Numerical Modeling of Soil-Pile Axial Load Transfer Mechanisms in Granular Soils

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Statement of the Problem

\[ Q_T = Q_s + Q_b \]

Pile resistance mobilization

Shaft resistance:

\[ Q_s = \int_0^L \int_0^{2\pi} \tau_f r \, d\theta \, dz \]

Base resistance:

\[ Q_b = \int_0^{2\pi} \int_0^R (\sigma_v) r \, dr \, d\theta \]

Berezantzev et al. (1961)
Statement of the Problem

Depends on:
- Nature of soil
- Initial soil conditions:
  - Initial state (undisturbed soil)
  - Installation effects
  - Residual loads
- Pile-soil Interface (Shaft)
  Relative movements for resistance mobilization
- Type of loading
  (Monotonic, cyclic, static or dynamic)
- Time effects

\[ Q_T = Q_s + Q_b \]

Pile resistance mobilization

Berezantzev et al. (1961)
I – Introduction

Background and Motivations

CONVENTIONAL DESIGN METHODS

- Predictions:
  - Theoretical solutions, Empirical methods
  - Broad range of predictions; Low reliability

IC 50 prediction event
Dunkirk sand (Jardine et al. [2005])

ISC2 prediction event
Porto residual soil (Santos et al. [2005])
Background and Motivations

CONVENTIONAL DESIGN METHODS

- Predictions:
  - Theoretical solutions, Empirical methods
  - Broad range of predictions; Low reliability

NEEDS

- Improve predictive methods: economies
- Understanding of all interaction mechanisms (installation and loading)
- Reliable and rational design method
Objectives (challenges)

- Improve the knowledge of the load transfer mechanism
- 3D numerical modeling of Pile-Soil Interface and soil behavior
- Determine load settlement response (base and shaft) to axial loads of non-displacement piles
- Contribute to the understanding of some of the installation effects:
  - Soil stress history
  - Residual loads
  - Friction fatigue
- Apply the model to an in-situ case study

Berezantzev et al. (1961)
Short Outline

I  - Introduction
II - Theory formulation and Numerical modeling
III - Soil-Structure Interface behavior
IV - Non-displacement Piles in Toyoura sand
V  - ISC’2 Experimental site
VI - Final remarks
Short Outline

I - Introduction
II - Theory and Numerical modeling
III - Soil-Structure Interface behavior
IV - Non-displacement Piles in Toyoura sand
V - ISC’2 Experimental site
VI - Final remarks
• Numerical Tools
  – GEFDYN FEM code [Aubry et al 1986]
  – SDT / Matlab post-processing

• ECP Elastoplastic Constitutive Models
  • Multimecanisms model “ECP” or “Huieux” model,
    (2D and 3D)
    [Aubry et al. 1982, Hujeux 1985]
  • Interface model, [Aubry et al. 1990]
    (2D)
    (1D)

• 3D Axisymmetric formulation of the Multimecanisms model
• 3D Interface model

Newly implemented
Employed and developed Tools

- **ECP elastoplastic Constitutive models:**
  - **Theoretical principles:**
    - Effective stress principle;
    - Incremental plasticity;
    - Critical state concept;
    - Mohr Coulomb type failure criterion;
    - Deviatoric primary yield surface of the k plane (3 planes):

\[
 f_k(\sigma, \varepsilon_p^e, r_k) = q_k - \sin \phi_{pp}^l \cdot p'_k \cdot F_k \cdot r_k
\]

\[
 F_k = 1 - b \ln \left( \frac{p_k'}{p_c} \right) \quad p_c = p_{co} \exp(\beta \varepsilon_p^o) \quad \varepsilon_p^o = \sum_{k=1}^{3} (\varepsilon_{p_{iso}}^p) + (\varepsilon_{p_{iso}}^p)
\]

- Isotropic yield surface:
  \[
  f_{iso} = |p'| - d p_c r_{iso}
  \]

- Progressive mobilization of shear:
  \[
  r_k = r_{cl}^k + \frac{\int_0^t \varepsilon_p^e \, dt}{a + \int_0^t \varepsilon_p^e \, dt} \quad a = a_1 + (a_2 - a_1) \alpha_k(r_k)
  \]

- Roscoe’s dilatancy law
II – Theory and Numerical modeling

Employed and developed Tools

• **3D Axisymmetric** formulation of the ECP multimechanisms model:

  *Induced anisotropy* depends on the choice of the deviatoric mechanisms planes direction

3D CV shear test (axisymmetric conditions)

\[ \varepsilon^p_v = \sum_{k=1}^{3} \left( \varepsilon^p_{vh,k} + (\varepsilon^p_i)_{iso} \right) \]
• **3D Axisymmetric formulation of the multimechanisms model:**

**Induced anisotropy** depends on the choice of the deviatoric mechanisms planes direction

3D CV shear test (axisymmetric conditions)

Deviatoric mechanisms:
Transformation of three orthogonal planes:

- $q_{rx}$, $p_{rx}$
- $q_{rz}$, $p_{rz}$
- $q_{yz}$, $p_{yz}$

**New formulation**

Induced anisotropy depends on the choice of the deviatoric mechanisms planes direction.
II – Theory and Numerical modeling

Employed and developed Tools

• 3D ECP Interface

Hypotheses:
- Isotropy in the sliding plane:
\[
\tau = \tau_s \cdot \overrightarrow{e_s} + \tau_t \cdot \overrightarrow{e_t}
\]

- The tangential flow rule of the sliding plane:
\[
\dot{u}_\tau^P = \lambda^P \cdot \Psi_\tau
\]
\[
\Psi_\tau = \frac{\partial f}{\partial \tau}
\]

- The normal plastic displacement rate variation:
\[
\dot{u}_n^P = \dot{\lambda}^P \cdot \Psi_n
\]
I - Introduction
II - Theory and Numerical modeling
III - Soil-Structure Interface behavior
   • Background
   • Strategy for parameters’ identification
   • Simulation of direct shear tests
   • Calibration of ECP interface model
IV - Non-displacement Piles in Toyoura sand
V - ISC’2 Experimental site
VI - Final remarks
III – Interface behavior

Background

• Soil-Structure Interface behavior depends on:

  Pile nature
  - \( R_t \) pile surface roughness
  - \( R_n \) - Normalized surface roughness \( \frac{R_t}{D_{50}} \) [Uesugi & Kishida 1986]

  Soil nature
  - \( D_{50} \) soil grain size
  - \( \sigma_{n0} \) initial normal stress
  - \( D_r \) initial relative density

  Soil initial state
  - \( t \) - interface layer thickness

  Boundary conditions
  - Shear tests:
    - CNL - Constant normal load
    - CV - Constant volume
    - CNS - Constant normal stiffness
III – Interface behavior

Strategy for parameters’ identification

- Boundary conditions, $CV, CNS, CNL$
- Initial normal stress, $\sigma_{no}$
- Initial relative density
- Type of loading
- Interface layer thickness, $t$
- Normalized roughness, $R_n$

Similitude with soil behavior
Uesugi et al. [1988], Boulon and Nova [1990] …

ECP soil model parameter’s identification strategy
(extensively studied at ECP)
(Hicher & Rahma [1994]... Kordjani [1995]...
Lopez [2003]...)
Strategy for parameters’ identification

- Boundary conditions, $CV$, $CNS$, $CNL$
- Initial normal stress, $\sigma_{no}$
- Initial relative density
- Type of loading
  - Interface layer thickness, $t$
  - Normalized roughness, $R_n$

Similitude with soil behavior

Uesugi et al. [1988], Boulon and Nova [1990] …

Taken into account implicitly in the parameters
### Simulation of Direct Shear Test

- **Boundary conditions, CV, CNS, CNL**
- **Initial normal stress, \( \sigma_{no} \)**
- **Initial relative density**
- **Type of loading**
- **Interface layer thickness, \( t \)**
- **Normalized roughness, \( R_n \)**

#### Observation

<table>
<thead>
<tr>
<th>Observation</th>
<th>( t )</th>
<th>Controlling parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quasi-steady state</td>
<td>Shifting for lower ( u_s )</td>
<td>( r(\dot{u}_d^p) = \dot{r}^{ed} + \frac{\dot{u}_d^p}{\dot{\theta} + \dot{u}_d^p} )</td>
</tr>
<tr>
<td>Dilatancy at the critical state</td>
<td>Reduction</td>
<td>( \sigma_c = \sigma_{co} \exp(\beta \dot{u}_d^p) )</td>
</tr>
<tr>
<td>Elasticity</td>
<td>Domain reduction</td>
<td>( G_{1}^{\uparrow} = \frac{G}{t} )</td>
</tr>
<tr>
<td>Friction angle at the critical state</td>
<td>Unaffected</td>
<td>--</td>
</tr>
</tbody>
</table>

- **FEM model (solid element)**
- **CNL shear test: (ECP multimechanisms)**
  - \( \sigma_{no}=100 \text{ kPa} \)

- **Graphs**
  - \( \tau_s \text{ (kPa) - Shear stress} \)
  - \( u_s \text{ (mm) - Tangential displacement} \)

**Figures**

- **Figure 1**: Diagram showing \( u_s \) and \( t \) with \( \tau_s \) and \( \sigma_{no} \) indicated.
- **Figure 2**: Graph plotting \( \tau_s \) vs. \( u_s \) with different lines for \( \frac{1}{t} \) values.
- **Figure 3**: Graph showing \( \sigma_{no} \) effect on \( \tau_s \).

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**Table**

- **Parameter Table**:
  - | Observation               | \( t \)     | Controlling parameters                                      |
  - |---------------------------|-------------|-------------------------------------------------------------|
  - | Quasi-steady state        | Shifting for lower \( u_s \) | \( r(\dot{u}_d^p) = \dot{r}^{ed} + \frac{\dot{u}_d^p}{\dot{\theta} + \dot{u}_d^p} \) |
  - | Dilatancy at the critical state | Reduction | \( \sigma_c = \sigma_{co} \exp(\beta \dot{u}_d^p) \) |
  - | Elasticity                | Domain reduction | \( G_{1}^{\uparrow} = \frac{G}{t} \) |
  - | Friction angle at the critical state | Unaffected | -- |
III – Interface behavior

Simulation of Direct Shear Test

- Boundary conditions, CV, CNS, CNL
- Initial normal stress, $\sigma_{no}$
- Initial relative density
- Type of loading
- Interface layer thickness, $t$
- Normalized roughness, $R_n$

\[ R_n = \frac{R_t}{D_{50}} \]  
[Uesugi & Kishida 1986]

Contraction / Dilation at the interface zone due to shearing
III – Interface behavior

Calibration of ECP interface model

- Normalized roughness, $R_n$, (calibration)

\[
\frac{d\sigma_n}{du_n} \neq 0
\]

CNS shear test $k=1000\text{kPa/mm}$

Experimental Data from Fioravante [2002]
Calibration of ECP interface model

- Normalized roughness, $R_n$, (calibration)

$R_n$
- Affects the failure mode:
  - $R_n < R_{n \text{ crit smooth}}$ ($\phi_{\text{int}} << \phi_{\text{soil}}$)
  - $R_n > R_{n \text{ crit rough}}$ ($\phi_{\text{int}} = \phi_{\text{soil}}$)

**FEM model**

**CNS shear test** $k=1000\text{kPa/mm}$

Experimental Data from Fioravante [2002]

Simulation
Short Outline

I - Introduction
II - Theory and Numerical modeling
III - Soil-Structure Interface behavior
IV - Non-displacement Piles in Toyoura sand
  • Validation with physical models
  • Soil-Pile interaction: parametric studies
  • Cyclic loading at constant force
V - ISC’2 Experimental site
VI - Final remarks
Centrifuge tests

• Static load test

- Tested sand: Toyoura
- Installation method: Non-displacement (in store before sand pluviation)
- Applied Load: Compression
- Soil density: $D R \approx 93\%$
- Roughness: $R_n = 0.01$ and 0.45
- Prototype: $L = 7.4\, \text{m}; D = 0.3\, \text{m}$
- Acceleration: 50g, 30g

[Fioravante 2002]
IV – Non-Displacement Piles

Toyoura sand

• Soil parameters calibration:
  (ECP multimechanism model)

<table>
<thead>
<tr>
<th>Sand</th>
<th>$\gamma_{\text{min}}$ [kN/m$^3$]</th>
<th>$\gamma_{\text{min}}$ [kN/m$^3$]</th>
<th>$e_{\text{max}}$ [-]</th>
<th>$e_{\text{min}}$ [-]</th>
<th>$G_s$ [-]</th>
<th>$D_{50}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Toyoura</td>
<td>13.1</td>
<td>16.2</td>
<td>0.977</td>
<td>0.605</td>
<td>2.64</td>
<td>0.16</td>
</tr>
</tbody>
</table>

Drained triaxial test
($p'_o$=100kPa)

TS40 (Dr=40%)
TS90 (Dr=93%)

Fukushima and Tatsuoka [1984]
IV – Non-Displacement Piles

Numerical model

• Model and boundary conditions:

SAME SIZE AS PROTOTYPE
Load settlement response

\[ Q_T = Q_s + Q_b \]

**IV – Non-Displacement Piles**

Validation with experimental results (1/2)

Expressions:
- \( Q_s \) - Shaft resistance
- \( Q_b \) - Base resistance

Parameters:
- \( s \) – pile head displacement
- \( d \) – pile diameter

- Rough, \( R_n = 0.45 \)
- Smooth, \( R_n = 0.01 \)

Experimental Data from Fioravante [2002]

Simulation

Reduction of \( R_n \): reduction of \( Q_s \rightarrow Q_T \)
IV – Non-Displacement Piles

Validation: Load Transfer mechanisms

Rough, smooth interface stress paths: Simulation

Control points:
- 1.7m
- 3.4m
- 6.0m

Graphs showing load transfer mechanisms for rough and smooth interfaces.
Load transfer coefficient \( (\beta = \frac{\tau}{\sigma_v}) \)

\( R_n = 0.45 \) (Rough interface)

\[ \tau = \beta \cdot \sigma_v \]

Validation with experimental results (2/2)

IV – Non-Displacement Piles

![Graph showing experimental data and simulation results](image)
Load transfer coefficient ($\beta = \tau / \sigma_{v0}$)

$R_n = 0.45$ (Rough interface)

$\beta$ decreasing with depth:
- progressive inhibition of dilatancy
- Increase in ambient stress

Final local $\beta$ ($\tau / \sigma_{v0}$)

Control points:
- 0.9m
- 2.9m
- 4.6m
- 6.4m

[Experimental results: Fioravante 2002]
Critical state approach

Importance of the soil initial state parameter?

- Regarding the interface roughness
- Regarding the pile length
- Regarding the type of loading
- Pile resistance for a soil with previous loading history

State parameter: $p_0/p_{co}$
Rough soil-pile interface:
Toyoura sand: different initial states, TS24, 40, 93

$Q_s$ - Shaft resistance

$Q_b$ - Base resistance
**IV – Non-Displacement Piles**

**Influence of soil initial state**

**Rough soil-pile interface:**

1) Different initial states, TS\(24, 40, 93\)
2) Initial stress: \(z=1.61\)

![Graph showing control points and soil behavior](image)

<table>
<thead>
<tr>
<th>TS24</th>
<th>TS40</th>
<th>TS93</th>
</tr>
</thead>
<tbody>
<tr>
<td>(p_{co(TS24)})</td>
<td>(p_{co(TS40)})</td>
<td>(p_{co(TS93)})</td>
</tr>
<tr>
<td>e</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ln p</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Graphs showing stress vs. normalized length for different initial states:**

- TS24
- TS40
- TS93

**Headings:**

- I
- II
- III
- IV
- V
- VI

**Footnote:**

- \(z= 1.61\) m
Rough soil-pile interface:

1) Different initial states, TS24, 40, 93
2) Different initial stress

Final local load transfer coefficient: $\beta = \tau / \sigma_{vo}$
IV – Non-Displacement Piles

Parametric studies

Influence of:
- Soil initial state
- Pile-soil surface roughness
- Pile length

In:
- Shaft resistance
- Base resistance

Average load transfer coefficient: $\beta_{\text{avg}}$

$$\beta_{\text{avg}} = \frac{Q_s}{\left( A_s \sigma_{v_0 \text{ avg}} \right)}$$

TS24, 40, 93
IV – Non-Displacement Piles

Parametric studies

Influence of:
- Soil initial state
- Pile-soil surface roughness
- Pile length

In:
- Shaft resistance
- Base resistance

Average load transfer coefficient: $\beta_{avg}$

$\beta_{avg} = \frac{Q_s}{(A_s \sigma_{vo \ avg})}$

Smooth interface:
No soil-pile interaction

TS24, 40, 93
IV – Non-Displacement Piles

Parametric studies

Influence of:

- Soil initial state
- Pile-soil surface roughness
- Pile length

In:

- Shaft resistance
- Base resistance

End bearing capacity factor: $N^*q$

$N_q = \frac{q_b}{\sigma_v}$

TS 24, 40, 93
Cyclic Load transfer Mechanisms

Motivation:
Reduction of radial and shear stress during pile penetration
Linked to: cyclic stress history

Main issue:
Identify the main mechanisms controlling friction fatigue

Key issues:
- Effect of the load level (5 amplitudes)
- Effect of the number of cycles
- Effect of cycling in subsequent reload to failure
Cyclic Load transfer Mechanisms

Effect of the load level:

- **Reduction of shaft resistance** with N;
- Progressive increase of the permanent settlement (s);
- Progressive mobilization of base resistance;
- Different levels of residual loads.

![Graphs showing Q_s - Shaft resistance and Q_b - Base resistance with various load levels.](image)
Cyclic Load transfer Mechanisms

Analysis of volume changes in the soil:

(N=5, Q_T =1300 kN)

Mechanism 1
Cyclic Load transfer Mechanisms

Analysis of volume changes in the soil:

Mechanism 2

(N=5, $Q_T = 400$ kN)
Analysis of volume changes in the soil:

(N=5, $Q_T = 200$ kN)

Mechanism 3
Cyclic Load transfer Mechanisms

In reload: soil-pile system improved with the **previous** application of **cycling**?

- shaft resistance

State after cycling (z=-4.4 m)

**Q_s** - Shaft resistance

**Q_b** - Base resistance
Cyclic Load transfer Mechanisms

Is the soil-pile system improved with the previous application of cycling?

- **Shaft** may not recover
- **Base** can be **improved**;
- **Total resistance** **beneficiates** from previous cycling at large load amplitude;
- **Friction fatigue**: compensated by the base mobilization during cycling

**Q_s** - Shaft resistance

**Q_T** - Total resistance

**Q_b** - Base resistance
Short Outline

I - Introduction
II - Theory and Numerical modeling
III - Soil-Structure Interface behavior
IV - Non-displacement Piles in Toyoura sand

V - ISC’2 Experimental site
   • ISC’2 Site description
   • Residual soil modeling
   • Static load test
   • Dynamic load test

VI - Final remarks
Used data

- **2nd International Conference on Site Characterization - University of Porto, Portugal**

**International Prediction Event on the Behavior of Bored, CFA and Driven Piles**

Viana da Fonseca et al. (2004)

- **Static and Dynamic Load tests**
  - Bored Piles
  - CFA Piles
  - Driven Piles

- **in situ tests**
  - SPT, CPT, DMT, PMT, CH

- **Laboratory tests**
  - Triaxial compression, extension; Resonant column and oedometer

Used to calibrate soil model’s parameters

(D’Aguiar (2008))
Residual soil modeling

Laboratory tests carried out in undisturbed samples:
Triaxial tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>S2/1</th>
<th>S2/5</th>
<th>S2/6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>3.2</td>
<td>5.5</td>
<td>7.0</td>
</tr>
<tr>
<td>$S_r$ (%)</td>
<td>62</td>
<td>86</td>
<td>81</td>
</tr>
<tr>
<td>$\sigma_{cv}$</td>
<td>60</td>
<td>100</td>
<td>140</td>
</tr>
<tr>
<td>$\sigma_{ch}$</td>
<td>30</td>
<td>50</td>
<td>70</td>
</tr>
</tbody>
</table>

$\phi'_p = 32^\circ$
$K_0 = 0.5$

---
experimental
---
simulation
Residual soil modeling

Laboratory tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>S5/2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>6.3</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^2$)</td>
<td>17.8</td>
</tr>
<tr>
<td>$e$</td>
<td>0.818</td>
</tr>
<tr>
<td>$S_r$ (%)</td>
<td>73</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen</th>
<th>S5/1</th>
<th>S5/3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>4.0</td>
<td>8.0</td>
</tr>
<tr>
<td>$S_r$ (%)</td>
<td>74</td>
<td>80</td>
</tr>
<tr>
<td>$\sigma_{cv}$</td>
<td>80</td>
<td>160</td>
</tr>
<tr>
<td>$\sigma_{ch}$</td>
<td>40</td>
<td>80</td>
</tr>
</tbody>
</table>

Oedometer test column test

Resonant column test simulated with drained cyclic shear test
Static Load Tests

- Static load: Bored and CFA Pile (E9 and T1)
  - Assumed hypothesis:
    - no modeling of installation effects
    - Rough interface

Bored pile (E9)

- Bored Pile: E=20GPa

CFA pile (T1)

- CFA Pile: E=40GPa
Static Load Tests

- Static load: Bored and CFA Pile (E9 and T1):
  
  Load distribution along depth

Bored pile E9

CFA pile T1

Pile Axial Load (N) distribution along depth

Pile instrumentation
Dynamic load test modelling

• Different simulations

Drop height: $h_1$

Wave propagation

Drop hammer

Soil

Pile

Blow 1

Simulation 1

Independent blow simulation

Blow 2

Simulation 2

Blow 3

Simulation 3

Blow 4

Simulation 4

Sequentially applied blow simulation

$I$  $II$  $III$  $IV$  $V$  $VI$
Dynamic load test records

• Forcing functions

Forces at the pile head for the DLT, bored pile (E9)

Drop hammer: 80kN

<table>
<thead>
<tr>
<th>Blow</th>
<th>Fall height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.6</td>
</tr>
<tr>
<td>2</td>
<td>0.8</td>
</tr>
<tr>
<td>3</td>
<td>1.1</td>
</tr>
<tr>
<td>4</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Fall heights for bored pile (E9)
Independent blow simulation

- Velocities at the pile head

![Graphs showing measured and computed velocities for different blows](image-url)
Sequentially applied blows simulation

- Complete load history ("pre-blow" + blow 1, 2, 3 and 4)
- Consistency of the dynamic load tests results

Velocity at the pile head

- Measured
- Computed
• Influence of the loading history in each blow response ("pre-blow" + blow 1 / blow 1)

Sequently applied blows simulation

- Seq. ("pre-blow" + blow 1)
- Indp. (blow 1)

Velocity at the pile head

Q_s - shaft resistance

Q_b - base resistance
Short Outline

I - Introduction
II - Theory and Numerical modeling
III - Soil-Structure Interface behavior
IV - Non-displacement Piles in Toyoura sand
V - ISC’2 Experimental site
VI - Conclusions and further research
• **The main issue:** 3D non-linear numerical modelling of soil-pile load transfer mechanisms of non-displacement piles axially loaded

• **The base of the work:**
  - Field observations
  - Computer simulations
  - Laboratory experimental tests

• **Aiming** to include the main **physical constraints** of the soil-pile interaction problem in a FEM model

adapted from Barends [2005]
VI – Conclusions and further research

Conclusions

• Spoted key aspects:
  – Soil-pile surface normalized roughness
  – Similitude between soil and soil-structure interface
  – Soil initial state
  – Soil loading history

• Allowing:
  – The validation of a complex model but sufficiently flexible and adapted to real case studies
  – Gained important insight, even if qualitative, concerning installation effects

• However:
  – Parameters identification
Further research

• Soil-Structure interface model:
  – More validations for cyclic loading and different soils;

• Water presence (coupled hydromechanical simulations):
  – Soil model is adapted;
  – Adapt the interface model;
  – Validations;

• Installation effects of displacement piles
Thank you …
Appendix
Concluding Remarks

- The GEFDYN FEM code
  - Enhanced:
    - 3D Axisymmetric formulation of the 3D ECP soil model (Hujeux model)
    - 3D interface model based on the critical state concept
  - Validated:
    - With other published numerical results
Concluding Remarks

Soil-Structure interface behavior:

- **3D ECP Interface model**
  - **Validated** with laboratory CNS shear test
  - **Correctly captures:**
    - Contractancy/dilatancy
    - Influence of normal stress
    - Coupling of shear and normal displacements
    - Critical state

- **Advantage of the 3D ECP interface:**
  - Same physical principles of the ECP soil model

- **Inconveniences of the 3D ECP interface:**
  - Large number of parameters

- **Coherent strategy for parameters’ identification**

- **Flexibility** in application
Concluding Remarks

Newly implemented model:
- Adapted for pile applications
- Validated with centrifuge test results (non-displacement piles)
- Flexible application to different soil-pile interface conditions

Parametric studies in Toyoura sand:
- Spot the key features of soil-pile interaction
  - normalized roughness
  - soil initial state
- Proposal of $\beta$ and $N_q$ parameters

Friction fatigue:
- Three different cyclic mechanisms control the rate of friction degradation
  - Related to the state parameter
  - Amplitude of the first load cycle
  - Number of cycles
- Soil memory in the reload resistance (parallel with installation effects)
Concluding Remarks

ISC’2 Experimental site: non-displacement piles

- **Consistent results** compared with *in-situ* static load tests results (bored and CFA piles)

- **Consistency** of the modeling based on:
  - Non linear behavior of:
    - Soil
    - Interface
  - Important work in parameters identification
  - Correct assumption of soil-pile interface (*rough interface*)

- **Compared performance** of bored and CFA piles

- Estimation of CFA pile **residual loads**
**Employed and developed Tools**

- **Comparison between Multimechanism and ECP interface models:**

<table>
<thead>
<tr>
<th>Multimechanisms</th>
<th>Interface</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Elasticity</strong></td>
<td></td>
</tr>
<tr>
<td>$K_{\text{max}} = K_{\text{ref}} \left( \frac{p'}{p_{\text{ref}}} \right)^{n_e}$</td>
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<tr>
<td>$G_{\text{max}} = G_{\text{ref}} \left( \frac{p'}{p_{\text{ref}}} \right)^{n_e}$</td>
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<td><strong>Yielding Function</strong></td>
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<tr>
<td>$q_k - \sin \phi_{\text{pp}}' p_k' F_k$ if $r_k \leq 0$</td>
<td>$</td>
</tr>
<tr>
<td>$F_k = 1 - b \ln \left( \frac{p_k'}{p_c} \right)$</td>
<td>$F = 1 - b \ln \left( \frac{\sigma'<em>{\text{in}}}{\sigma</em>{\text{in}}} \right)$</td>
</tr>
<tr>
<td>$p_c = p_{\text{co}} \exp(\beta \cdot \varepsilon'_{v})$</td>
<td>$\sigma_{\text{c}} = \sigma_{\text{co}} \exp(\beta \cdot \varepsilon_{\text{u}})$</td>
</tr>
<tr>
<td>$r_k = \varepsilon'<em>{v} + \varepsilon'</em>{v} F_k$</td>
<td>$r(\varepsilon_{\text{u}}) = \varepsilon_{\text{u}} + \frac{\varepsilon_{\text{u}}}{a + b \varepsilon_{\text{u}}}$</td>
</tr>
<tr>
<td>$a = a_1 + (a_2 - a_1) \alpha_k(r_k)$</td>
<td>$a = a_1 + (a_2 - a_1) \alpha(\varepsilon_{\text{u}})$</td>
</tr>
</tbody>
</table>

**Multimechanisms**

- $\varepsilon'_{v} = \lambda'_{v} \Psi_{v}$
- $\Psi_{v} = \alpha_{v} \alpha_{k}(r_k) \left( \sin \psi - \frac{q_k}{p_k} \right)$
- if $r_{\text{clas}} < r_k < r_{\text{hys}}$
  - $\alpha_k(r_k) = 0$
- if $r_{\text{hys}} < r_k < r_{\text{mob}}$
  - $\alpha_k(r_k) = \left( \frac{r_k - r_{\text{hys}}}{r_{\text{mob}} - r_{\text{hys}}} \right)^m$
- if $r_{\text{mob}} < r_k < 1$
  - $\alpha_k(r_k) = 1$

**Interface**

- $\varepsilon'_{n} = \lambda'_{p} \Psi_{n}$
- $\Psi_{n} = \alpha_{v} \alpha(r) \left( \tan \psi - \frac{r}{\sigma_{\text{in}}} \right)$
- if $r_{\text{clas}} < r < r_{\text{hys}}$
  - $\alpha(r) = 0$
- if $r_{\text{hys}} < r < r_{\text{mob}}$
  - $\alpha = \left( \frac{r - r_{\text{hys}}}{r_{\text{mob}} - r_{\text{hys}}} \right)^m$
- if $r_{\text{mob}} < r < 1$
  - $\alpha(r) = 1$

**Isotropic Mechanism**

- $f_{\text{iso}} = |p| - d \cdot p_c \cdot r_{\text{iso}}$
- $r_{\text{iso}} = \gamma_{\text{iso}} + \frac{\varepsilon'_{v} \cdot r_{\text{iso}}}{p_{\text{ref}} \cdot \varepsilon'_{v} \cdot r_{\text{iso}}}$
## Employed and developed Tools

- **ECP elastoplastic Constitutive models:**

### Parameters

<table>
<thead>
<tr>
<th>Model</th>
<th>Elasticity</th>
<th>Yield Function</th>
<th>Hardening</th>
<th>Threshold Domains</th>
<th>Initial State</th>
</tr>
</thead>
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<td>Multimechanism</td>
<td>$K_{ref}, G_{ref}$, $n_e, \rho_{ref}$</td>
<td>$\phi'_p, \beta, b, d$</td>
<td>$a_1, a_2, \psi, \alpha_p, m, c_1, c_2$</td>
<td>$r^{\text{ela}}, r^{\text{hys}}, r^{\text{mob}}, r^{\text{is}}$</td>
<td>$p_{co}$</td>
</tr>
<tr>
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<td>$E_{ref}, G_{ref}$, $n_e, \sigma_{ref}$</td>
<td>$\phi'_p, \beta, b$</td>
<td>$a_1, a_2, \psi, \alpha_p, m$</td>
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<tr>
<td>$p'<em>c = p'</em>{co} \exp(\beta \varepsilon''_{v_k})$</td>
<td>$\sigma'<em>c = \sigma'</em>{co} \exp(\beta u''_{p_n})$</td>
</tr>
<tr>
<td><strong>Deviatoric hardening</strong></td>
<td></td>
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<tr>
<td>$r'<em>k = r^{el}<em>k + \frac{\varepsilon''</em>{p_k}}{a + \varepsilon''</em>{p_k}}$</td>
<td>$r(u''<em>{p_n}) = r^{el} + \frac{u''</em>{p_n}}{u'<em>n + u''</em>{p_n}}$</td>
</tr>
<tr>
<td><strong>Plastic potential</strong></td>
<td></td>
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<tr>
<td>$\dot{\varepsilon''<em>{v_k}} = \dot{\lambda}'</em>{k} \Psi'_v$</td>
<td>$\dot{u''<em>{p_n}} = \dot{\lambda}'</em>{p} \Psi'_n$</td>
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<td>$\Psi'<em>{v} = \alpha</em>{psi} \alpha_k(r'_k) \left(\sin \psi - \frac{q_k}{p'_k}\right)$</td>
<td>$\Psi'<em>n = \alpha</em>{psi} \alpha(r) \left(\tan \psi - \frac{\tau}{\sigma'_n}\right)$</td>
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<tr>
<td>$\varepsilon''<em>{v</em>{iso}} = \sum_{k=1}^{3} \left(\varepsilon''<em>{v_k} + \varepsilon''</em>{p_{iso}}\right)$</td>
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</table>
II – Theory and Numerical modeling

Employed and developed Tools

- **3D Axisymmetric formulation of the ECP multimechanisms model:**

3D CV shear test (axisymmetric conditions)
Numerical Validation

- **GEFDYN FEM code for pile applications**
  - Soil: Mohr-Coulomb model
  - Interface: Mohr-Coulomb model
  - FEM codes: ABAQUS and CESAR (LCPC)

**Case 1**

- Trochanics [1991]

**Case 2**

- Neves et al. [2001]

Graphs showing load vs. settlement for different cases, comparing ABAQUS and GEFDYN results.
Simulation of Direct Shear Test

III – Interface behavior

- Boundary conditions and model

Case 1: CNL
Case 2: CV
Case 3: CNS
III – Interface behavior

Simulation of Direct Shear Test

(a) Simulation of shear stress $\tau_s$ (kPa) vs. displacement $u_s$ (mm) for $K=0$ and $K=\infty$.

(b) Simulation of shear stress $\tau_s$ (kPa) vs. normal stress $\sigma_n$ (kPa) for $K=0$ and $K=\infty$.

(c) Simulation of normal stress $\sigma_n$ (kPa) vs. displacement $u_n$ (mm) for $K=0$ and $K=\infty$.

(d) Simulation of normal stress $\sigma_n$ (kPa) vs. displacement $u_n$ (mm) for $K=0$ and $K=\infty$. 

Legend: CNL, CV, CNS
Simulation of Direct Shear Test

III – Interface behavior

- CNL test
- Influence of interface thickness \((t)\)

\[
\dot{u}_t = \dot{\gamma}_t \cdot t
\]

\[
\dot{u}_n = \dot{\varepsilon}_v \cdot t
\]
Simulation of Direct Shear Test

- Parameters determination: CNL test
- Influence of interface thickness ($t$)

\[
\dot{u}_t = \dot{\gamma}_t \cdot t \\
\dot{u}_n = \dot{\varepsilon}_v \cdot t
\]
Simulation of Direct Shear Test

• Parameters determination: **CNL test**
• influence of interface thickness \((t)\)

\[
G^* = G_{\text{soil}} \frac{t}{t}
\]
IV – Non-Displacement Piles

Parametric studies

Influence of:
- Soil initial state
- Pile-soil surface roughness
- Pile length

In:
- Shaft resistance
- Base resistance

Average load transfer coefficient: $\beta_{avg}$

$\beta_{avg} = \frac{Q_s}{(A_s \cdot \sigma_{vo avg})}$

Smooth interface:
No soil-pile interaction

$TS24, 40, 93$
IV – Non-Displacement Piles

Parametric studies

Pile length

Local load transfer coefficient: $\beta$

$\beta = \tau/\sigma_v$
Increasing number of cycles for mechanism 2

Reduction of the maximum resistance

Total Load
Cyclic Load transfer Mechanisms

Increasing number of cycles for mechanism 3

2 depths stress path

No stress \((\sigma_3=0)\)
IV – Non-Displacement Piles

Cyclic Load transfer Mechanisms

Increasing number of cycles for mechanism 1

(N=12, $Q_T = 200$ kN)
Compared performance of bored and CFA piles (E9 and T1):

- **Bored – E9**
  - Measured resistance
  - Simulated base resistance

- **CFA – T1**
  - Measured resistance
  - Simulated base resistance
Estimation of residual loads: **CFA pile**

- **Two steps modeling:**
  1) Base pre-loading with $s/d=5$
  2) Load removal → residual loads locked-in soil state modification

**Calculation 1: load+unload**

Static Load Tests
Estimation of residual loads: CFA pile

Residual load = 76 kN

- Experimental Static load test
- Simulated: load + unload
- Simulated: reload after load + unload

$Q_b$ - Base resistance

$Q_b = 76.4$ kN
Dynamic Load test

Drop height: $h$

Wave propagation

$V(t)$

$\Delta x$

$\Delta t$

$L \cdot c$

Wave refraction

$c = \frac{\Delta x}{\Delta t} = \sqrt{\frac{E}{\rho}}$

$F = Z \cdot v$

$Z = \frac{E \cdot A}{c}$

F, Force
v, velocity
Z, impendancy
Independent blows simulation

- Comparison between static and dynamic load tests
- Resistance mobilization – load settlement results

\[ Q_T = Q_s + Q_b \]
Sequentially applied blows

- Complete load history ("pre-blow" + blow 1,2,3 and 4)
- Load settlement response

\[ Q_T = Q_s + Q_b \]
Resistance after dynamic test

- Influence of the soil state and loading history: application of dynamic load modifies the pile static resistance

Simulation 1: “Virgin SLT”
Static load test from initial stress field
STATIC LOAD TEST

Simulation 2: “After DLT”
Static reload from stress field generated in the DLT
DYNAMIC LOAD TEST + STATIC LOAD TEST