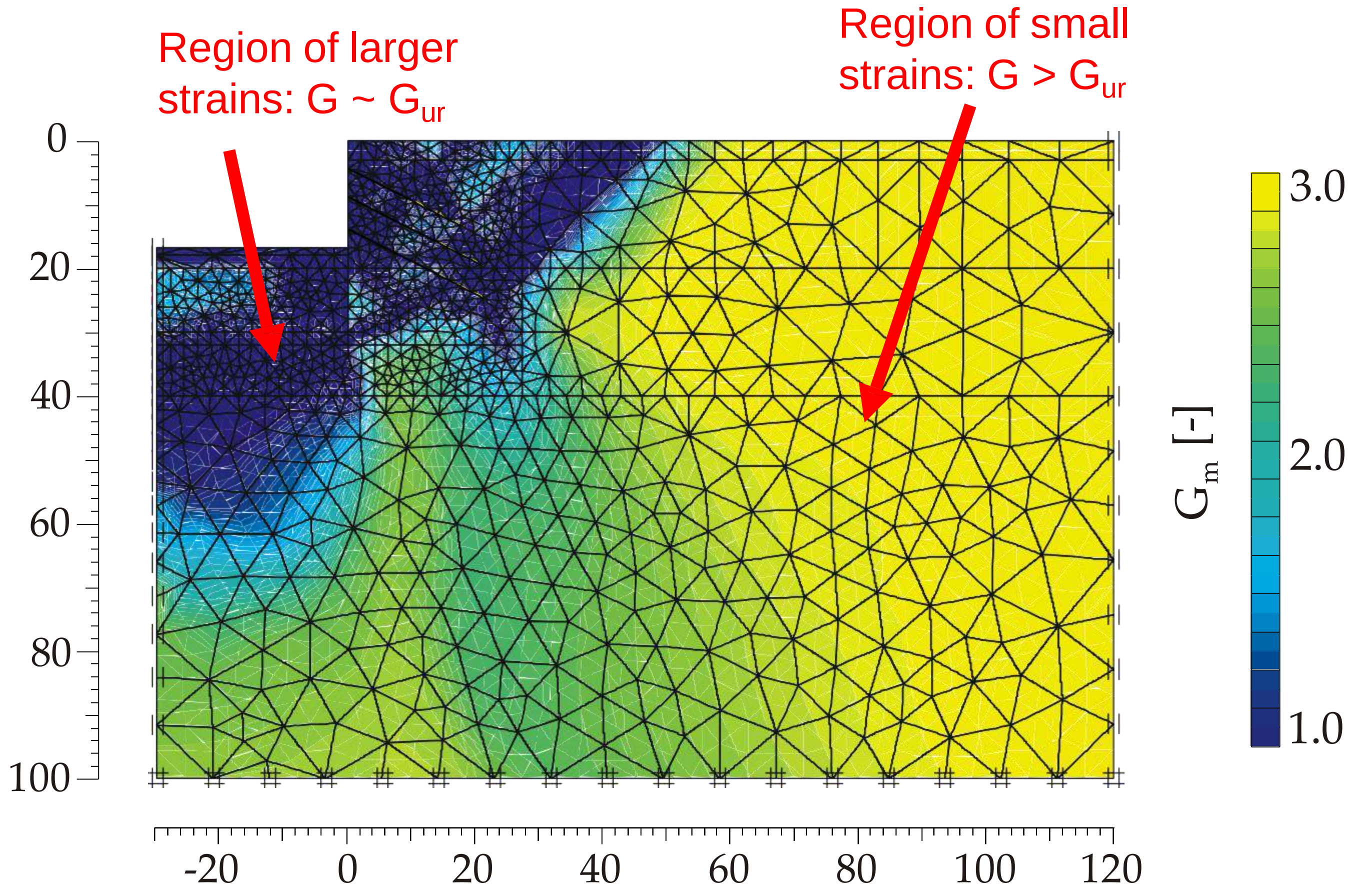
Order of magnitude:

$$G_0^{ref} = (2.5\ to\ 10) G_{ur}^{ref}$$

$$\gamma_{0.7} = (1\ to\ 2) \cdot 10^{-4}$$

where $G_{ur}^{ref} = \frac{\square_{ur} E^{ref}}{2(1 + \nu_{ur})}$

EXAMPLE DEEP EXCAVATION



$G_m = G^{ref} / G_{ur}^{ref}$

see also: Thomas Benz, *Small-Strain Stiffness of Soils and its Numerical Consequences*, Mitteilung 55 des Instituts für Geotechnik, Universität Stuttgart, 2007.

	Mohr-Coulomb Model	Hardening Soil Model	Hardening Soil Small Model
Stress dependent stiffness*	NO	YES	YES
Distinction in stiffness for primary loading and unloading / reloading	NO	YES	YES
Plastic strains for stress states below MC - failure line **(deviatoric and volumetric hardening)**	NO	YES	YES
Failure according to Mohr-Coulomb	YES	YES	YES
Small strain stiffness	NO	NO	YES

* (not only dependent on σ_0, this is possible also with MC-Model)

CALIBRATION OF HS MODEL

PLAXIS ADVANCED COURSE, BANDUNG, INDONESIA

INTRODUCTION

In daily engineering practice soil parameters are obtained from one or more laboratory tests. In order to perform the best possible Plaxis calculation these soil parameters have to be translated into input parameters for the constitutive model used, taking into account the possibilities and limitations of the constitutive model. Most parameters for the constitutive models used in Plaxis can be determined directly from standard laboratory tests as triaxial tests and oedometer tests. However, due to the complexity of the models it is recommended to not simply accept the parameters determined from those tests, but to actually model the tests and see if the parameters found actually give a proper representation of the real laboratory test results within the limits of the constitutive models.

In the past simulation of laboratory tests with Plaxis could only be done by creating a complete finite element model taking into account the appropriate loads and boundary conditions. Since Plaxis V8, however, a special tool is included in the software to simply simulate laboratory tests without the need for making a finite element model: the SoilTest tool.

In this exercise the SoilTest tool will be used for the simulation of both oedometer and triaxial tests on sand.

CONTENT

SIMULATION OF LABORATORY TESTS

In this exercise results from oedometer and triaxial tests are presented. The aim is to determine the parameters for the Hardening Soil model such that a simulation of the tests within Plaxis gives the best possible results compared to the original laboratory tests. In short:

1. Determine soil parameters based on the given soil laboratory tests results
2. Perform the laboratory tests using SoilTest with the parameters found
3. Match SoilTest results with the original laboratory results to find the best matching model parameters for the Hardening Soil model.

Exercise 1: LABORATORY TESTS ON SAND

Parameter determination

On a sample of dense sand both oedometer tests and triaxial tests have been performed. The results of those tests are given in the figures below. Use these figures to determine the parameters for the Hardening Soil model and collect the parameters in Table 1. Note that it is possible that some parameters cannot be determined with the given laboratory results, these parameters have to be estimated.

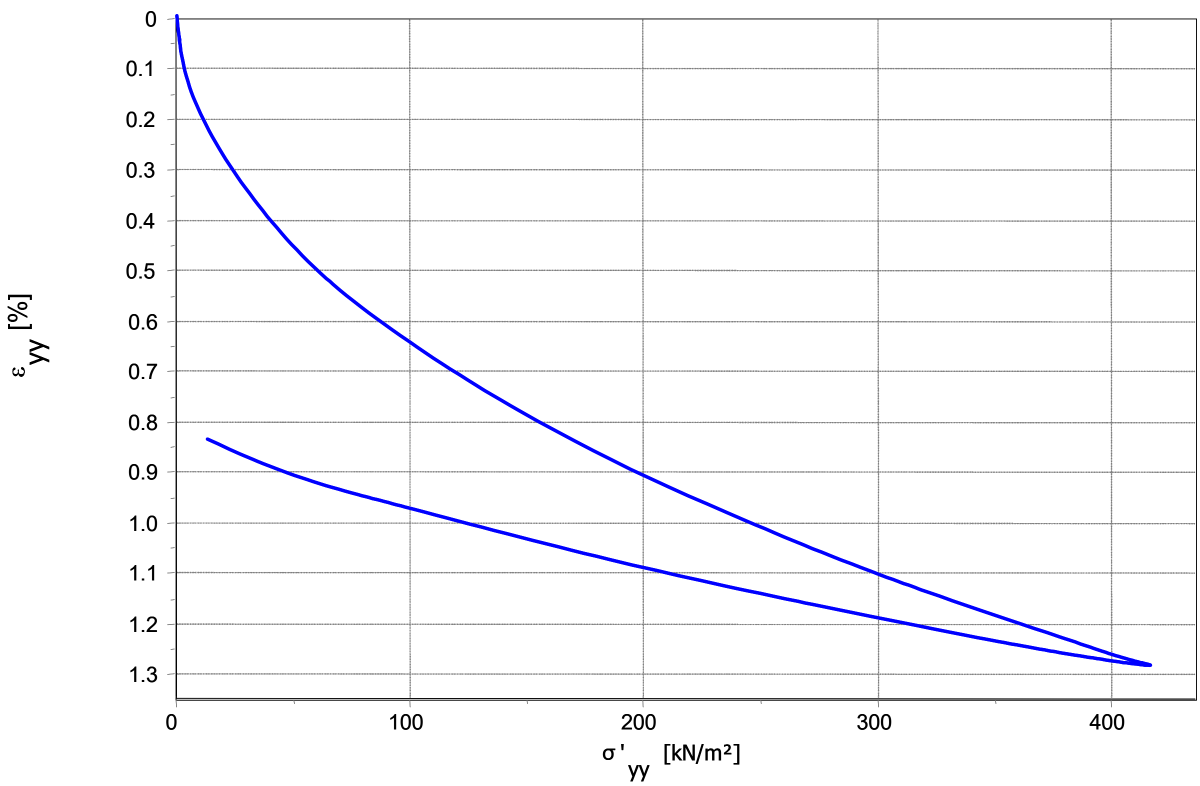


Figure 1: Oedometer test results

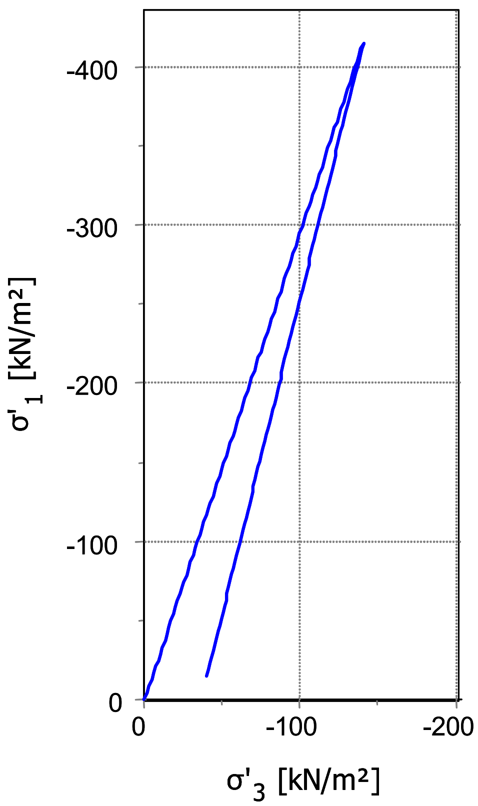


Figure 2: Development of horizontal and vertical stress in oedometer test

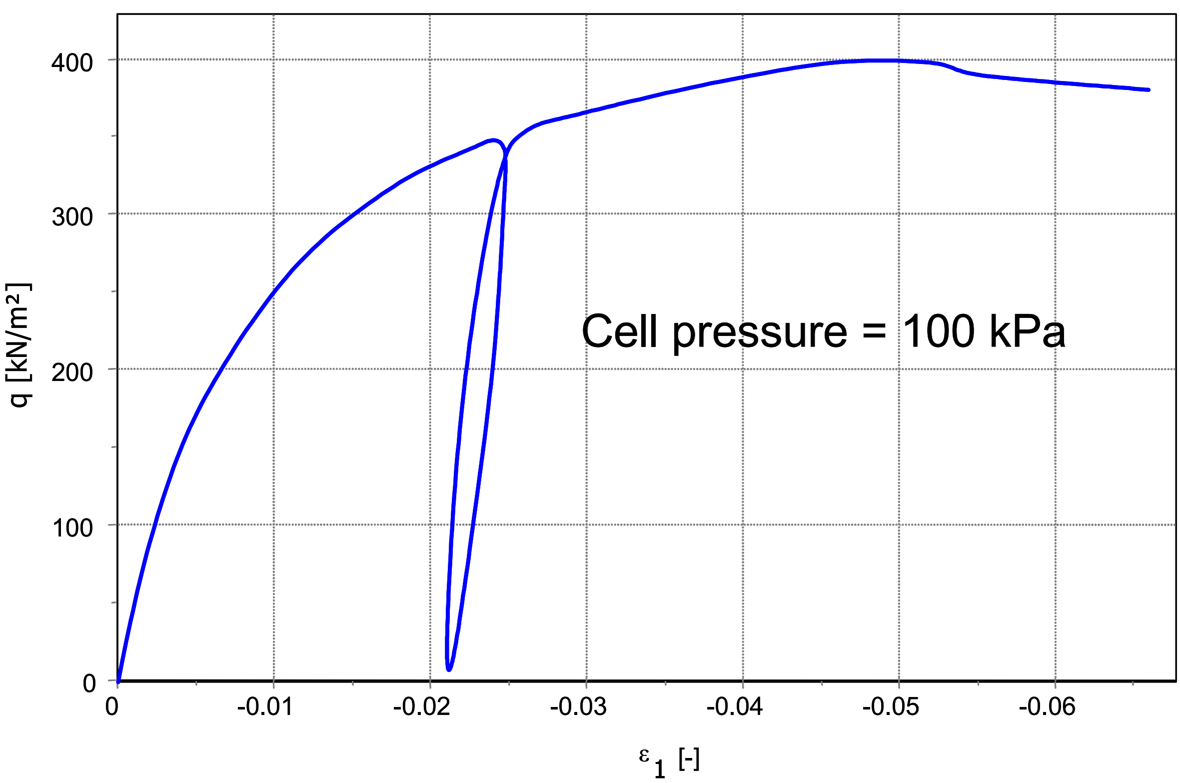


Figure 3: Drained triaxial test unloading-reloading (cell pressure = 100 kPa)

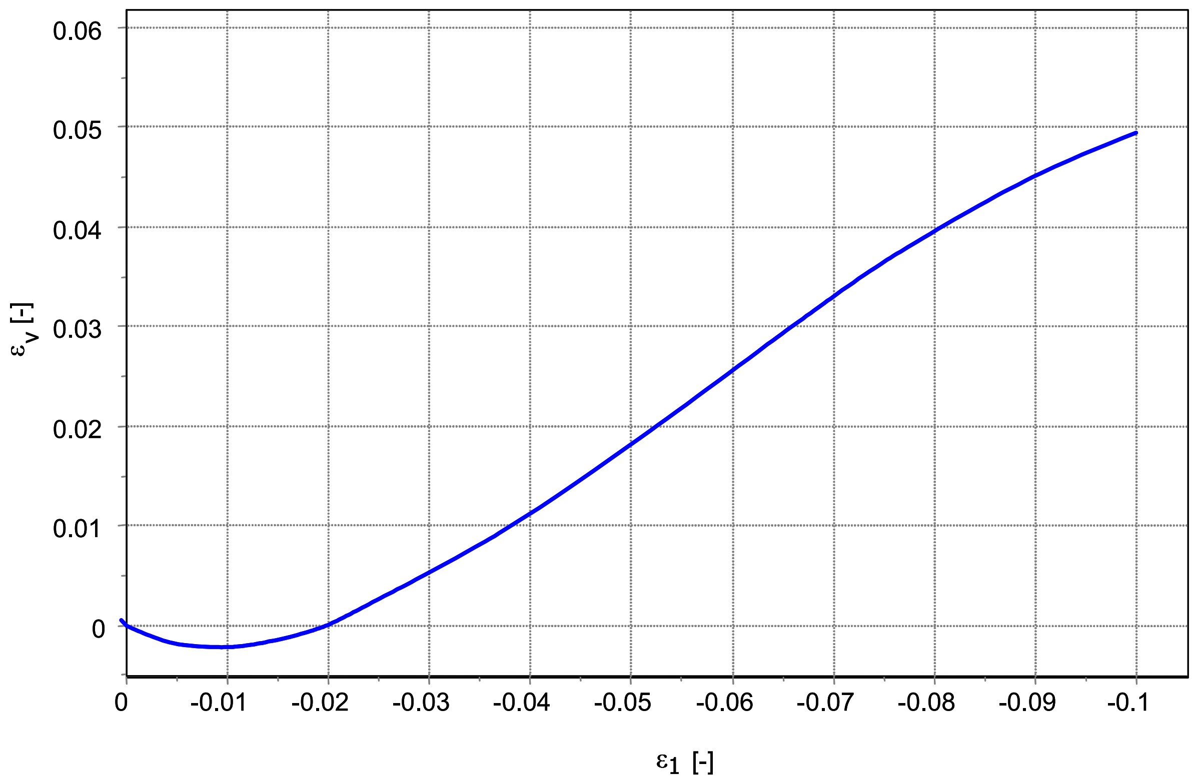


Figure 4: Axial vs. volume strain in drained triaxial test

Collect the soil parameters in table 1:

Parameter	Unit	Value
E_{50}^{ref}	[kPa]	
E_{oed}^{ref}	[kPa]	
E_{ur}^{ref}	[kPa]	
ν_{ur}	[-]	
c'	[kPa]	
ϕ'	[°]	
ψ [	[°]	
m	[-]	
K_0^{NC}	[-]	

Table 1: Hardening Soil Parameters of the Sand

With these data perform a triaxial test in the SoilTest program.

APPENDIX B: INTRODUCTION TO THE SOILTEST TOOL

For the simulation of laboratory tests Plaxis offers the SoilTest tool based on a single stress point calculation that makes it possible to do fast simulations without the need for a finite element mesh. The SoilTest tool can be called from within the material sets database or from within the definition of a material set. (see Figure 9).

In the following paragraphs a step-by-step description is given on how to model both an oedometer test and a triaxial test with the help of many screen shots of the SoilTest tool.

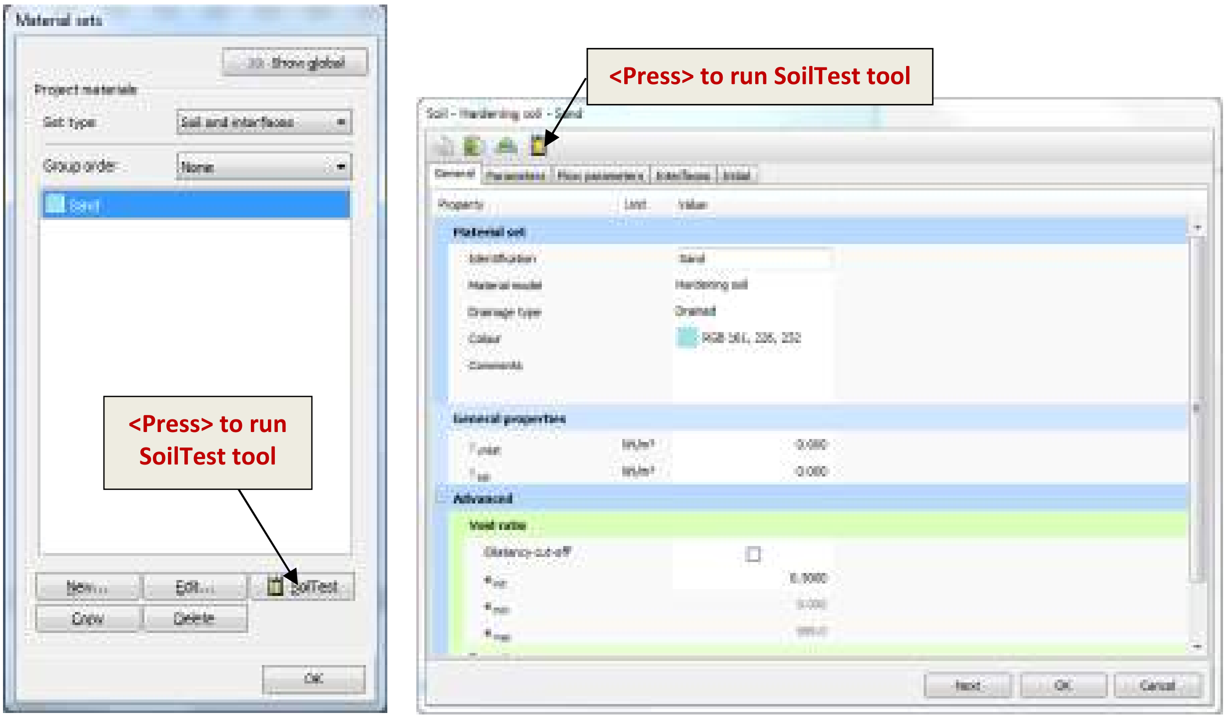


Figure 5: The SoilTest tool

How to Model an oedometer test

In order to model an oedometer test first the material data set has to be created. After doing so, press the <SoilTest> button to start the SoilTest tool. The window that opens is shown in Figure 10.

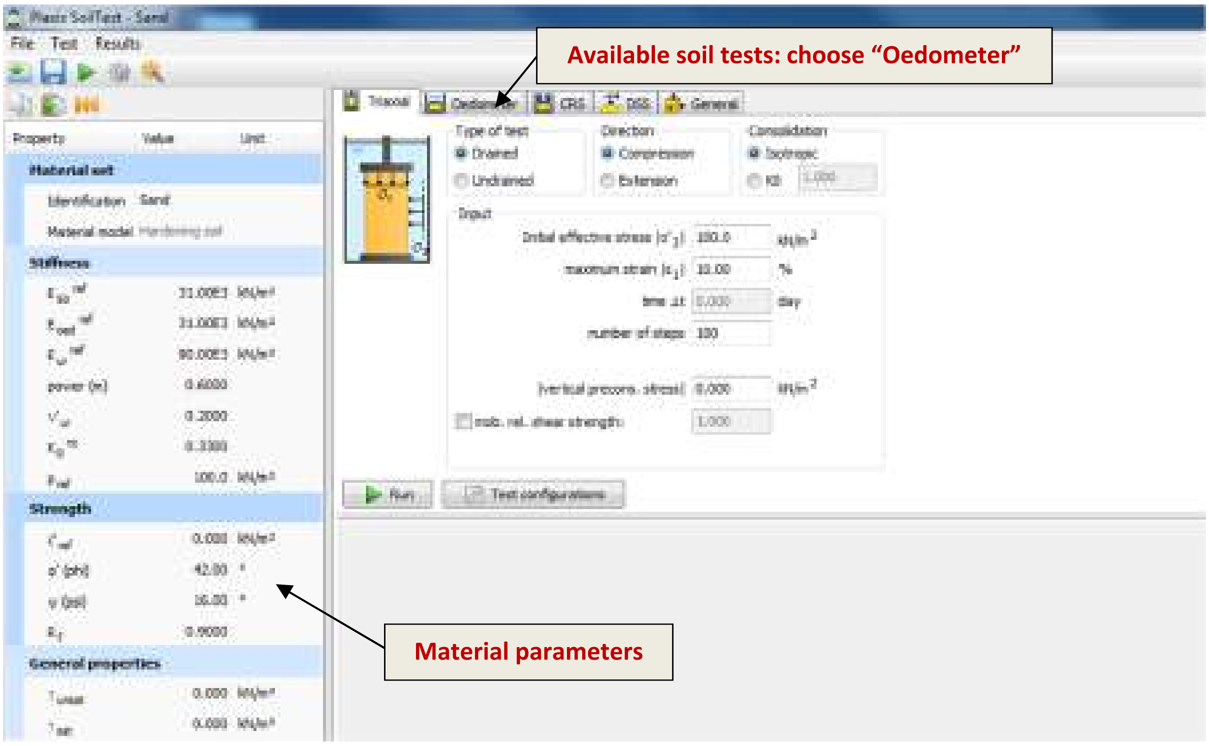


Figure 6: SoilTest tool

In the main window select the *Oedometer* tabsheet and set the parameters as indicated in Figure 11.

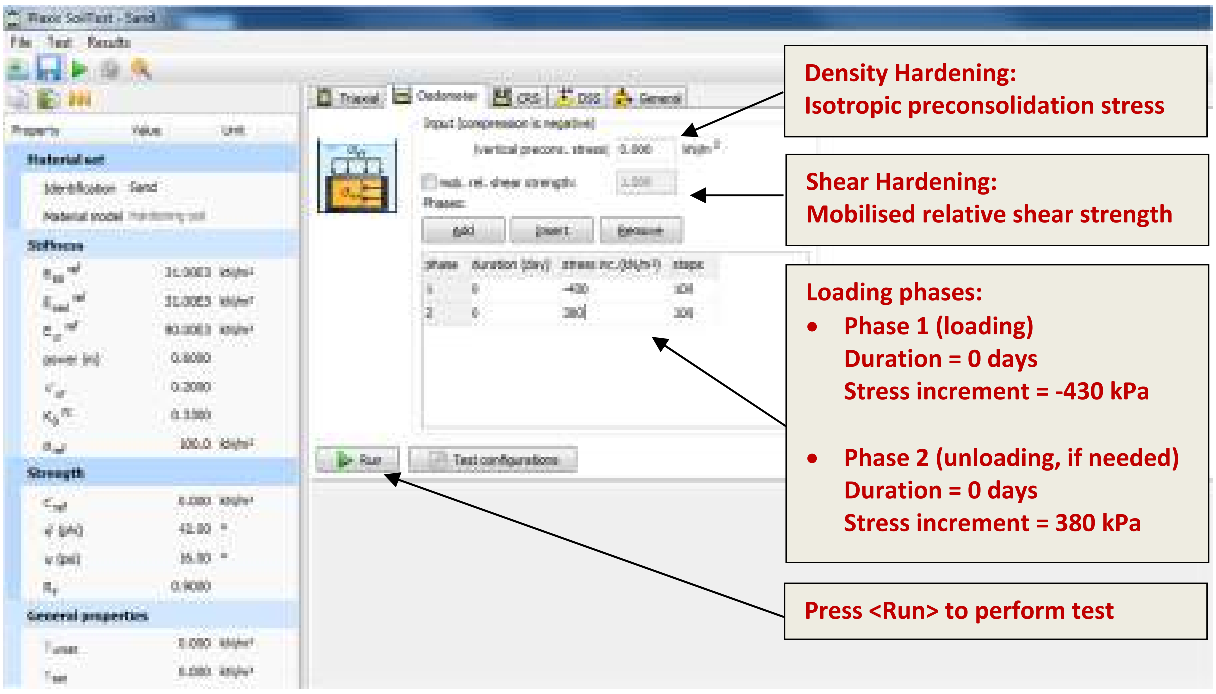


Figure 7: Setting the oedometer parameters

After the oedometer test has been calculated, graphs with results appear at the bottom of the SoilTest window. The user can double-click these graphs to view them in separate windows. Furthermore, custom charts can be added, see Figure 12.

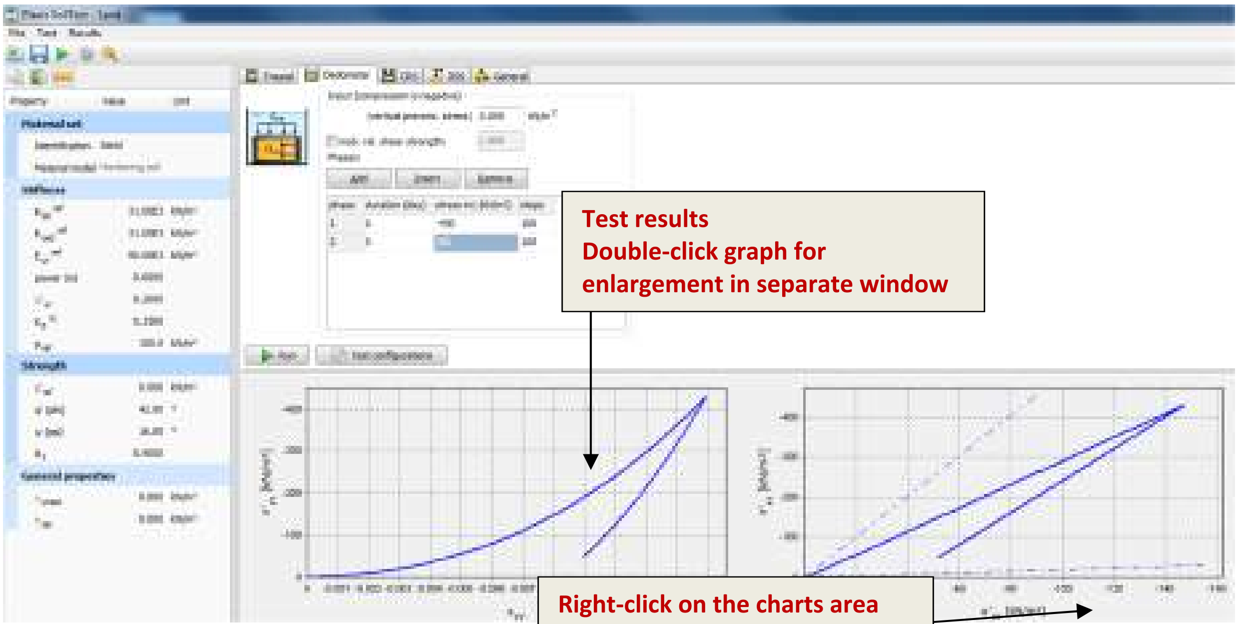


Figure 8: Inspecting the oedometer test results

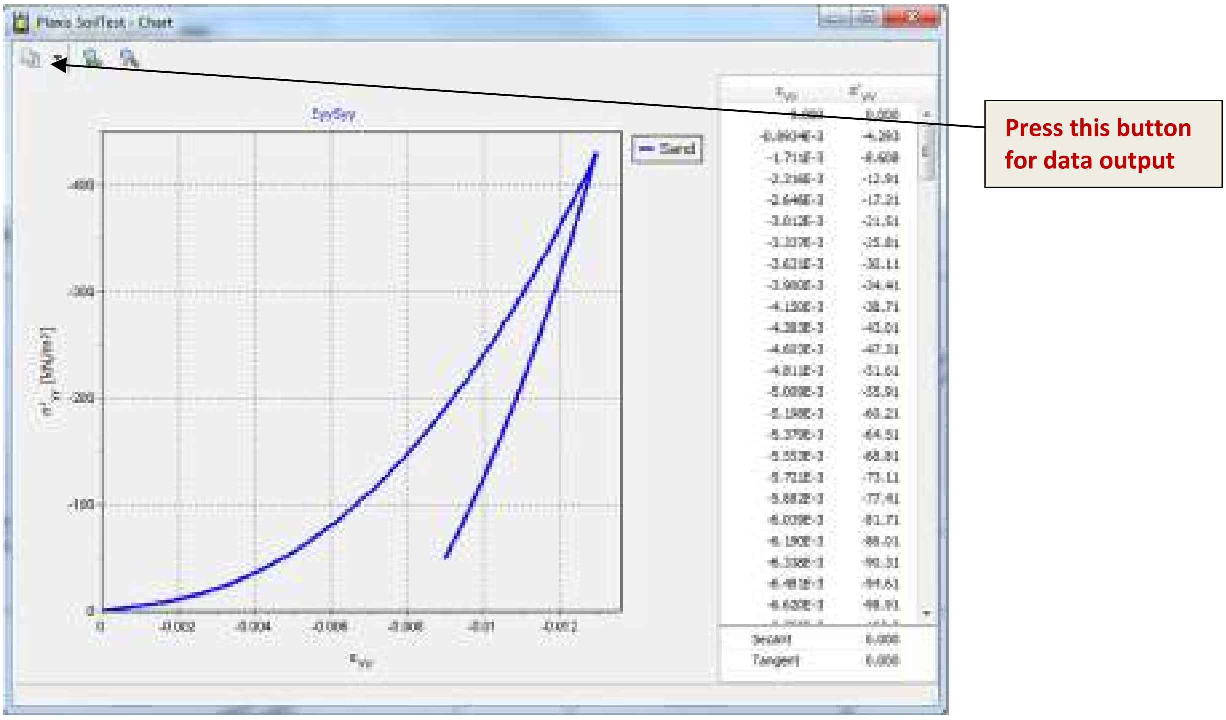


Figure 9: Separate window

Figure 10: Data output

How to Model a triaxial test

From the material database or the material set definition window press the <SoilTest> button to start the SoilTest tool. In the main window choose the tabsheet *Triaxial* and set the type of test as well as the test parameters as shown in Figure 13.

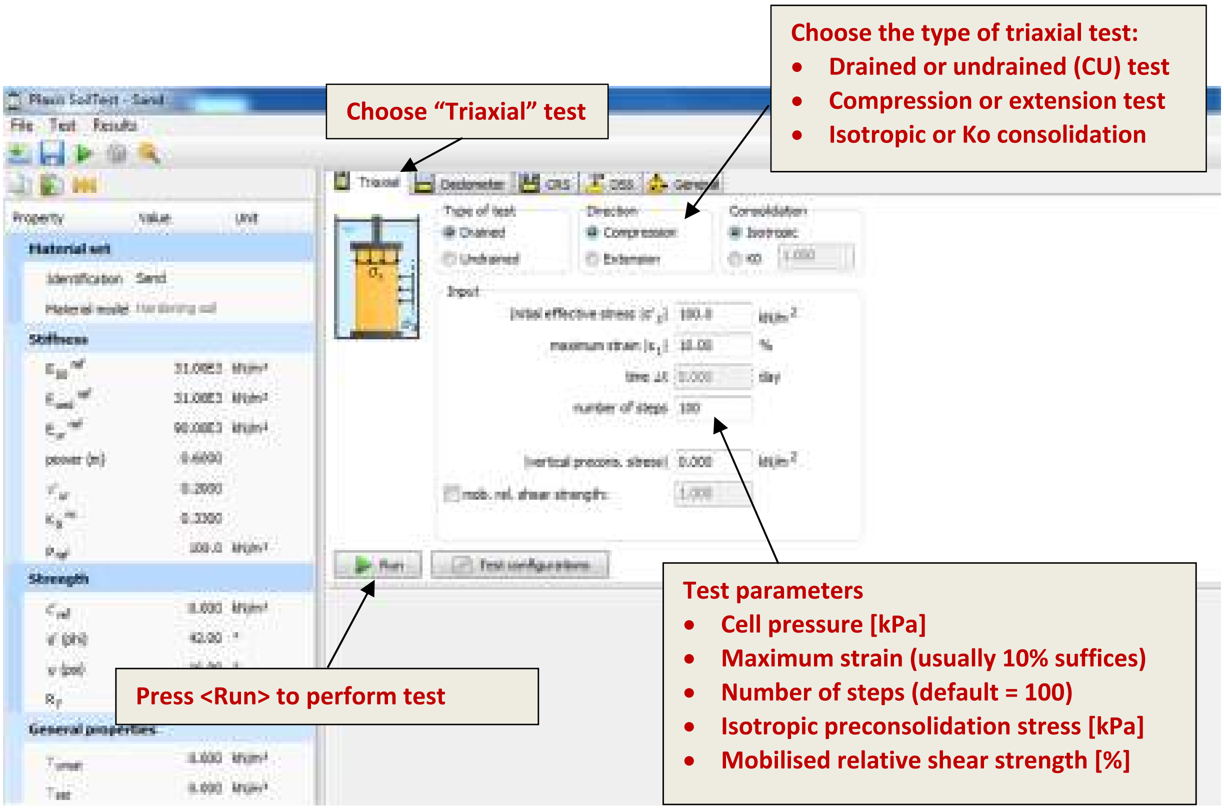


Figure 11: Defining a triaxial test

After the triaxial test has been calculated, graphs with results appear at the bottom of the SoilTest window. As described above for the oedometer test, the user can double-click this graphs to view them in separate windows as well as add custom charts.

Modelling a triaxial test with unloading/reloading

The standard functionality in SoilTest for simulation of a triaxial test does not allow intermediate unloading-reloading paths. However, the SoilTest functionality contains a *General* option with which soil test can be defined in terms of boundary stresses or strains on all sides of a soil test cube. Hereafter it will be shown how this can be used for the simulation of a triaxial test with an unloading/reloading path.

After opening the SoilTest option from the material set definition window, the tabsheet General should be chosen. On this tabsheet a list of calculation phases can be defined where stress or strain increments can be applied.

Initial phase

First of all we have to specify whether stresses or strains will be applied on the boundaries during the test. For this exercise stresses will be applied. Now the values of the initial stresses on the soil sample have to specified. For a triaxial test the initial stresses are the cell pressures acting on the soil, hence for σ_{xx}, σ_{yy} and σ_{zz} the cell pressure has to be entered. The cell pressure is a water pressure, there will be no shear stress acting on the soil: $\tau_{xy} = 0$. See Figure 14 for details.

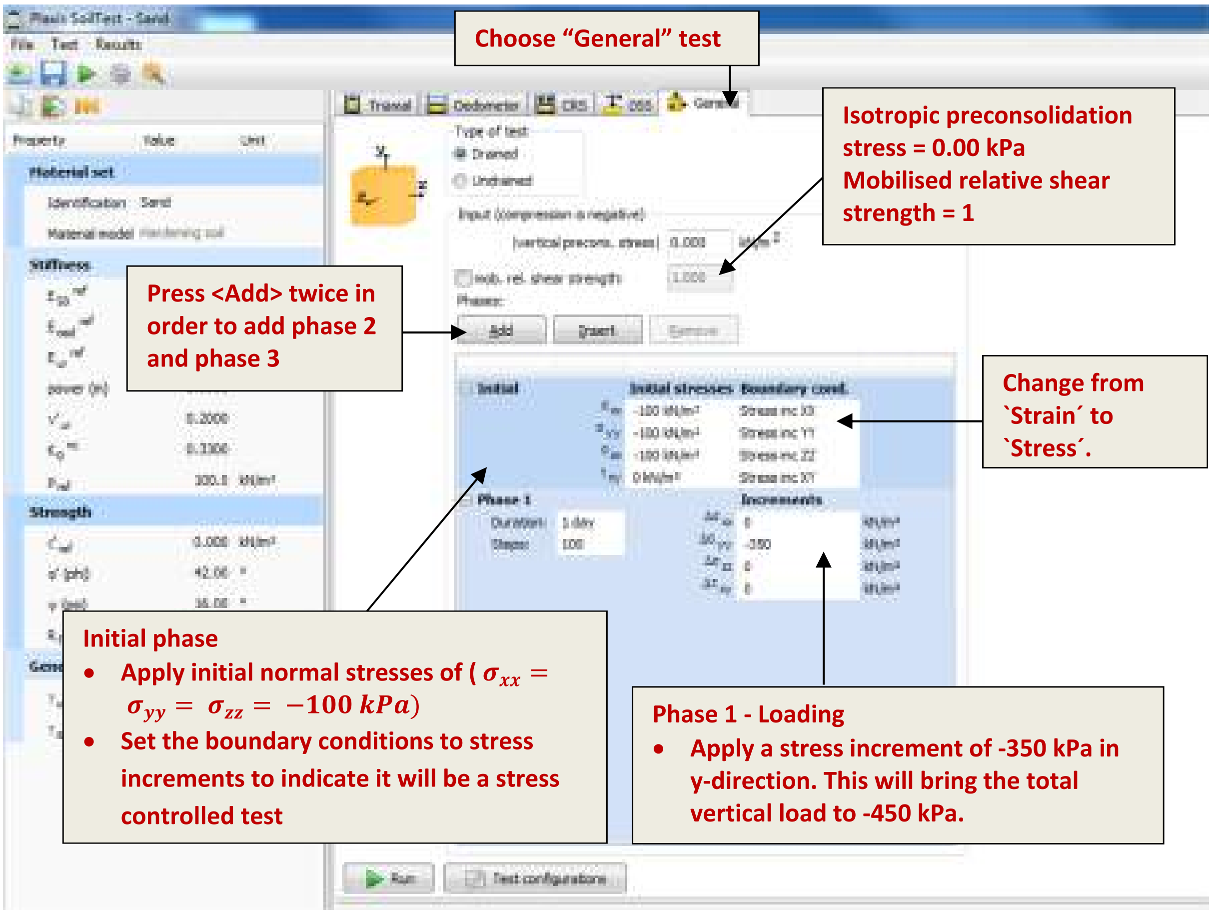


Figure 12: General option for simulation of laboratory tests used for triaxial test

Phase 1

Apply a stress increment in vertical direction (σ_{yy}) until the stress level where the unloading path should start. Note that the horizontal stresses (σ_{xx} and σ_{zz}) remain the same as they represent the cell pressure. Hence, the horizontal stress increments are zero in this phase.

Phase 2

Press the <Add> button to add another phase to the phase list. This phase represents the unloading phase. See Figure 17 for details.

Phase 3

Press the <Add> button once more in order to add the 3rd phase. This phase represents the reloading of the soil as well as the continuation of primary loading until either failure or a higher stress level from where, for instance, another unloading/reloading cycle can be defined.

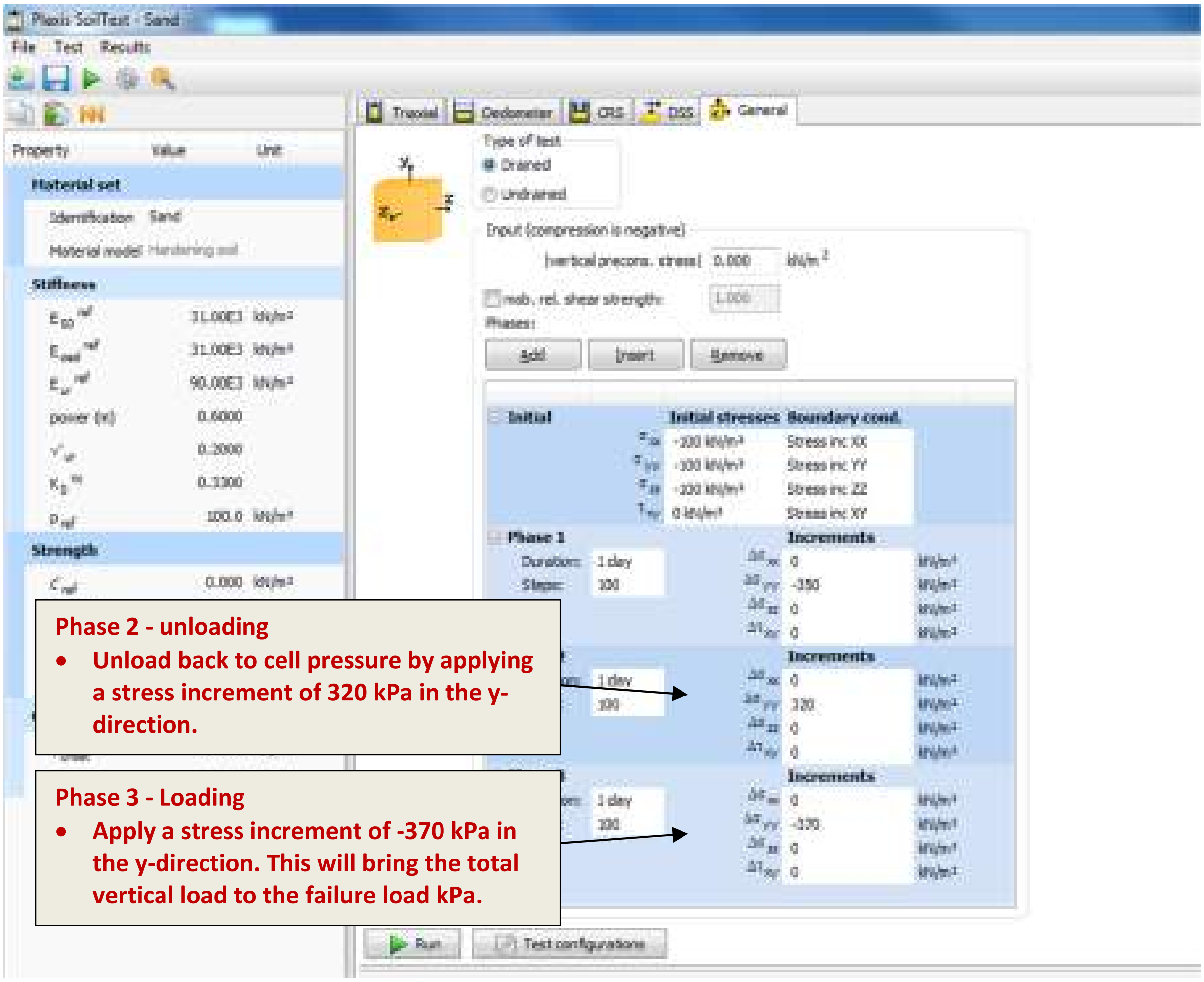


Figure 13: Unloading/reloading cycle in a triaxial test using the General option

APPENDIX C: COMPARISON OF THE CALCULATED AND ORIGNIAL TEST RESULTS

The blue data is the original soil laboratory data and the red one is the calculated behaviour of the HS-Material based on the laboratory data. As indicated in the following figures, the numerical results with the obtained HS parameters give a proper representation of the soil tests.

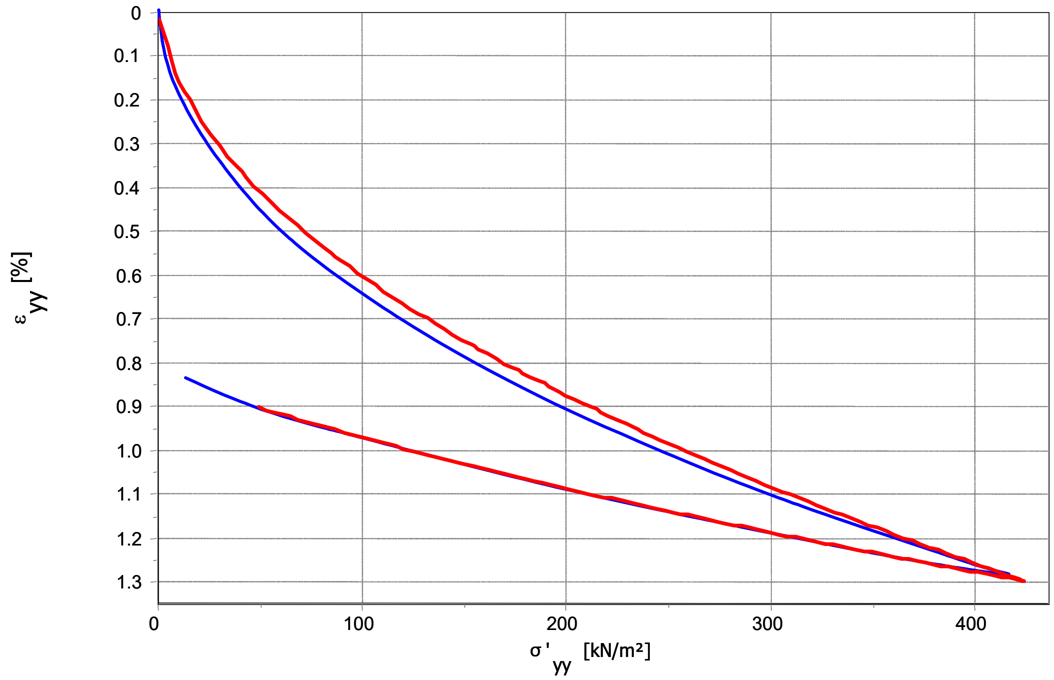


Figure 14: Oedometer test results

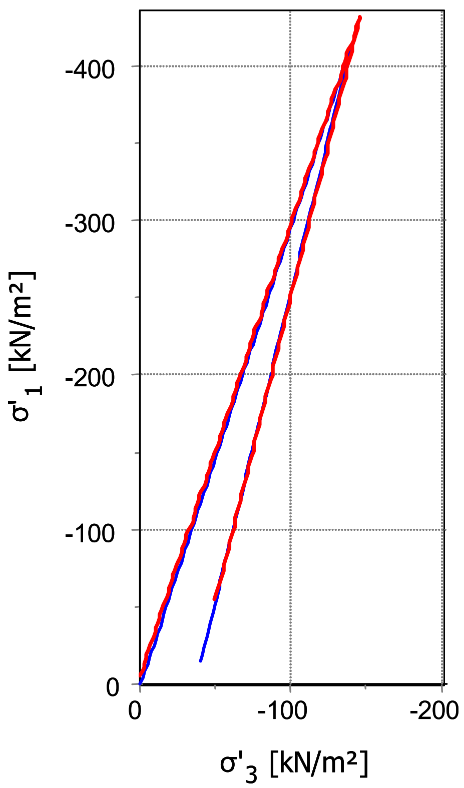


Figure 15: Development of horizontal and vertical stress in oedometer test

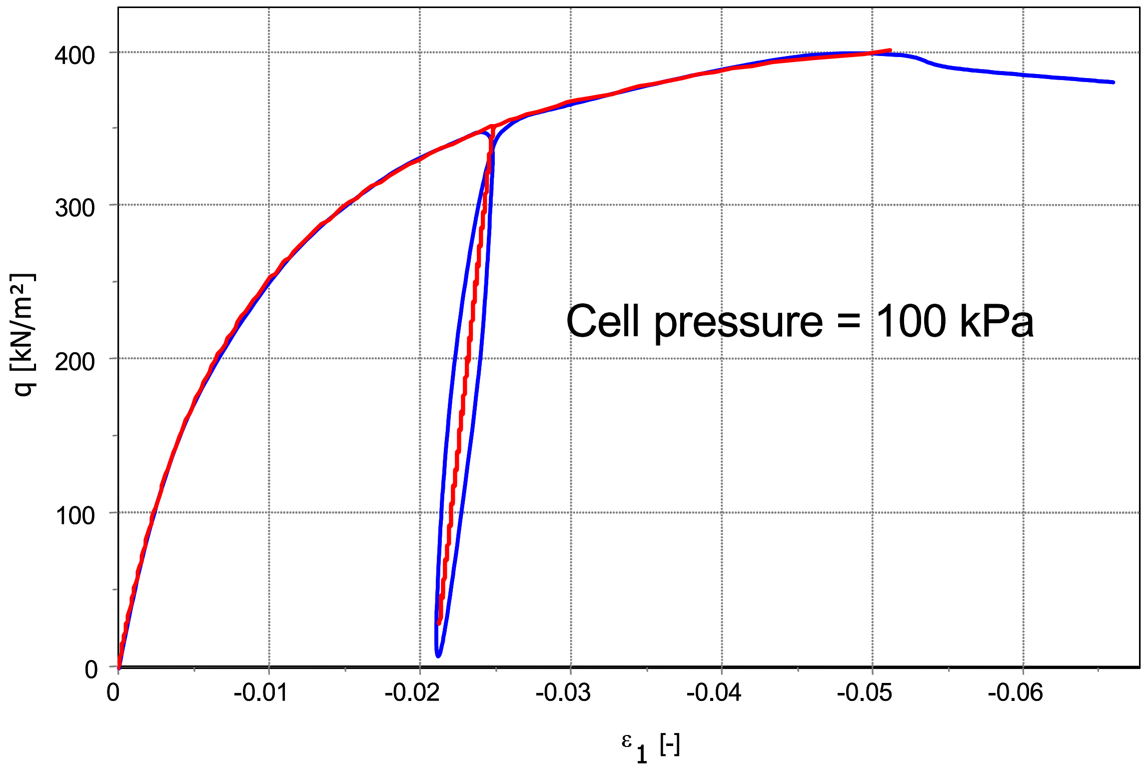


Figure 16: Drained triaxial test unloading-reloading (cell pressure = 100 kPa)

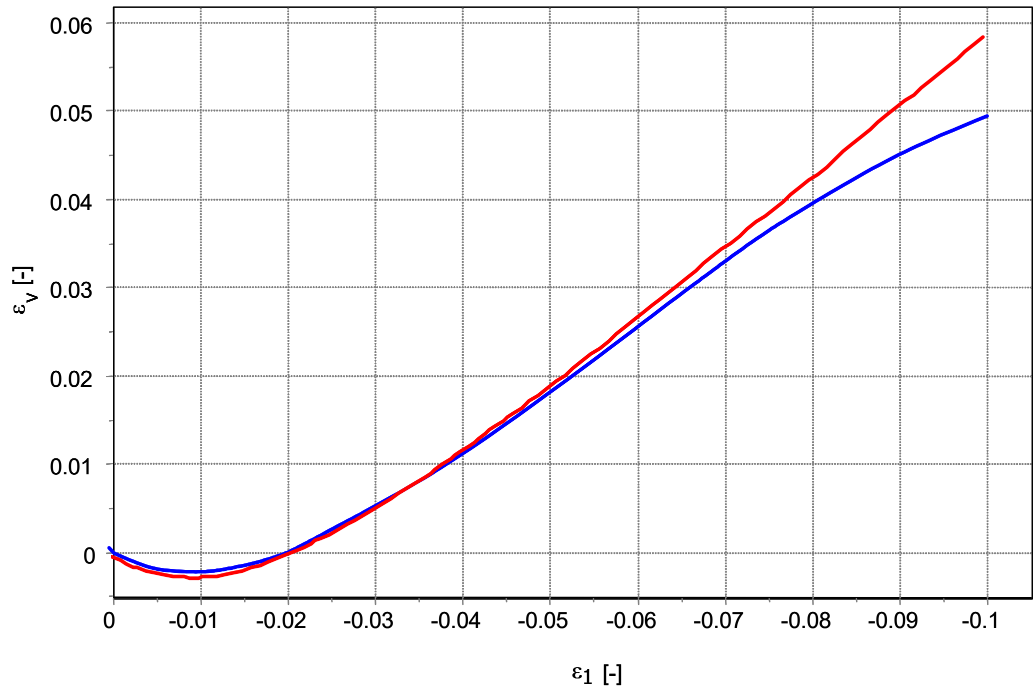


Figure 17: Axial vs. volume strain in drained triaxial test (Standard Triaxialtest - cell pressure=100[kPa])

APPENDIX A: PARAMETER DETERMINATION

Sand

First we determine parameters from the triaxial test data.

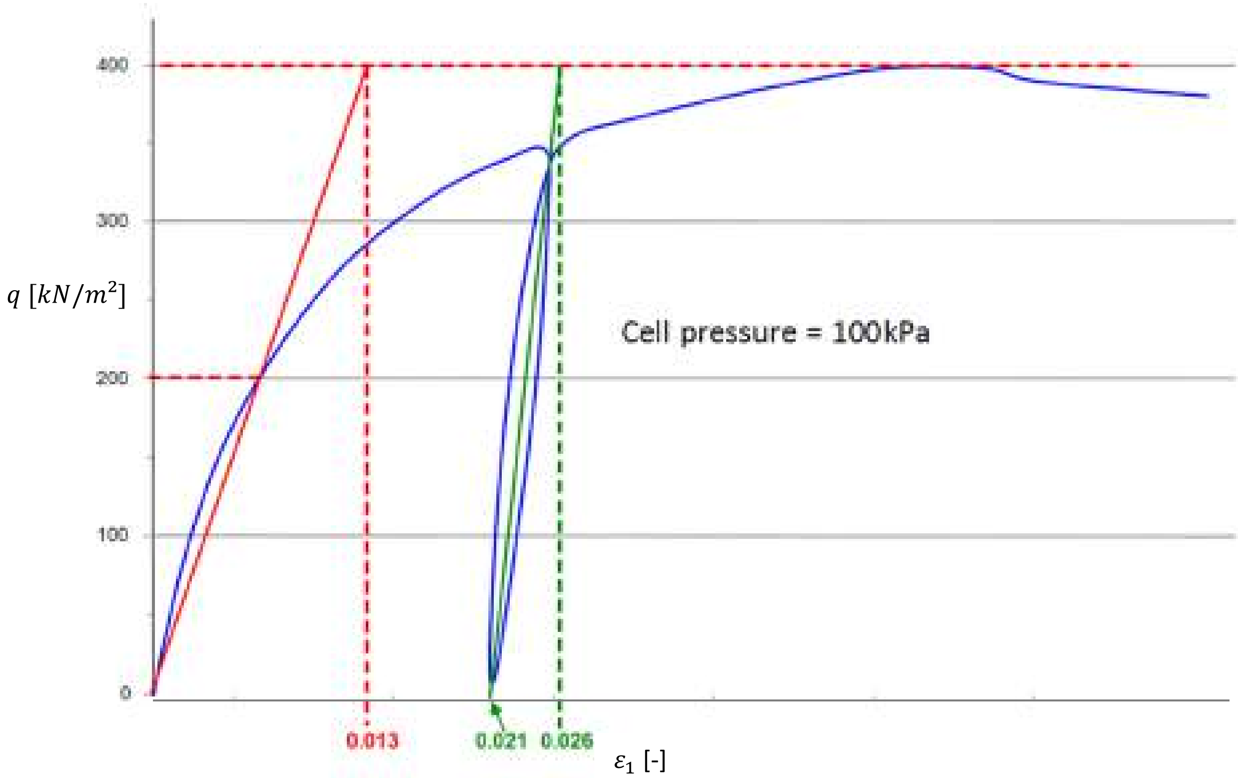


Figure 1: Determine stiffness parameters from drained triaxial test

Cohesion and friction angle

For a cell pressure σ_3 = 100 kPa a maximum value of approximately $|\sigma_1\text{-}\sigma_3|$ = 400 kPa is reached at failure. The Mohr-Coulomb failure criterion is:

$$\frac{1}{2} * (\sigma_1 - \sigma_3) - \frac{1}{2} * (\sigma_1 + \sigma_3) * \sin\varphi - \mathrm{c} * \cos\varphi = 0$$

Considering it is sand we assume that the cohesion is zero and so the Mohr-Coulomb failure criterion reduces to:

$$\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \sin\varphi$$

Filling in σ_3 = 100 kPa and σ_1 = 500 kPa as obtained from the test we find for the friction angle ϕ=42°

Reference stiffness from triaxial test

The triaxial test stiffness E_{50} is the secant stiffness over the first 50% of the failure value for $|\sigma_1\text{-}\sigma_3|$. This is indicated in red in the triaxial test graph of Figure 5.

$$E_{50}^{\sigma_3=100kPa} = \frac{400}{0{,}013} = 30800\ kPa$$

The triaxial test stiffness E_{50} is within the Hardening Soil model defined as:

$$E_{50} = E_{50}^{ref} * \left(\frac{c\ * \cos\varphi\ -\ \sigma_3 * \sin\varphi}{c\ *\ \cos\varphi + \ p_{ref}\ *\ \sin\varphi}\right)^m\ , c = 0 \rightarrow\ E_{50} = E_{50}^{ref} * \left(-\frac{\sigma_3}{p_{ref}}\right)^m$$

The reference stress p_{ref} is chosen equal to the cell pressure of this triaxial test (100kPa).

$E_{50}^{ref} = E_{50}^{\sigma_3=100kPa} \approx 31000\ \mathrm{kPa}$

Reference unloading-reloading stiffness

Similar to the determination of the E_{50}^{ref} value the reference unloading-reloading stiffness can be determined. Therefore an unloading-reloading cycle is performed.
The Hardening Soil model does not have unloading-reloading behaviour with hysteresis but simple non-linear elastic unloading-reloading behaviour. A secant value is taken for the unloading-reloading behaviour, as indicated with the green line in the triaxial test results.

$$E_{ur}^{\sigma_3=100\,\mathrm{kPa}} = \frac{400}{0.026-0.021} = 80000\,\mathrm{kPa}$$

Under the same assumptions as for the stiffness in triaxial testing counts:

$E_{ur}^{ref} = E_{ur}^{\sigma_3=100\,\mathrm{kPa}}$

But this is a low value for the unloading reloading stiffness and so

$E_{ur}^{ref} = 90000$ is chosen

Dilatancy angle

From the plot of axial strain versus volume strain the dilatancy angle can be determined according to

$$\sin\psi = \frac{\Delta\varepsilon_v}{-2\Delta\varepsilon_1 + \Delta\varepsilon_v}$$

See Figure 6 for details.

With $\Delta\varepsilon_v$=0.048-0.004 = 0.044 and $\Delta\varepsilon_1$ = -0.09-(-0.03) = -0.06 the dilatancy can be calculated as ψ=16°.

Note: The Poisson's ratio needed for the Hardening Soil model ***cannot*** be determined from the given graphs since the Poisson's ratio needed is an <u>unloading-reloading</u> Poisson's ratio.

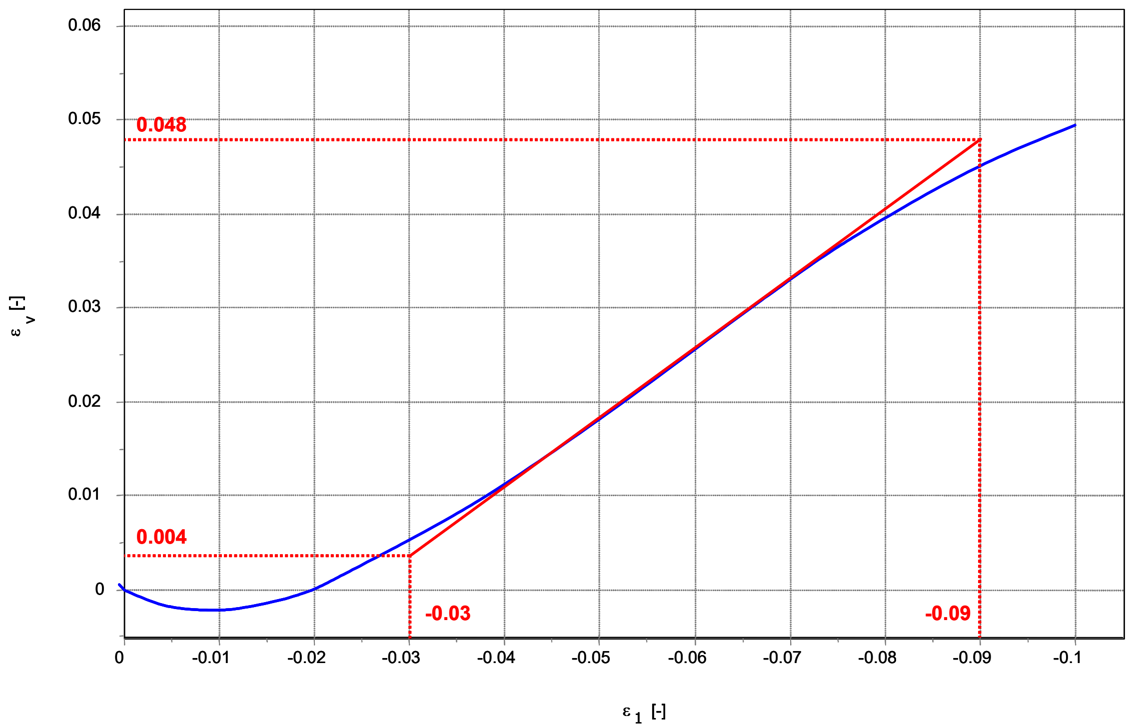


Figure 2: Determination of dilatancy angle from drained triaxial test

Oedometer stiffness and power of stress dependent stiffness

From the oedometer test results we determine the stiffness E_{oed} for vertical stresses σ_{yy} = 100 kPa and σ_{yy} = 200 kPa, see Figure 7. Note that E_{oed} is a tangent stiffness.
Make sure to use the primary loading part of the oedometer test results.

$$E_{oed}^{\sigma_y=100kPa} = \frac{320 \; - \; 0}{1.35\% \; - \; 0.33\%} = 31400 \text{ kPA}$$

$$E_{oed}^{\sigma_y=200kPa} = \frac{400 \; - \; 0}{1.35\% \; - \; 0.47\%} = 45450 \text{ kPA}$$

Within the Hardening Soil model the stress dependent oedometer stiffness is defined as:

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cos\varphi - \sigma_{yy}\sin\varphi}{c\cos\varphi + p_{ref}\sin\varphi}\right)^m, c = 0 \;\Rightarrow\; E_{oed} = E_{oed}^{ref}\left(-\frac{\sigma_{yy}}{p_{ref}}\right)^m$$

Choosing the reference pressure p_{ref} = 100 kPa gives

$$E_{oed}^{ref} = E_{oed}^{\sigma_y=100kPa} \approx 31000 \text{ kPA}$$

The power *m* for stress dependent stiffness can now be determined as:

$$\frac{E_{oed}^{\sigma_y=200kPa}}{E_{oed}^{ref}} = \left(\frac{\sigma_{yy}}{p_{ref}}\right)^m \Rightarrow \frac{45500}{31000} = \left(\frac{200}{100}\right)^m \Rightarrow m \approx 0.6$$

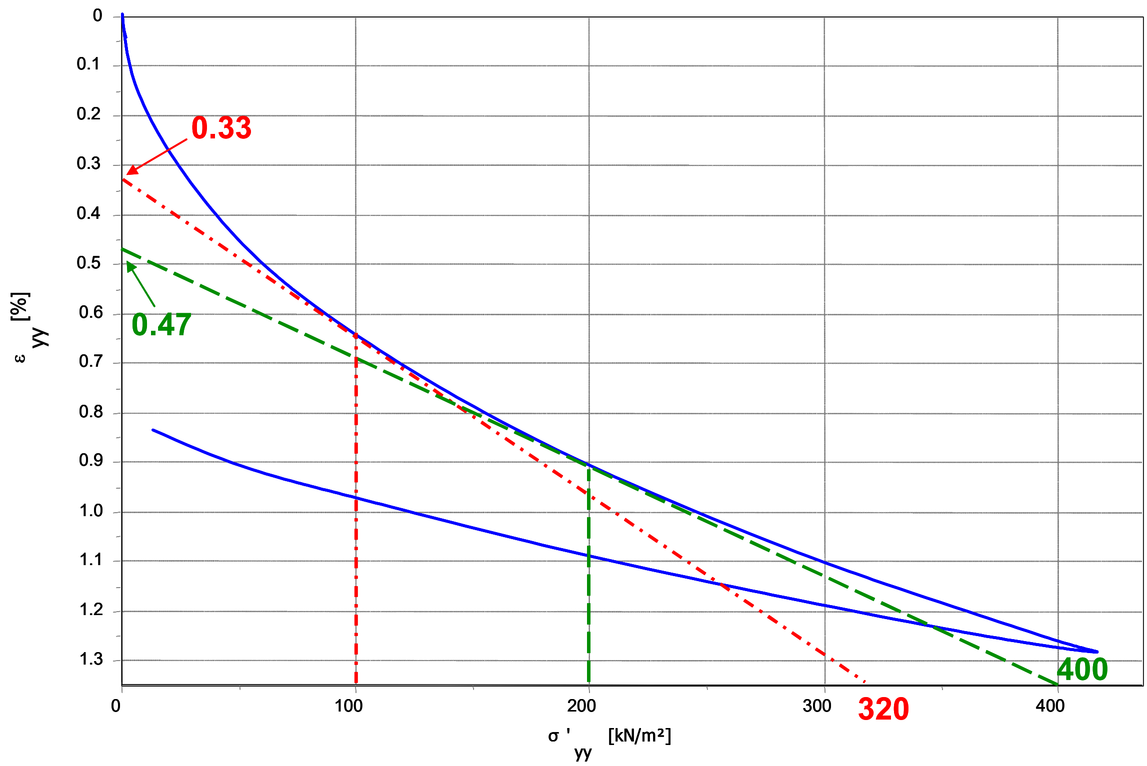


Figure 3: Determination of oedometer stiffness and power of stress dependency

K₀ value for normal consolidation

The K0-value for normal consolidation can only be obtained if measurements of horizontal stresses have been performed during the oedometer test. If so, results as given in Figure 8 may be obtained.
From the primary loading line it is possible to work out K_0.

$$K_0^{nc} = \frac{\Delta\sigma_x}{\Delta\sigma_y} = \frac{\Delta\sigma_3}{\Delta\sigma_1} = \frac{100}{300} = 0.33$$

Alternatively one can use Jaki's formula

$$K_0^{nc} \approx 1 - \sin\varphi = 1 - \sin(42) = 0.33$$

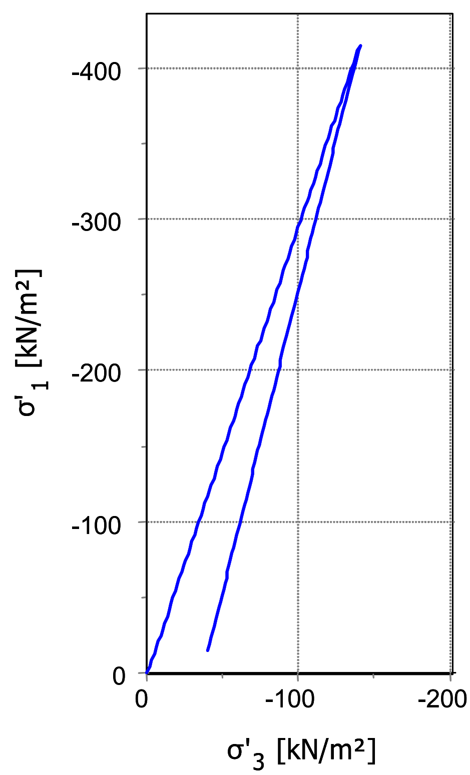


Figure 4: Horizontal/vertical stress ratio during oedometer test

Parameter	Unit	Value
$E_{50}{}^{ref}$	[kPa]	31,000
$E_{oed}{}^{ref}$	[kPa]	31,000
$E_{ur}{}^{ref}$	[kPa]	90,000
ν_{ur}	[-]	0.2
c’	[kPa]	0
ϕ’	[°]	42
ψ [	[°]	16
m	[-]	0.6
$K_0{}^{NC}$	[-]	0.33

Table 2: Hardening Soil Parameters of the Sand

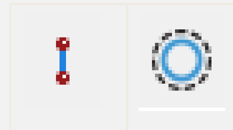
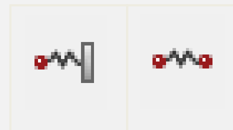
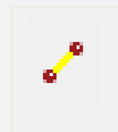
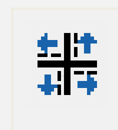
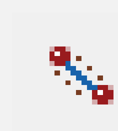
CG07 STRUCTURAL ELEMENTS IN PLAXIS

William W.L CHEANG PhD MSc PGDip BEng (Hons)

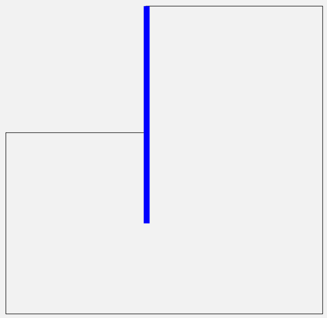
Contents

1. Structural elements available in Plaxis
2. Usage of structural elements in FE modelling
3. **Plate** elements (*Beam and Shell element*)
4. **Anchor** elements *(Spring element)*
5. **Geotextile** elements *(Membrane element)*
6. **Interface** elements *(Zero thickness element)*
7. **Embedded Beam Row** elements

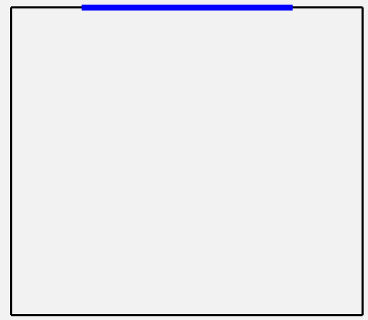
1.Structural elements in Plaxis

1. **Plate element** 
2. **Anchor element** 
3. **Geogrids element** 
4. **Interface element** 
5. **Embedded beam row element** 

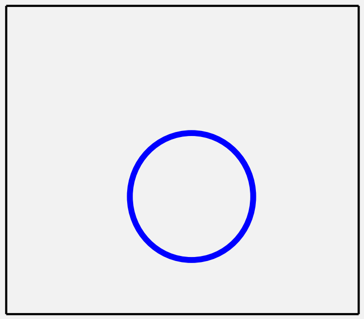
2. Application of structural elements



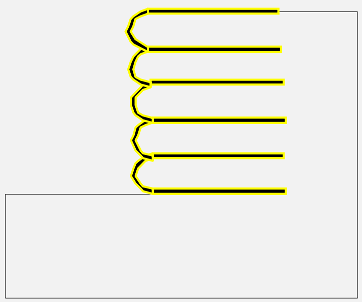
wall



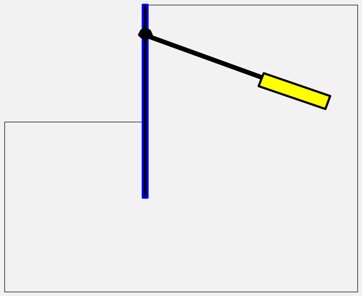
strip footing



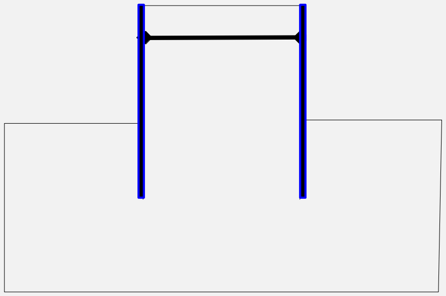
tunnel



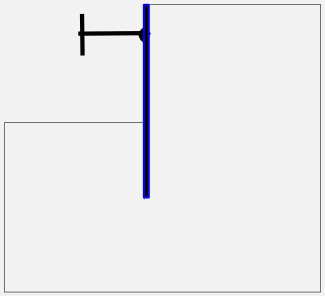
geotextile wall



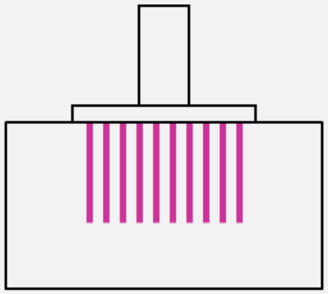
ground anchor



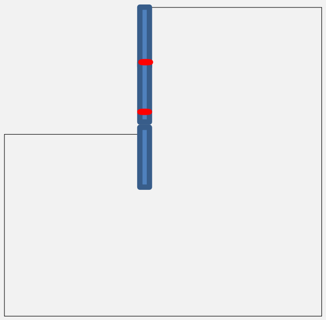
cofferdam



strut



Pile foundations



Incremental wall

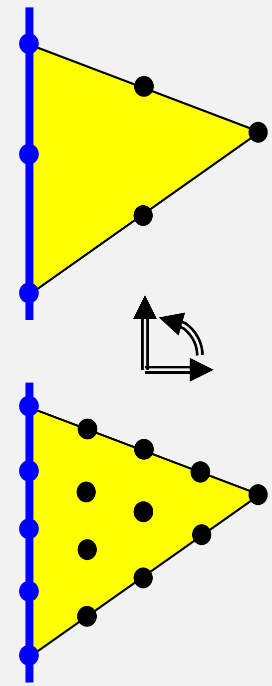
STRUCTURAL ELEMENTS

3. PLATE ELEMENTS

3.1 Plate Element

Overview:

1. 3 or 5 noded line elements (for 6-noded or 15-noded element mesh)
2. 3 degrees of freedom per node
3. Plates have:
    - Axial forces
    - Shear forces
    - Bending moments
    - Hoop forces (axisymmetry)
4. Elastic or elastoplastic behaviour
5. For modelling walls, floors, tunnels



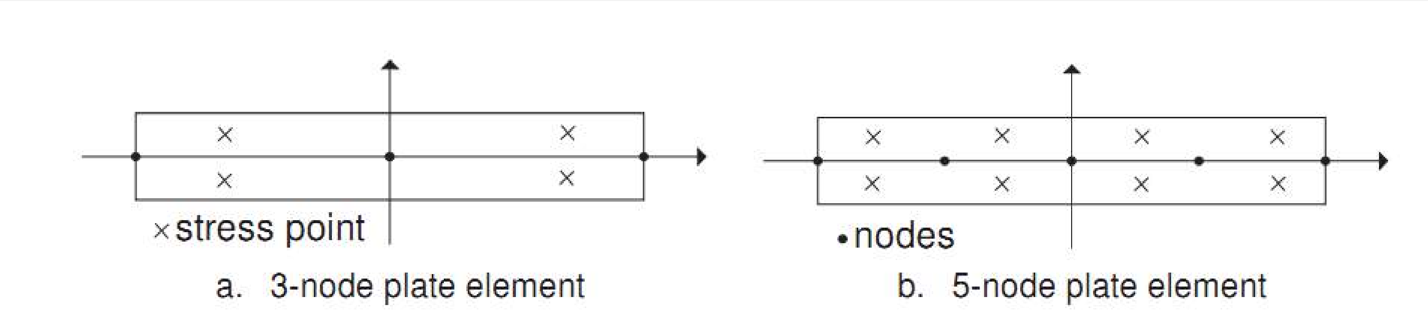
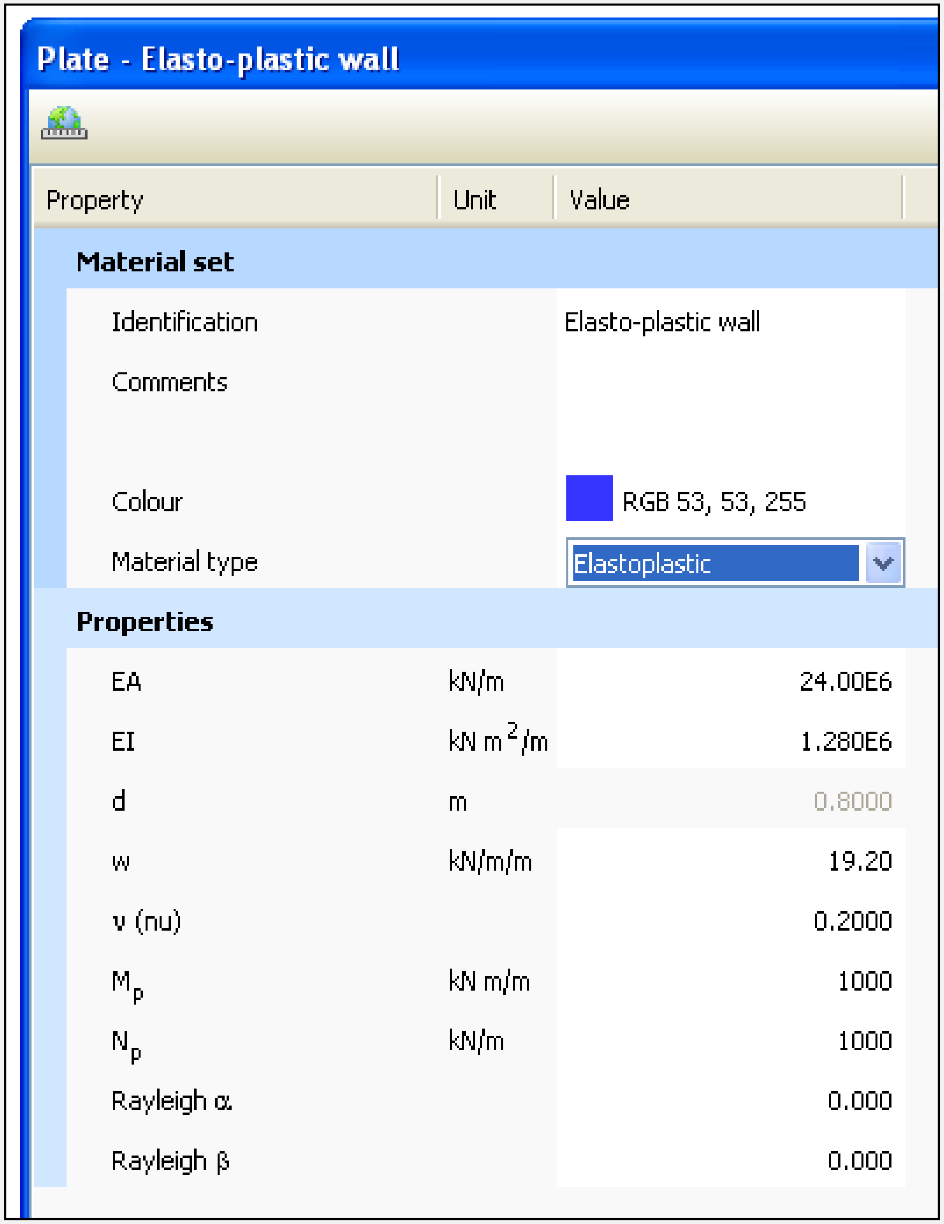


Figure 3.10 Position of nodes and stress points in plate elements

3.2 Plate Element

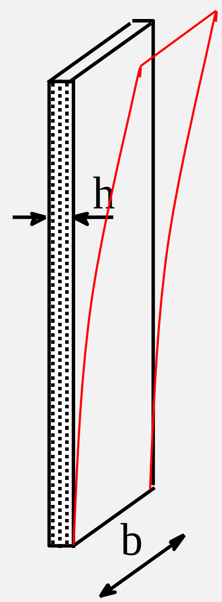
Plates – elastic parameters

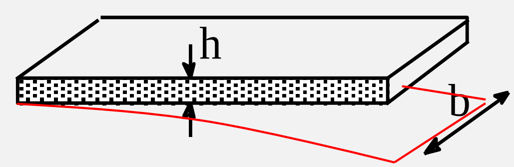


$$EI = E \cdot \frac{h^3 \cdot b}{12}$$ **(*b* = 1 m)**

$$EA = E \cdot h \cdot b$$ **(*b* = 1 m)**

$$d = h = \sqrt{12 \frac{EI}{EA}}$$ (Equivalent rectangular plate thickness)



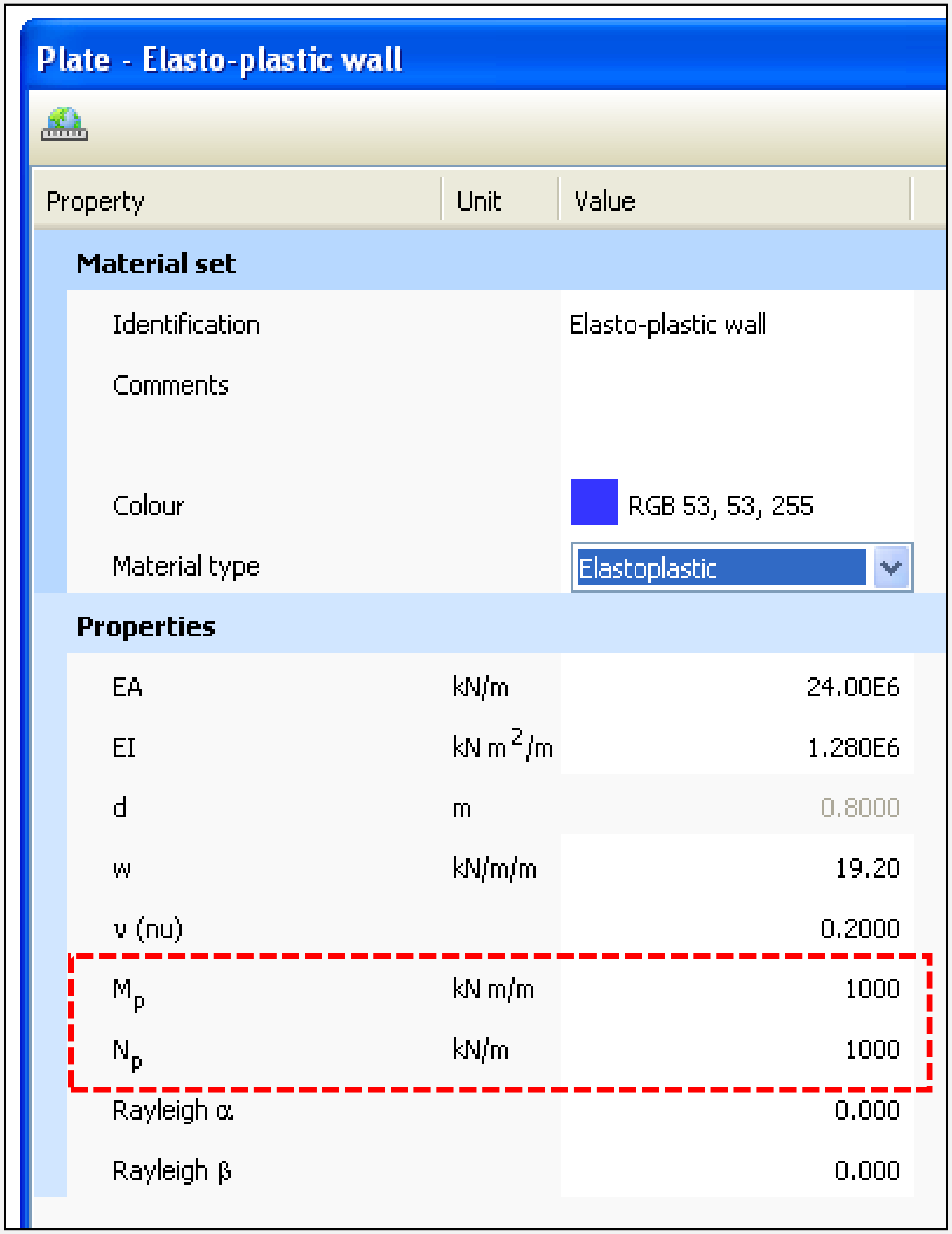


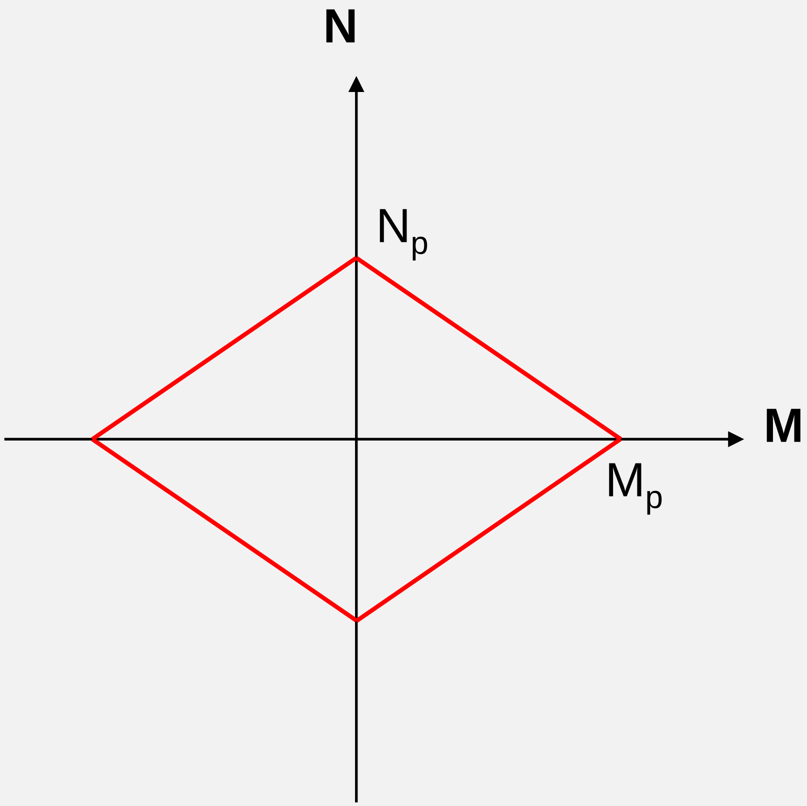
b = 1 m in plane strain

b = 1 meter in axisymmetry

3.3 Plate Element

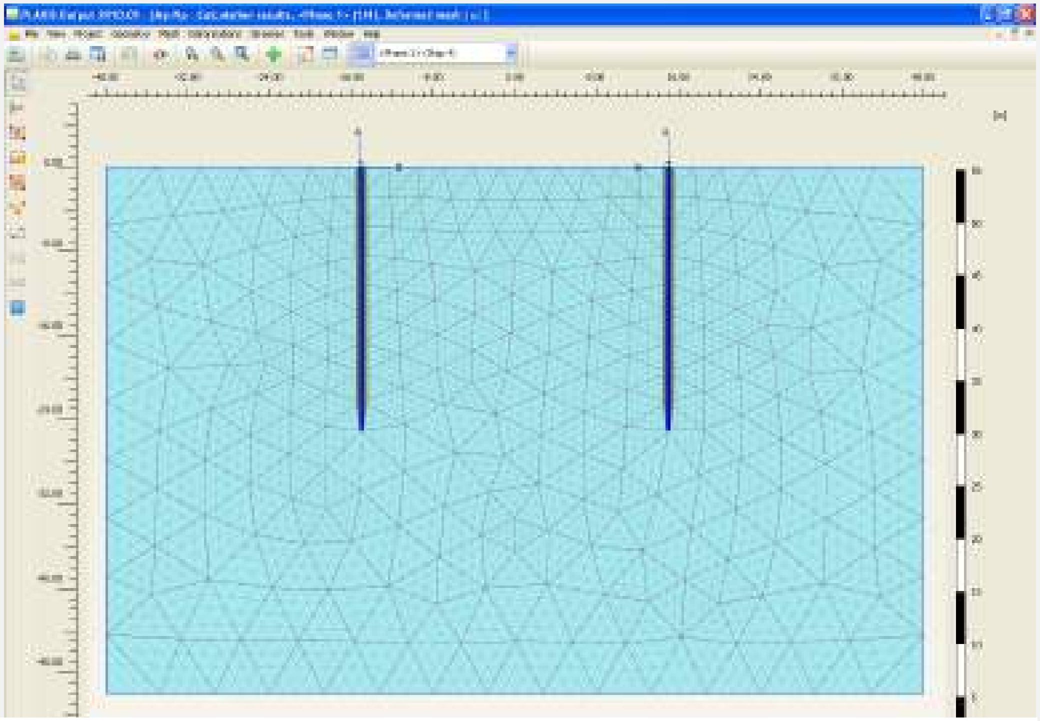
Plates – elasto-plastic behaviour

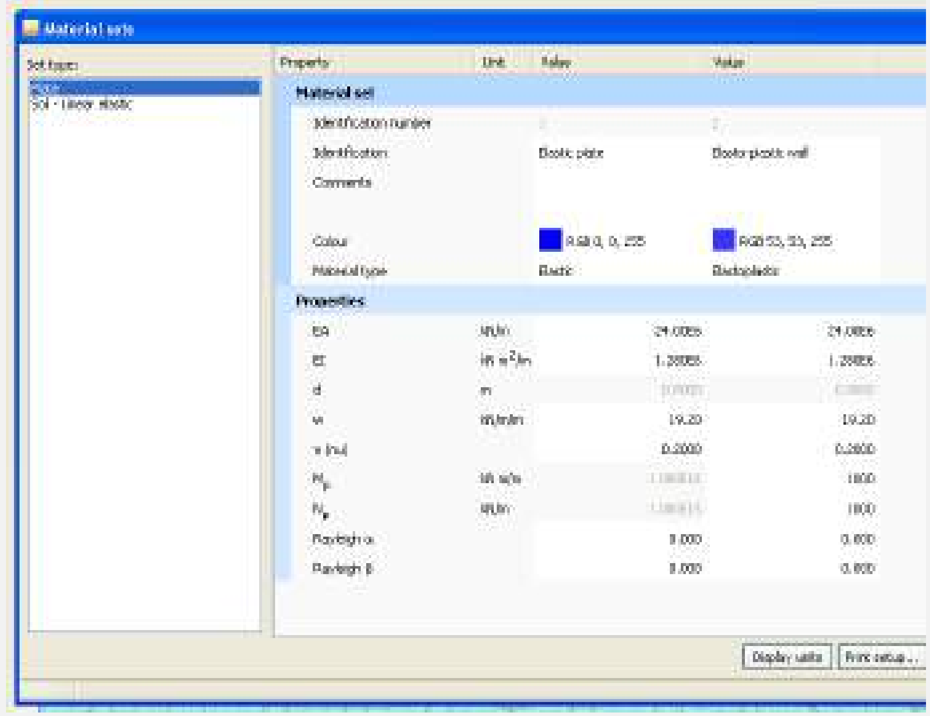


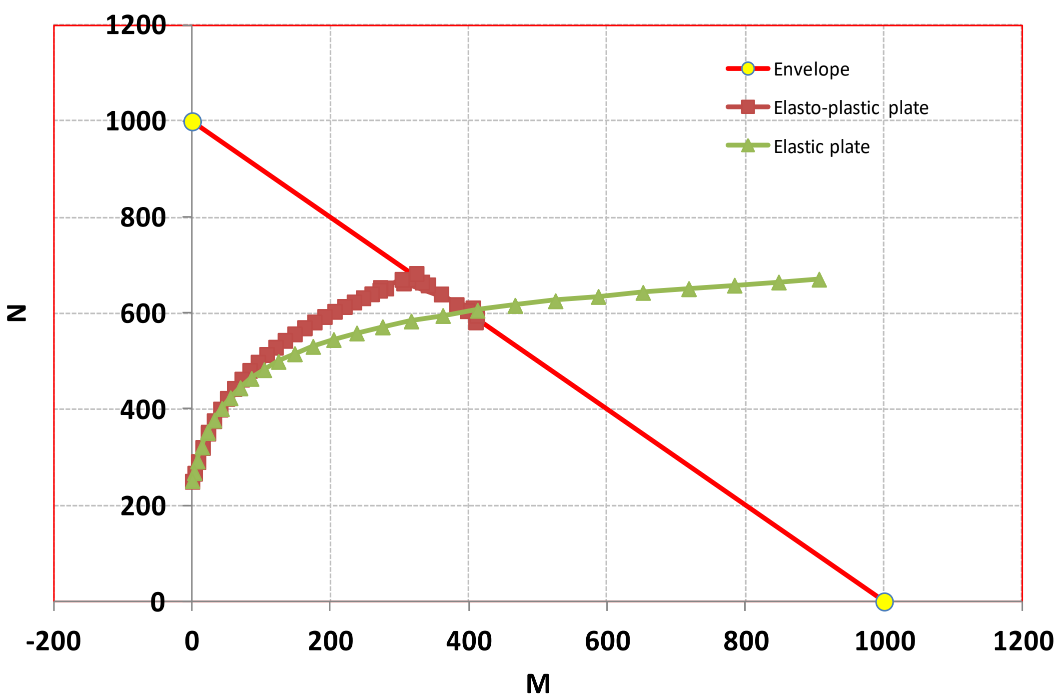


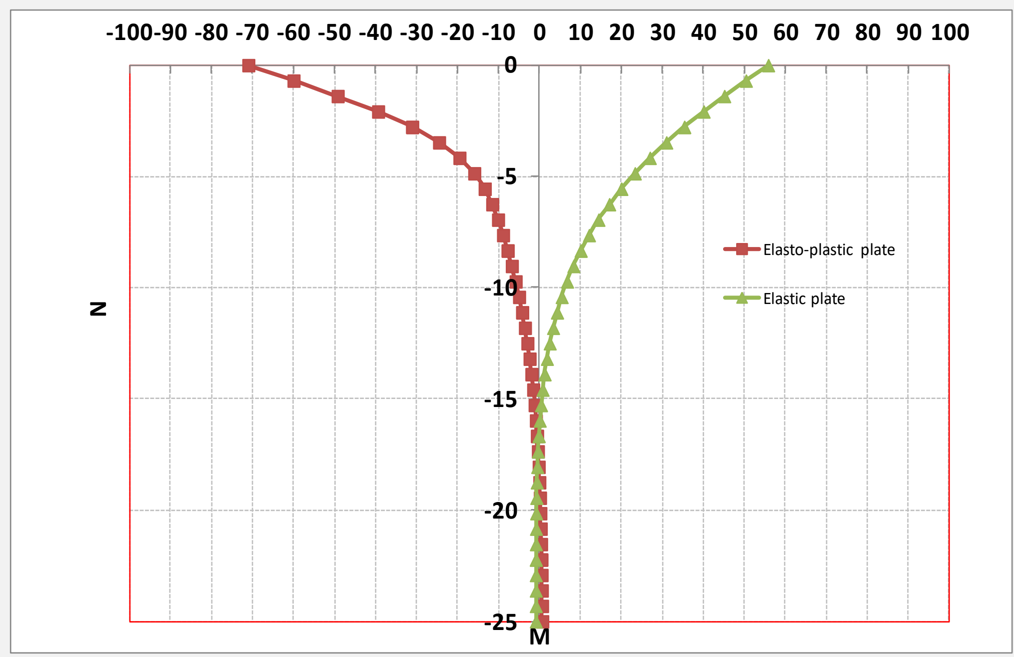
3.4 Plate Element

(Illustration: Mp-Np.P2D):



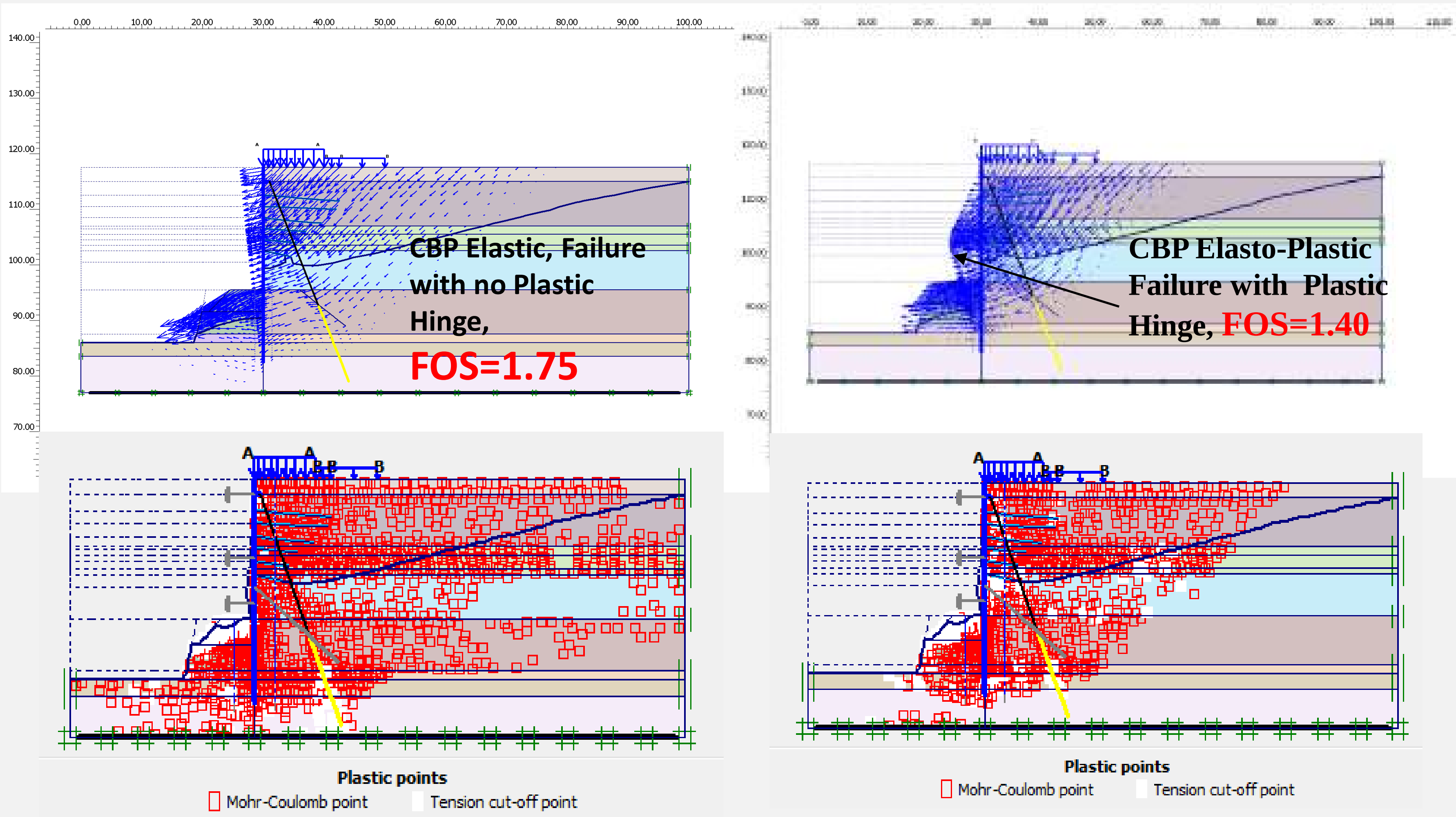






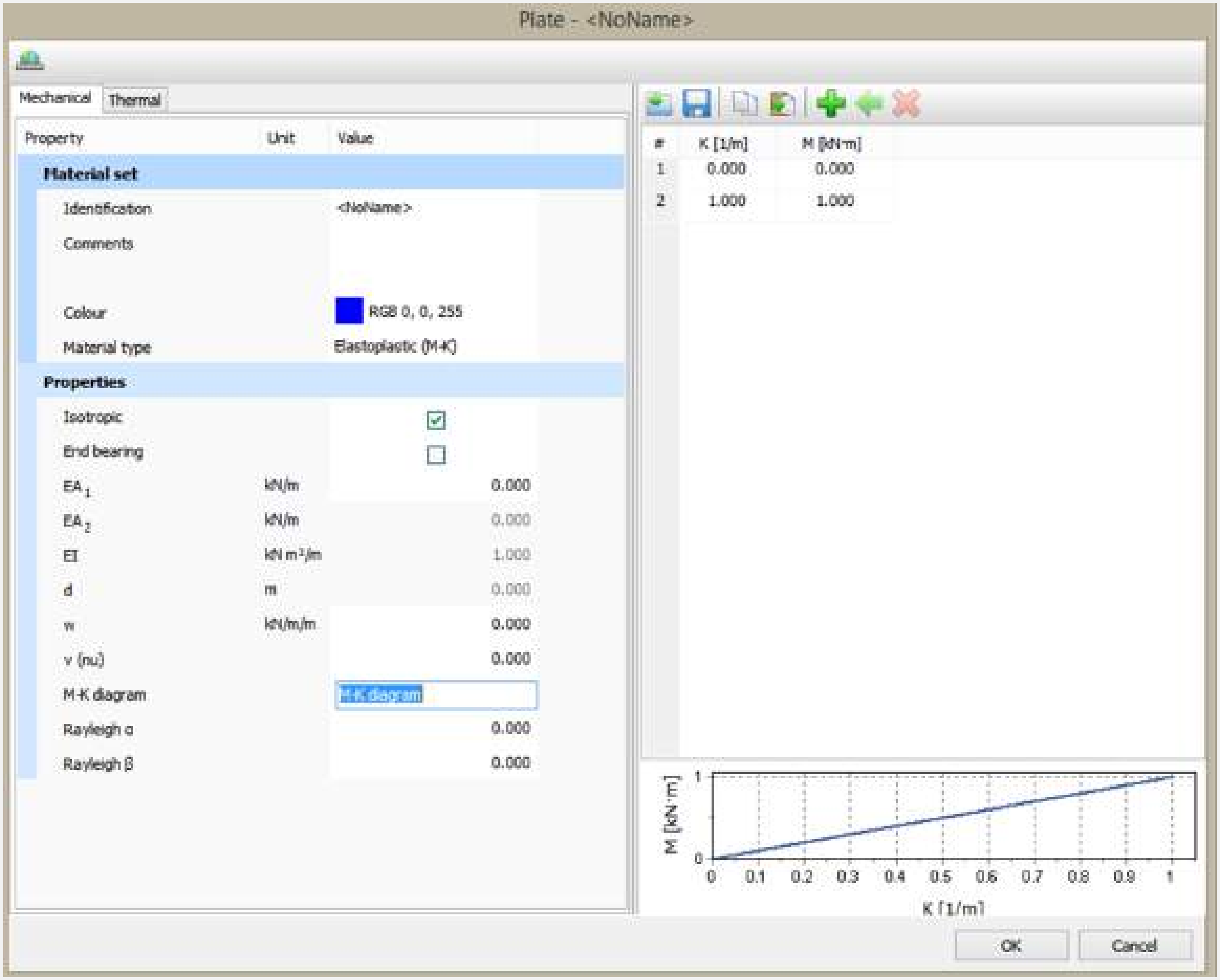
3.5 Plate Element

Effect on Global FOS by c/phi Reduction



1. Elastic wall excludes possibility of wall plastic hinge; and over-estimate FOS=1.75
2. Allowing for wall plastic hinge (Elasto-plastic wall) gave lower FOS=1.40 and smaller soil yielded zone behind the wall

Elastoplastic M-Kappa (Non-linear Behaviour)



[Elastoplastic Non-linear behaviour (M-K) Verification]

Elastoplastic M-Kappa (Non-linear Behaviour)

Load case	Phase	M [kNm/m]	Comment
1	1	-41	Loading to failure (clockwise)
2	2	+41	Loading to failure (anticlockwise)
3	3	-25	Loading
	4	0	Unloading
	5	-35	Reloading
	6	0	Unloading
	7	-41	Reloading to failure
4	8	-25	Loading
	9	+25	Reverse loading
	10	-35	Reloading
	11	+25	Reverse reloading
	12	-41	Reloading to failure
5	13	-39.5	Loading
	14	+39.5	Reverse loading
	15	-39.5	Reloading

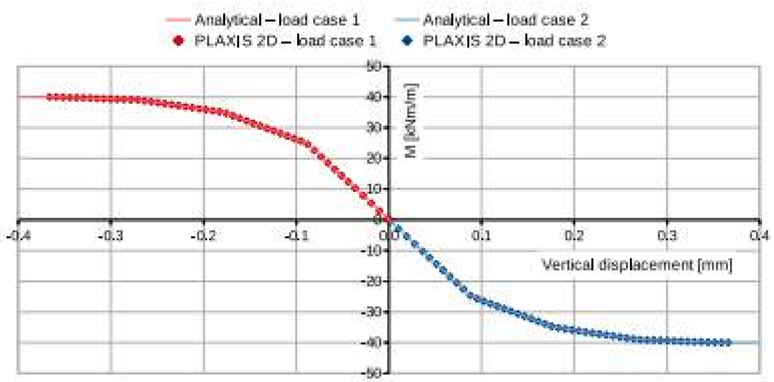


Figure 3 Bending moment versus vertical displacement (load cases 1 and 2)

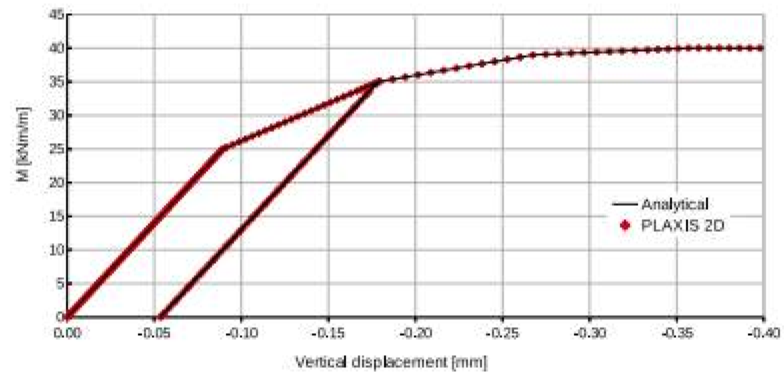


Figure 4 Bending moment versus vertical displacement (load case 3)

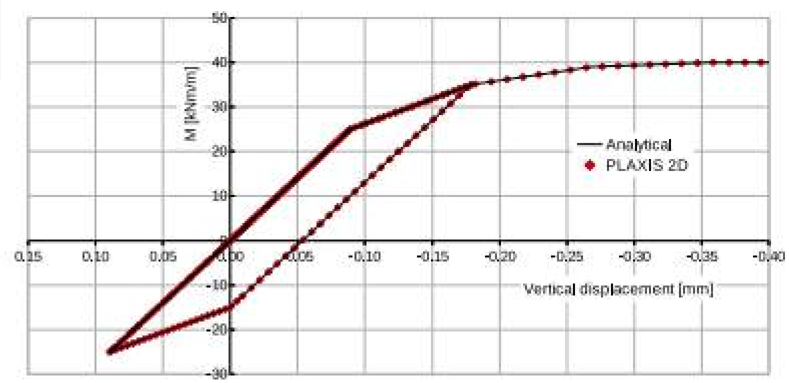


Figure 5 Bending moment versus vertical displacement (load case 4)

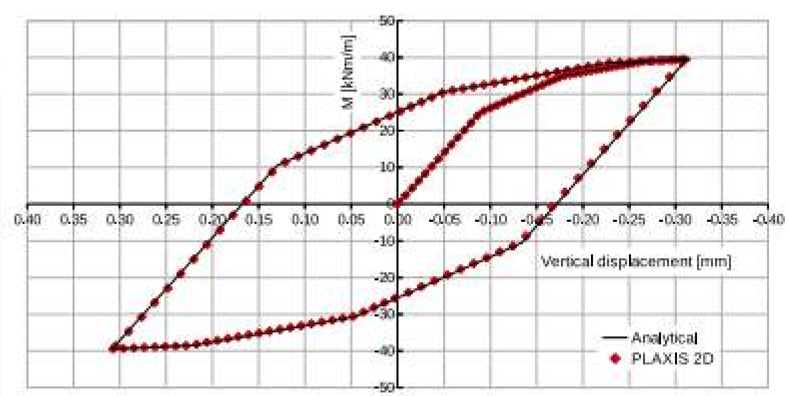
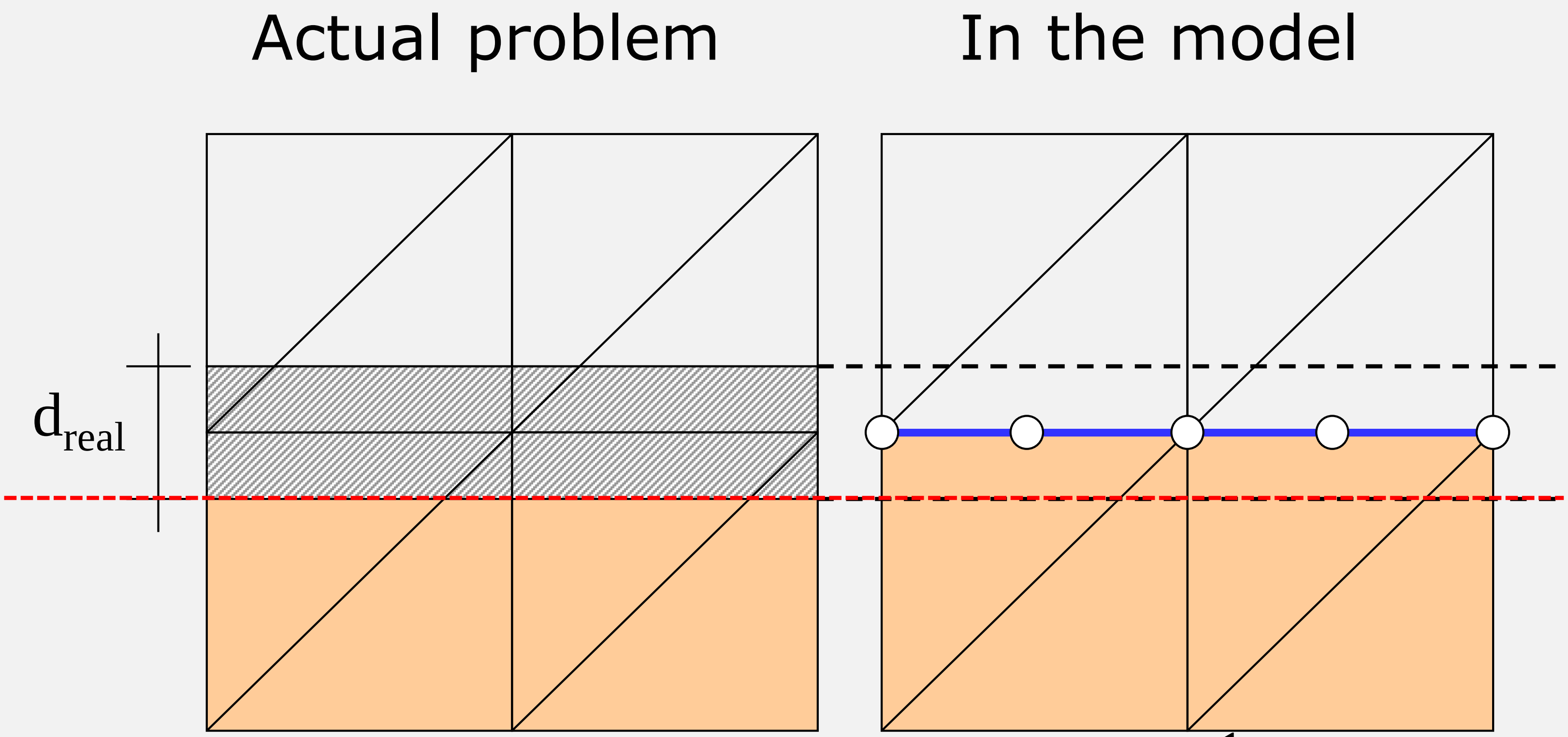


Figure 6 Bending moment versus vertical displacement (load case 5)

3.6 Plate Element

Plates – weight, excavation



$w_{real} = \gamma_{concrete} \cdot d_{real}$

$w_{model} = \gamma_{soil} \cdot \frac{1}{2} d_{real} + w_{plate}$

$$w_{model} = w_{real} \Rightarrow w_{plate} = (\gamma_{concrete} - \frac{1}{2}\gamma_{soil}) \cdot d_{real}$$

Below GT

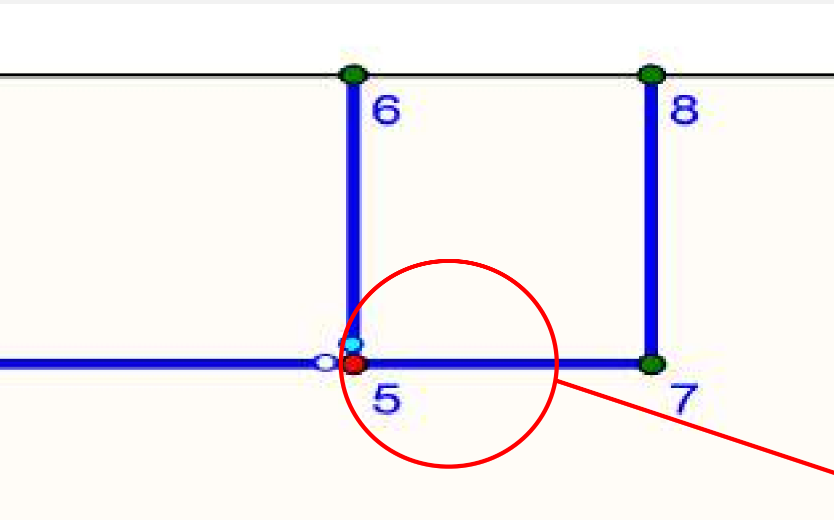
$\gamma_{soil} = \gamma_{sat}$

Above GT

$\gamma_{soil} = \gamma_{unsat}$

3.7 Plate Element

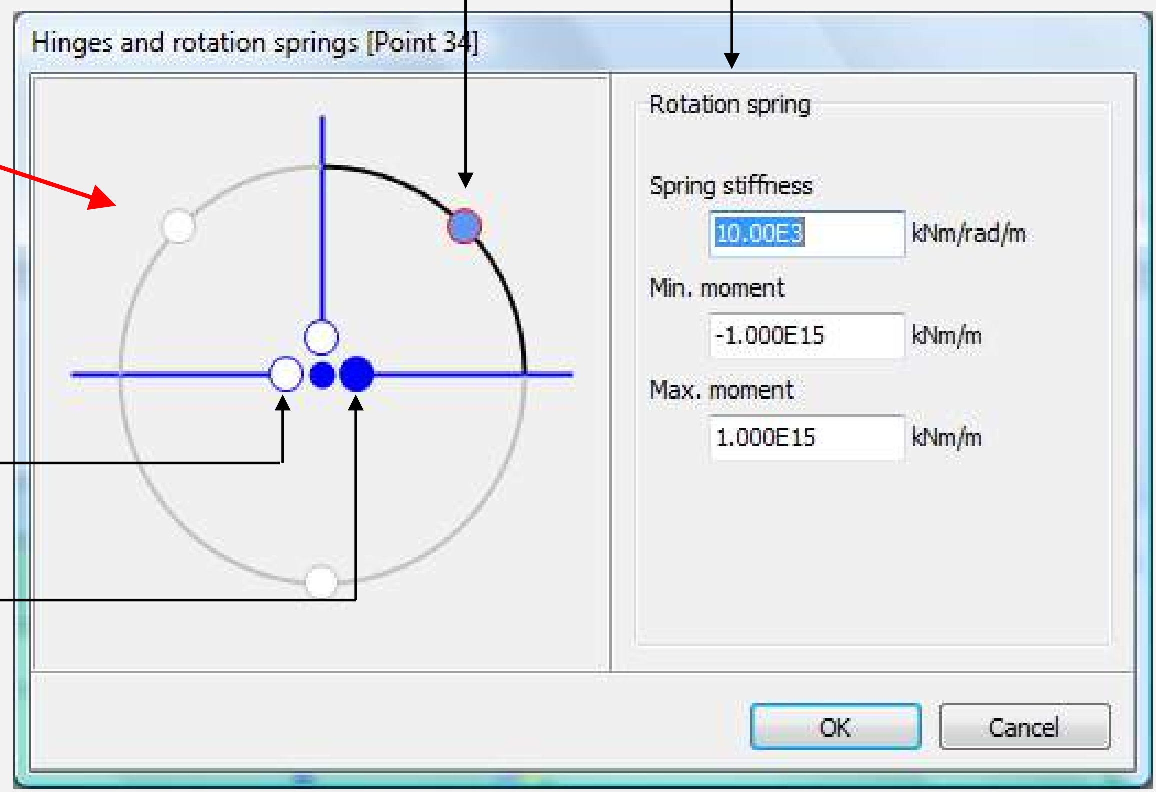
Plates – connections



Spring data:

- Stiffness
- Min/Max moment

Rotation spring



Hinged connection

Rigid connection (default)

Illustration: Connection.P2D

3.8 Plate Element

Walls – thin wall vs. thick wall

- **Thin wall**
  - Wall thickness << wall length
  - No much end-bearing, only friction

→ **Plate element suffices**

- **Thick wall**
  - Wall thickness significant
  - End-bearing capacity needed

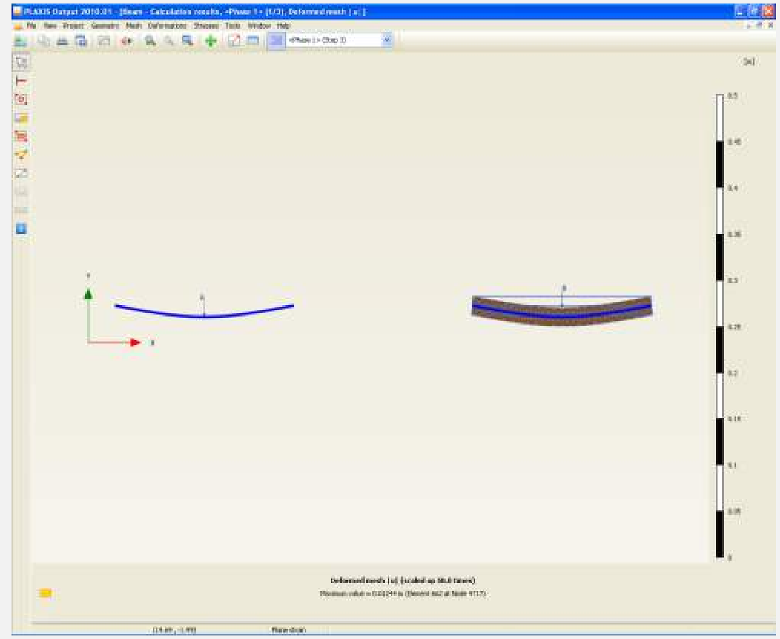
→ **Use soil elements with material set representing wall material**

→ **In order to obtain structural forces a plate with fictitious properties may be inserted**

3.9 Plate Element

Walls – thick wall

1. Soil elements with material set representing wall material
2. Difficult to obtain structural forces from soil elements, therefore introduce very flexible plate within the solid wall elements:
    - No influence on deformation: low stiffness, no weight
    - Located in on the neutral line (usually the middle)
    - Tight bonding to the concrete elements: no interfaces

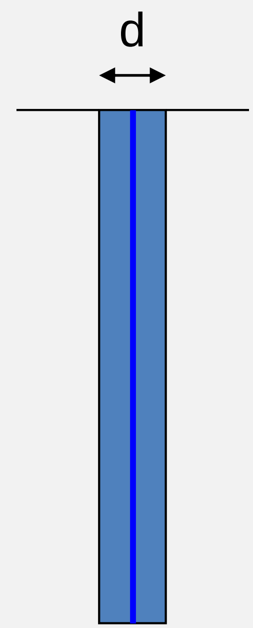


(Illustration: Beam.P2D):

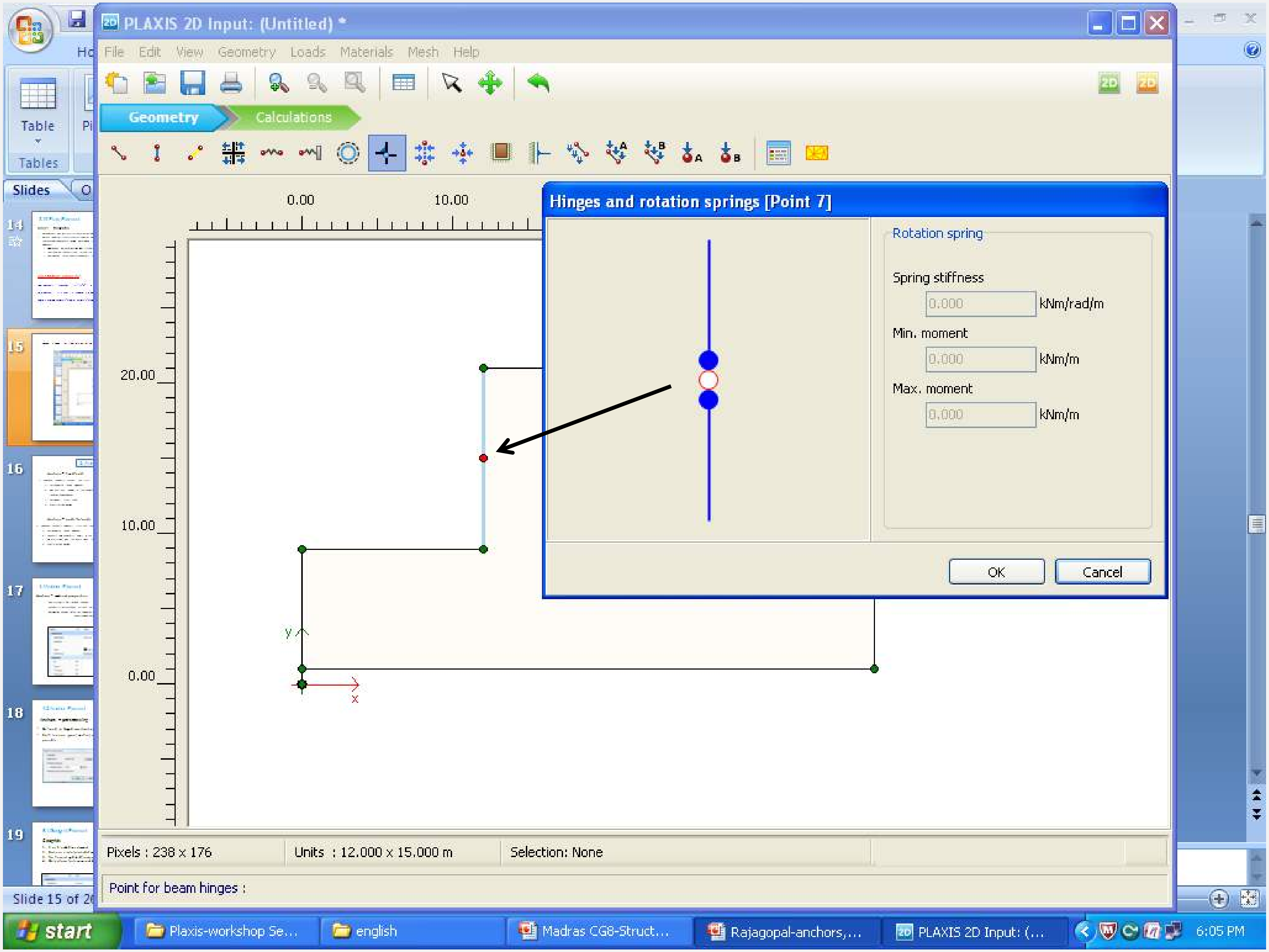
Solid elements: $E_{soil}=E_{wall}$, $I = {}^{1}/_{12} * d^3$, d = wall thickness

Plate element: $EI = E_{soil} I / x$, choose x large (e.g. 10^6)

$u_{plate} = u_{soil} \rightarrow M_{wall} = x * M_{plate}$, $Q_{wall} = x * Q_{plate}$



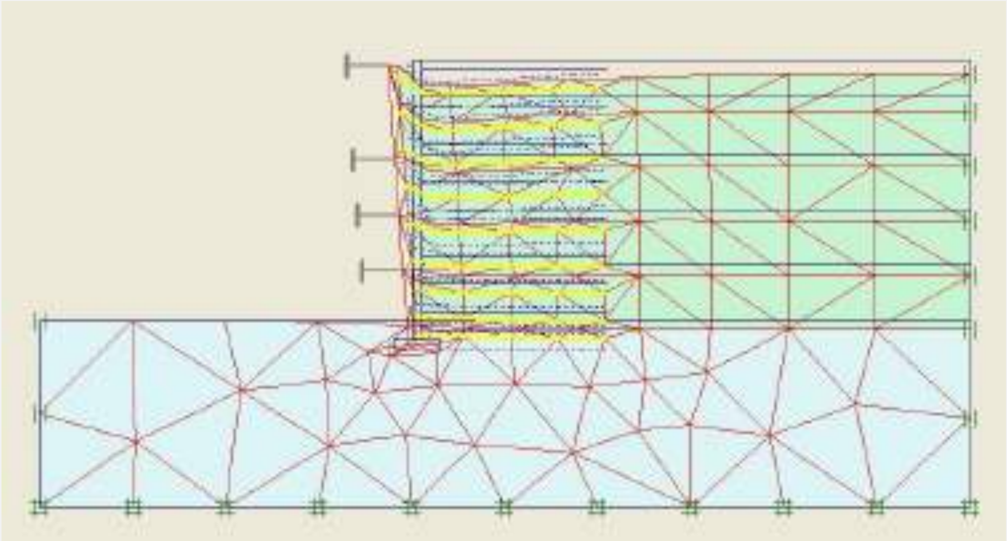
Placing a Hinge for free rotation of plates

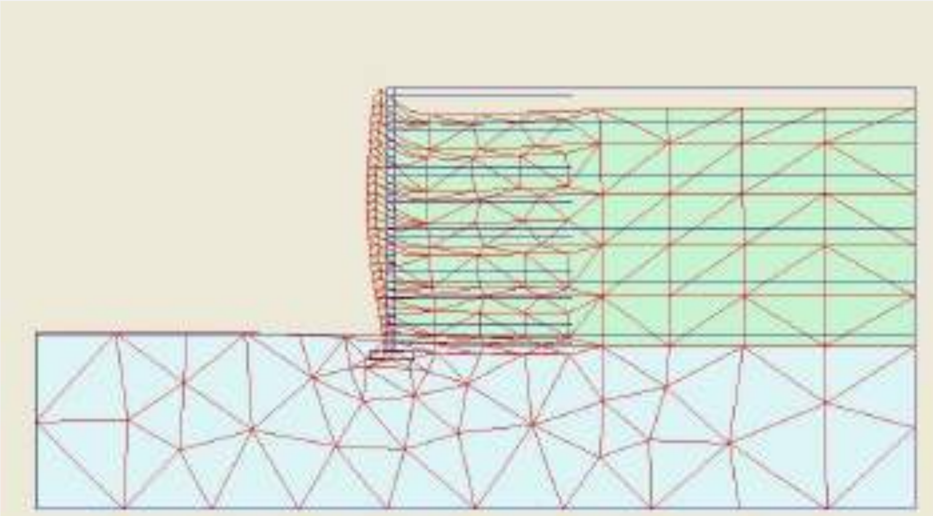


Can be placed at joint between plate elements – allows for free rotation

Full-height panel support to reinforced soil walls – rigid body rotation of facing

Incremental panel support to reinforced soil walls – bulging type deformation at front end



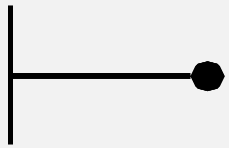


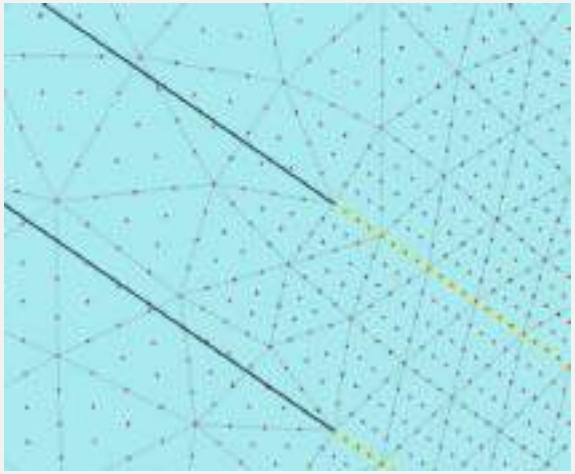
STRUCTURAL ELEMENTS

4. ANCHOR ELEMENTS

4. Anchor Element

Anchors – fixed-end





a) To model supports, anchors and struts
   a) Elasto-plastic spring element
   b) One end fixed to point in the geometry, other end is fully fixed for displacement
   c) Positioning at any angle
   d) Pre-stressing option

Anchors – node-to-node

4.1 Anchor Element

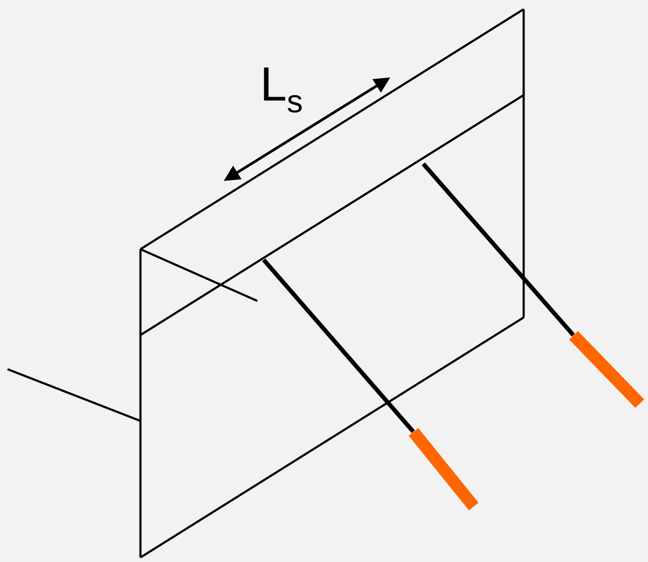
Anchors – material properties

Axial stiffness, *EA* (for one anchor) [kN]

Spacing, L_s (out-of-plane distance between anchors) [m]

Maximum anchor force for compression and tension, $|F_{max,comp}|$ and $|F_{max,tens}|$ [kN]

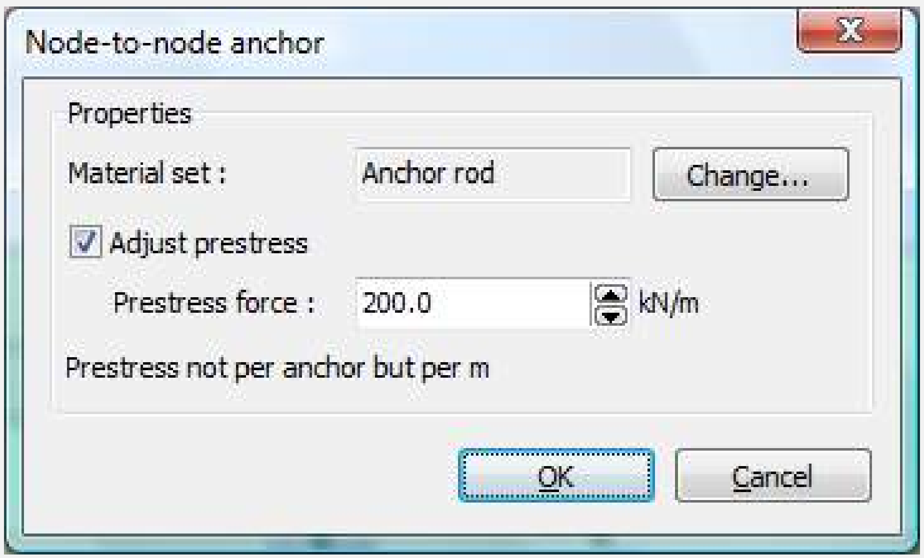
Property	Unit	Value
Material set		
Identification		Anchor rod
Comments		
Colour		RGB 51, 89, 91
Material type		Elastoplastic
Properties		
EA	kN	500.0E3
$L_{spacing}$	m	2.500
$\|F_{max,tens}\|$	kN	1.000E12
$\|F_{max,comp}\|$	kN	1.000E12



4.2 Anchor Element

Anchors – pre-stressing

- Defined in Staged construction phase
- Both tension (grout anchor) or compression (strut) possible



Tension = positive

STRUCTURAL ELEMENTS

5. GEOGRID ELEMENTS

5.1.Geogrid Element

Geogrids

1. 3 or 5 noded line element
2. Elastic or elasto-plastic behaviour
3. No flexural rigidity (EI), only axial stiffness (EA)
4. Only allows for tension, not for compression

Property	Unit	Value
Material set		
Identification		Grout body
Comments		
Colour		RGB 255, 255, 0
Material type		Elastoplastic
Properties		
Isotropic		☐
EA_1	kN/m	100.0E3
EA_2	kN/m	0.000
$N_{p,1}$	kN/m	10.00E6
$N_{p,2}$	kN/m	10.00E6

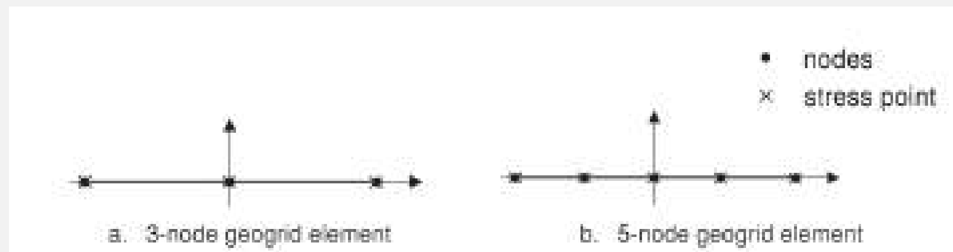
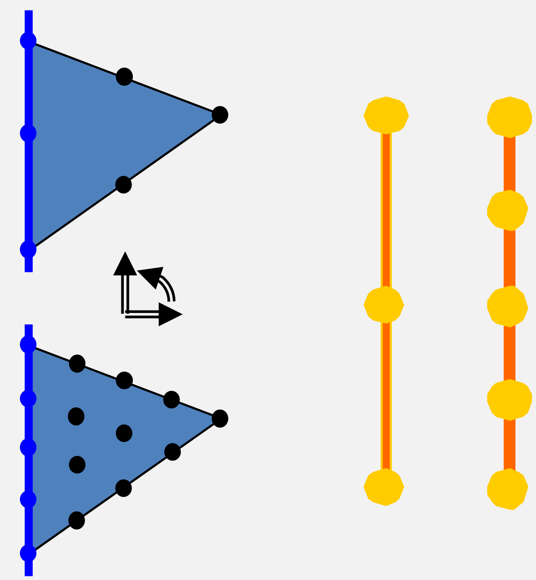
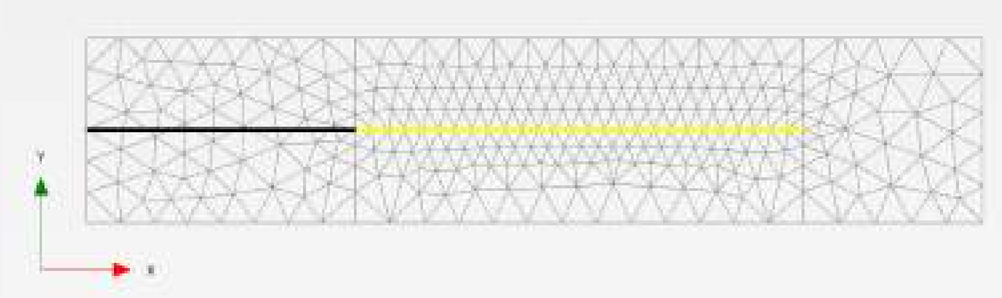


Figure 3.12 Position of nodes and stress points in geogrid elements



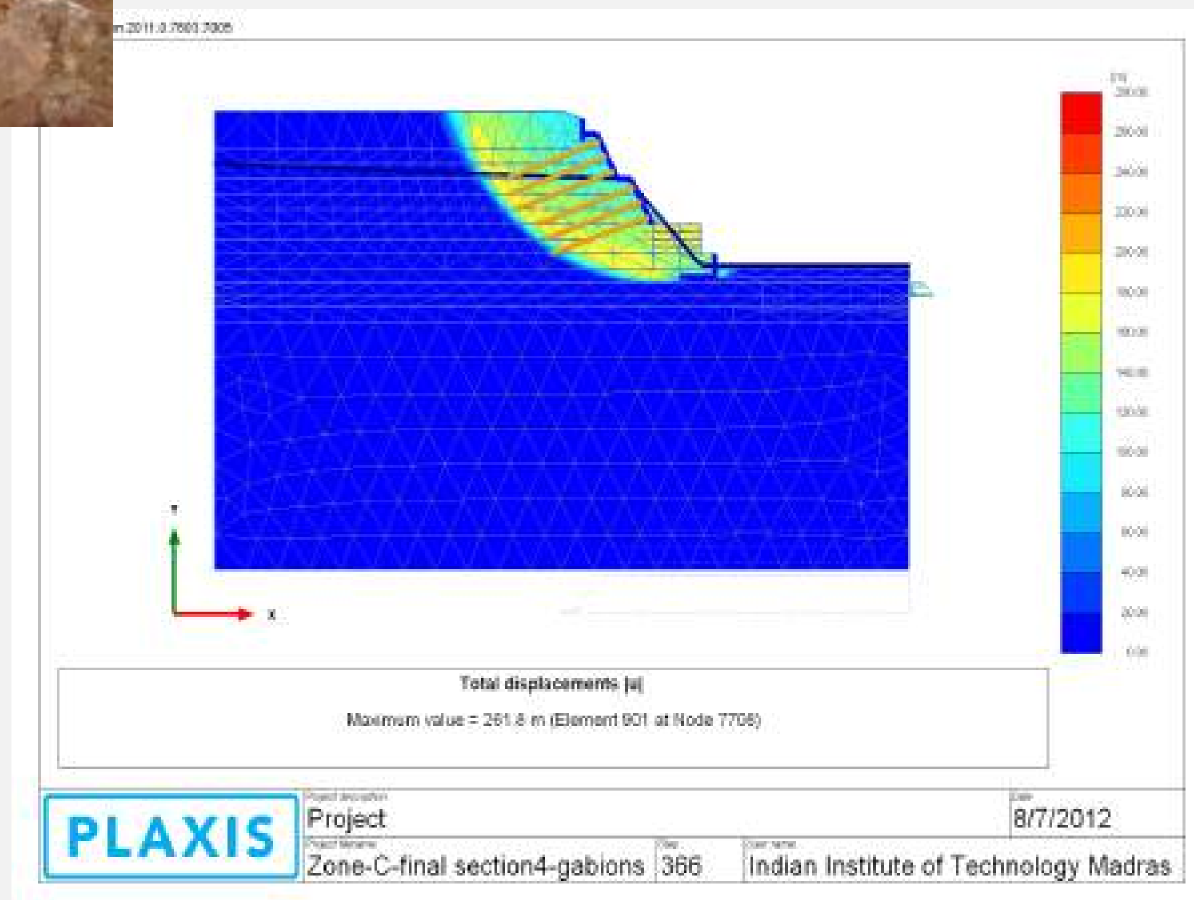
5.2 Anchor Element + Geogrid Element

Ground anchors



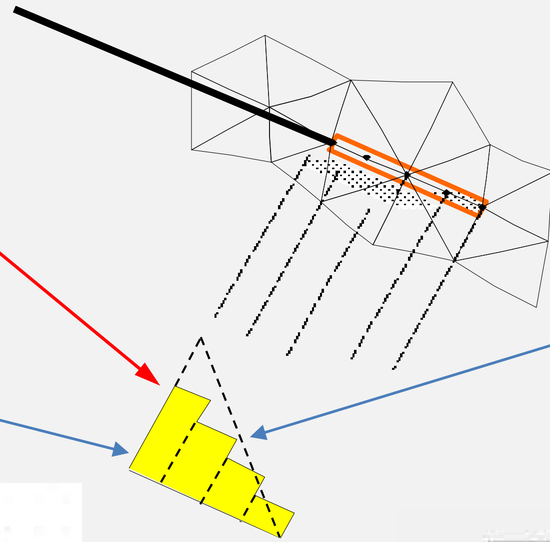
1. Combination of node-to-node anchor and geogrid
2. Node-to-node anchor represents anchor rod (free length)
   (no interaction with surrounding soil)
3. Geogrid represents grouted part (full interaction with surrounding soil)
4. No interface around grouted part; interface would create unrealistic failure surface
5. Working load conditions only – no pullout
6. If pullout force is known this can be used by limiting anchor rod force

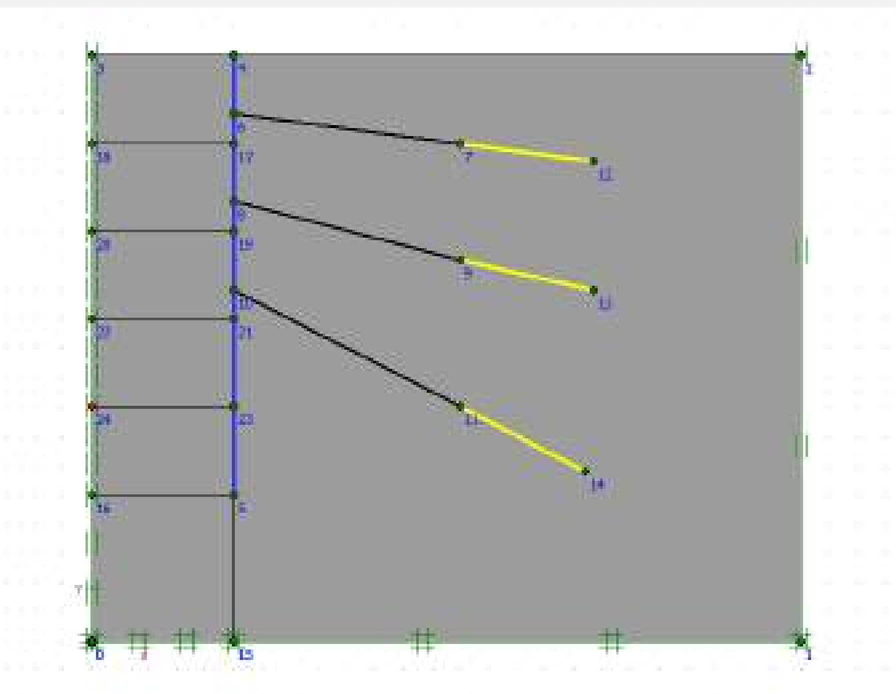
Modeling of soil nails using geogrid elements



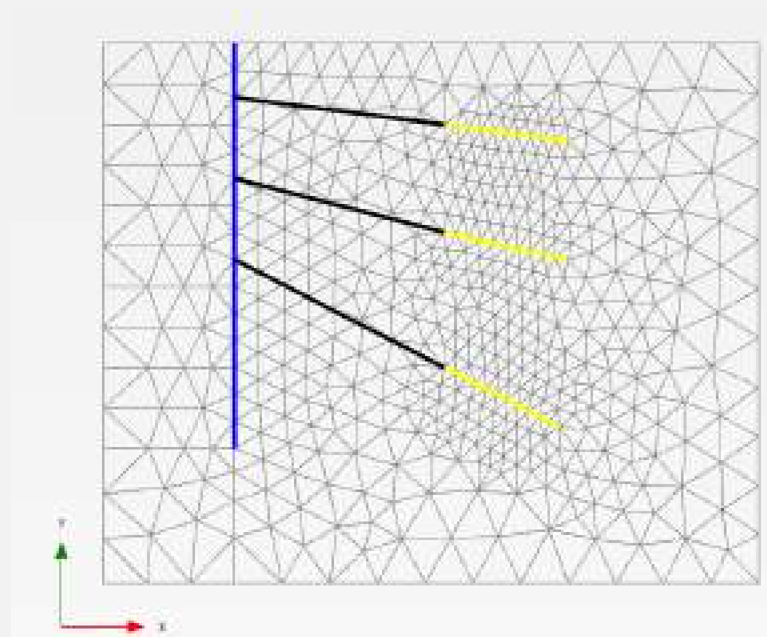
5.3 Ground anchors

Axial force distribution along fixed length (modelled using geogrid)



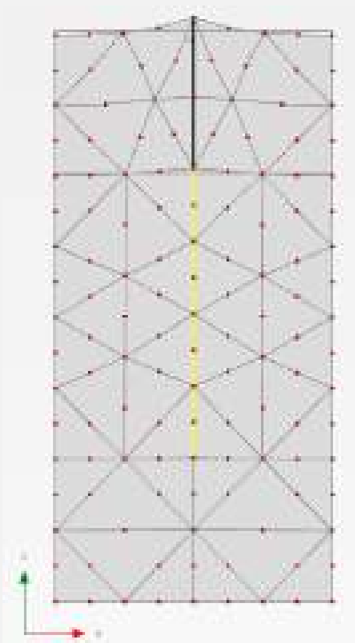


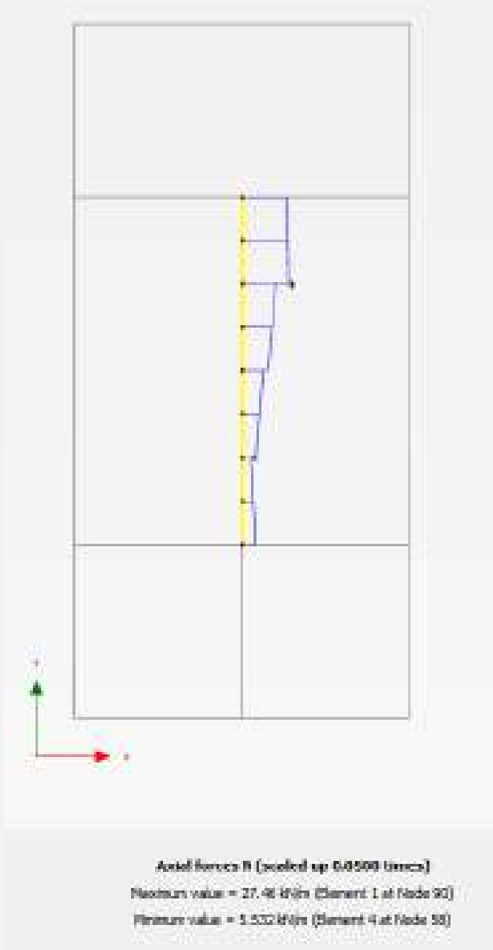
Input geometry

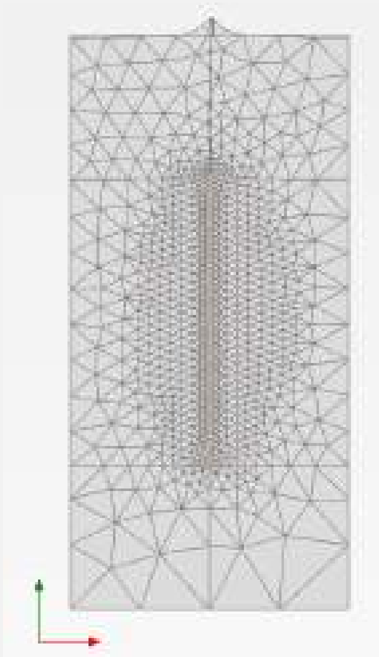


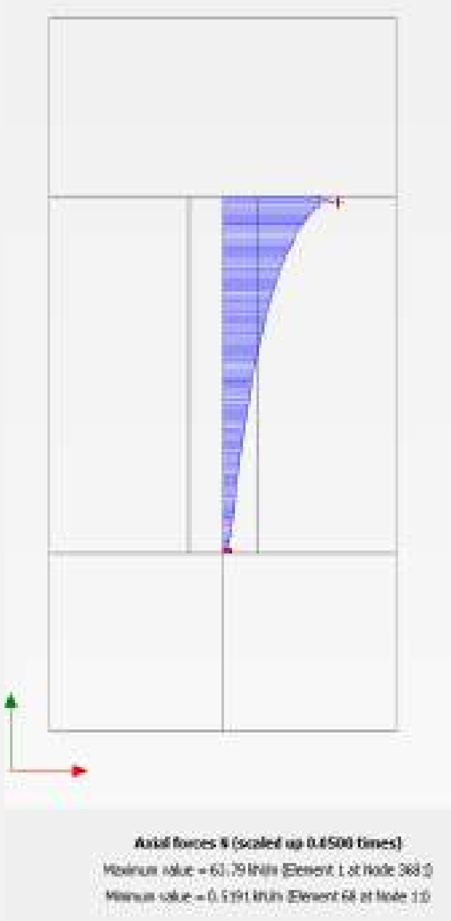
Generated mesh

5.4 Ground Anchors: Influence of node numbers along structural elements









STRUCTURAL ELEMENTS

6. INTERFACE ELEMENTS

6.1 Interface Element

Interfaces – material properties

1. Soil-structure interaction
    1. Wall friction
    2. Slip and gapping between soil and structure
2. Soil material properties
    A. Taken from soil using reduction factor R_{inter}
3. Individual material set for interface possible

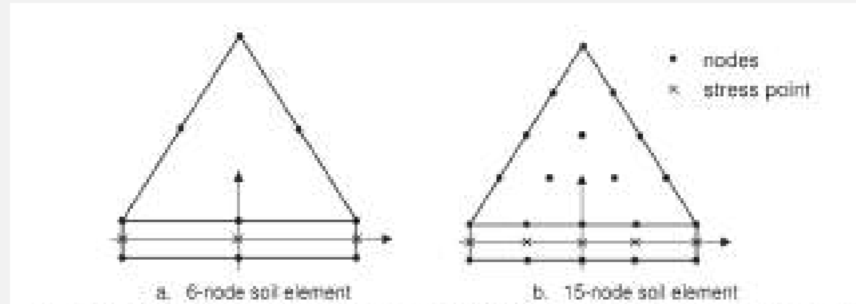
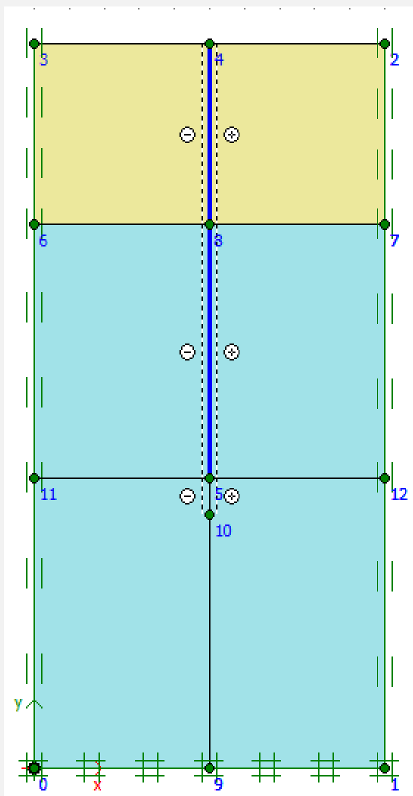
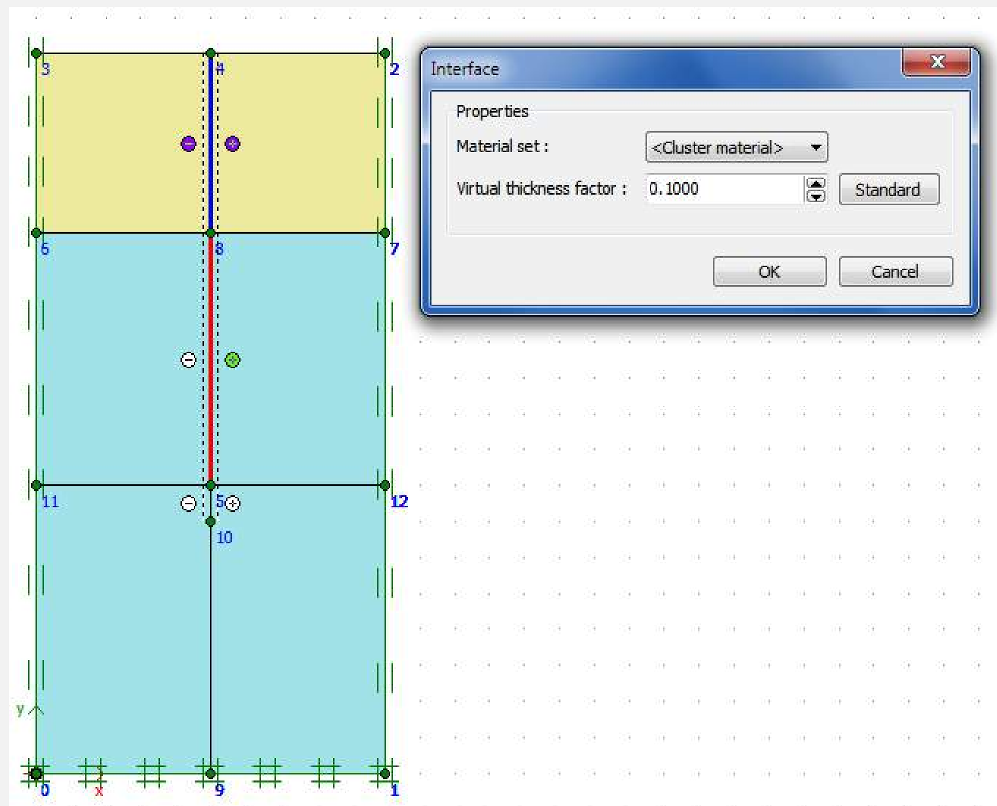


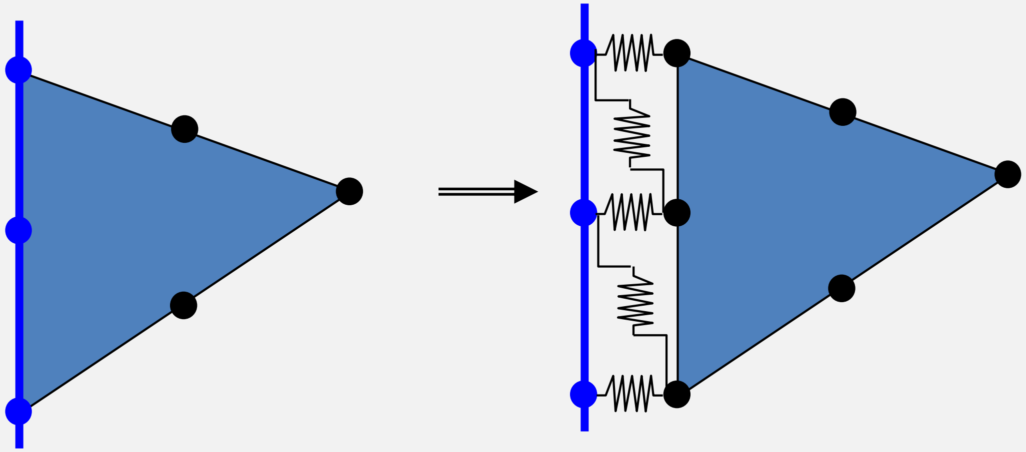
Figure 3.13 Distribution of nodes and stress points in interface elements and their connection to soil elements





Interfaces – soil structure interaction

- Doubling of nodes to (partially) uncouple soil and structural element
- Spring connection between soil nodes and structural nodes
  - Normal spring
  - Shear spring
- Allows for modelling of slip, gapping and closing between soil and structure
- Can also be used between to soil materials



Output:
- Normal stresses
- Shear stresses
- Displacements

Interfaces – material properties

- Soil material properties

  Taken from soil using reduction factor R_{inter}

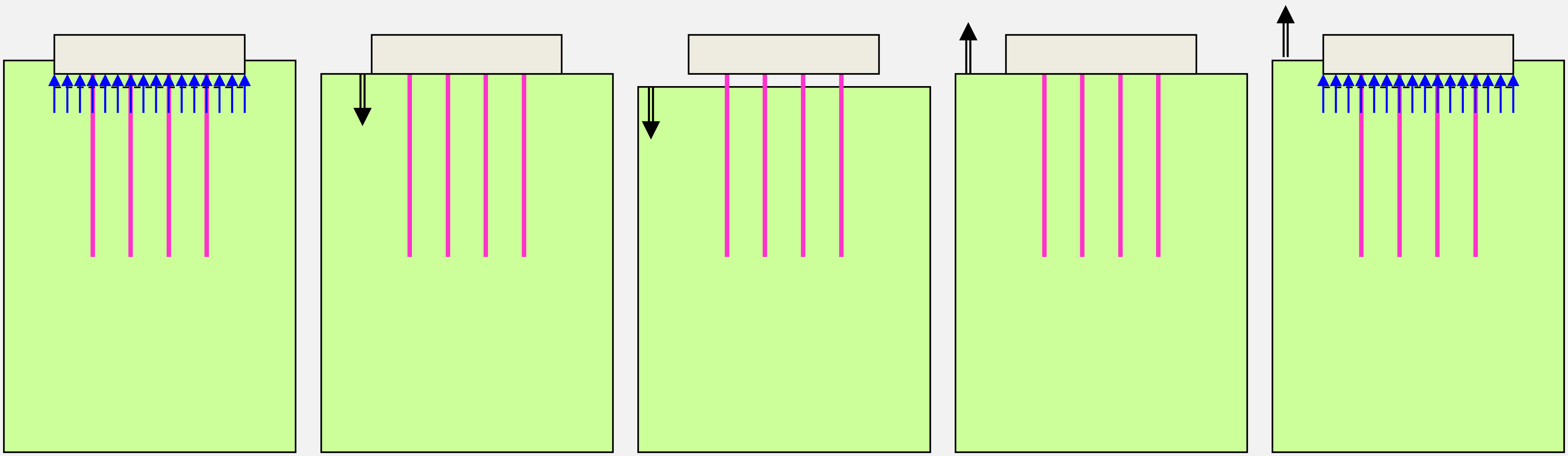
C_{inter}	$= R_{inter} * C_{soil}$
$\tan(\phi_{inter})$	$= R_{inter} * \tan(\phi_{soil})$
ψ_{inter}	$= 0$ for $R_{inter} < 1$
	$= \psi_{soil}$ $R_{inter} = 1$
$\sigma_{t,inter}$	$= R_{inter} * \sigma_{t,soil}$
G_{inter}	$= (R_{inter})^2 * G_{soil}$

1. Residual reduction factor $R_{inter,res}$ only affects strength, not stiffness

2. Individual material set for interface possible

Interfaces – material properties

1. Residual strength after reaching maximum shear strength
2. Gap closure



Foundation pressure | Pressure reduction (soil shrinkage) | Gap opening (soil shrinkage) | Gap closing (soil swelling) | Foundation pressure (soil swelling)

6.2 Interface Element

Interfaces – reduction factor

Suggestions for R_{inter}:

- Interaction sand/steel = $R_{inter} \approx 0.6 – 0.7$
- Interaction clay/steel = $R_{inter} \approx 0.5$
- Interaction sand/concrete = $R_{inter} \approx 1.0 – 0.8$
- Interaction clay/concrete = $R_{inter} \approx 1.0 – 0.7$
- Interaction soil/geogrid **(grouted body)** = $R_{inter} \approx 1.0$
  (interface may not be required)
- Interaction soil/geotextile = $R_{inter} \approx 0.9 – 0.5$ (foil, textile)

With reference to BS8002:

3.2.6 Design values of wall friction, base friction and undrained wall adhesion

These should be derived from the representative strength determined in accordance with **2.2.8**, using the same mobilization factors as for the adjacent soil.

The design value of the friction or adhesion to be mobilized at an interface with the structure should be the lesser of:

a) the representative value determined by test as described in **2.2.8** if such test results are available; or

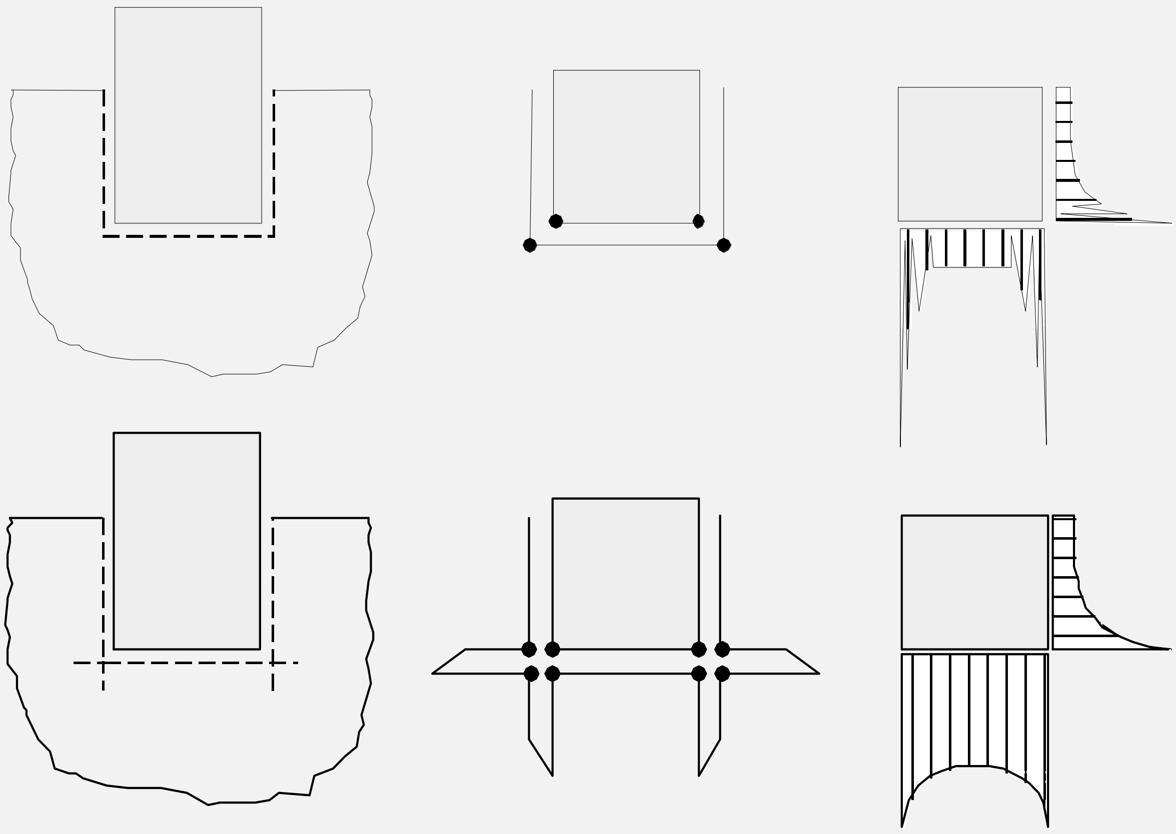
b) 75 % of the design shear strength to be mobilized in the soil itself, that is using:

$$\text{design } \tan\delta = 0.75 \times \text{design } \tan\varphi' \qquad (5)$$

$$\text{design } c_{\mathrm{w}} = 0.75 \times \text{design } c_{\mathrm{u}} \qquad (6)$$

Interfaces

- Try to omit stress oscillations at corners of stiff structures



Inflexible corner points, may cause bad stress results

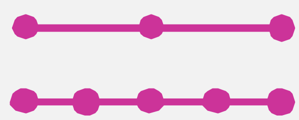
Flexible corner points with improved stress results

STRUCTURAL ELEMENTS

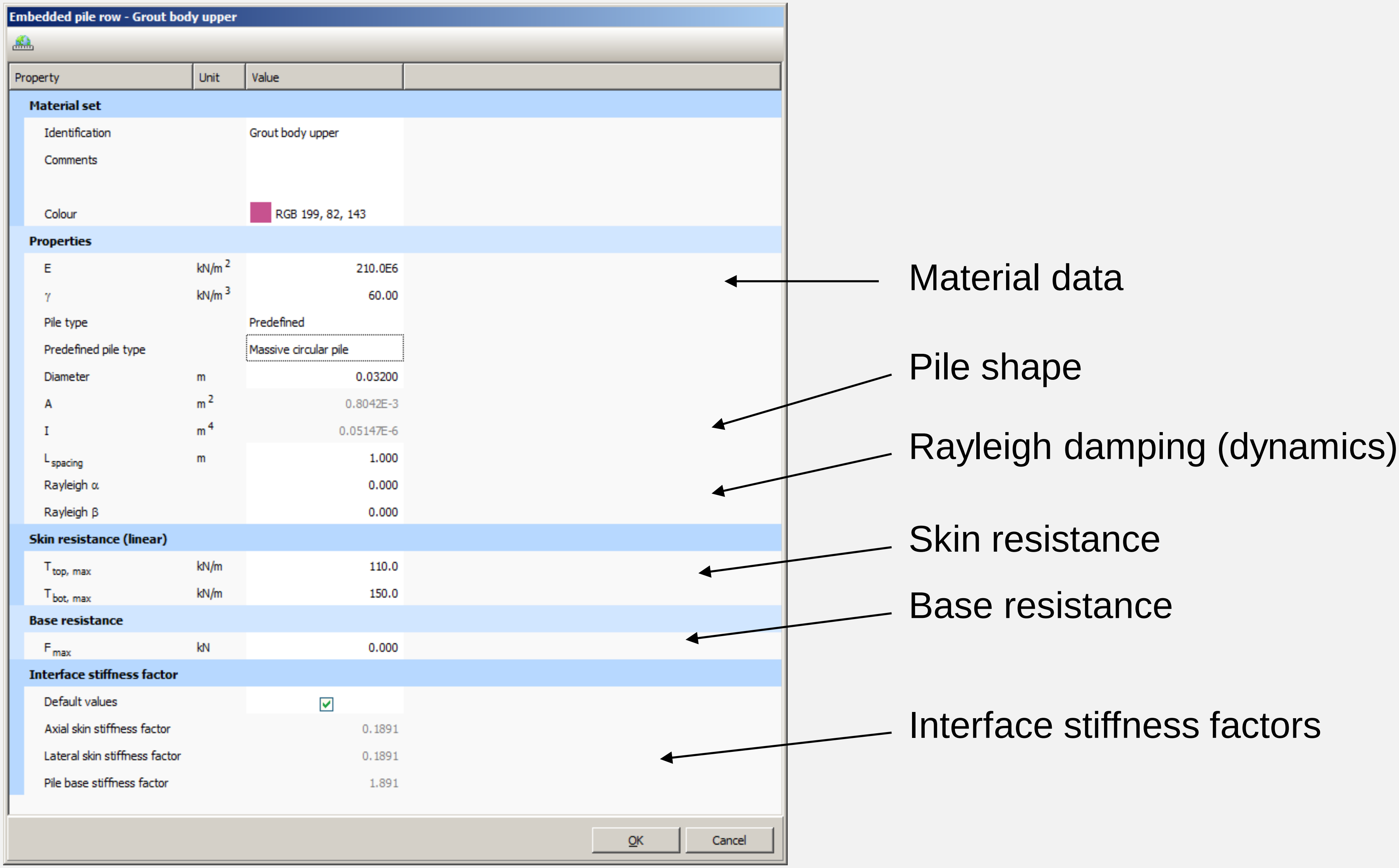
7.EMBEDDED BEAM ROW ELEMENTS

Embedded pile row (Embedded Beam Row)

- 3 or 5-noded line element
- Interaction with soil through an interface allowing for skin friction (linear skin resistance distribution)
- End-bearing capacity through spring connection
- Soil can “flow” in between piles
- Available from Plaxis 2D 2012



Embedded pile row - parameters



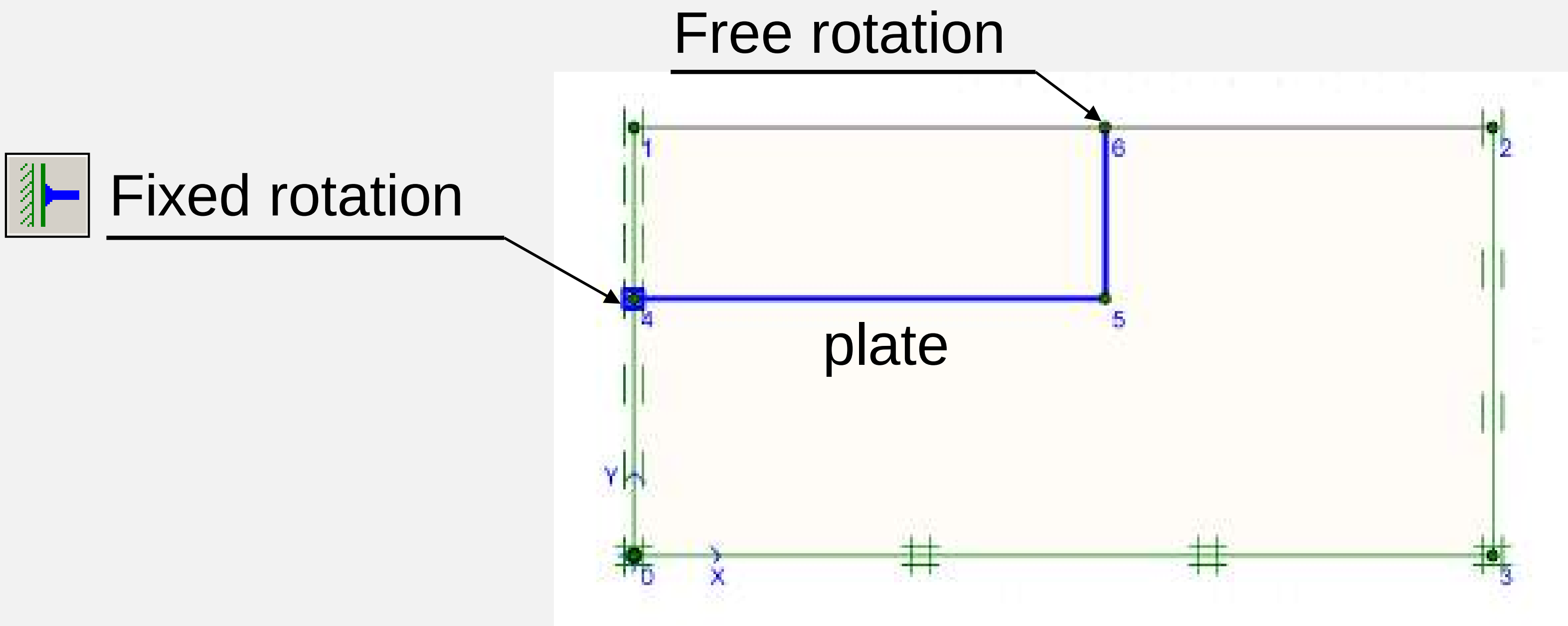
Embedded pile row – parameters

1. Material data (stiffness *E* and material weight *γ*)
2. Pile shape (circular or square, hollow or massive, user-defined)
3. Cross sectional data (area *A*, moment of inertia *I*)
4. Out-of-plane spacing
5. Skin resistance at top and bottom of the pile row ($T_{top,max}$ and $_{Tbottom, max}$)
6. Base resistance (F_{max})
7. Interface stiffness factors
   - Determined by curve fitting on predefined load-displacement curves
   - Default values obtained from the load-displacement curve for a pile in Dutch soil conditions (bored pile in sand, submerged) according to the national annex of Eurocode 7.

GOOD USAGE AND APPLICATION OF STRUCTURAL ELEMENTS

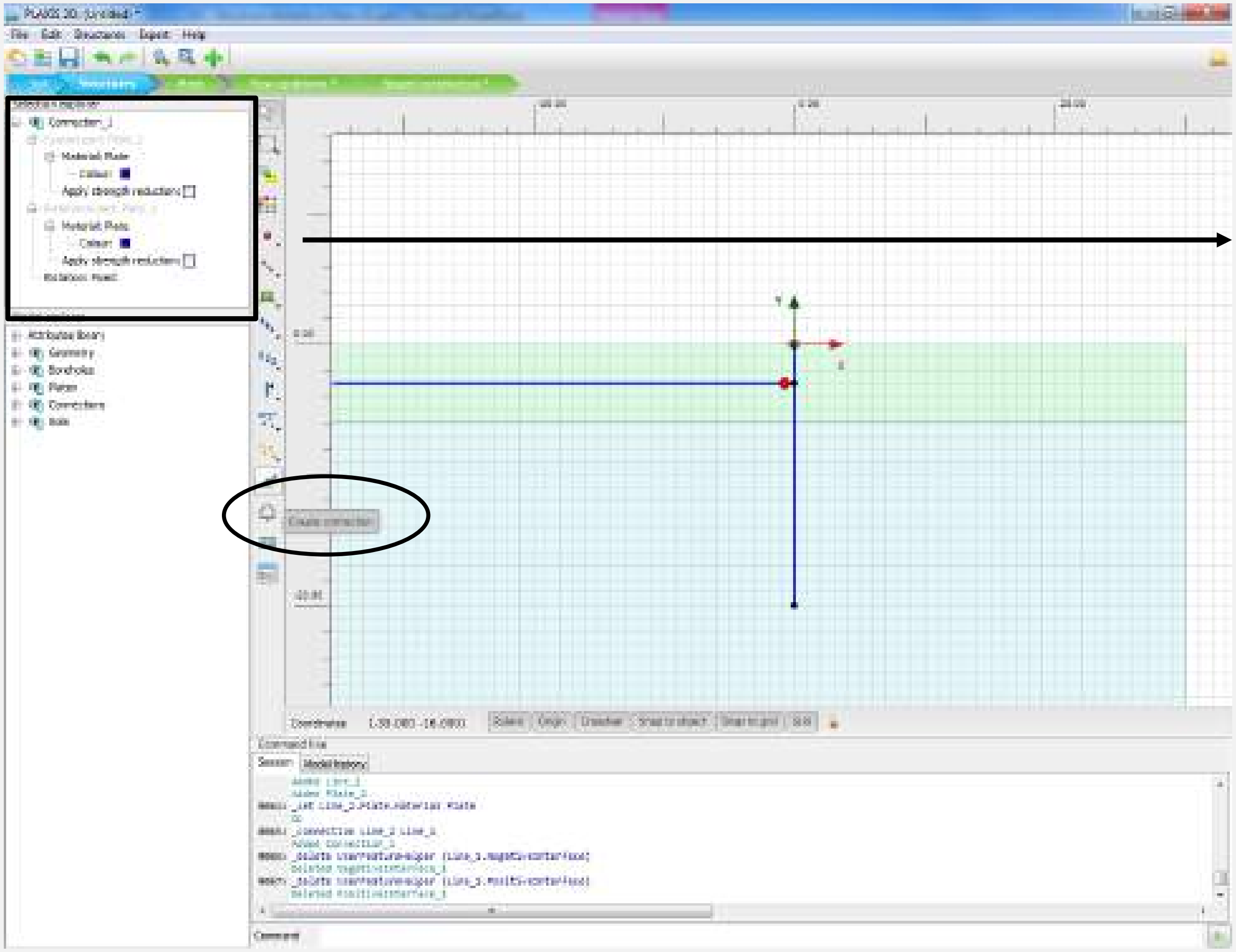
ADDITIONAL INSIGHTS

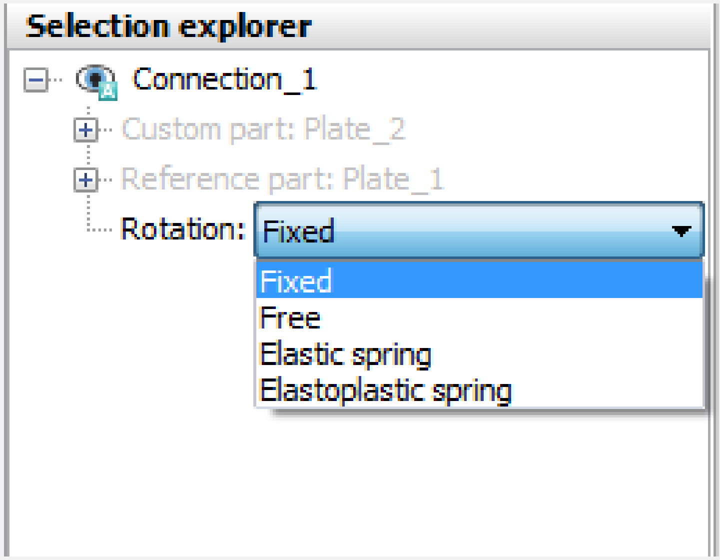
Plates / Shells - Boundary Conditions



- Rotation fixed at (partly) fixed boundaries – axis of symmetry
- Rotation free at free boundaries

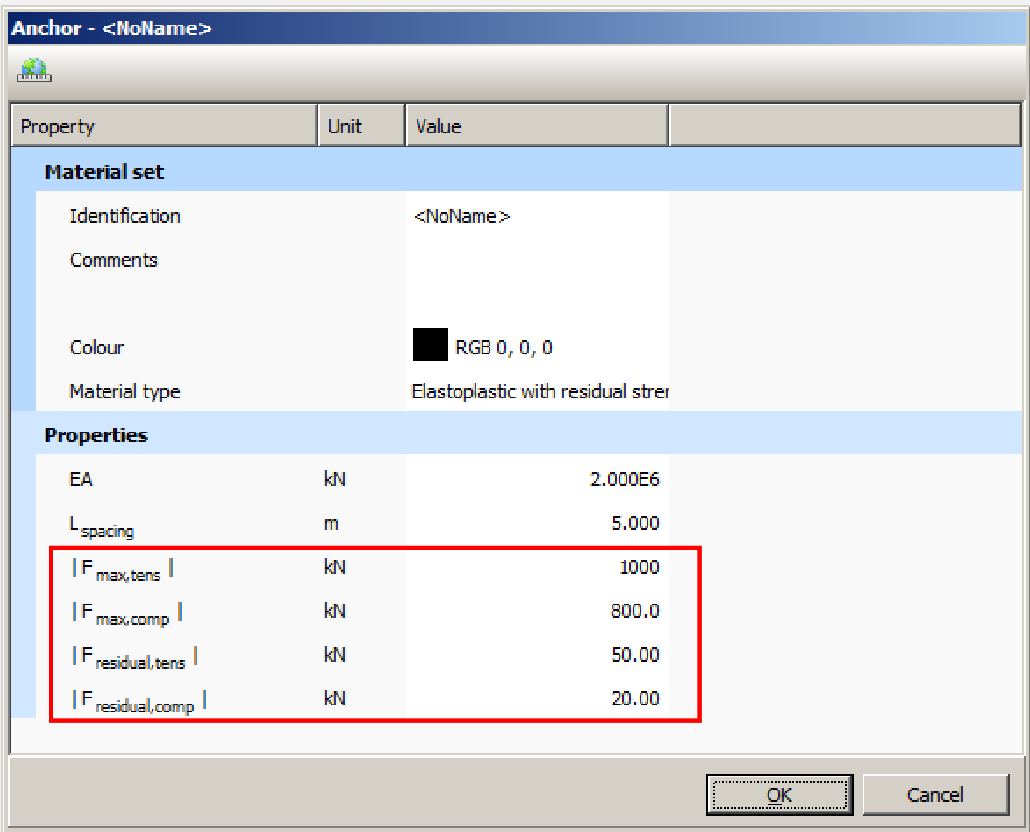
Plates / Shells – Connections

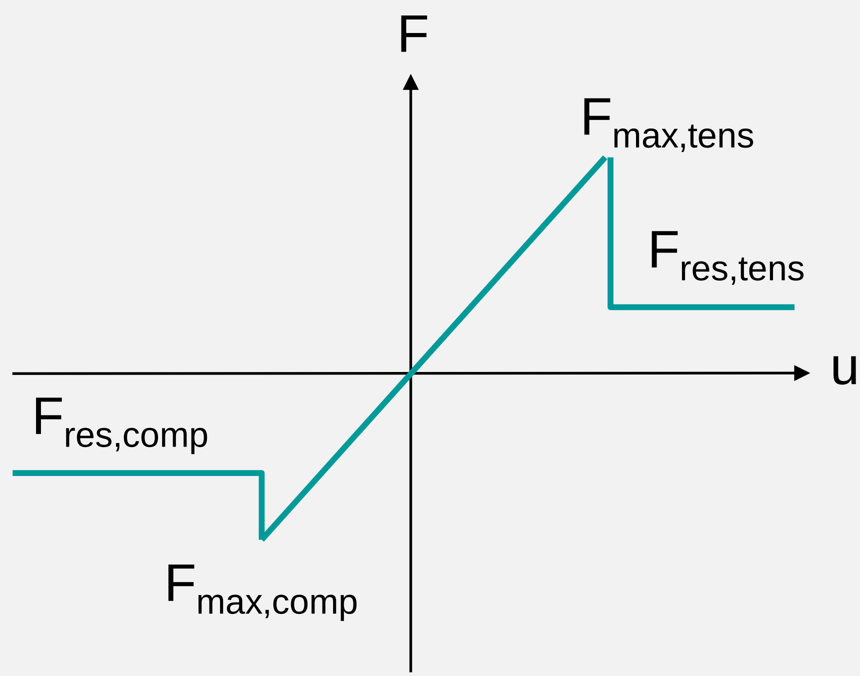




Anchors - Material Properties

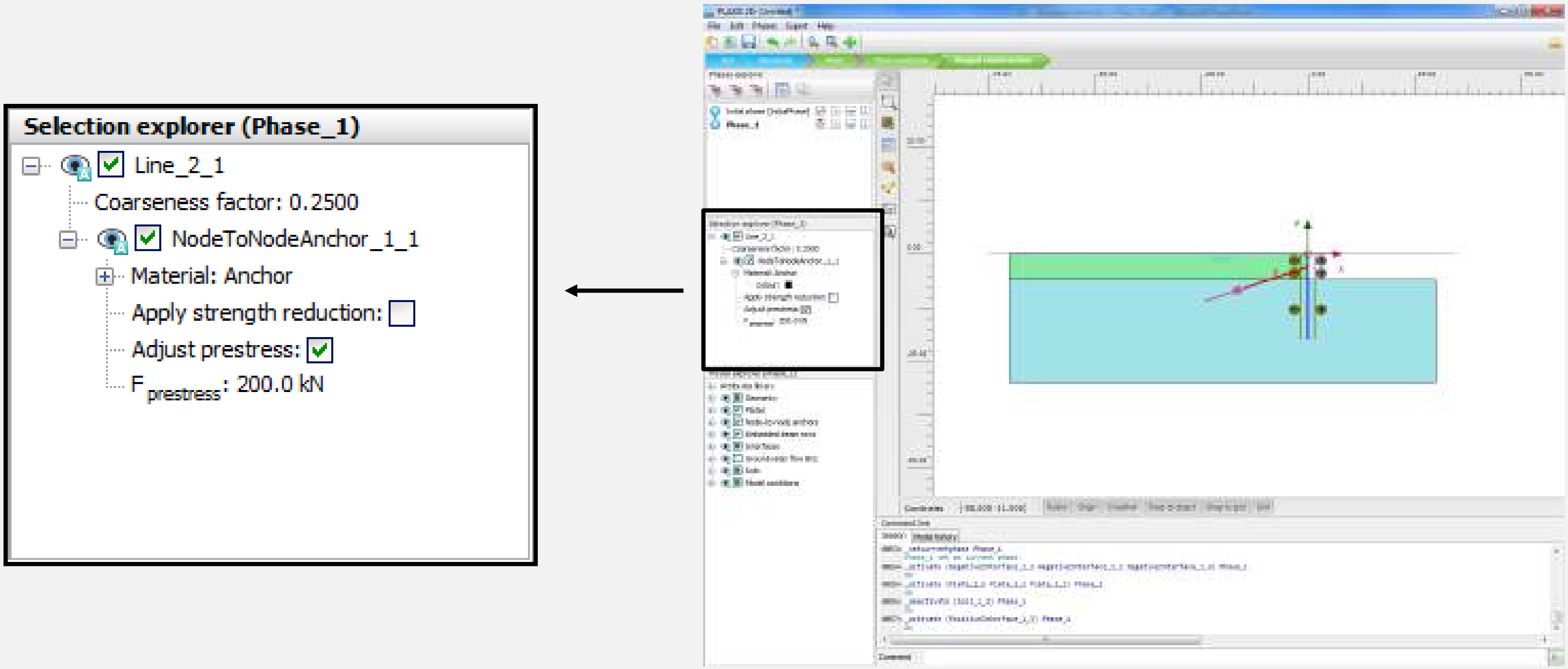
Residual strength (Plaxis 2D 2012)





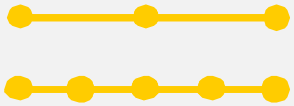
Anchors - Pre-Stressing

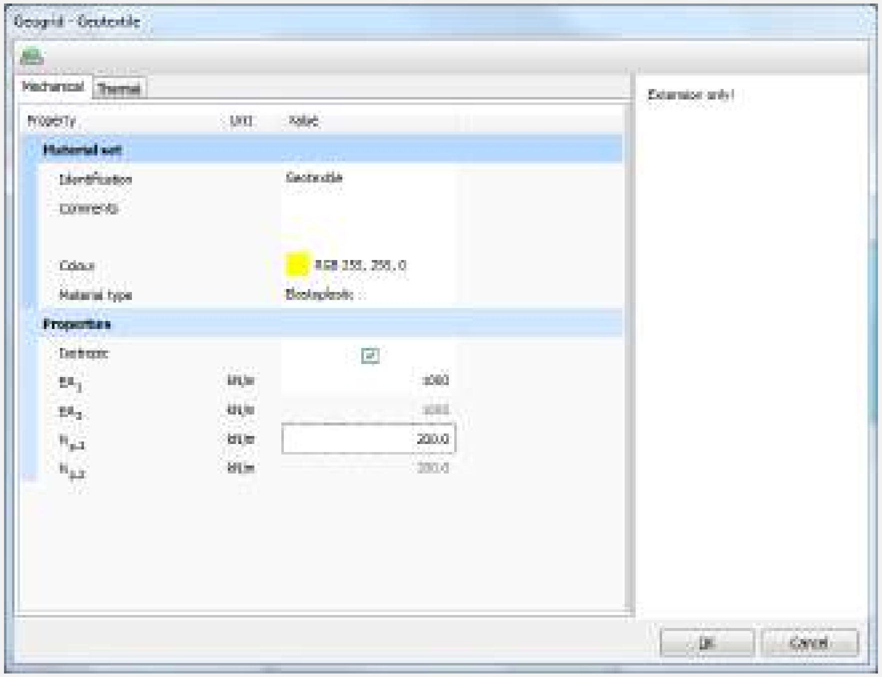
- **Defined in Staged Construction phase**
- **Both tension (grout anchor) or compression (strut) possible**



Geogrids

1. 3 or 5 noded line element
2. Elastic or elasto-plastic behaviour
3. No flexural rigidity (EI), only axial stiffness (EA)
4. Only allows for tension, not for compression
5. Updated Mesh option relevant





Grouted Anchors

Combination of free length and bonded length

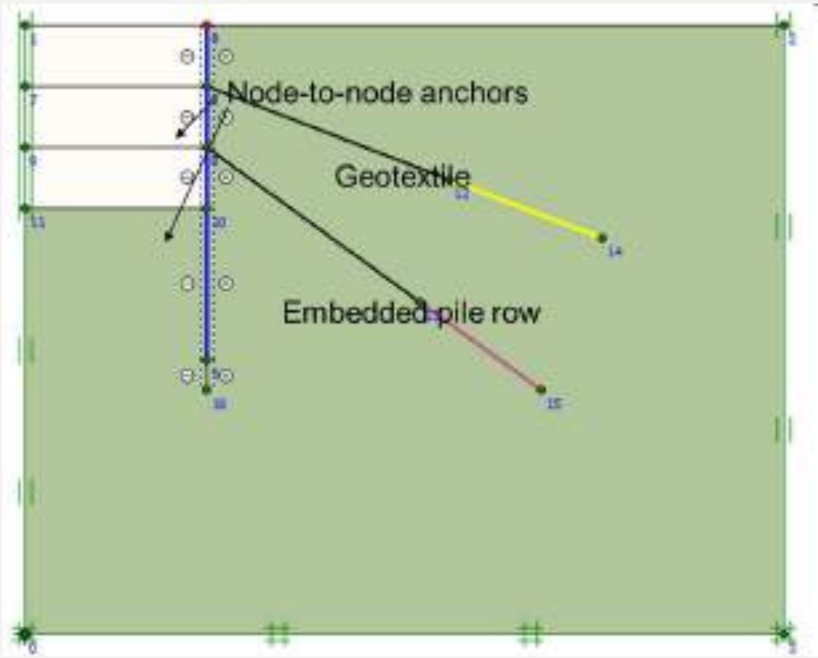
1. **<u>Free length</u>** is modelled using a node-to-node anchors (no interaction with surrounding soil)
2. **<u>Bonded length</u>** is modelled using either as:

   A. Geotextile

   - Full bonding with the soil
   - No interface around grouted part (interface may create unrealistic failure surface)

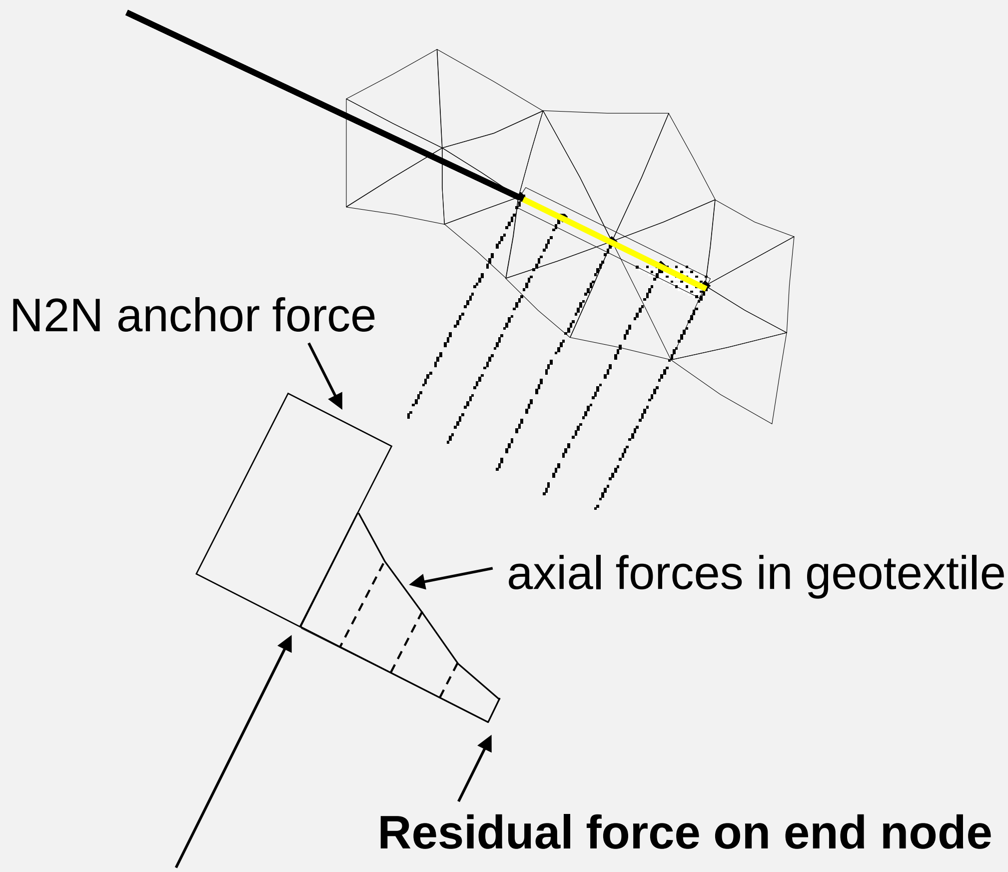
   B. Embedded pile row (Plaxis 2D 2012 and higher)

   - Allows for slip
   - Allows for soil to move in between anchors (more realistic)

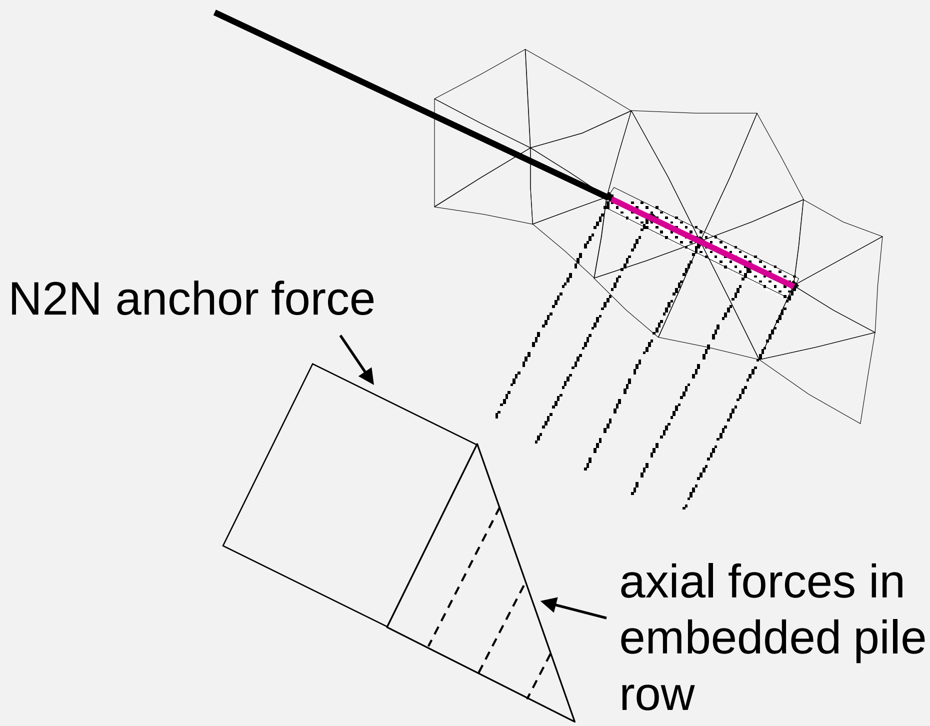


Grouted Anchors - Axial Forces

Geotextile as grout body



Embedded pile row as grout body



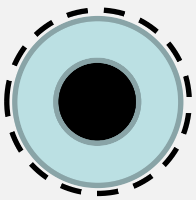
N_{rod} <> N_{grout} due to shared node between anchor, geotextile and soil

Grouted Anchors – Practical Use

Pullout mechanism technically possible with embedded pile row

1. Properties of the steel rod for the beam

q_s(kPa)



2. Linear skin resistance based on cylindrical grout body shape and pull-out strength qs

$T_{skin,start} = T_{skin,end} = q_s \pi D_{grout}$

3. Base resistance set to 0

4. Interface stiffness factor set to default values

Grouted Anchors – Practical Use

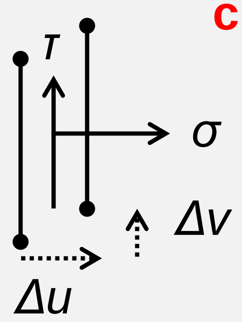
1. **Some practical uncertainties still remains:**
   - **Size and shape of the grout body**
   - **Degree of bonding between grout body and soil**

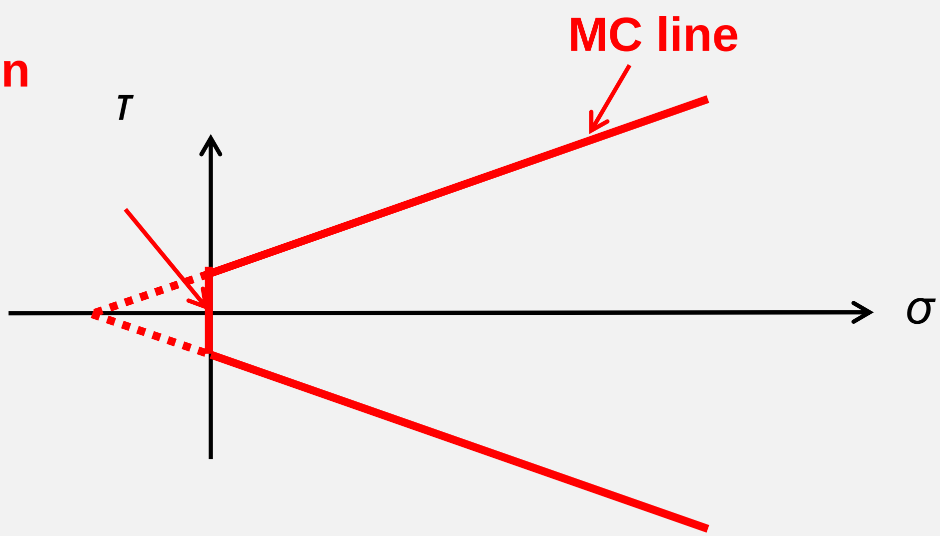
2. **Alternative: If pull-out force is known this can be used by limiting anchor rod force**

Soil-Structure Interaction: Interface Elements

1. **Interface elements handle normal and shear behaviour**
    - Linear elastic perfectly plastic behaviour
    - Shear strength defined according MC criterion (slipping)
    - Tension cut-off (gapping)

$$\begin{cases} \boldsymbol{\sigma} = \boldsymbol{f}(\Delta \boldsymbol{u}) \\ \boldsymbol{\tau} = \boldsymbol{g}(\Delta \boldsymbol{v}) \end{cases}$$





Interfaces – Material Properties

Soil material properties

1. Taken by default from soil using reduction factor R_{inter}

$$C_{inter} = R_{inter} * C_{soil}$$

$$\tan(\varphi_{inter}) = R_{inter} * \tan(\varphi_{soil})$$

$$\psi_{inter} = 0 \text{ for } R_{inter} < 1$$

$$= \psi_{soil}\ R_{inter} = 1$$

$$\sigma_{t,inter} = R_{inter} * \sigma_{t,soil}$$

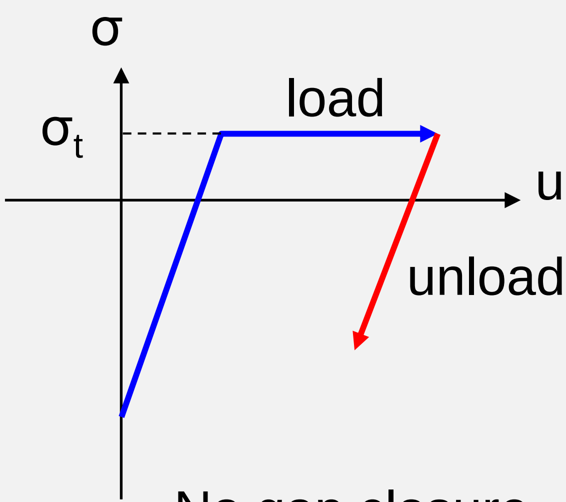
$$G_{inter} = (R_{inter})^2 * G_{soil}$$

2. Residual reduction factor $R_{inter,res}$ only affects strength, not stiffness.
3. Individual material set for interface possible

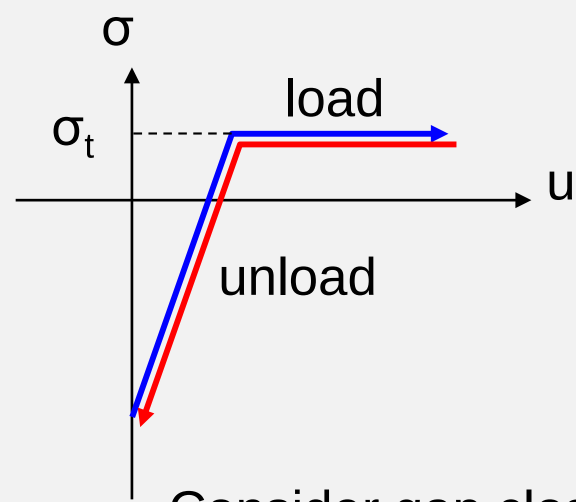
Interfaces – Material Properties

Additional features

- Residual strength when reaching maximum shear strength
- Gap closure



No gap closure



Consider gap closure

END

SCIENCE ▪ PASSION ▪ TECHNOLOGY

CG08

DEEP EXCAVATIONS

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Pelatihan software Plaxis di Tembagapura (In-house training) PT. Freeport Indonesia/ Papua, 22-30 September 2018

CONTENTS

NUMERICAL ANALYSIS OF DEEP EXCAVATIONS

involves modelling of

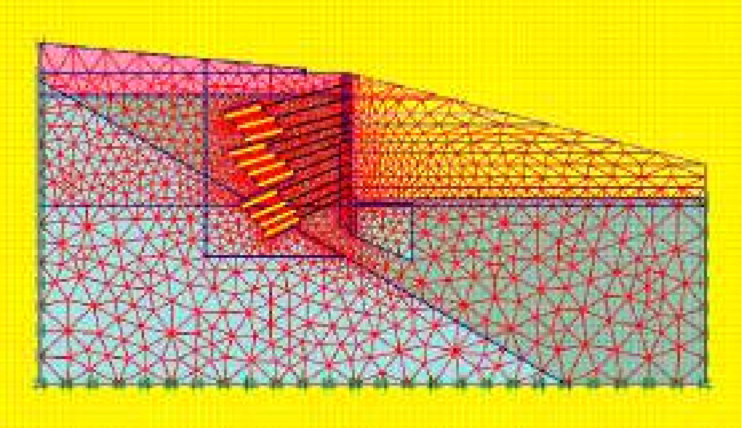
- various excavation stages
- interaction wall / soil > interface elements
- changes in groundwater level
- struts or anchors (including load transfer to soil)
- adjacent structures (buildings, tunnels, piles, …)

requires advanced constitutive model because

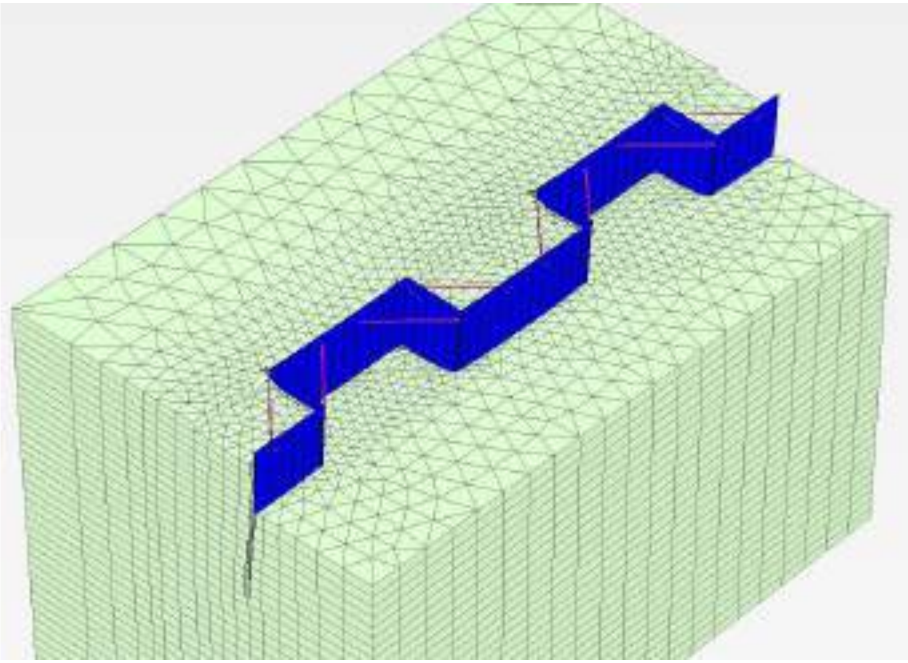
- stress paths in soil are not monotonic (significant change in stress path direction)
- primary loading and unloading / reloading occurs in different parts of the domain analysed
- some areas will experience large strains with significant plastic deformations, others will be in the very small strain range

> simple elastic - perfectly plastic models not sufficient

2D model sufficient

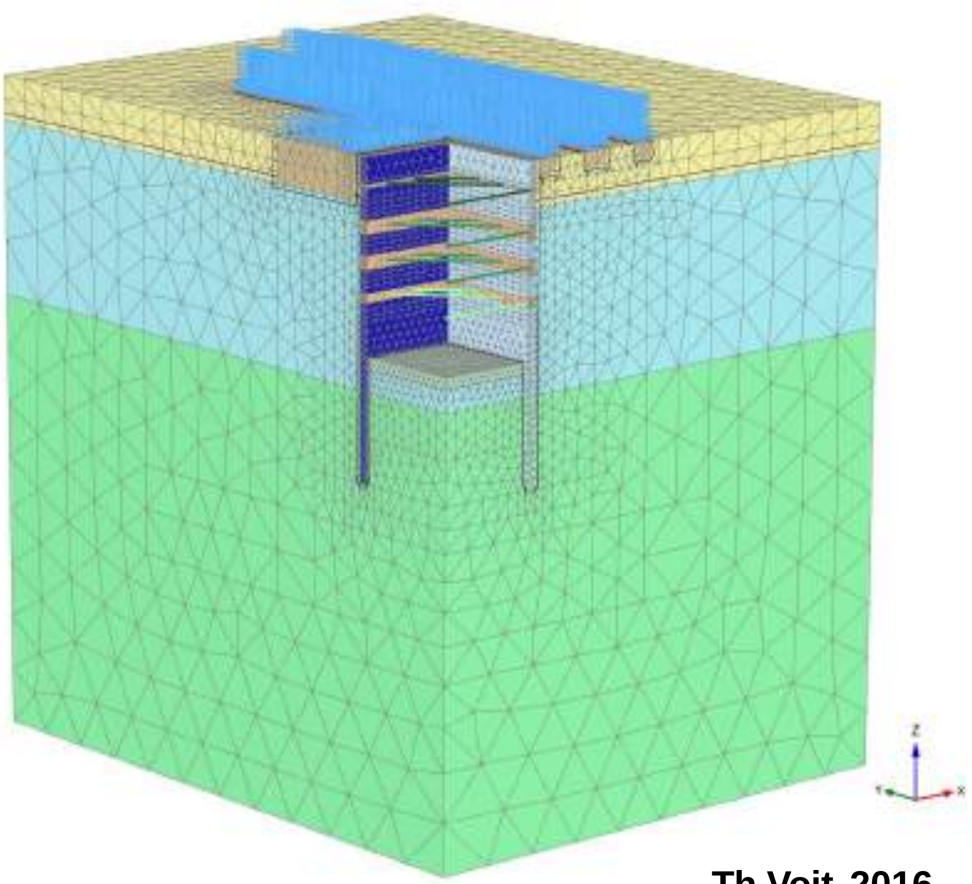


3D model preferable, but 2D approximation possible



INTRODUCTION

3D model required



Th.Voit, 2016

HYDRAULIC CONDITIONS

- **Setting of new hydraulic conditions**
  - Consider each individual case carefully
  - Select the relevant situation: undrained, drained or consolidation, it may be different for different materials

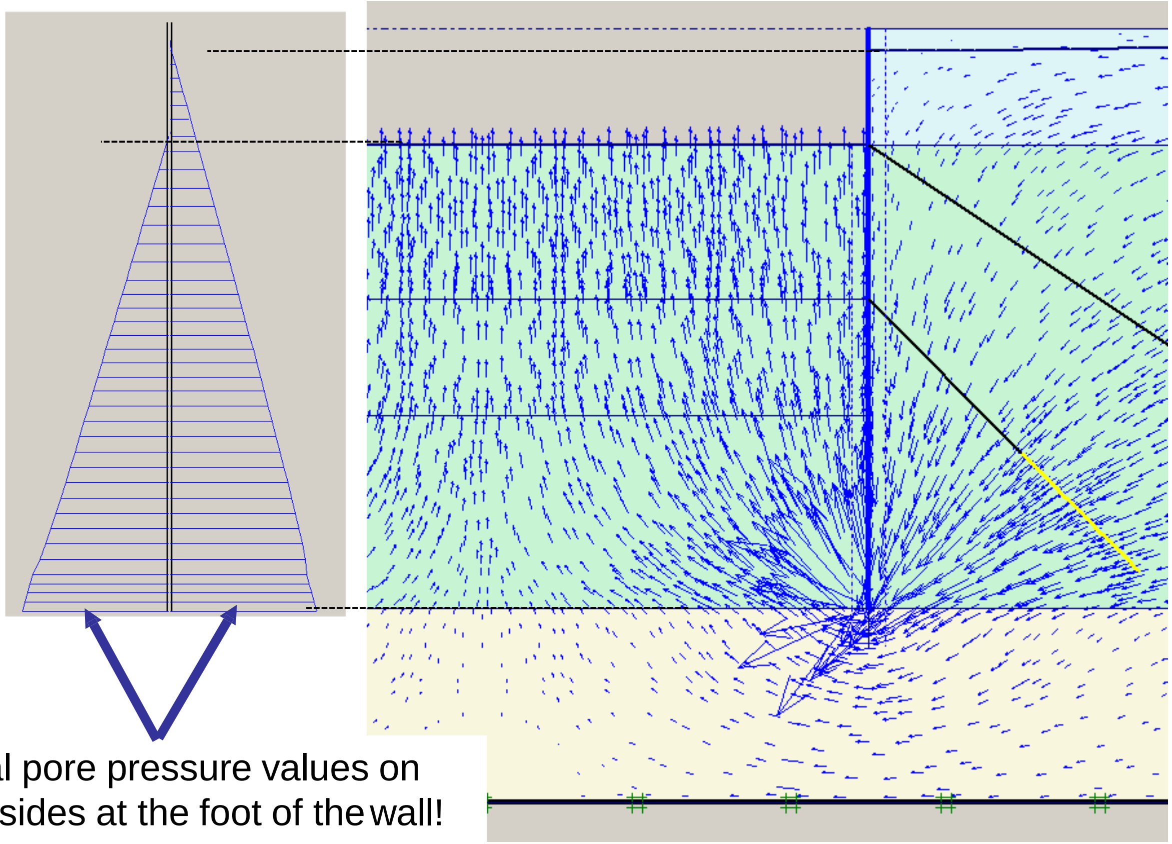
- **Specifying pore pressure distributions**
  - **General phreatic level**
    > applies to all clusters that have not been separately defined
  - **Cluster phreatic level** > applies to one specific cluster
  - **Cluster dry** > makes a specific cluster dry
  - **Interpolate** > interpolates pore pressures between clusters above and below
  - **User-defined pore pressure**

    > specify pore pressure at reference level and the rate of increase in the *y*-direction
  - **Groundwater flow calculation** > gives the steady-state solution

HYDRAULIC CONDITIONS

Groundwater flow calculations

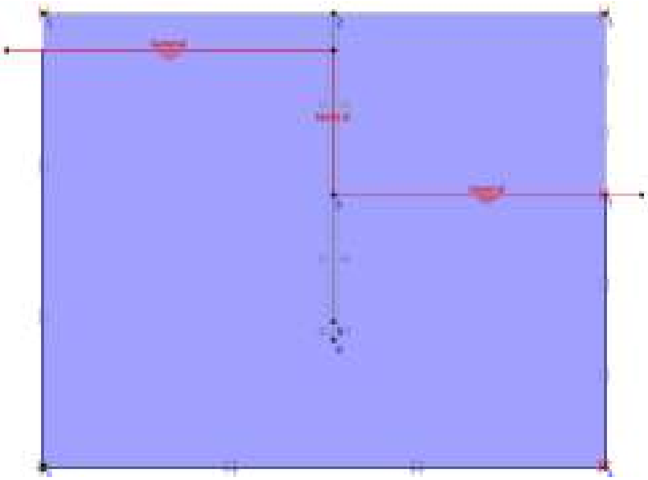
- Plate elements are permeable
- Interface elements are impermeable (unless deactivated in the groundwater flow calculation)

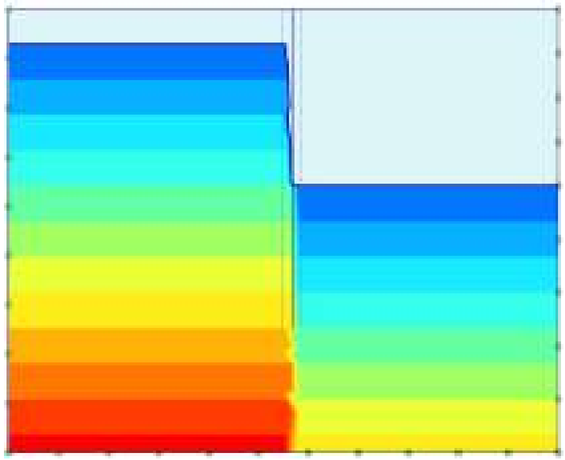


Equal pore pressure values on both sides at the foot of the wall!

HYDRAULIC CONDITIONS

Dewatering



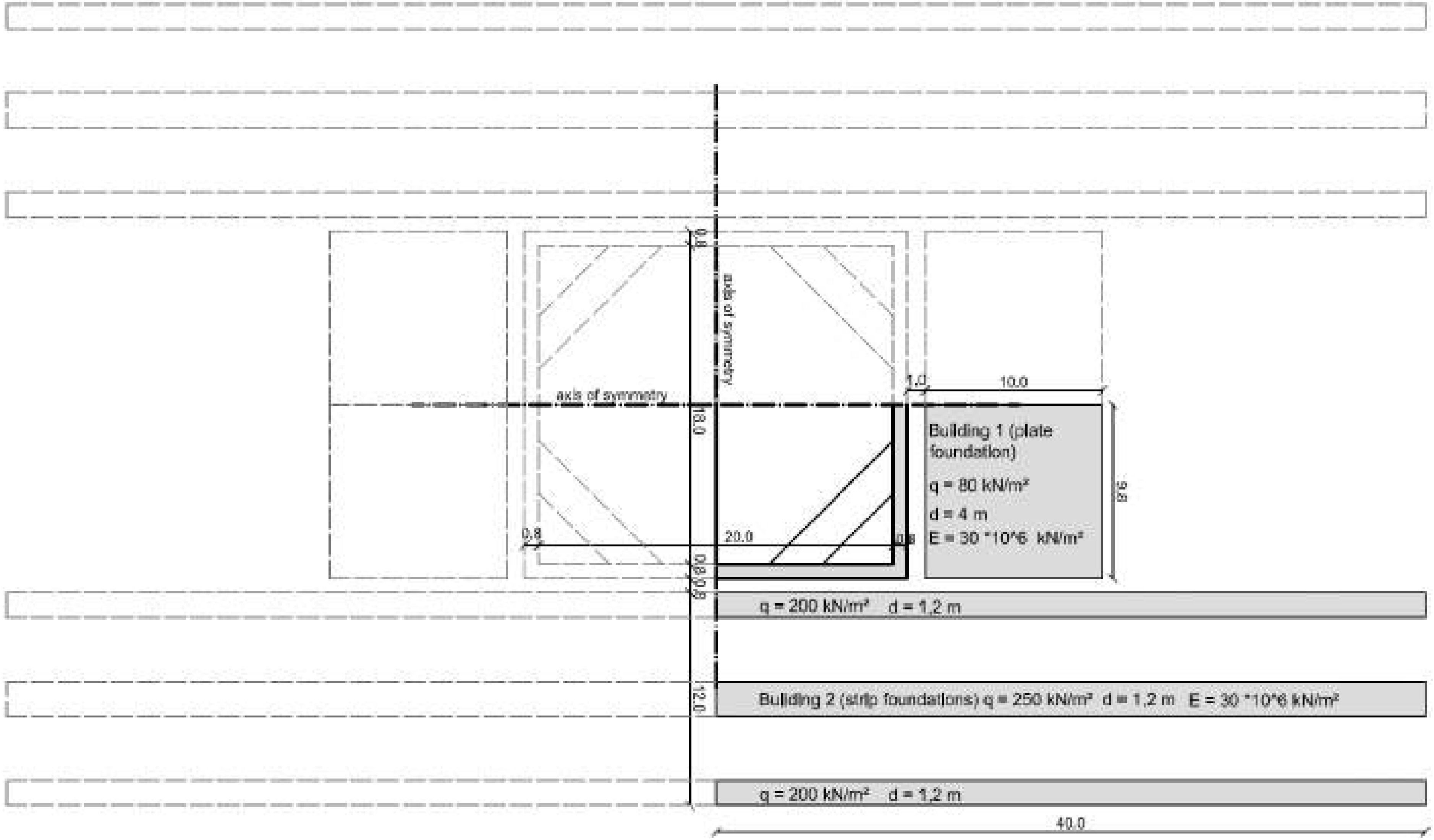


Z-shape phreatic level gives wrong results:

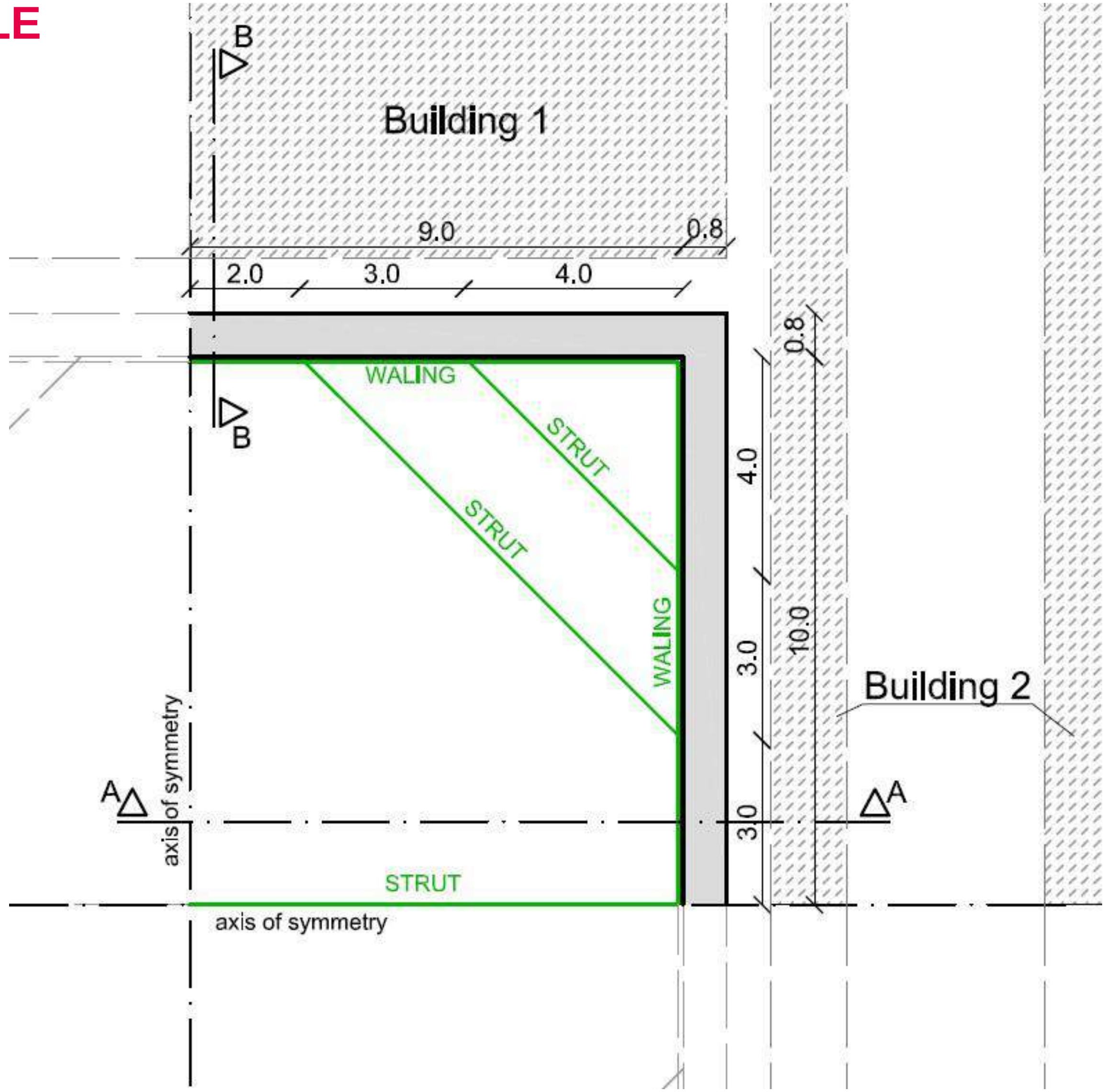
- No equilibrium in horizontal water pressures:
  - Local peak stresses
  - Local peak strains
  - Non-physical horizontal displacements
  - Non-physical excess pore pressures

- Possible incorrect water pressure acting on wall

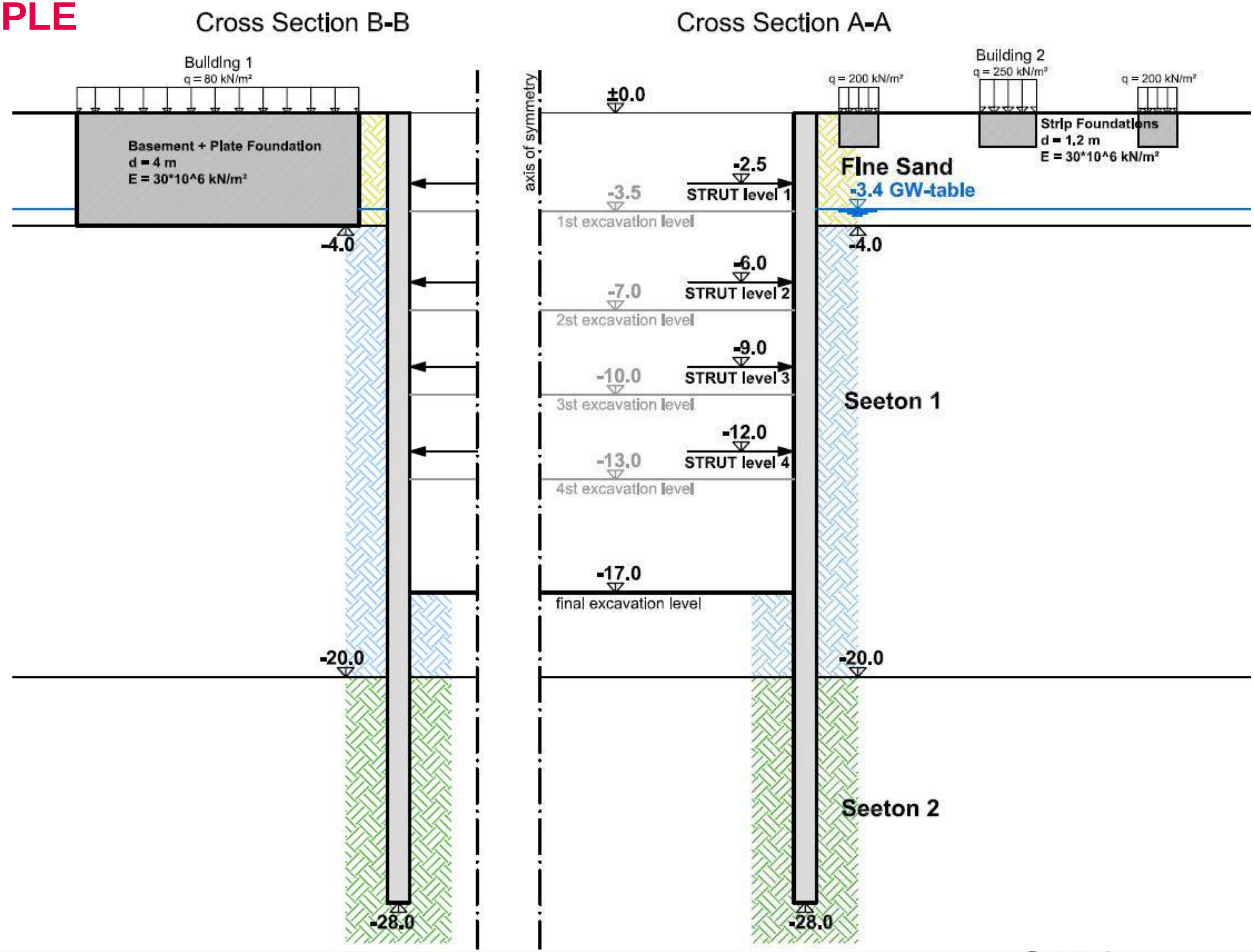
3D EXAMPLE


axis of symmetry
axis of symmetry
0.8
18.0
0.8
20.0
0.8
0.8
0.8
1.0
10.0
9.8
Building 1 (plate foundation)
q = 80 kN/m²
d = 4 m
E = 30 *10^6 kN/m²
q = 200 kN/m²   d = 1.2 m
12.0
Building 2 (strip foundations) q = 250 kN/m²   d = 1.2 m   E = 30 *10^6 kN/m²
q = 200 kN/m²   d = 1.2 m
40.0

3D EXAMPLE


B
Building 1
9.0
0.8
2.0
3.0
4.0
WALING
B
STRUT
STRUT
WALING
4.0
3.0
3.0
0.8
10.0
Building 2
A
A
axis of symmetry
STRUT
axis of symmetry

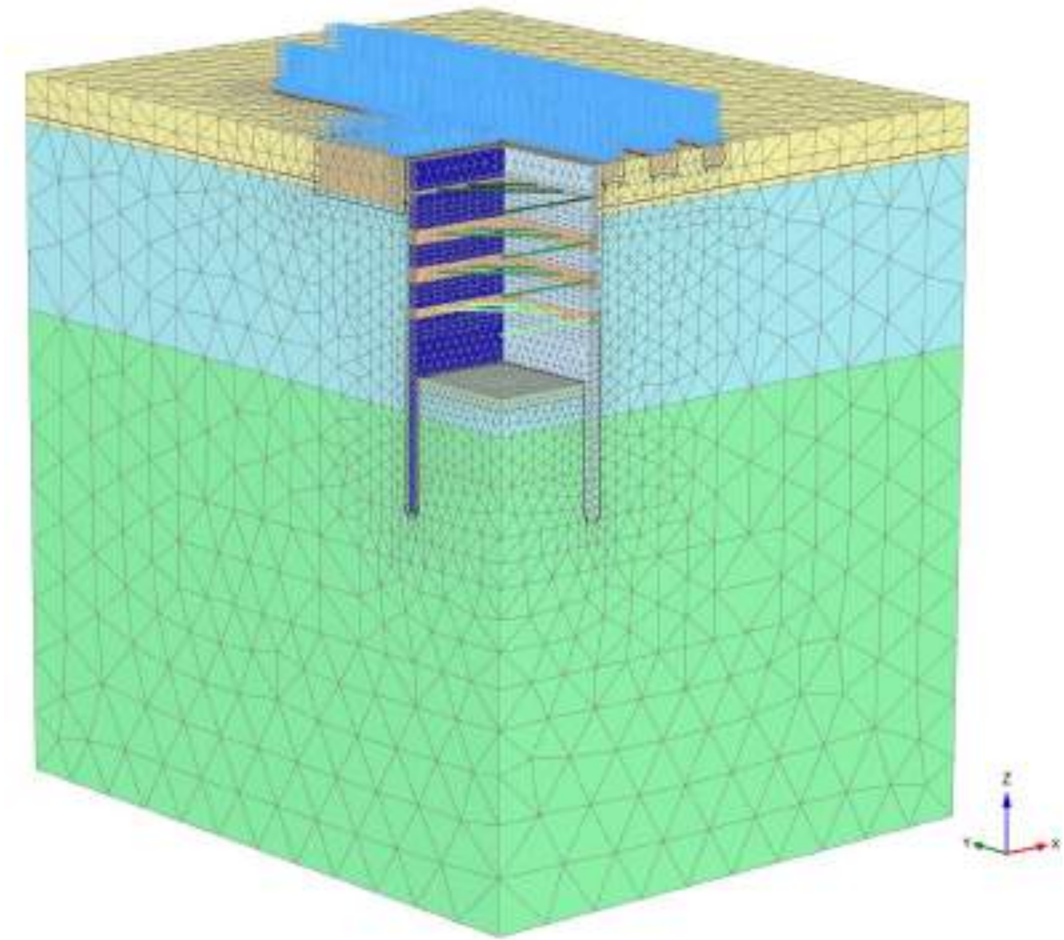
3D EXAMPLE



Cross Section B-B
Cross Section A-A
Building 1
q = 80 kN/m²
Basement + Plate Foundation
d = 4 m
E = 30*10^6 kN/m²
-4.0
-20.0
-28.0
axis of symmetry
±0.0
-2.5
STRUT level 1
-3.5
1st excavation level
-6.0
STRUT level 2
-7.0
2st excavation level
-9.0
STRUT level 3
-10.0
3st excavation level
-12.0
STRUT level 4
-13.0
4st excavation level
-17.0
final excavation level
Building 2
q = 250 kN/m²
q = 200 kN/m²
q = 200 kN/m²
Strip Foundations
d = 1,2 m
E = 30*10^6 kN/m²
Fine Sand
-3.4 GW-table
-4.0
Seeton 1
-20.0
Seeton 2
-28.0

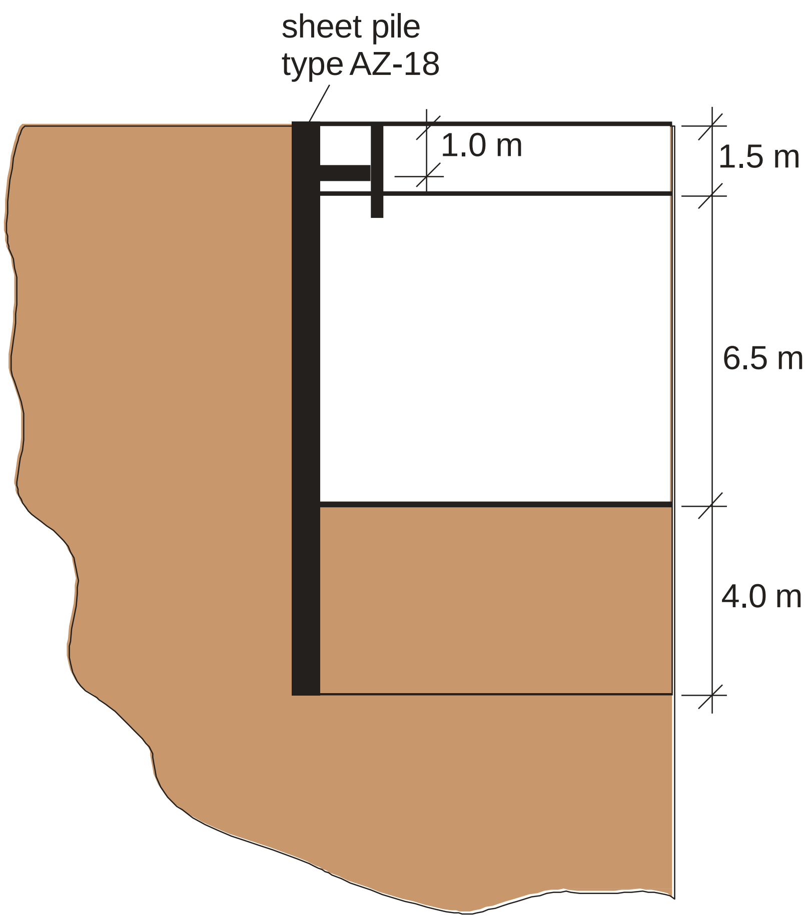
3D EXAMPLE

- Model dimensions
  - x/y/z = 50/60/60 m
- Mesh
  - ~ 150 000 elements



Voit, 2016

NOTE ON φ/c REDUCTION



Material parameters soil layer:

$\varphi = 35°$

$c = 0.1$ kN/m²

$\gamma = 17$ kN/m³

$\psi = 0°$

Properties sheet pile wall:

EA = 3.008E6 kN/m

EI = 6.84E4 kNm^2/m

M_{pl} = 505 kNm/m

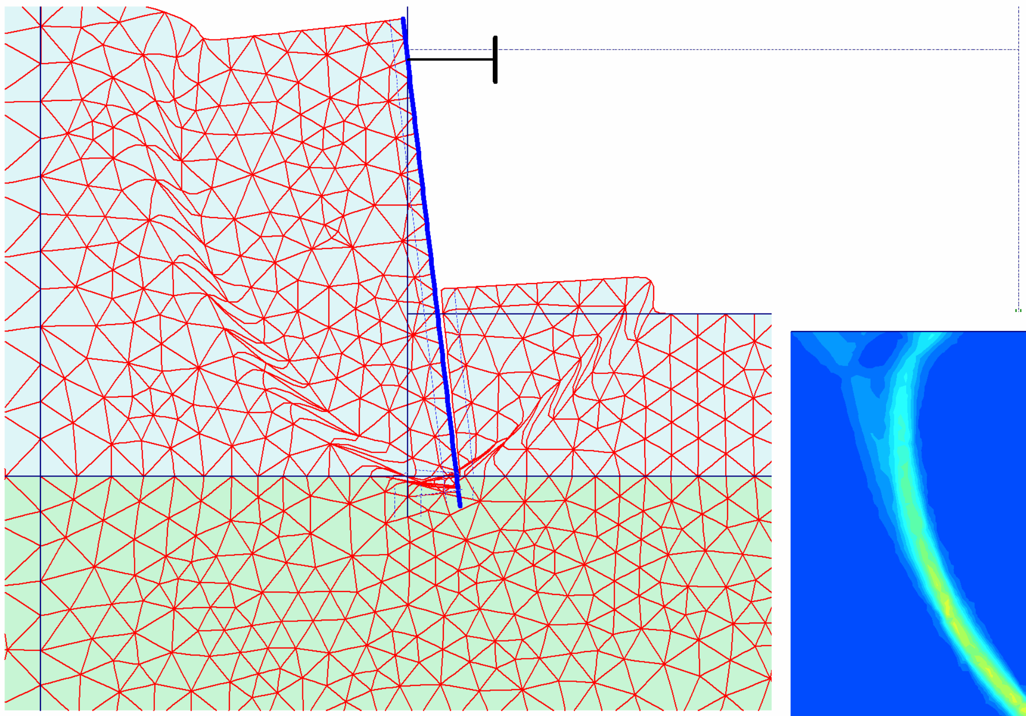
Properties strut:

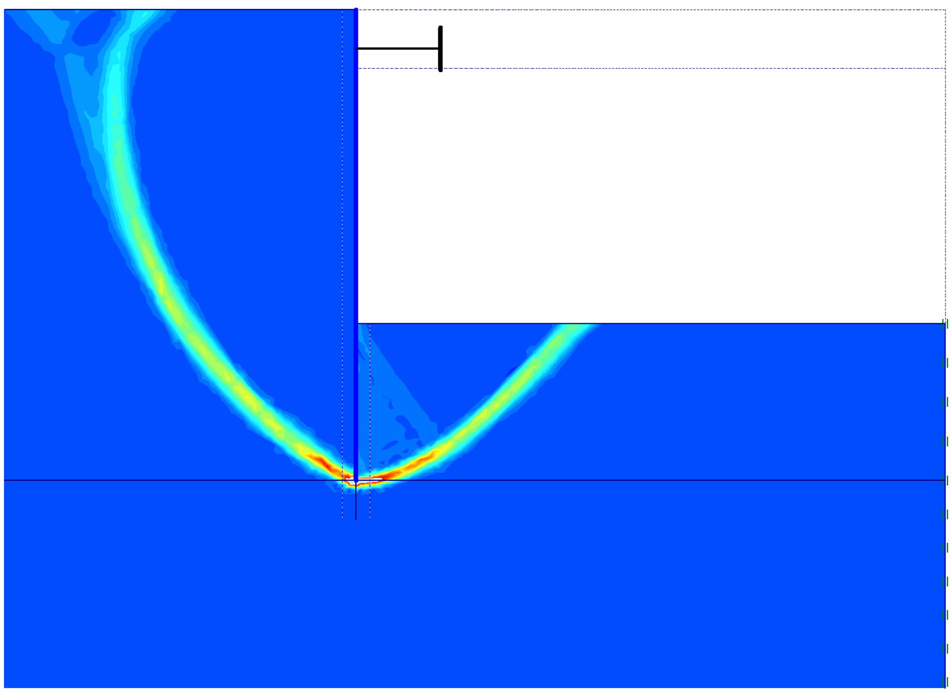
E = 3.0E7 kN/m^2

A = 0.24 m^2

Horizontal strut distance: 1 m

NOTE ON φ/c REDUCTION

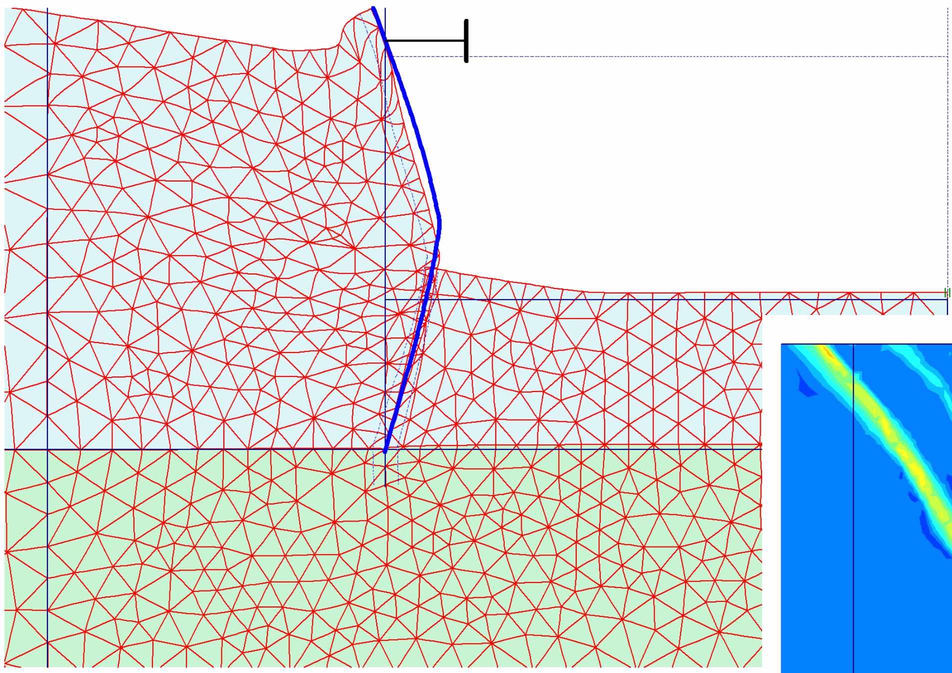


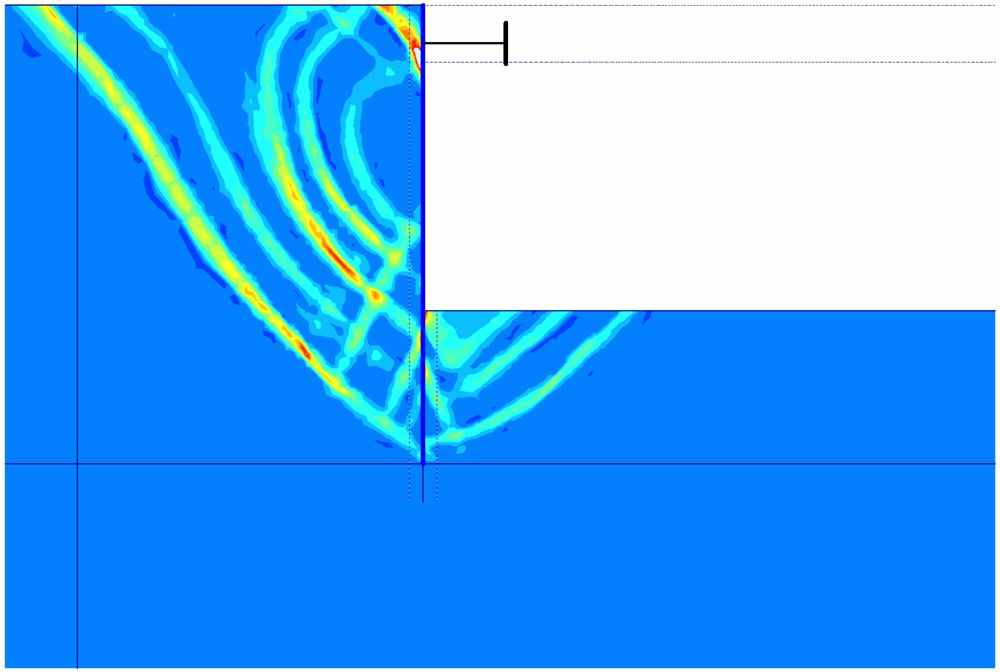


wall elastic

Msf = 1.95

NOTE ON φ/c REDUCTION

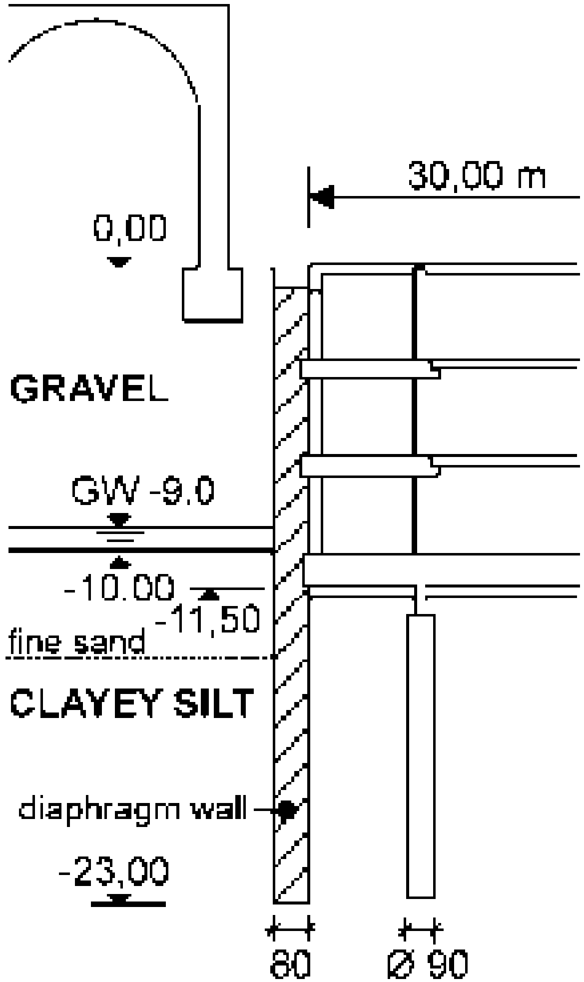




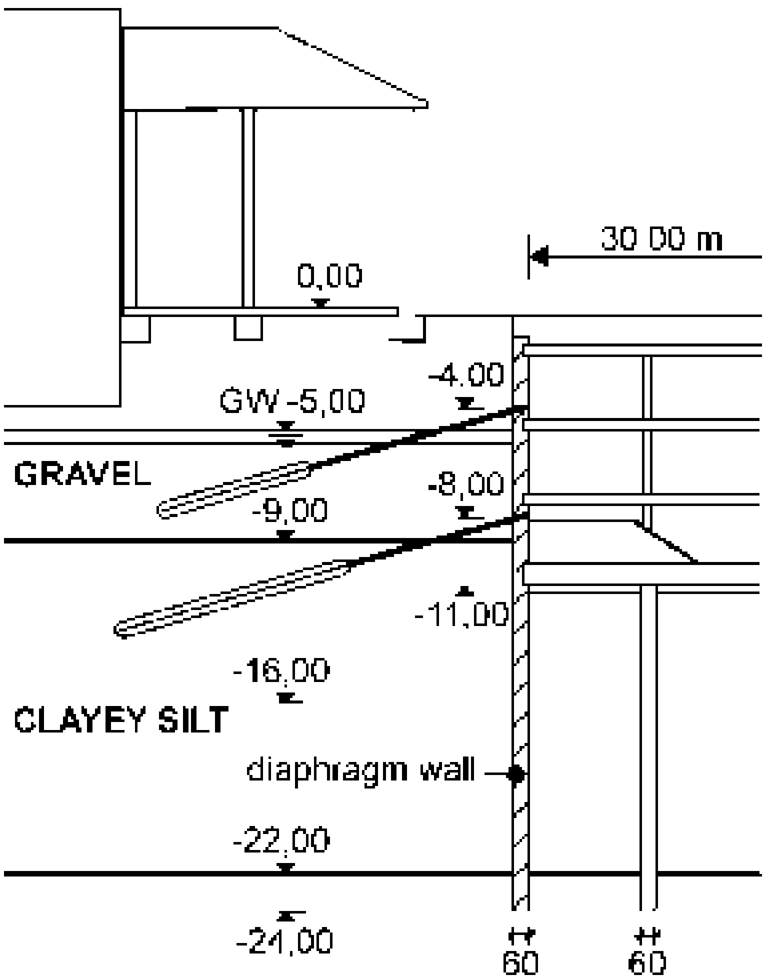
wall elastic - perfectly plastic

Msf = 1.73

BACK ANALYSIS OF DIFFERENT EXCAVATIONS

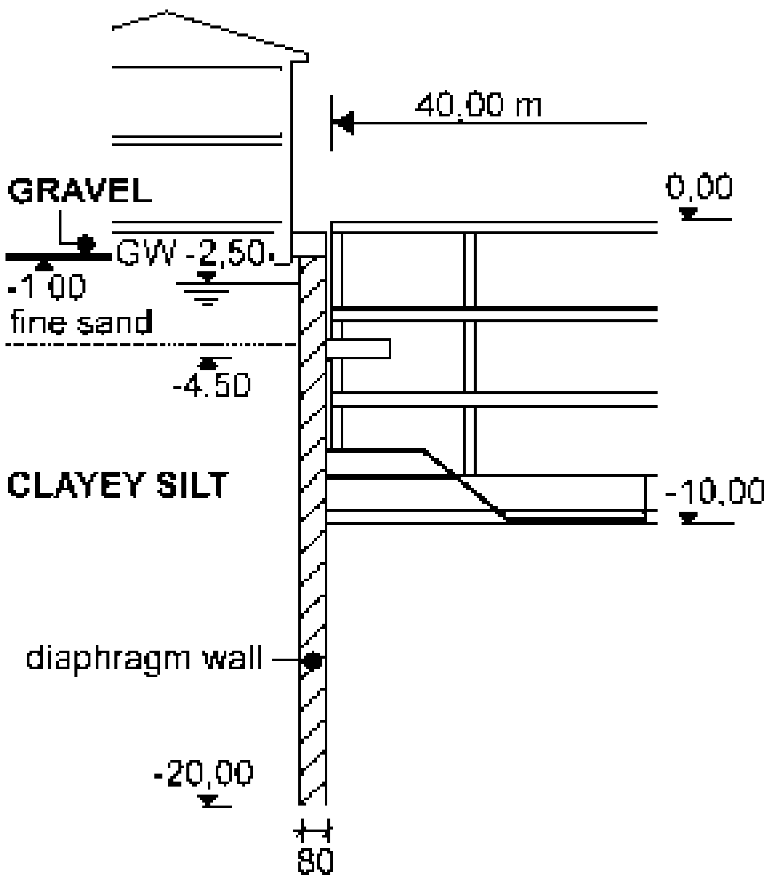


Layout of project "Toskanatrakt"

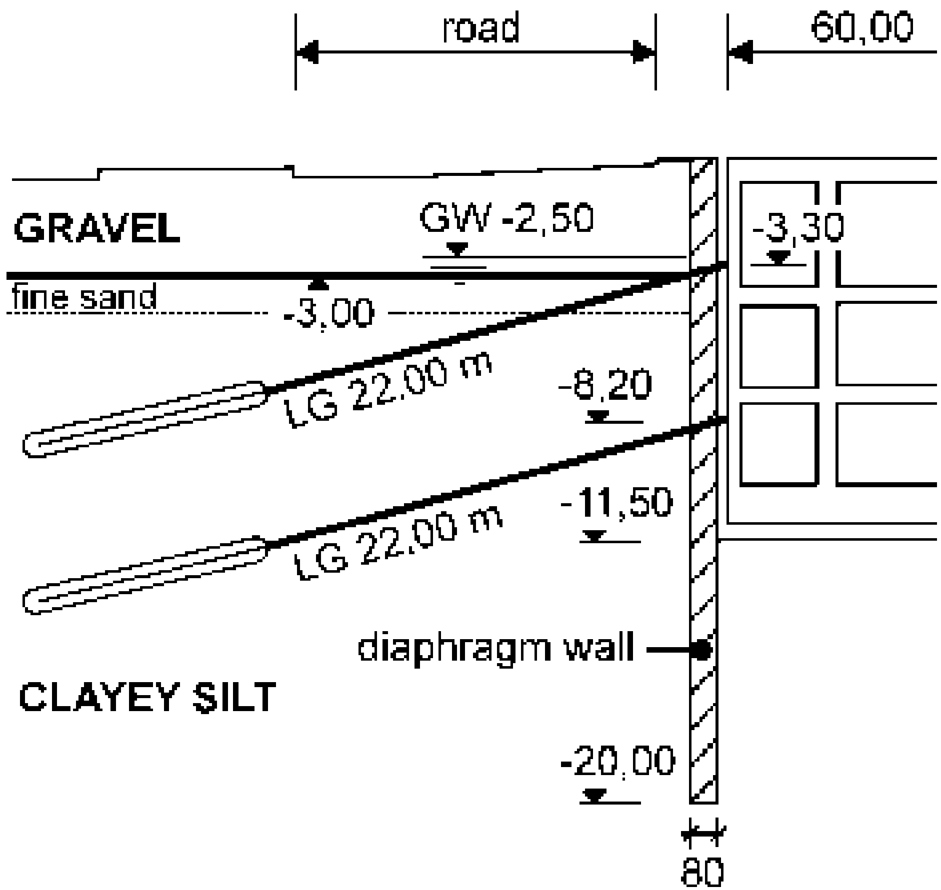


Layout of project "Hypobank"

BACK ANALYSIS OF DIFFERENT EXCAVATIONS

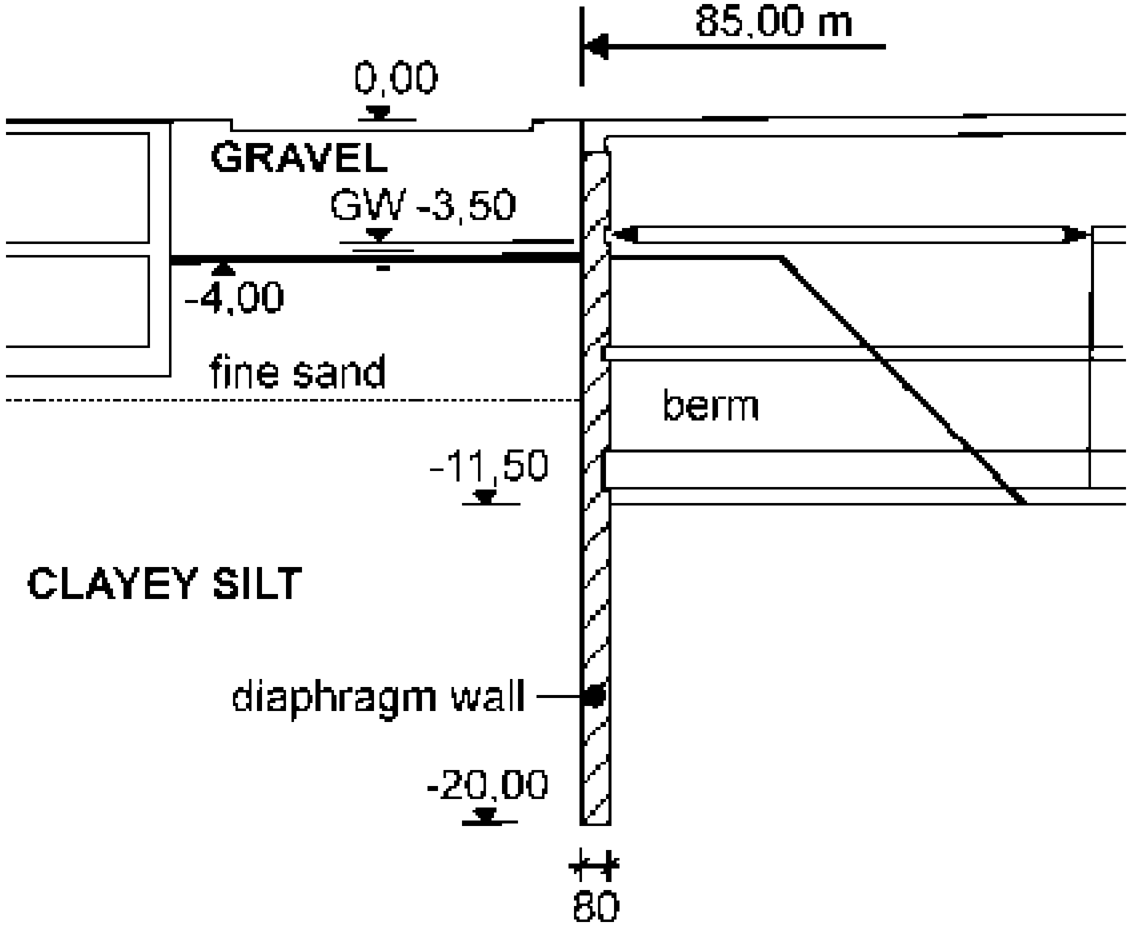


Layout of project "AMV"



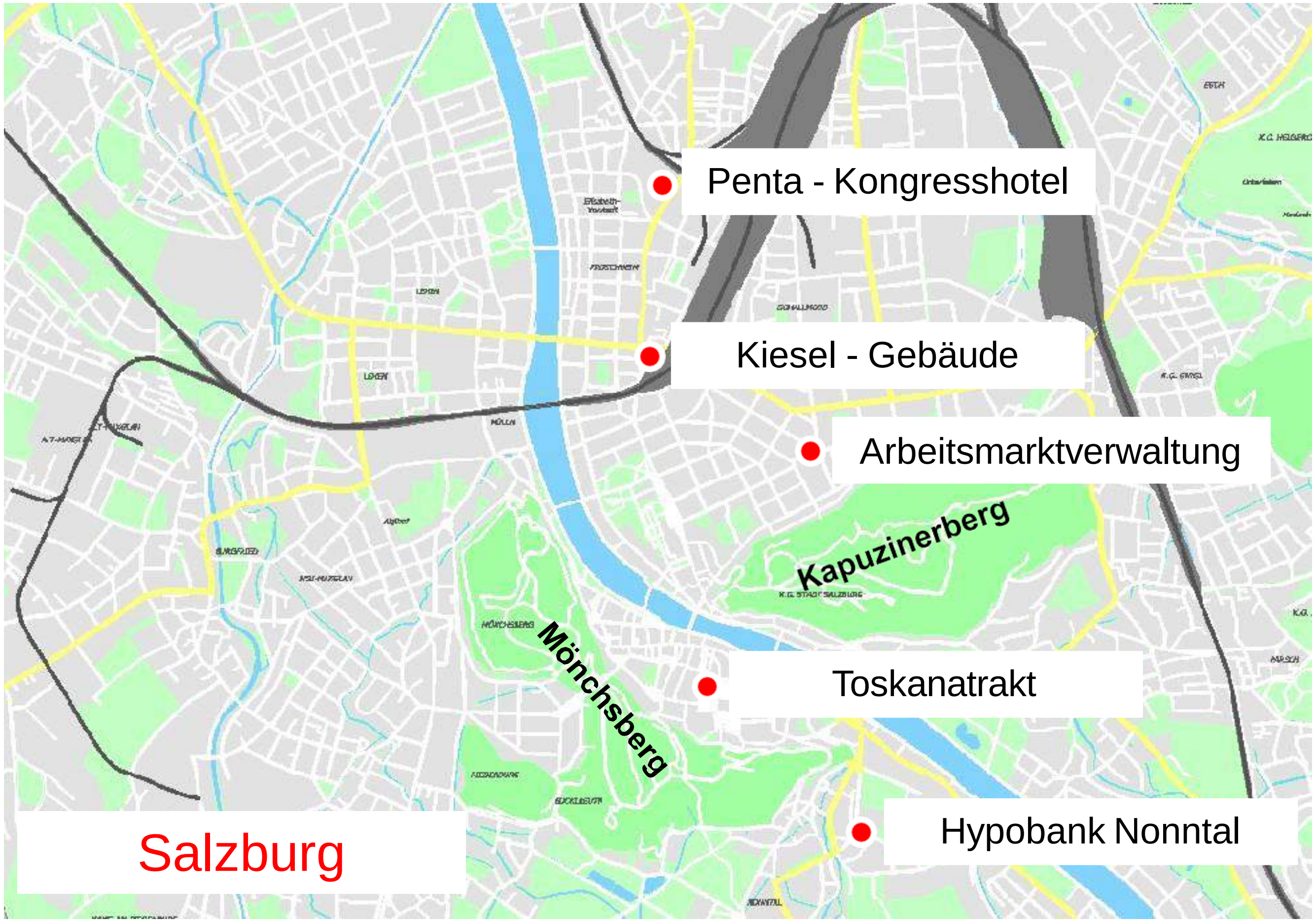
Layout of project "Kiesel"

BACK ANALYSIS OF DIFFERENT EXCAVATIONS



Layout of project "Penta"

BACK ANALYSIS OF DIFFERENT EXCAVATIONS



PROJECT KIESEL

Soil layer	Typ	γ [kN/m³]	γ_{SAT} [kN/m³]	$E_{50\ ref}$ [kN/m²]	$E_{ur\ ref}$ [kN/m²]	$E_{oed\ ref}$ [kN/m²]	φ' [°]	ψ [°]	c' [kN/m²]	ν_{ur} -	p_{ref} [kN/m²]	m -
Gravel	D	19.50	21.50	65.000	194 000	65 000	35	5	0	0.20	100	0.50
Silty sand	D	20.00	20.50	40.000	120 000	40 000	32	0	2	0.20	100	0.60
Fine sand („Seeton 1“)	UD	20.00	20.00	35.000	140 000	35 000	30	0	2	0.20	100	0.70
Coarse silt („Seeton 2“)	UD	20.00	20.00	25.000	100 000	20 000	28	0	10	0.20	100	0.70

Parameters HS / HS-small model

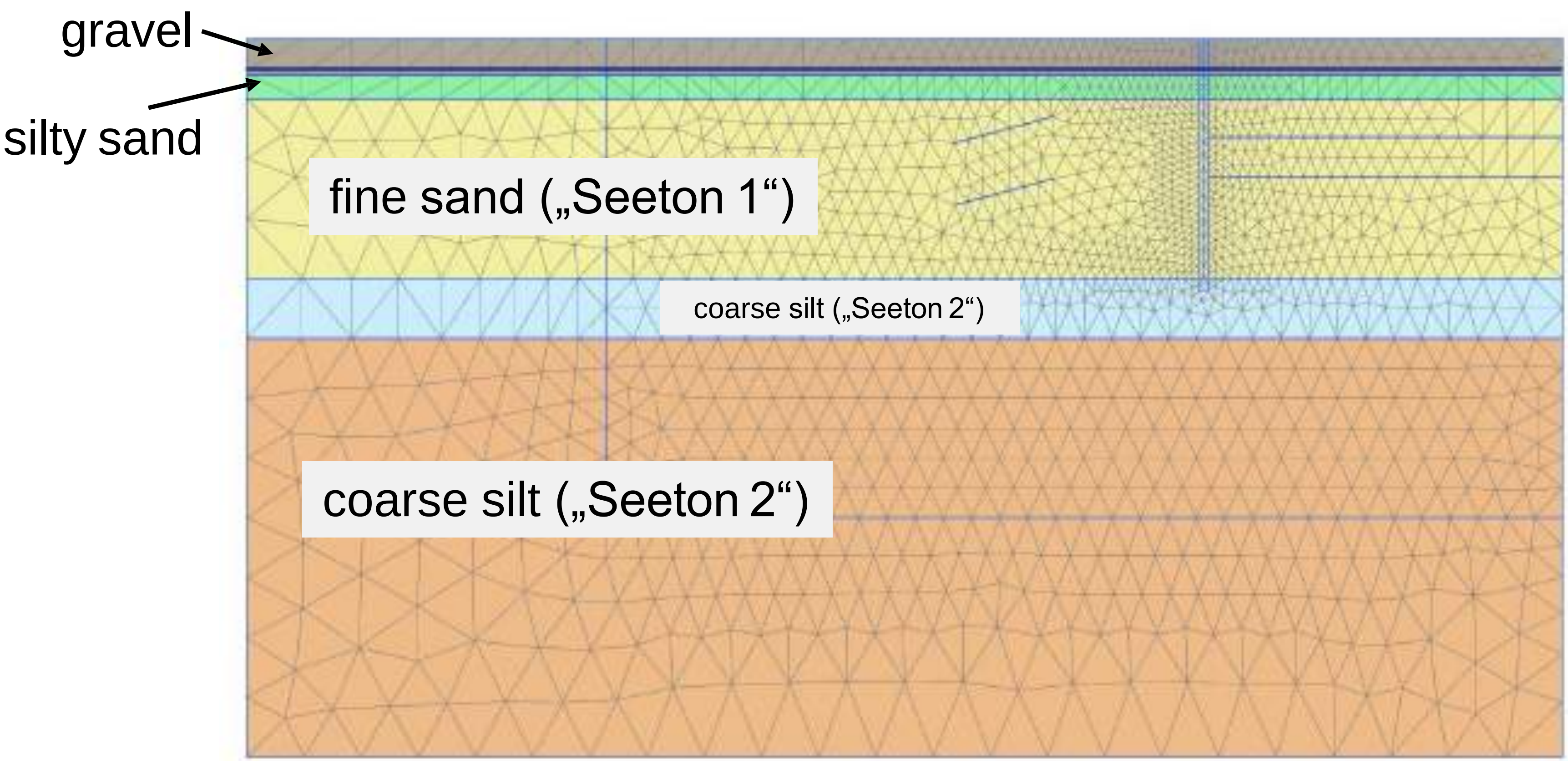
PROJECT KIESEL

Soil layer	K_{0nc}	$K_{0,\ manual}$	R_f	R_{inter}	k_x	k_y
	-	-	-	-	[m/d]	[m/d]
Gravel	0.426	0.55	0.90	0.67	8.64E+01	4.32E+01
Silty sand	0.470	0.55	0.90	0.67	8.64E-01	8.64E-01
Fine sand („Seeton 1")	0.500	0.55	0.90	0.67	4.32E-03	4.32E-04
Coarse silt („Seeton 2")	0.531	0.55	0.90	0.67	8.64E-04	8.64E-05

$\gamma_{0.7}$	G_o
[kN/m²]	[kN/m²]
1.50E-04	242 000
1.50E-04	150 000
2.00E-04	175 000
2.00E-04	125 000

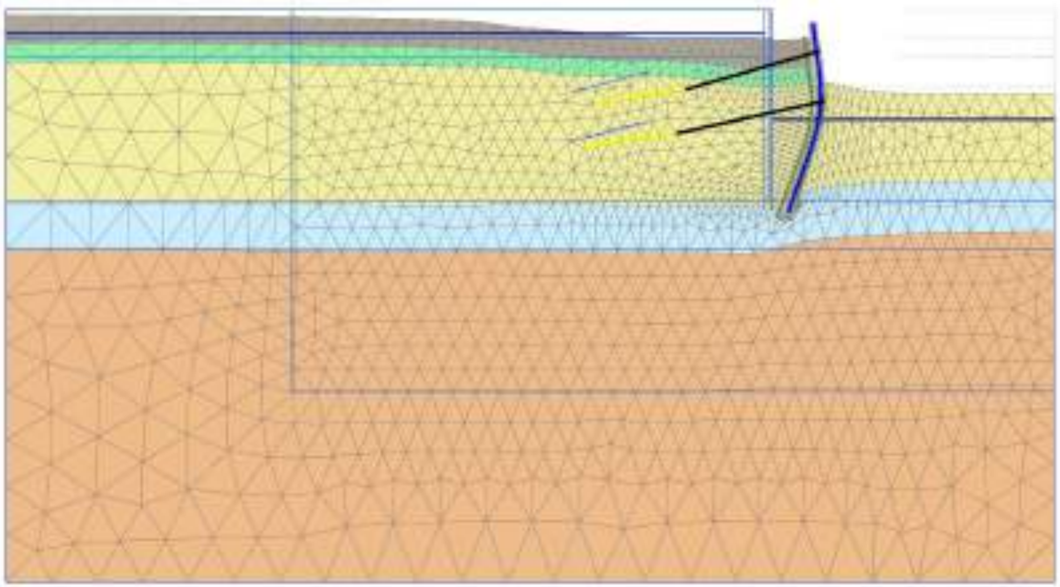
Parameters HS / HS-small model

PROJECT KIESEL

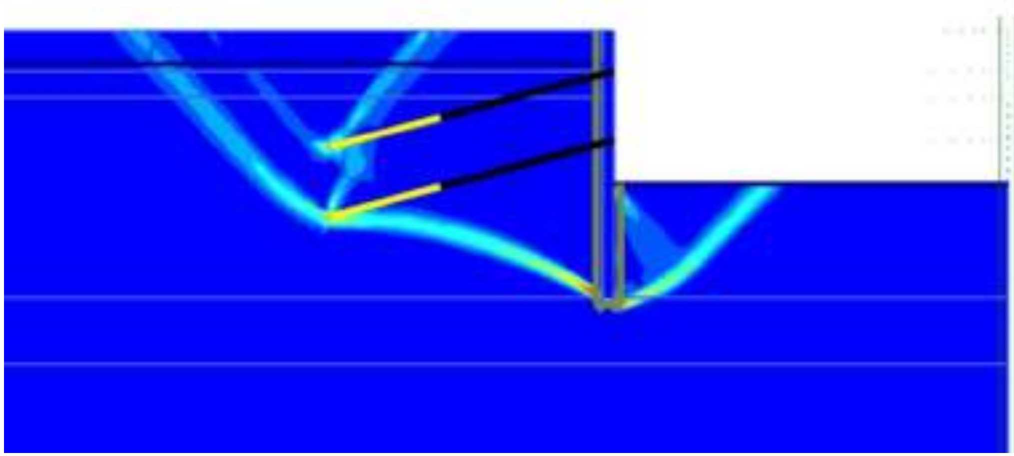


Soil layers and mesh for project „Kiesel“

PROJECT KIESEL



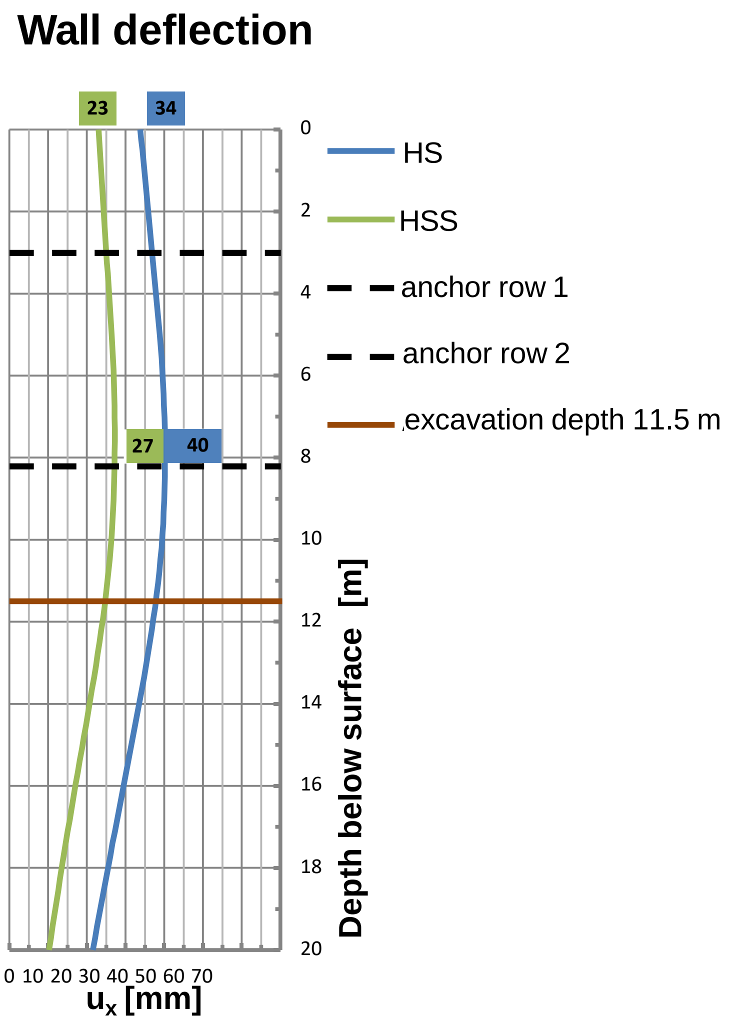
Deformed mesh - HSS model

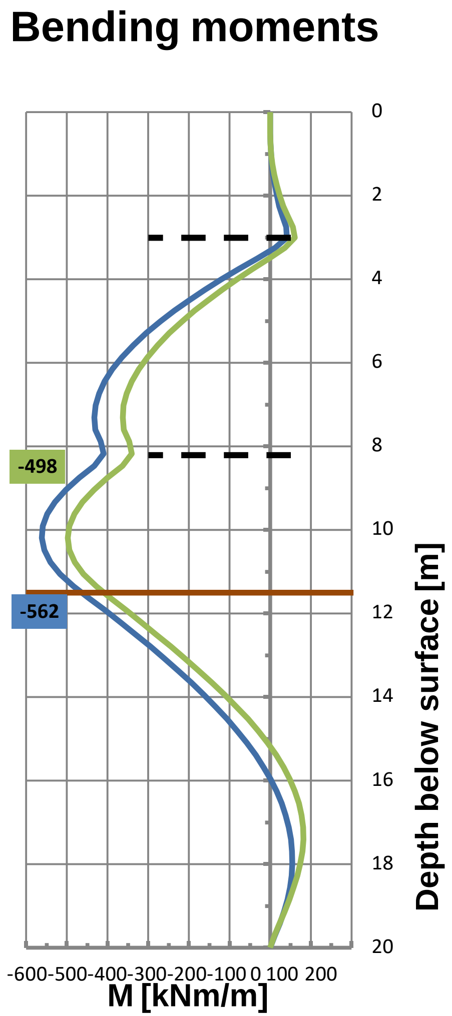


Failure mechanism - wall elastic

FoS ≈ 1.40

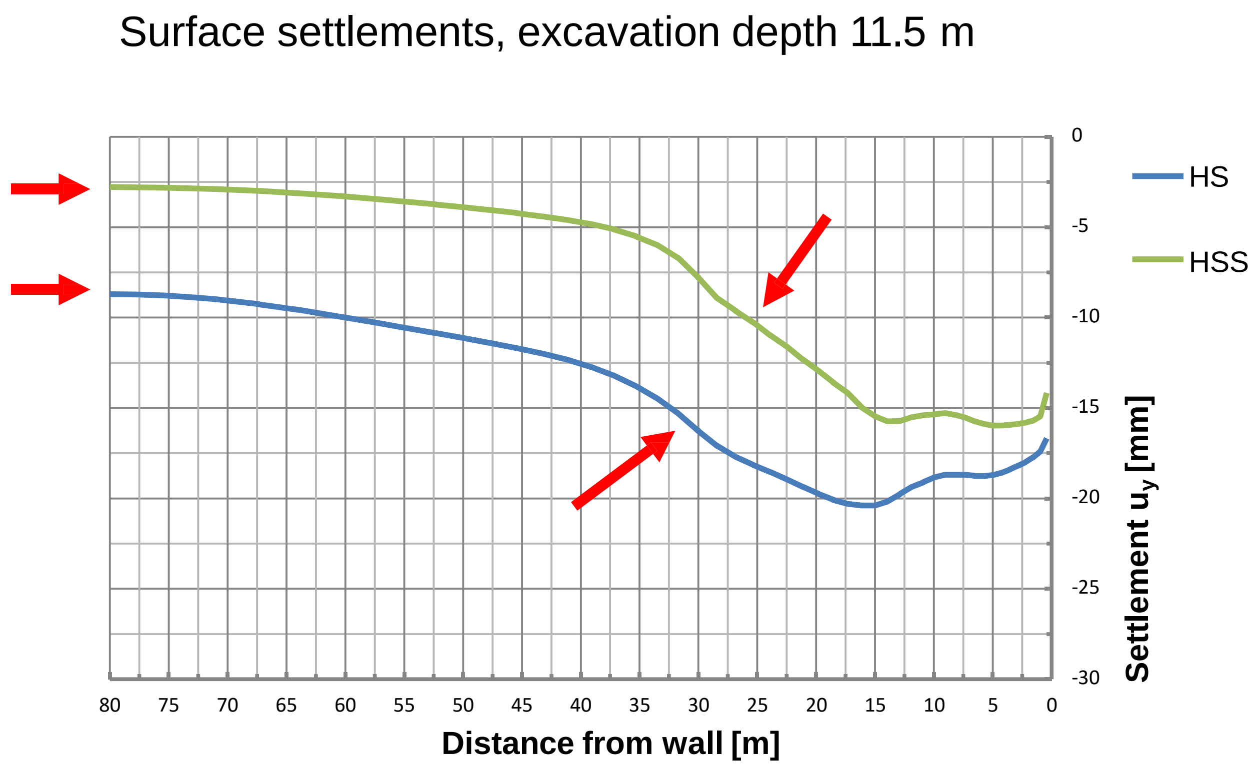
PROJECT KIESEL





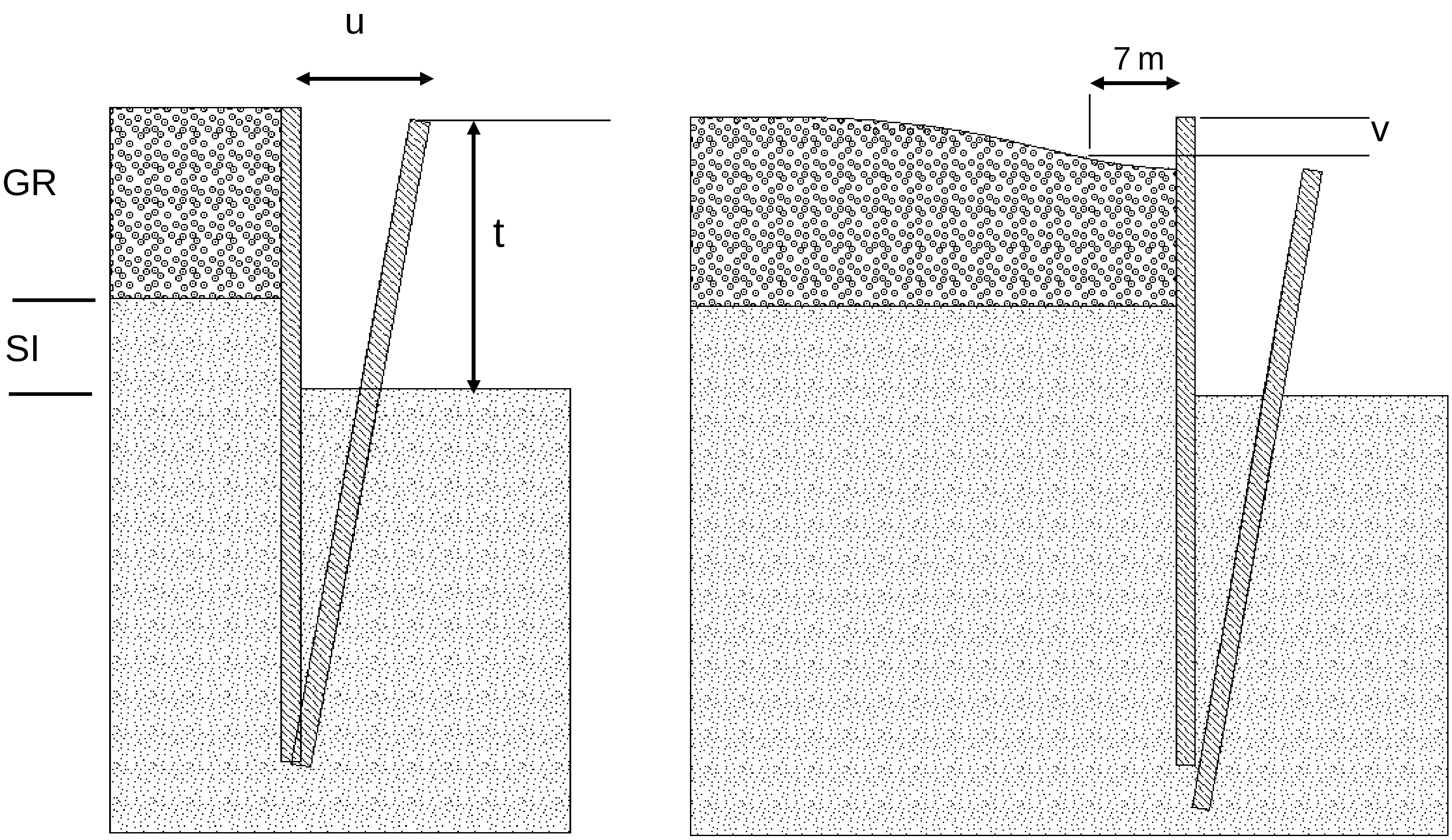
Kern, 2013

PROJECT KIESEL



Kern, 2013

BACK ANALYSIS OF DIFFERENT EXCAVATIONS

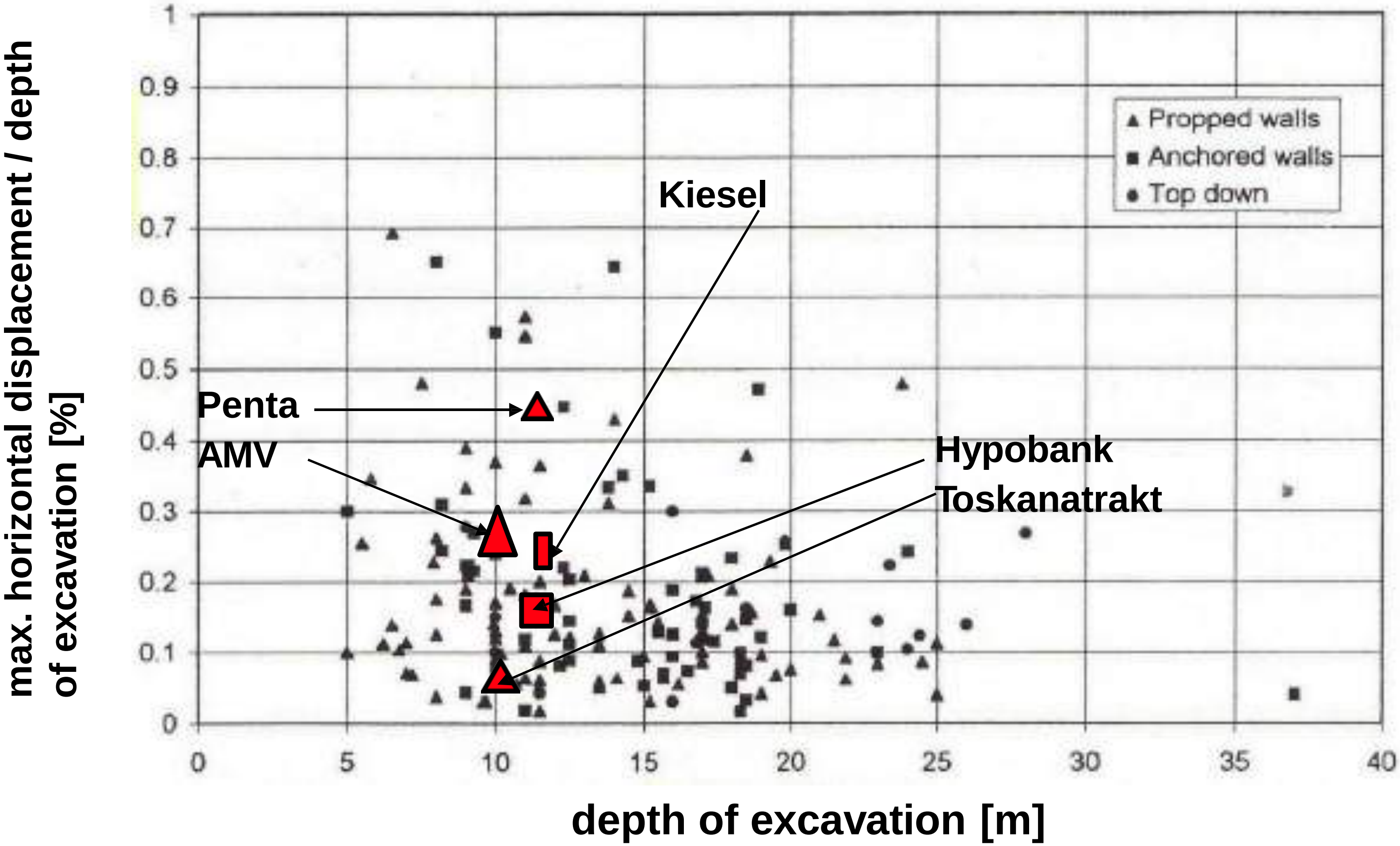


GR / SI ……. ratio of depth of gravel layer to clayey silt layer with reference to excavation level

BACK ANALYSIS OF DIFFERENT EXCAVATIONS

		size					
				measured	calculated	measured	calculated
	GR/SI	L * B	Vol	u		v	
Project	m/m	m²	m³	mm		mm	
Toskanatrakt	6.7	850	9800	5	0	8	0
Hypobank	4.5	1000	11000	13	13	11	7
AMV	0.1	1800	18000	17	20	26	15
Kiesel	0.4	2200	25000	20	23	28	16
Penta	0.5	6500	75000	100	100	30	64

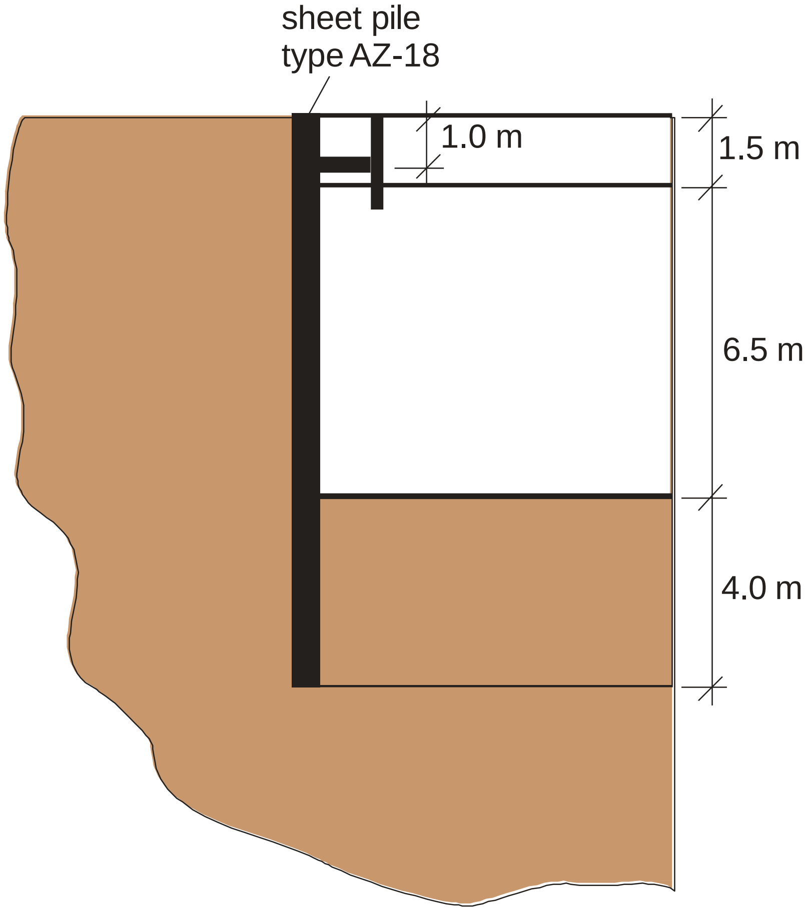
BACK ANALYSIS OF DIFFERENT EXCAVATIONS



Long, M. (2001). Database for retaining wall and ground movements due to deep excavation, Journal of Geotechnical and Geoenvironmental Engineering, 203-224.

Clough, G.W., Smith, E.M., Sweeney, B.P. (1989). Movement control of excavation support systems by iterative design. Proc. Foundation Engineering-Current Principles and Practices, Vol. 2, ASCE, New York, 869-884.

NOTE ON φ/c REDUCTION



Material parameters soil layer:

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Horizontal strut distance: 1 m

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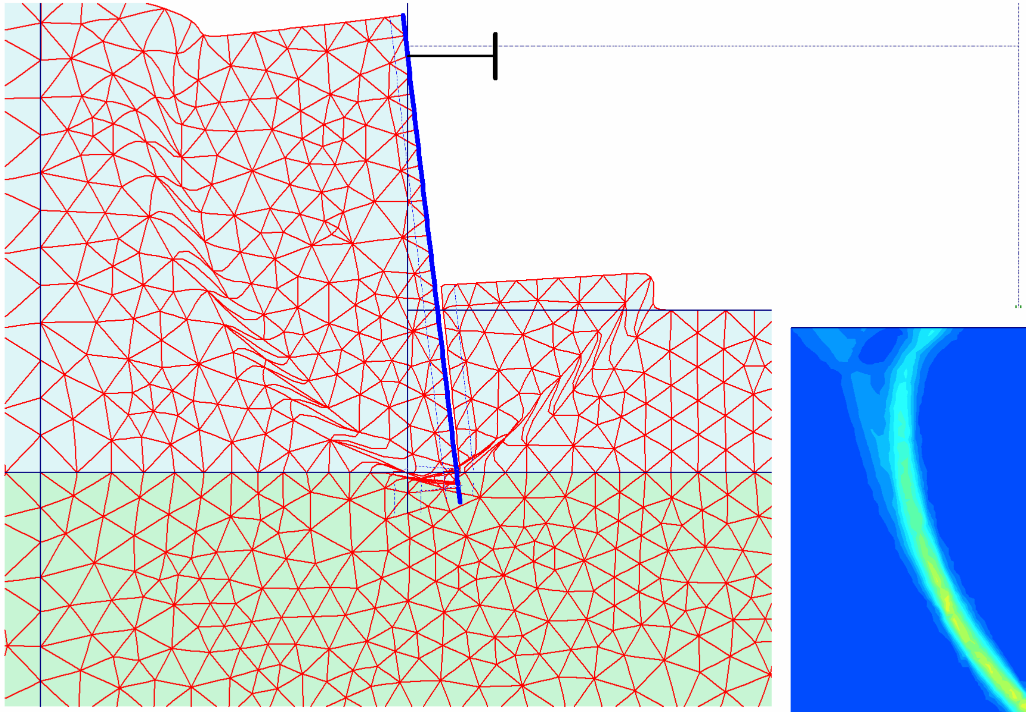
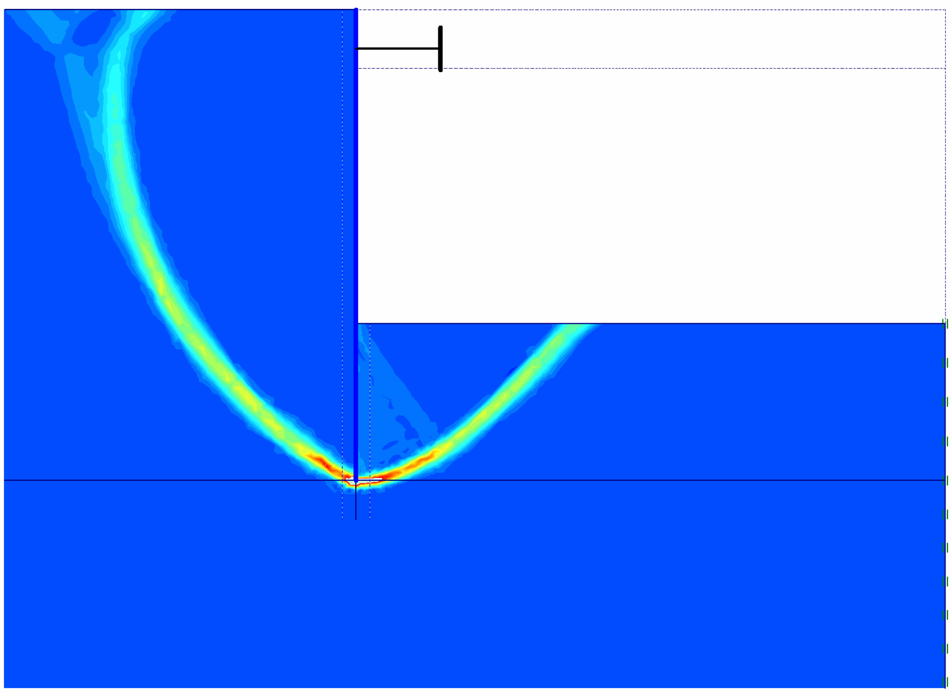


wall elastic

Msf = 1.95



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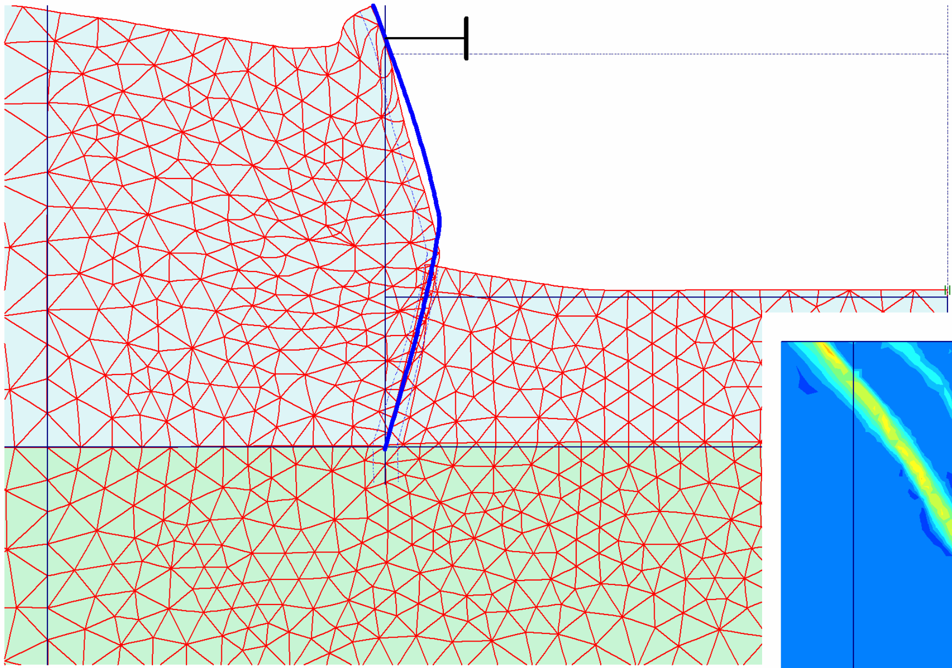
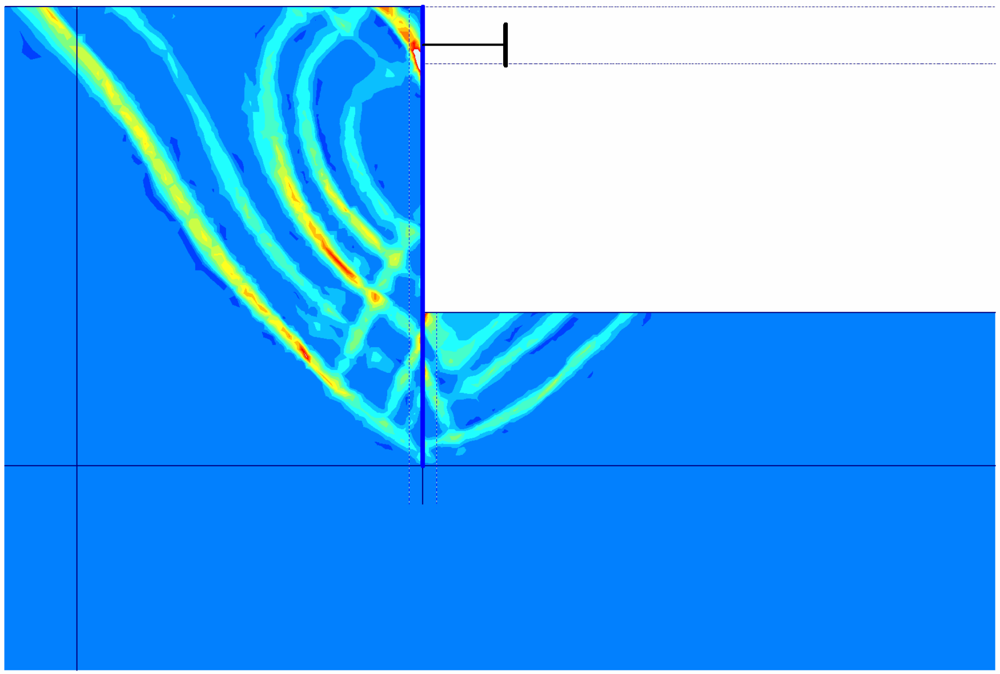
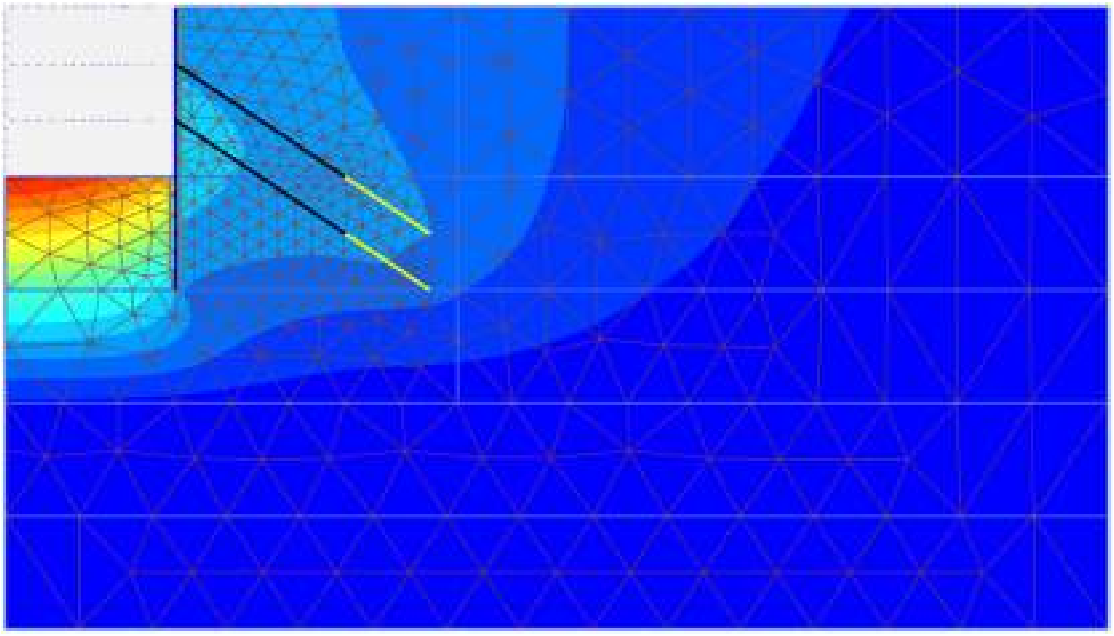




wall elastic - perfectly plastic
Msf = 1.73

EXCAVATION OF BUILDING PIT (MC-Model)



PLAXIS ADVANCED COURSE

INTRODUCTION

A building pit was constructed in the south of the Netherlands. The pit is 15 m deep and 30 m wide. A diaphragm wall is constructed using 60 cm diameter bored piles; the wall is anchored by two rows of pre-stressed ground anchors. In this exercise the construction of this building pit is simulated and the deformation and bending moments of the wall are evaluated.
The upper 40 m of the subsoil consists of a more or less homogeneous layer of medium dense fine sand with a unit weight of 18 kN/m³. Triaxial test data of a representative soil sample is given in Figure 2. Underneath this layer there is very stiff layer of gravel, which is not to be included in the model. The groundwater table is very deep and does not play a role in this analysis.

AIMS

- Determination of soil stiffness parameters
- Using interface elements
- Using ground anchors
- Pre-stressing of anchors
- Combination of structural elements

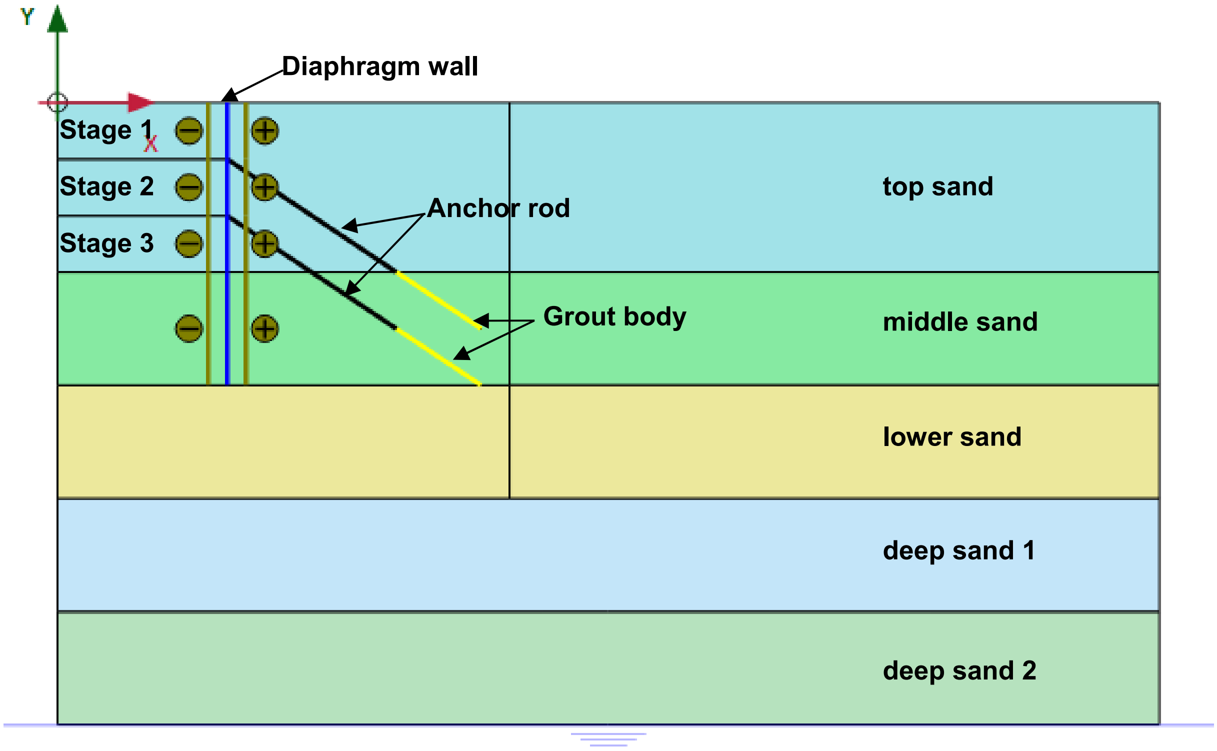


Figure 1: Geometry for tied-back excavation.

DETERMINATION OF STIFFNESS & STRENGTH PROPERTIES (SAND)

Use the Mohr-Coulomb model and extract model parameters for the sand layer from the triaxial test data. Concerning stiffness behaviour, take into account the fact that the excavation involves **unloading rather than primary loading**. It is suggested to divide the sub-soil into three regions: Top Sand (0 m → −15 m), Middle Sand (−15 m → −25 m) and Lower Sand (−25 m → −35 m). This allows for the input of an average stiffness in each individual region. As the simple Mohr-Coulomb model cannot take into account the stress-dependency of the stiffness, the input of an average stiffness per region, by the user, is the next best option to enhance the model. Please note there are other models (HS and SS model) available in PLAXIS that take into account the stress-dependency of the soil stiffness. These advanced models will be introduced in a later exercise. The soil parameters can be entered into Table 1.

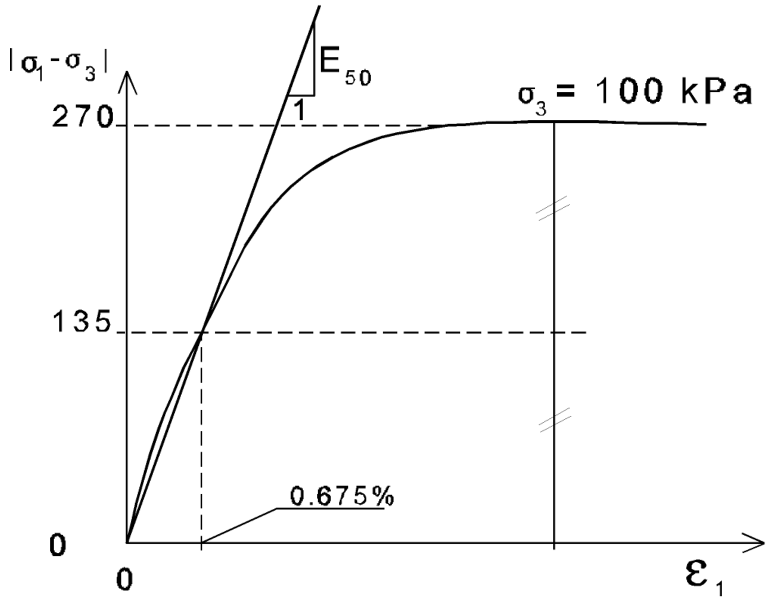


Figure 2: Mohr-Coulomb model for the sand layer.

Table 1: Soil material set parameters.

Parameter	Symbol	Top sand	Middle sand	Lower sand	Deep sand 1	Deep sand 2	Unit
Material model	Model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	-
Type of behaviour	Type	Drained	Drained	Drained	Drained	Drained	-
Dry weight	γ_{unsat}	18.0	18.0	18.0	18.0	18.0	kN/m^3
Wet weight	γ_{sat}	18.0	18.0	18.0	18.0	18.0	kN/m^3
Permeability x-dir.	k_x	0.0	0.0	0.0	0.0	0.0	m/d
Permeability y-dir.	k_y	0.0	0.0	0.0	0.0	0.0	m/d
Young's modulus*	E	?	?	?	280.000	470.000	kN/m^2
Poisson's ratio	ν	0,33	0,33	0,33	0,33	0,33	-
Cohesion	c	?	?	?	1,0	1,0	kN/m^2
Friction angle	φ	?	?	?	35	35	°
Dilatancy angle	ψ	?	?	?	5	5	°
Interface strength reduction	R_{inter}	0.6	0.6	0,6	Rigid (1.0)	Rigid (1.0)	-

GEOMETRY AND MATERIAL PROPERTIES INPUT

Start a new project

Enter general settings

- Accept the default values in the Project tab sheet of the *General settings (15-node elements)*. For the dimensions see Figure 3.

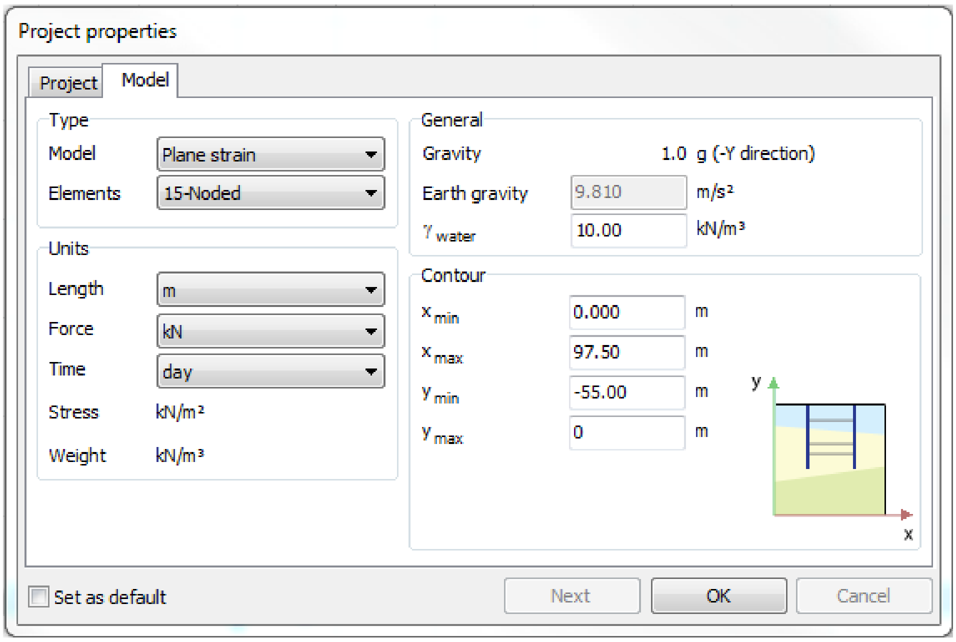
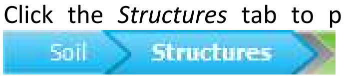


Figure 3: General settings, tab sheet dimensions.

Enter GEOMETRY, PLATE, interfaces, anchors, and geotextiles

- Start by using the borehole tool  to define the virgin soil conditions.
- Enter the material properties for the three soil data sets, as determined in the first table of this exercise.
- After entering all properties for the three soil types, drag and drop the properties to the graph of the soil column on the left hand side.
- Set the *Head* in the borehole to -55 m.
- Click the *Structures* tab to proceed with the input of structural elements in the Structures mode. 

- Enter the geometry as proposed in Figure 4

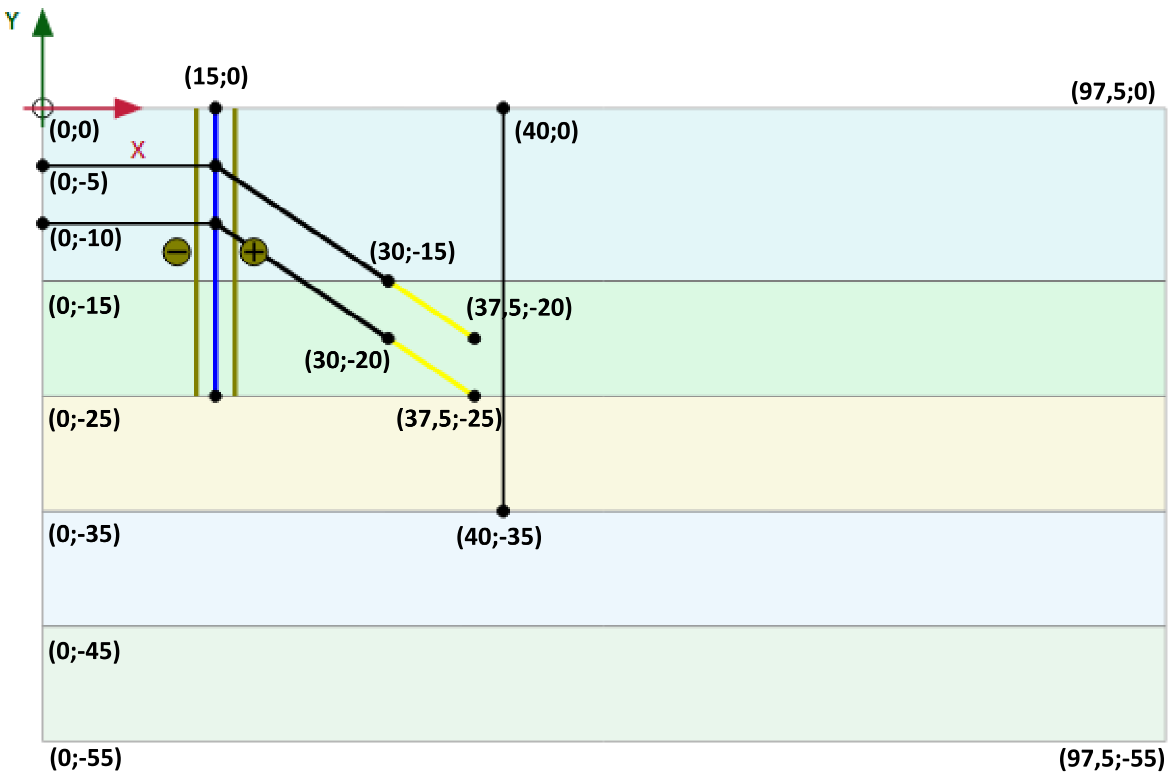
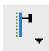
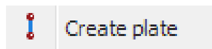
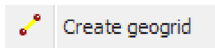


Figure 4: Geometry of the model

- Click the *Create structure* button  in the side toolbar and select the *Create Plate* 

option to introduce the diaphragm wall.
- Click the *Create structure* button again and select the *Create geogrid* 

option to introduce the two geotextile elements that represent the grout body.
- Right-click the plate representing the diaphragm wall. Point to *Create* and click on the *Positive interface* option in the appearing menu (Figure 5). In the same way assign a negative interface as well.

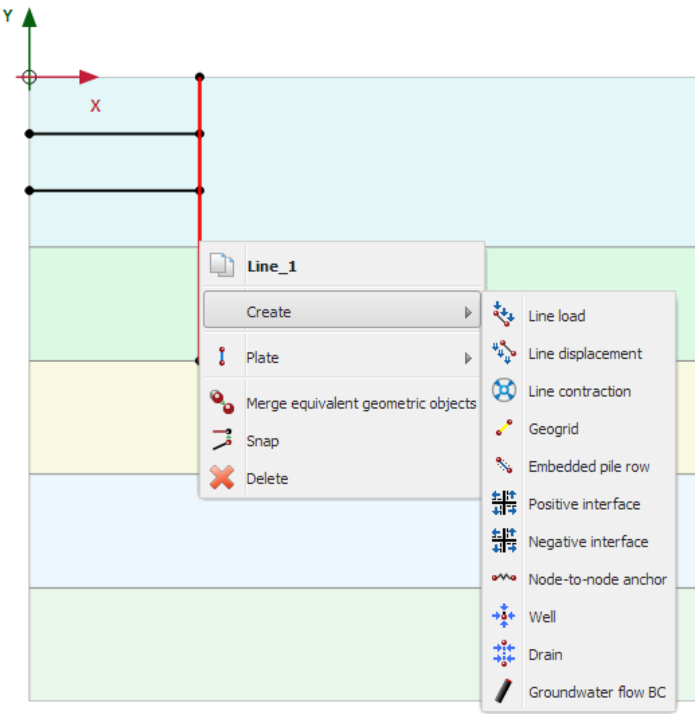


Figure 5: Interface assignment to existing geometry

- Click the *Create structure* button again and select the *create node to node anchor* Create node-to-node anchor option to introduce the two anchors. These anchors connect the beginning of the grout-body to the wall.
- Enter material properties for the plates, anchors and `geogrids` as indicated in Table 2, Table 3 and Table 4. After entering all properties, assign the appropriate properties to the structures.

Table 2: Properties of the diaphragm wall (plate)

Parameter	Symbol	Diaphragm wall	Unit
Material Model	Model	Linear Elastic	-
Normal stiffness	EA	$8.0*10^7$	kN/m
Flexural rigidity	EI	$1.5*10^6$	kNm^2/m
Weight	w	8.0	kN/m/m
Poisson's ratio	ν	0.0	-

Table 3: Properties of the anchor rod (node-to-node anchors).

Parameter	Symbol	Anchor rod	Unit
Material Model	Model	Elastic	-
Normal stiffness	EA	$2.0*10^5$	kN/m
Spacing	L_s	1.0	m

Table 4: Property of the grout body (geogrid).

Parameter	Symbol	Grout body	Unit
Normal stiffness	EA	$2.0*10^5$	kN/m

MESH GENERATION

- Proceed to the *Mesh* mode. Generate the mesh and use the default option for the Element distribution (medium).
- View the mesh. The resulting mesh is shown in Figure 6.

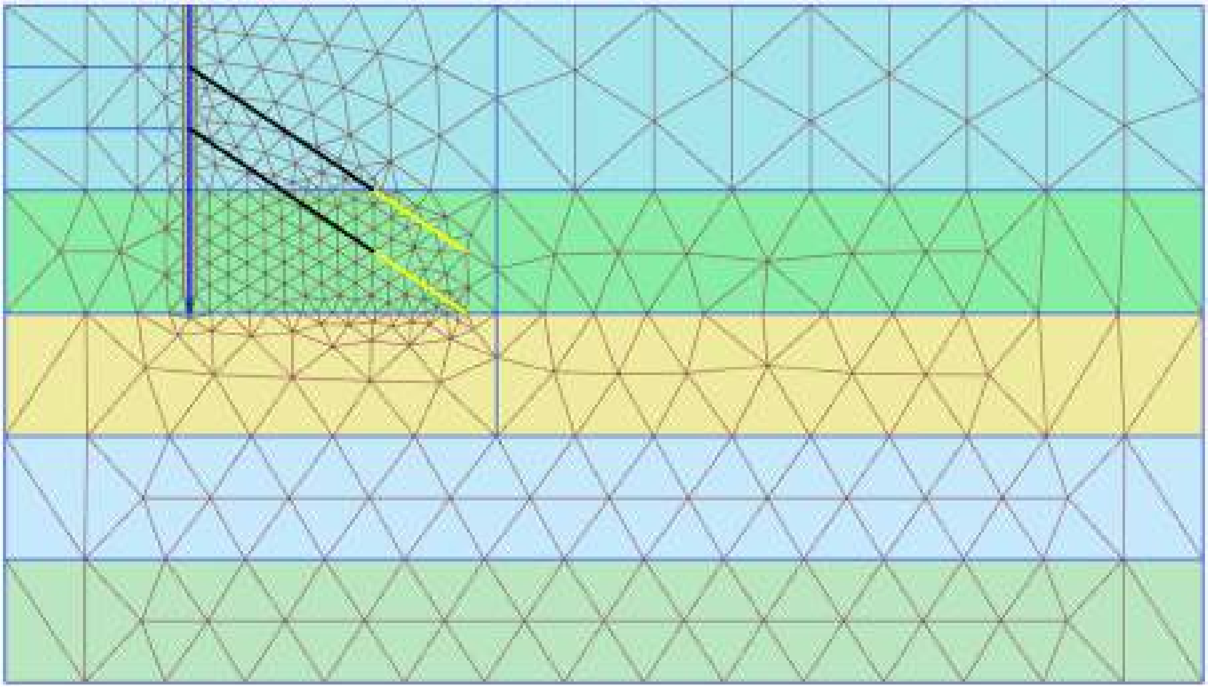
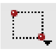


Figure 6: Medium Finite Element mesh

- Select the *Clusters*  around the diaphragm wall and press *Refine Clusters*  button. Generate the mesh again. This will result in a refinement around the selected *Clusters* as shown in Figure 7.

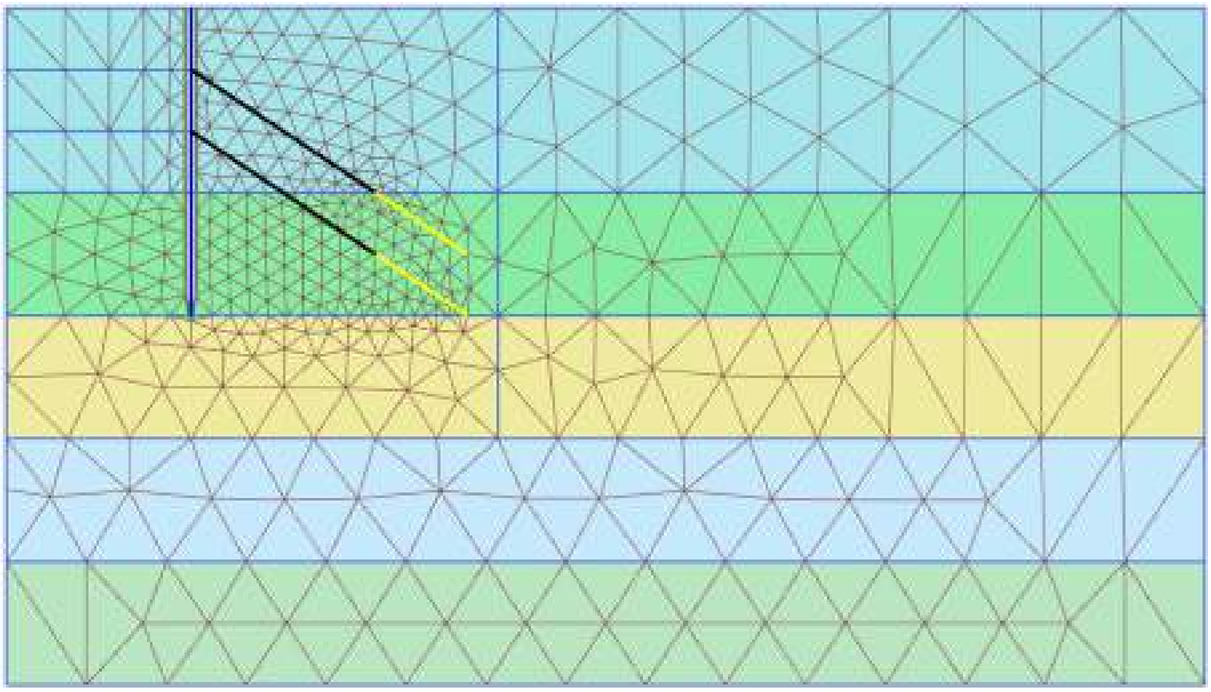


Figure 7: Refined Finite Element mesh

WATER CONDITIONS

- As the phreatic surface is located below the deep send 2 layer, the generation of pore pressures can be skipped.

CALCULATION

- Click on the *Staged construction* tab to proceed with the definition of the calculation phases.
- In the Phases explorer the *Initial phase* is already introduced. Double click the *Initial phase* to check if the calculation type is *KO procedure*.

  Make sure all the soil volumes are active and all the structural elements are inactive.

Construction process

The entire construction process consists of five phases. Define the phases, as shown graphically below. For each phase, use *plastic* for the *calculation type* and *Staged construction* for the *loading type*.

- In the first phase (Figure 10) the diaphragm wall is activated and the first excavation takes place. Note that though the interfaces along the wall are activated automatically with the activation of the wall.
- In the second phase (Figure 11), a new option is used, namely the pre-stressing of anchors.
- First the grout-body (the geogrid) is switched on by right clicking on the 'geogrid' element and activating it. The element will appear in yellow as soon as it is switched on. The light grey color indicates non-active elements.
- Now the grout-body is active and the anchor element needs to be pre-stressed. In the Selection explorer set the Adjust prestress parameter to true and assign a pre-stress force of 300kN (Figure 8). The letter P indicates that a pre-stress force will be active in the anchor.

Create the remaining phases, as indicated in Figure 12, Figure 13 and Figure 14. Select some nodes for the load displacement curves (e.g. top of wall).

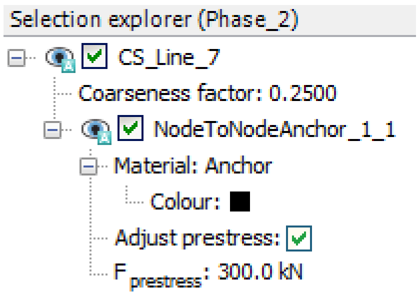


Figure 8: Selection explorer

Hint: A pre-stress force is exactly matched at the end of a finished staged construction calculation and turned into an anchor force. In successive calculation phases the force is considered to be just an anchor force and can therefore further increase or decrease, depending on the development of the surrounding stresses and forces.

- Start the Calculation. 

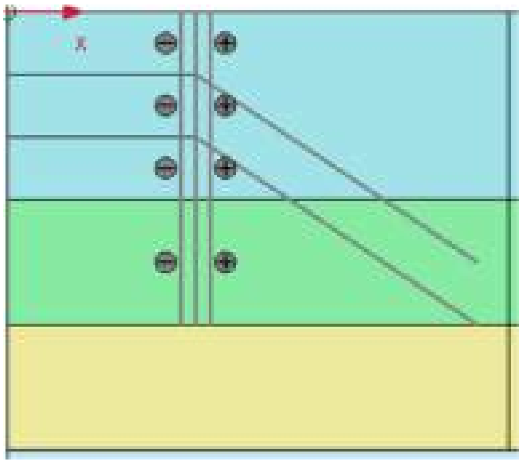


Figure 9: Initial conditions, diaphragm wall and anchors are not active.

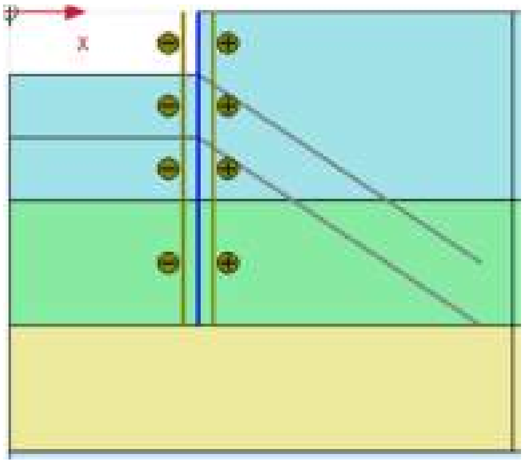


Figure 10: Phase 1, excavation and activation of the diaphragm wall.

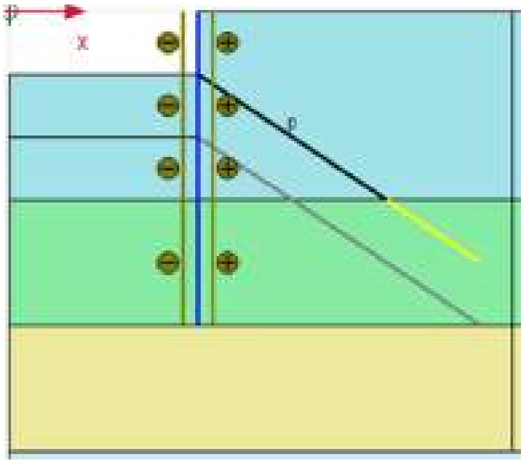


Figure 11: Phase 2, activation of the anchor and geotextile (grouting). Pre-stressing of the anchor to a value of 300 kN/m.

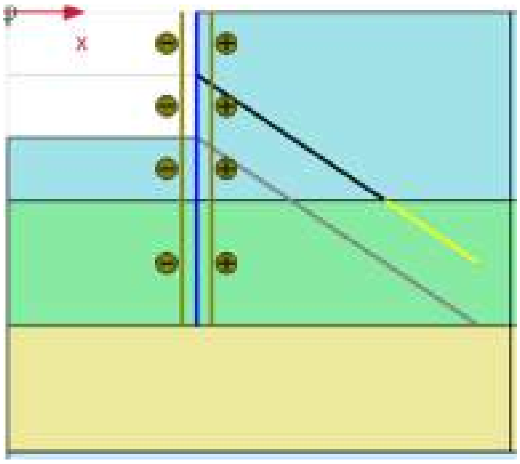


Figure 12: Phase 3, excavation of the second part.

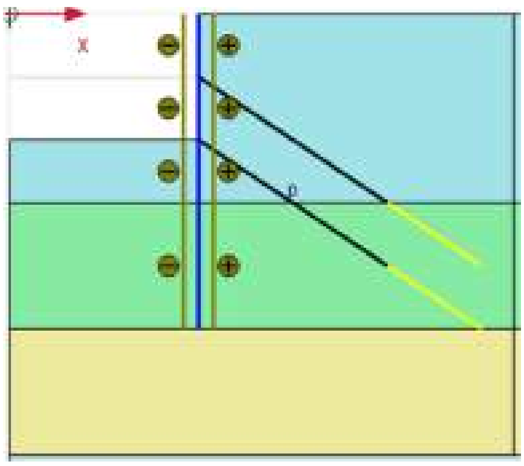


Figure 13: Phase 4, activation of the second anchor and geotextile (grouting). Pre-stressing of the anchor to a value of 300 kN/m.

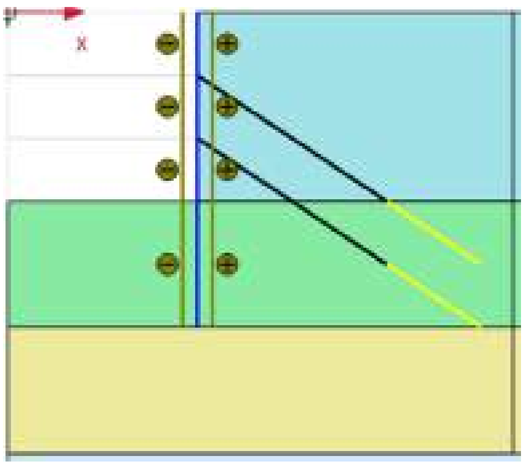


Figure 14: Phase 5, excavation of the third part

INSPECT OUTPUT

By double clicking on the node-to-node anchors, Plaxis will present a box, in which the stress in the anchor may be inspected.

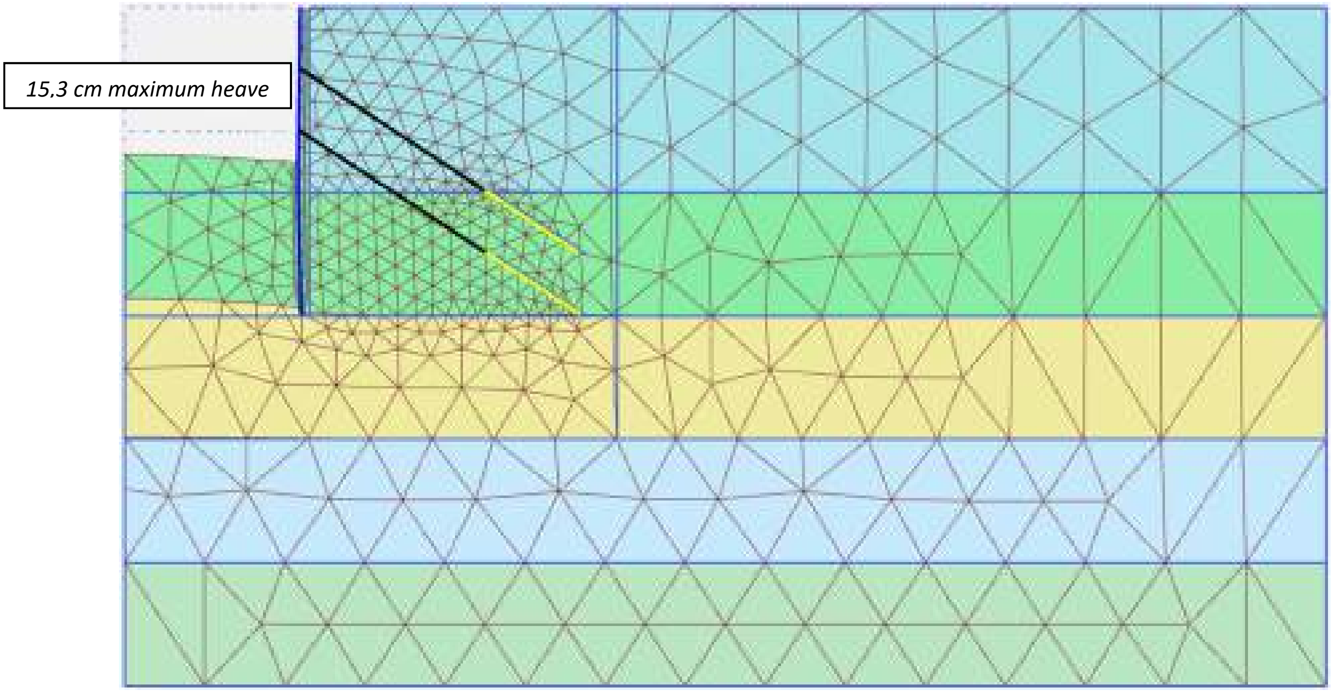


Figure 15: Deformed mesh (MC-model with E_{50} - modulus)

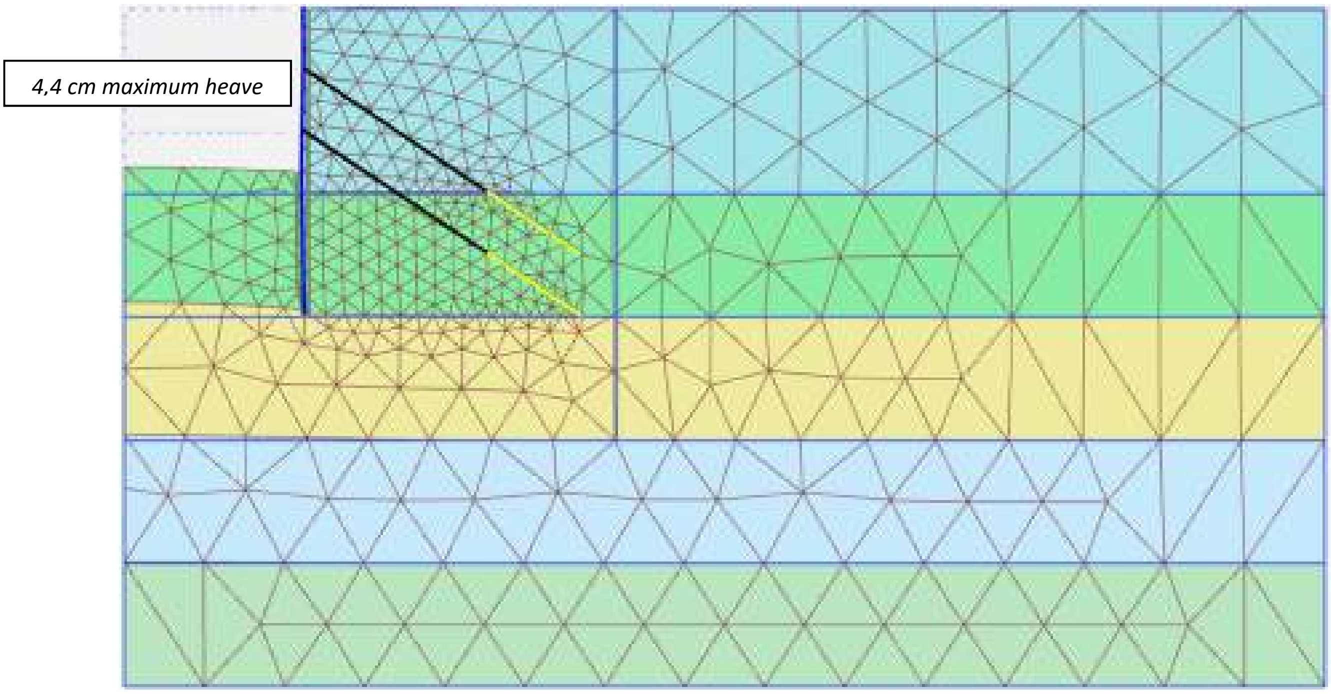


Figure 16: Deformed mesh (MC-model with E_{ur} - modulus)

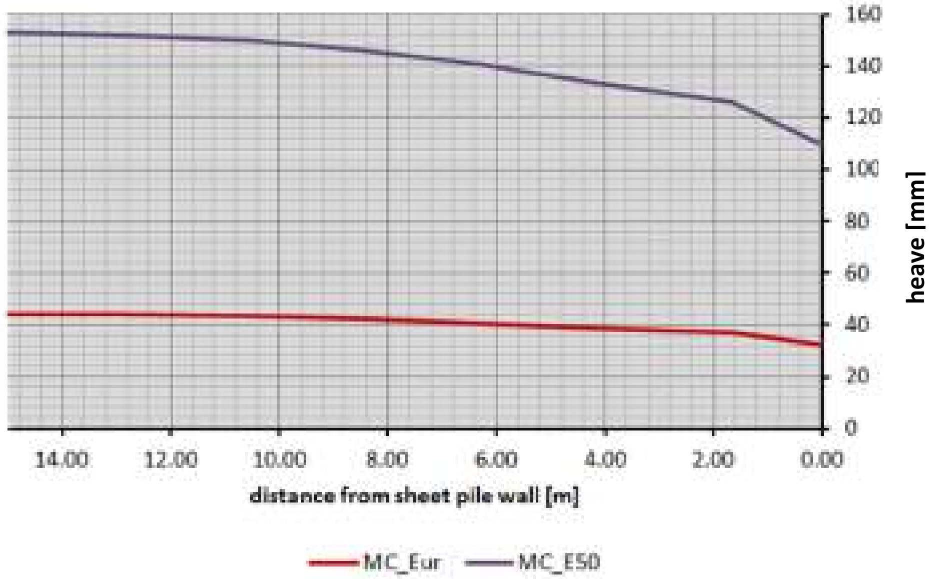


Figure 15: Sole heave after the last excavation step

Figure 16: Bending moments after the last excavation step

DETERMINATION OF PARAMETERS FROM TRIAXIAL TEST

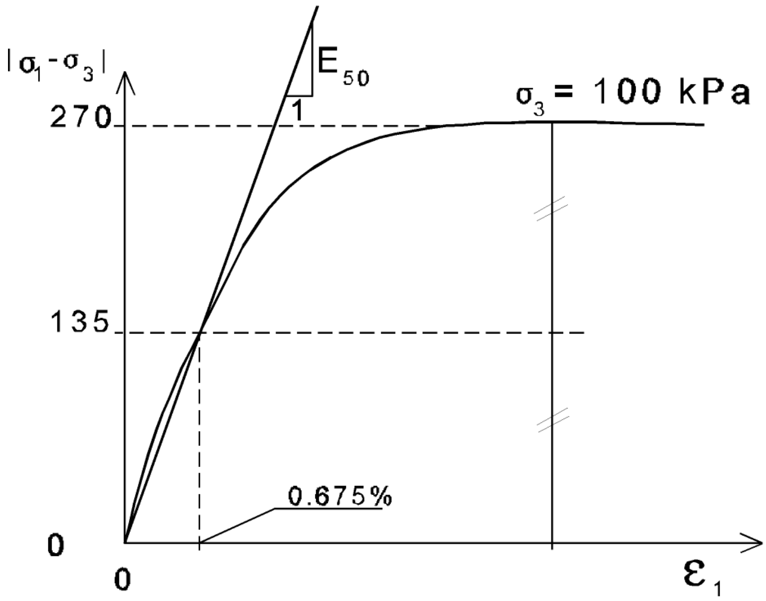


Figure 1: Mohr-Coulomb model for the sand layer.

STRENGTH PARAMETERS

Fill in σ_1 and σ_3 in the Mohr-Coulomb criteria:

$$\sigma_1 - \sigma_3 = (\sigma_1 + \sigma_3)\sin\varphi + 2c\cos\varphi$$

Since c will be small, assume c = 0:

$$\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \sin\varphi$$

$$\frac{370 - 100}{370 + 100} = \sin\varphi$$

$\varphi = 35\ °$ For reasons of numerical stability use c = 1 kPa

$\psi = \varphi - 30$ $\psi = 5$

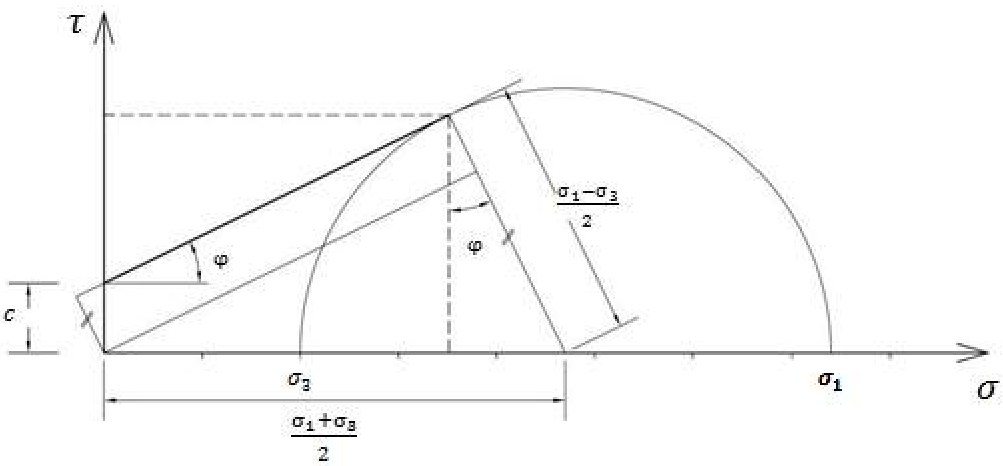


Figure 2: Strength parameter of the Mohr-Coulomb model.

STIFFNESS PARAMETERS

Since excavation is considered in this exercise, the input of Young's modulus E should be based on unloading, rather than on primary loading.

σ_3 = 100 kPa (corresponds with reference pressure, $E_{50} = E_{50}^{ref}$)

$E_{50} = \frac{\Delta\sigma_v}{\Delta\varepsilon_v} = \frac{135}{0{,}675\%} = 2{,}0*10^4 kN/m^2$ assume $E_{ur} \approx 4\ E_{50}$

For improved modelling, three regions are distinguished, for each individual layer an average stiffness is determined by using:

$E_{50} = E_{50}^{ref}\sqrt{\frac{\sigma_3}{p^{ref}}}$ $E_{50} \approx E_{load} \rightarrow E_{ur} \approx E_{unload}$

We distinguish:
Top layer (from top to - 15 m)
Middle layer (from -15 m to -25 m)
Lower layer (from -25 m to -35 m)

Top layer:
σ_3 = depth $\cdot\ \gamma_{soil} \cdot K_0$
σ_3 = average horizontal stress (reference point at - 7.5 m)
$\sigma_3 = 7.5 \cdot 18 \cdot (1-\sin\varphi) =$ **58 kPa**

$E_{50}^{top} = 2{,}0 \cdot 10^4 \sqrt{\frac{58}{100}} \sim 1{,}5 \cdot 10^4$ kPa

$E_{ur}^{top} \approx 4 \cdot E_{50}^{top} = 6.0 \cdot 10^4$ kPa

Middle layer:
$\sigma_3 = 20 \cdot 18 \cdot (1-\sin\varphi) =$ **153 kPa**

$E_{50}^{middle} = 2{,}0 \cdot 10^4 \sqrt{\frac{153}{100}} \sim 2{,}5 \cdot 10^4$ kPa

$E_{ur}^{middle} \approx 4 \cdot E_{50}^{middle} = 1.0 \cdot 10^5$ kPa

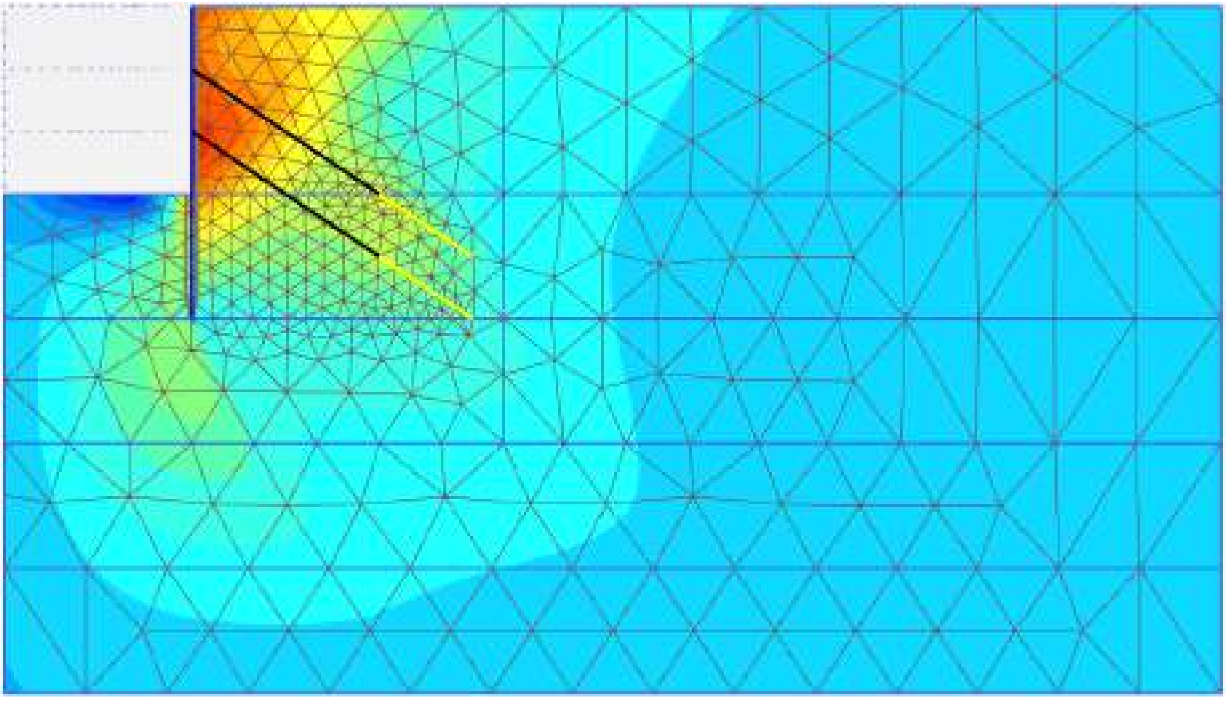
Lower layer:
$\sigma_3 = 30 \cdot 18 \cdot (1-\sin\varphi) =$ **230 kPa**

$E_{50}^{bottom} = 2{,}0 \cdot 10^4 \sqrt{\frac{230}{100}} \sim 3{,}0 \cdot 10^4$ kPa

$E_{ur}^{bottom} \approx 4 \cdot E_{50}^{bottom} = 1.2 \cdot 10^5$ kPa

Assume for all layers $\nu = 0.33$.

EXCAVATION OF BUILDING PIT (HS-Model and HSS-Model)



PLAXIS ADVANCED COURSE

INTRODUCTION

In the next exercise the geometry from 'Excavation of building pit (MC-Model) ` is used. Instead of the Mohr-Coulomb model, the Hardening Soil and the Hardening Soil Small model is used.
A building pit was constructed in the south of the Netherlands. The pit is 15 m deep and 30 m wide. A diaphragm wall is constructed using 60 cm diameter bored piles; the wall is anchored by two rows of pre-stressed ground anchors. In this exercise the construction of this building pit is simulated and the deformation and bending moments of the wall are evaluated.
The upper 40 m of the subsoil consists of a more or less homogeneous layer of medium dense fine sand with a unit weight of 18 kN/m^3. Triaxial test data of a representative soil sample is given in the exercise 'Excavation of building pit in (MC-Model) `. The groundwater table is very deep and does not play a role in this analysis.

GEOMETRY INPUT

- Select the existing project of the 'Excavation of building pit (MC-Model) ´ exercise.
- From the File menu select Save As and save the existing project under a new file name.

GEOMETRY INPUT – HS MODEL

- Modify the material properties by selecting the item Soils & Interfaces from the Materials menu. Use the material properties as given in Table 1. Assign the Sand material set to all soil clusters.

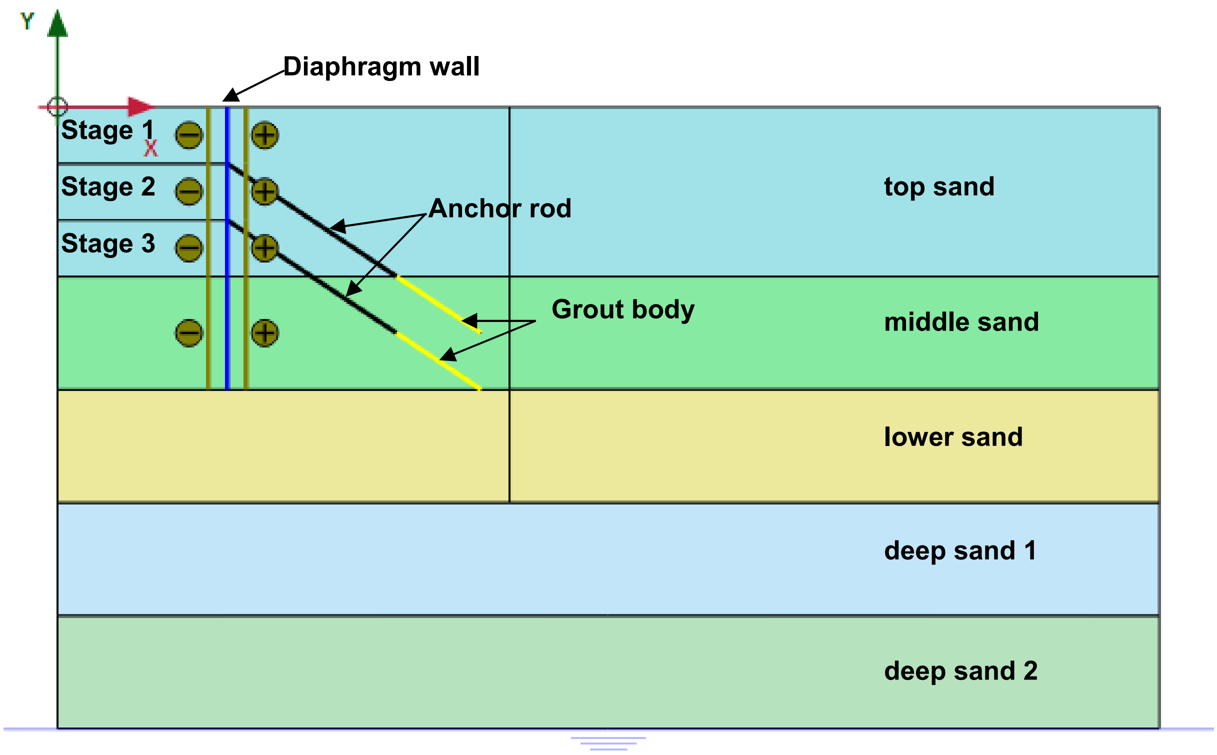


Figure 1: Geometry for tied-back excavation.

Table 1: Soil material set parameters for HS-model.

Parameter	Symbol	Top/Middle/ Lower sand	Deep sand 1	Deep sand 2	Unit
Material model	Model	HS-model	HS-model	HS-model	-
Type of behaviour	Type	drained	drained	drained	-
Dry weight	γ_{unsat}	18	18	18	kN/m^3
Wet weight	γ_{sat}	18	18	18	kN/m^3
Young's modulus	E_{50}^{ref}	20.000	40.000	60.000	kN/m^2
Oedometer mod.	E_{oed}^{ref}	20.000	40.000	60.000	kN/m^2
Unloading modulus	E_{ur}^{ref}	80.000	160.000	240.000	kN/m^2
Power	m	0,5	0,5	0,5	-
Poisson's ratio	ν	0,2	0,2	0,2	-
Reference stress	P_{ref}	100	100	100	kN/m^2
Cohesion	c	1,0	1,0	1,0	kN/m^2
Friction angle	φ	35	35	35	°
Dilatancy angle	ψ	5,0	5,0	5,0	°
Interface strength reduction	R_{inter}	0,6	1,0	1,0	-
K_0 ^{nc}	K_0=1-sinƒ	0,426	0,426	0,426	-

CALCULATION

Construction process

The entire construction process consists of five phases. Define the phases, as shown graphically below. For each phase, use the *Plastic calculation*, *Staged construction*.
Create the remaining phases, as indicated in Figure 2, Figure 3, Figure 4, Figure 5, Figure 6 and Figure 7. Select some nodes for the load displacement curves (e.g. top of wall).

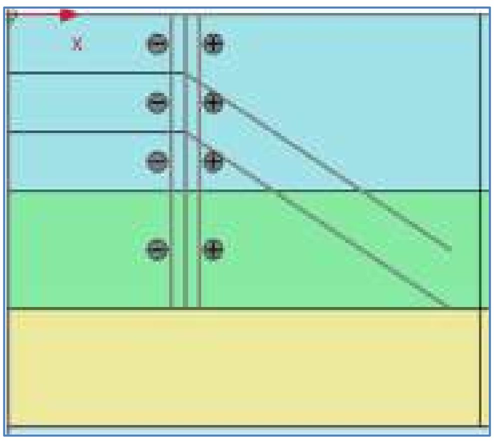


Figure 2: Initial conditions, diaphragm wall and anchors are not active.

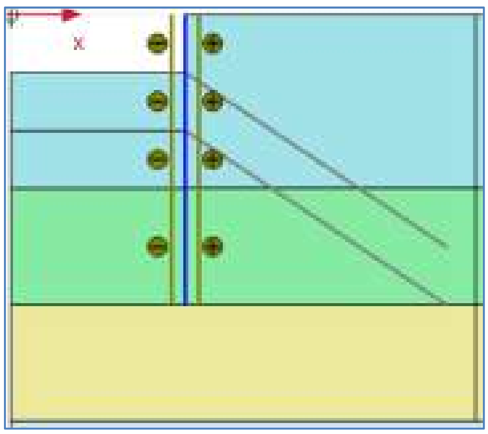


Figure 3: Phase 1, excavation and activation of the diaphragm wall.

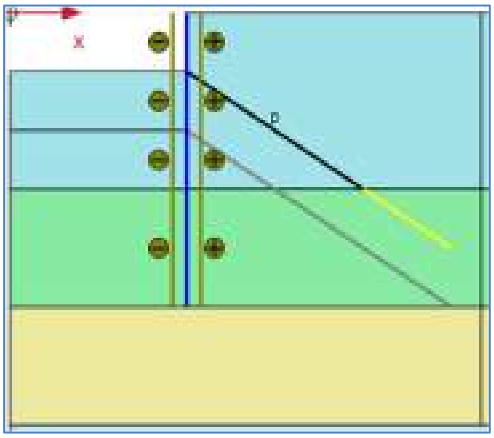


Figure 4: Phase 2, activation of the anchor and geotextile (grouting). Pre-stressing of the anchor to a value of 300 kN/m.

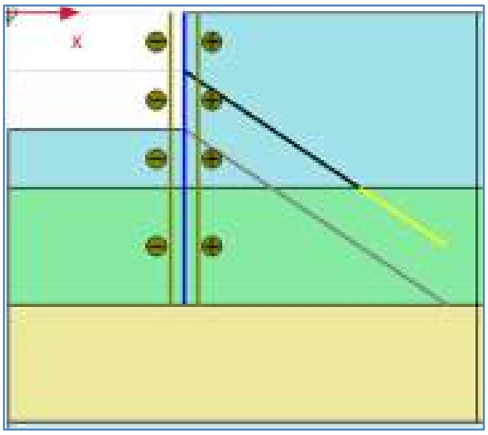


Figure 5: Phase 3, excavation of the second part.

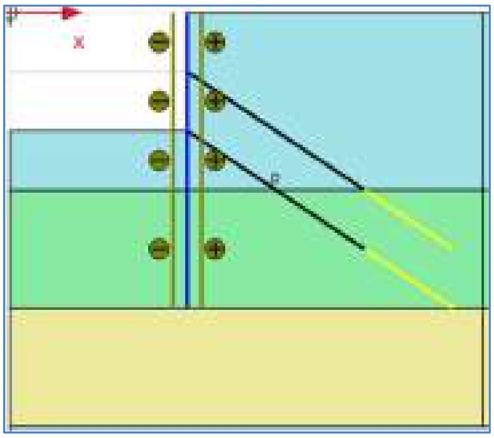


Figure 6: Phase 4, activation of the second anchor and geotextile (grouting). Pre-stressing of the anchor to a value of 300 kN/m.

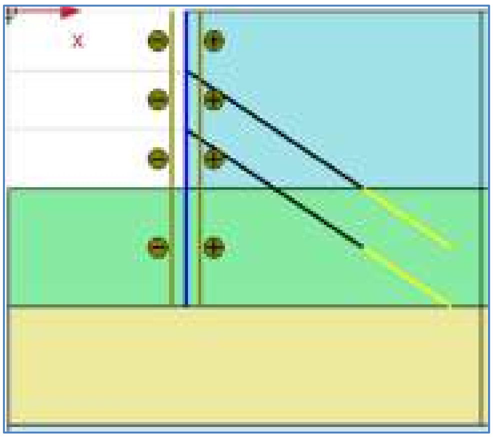


Figure 7: Phase 5, excavation of the third part.

GEOMETRY INPUT - CHANGE MATERIAL (HSS MODEL)

- Modify the material properties by selecting the item Soils & Interfaces from the Materials menu. Use the material properties as given in Table 2. Assign the Sand material set to all soil clusters.

Table 2: Soil material set parameters for HSS-model.

Parameter	Symbol	Top sand / Middle sand / Lower sand / Deep sand 1 + 2	Unit
Material model	Model	HSS-model	-
Type of behaviour	Type	drained	-
Dry weight	γ_{unsat}	18	kN/m^3
Wet weight	γ_{sat}	18	kN/m^3
Young's modulus	E_{50}^{ref}	20.000	kN/m^2
Oedometer mod.	E_{oed}^{ref}	20.000	kN/m^2
Unloading modulus	E_{ur}^{ref}	80.000	kN/m^2
Power	m	0,5	-
Poisson's ratio	ν	0,2	-
Reference stress	P_{ref}	100	kN/m^2
Cohesion	c	1,0	kN/m^2
Friction angle	φ	35	°
Dilatancy angle	ψ	5,0	°
Interface strength reduction	R_{inter}	0,6	-
K_0 ^{nc}	K_0=1-sinf	0,426	-
Shear modulus	G_0^{ref}	100.000	kN/m^2
Shear strain	$g_{0.7}$	0,0001	-

INSPECT OUTPUT

The main results for this exercise are illustrated in Figure 8, Figure 9 and Figure 10. The maximum values are listed below in Table 7.

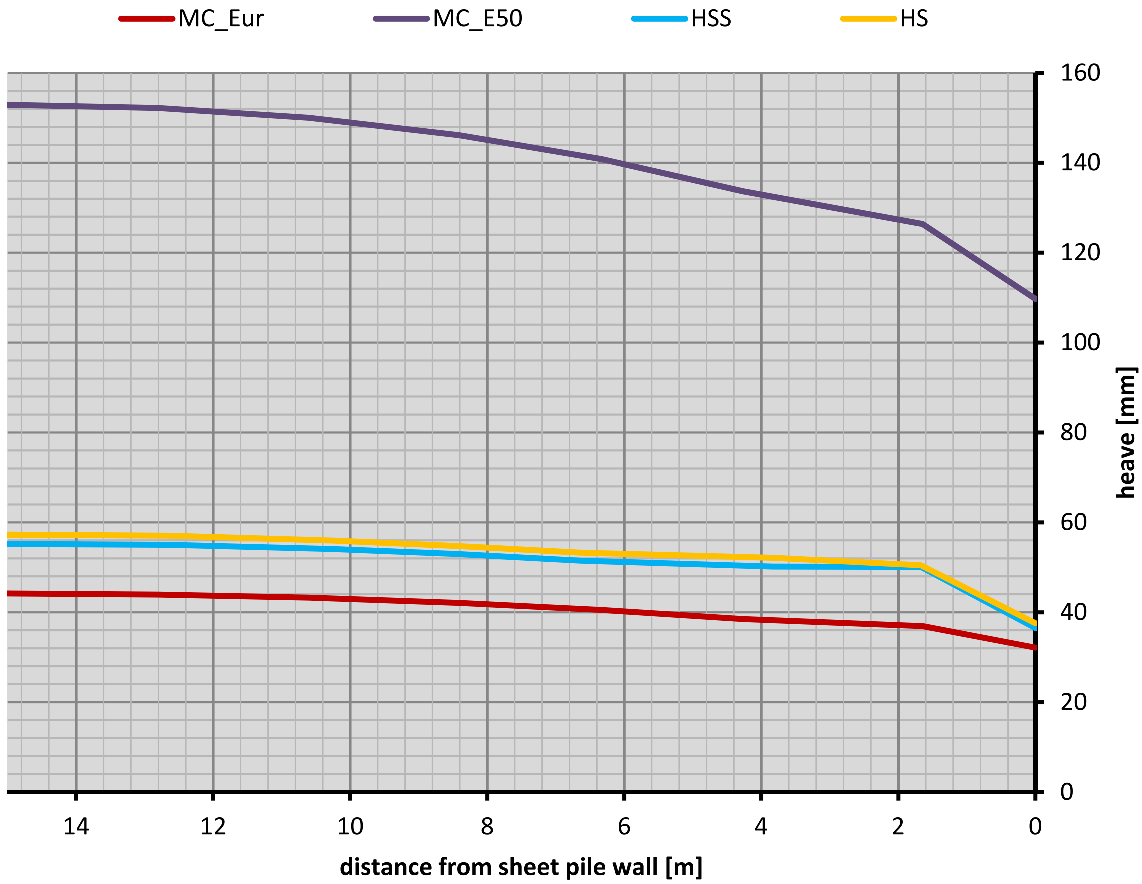


Figure 8: heave after the last excavation step [mm]

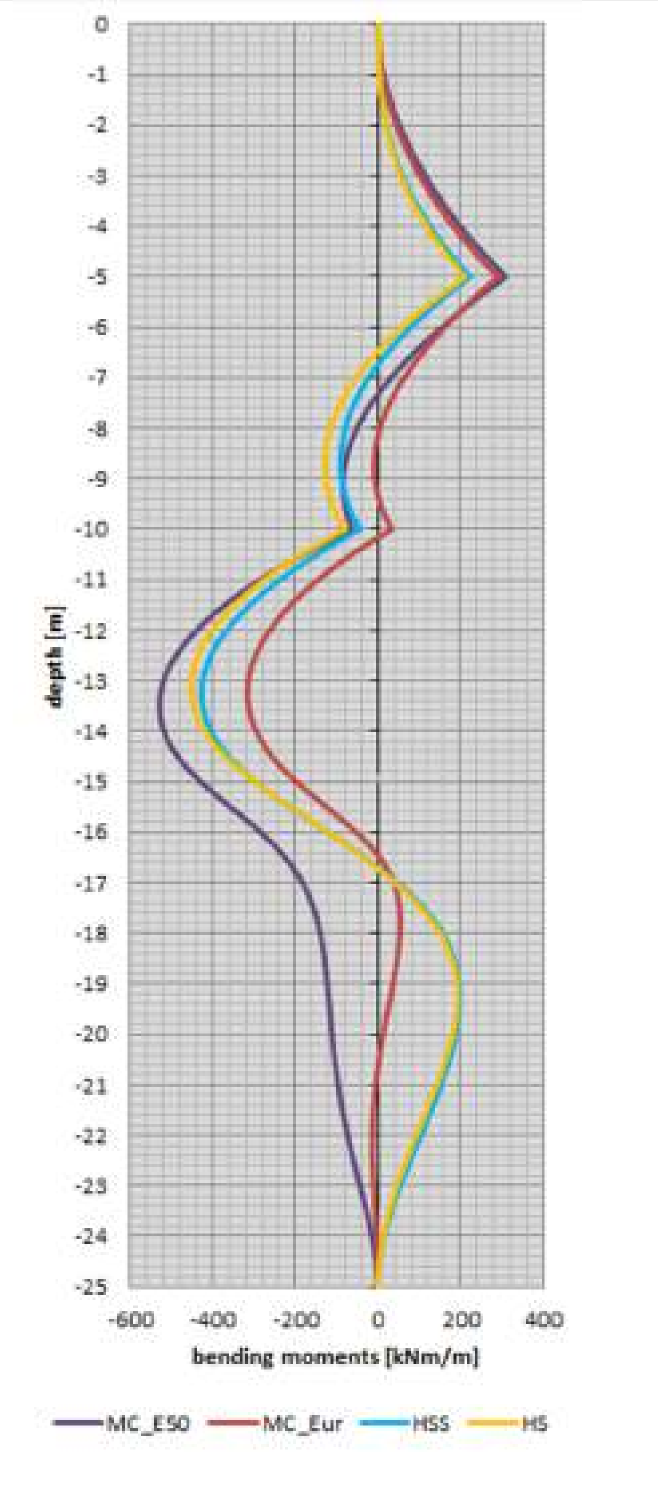


Figure 9: Bending moments after the last excavation step [kNm/m]

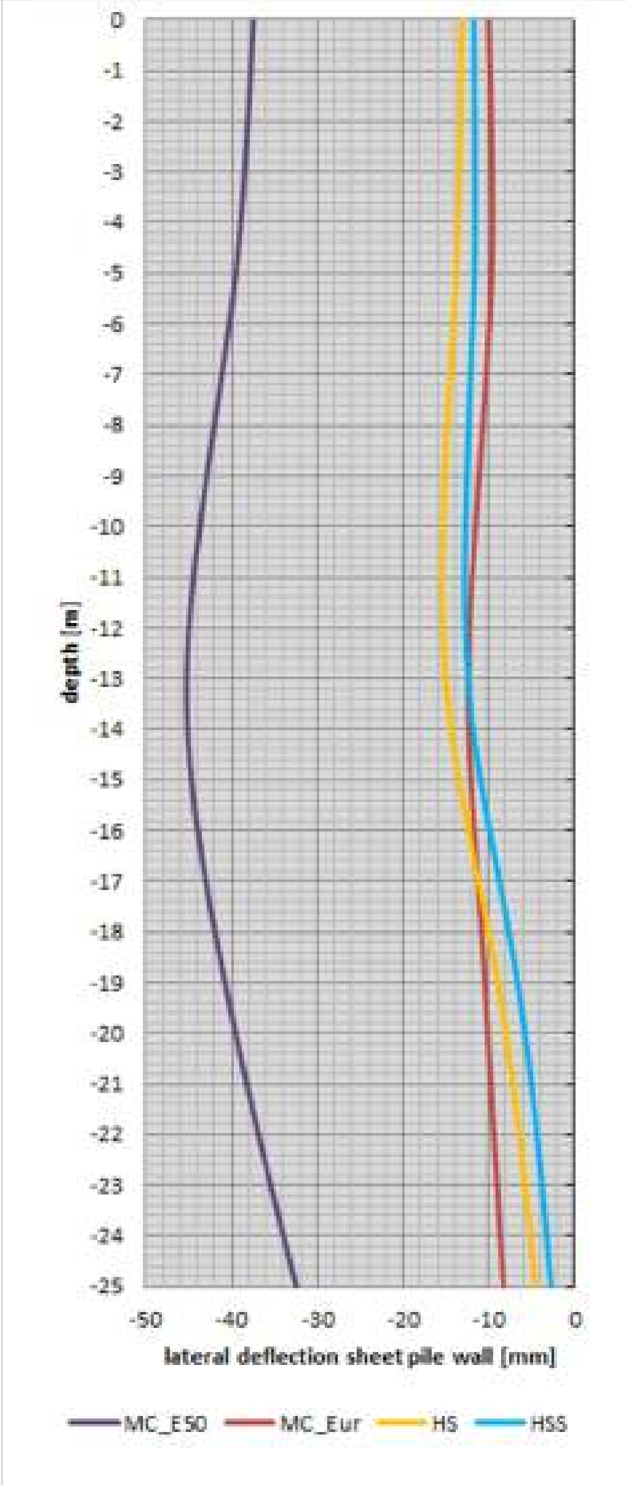


Figure 10: lateral deflection of the sheet pile wall after the last excavation step [mm]

Table 3: Maximum value of the results after the last excavation step

Parameter	Symbol	MC (E_{50})	MC ($_{Eur}$)	HS	HSS	Unit
Max. Sole heave	u_y	153	44	57	56	mm
Max. Lateral deflection	u_x	-45	-12	-15	-13	mm
Max. Surface settlements	u_y	-8	0	-7	-9	mm
Max. Bending moments	M_y	-537	-320	-451	-426	kNm/m

CG10 MODELLING OF GROUNDWATER

William W.L CHEANG PhD MSc PGDip BEng (Hons)
Dr. techn. Indra Noer Hamdhan

CONTENTS

Modelling of Groundwater in Plaxis

PART A: INTRODUCTION

Groundwater Analysis and Conditions

A. Geotechnical problems are related to groundwater

B. Two extreme conditions of porewater response are normally considered, they are:

   1. Drained
   2. Undrained (Method A, B & C)

C. Real soil behaviour is related to time , i.e. transient, with the porewater pressure being dependent on imposed:

   1. Permeability
   2. Rate of loading
   3. Hydraulic (Flow) boundary conditions

D. The interstitial voids of the soil skeleton can be fully or partially filled with pore fluid and therefore effective stresses are influenced by this action

E. This lecture will look into the following issues:

   1. The setup of pore pressures in PLAXIS
   2. Input parameters
   3. Some examples of groundwater regimes

Modelling of Groundwater in Plaxis

PART B: DEFINITIONS OF PORE PRESSURES IN PLAXIS

A. DEFINITIONS AND MODES

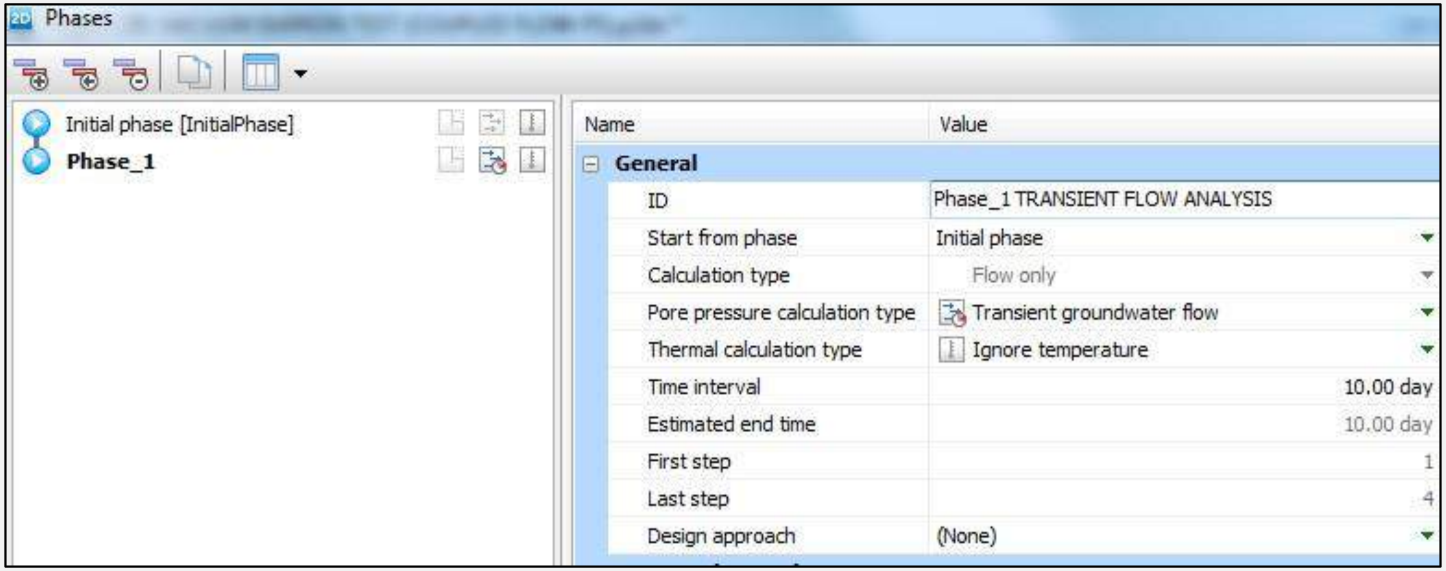
A. Calculation Type (Staged Construction Mode)

1. Ground Water Flow Only
2. Consolidation (based on P_excess ONLY)
3. Fully Coupled Flow-Deformation Consolidation (based on Total PWP ie Pactive)

B. Definition of Pore Water Pressure Terms in Plaxis

1. Pactive - Active State Pore Pressures (idea of Total Pore Water Pressure)
2. Pwater - Pore Water Pressures (Psteady + Pexcess)
3. Psteady - Steady State Pore Pressures (Background)
4. Pexcess - Excess Pore Pressures

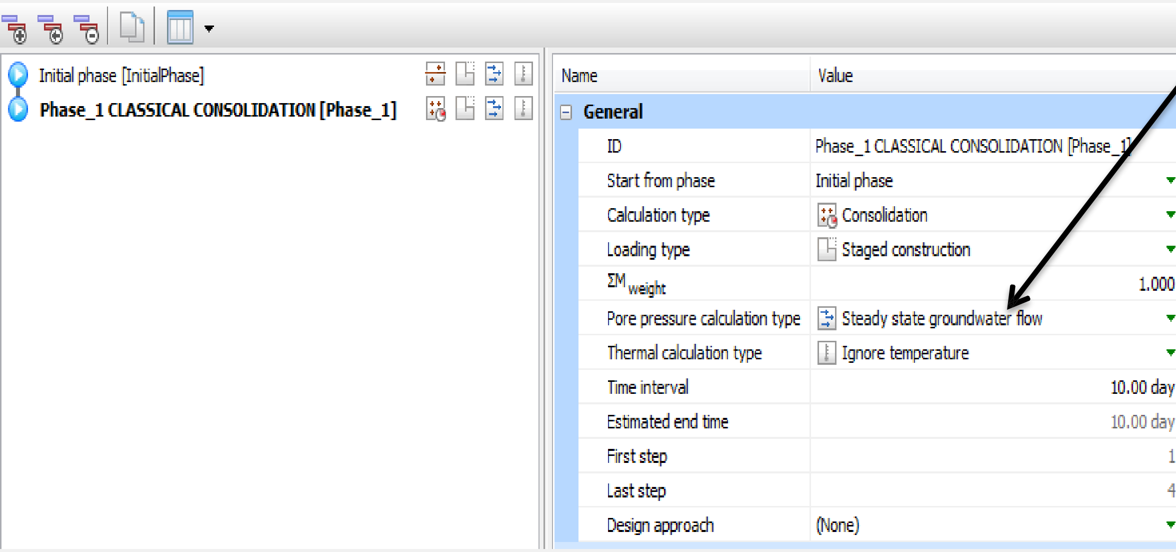
GROUNDWATER FLOW ONLY MODE



Groundwater Flow mode:

- Similar to PlaxFlow but with huge improvements in the kernel (see Galavi, 2010)
- All functionalities of PlaxFlow rewritten in PLAXIS code (new)
- Steady state groundwater flow
- Transient groundwater flow (ONLY POSSIBLE IN THIS MODE)
- All types of boundary conditions
- New features in wells and drains
- Faster calculation (new)

PLASTIC OR CONSOLIDATION TYPE

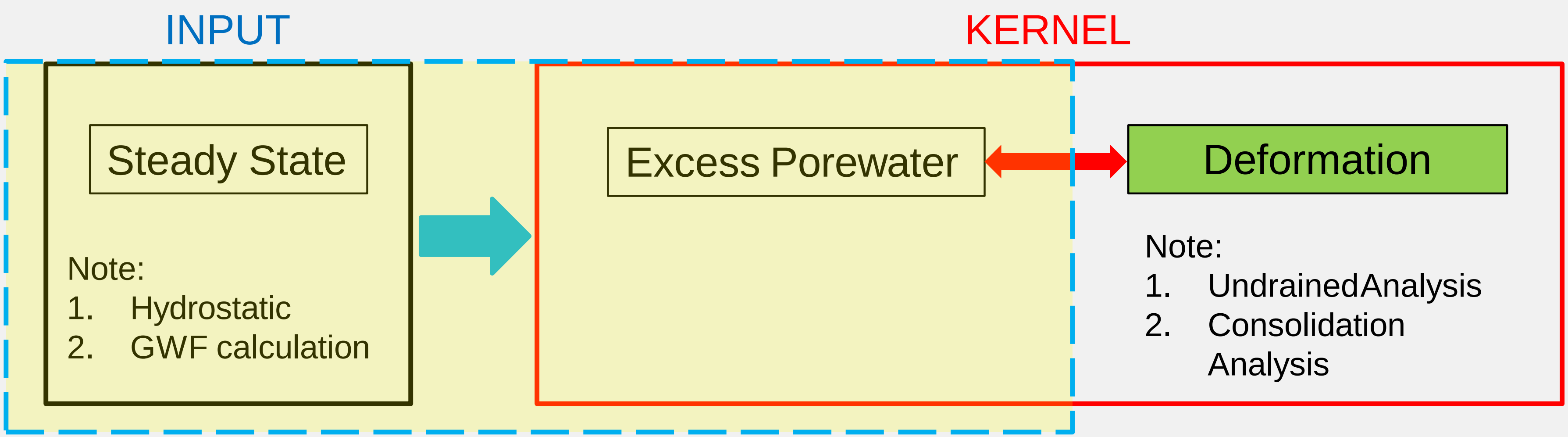


A. Steady-state pore pressures

1. Phreatic lines
2. Use pressures from previous phase
3. Steady-state groundwater flow analysis

B. Excess pore pressures

1. Undrained material type (A) in combination with "Plastic" calculation, combine with
2. Consolidation analysis



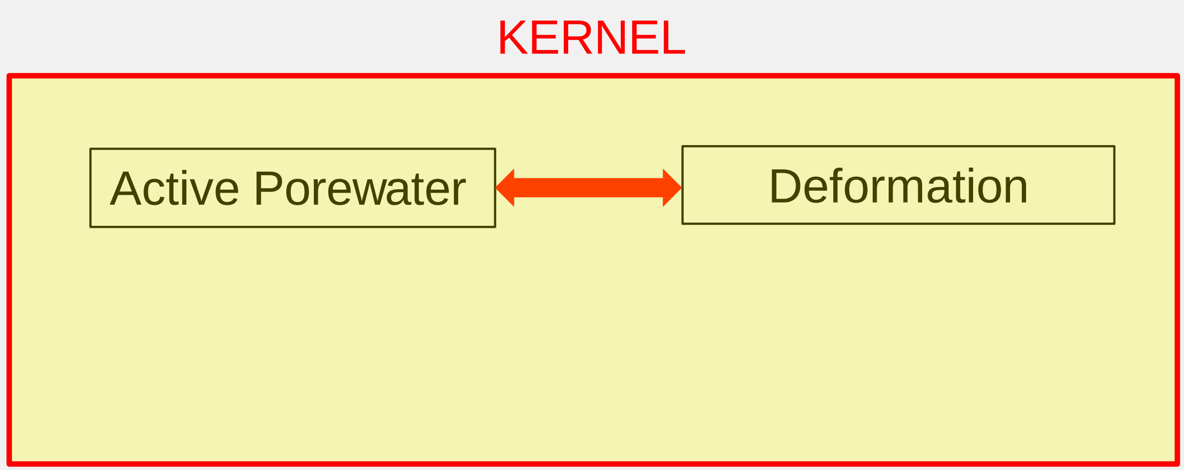
Active Porewater = Steady-state + Excess_PP

FULLY COUPLED FLOW-DEFORMATION TYPE

Name	Value
General	
ID	Phase_1 COUPLED FLOW CONSOLIDATION
Start from phase	Initial phase
Calculation type	Fully coupled flow-deformation
Loading type	Staged construction
ΣM_{weight}	1.000
Thermal calculation type	Ignore temperature
Time interval	10.00 day
Estimated end time	10.00 day
First step	1
Last step	4
Design approach	(None)

A. Consolidation analysis ON Total PP

B. Transient groundwater flow analysis is implied in the calculations



ACTIVE PORE PRESSURE

P_{active}

9.3.6 PORE PRESSURES

Pore pressures are the quantities that relate to the stress in the pores of the material. The pores in soil are usually filled with a mixture of water and air. In most cases, stresses and pore pressures in the soil are negative (pressure). However, due to capillary action or undrained unloading, pore pressures may become positive (tension), which is denoted as suction.

- PLAXIS distinguishes between different types of pore pressures and related quantities. It is important to understand the differences between these quantities in order to properly interpret the results from a PLAXIS calculation.

Total stresses are divided into effective stresses, σ', and active pore pressures, p_{active}.

$$\sigma = \sigma' + p_{active}$$

Active pore pressures are defined as the effective saturation, S_{eff}, times the pore water pressures, p_{water}.

$$\boxed{p_{active} = S_{eff} \cdot p_{water}}$$

Effective degree of saturation: $S_{eff} = (S - S_{res})/(S_{sat} - S_{res})$

Degree of saturation: S

Residual degree of saturation: S_{res}

Pore water pressure differs from active pore pressure when the degree of saturation is less than unity, which is usually the case above the phreatic level. Below the phreatic level p_{active} and p_{water} are generally equal.

PORE WATER PRESSURE

P_{water}

Total stresses are divided into effective stresses and pore pressures. Pore pressures are composed of steady-state pore pressures and excess pore pressures, including suction. Since decades, PLAXIS uses the term 'active pore pressure' to denote the contribution of the pore stresses in the total stresses. In the context of unsaturated soil behaviour, active pore pressure is defined as:

Active pore pressure: $p_{active} = S_{eff}(p_{steady} + p_{excess}) = S_{eff}\, p_w$

Pore water pressure: $p_{water} = p_{steady} + p_{excess}$

Note that for fully saturated soil the 'active pore pressure' is equal to the 'pore water pressure', but there is a clear difference in the unsaturated zone.

As an alternative to the pore water pressure, p_{water}, the groundwater head, h, can be viewed:

$$h = z - \frac{p_{water}}{\gamma_w}$$

where z is the vertical coordinate and γ_w is the unit weight of water. In the pore water pressure a further distinction is made between steady state pore pressure, p_{steady}, and excess pore pressure, p_{excess}.

$$p_{water} = p_{steady} + p_{excess}$$

where steady-state pore pressure is the steady-state or long term part of pore pressure, which is supposed to be input data for a deformation analysis.

Excess pore pressure is the result from undrained behaviour (*Undrained (A)* or *(B)*, or low-permeable materials), and is affected by stress changes due to loading or unloading, a (sudden) change in hydraulic conditions and consolidation.

STEADY-STATE AND EXCESS PORE PRESSURE

P_{steady} P_{excess}

Steady-state pore pressure: p_{steady}

- Input for a deformation analysis Plastic Drained or Undrained Analysis
- Direct generation based on phreatic levels and cluster-related pore pressure distribution
- Or, calculated from steady-state groundwater flow calculation GWF- STEADY FLOW

Excess pore pressure: p_{excess}

- Result from undrained behaviour (*Undrained (A)* or *(B)*) and K_w/n from Table C.1
- Affected by loading, a (sudden) change in hydraulic conditions and consolidation.
- In the case of a fully coupled flow-deformation analysis, p_{steady} is calculated from a preliminary steady-state groundwater flow calculation using the hydraulic boundary conditions at the end of the calculation phase. This enables the calculation and output of p_{excess} for a fully coupled analysis in all steps. $p_{excess} = p_{water} - p_{steady}$.

Consolidation process is dissipation of P_{active} or P_{excess} to P_{steady} values in the long-term condition

SATURATION AND SUCTION

SATURATION

Effective degree of saturation: $S_{eff} = (S - S_{res})/(S_{sat} - S_{res})$

Degree of saturation: S

Residual degree of saturation: S_{res}

Saturated degree of saturation: S_{sat} (usually 1.0)

- There is a relationship between the (effective) degree of saturation (S or S_{eff}) and the suction in the unsaturated zone, which depends on the selected soil-water retention curve in the material data set. By default, the data set for 'fine sand' is selected. SWRC (OR SWCC OR RC) = SATURATION – SUCTION RELATIONSHIP

SUCTION

The term 'suction' is used to denote any positive value of pore water stress (pore water tension):

Suction: Tension component in p_{water}: $Suction = Max(p_{water}, 0)$

- Suction could be a result of a positive component in p_{steady} as well as p_{excess}, but it is evaluated from the sum of both. Include Steady (capillary action) and Excess_PP components (undrained unloading)

Effective suction: $Suction_{eff} = S_{eff} Suction$

- When multiplied by the tangent of the friction angle, this quantity gives a kind of 'artificial cohesion' in the soil. This is know as the apparent cohesion

Similar in concept to Prof Fredlund's strength model Ca = (ua-uw)*tan ϕ_b = S_{eff} *tanϕ

TERZAGHI OR BISHOP EFFECTIVE STRESS

Terzaghi Effective Stress (**only for Saturated Soil with pressure or suction**)

Effective stress: σ' (stress in the soil skeleton)

Total stress: $\sigma = \sigma' + p_{active}$

Terzaghi's effective stress: $\sigma' = \sigma - p_{active}$

- Terzaghi's definition of effective stress applies to saturated soils.

Bishop Effective Stress (**only for Saturated/Unsaturated Soil** with pressure or suction)

Bishop' s stress: $\sigma' = \sigma - p_a + \chi(p_a - p_{water})$

Atmospheric pressure: p_a (100 kPa, but in PLAXIS taken as the zero reference level)

Matric suction coefficient: χ

S-eff = Effective Degree of Saturation

In PLAXIS, p_a is assumed to be 0 and χ is supposed to be equal to S_{eff}, so Bishop' s stress in PLAXIS:

$$\sigma' = \sigma - S_{eff} \cdot (p_{steady} + p_{excess}) = \sigma - p_{active}$$

- When the soil reaches full saturation ($S_{eff} = 1$), Bishop's stress coincides with Terzaghi's effective stress.
- Using Bishop's effective stress as a basis for all calculations, distinction can still be made between conventional calculations (fully saturated or dry materials) and calculations in which unsaturated soil behaviour is taken into account.
- The aforementioned distinction can be made by ignoring suction or allowing suction.

Modelling of Groundwater in Plaxis

PART C: GENERATION OF POREWATER PRESSURES

GENERATION OF STEADY-STATE PORE PRESSURES

Steady-state (Background) pore pressures can be generated by:

A. Phreatic and Cluster Approach (Hydrostatic)

B. Groundwater Flow Analysis

PHREATIC AND CLUSTER APPROACH

GENERATION OF STEADY-STATE PRESSURES

PHREATIC AND CLUSTER APPROACH

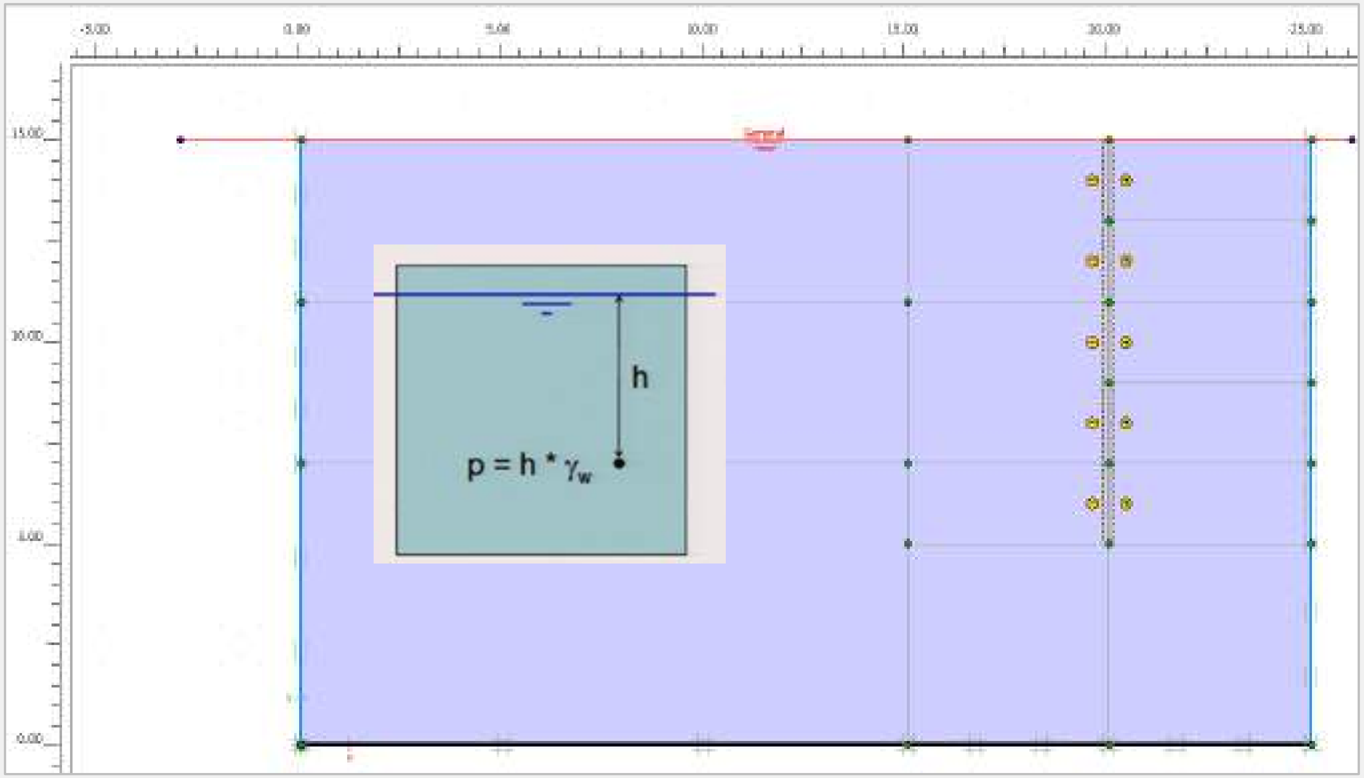
A. Phreatic Level

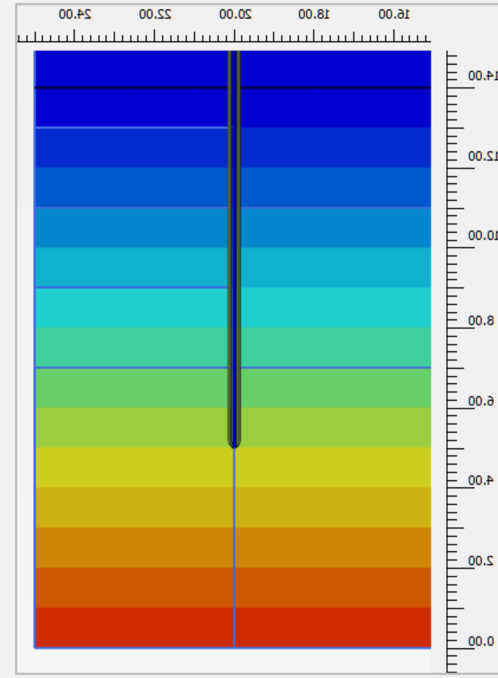
1. General Phreatic Level
2. Cluster Phreatic Level

B. Cluster Pore Pressure Distribution

1. Interpolation (Adjacent to clusters or phreatic lines)
2. Cluster Dry
3. User-defined Pore Pressure Distribution (by Customised BH)

C1. PHREATIC LINE





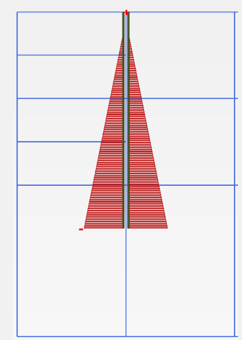
a) Porewater pressures are hydrostatic

b) Calculated based on gamma-water * height of the water column

c) Simple situations (water-table is horizontal)

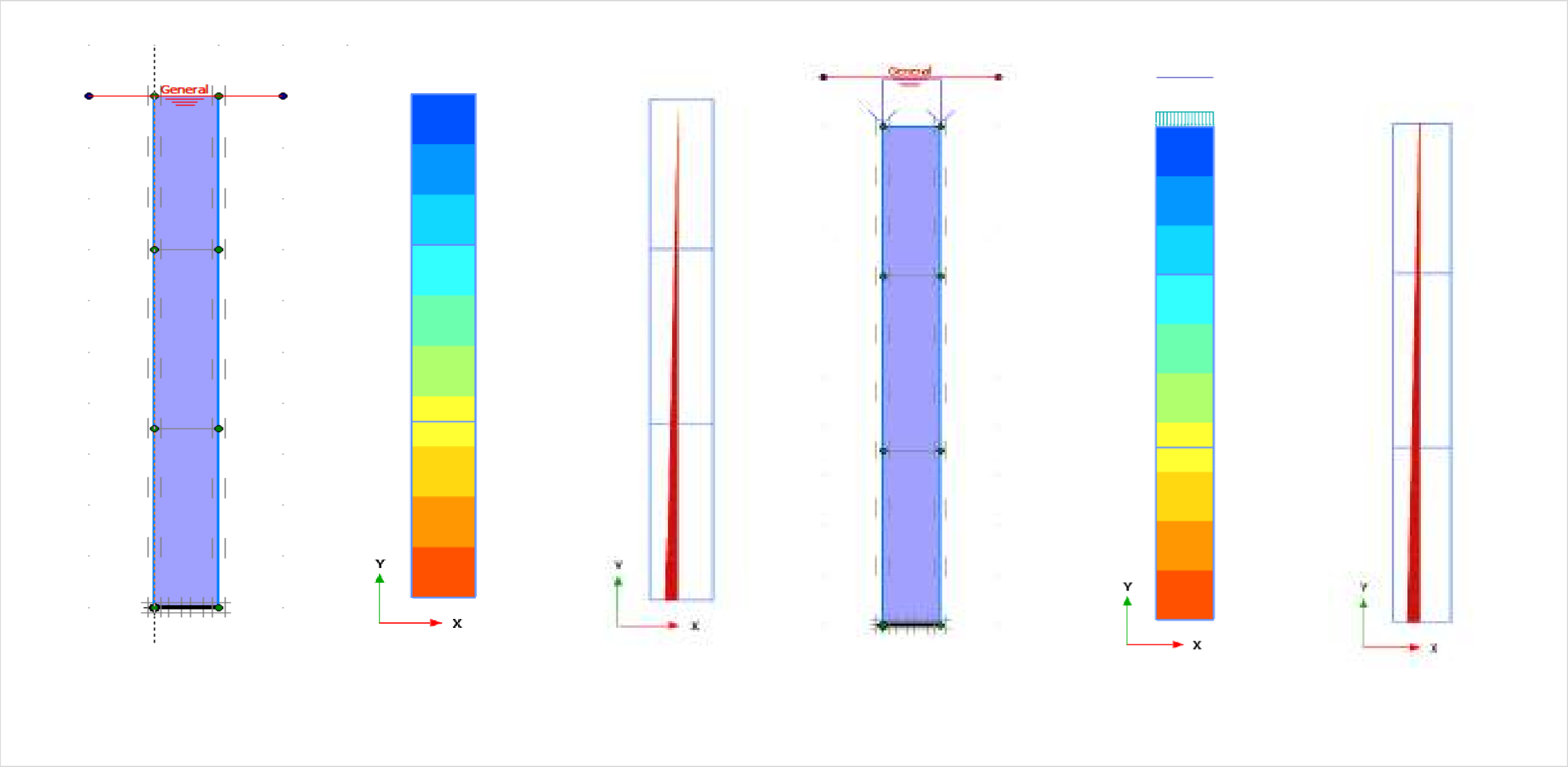
d) No flow

e) For cases, i.e. simple excavations, foundations or embankments

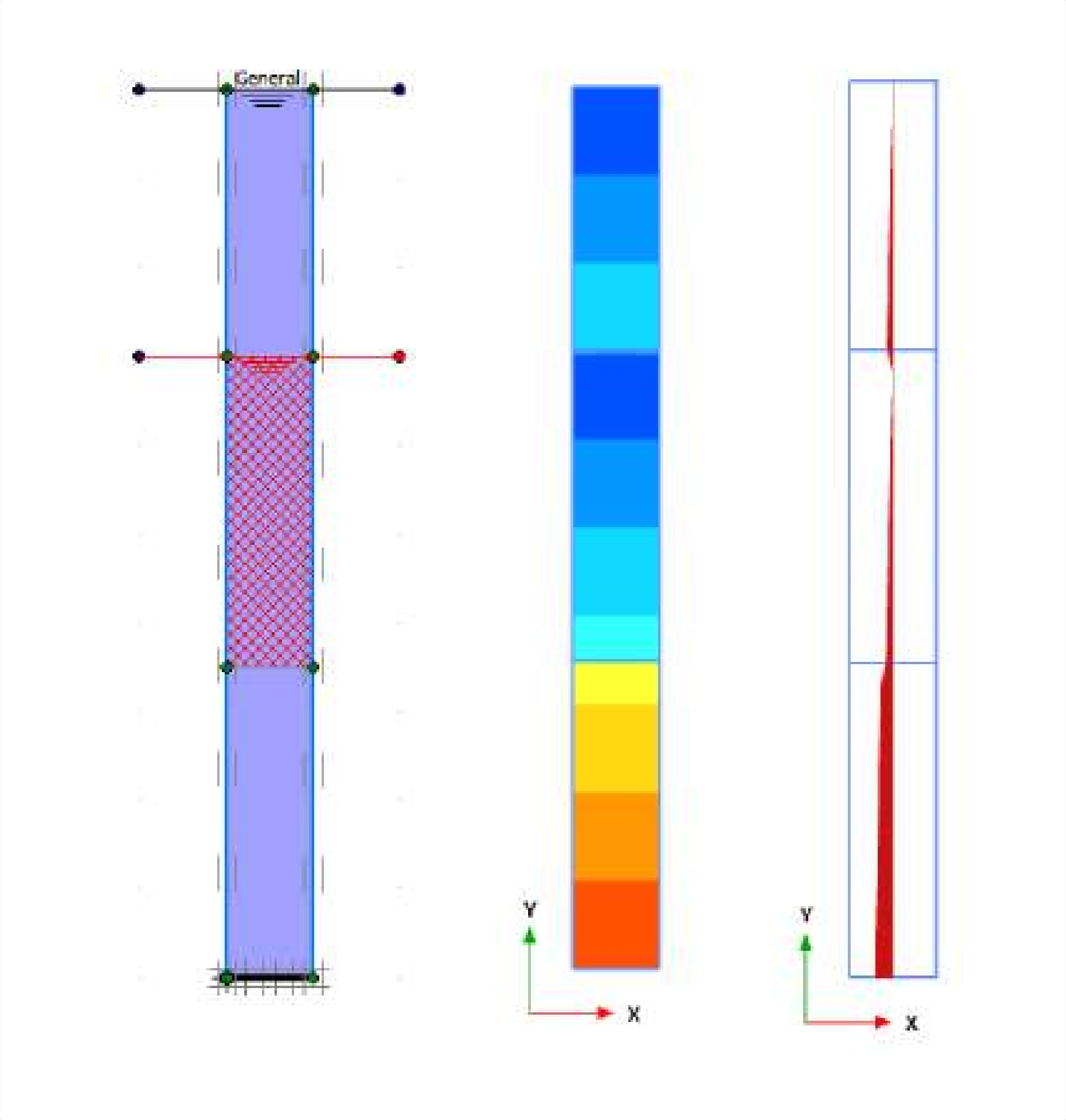


Hydrostatic steady-state pressure along interface

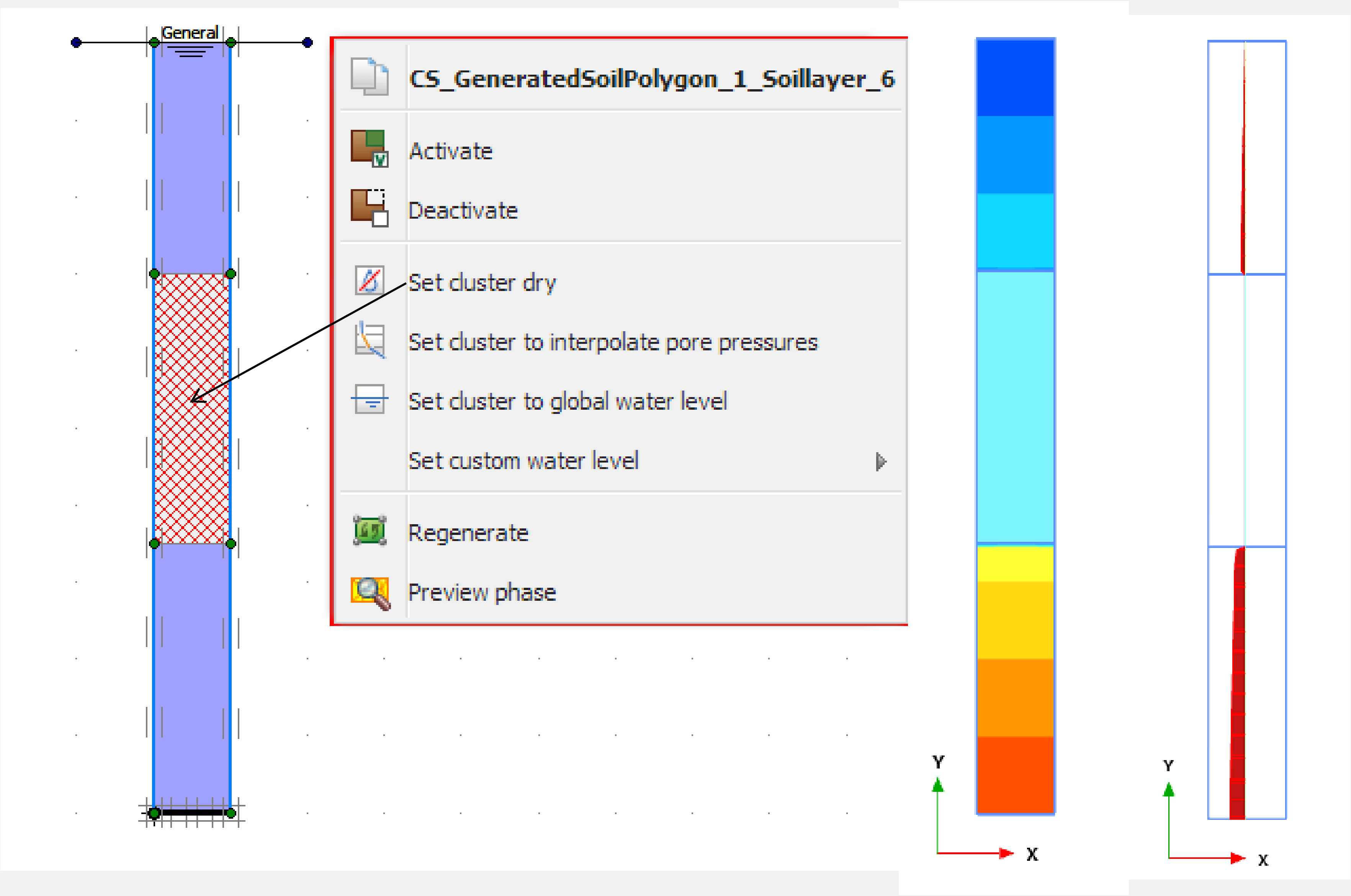
GENERAL PHREATIC LINE



GENERAL & CLUSTER PHREATIC LINES

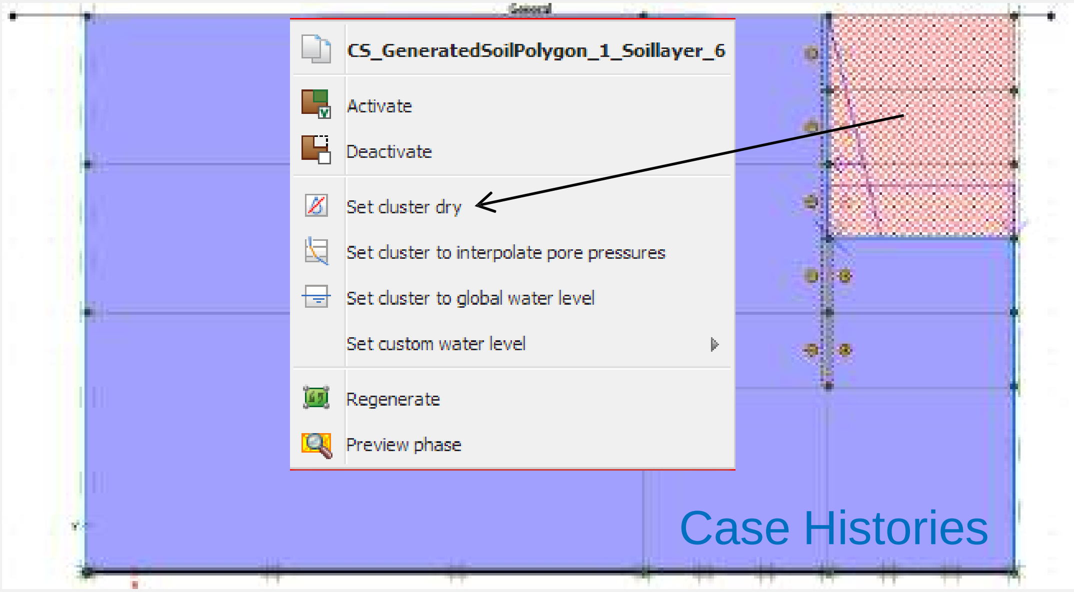


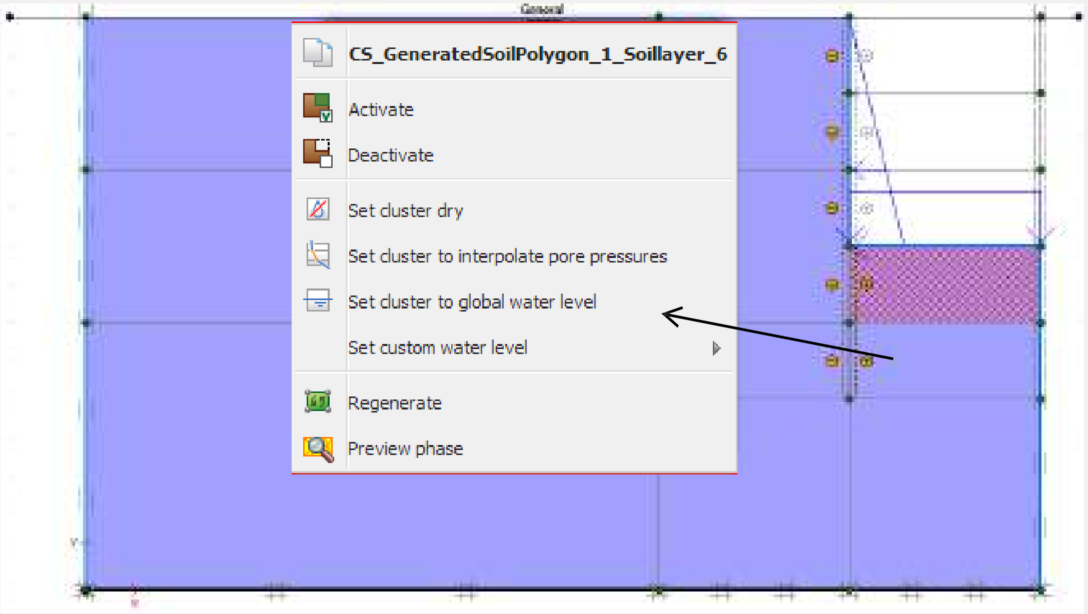
Cluster: Dry

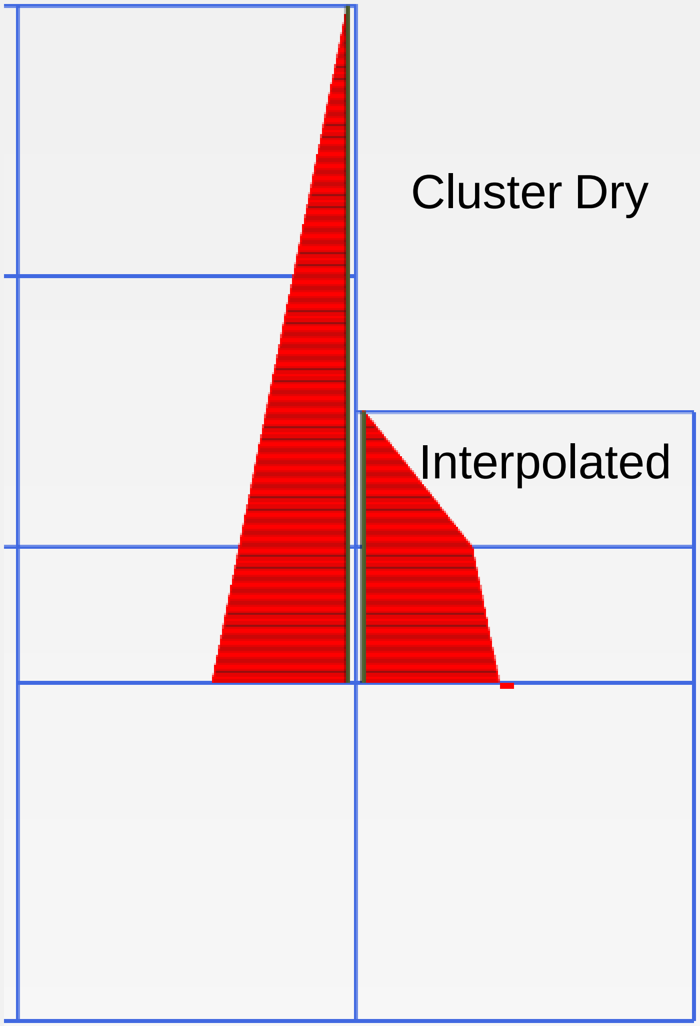


General
CS_GeneratedSoilPolygon_1_Soillayer_6
Activate
Deactivate
Set cluster dry
Set cluster to interpolate pore pressures
Set cluster to global water level
Set custom water level
Regenerate
Preview phase
Y
X
Y
X

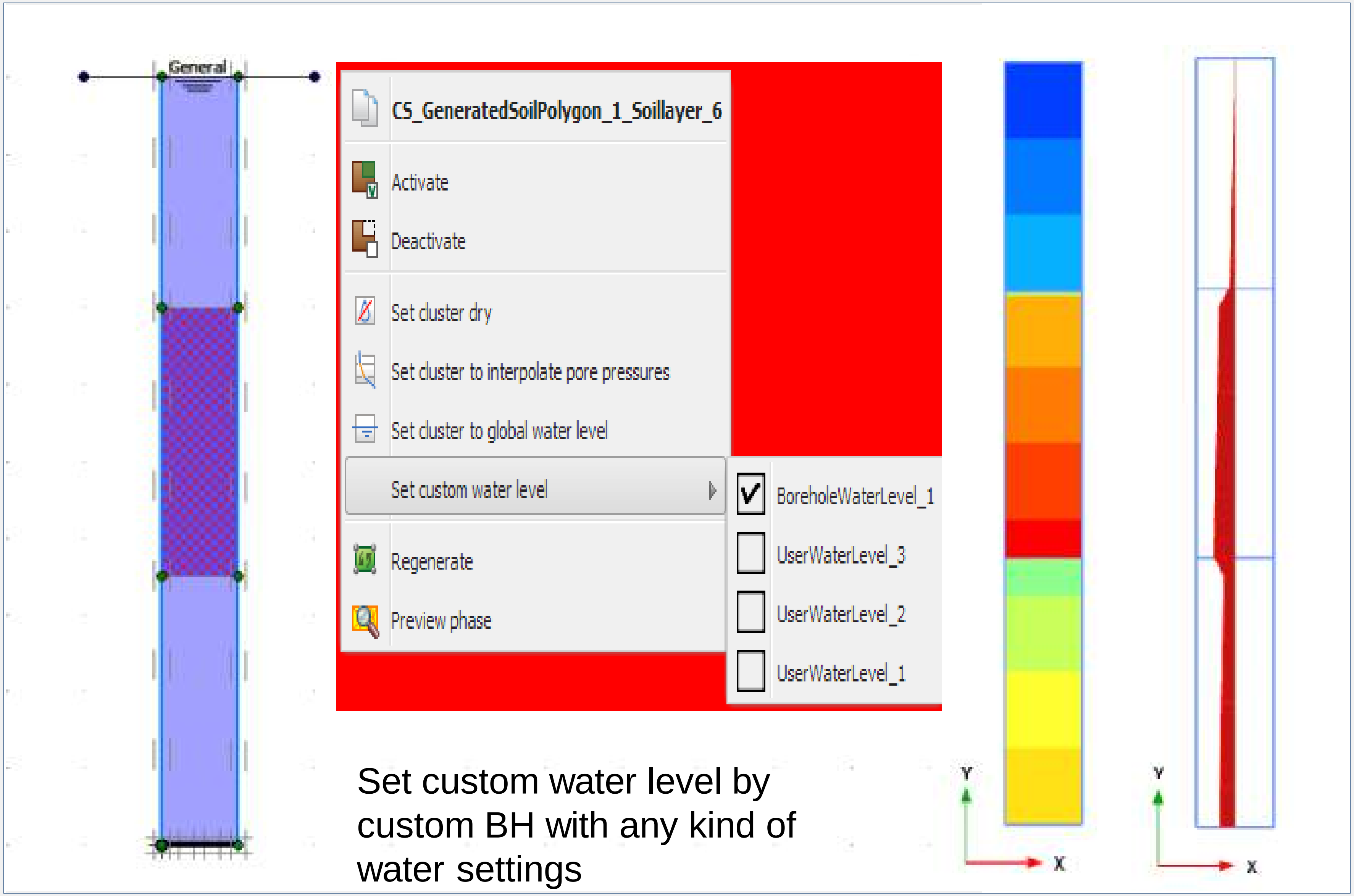
CLUSTER DRY + INTERPOLATION COMBO







Cluster: User-defined Pore Pressure Distribution



Set custom water level by custom BH with any kind of water settings

DEFINE WATER CONDITIONS IN INDIVIDUAL SOIL LAYERS IN BH IN SOIL MODE

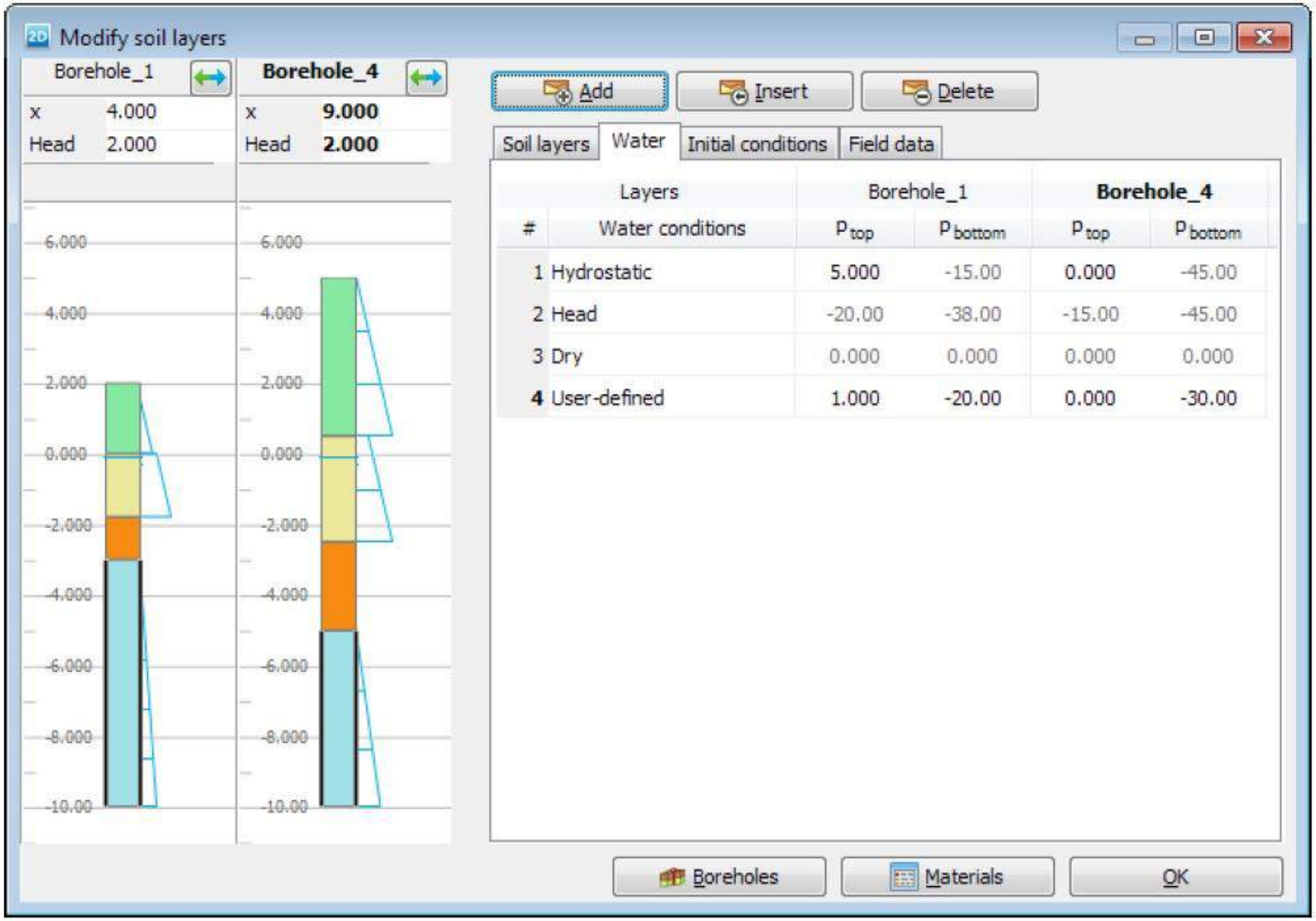


Figure 4.5 *Water* tabsheet of the *Modify soil layers* window

DEFINE HYDRAULIC CONDITIONS IN STRUCTURE MODE

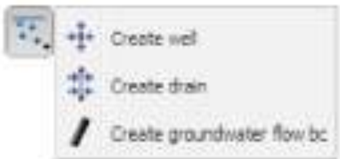


Figure 5.41 The *Create hydraulic conditions* menu

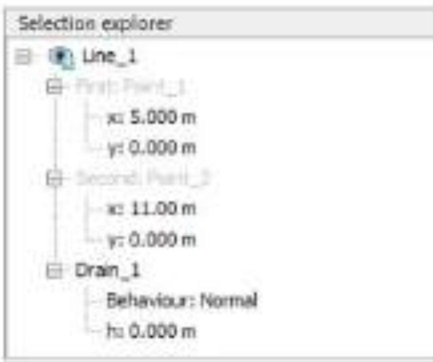


Figure 5.43 *Normal* drain definition in the *Selection explorer*

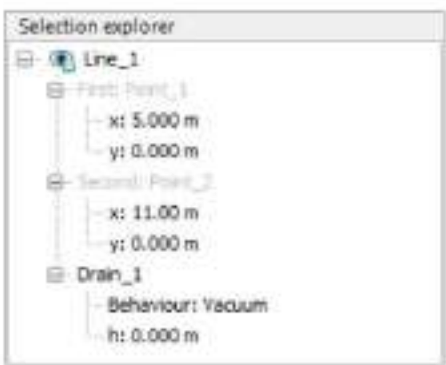
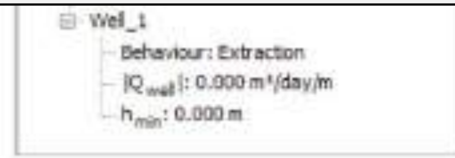


Figure 5.44 *Vacuum* drain definition in the *Selection explorer*



Extraction or Infiltration well

Figure 5.42 *Well* definition in the *Selection explorer*

The minimum possible head in the well. When the groundwater head reduces below the h_{min} level no further extraction will occur. As a habit, h_{min} may be set equal to the bottom level of the well in the ground.

- Drains are lines where Excess PP are set to reduce to the specified head h. These are relevant only to GW Flow, Consolidation or Fully Coupled Analysis
- Normal drains setting will cause Excess PP to reduce to given head h, usually set at hydrostatic value. PP lower than given head are not affected (ie it works only for compressive Excess PP)
- Vacuum drains allow drains to reduce Excess PP to below atmospheric pressures (tensile Excess PP). The value of h is set to –ve value usually to a maximum of -10m (-100 kPa). Real drains can only achieve between -60 and -80 kPa

Groundwater Flow Analysis

GENERATION OF STEADY-STATE PRESSURES

GROUNDWATER FLOW ANALYSIS OPTIONS

1. Steady-state Pore Pressure Generation based on Groundwater Flow Calculation
   - PLASTIC CALCULATION TYPE
   - CONSOLIDATION CALCULATION TYPE
2. Transient-state groundwater flow
   - IMPLIED IN FULL-COUPLED FLOW DEFORMATION ANALYIS
   - POSSIBLE TO PERFORM THIS ANALYSIS IN “PURE” GWF MODE

GROUNDWATER FLOW: STEADY STATE

Calculation based on setup of:

1. Boundary conditions:
   a) Prescribed water levels (constant)
   b) Closed flow boundaries (bottom, axis of symmetry)
   c) Wells and drains (constant)
   d) Interface elements (on=impermeable, off=permeable)
   e) Inflow / outflow (constant)
2. Soil permeabilities of layers
3. Phreatic level in the soil is being calculated for t=infinite

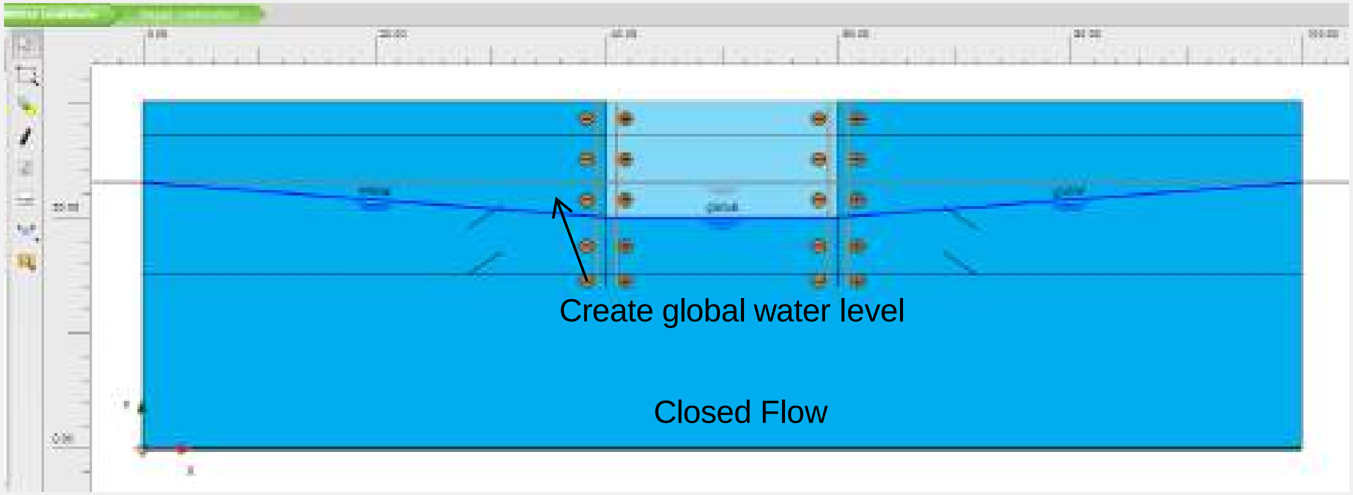
GROUNDWATER FLOW: TRANSIENT-STATE

Transient groundwater flow:

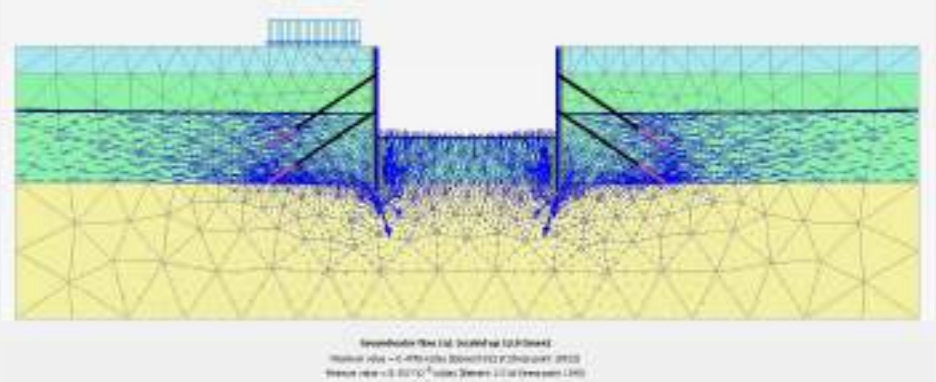
1. Boundary conditions
   a) Prescribed water levels (changing with time)
   b) Closed flow boundaries (bottom, axis of symmetry)
   c) Wells and drains (changing with time)
   d) Interface elements (on=impermeable, off=permeable)
   e) Inflow / outflow (changing with time)
2. Flow field changes in time:
   a) Constantly changing natural water conditions
   b) Relatively fast building process, pumping, wells
3. Embankments with river changes, tidal change
4. Reservoir impoundment and drawdown
5. Precipitation problems

Pore pressures – steady-state

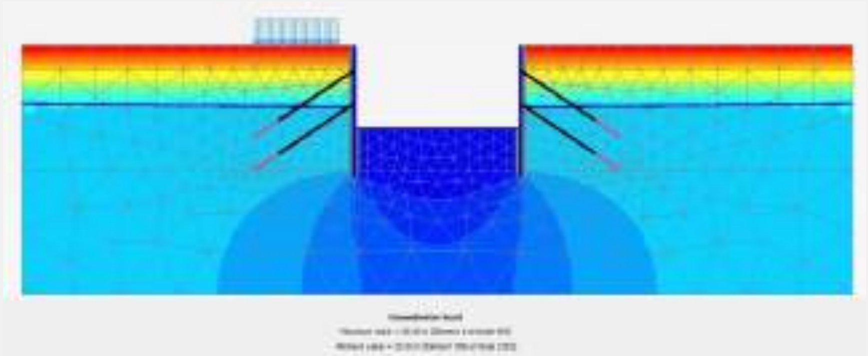
Steady-state flow:
Water conditions settings



RESULTS – FLOW FIELD

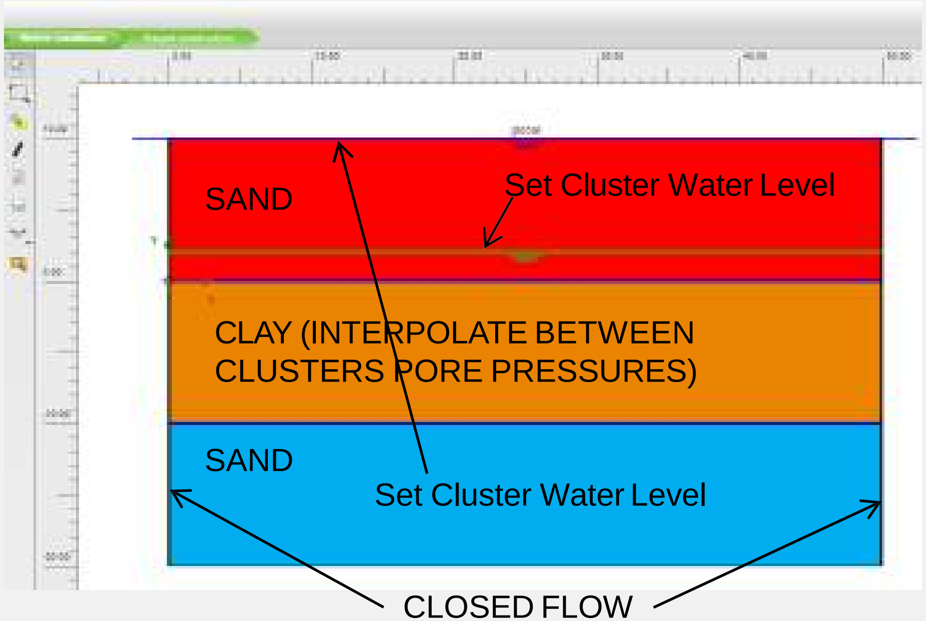


RESULTS – GROUNDWATER HEAD

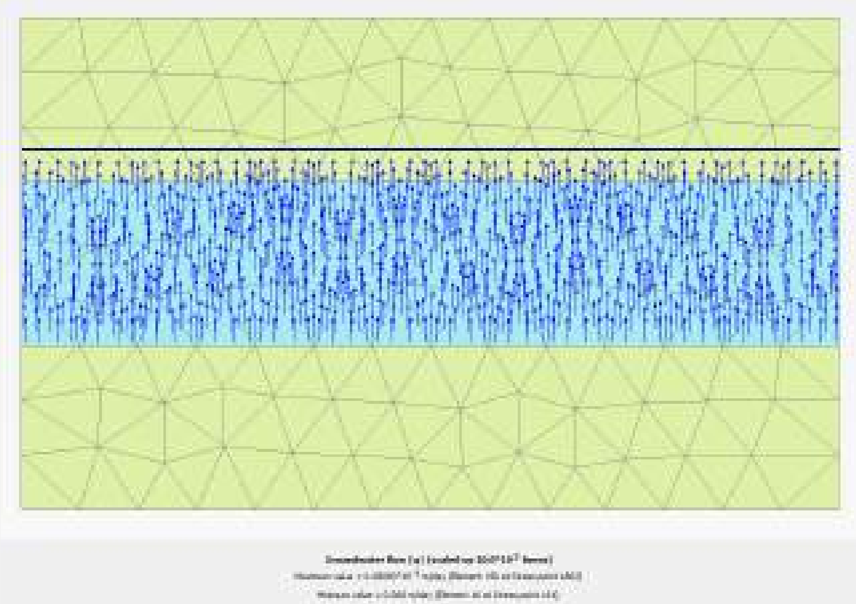


STEADY STATE – ARTESIAN CONDITION

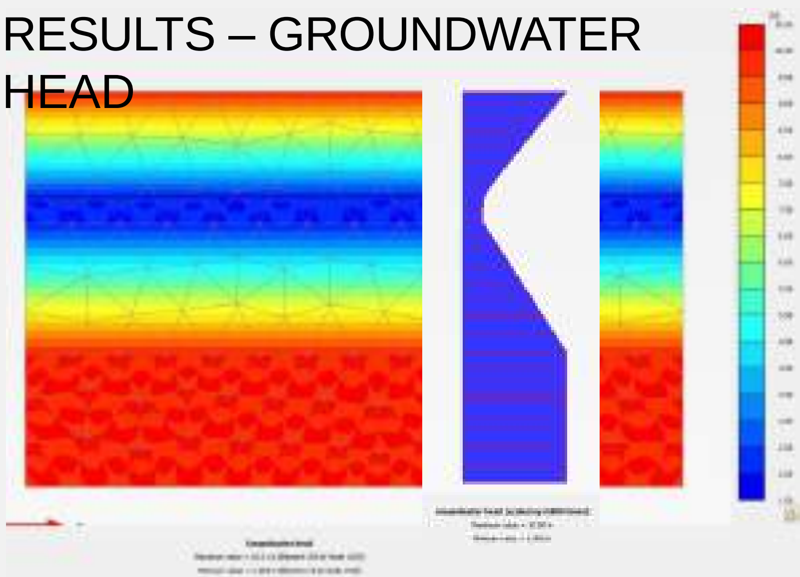
WATER CONDITIONS SETTINGS:



RESULTS – FLOW FIELD



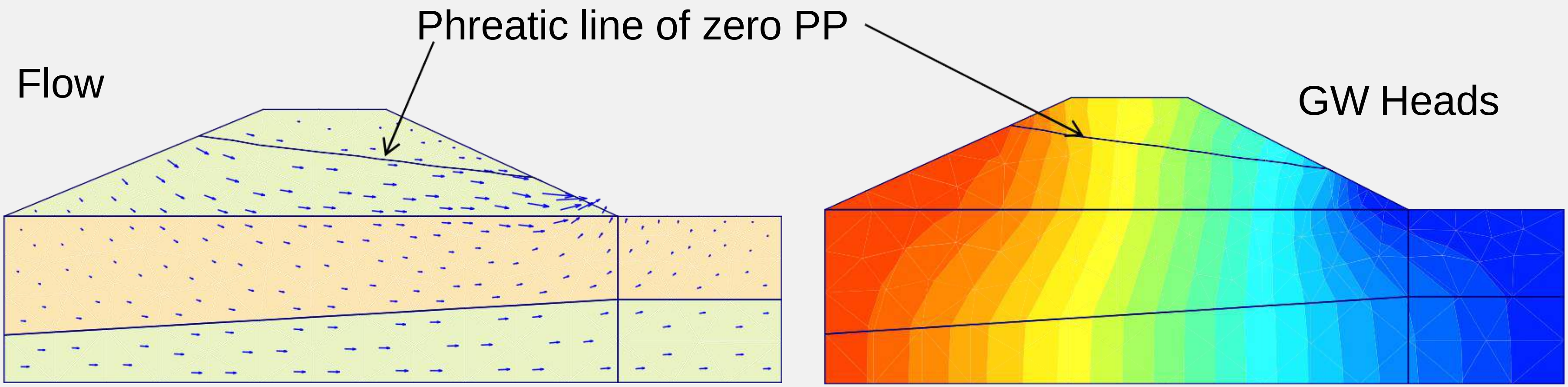
RESULTS – GROUNDWATER HEAD



SOME POINTS: FINITE ELEMENT MODELING

1. GWF calculation generally needs finer mesh than deformation analysis
2. GWF calculation generally needs larger number of steps than deformation analysis
3. GWF calculation usually converges, but can be problematic when:
    a) Mesh is too coarse
    b) Elements are distorted
    c) Large differences in permeability of adjacent layers

SOME POINTS- FINITE ELEMENT MODELING



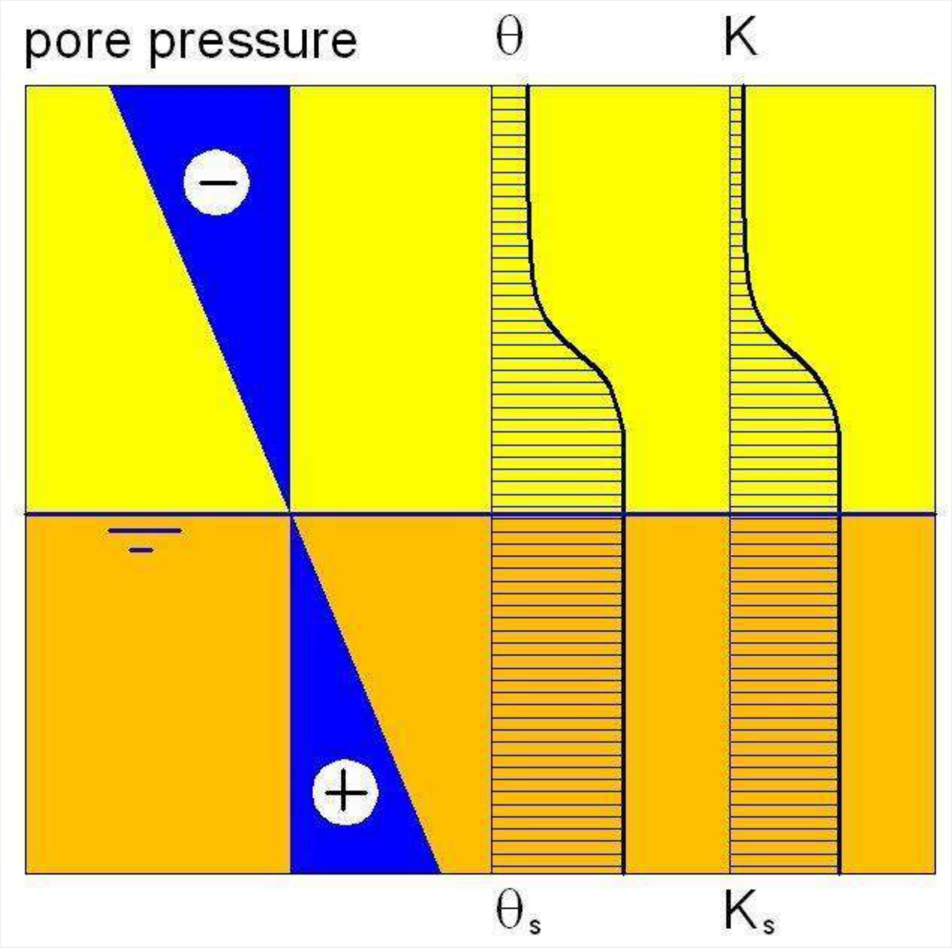
1. Qualitative evaluation:
   - Flow field
   - Location of phreatic line
2. Quantitative evaluation:
   - Heads, pore pressures compared to hydrostatic,
   - Compare with measurements or field experience

Modelling of Groundwater in Plaxis

PART D: HYDRAULIC MODELS IN PLAXIS

Groundwater flow – flow in unsaturated soil

Water content and permeability in unsaturated zone



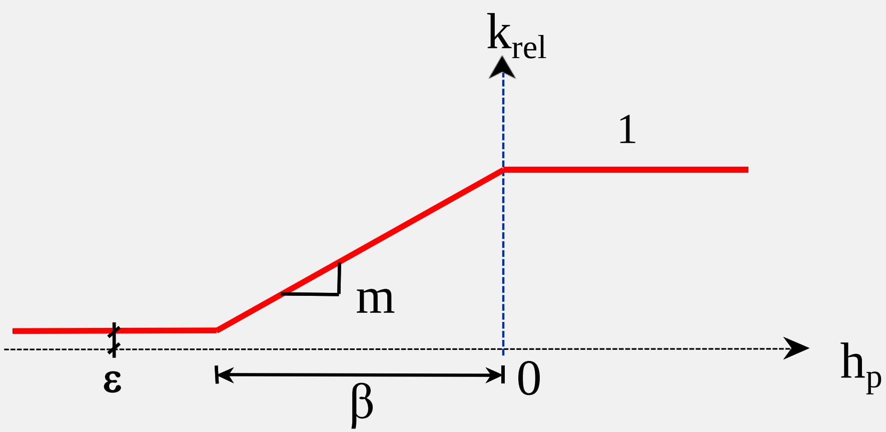
$$k = k_{rel}\, k_{sat}\,, \quad k_{rel} = f\left(h_p\,, S\right)$$

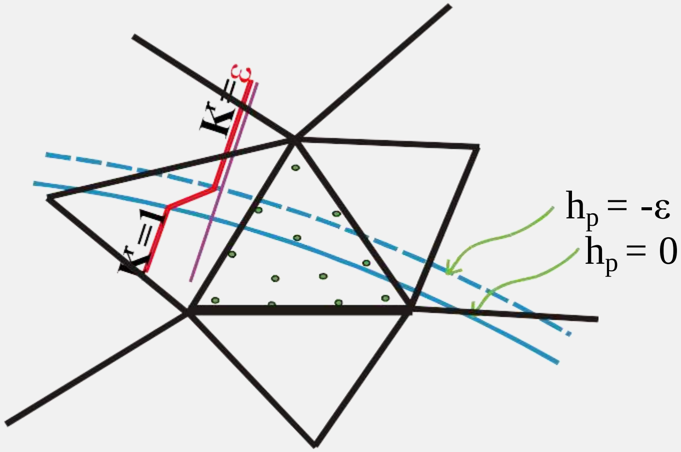
$$S(h) = \frac{\theta(h_p)}{n}$$

$h_p = \Psi$ => pressure head

Groundwater flow – flow in unsaturated soil

A. Linear Model





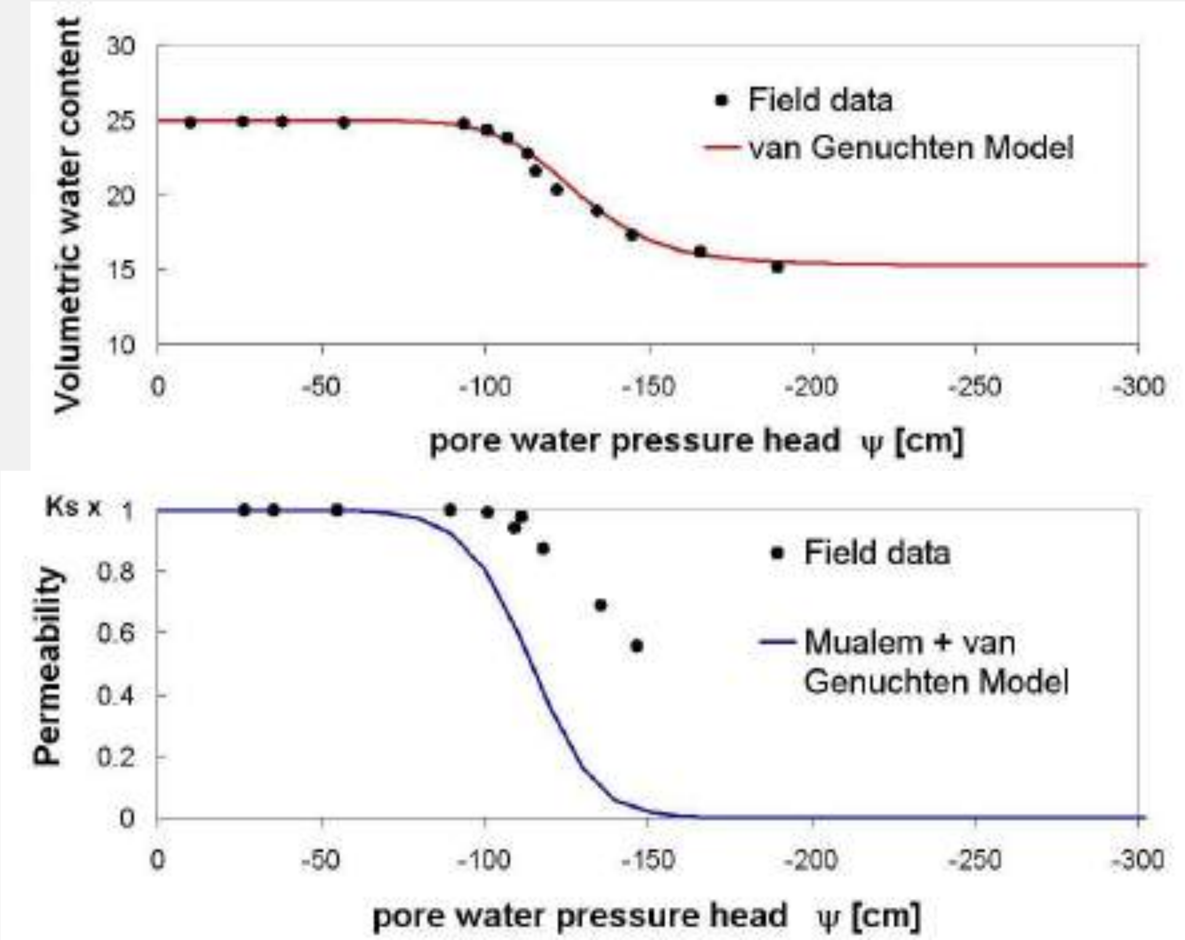
$$k_{rel} = \begin{cases} 1 & h_p > 0 \quad \text{Saturated} \\ 1 - m h_p & 0 > h_p > \beta \quad \text{Partially saturated} \\ \varepsilon & h_p < \beta \quad \text{Dry} \end{cases}$$

For numerical stability

$$\beta > \frac{1}{3} \frac{\sqrt{A_e}}{N_{\text{int}}}$$

Groundwater flow – flow in unsaturated soil

Soil Water Characteristic Curve (SWRC or RC)



Groundwater flow – flow in unsaturated soil

van Genuchten model

$$S(h_p) = S_{\mathrm{res}} + (S_{\mathrm{sat}} - S_{\mathrm{res}})\left(1 + \left(g_{\mathrm{a}}\left|h_p\right|\right)^{g_{\mathrm{n}}}\right)^{\left(\frac{1-g_{\mathrm{n}}}{g_{\mathrm{n}}}\right)}$$

$$k_{rel}(S) = (S_{\mathrm{e}})^{g_{\mathrm{l}}}\left(1 - \left(1 - S_{\mathrm{e}}^{\left(\frac{g_{\mathrm{n}}}{g_{\mathrm{n}}-1}\right)}\right)^{\left(\frac{g_{\mathrm{n}}-1}{g_{\mathrm{n}}}\right)}\right)^{2}$$

with $$S_{\mathrm{e}} = \frac{S - S_{\mathrm{res}}}{S_{\mathrm{sat}} - S_{\mathrm{res}}}$$

S_{sat},S_{res}: saturated and residual saturation

g_a, g_n and g_l: curve fitting parameters

Groundwater flow – flow in unsaturated soil

Approximate van Genuchten model

$$S(h_p) = \begin{cases} 1 & \text{if} \quad h_p \geq 0 \\ 1 + \dfrac{h_p}{|h_{ps}|} & \text{if} \quad h_{ps} < h_p < 0 \\ 0 & \text{if} \quad h_p \leq h_{ps} \end{cases}$$

Linear in Saturation

$$k_{rel}(h_p) = \begin{cases} 1 & \text{if} \quad h_p \geq 0 \\ 10^{\frac{4h_p}{|h_{pk}|}} & \text{if} \quad h_{pk} < h_p < 0 \\ 10^{-4} & \text{if} \quad h_p \leq h_{pk} \end{cases}$$

Log-linear in Permeability

h_{ps}: length of partially saturated zone under hydrostatic conditions

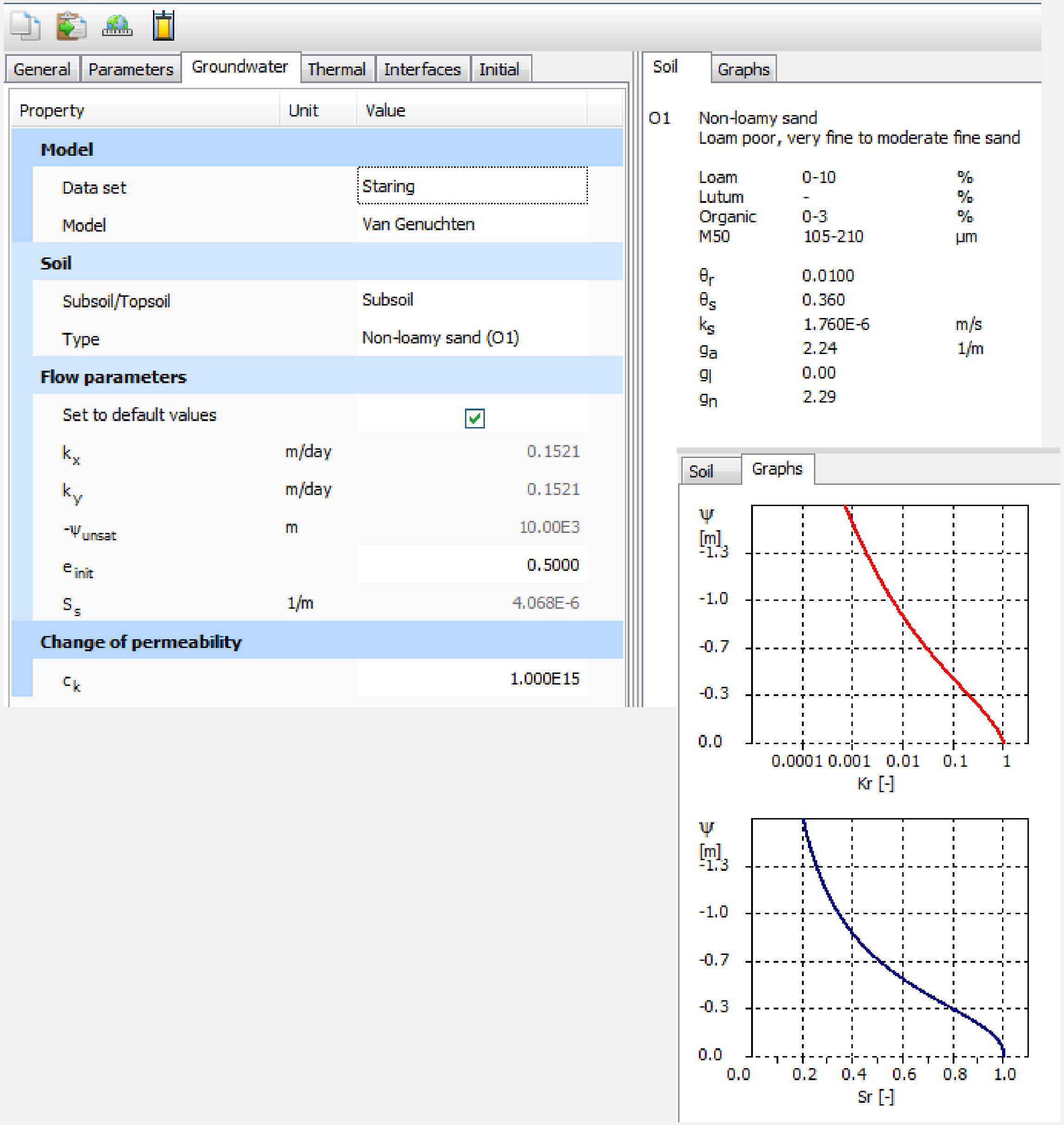
h_{pk}: pressure head at $k_{rel}=10^{-4}$

Groundwater flow - material data sets

A. Parameters:
   1. Permeabilities (k_x, k_y)
   2. Void ratio (to calculate storage)
   3. Elastic storage coefficient
      (The volume of water that a unit volume of saturated soil loses due to a unit reduction in the applied water head)
   4. Maximum unsaturated zone height

A. Soil classification
   1. Particle fractions
   2. Predefined series (Staring, Hypres, USDA) with Van Genuchten and Approx. van Genuchten parameters.
   3. User-defined

Groundwater flow - material data sets



Soil classification - Staring

Dutch soil classification system

18 upper soils data sets

18 lower soil data sets

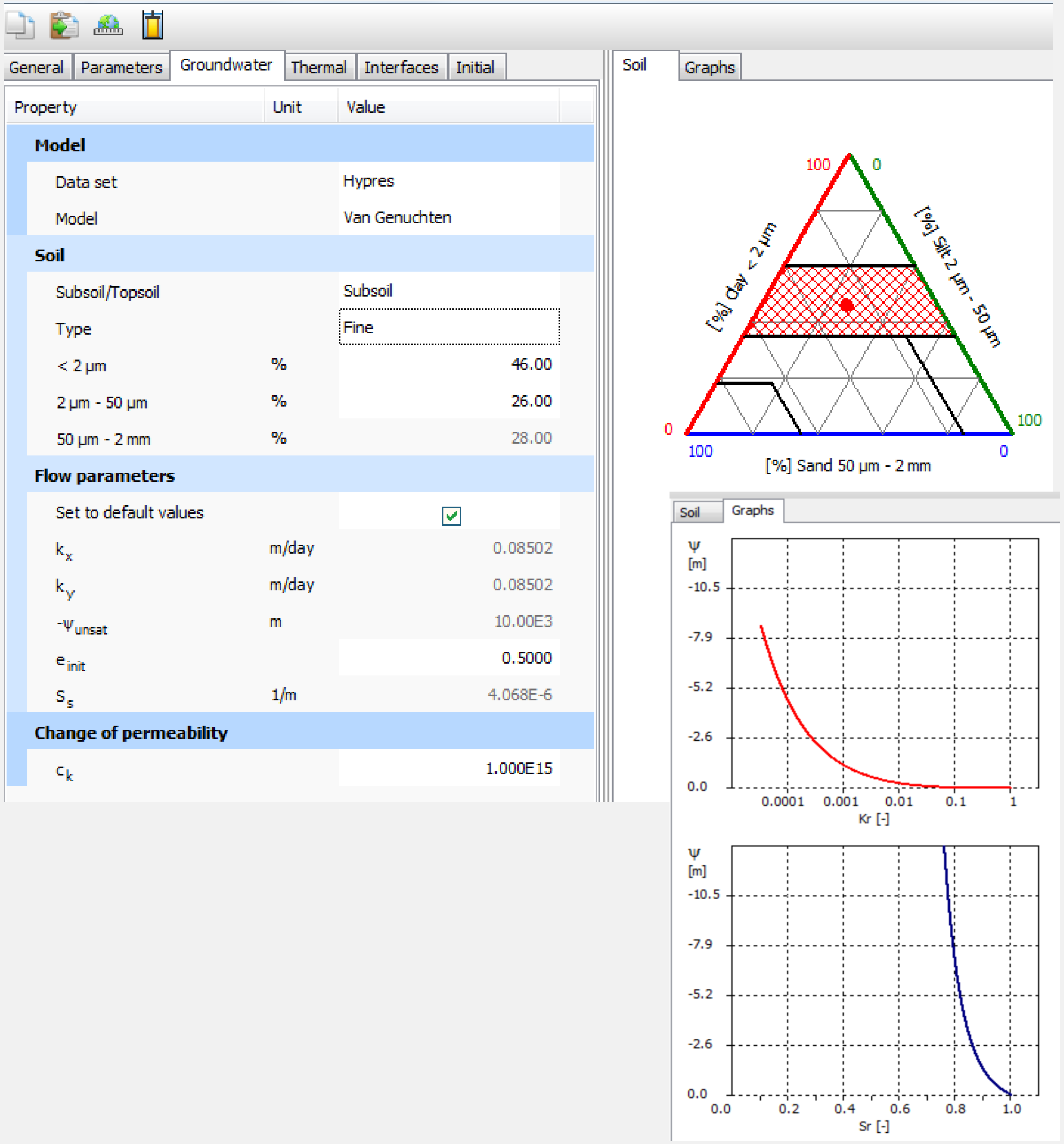
Upper soils:

< 1m below soil surface

Lower soils:

all deeper soils

Groundwater flow - material data sets



Soil classification: Hypres

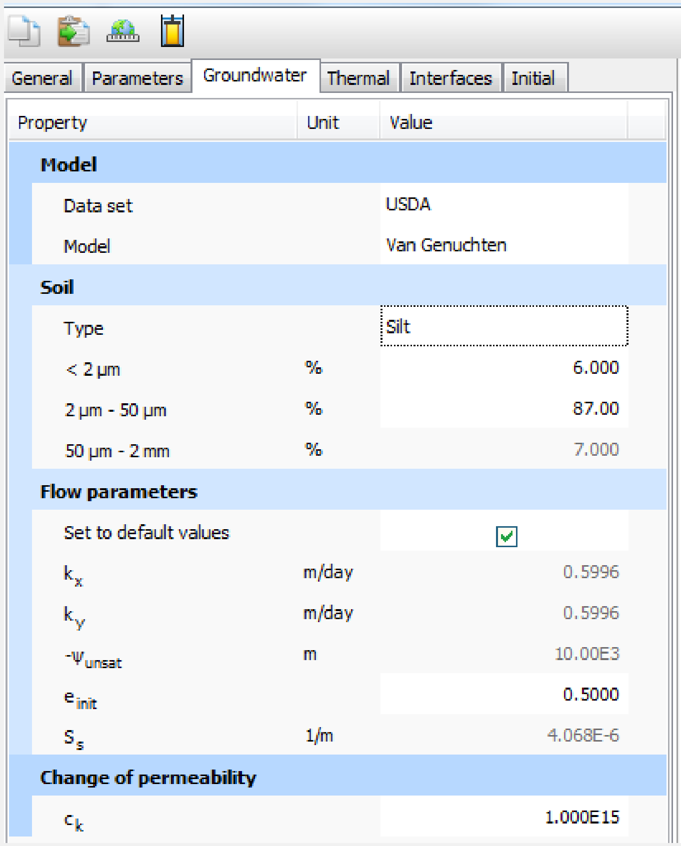
Hydraulic Properties of European Soils

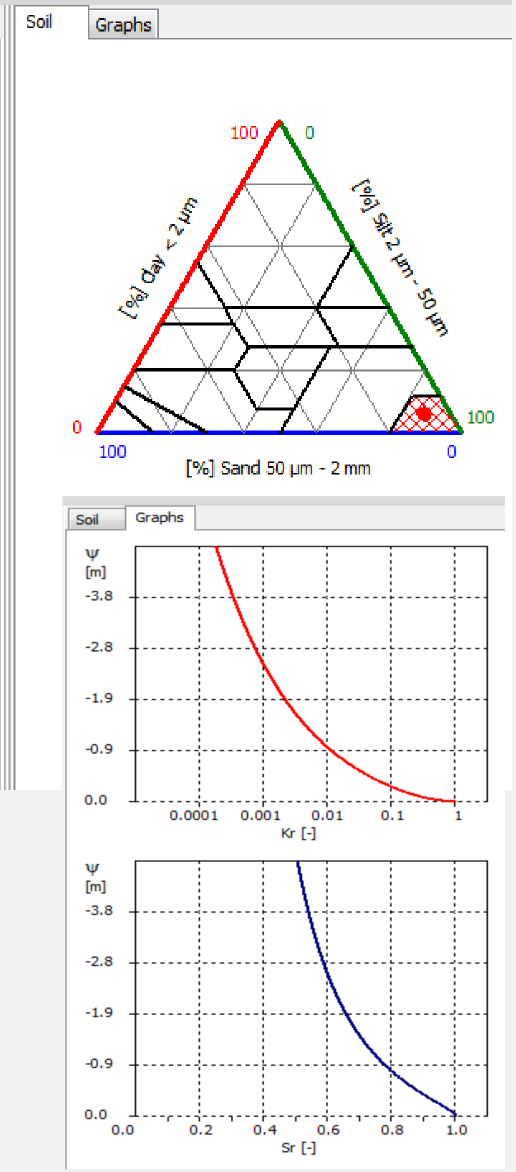
Particle distribution:

- < 2μm
- 2μm - 50μm
- 50μm – 2mm

5 upper soils data sets
5 lower soil data sets
1 organic soil data set

Groundwater flow - material data sets





Soil classification: USDA

United States Department of Agriculture

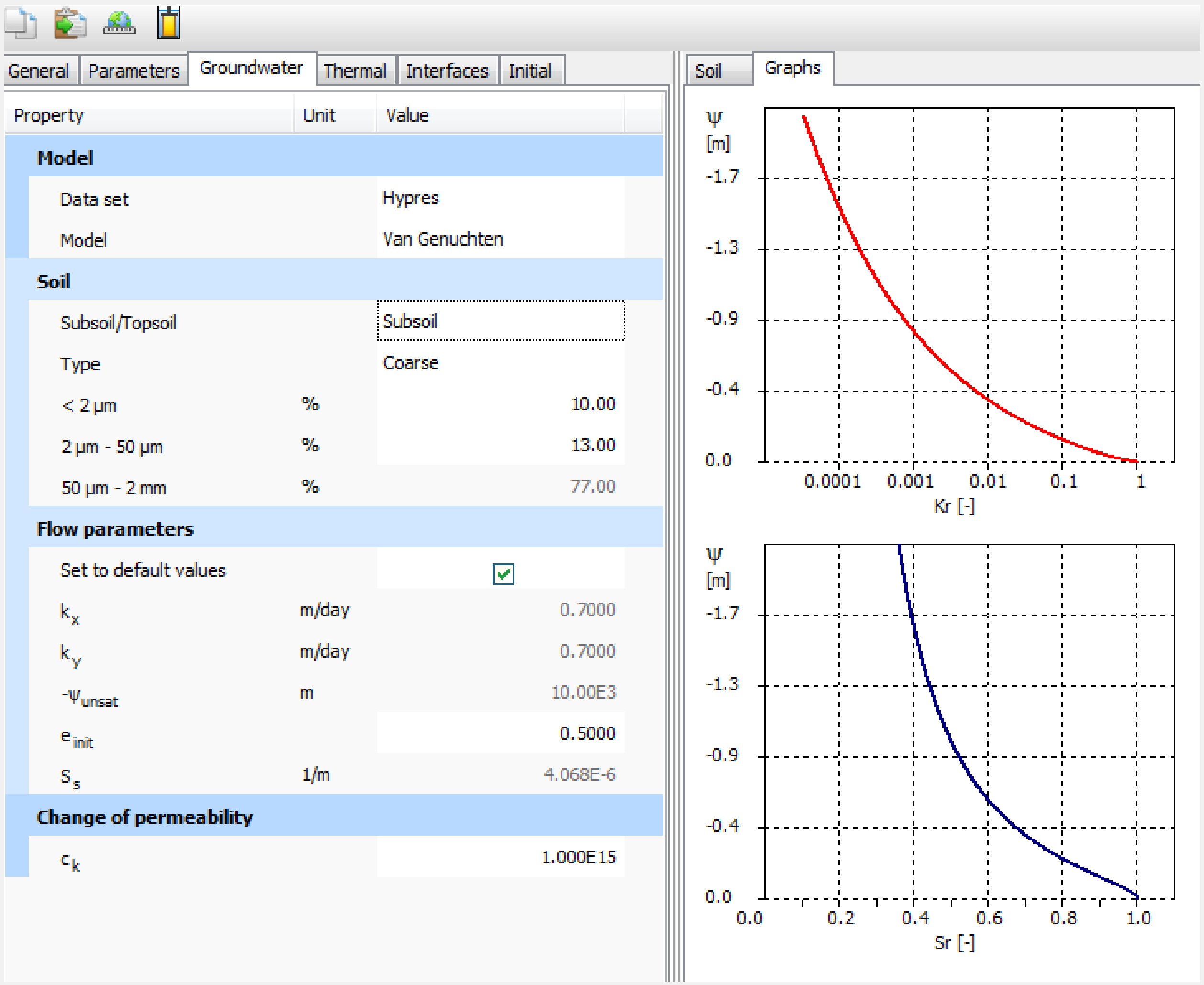
Particle distribution:

- < 2μm
- 2μm - 50μm
- 50μm – 2mm

12 soils data sets
No difference between upper and lower soils

Groundwater flow - material data sets

Soil classification and Van Genuchten parameters



Relative permeability

For SS Flow analysis, only need Relative Permeability

Degree of Saturation SWCC (Soil Water Characteristic Curve)

Both Kr and Sr functions are needed for Transient Seepage and Fully Coupled Flow Consolidation analysis

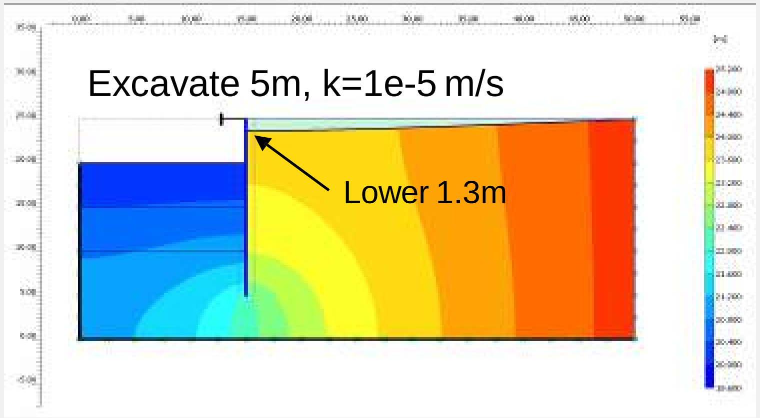
Modelling of Groundwater in Plaxis

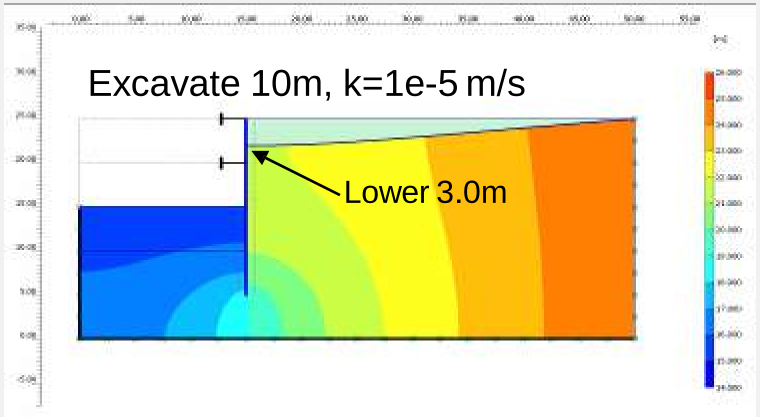
PART E: EXCAVATION EXAMPLES

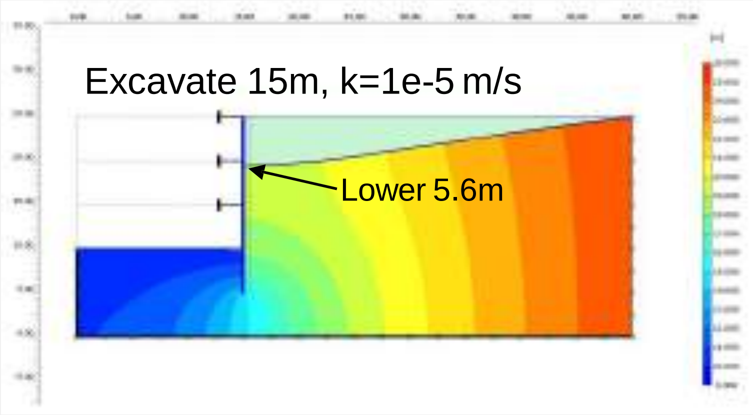
GWT LOWERING BY STEADY STATE SEEPAGE

GWT LOWERING BY TRANSIENT SEEPAGE

GWT lowering SS Seepage



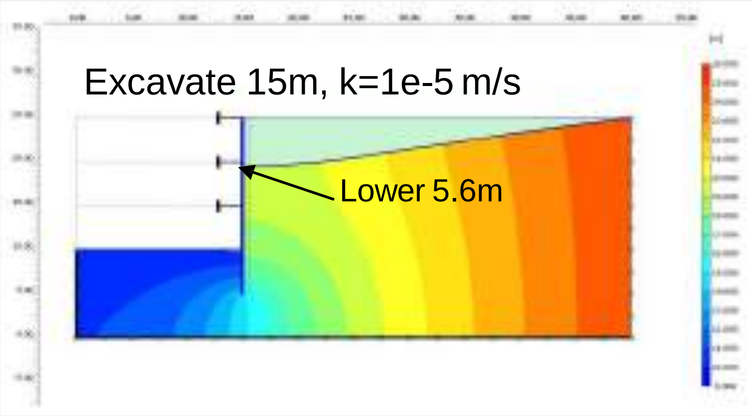


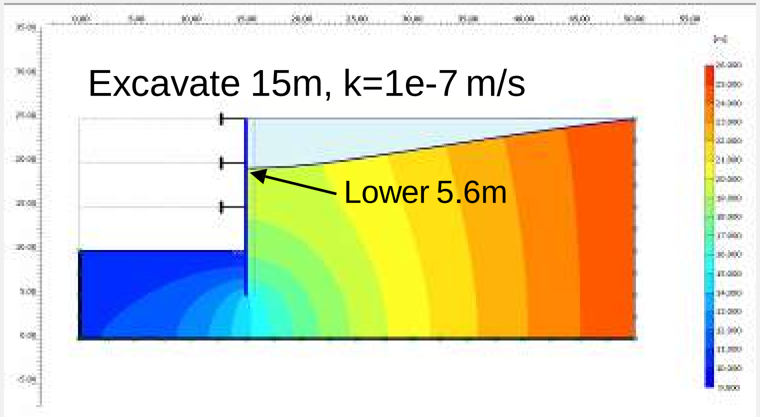


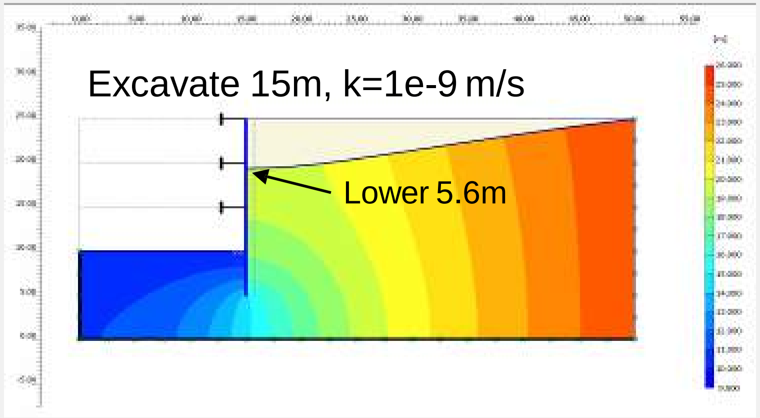
For a given permeability value and Steady-state

- GWT drawdown is nearly proportional to excavation depth.

GWT lowering SS Seepage



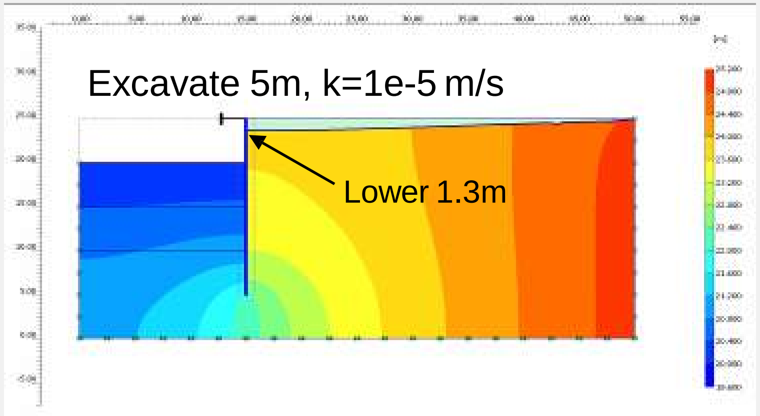


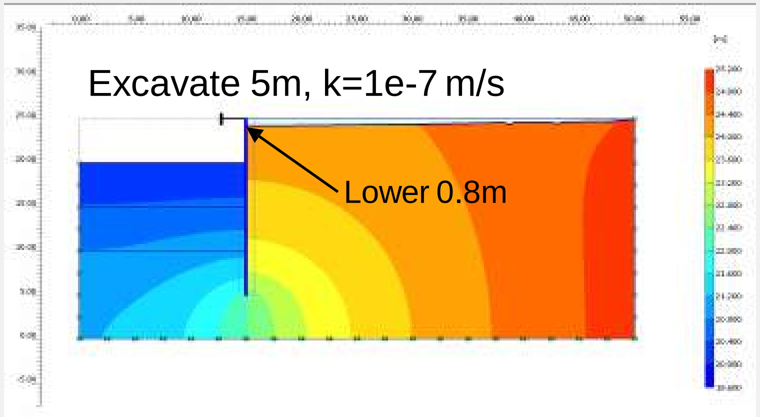


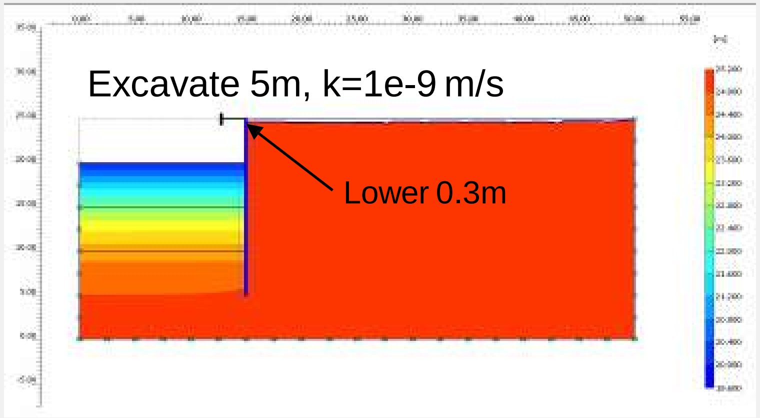
Fixed geometry configuration but variation in permeability:

- For SS case, GWT is not dependent on k.
- Pattern of GW heads is function of geometry only and soil layer arrangements

GWT and Transient Seepage





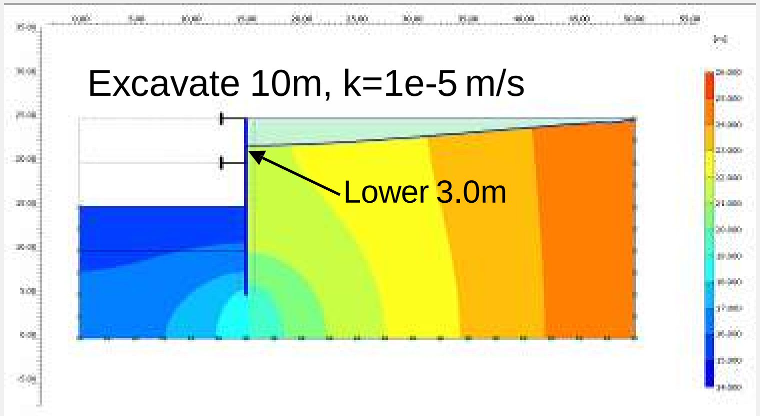


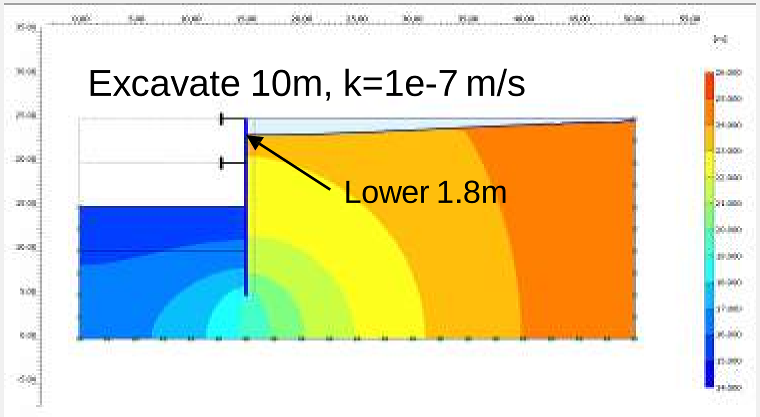
Excavate 5m in 30 days.

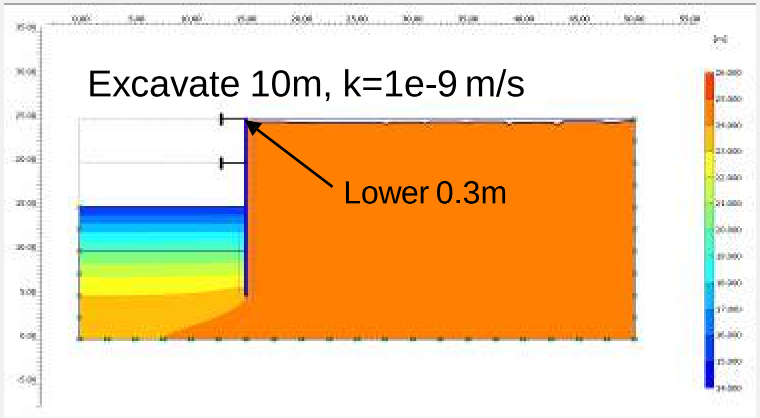
Sands, k=1e-5 m/s is like SS case

Clays, k=1e-9 m/s very little GWT lowered

GWT and Transient Seepage





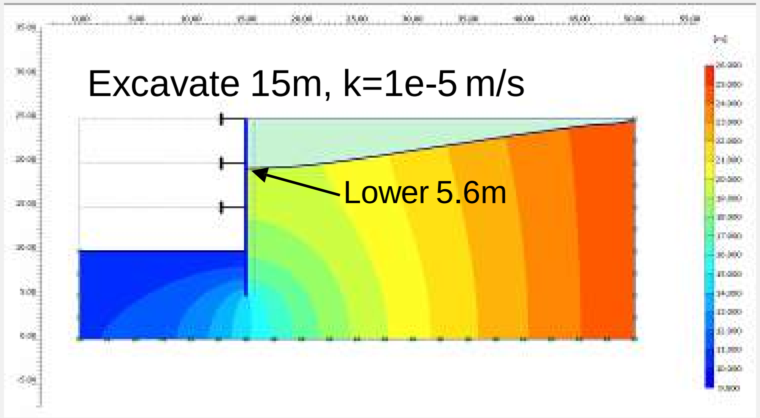


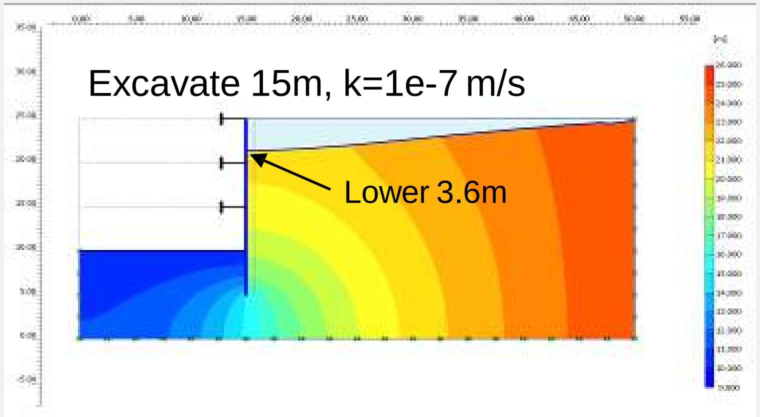
Excavate next 5m in 30 days.

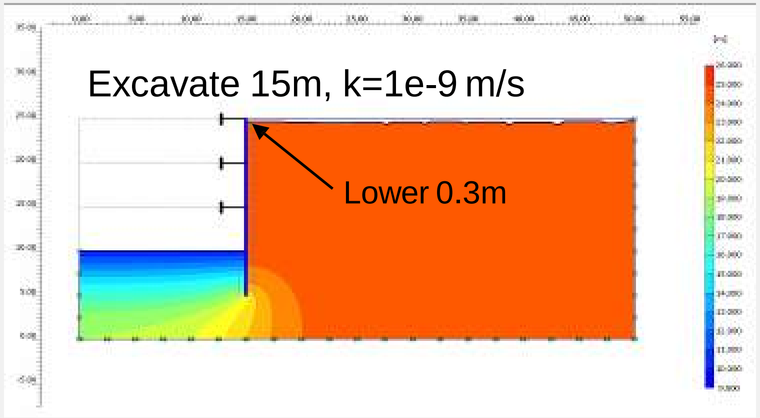
Sands, k=1e-5 m/s is like SS case

Clays, k=1e-9 m/s very little GWT lowered

GWT and Transient Seepage







Excavate next 5m in 30 days.

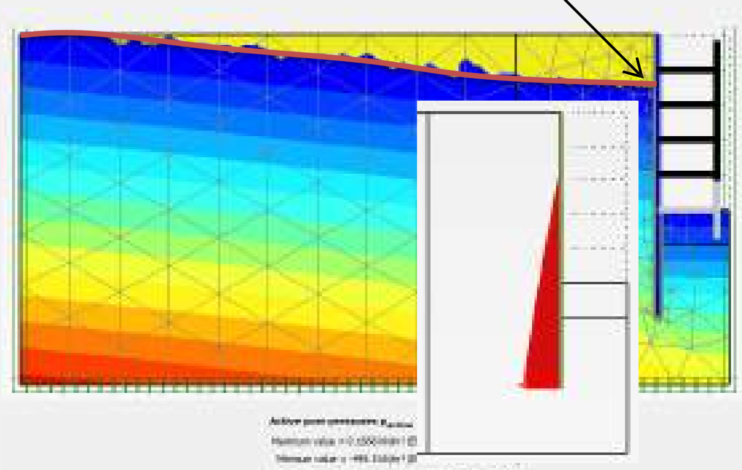
Sands, k=1e-5 m/s is like SS case

Clays, k=1e-9 m/s very little GWT lowered

GWT in Fully Coupled Flow Models

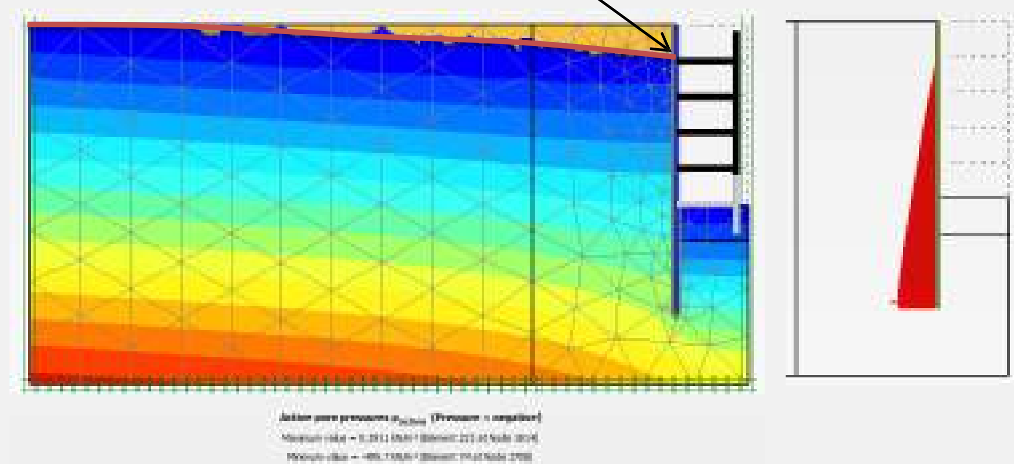
Excavate 25m, k=1e-5 m/s

Lower 8.0m



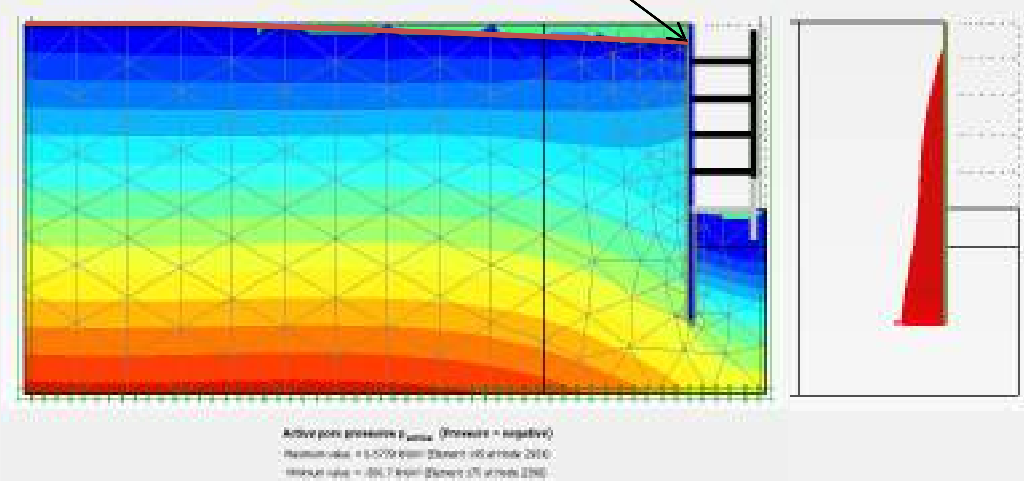
Excavate 25m, k=1e-7 m/s

Lower 4.7m



Excavate 25m, k=1e-9 m/s

Lower 2.0m



Excavate 25m in 150 days.

Sands, k=1e-5 m/s is like SS case

Clays, k=1e-9 m/s very little GWT lowered

END

QUESTIONS?

SCIENCE ▪ PASSION ▪ TECHNOLOGY

CG11

UNDRAINED SOIL BEHAVIOUR AND CONSOLIDATION

Helmut F. Schweiger [1] **dan Indra Noer Hamdhan** [2]

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Pelatihan software Plaxis di Tembagapura (In-house training) PT. Freeport Indonesia/ Papua, 22-30 September 2018

CONTENTS

- **Drained analysis appropriate when**
  - permeability is high
  - rate of loading is low
  - short term behaviour is not of interest for problem considered
- **Undrained analysis appropriate when**
  - permeability is low and rate of loading is high
  - short term behaviour has to be assessed
- **Implications of undrained soil behaviour**
  - excess pore pressures are generated

$$\Delta u \neq 0, \Delta\sigma \neq \Delta\sigma'$$

  - no volume change

in fact small volumetric strains develop because a finite (but high) bulk modulus of water is introduced in the finite element formulation

  - predicted undrained shear strength depends on soil model used
  - assumption of dilatancy angle has serious effects on results

PORE PRESSURE PARAMETERS A AND B

Skempton 1954: $$\Delta p_w = B\left[\Delta\sigma_3 + A\left(\Delta\sigma_1 - \Delta\sigma_3\right)\right]$$

- fully saturated soil
- no inflow / outflow of pore water
- bulk modulus of soil *grains* >>>
- isotropic linear elastic material behaviour (Hooke´s law)

$$\Delta\varepsilon_{vol,\ skeleton} = \Delta\varepsilon_{vol,\ pore\ water}$$

$$\Delta\varepsilon_{vol,\ skeleton} = \frac{\Delta p'}{K'}$$

$$K' = \frac{E'}{3\left(1-2\nu'\right)}$$

$$\Delta\varepsilon_{vol,\ pore\ water} = \frac{n\,\Delta p_w}{K_w}$$

PORE PRESSURE PARAMETERS A AND B

Assuming triaxial compression:

$$\Delta\sigma_1 ; \quad \Delta\sigma_2 = \Delta\sigma_3$$

$$\Delta p_w = \frac{\Delta\sigma_1 + 2\Delta\sigma_3 - 3\Delta p_w}{3K'} \cdot \frac{K_w}{n}$$

leading to
$$\Delta p_w = \frac{1}{1+\dfrac{nK'}{K_w}} \left[\Delta\sigma_3 + \frac{1}{3}\left(\Delta\sigma_1 - \Delta\sigma_3\right) \right]$$

$$\Delta p_w = B \left[\Delta\sigma_3 + A\left(\Delta\sigma_1 - \Delta\sigma_3\right) \right]$$

with
$$B = \frac{1}{1+\dfrac{nK'}{K_w}} \qquad A = \frac{1}{3}$$

PORE PRESSURE PARAMETERS A AND B

Notes on parameters A and B:

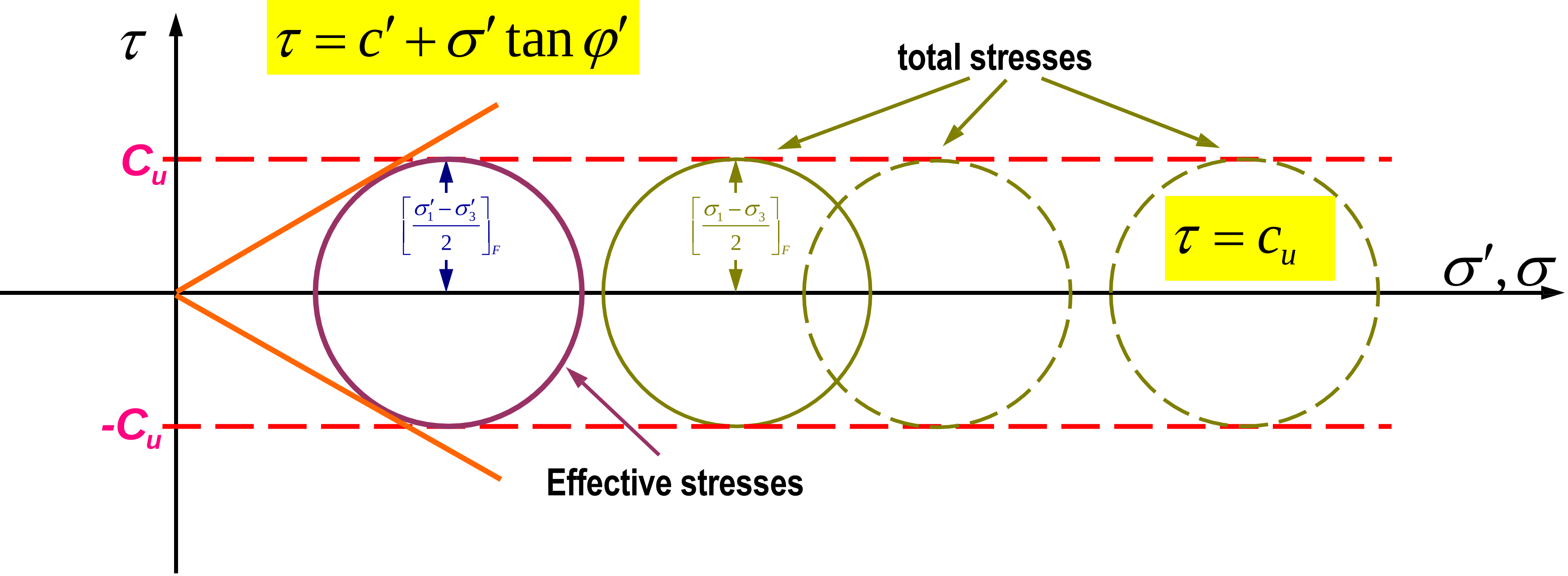
- for K_w large compared to K´, parameter B ~ 1.0 (corresponds to $\Delta p_w = \Delta p$ > $\Delta p' = 0$)

- small amount of drapped air reduces parameter B significantly

- parameter A depends on stress path, even for elastic material behaviour

- parameter A cannot be determined a priori for complex elastic-plastic constitutive models > is a result of the model behaviour for the stress path followed

UNDRAINED SHEAR STRENGTH



- Soil behaves as if it was cohesive
- c_u $(= s_u)$: undrained shear strength
- c_u only changes if drainage occurs (no change if undrained conditions prevail)

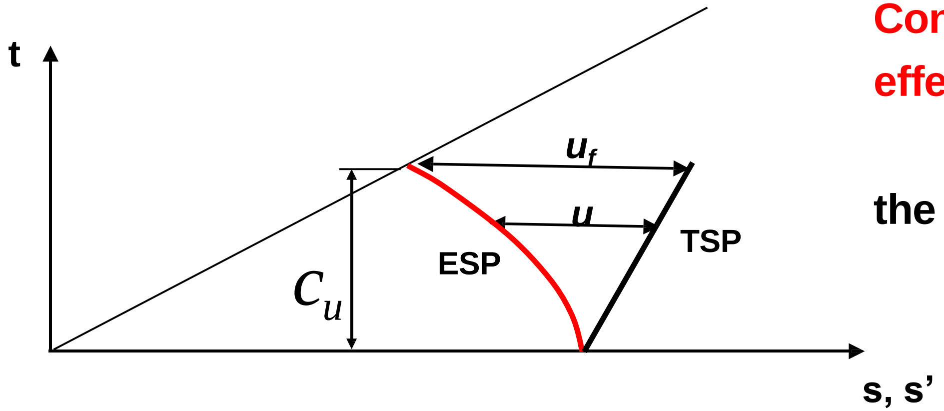
UNDRAINED BEHAVIOUR WITH PLAXIS

UNDRAINED A (analysis in terms of *effective* stresses):
effective strength parameters c´, φ´, ψ´
effective stiffness parameters E_{50}´, ν´

UNDRAINED B (analysis in terms of *effective* stresses):
undrained strength parameters $c = c_u$, $\varphi = 0$, $\psi = 0$
effective stiffness parameters E_{50}´, ν´

UNDRAINED C (analysis in terms of *total* stresses):
total strength parameters $c = c_u$, $\varphi = 0$, $\psi = 0$ **undrained** stiffness parameters E_u, $\nu_u = 0.495$

UNDRAINED A



Constitutive equations in terms of effective stresses: $\Delta\sigma' = D'\Delta\varepsilon$

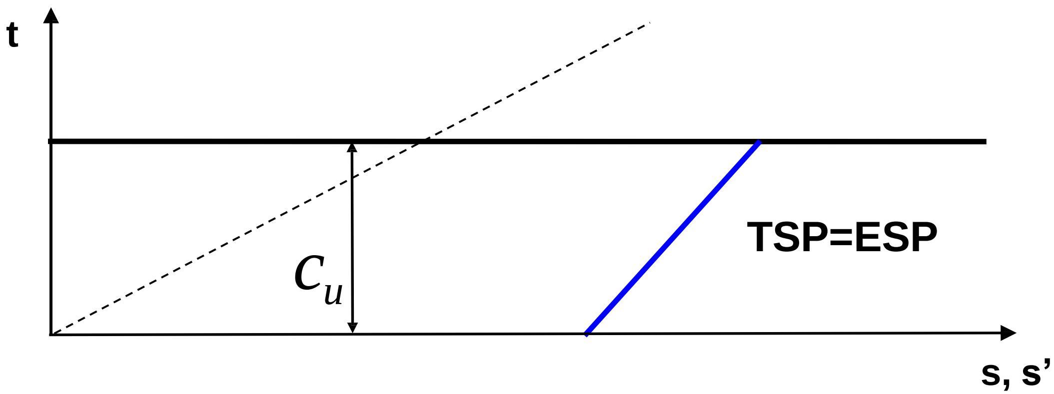
the total stiffness matrix is computed as:

$$D = D' + D_f$$

- **single set of parameters in terms of effective stress (undrained, drained, consolidation analysis consistent)**
- **realistic prediction of pore pressures (if model is appropriate)**
- **the undrained analysis can be followed by a consolidation analysis (correct pore pressures, correct drained parameters and c_u is updated automatically)**
- **c_u is a consequence of the model, not an input parameter!!**

Available for all models (including user defined models)!

UNDRAINED C

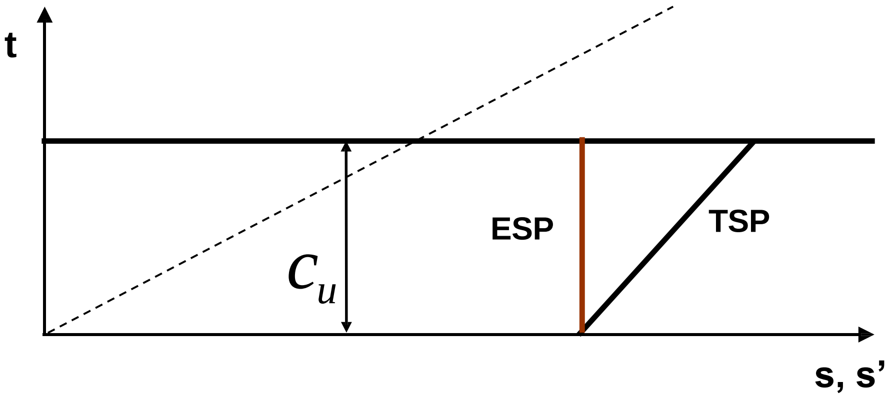


Constitutive equations in terms of total stresses: $\Delta\sigma = D\,\Delta\varepsilon$

- **parameters in terms of total stress**
- **no prediction of pore pressures (only total stresses are obtained)**
- **the undrained analysis can not be followed by a consolidation analysis**
- **c_u is an input parameter!!**

Available for Linear elastic, Mohr Coulomb, NGI-ADP models

UNDRAINED B



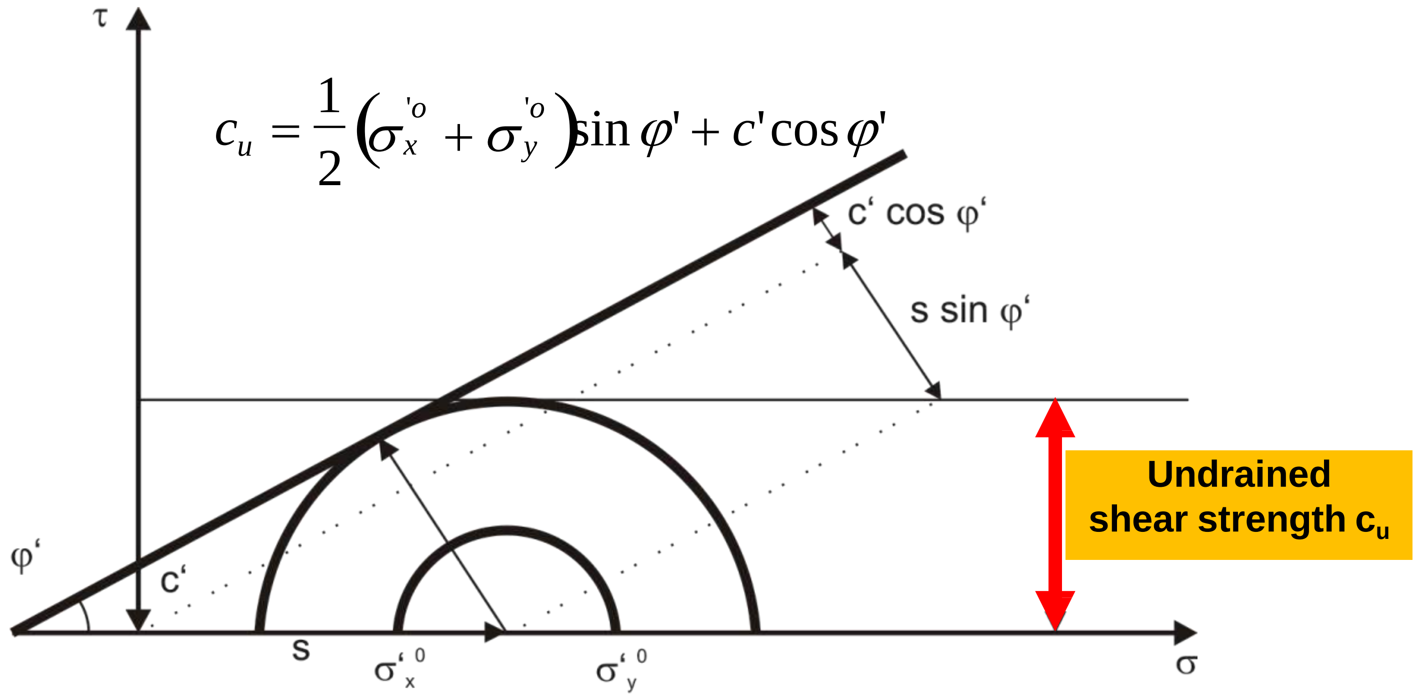
- **parameters in terms of total stress and effective stress**
- **prediction of pore pressures (but generally unrealistic)**
- **the undrained analysis should not be followed by a consolidation analysis (pore pressures unrealistic and c_u is not updated automatically). Note:** "Model change" to effective parameters also not recommended because of unrealistic pore pressure distribution.
- **c_u is an input parameter!!**

Available for Mohr Coulomb, HSM, HSSM, NGI-ADP models

UNDRAINED STRENGTH FROM MOHR-COULOMB

Consider fully undrained isotropic elastic behaviour (Mohr Coulomb in elastic range)

$\Delta p_w = \Delta p > \Delta p' = 0 \rightarrow$ centre of Mohr Circle remains at the same position



Mohr Circle for evaluating undrained shear strength (plane strain)

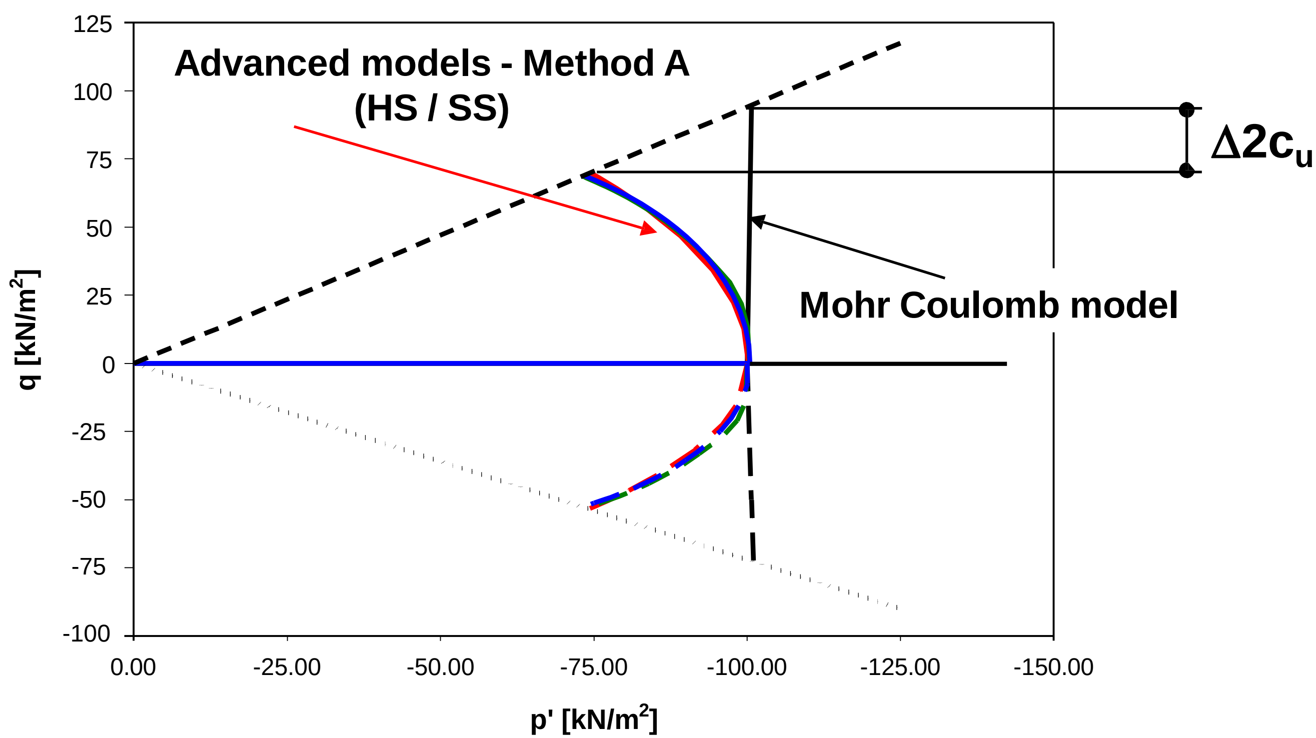
Parameters used for Hardening Soil model (HS)

Parameter		Meaning	Value
γ	[kN/m³]	Unit weight (unsaturated)	16
γ_r	[kN/m³]	Unit weight (saturated)	16
φ'	[°]	Friction angle (Mohr-Coulomb)	24
c'	[kPa]	Cohesion (Mohr-Coulomb)	0
ψ	[°]	Angle of dilatancy	0
ν_{ur}	[-]	Poisson's ratio unloading-reloading	0.20
E_{50}^{ref}	[kPa]	Secant modulus for primary triaxial loading	2 000
E_{oed}^{ref}	[kPa]	Tangent modulus for oedometric loading	1 000
E_{ur}^{ref}	[kPa]	Secant modulus for un- and reloading	7 500
m	[-]	Exponent of the Ohde/Janbu law	1.0
p^{ref}	[kPa]	Reference stress for the stiffness parameters	100
K_0^{nc}	[-]	Coefficient of earth pressure at rest (NC)	1-sin(φ')
R_f	[-]	Failure ratio	0.90
$\sigma_{Tension}$	[kPa]	Tensile strength	0

Parameters for Soft Soil model and Mohr Coulomb model accordingly

Undrained triaxial test (numerical simulation - UNDRAINED A)

Preconsolidation to 100 kPa > compression / extension



Triaxial compression:

$c_u/\sigma_v' = 0.47$ (MC)

$c_u/\sigma_v' = 0.35$ (HS)

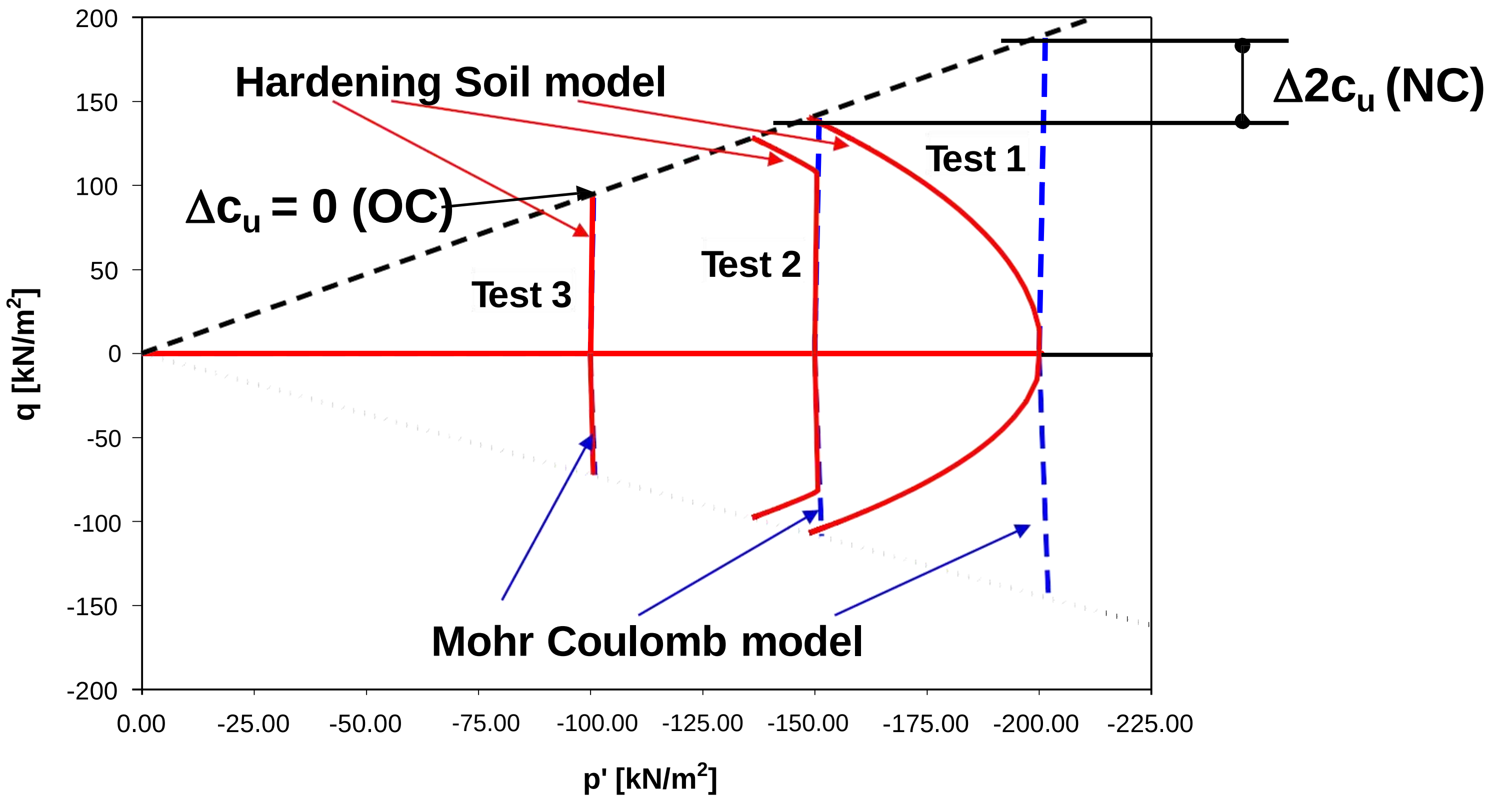
Triaxial extension:

$c_u/\sigma_v' = 0.36$ (MC)

$c_u/\sigma_v' = 0.26$ (HS)

Undrained triaxial test (numerical simulation - UNDRAINED A)

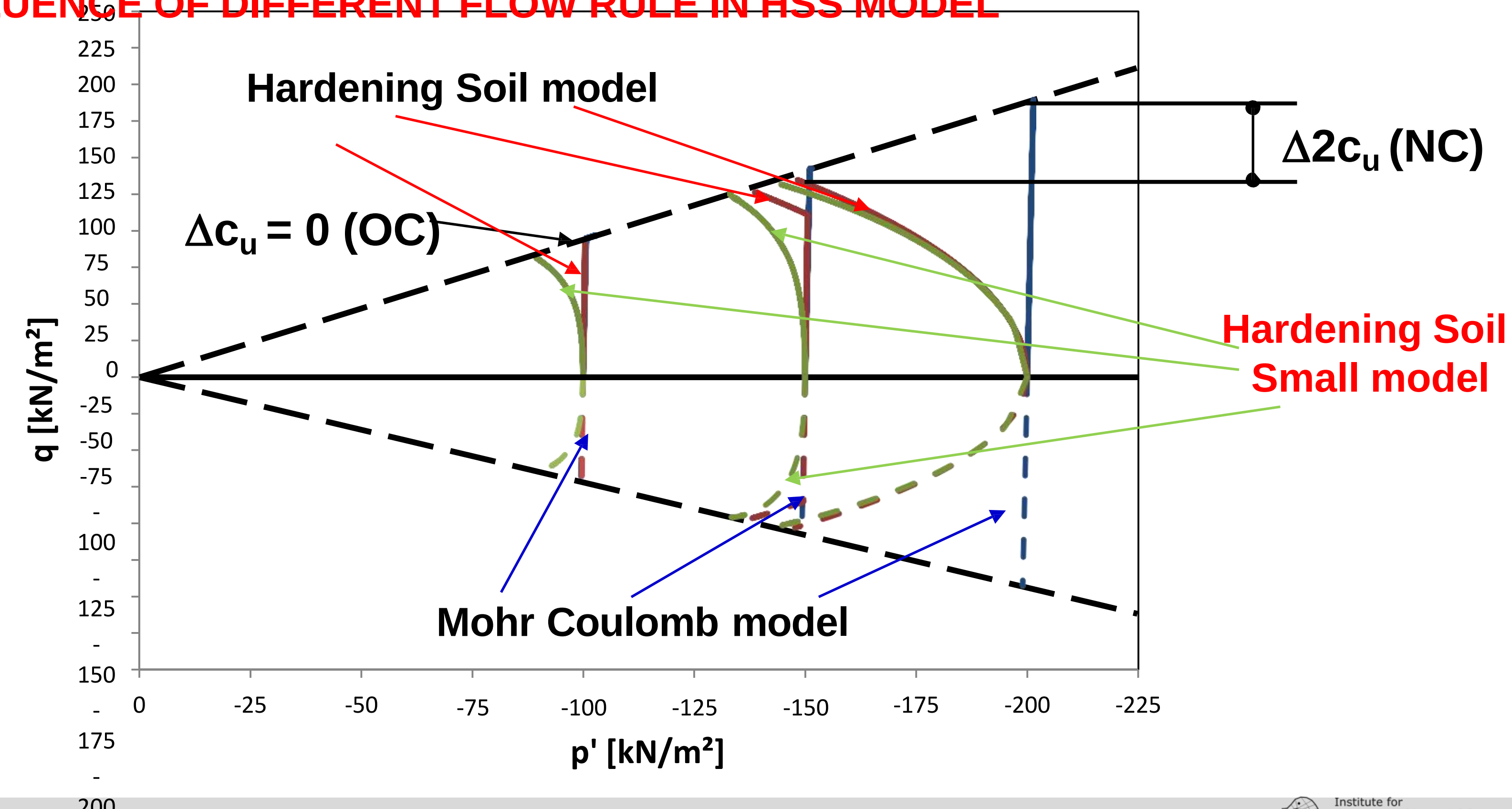
Test 1: Preconsolidation to 200 kPa > compression / extension
Test 2: Preconsolidation to 200 kPa > isotropic unloading to 150 kPa > compression / extension Test 3: Preconsolidation to 200 kPa > isotropic unloading to 100 kPa > compression / extension



Undrained triaxial test (numerical simulation - UNDRAINED A)

Test 1: Preconsolidation to 200 kPa > compression / extension
Test 2: Preconsolidation to 200 kPa > isotropic unloading to 150 kPa > compression / extension Test 3: Preconsolidation to 200 kPa > isotropic unloading to 100 kPa > compression / extension

INFLUENCE OF DIFFERENT FLOW RULE IN HSS MODEL

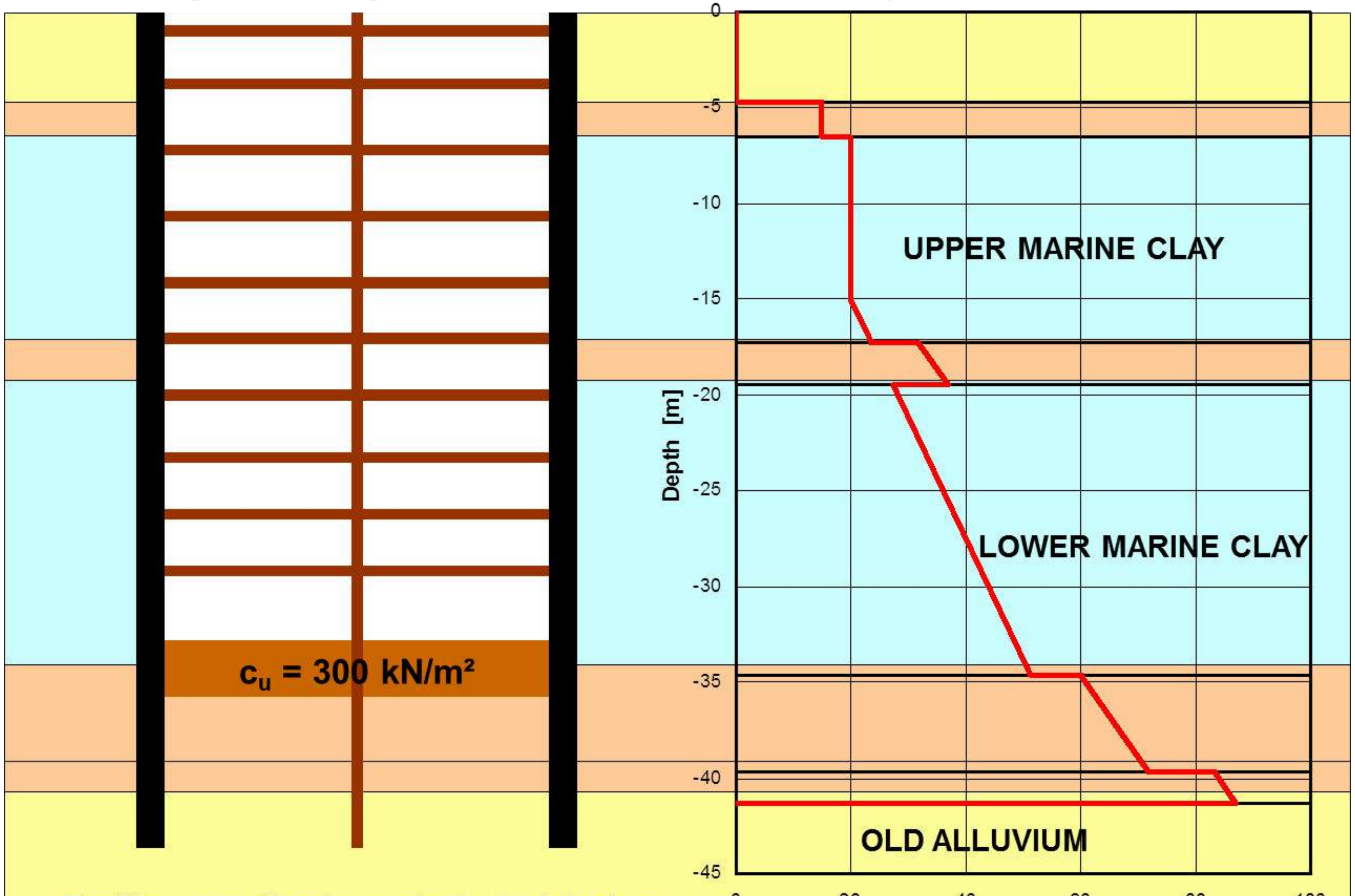


INFLUENCE INPUT PARAMETERS - HS / HSS MODELS

	E_{50}^{ref}	E_{ur}^{ref}	E_{oed}^{ref}	φ'	ψ	c'	$\gamma_{0.7}$	G_0	E_{50}/E_{oed}
HSS_a	2 000	10 000	1 000	24	0	0	0.0003	15 000	2.00
HSS_f	3 000	10 000	1 000	24	0	0	0.0003	15 000	3.00
HSS_g	3 500	10 000	1 000	24	0	0	0.0003	15 000	3.50
HSS_h	4 000	10 000	1 000	24	0	0	0.0003	15 000	4.00
HSS_i	5 000	10 000	1 000	24	0	0	0.0003	15 000	5.00
HS_a	2 000	10 000	1 000	24	0	0			2.00

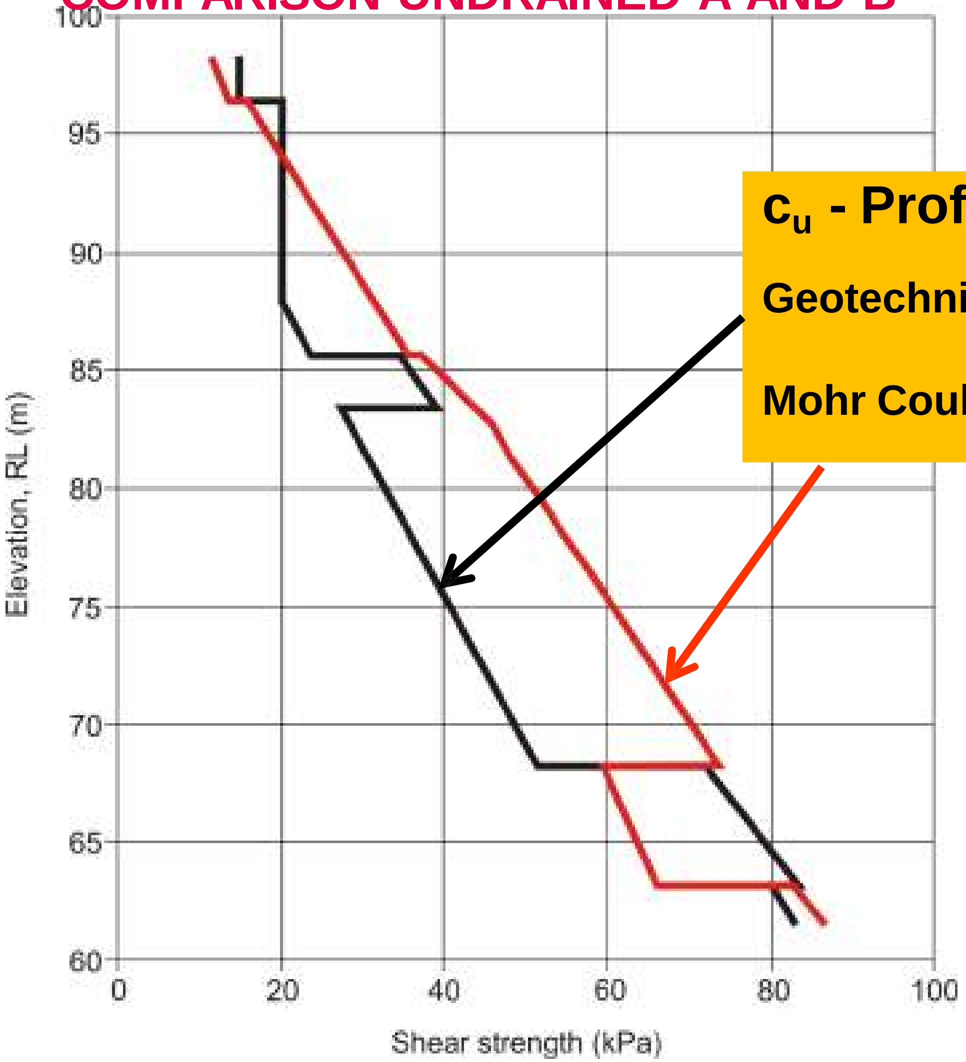
COMPARISON UNDRAINED A AND B

Practical example: deep excavation in soft clay



c_u-Profile according to geotechnical design parameter table

COMPARISON UNDRAINED A AND B



c_u - Profile:

Geotechnical design parameter table

Mohr Coulomb model (UNDRAINED A)

UNDRAINED A:

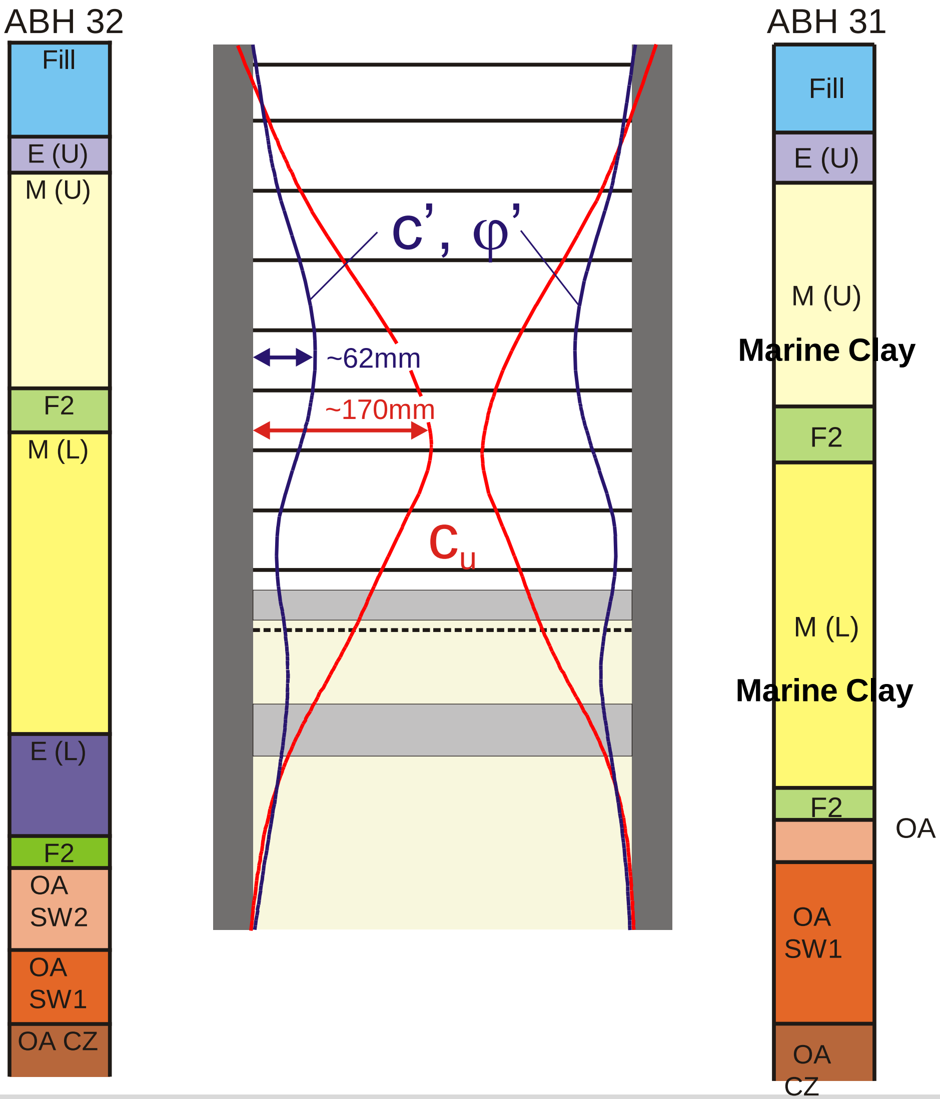
Mohr-Coulomb effective (φ' and c')

UNDRAINED B:

Mohr-Coulomb (c_u)

> significant difference in undrained shear strength

COMPARISON UNDRAINED A AND B



Practical example: deep excavation in soft clay

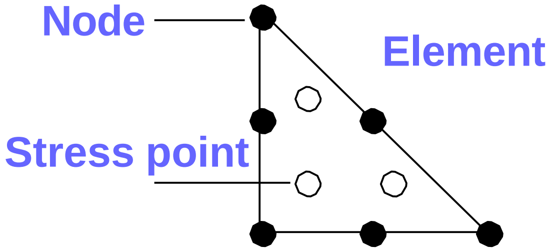
Method A with **Mohr-Coulomb (!)** overestimates undrained shear strength for normally consolidated soft soils

- difference in calculated horizontal displacements significant
- bending moments differ by a factor of 2
- minor differences in earth

SUMMARY - UNDRAINED BEHAVIOUR

- **FE analysis of undrained conditions should be performed in effective stresses with effective stiffness and strength parameters (Method A)**
- **Method A must be used:**
  - if consolidation/long term analysis is required
  - to exploit all features of advanced soil models
- **Undrained shear strength is a result of the constitutive model**

  (check what c_u model predicts; be careful when soil parameters are part of the contract)
- **Dilatancy angle has to be zero**
- **Methods (B) and C provide alternative ways to analyze undrained problems but:**
  - the constitutive model dos not generally represent the true soil behaviour (before failure)
  - potentially useful for stability problems in undrained conditions (specification of undrained shear strength is straightforward)

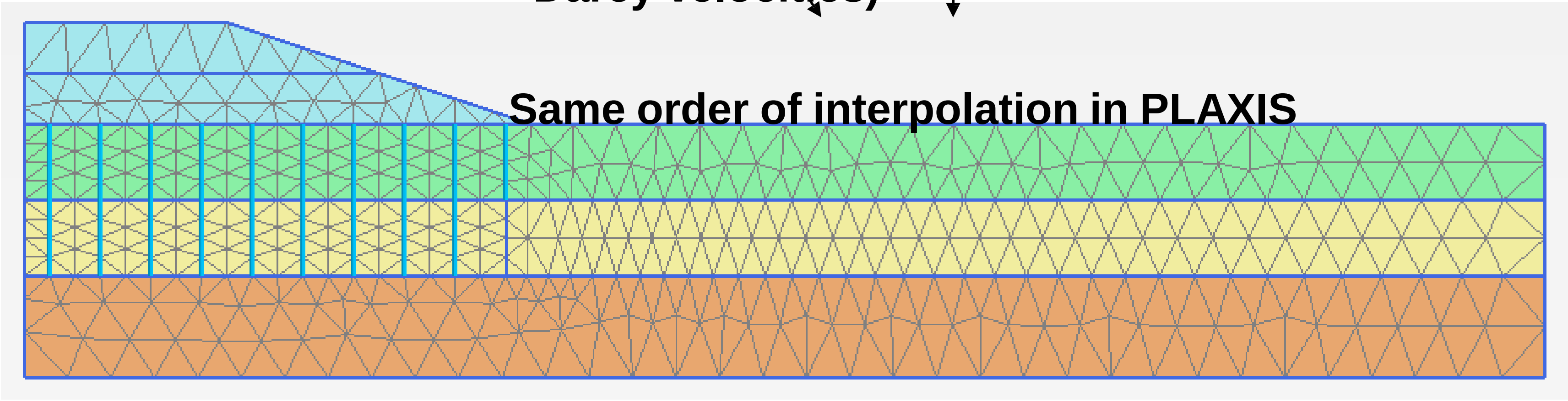


Mesh:

Elements: Interpolation of primary variables

Nodes: Primary variables (displacements, pore pressures)

Stress points: Derived variables (strains, stresses, Darcy velocities)



Same order of interpolation in PLAXIS

PERMEABILITY

- **PLAXIS allows consideration of change of permeability with void ratio**

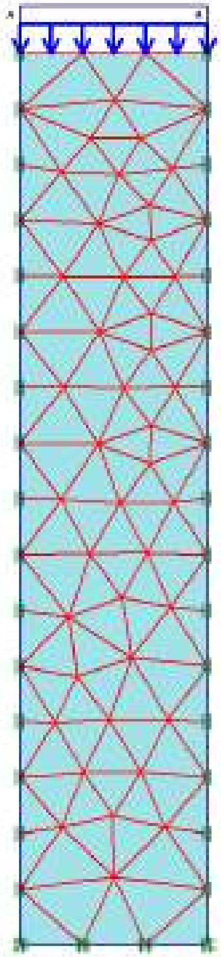
$$\log\left[\frac{k}{k_0}\right] = \frac{\Delta e}{c_k}$$

Default value for c_k is 10^{15}

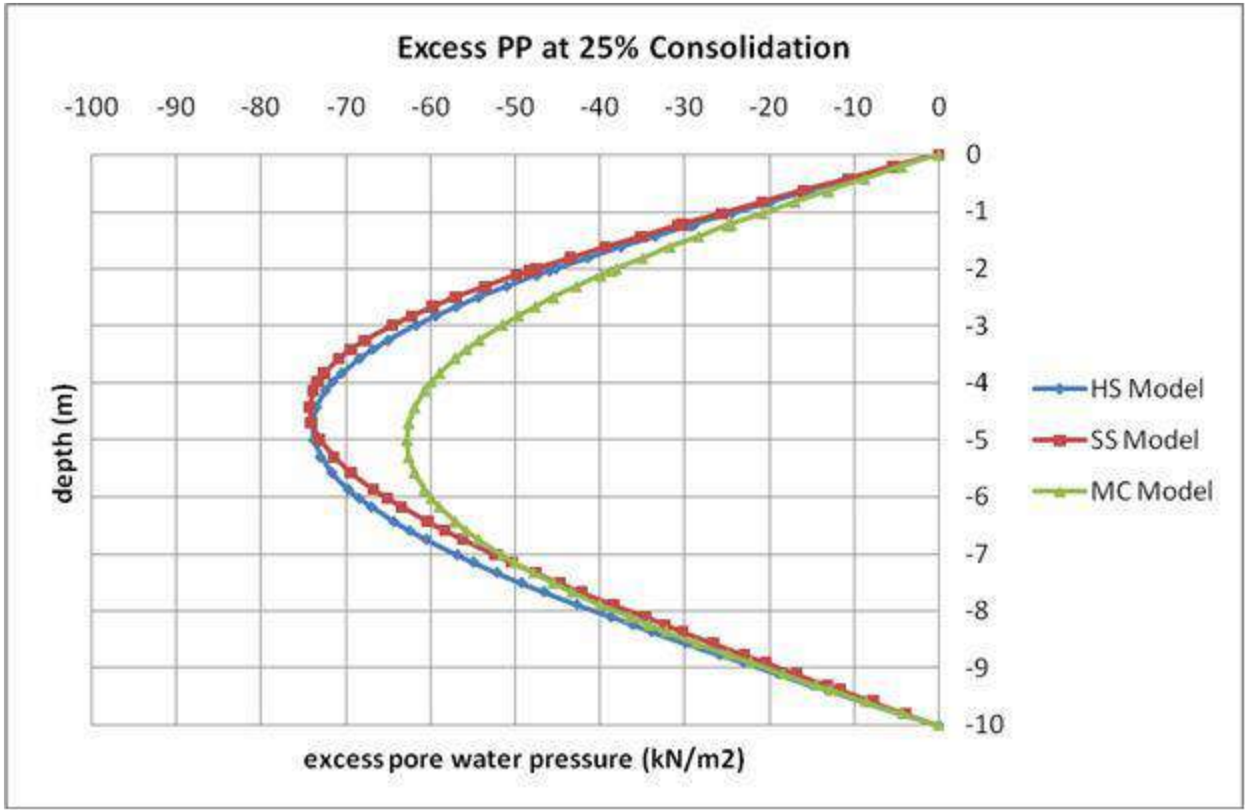
- **There may be large contrasts of permeability between different materials in the same problem**
  - **Too much permeability contrast may cause numerical difficulties**
  - **The ratio between the highest and lowest permeability value should not exceed 10^5**
  - **To simulate an almost impermeable material (e.g. concrete), a value lower by a factor 1000 is usually sufficient**

1D CONSOLIDATION - NUMERICAL SIMULATION

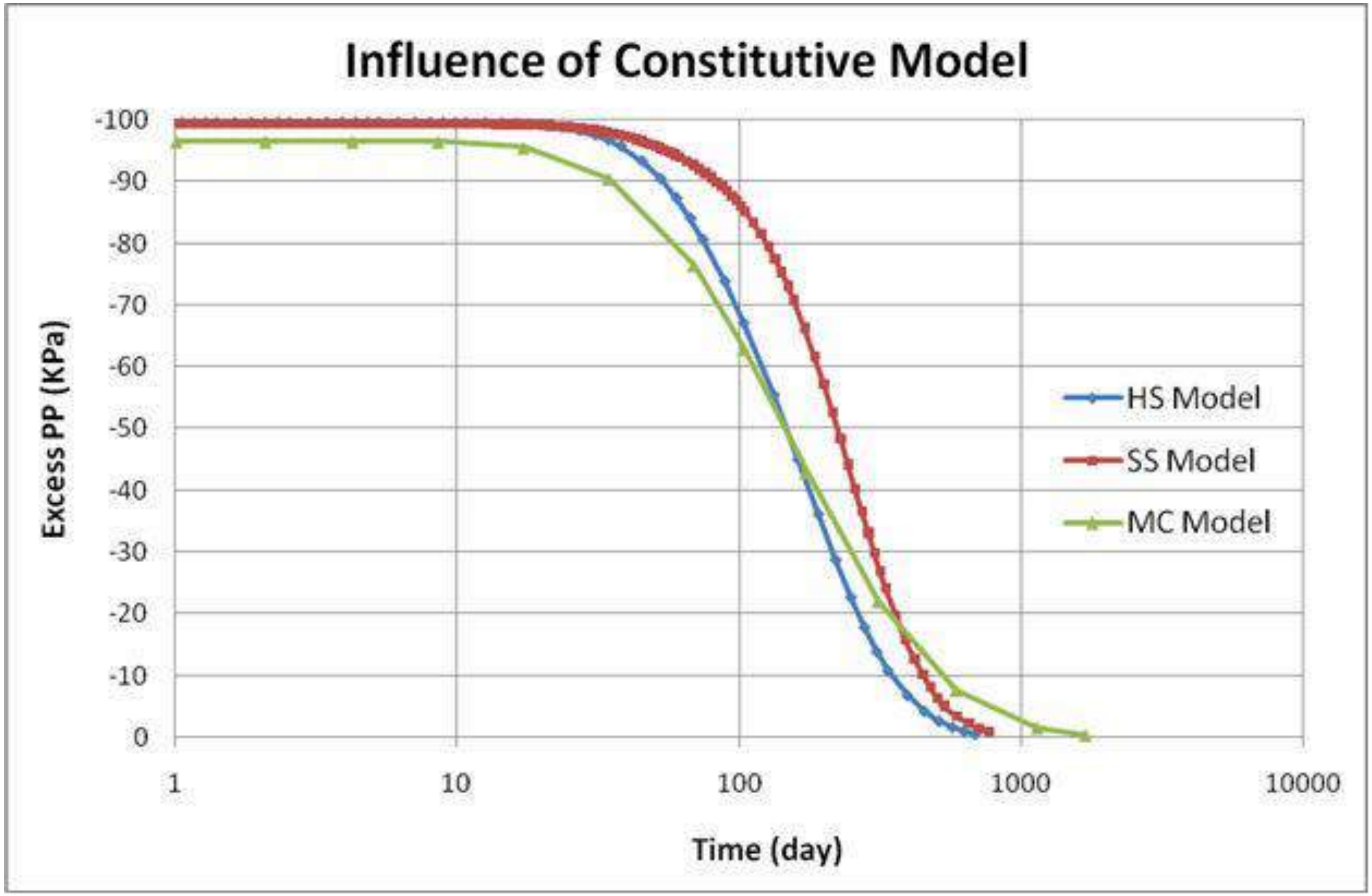
drainage at top and bottom



Isochrones at 25% consolidation:
not symmetric for advanced models

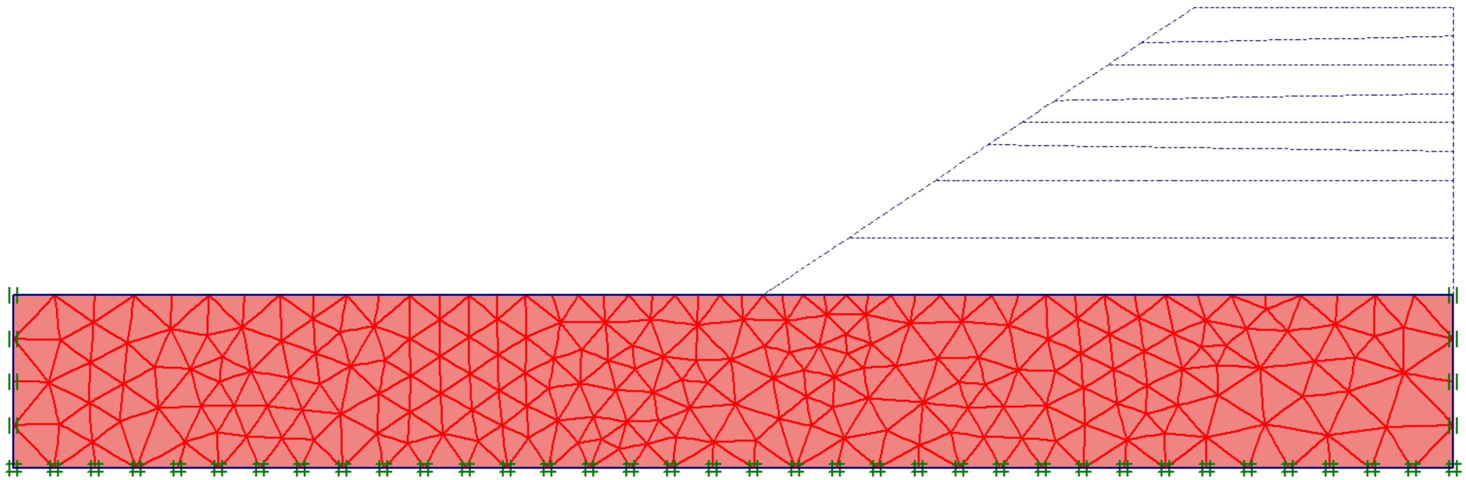


1D CONSOLIDATION - NUMERICAL SIMULATION



EXAMPLE - EMBANKMENT CONSTRUCTION

Influence of consolidation on stability
(only possible with UNDRAINED A and advanced model)



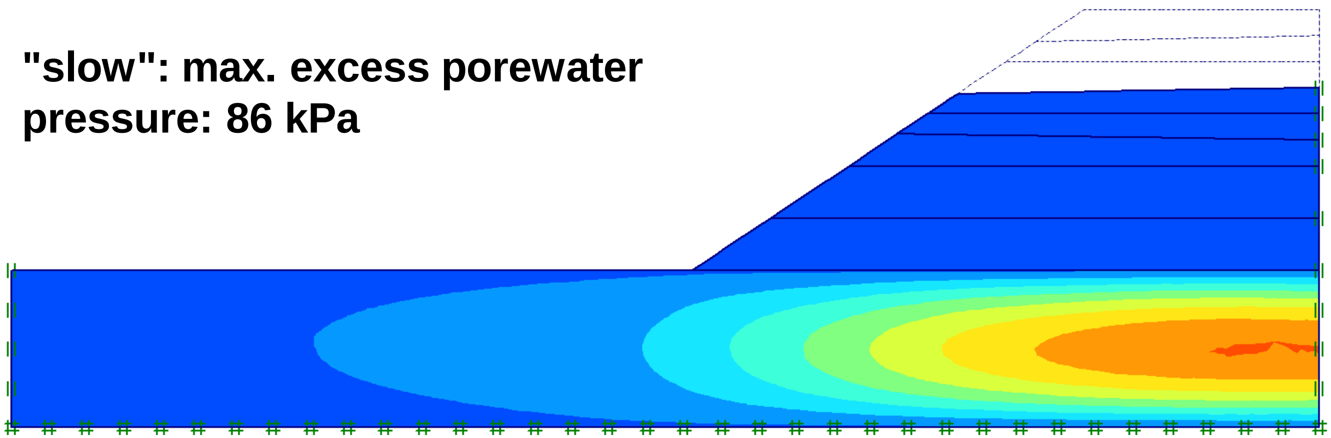
influence of construction speed is investigated

"fast" construction: 2 days of consolidation per placement of 1 m embankment

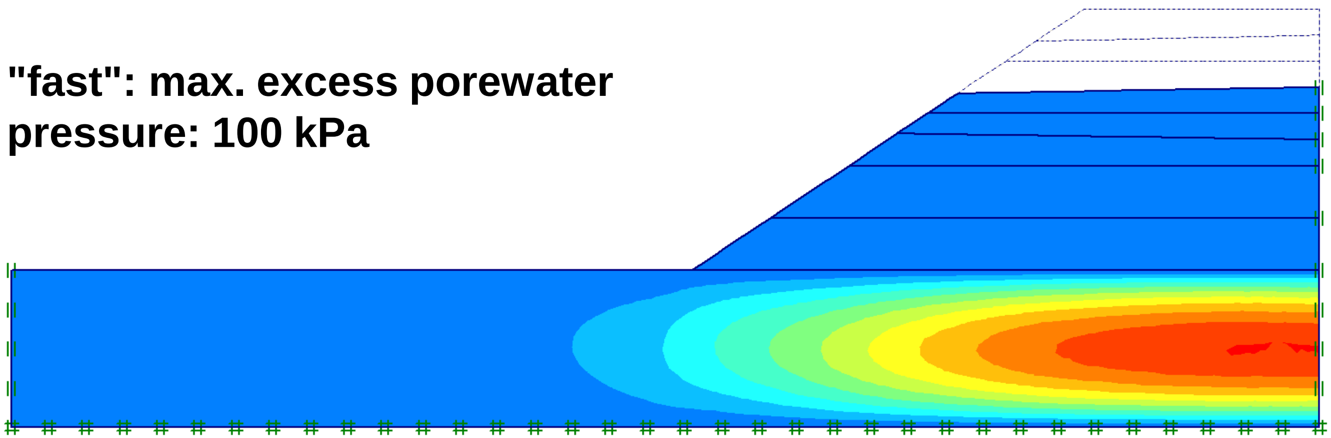
"slow" construction: 3 days of consolidation per placement of 1 m layer embankment

EXAMPLE - EMBANKMENT CONSTRUCTION

"slow": max. excess porewater pressure: 86 kPa

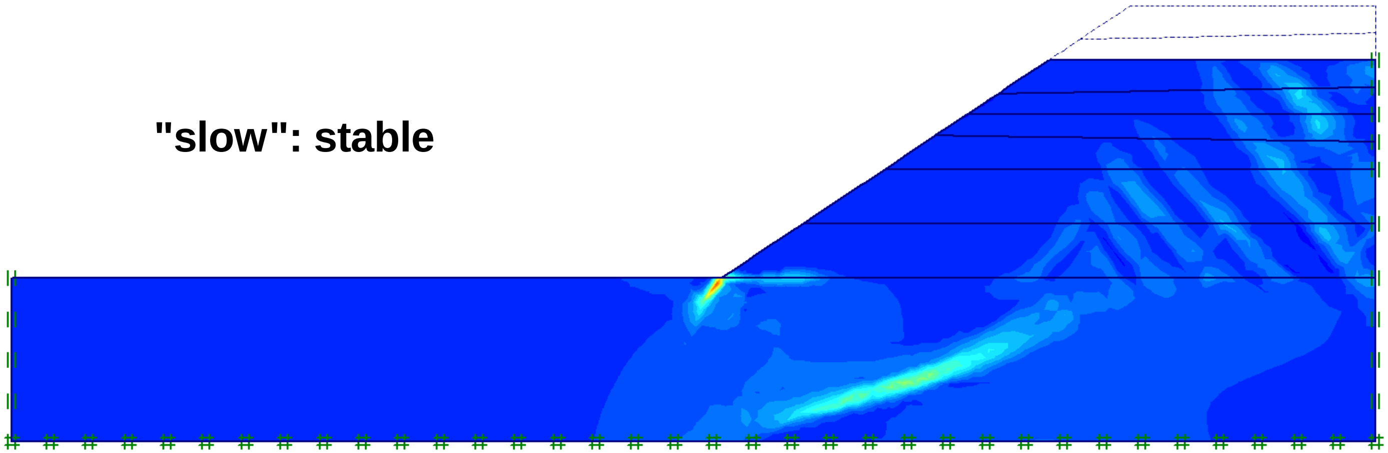


"fast": max. excess porewater pressure: 100 kPa

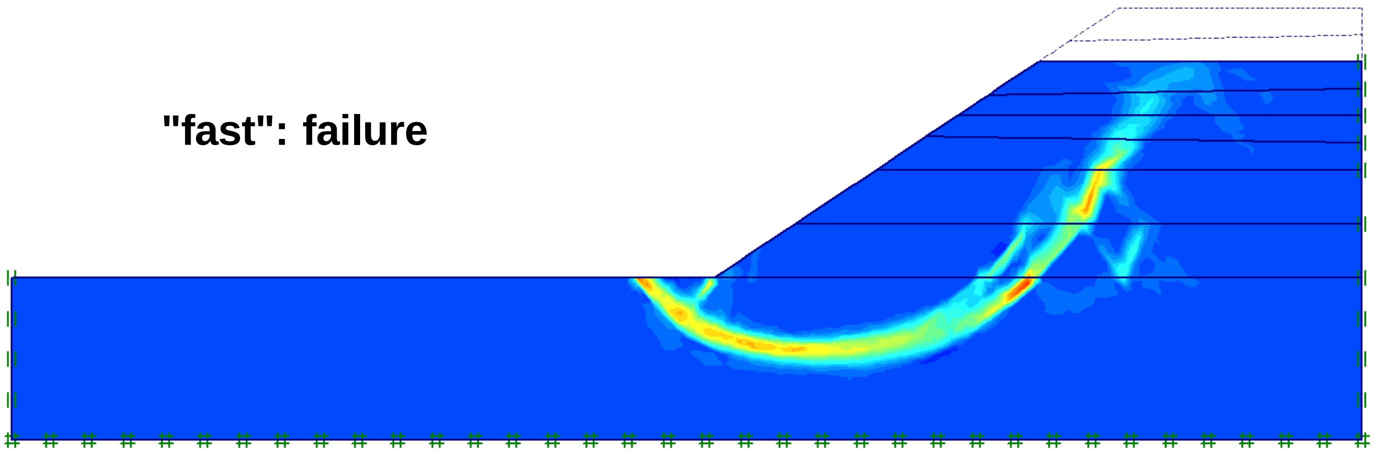


EXAMPLE - EMBANKMENT CONSTRUCTION

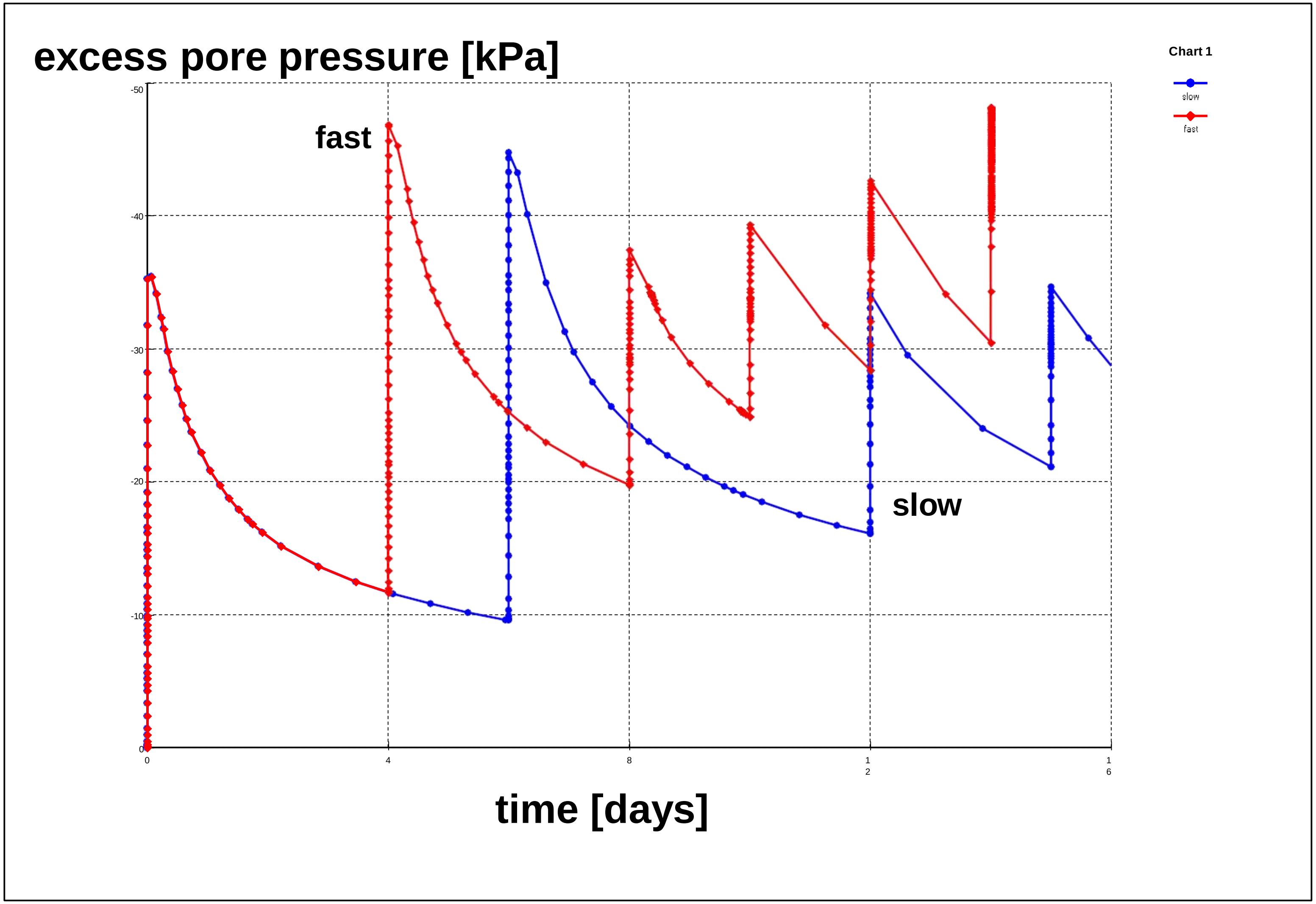
"slow": stable



"fast": failure

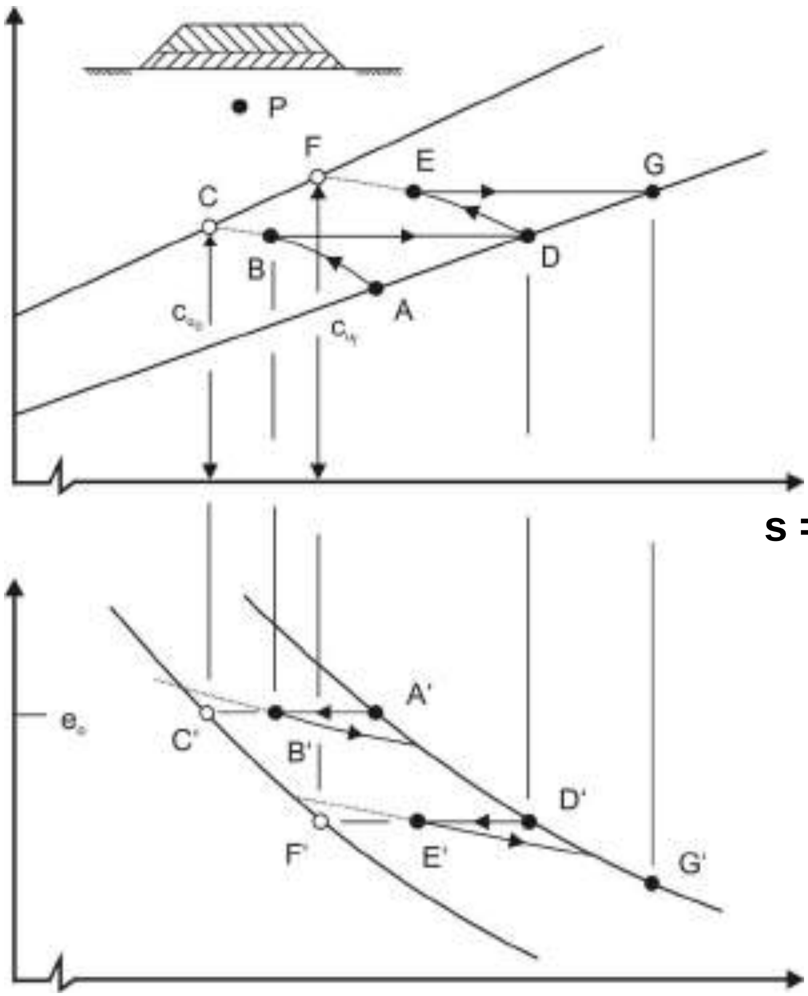


EXAMPLE - EMBANKMENT CONSTRUCTION



SIMPLIFIED REPRESENTATION OF STRESS PATH

$t = ½ (\sigma_1 - \sigma_3)$



$s = ½ (\sigma_1 + \sigma_3)$

$s = ½ (\sigma_1 + \sigma_3)$

from Ortigao, 1995

CG12
Ground Improvement with Vertical Drains

Ikhya Ikhya 1) and Helmut F. Schweiger 2)

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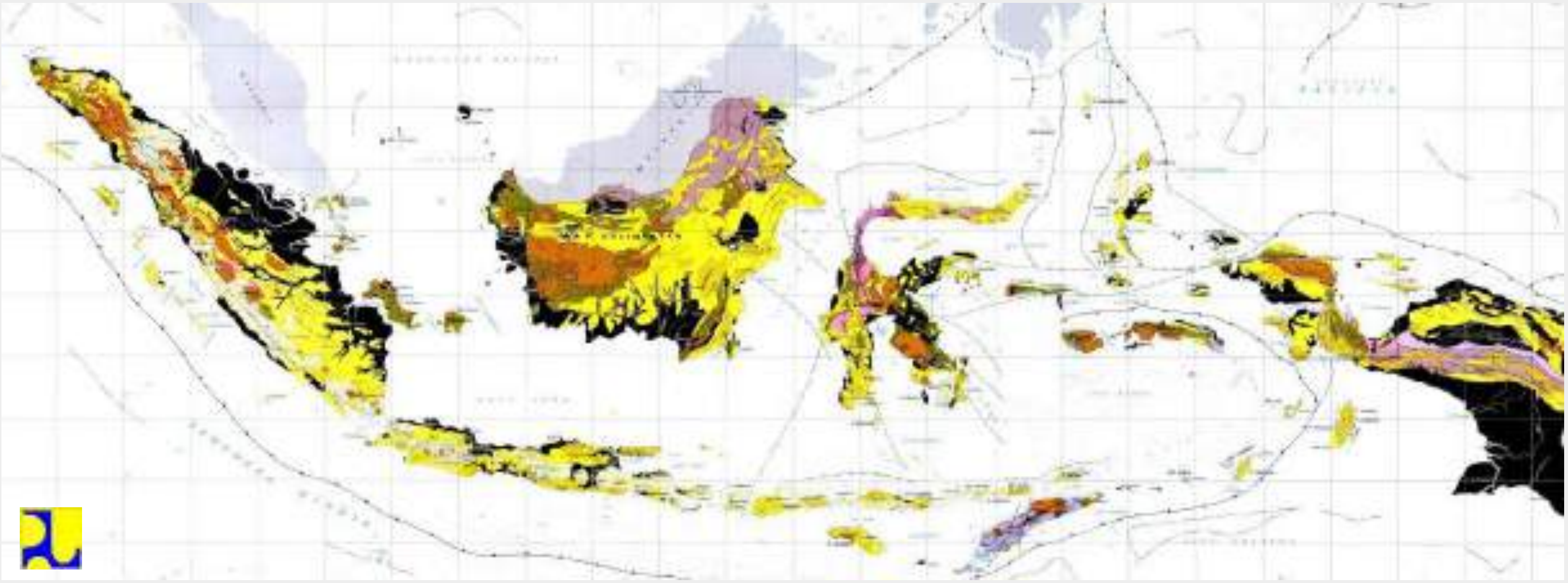
2) Computational Geotechnics Group
Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology

Pelatihan software Plaxis di Tembagapura (In-house training) PT. Freeport Indonesia/ Timika, Papua, 22-30 September 2018

Content

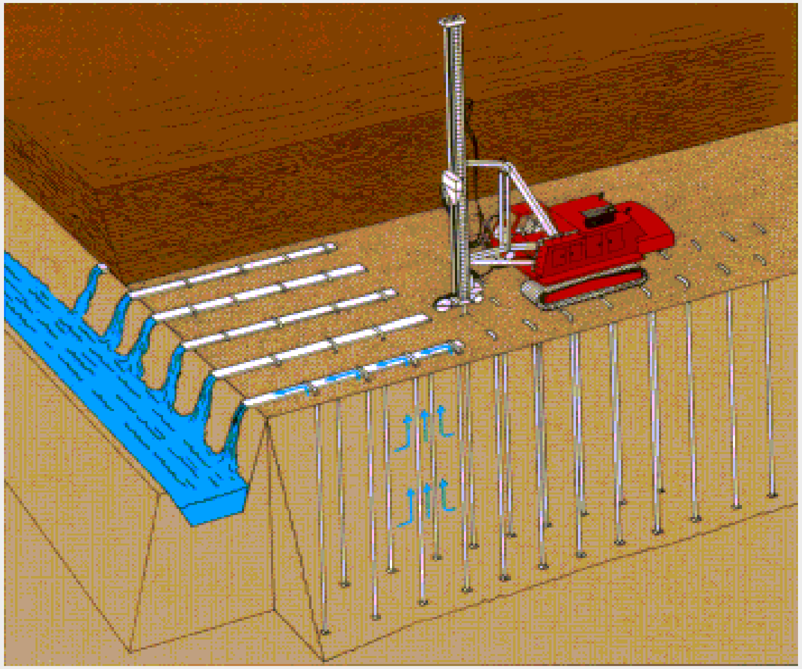
Soft Soil and Soft Soil in Indonesia



- Soft soil deposits (soft clays, organic soils, and peats) are normally characterized by low shear strength, high compressibility and low coefficient of permeability. Characteristics which make them a difficult soil for engineering .
- Soft Soil deposits can be found throughtout world and are also prevailing in many parts of Indonesia.
- Indonesia possesses one of the longest coastal-line in the world with many of them having soft soil deposits.
- The area of soft soil deposits in Indonesia is around 30% of Indonesia’s total land area (60 million-ha with 20 million-ha is peats) with varying depth (black area and some white area on the map).

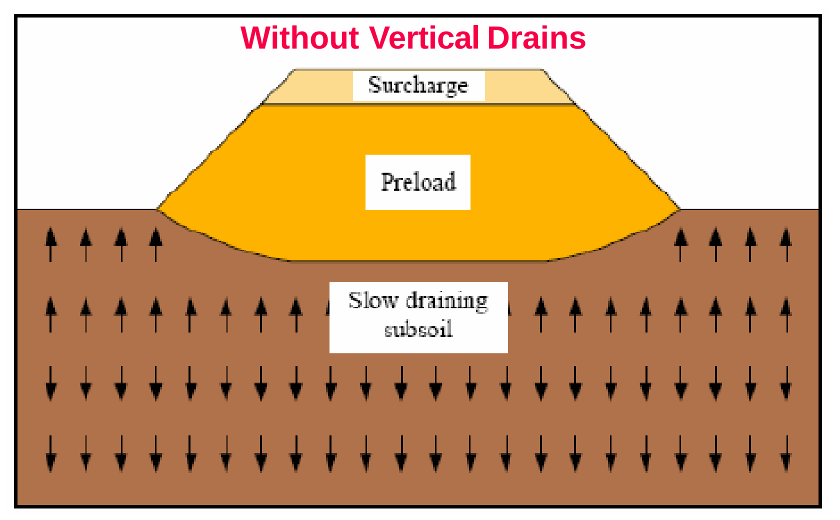
Ground Improvement and Vertical Drains

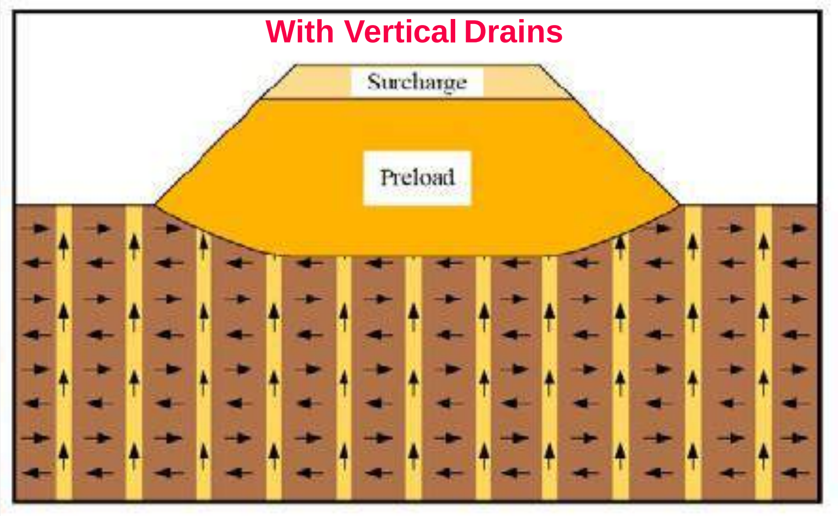
- Ground improvement techniques play an imprortant role in extending the infrastructure across the country in difficult soils.
- Preloading and vertical drains, vibro compaction and replacement, in situ soil mixing, grouting, dynamic compaction, piles, geosynthetics, electro-osmotic, etc, are some ground improvement techniques.
- The selection of the correct ground improvement technique can have an important effect on the foundation choice and can often lead to more economical solutions when compared to traditional approaches.



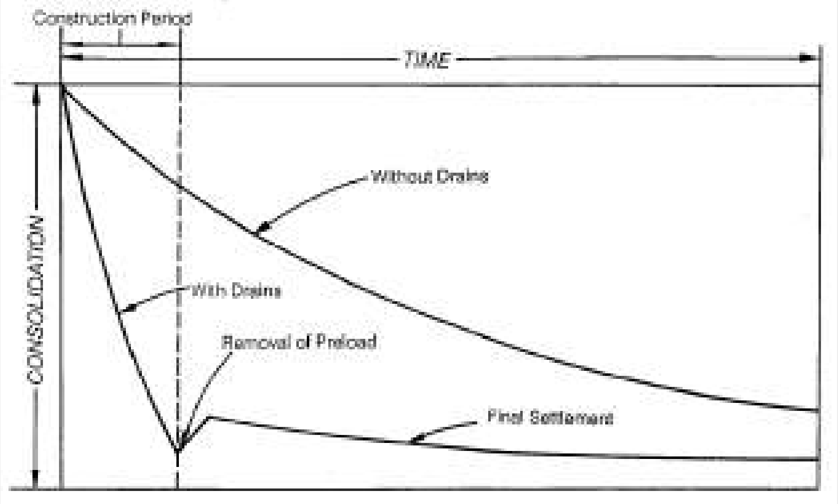
- Vertical drains combined with preloading have become common practice and are among the most effective procedures for ground improvement accelerating the consolidation process (decrease the time for reaching final settlements).
- The installation of vertical drains reduces the drainage path and speeds up the dissipation of excess pore water pressure generated during the application of surcharge loads in saturated fine grained soils.

Vertical Drains

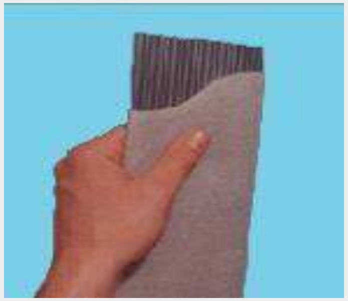


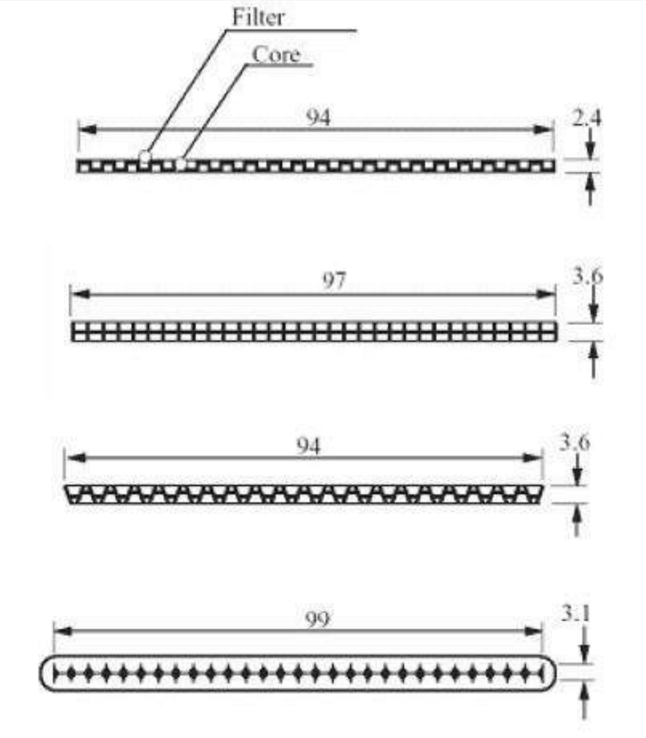


- The installation of vertical drains provides a shorter drainage path and a faster dissipation of excess pore water, thereby resulting in faster development of settlements and a quicker gain of strength due to consolidation.



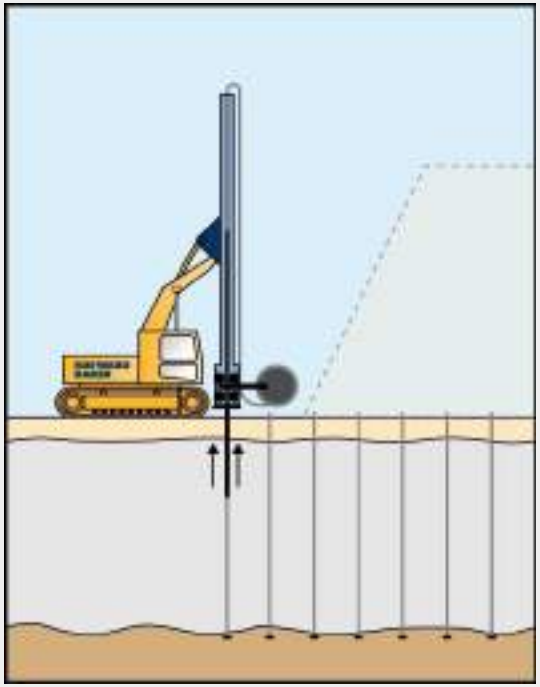
Prefabricated Vertical Drains (PVD)

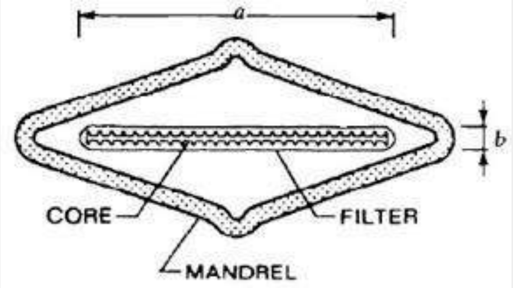




- Prefabricated vertical drains (PVD) are typically 10 cm wide, 3 to 9 mm thick, and packed in rolls.
- The drain consists of a geotextile filter wrapped on a plastic strip with molded channels that allow water to travel to the ground surface, relieving excess subsurface pore water pressure.
- The geotextile filter prevents soil particles from entering the channels and clogging the drain.

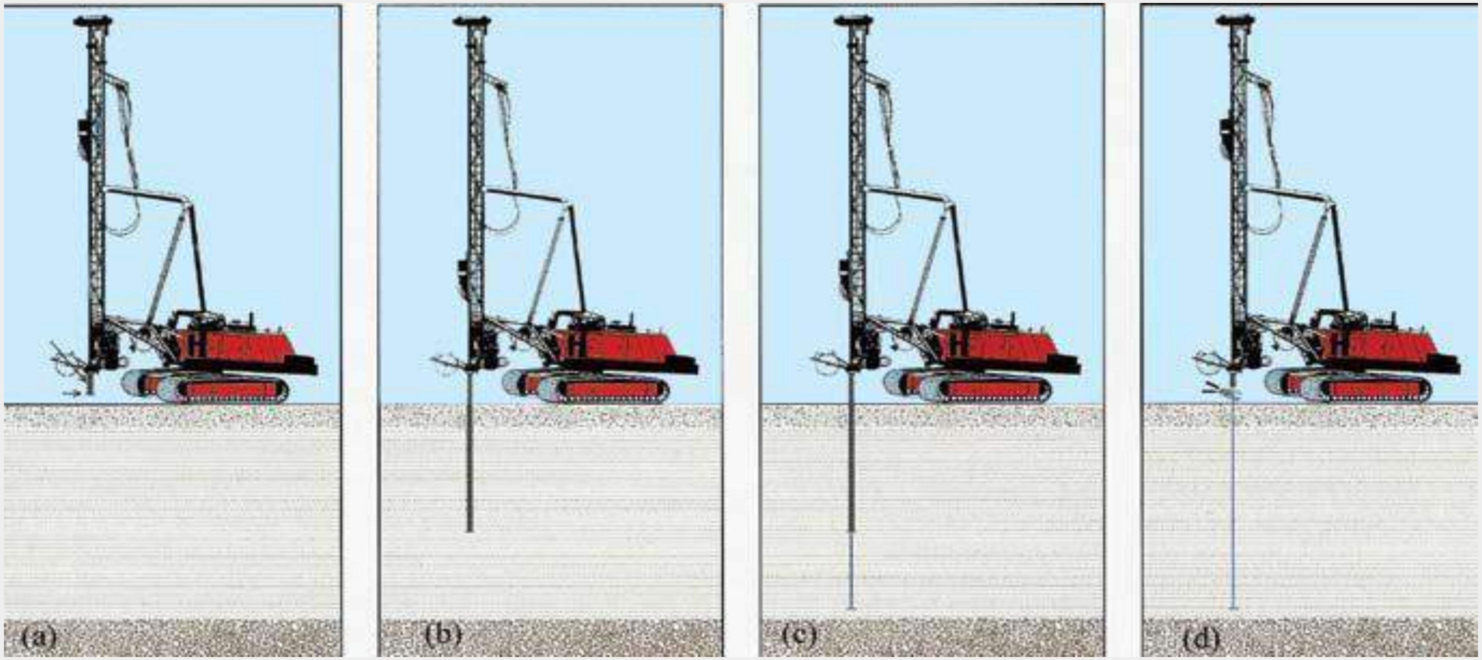
Installation of Prefabricated Vertical Drains (PVD)





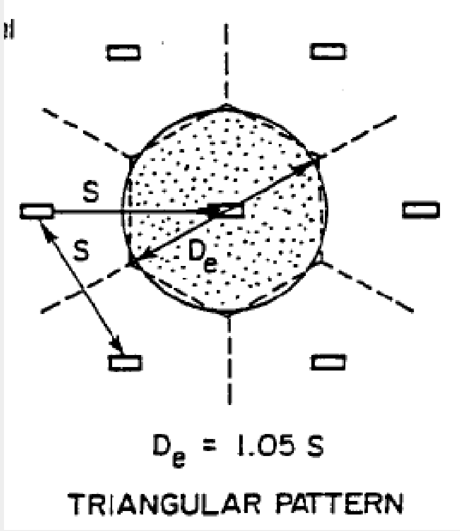
- The installation equipment consists of the drain spools, mandrel, anchor and PVD delivery route excavator or crane.
- The mandrel cross-sectional area is typically 60 - 80 square cm.
- The installation force is typically provided by vibratory methods, static methods or a combination of these methods depending on the soil conditions.
- The majority of depths for PVD applications are between 5 - 40 m, however, drains can be installed up to 60 m.

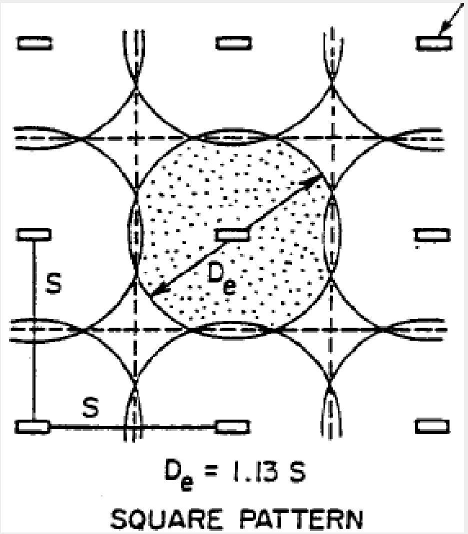
Installation of Prefabricated Vertical Drains (PVD)

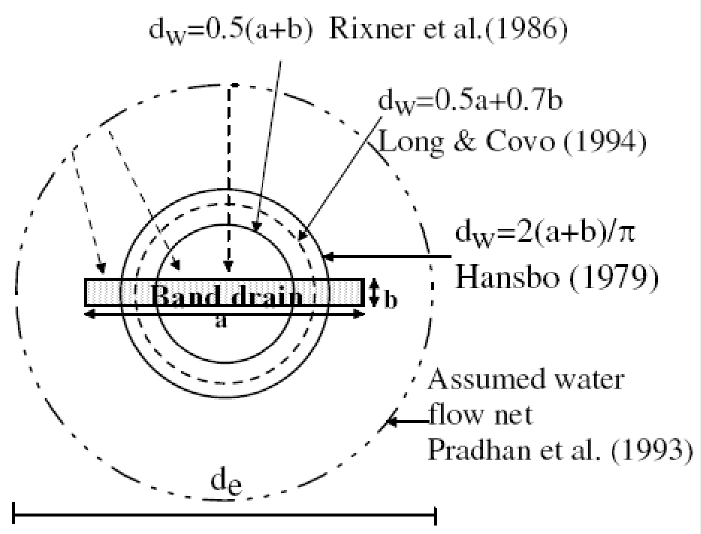


- The wick drain is threaded through the mandrel which protects it from damage as it is driven through the soil.
- After reaching the target depth, the drain is anchored to keep it in place while the mandrel is withdrawn.
- The drain is then cut approximately 10 to 30 cm above ground, and a new anchor fastened to the wick at the bottom of the mandrel in preparation for the next installation point.
- The drains layout usually consists of triangular or square grid patterns and center-to-center spacing is usually in the range of 0.5 - 2.5 m.

Vertical Drain Parameters





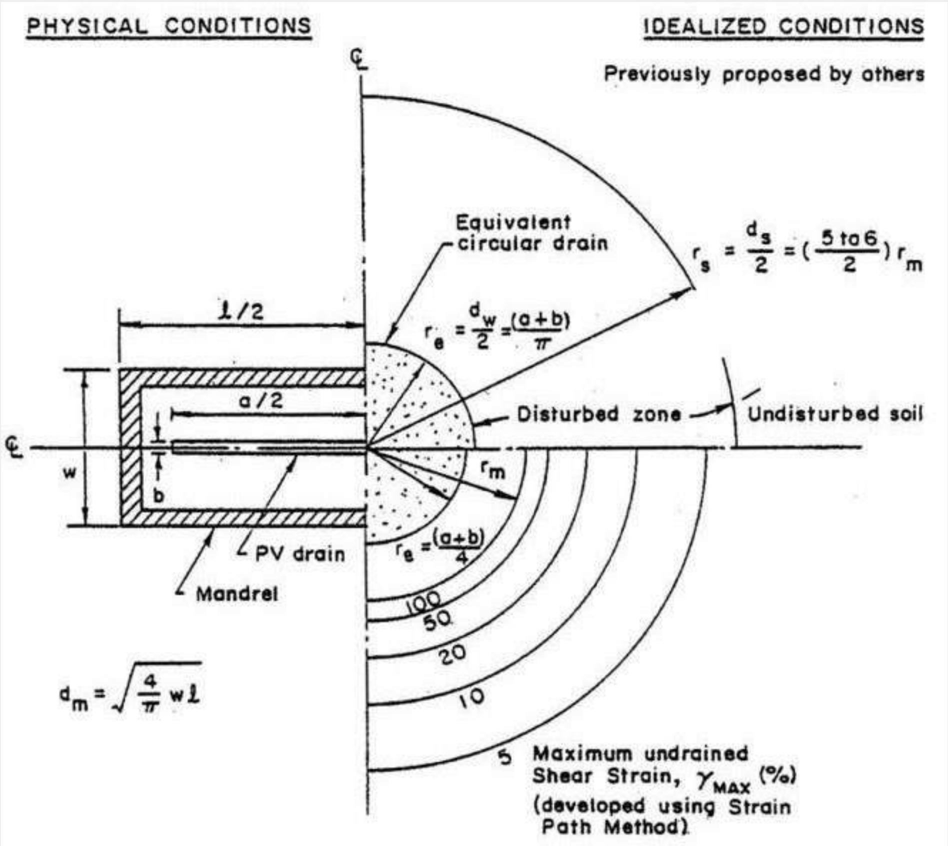


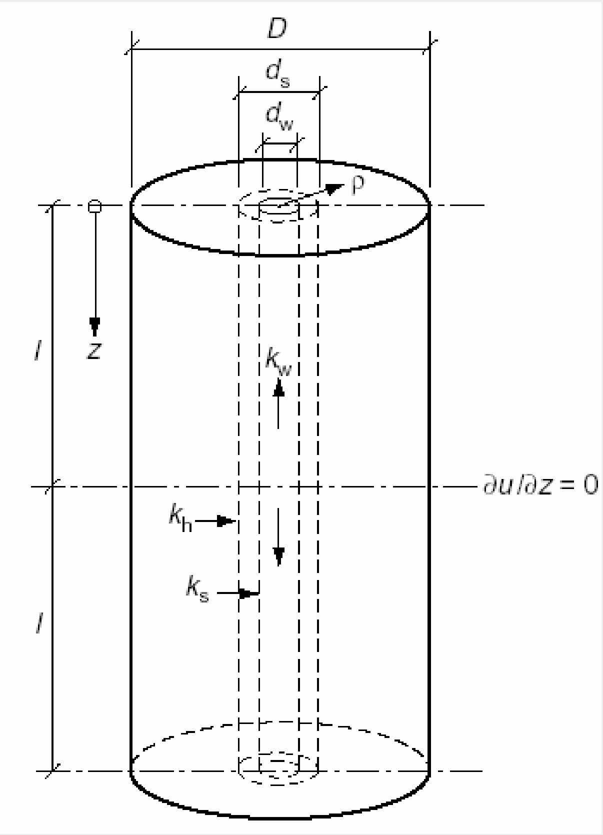
Authors	Smear zone radius
Holtz and Holm (1973); Hansbo (1986), (1987); Bergado et al. (1991), (1993b)	2 $r_{m,eq}$
Jamiolkowski et al. (1983)	(2.5 to 3) $r_{m,eq}$
Mesri et al. (1994)	(2 to 4) $r_{m,eq}$
Hansbo (1997); Chai and Miura (1999); Hird and Moseley (2000)	(2 to 3) $r_{m,eq}$
Sathananthan and Indraratna (2006)	2.5 $r_{m,eq}$

S = center-to-center spacing,
De = equivalent influence zone diameter,
rm = equivalent mandrel radius,

axb = dimension of drain,
dw = equivalent drain diameter,
rs = equivalent smeared zone radius.

Vertical Drain Parameters





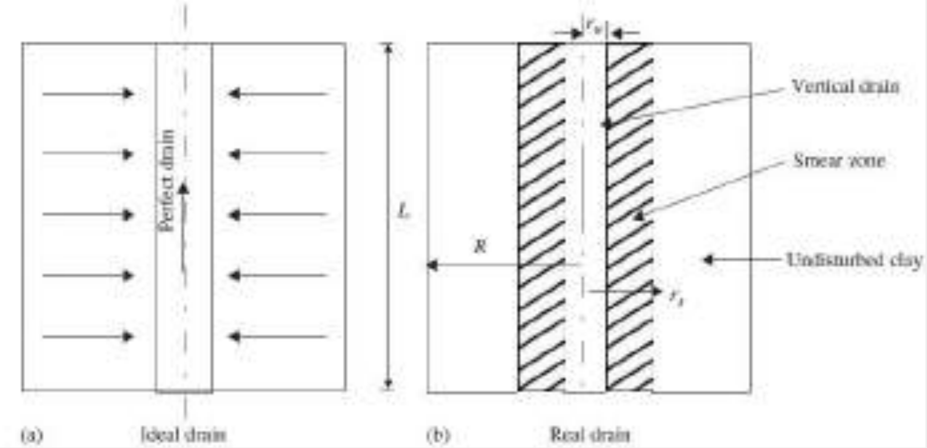
axb = dimension of drain,
r_e = equivalent drain radius,
r_s = equivalent smear zone radius,
r_m = equivalent mandrel radius,

lxw = dimension of mandrel,
d_w = equivalent drain diameter,
d_s = equivalent smear zone diameter,
d_m = equivalent mandrel diameter,

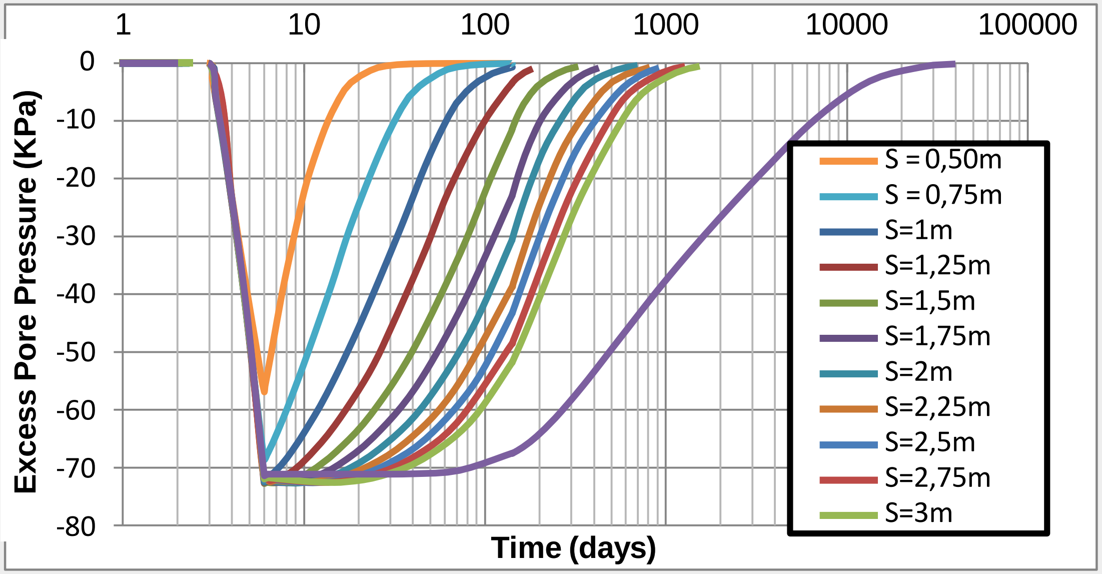
D = diameter of soil cylinder,
k_w = vertial permeability of the drain,
k_h = horizontal permeability of soil,
k_s = horizontal permeability of smear zone.

Vertical Drain Parameters

Vertical Drain Parameters				
S	-	1.9	m	spacing
axb	-	100x4	mm^2	dimension of drain
AxB	-	120x50	mm^2	dimension of mandrel (rectangular)
l	-	10	m	length
d_e	$d_e = 1.05S$	2	m	equivalent influence zone diameter (triangular pattern)
d_w	$d_w = 0.5(a+b)$	52	mm	equivalent drain diameter
d_m	$d_m = 2\sqrt{(AB/\pi)}$	88	mm	equivalent mandrel diameter
d_s	$d_s = 2.5d_m$	220	mm	equivalent smeared zone diameter
n	$n = d_e/d_w$	38.46	-	ratio of d_e to d_w
s	$s = d_s/d_w$	4.23	-	ratio of d_s to d_w
k_v	-	4.32×10^{-4}	m/day	vertical permeability in the undisturbed zone
k_h	$k_h = k_v$	4.32×10^{-4}	m/day	horizontal permeability in the undisturbed zone
k_s	$k_s = 0.5k_h$	2.15×10^{-4}	m/day	horizontal permeability in the smear zone

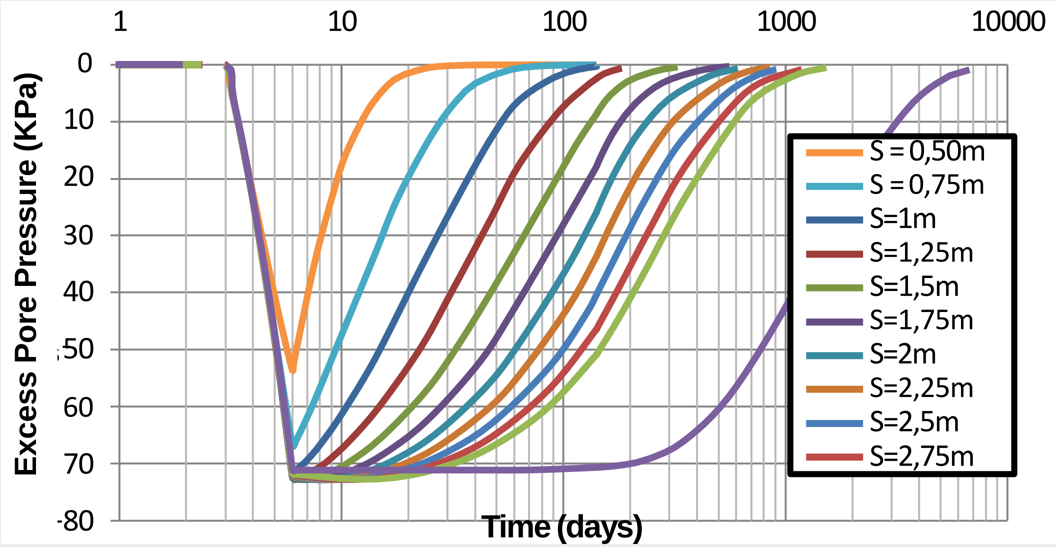


Influence of PVD Spacing (S)

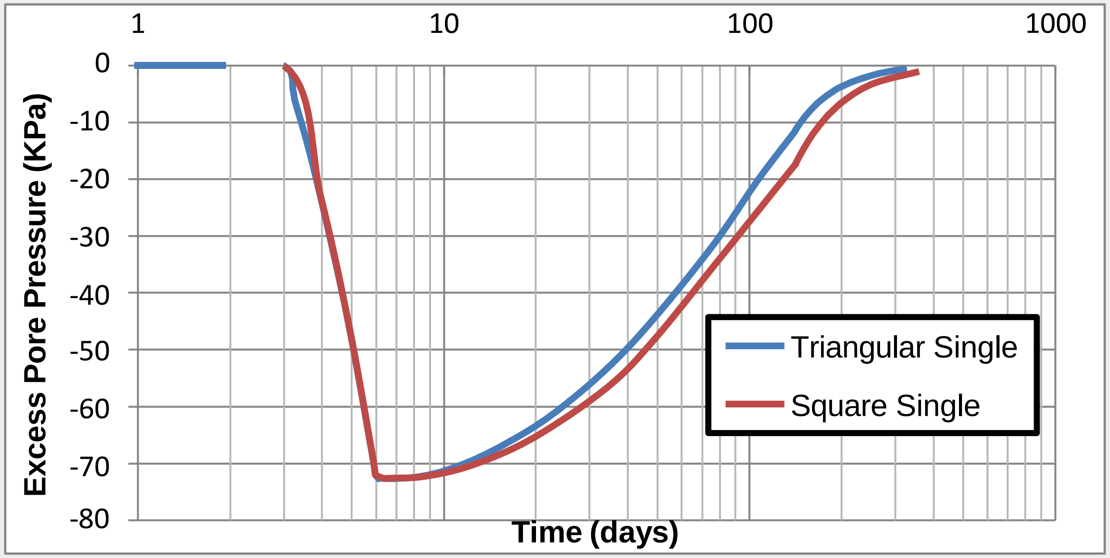


Single Drained
S= 0.5m - 3.0m

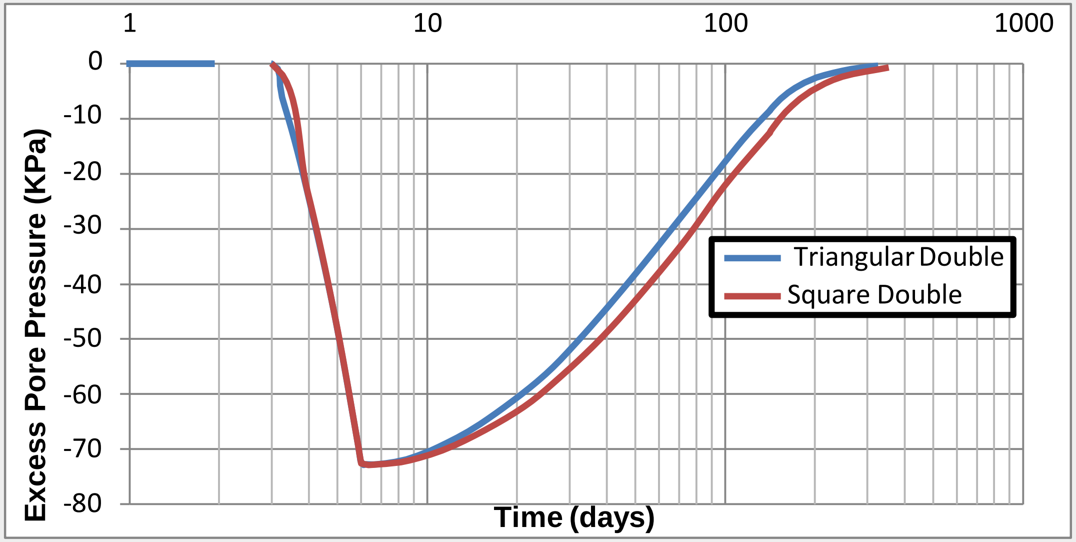
Double Drained
S= 0.5m – 3.0m



Influence of PVD Configuration (de) (Triangular and Square Patterns)

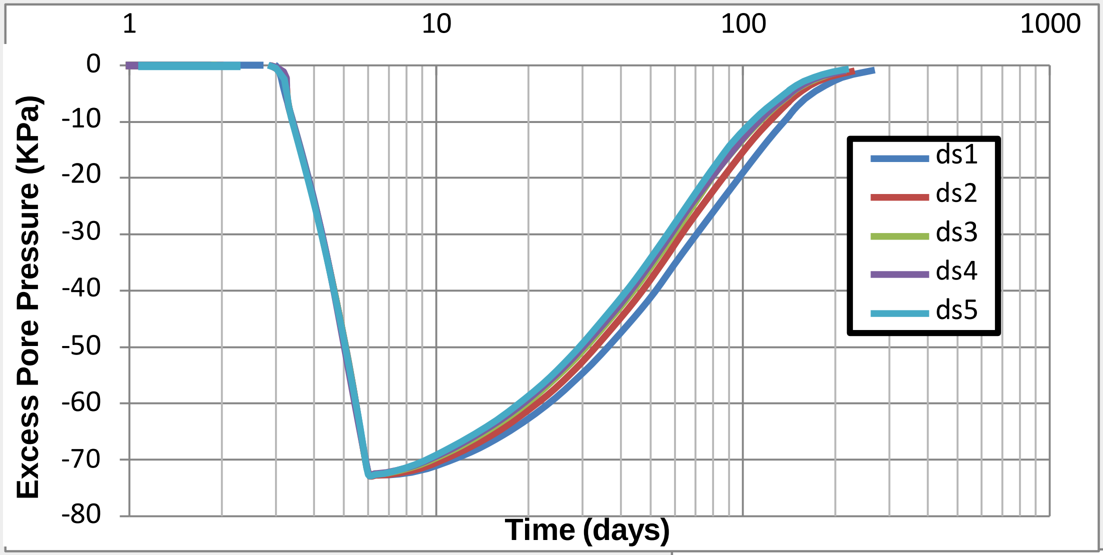


Single Drained



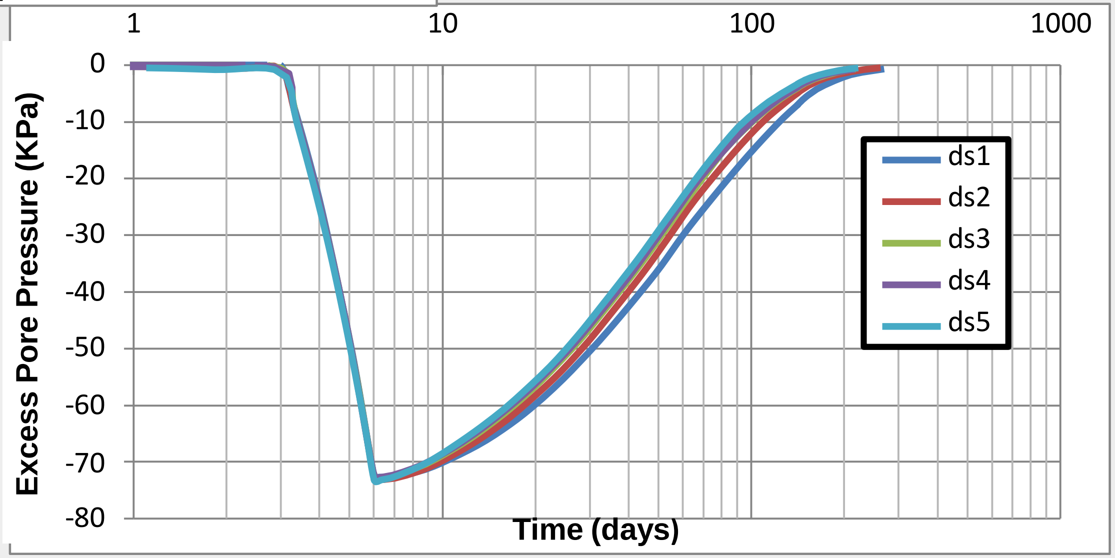
Double Drained

Influence of Smear Zone Diameter (ds)

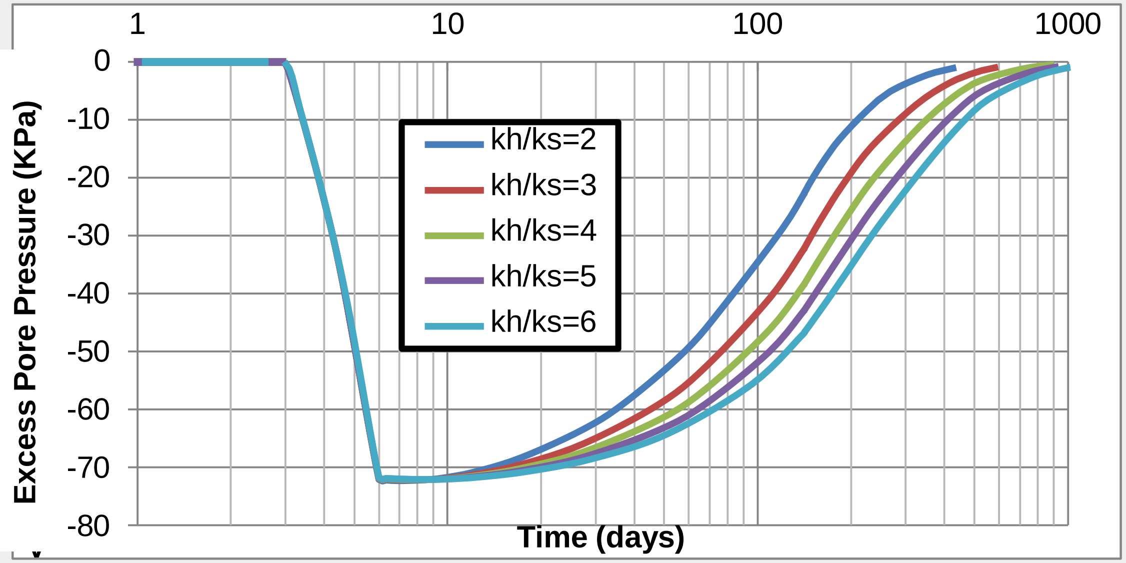


Single Drained
ds1= 99 mm
ds2= 240 mm
ds3= 360 mm
ds4= 396 mm
ds5= 480 mm

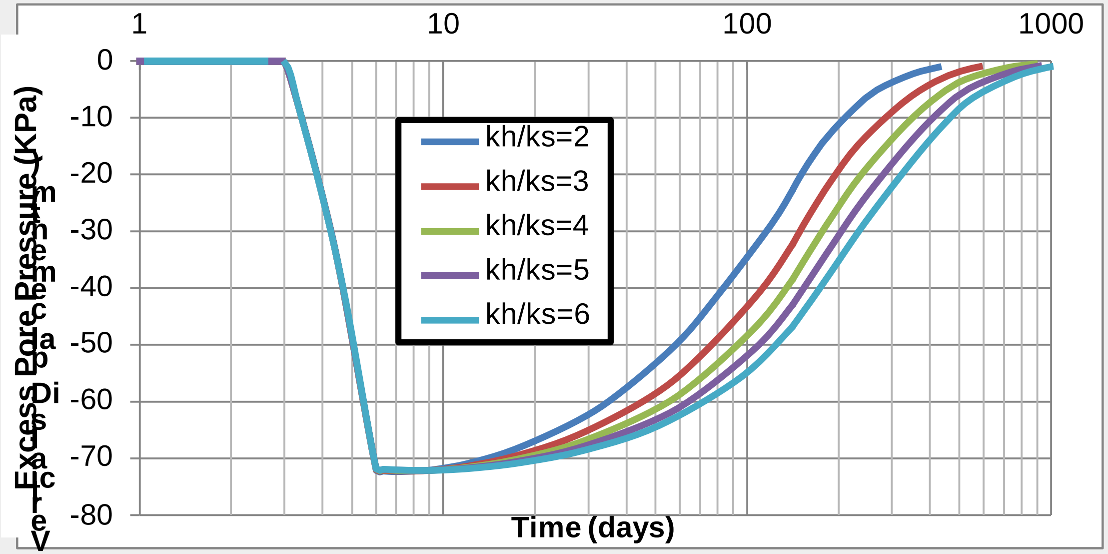
Double Drained
ds1= 99 mm
ds2= 240 mm
ds3= 360 mm
ds4= 396 mm
ds5= 480 mm



Influence of Smear Zone Permeability (ks)



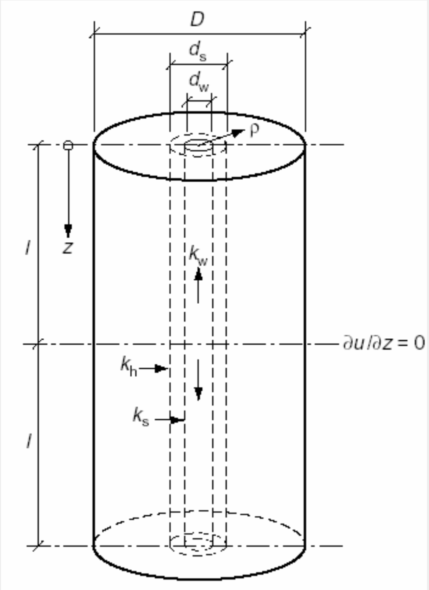
Single Drained



Double Drained

Assumptions made in analytical solution :

- Single cylindrical drainage cell model (each drain is independent at the centre of a cylindrical soil mass, i.e. valid only below centreline of embankments neglecting lateral deformations).
- Homogeneous soil layer or limited number of soil layer (one or two soil layer problems).
- Load distribution is uniform.
- Instantaneously applied load or limited steps and rate of loading.
- Based on simple soil models.



Horizontal Consolidation – Barron (1948)

$$\overline{U}_h = 1 - e^{\left(\frac{-8T_h}{\upsilon}\right)} \qquad T_h = \frac{c_h \cdot t}{D^2} \qquad c_h = \frac{k_h \cdot E_{oed}}{\gamma_w}$$

$$\upsilon = \frac{n^2}{n^2-1} \cdot \ln(n) - \frac{3}{4} + \frac{1}{4n^2}$$ ...(drain spacing factor)

$$\upsilon = \frac{n^2}{n^2 - s^2} \cdot \ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{s^2}{4n^2} + \frac{k_h}{k_s} \cdot \frac{n^2 - s^2}{n^2} \cdot \ln(s)$$ ...(drain spacing and soil disturbance factor)

$$n = \frac{D_e}{d_w} \qquad s = \frac{d_s}{d_w}$$

Where,

Uh = average degree of consolidation on a horizontal plane,

υ = parameter for perfect drain condition and with smear condition,

Th = time factor for radial drainage,

ch = coefficient of consolidation for radial drainage,

kh = undisturbed horizontal permeability,

ks = horizontal permeability in the smear zone,

de = equivalent influence zone diameter,

dw = equivalent drain diameter,

ds = equivalent smear zone diameter.

Vertical Consolidation – Terzaghi (1925)

$$U_z = 1 - \sum_{m=0}^{m=\infty} \left(\frac{2}{M} \sin(MZ) e^{\left(-M^2 T_v\right)} \right)$$

$$M = \frac{2m+1}{2}\pi \qquad Z = \frac{z}{H}$$

$$T_v = \frac{c_v \cdot t}{H^2} \qquad c_v = \frac{k_v \cdot E_{oed}}{\gamma_w}$$

Where:

Uz = vertical degree of consolidation,

Tv = vertical time factor,

cv = vertical coefficient of consolidation,

kv = undisturbed vertical permeability,

H = height of drainage path (*H*/2 for double drainage and *H* for single drainage).

Combination of Vertical and Horizontal Consolidation – Carillo (1942)

$$\overline{U} = 1 - (1 - \overline{U}_v)(1 - \overline{U}_h)$$

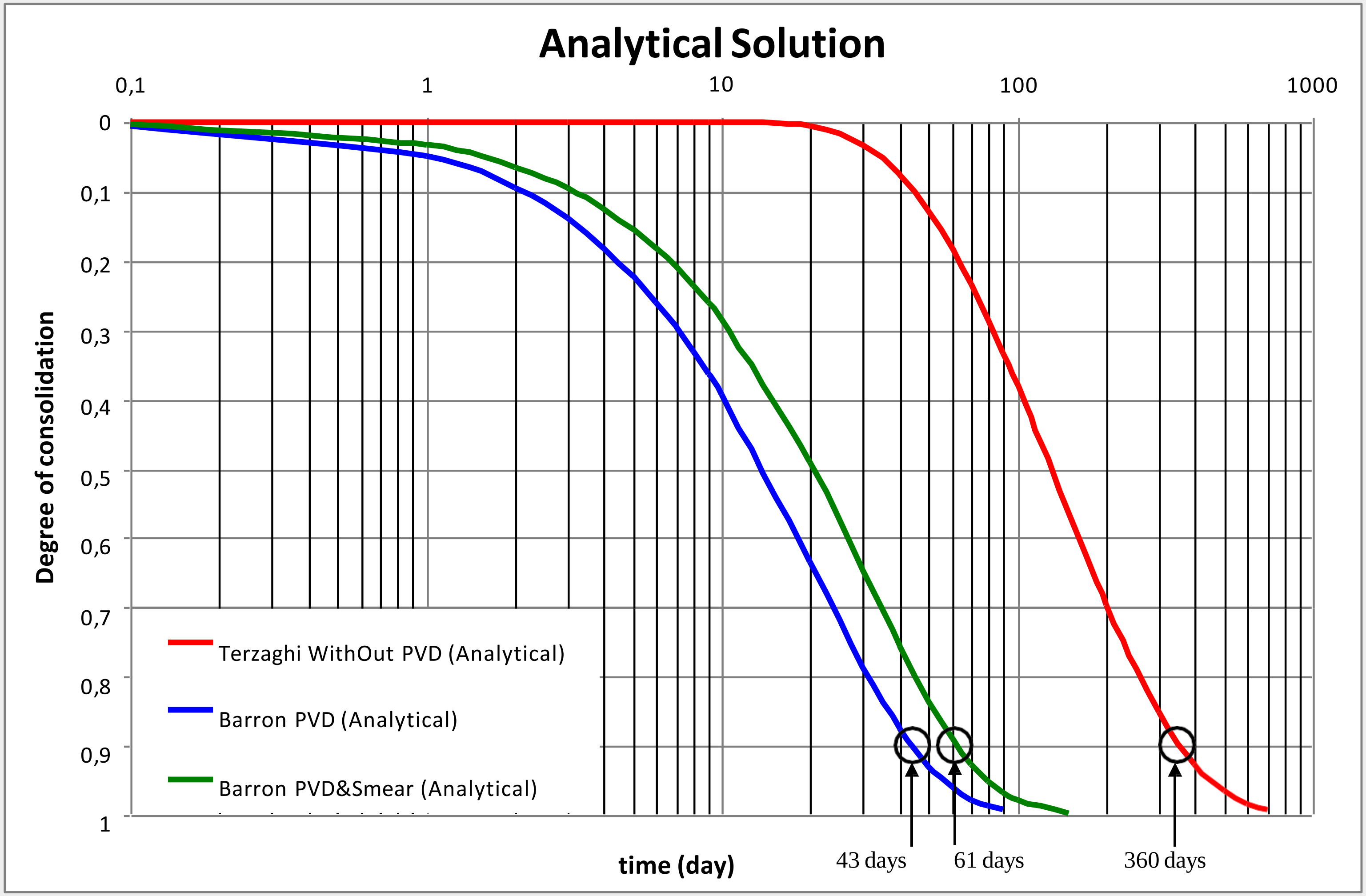
Where,

U = combination of vertical and horizontal degree of consolidation

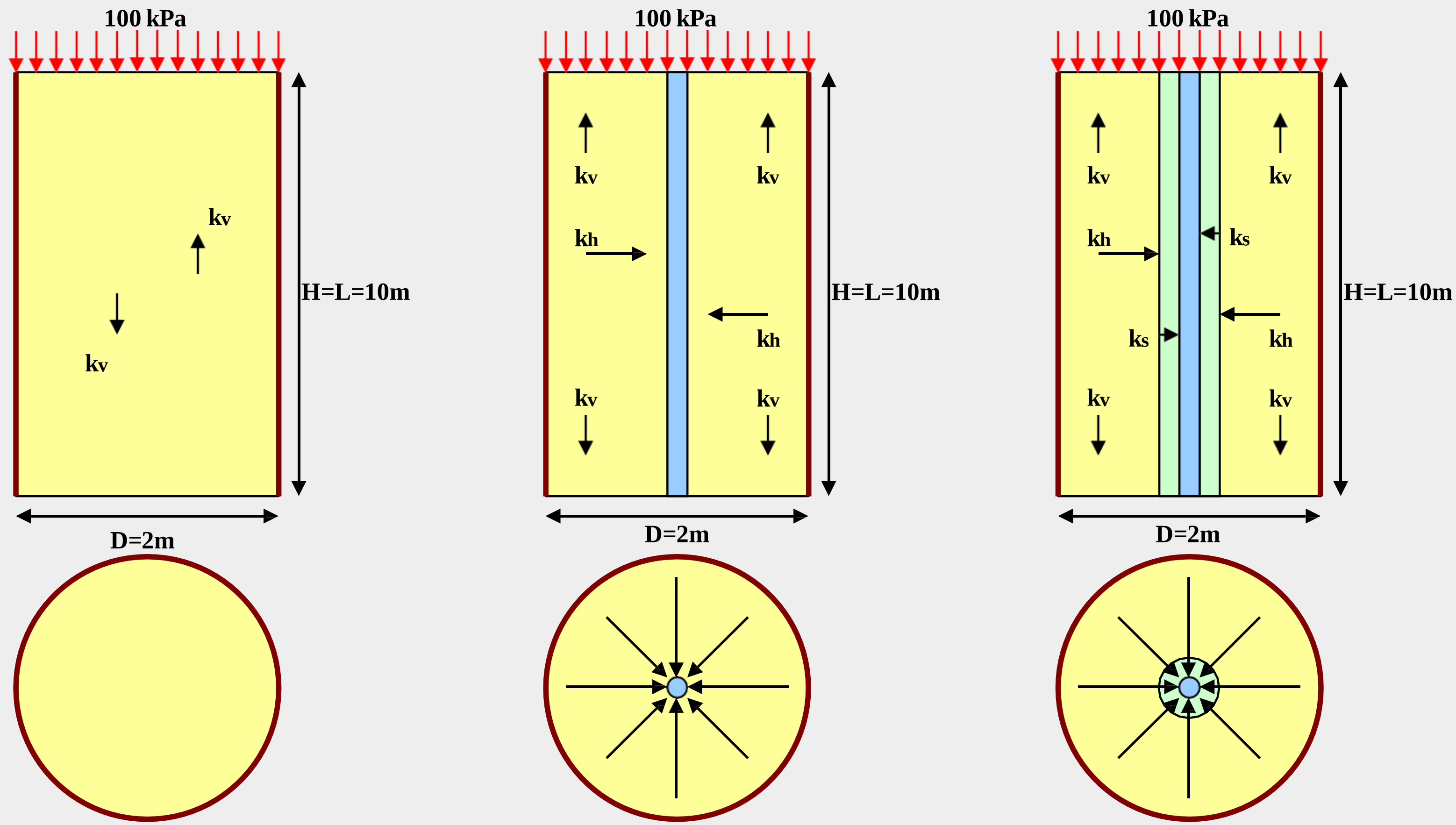
Uv = vertical degree of consolidation,

Uh = horizontal degree of consolidation.

Degree of Consolidation for Analytical Solution (Analytical)



2D Axisymmetric Model (MC and HS)



Without PVD — With PVD — With PVD and Smear Zone

(De=2m, Ds=220mm, Dw=30mm, Hsoil=10m, Lpvd=10m, P=100KPa, GWT=0m, Double Drained)

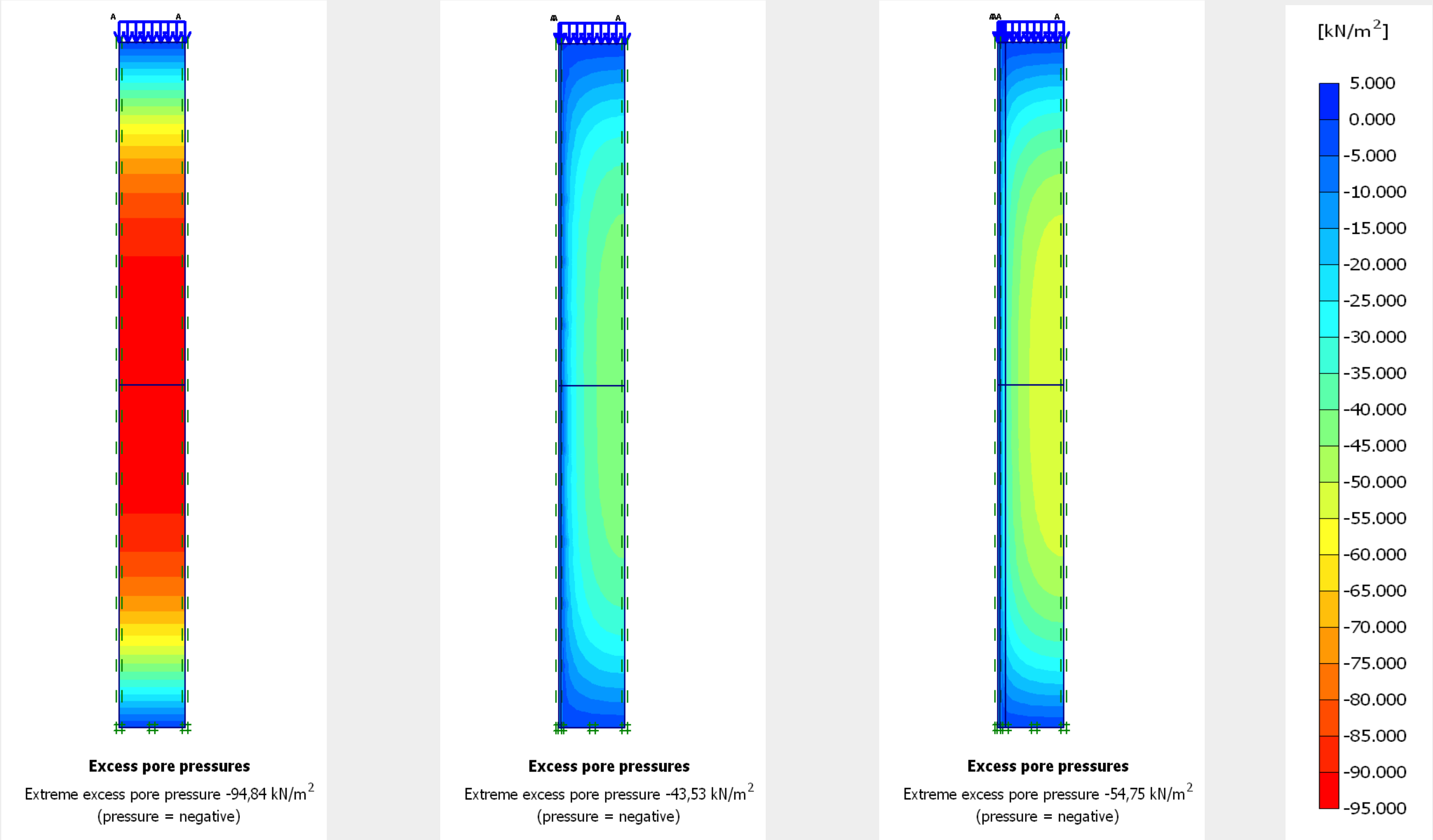
Soil Parameters (Rotterdam Klei)

Soil Parameters MC Model		
Parameters		Clay
Type	-	UnDrained
γ_{unsat}	kN/m³	15
γ_{sat}	kN/m³	16
k_x	m/day	4.32 x 10^{-4}
k_y	m/day	4.32 x 10^{-4}
υ	-	0.3
E_{ref}	kN/m²	1250
c_{ref}	kN/m²	15
φ	degree	27
Ψ	degree	0
E_{incr}	kN/m³	0
c_{incr}	kN/m³	0
y_{ref}	m	0
T-Strength	kN/m²	0
R_{inter}	-	0.64

Vertical Drain Parameters				
l		10	m	
d_e		2	m	
d_w		52	mm	
d_s		220	mm	
k_s		2.15 x 10^{-4}	m/day	

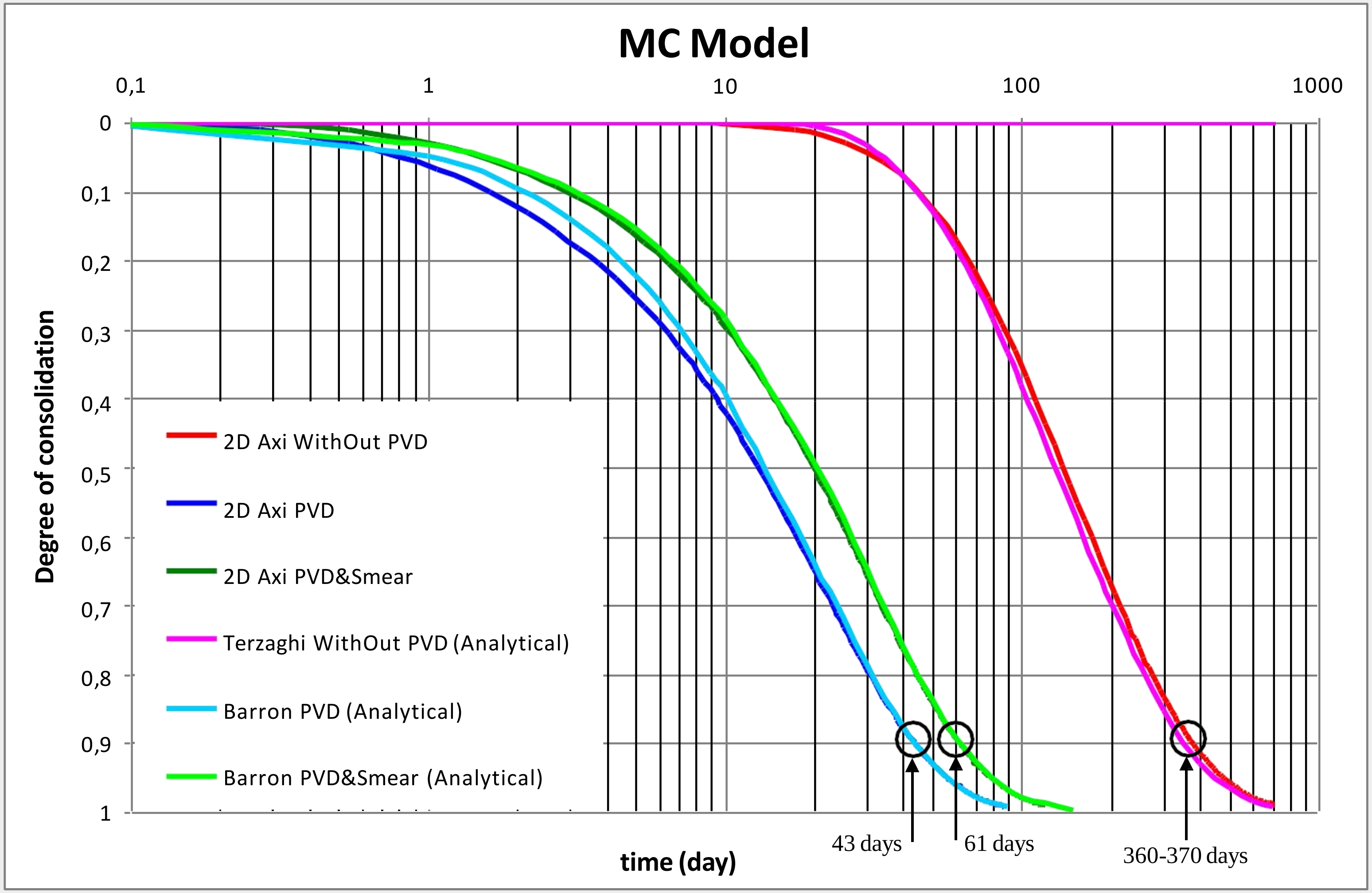
Soil Parameters HS Model		
Parameters		Clay
Type	-	UnDrained
γ_{unsat}	kN/m³	15
γ_{sat}	kN/m³	16
k_x	m/day	4.32 x 10^{-4}
k_y	m/day	4.32 x 10^{-4}
E_{50}^{ref}	kN/m²	4300
E_{oed}^{ref}	kN/m²	1800
E_{ur}^{ref}	kN/m²	14400
c_{ref}	kN/m²	15
φ	degree	27
Ψ	degree	0
υ_{ur}	-	0.2
p_{ref}	kN/m²	100
m	-	0.9
K_0^{nc}	-	0.546
c_{incr}	kN/m³	0
y_{ref}	m	0
R_f	-	0.9
T-Strength	kN/m²	0
R_{inter}	-	0.64
δ_{-inter}	-	0

Excess Pore Pressure - Mohr Coulomb - 20 days

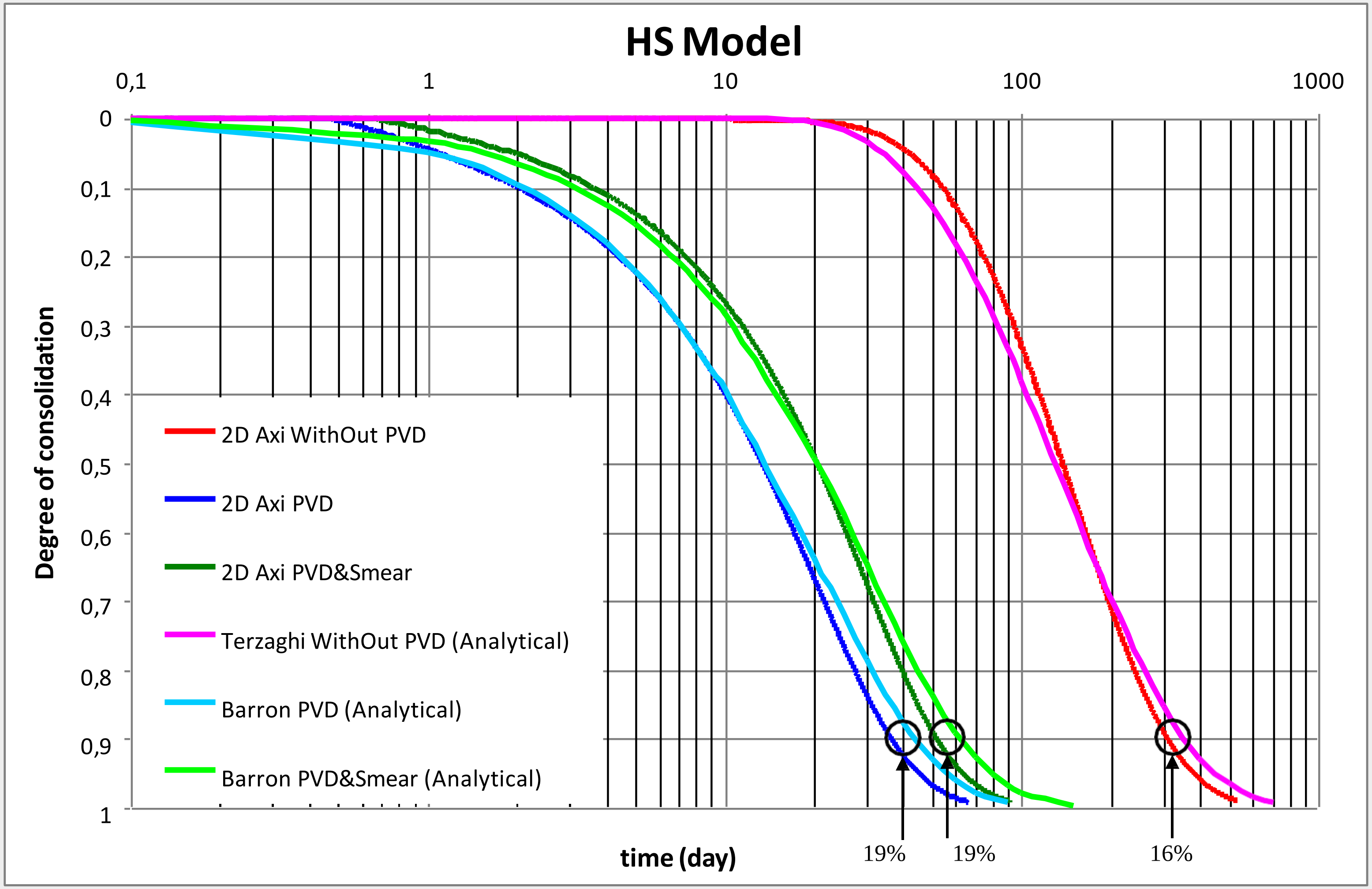


Without PVD | **With PVD** | **With PVD and Smear Zone**

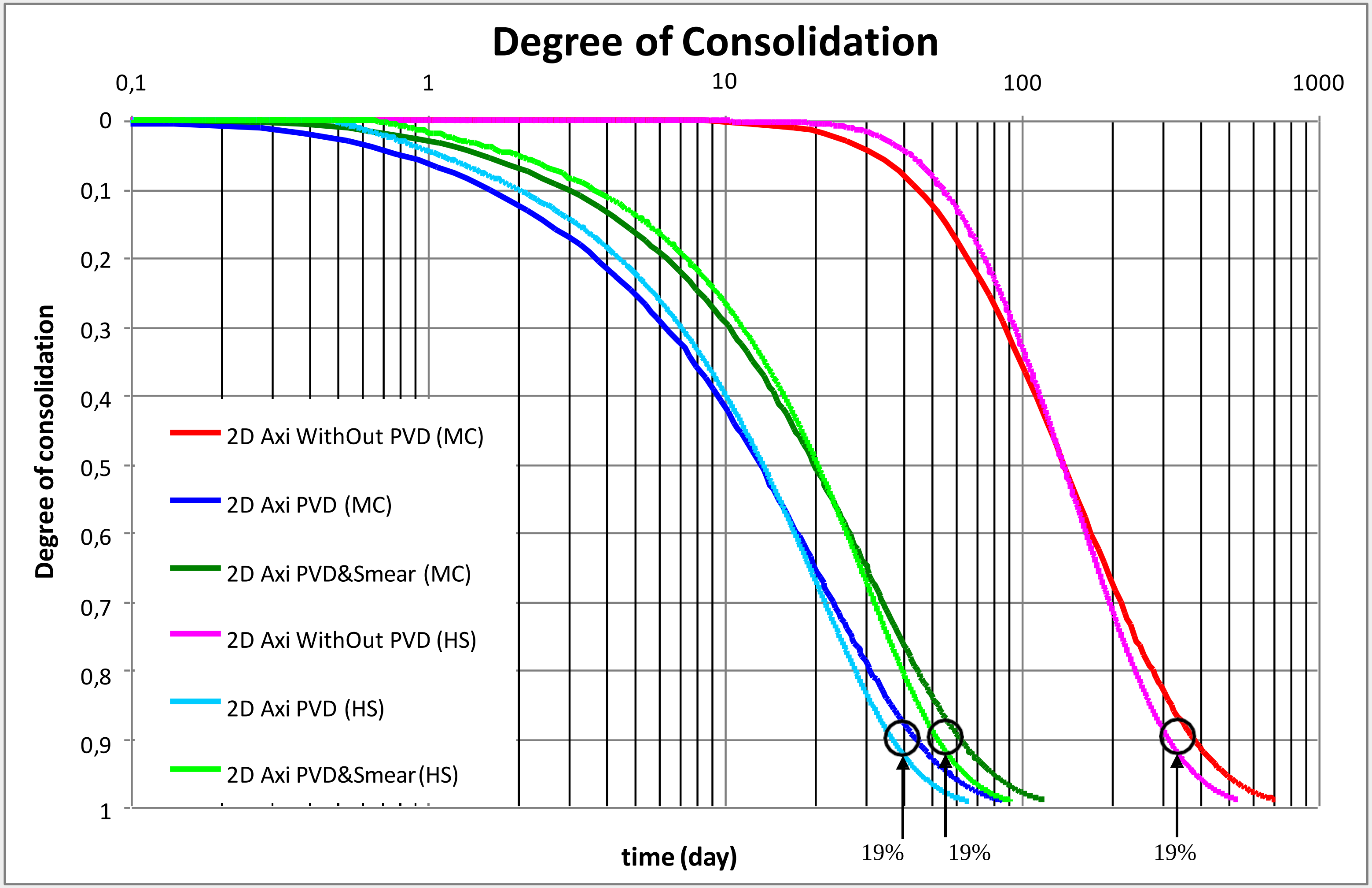
Degree of Consolidation for Different Models (Numerical vs Analytical)



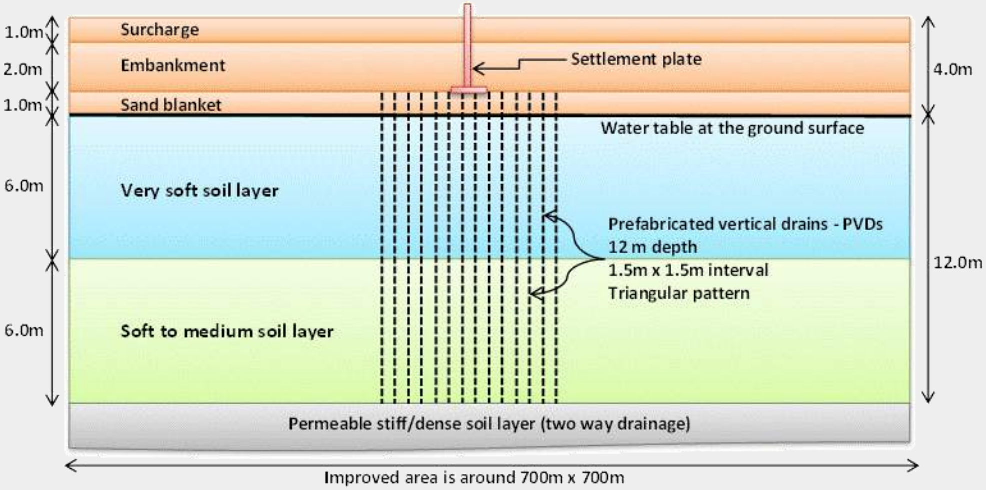
Degree of Consolidation for Different Models (Numerical vs Analytical)

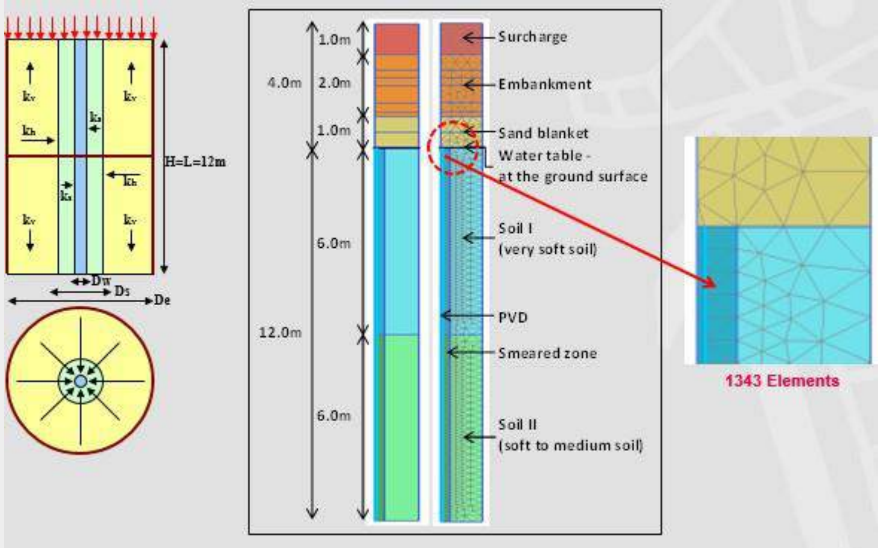


Degree of Consolidation for Different Constitutive Models (MC vs HS)



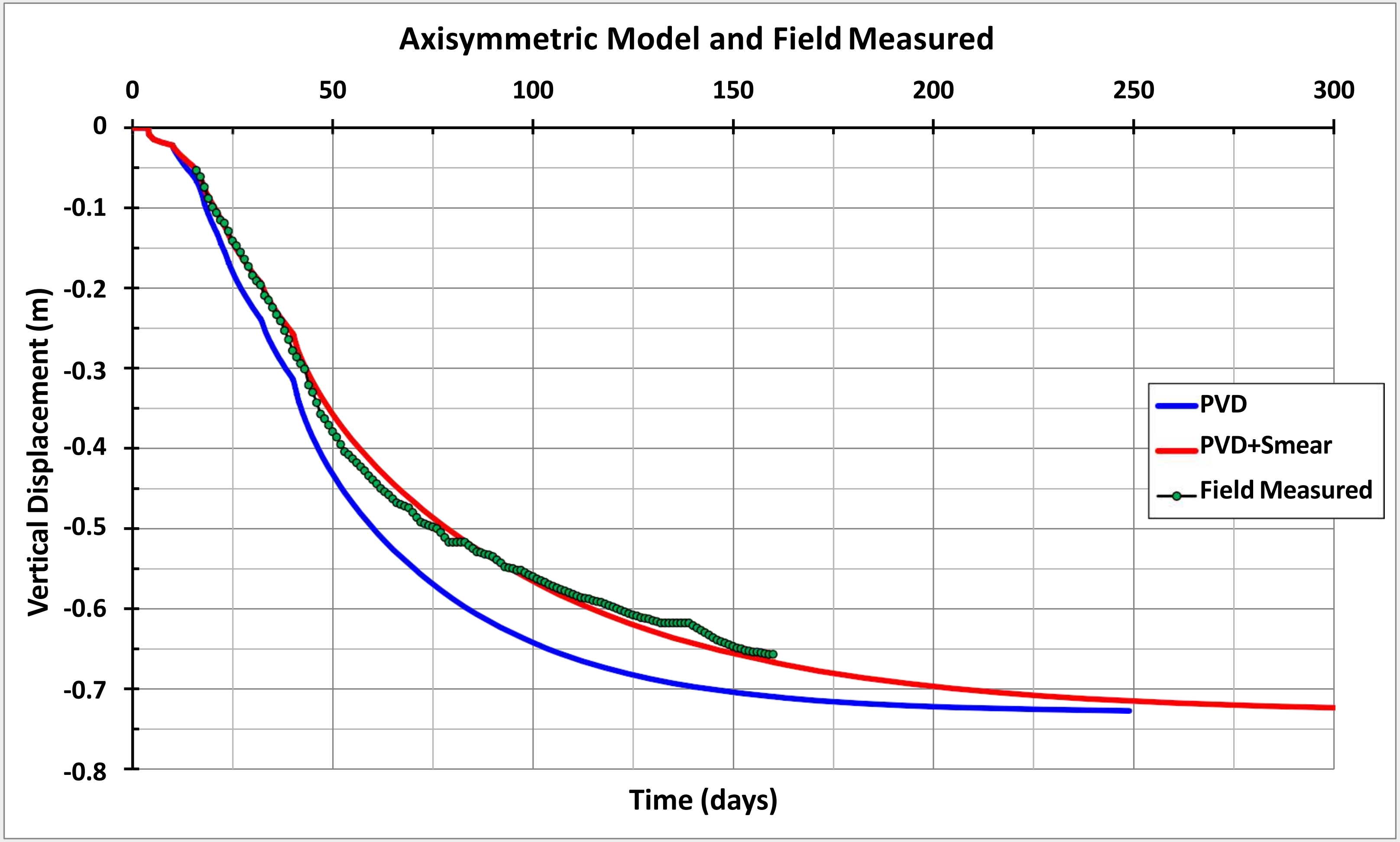
Comparison field measured and numerical model





A good agreement between the field measurement result and the numerical model prediction in the 2D axisymmetric condition at Cirebon Power Plant Project.

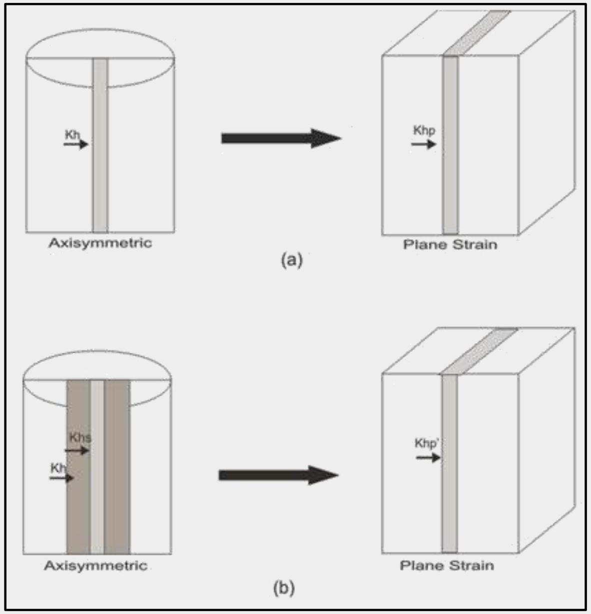
Vertical Displacement – Comparison field measured and numerical model



A good agreement between the field measurement result and the numerical model prediction in the 2D axisymmetric condition at Cirebon Power Plant Project.

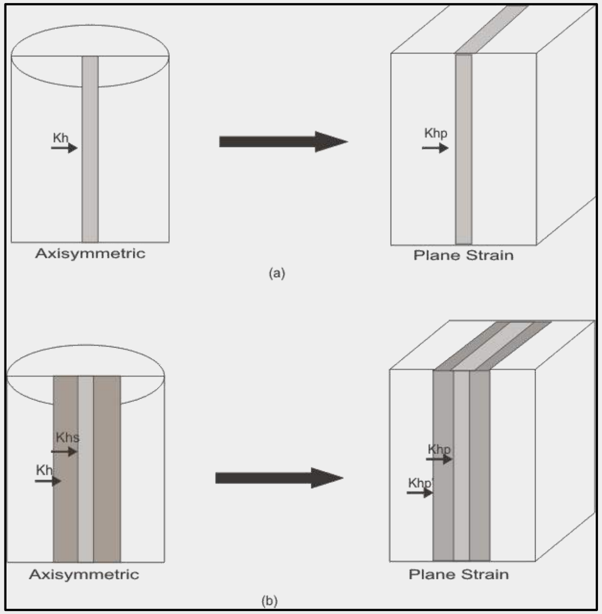
Conversion of an axisymmetric unit cell into plane strain condition

Hird et al. (1992)



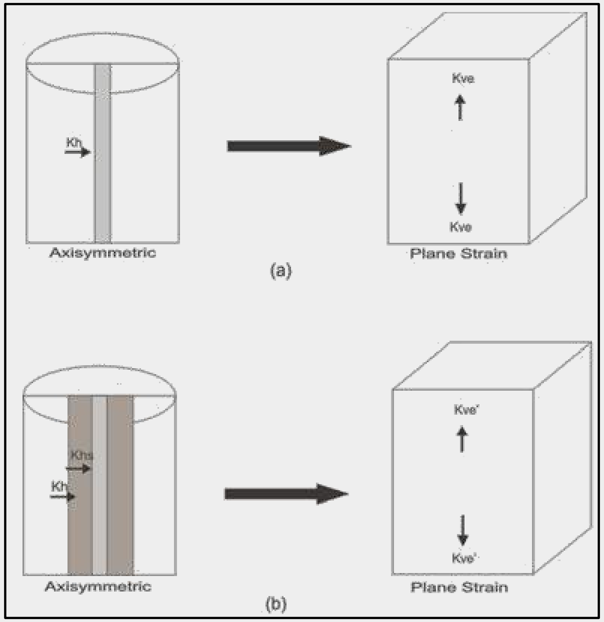
$$\frac{k_{hp}'}{k_h} = \frac{2}{3\left[\ln\left(\frac{R}{r_s}\right) + \left(\frac{k_h}{k_s}\right) ln(s) - \frac{3}{4}\right]}$$

Indraratna et al. (1997)



$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\frac{k'_{hp}}{k_{hp}}\left[\ln\left(\frac{n}{s}\right)+\left(\frac{k_h}{k'_h}\right)\ln(s)-0{,}75\right]-\alpha}$$

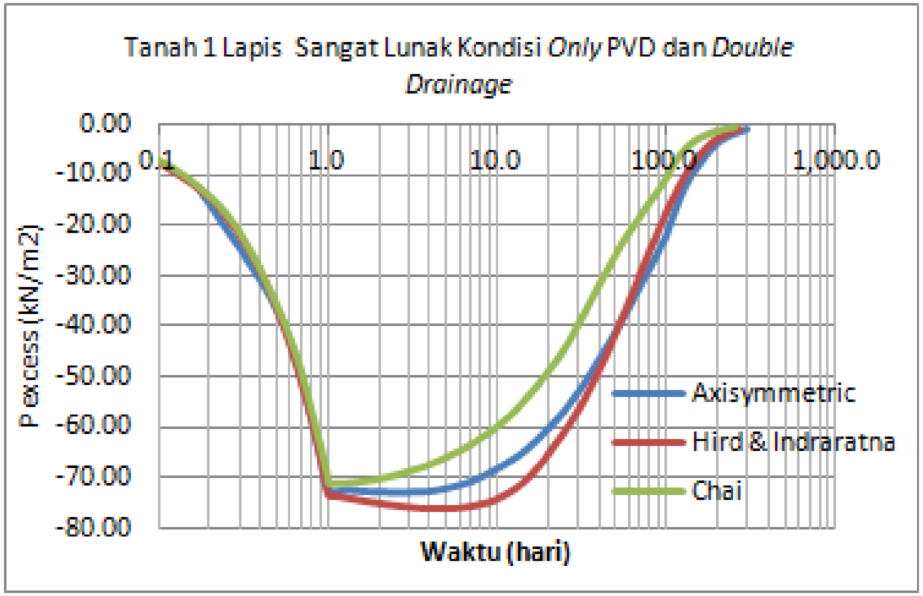
Chai et al. (2001)

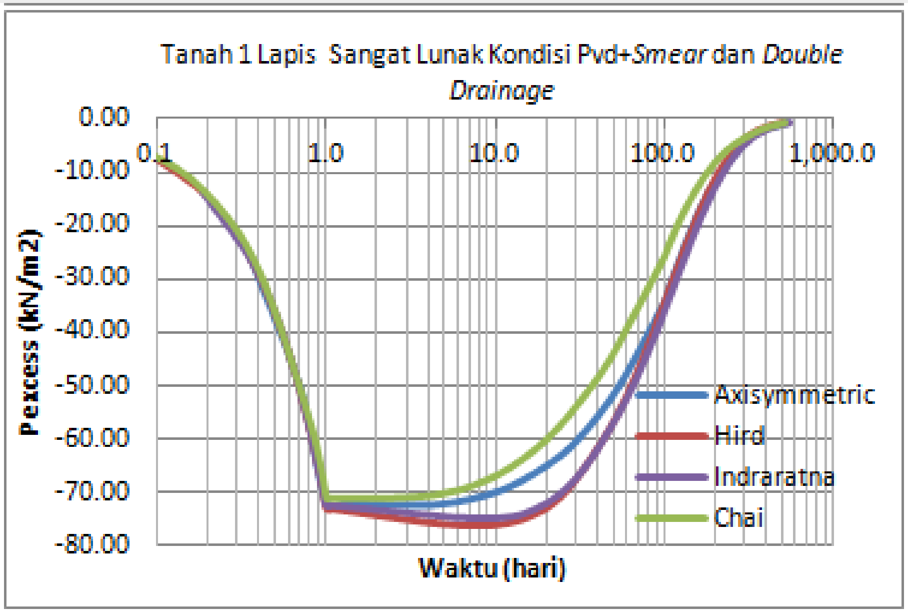


$$K_{ve} = \left(1 + \frac{2{,}5L^2 k_h}{\mu d_e^2 k_v}\right) k_v$$

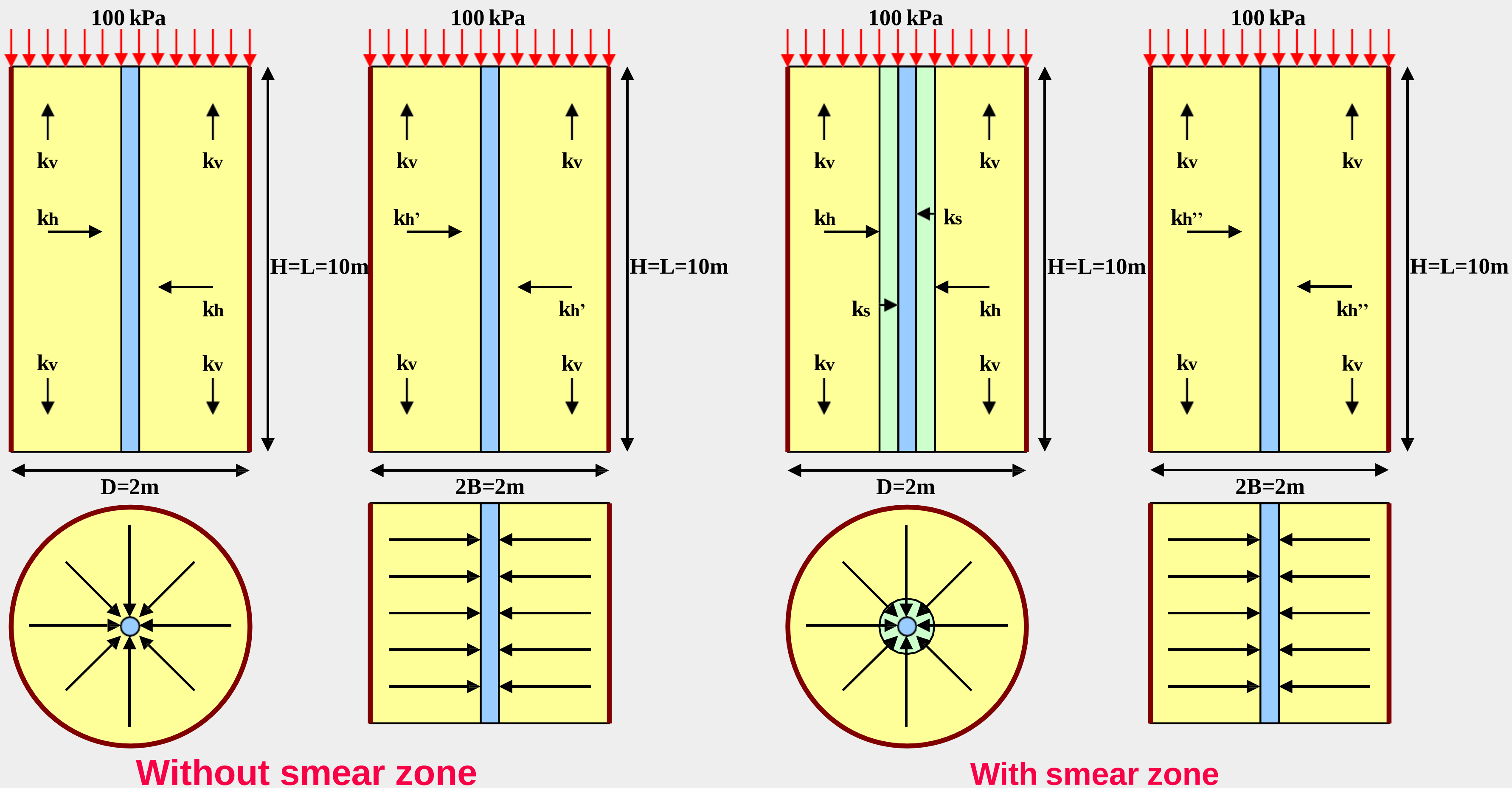
Transforming the in-situ 3D unit-cell axisymmetric condition into equivalent 2D multi-drain plane strain condition

Axisymmetric vs Equivalent Plane Strain Condition (Hird et al., Indraratna et al., Chai et al.)





Equivalent Horizontal Permeability for Plane Strain Condition – Hird et al. (1992)



Transforming the in-situ 3D unit-cell axisymmetric condition into equivalent 2D multi-drain plane strain condition

Equivalent Horizontal Permeability for Plane Strain Condition – Hird et al. (1992)

$$\frac{k_{hp}}{k_h} = \frac{2B^2}{3R^2\left[\ln\left(\frac{R}{r_s}\right) + \left(\frac{k_h}{k_s}\right)\ln\left(\frac{r_s}{r_w}\right) - \frac{3}{4}\right]}$$

(for the permeability matching is obtained by substituting *B=R=1m)*

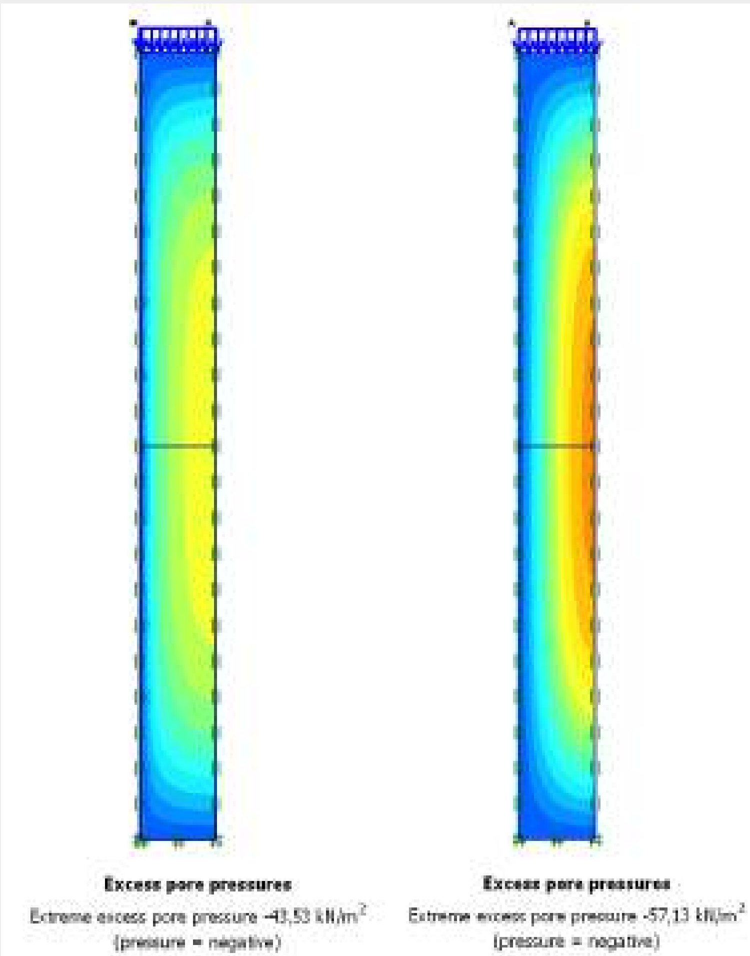
$$\frac{k_{hp}}{k_h} = \frac{0.67}{\left[\ln(n) - 0.75\right]} \qquad n = \frac{d_e}{d_w}$$

(if smear effects are ignored)

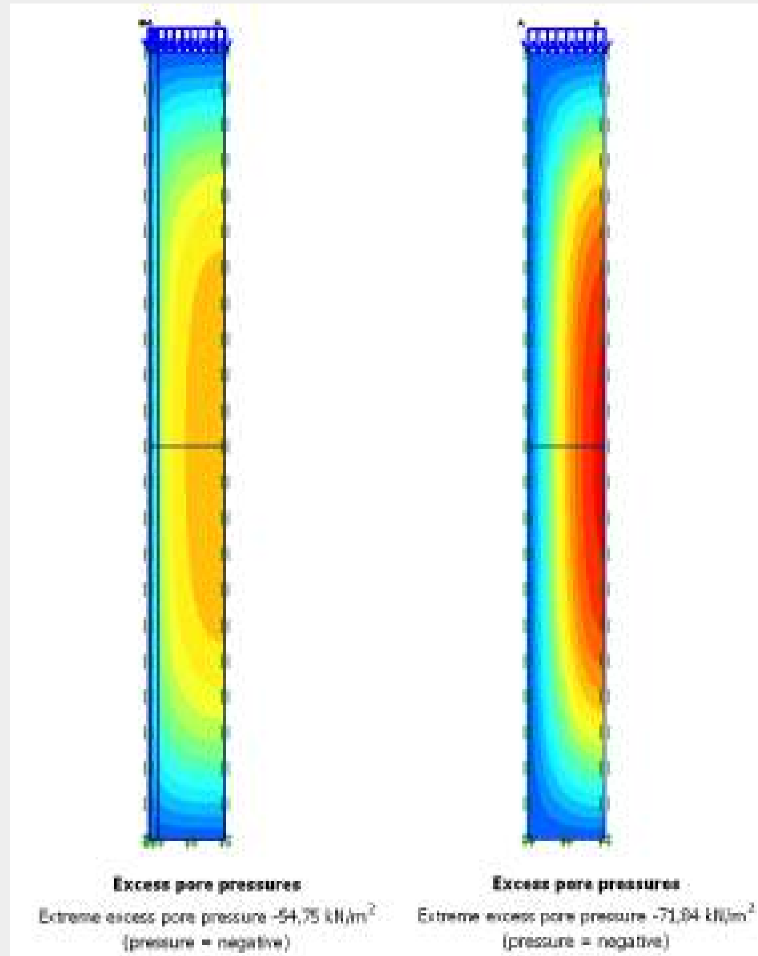
Where,
khp = equivalent horizontal permeability in plane strain condition (with and without smear zone),
kh = undisturbed horizontal permeability = 4.32x10-4 m/day (kh=kv),
ks = horizontal permeability in smear zone = 2.15x10-4 m/day,
de = equivalent influence zone diameter = 2m,
dw = equivalent drain diameter = 52mm,
ds = equivalent smear zone diameter = 220mm.

Equivalent Horizontal Permeability (Hird et al. 1992)			
k_{hp}	9.98×10^{-5}	m/day	equivalent horizontal permeability without smear zone
$k_{hp'}$	6.63×10^{-5}	m/day	equivalent horizontal permeability with smear zone

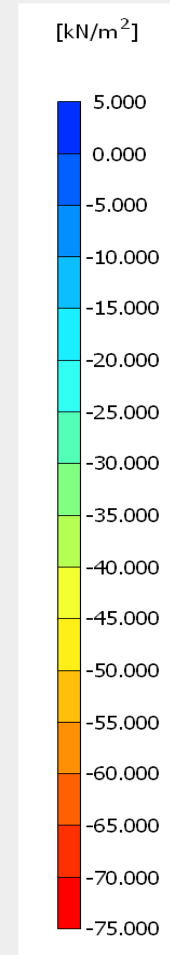
Excess Pore Pressure - Mohr Coulomb (Plane Strain - Hird et al.) - 20 days



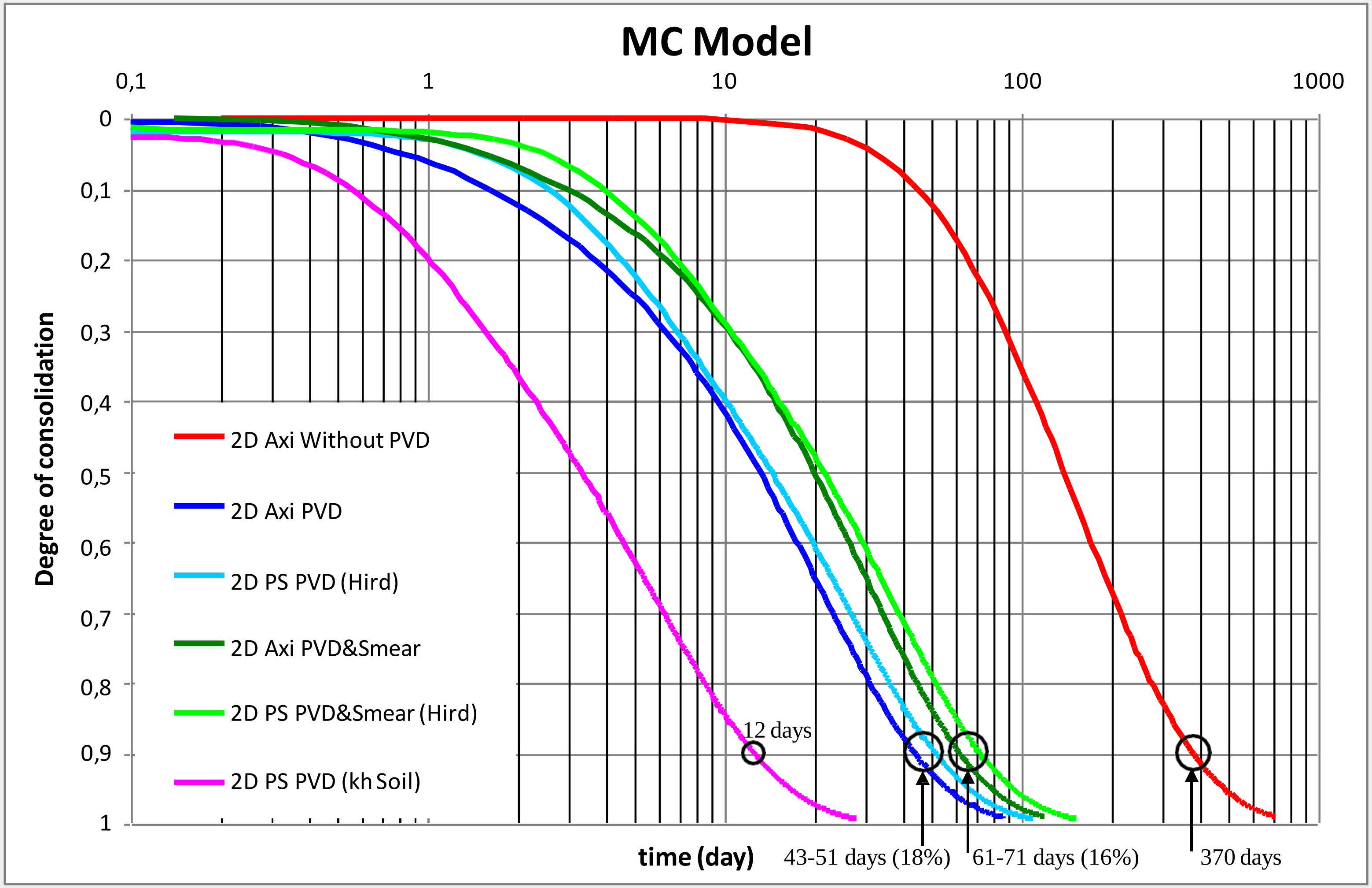
With PVD (Axi vs PS)



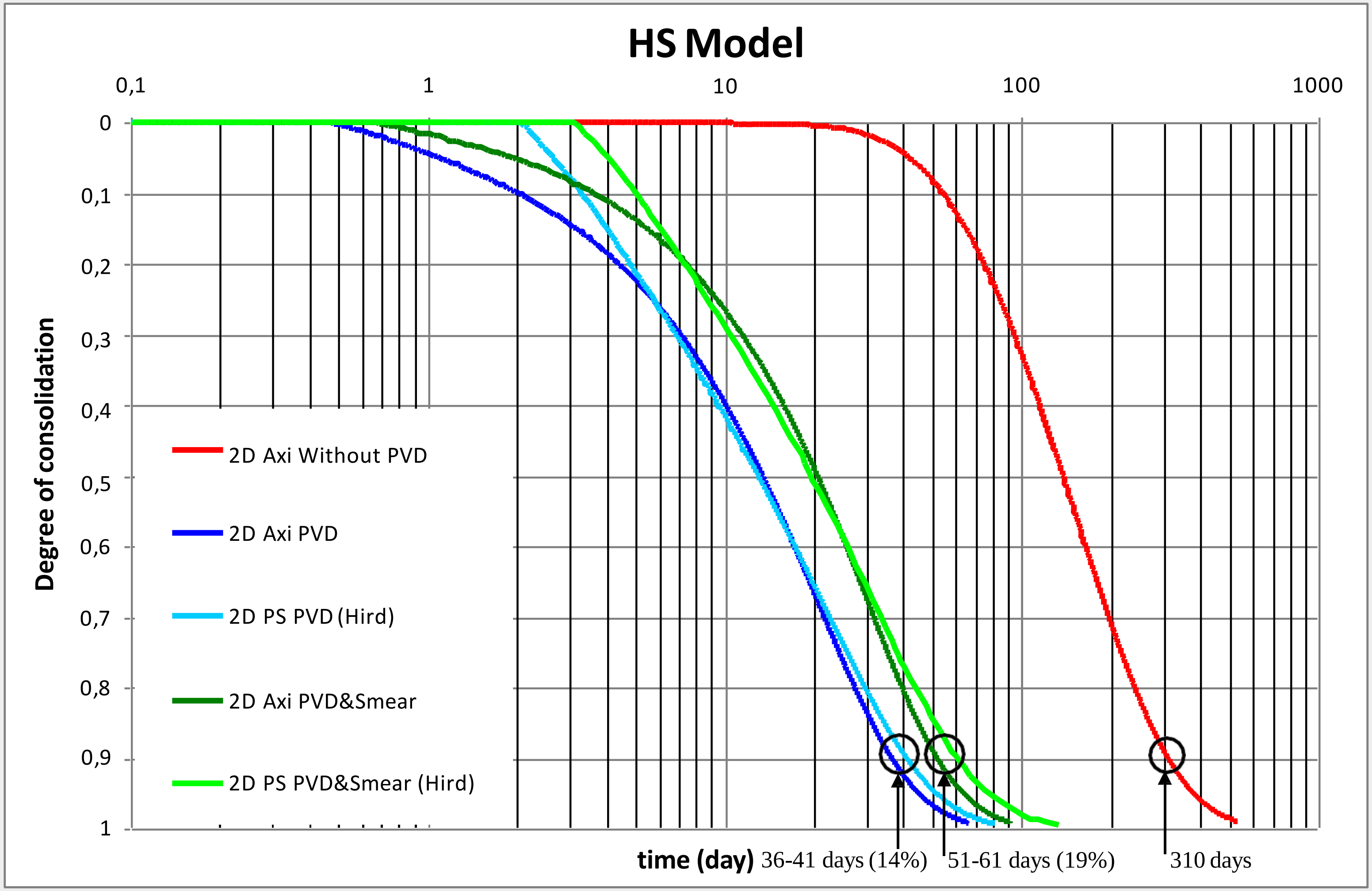
With PVD and Smear Zone (Axi vs PS)



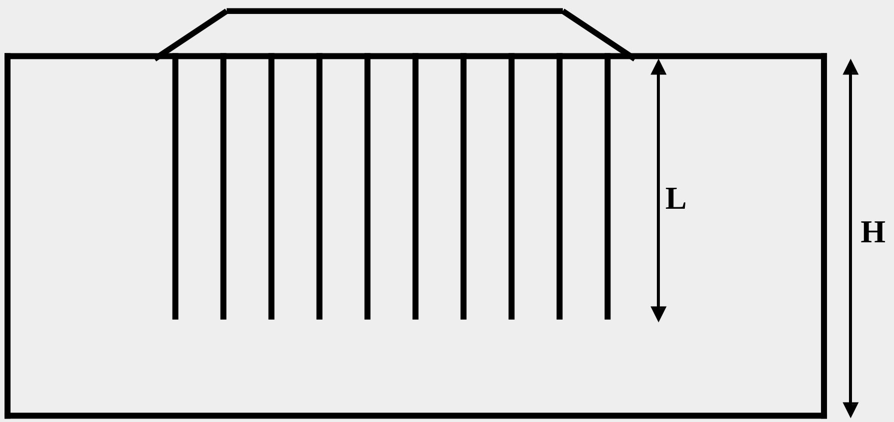
Degree of Consolidation for Axisymmetric vs Equivalent Plane Strain Condition (Hird et al.)



Degree of Consolidation for Axisymmetric vs Equivalent Plane Strain Condition (Hird et al.)



Partially Penetration of Vertical Drains (Floating PVD)

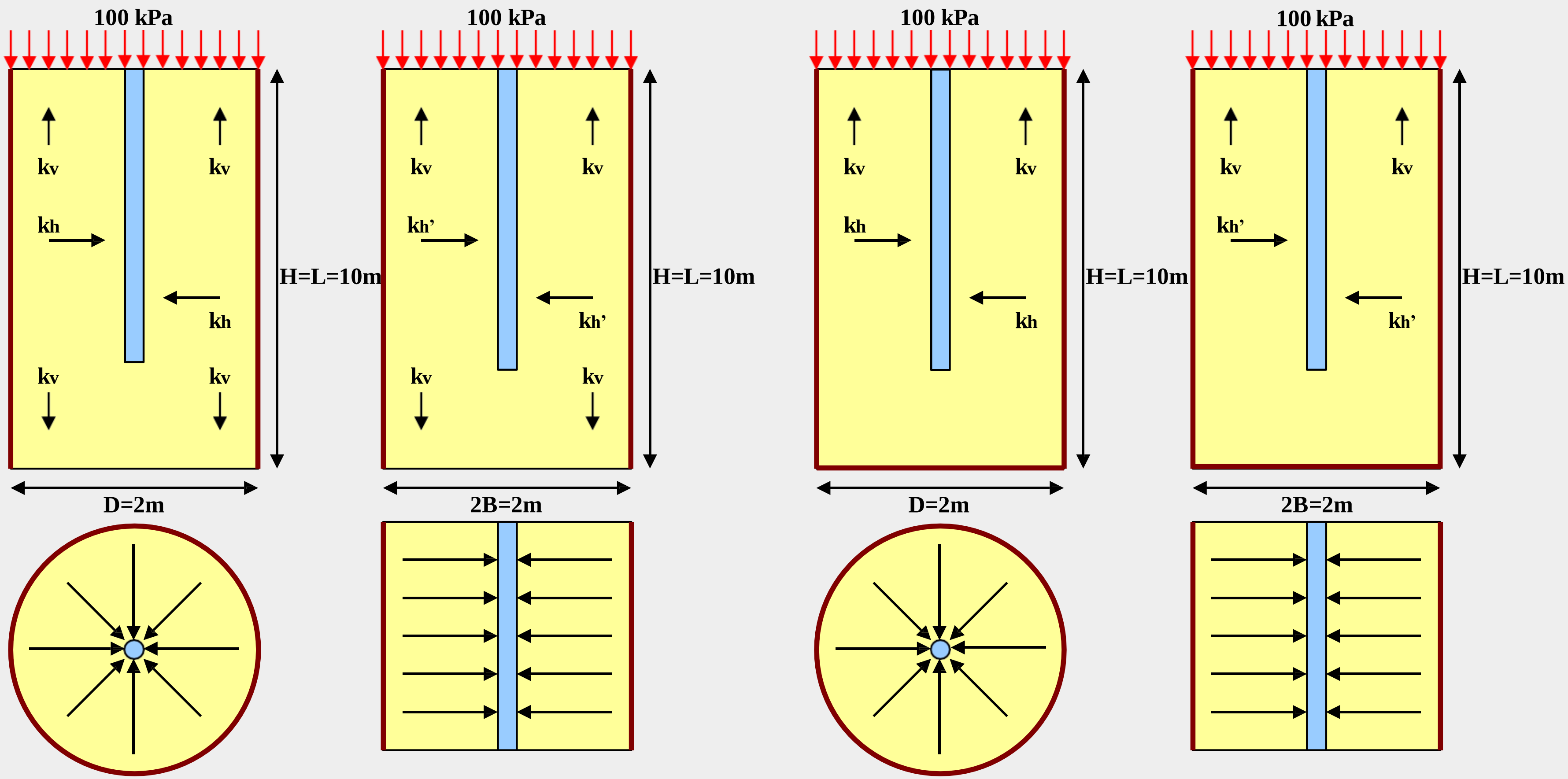


- Double Drained
- Single Drained

L/H = 1; 0.9; 0.8; 0.7; 0.6; 0.5; 0.4; 0.3; 0.2; 0.1; 0

Influence of PVD Penetration Depth

(Lpvd = 10m, 9m, 8m, 7m, 6m, 5m, 4m, 3m, 2m, 1m, 0m)



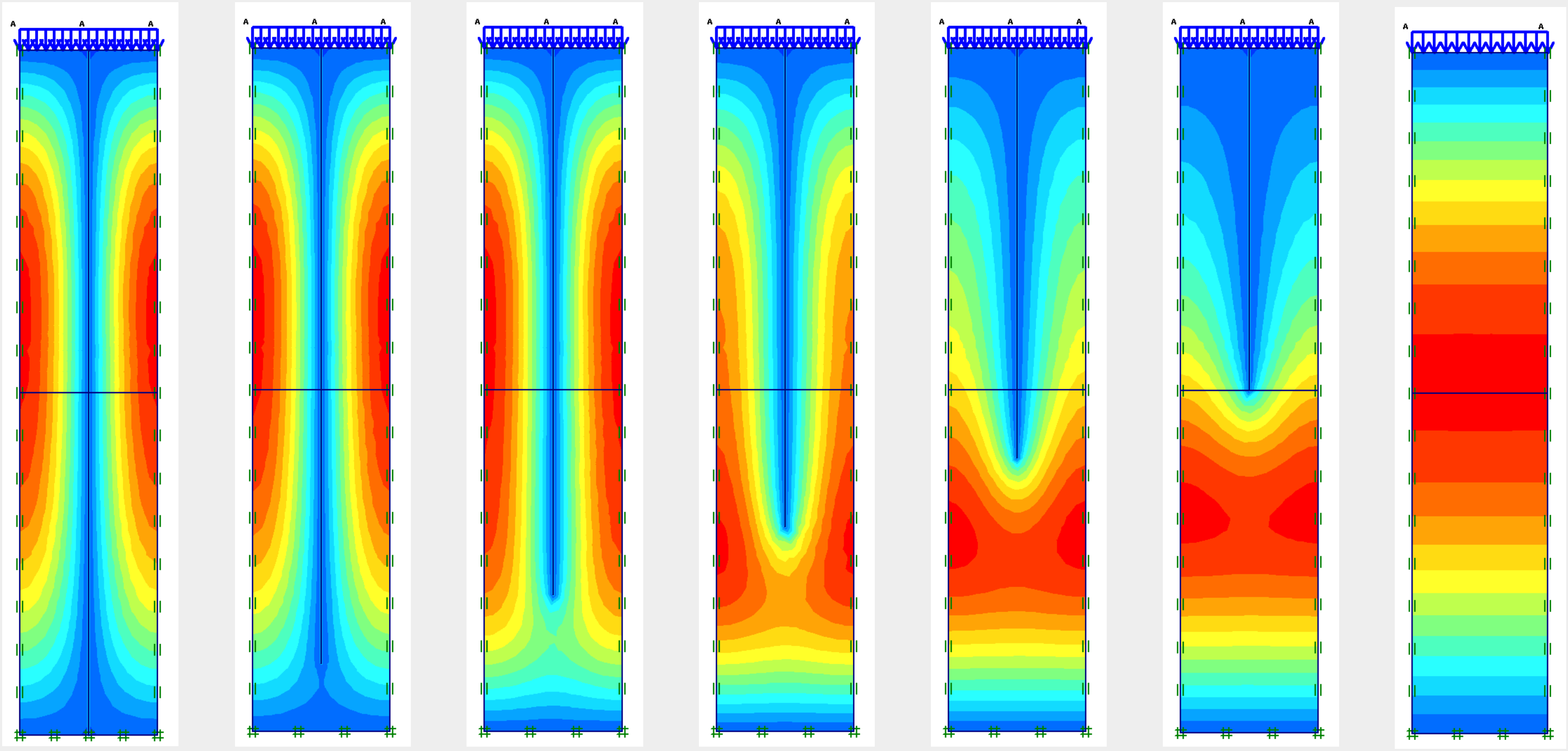
Double Drained

Single Drained

(De=2m, Hsoil=10m, P=100KPa, GWT=0m, PVD Without smear zone, Axi and PS)

Influence of PVD Penetration Depth (Double Drained)

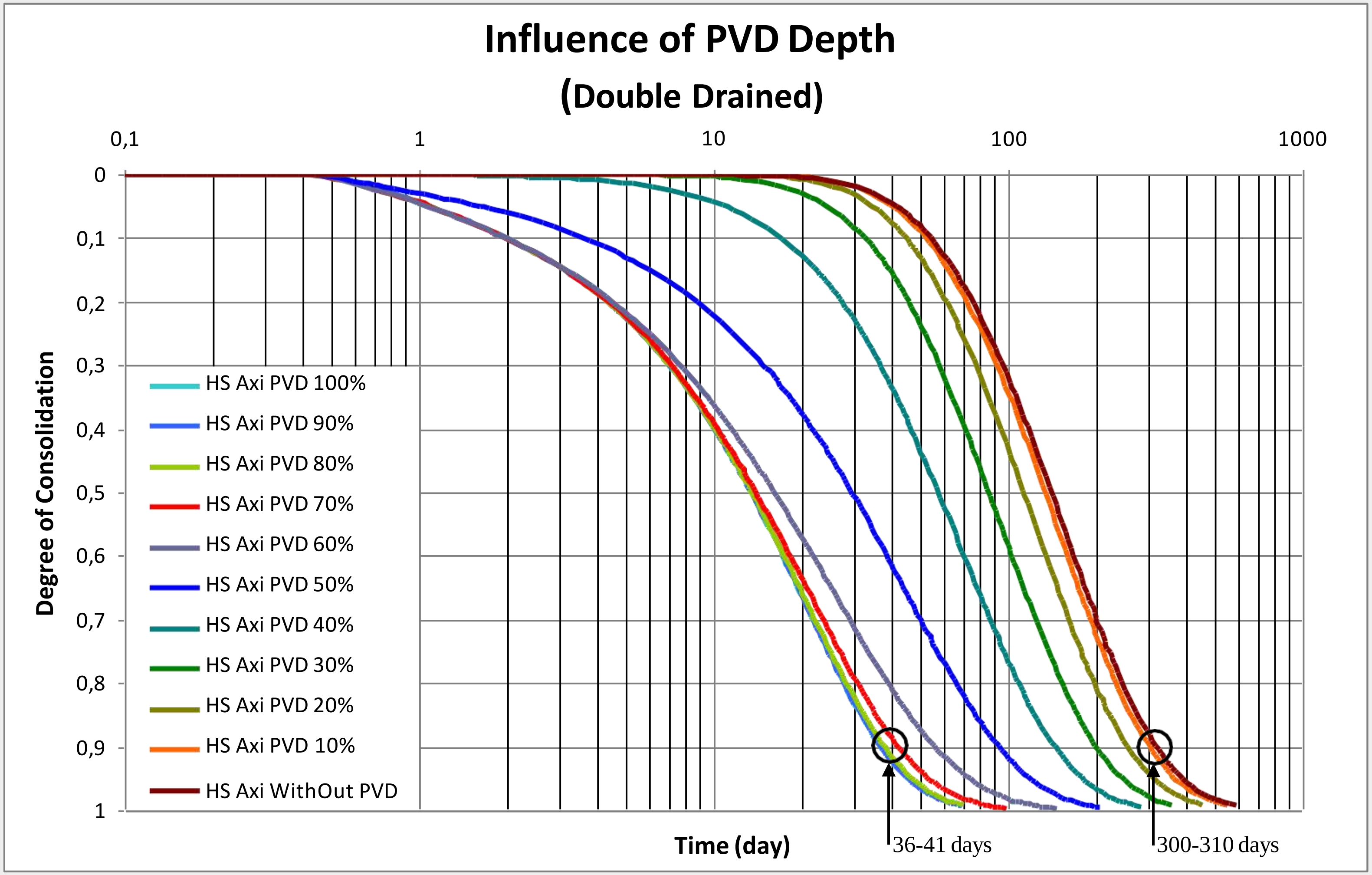
(Lpvd = 10m, 9m, 8m, 7m, 6m, 5m, 4m, 3m, 2m, 1m, 0m)



100% 90% 80% 70% 60% 50% 0%

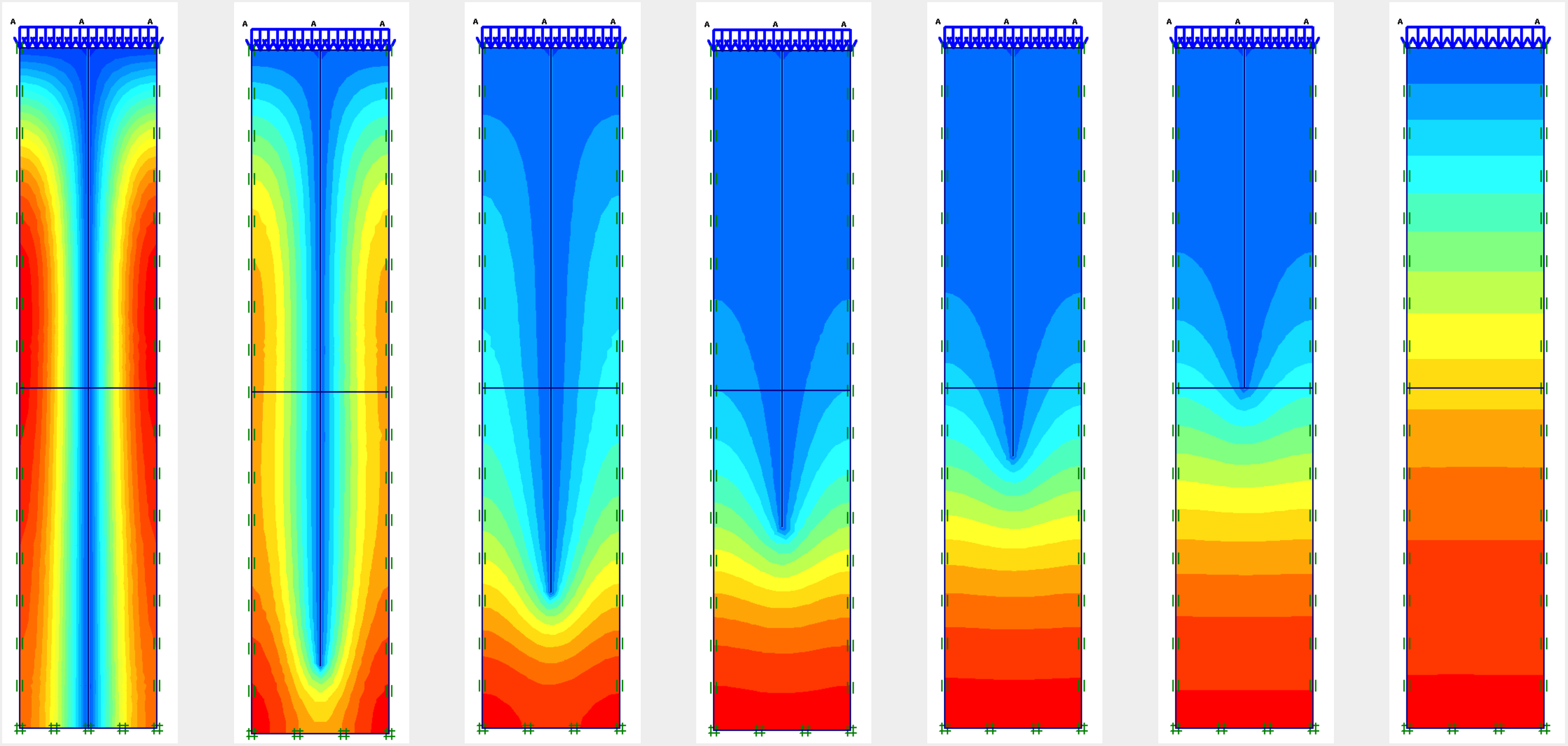
(De=2B=2m, Hsoil=10m, P=100KPa, GWT=0m, Double Drained)

Degree of Consolidation for Different PVD Penetration Depth (Axi)



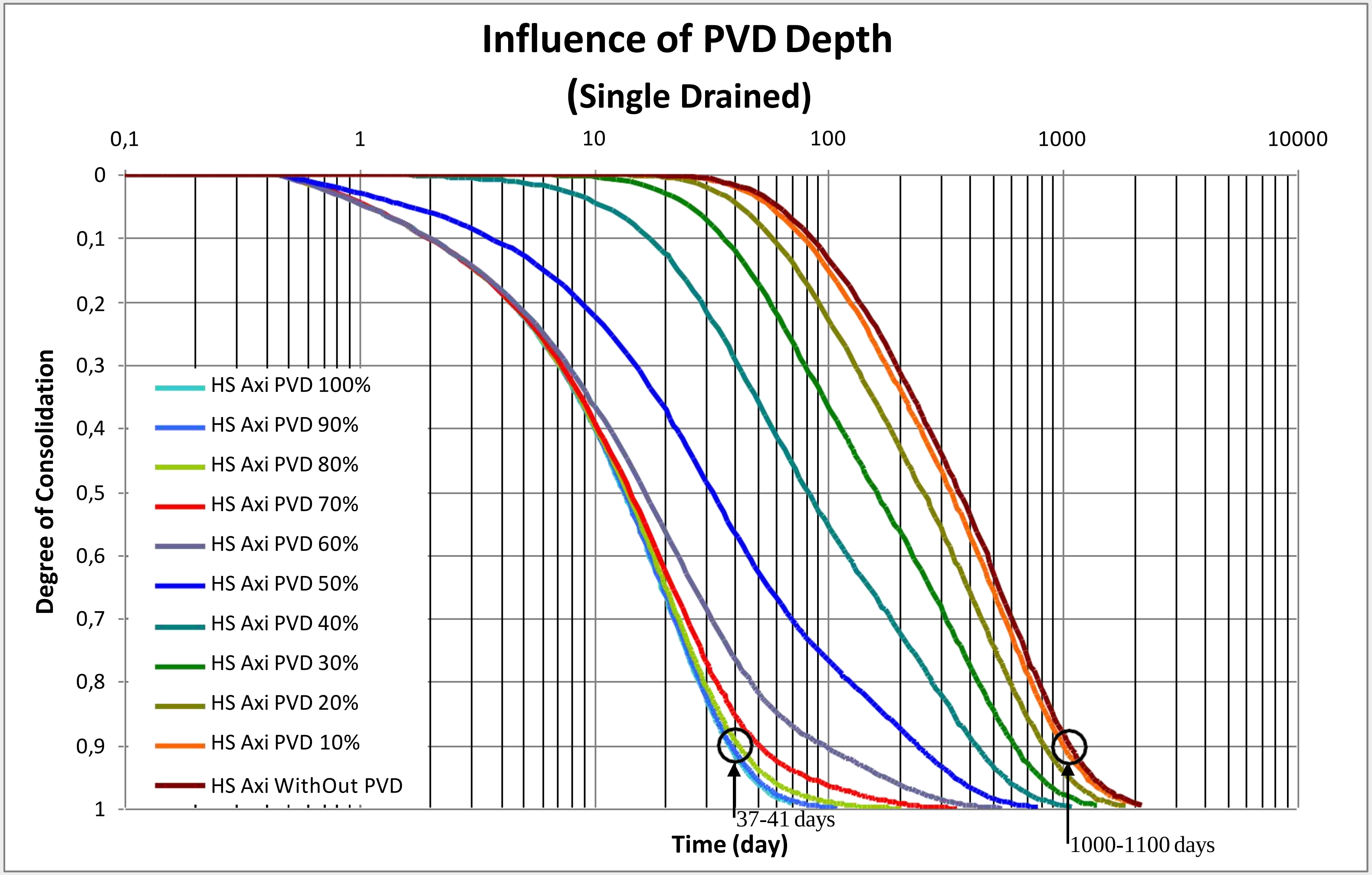
Influence of PVD Penetration Depth (Single Drained)

(Lpvd = 10m, 9m, 8m, 7m, 6m, 5m, 4m, 3m, 2m, 1m, 0m)

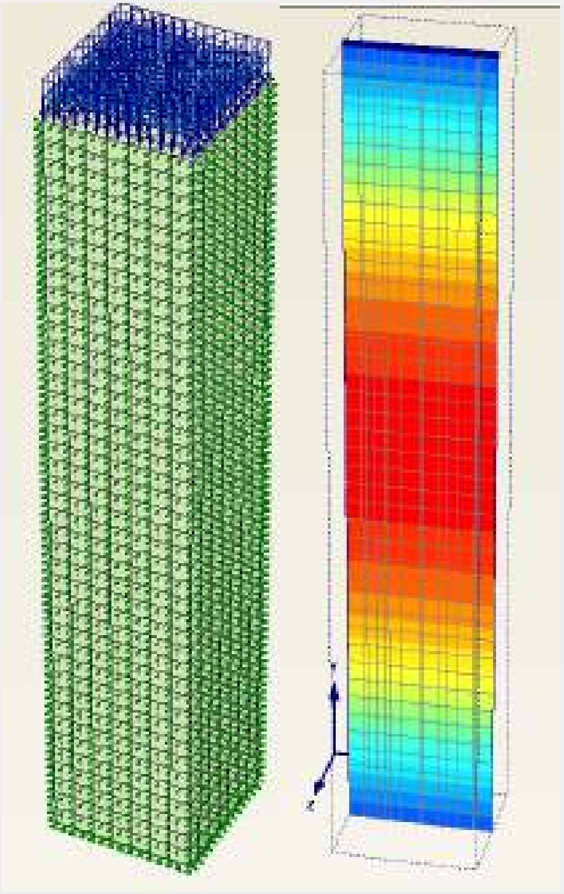


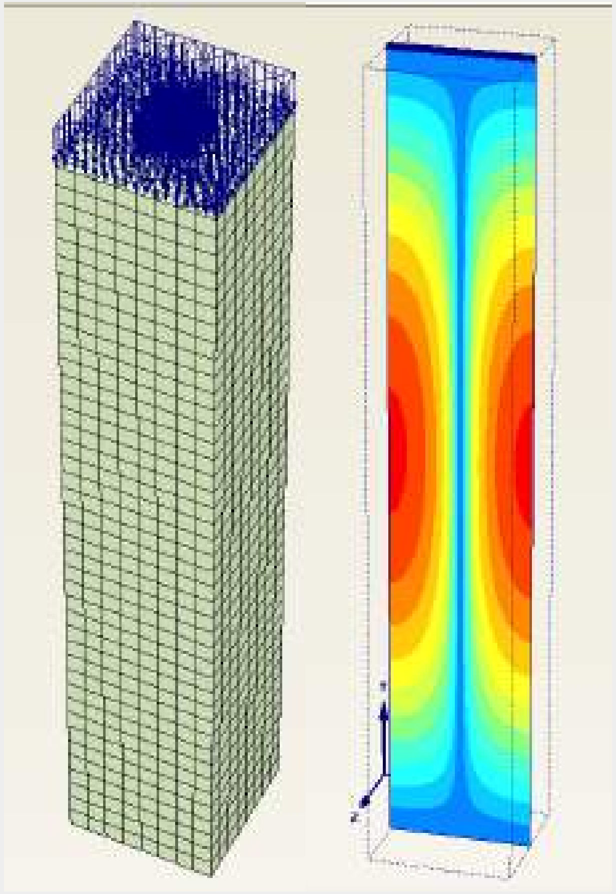
(De=2m, Hsoil=10m, P=100KPa, GWT=0m, Single Drained)

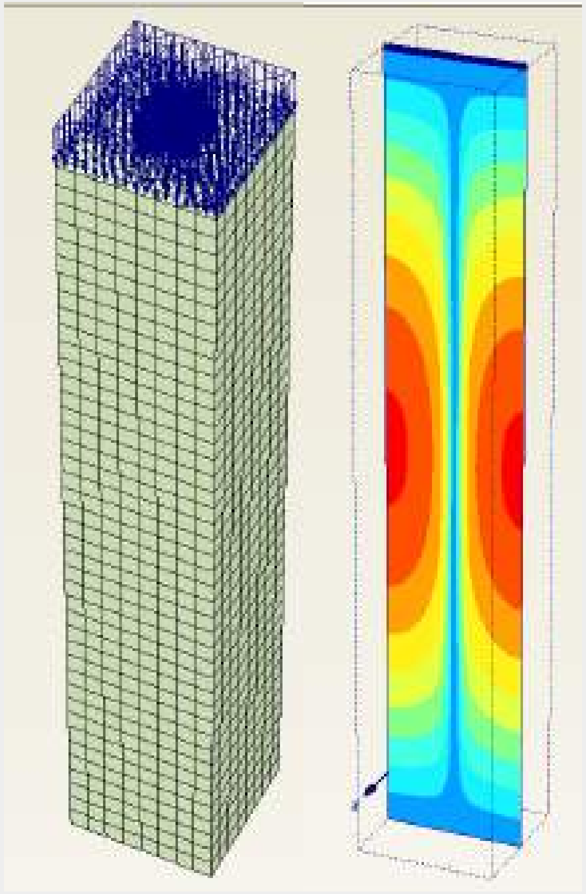
Degree of Consolidation for Different PVD Penetration Depth (Axi)



3D Model - Square Pattern

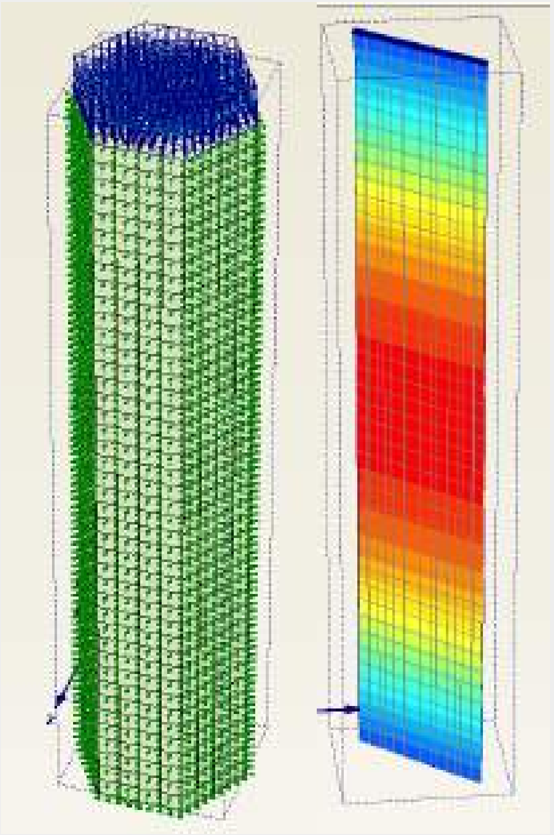




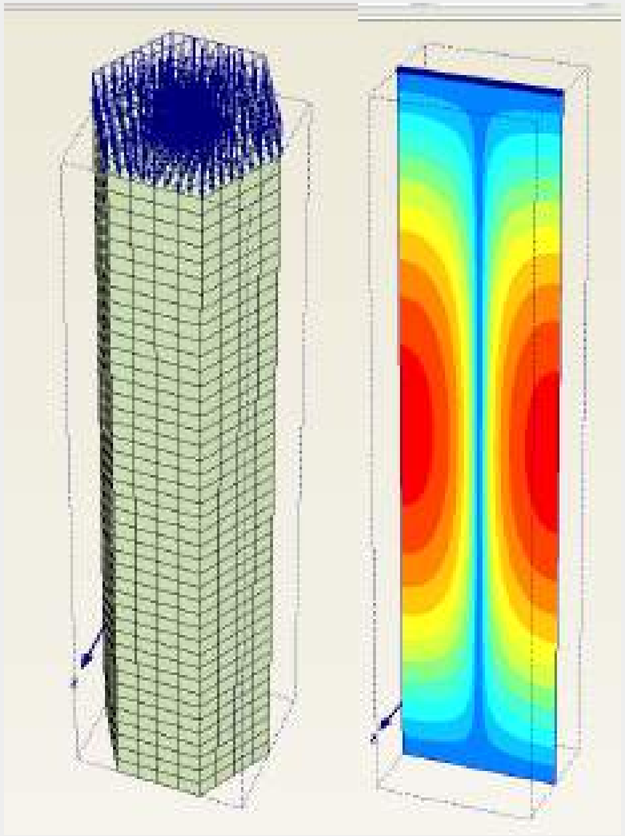


Without PVD **With PVD** **With PVD and Smear Zone**

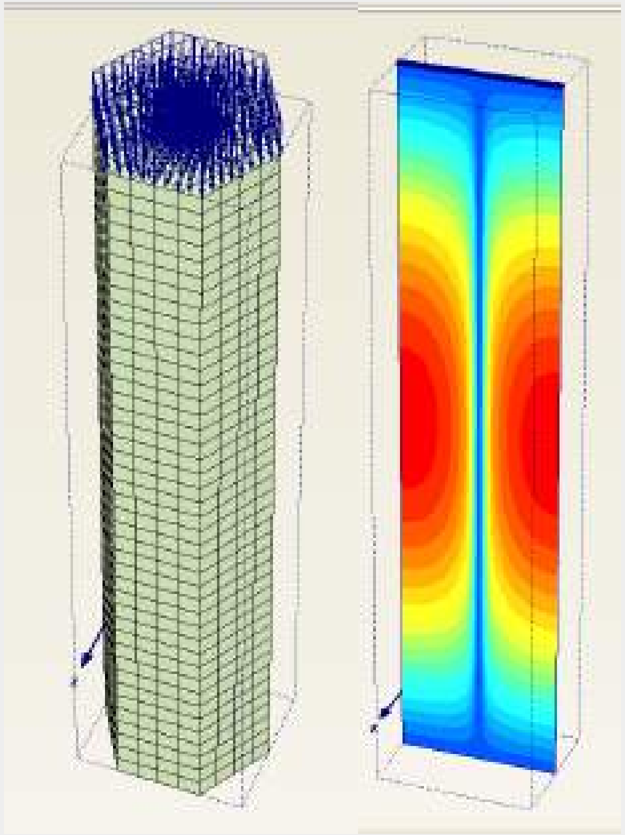
3D Model - Triangular Pattern



Without PVD

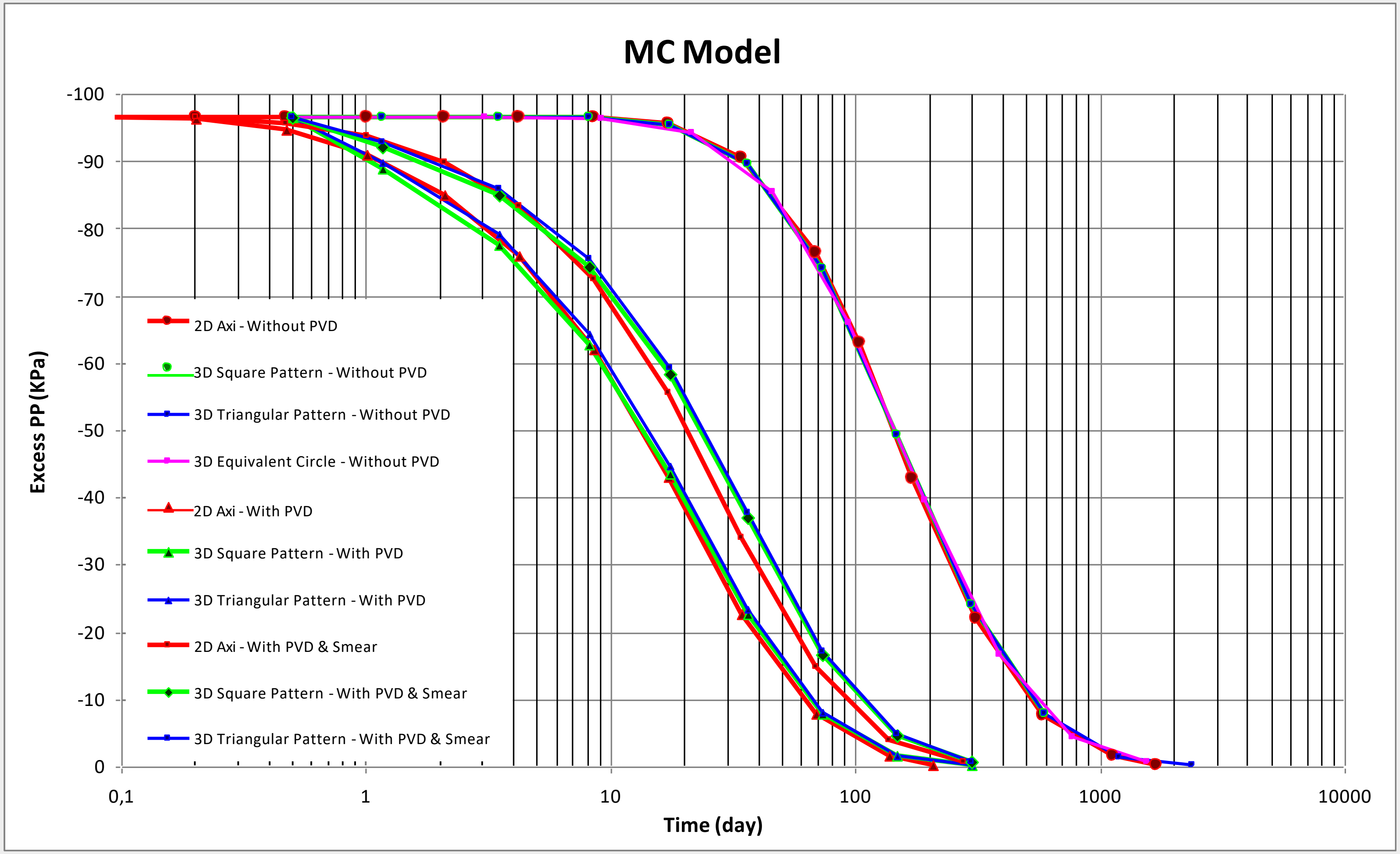


With PVD

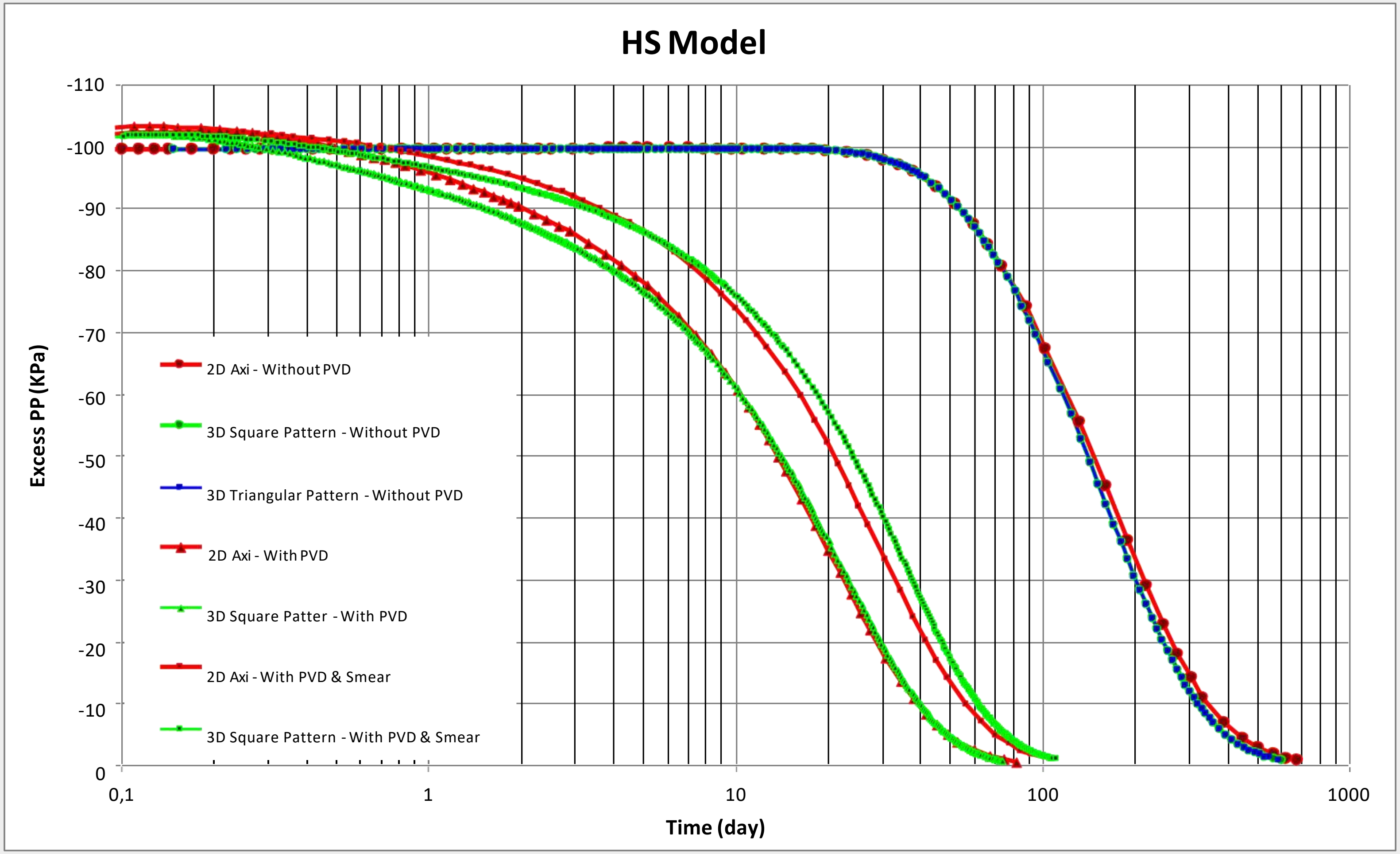


With PVD and Smear Zone

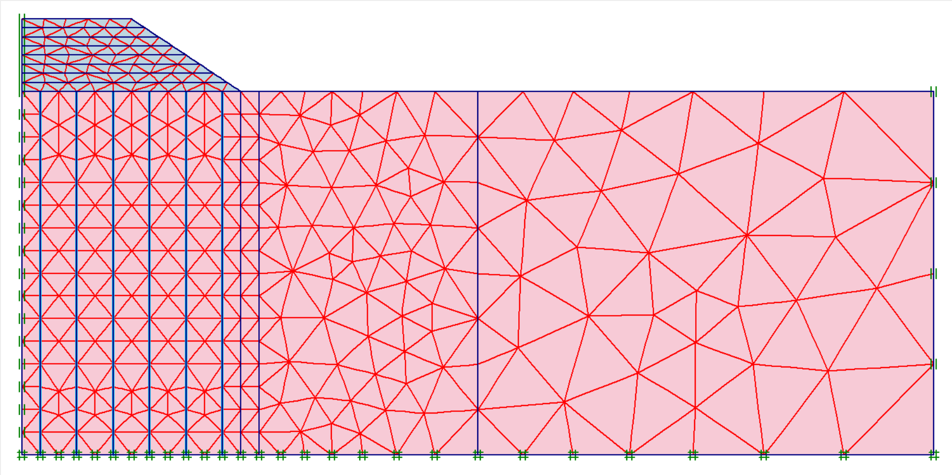
Excess PP for Different 3D Condition (2D axi vs 3D)



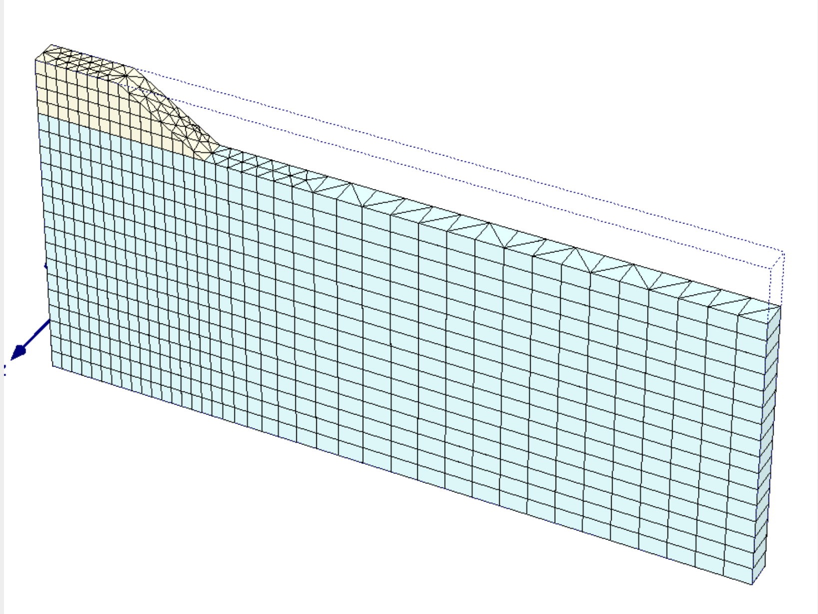
Excess PP for Different 3D Condition (2D axi vs 3D)



2D and 3D Embankment Model

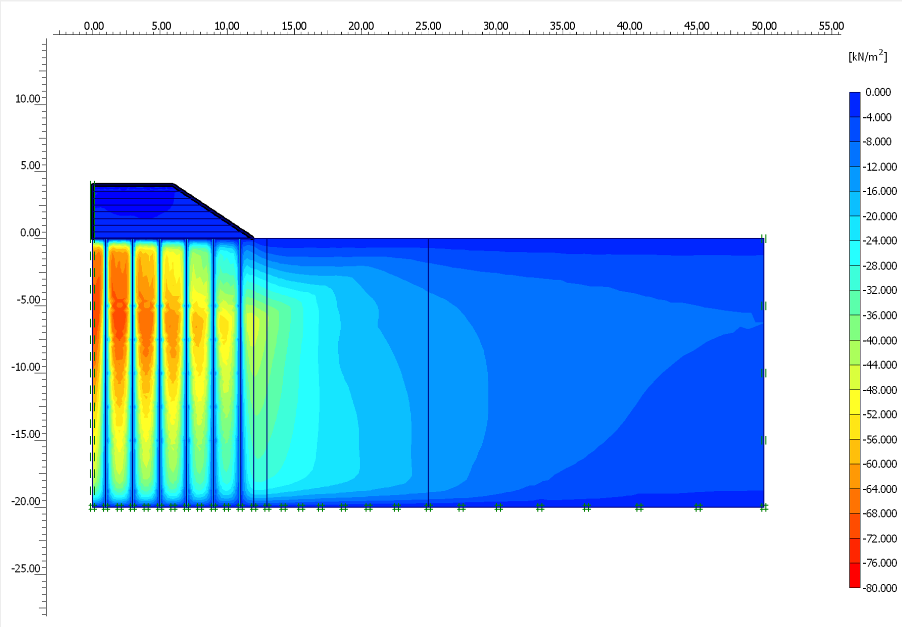


2D Plane Strain Model

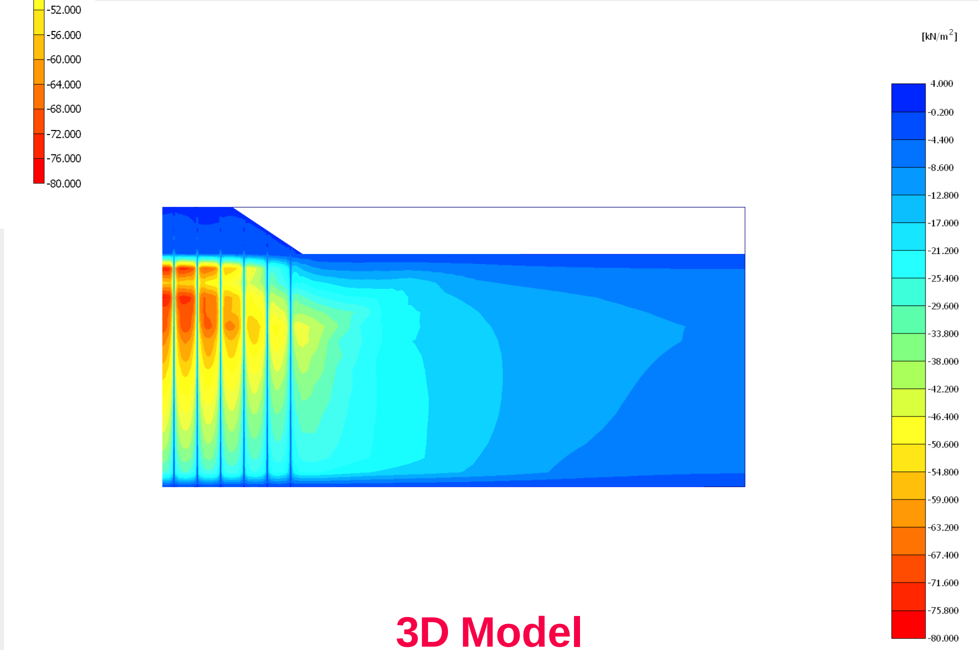


3D Model

Excess PP at end of Construction – HS Model

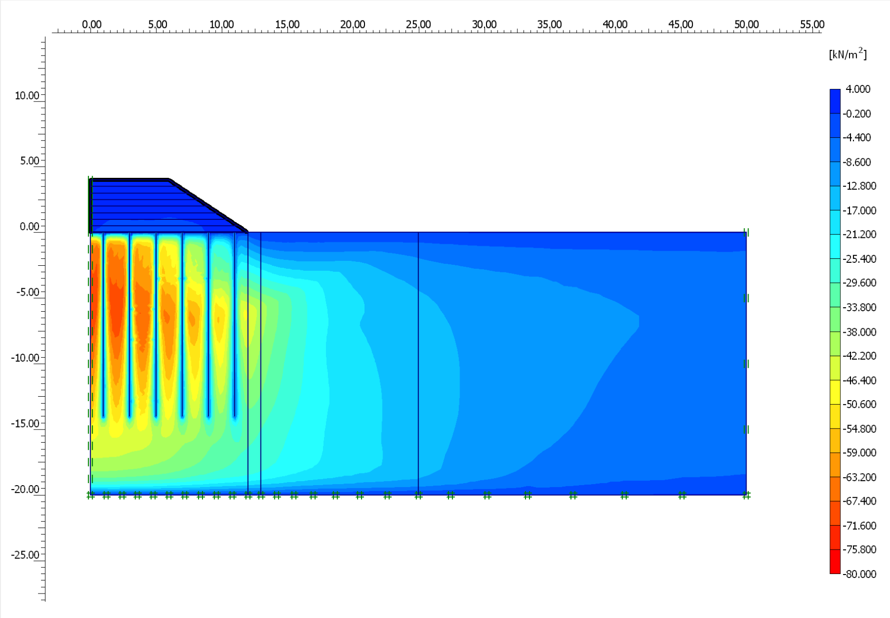


2D Plane Strain Model

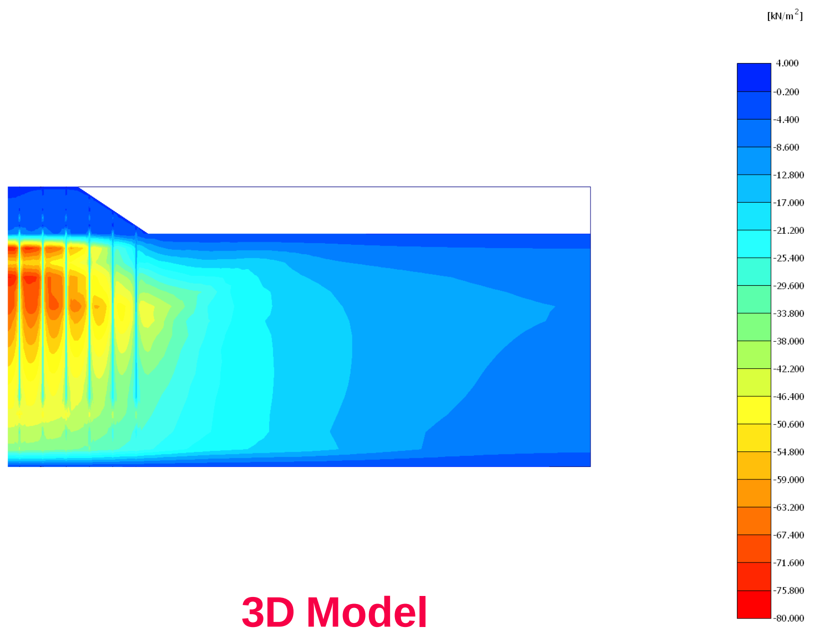


3D Model

Excess PP at end of Construction – HS Model (Floatiing PVD)

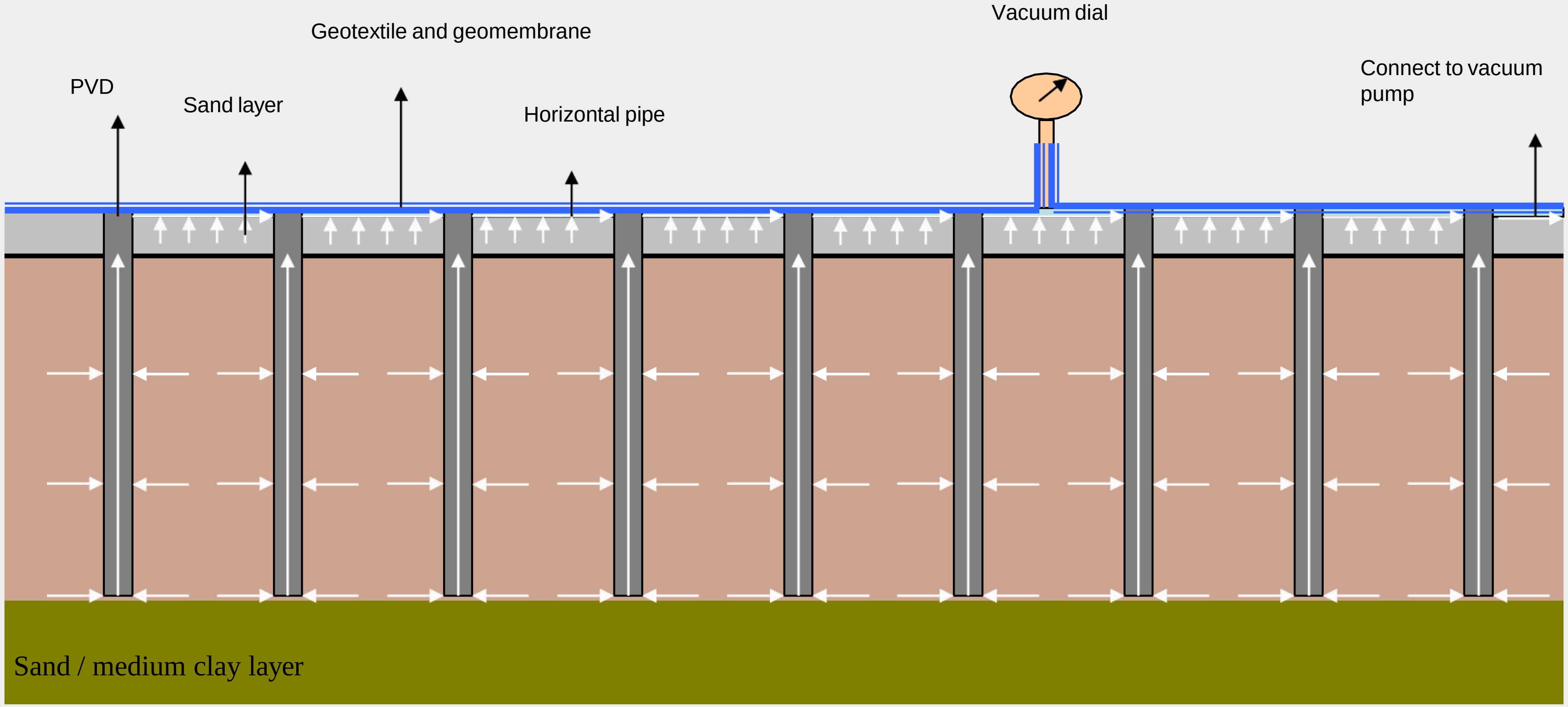


2D Plane Strain Model



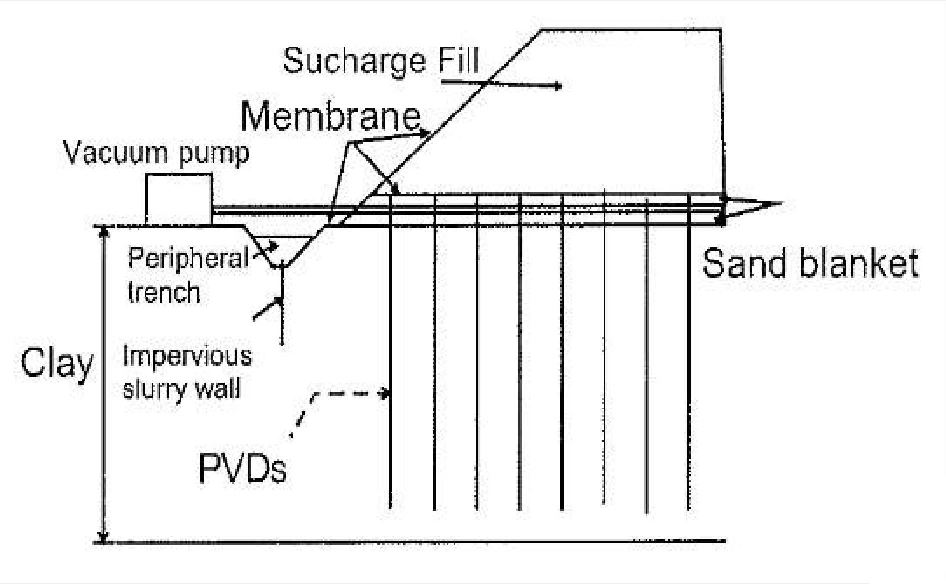
3D Model

PVD with Vacuum Preloading

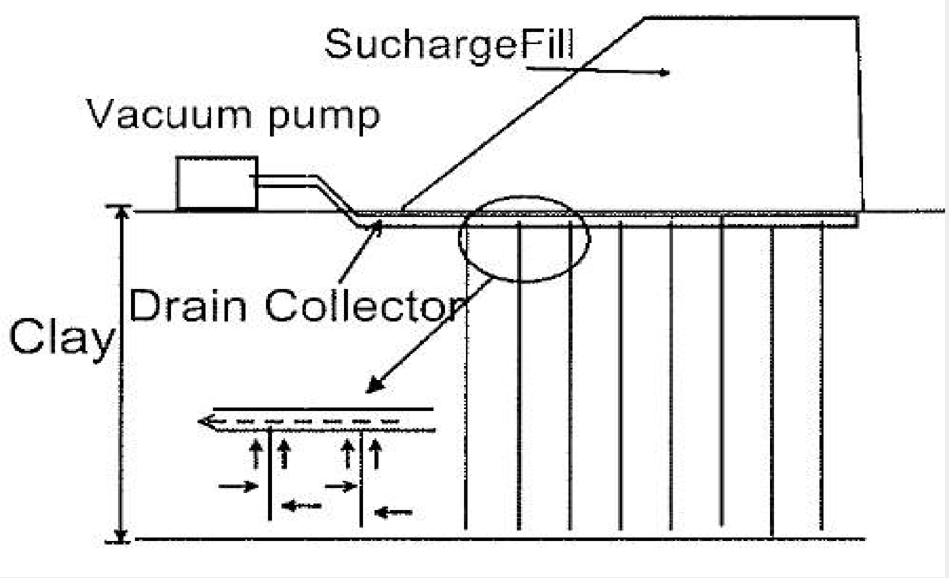


- Kjellman first introduce a vacuum preloading to accelerate consolidation in 1952 and a small field test was perform.
- In this method the soil load is replace by atmospheric pressure

PVD with Vacuum Preloading

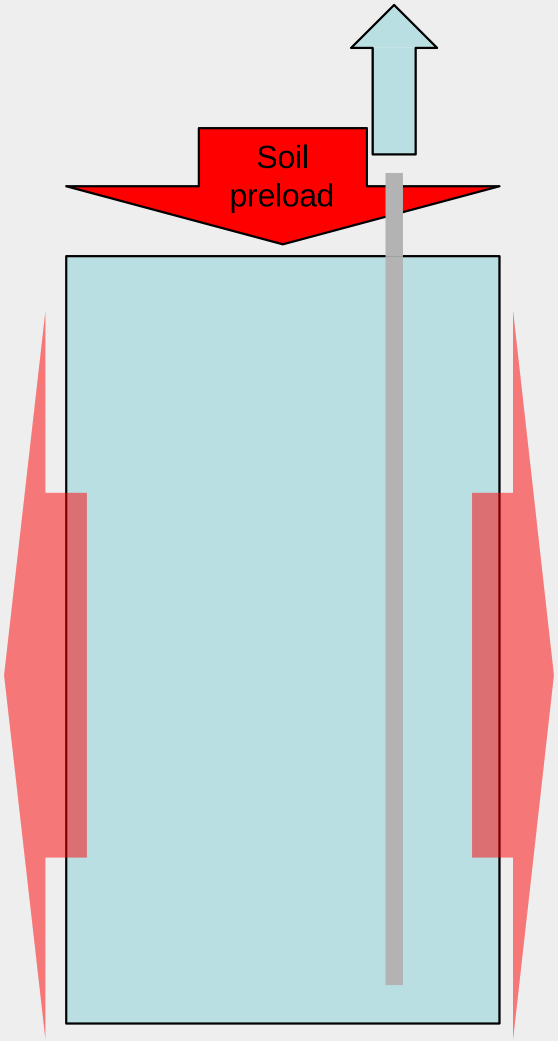


Vacuum with Membrane



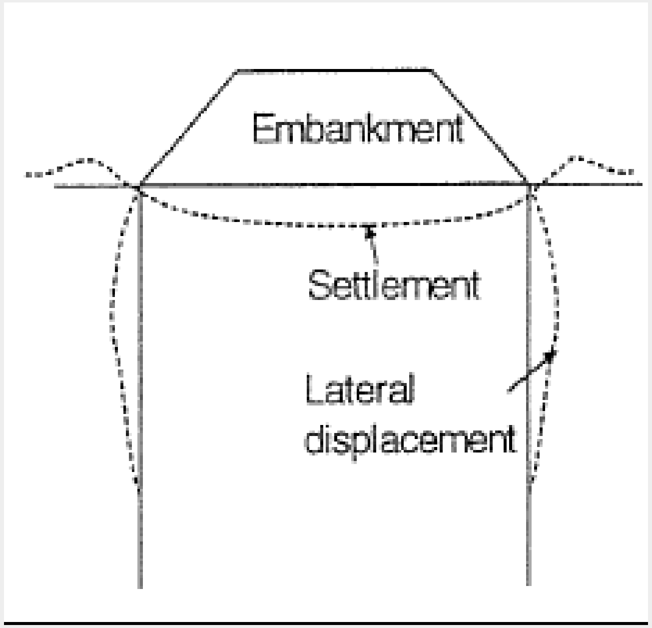
Vacuum without Membrane

Vacuum vs Non Vacuum

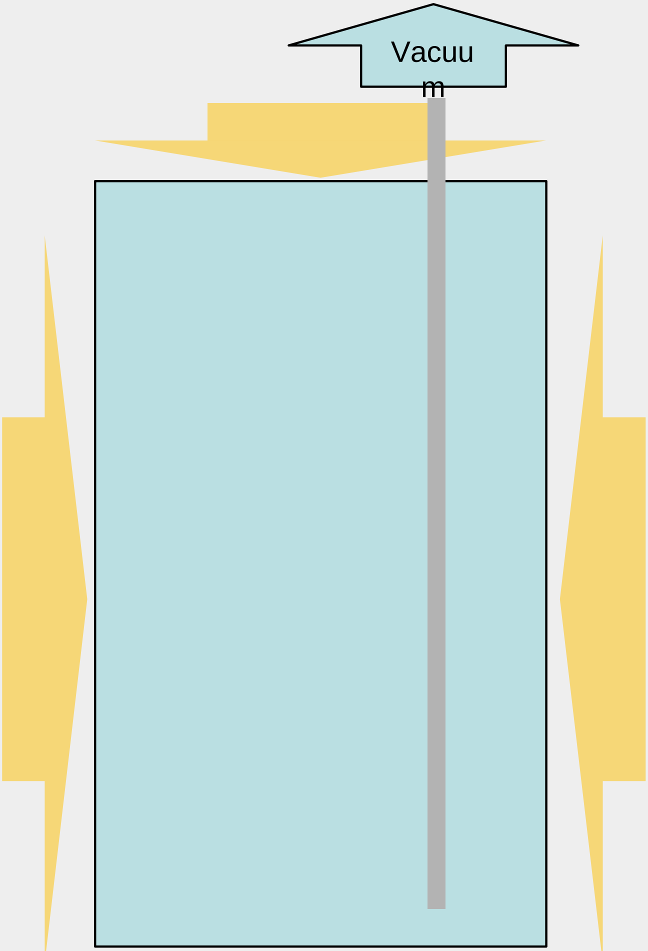


Type	Total stress	Initial /boundary condition	Process	Effect
Vertical drain with surcharge	$\sigma\uparrow(\Delta\sigma)$	$u\uparrow(\Delta u)$ σ'	$\Delta u \rightarrow \sigma'$	$\sigma'\uparrow$ $(\Delta u \approx \Delta\sigma)$

$$\sigma = u + \sigma'$$

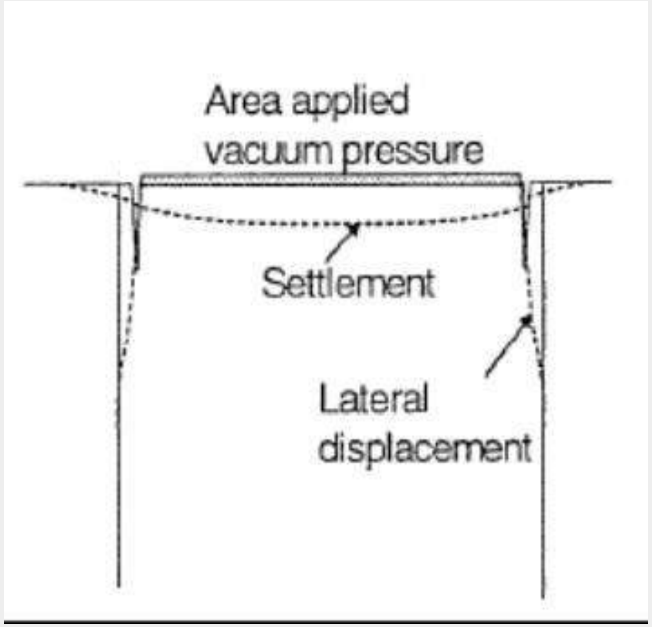


Vacuum vs Non Vacuum



Type	Total stress	Initial /boundary condition	Process	Effect
Vacuum	σ	$u(soil)$ $-u^*(boundary)$ σ'	$u\downarrow$ $(\Delta u \approx -u^*)$	$\sigma'\uparrow$ $(\Delta u \approx -u^*)$

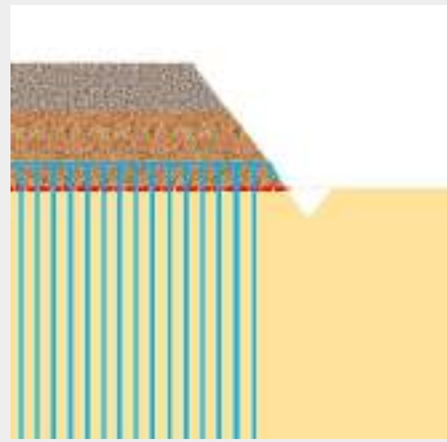
$$\sigma = u + \sigma'$$

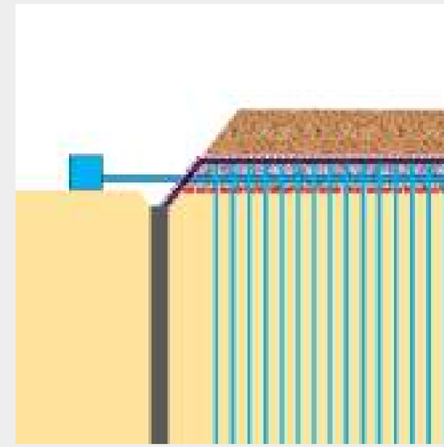


Vacuum vs Non Vacuum

PVD with Conventional Soil Prelading

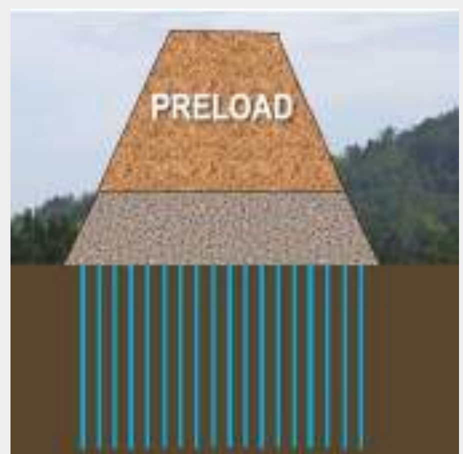
1. Higher Surcharge
2. Lower Stability
3. Greater Lateral Movement
4. Many heavy equipment
5. Longer Construction Time



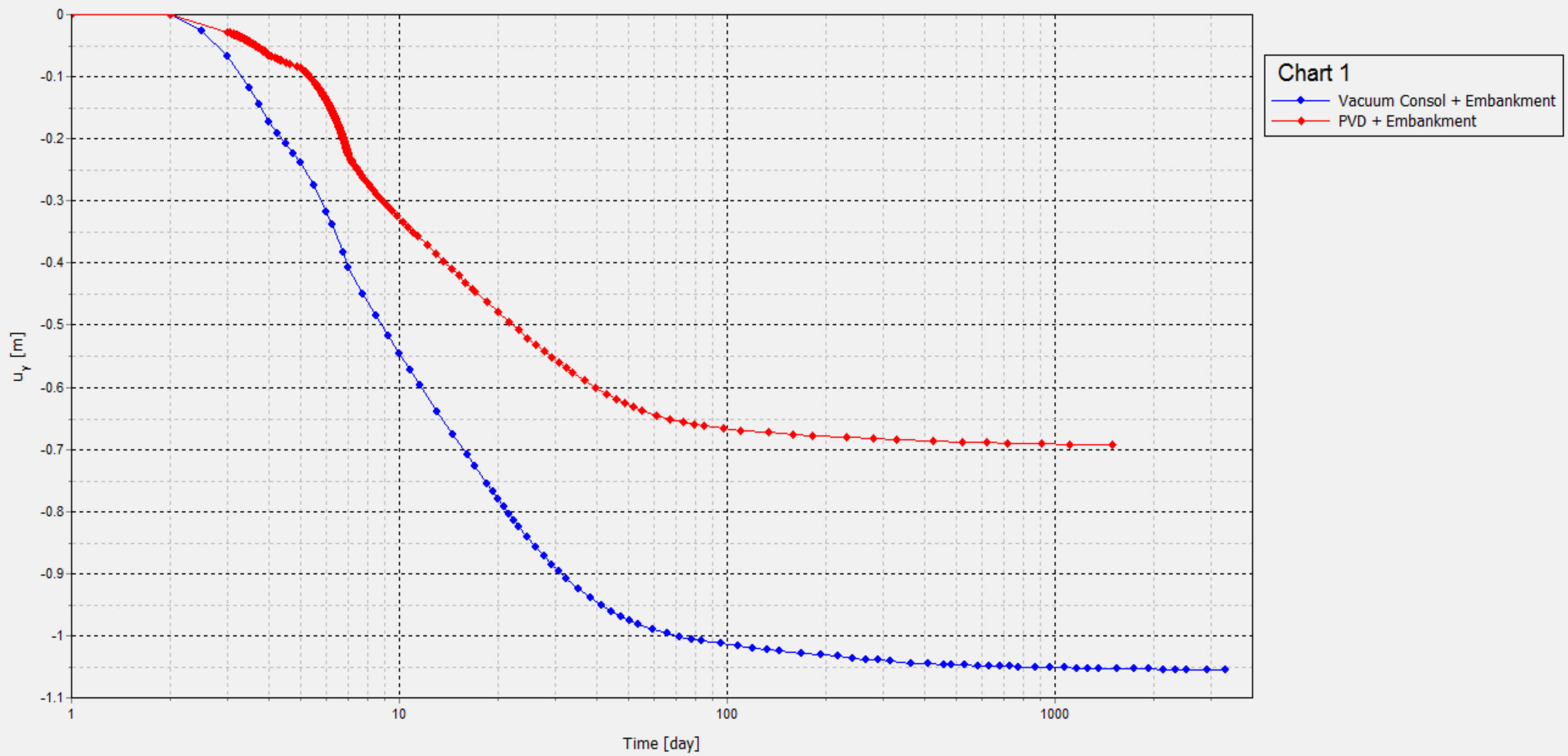


PVD with Vacuum Preloading

1. Lower or no Surcharge
2. Better Control of Stability
3. Less Lateral Movement
4. Less heavy equipment
5. Shorter Construction Time



Vacuum vs Non Vacuum



Vertical Displacement – Comparison PVD with Conventional Soil Prelading (four meters surcharge) vs PVD with Conventional Soil Prelading + Vacuum Preloading (four meters surcharge + 80 KPa Vacuum Pressure)

Conclusion

- Analytical methods for evaluating the effect of vertical drains usually assume consolidation within a uniform soil column with linear compressibility characteristics without any lateral movement.

- Numerical methods provide a less restrictive analysis compared to analytical methods, such as: advanced soil models, multiple drain analysis, multi layer subsoil condition, staged construction, non uniform load distribution (lateral displacement and stability analysis), variation of drains configuration (spacing and floating), effects of a sand mat and reinforcement with geosynthetic, etc.

- The equivalent 2D plane strain approach is sufficient to conduct multi-drain analysis in large projects, where the application of a 2D plane strain model is efficient both from a computational time point of view and with respect to the accuracy of the predictions (compared with a full 3D model and 2D Axisymmetric model).

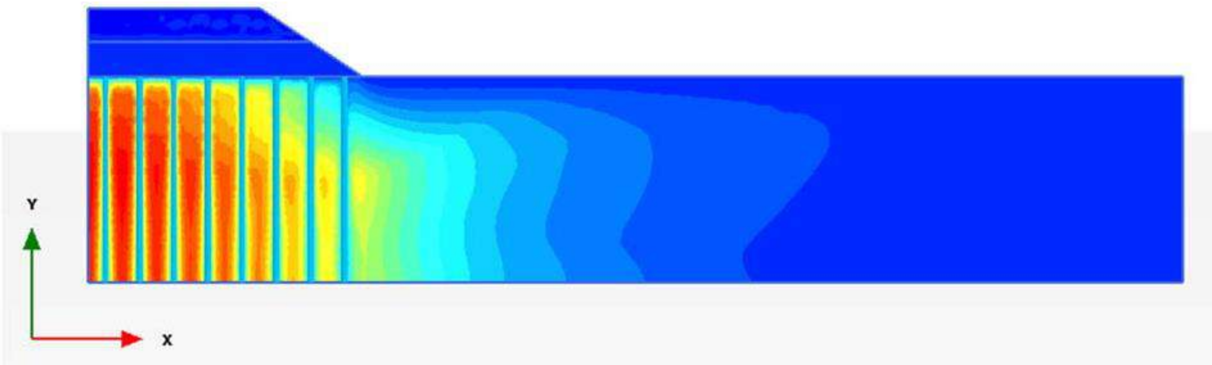
Conclusion

- For floating PVDs, the drain length can be reduced to 70% (double drained) and 80% (single drained) of the entire soft clay thickness without significantly affecting the consolidation process.

- The Advantages Vacuum Consolidation Method compare with conventional soil preloading are Lower or no surcharge, better control of stability, less lateral movement, less heavy equipment, shorter construction time.

…Thank You…

CG13
EXERCISE 4: Modelling of Embankment Improved with Vertical Drains



Ikhya Ikhya

COMPUTIONAL GEOTECHNICS

INTRODUCTION

The exercise concerns in the **consolidation analysis with vertical drains** (**with and without drains**, **with and without smear effeect**, **with and without vacuum**). In order to keep the problem as simple as possible, only one soil layer is considered.

Objectives:

- Consolidation analysis
- Modelling drains
- Change of horizontal permeability (kx)
- Updated mesh analysis (large deformations)
- Introduction Vacuum Consolidation

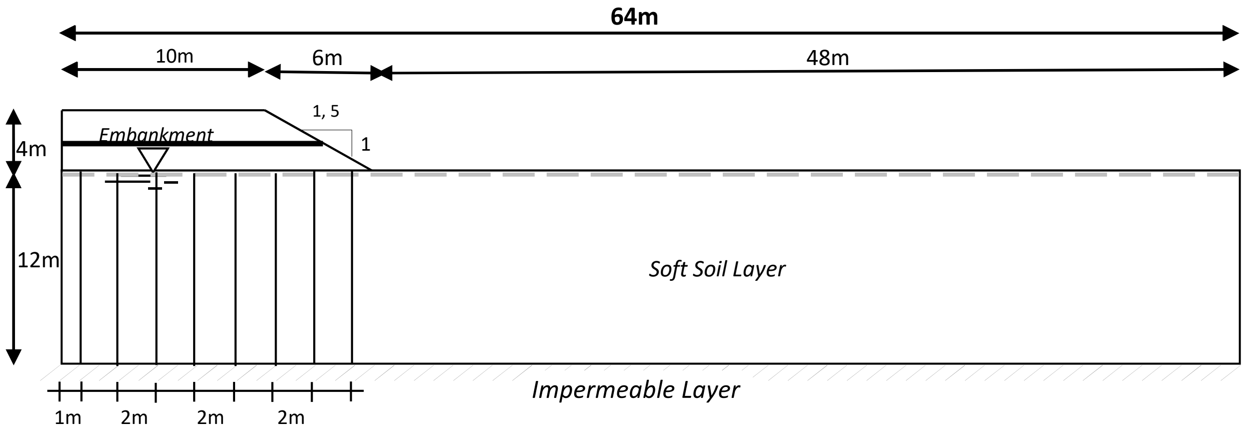


Figure 1: Scheme of the exercise

INPUT

Figure 1 shows a cross section of a **road embankment**. The embankment is **20.0 m wide** and **4.0 m high**. **The slopes** have an inclination of **1:1.5**. The problem is symmetric, so **only one half is modelled** (in this case the right half is chosen). **The embankment** itself is composed of **sandysoil**. **The subsoil** consists of **12.0 m of soft soil**.
The phreatic level is located **0 m** (at the original ground surface). **Under the soft soil layer** there is a **Impermeable Soil Layer**.

General settings

- Start the Input program and select Start a new project from the Quick select dialog box.
- In the Project tabsheet of the Project properties window, enter an appropriate title *(PVD Consolidation*).
- In the Model tabsheet make sure that Model is set to **Plane strain** and that Elements is set to **15-Noded**.
- Define the limits for the soil contour as xmin = **0.0, xmax = 64.0, ymin = −12.0 and ymax = 4.0**.

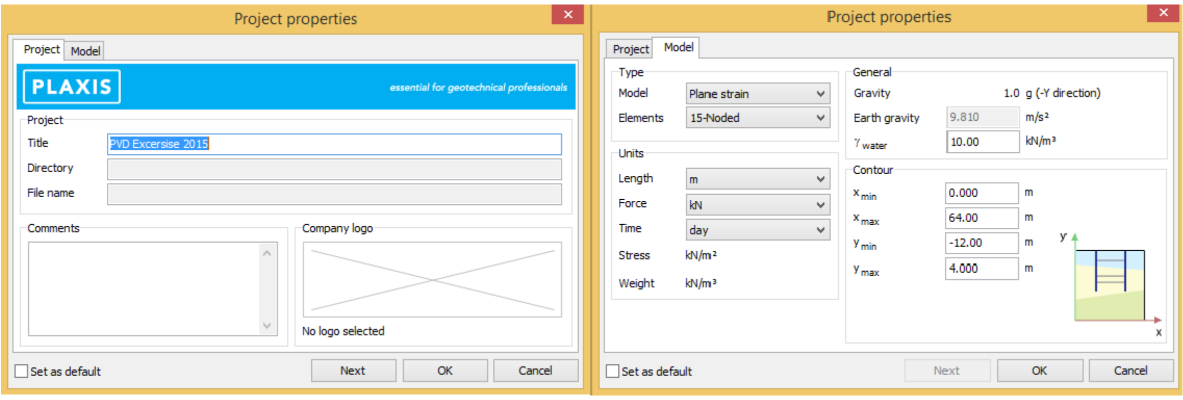


Figure 2: Project tabsheet and Model tabsheet of the Project properties window

Definition of soil stratigraphy

The sub-soil layer are defined using a **borehole**. The embankment layers are defined in **the Structures mode (soil polygon)**.

To define the soil stratigraphy:

- **Create a borehole** at **x = 0**. The Modify soil layers window pops up. 
- **Define single soil layer** as shown in Figure 3.
- **The water level is located at y = 0 m**. In the borehole column specify a value of 0 to Head.

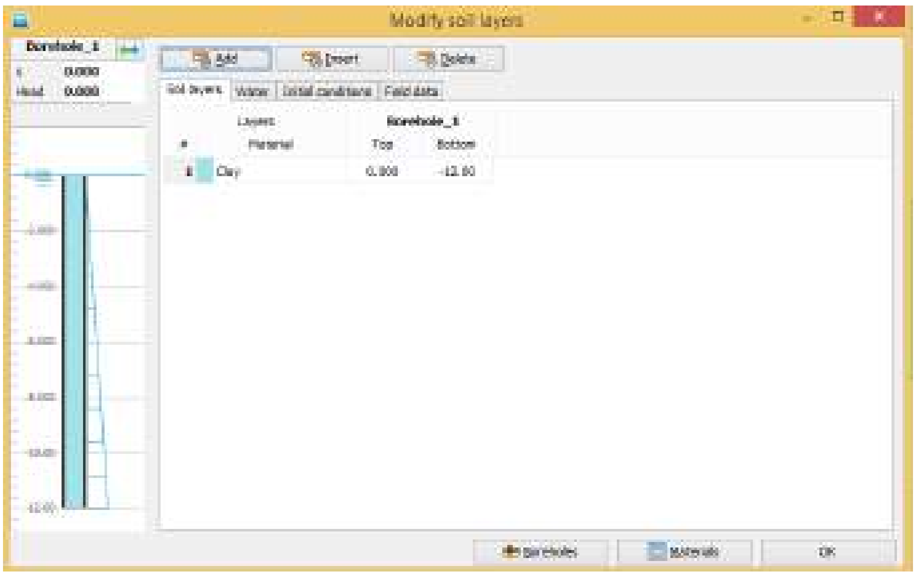


Figure 3: Modify soil layer window.

- **Open the Material sets window.** 
- **Create soil material data set** according to **Table 1** and **assign them to the corresponding layer in the borehole** (Figure 3).

- **Close the Modify soil layers window** and proceed to the **Structures mode to define the embankment and drains**.

Table 1: Material properties

Hardening Soil Model			
Parameters		**Embankment**	**Clay**
Type		Drained	UnDrained A
γ_{unsat}	kN/m^3	18	15
γ_{sat}	kN/m^3	20	16
k_x	m/day	8,64 x 10^{-1}	4.32 x 10^{-4}
k_y	m/day	8,64 x 10^{-1}	4.32 x 10^{-4}
F_{50}^{ref}	kN/m^2	20000	4300
$F_{oed}{}^{ref}$	kN/m^2	20000	1800
$E_{ur}{}^{ref}$	kN/m^2	60000	14400
c_{ref}	kN/m^2	10	15
Φ	degree	30	27
Ψ	degree	0	0
υ_{ur}	-	0.2	0.2
p_{ref}	kN/m^2	100	100
M	-	0.5	0.9
K_0^{nc}	-	0.500	0.546
R_f	-	0.9	0.9
R_{inter}	-	1.0	0.64
δ_{-inter}	-	0	0

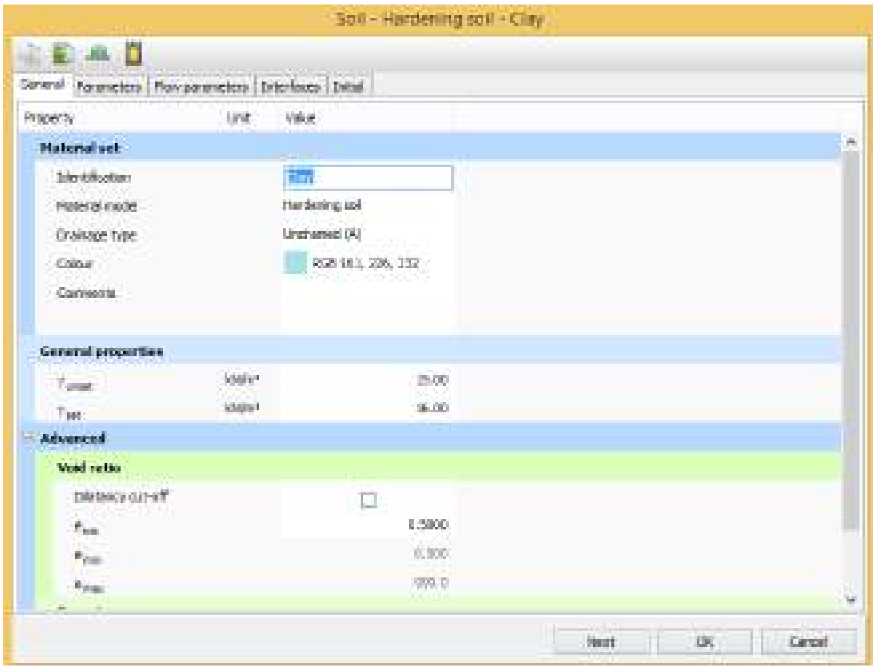


Figure 4: General tabsheet of the soil and interface data set window

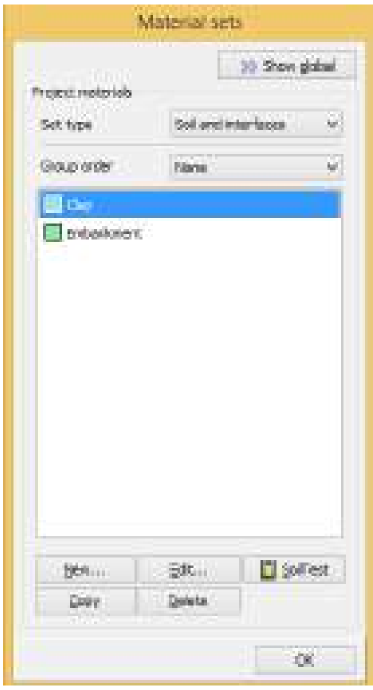


Figure 5: Material sets window.

Definition of Embankment and Drains

The embankment and **the drains** are defined in the **Structures mode**.

To define the embankment layers:

- **Click the Create soil polygon button** in the **side toolbar** and select the **Create soil**

polygon option in the appearing menu. 

- **Define the embankment** in the **draw area** by **clicking on (0.0 0.0), (0.0 4.0), (10.0 4.0) and (16.0 0.0)**.
- **Right-click** the created polygon and **assign the Embankment data set to the soil polygon** (Figure 6).
- **To define** the **embankment construction level** click the **Cut polygon** in the **side toolbar** and **define a cutting line** by **clicking on (0.0 2.0) and (13.0 2.0)**. The **embankment cluster is split into two sub-clusters**.

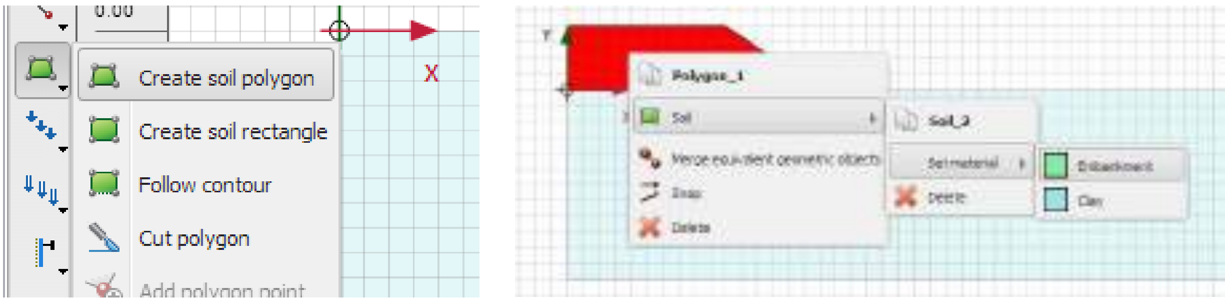


Figure 6: Assignment of a material dataset to a soil claster in the draw area.

In this project the effect of the drains on the consolidation time will be investigated by comparing the results with a case without drains.

- **Click the Create hydraulic conditions button** in the **side toolbar** and **select the Create drain option** in the appearing menu (Figure 7).

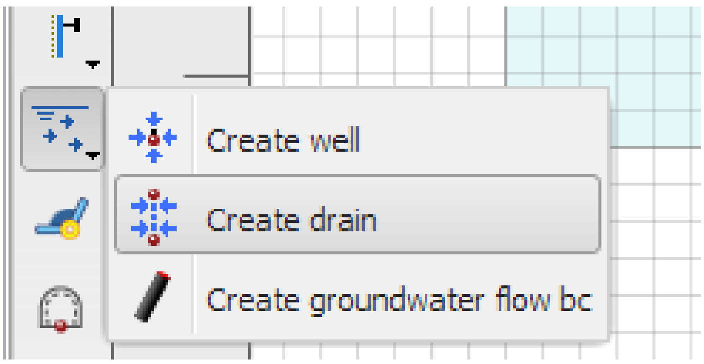


Figure 7: The Create drain option in the Create hydraulic conditions menu

- **Drains are defined in the soft layer (y = 0.0 to y = -12.0)**. The **distance between two** consecutive **drains** is **2 m**. **Considering the symmetry**, the **first drain** is **located at 1 m distance from the model boundary**.
- **8 drains** will be created in total (Figure 8) 

Figure 8: Final geometry of the model

MESH GENERATION

- Proceed to the **Mesh mode**
- Generate the mesh. **Use the default option** for the **Element distribution parameter (Medium)**. (Figure 9).
- **View the generated mesh**. The resulting mesh is shown in Figure 10.
- **Click on the Close tab** to **close the Output program**.

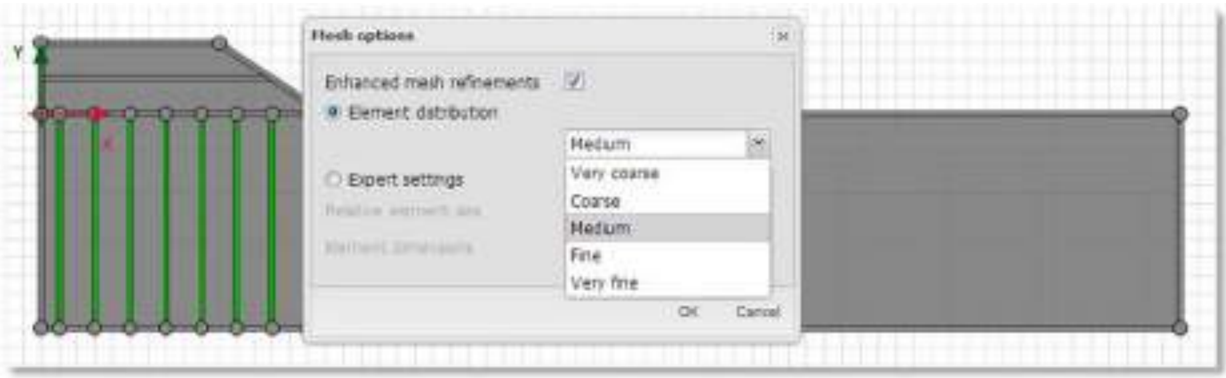


Figure 9: Mesh options window.

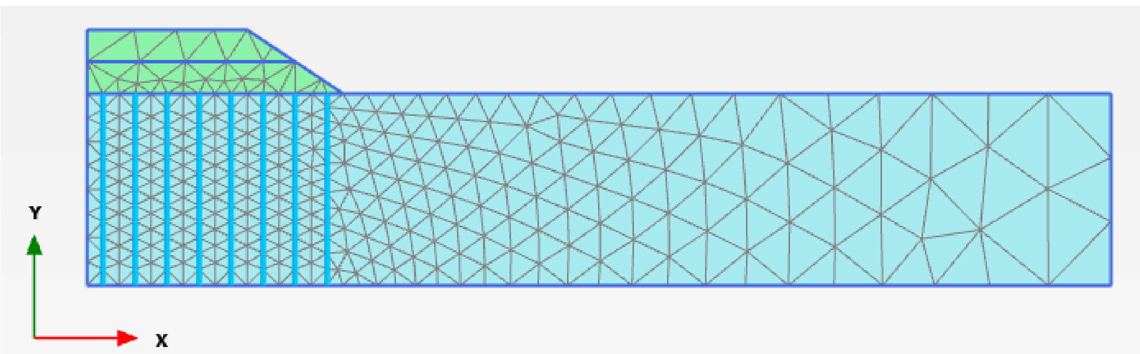


Figure 10: The generated mesh in the Output window.

CALCULATIONS

Once the mesh has been generated, the finite element model is complete. **The calculation phases** to simulate the stages of construction are shown in **Table 2**.

Table 2. Calculation phases

No	Staged Construction	Days
1	*Initial condition*	-
2	*Install PVD*	2
3	*2.0 Embankment (+2.0m)*	2
4	*Consolidation*	1
5	*2.0 Embankment (+4.0m)*	2
6	*Consolidation*	10
7	*Final Consolidation*	-

Initial Phase: Initial Conditions

In the **initial situation** the **embankment is not present**. In order to generate the initial stresses therefore, **the embankment must be deactivated first in the Staged constructio n mode**.

In the **Staged construction mode deactivate the two clusters that represent the embankment**, just like in a staged construction calculation. When the embankment has been deactivated (the corresponding clusters should have the background colour), the remaining **active geometry is horizontal with horizontal layers**, so the **K0 procedure can be used to calculate the initial stresses** (Figure 11).

- In the **Model explorer** expand the **Soils subtree**.
- Expand the **Soils subtree** and **deactived** The Embankment.

The **initial water pressures are fully hydrostatic and based on a general phreatic level located at y = 0**. Note that a **phreatic level is automatically created at y = 0, according to the value specified for Head in the borehole**.

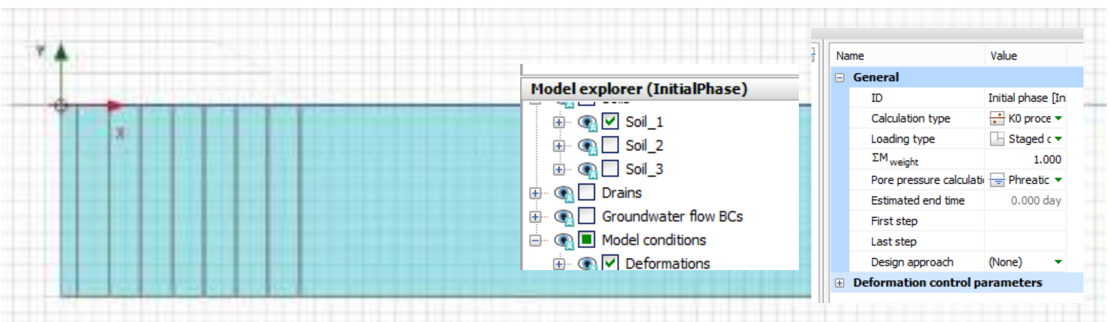


Figure 11: Configuration of the initial phase.

In addition to the phreatic level, attention
must be paid to the **boundary conditions for the consolidation analysis** that will be performed during the calculation process. **Without giving any additional input, all boundaries except for the bottom boundary are draining so that water can freely flow out**
of these boundaries and excess pore pressures can dissipate. In the **current situation**, however, **the left vertical boundary** must be **closed because** this is a **line of symmetry**, so **horizontal flow should not occur**. **The bottom horizontal boundary** also **closed because the excess pore pressures cannot be dissipated through Impermeable Soil Layer**. The remaining boundaries are open because the excess
pore pressures can be dissipated through these boundaries. In order to define the appropriate consolidation boundary conditions, follow these steps:

- In the **Model explorer** expand the **Model conditions subtree**.
- Expand the **GroundwaterFlow subtree** and set Boundary **XMin and YMin to Closed** (Figure 12).

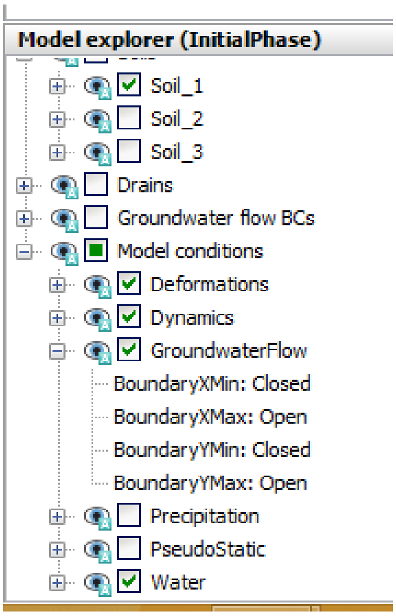


Figure 12: The boundary conditions of the problem.

Consolidation Analysis

A consolidation analysis introduces the dimension of time in the calculations. In order to correctly perform a consolidation analysis a proper time step must be selected. The use of time steps that are smaller than a critical minimum value can result in stress oscillations.

The consolidation option in PLAXIS allows for a fully automatic time stepping procedure that takes this critical time step into account. Within this procedure there are three main possibilities:

- Consolidate for a predefined period, including the effects of changes to the active geometry **(Staged construction)**.
- Consolidate until all excess pore pressures in the geometry have reduced to a

predefined minimum value **(Minimum excess pore pressure)**.

- Consolidate until a specified degree of saturation is reached **(Degree of consolidation)**.

The **first two possibilities will be used in this exercise**. To define the calculation phases, follow these steps:

Phase 1: Install PVD

The first calculation stage is a **Consolidation analysis**, **Staged construction**.

- **Add a new phase.**
- In the Phases window select the **Consolidation option** from the **Calculation type** drop-down menu in the General subtree.
- Make sure that the **Staged construction** option is selected for the **Loading type.**
- Enter a **Time interval** of **2 days**. The default values of the remaining parameters will be used.
- In the **Model explorer** expand the **Drains subtree** and **actived The PVD** (figure 13).
- **Click the Material sets windows button in the side toolbar**. **Copy clay material (clay ps)** and **select the flow parameters to change horizontal soil permeability (kx)** with **equivalent horizontal permeability for plane strain condition (khp)** (table 4). Change (**clay layer**) with (**clay ps**).

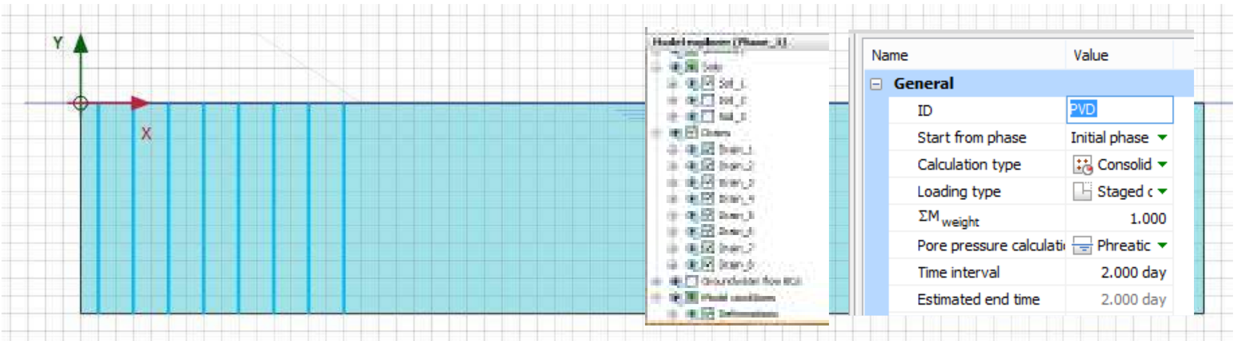


Figure 13: Phase 1.

Table 3: PVD parameters

No	PVD parameters	Formula	Units
1	*Space of triangular grid pattern(s)*		1.9 m
2	*Installation depth (l)*		-12m
3	*Mandrel diameter (dm)*		88 mm
4	*Efektif space(de)*	1.05 x s	2 m
5	*Equivalence PVD diameter (dw)*	$\frac{2(a+b)}{n}$	52mm
6	*Smear are diameter (ds)*	$\frac{5}{2}dn$	220mm
7	*The permeability in the smeared zone(Ks)*	Kh/ks = 2	2.15×10^{-4} m/day

Table 4: Equivalent Horizontal Permeability

Equivalent Horizontal Permeability (Hird et al. 1992)			
Horizontal Permeability	**Formula**	**Units**	**Description**
Kh (kx)		4.32 x 10^{-4} m/day	Horizontal permeability undisturb soil
k_{hp}	$\frac{k_{hp}}{k_h} = \frac{0{,}67}{[\ln(n) - 0{,}75]}$	**9.98 x 10^{-5} m/day**	equivalent horizontal permeability without smear zone
$k_{hp'}$	$\frac{k_{hp}'}{k_h} = \frac{2}{3\left[\ln\left(\frac{R}{r_c}\right) + \left(\frac{k_h}{k_c}\right)\ln(s) - \frac{3}{4}\right]}$	6.63 x 10^{-5} m/day	equivalent horizontal permeability with smear zone

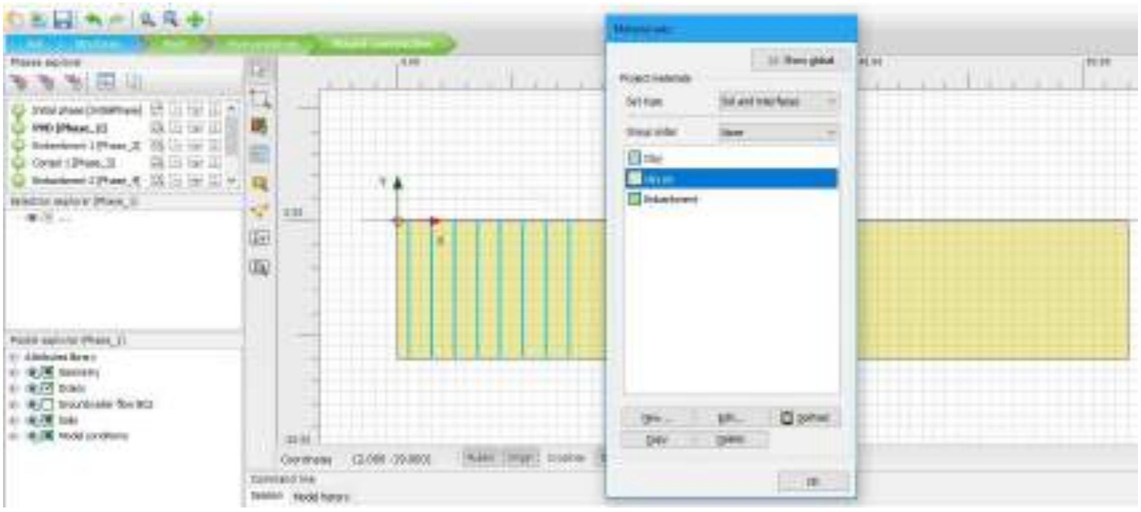


Figure 14: Phase 1.

Phase 2: Embankment 1

The second calculation stage is also a **Consolidation analysis**, **Staged construction**.

- **Add a new phase**. 
- In the Phases window select the **Consolidation option** from the **Calculation type** drop-down menu in the General subtree.
- Make sure that the **Staged construction** option is selected for the **Loading type**. 
- Enter a **Time interval** of **2 days**. The default values of the remaining parameters will be used.
- In the **Model explorer** expand the **Soils subtree** and **activate the first part of the Embankment** (figure 14).

- Change **max steps from 250 (default)** to **1000** by click Edit Phase

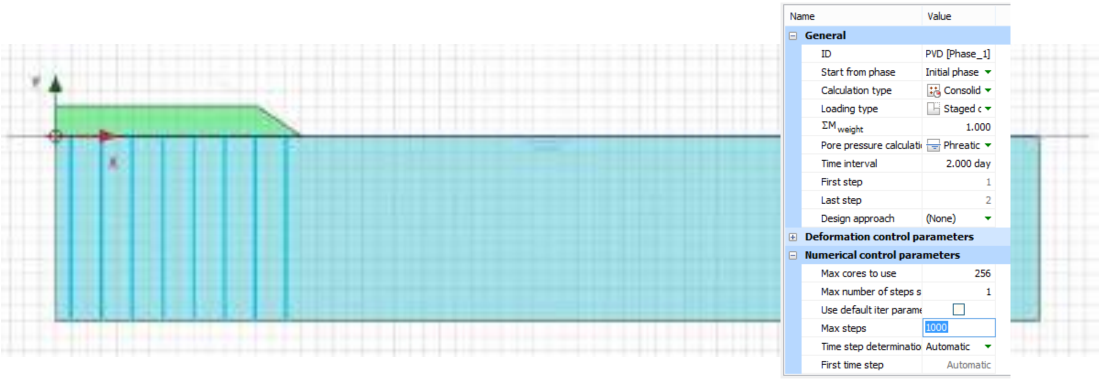


Figure 15: Phase 2.

Phase 3: Consolidation 1

The third phase is also a **Consolidation analysis**, **Staged construction**. In **this phase no changes to the geometry** are made as **only a consolidation analysis to ultimate time is required**.

- **Add a new phase**. 
- In the Phases window select the **Consolidation option** from the **Calculation type** drop-down menu in the General subtree.
- Make sure that the **Staged construction** option is selected for the **Loading type**. 
- Enter a **Time interval** of **1 day**. The default values of the remaining parameters will be used.

Phase 4: Embankment 2

The fourth calculation stage is once again a **Consolidation analysis**, **Staged construction**.

- **Add a new phase**. 
- In the Phases window select the **Consolidation option** from the **Calculation type** drop-down menu in the General subtree.
- Make sure that the **Staged construction** option is selected for the **Loading type**. 
- Enter a **Time interval** of **2 days**. The default values of the remaining parameters will be used.
- In the **Model explorer** expand the **Soils subtree** and **activate the second part of the Embankment** (figure 15).
- Change **max steps from 250 (default)** to **1000** by select Edit Phase

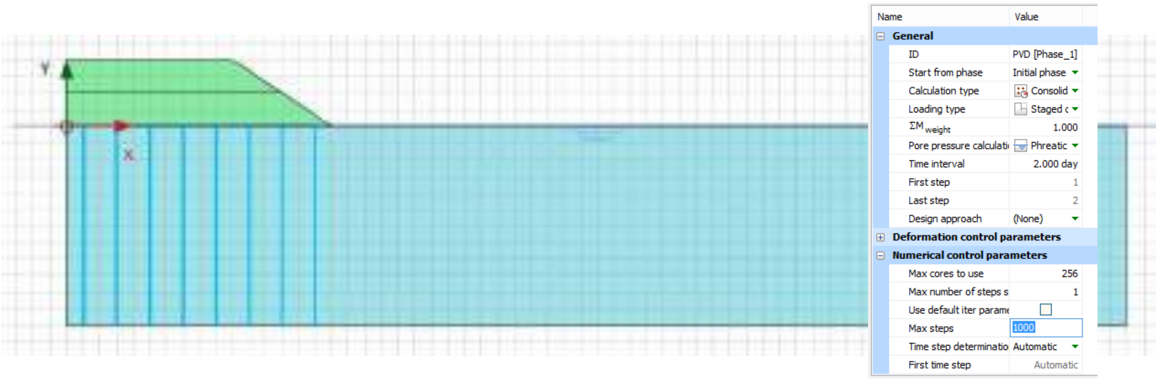


Figure 16: Phase 4.

Phase 5: Consolidation 2

The fifth phase is also a **Consolidation analysis**, **Staged construction**. In this phase **no changes to the geometry are made as only a consolidation analysis to ultimate time is required**.

- **Add a new phase.** 
- In the Phases window select the **Consolidation option** from the **Calculation type** drop-down menu in the General subtree.
- Make sure that the **Staged construction** option is selected for the **Loading type.** 
- Enter a Time interval of **10 day.** The default values of the remaining parameters will be used.

Phase 6: Final Consolidation

The sixth phase is a **Consolidation analysis to a minimum excess pore pressure**.

- **Add a new phase.** 
- In the Phases window select the **Consolidation option** from the **Calculation type** drop-down menu in the General subtree.
- Select the **Minimum excess pore pressure option** in the **Loading input** drop-down menu and accept the default value of 1 kN/m2 for the minimum pressure. The default values of the remaining parameters will be used.

Execution of calculation:

Before starting the calculation, click the Select points for curves button and select the following points (figure 16): 

Point A (0 0) will be used to plot the development of vertical displacement.
Point B (0 -6) will be used to plot the development (and decay) of excess pore pressures.

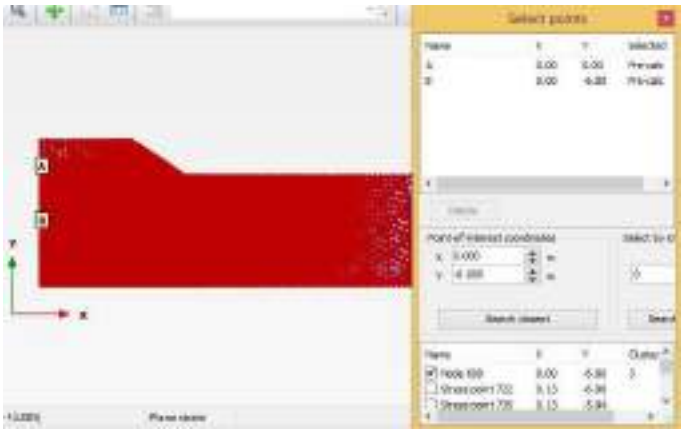


Figure 17: Point A and B.

After selecting these points, start the calculat 

During a consolidation analysis the development of time can be viewed in the upper part of the calculation info window.

In addition to the multipliers, a parameter Pexcess,max occurs, which indicates the current maximum excess pore pressure. This parameter is of interest in the case of a Minimum excess pore pressure consolidation analysis, where all pore pressures are specified to reduce below a predefined value.

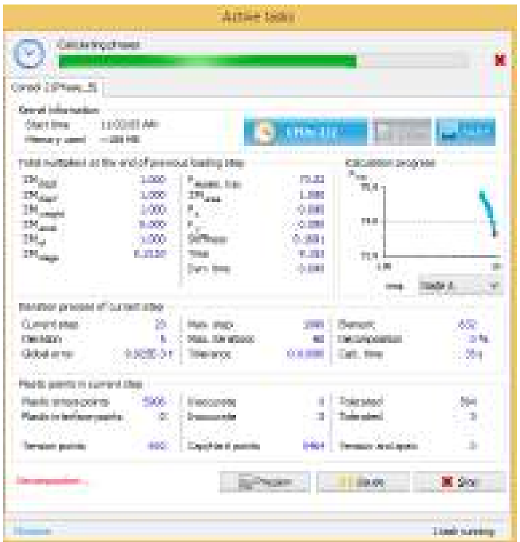


Figure 18: Calculation progress displayed I the Active task window.

RESULTS

After the calculation has finished, **select** the ***'View calculation'* button** that will start the output program. 

Check the various types of output, such as the **deformed mesh**, **displacement contours**, **excess pore pressure (Pexcess)**, **etc**. These can be found from the ***'Deformations'* and *'Stresses'* menus**.

Figure 19: Vertical Displacement (uy)

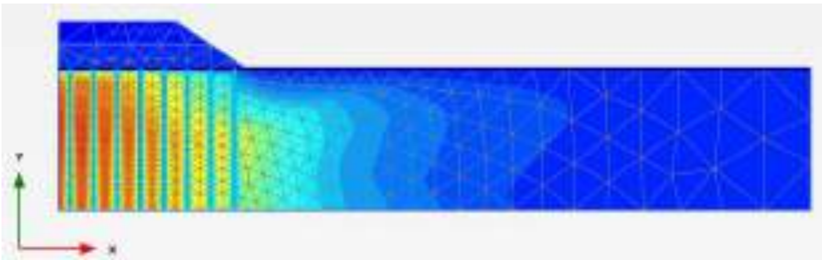


Figure 20: excess pore pressure (P_{excess}) with PVD

The Curves manager can be used to view the development, with time, of the excess p ore pressure under the embankment (Point B) (figure 20).

In order to create such a curve, follow these steps: 

- **For the x-axis**, select the Project option from the drop-down menu and **select Time in the tree**.
- **For the y-axis select the point** in the middle of the soft soil layers **(Point B)** from the drop-down menu. In the tree select **Stresses → Pore pressure → pexcess**.

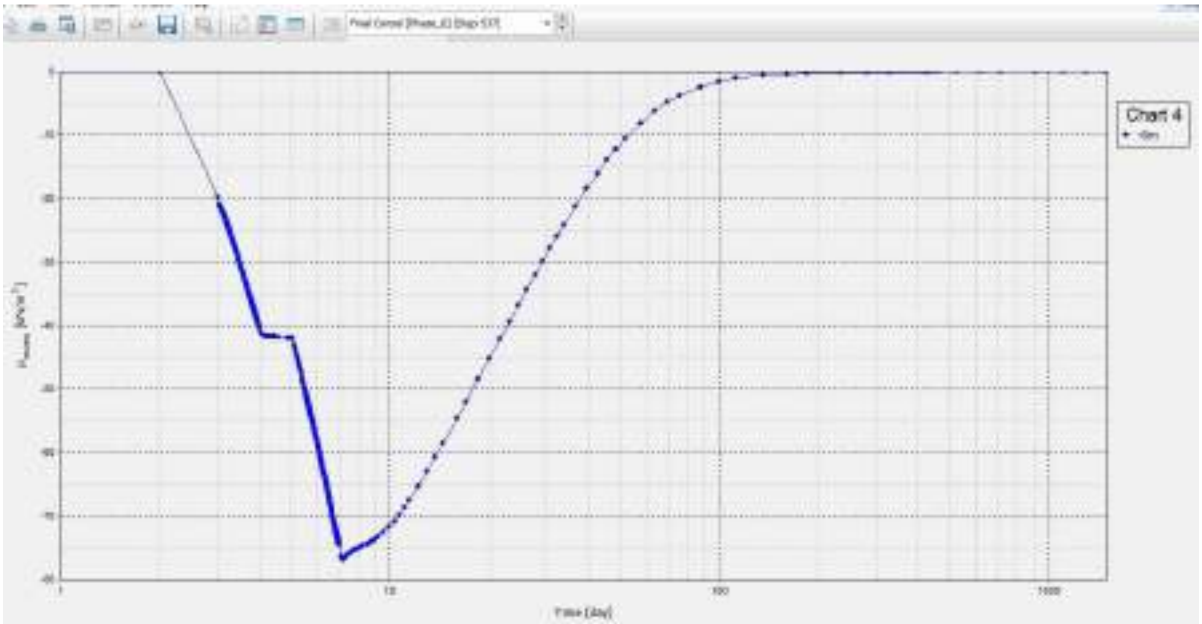


Figure 21: Development of excess pore pressure under the embankment (Point B)

Also
the Curves manager can be used to view the development, with time, of the vertical displacement (Uy) of the embankment (Point A and B).

In order to create such a curve, follow these steps: 

- **For the x-axis**, select the Project option from the drop-down menu and select **Time** in the tree.
- **For the y-axis select the point** in the top of the soft soil layers (Point A) and also can be select the point in the middle of the soft soil layers (Point B) from the drop-down menu. In the tree select **Deformation → Total Displacements → uy**.

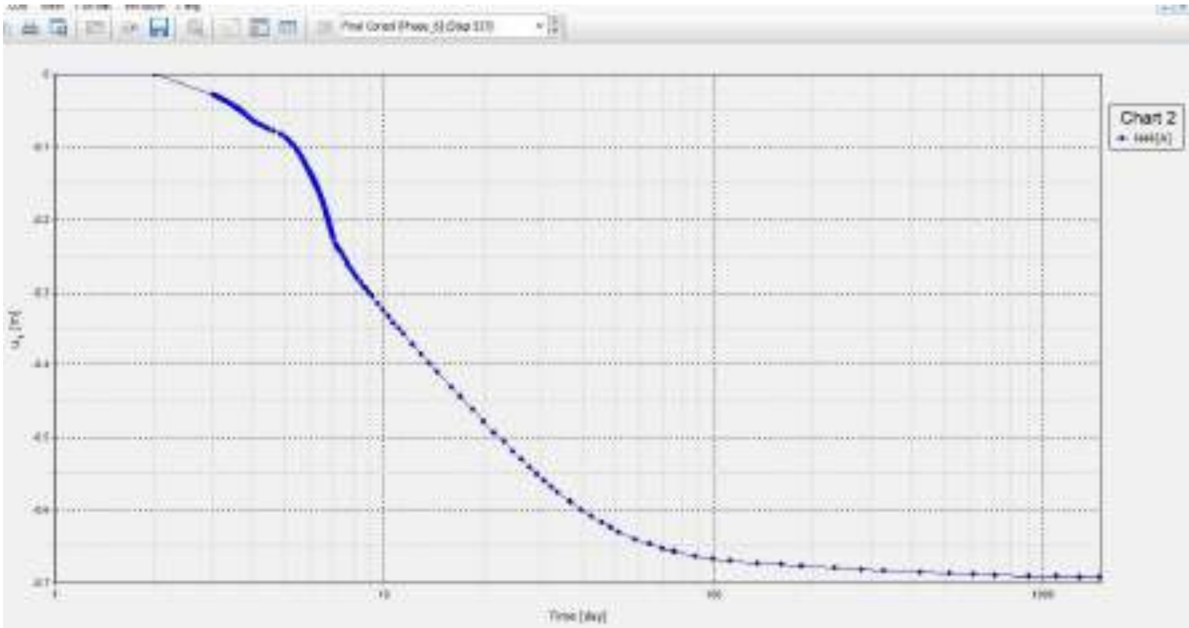


Figure 22: Displacement versus time for the point A

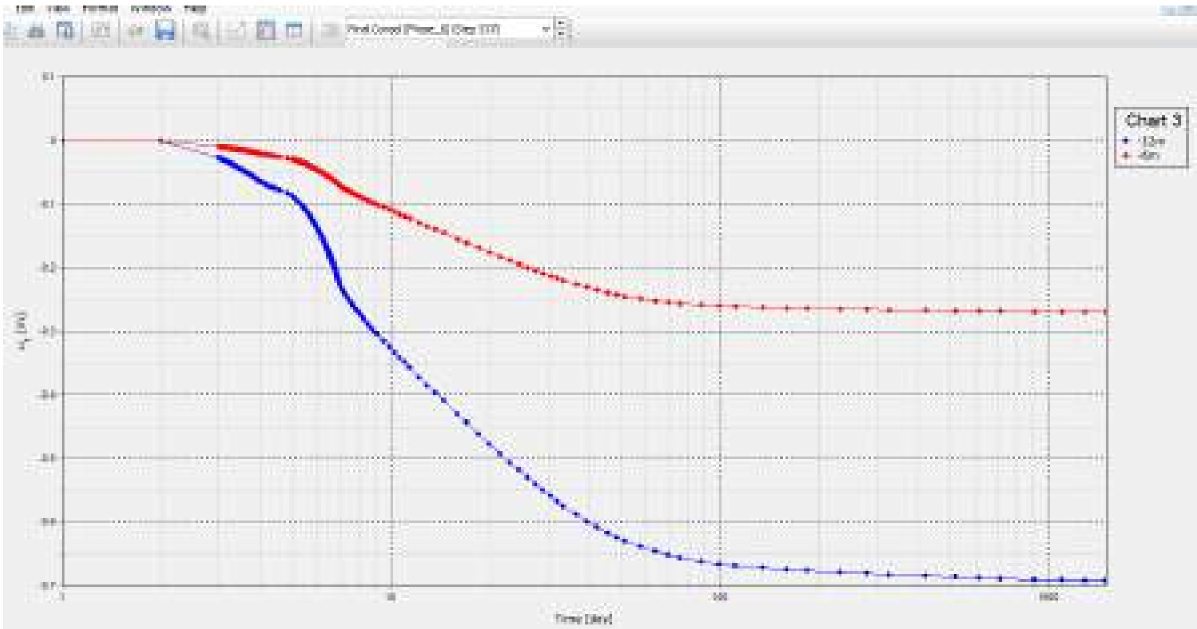


Figure 23: Displacement versus time for the point A and B

WITHOUT PVD VS ONLY PVD VS PVD WITH SMEAR

Consolidation analysis PVD with Smear zone

- **Save as** this work.
- Click the **Material** sets windows button in the side toolbar and **select "clay ps" material to change the flow parameters** from equivalent horizontal permeability for plane strain condition (without smear) (**khp**) with equivalent horizontal permeability for plane strain condition (with smear) **(khp')** (table 4) 

Consolidation analysis without PVD

- **Save as** this work.
- In the **Model explorer** expand the Model conditions subtree, expand the **Drains subtree** and **deactived The PVD**.
- Click the **Material** sets windows button in the side toolbar and **select "clay ps" material to change the flow parameters** from equivalent horizontal permeability for plane strain condition (without smear) (**khp**) with original horizontal soil permeability **(kx)**.

Figure 24: excess pore pressure (P_{excess}) Without PVD

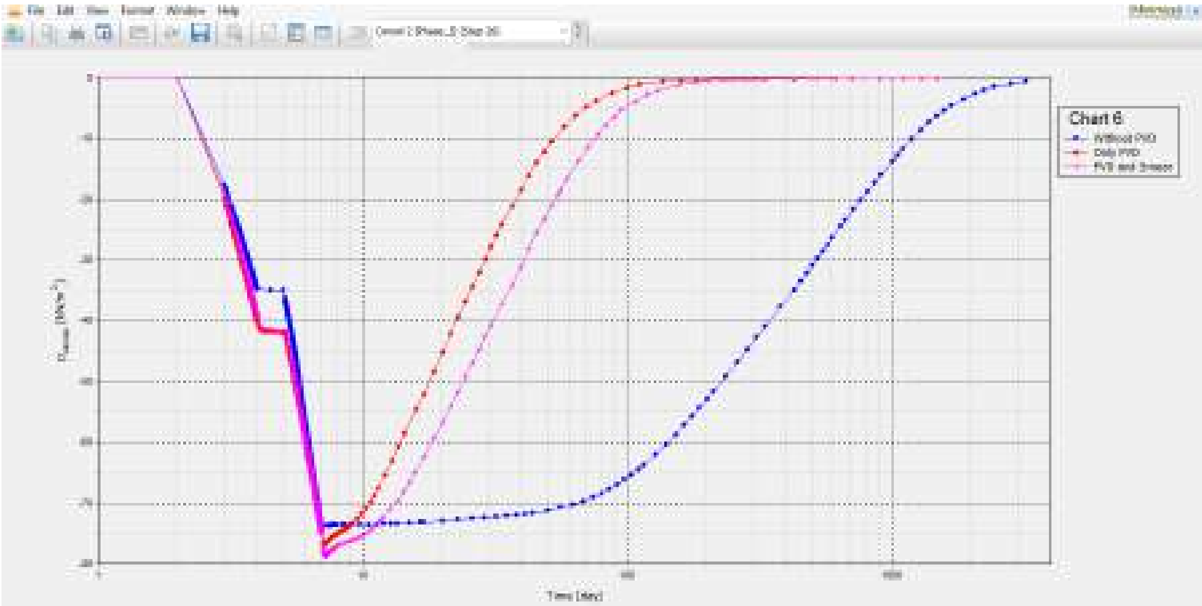


Figure 25: Excess pore pressure versus time (Point B)

Comparison curve using the PVD only, PVD + smear effect, and without PVD

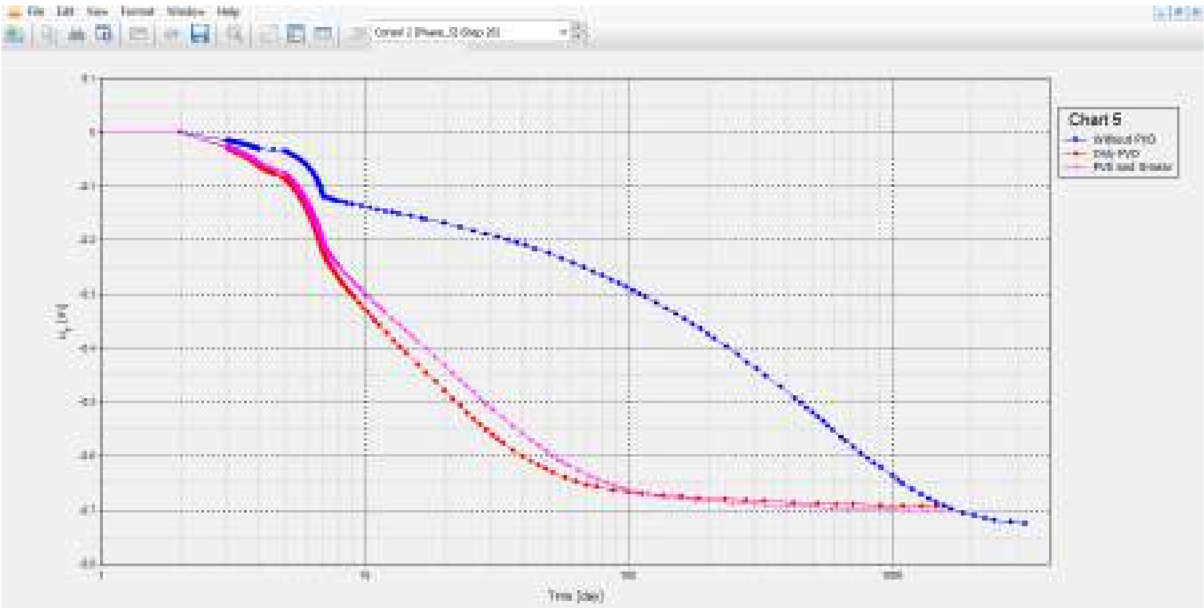
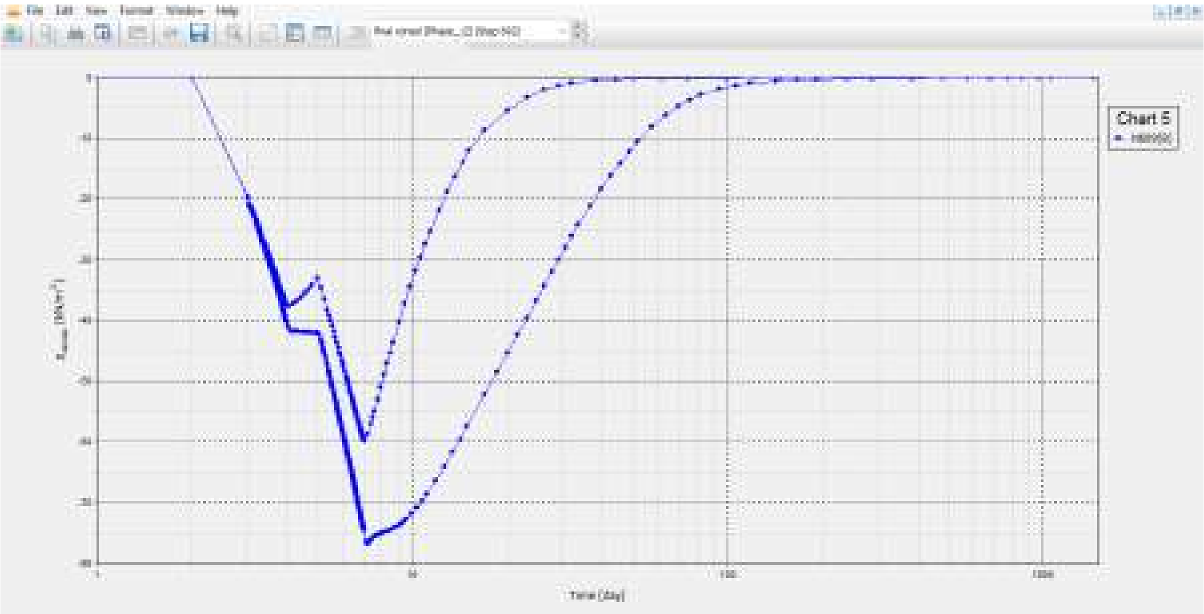
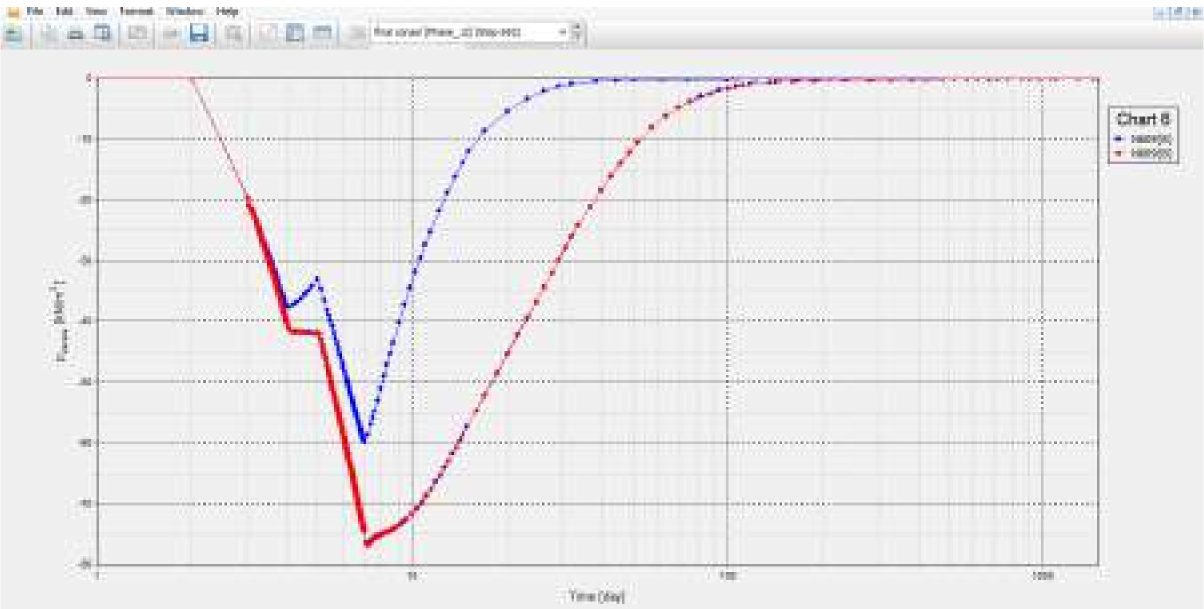


Figure 26: Displacement versus time (Point A)

Comparison curve using the PVD only, PVD + smear effect, and without PVD


File Edit View Format Window Help
Chart 5
Time [day]


File Edit View Format Window Help
Chart 6
Time [day]

Modelling Vacuum Consolidation

There are various methods of vacuum consolidation in the real world, but they are all modelled in similar way in Plaxis. Most methods in reality are using vertical drains, which are somehow connected at the top to an air pump that reduces the air pressure in the drains until near-vacuum exist. In Practice, a complete vacuum (100 kN/m^2 pressure) is not achievable, but an effective under-pressure of 60-90 kN/m^2.

Since Plaxis does not take air pressure into account (atmospheric pressure is assume to be the zero reference pressure level), a reduction of the groundwater head is used instead to simulate vacuum consolidation). It is modelled leads to negative pore stresses (suction), which are not there in reality.

Vacuum Consolidation analysis

Open File Analysis Consolidation with PVD (without smear).

- **Save as** this work.
- Click the **Material** sets windows button in the side toolbar and select the material data set. Arrange the parameters in material data set, so the saturated condition apply to this volume. 
    - o Change unsaturated unit weight (γ_{unsat}) in clay soil parameter (General tabsheet of the material data set), must be set equal to the saturated unit weight, γ_{sat}

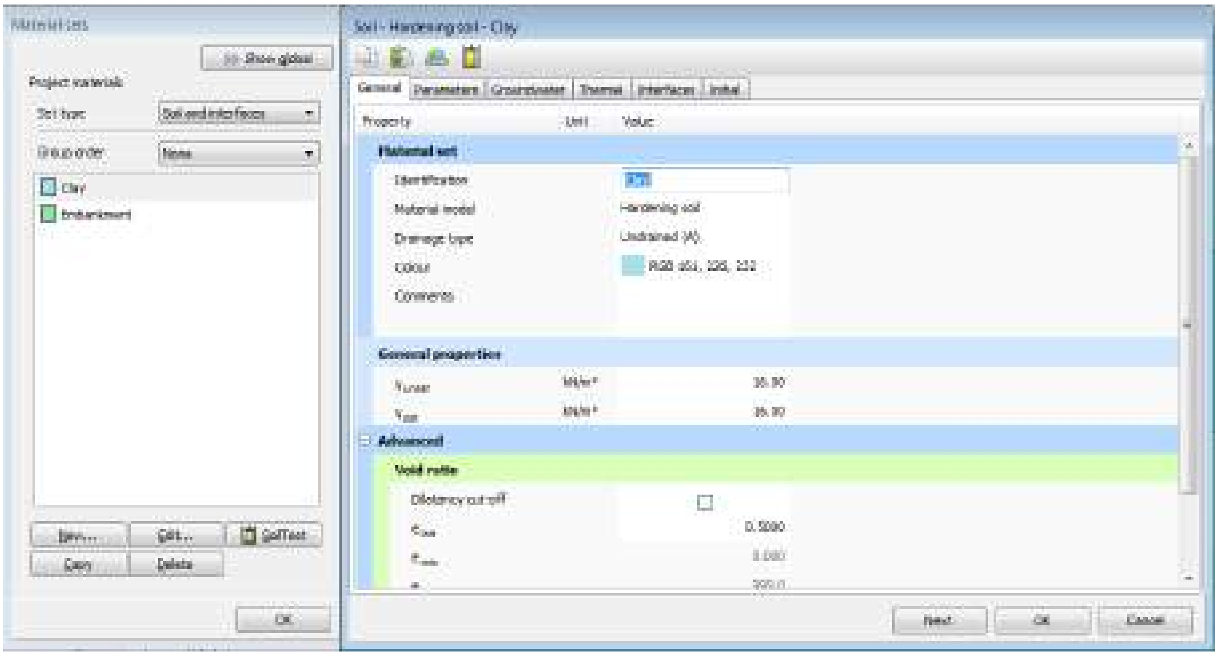
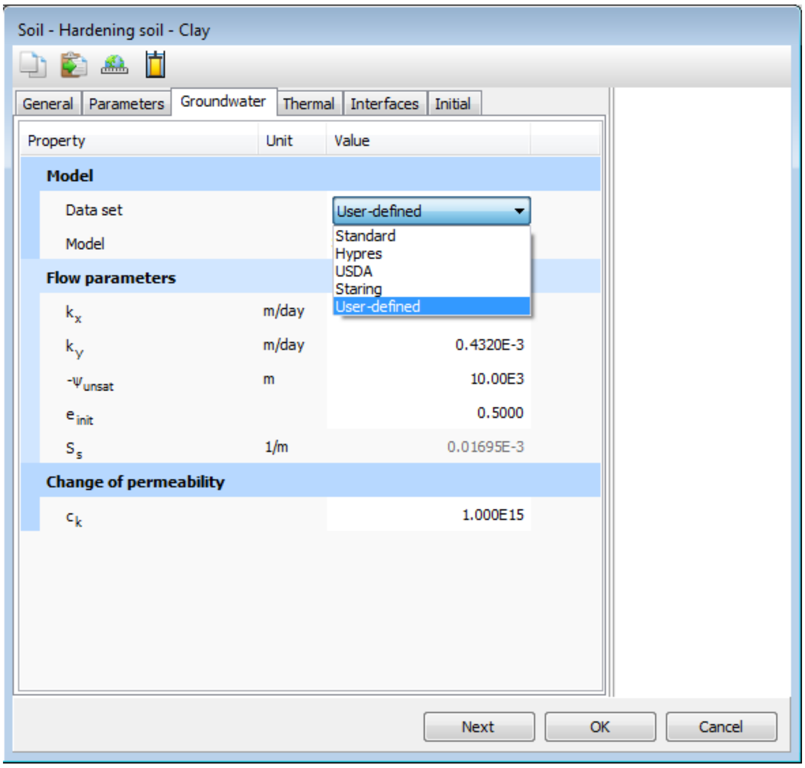
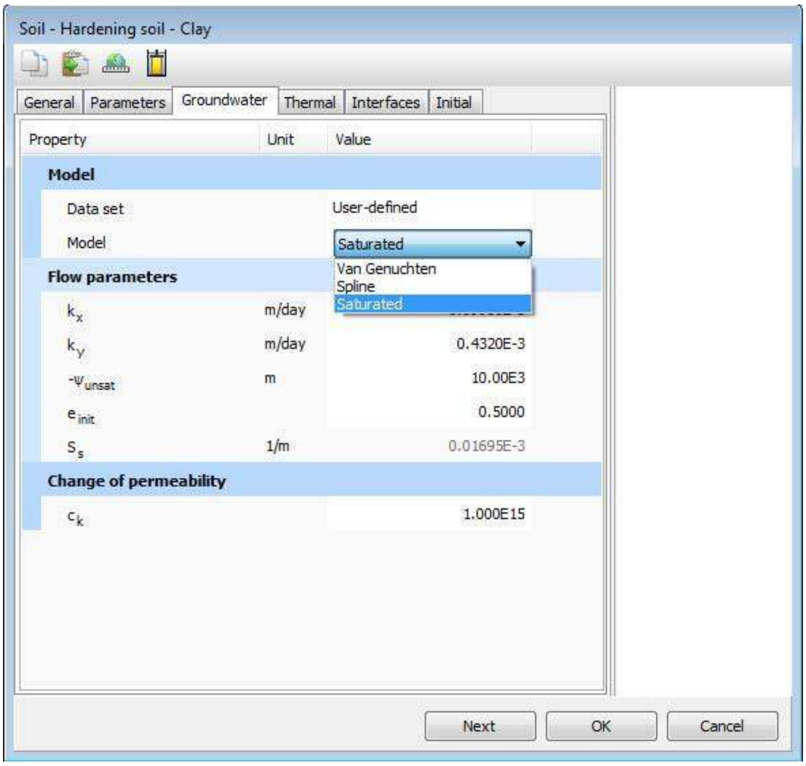


Figure 273: Material set for clay parameter $\gamma_{unsat} = \gamma_{sat}$

- o In Groundwater tabsheet for clay parameter, hydraulic model must be set to *saturated* after selecting Data Set : *User-defined*



(a)



(b)

Figure 284: Groundwater model for Vacuum Consolidation :

(a) Data Set : User Defined, (b) Model : Saturated

- In ***Phase Explorer*** for stages construction,
  - Click 'initial phase' and de-select the ***ignore suction*** option in Deformation control parameters sub tree. Do exactly same for all phases.

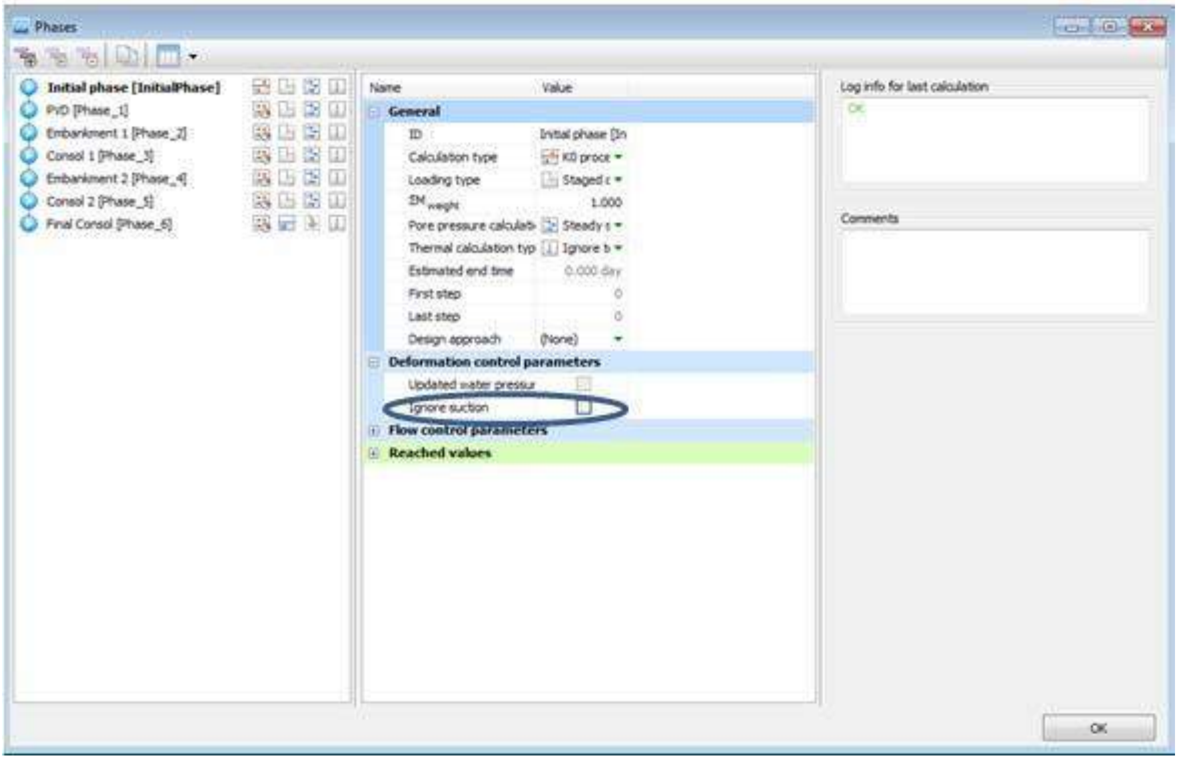


Figure 295: De-select Ignore suction

- from Initial Phase to ‘Consol 2 (Phase 5)’. Change Phreatic to ***Steady state groundwater flow*** In pore pressure calculation (general tab)

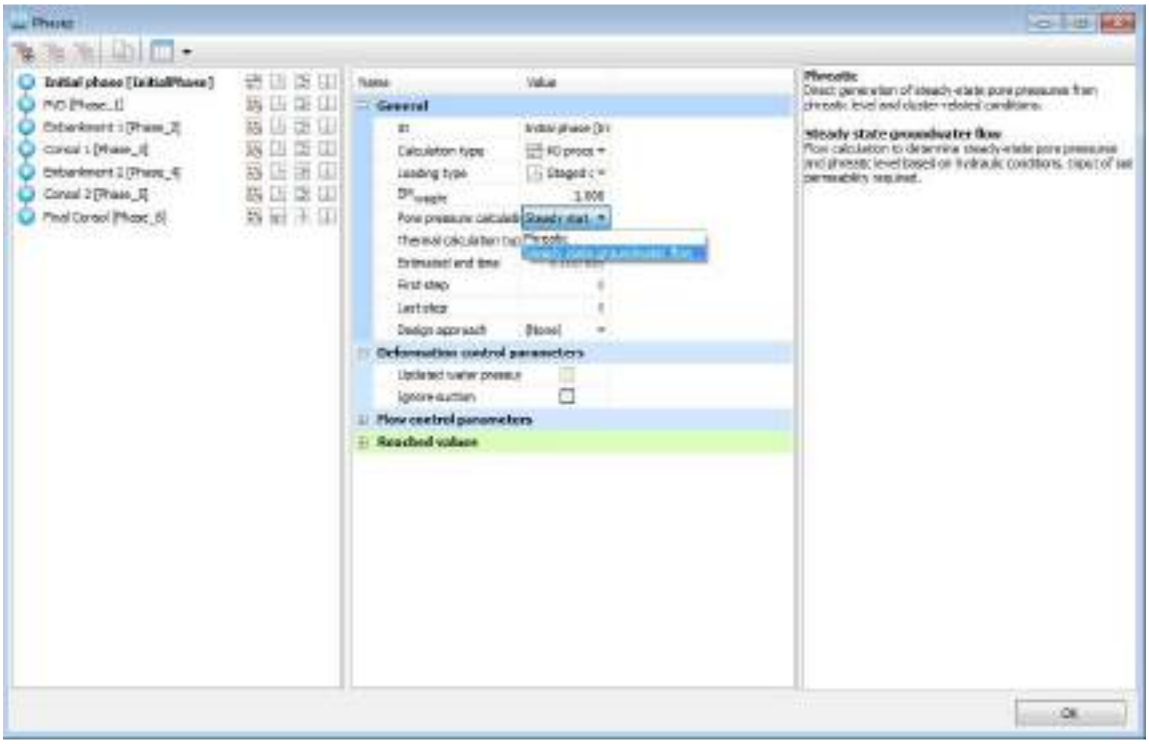


Figure 306: Steady state groundwater flow for pore pressure calculation

- In ‘Embankment 1 (Phase 2)’, **Select all drains**. Expand *selection explorer* and *sub tree* of drain. Then **change behaviour of drain from normal to vacuum**. **Input head (h)** must be underpressure **(-8 m)**. Do exactly same

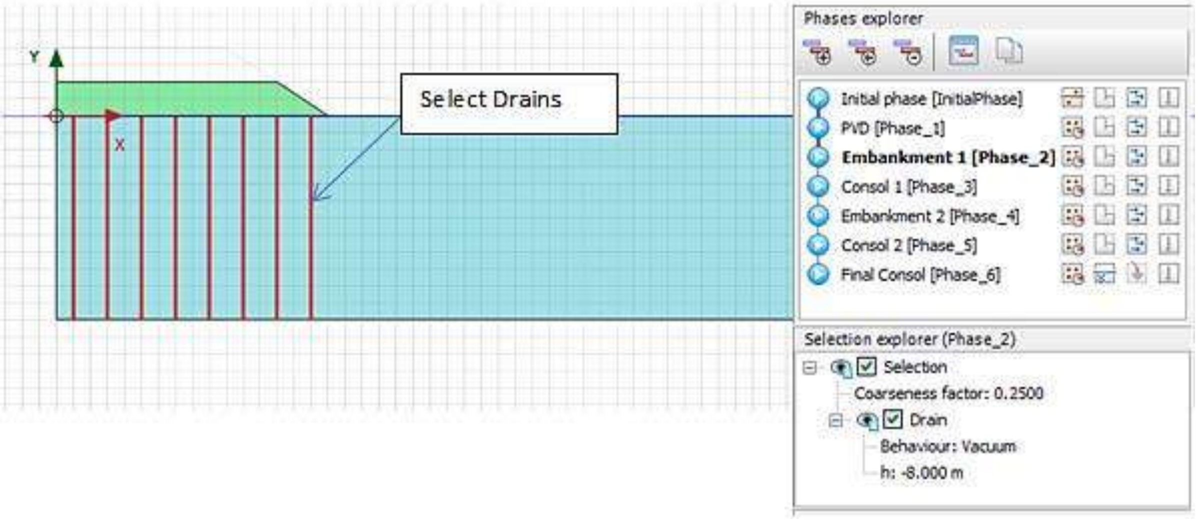


Figure 317: Modelling vacuum behavior

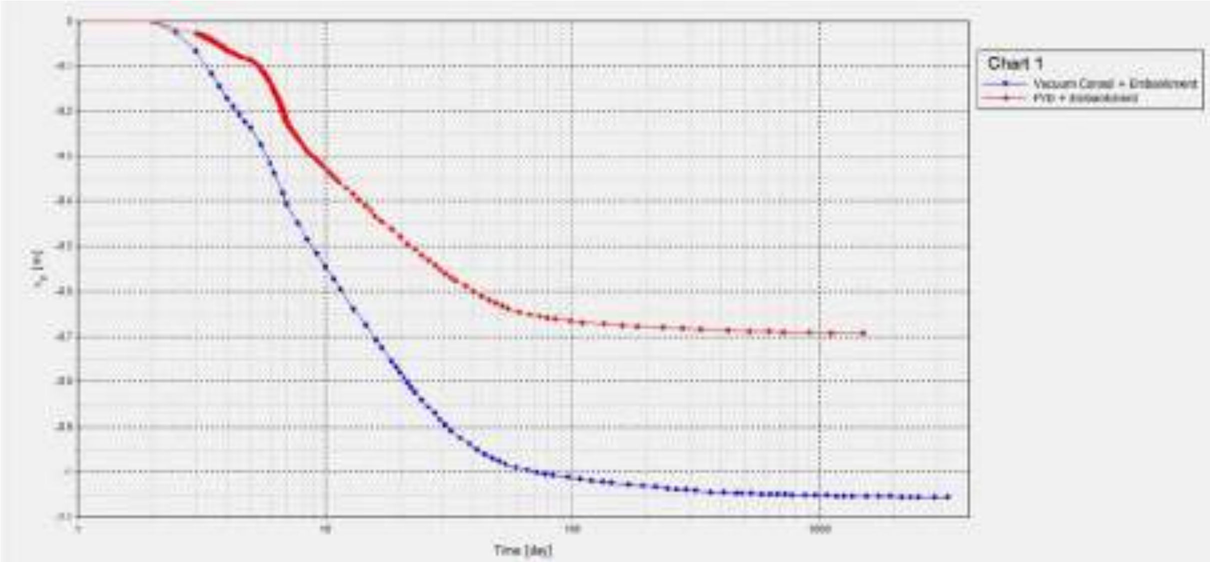


Figure 28: Displacement versus time (Point A)

Comparison curve using the Vacuum Consolidation and without Vacuum Consolidation

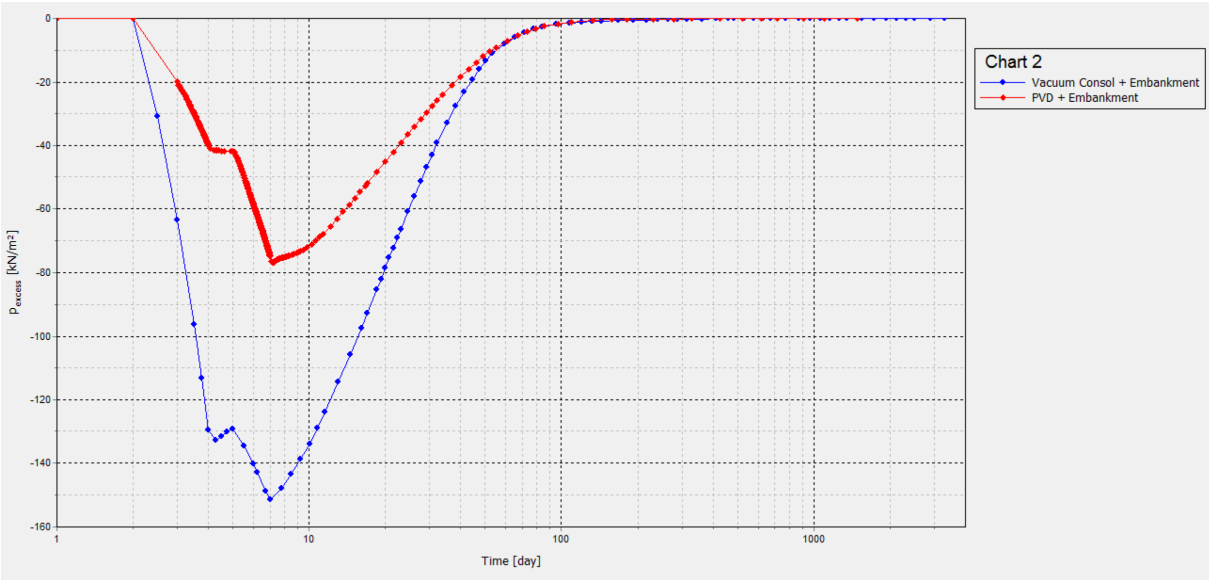


Figure 29: Excess pore pressure versus time (Point B)

Comparison curve using the Vacuum Consolidation, and without Vacuum Consolidation

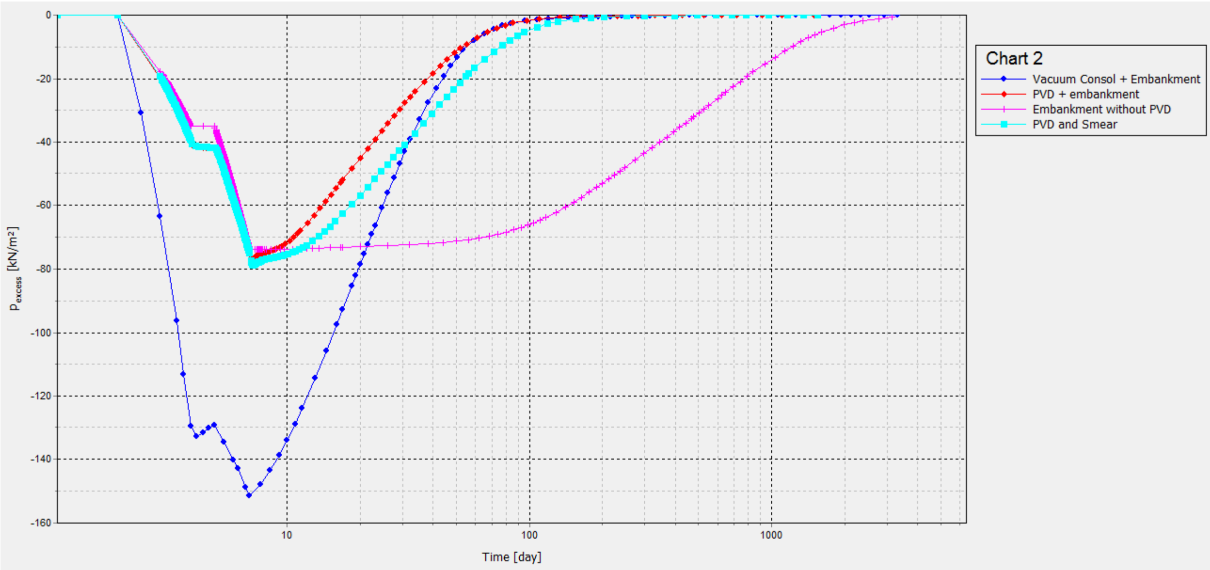


Figure 29: Excess pore pressure versus time (Point B)

Comparison curve for all cases

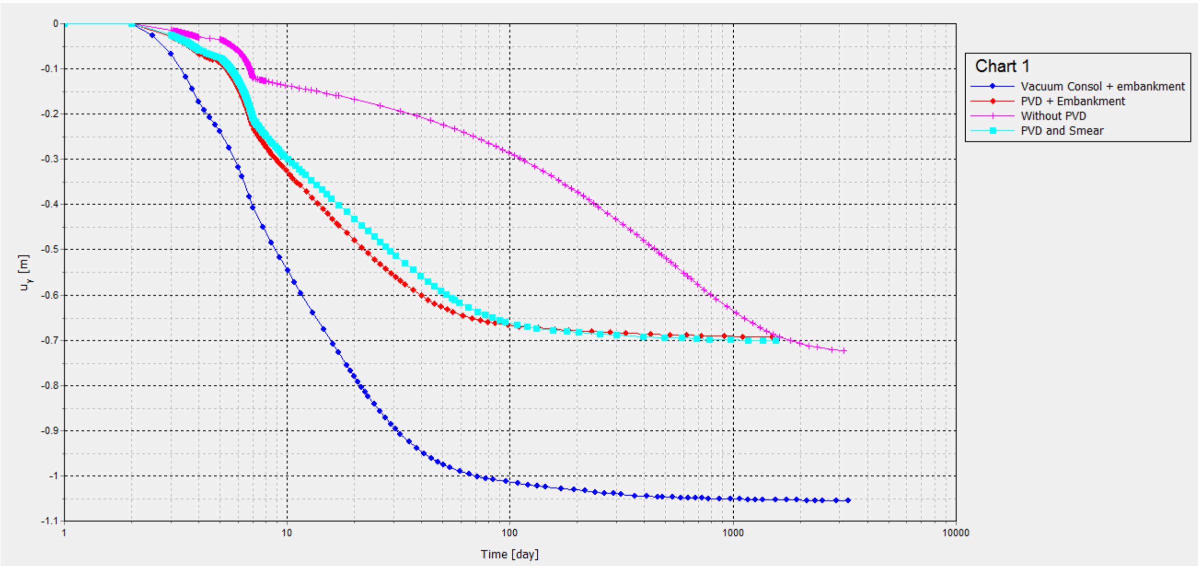


Figure 29: Displacement versus time (Point A)

Comparison curve for all cases

SCIENCE ▪ PASSION ▪ TECHNOLOGY

CG15
INITIAL STRESSES
SAFETY ANALYSIS

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Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology
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National Institute of Technology (Itenas) Bandung

CONTENT

- **Initial stresses**
  - general
  - K0-procedure
  - gravity loading
  - special cases
- **Phi-c-reduction**
  - safety factor
  - safety factor analysis
  - examples
  - final advice
- **Comparison Phi-c-reduction – Limit equilibrium analysis**
- **Comparison Phi-c-reduction – Limit analysis**
- **Non-associated plasticity**

- **Initial stresses represent the equilibrium state of stress of the undisturbed soil and consist of:**
  - Soil weight
  - Loading history

- **In Plaxis two possibilities exist:**
  - K0-procedure
  - Gravity loading

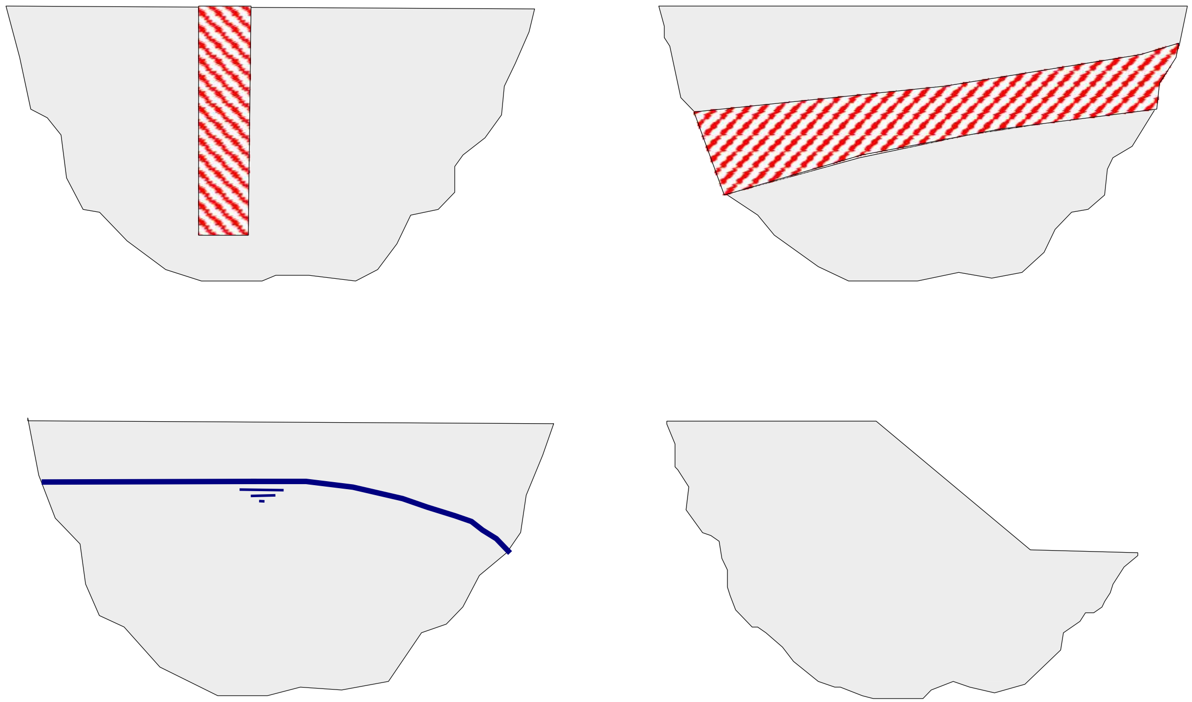
GENERATION OF INITIAL STRESSES AS FIRST STEP IN CALCULATION K_0-PROCEDURE

- **Needed:**
  coefficient for lateral earth pressure K_0

- **Disadvantage:**
  No equilibrium for inclined surface (a nil-step can be used)

- **Advantage:**
  No displacements are generated, only stresses, can take into account initial OCR or POP

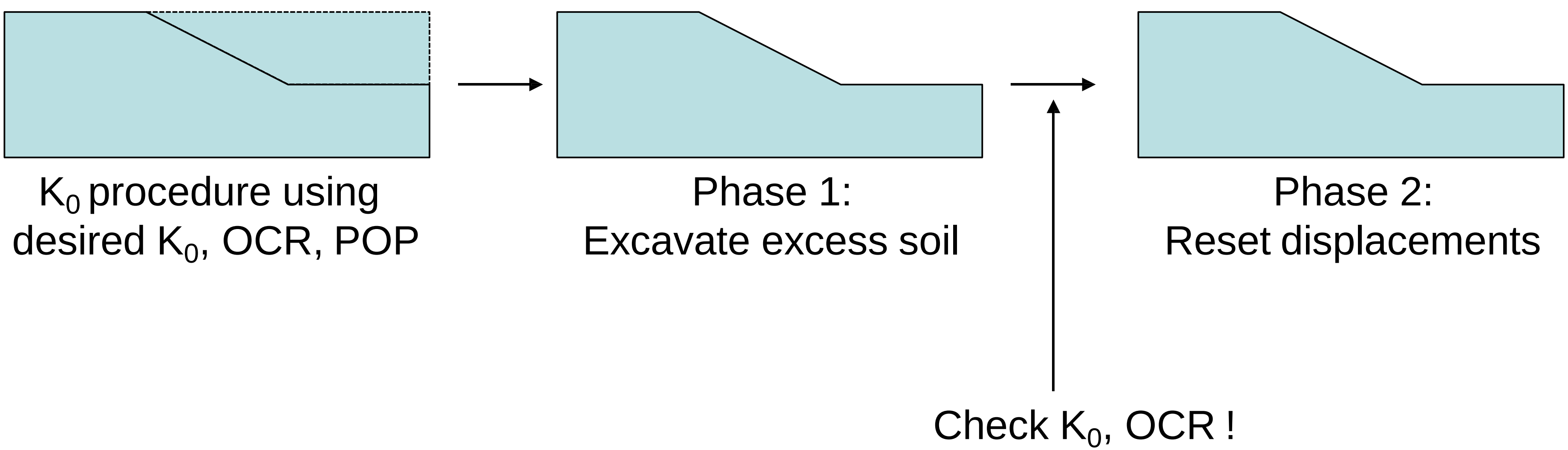
CASES WHERE GRAVITY LOADING SHOULD BE USED INSTEAD OF K_0-PROCEDURE



SPECIAL CASES

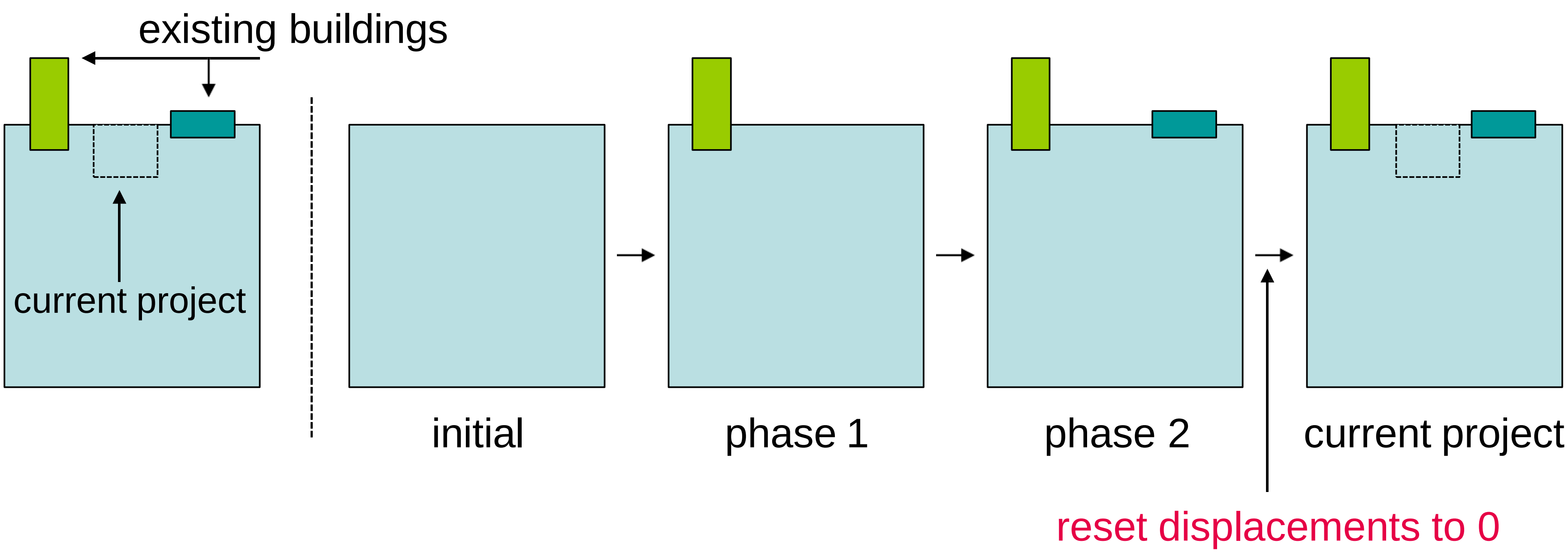
For example a slope in overconsolidated soil or if (approximate) modelling of geological history is required

Gravity loading needed due to geometry,
but initial OCR or POP required



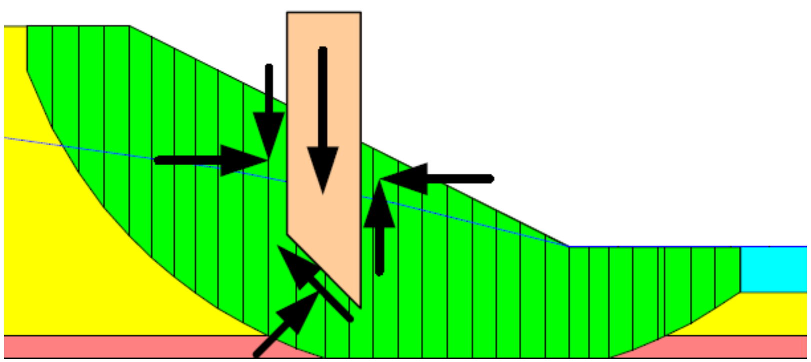
SPECIAL CASES

For complex initial situations like inner city building projects it may be needed to use several calculation phases to model the current situation before starting the calculation for the actual project.



Methods of assessing the factor of stability problems:

- Limit equilibrium method > generally used in practical engineering
- Displacement finite element method > phi-c-reduction (strength reduction method)
- Finite element limit analysis

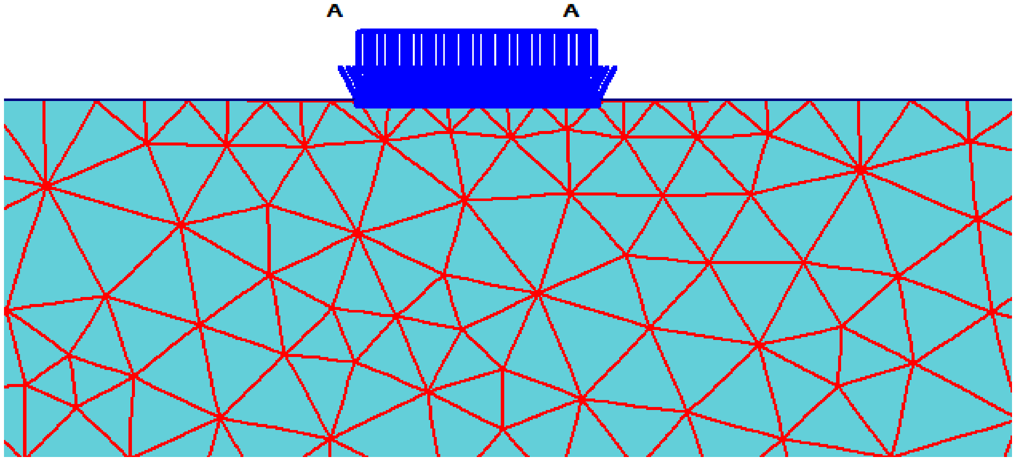


Limit equilibrium analysis

- Overall form of the failure surface needs to be determined in advance
- Distribution of the interslice forces is assumed differently in various methods
- Kinematic admissibility is not ensured
- Need to perform a global search for identifying the failure mechanism with the lowest factor of safety

DISPLACEMENT FINITE ELEMENT METHOD > PHI-C-REDUCTION

Factor of safety: Many possible definitions

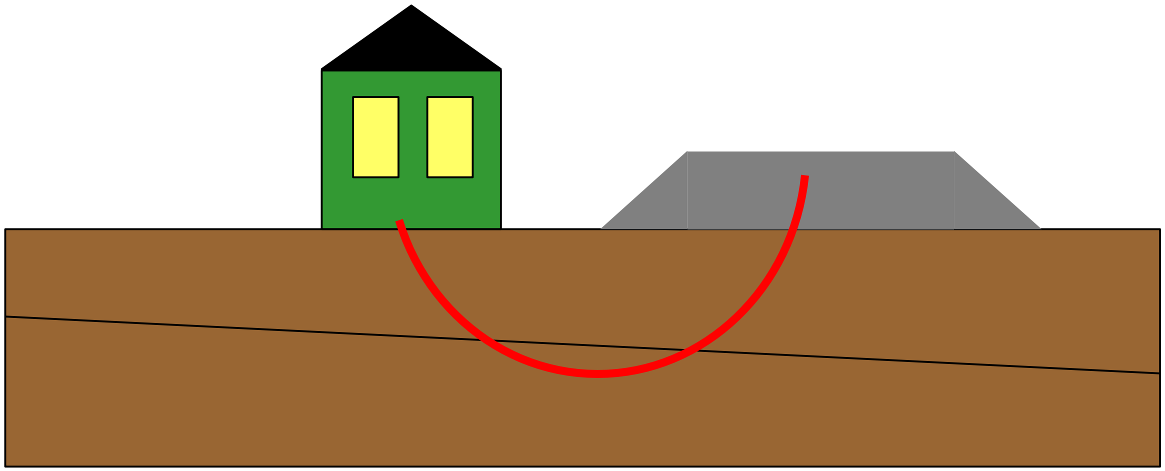


$$\frac{\textit{available soil resistance}}{\textit{mobilized soil resistance}} = 1.8$$

$$\frac{\textit{failure load}}{\textit{working load}} = 5.9$$

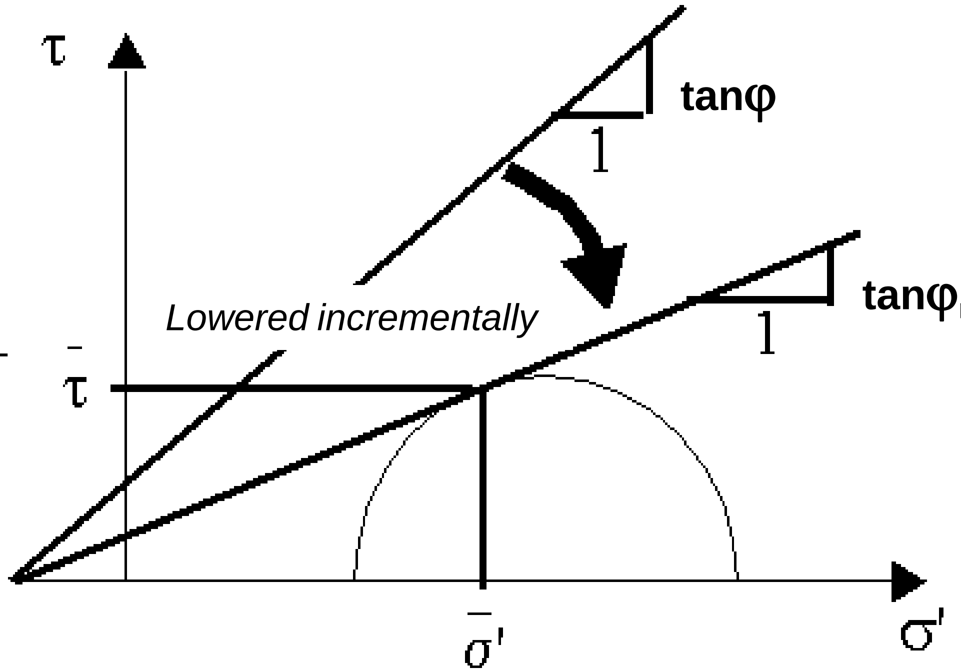
PLAXIS: Safety factor on soil resistance

- Strength reduction method: Phi-c-reduction (safety analysis)
- Same numerical tool as for serviceability design
- Automatically detects most critical failure mechanism



- Reduction of strength parameters *c* and *tan(φ)* until failure is reached.
- The factor of safety is then defined as:

$$\Sigma Msf = \frac{c}{c_{reduced}} = \frac{\tan \varphi}{\tan \varphi_{reduced}}$$



Calculation procedure:

- Create a Safety Phase
- Accept the default increment for $M_{sf} = 0.1$ from the multiplier tab-sheet
- Calculate
- Carefully examine ΣM_{sf} vs. displacement curve in Plaxis Curves to assure that failure is indeed reached
- If so, the value of ΣM_{sf} is assumed to be the factor of safety on soil resistance

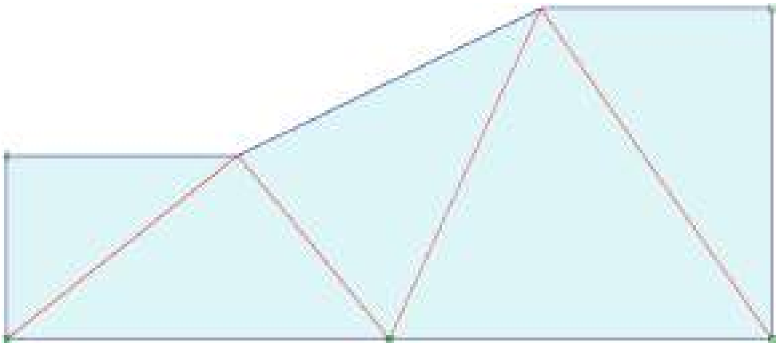
Notes:

- In order to check failure, select a control point within the (expected) failing body
- Use sufficient number of load steps
- Choose elasto-plastic behaviour for wall, anchors and geotextiles with realistic full plastic values in order to prevent excessively high structural forces
- Displacements etc. AFTER safety analysis are meaningless

MAIN ADVANTAGES OF PHI-C-REDUCTION METHOD

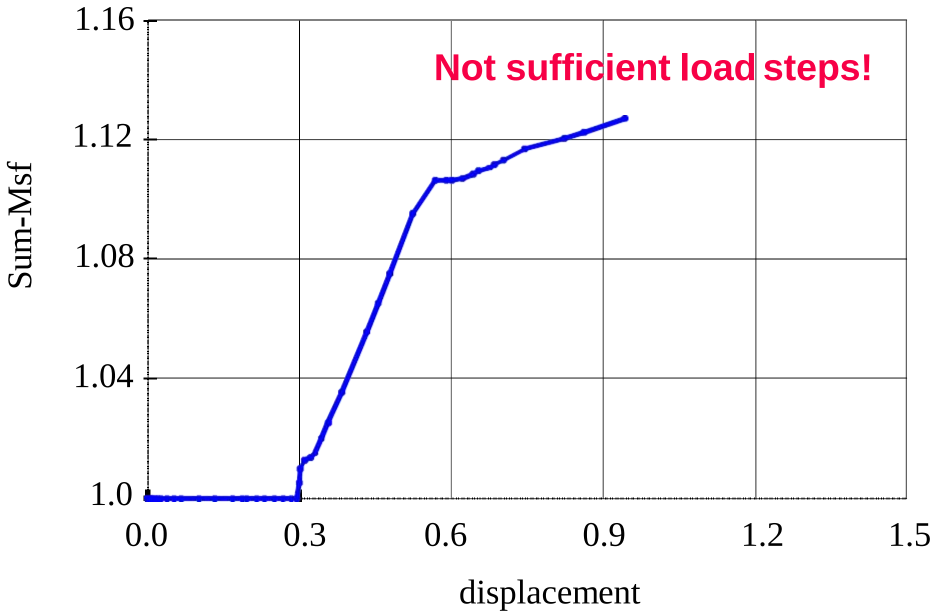
- Requires no a-priori assumptions on the failure mechanism
- Critical surface is found automatically as slope failure occurs naturally through the zones due to insufficient shear strength to resist shear stresses.
- No requirement of assumptions on e.g. inter-slice shear force distribution
- Applicable to complex conditions
- Numerical tool same as for deformation analysis
- Powerful alternative approach

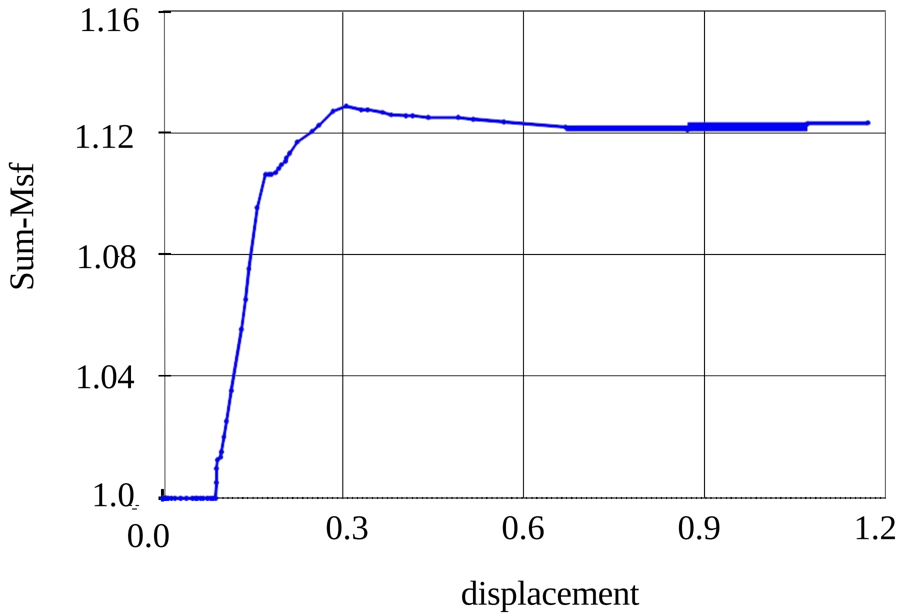
INFLUENCE OF MESH DISCRETIZATION

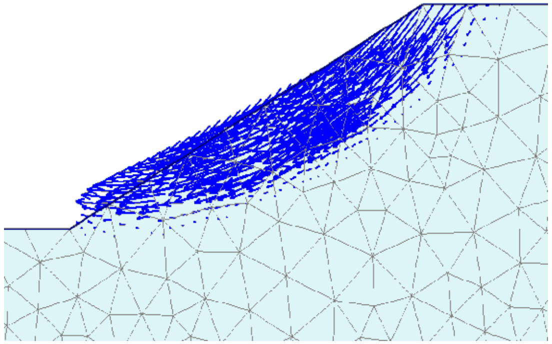


15-noded elements	Factor of Safety
5	1.90
11	1.62
(very coarse) 38	1.52
(coarse) 82	1.51
(medium) 170	1.50
(fine) 414	1.45
(very fine) 871	1.43
3733	1.43
15749	1.43

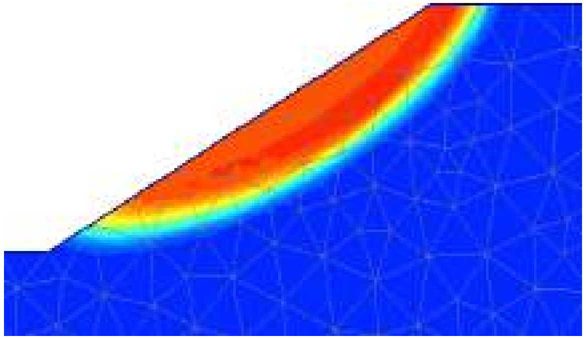
NUMBER OF LOAD STEPS



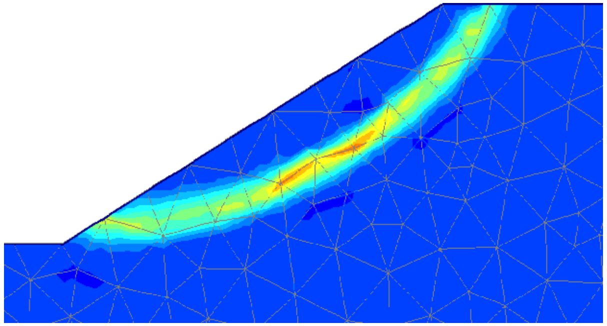




1. Arrows of incremental displacements

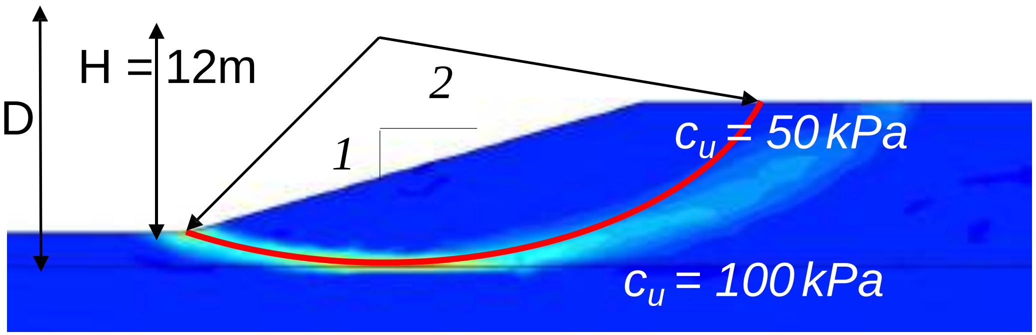


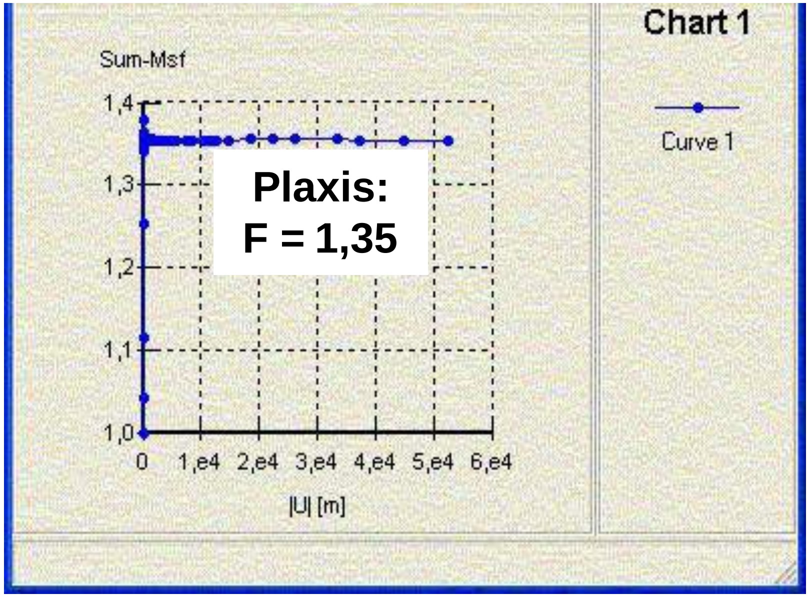
2. Shadings of incremental displacements



3. Shadings of incremental shear strains

UNDRAINED STABILITY OF A SLOPE

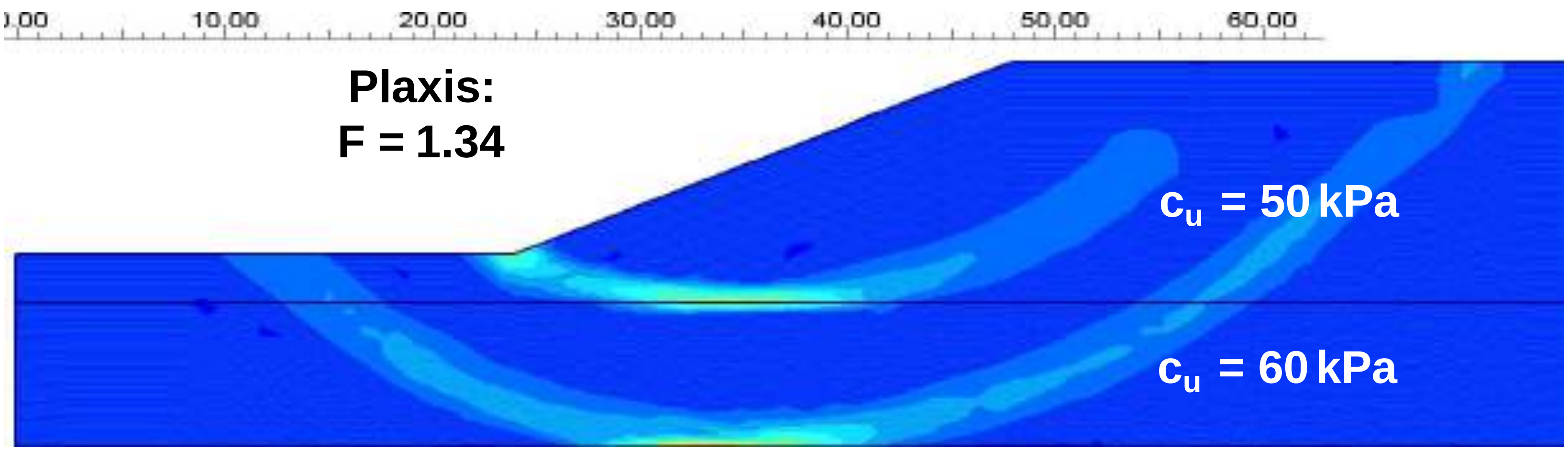




Stability charts: $F = N_0 \frac{c_u}{P_d} = 6.6 \ \frac{50}{12 \cdot 20} = 1.38$, $N_0 = f(\theta, \frac{D}{H})$ (Taylor,1948)

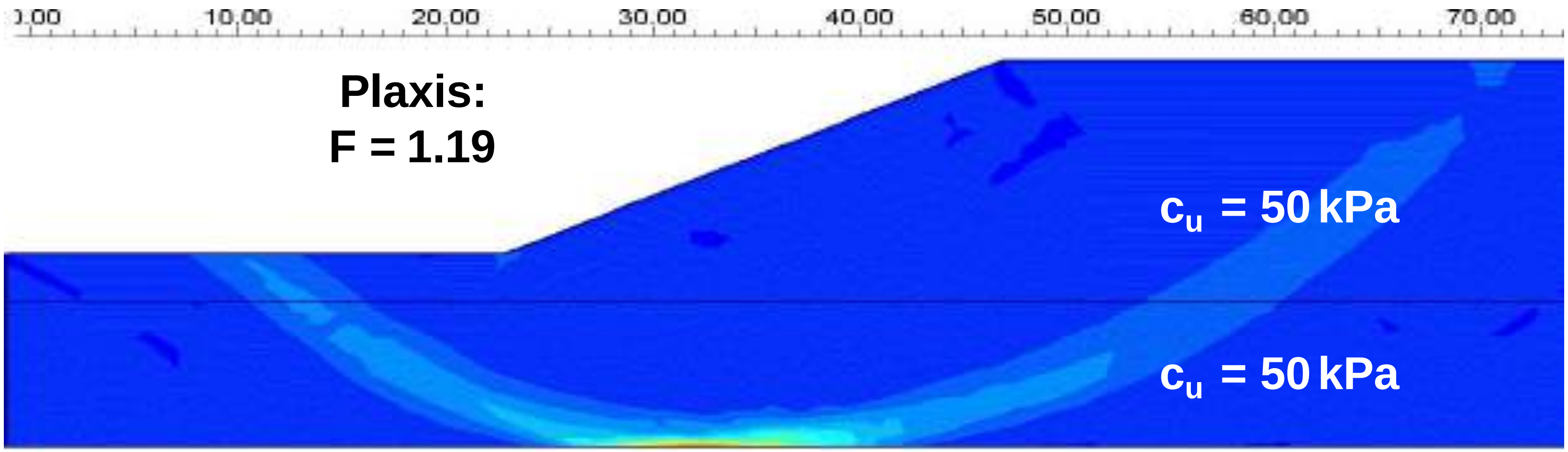
UNDRAINED STABILITY OF A SLOPE

Automatic detection of most critical shear surface

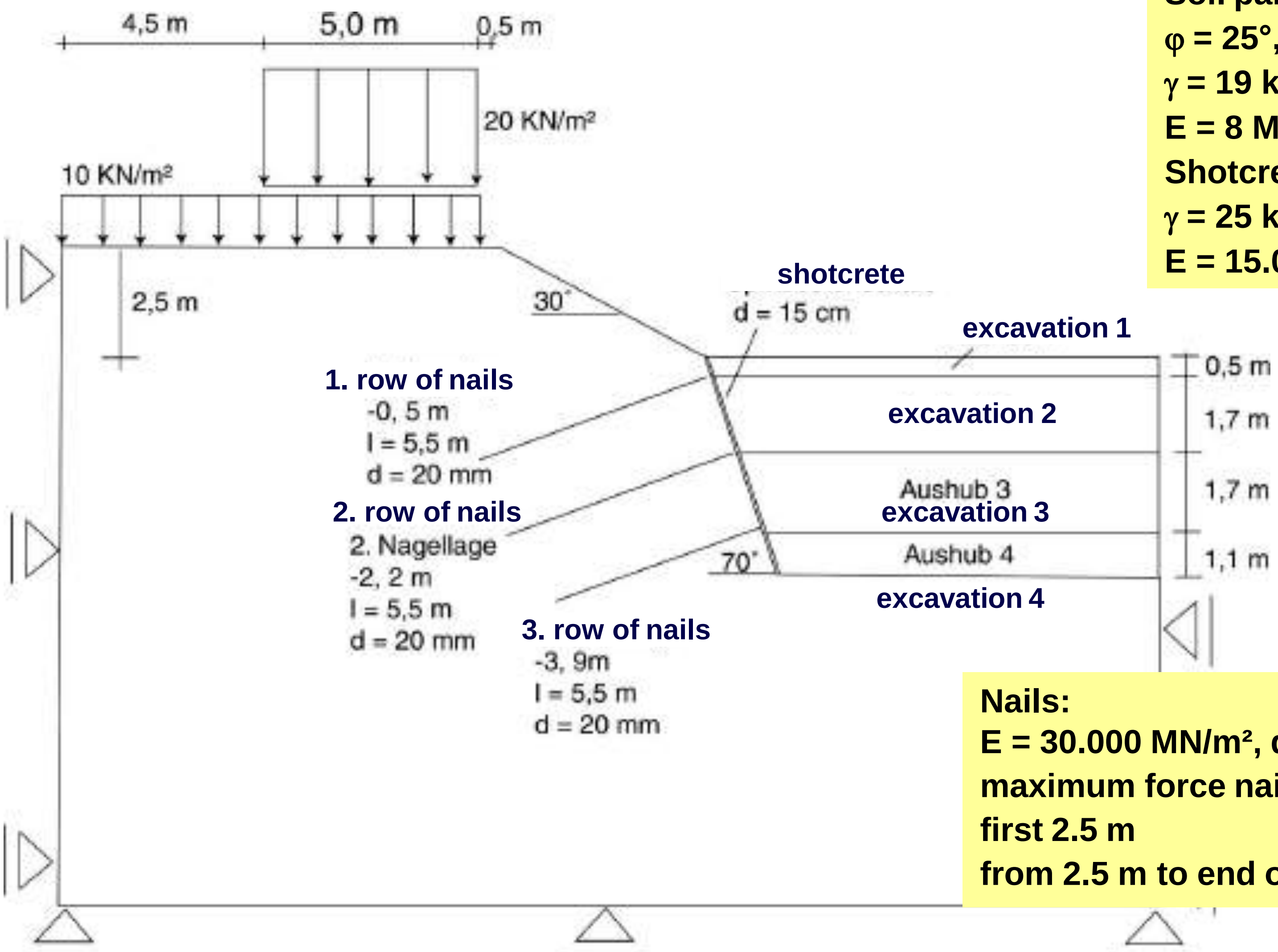


UNDRAINED STABILITY OF A SLOPE

Automatic detection of most critical shear surface



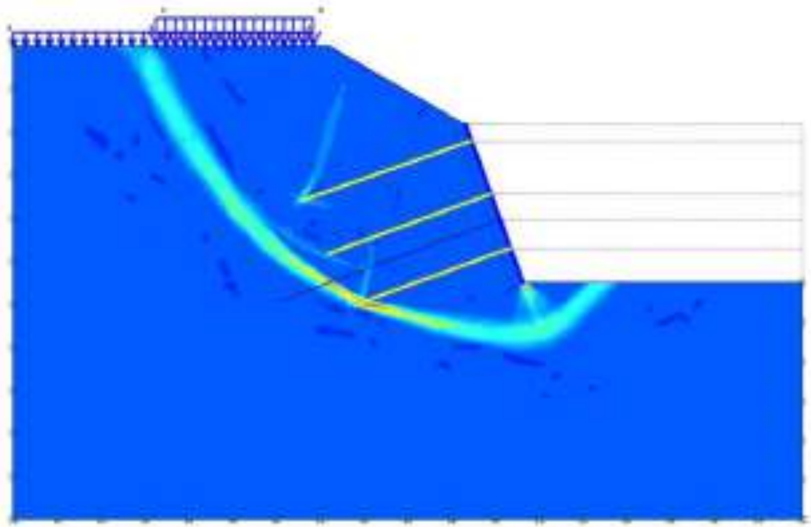
EXAMPLE - SOIL NAILING

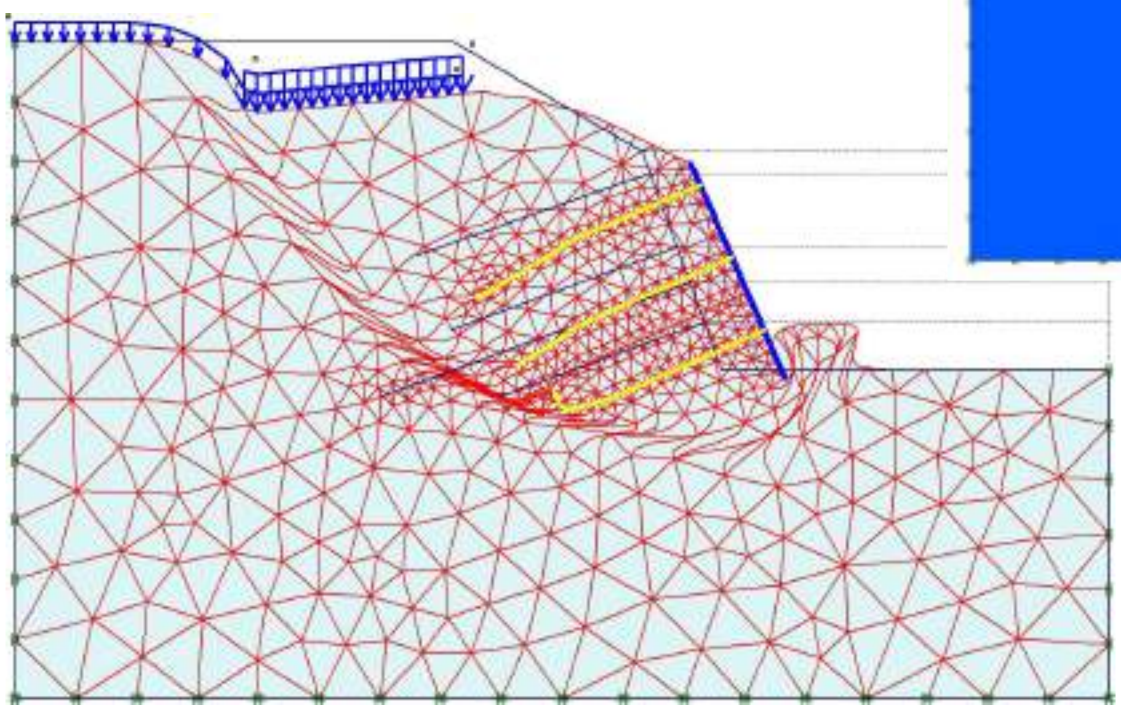


Soil parameters:
φ = 25°, c = 5 kN/m²
γ = 19 kN/m³
E = 8 MN/m², ν = 0.37
Shotcrete:
γ = 25 kN/m³
E = 15.000 MN/m², ν = 0.18

Nails:
E = 30.000 MN/m², d = 20 mm
maximum force nails can take:
first 2.5 m F_{max} = 75 kN
from 2.5 m to end of nail F_{max} = 40 kN

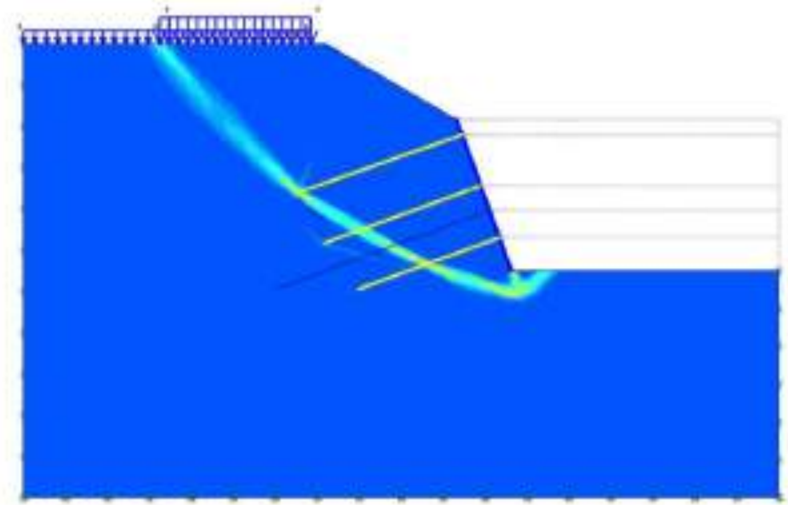
EXAMPLE - SOIL NAILING

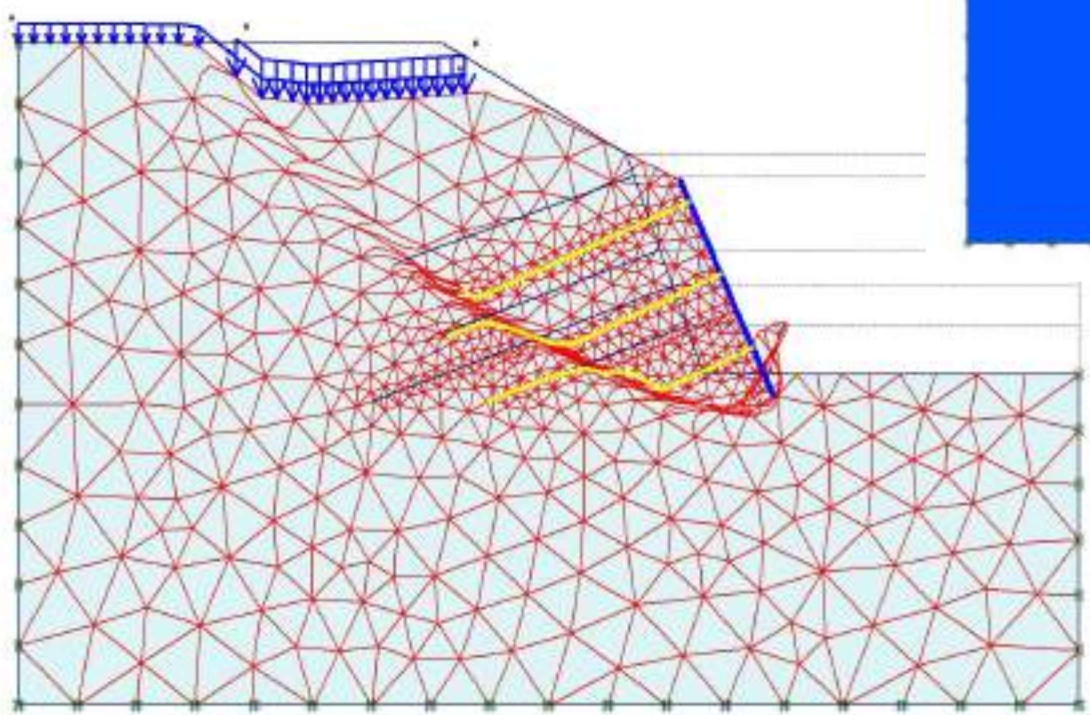




Nails elastic: safety factor from φ/c - reduction: 1.18

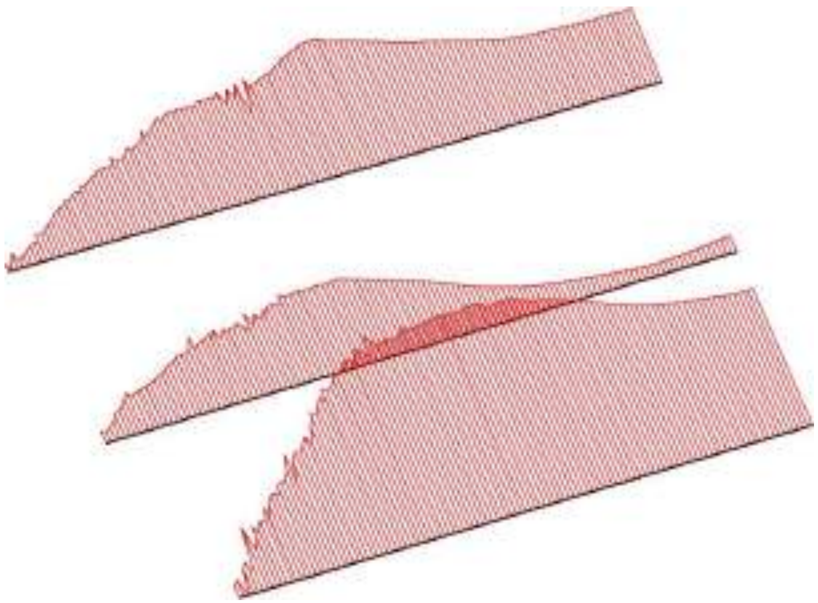
EXAMPLE - SOIL NAILING



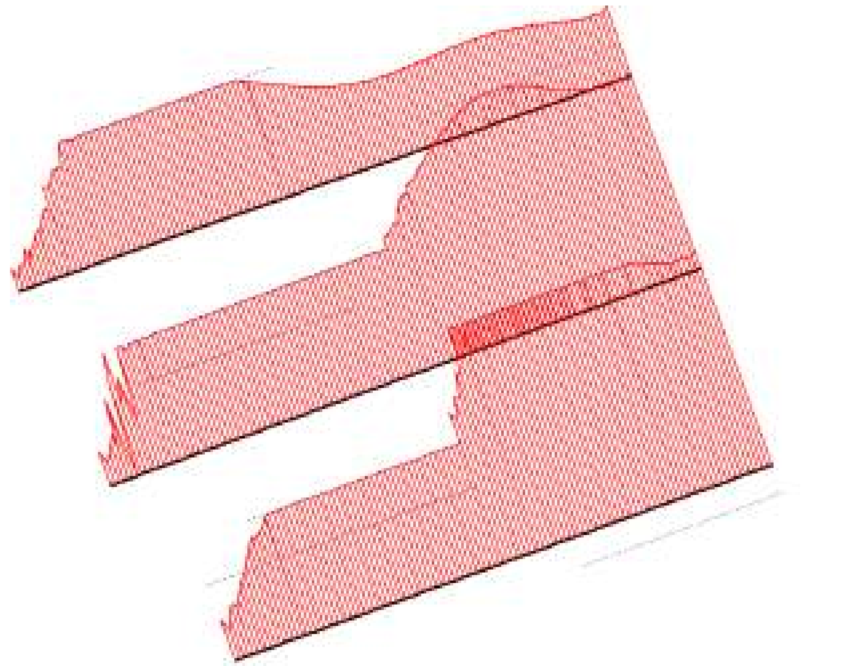


Nails plastic: safety factor from φ/c - reduction: 1.07

EXAMPLE - SOIL NAILING



nails elastic: max. force 181 kN/m

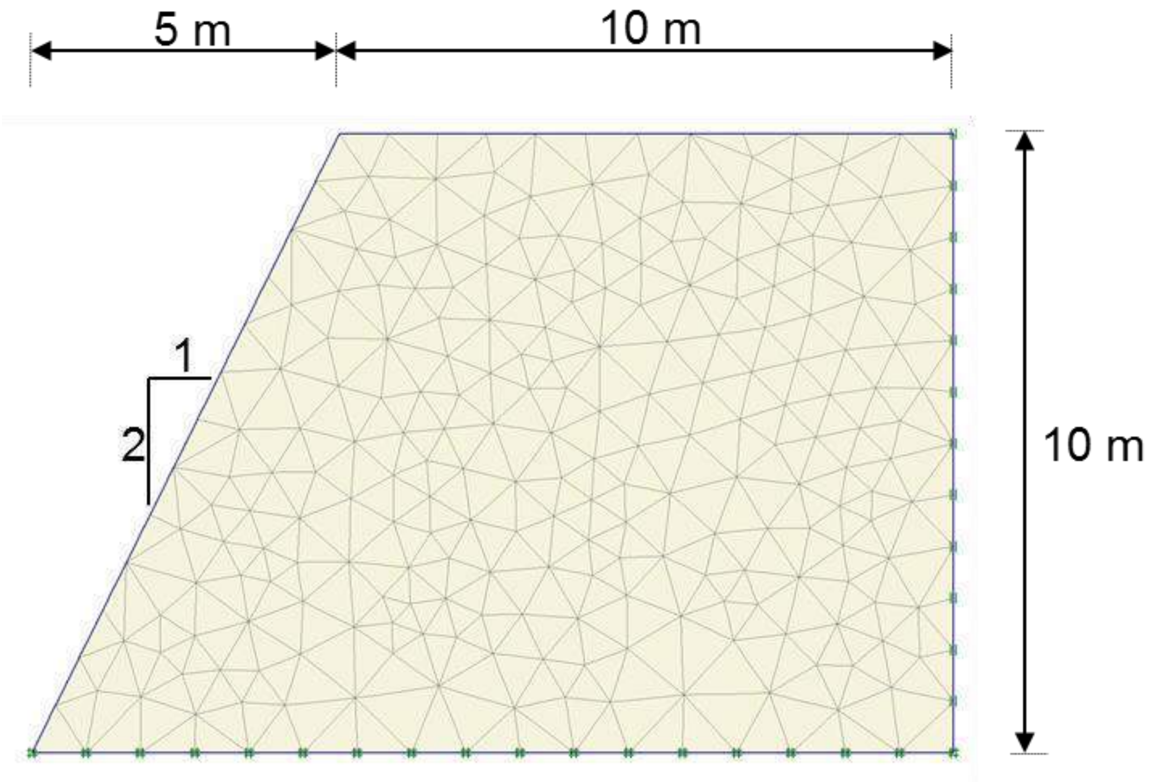


nails plastic: max. force 75 / 40 kN/m as specified

SOME FINAL ADVICE FOR PHI-C-REDUCTION

- Always inspect the incremental displacements or strains as computed in the last load step to make sure that failure is reached.
- The mesh used in the calculation needs to be sufficiently fine.
- Mesh: Refine and redo the phi-c analysis until the factor of safety remains constant upon further refinement of the mesh.
- Always use the arc-length time stepping procedure within the phi-c-reduction (default)
- Use a small tolerated error (maximum should be the default error of 1%)
- Beware of three-dimensional effects
- Check for local, not relevant, failure mechanisms

EXAMPLE OF STEEP SLOPE



Reference: Conte, E.; Silvestri, F.; Troncone, A. (2010)
Stability analysis of slopes in soils with strain-softening behaviour.
Computers and Geotechnics, Vol. 37 (5), 710-722.

RESULTS

high cohesion

Plaxis 2D vs Slide 6.0				
		Slide 6.0 - FoS	Plaxis 2D	
friction angle φ‘	cohesion c‘	Morgenstern-Price	FoS for φ = ψ	FoS for ψ = 0
[°]	[kPa]	[-]	[-]	[-]
15°	29.0	1.194	1.035	1.013
	33.0	1.300	1.131	1.112
22,5°	21.5	1.097	1.018	KOLLAPS
	28.0	1.321	1.187	1.134
30°	17.0	1.097	1.056	KOLLAPS
	23.0	1.308	1.230	1.139
35	14.0	1.103	1.074	KOLLAPS
	19.5	1.299	1.247	.,126
40°	11.0	1.105	1.082	KOLLAPS
	16.0	1.294	1.260	1.108
45°	8.0	1.098	1.080	KOLLAPS
	12.5	1.292	1.266	1.083
50°	5.5		1.085	KOLLAPS
	9.5	1.305	1.286	1.060

high friction

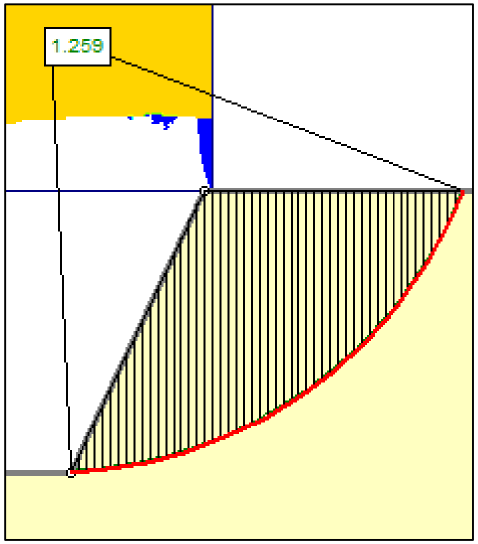
HOMOGENEOUS SLOPE

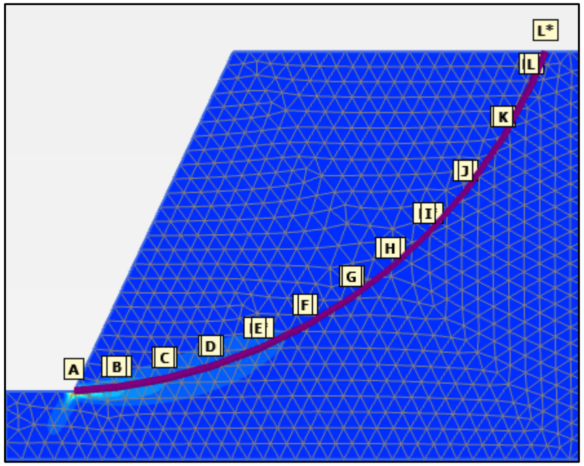
φ' = 15 [°]	cohesion c'	Slide	Plaxis (φ=ψ)	Plaxis (ψ=0)	Plaxis (φ=ψ)	Plaxis (ψ=0)
	[kPa]	Morgenstern-Price	no tension	no tension	with tension	with tension
	26.5	1.135	1.035	1.013	1.044	1.018
	33.0	1.300 ~	1.131	1.112		1.198
φ' = 30 [°]	**cohesion c'**		**Plaxis (φ=ψ)**	**Plaxis (ψ=0)**	**Plaxis (φ=ψ)**	**Plaxis (ψ=0)**
	[kPa]	Morgenstern-Price	no tension	no tension	with tension	with tension
	17.0	1.101	1.056	0.930	1.098	1.050
	23.0	1.312	1.230	1.139	1.288	1.178
φ' = 45 [°]	**cohesion c'**		**Plaxis (φ=ψ)**	**Plaxis (ψ=0)**	**Plaxis (φ=ψ)**	**Plaxis (ψ=0)**
	[kPa]	Morgenstern-Price	no tension	no tension	with tension	with tension
	8.0	1.100	1.080	0.814	1.100	1.050
	12.5	1.293	1.266	1.083	1.298	1.000

INFLUENCE OF TENSION CUT OFF / TENSION CRACK

Extreme case:
$\varphi = 0°$
$c = 50$ kPa
$\beta = 65°$

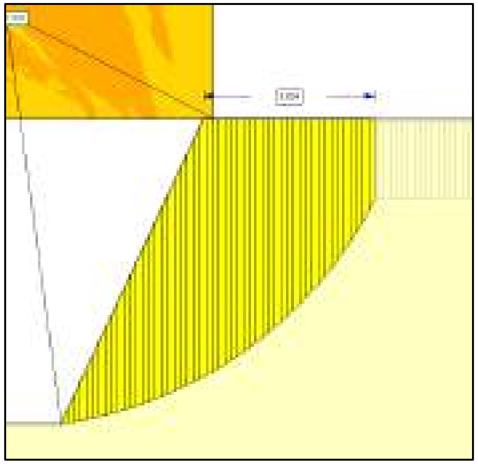
Slide - no tension crack: FoS = 1.259

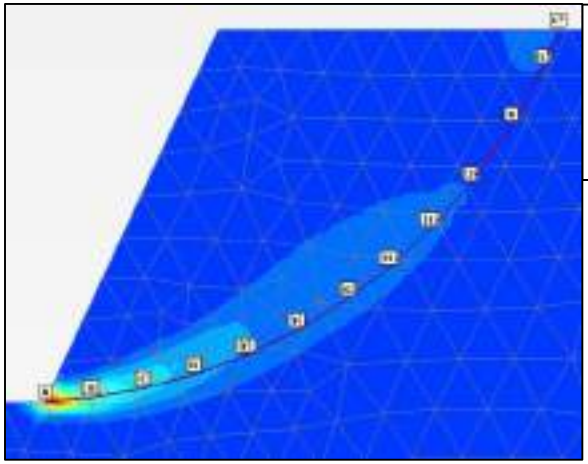




Plaxis:
FoS = 1.255
tco = 100 kPa

Slide - with tension crack: FoS = 1.094





Plaxis:
FoS = 1.092
tco = 0 kPa

SUMMARY

- **Strength reduction techniques (phi-c-reduction) are well suited to calculate factors of safety for slopes**
- **It can be shown that results compare extremely well with rigorous limit analyses**
- **For high values of friction angles and low values of dilatancy angles numerical instabilities may occur**
- **Numerical instabilities of non-associated displacement finite element analyses can be avoided by employing a modified Davis approach.**
- **The modified Davis procedure can be used with displacement based finite element analysis and finite element limit analysis**

References:

Tschuchnigg, F., Schweiger, H.F., Sloan, S.W. (2015): Slope stability analysis by means of finite element limit analysis and finite element strength reduction techniques - Part I: Numerical studies considering non-associated plasticity. Computers and Geotechnics, 70, 2015, 169-177.

Tschuchnigg, F., Schweiger, H.F., Sloan, S.W. (2015): Slope stability analysis by means of finite element limit analysis and finite element strength reduction techniques - Part II: Back analyses of a case history. Computers and Geotechnics, 70, 2015, 178-189.

Tschuchnigg, F., Schweiger, H.F., Sloan, S.W., Lyamin, A.V., Raissakis, I. (2015): Comparison of finite-element limit analysis and strength reduction techniques. Geotechnique, 65, 2015, 249-257.

SCIENCE ▪ PASSION ▪ TECHNOLOGY

CG16

UNSATURATED SOILS

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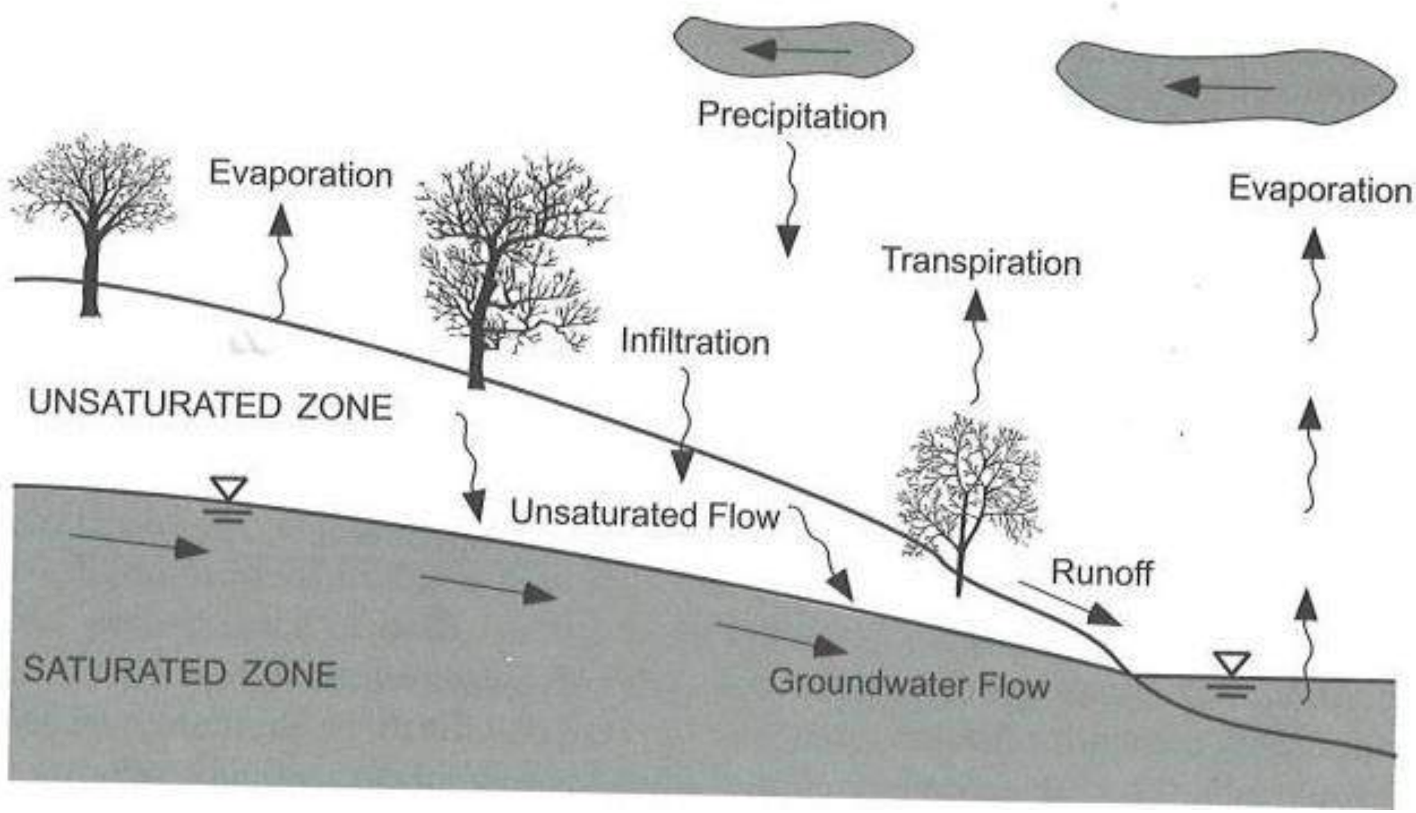
Pelatihan software Plaxis di Tembagapura (In-house training) PT. Freeport Indonesia/ Timika, Papua, 22-30 September 2018

CONTENT

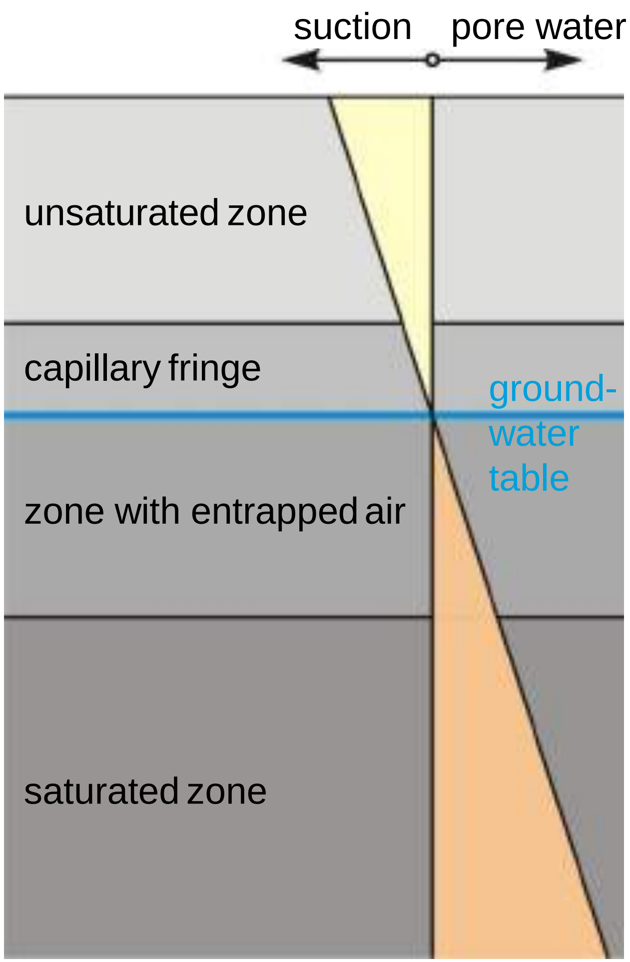
PARTIAL SATURATION OF SOILS



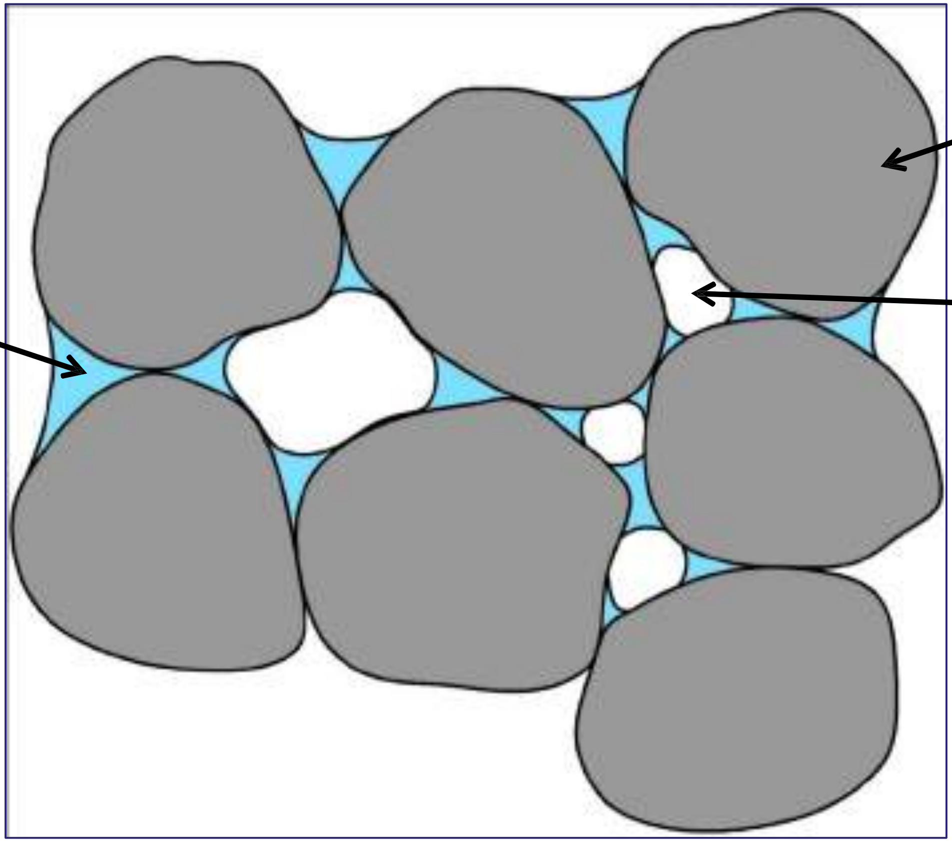
PARTIAL SATURATION OF SOILS

Regions in a soil segment:

- suction:
  - unsaturated zone (unsaturated)
  - capillary fringe (quasi-saturated – suction)
- pore water pressure:
  - zone with entrapped air (quasi-saturated – pore water pressure)
  - saturated zone (saturated)
- for fluctuating water tables above ground surface zone with entrapped air is relevant



PARTIAL SATURATION OF SOILS



$$n = \frac{V_{pores}}{V_{total}} = \frac{V_{liquid} + V_{gas}}{V_{total}}$$

Porosity

$$S_r = \frac{V_{liquid}}{V_{pores}} = \frac{V_{liquid}}{V_{liquid} + V_{gas}} = 1 - S_g$$

Degree of saturation

EXAMPLES FOR THE INFLUENCE OF PARTIAL SATURATION OF SOILS

Shum Wan Road landslide, Hong Kong Island on August 13th,1995

Photographs from Geotechnical Engineering Office, Hong Kong

EXAMPLES FOR THE INFLUENCE OF PARTIAL SATURATION OF SOILS

Failure triggered by extensive rainfall (Indonesia, 2010)

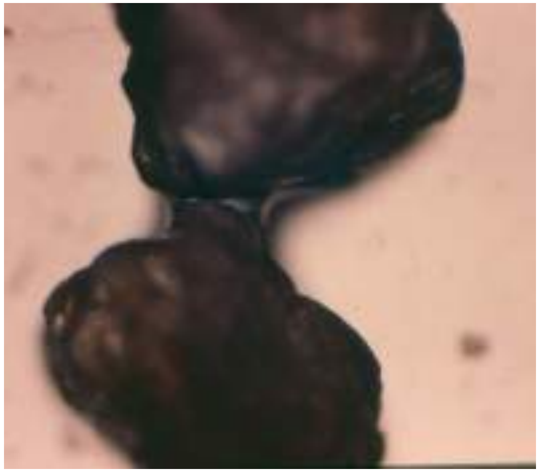
EXAMPLES FOR THE INFLUENCE OF PARTIAL SATURATION OF SOILS

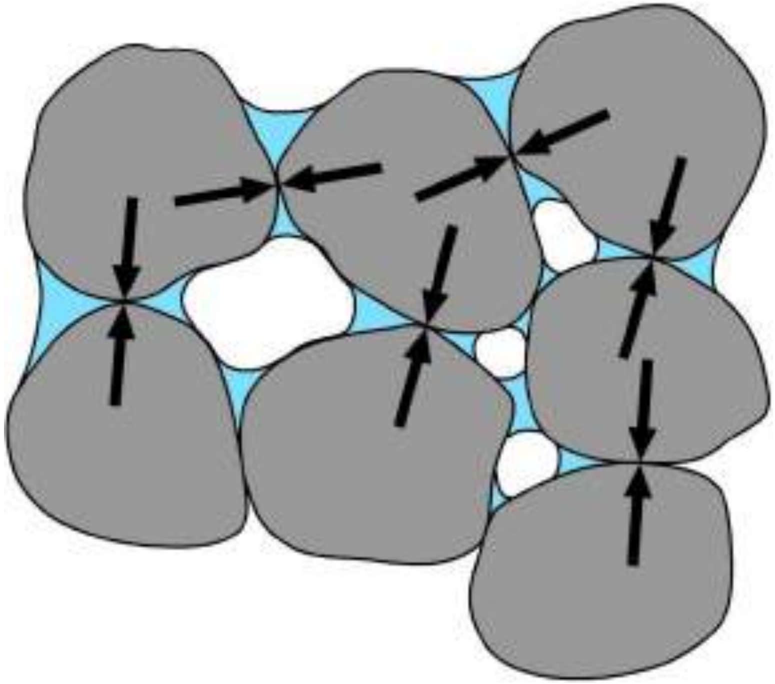
Collapse in Via Luigi, Settembrini, Naples, (15-09-2001)

CAPILLARY ACTION

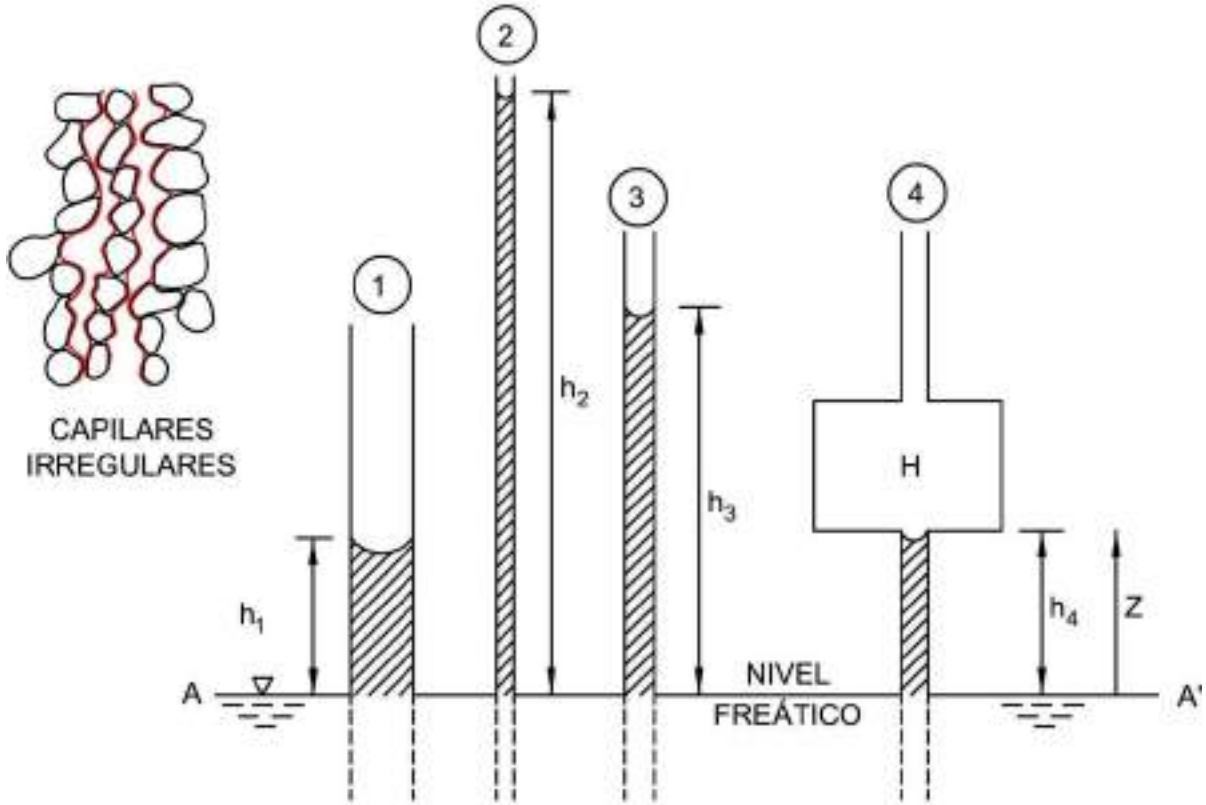
$$\boxed{s = p_a - p_w}$$

w …. water
a …. air





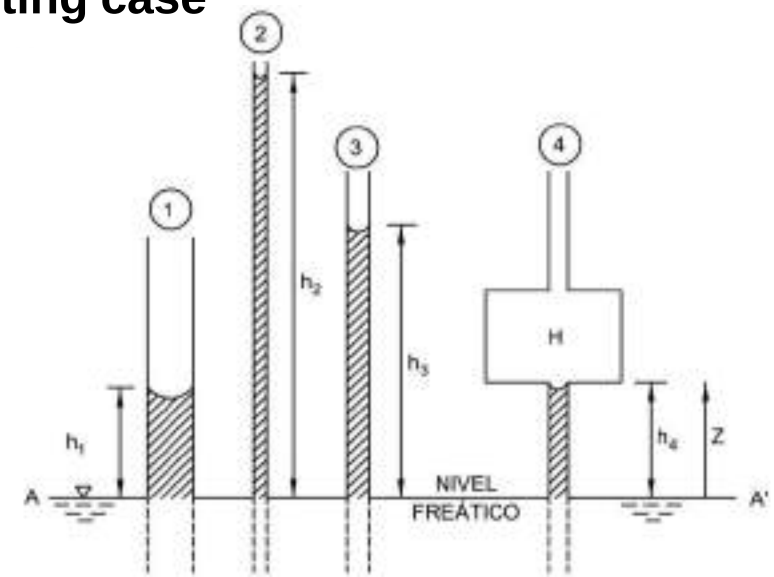
CAPILLARY RISE HEIGHT



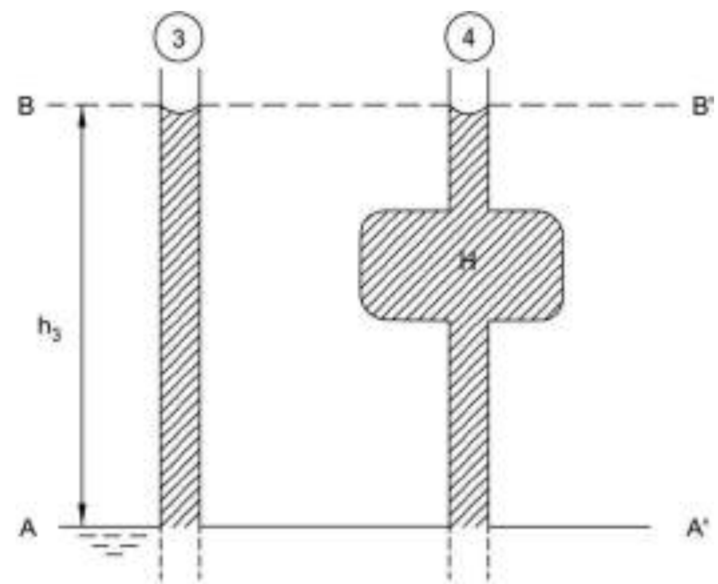
Capillary rise height is irregular and depends on the geometry of the pore space.

CAPILLARY HYSTERESIS

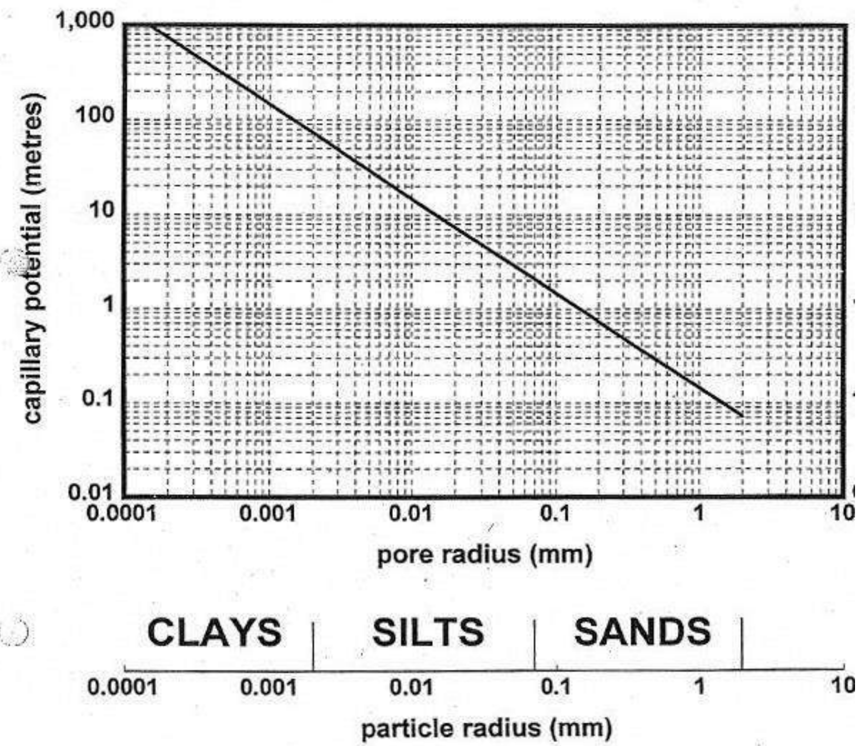
Wetting case



Drying case

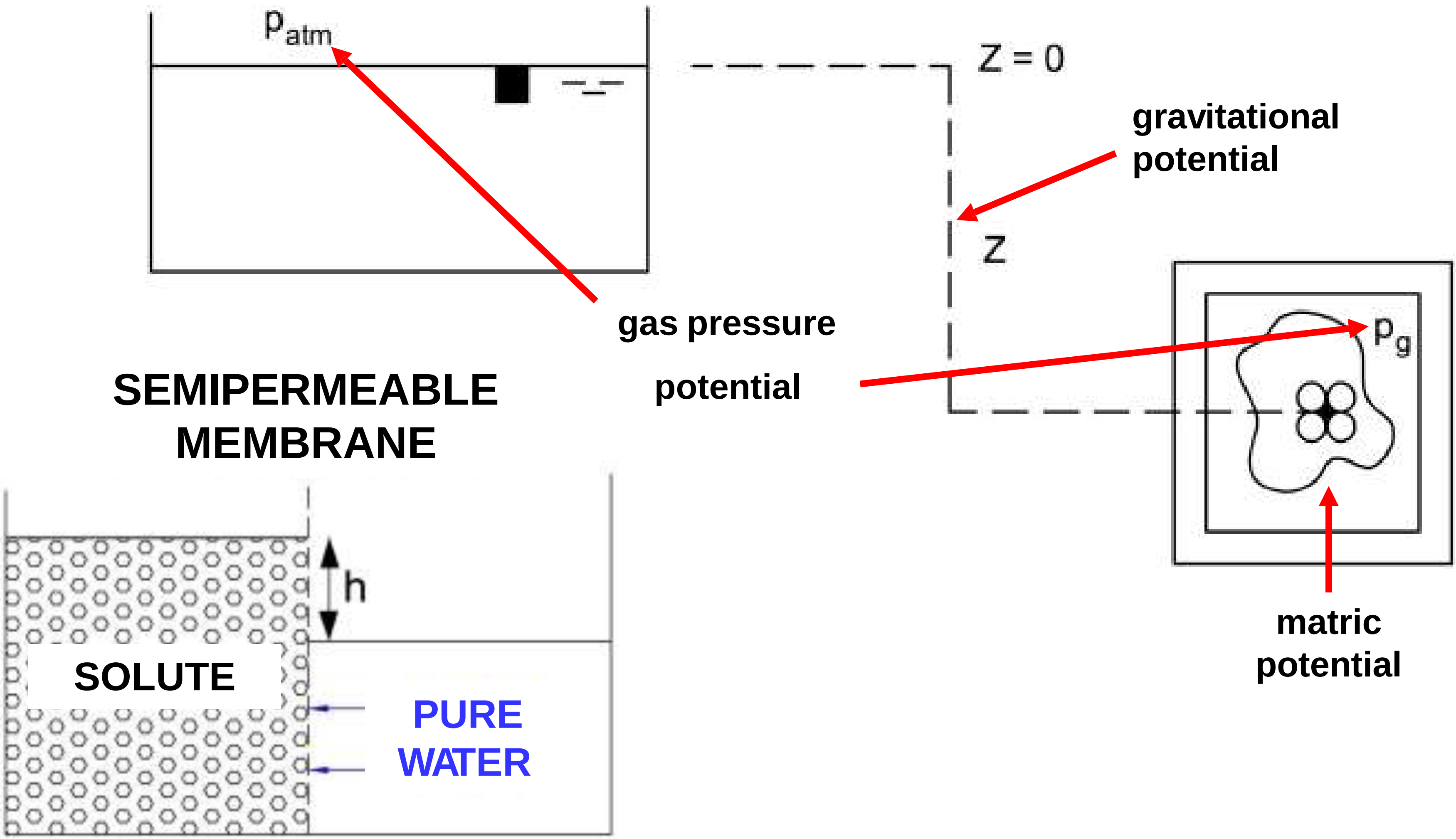


CAPILLARY POTENTIAL



soil type	grain size [mm]	$H_{k\ max}$
sand	*2.0 – 0.6* *0.6 – 0.2* *0.2 – 0.1* *0.1 – 0.06*	*3 – 10 cm* *10 – 30 cm* *30 – 60 cm* *60 – 100 cm*
silt	*0.06 – 0.02* *0.02 – 0.006* *0.006 – 0.002*	*1 – 3 m* *3 – 10 m* *10 – 30 m*
clay	*0.002*	*30 – 300 m*

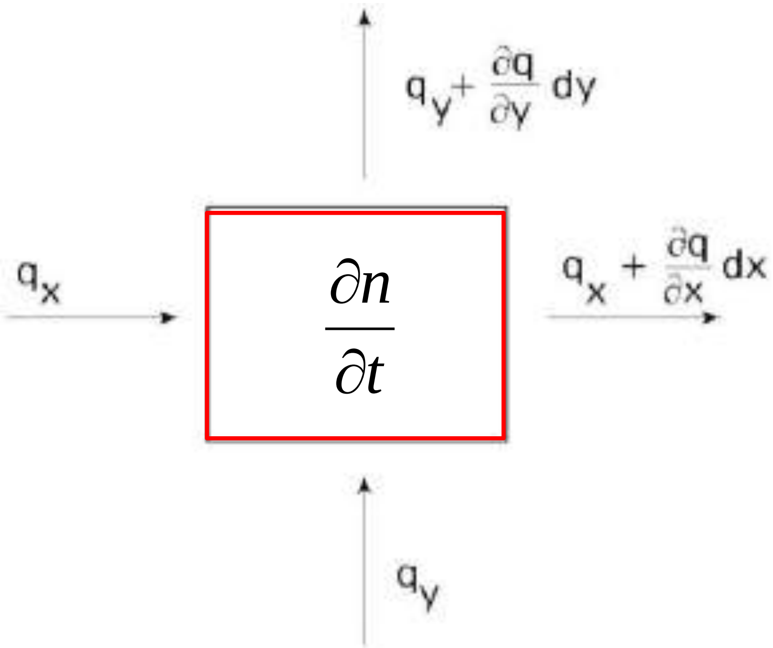
WATER POTENTIAL Ψ



FLOW AND CONSOLIDATION

- **Saturated soils: equation of continuity (with soil deformation)**

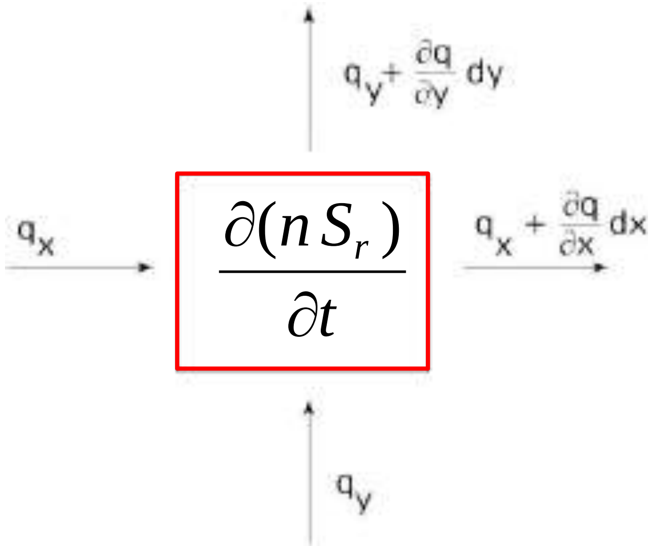
$$\frac{\partial n}{\partial t}+\left(\frac{\partial q_x}{\partial x}+\frac{\partial q_y}{\partial y}\right)=0$$



FLOW AND CONSOLIDATION

- **Unsaturated soils: equation of continuity (with soil deformation)**

$$\frac{\partial (n S_r)}{\partial t} + \left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} \right) = 0$$

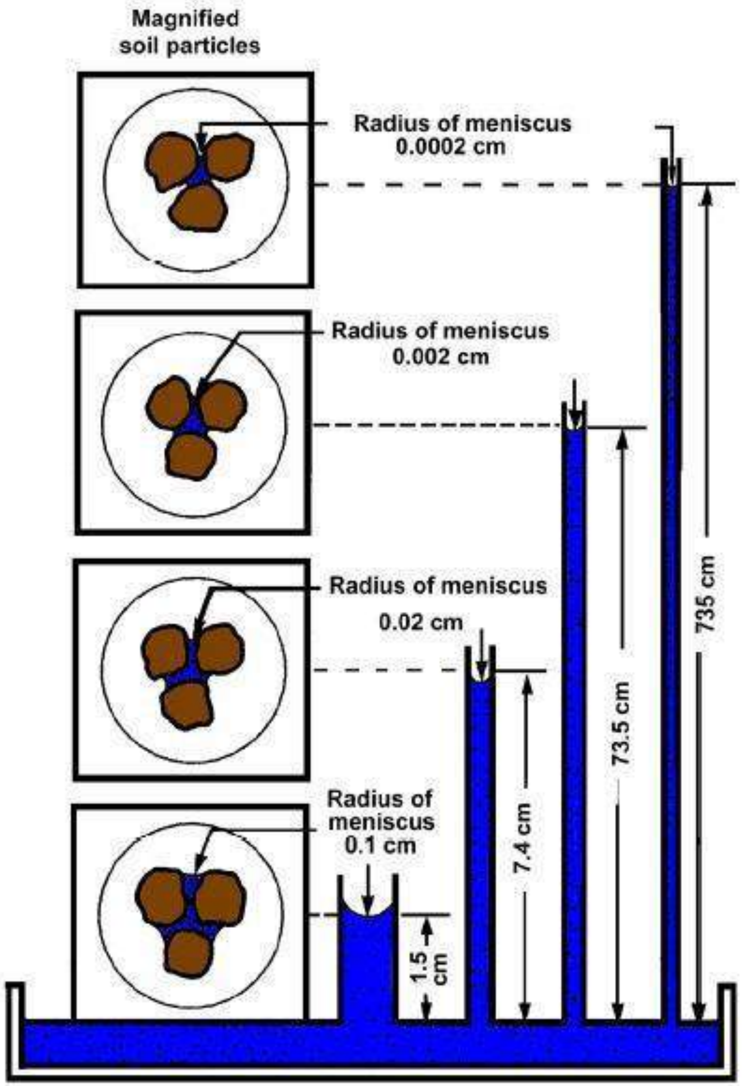


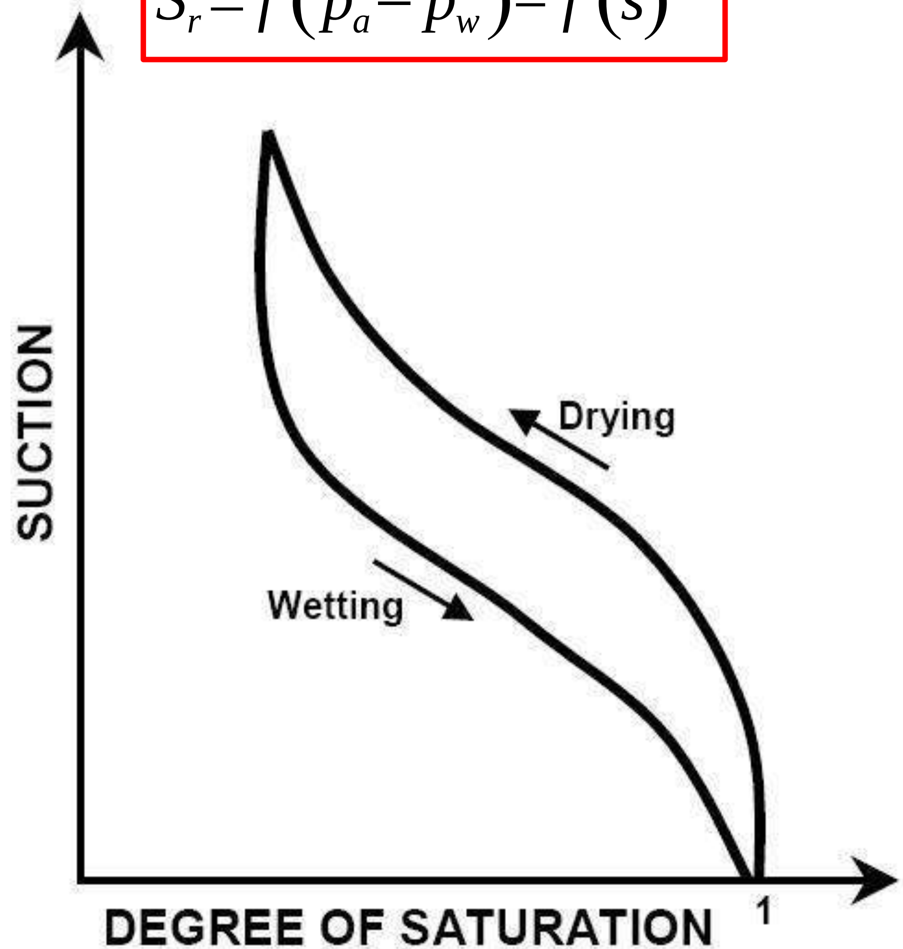
$$\frac{\partial n(\sigma, s)}{\partial t} S_r + \frac{\partial S_r(\sigma, s)}{\partial t} n + \left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} \right) = 0$$

Constitutive law **Retention curve** **Darcy's law**

SOIL WATER CHARACTERISTIC CURVE (SWCC) / RETENTION CURVE

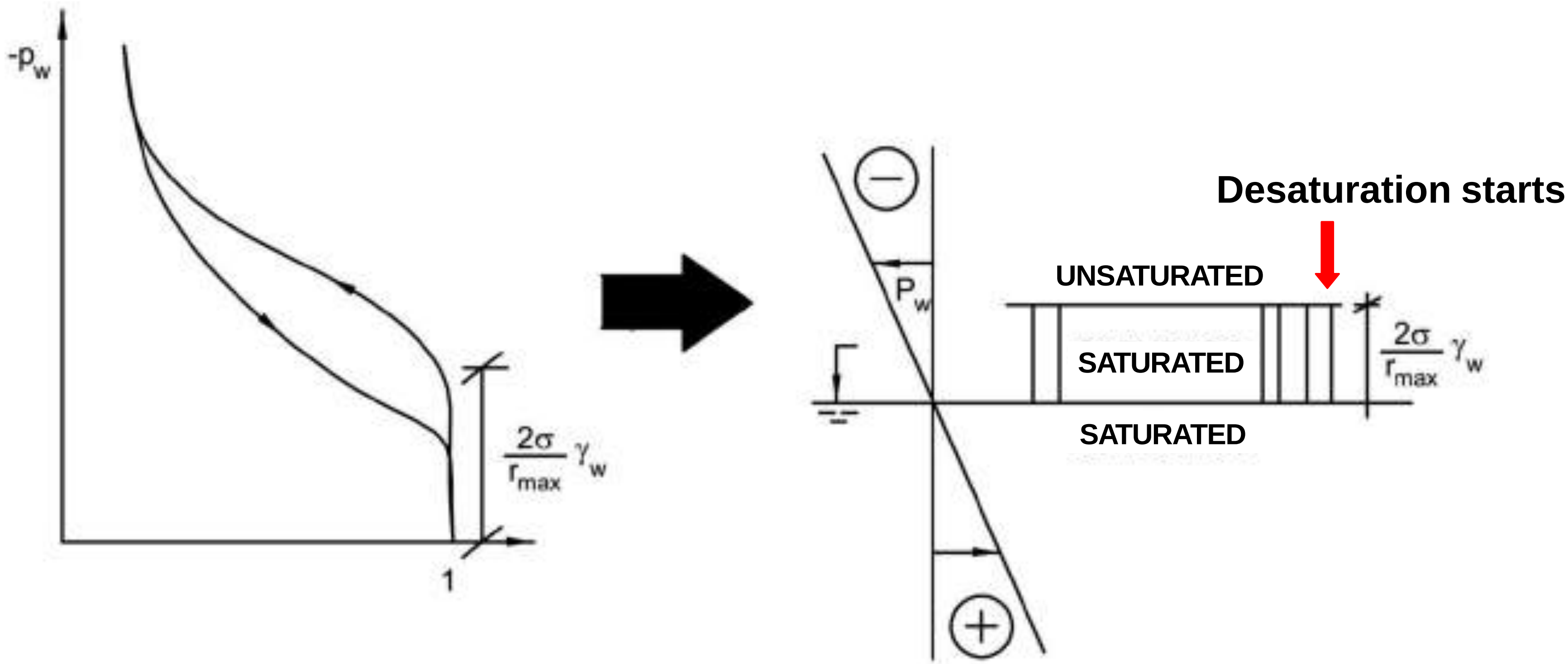
$$S_r = f(p_a - p_w) = f(s)$$



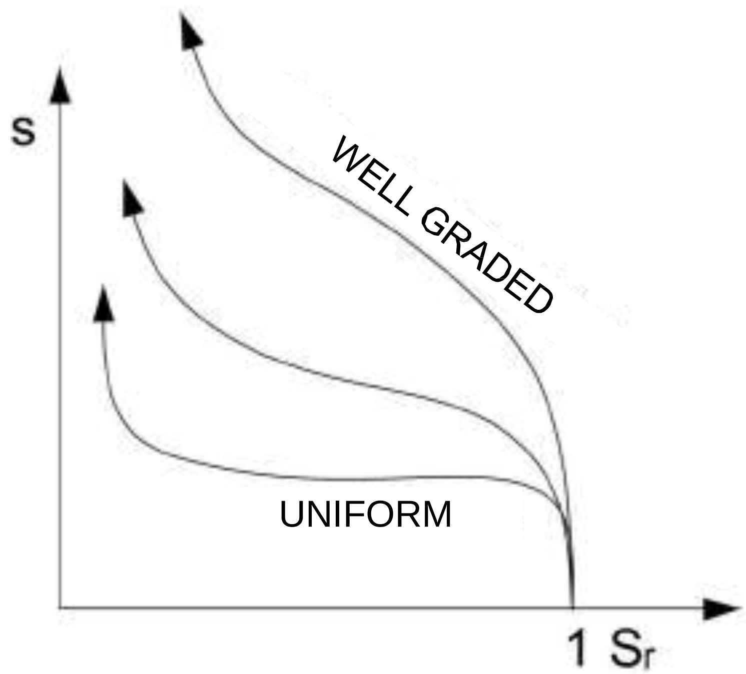


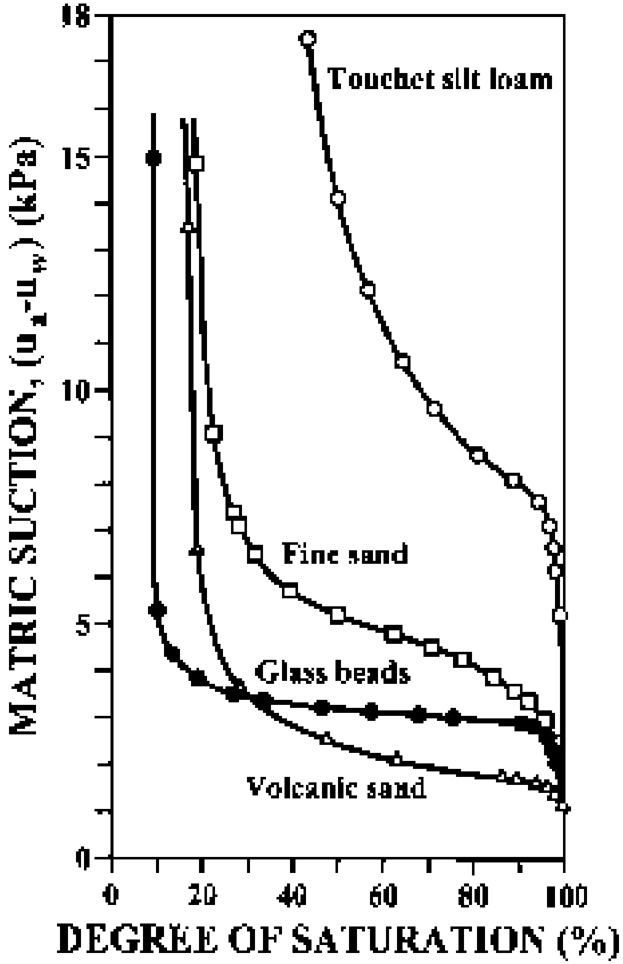
RETENTION CURVES

- **Retention curves exhibit hysteresis effects**
- **This value of negative water pressure is called the air entry value (small for sands, large for clays) for that soil (suction is not strong enough to draw out water from the soil)**
- **Negative pore pressures can exist in saturated soils**



RETENTION CURVES - EXAMPLES

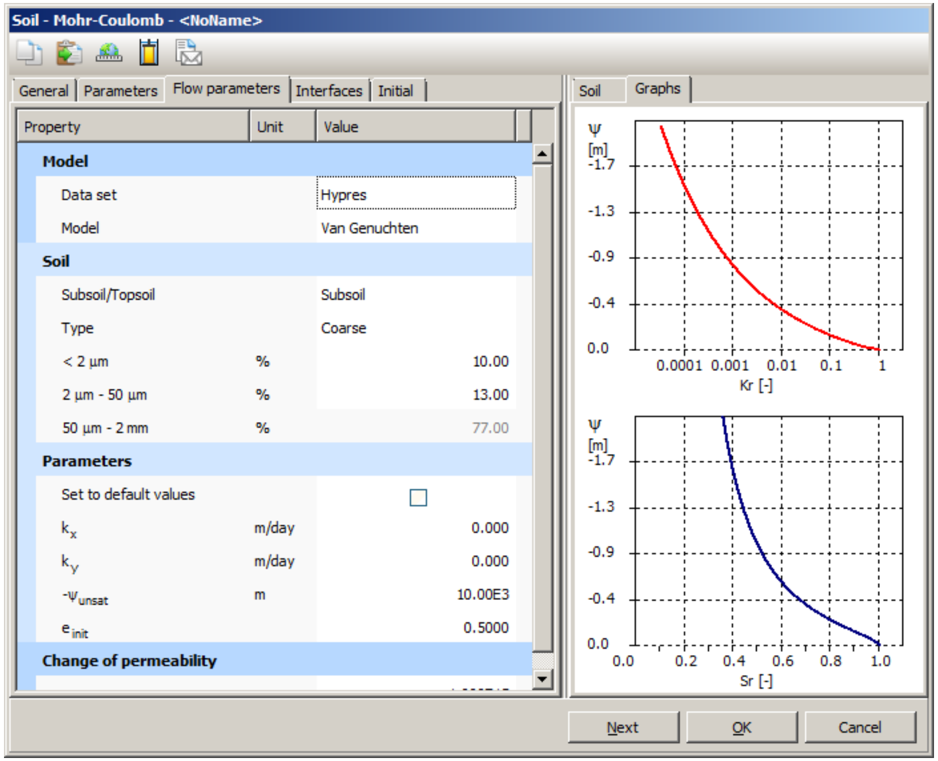




(Brooks and Corey, 1964)

HYDRAULIC PARAMETERS: DATA SET

- **Standard (Topsoil part of Hypres)**
- **Hypres (International soil classification system)**
- **USDA (International soil classification system)**
- **Staring (Dutch soil classification system)**
- **User-defined**



- **Parameter values are provided depending on soil classification**
- **Hydraulic parameters must be chosen very carefully!**

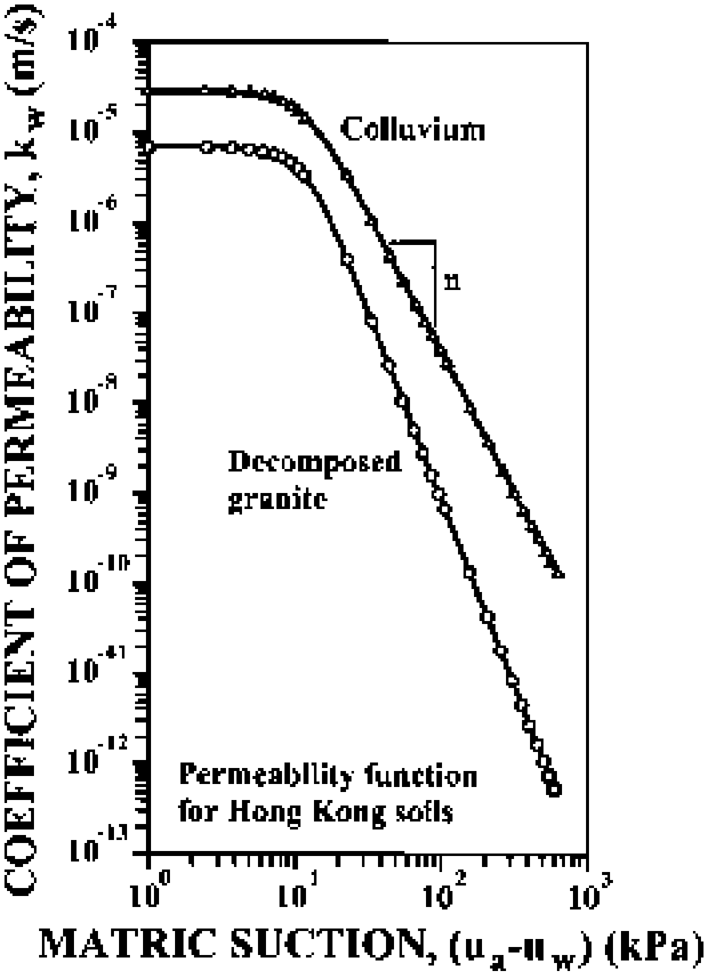
PERMEABILITY

Unsaturated soils: Darcy's law

$$q = -k\frac{\partial h}{dy} = -k\frac{\partial \phi}{dy}$$

$$h = \phi = y + \frac{p_w}{\gamma_w}$$

$$k = k_{rel}(S_r)\,k_{sat}$$

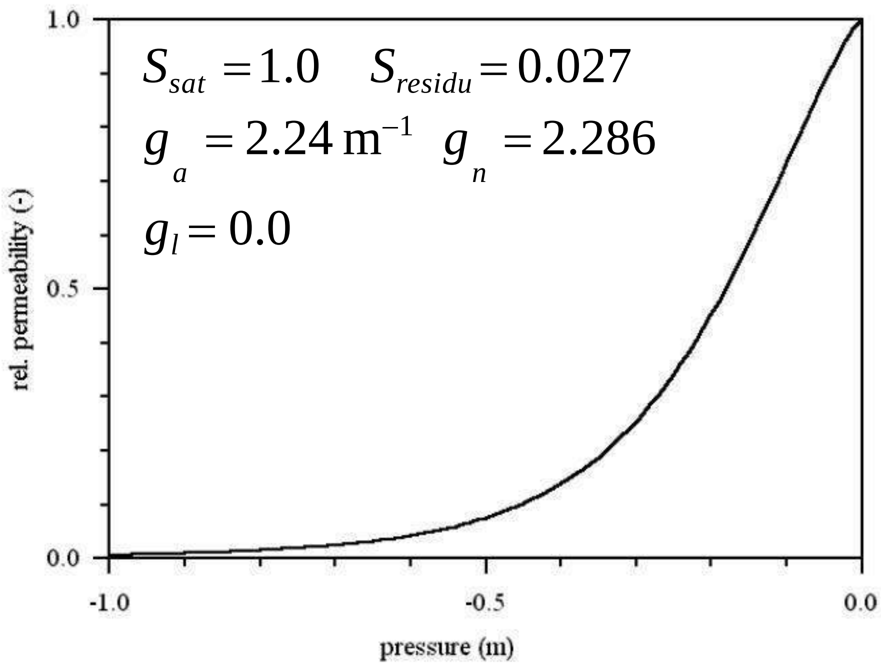


Fredlund & Rahardjo (1993)

Permeability decreases in partially saturated soils

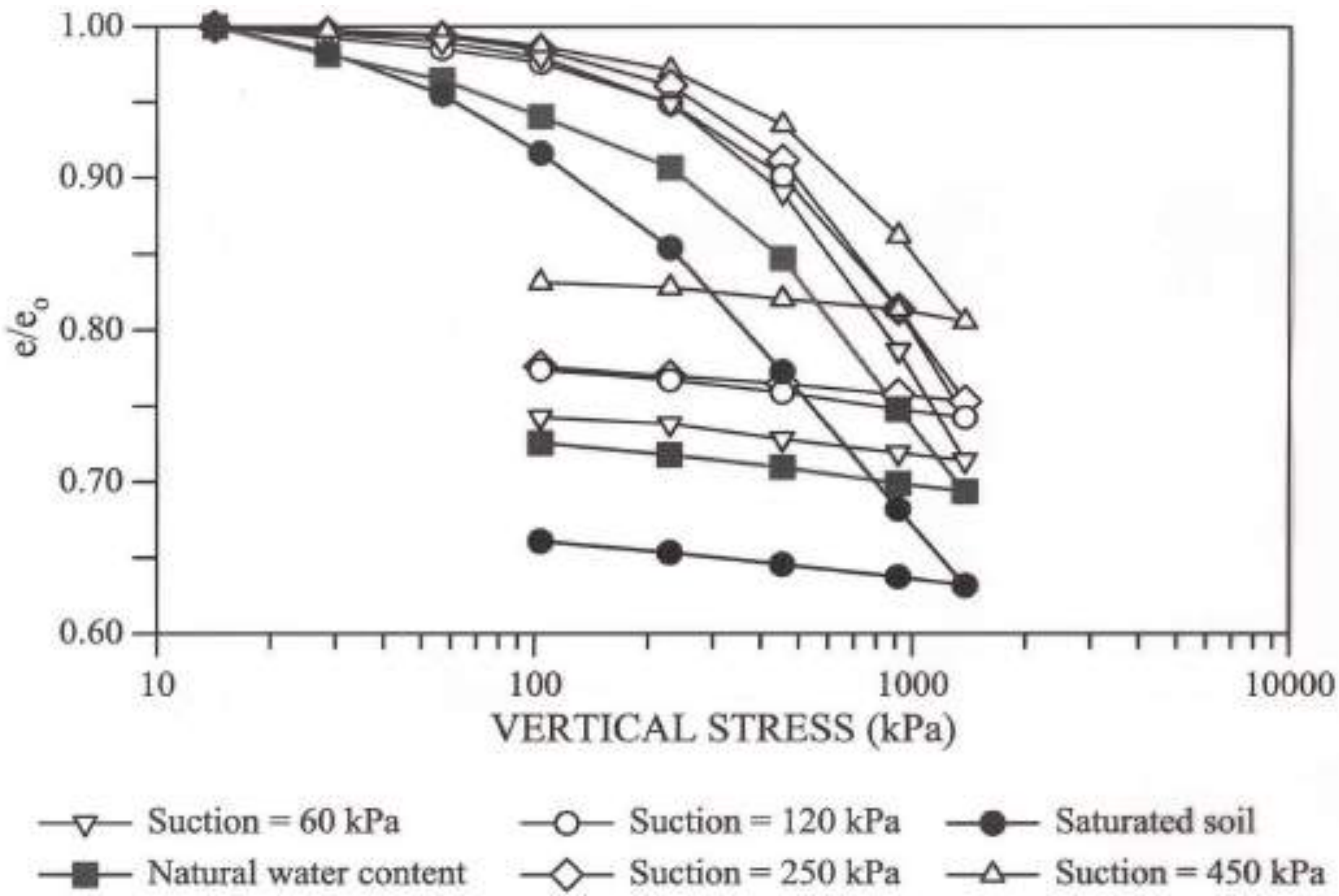
PERMEABILITY

$$k_{rel}(S_r) = (S_e)^{g_l}\left(1-\left(1-S_e^{\left(\frac{g_n}{g_n-1}\right)}\right)^{\left(\frac{g_n-1}{g_n}\right)}\right)^2 \qquad S_e = \frac{S_r - S_{residu}}{S_{sat} - S_{residu}}$$



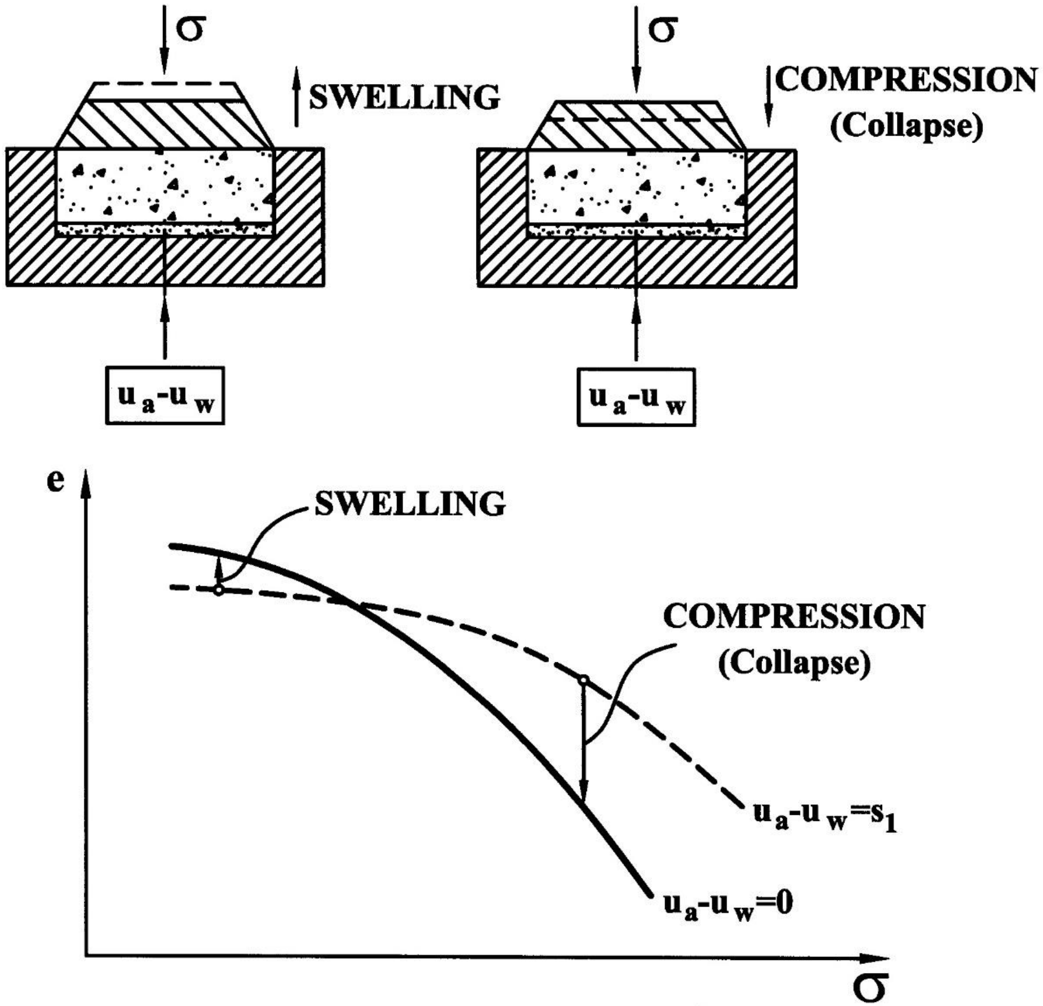
VOLUMETRIC BEHAVIOUR

Suction increases the apparent preconsolidation stress and (often) soil stiffness



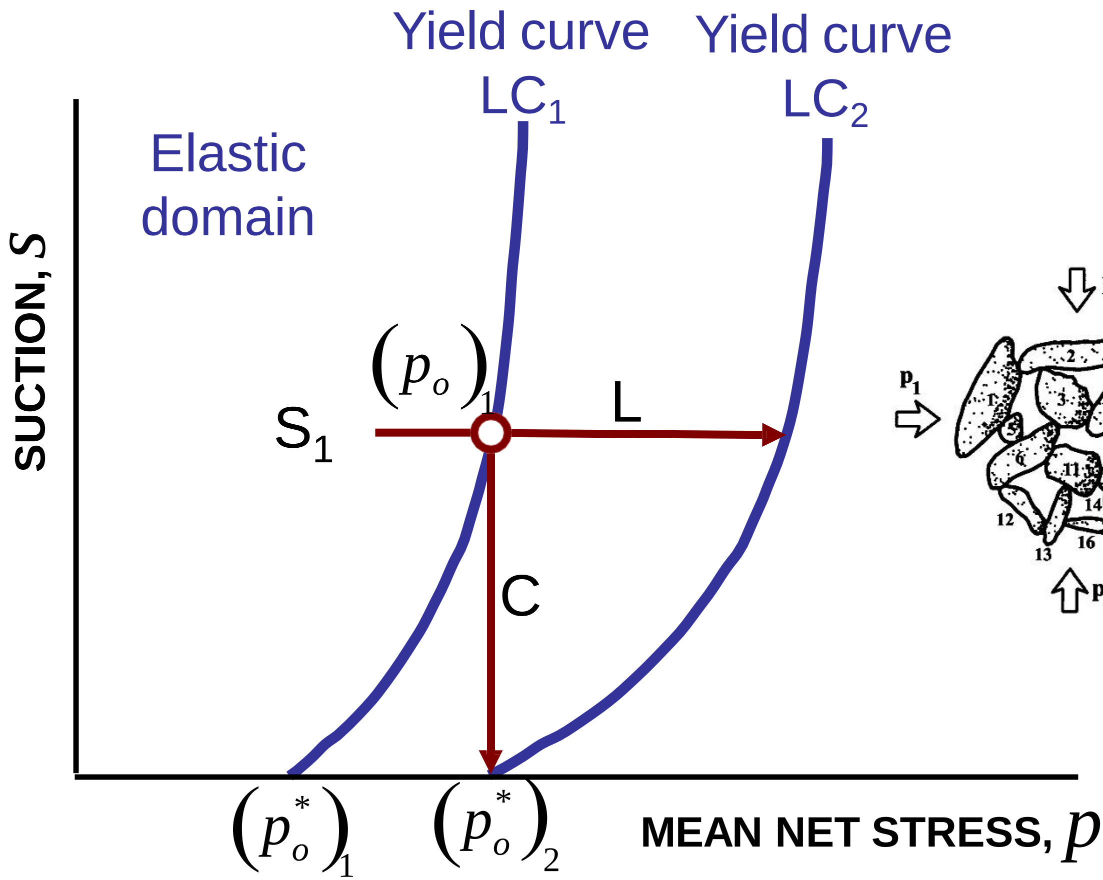
(Oedometer tests on a Brazilian residual soil; Lemos, 1998)

VOLUMETRIC BEHAVIOUR

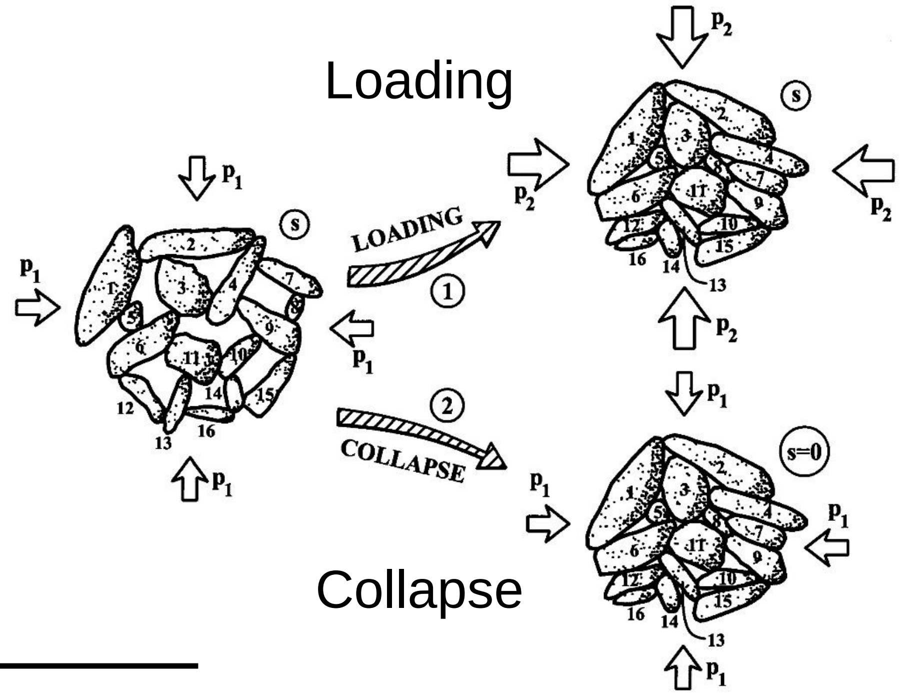


Volume change behaviour on saturation depends on applied stress level

VOLUMETRIC BEHAVIOUR



Isotropic plane



VOLUMETRIC BEHAVIOUR - SUMMARY

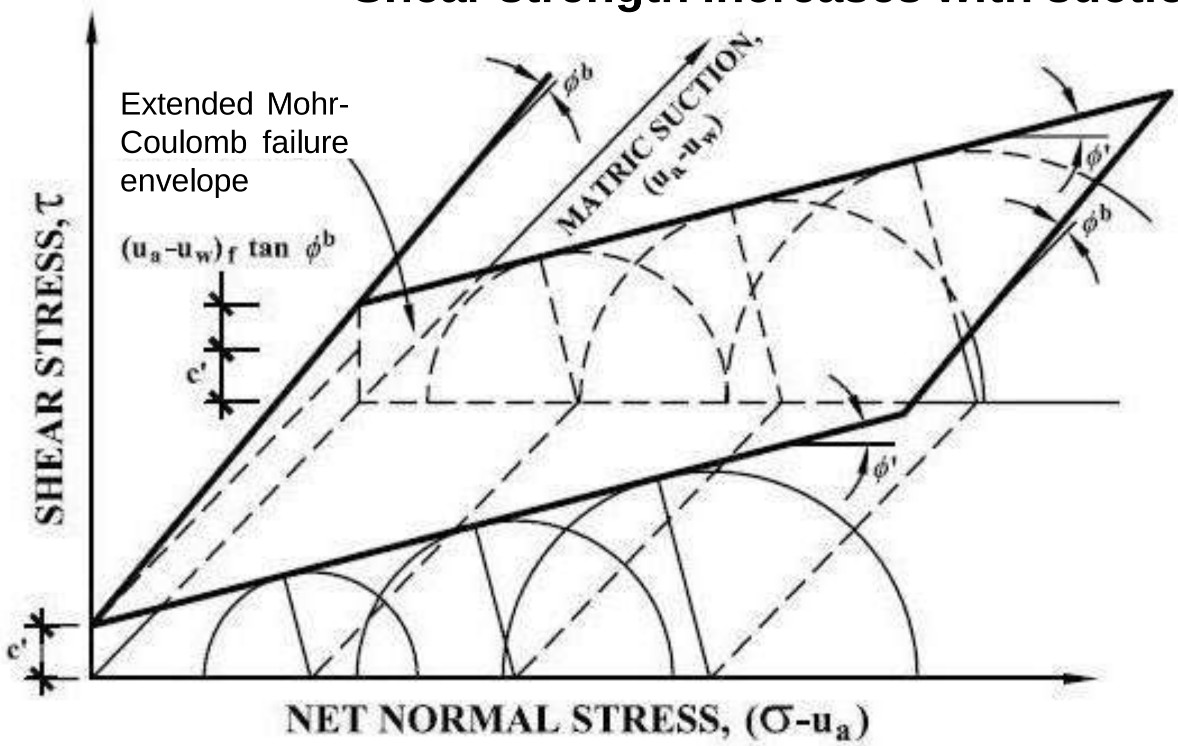
Features of volumetric behaviour:

- suction increases the apparent preconsolidation pressure (yield stress) and (often) soil stiffness
- volumetric behaviour depends on stress level. Swelling or compression (“collapse”) may occur depending on applied load
- collapse behaviour
  - after collapse soil lies on saturated consolidation line
  - volume change reversal may occur during collapse
  - in loose soils, collapse strains reach a maximum at a certain intermediate stress level
- volume change behaviour is path independent only for a certain class of stress paths
- suction changes may lead to irreversible deformations

SOIL STRENGTH

$$\tau_f = c' + (\sigma_n - p_a)\tan\phi' + s\tan\phi^b$$

Shear strength increases with suction



Fredlund & Rahardjo (1985)

Shortcoming: close to saturation ϕ^b should be equal to ϕ' but this gives jump and too high strength with very low saturation

SOIL STRENGTH

Effective stresses in partially saturated soils (Bishop 1959) – other definitions possible

$$\sigma' = (\sigma - u_a) + \chi\,(u_a - u_w)$$

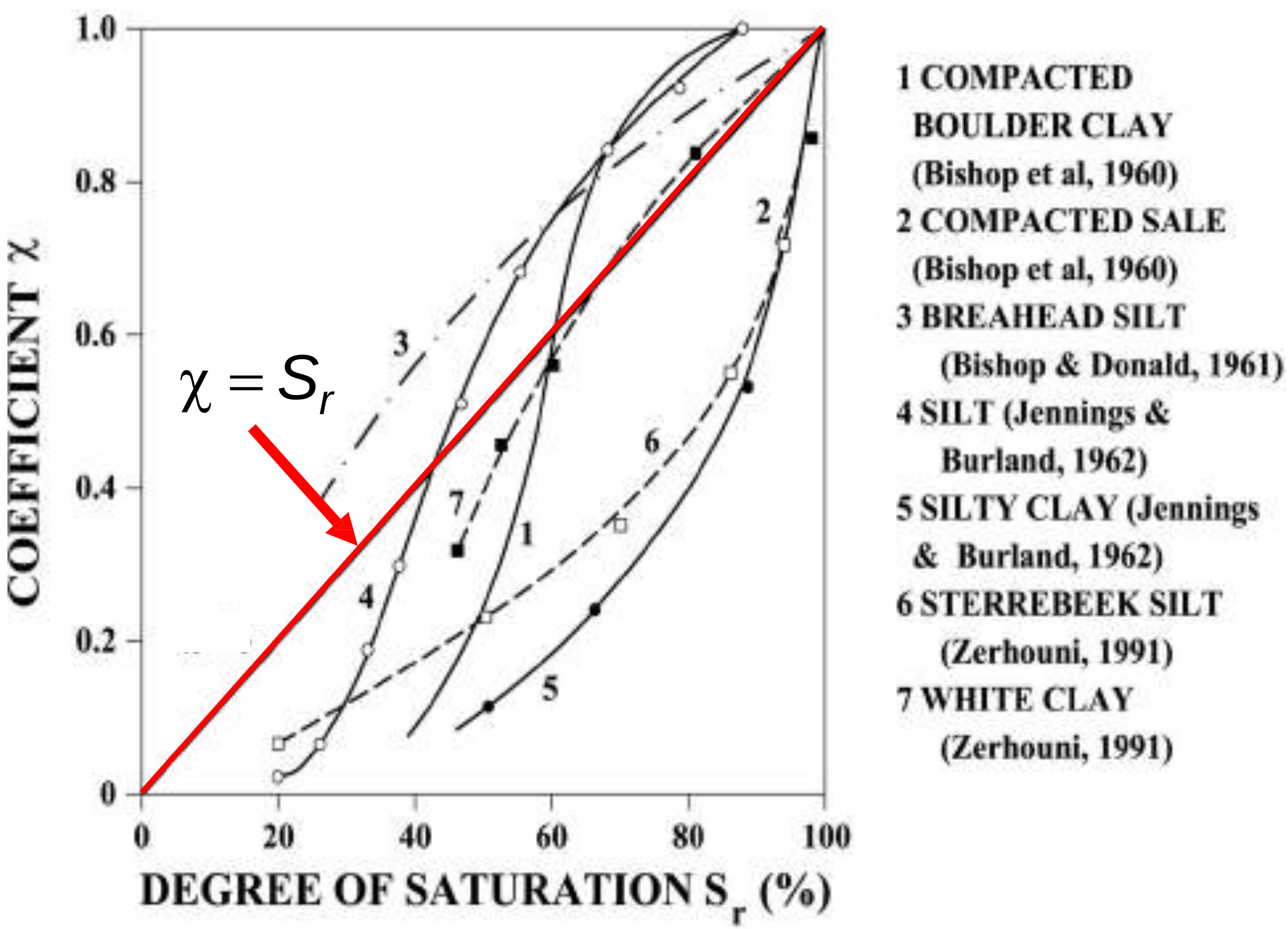
- σ' and σ = effective and total stresses
- u_a = pore air pressure
- u_w = pore water pressure
- $(u_a - u_w)$ = capillary tension
- χ = Bishop parameter

  varies from 0 (dry) to 1 (fully saturated)

Assuming that, $u_a \approx 0$ and $\chi \approx S$ the equation simplifies to:

$$\sigma' = (\sigma - S\,u_w)$$

COEFFICIENT χ



SOIL STRENGTH

$$\sigma' = (\sigma - S_r u_w)$$

Check this product!

$$\tau_f = c' + \sigma_n' \tan\varphi' = c' + (\sigma_n - S_r u_w)\tan\varphi' = c' + \sigma_n \tan\varphi' - S_r u_w \tan\varphi'$$

The variation of shear strength with suction depends on the variation of S_r with suction

- **Often S_r is replaced by S_e *(PLAXIS!)***

$$S_e = \frac{S_r - S_{res}}{S_{sat} - S_{res}}$$

S_e: effective saturation

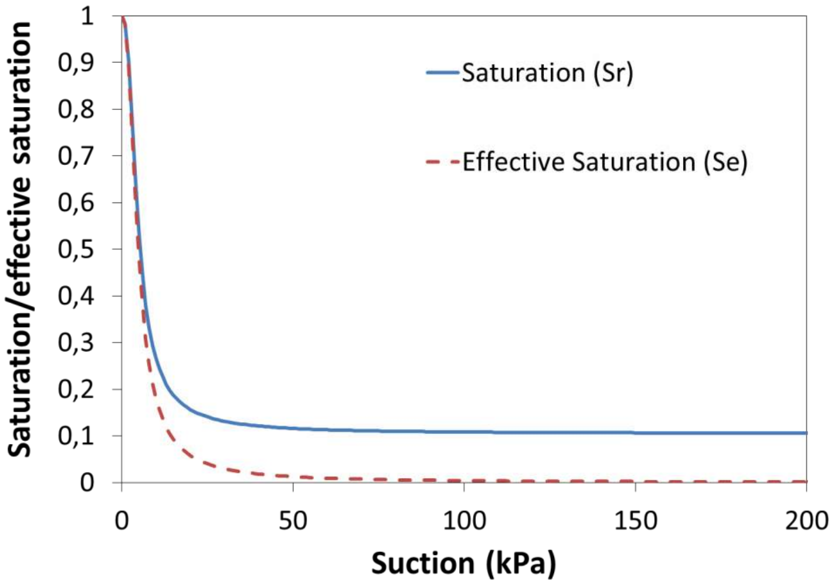
S_r: degree of saturation

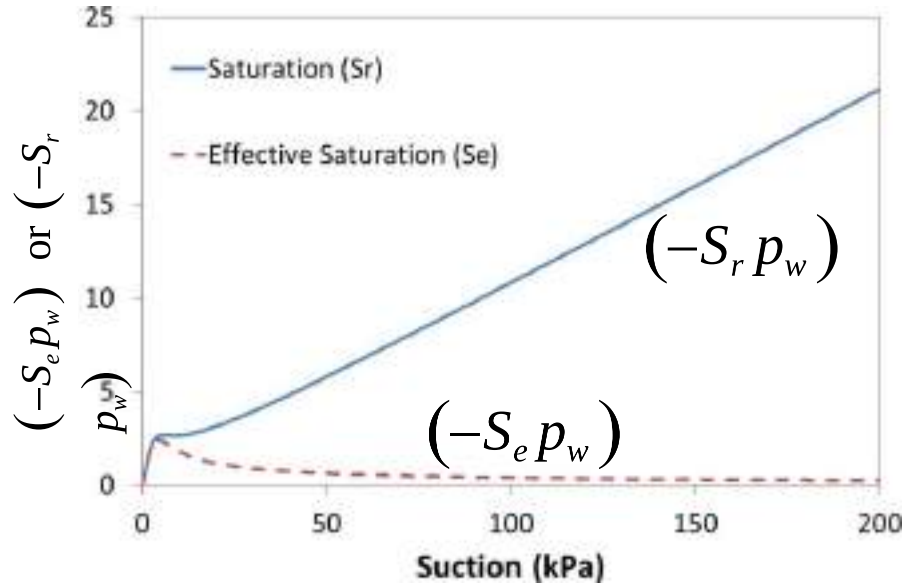
S_{res}: residual saturation (suction $\rightarrow \infty$)

S_{sat}: saturation when (suction=0)

SOIL STRENGTH

Sand (*USDA*): (S_{res}=10,5%)





DEFINITION OF QUASI-SATURATED

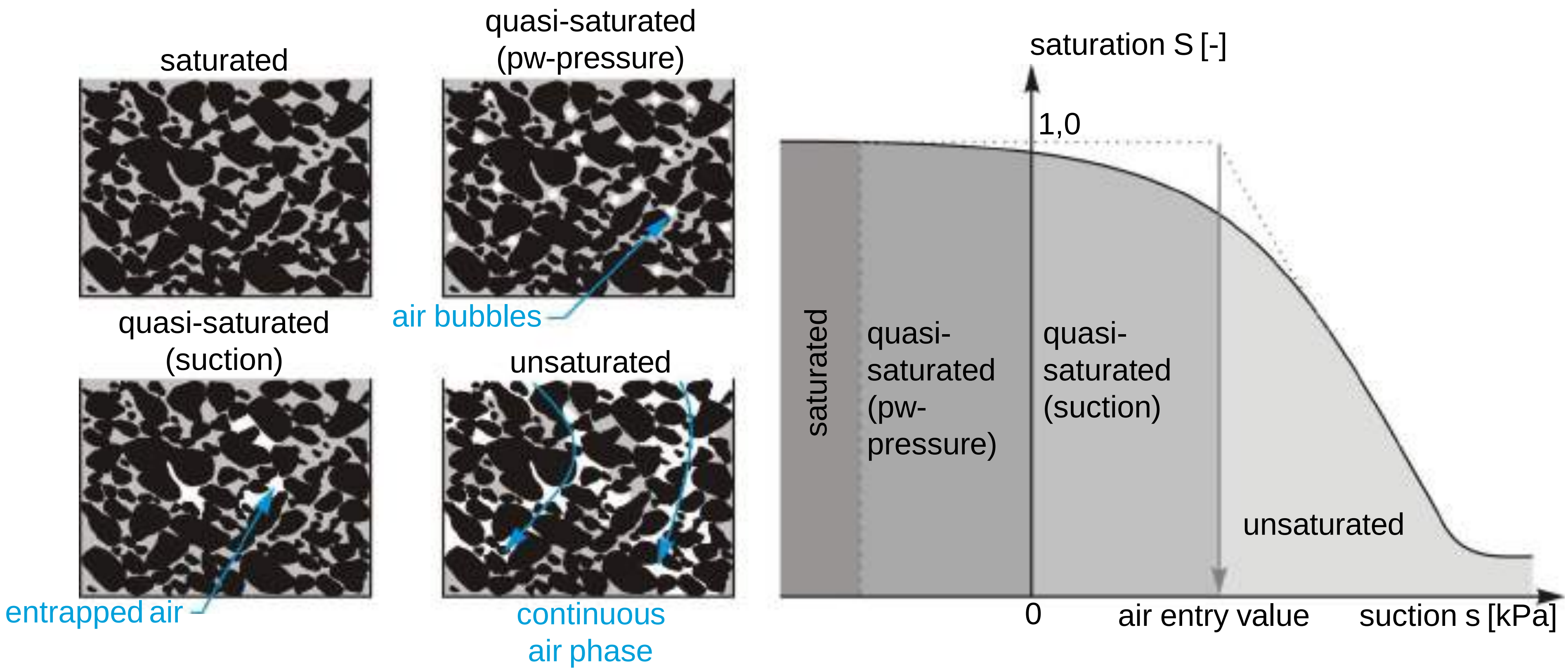
„Where air is entrapped in soils beneath the groundwater table, the soils are not fully saturated……we would like to introduce a new term, „**quasi-saturated soils**“, that will be used to define soils beneath the water table which contain entrapped air.“

(Faybishenko, B.)

DEFINITION

Stages of saturation:

- closer look to area near saturation
- distinction of quasi-saturated stage between zone with positive and negative pore water pressure

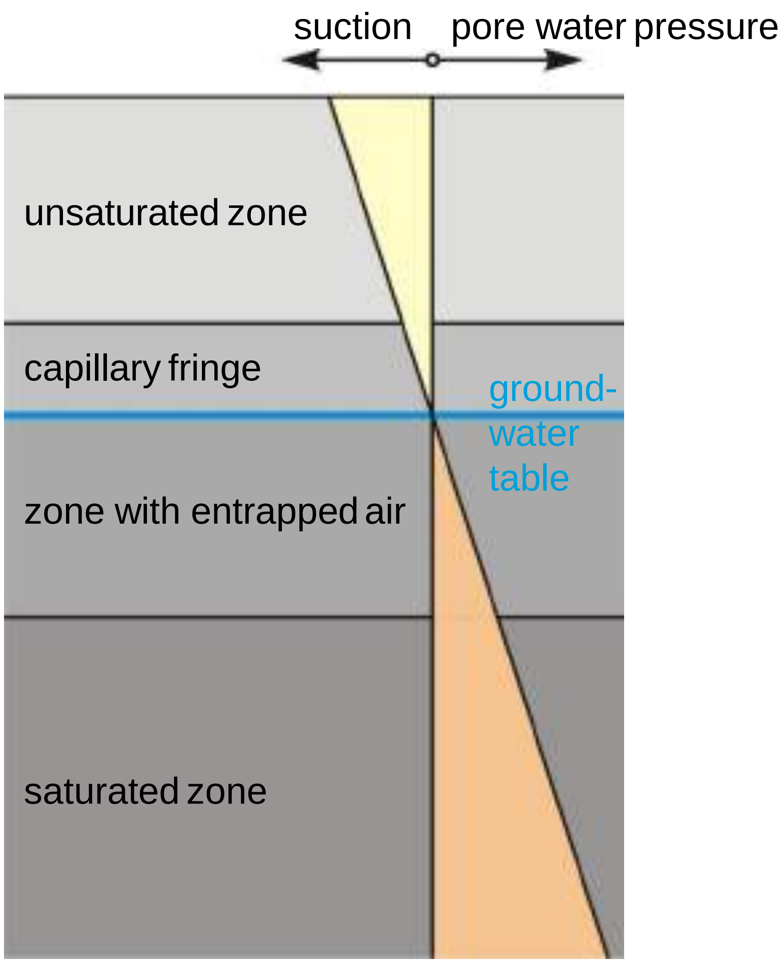


Reference: Boutonnier, 2013

DEFINITION

Regions in a soil segment:

- suction:
  - unsaturated zone (unsaturated)
  - capillary fringe (quasi-saturated – suction)
- pore water pressure:
  - zone with entrapped air (quasi-saturated – pore water pressure)
  - saturated zone (saturated)
- for fluctuating water tables above ground surface zone with entrapped air is relevant



itenas

CG17
Modelling of Slope Instability and Influence of Unsaturated Soil Behaviour

Indra Noer Hamdhan [1] and Helmut F. Schweiger [2]

1) Civil Engineering Department
National Institute of Technology (Itenas) Bandung

2) Computational Geotechnics Group
Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology

COMPUTATIONAL GEOTECHNICS GROUP

CONTENTS

Definition of safety factor obtained by FEM
(available = characteristic value)

$$\eta_{fe} = \frac{\tan\varphi_{available}}{\tan\varphi_{failure}} = \frac{c_{available}}{c_{failure}}$$

Basically 2 possibilities to obtain factor of safety:

1: Calculation with characteristic parameters > automatic reduction of strength parameters of soil until equilibrium is no longer achieved in numerical analysis

Some FE-codes do this automatically > strength-reduction technique

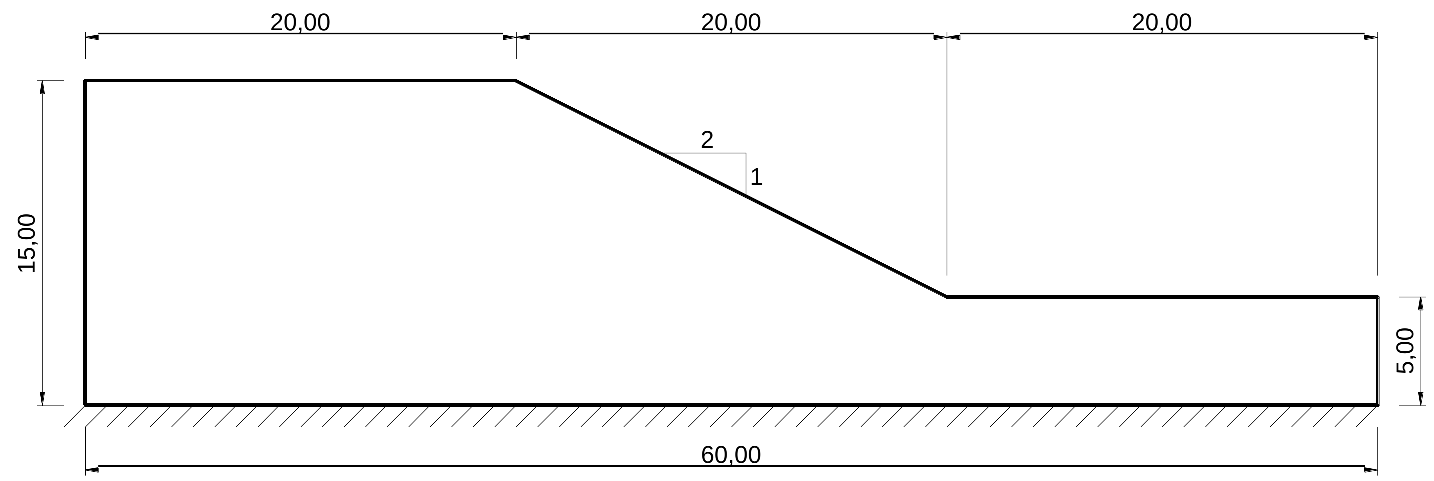
2: Calculation with reduced parameters > perform *new* calculation with different factors until equilibrium is no longer achieved in numerical analysis

Some codes do this automatically

see also: e.g. Griffiths (1980), Naylor (1981), Brinkgreve & Bakker (1991), Matsui & San (1992)

Homogeneous, drained soil layer

Inclination of slope: 2:1



Soil parameters (Mohr-Coulomb failure criterion):

$\varphi' = 20.0$ [°] , $c' = 10.0$ [kN/m²]

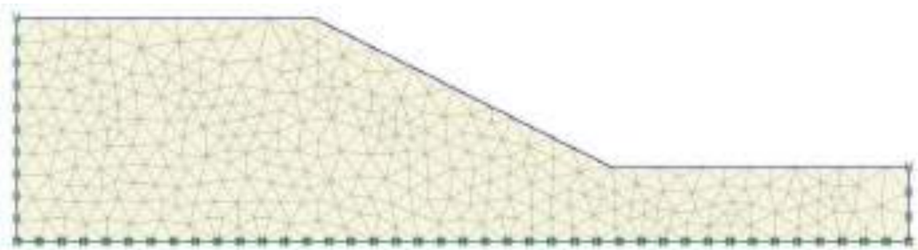
$E = 10^5$ [kN/m²] , $\nu = 0.3$ [-] , $\gamma = 20.0$ [kN/m³]

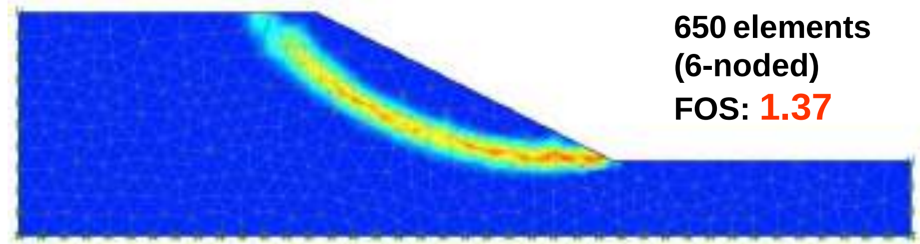
Example from: Griffiths and Lane, Slope stability analysis by finite elements, Geotechnique 49, 387-403, 1999

Check of Mesh Dependency:

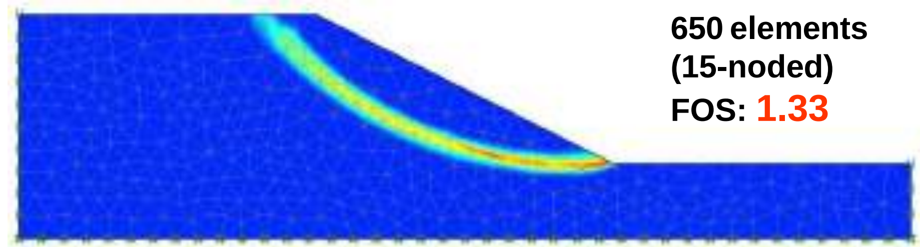
Incremental shear strains after φ/c-reduction

Coarse Mesh (650 elements):



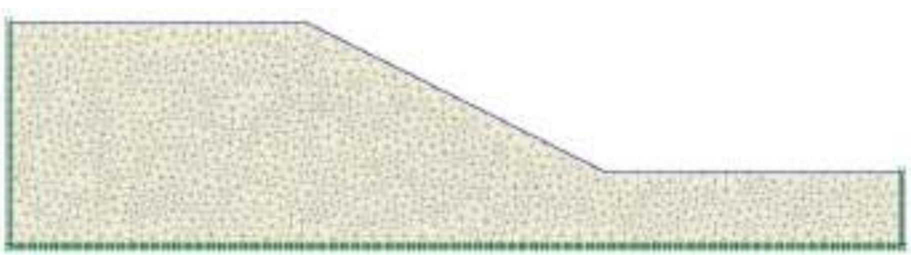


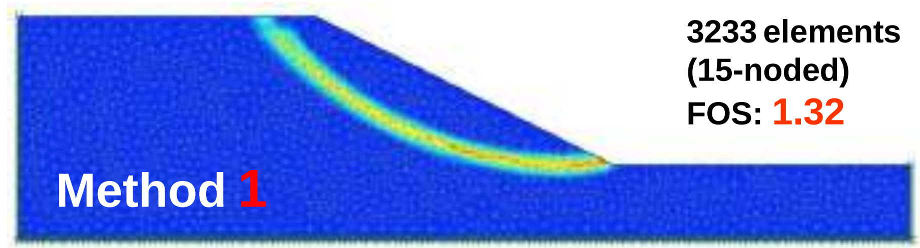
650 elements
(6-noded)
FOS: 1.37



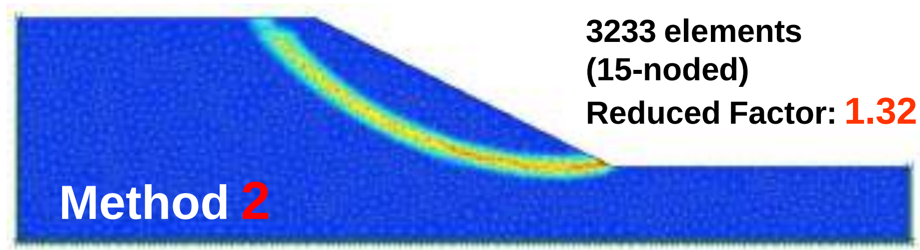
650 elements
(15-noded)
FOS: 1.33

Fine Mesh (3233 elements):





3233 elements
(15-noded)
FOS: 1.32



3233 elements
(15-noded)
Reduced Factor: 1.32

No difference in results for Method 1 and 2

Advantage of FEM vs. LEM for Slope Stability Analysis

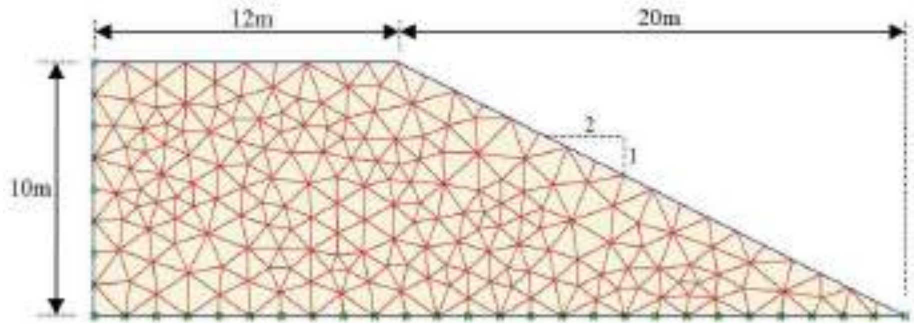
1. In the **FEM**, failure occurs naturally through the zones within the soil mass wherein the shear strength of the soil is not capable to resist the applied shear stress, so there is **no need to make assumption about the shape or location of the failure surface**.
2. There is no theory of slices in the **FEM**, so **no need to make assumption about slide side forces**. The FEM maintains overall equilibrium until failure is reached.
3. As long as the compressibility data of soils is available, the **FEM** will **provide deformations** result at the working stress levels.
4. The **FEM** is capable to check **the progressive failure** up to and including shear failure.

Source: Griffiths and Lane, Slope stability analysis by finite elements, Geotechnique 49, 387-403, 1999

(1) Homogeneous slope with no foundation layer

FEM

Geometry and mesh:



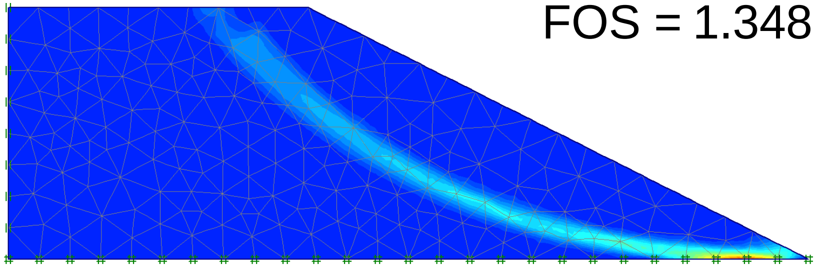
Soil parameters with Mohr-Coulomb model:

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (effective shear strength)	c'	[kPa]	10
Friction angle (effective shear strength)	ϕ'	[^{o}]	20

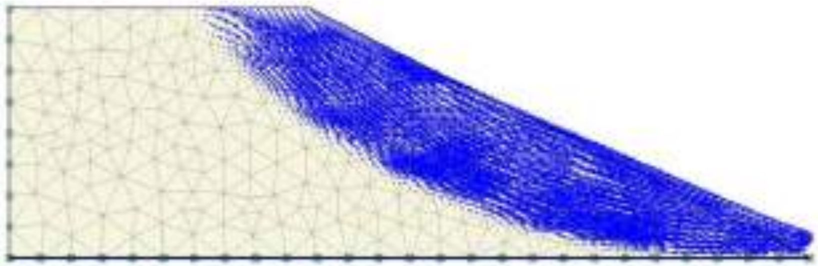
Failure mechanism:

Incremental strains:

FOS = 1.348



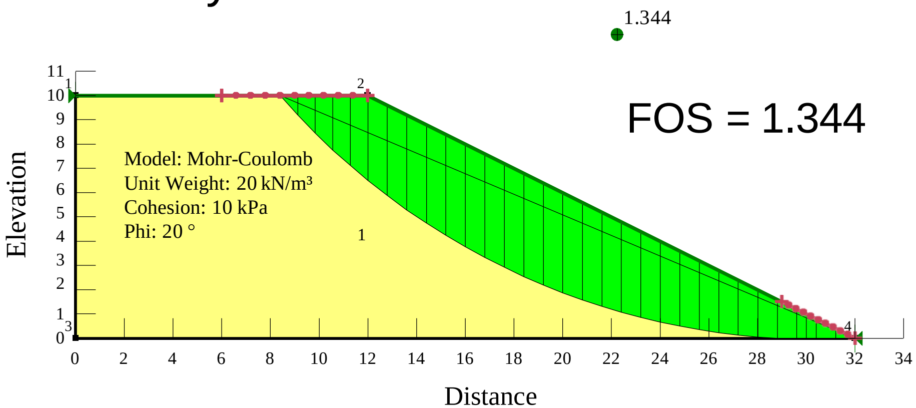
Incremental displacement:



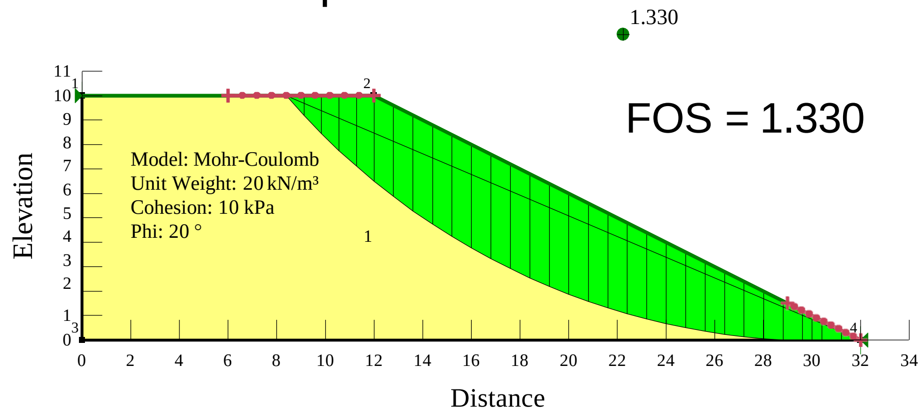
(1) Homogeneous slope with no foundation layer

LEM

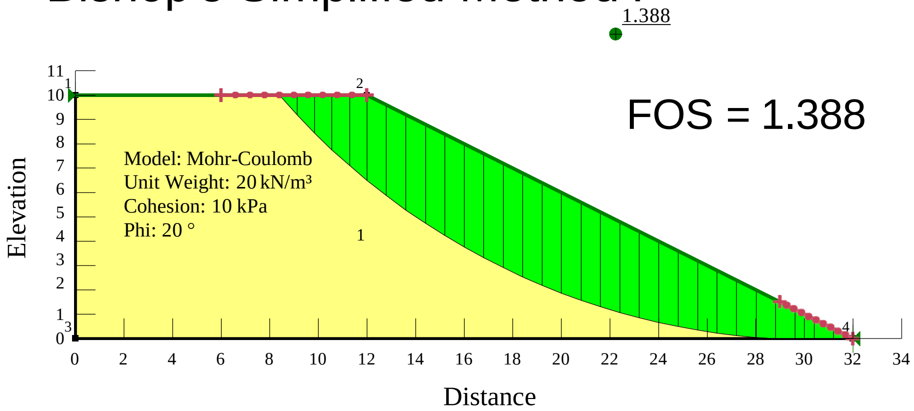
Ordinary Method of Slice:



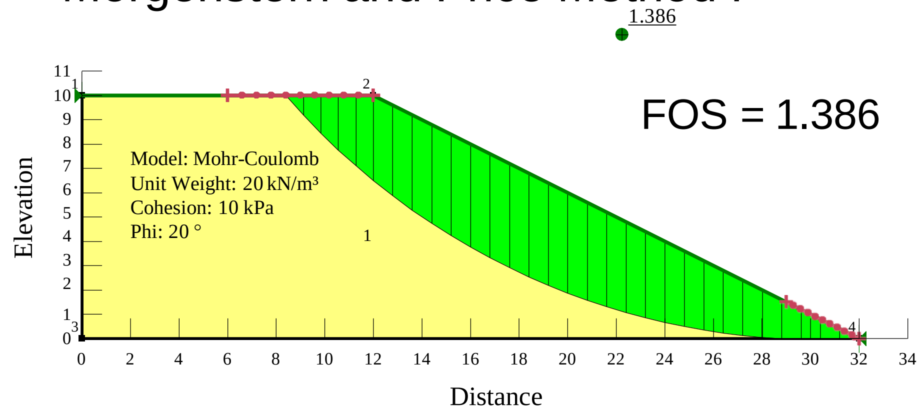
Janbu’s Simplified Method :



Bishop’s Simplified Method :



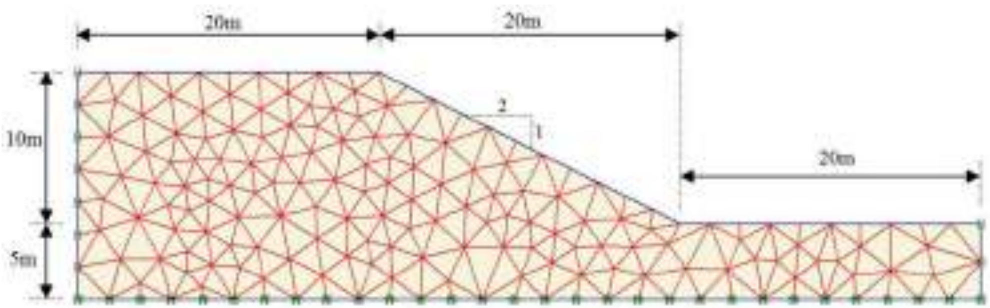
Morgenstern and Price Method :



(2) Homogeneous slope with foundation layer

FEM

Geometry and mesh:

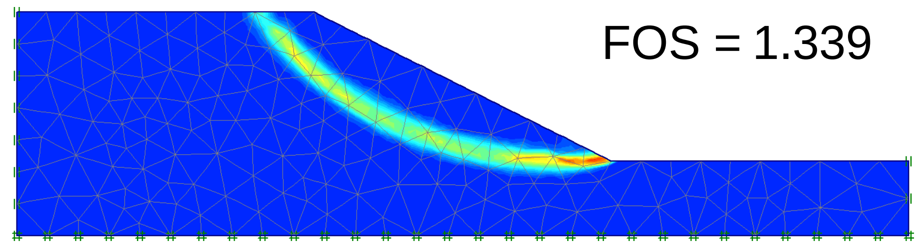


Soil parameters with Mohr-Coulomb model:

Description	Symbol	Unit	Value
Unit weight	γ	[kN/m^3]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (effective shear strength)	c'	[kPa]	10
Friction angle (effective shear strength)	ϕ'	[º]	20

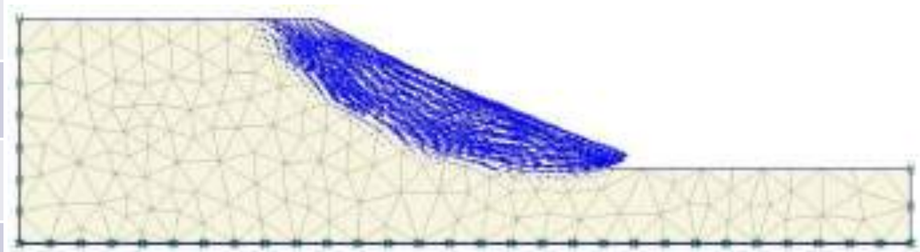
Failure mechanism:

Incremental strains:



FOS = 1.339

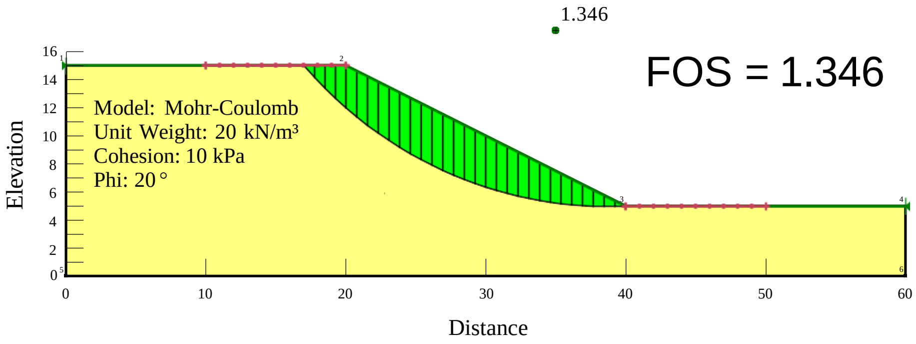
Incremental displacement:



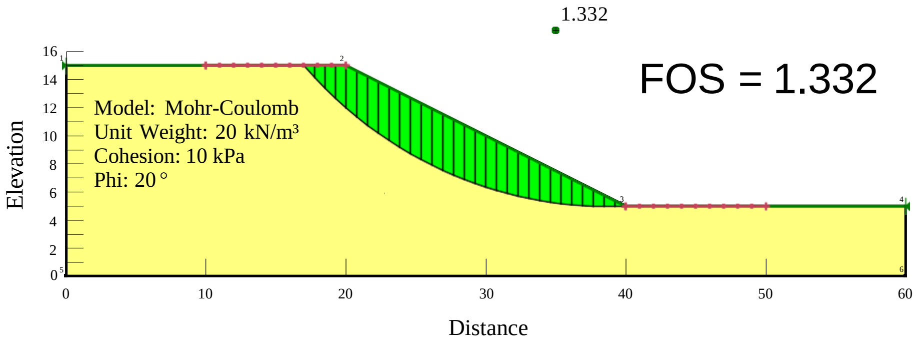
(2) Homogeneous slope with foundation layer

LEM

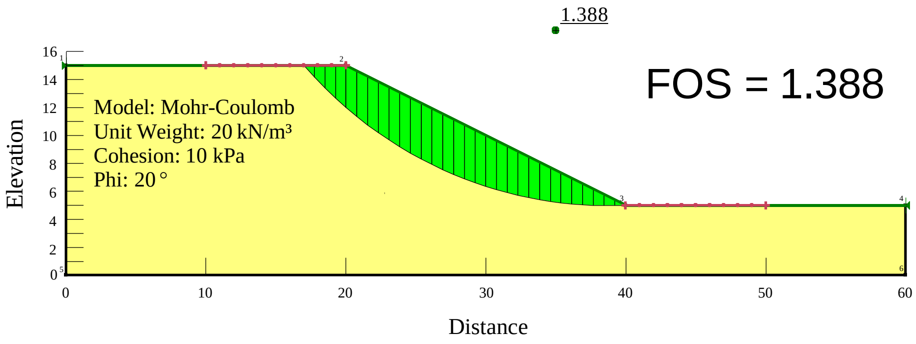
Ordinary Method of Slice:



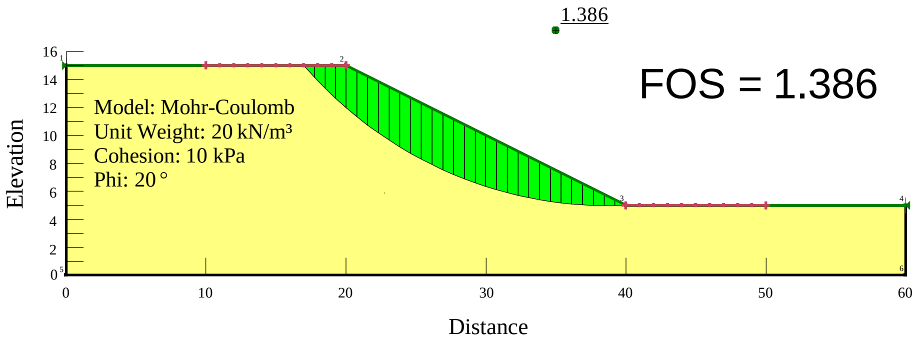
Janbu’s Simplified Method :



Bishop’s Simplified Method :



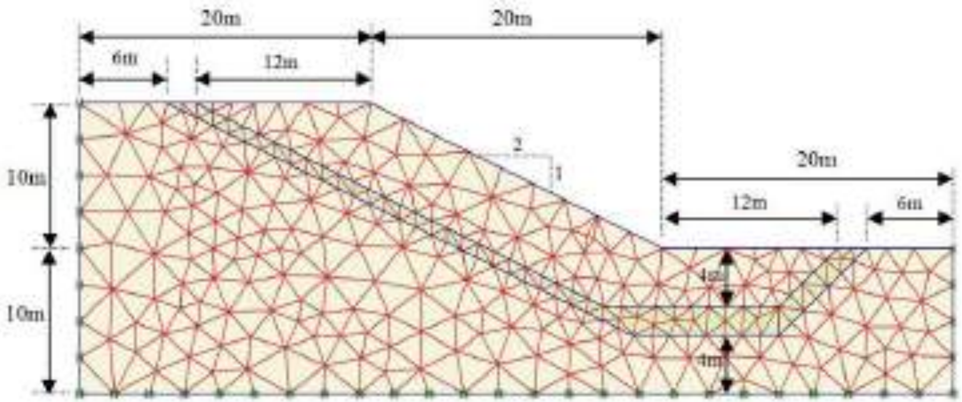
Morgenstern and Price Method :



(3) An Undrained Clay Slope with a Thin Weak Layer

FEM

Geometry and mesh:



Soil parameters with Mohr-Coulomb model:

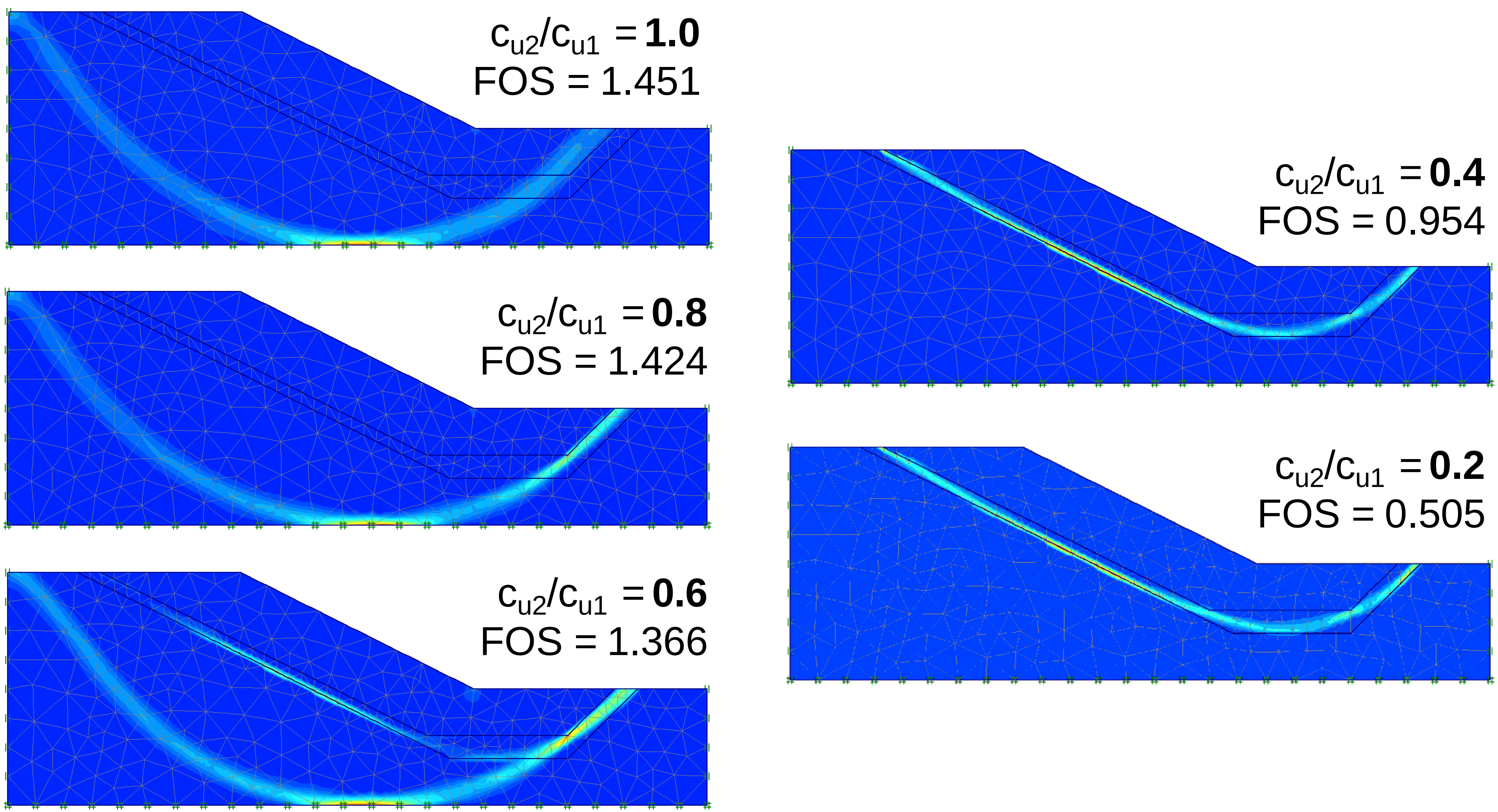
Description	Symbol	Unit	Value
Unit weight	γ	[kN/m³]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (undrained shear strength)	c_{u1}	[kPa]	50
Friction angle (undrained shear strength)	ϕ_u	[º]	0

The analysis are carried out using a constant value of undrained shear strength of soil (cu_1) and five different values of undrained shear strength of the thin layer (cu_2) with ratio cu_2/cu_1 equal to **1**, **0.8**, **0.6**, **0.4**, and **0.2**.

(3) An Undrained Clay Slope with a Thin Weak Layer

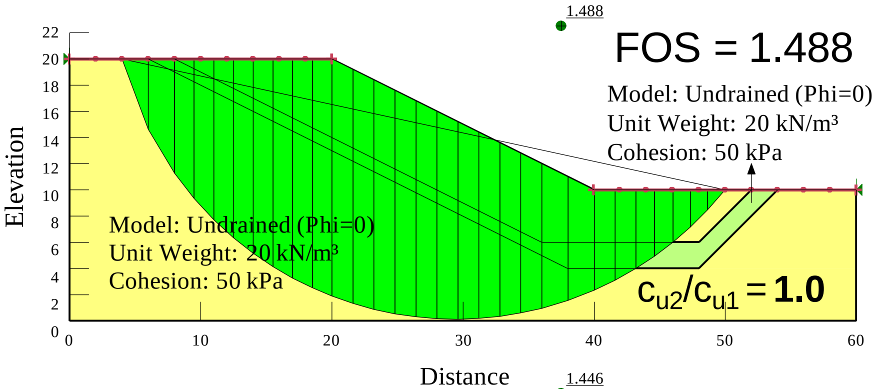
FEM

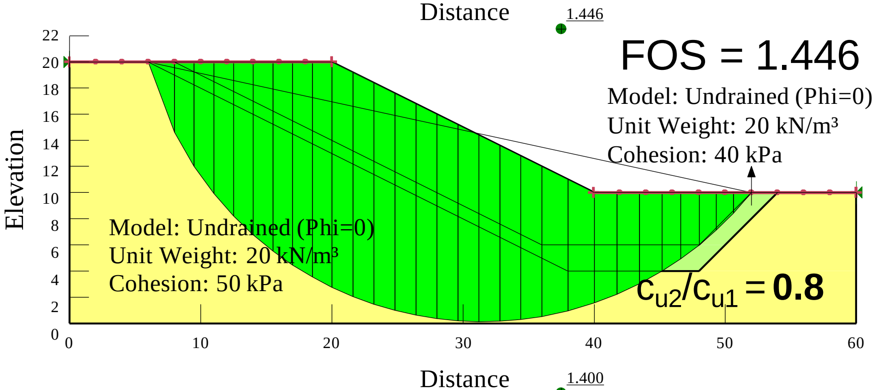
Failure mechanism (incremental strains):

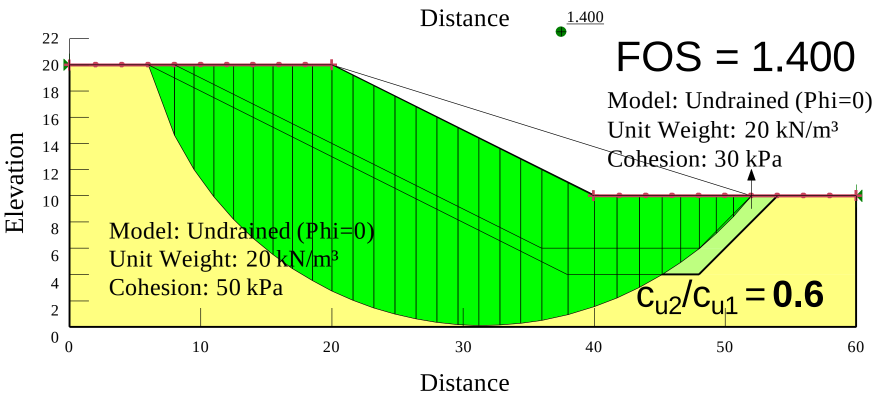


(3) An Undrained Clay Slope with a Thin Weak Layer

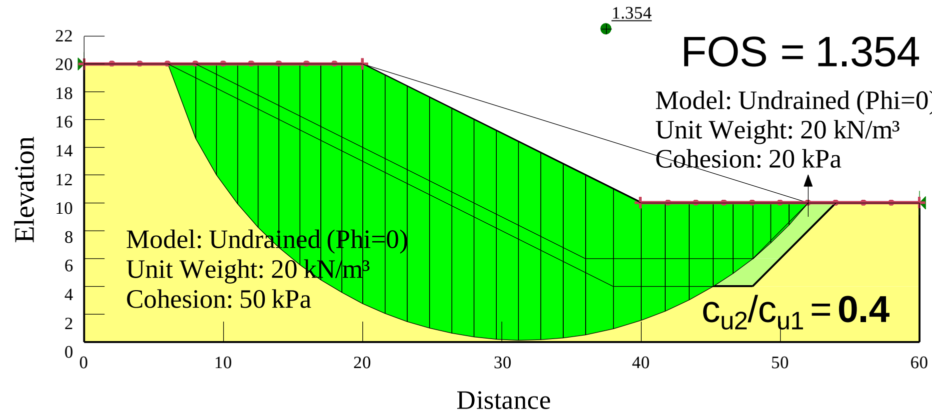
LEM

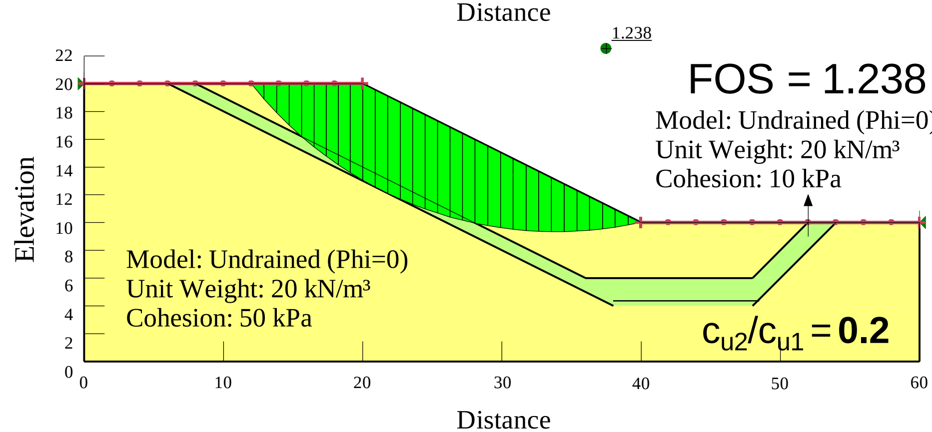






Morgenstern and Price Method :

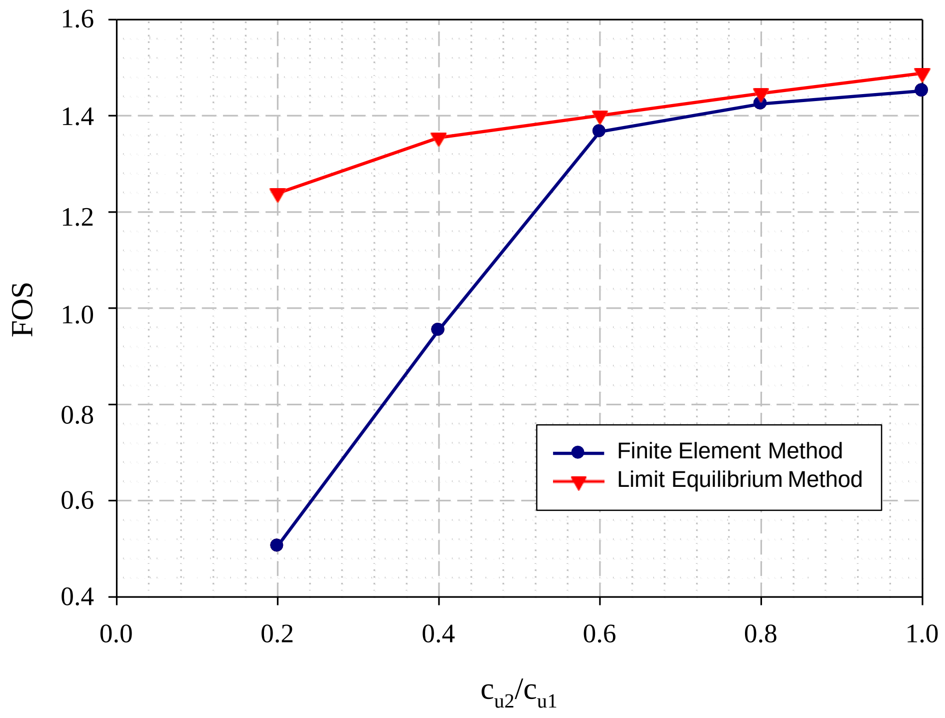




(3) An Undrained Clay Slope with a Thin Weak Layer

Computed FOS for an undrained clay slope with a thin weak layer with variations of c_{u2}/c_{u1}:

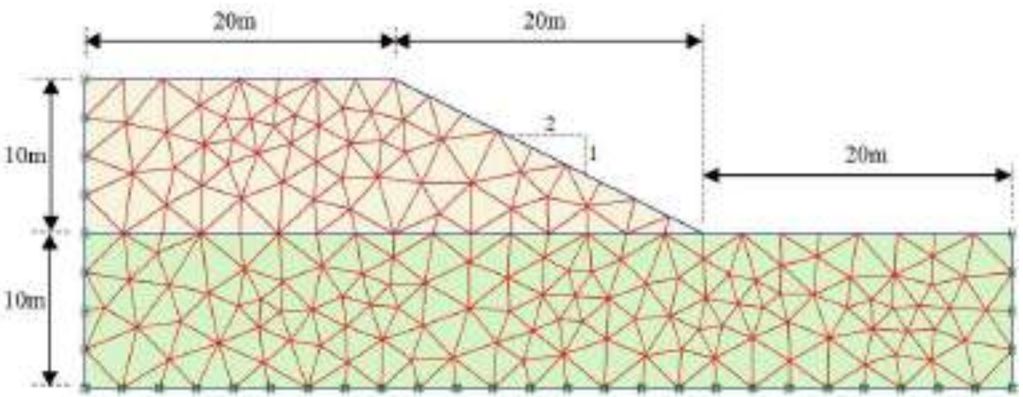
FEM vs. LEM



(4) An Undrained Clay Slope with a Weak Foundation Layer

FEM

Geometry and mesh:



Soil parameters with Mohr-Coulomb model:

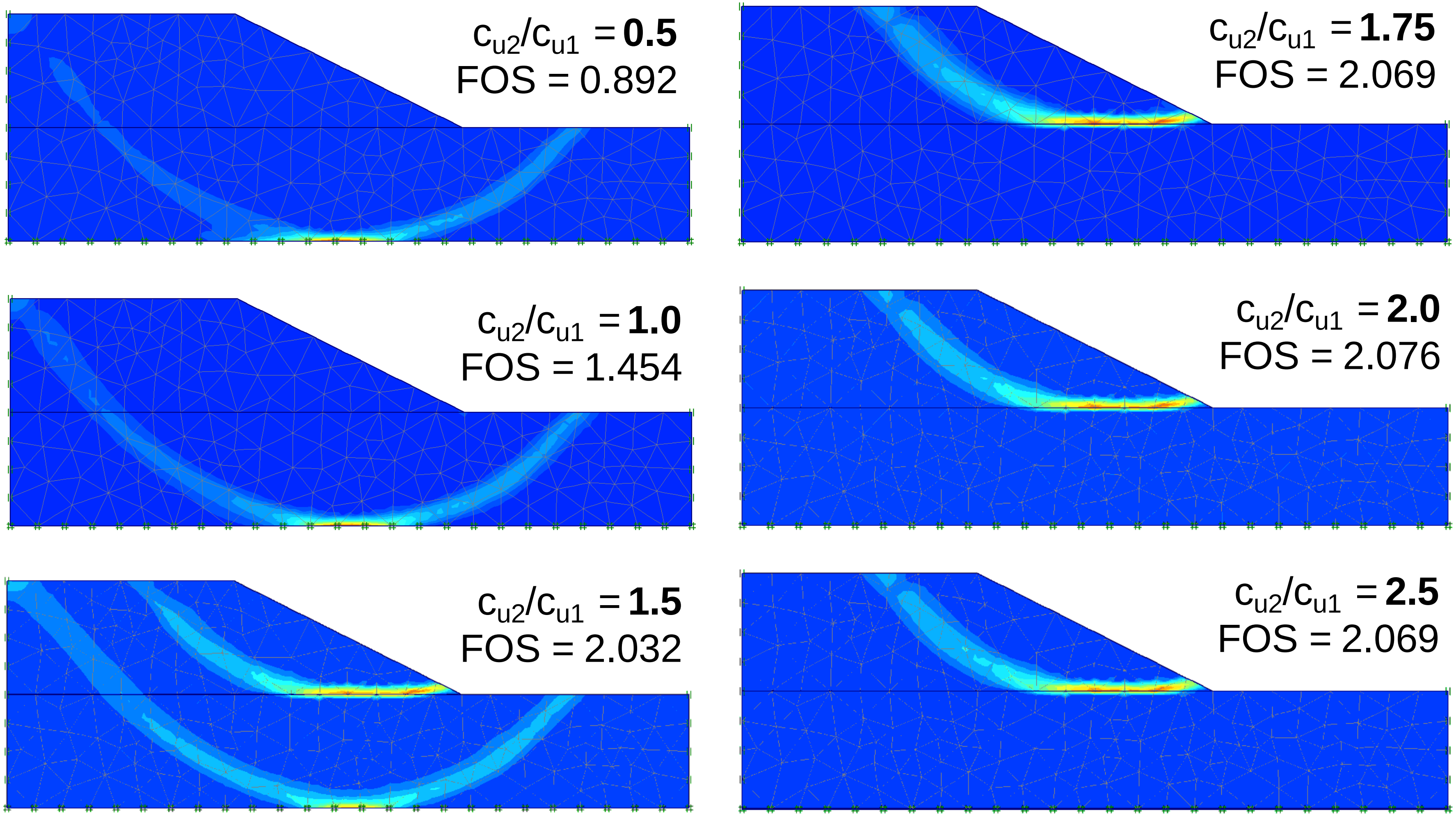
Description	Symbol	Unit	Value
Unit weight	γ	[kN/m³]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (undrained shear strength)	c_{u1}	[kPa]	50
Friction angle (undrained shear strength)	ϕ_u	[º]	0

The analysis are carried out using a constant value of undrained shear strength of soil (cu_1) and six different values of undrained shear strength of the foundation layer (cu_2) with ratio cu_2/cu_1 equal to **0.5**, **1.0**, **1.5**, **1.75**, **2.0** and **2.5**.

(4) An Undrained Clay Slope with a Weak Foundation Layer

FEM

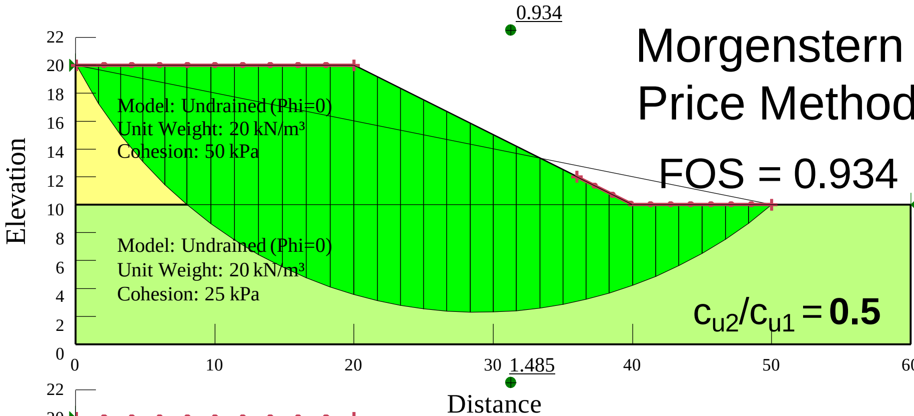
Failure mechanism (incremental strains):

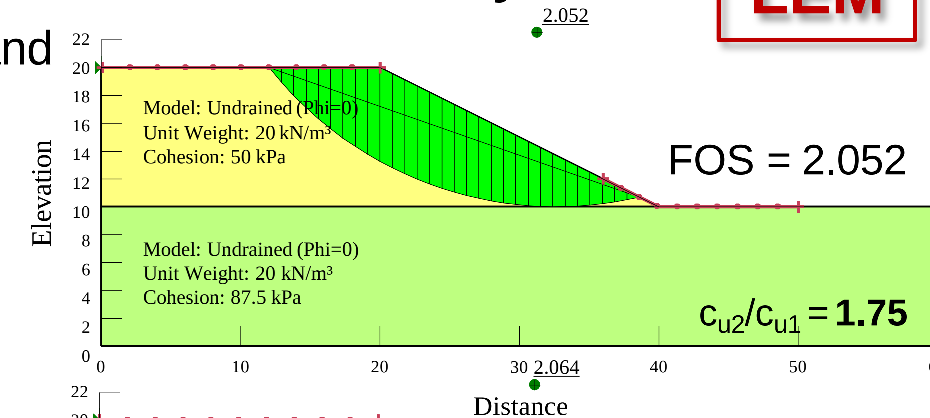


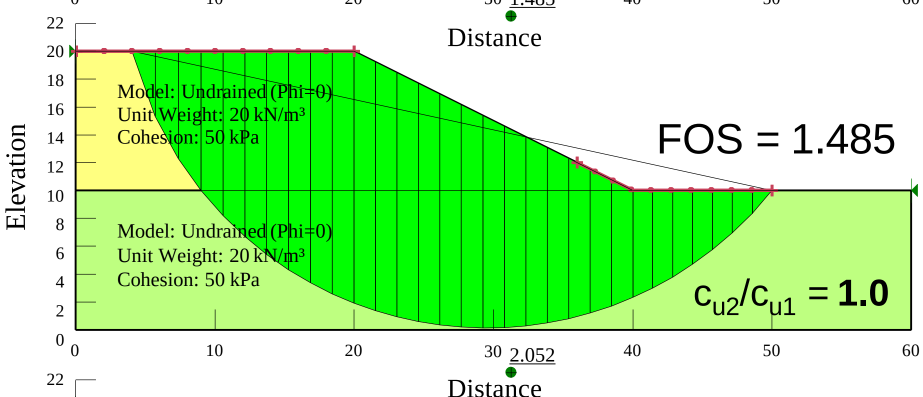
(4) An Undrained Clay Slope with a Weak Foundation Layer

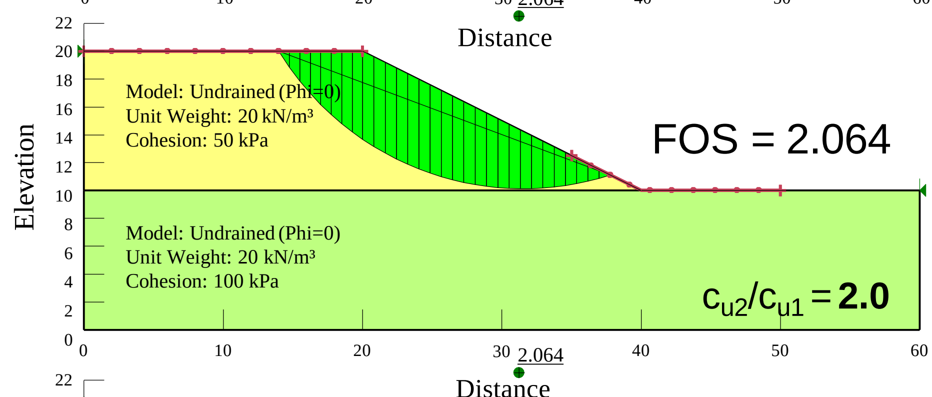
LEM

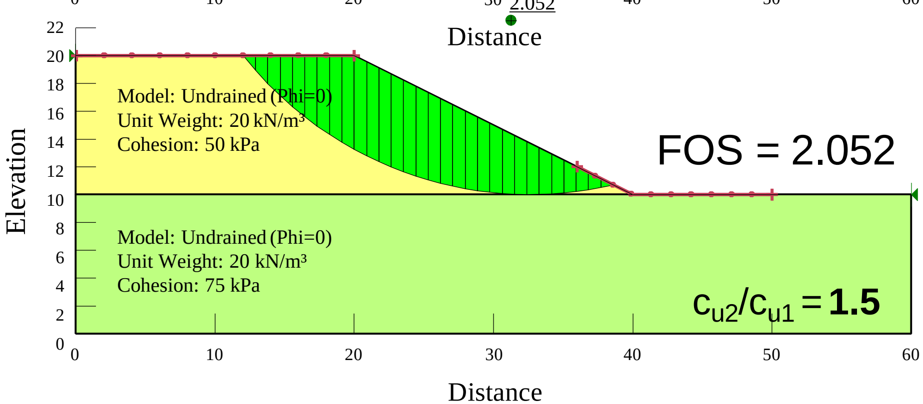
Morgenstern and Price Method :

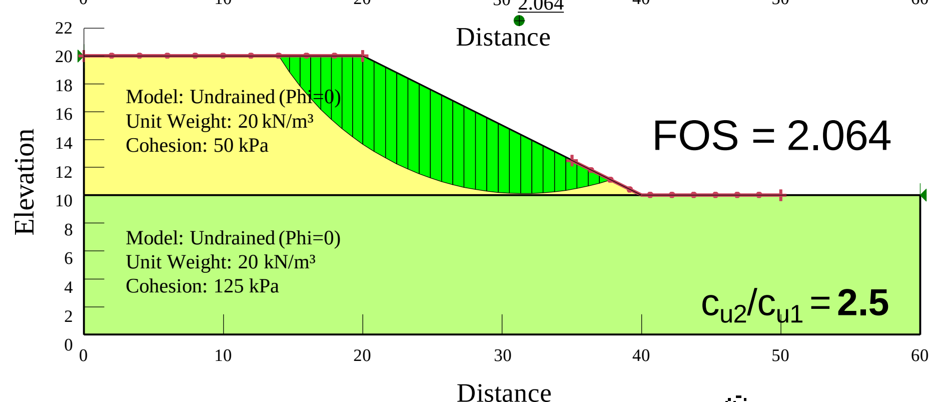








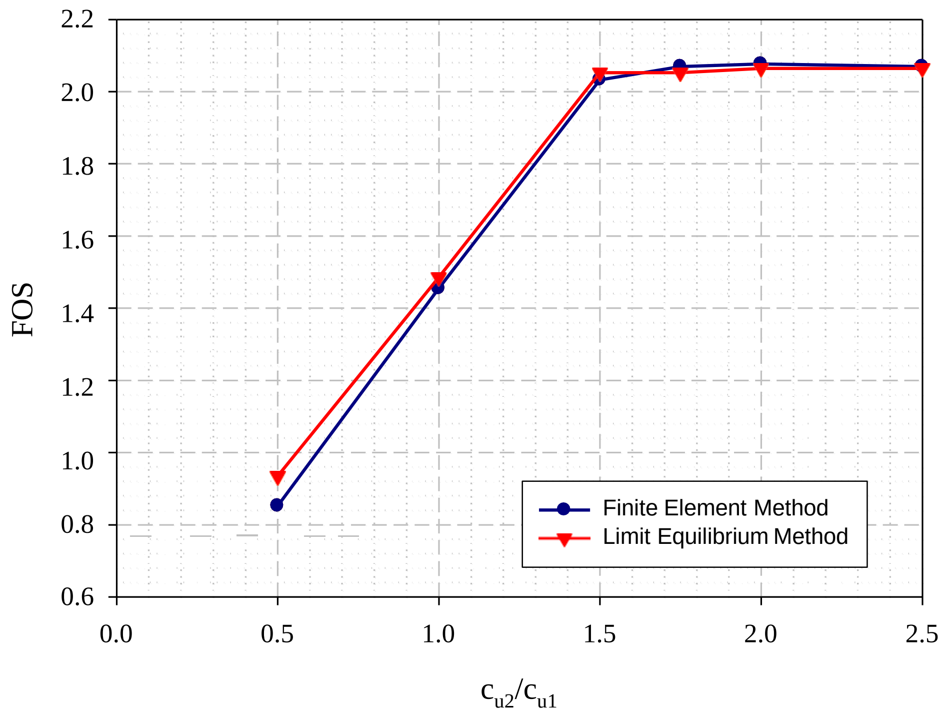




(4) An Undrained Clay Slope with a Weak Foundation Layer

Computed FOS for an undrained clay slope with a weak foundation layer with variations of c_{u2}/c_{u1}:

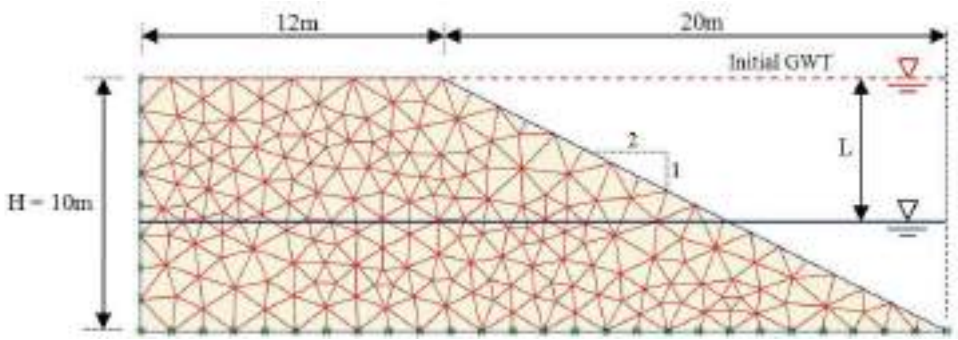
FEM vs. LEM



(5) Homogeneous Slope with Horizontal Free-Surface

FEM

Geometry and mesh:



Soil parameters with Mohr-Coulomb model:

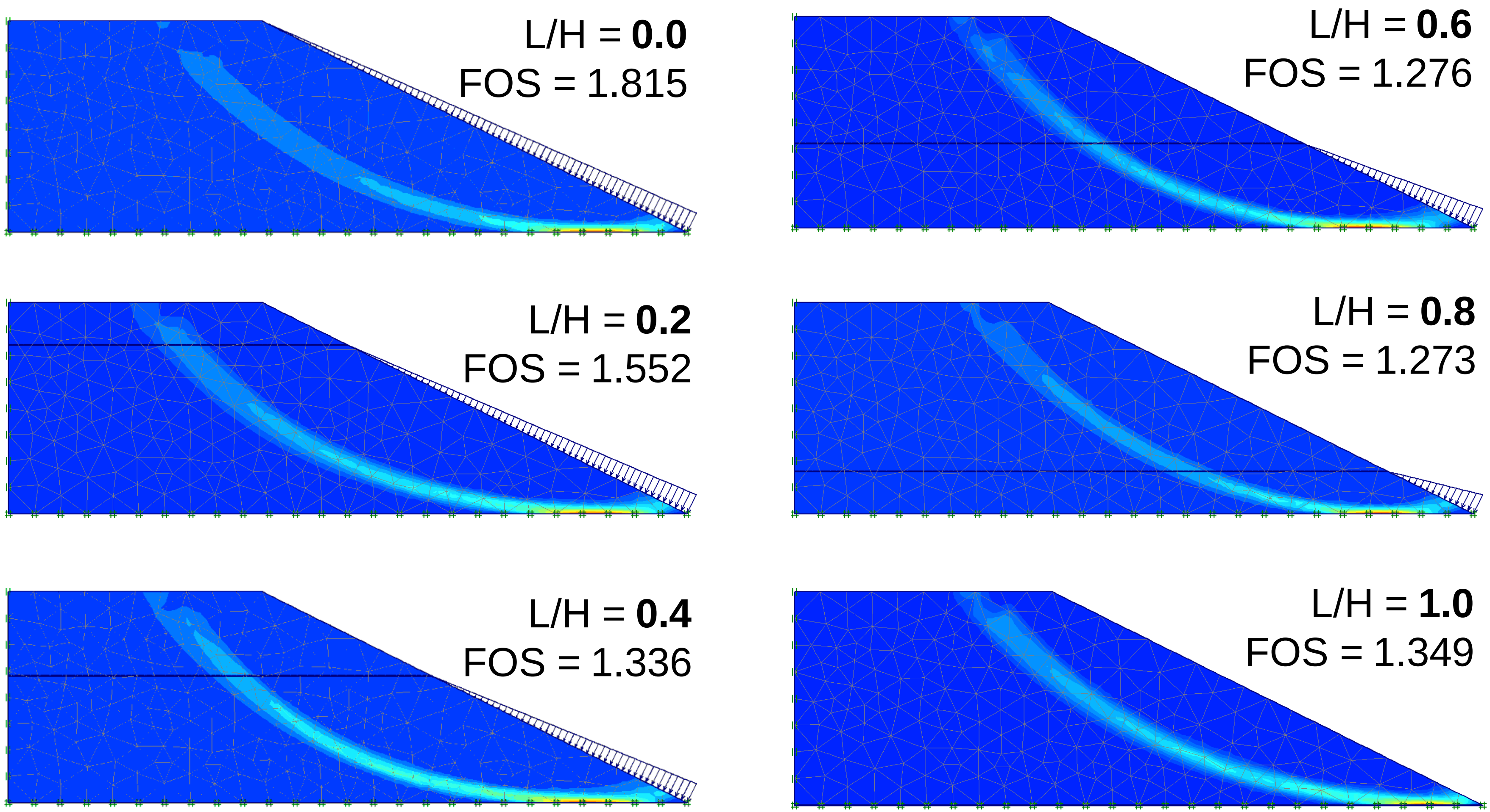
Description	Symbol	Unit	Value
Unit weight	γ	[kN/m³]	20
Effective secant modulus	E'	[kPa]	100.000
Effective poisson's ratio	ν'	[-]	0.3
Cohesion (effective shear strength)	c'	[kPa]	10
Friction angle (effective shear strength)	ϕ'	[º]	20

In this analysis, a slope with different drawdown ratio, L/H which has been varied from 0.0 (slope completely submerged with water level at the crest of the slope) to 1.0 (water level at the toe of the slope) were considered

(5) Homogeneous Slope with Horizontal Free-Surface

FEM

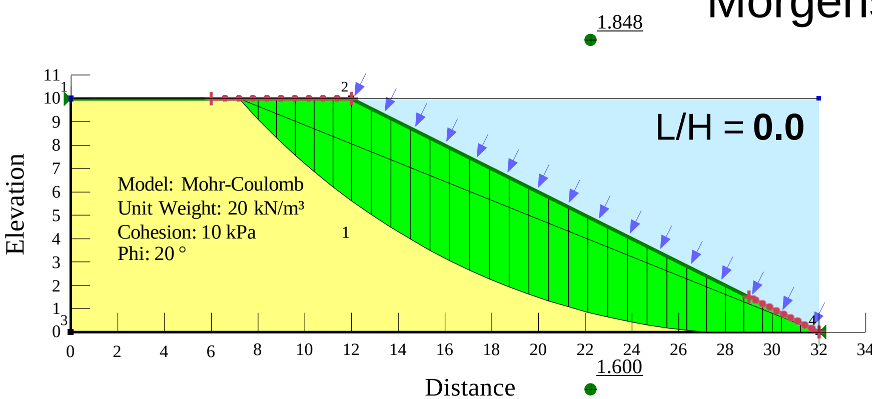
Failure mechanism (incremental strains):

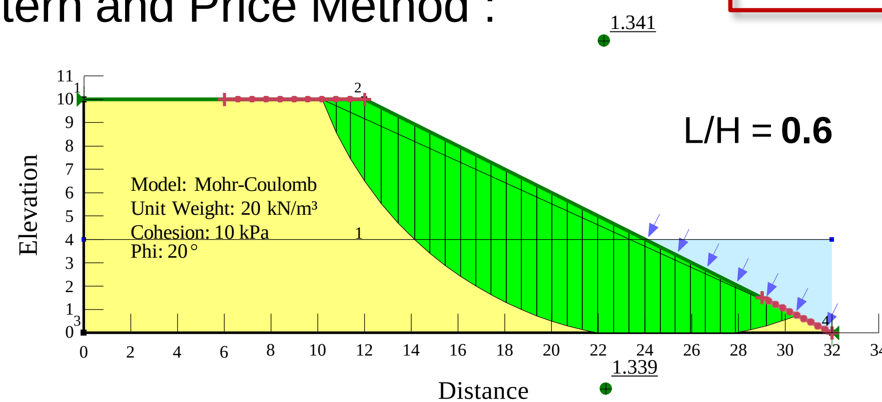


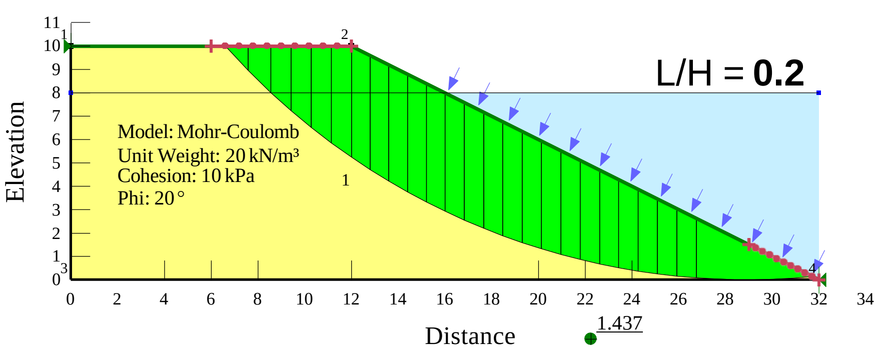
(5) Homogeneous Slope with Horizontal Free-Surface

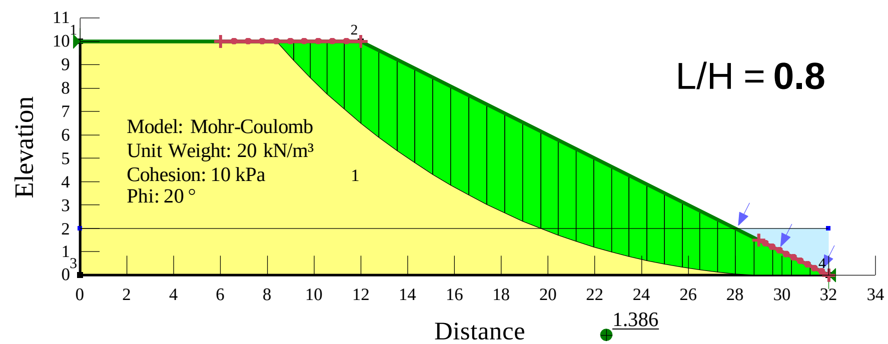
LEM

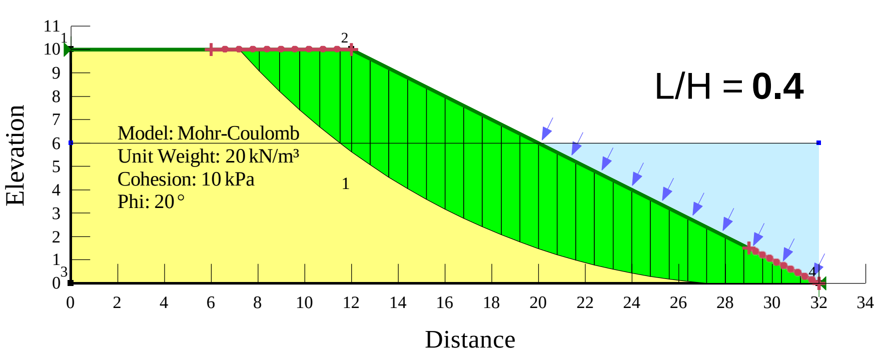
Morgenstern and Price Method :

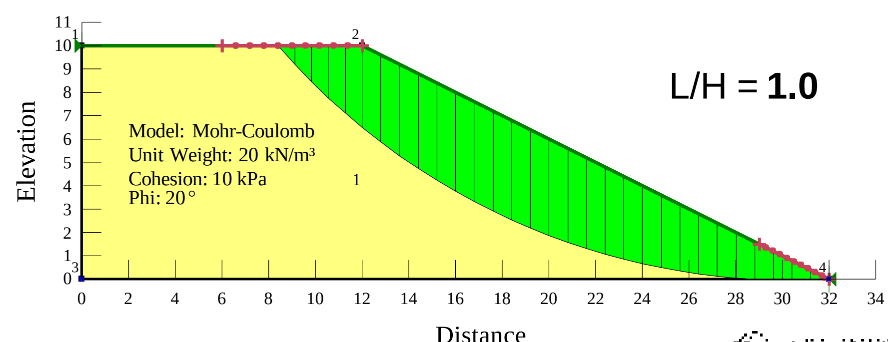








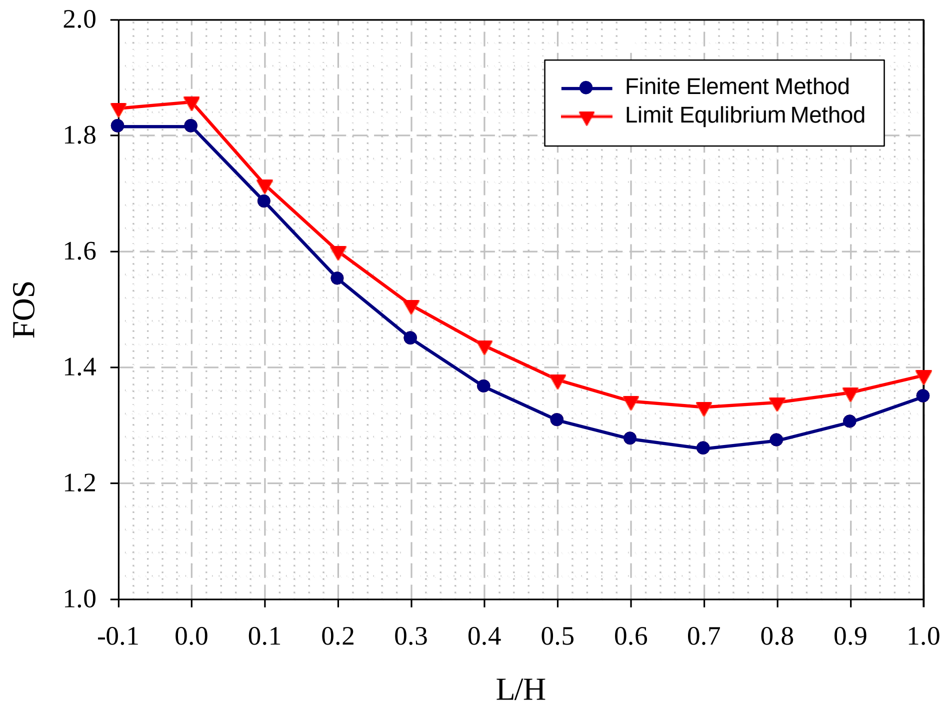




(5) Homogeneous Slope with Horizontal Free-Surface

Computed FOS for homogeneous slope with horizontal free-surface with variations of L/H:

FEM vs. LEM



Unsaturated soil slope subjected to rainfall infiltration

In slope stability analysis, the effect of negative pore water pressure or suction is usually not taken into account because suction will reduce with rainfall infiltration and therefore it can be assumed that matric suction does not influence the long term stability of the slope.

However, to reduce matric suction from the soil, the rainfall needs to be sustained over a significant time period and also the rainfall intensity needs to approximate the saturated coefficient of permeability of the soil at the ground surface.

Hydraulic characteristics such as saturated coefficient of permeability and initial degree of saturation, intensity and duration of rainfall are parameters which are important in the analysis of slope stability considering rain infiltration.

According to Biot's theory of consolidation, to analyze the behaviour of unsaturated soils, is required to simultaneously compute deformation and groundwater flow with time dependent boundary conditions (fully coupled flow deformation analysis).

Hydraulic Models

Van Genuchten (1980) presented a set of closed-form equations of hydraulic characteristics for unsaturated soils which is based on the capillary model of Mualem (1976). The Van Genuchten model introduced the relation between saturation and suction pore pressure head (ϕ_p):

$$S \quad \phi_p \; = S_{residu} + \; S_{sat} - S_{residu} \quad 1 + \; g_a \; \phi_p^{\quad g_n \;\; g_c}$$

$$\phi_p = -\frac{u_w}{\rho_w g}$$

where S_{residu} is the residual degree saturation of the soil that describes the part of water that remains in the soil even at high suction heads. S_{sat} is the degree saturation of the soil when the pores are filled with water. g_a, g_n and g_c are empirical parameters, and it is assumed that:

$$g_c = \frac{1-g_n}{g_n}$$

Hydraulic Models

The Van Genuchten Model is used in which the effective degree of saturation (S_e) is obtained as:

$$S_e = \frac{S - S_{residu}}{S_{sat} - S_{residu}}$$

The relative permeability in relation to Mualem – Van Genuchten is:

$$k_{rel}S = S_e^{g_l}\left[1-\left(1-S_e^{\frac{g_n}{g_n-1}}\right)^{\frac{g_n-1}{g_n}}\right]^2$$

where g_l is an empirical parameter.

Shear Strength of Unsaturated Soil

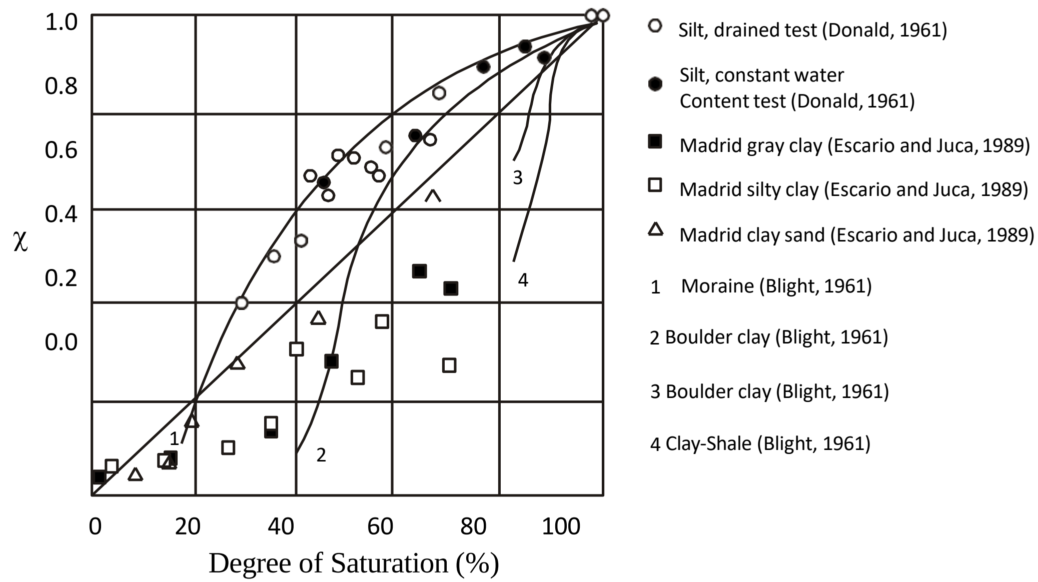
The principle of effective stress is applicable for saturated soils. For unsaturated soils, the water phase fills only parts of the pore volume, whereas the remainder is occupied by air. Bishop (1959) has modified Terzaghi's classical effective stress theory and presented the matric suction coefficient (χ) for the effective stress of unsaturated soils:

$$\sigma' = \sigma - u_a + \chi\, u_a - u_w$$

where σ' and σ are the effective and total stress respectively, u_a is the pore air pressure, and u_w is pore water pressure. The term ($u_a - u_w$) is called matric suction and χ is the matric suction coefficient and varies from 0 to 1 covering the range from dry to fully saturated conditions.

By assuming that the pore air pressure is constant and is small enough to be neglected ($u_a \approx 0$), consequently for a dry soil, effective stress and total stress are the same. The matric suction coefficient (χ) is usually obtained from laboratory tests on both saturated and unsaturated samples.

Oberg and Sallfors (1997) and Vanapalli et al. (1996) suggested that the factor χ can approximately be replaced by the degree of saturation or the effective degree of saturation, because the shear strength of unsaturated soils is strongly related to the amount of water in voids of soils and in turn to the matric suction.



Consequently, the effective stress equation can be simplified to:

$$\sigma' = \sigma - S\, u_w$$

where *S* is the degree saturation of the soil.

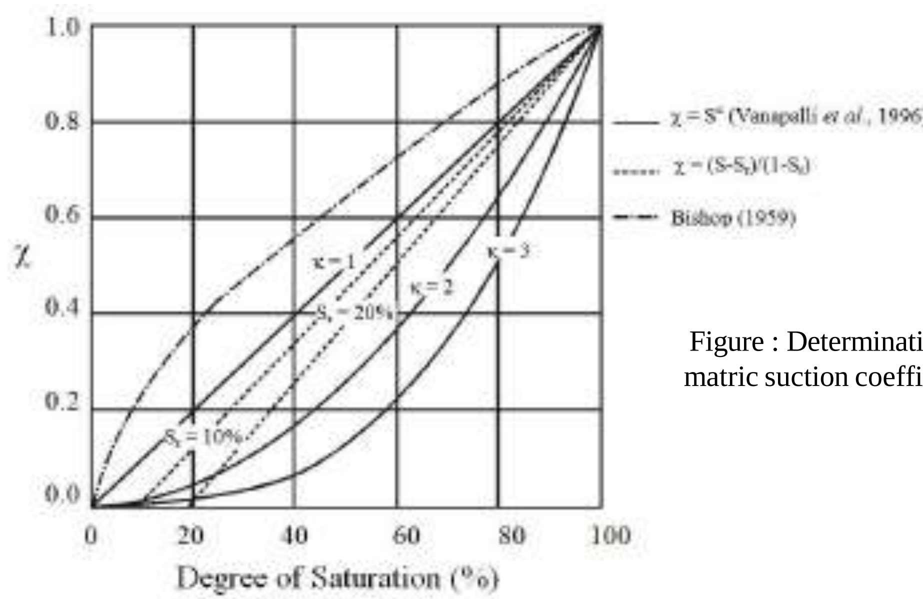


Figure : Determination of matric suction coefficient.

Matric Suction Profile

The matric suction profile will come to equilibrium at a hydrostatic condition when there is a zero net flux from the ground surface. If moisture is extracted from the ground surface such as evaporation, the matric suction profile will be drawn to the left. If moisture enters at the groundwater surface such as infiltration, the matric suction profile will be drawn to the right.

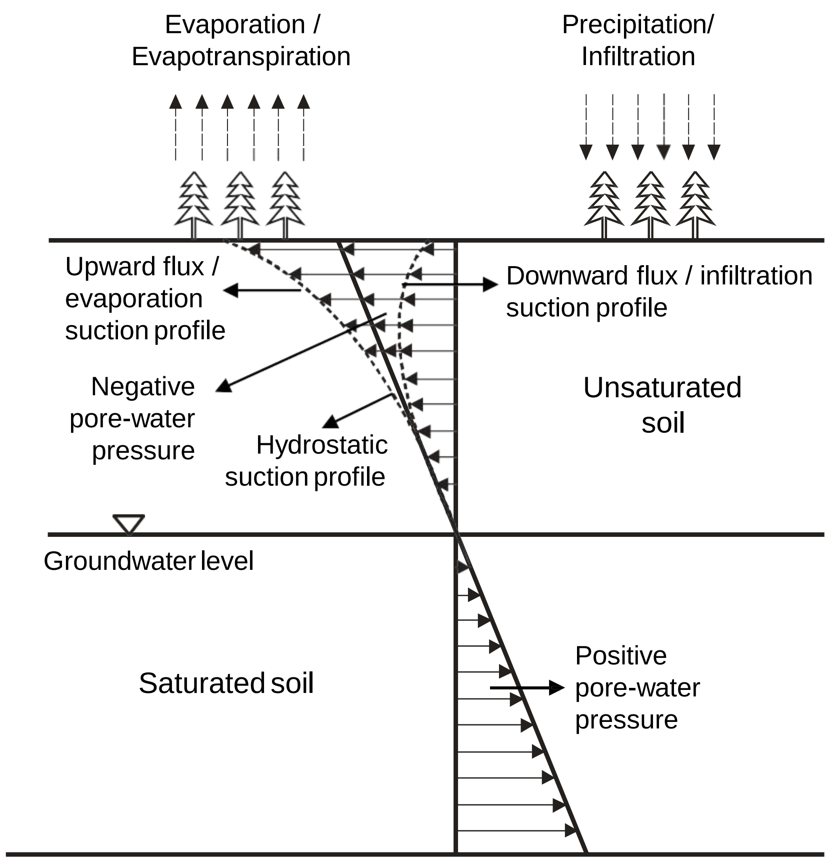


Figure : Matric suction profile in horizontally layered unsaturated soil profiles

Example of Unsaturated Soil Slope

In this part slope stability analysis of unsaturated soils due to rain infiltration will be discussed. A simple case of a homogeneous slope has been chosen. The international soil classification system USDA series is used for determining the hydraulic data for the analysis. The mechanical and hydraulic models used in the analysis are the Mohr Coulomb failure criterion and the Van Genuchten model respectively. The height of the slope is 10 m and the gradient (horizontal to vertical) is 2:1.

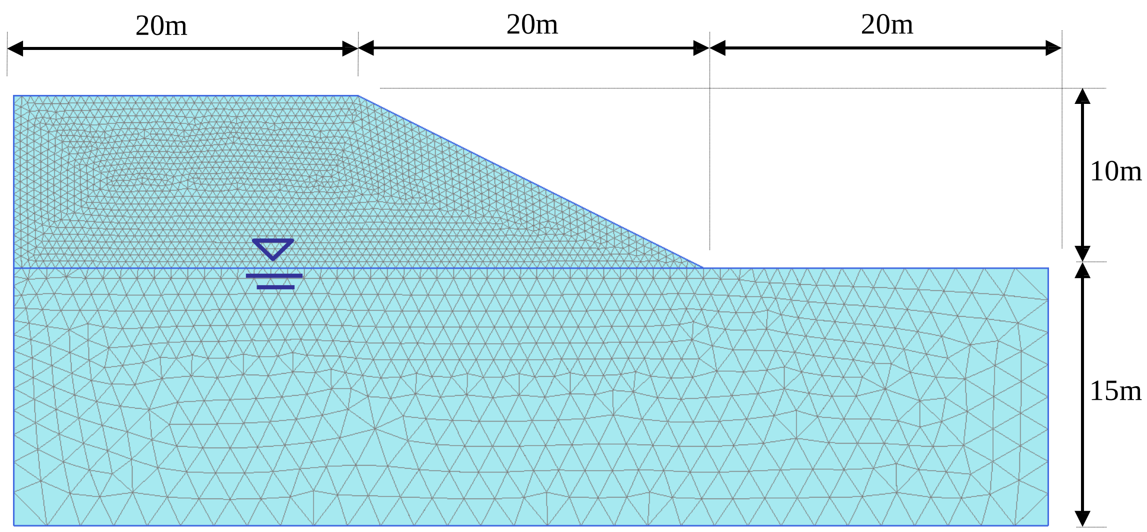


Figure : Geometry and two dimensional finite element mesh (4800 15-noded elements)

Soil Parameters

Soil parameters for the Mohr Coulomb model used in the analysis:

Description	Symbol	Unit	Value
Unit weight	γ	kN/m³	20
Elasticity modulus	E'	kPa	7500
Effective poisson's ratio	ν'	-	0.35
Effective cohesion	c'	kPa	20
Effective friction angle	ϕ'	o	20

Hydraulic Parameters

Four different hydraulic parameter sets of the USDA series for the Van Genuchten Models are used to evaluate the effect of these parameters in slope stability during rain infiltration:

Texture	K_{sat} (m/s)	g_a (1/m)	g_n (-)	g_l (-)
Sand	8.25E-05	14.50	2.68	0.50
Loamy Sand	4.05E-05	12.40	2.28	0.50
Sandy Loam	1.23E-05	7.50	1.89	0.50
Loam	2.89E-06	3.60	1.56	0.50
Silt	6.94E-06	1.60	1.37	0.50
Silty Loam	1.25E-06	2.00	1.41	0.50
Sandy Clay Loam	3.63E-06	5.90	1.48	0.50
Clayey Loam	7.22E-06	1.90	1.31	0.50
Silty Clay Loam	1.94E-06	1.00	1.23	0.50
Sandy Clay	3.33E-06	2.70	1.23	0.50
Silty Clay	5.50E-07	0.50	1.09	0.50
Clay	5.50E-08	0.80	1.09	0.50

Source: Plaxis 2D Reference Manual 2010 Beta

Boundary Conditions

The initial ground water level was assumed to be horizontal at level of the toe of the slope. A rainfall with intensity of 10 mm/hour lasting 3 days (72 hours) was applied on the crest and the slope. The minimum and the maximum pore pressure head respectively are -0.1 m (θ_{min}) and 0.1m (θ_{max}). The left boundary, right boundary and lower boundary of the model were assumed impervious boundaries.

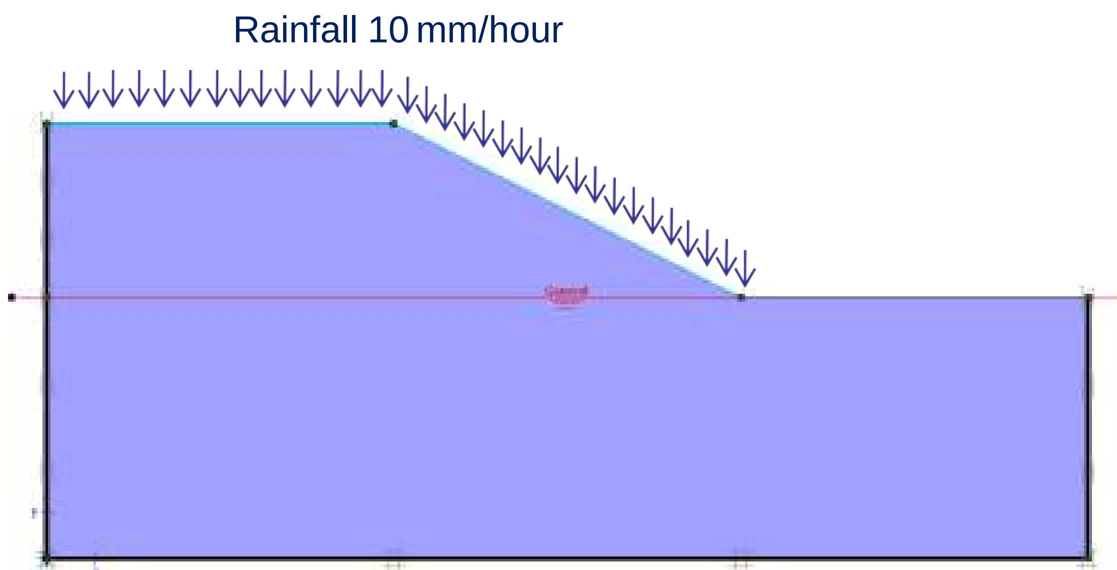


Figure : Boundary conditions of the model.

Initial Degree of Saturation

Initial degree of saturation for the four different hydraulic parameters leading to different initial degree of saturation at the same suction:

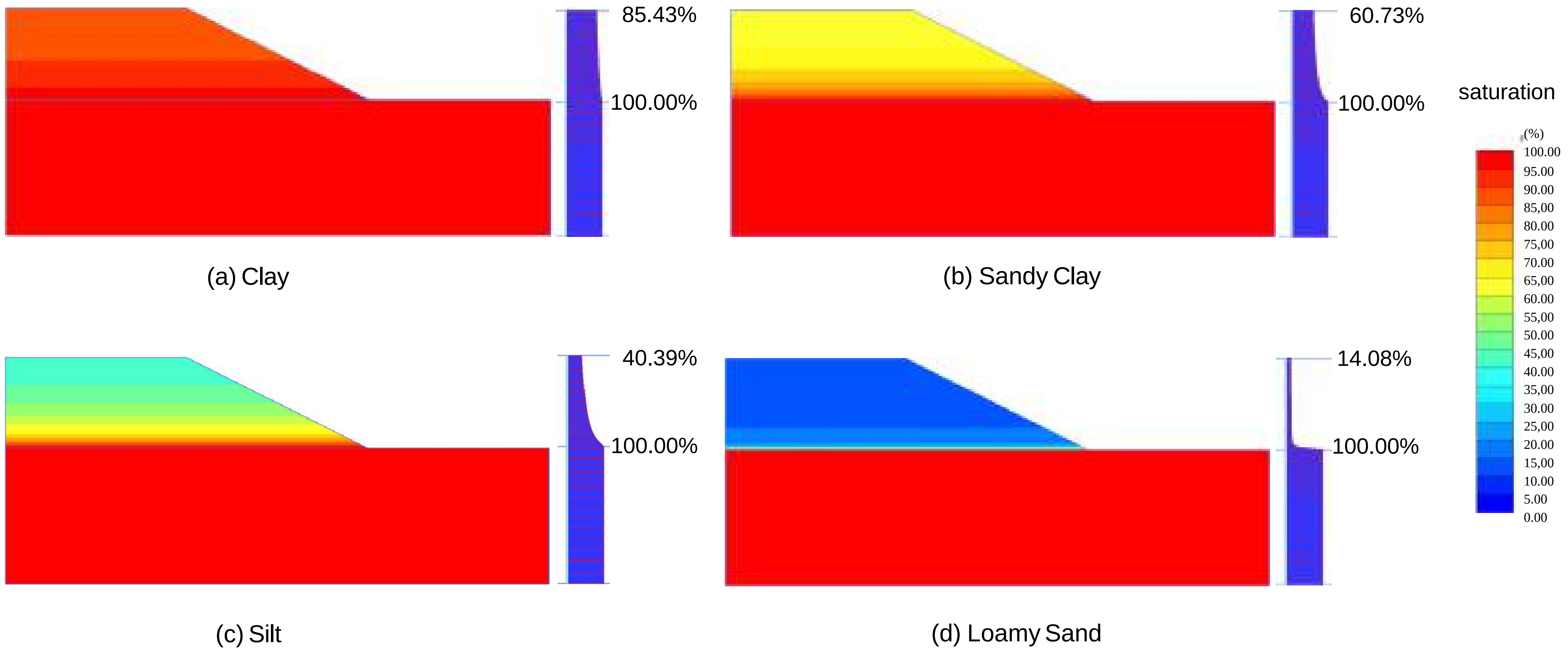


Figure : Initial conditions: degree of saturation.

Initial Suction

Initial suction in the model is assumed to increase linearly above ground water level until ground surface.

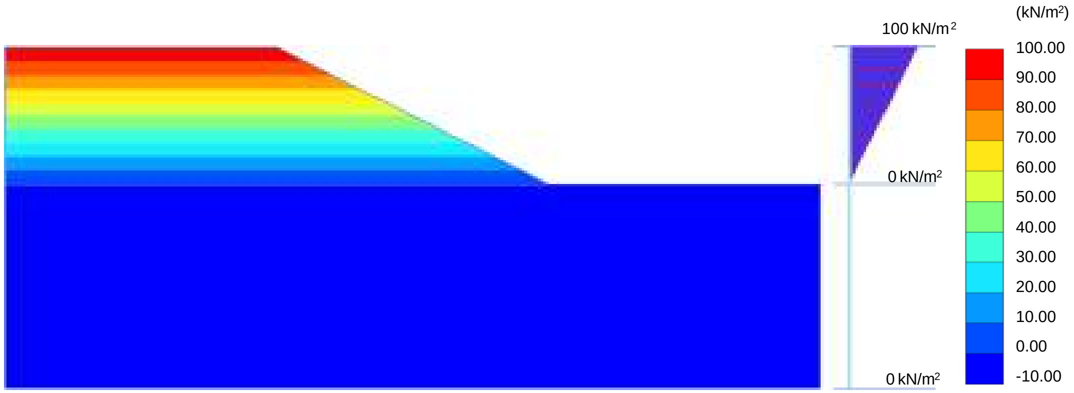


Figure : Initial conditions: suction.

The relation between suction and saturation, i.e. the Soil Water Characteristic Curve (SWCC):

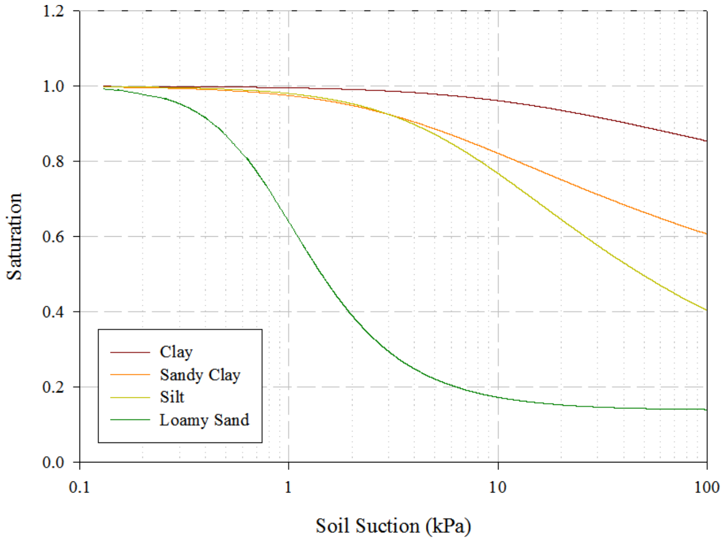
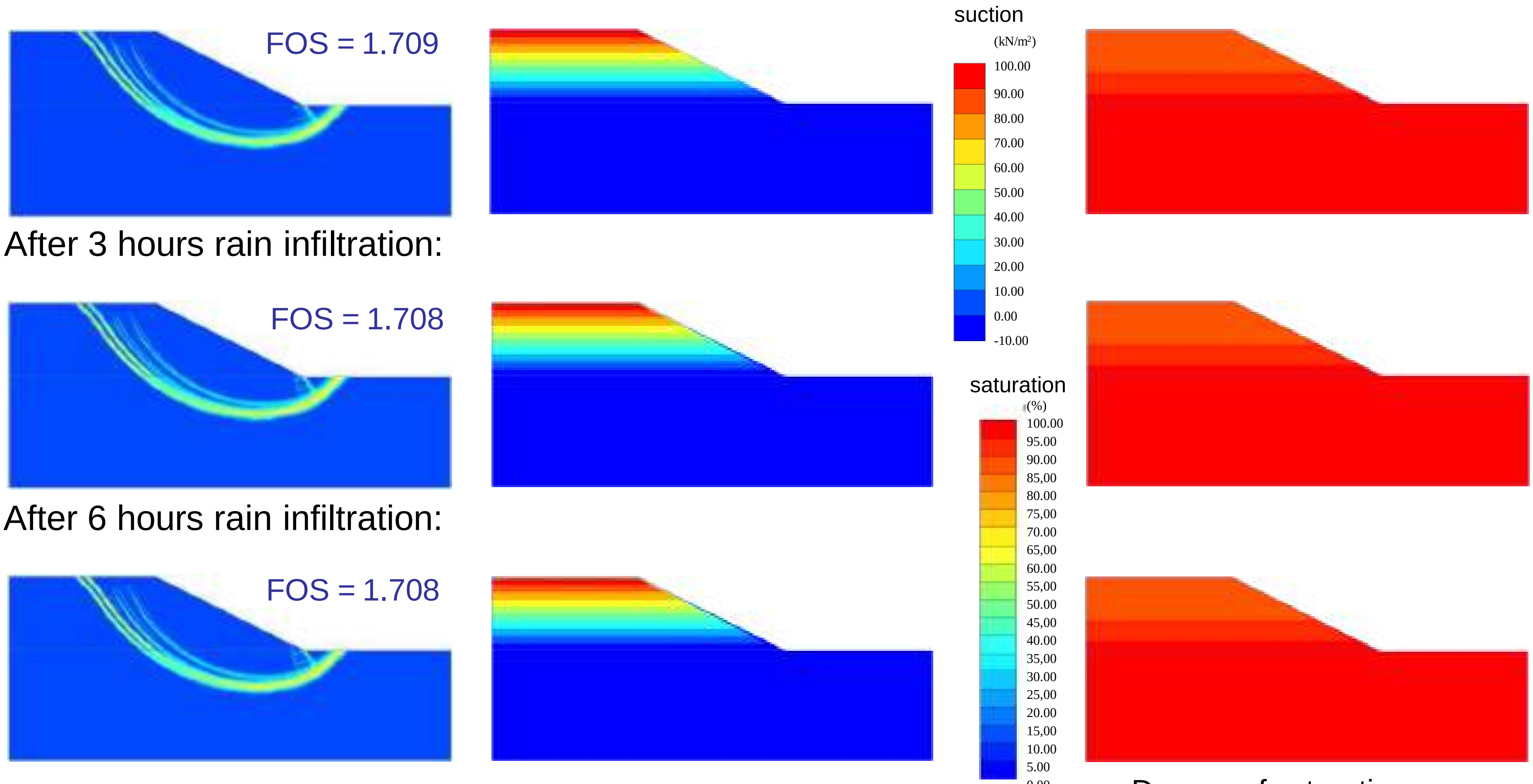


Figure : Soil Water Characteristic Curve (SWCC) for chosen soil types.

Result: Failure mechanism, distribution of suction and degree of saturation (Clay)

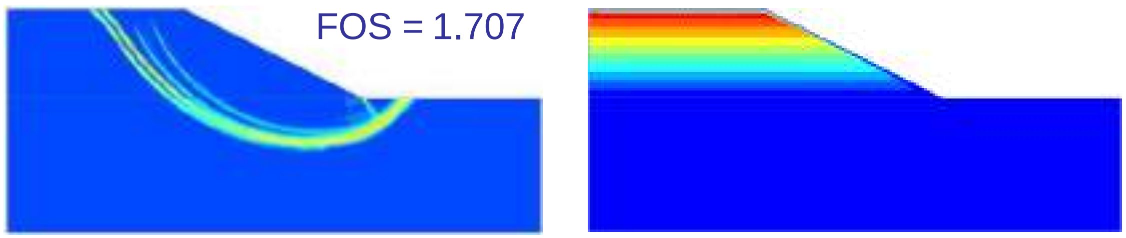
Before rain infiltration:

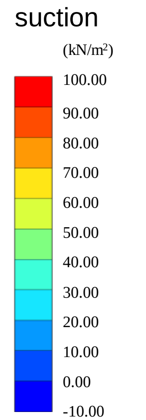


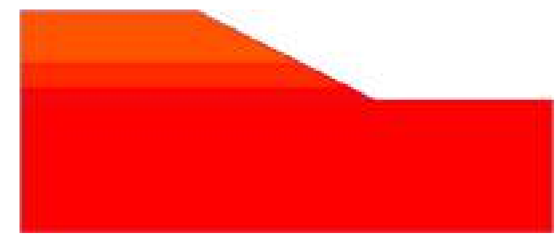
Failure mechanism | Distribution of suction | Degree of saturation

Result: Failure mechanism, distribution of suction and degree of saturation (Clay)

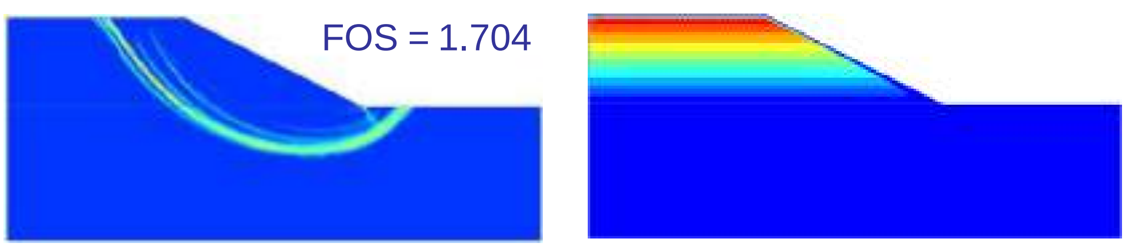
After 9 hours rain infiltration:

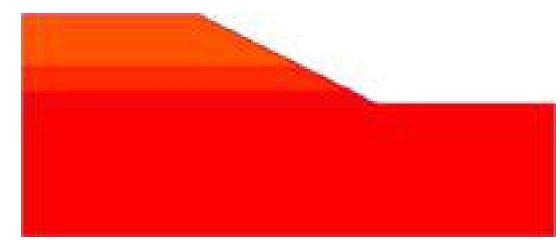


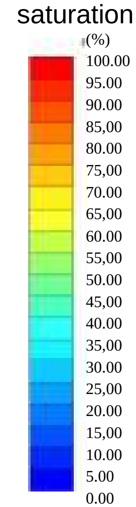




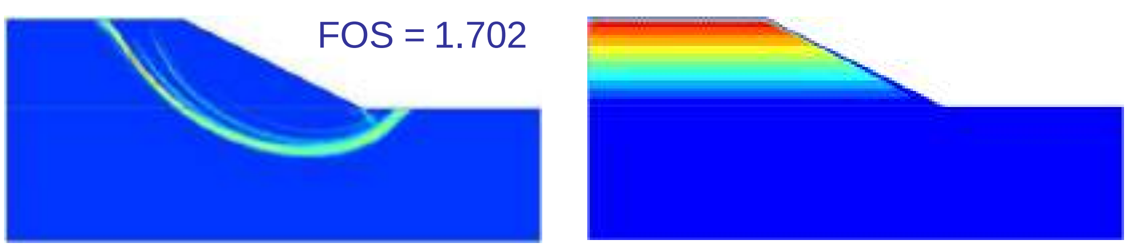
After 18 hours rain infiltration:







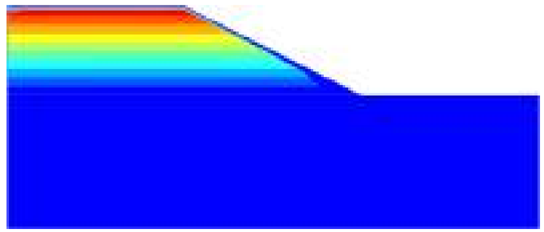
After 36 hours rain infiltration:

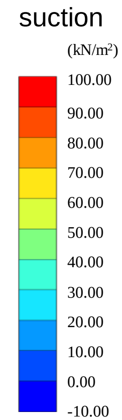


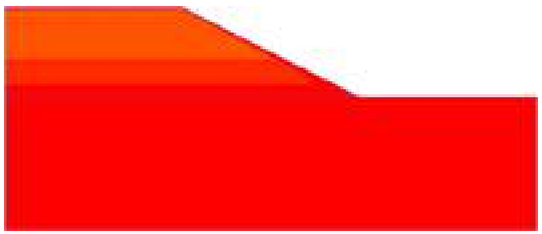
Failure mechanism Distribution of suction Degree of saturation

Result: Failure mechanism, distribution of suction and degree of saturation (Clay)

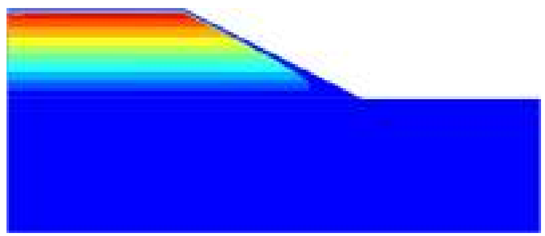
After 48 hours rain infiltration:

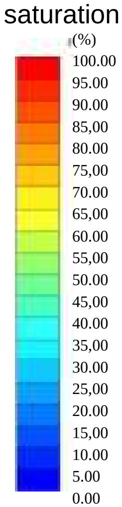


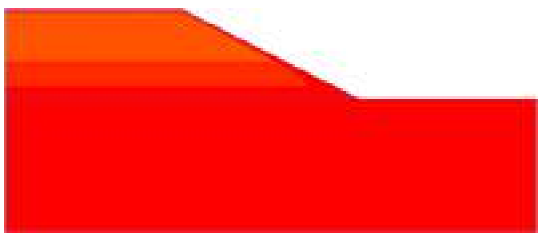




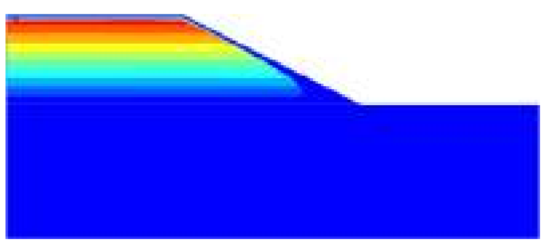
After 60 hours rain infiltration:

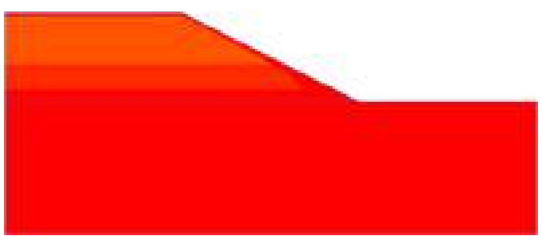






After 72 hours rain infiltration:

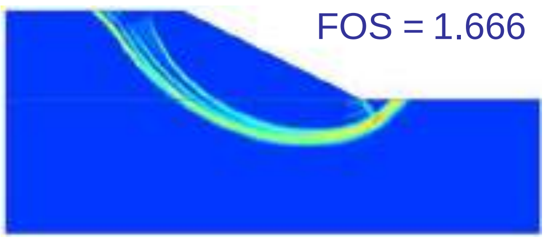


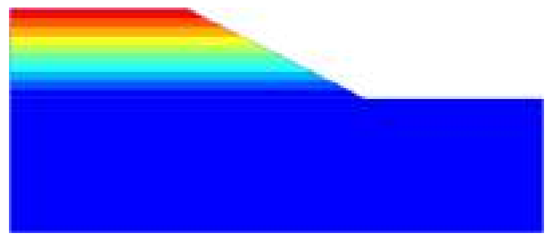


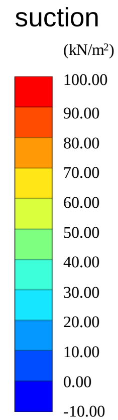
Failure mechanism | Distribution of suction | Degree of saturation

Result: Failure mechanism, distribution of suction and degree of saturation (Sandy Clay)

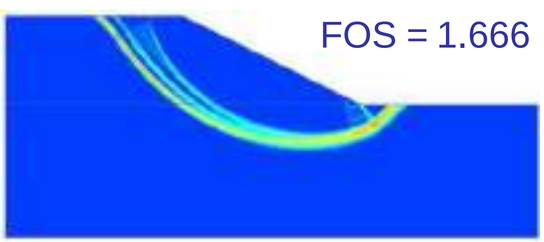
Before rain infiltration:

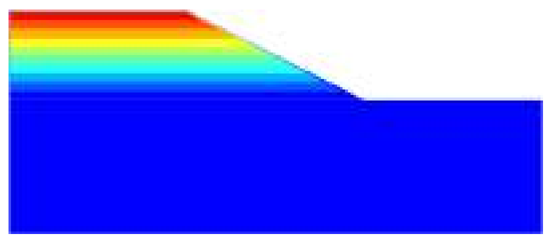


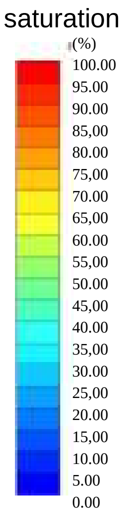




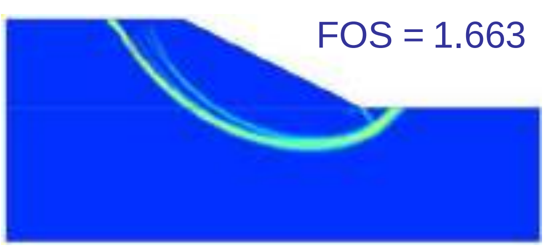
After 3 hours rain infiltration:

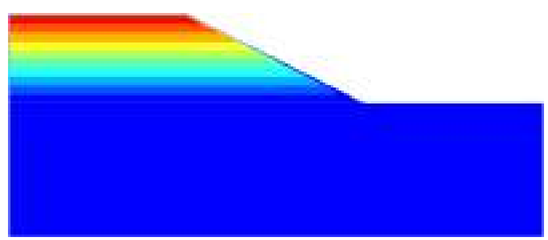


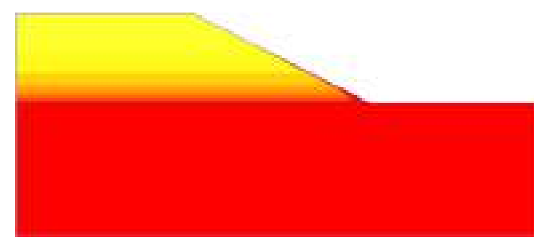




After 6 hours rain infiltration:



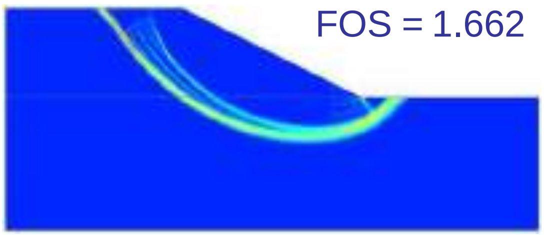


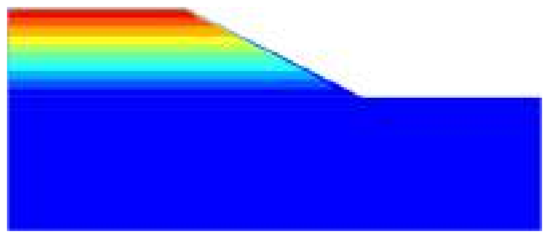
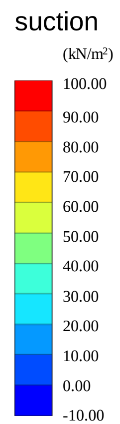


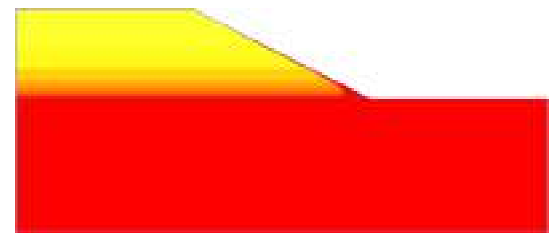
Failure mechanism | Distribution of suction | Degree of saturation

Result: Failure mechanism, distribution of suction and degree of saturation (Sandy Clay)

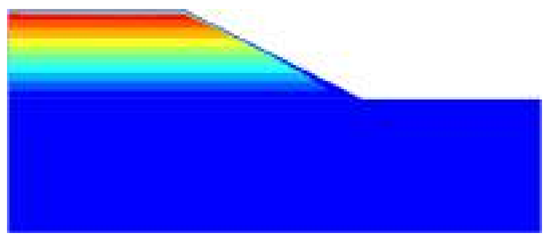
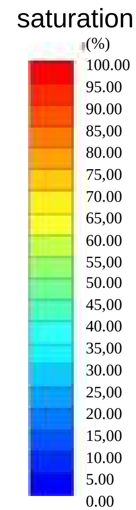
After 9 hours rain infiltration:



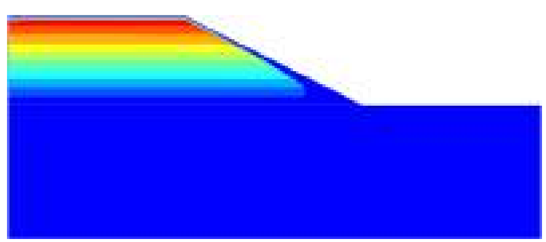





After 18 hours rain infiltration:

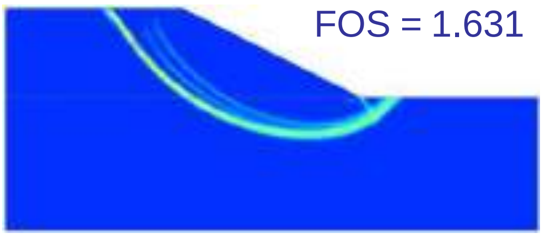
After 36 hours rain infiltration:

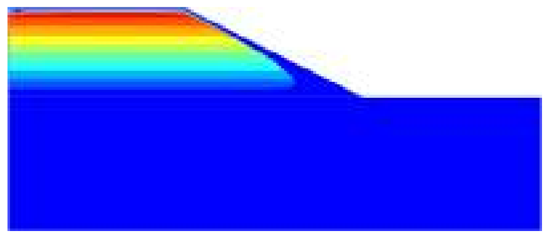



Failure mechanism | Distribution of suction | Degree of saturation

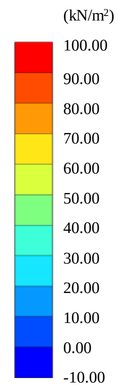
Result: Failure mechanism, distribution of suction and degree of saturation (Sandy Clay)

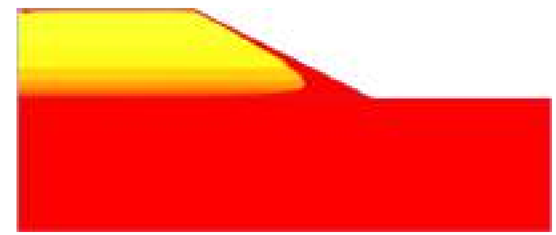
After 48 hours rain infiltration:



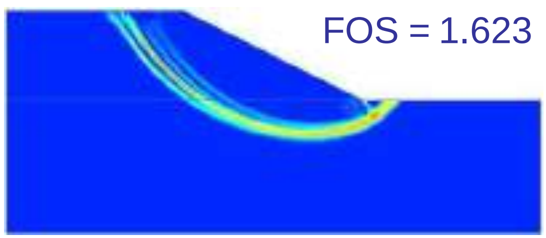


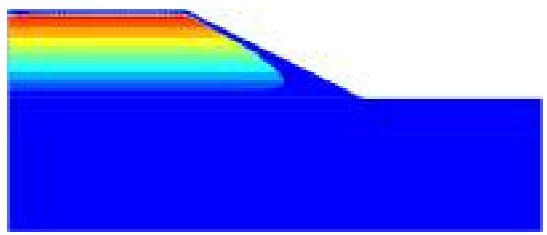
suction

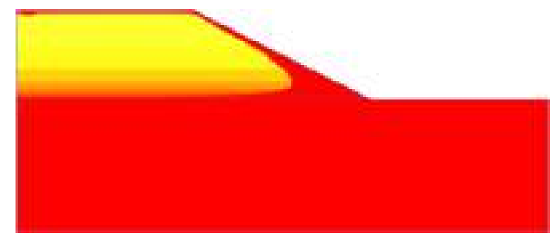




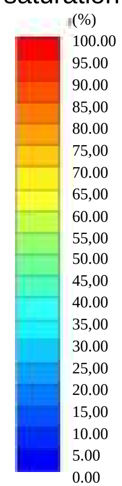
After 60 hours rain infiltration:



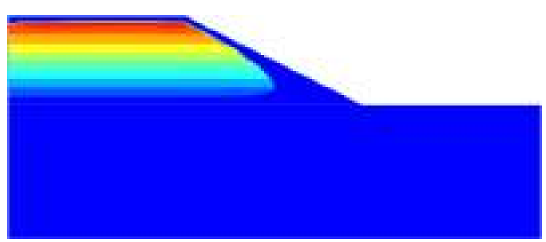


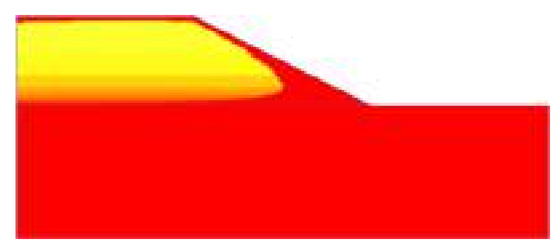


saturation



After 72 hours rain infiltration:





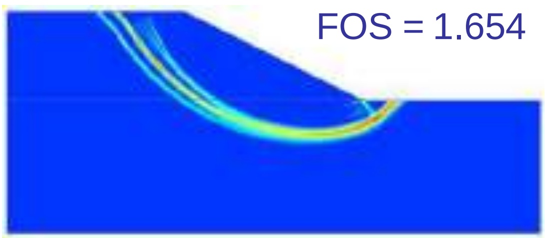
Failure mechanism

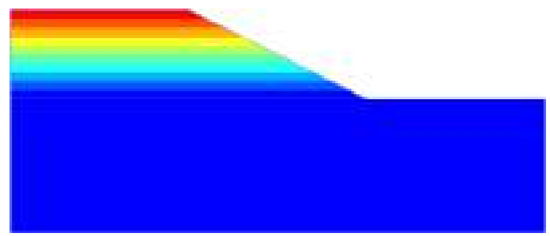
Distribution of suction

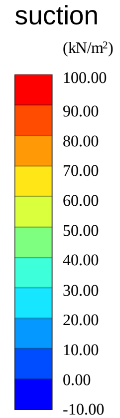
Degree of saturation

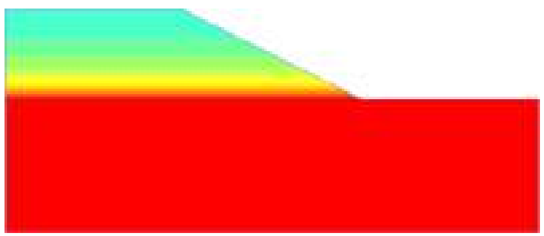
Result: Failure mechanism, distribution of suction and degree of saturation (Silt)

Before rain infiltration:

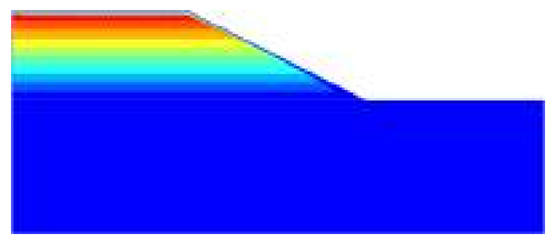


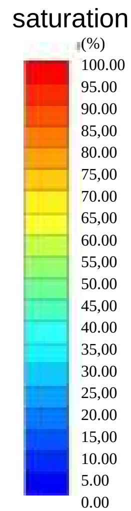


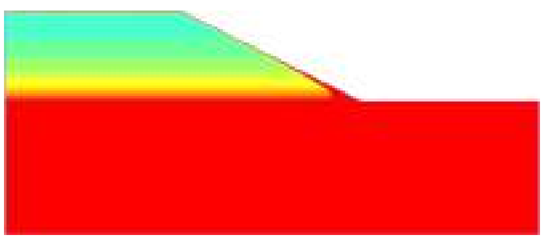




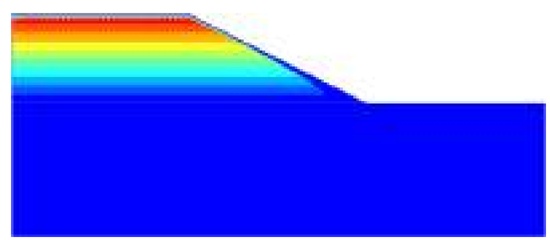
After 3 hours rain infiltration:

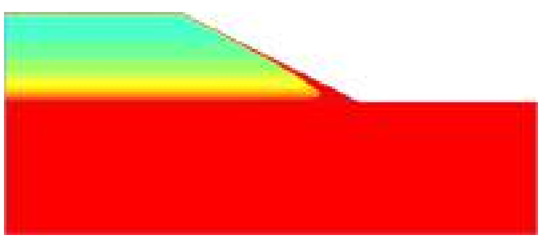






After 6 hours rain infiltration:





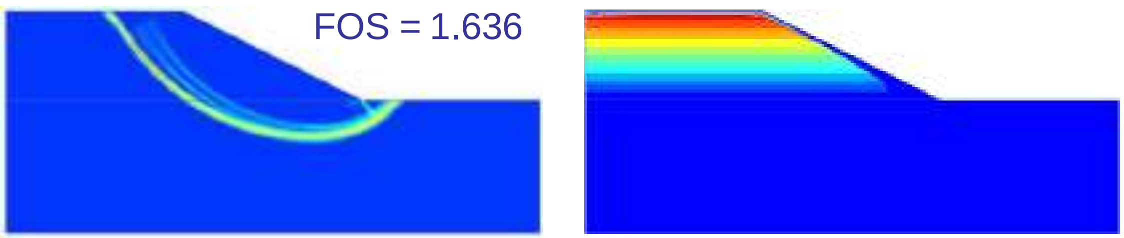
Failure mechanism

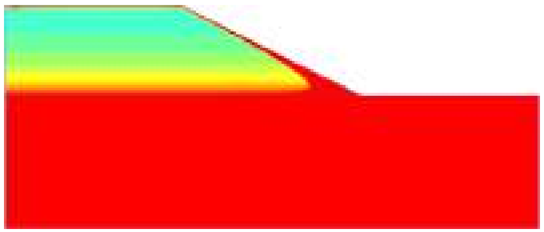
Distribution of suction

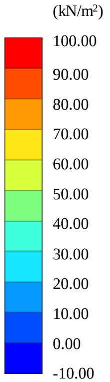
Degree of saturation

Result: Failure mechanism, distribution of suction and degree of saturation (Silt)

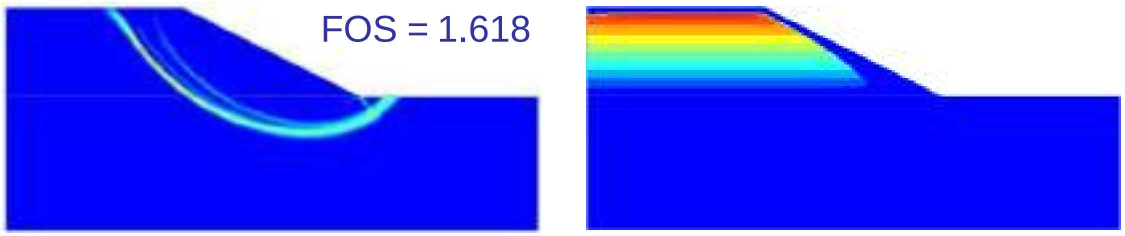
After 9 hours rain infiltration:

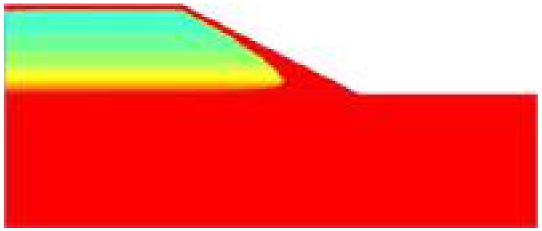


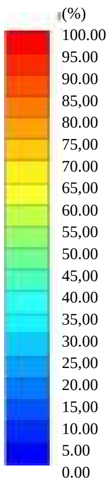




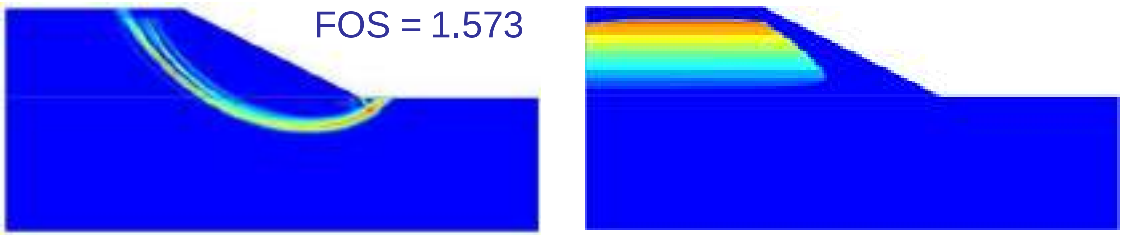
After 18 hours rain infiltration:

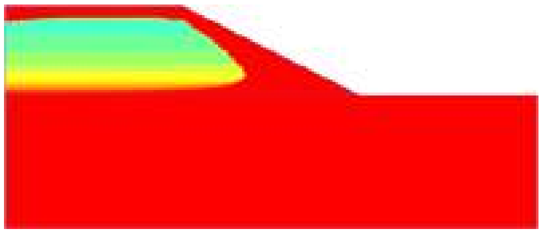






After 36 hours rain infiltration:

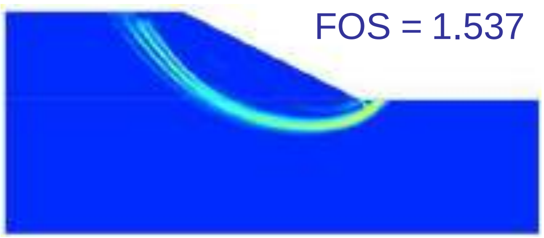




Failure mechanism　　Distribution of suction　　Degree of saturation

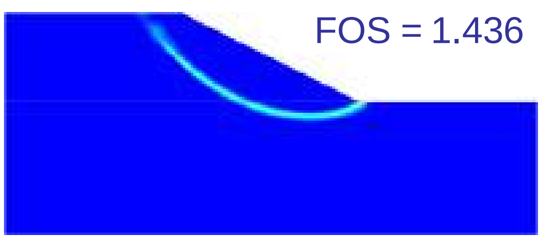
Result: Failure mechanism, distribution of suction and degree of saturation (Silt)

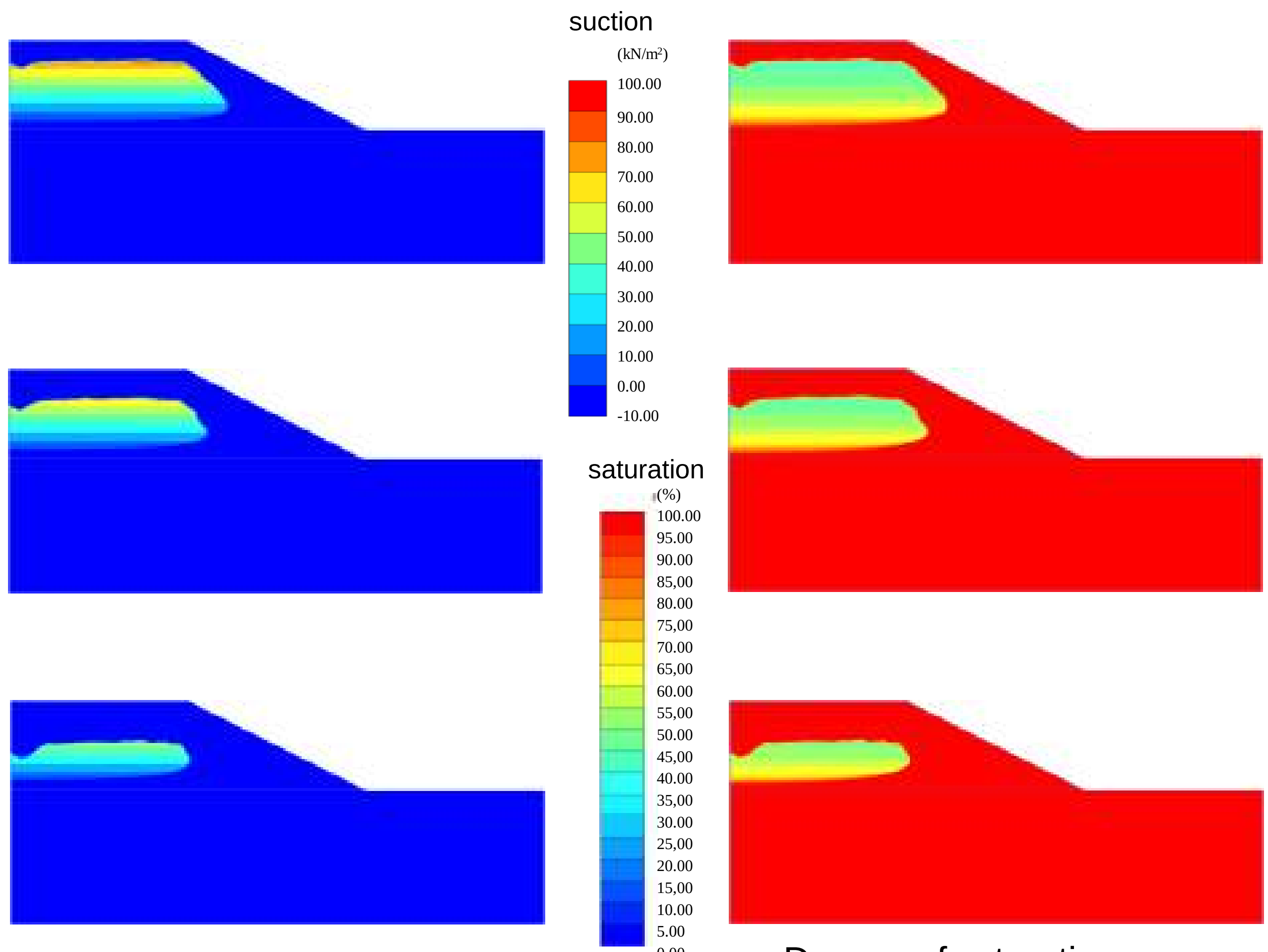
After 48 hours rain infiltration:



After 60 hours rain infiltration:

After 72 hours rain infiltration:

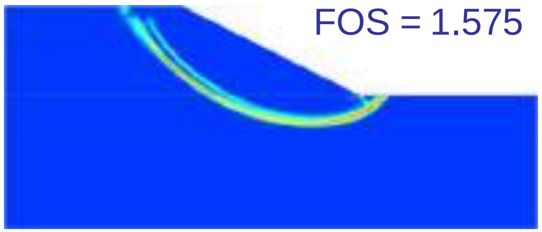


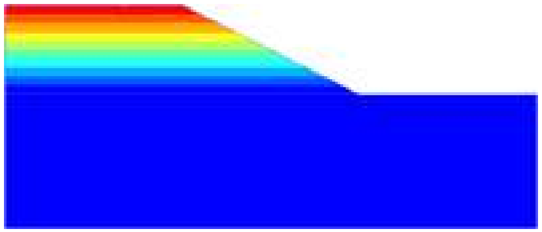


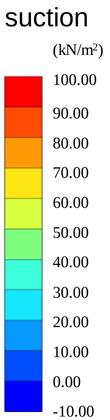
Failure mechanism | Distribution of suction | Degree of saturation

Result: Failure mechanism, distribution of suction and degree of saturation (Loamy Sand)

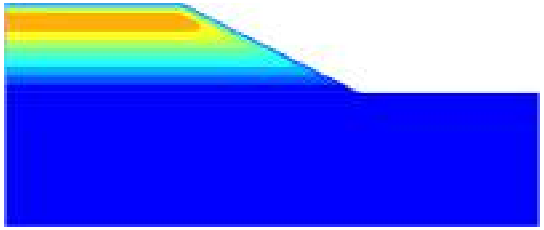
Before rain infiltration:

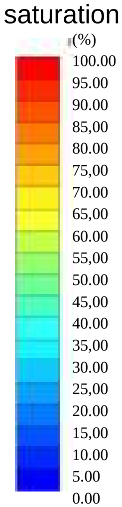




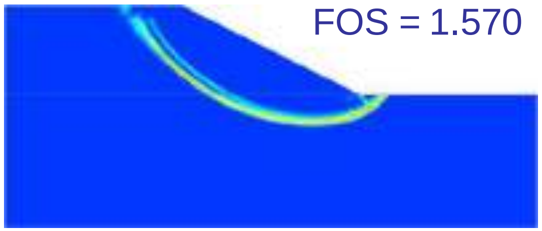


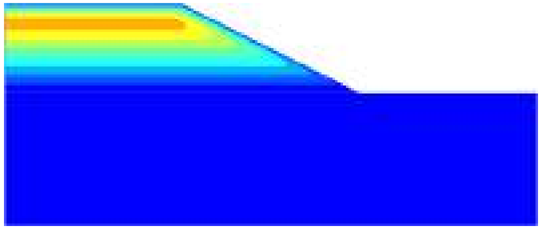
After 3 hours rain infiltration:





After 6 hours rain infiltration:





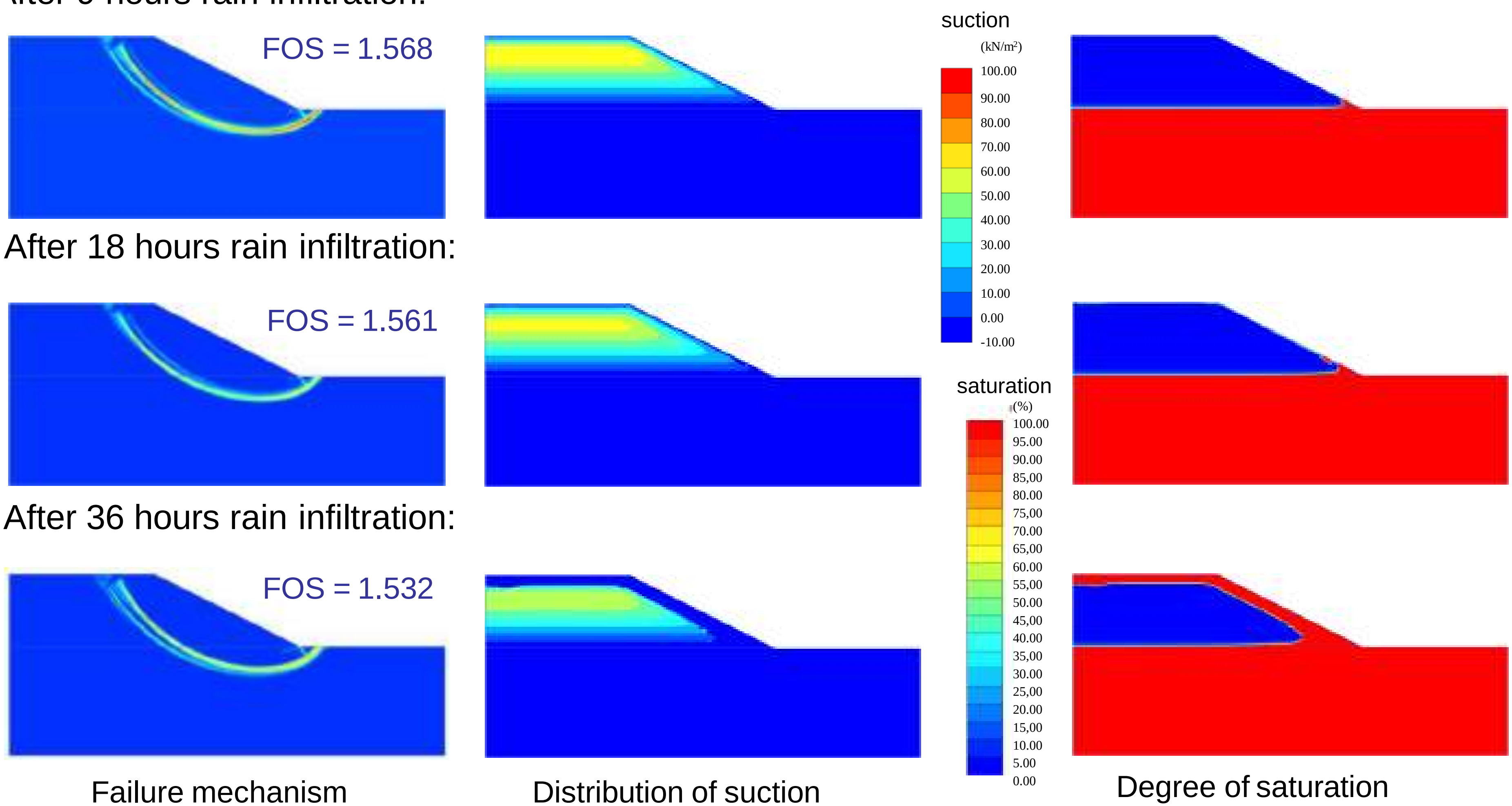
Failure mechanism

Distribution of suction

Degree of saturation

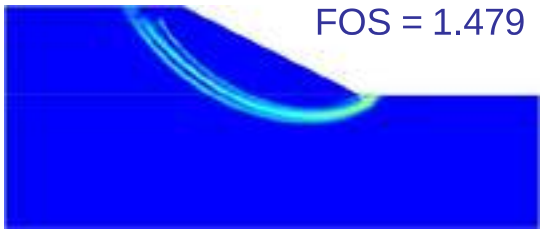
Result: Failure mechanism, distribution of suction and degree of saturation (Loamy Sand)

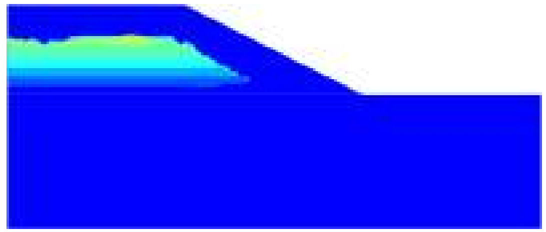
After 9 hours rain infiltration:

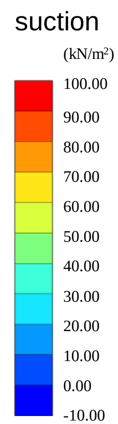


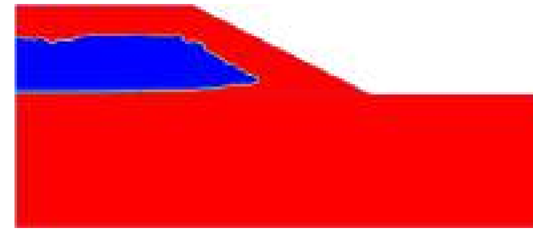
Result: Failure mechanism, distribution of suction and degree of saturation (Loamy Sand)

After 48 hours rain infiltration:

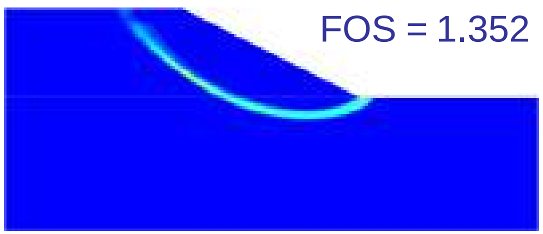


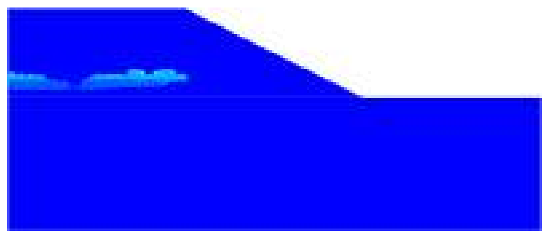


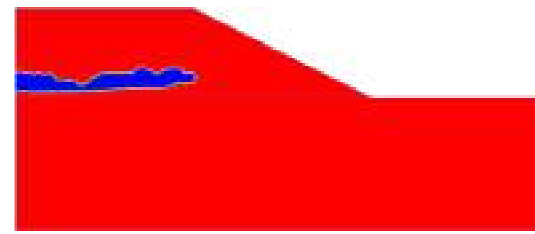


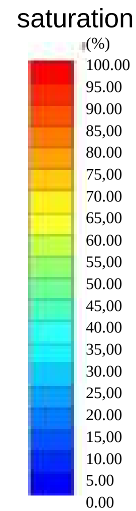


After 60 hours rain infiltration:

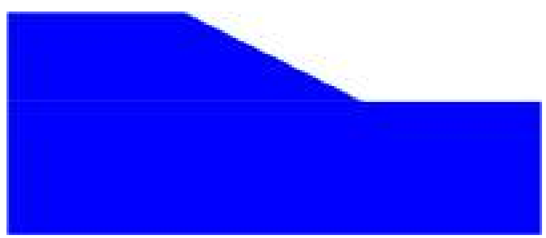


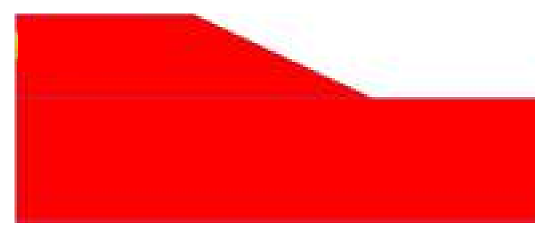






After 72 hours rain infiltration:

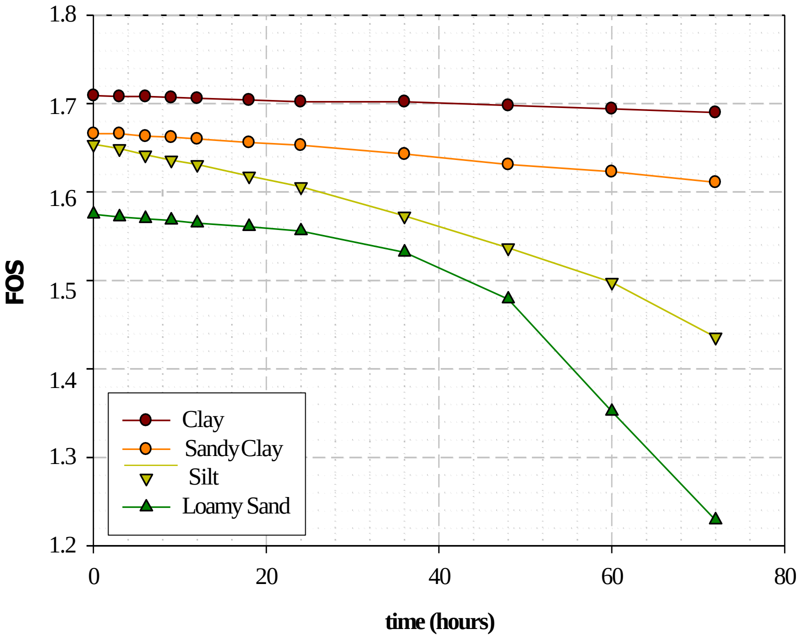




Failure mechanism | Distribution of suction | Degree of saturation

Resume of FOS of unsaturated soil slope during rain infiltration:

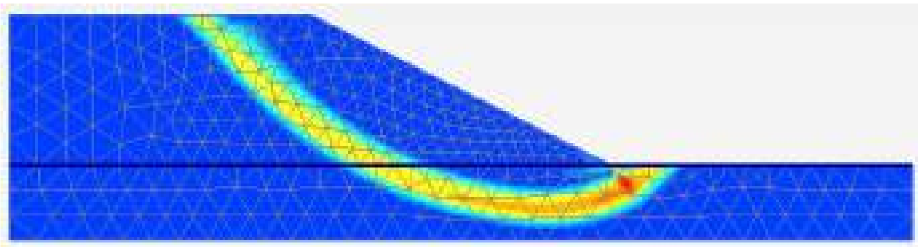
Time (hours)	FOS			
	Clay (k_{sat}=5.5E-07 m/s)	Sandy Clay (k_{sat}=3.3E-06 m/s)	Silt (k_{sat}=6.9E-06 m/s)	Loamy Sand (k_{sat}=4.1E-05 m/s)
0	1.709	1.666	1.654	1.575
3	1.708	1.666	1.649	1.572
6	1.708	1.663	1.642	1.570
9	1.707	1.662	1.636	1.568
12	1.706	1.660	1.631	1.565
18	1.704	1.656	1.618	1.561
24	1.702	1.653	1.606	1.556
36	1.702	1.643	1.573	1.532
48	1.698	1.631	1.537	1.479
60	1.694	1.623	1.498	1.352
72	1.690	1.611	1.436	1.229



Conclusions

- Safety factors from FEM compare well with factors obtained from LEM.
- FEM for slope stability is more powerful than LEM. The failure mechanisms in FEM are computed automatically as part of the stress equilibrium process.
- Effect of rainfall infiltration, leading to change of suction and saturation in slope, on factor of safety can be assessed.
- During the time of rain infiltration, suction decreases and thus the FOS of the slope reduces, whereas the reduction is faster for soils with high permeability than for soils with low permeability.

EXERCISE 1: Slope stability analysis (*fully saturated slopes*)



Indra Noer Hamdhan

COMPUTIONAL GEOTECHNICS

INTRODUCTION

The exercise concerns the stability of a slope in short term condition. In order to keep the problem as simple as possible, only the Mohr Coulomb material model is considered. Besides the procedure to generate the finite element mesh, attention is paid to the input of boundary conditions, material properties, the actual calculation and inspection of some output results.

AIMS

- Geometry input
- Parameters
- Calculation of FOS of the slope

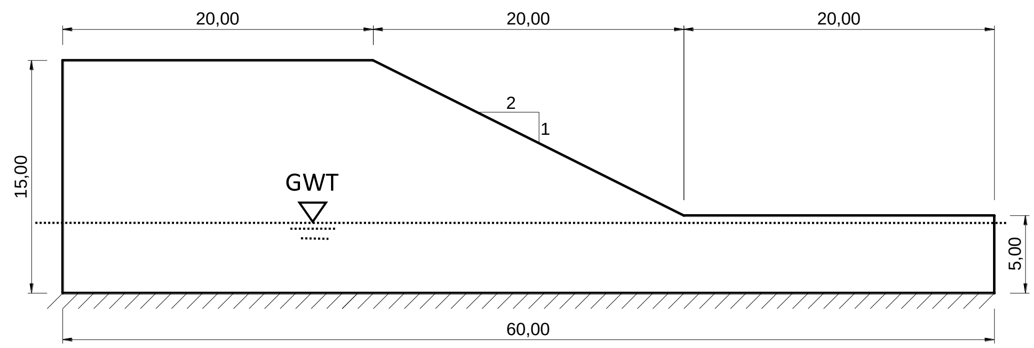


Figure 1: Scheme of the exercise (Griffiths and Lane, 1999)

SCHEME OF OPERATIONS:

GEOMETRY INPUT

- Project properties
- Defining of soil stratigraphy
- Material data sets
- Defining of slope geometry
- Mesh generation

PERFORMING CALCULATIONS

- Initial conditions
- Phase 1 (safety analysis)
- Execution of calculation
- Viewing calculation results

THE SAFETY FACTOR

- The safety factor

GEOMETRY INPUT

Start PLAXIS by double-clicking the icon of the Input program.

- A *'Quick Select'* dialog box will appear in which you can select an existing project or create a new one.
- Choose *'Start a new project'* (see Figure 2). Now the *'Project properties'* window appears, consisting of the two tabsheets *'Project'* and *'Dimensions'* (see Figure 3 and Figure 4).

Figure 2: Quick Select dialog box

Project Properties

The first step in an analysis is to define the basic parameters of the finite element model. This is done in the *'Project properties'* window. These settings include the description of the problem, the type of analysis, the basic type of elements, the basic units and the size of the drawing area.

In order to enter the proper settings for the footing project, follow these steps:

- In the *'Project'* tabsheet, enter “Slope Exercise 1” in the *'Title'* box and type “Slope stability analysis without considering suction” or any other text in the *'Comments'* box.

- In the *'Model'* tabsheet, keep the default units in the Units box (Unit of Length = m; Unit of Force = kN; Unit of Time = day).
- In the *'Model'* tabsheet, the type of the analysis (*Model*) and the basic element type (*Elements*) are specified. As this exercise concerns a slope stability, choose *'Plane strain'* from the *'Model'* combo box. Select *'15-Node'* from the *'Elements'* combo box.
- The *'General'* group the gravity is 1.0g (-Y direction, which is in the vertical direction) and earth gravity is 9.810 m/s^2 and the unit weight of water γ_{water} is set to 10 kN/m^3.

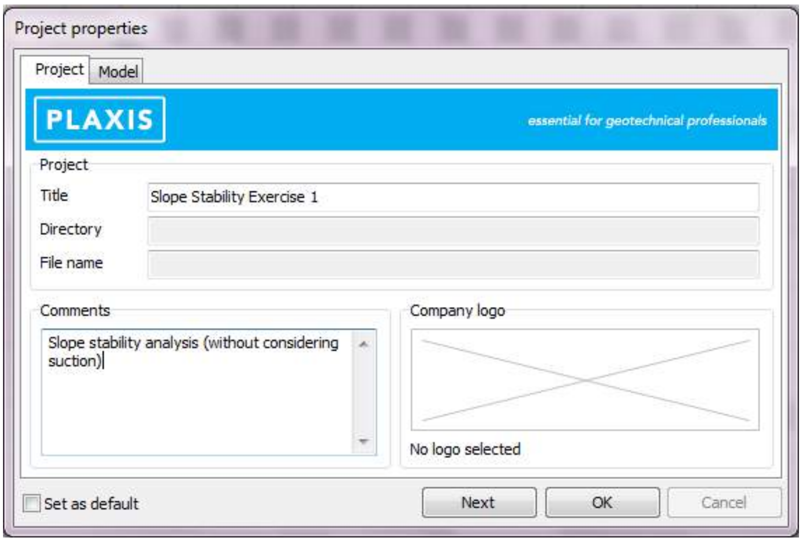


Figure 3: Project tabsheet of the Project properties window

- In the *'Contour'* group the size of the considered geometry must be entered. The values entered here determine the size of the drawing area in the Input window. PLAXIS 2D will automatically add a small margin so that the geometry will fit well within the draw area. Enter the following values in the boxes *'x_{min}', 'x_{max}', 'y_{min}', 'y_{max}'* – 0.0; 60.0; 0.0; 15.0.
- Click on the  button to confirm the settings. Now the drawing area appears in which the geometry model can be drawn.

Hint: For changing the general settings, one can access the *'Project properties'* window by selecting the *'Project properties'* option from the *'File'* menu.

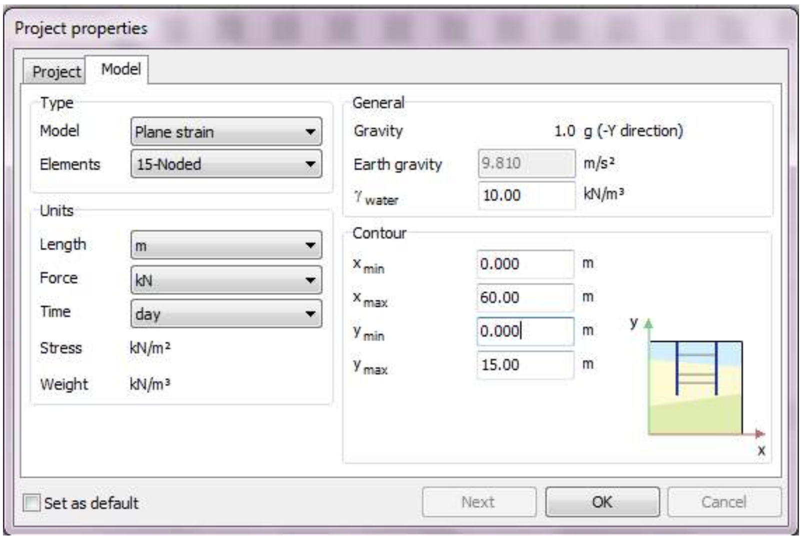


Figure 4: Model tabsheet of the Project properties window

Defining of soil stratigraphy

Information on the soil layers is entered in boreholes. Boreholes are locations in the draw area at which the information on the position of soil layers and the water table is given. If multiple boreholes are definied, PLAXIS 2D will automatically interpolate between the boreholes. The layer distribution beyond the boreholes is kept horizontal. In order to construct the soil stratigraphy follow these steps:

Click the *Create borehole* button in the side (vertical) toolbar to start defining the soil stratigraphy.

- Click at x = 0 in the draw area to locate the borehole. The *Modify soil layers* window will appear.
- In the *Modify soil layer window* add a soil layer by clicking the *Add* button.
- Set the top boundary of the soil layer at y = 5 and keep the bottom boundary at y = 0 m.
- By default the *Head* value (groundwater head) in the borehole column is set to 0 m. Set the Head to 5.0m (Figure 5).

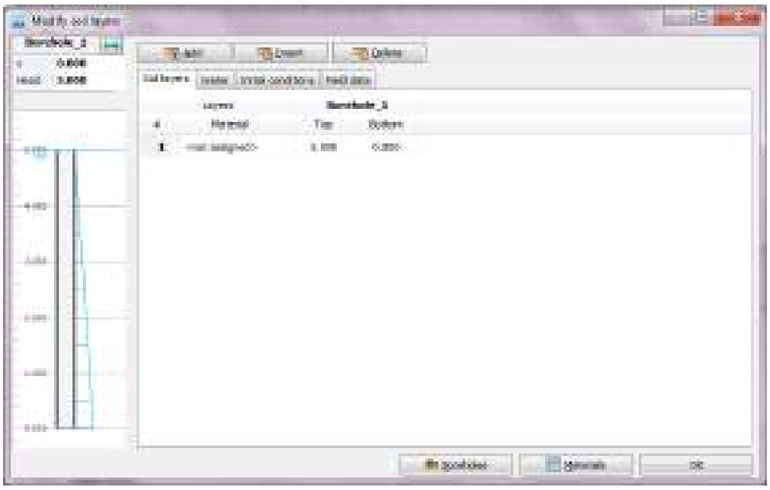


Figure 5: Modify soil layer window.

The creation of material data sets and their assignment to soil layer is described in the following section.

Material data sets

In order to simulate the behaviour of the soil, a suitable soil model and appropriate material parameters must be assigned to the geometry. In PLAXIS 2D, soil properties are collected in material data sets and the various data sets are stored in a material database. From the database, a data set can be assigned to one or more soil layers. For structures (like walls, plates, anchors, geogrids, etc.) the system is similar, but different types of structures have different parameters and therefore different types of material data sets. PLAXIS 2D distinguishes between material data sets for *'Soil and Interfaces', 'Plates', 'Anchors', 'Embedded pile row' and 'Geogrids'*.
The creation of material data sets is generally done after the input of boundary conditions. Before the mesh is generated, all material data sets should be defined and all clusters and structures must have their appropriate data set.

To create of material set for soil layer, follow these steps:

Open the *Material sets* window by clicking the *Materials* button in the *Modify soil layers* window. The material sets windows pops up (Figure 6).

Figure 6: Material sets window.

- Click the *New* button at the lower side of the Material sets window. A new window will appear with five tabsheets: General, Parameters, Flow Parameters, Interface and Initial.
- Enter the proper values in the *'General properties'* box according to the material properties listed in **Error! Reference source not found.** (see also **Error! Reference source not found.** and **Error! Reference source not found.**).
- Hydraulic model, data set, soil type, values of permeability, and initial void ratio can be entered in the *'Parameter'* box of the *'Flow parameters' tabsheet.* (see **Error! Reference source not found.**)
- Since the geometry model does not include interfaces, the fourth tabsheet *'Interfaces'* can be skipped.
- Click OK to confirm the input of the current material data set. Now the created data set will appear in the tree view of the *'Material Sets'* window.

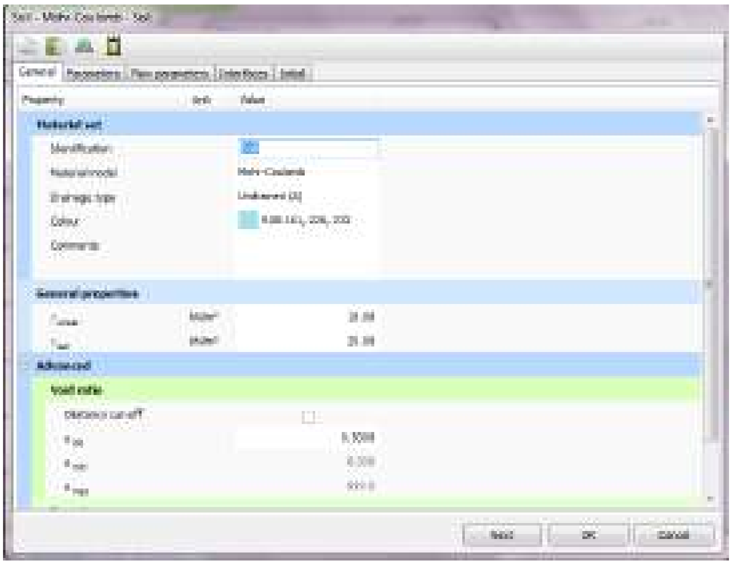


Figure 7: General tabsheet of the soil and interface data set window.

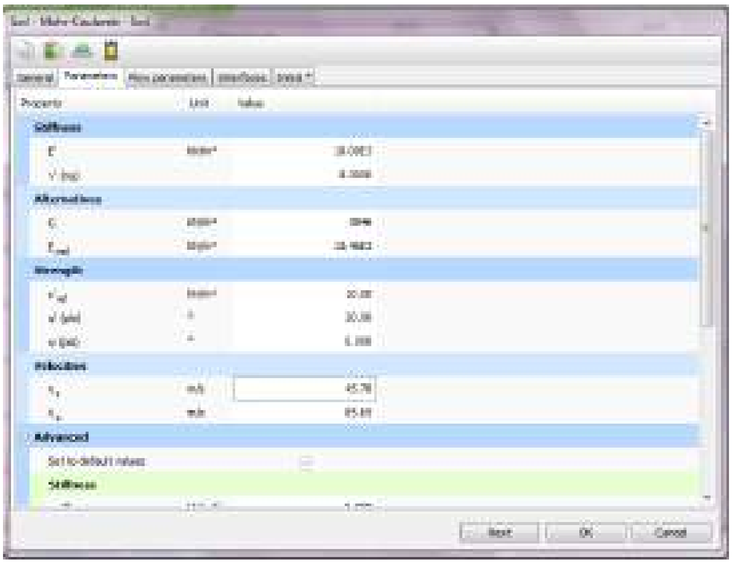


Figure 8: Parameters tabsheet of the soil and interface data set window.

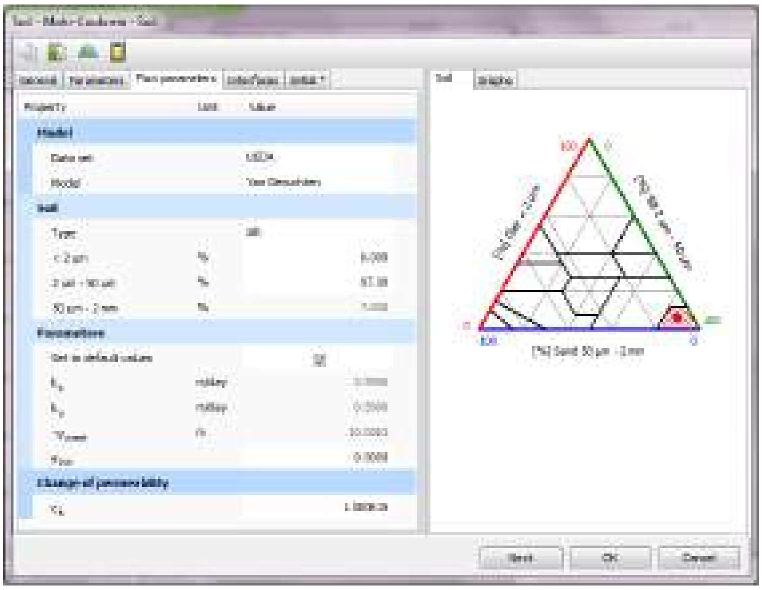


Figure 9: Flow parameters tabsheet of the soil and interface data set window.

The material properties of the soil layer are listed in the following table.

Parameter	Symbol	Soil	Unit
General			
Material model	Model	Mohr Coulomb	-
Type of material behaviour	Type	Undrained A	-
Soil unit weight above phreatic level	γ_{unsat}	18,0	kN/m^3
Soil unit weight below phreatic level	γ_{sat}	20,0	kN/m^3
Parameters			
Young's modulus	E_{ref}	10000	kN/m^2
Poisson's ratio	ν	0,30	-
Cohesion	c'	10,0	kN/m^2
Friction Angle	ϕ'	20,0	º
Flow parameters			
Data set	-	USDA	-
Model	-	Van Genuchten	-
Soil type	-	Silt	-
Permeability x-dir.	k_x	0,5996	m/day
Permeability y-dir.	k_y	0,5996	m/day

Table 1: Material properties

Van Genuchten model is the model that describes the hydraulic behaviour of unsaturated soil. In this model Soil Water Characteristic Curve (SWCC) is introduced to describe hydraulic parameters of the groundwater flow in unsaturated zones (usually above the phreatic surface). This input information is not used in Exercise 1, but will be used in the following exercises.

- Drag the set *Soil* from the *Material sets* window (select it and hold down the left mouse button while moving) to the graph of the soil column on the left hand side of the Modify soil layers window and drop it there (*release the left mouse button*).
- Click *OK* in the *Material sets* window to close the database.
- Click *OK* to close the *Modify soil layers* window.

Defining of slope geometry

The slope is defined in the *Structures* mode. To define slope layers:

Click the *Create soil polygon* button in the side toolbar and select the Create soil polygon option in the appearing Menu.

- Define the slope in the draw area by clicking on on (0.0 5.0), (0.0 15.0), (20.0 15.0) and (40.0 5.0).
- Right-click the created polygon and assign the slope data set to the soil polygon (Figure 10).
- Final model geometry will be appear (Figure 11)

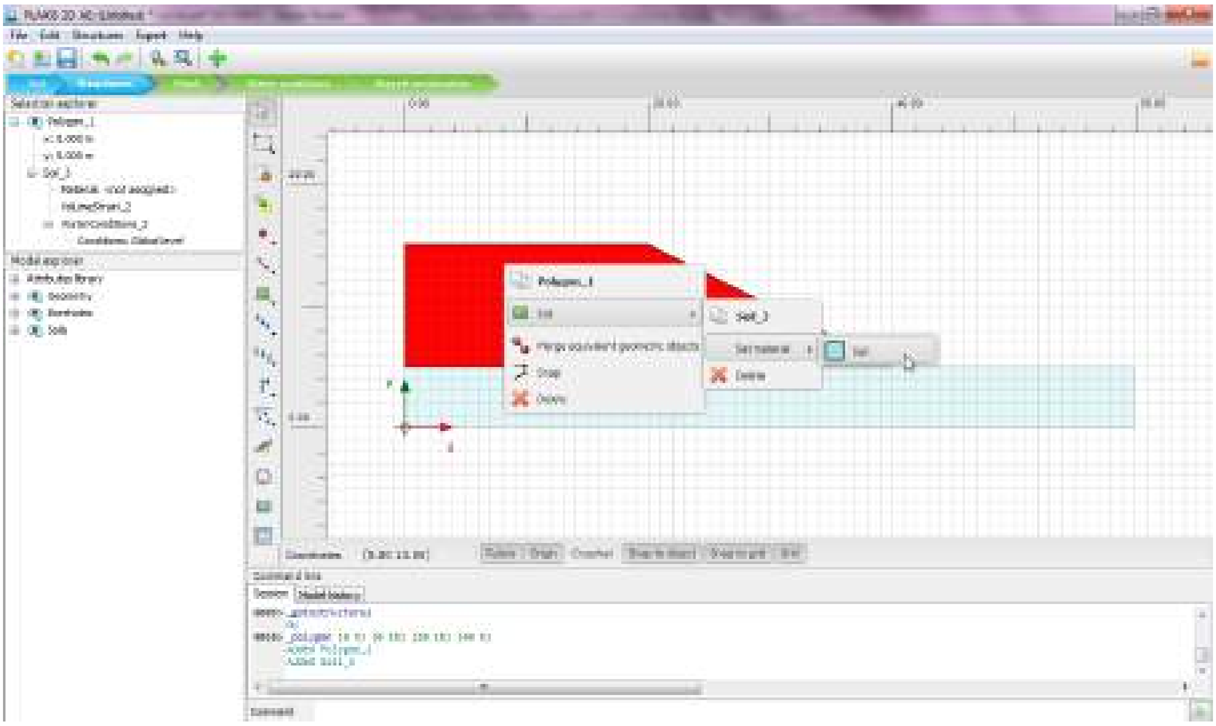


Figure 10: Assignment of a material dataset to a soil claster in the draw area.

The proposed geometry does not include plates, hinges, geogrids, interfaces, anchors or tunnels. Hence, you can skip the corresponding buttons in the toolbar.

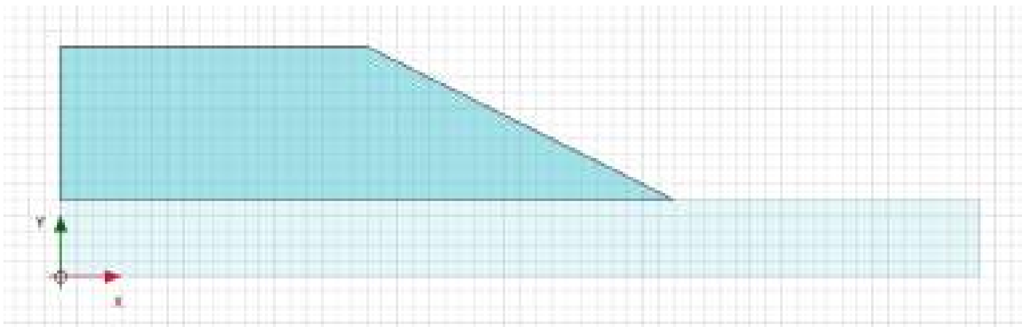


Figure 11: Final geometry of the model.

Mesh Generation

In order to generate the mesh, follow these steps:

- Proceed to the *Mesh* mode by clicking the corresponding tab.

 Click the *Generate mesh* button in the side toolbar. The *Mesh options* window pops up.

- Select the *Very Fine* option as element distribution.
- Click OK to start the mesh generation.

Figure 12: Mesh options window.

As the mesh is generated, click the *View mesh* button. A new window is opened displaying the generated mesh (Figure 13).

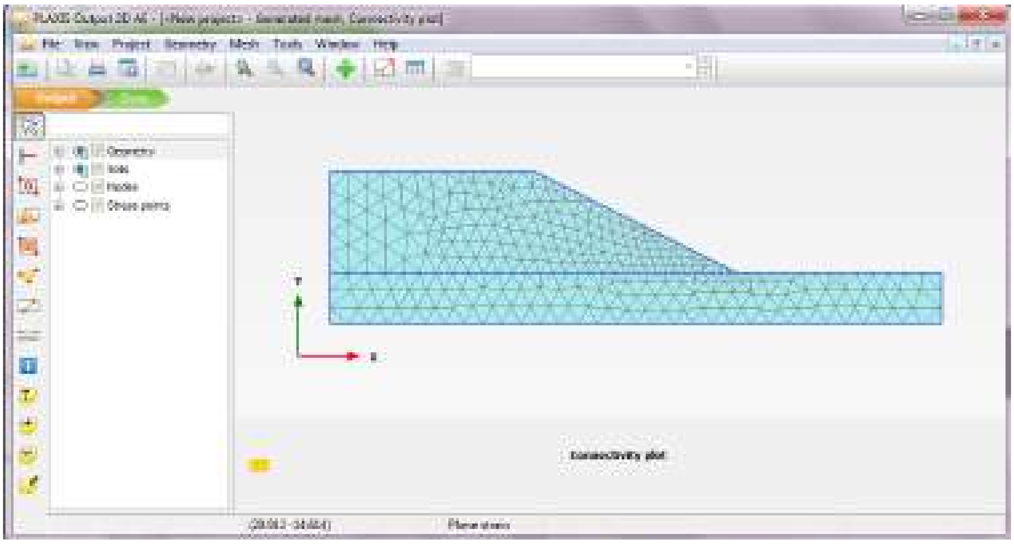


Figure 13: The generated mesh in the *Output* window.

Click on the *'Close'* tab to close the Output program and back to the *Mesh* mode.

PERFORMING CALCULATIONS

Once the mesh has been generated, the finite element model is complete.

Initial Conditions

Click the *water conditions* tab to show the initial water conditions that already defined (Figure 14).

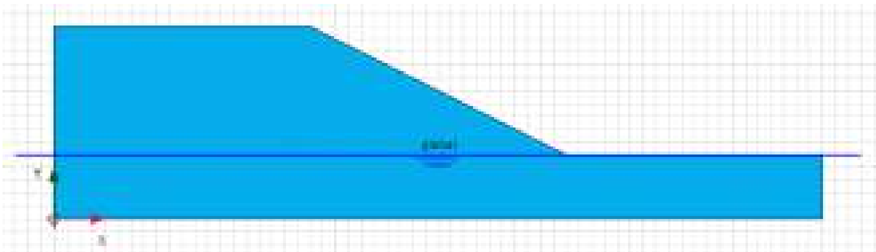


Figure 14: The initial water conditions.

The 'Initial phase' always involves the generation of initial conditions. In general, the initial conditions comprise the initial geometry configuration and the initial stress state, i.e. effective stresses, pore pressures and state parameters, if applicable.
Click the *Staged construction* tab to proceed with the definition of calculation phases. When a new project has been defined, a first calculation phase named "Initial phase", is automatically created and selected in the *Phases explorer* (Figure 15). All structural elements and loads that are present in the geometry are initially automatically switched off; only the soil volumes are initially active.

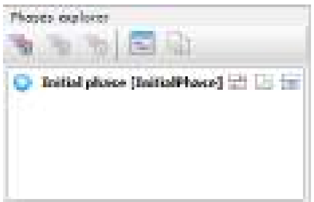


Figure 15: Phases explorer.

The *Phases* window (Figure 16) is displayed by clicking the *Edit phase* button or by double clicking on the phase in the *Phase explorer.*

Choose *'Gravity Loading'* from the *'Calculation type'* combo box of the *'General'* tabsheet.

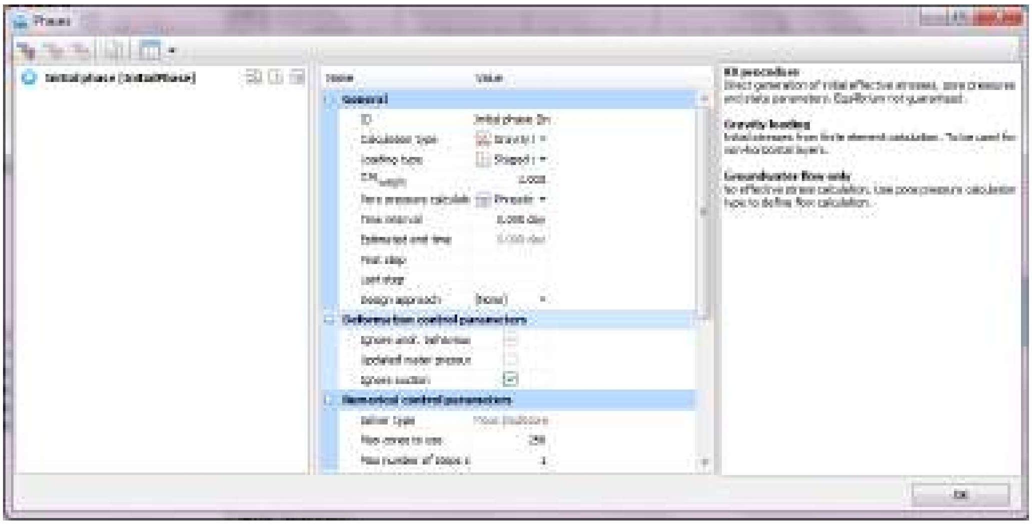


Figure 16: The *Phases* window for Initial phase.

The *Staged construction* option is available as Loading type.

The *Phreatic* option is selected by default as the *Pore pressure calculation type*.

Hint: As a rule, one should use The '*K_0-procedure*' only in cases with a horizontal surface and with all soil layers and phreatic levels parallel to the surface. For all other cases, *Gravity Loading* should be used.

- In the *Model explorer* expand the *Model conditions* subtree.
- Expand the *Deformations* subtree. Note that the *Use default fixities* box is checked. By default, a full fixity is generated at the base of the geometry, whereas roller supports are assigned to the vertical boundaries (u_x = 0; u_y = free).
- Expand the *Water* subtree. The water level generated according to the *Head value* assigned to boreholes in the *Modify soil layer* window (BoreholeWaterLevel_1) is automatically assigned to *GlobalWaterLevel* (Figure 17).

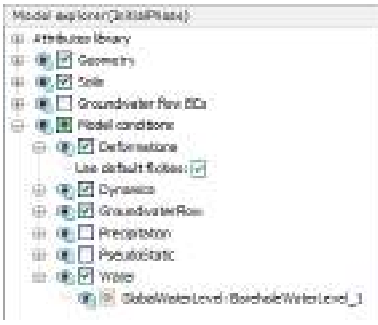


Figure 17: The *Deformations* and *Water* subtrees in the *Model explorer.*

Phase 1 (safety analysis):

In order to simulate slope stability analysis, a safety analysis is required. In order to define the calculation phase follow these steps:

Click the *Add phase* button in the *Phases explorer*. A new phase, named *Phase_1* will be added in the *Phases explorer.*

- Double-click *Phase_1* to open the *Phases* window.
- In the *ID* box of the *General* subtree, write (optionally) an appropriate name for the new phase (for example “safety analysis”.
- The current phase starts from the *Initial phase*, which contains the initial stress state. The calculation type is *Safety,* Loading type Incremental multiplier with Msf 0.1 (see **Error! Reference source not found.**).
- Check list *Ignore suction* in Deformation control parameters.

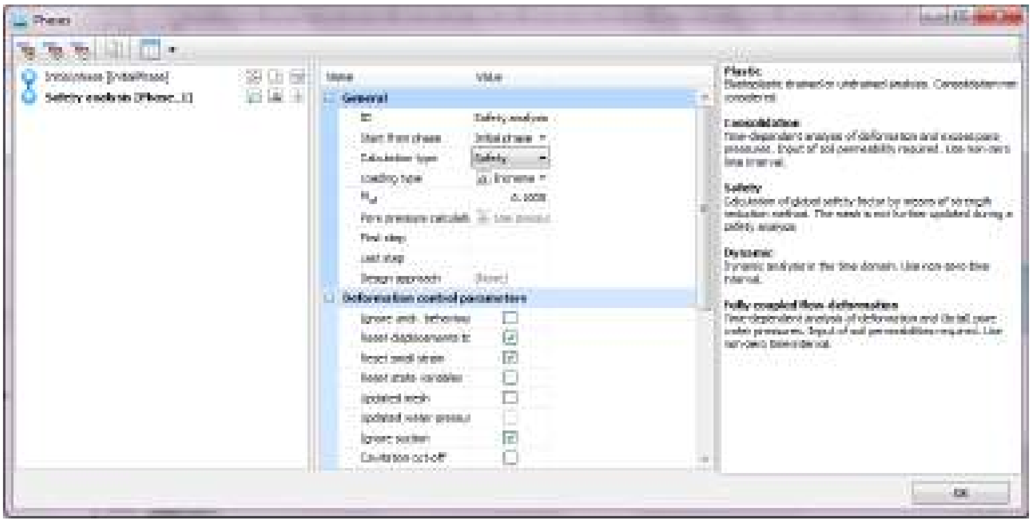


Figure 18: Parameters tabsheet of the first calculation phase

Execution of calculation:

All calculation phases (two phases in this case) are marked for calculation (indicated by a blue arrow). The execution order is controlled by the *Start from phase* parameter.

To find the safety factor of the slope, ΣMsf -displacement curves should be generated in this exercise.

Click on the *'Select points for curves'* button in the toolbar. This will result in a plot of the mesh, showing all generated nodes. Click on the node and the stress point to be inspected.

For the selection of a node it is sometimes necessary to use the zoom option. After selection of the node it will be indicated as white colour at node and stress point that selected. The name of selected node called *'A'* points and the name of selected stress point called *'K'* point. Press the *'Update'* button to proceed to calculations (see Figure 19).

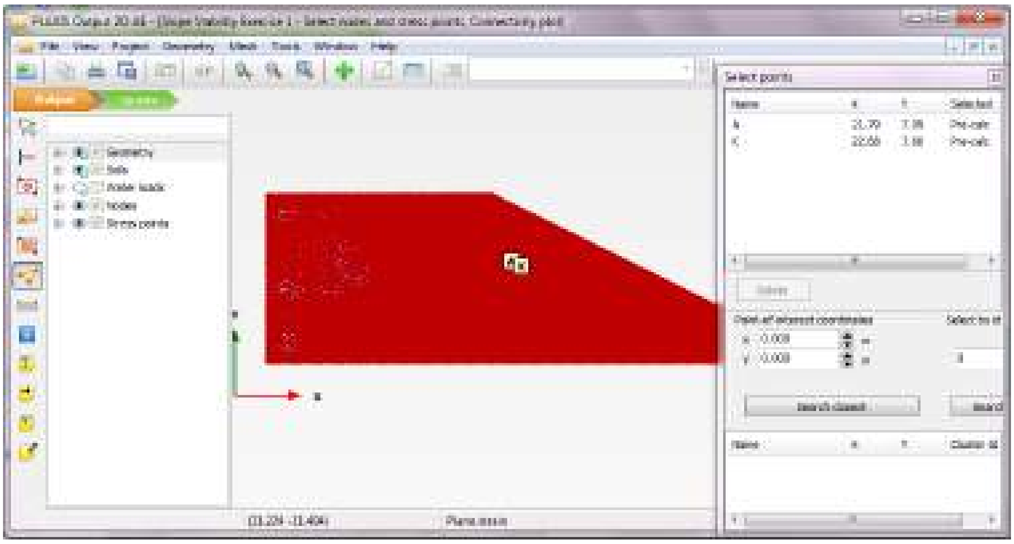


Figure 19: Node and stress point selected

Click the *Calculate* button to start the calculation process. Ignore the warning that no nodes and stress points have been selected for the curves. During the execution of a calculation, a window appears which gives information about the progress of the actual calculation phase (Figure 20).

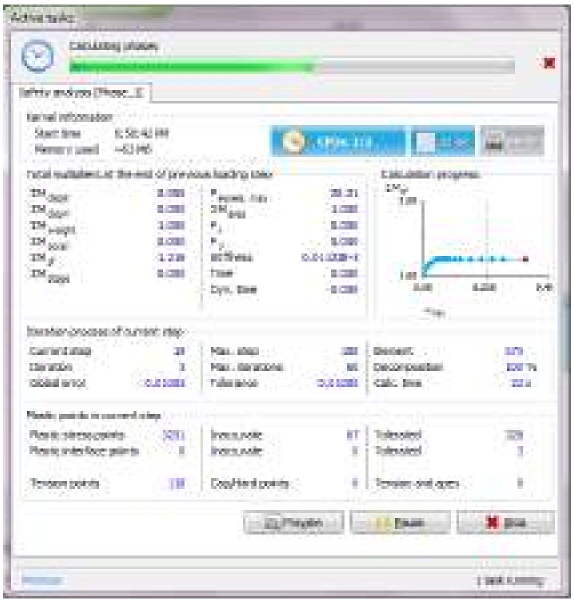


Figure 20: Active task window displaying the calculation progress.

The information, which is continuously updated, shows the calculation progress, the current step number, the global error in the current iteration and the number of plastic points in the current calculation step. It will take a few seconds to perform the calculation. When a calculation ends, the window is closed and focus is returned to the main window.

The phase list in the *Phases explorer* is updated. A successfully calculated phase is indicated by a check mark inside a green circle.

Save the project before viewing results.

Viewing calculation results:

Press the *'View calculation'* button that will start the output program.

Check the various types of output, such as the deformed mesh, displacement contours, effective (principal) stresses, incremental strain, etc. These can be found from the *'Deformations'* and *'Stresses'* menus. Figure 21 illustrates incremental deviatoric strain $\Delta\gamma$ at the end of safety phase that indicates the failure mechanism of the slope.

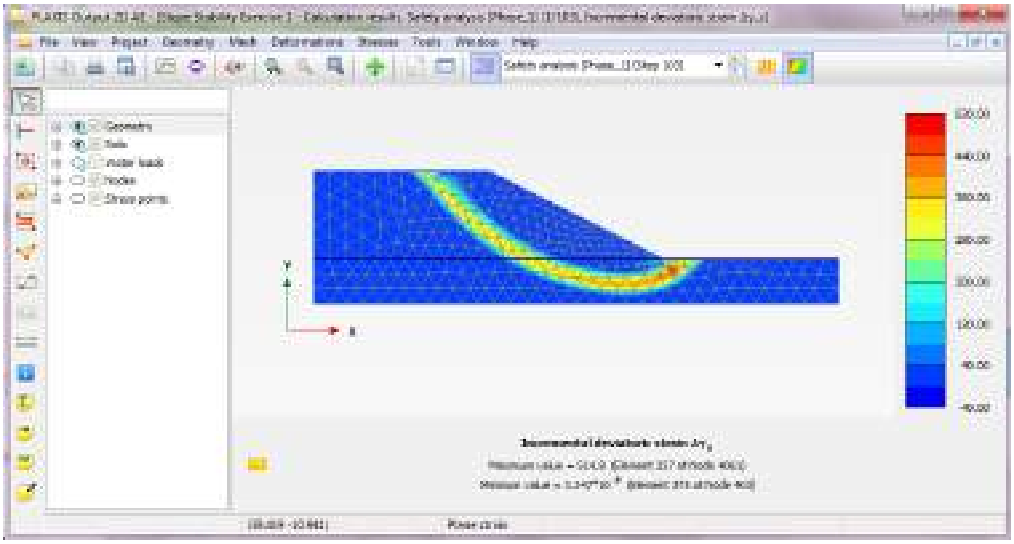


Figure 21: Incremental deviatoric strain $\Delta\gamma_s$ at the end of safety phase

THE SAFETY FACTOR

The safety factor can be obtained from edit phases button at the Phases explorer. The reach values of ΣMsf – Reached safety factor represent the safety factor, provided that this value is indeed more or less constant during the previous few steps.

The best way to evaluate the safety factor is to plot a curve in which the parameter *ΣMsf* is plotted against the displacement of a certain node. Although the displacements are not relevant, they indicate whether or not a failure mechanism has developed.

In order to evaluate the safety factor in this exercise, follow these steps:

Start the curves program by clicking on the *'Curves manager'* button in the output program.

- Create a new chart by clicking the New... button and select the appropriate problem in the file requester.
- The *'Curve Generation'* window as indicated below (see Figure 22) will appear.
- In the *Curve generation* window, select Point A for the *x*-axis. Select *Deformations* → *Total displacements* → *|u|* (absolute)
- For the *y*-axis, select *Project* and then select *Multipliers* → *ΣMsf*. The safety phases are considered in the chart. As a result, the curve of Figure 23 appears.

The maximum displacements plotted are not relevant. If the mouse cursor moves over a point on the curve, a box showing the exact value can be obtained.

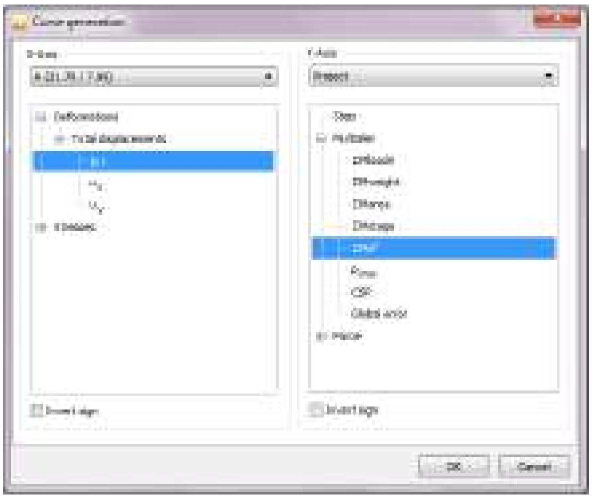


Figure 22: Curve generation

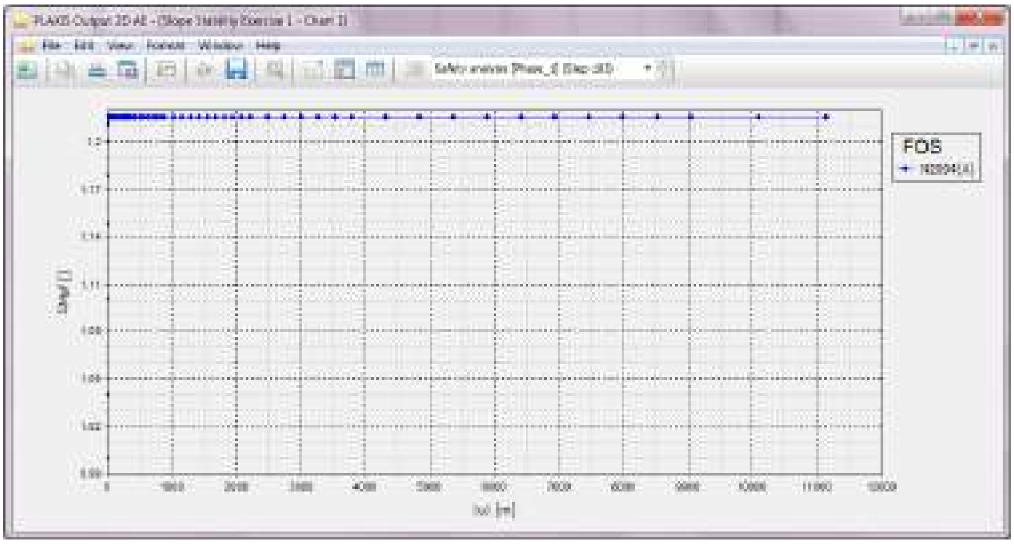
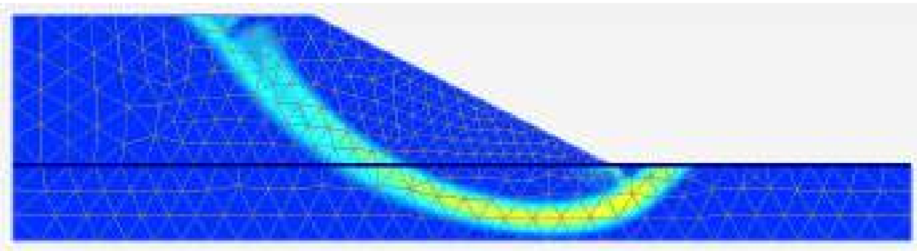


Figure 23: Evaluation of safety factor

EXERCISE 2: Slope stability analysis (*partially saturated slopes*)



COMPUTIONAL GEOTECHNICS

INTRODUCTION

When unsaturated soils are considered and the effect of negative pore water pressure or suction is taken into account. This condition uses Bishop's definition of stress instead of Terzaghi's stress and is suitable for calculating unsaturated response of soils and for performing fully coupled hydro-mechanical behaviour of soils.

Terzaghi's principle effective stress theory is given by: $\sigma' = \sigma - p_w$

Bishop's principle effective stress theory is given by: $\sigma' = (\sigma - p_a) + \chi(p_a - p_w)$

Where σ' is effective stress, σ is total stress, p_w is pore water pressure, the term ($p_a - p_w$) is called matric suction and χ is the matric suction coefficient and varies from 0 to 1 covering the range from dry to fully saturated conditions. By assuming that the pore air pressure p_a is constant and is small enough to be neglected ($p_a \approx 0$) and matric suction coefficient (χ) is can approximately be replaced by the effective degree of saturation, consequently, the effective stress equation can be simplified to: $\sigma' = \sigma - S_e p_w$

where S_e is the effective degree of saturation. This modified principle effective stresses is used when the suction is taken into account.

AIMS:

- To understand the effect of negative pore water pressure or suction in a slope stability analysis.

SCHEME OF OPERATIONS:

Geometry input

- Use previous input file without change of:
    - Project properties
    - Defining of soil stratigraphy
    - Material data sets
    - Defining of slope geometry
    - Mesh generation
- Save as new data file

PERFORMING CALCULATIONS

- Use previous initial conditions
- Phase 1 (safety analysis)
- Execution of calculation
- Viewing calculation results

THE SAFETY FACTOR

- The safety factor
- Inspect the safety factor and compare with slope stability analysis without considering suction.

GEOMETRY INPUT - USE PREVIOUS INPUT FILE

Start PLAXIS by clicking on the icon of the Input program.

- Select the existing project file from the last exercise (Slope Exercise 1).
- From the *'File'* menu select *'Save As'* and save the existing project under a new file name (e.g. 'Slope Exercise 2')

PERFORMING CALCULATIONS - Re-run existing calculation-list

- Uncheck list *Ignore suction* in option of Deformation control parameters in the *Phases explorer* (Figure 24).

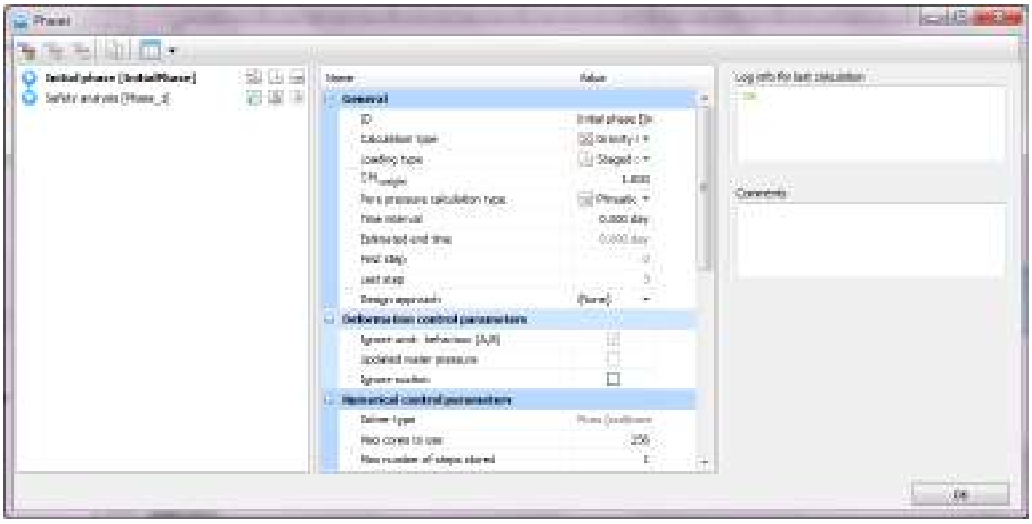


Figure 24: Uncheck *Ignore suction* in *the Phases explorer*

Click the *'Calculate'* button. This will start the calculation process (Figure 25).

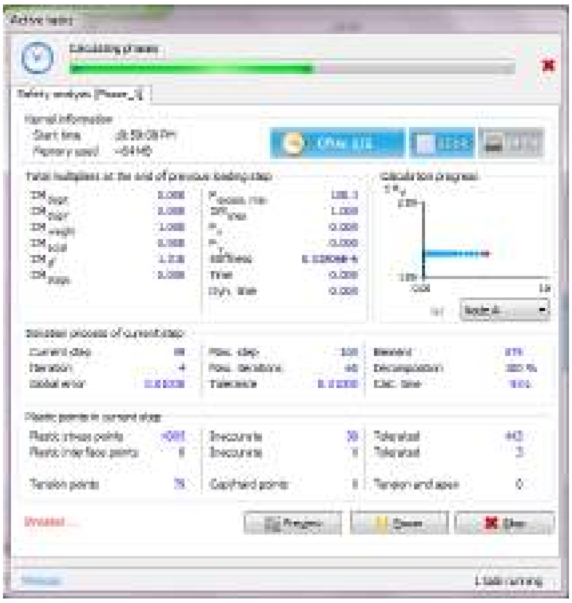


Figure 25: Active task window displaying the calculation progress.

The information, which is continuously updated, shows the calculation progress, the current step number, the global error in the current iteration and the number of plastic points in the current calculation step. It will take a few seconds to perform the calculation. When a calculation ends, the window is closed and focus is returned to the main window.

The phase list in the *Phases explorer* is updated. A successfully calculated phase is indicated by a check mark inside a green circle.

Save the project before viewing results.

Viewing calculation results:

Press the *'View calculation'* button that will start the output program.

- **Error! Reference source not found.** illustrates incremental deviatoric strain $\Delta\gamma$ at the end of safety phase that indicates the failure mechanism of the slope. Figure 27 and Figure 28 show the suction profile of the slope. Figure 29 and Figure 30 show the profile of degree of saturation of the slope.

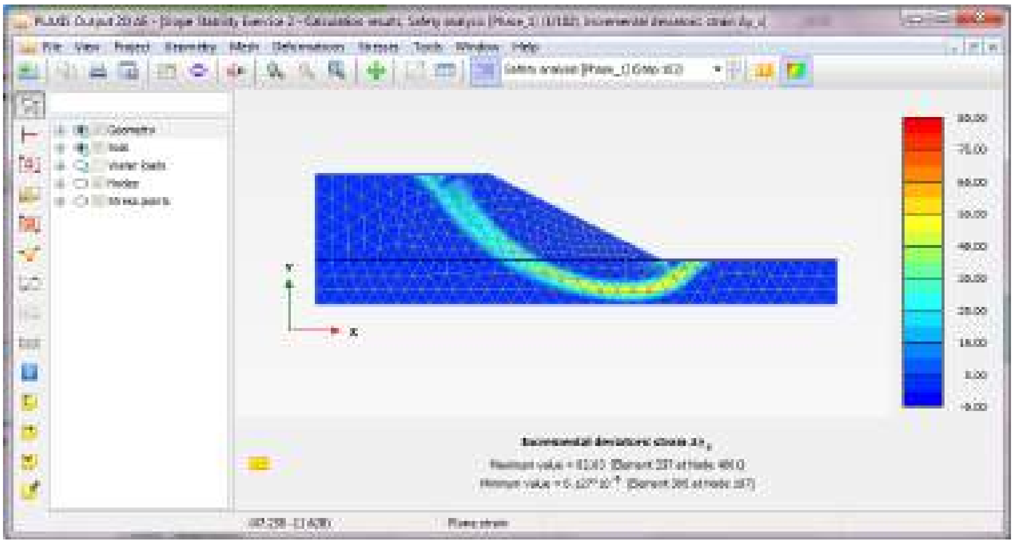


Figure 26: Incremental deviatoric strain $\Delta\gamma_s$ at the end of safety phase

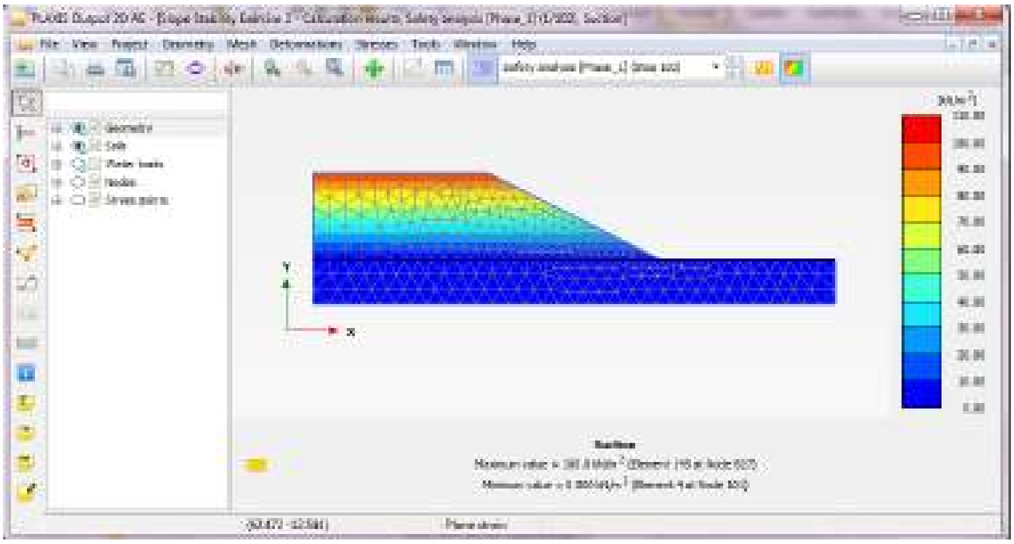


Figure 27: Suction

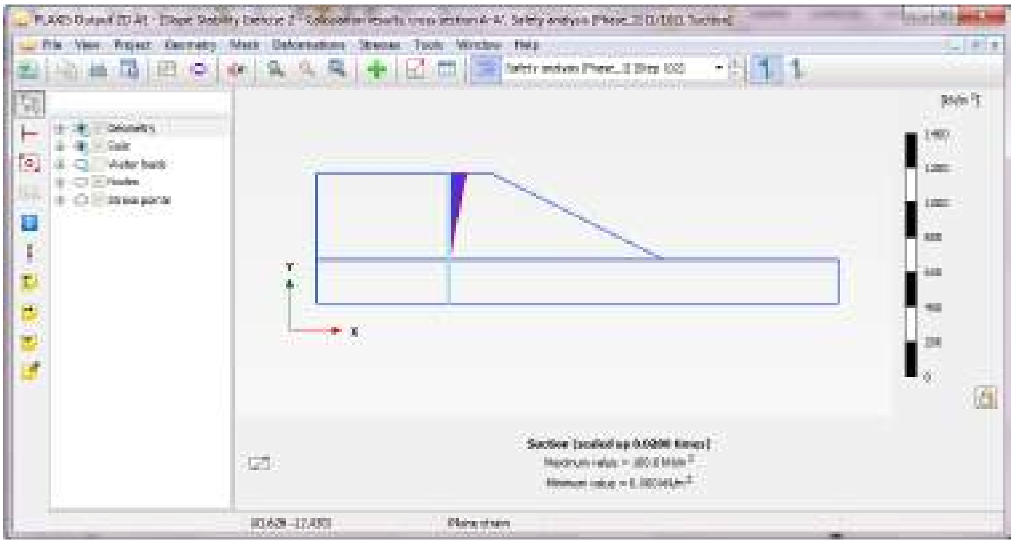


Figure 28: Cross section of suction

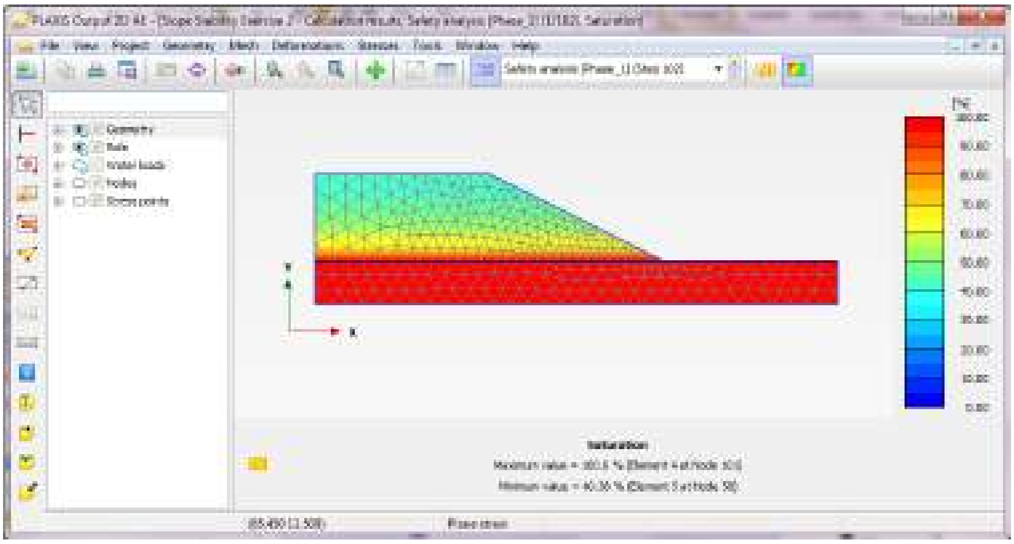


Figure 29: Degree of saturation

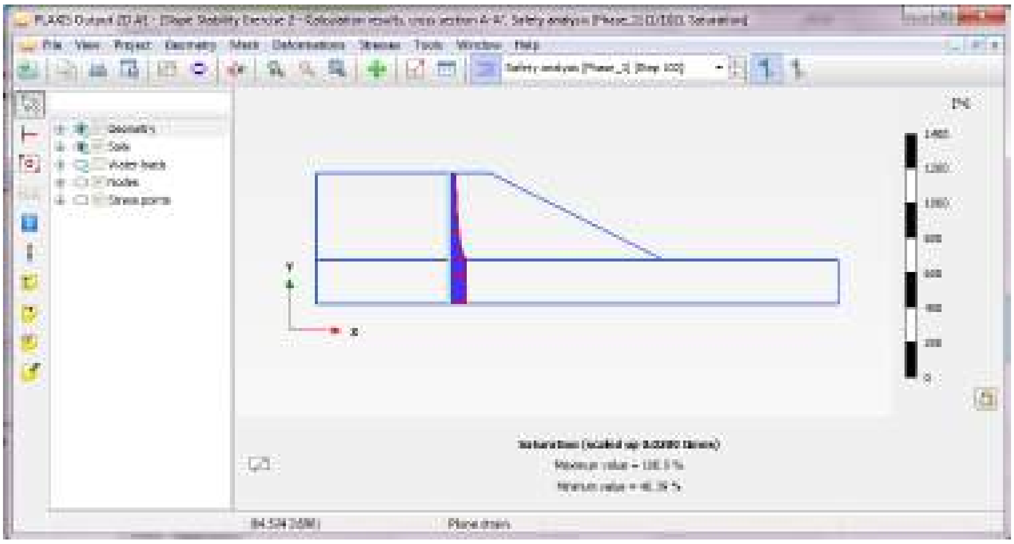


Figure 30: Cross section of degree of saturation

THE SAFETY FACTOR

In order to evaluate the safety factor in this exercise, use the same procedure as used in the previous exercise. Figure 31 shows the *ΣMsf* against the displacement curve.

To compare the results obtained with both calculation modes plot both ΣMsf -|u| curves in one chart.

In order to make a comparison of result in the curve generated, follow these steps:

- From the *Format* menu select *Settings* and Figure 32 will appear.
- Select *Add curve* and select *From another project (all)* and then select and open previous project file *Slope Stability Exercise 1*. Use the same steps to generate the curve and Figure 33 will appear. Use the manual scaling in the *Chart* tab to change the scale of the chart.

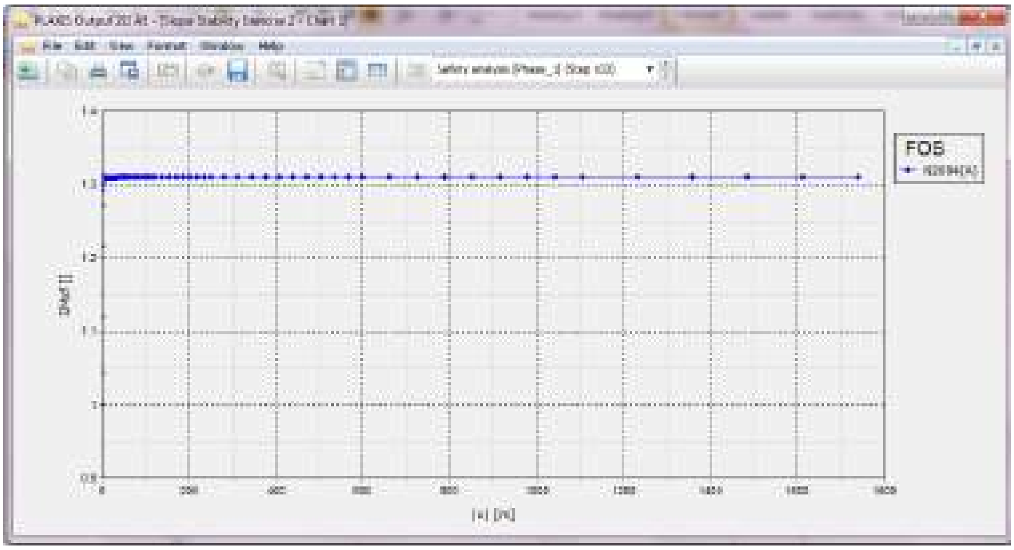


Figure 31: Evaluation of safety factor

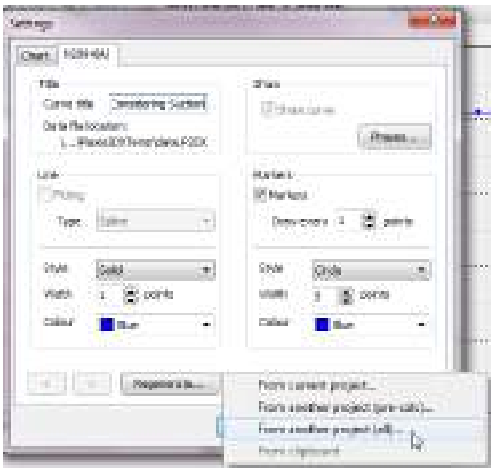


Figure 32: Format setting window

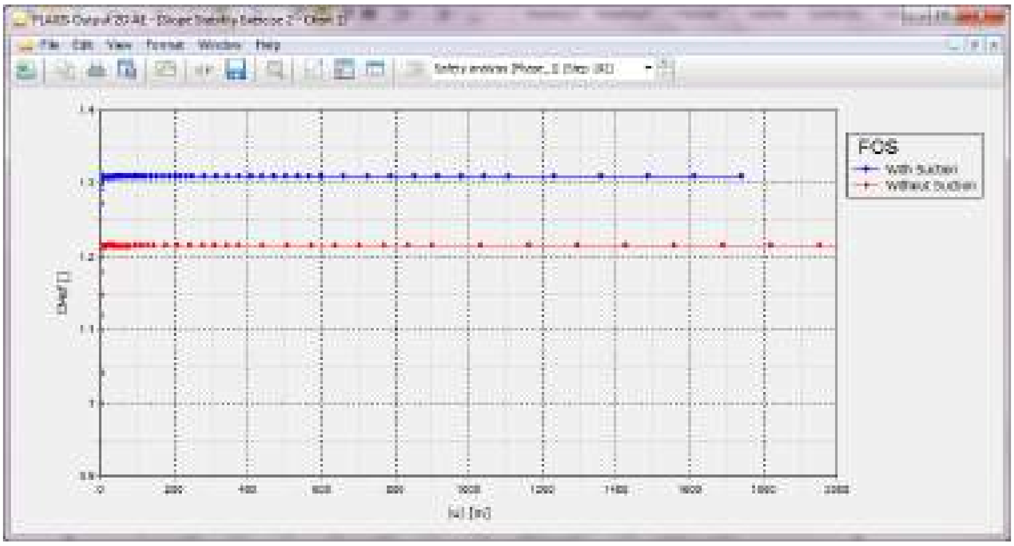
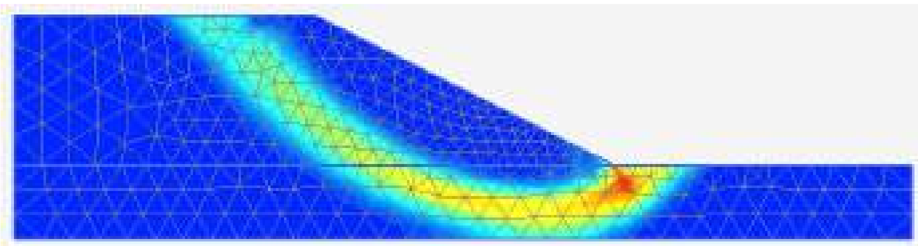


Figure 33: Safety factors with two different Calculation modes

EXERCISE 3: Slope stability analysis subjected to Rainfall Infiltration (*partially saturated slopes*)



COMPUTIONAL GEOTECHNICS

INTRODUCTION

Slope stability analysis of unsaturated soil subjected to rainfall infiltration requires to compute simultaneously the deformations and groundwater flow with time dependent boundary conditions (fully coupled flow-deformation analysis). The calculation will be performed that considering the suction uses Bishop's definition of stress instead of Terzaghi's stress and Biot's theory of consolidation.

AIMS:

- To understand the change of negative pore-water pressure or suction during rainfall infiltration. This change of suction will influence the stability of the slope.

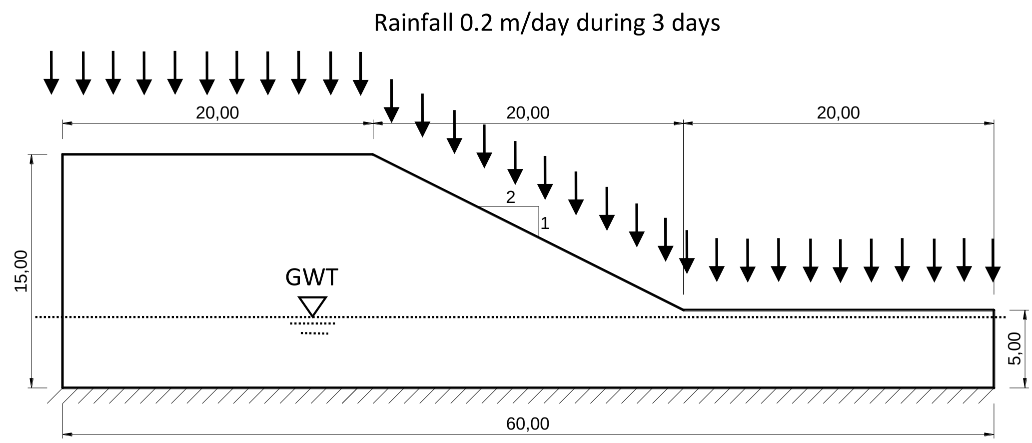


Figure 34: Scheme of the exercise (Griffiths and Lane, 1999)

SCHEME OF OPERATIONS:

Geometry input

- Use previous input file with change of:
    - Project properties
        - To reduce the computational time, 6-noded elements are used in this exercise. But nevertheless, 15-noded elements would give more accurate results for safety analysis.
    - Mesh generation
- Save as new data file

PERFORMING CALCULATIONS

- Use previous initial conditions
- Phase *fully coupled flow-deformation*.
- Phase *safety analysis*.
- Execution of calculation
- Viewing calculation results

THE SAFETY FACTOR

- Inspect the safety factor during rainfall infiltration.

GEOMETRY INPUT - USE PREVIOUS INPUT FILE

Start PLAXIS by clicking on the icon of the Input program.

- Select the existing project file from the last exercise (Slope Exercise 2).
- From the *'File'* menu select *'Save As'* and save the existing project under a new file name (e.g. 'Slope Exercise 3')

GEOMETRY INPUT

Project properties

- Change the elements by selecting the item *'Project properties'* from the *'File'* menu. Select the "6-Node" for Elements in Model tabsheet. (see Figure 35).

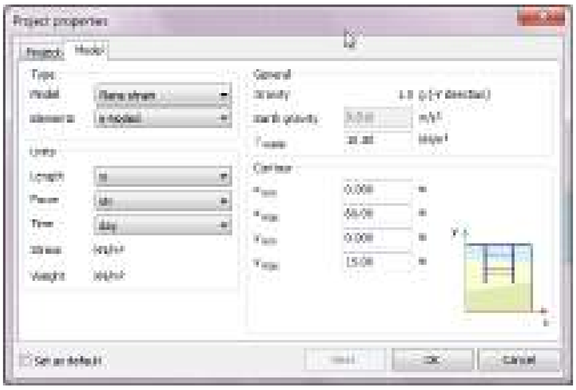


Figure 35: Elements: 6-Noded

Mesh generation

Use the procedure described before to generate the mesh after the change to 6-noded elements.

PERFORMING CALCULATIONS

- Delete old calculation phases.

Calculation Phases

This analysis consists of six phases. From the first phase until third phase a *Fully coupled flow-deformation analysis* is performed. During rainfall infiltration deformations and groundwater flow are computed simultaneously with time dependent boundary conditions (fully coupled flow-deformation analysis). From the fourth phase until seventh phase, safety analyses are performed.

In order to perform all the calculation phases, follow these steps:

Phase 1 (1st day of Rainfall):

Click the *Add phase* button in the *Phases explorer*. A new phase, named *Phase_1* will be added in the *Phases explorer.*

- Double-click *Phase_1* to open the *Phases* window.
- In the *ID* box of the *General* subtree, write (optionally) an appropriate name for the new phase (for example “1st day of rainfall”.
- The current phase starts from the *Initial phase*, which contains the initial stress state. The calculation type is *fully coupled flow-deformation* and time interval is 1 day (see **Error! Reference source not found.**).

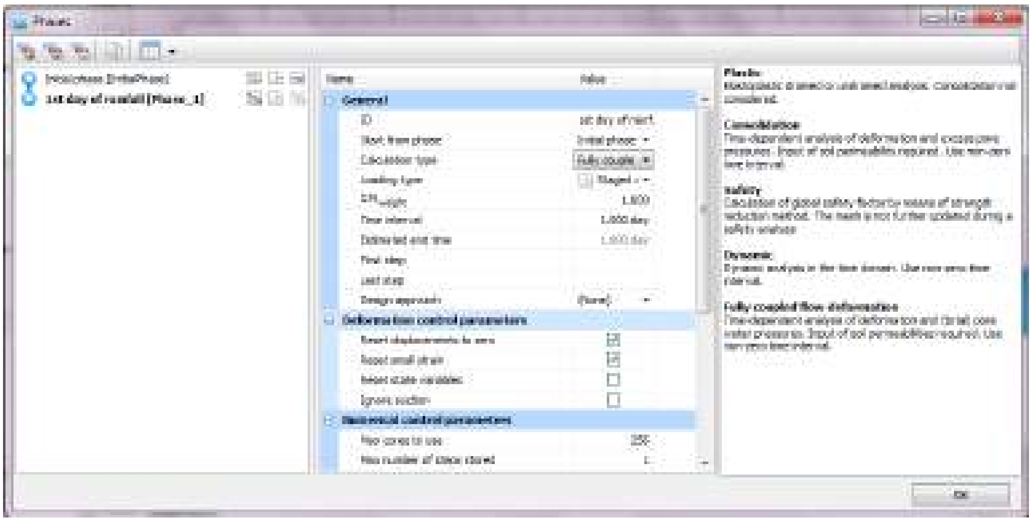


Figure 36: Phases explorer – Calculation type: fully coupled flow-deformation.

- Uncheck list *Ignore suction* in Deformation control parameters.
- In the *Model explorer* expand the *Model conditions* subtree.
- Expand the *Precipitation* subtree. Fill the parameters q (infiltration) = 0.2 m/day, ψ_{max} = 0.1 m, and ψ_{min} = -1.0 m (see **Error! Reference source not found.**).

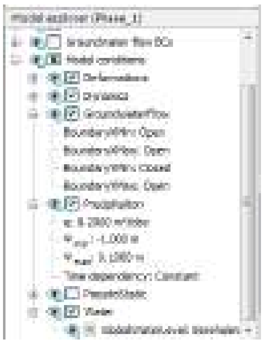


Figure 37: Model explorer – Model condition – Precipitation subtree

Phase 2 (2nd day of Rainfall):

Click the *Add phase* button in the *Phases explorer*. A new phase, named *Phase_2* will be added in the *Phases explorer.*

- Double-click *Phase_2* to open the *Phases* window.
- In the *ID* box of the *General* subtree, write (optionally) an appropriate name for the new phase (for example "2nd day of rainfall".
- Use the same procedure as used in the Phase 1 for the input of the *General* in *Phases explorer* but without select *Reset displacement to zero*.

Phase 3 (3rd day of Rainfall):

Click the *Add phase* button in the *Phases explorer*. A new phase, named *Phase_3* will be added in the *Phases explorer.*

- Double-click *Phase_3* to open the *Phases* window.
- In the *ID* box of the *General* subtree, write (optionally) an appropriate name for the new phase (for example "3rd day of rainfall".
- Use the same procedure as used in the Phase 2 for the input of the *General* in *Phases explorer.*

The calculation-list after Phase 3 from Slope Exercise 3 appears, as indicated below (see Figure 38).

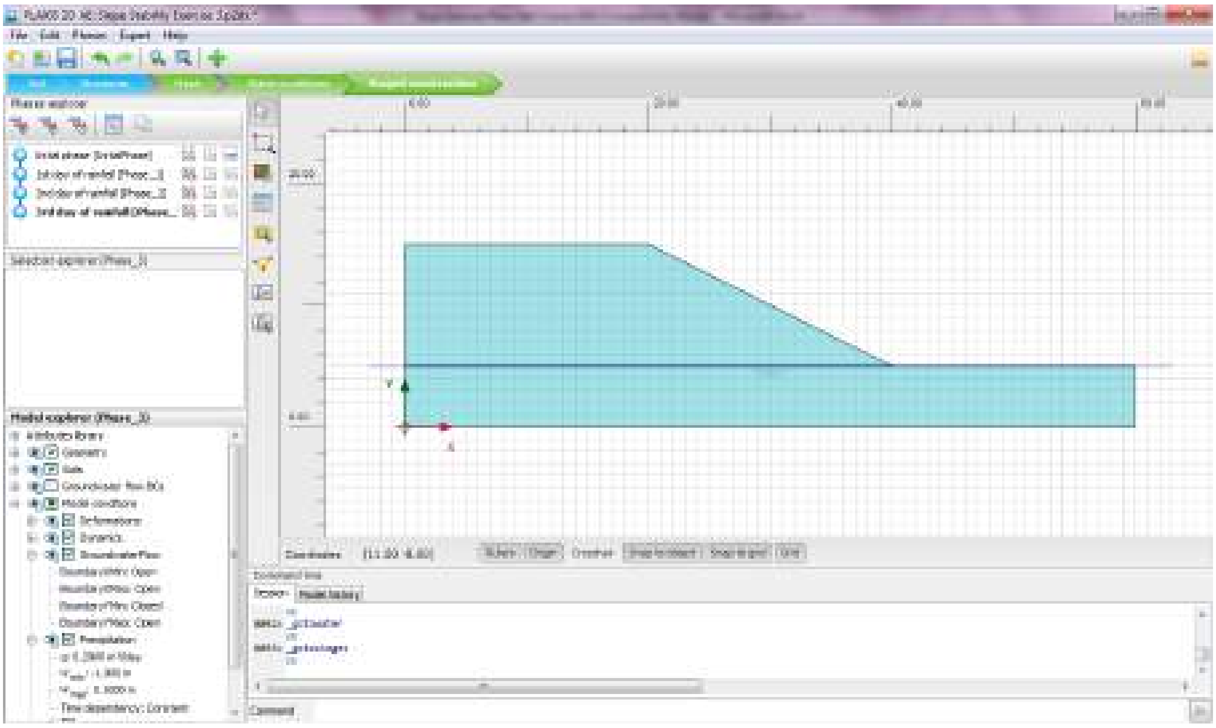


Figure 38: The calculation-list after third calculation phase

Phase 4 to 7 (safety analysis):

- In Phase 4 to 7, stability calculations are defined for the phases 0 to 3 respectively. Initial factor of safety (FOS), FOS after 1st day of rainfall, FOS after 2nd day of rainfall and FOS after 3rd day of rainfall will be calculated in Phase 4 to 7 respectively.

Click the *Add phase* button in the *Phases explorer*. A new phase, named *Phase_4* will be added in the *Phases explorer.*

- Double-click *Phase_4* to open the *Phases* window.
- In the *ID* box of the *General* subtree, write (optionally) an appropriate name for the new phase (for example "Initial factor of safety (FOS)".
- The current phase starts from the *Initial phase*, which contains the initial stress state. The calculation type is *Safety,* Loading type Incremental multiplier with Msf 0.1.
- The procedure repeat for Phase 5, Phase 6 and Phase 7 for FOS after 1st day of rainfall, FOS after 2nd day of rainfall and FOS after 3rd day of rainfall respectively.

he full calculation-list from Slope Exercise 3 appears, as indicated below (see Figure 39).

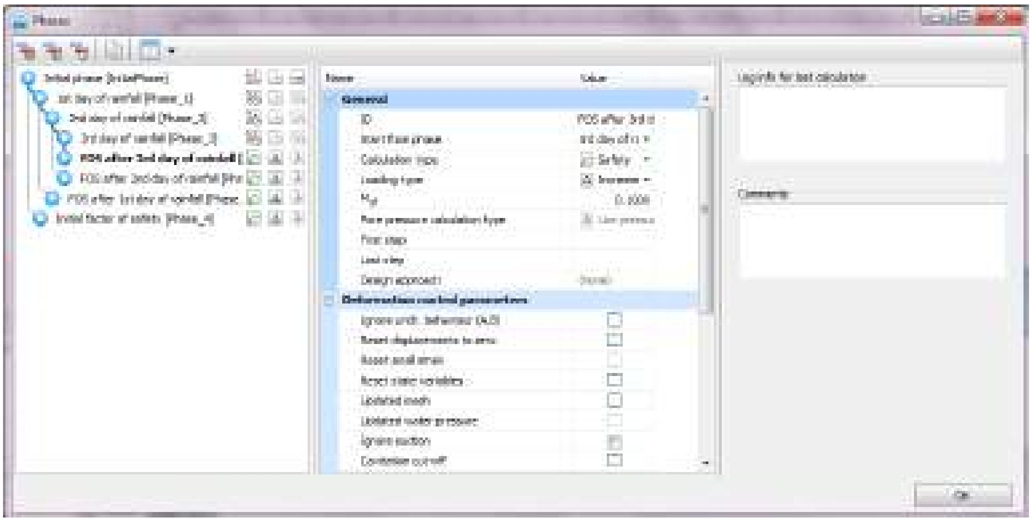


Figure 39: Calculation phase

Viewing calculation results:

Press the *'View calculation'* button that will start the output program.

- Check the various types of output showing the suction, the degree of saturation and the failure mechanism, as presented in Figure 40 to Figure 51.

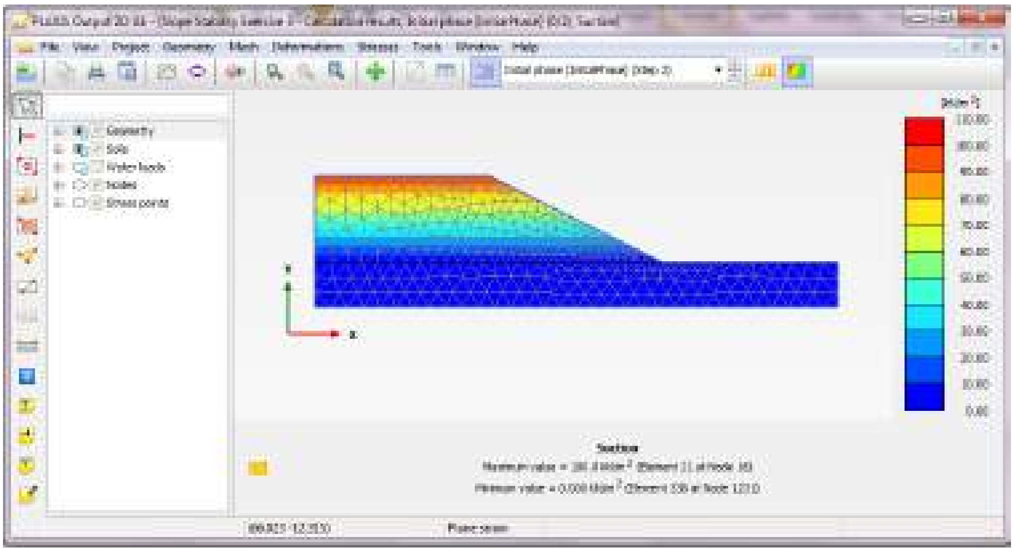


Figure 40: Initial suction

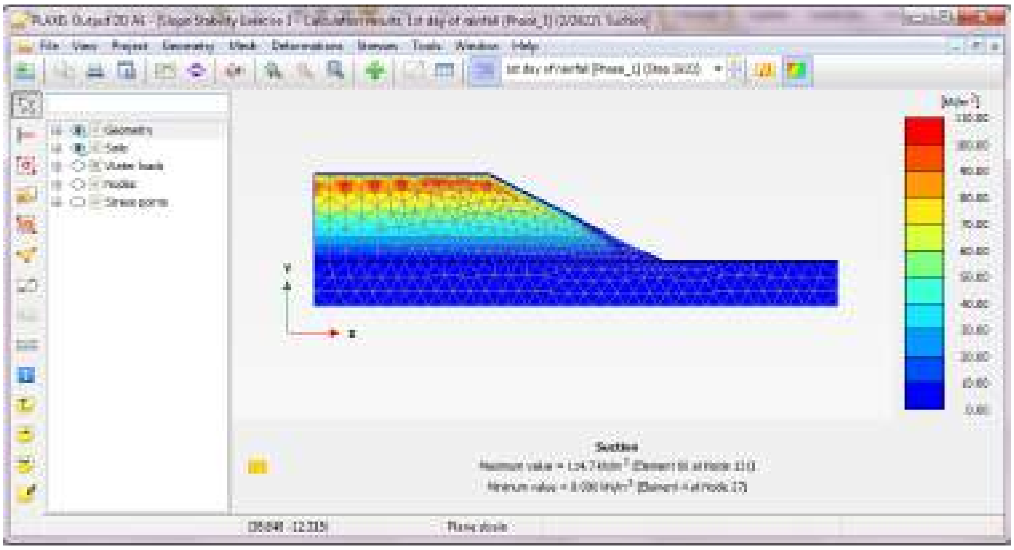


Figure 41: Suction after 1st day of rainfall

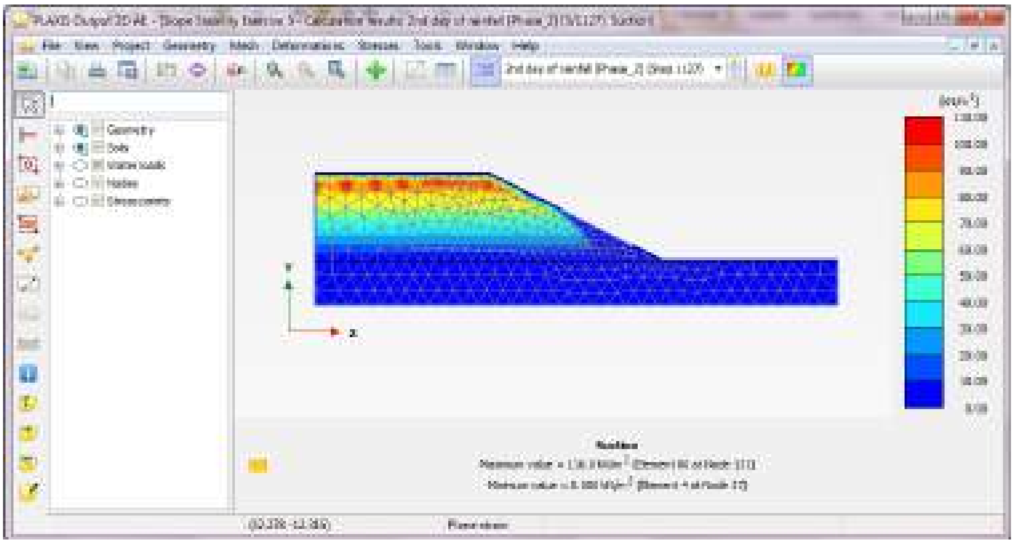


Figure 42: Suction after 2nd day of rainfall

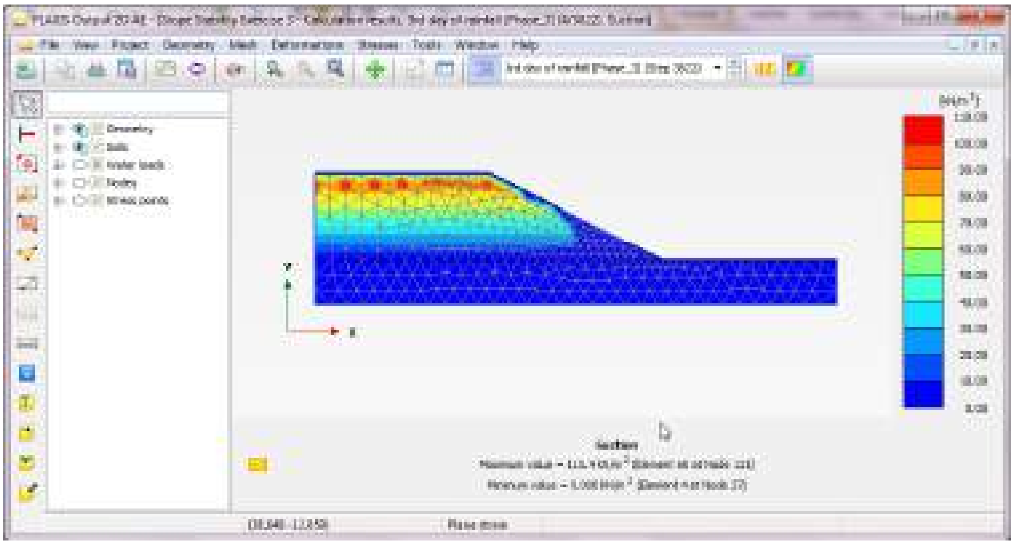


Figure 43: Suction after 3rd day of rainfall

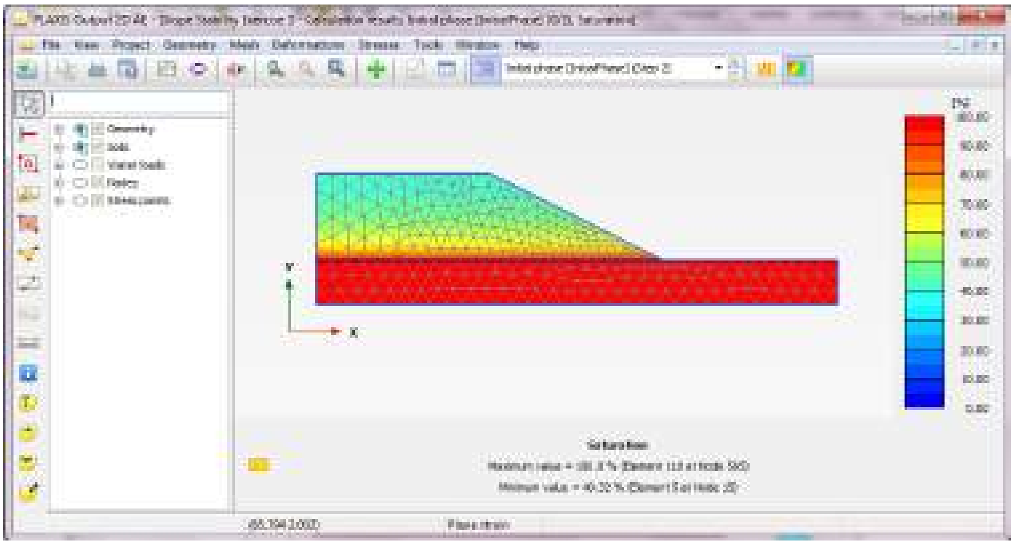


Figure 44: Initial degree of saturation

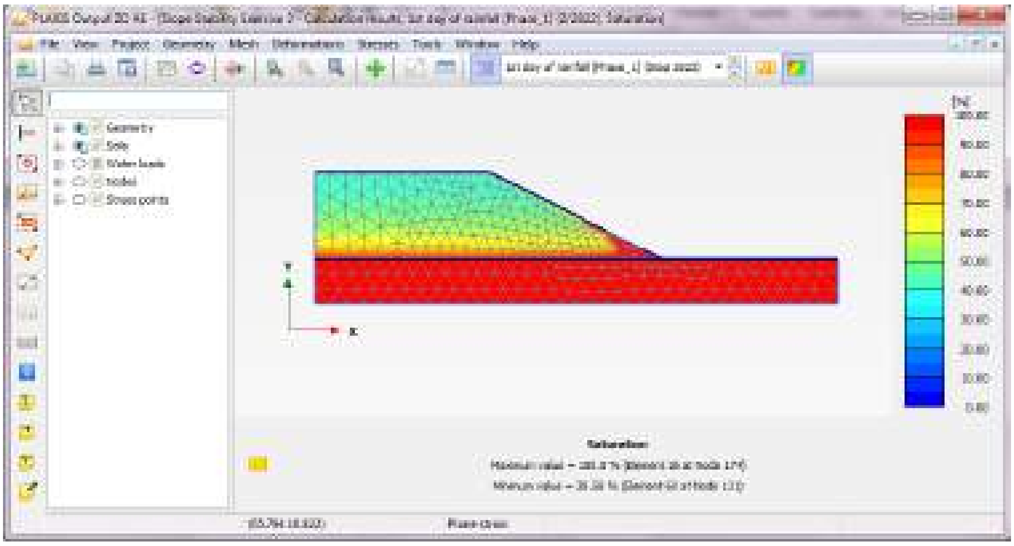


Figure 45: Degree of saturation after 1st day of rainfall

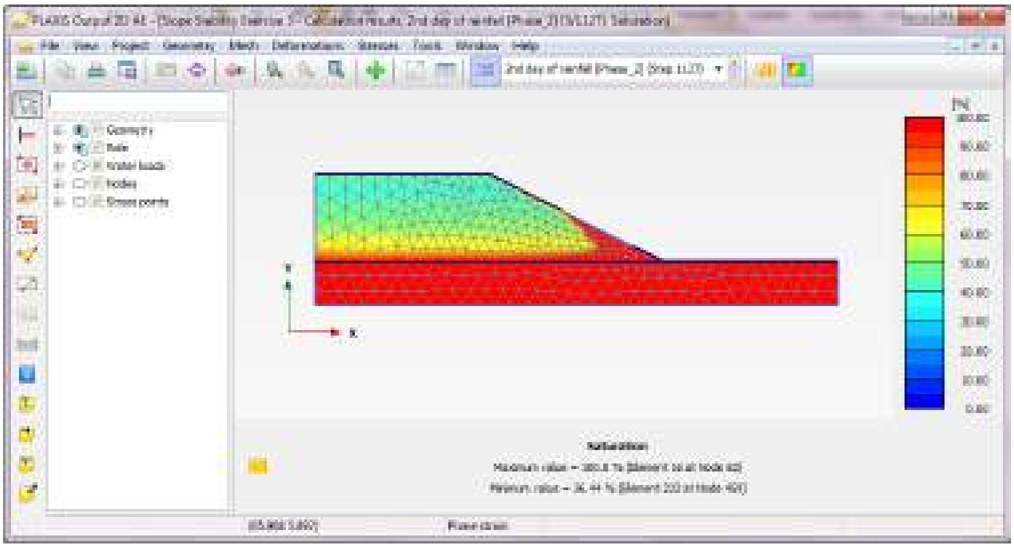


Figure 46: Degree of saturation after 2nd day of rainfall

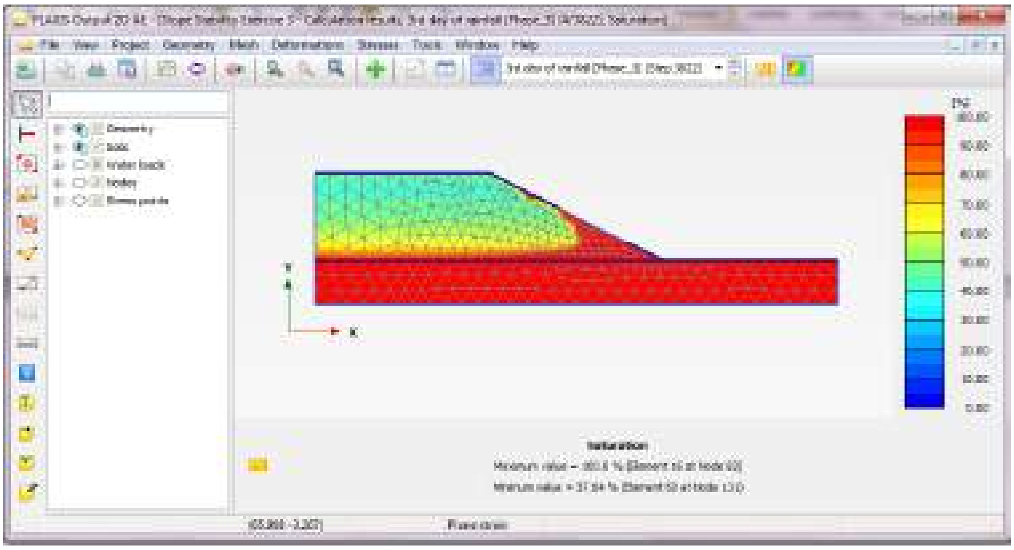


Figure 47: Degree of saturation after 3rd day of rainfall

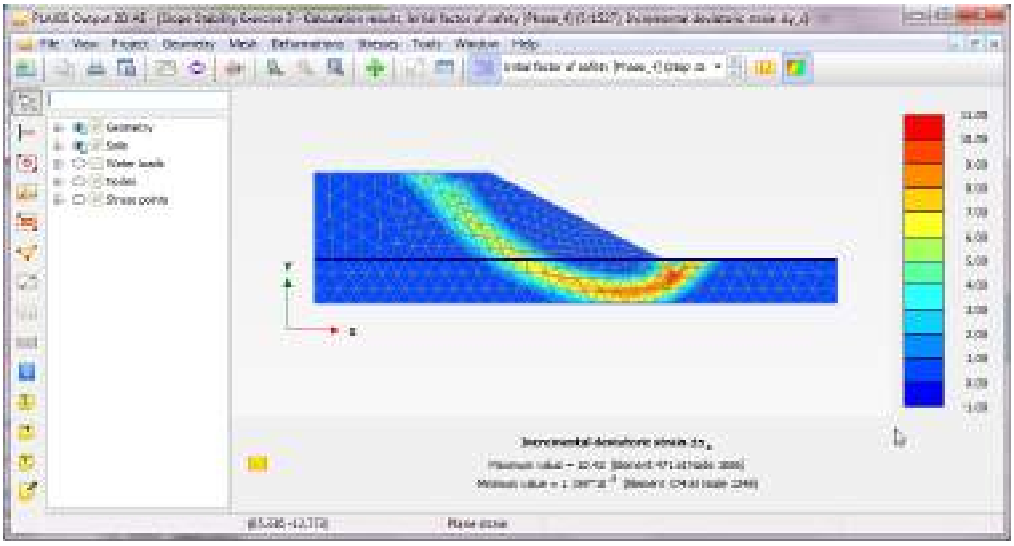


Figure 48: Failure mechanism before rainfall

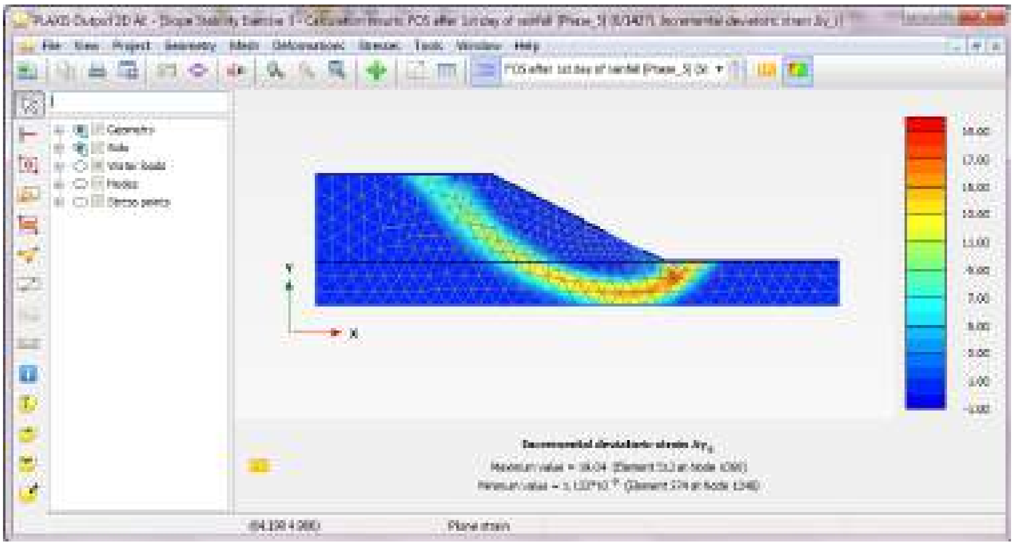


Figure 49: Failure mechanism after 1st day of rainfall

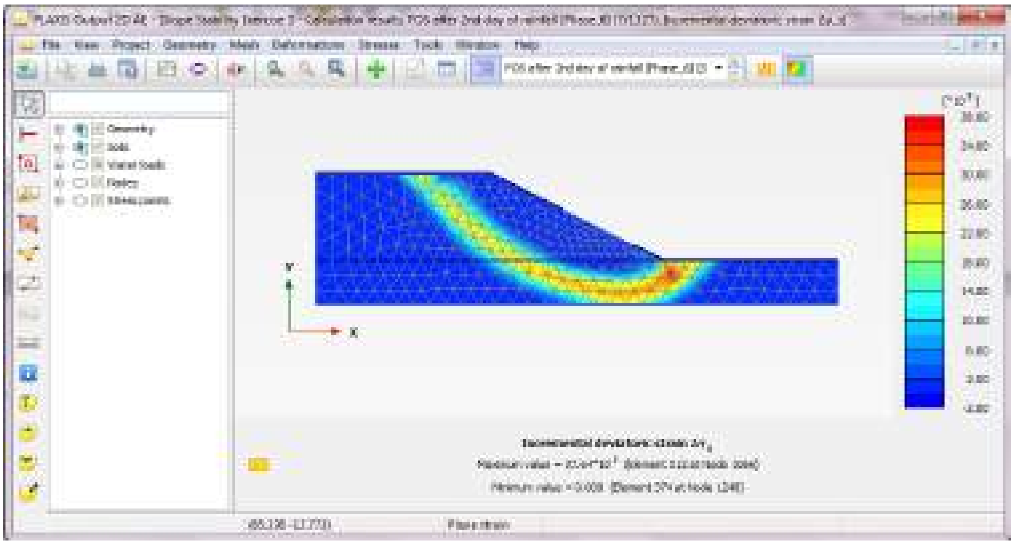


Figure 50: Failure mechanism after 2nd day of rainfall

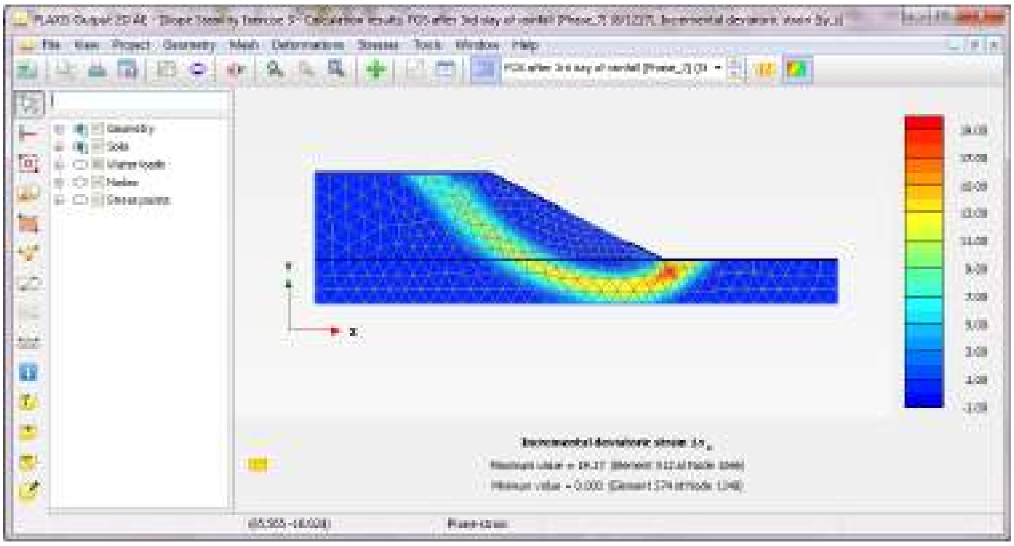


Figure 51: Failure mechanism after 3rd day of rainfall

THE SAFETY FACTOR

In order to evaluate the safety factor in this exercise, use the same steps as described before. Figure 52 show the *ΣMsf* against the displacement curve. The curve shows that the safety factor reduces during rainfall infiltration.

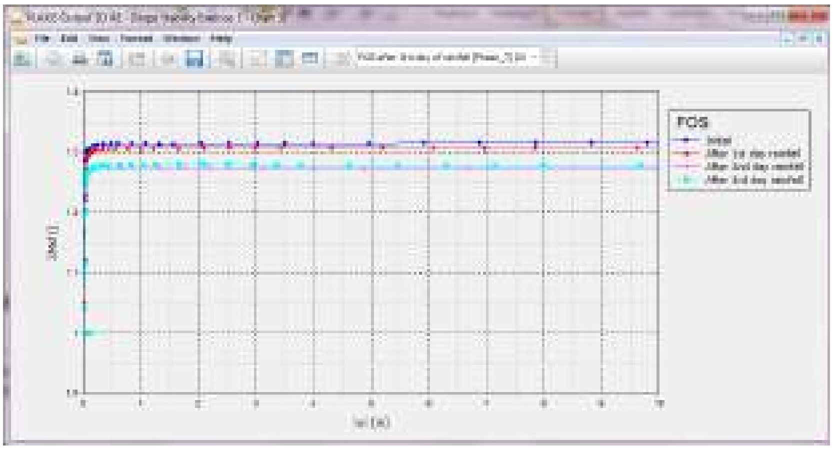


Figure 52: Evaluation of safety factor