# Coupled Stress/Pore Fluid Analysis Of a Stress Test of a Levee on Soft Subsoil

**Camilo Marulanda Escobar, PhD**
**Juan Sebastian Tello, PhD**
**David Leon Vanegas**

# PROBLEM DESCRIPTION

- Objective
  - Evaluate accuracy of current models to assess safety of dykes on soft subsoil
- Background
  - STOWA funded a full scale test for inducing failure by means of excavations and dewatering operations at the toe of the dyke
  - Materials were characterized by means of CPTUs and laboratory tests
- The Benchmark
  - Develop a Coupled Pore-Fluid/Stress FE Models based on Laboratory Tests and Assess Accuracy

# PAPER QUESTIONS

1. Prediction of failure
   a) When did failure occur? (based on observations)
   b) Based on monitoring data, will failure be predicted best based on drained or undrained conditions?
   c) What is the shape of the failure surface?
   d) Which elements of the model are most affected by uncertainties?
2. Pre-failure response
   a) Which material model best fits the lab tests?
   b) How did the soil respond? Is the model accurate?
      - Plot U1 vs t @ two inclinometer locations
      - Plot POR vs t @ 3 piezometers
   c) Are lab tests representative of material in the field?
   d) Can the prediction be refined by back-analysis?

***Method I***

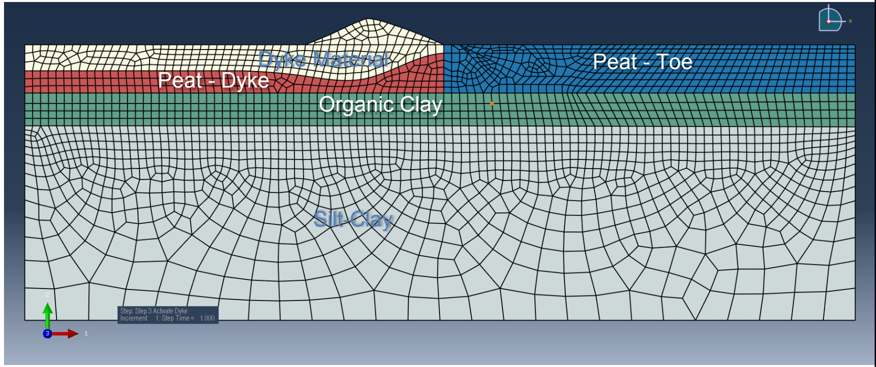
*Lab Tests & Only*

***Method II***

*Include back analysis of monitoring data*

# MODEL DESCRIPTION

- Consists of a 2D geometry of the dyke section
  - Plane strain
  - Establishes an initial geostatic stress state with steady-state fluid flow
  - Consists of 15 additional transient soils analysis steps for
    - Excavations
    - Dewatering and Watering Ops
    - Consolidation times




Step Manager

*Simulation Steps*

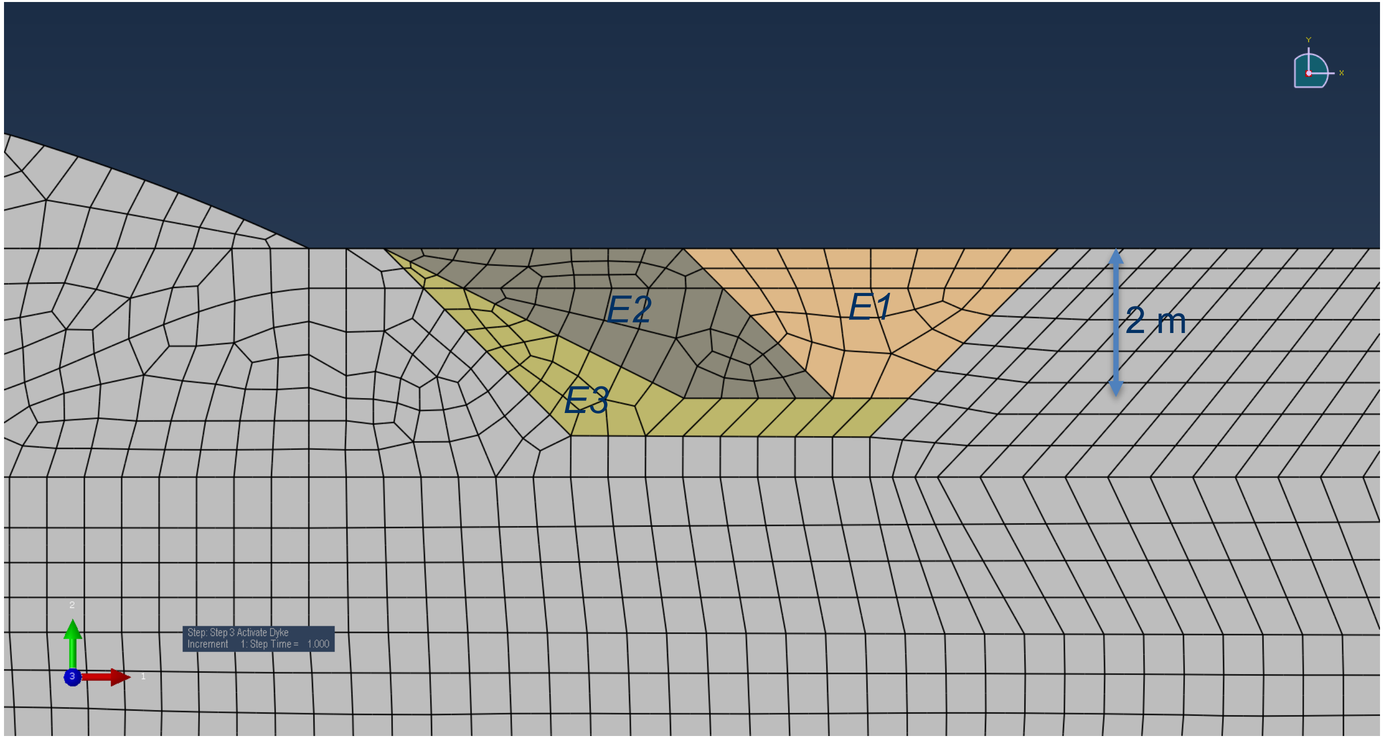
| Name | Procedure | Nlgeom | Time |
|---|---|---|---|
| Initial | (Initial) | N/A | N/A |
| Initial Step | Geostatic | ON | 1 |
| Excavation 1 | Soils (Steady-State) | ON | 28800 |
| Excav 1 - end | Soils (Transient) | ON | 144000 |
| Dewatering 1 | Soils (Transient) | ON | 10800 |
| Consolidate Dewatering 1 | Soils (Transient) | ON | 86400 |
| Water 1 - start | Soils (Transient) | ON | 7200 |
| Water 1 - end | Soils (Transient) | ON | 324000 |
| Excavation 2 - start | Soils (Steady-State) | ON | 36000 |
| Excavation 2 - end | Soils (Transient) | ON | 140400 |
| Dewatering 2 - start | Soils (Transient) | ON | 14400 |
| Dewater 2 - end | Soils (Transient) | ON | 68400 |
| Water 2 - start | Soils (Transient) | ON | 7200 |
| Water 2 - end | Soils (Transient) | ON | 334800 |
| Excavation 3 - start | Soils (Steady-State) | ON | 36000 |
| Excavation 3 - end | Soils (Transient) | ON | 111600 |
| Dewatering Excavation 3 (150 | Soils (Transient) | ON | 111600 |

# WATERING AND DEWATERING BOUNDARY CONDITIONS

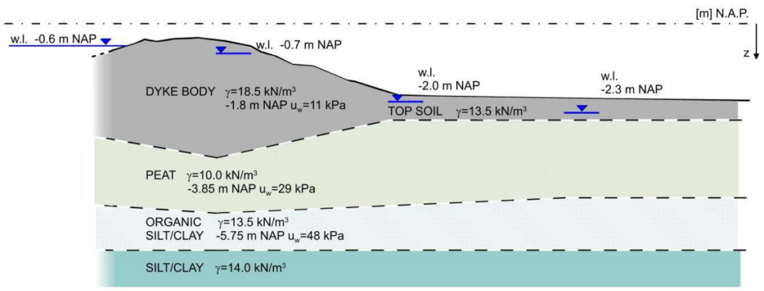
- In order to model a moving water level, two user subroutines are implemented in Fortran
  - Hydrostatic Pressure (DLOAD.f)
  - Pore fluid flow (FLOW.f)
    - Separates the boundary into
      - Free drainage
      - Flow velocity driven by sink pressure

```fortran
      SUBROUTINE FLOW(H,SINK,U,KSTEP,KINC,TIME,NOEL,NPT,COORDS,
     1 JLTYP,SNAME)
C
      INCLUDE 'ABA_PARAM.INC'
C
      DIMENSION TIME(2), COORDS(3)
      CHARACTER*80 SNAME



      Z = COORDS(3)
      GW = 0.001d0 * 9.81

      IF (KSTEP.EQ.1) THEN          ! ----------- Wetting   ***** Not Called ******
         HO = -1.217832             ! Initial Height
         HF = -1.217832             ! Final Height
         TF = 575844                    ! Final Step Time
      ELSEIF (KSTEP.EQ.2 ) THEN     ! ----------- E1
         HO = -1.217832             ! Initial Height
         HF = -1.217832             ! Final Height
         TF = 28800                 ! Final Step Time
      ELSEIF (KSTEP.EQ.3 ) THEN     ! ----------- Consolidate E1
         HO = -1.217832             ! Initial Height
         HF = -1.217832             ! Final Height
         TF = 144000            ! Final Step Time
      ELSEIF (KSTEP.EQ.4 ) THEN     ! ----------- Pump 1
         HO = -1.217832             ! Initial Height
         HF = -2.217832             ! Final Height
         TF = 10800                 ! Final Step Time
      ELSEIF (KSTEP.EQ.5 ) THEN     ! ----------- Consolidate PUMP 1
         HO = -2.217832             ! Initial Height
         HF = -2.217832             ! Final Height
         TF = 86400                 ! Final Step Time
      ELSEIF (KSTEP.EQ.6 ) THEN     ! ----------- Fill 1
         HO = -2.217832             ! Initial Height
         HF = -1.217832             ! Final Height
         TF = 7200              ! Final Step Time
      ELSEIF (KSTEP.EQ.7 ) THEN     ! ----------- Consolidate FILL 1
      ELSEIF (KSTEP.EQ.8 ) THEN     ! ----------- EXCAVATION 2
      ELSEIF (KSTEP.EQ.9 ) THEN     ! ----------- Consolidate E2
      ELSEIF (KSTEP.EQ.10 ) THEN    ! ----------- PUMP 2
      ELSEIF (KSTEP.EQ.11 ) THEN    ! ----------- Consolidate PUMP 2
      ELSEIF (KSTEP.EQ.12 ) THEN    ! ----------- FILL 2
      ELSEIF (KSTEP.EQ.13 ) THEN    ! ----------- Consolidate FILL 2
      ELSEIF (KSTEP.EQ.14 ) THEN    ! ----------- EXCAVATE 3
      ELSEIF (KSTEP.EQ.15 ) THEN    ! ----------- Consolidate E3
      ELSEIF (KSTEP.EQ.16 ) THEN    ! ----------- PUMP E3

      HW = HO + ( HF - HO ) * TIME(1) / TF ! Water Level
      ELSIZE = 0.5
    ! Drainage Only
    PERM = 1/1.d7
    if (Z > HW ) THEN             ! IF point is above phreatic surf

        IF (U.LT.0.D0 ) THEN      ! IF U < 0
            H = 0.d0              ! No Flow
            sink = 0.d0
        ELSE                      ! IF U > 0
            SINK = 0.D0           ! So that Vn = ks * U
            H = PERM / GW * 1.d5/ 1.0
        ENDIF
```

# EXCAVATION GEOMETRIES



# MATERIAL CHARACTERIZATION




## ***Oedometer Tests***

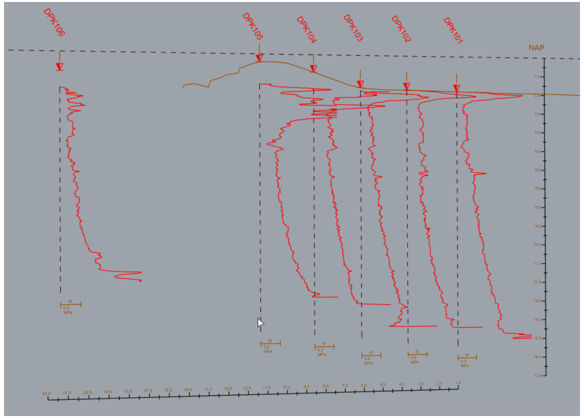
*TxCU & $K_0$*

## ***Shear Tests***

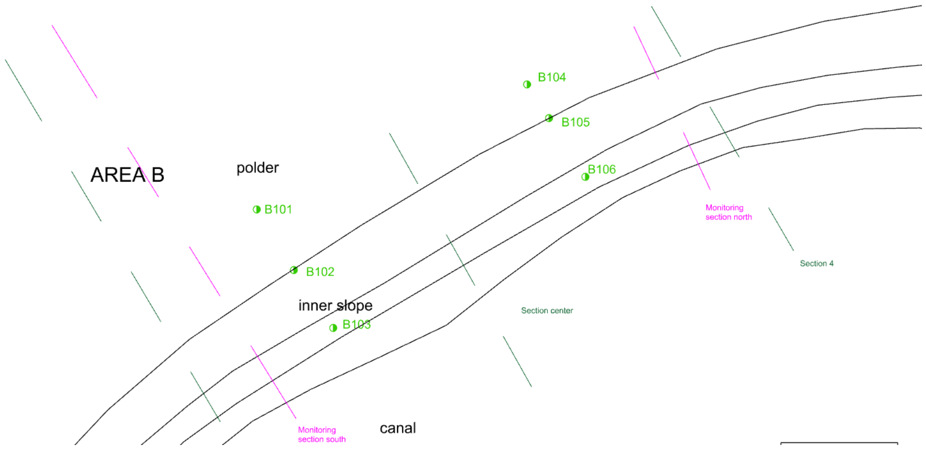
*TxCU & $K_0$*

## ***LABORATORY TESTS GIVEN***

*CPT Data*

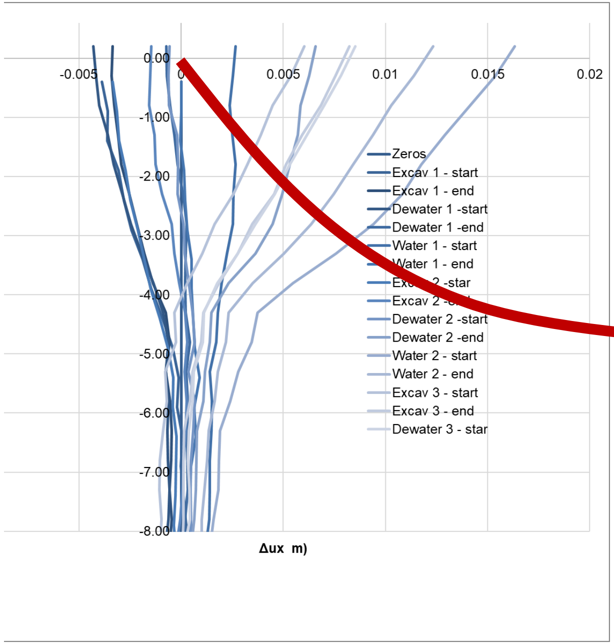


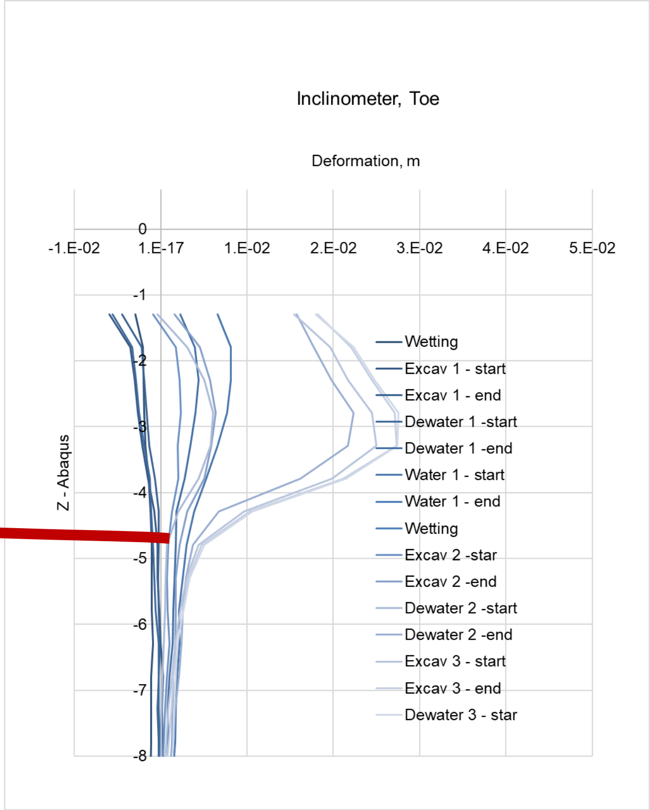

*Boreholes B101-B106*



# INCLINOMETERS | CREST & TOE

Approximate shape of failure surface



# PREDICTION OF FAILURE

# Prediction of failure

1.a) Given the stress test plan and the geotechnical characterization of the site. when did failure occur?

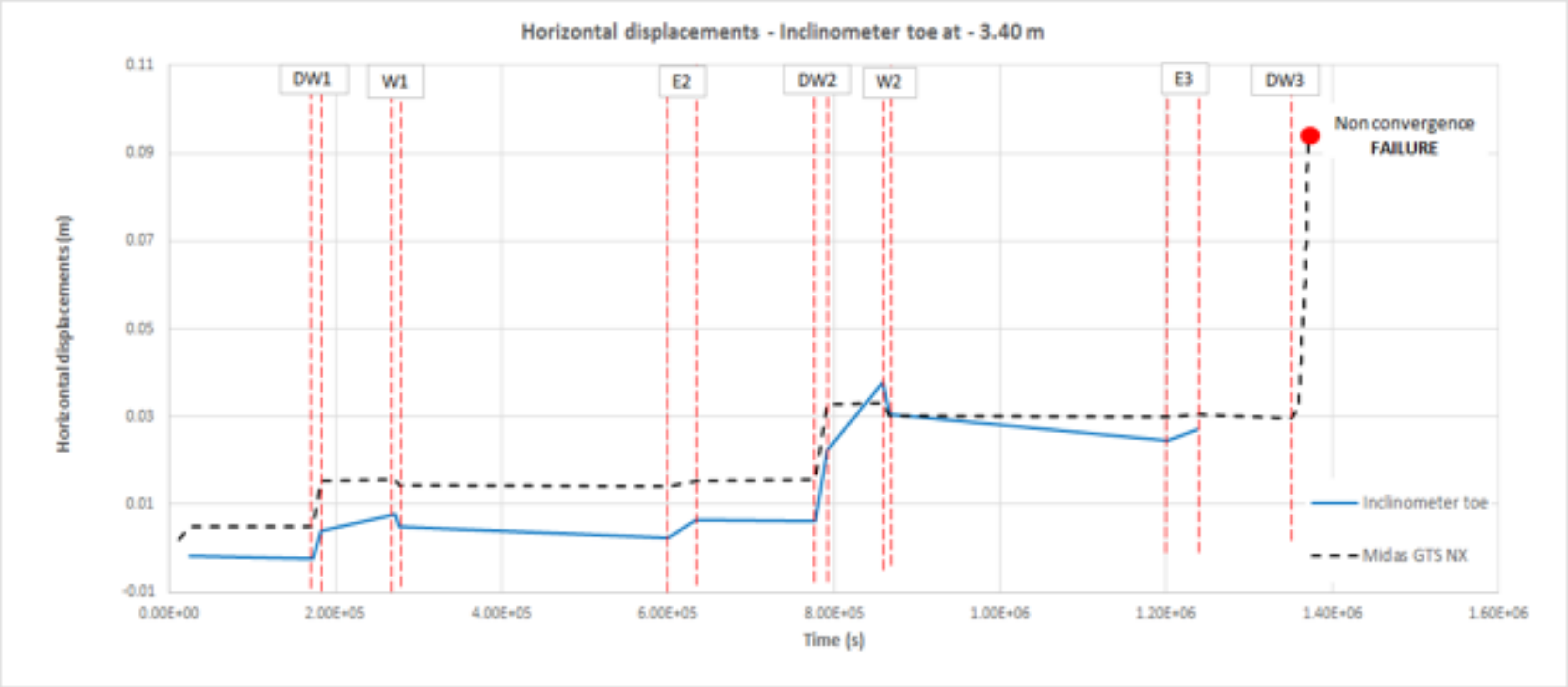
The Finite Element model in Midas GTS NX (Hardening Soil model) predicted the failure during the dewatering 3, when the water level in the excavation was at -2.97 m NAP, as evidenced by a drastic increase in deformations and plastic strains and an inability to achieve convergence, implying that there is no stress distribution that is able to satisfy the failure criterion of the constitutive model and be in equilibrium with the applied loads (Griffths, 1999).

According to the stress test plan, the failure happened approximately on 10/14/2015 at 5:00: AM.

However, significant plastic (permanent) deformations were developed during the 2nd dewatering operation, as shown here, which were not recovered during watering. It is likely that the 3rd dewatering operation merely exacerbated this accumulation of plasticity, eventually leading to failure.

# Prediction of failure

## 1.a) Given the stress test plan and the geotechnical characterization of the site. when did failure occur?



# Prediction of failure

1.b) Based on the monitoring data provided, will the failure be best reproduce by drained or undrained conditions?

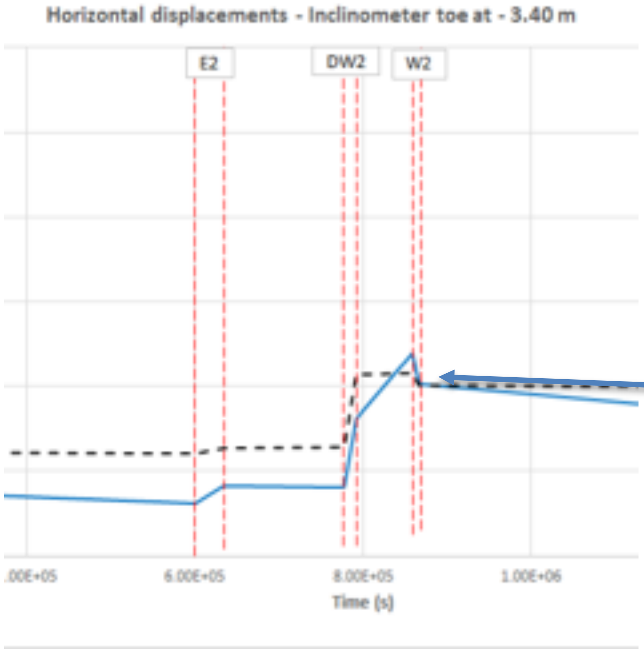
The evolution of pore pressure during the stress test shows that the maximum change of pore pressure head was 26 cm during the drawdown of 100 cm in the excavation 2 (in piezometer PtC2 located about 5 m away from the excavation).

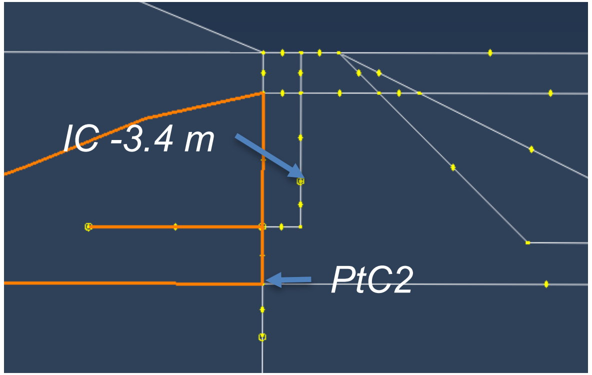
The inclinometers also show the highest deformations in this region and at this time, therefore, suggesting that changes in pore pressure will affect the failure response.

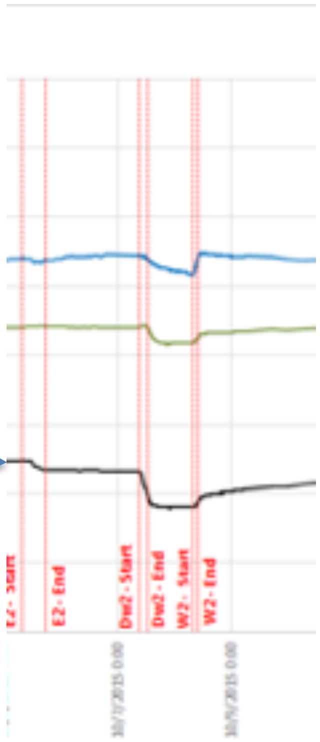
Hence, failure will be best reproduced by drained conditions, because it is to be expected that the change in pore pressure at the same location during the 3rd dewatering would be even higher, since it is closer to the piezometer and the planned drawdown was 150 cm.

# Question 1.b continued…

## Deformations and pore pressures near the failure region



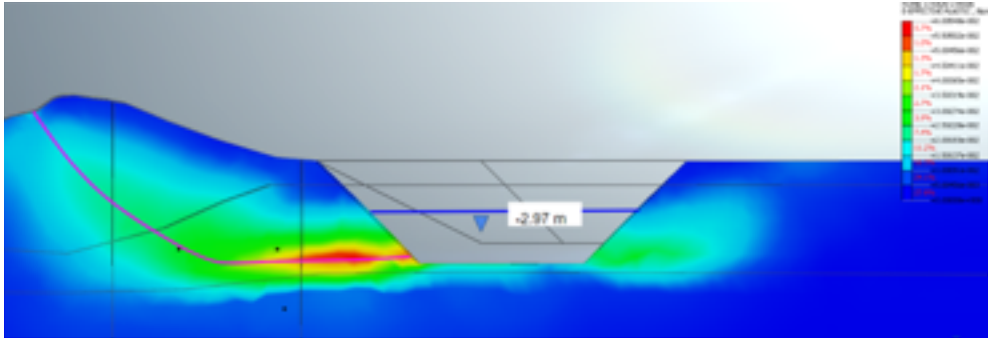



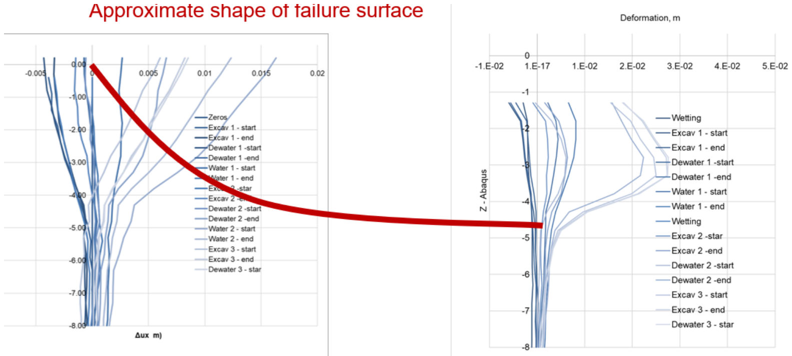



# QUESTION 1.C

## 1.c) What was the shape of the failure surface?

The shape of the failure surface can be evidenced by the accumulation of plastic strain right before non convergence in the Midas Model. The failure starts in the toe of the excavation and it extends along an horizontal plane in the peat and then through the dyke still the reservoir level. This kind of failure is in agreement with the failure in peat slope and peat dykes, which tend to fail along a horizontal plane (Pigott, 1992, Van Barras 2005, Boylan 2008)




See Slide 22

# Prediction of failure

1.d) Which elements of the model are most affected by uncertainty?

- According to the horizontal displacements register by in the inclinometers, the failure surface seems to be lower than the one predicted in the FE model. Therefore, it is possible the failure goes throw the interface between the Peat and the Organic Clay, which made difficult to accurately predicted the failure, because there is no information about the mechanical behavior of this interface.
- The failure is trigger by the drawdown in the excavation and hence it is considerably affected by the permeability of the soil, since to low permeabilities the pore pressure could remain high meanwhile the stabilizing effect of the water in the excavation is lost, reducing considerably the factor of safety. In consequence, the permeability is an important parameter in the prediction of failure and in the same time is one of the most affect by uncertainty, due to the soil permeability was considered with the empirical correlation of the CPTu and with typical values in literature.

# PRE-FAILURE RESPONSE

# QUESTION 2.A

(2.a) Which material model can be used to fit the best the subsoil and the material behaviour observed in the laboratory?

Based on preliminary results of element tests carried out in Midas GTS-NX, the Modified Mohr Coulomb model (Hardening Soil) offers important advantages in relation to the Modified Cam-Clay models tested in Abaqus.

One difficulty with the MCC model in Abaqus is the Poisson ratio/Shear behavior. In one option, the Poisson Ratio is specified and the shear modulus varies with confinement, whereas in another, the shear modulus is specified directly and Poisson's ratio varies with confinement.

Also, in many of the lab tests provided, not enough information is available to determine the Poisson ratio, and values of 0.3-0.45 have been tested in order to attempt to reproduce the results of the monitoring data.

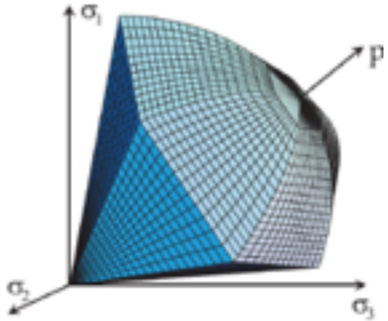
Some comparisons between the lab tests and element tests that attempt to reproduce them are shown in slides 13-15.

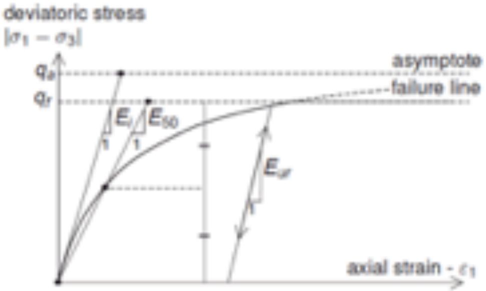
# MODIFIED MOHR-COULOMB MODEL IN MIDAS

## Modified Mohr-Coulomb (Hardening soil model)

- Hyperbolic stress-strain relationship in axial compression-
- Plastic strain in mobilizing friction (Shear hardening).
- Plastic strain in primary compression (Volumetric hardening).
- Stress-dependent stiffness.
- Elastic unloading / reloading compared to virgin loading.
- Memory to pre-consolidation stress.
- Dilatancy below the MC line.

*Ezzat M. (2018) Lecture 6: Numerical analysis in geotechnical Engineering*



# POROUS ELASTICITY IN ABAQUS

- Elastic behavior can be defined by:
  - Linear Elasticity
    - $E, \nu$
  - Porous Elasticity
    - Either
    - Shear Behavior either:
      - Constant Poisson Ratio (variable shear modulus)
      - Constant Shear Modulus
- Plastic Behavior can be modeled either by
  - Logarithmic Plastic Bulk Moduls, λ
  - Tabular form (see next slide)

$$E = E_{ref}\left[\frac{p+p_0}{p_{ref}+p_0}\right]^n \quad p>0; E = fE_{ref} \quad p \le 0,$$

$$\nu = \nu_0 + (\nu_\infty - \nu_0)\left(1 - e^{-mp}\right) \quad p>0; \nu = \nu_0 \quad p \le 0,$$

$$\frac{\kappa}{(1+e_0)} \ln\left(\frac{p_0 + p_t^{el}}{p + p_t^{el}}\right) = J^{el} - 1,$$

$$G = \frac{3(1-2\nu)(1+e_0)}{2(1+\nu)\kappa}\left(p + p_t^{el}\right)\exp\left(\varepsilon_{vol}^{el}\right),$$

$$\mathbf{S} = 2G\,\mathbf{e}^{el}.$$

# CLAY PLASTICITY CONTINUED

If plasticity is defined by $\lambda$ hardening follows the exponential form

$$a = a_0 \exp\left[(1+e_0)\,\frac{1-J^{pl}}{\lambda-\kappa J^{pl}}\right]$$

But the material can't exhibit a tensile yield stress, $p_t$,
And it requires the elastic behavior to be defined through $\kappa$, not allowing the definition of linear elasticity.

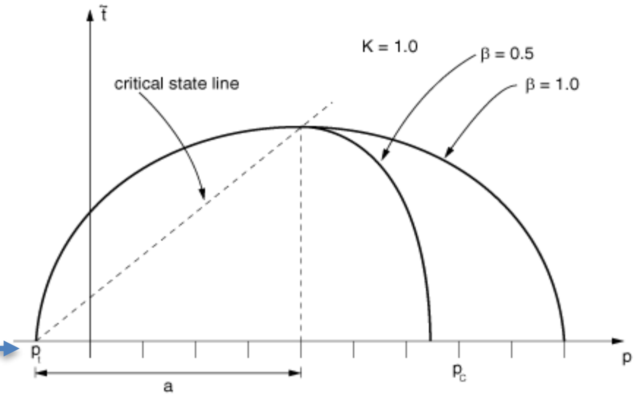
An alternative is to define the compressive hardening behavior through tabular forms:

$$p_c = p_c\left(\varepsilon^{pl}_{\text{vol}}\right),$$

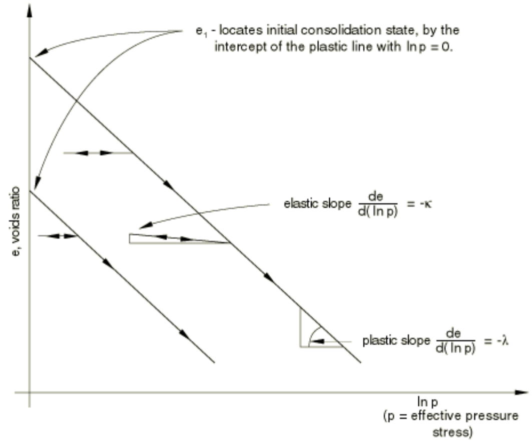
$$p_t = p_t\left(\varepsilon^{pl}_{\text{vol}}\right).$$

This form of clay plasticity allows the elastic part to be Defined through $E$ as well as $\kappa$

*MCC Yield Surface in p-q space*




*MCC Consolidation behavior*

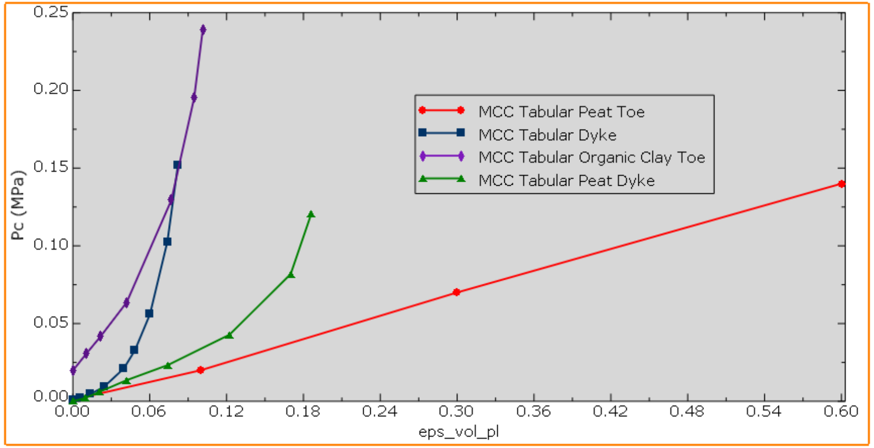


# TABULAR COMPRESSIVE CLAY HARDENING

$$p_c = p_c\left(\varepsilon^{pl}_{\text{vol}}\right),$$

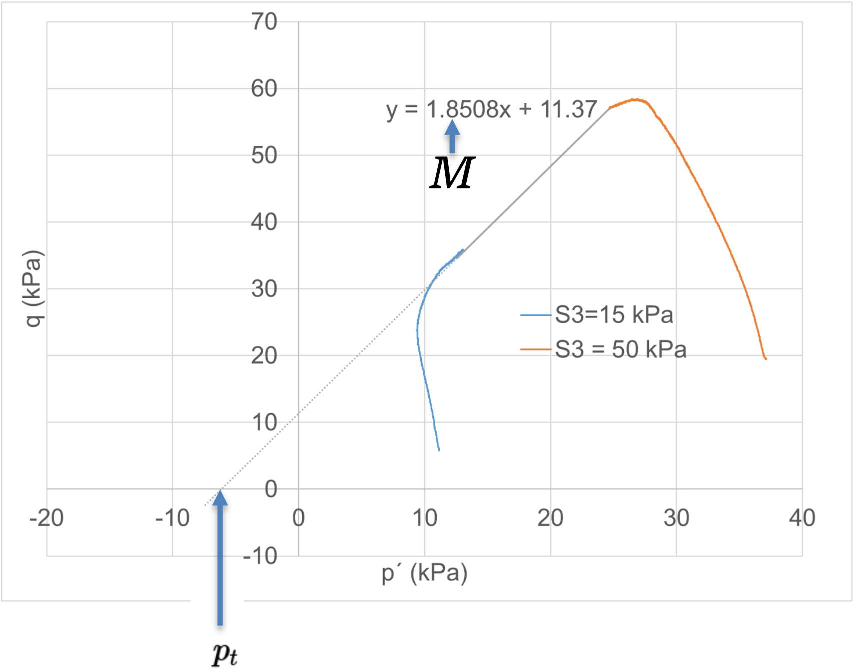
From the Oedometer tests, these curves may be estimated.

Tabular Compressive Clay Hardening Curves for Modified Cam Clay



# CLAY PLASTICITY – YIELD STRESS IN HYDROSTATIC TENSION

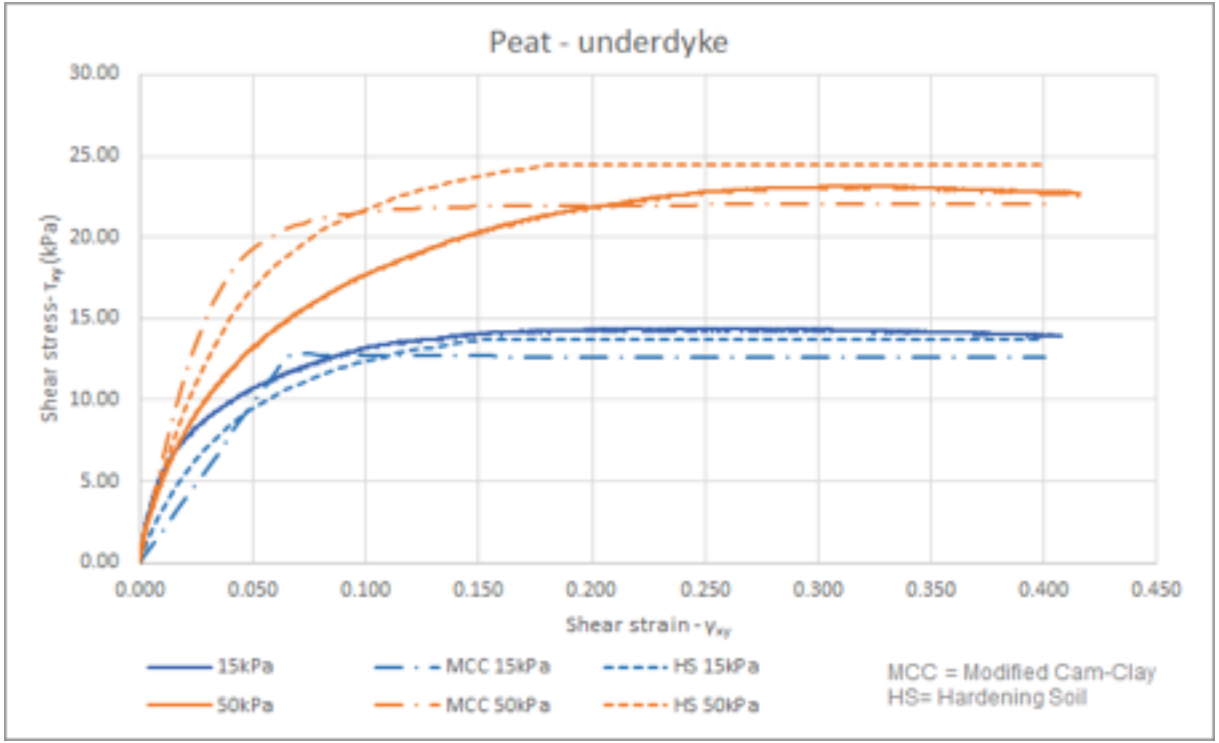
*Direct Simple Shear Tests*



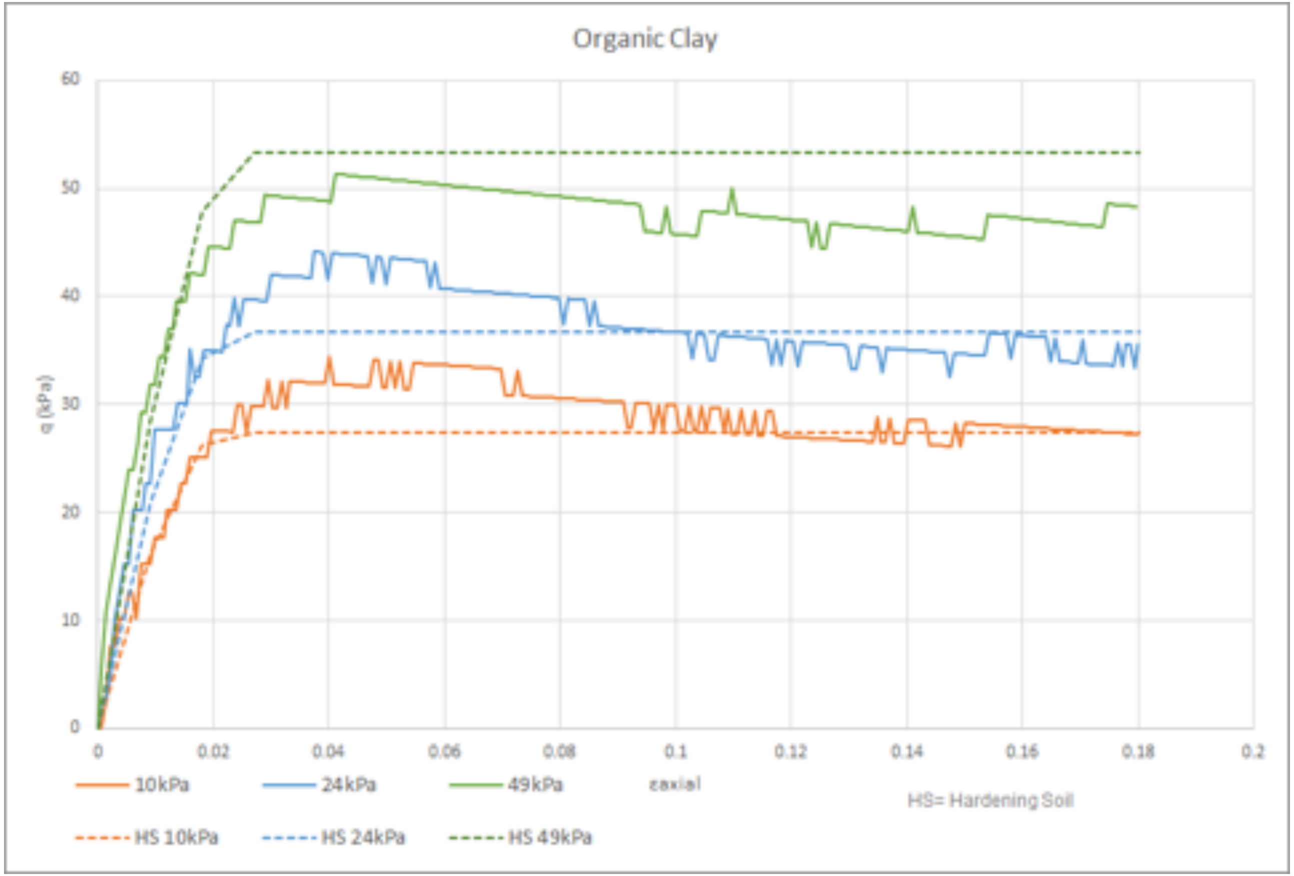

*Critical State Surface for Peat Under Dyke*

# CU ELEMENT TESTS VS LAB TEST| PEAT UNDER DYKE

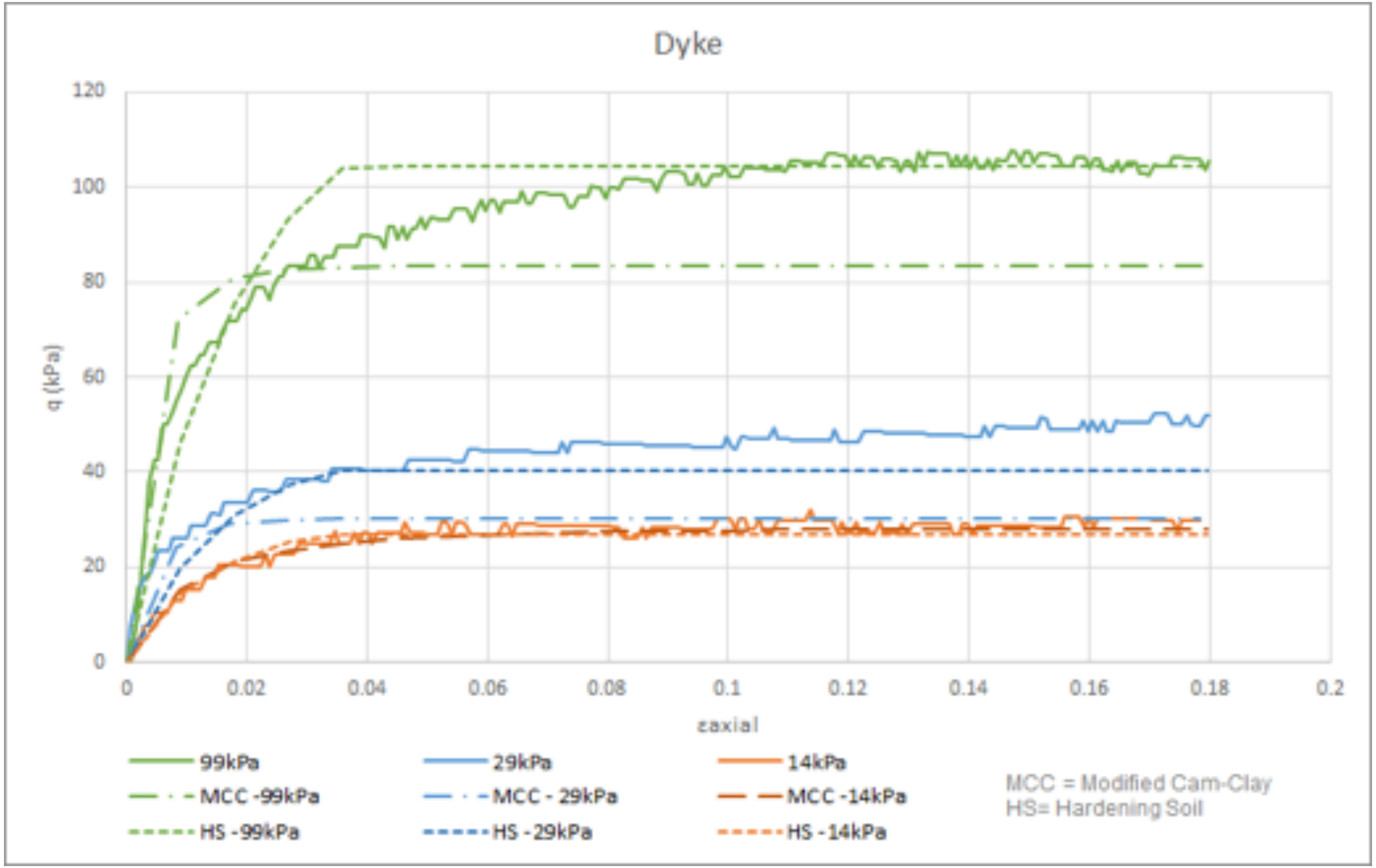
Parameter calibration with the numerical simulation of CU Triaxial test and direct simple shear test with one single algorithm (soil test wizard).



# TRIAXIAL ELEMENT TESTS | ORGANIC CLAY VS HS MODEL



# TRIAXIAL ELEMENT TESTS @ DYKE MAT | MCC & HS



# QUESTION 2.B.

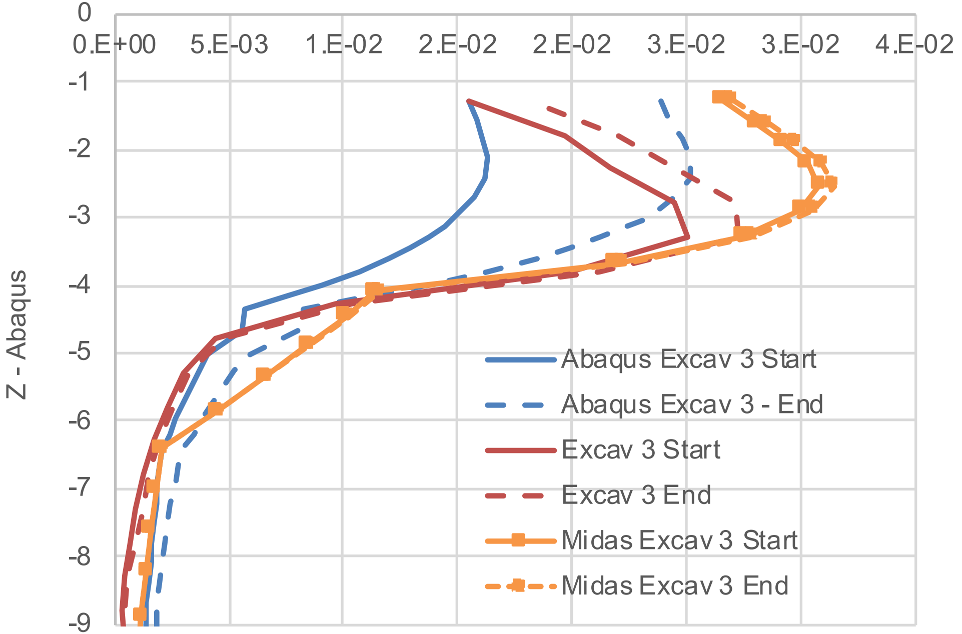
2.b) Given the stress test plan and the geotechnical characterization of the site, how did the soil respond from the start of the test to the beginning of the final drawdown stage? to this aim, provide the time history of:

-2.b.1) Horizontal displacement in correspondence of the two vertical lines monitored by the inclinometers. (see slides 29-30)

-2.b.2) Pore pressure in correspondence of the three locations monitored by piezometers. (see slides 31-32)

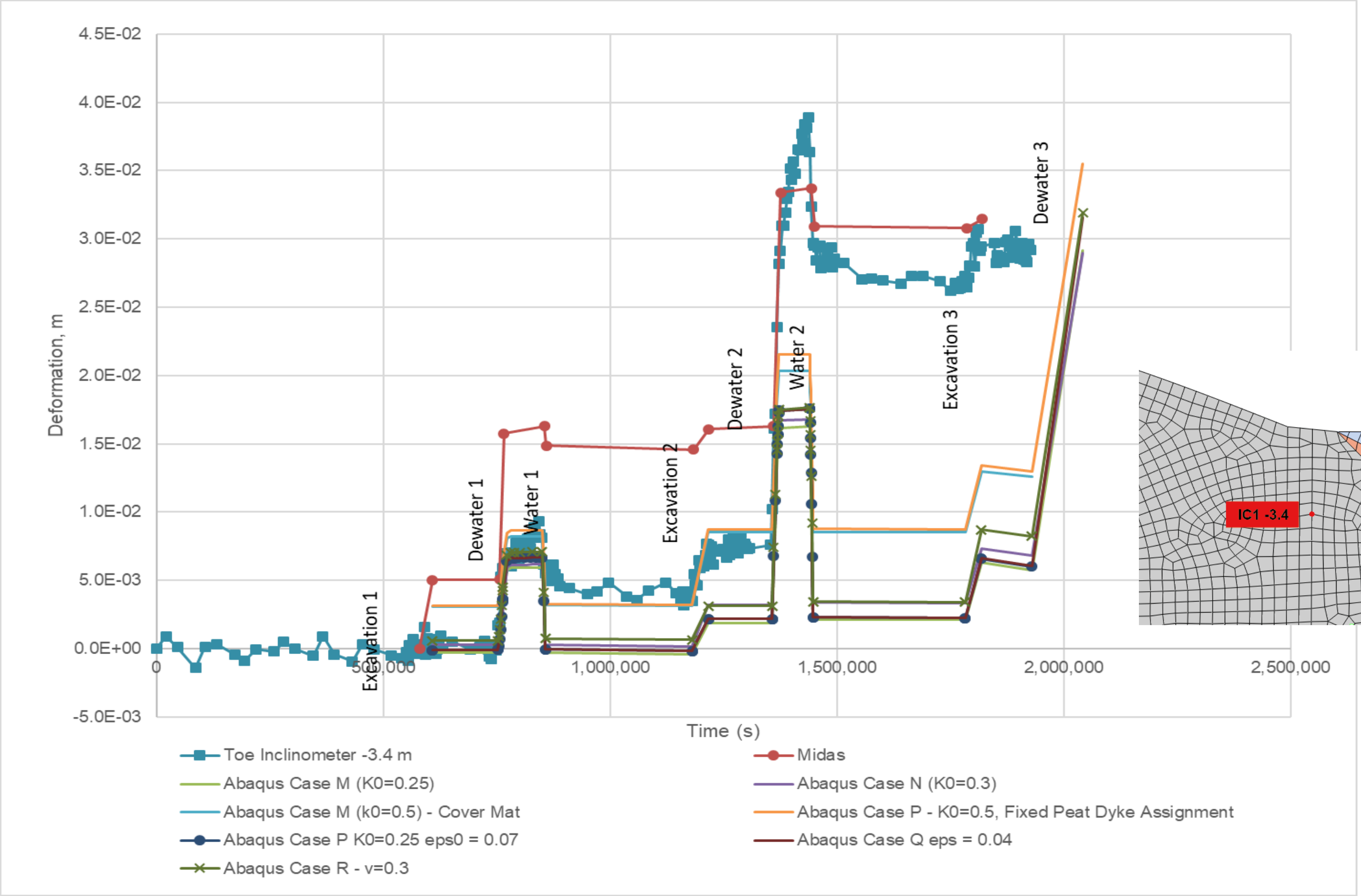
# INCLINOMETER TOE | EXCAVATION 3

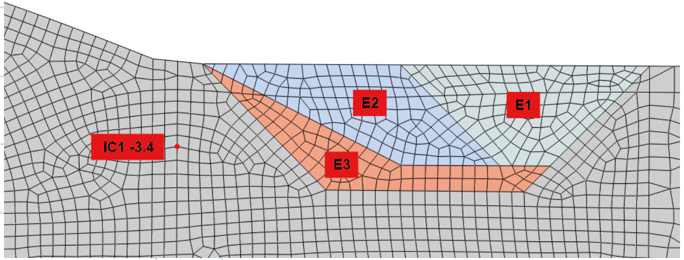
Excavation 3 - Before and After | Inclinometer vs Abaqus vs Midas



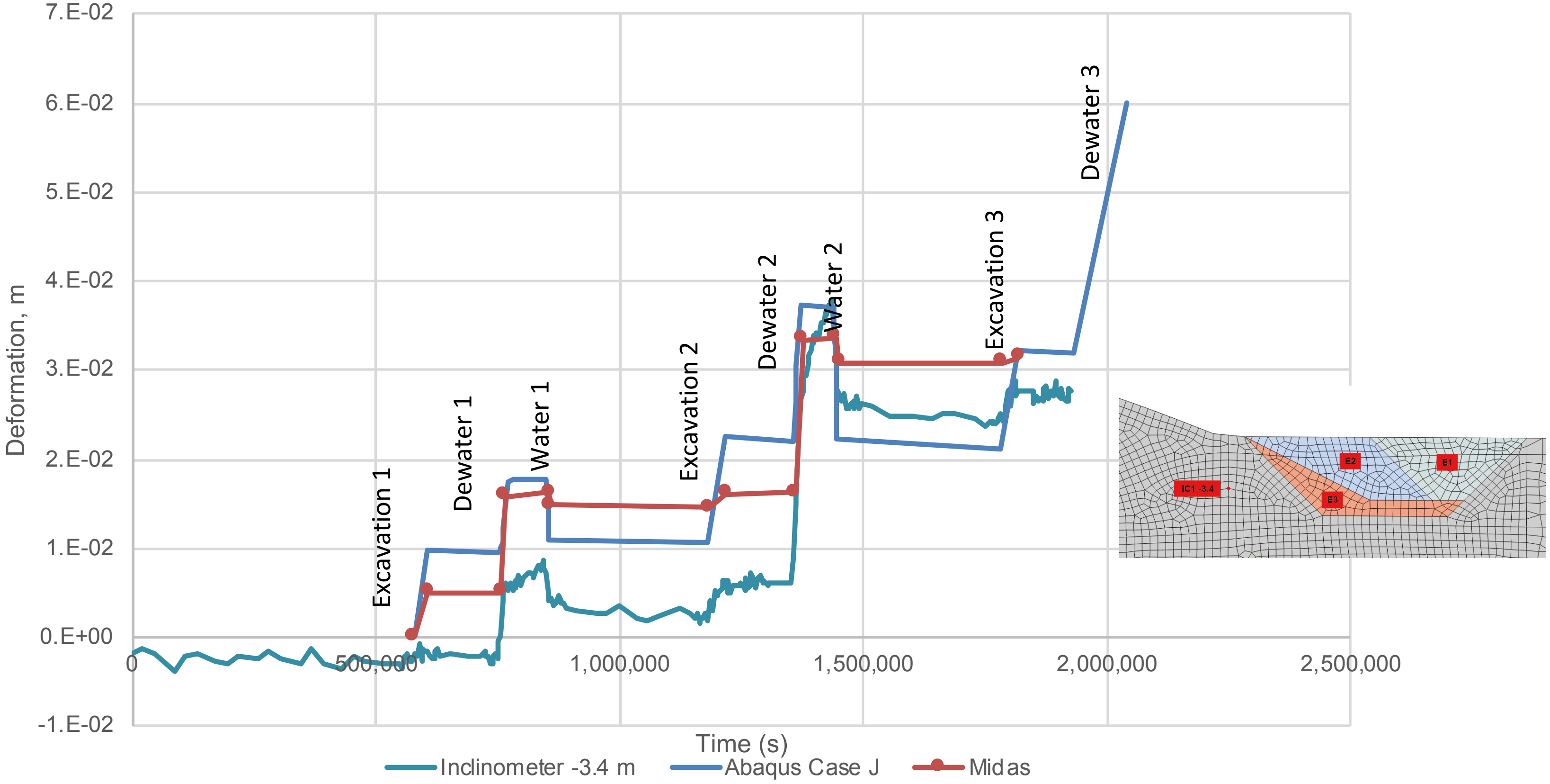

*The Abaqus model is overestimating the displacement jump during the 3rd excavation, whereas the Midas Model predicts a very small jump. The instrumentation also shows a small jump in deformation during the 3rd excavation.*

# INCLINOMETER TOE VS ABAQUS & MIDAS

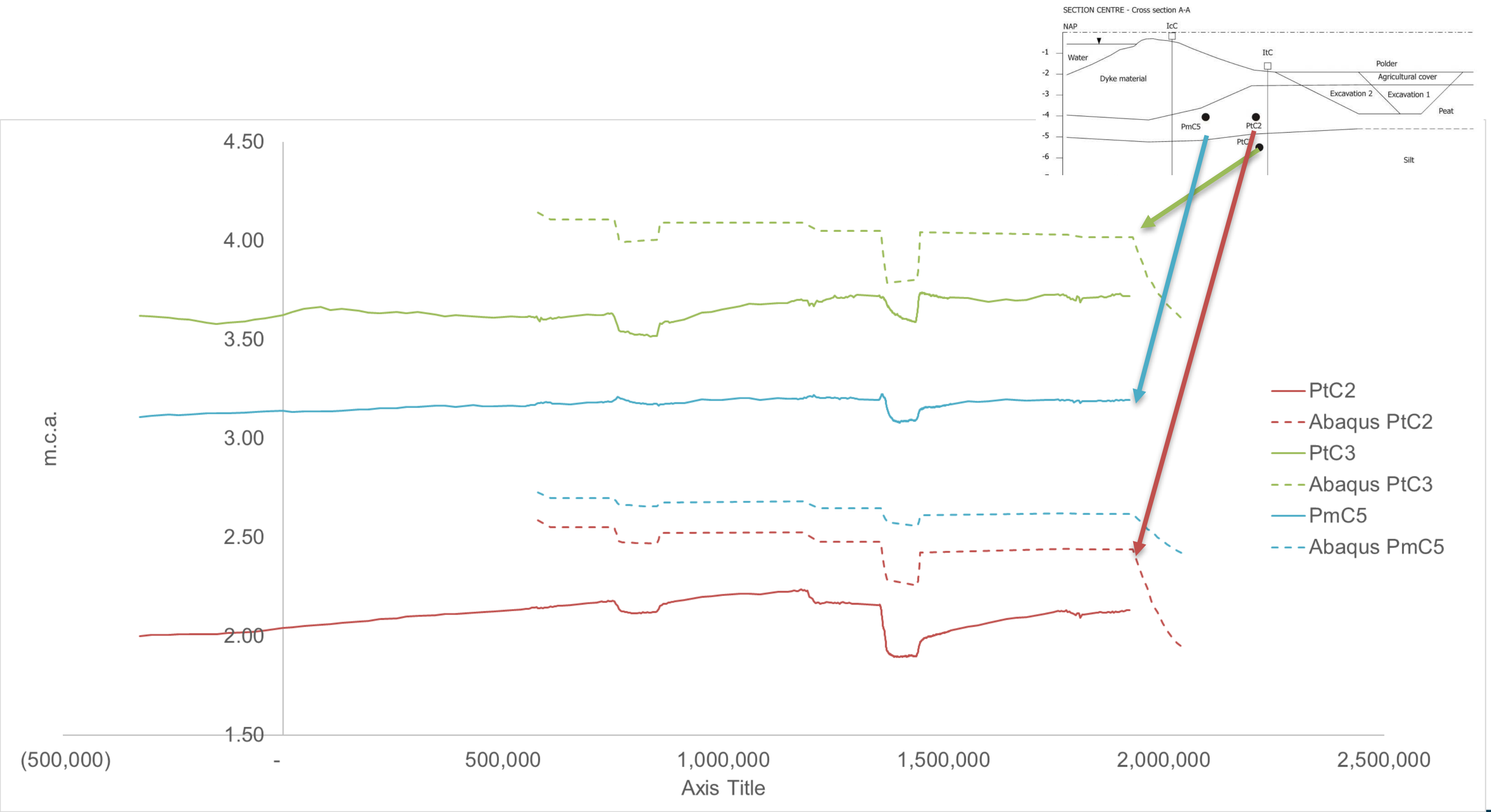


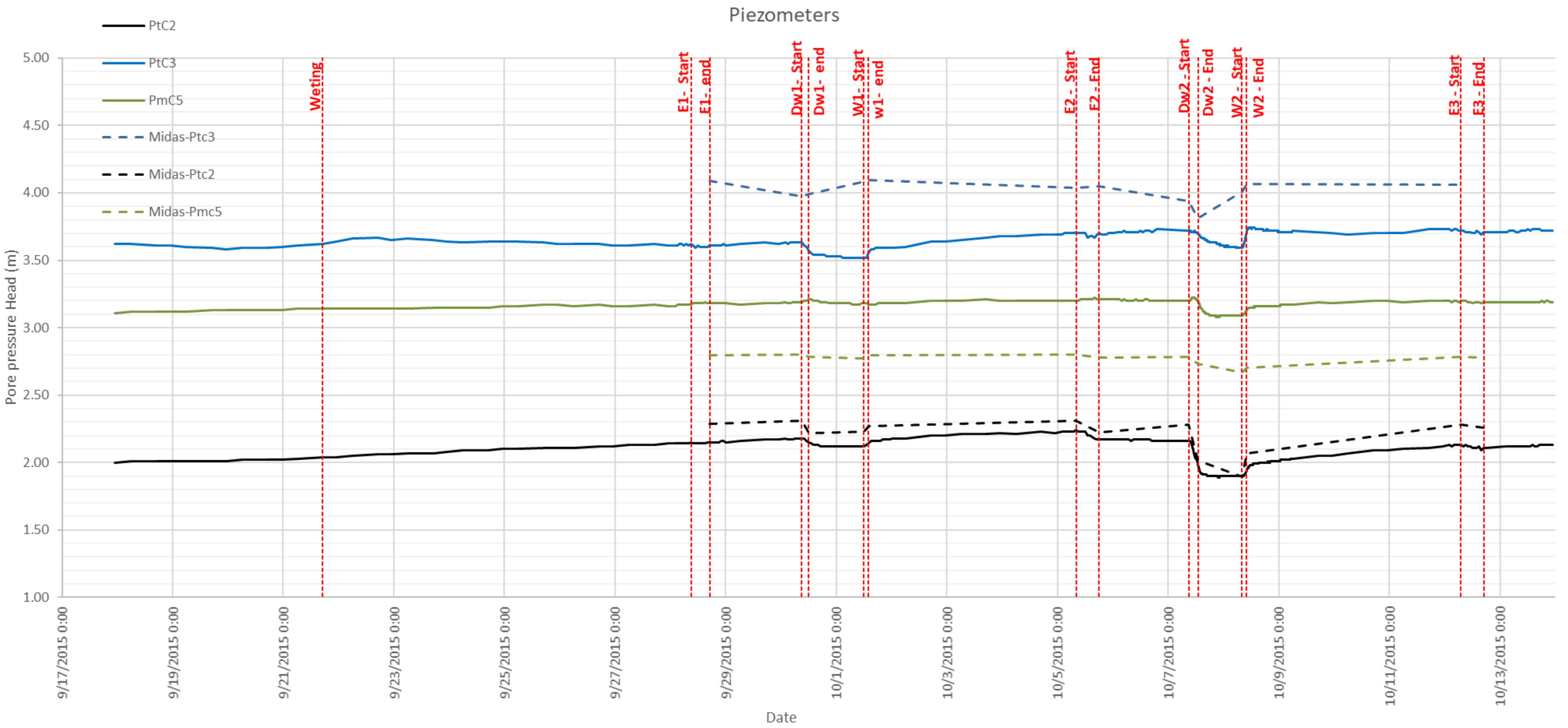
# DEFORMATION VS TIME | TOE @ -3.4M DEPTH



# PIEZOMETERS VS MODEL PORE PRESSURES| ABAQUS



# PIEZOMETERS VS PORE PRESSURES | MIDAS



# QUESTION 2.B.

3.b) Given the measurements taken, can the pre-failure displacements and pore pressures be predicted accurately with current models?

- The change of pore pressure were predicted accurately with the Modified Mohr-Coulomb and the Modified Cam-Clay models, and the difference in the results are mainly due to not correctly represent the initial state of the pore pressure (water level in the polder stablish by pumps, wetting stage, etc.).
- The two model overestimated the deformation in the 1$^{st}$ excavation. In the Midas model this is because of the high horizontal effective stress calculated in the initial state.
- Based on current assumptions, the deformations predicted by the Abaqus model are:

  Adequately estimated during **excavations**

  Underestimated during **dewaterings**