The Hardening Soil model with small strain stiffness

in Zsoil v2011

Rafal OBRZUD

GeoMod Ing. SA, Lausanne
Content

- Introduction
- Framework of the Hardening Soil model
  - Hardening Soil SmallStrain
  - Hardening Soil Standard
- Model parameters
- Parameter Identification
- Assistance in Parameter Selection
- Practical applications
Why do we need advanced constitutive models?
Approximation of soil behaviour for reliable simulations of practical cases

Triaxial drained compression test

Plasticity (yielding) before reaching the ultimate state

- Introduction

Experiment: Texas sand
Simulation: Hardening Soil
Simulation: Mohr-Coulomb
Why do we need advanced constitutive models?

**ENGINEERING CALCULATIONS**

**LIMIT STATE ANALYSIS**
- Bearing capacity
- Slope stability

**DEFORMATION ANALYSIS**
- Pile, retaining wall deflection
- Supported deep excavations
- Tunnel excavations
- Consolidation problems

Basic models e.g. Mohr-Coulomb

Advanced soil models
Basic differences between implemented soil models

Mohr-Coulomb

$q$

$p'$

$K_0$-line

yield surface

Linear elastic domain

$q$

$p'$

$E = E_{ur}$

$E$

$E_{ur}$

$q$

$K_0$-unloading

Excavations
Practical applications of the Hardening Soil model

Tunneling

Standard Mohr-Coulomb  Hardening-Soil
Basic differences between implemented soil models

Mohr-Coulomb

Volumetric cap models

- Introduction
Practical applications of the Hardening Soil model

Berlin sand excavation

Standard Mohr-Coulomb

Hardening-Soil

The Hardening Soil model with small strain stiffness  - Introduction
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Basic differences between implemented soil models

Mohr-Coulomb

Volumetric cap models

Hardening Soil models

$K_0$-line

Linear elastic domain

$\sigma$-yield surface

isotropic hardening mechanism

+ shear hardening mechanism

NON-LINEAR elastic domain

Linear elastic domain

Stiffness degradation

Stiffness degradation

$E=E_{ur}$

$E\leq E_{ur}$

$E\leq E_{0}$
Small strain stiffness in geotechnical practice

e.g. Atkinson, Jardine and many others
Introduction

The Hardening Soil model with small strain stiffness
## Perceived application of ZSoil models

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<td>HS-Small Strain</td>
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<tr>
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<td>ULS</td>
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**ULS** - Ultimate Limit State analysis  
**SLS** - Serviceability Limit State analysis
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- Introduction
- **Hardening Soil model framework**
  - Hardening Soil SmallStrain
  - Hardening Soil Standard
- Model parameters
- Parameter Identification
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Hardening Soil model
to describe macroscopic phenomena exhibited by soils

**HS-Standard**

- **Densification** (decrease of voids volume in soil due to plastic deformations)
- **Soil stress history** (accounting for preconsolidation effects)
- **Plastic yielding** (irreversible strains with reaching a yield criterion)

Idealized plot of one-dimensional oedometric compression test.
Hardening Soil model to describe macroscopic phenomena exhibited by soils

**HS-Standard**

- **Densification** (decrease of voids volume in soil due to plastic deformations)
- **Soil stress history** (accounting for preconsolidation effects)
- **Plastic yielding** (irreversible strains with reaching a yield criterion)
- **Stress dependent stiffness** (increasing stiffness moduli with increasing depth or stress level)

![Triaxial drained compression test - Texas sand](image)

- SIG3 = 34.5kPa
- SIG3 = 138kPa
- SIG3 = 345kPa

**Graph depiction:**

- Deviatoric stress $q = \sigma_1 - \sigma_3$
- Axial strain $\varepsilon_1$
- Three different SIG3 values shown with different markers.
Hardening Soil model
to describe macroscopic phenomena exhibited by soils

HS-Standard

- **Densification** (decrease of voids volume in soil due to plastic deformations)
- **Soil stress history** (accounting for preconsolidation effects)
- **Plastic yielding** (irreversible strains with reaching a yield criterion)
- **Stress dependent stiffness** (increasing stiffness moduli with increasing depth or stress level)
- **Dilatation** (occurrence of negative volumetric strains during shearing)

+ **HS-SmallStrain** for handling stiffness in a small strain range

- **Stiffness variation** (degradation of $G_0$ modulus with increasing shear strain amplitudes between $\gamma \approx 0.0001 - 0.1\%$)

- **Hysteretic, non-linear stress-strain** relationship applicable in the range of small strains *

* HS-SmallStrain can be used to some extent to model hysteretic behavior under cycling loadings with the exception of gradual softening for increasing number of cycles.
Hardening Soil model in ZSoil

- Framework of HS model

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Hardening Soil model framework

Activation of $G_0$ degradation in the small strain range
Hardening Soil model framework

Triaxial test simulation

Secant stiffness in small strains

- Framework of HS model
Hardening Soil model framework

Triaxial test simulation with the unloading/reloading cycle

0,019 0,02 0,021 0,022 0,023 0,024
Deviatoric stress    q [kPa]

0 0,01 0,02 0,03 0,04 0,05
Deviatoric stress    q [kPa]

Axial strain $\varepsilon_1 [-]$
Main mechanical features of HS model

**HS-Standard**

- Smooth hyperbolic approximation of the stress-strain curve of soil (Duncan-Chang concept)

- Two plastic mechanisms:
  - isotropic (to handle important plastic volumetric strains observed in normally-consolidated soils)
  - deviatoric (to handle domination of plastic shear strains which are observed in coarse materials and heavily consolidated cohesive soils)

- Ultimate state described by Mohr-Coulomb criterion

- Rowe’s dilatancy

**HS-SmallStrain**

- Nonlinear elasticity which includes hysteretic effects (Hardin-Drnevich concept)
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- Hardening Soil model framework
  - *Hardening Soil SmallStrain*
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Non-linear elasticity for very small strains

Hyperbolic Hardin-Drnevich relation to describe S-shaped stiffness

for primary loading:
\[ \frac{G_s}{G_0} = \frac{1}{1 + a \frac{\gamma}{\gamma_{0.7}}} \]

for unloading/reloading:
\[ \frac{G_s}{G_0} = \frac{1}{1 + a \frac{\gamma}{2\gamma_{0.7}}} \]

\[ E_0 = 2(1 + \nu_{ur})G_0 \]

a = 0.385
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Hyperbolic approximation of the stress-strain curve

\( E_0 \) – tangent initial stiffness modulus

\( E_{50} \) – secant stiffness modulus corresponding to 50% of the ultimate deviatoric stress \( q_f \) described by Mohr-Coulomb criterion

\( E_{ur} \) – unloading/reloading modulus (parameter corresponding to \( r \) in Modified Cam Clay)

\[
q_a = \frac{q_f}{R_f}
\]

by default \( R_f = 0.9 \)

For most soils \( R_f \) falls between 0.75 and 1

\[
f_1 = \frac{q_a}{E_{50}} \left( \frac{q}{q_a} - q \right) - 2 \left( \frac{q}{E_{ur}} \right) - \gamma^{PS} \quad \text{for } q < q_f
\]
The Hardening Soil model with small strain stiffness - HS Standard

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Hyperbolic approximation of the stress-strain curve
Stress stiffness dependency

Triaxial drained compression test - Texas sand

- SIG3 = 34.5kPa
- SIG3 = 138kPa
- SIG3 = 345kPa

Deviatoric stress $q = \sigma_1 - \sigma_3$

Axial Strain $\varepsilon_1$
Stress stiffness dependency

$E_0^{ref}, E_{50}^{ref}, E_{ur}^{ref}$ correspond to reference minor stress $\sigma^{ref}$

$$E_{50} = E_{50}^{ref} \left( \frac{\sigma_3^* + c \cot \phi}{\sigma^{ref} + c \cot \phi} \right)^m$$

where $\sigma_3^* = \max(\sigma_3, \sigma_L)$

i.e. stiffness degrades with decreasing $\sigma_3$ up to the limit minor stress $\sigma_L$ which can be assumed by default $\sigma_L = 10$ kPa

In natural soils: $m = 0.3 - 1.0$

e.g.
Norwegian Sands and silts $\approx 0.5$ (Janbu, 1963)
Soft clays $m = 0.38 - 0.84$ (Kempfert, 2006)
1. Find three values of $E_{50}^{(i)}$ corresponding $\sigma_3^{(i)}$ to respectively

2. Find a trend line $y = ax + b$ by assigning variables

\[ y = \ln E_{50}^{(i)} \]
\[ x = \ln \left( \frac{\sigma^{(i)} + c \cot \phi}{\sigma^{ref} + c \cot \phi} \right) \]

3. Then the determined slope of the trendline $a$ is the parameter $m$
Stress stiffness dependency at initial state

\[ E_{50} = E_{50}^{\text{ref}} \left( \frac{\sigma_3^* + c \cot \phi}{\sigma_{\text{ref}} + c \cot \phi} \right)^m \]

- Level of reference stress
- \[ \sigma_{\text{ref}} \]
- \[ c \]
- \[ \phi \]
Stiffness exponent $m$

Geotechnical evidence  In natural soil, the exponent $m$ varies between 0.3 and 1.0. Janbu (1963) reported values of 0.5 for Norwegian sands ans silts.

*Figure 3.19:* Typical values for $m$ obtained for sands from triaxial test vs. initial porosity $n_0$.

*Figure 3.20:* Typical values for $m$ obtained for sands from oedometric test vs. initial porosity $n_0$. 
Unloading/reloading Poisson’s ratio $\nu_{ur}$

Experimental measurements from local strain gauges show that the initial values of Poisson’s ratio in terms of small mobilized stress levels $q/q_{\text{max}}$ varies between 0.1 and 0.2 for clays, sands.

![Typically elastic domain](image)

- Toyoura Sand (Dr=56%)
- Ticino Sand (Dr=77%)
- Pisa Clay (Drained Triaxial)
- Sagamihara soft rock
Unloading/reloading Poisson’s ratio $\nu_{ur}$

Therefore, the characteristic value for the elastic unloading/reloading Poisson’s ratio of $\nu_{ur} = 0.2$ can be adopted for most soils.
The Hardening Soil model with small strain stiffness - HS Standard

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Double hardening model

\[ f_2 = \frac{q^2}{M^2 r^2(\theta)} + p'^2 + p^2_c \]

\( r(\theta) \) – follows van Eekelen’s formula in order to assure a smooth and convex yield surface (cf. Modified Cam clay model)
Double hardening model

$\sigma_1$, $\sigma_2$, $\sigma_3$, $p'$

cap yield surfaces described with van Ekelen’s formula

Mohr-Coulomb failure surface

$p'$-q section through shear and cap yield surfaces

The Hardening Soil model with small strian stiffness - HS Standard
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Dilatancy

\[
\sin \psi_m = 0 \quad \text{if} \quad \phi_m < \phi_{cs} \quad \text{(cut-off in contractancy domain)}
\]

\[
\sin \psi_m = \frac{\sin \phi_m - \sin \phi_{cs}}{1 - \sin \phi_m \sin \phi_{cs}} \quad \text{if} \quad \phi_m \geq \phi_{cs} \quad \text{(Rowe’s dilatancy)}
\]

2The mobilized friction angle $\phi_m$ describes the stress ratio $\tau/\sigma$ (at the critical state $\phi_m = \phi_{cs}$).
The Hardening Soil model with small strain stiffness - HS Standard

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Since the cut-off for the contractancy domain could yield too small volumetric strain, the scaling parameter $D$ is introduced.

\[
\sin \psi_m = D \frac{\sin \phi_m - \sin \phi_{cs}}{1 - \sin \phi_m \sin \phi_{cs}}
\]

For $\sin \psi_m < \sin \phi_{cs}$:

$D = 0.25$

For $\sin \psi_m \geq \sin \phi_{cs}$:

$D = 1.00$
Parameters *M* and *H*

**Evolution of the hardening parameter** $p_c$:

$$dp_c = -H \left( \frac{p_c + c \cot \phi}{\sigma_{ref} + c \cot \phi} \right)^m$$

*H* controls the rate of volumetric plastic strains.

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The Hardening Soil model with small strain stiffness - HS Standard
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The Hardening Soil model with small strain stiffness

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**Parameters $M$ and $H$**

$M$ and $H$ must fulfil two conditions:

1. $K_0^{NC}$ produced by the model in oedometric conditions is the same as $K_0^{NC}$ specified by the user

2. $E_{oed}$ generated by the model is the same $E_{oed}^{ref}$ specified by the user
**Parameters $M$ and $H$**

**ASSUMPTIONS:**

1. At given $\sigma_{\text{oed}}^{\text{ref}}$ which is located at post-yield plastic curve, both shear an volumetric mechanism are active.

2. Initial stress point

\[ p^* = \frac{1 + 2K_0^{NC}}{3} \sigma_{\text{oed}}^{\text{ref}} \quad \text{and} \quad q^* = (1 - K_0^{NC})\sigma_{\text{oed}}^{ref} \]

3. A strain driven oedometer test is run with vertical strain amplitude $\Delta \varepsilon = 10^{-5}$ and the tangent oedometric modulus is computed as $E_{\text{oed}} = \delta \sigma/\delta \varepsilon \cong \Delta \sigma/\Delta \varepsilon$

4. Optimisation of $M$ and $H$ must fulfil two conditions:

- $K_0^{NC}$ produced by the model in oedometric conditions = $K_0^{NC}$ specified by the user
- $E_{\text{oed}}^{\text{ref}}$ generated by the model = $E_{\text{oed}}^{\text{ref}}$ specified by the user
Initial state variables - OCR

Notion of overconsolidation ratio:

\[ OCR = \frac{\sigma'_v}{\sigma'_v} \]

\( \sigma'_v \) – current in situ stress

\( \sigma'_v \) – past vertical preconsolidation pressure

NB. In natural soils, overconsolidation may stem from mechanical unloading such as erosion, excavation, changes in ground water level, or due to other phenomena such as desiccation, melting of ice cover, compression and cementation.
Initial state variables – \( OCR \) and \( K_0 \)

Normally-consolidated soil (\( OCR=1 \))

Overconsolidated soil (\( OCR>1 \))

Why does it appear and what should be inserted?

The Hardening Soil model with small strain stiffness - HS Standard
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Initial state variables – $K_0$ and $K_0^{NC}$

- **A**
  - Vertical effective stress: $\sigma'_v$
  - Horizontal effective stress: $\sigma'_h$

- **B**
  - Excavation
  - Vertical effective stress: $\sigma'_v$
  - Horizontal effective stress: $\sigma'_h$

**Graphical Representation**

- Vertical effective stress vs. Horizontal effective stress
  - **A** to **B**: Loading
  - **B** to **A**: Unloading
  - **K$_0^{NC}$-line**

The Hardening Soil model with small strian stiffness - HS Standard
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Initial state variables – $K_0$ and $K_0^{NC}$

$\sigma'_{SR}$ Preconsolidation stress configuration corresponding to $K_0^{NC}$

$\sigma'_0$ Current *in situ* stress configuration corresponding to $K_0$
(at the beginning of numerical simulation)
Initial state variables – $K_0$ vs. OCR

Estimation of earth pressure at rest

**Normally-consolidated soils**

\[
K_0^{NC} = 1 - \sin \phi'
\]

\[
K_0^{NC} = (\sqrt{2} - \sin \phi')/(\sqrt{2} + \sin \phi') \quad \text{(Simpson, 1992)}
\]

\[
K_0^{NC} = 0.95 - \sin \phi' \quad \text{Brooker & Ireland (1965)}
\]

**Overconsolidated soils**

\[
K_0 = K_0^{NC} \cdot OCR^m
\]

$m = 0.5 \quad \text{suggested by Meyerhof (1976)}$

$m = \sin \phi' \quad \text{suggested in Kulhawy & Mayne (1982)}$
Initial state variables – $K_0$ and $K_0^{NC}$

For most cases when soil was subject to “natural” consolidation before unloading $K_0^{SR} = K_0^{NC}$
Initial state variables – $K_0$ and $K_0^{NC}$

In the case of simulating the triaxial compression test after isotropic loading/unloading:

$K_0^{SR} \neq K_0^{NC}$

but

$K_0^{SR} = 1$

Isotropic loading/unloading
Initial state variables

1. User specifies initial stress conditions $\sigma'_0$
2. Zsoil at the beginning of a FE analysis calculates $\sigma^{SR}$ with

$$\sigma^{'SR} = \sigma'_y \cdot OCR \quad \text{or} \quad \sigma^{'SR} = \sigma'_y + q^{POP}$$

and the horizontal stress components

$$\sigma_x^{SR} = \sigma_y^{SR} K_0^{SR} \quad \text{and} \quad \sigma_z^{SR} = \sigma_y^{SR} K_0^{SR}$$

3. Then the hardening parameter $p_{c0}$ is computed
Setting initial state variables

1. Through $K_0$

\[ \sigma_y = \gamma \cdot \text{"depth"} \]
\[ \sigma_x = \sigma_x \cdot K_0(x) \]

2. Through Initial Stress
Setting initial state variables

If the model does not converge at Initial State for the specified $K_0$

1. try to start Initial State analysis from a small Initial loading Fact. and Increment

2. otherwise, define initial conditions through Initial Stress
Initial state variables - preconsolidation

1. through $OCR$ (gives constant OCR profile)

At the beginning of FE analysis, Zsoil sets the stress reversal point (SR) with:

$$\sigma_y^{SR} = \sigma_y \cdot OCR$$

or

$$\sigma_y^{SR} = \sigma_y' + q^{POP}$$

and

$$\sigma_x^{SR} = \sigma_y^{SR} K_0^{SR}$$

and

$$\sigma_z^{SR} = \sigma_y^{SR} K_0^{SR}$$

2. through $q^{POP}$ (gives variable OCR profile)
Stress history through OCR
Deposits with constant OCR over the depth (typically deeply bedded soil layers)
Deposits with varying OCR over the depth (typically superficial soil layers)

Profiles for Bothkennar clay
Stress history through $q^{\text{POP}}$

Deposits with varying OCR over the depth (typically superficial soil layers)

$$K_0^{\text{OC}} = K_0^{\text{NC}} \sqrt{\text{OCR}}$$

$$K_0^{\text{OC}} = K_0^{\text{NC}} \text{OCR}^{\sin(\phi)}$$

(Mayne & Kulhawy 1982)

Variable $K_0$ can be introduced merely through Initial Stress option.
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### Hardening Soil model

#### List of parameters to be provided by the user (page 1)

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<td>2. $\gamma_{0.7}$</td>
<td>Triaxial test with local gauges, geotechnical evidence</td>
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<td>Triaxial test</td>
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<td>4. $E_{ur}^{\text{ref}}$ kPa</td>
<td>Triaxial test</td>
</tr>
<tr>
<td>5. $\nu_{ur}$</td>
<td>Triaxial test</td>
</tr>
<tr>
<td>6. $m$</td>
<td>Triaxial test</td>
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</table>

<table>
<thead>
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<th>Auxiliary variable</th>
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</thead>
<tbody>
<tr>
<td>7. $\sigma_3^{\text{ref}}$ kPa</td>
<td>Triaxial test</td>
</tr>
</tbody>
</table>
# Hardening Soil model

## List of parameters to be provided by the user (page 2)

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<td><strong>Shear mechanism</strong></td>
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<tr>
<td>8.</td>
<td>$c$</td>
<td>kPa</td>
<td>Triaxial test, in situ test</td>
</tr>
<tr>
<td>9.</td>
<td>$\phi$</td>
<td>°</td>
<td>Triaxial test, in situ test</td>
</tr>
<tr>
<td>10.</td>
<td>$(R_f)$</td>
<td>-</td>
<td>Triaxial test or the default value 0.9</td>
</tr>
<tr>
<td>11.</td>
<td>$\psi$</td>
<td>°</td>
<td>Triaxial test</td>
</tr>
<tr>
<td>12.</td>
<td>$e_{\text{max}}$</td>
<td>°</td>
<td>Triaxial test on dense granular or overconsolidated cohesive soil</td>
</tr>
<tr>
<td><strong>Volumetric (cap) mechanism</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13.</td>
<td>$E_{\text{oed}}^{\text{ref}}$</td>
<td>kPa</td>
<td>Oedometric test</td>
</tr>
<tr>
<td>14.</td>
<td>$\sigma_{\text{oed}}^{\text{ref}}$</td>
<td>kPa</td>
<td>Oedometric test</td>
</tr>
<tr>
<td><strong>Initial state variable</strong></td>
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<td></td>
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<tr>
<td>15.</td>
<td>OCR / $q^{\text{POP}}$</td>
<td>- / kPa</td>
<td>Oedometric test or <em>in situ</em> tests</td>
</tr>
<tr>
<td>16.</td>
<td>$K_0^{SR}$</td>
<td>-</td>
<td>$K_0^{SR} = 1 - \sin \phi$ (or Ko-consolidation triaxial test)</td>
</tr>
<tr>
<td>17.</td>
<td>$\sigma_0$</td>
<td>kPa</td>
<td>Init. stress conditions accounting for $K_0 = K_0^{SR}$ (for normally-consolidated soil), $K_0 = K_0^{OC}$ (for overconsolidated soil)</td>
</tr>
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# Hardening Soil model

## Comparison of corresponding soil parameters for selected ZSoil models

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<th>Corresponding soil parameters</th>
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<th>Cap</th>
<th>Mohr-Coulomb</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small strain stiffness</td>
<td>$E_0$ and $\gamma_{0.7}$</td>
<td>none</td>
<td>none</td>
</tr>
</tbody>
</table>
| Elastic constants            | $E_{ur}$ and $\nu_{ur}$  
    $E_{50}$ and $m$ | $E$ and $\nu$ | $E$ and $\nu$ |
| Failure criterion            | $\phi$ and $c$ | $\phi$ and $c$ | $\phi$ and $c$ |
| Dilatancy                    | $\psi$ and $e_{max}$ | $\psi$ | $\psi$ and $e_{max}$ |
| Cap surface parameters       | $E_{oed}$ (can be determined from $\lambda$)  
    $OCR$, $K_0$ and $K_0^{SR}$ | $\lambda$ | None |
|                              | $OCR$, $K_0$ | $OCR$, $K_0$ | $K_0$ |
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Parameter identification

Report

THE HARDENING SOIL MODEL -
A PRACTICAL GUIDEBOOK

ZSoil PC 100701 report
revised 17.03.2011

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The Hardening Soil model with small strian stiffness - Parameter identification
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
The HS model – a practical guidebook

Contents:

- Theory
- Parameter identification
- Parameter estimation
Parameter identification

Stiffness parameters

Determination of $G_0$ from geophysical tests SCPT, SDMT and others

\[ G_0 = \rho V_s \quad E_0 = 2(1 + \nu_{ur})G_0 \]

with $\rho$ denoting density of soil and $V_s$ shear wave velocity. ($\nu=0.15..0.25$ for small strains)

In situ tests with seismic sensors:

• seismic piezocone testing (SCPTU)  
  (Campanella et al. 1986)

• seismic flat dilatometer test (SDMT)  
  (Mlynarek et al. 2006, Marchetti et al. 2008)

Geophysical tests: (see review by Long 2008)

• continuous surface waves (CSW)
• spectral analysis of surface waves (SASW)
• multi channel analysis of surface waves (MASW)
• frequency wave number (f-k) spectrum method
Parameter identification
Stiffness parameters

Determination of $G_0$ for sands from CPT

$$E_0 = 2(1 + \nu_{ur})G_0$$

after Robertson and Campanella (1983)
Parameter identification
Stiffness parameters

Estimation of $E_0$ from $E_{50}$

$E_{50} = 0.26E_0$
$E_{50} = 0.06E_0$

Observed secant stiffness modulus reduction curves from static torsional and triaxial shear data on clays and sands (from Mayne, 2007).
Parameter identification
Stiffness parameters

Estimation of $E_0$ from $E_{50}$

Cone resistance vs. maximal shear modulus $G_0$ for sands (after Robertson and Campanella, 1983).

\[ \frac{G_s}{G_0} = 1 - \left( \frac{q}{q_{\text{max}}} \right)^g \]  

where the exponent $g \approx 0.3 \pm 0.1$ fits uncemented, insensitive and not highly structured soils.
Parameter identification
Stiffness parameters

Approximative relation between "static" soil stiffness (here $E_s \approx E_{ur}$) and "dynamic" modulus $E_d$ corresponding to $E_0$ proposed by Alpan (1970).
HS-SmallStrain
Non-linear elasticity for small strains

Cohesionless soils
Influence of voids ratio

Influence of confining stress $p'$
after Wichtmann & Triantafyllidis

\[ \gamma_{0.7} = 8.3 \cdot 10^{-5} \frac{p'}{p_{\text{ref}}} + 1.1 \cdot 10^{-4} \]
where $p_{\text{ref}} = 100\text{kPa}$.
HS-SmallStrain
Non-linear elasticity for small strains

Cohesive soils

Influence of soil plasticity

\[ \gamma_{0.7} = 5 \cdot 10^{-6} I_P + 1 \cdot 10^{-4} \]

approximation proposed by Stokoe et al.

after Vucetic and Dobry (from Benz, 2007)
Parameter identification
Stiffness parameters

Determination of $E_{50}$ for sands from CPT

after Robertson and Campanella (1983)
Parameter selection

Stiffness parameters

Undrained vs drained moduli – theoretical relationship

Since the shear modulus is not affected by the drainage conditions

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

where $\nu_u$ is the Poisson’s coefficient in undrained conditions $\nu_u = 0.5$

$$\frac{E_u}{E} = \frac{3}{2(1 + \nu)}$$

for the drained Poison’s coefficient ranging for most soils between 0.12 and 0.4:

$$\frac{E_u}{E} \approx 1.07 \text{ to } 1.34$$
Parameter identification
Oedometric modulus $E_{oed}$

$$E_{oed} = \left( \frac{1 + e^{\text{ref}}}{C_c} \right) \sigma^*$$

where $C_c$ is the compression index

$$\sigma^* = \frac{\Delta\sigma'}{\log_{10} \left( \frac{\sigma_{oed}^{\text{ref}} + \Delta\sigma'}{\sigma_{oed}^{\text{ref}}} \right)}$$

Since we look for tangent $E_{oed}$, $\Delta\sigma' \to 0$, and

$$\sigma^* = 2.303\sigma_{oed}^{\text{ref}}$$

$$E_{oed} = \frac{2.3(1 + e^{\text{ref}})}{C_c} \sigma_{oed}^{\text{ref}}$$

If $\lambda$ is known

$$C_c = 2.3\lambda$$
**Parameter selection - oedometric modulus $E_{oed}$**

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

but

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

$\sigma_{oed}^{ref} = \sigma^{ref} / K_{0}^{NC}$

$\sigma^{ref}$

шение between triaxial stiffness moduli and oedometric moduli for three lacustrine clays in Germany, from Kemfert (2006).

<table>
<thead>
<tr>
<th></th>
<th>$E_{\varepsilon}/E_{oed}$</th>
<th>$E_{50}/E_{oed}$</th>
<th>$E_{ur}/E_{oed,ur}$</th>
<th>$E_{oed,ur}/E_{oed}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil 1</td>
<td>2.08</td>
<td>1.03</td>
<td>2.33-2.52</td>
<td>2.60</td>
</tr>
<tr>
<td>Soil 2</td>
<td>1.63</td>
<td>0.77</td>
<td>1.29-2.09</td>
<td>3.63</td>
</tr>
<tr>
<td>Soil 3</td>
<td>2.82</td>
<td>1.45</td>
<td>1.32-2.51</td>
<td>6.65</td>
</tr>
<tr>
<td>Average</td>
<td>2.17</td>
<td>1.08</td>
<td></td>
<td>4.29</td>
</tr>
</tbody>
</table>

$E_{\varepsilon}$ was derived from the initial slope of the triaxial curve $\varepsilon_1 - q$

$E_{oed,ur}$ denotes unloading/reloading oedometer modulus
Parameter identification
Preconsolidation pressure and OCR

Laboratory:
- Oedometer test (Casagrande’s method, Pacheco Silva method (1970))

Field tests:
- Static piezocone penetration (CPTU) (see reports by Mayne)
- Marchetti flat dilatometer (DMT) (correlations by Marchetti (1980), Lacasse and Lunne, 1988)
Parameter identification
Preconsolidation pressure and OCR

\[ OCR = k_{\sigma_t} \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \]

\[ OCR = k_{\sigma_e} \left( \frac{q_t - u_2}{\sigma'_{v0}} \right) \]

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Geographical region</th>
<th>Number of sites/points</th>
<th>( k_{\sigma_t} )</th>
<th>( R^2 )</th>
<th>Number of sites/points</th>
<th>( k_{\sigma_e} )</th>
<th>( R^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>[1]</td>
<td>Sweden</td>
<td>9/110</td>
<td>0.292</td>
<td>-</td>
<td>9/110</td>
<td>0.50</td>
<td>-</td>
</tr>
<tr>
<td>[2]</td>
<td>Canada</td>
<td>31/153</td>
<td>0.294</td>
<td>0.90</td>
<td>31/153</td>
<td>0.546</td>
<td>0.96</td>
</tr>
<tr>
<td>[3]</td>
<td>Worldwide</td>
<td>123/1121</td>
<td>0.305</td>
<td>0.84</td>
<td>84/811</td>
<td>0.50</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table 3.10: Comparison of the empirical coefficients obtained from multiple regression analyzes for non-fissured clays.

Example CPTU results showing excellent profiling capability (after Zuidberg et al., 1982).

(from Lunne et al. 1997)
Parameter identification
Preconsolidation pressure and OCR

DMT. Based on dilatometer measurements, estimation of OCR for clays can be carried out with the formula proposed by Marchetti which relates the horizontal stress index $K_D$ to OCR from oedometer tests with the following correlation:

$$OCR = (0.5K_D)^{1.56}$$  \hspace{1cm} (3.40)

The application of this correlation is restricted to materials with $I_D < 1.2$, free of cementation which have experienced simple one-dimensional stress histories (Totani et al., 2001).

An improved relationship which takes into account a large range of soil plasticity in the exponent was proposed by Lacasse and Lunne (1988):

$$OCR = 0.225K_D^{1.35 - 1.67}$$ \hspace{1cm} (3.41)

where the exponent varies from 1.35 for plastic clays, up to 1.67 for low plasticity materials.

![Figure 3.26: Various correlations $K_D - OCR$ for cohesive soils from various geographical areas (from Totani et al., 2001).](image-url)
## Parameter identification

### Other information from DMT

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>DESCRIPTION</th>
<th>BASIC DMT REDUCTION FORMULAE</th>
</tr>
</thead>
<tbody>
<tr>
<td>$p_0$</td>
<td>Corrected First Reading</td>
<td>$p_0 = 1.05 (A - Z_M + \Delta A) - 0.05 (B - Z_M - \Delta B)$</td>
</tr>
<tr>
<td>$p_1$</td>
<td>Corrected Second Reading</td>
<td>$p_1 = B - Z_M - \Delta B$</td>
</tr>
<tr>
<td>$I_D$</td>
<td>Material Index</td>
<td>$I_D = \frac{(p_1 - p_0)}{(p_0 - u_0)}$</td>
</tr>
<tr>
<td>$K_D$</td>
<td>Horizontal Stress Index</td>
<td>$K_D = \frac{(p_0 - u_0)}{\sigma_{v0}}$</td>
</tr>
<tr>
<td>$E_D$</td>
<td>Dilatometer Modulus</td>
<td>$E_D = 34.7 \left(p_1 - p_0\right)$</td>
</tr>
<tr>
<td>$K_0$</td>
<td>Coeff. Earth Pressure in Silt</td>
<td>$K_{0,DMT} = (K_0 / 1.5)^{0.47} - 0.6$</td>
</tr>
<tr>
<td>OCR</td>
<td>Overconsolidation Ratio</td>
<td>$\text{OCR}_{DMT} = (0.5 \cdot K_0)^{1.25}$</td>
</tr>
<tr>
<td>$C_u$</td>
<td>Undrained Shear Strength</td>
<td>$C_{u,DMT} = 0.22 \sigma'_{v0} (0.5 K_0)^{1.25}$</td>
</tr>
<tr>
<td>$\Phi$</td>
<td>Friction Angle</td>
<td>$\Phi_{safe,DMT} = 28^\circ + 14.6^\circ \log K_0 - 2.1^\circ \log^2 K_0$</td>
</tr>
<tr>
<td>$C_h$</td>
<td>Coefficient of Consolidation</td>
<td>$C_{h,DMTA} = 7 \text{ cm}^2 / t_{exp}$</td>
</tr>
<tr>
<td>$k_h$</td>
<td>Coefficient of Permeability</td>
<td>$k_h = c_h \gamma / M_h \ (M_h \approx K_0 \cdot M_{DMT})$</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Unit Weight and Description</td>
<td>(see chart in Fig. 16)</td>
</tr>
<tr>
<td>$M$</td>
<td>Vertical Drained Constrained Modulus</td>
<td>$M_{DMT} = R_M E_D$</td>
</tr>
<tr>
<td>$U_0$</td>
<td>Equilibrium Pore Pressure</td>
<td>$u_0 = p_2 = C - Z_M + \Delta A$</td>
</tr>
</tbody>
</table>

$Z_M =$ Gage reading when vented to atm. If $\Delta A$ & $\Delta B$ are measured with the same gage used for current readings $A$ & $B$, set $Z_M = 0$. $Z_M$ is compensated.

$E_D$ is NOT a Young's modulus $E$. $E_D$ should be used only AFTER combining it with $K_0$ (Stress History). First obtain $M_{DMT} = R_M E_D$, then e.g. $E = 0.8 M_{DMT}$.
Parameter identification
Strength parameters

then \( \phi = \arcsin \left( \frac{3M^*}{6 + M^*} \right) \)
\( c = c^* \frac{3 - \sin \phi}{6 \cos \phi} \) (3.10)

Compatibility of strength envelopes derived from drained and undrained triaxial tests (from Kempfert, 2006).

The Hardening Soil model with small strain stiffness  - Parameter identification
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
Parameter identification
Strength parameters

Friction angle $\phi$ for granular soils from \textit{in situ} tests

**SPT**

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$N_{30}$, blows/0.3m</th>
<th>Friction angle, $\phi$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose sand</td>
<td>&lt; 4</td>
<td>&lt; 29</td>
</tr>
<tr>
<td>Loose sand</td>
<td>4-10</td>
<td>29-30</td>
</tr>
<tr>
<td>Medium sand</td>
<td>10-30</td>
<td>30-36</td>
</tr>
<tr>
<td>Dense sand</td>
<td>30-50</td>
<td>36-41</td>
</tr>
<tr>
<td>Very dense sand</td>
<td>&gt; 50</td>
<td>&gt; 41</td>
</tr>
</tbody>
</table>

**CPTU**

$$\phi' = \arctan \left[ 0.10 + 0.38 \log \left( \frac{q_t}{\sigma_{v0}} \right) \right]$$  

(Robertson & Campanella, 1983)

**DMT**

Upper bound  
$$\phi_{max}' = 31 + \frac{K_D}{(0.236 + 0.066K_D)}$$  

(Totani et al., 1999)

Lower bound  
$$\phi_{min}' = 28 + 14.6 \log K_D - 2.1(\log K_D)^2$$

with $K_D$ denoting horizontal stress index which is calculated based on the first dilatometer reading $p_0$, i.e. $K_D = (p_0 - u_0)/\sigma_{v0}$. 

---

The Hardening Soil model with small strian stiffness  - Parameter identification
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
Content

- Introduction
- Hardening Soil - Small Strain
- Hardening Soil – Standard
- Model Parameters
- Parameter Identification
- Assistance in Parameter Selection
- Practical applications
The Hardening Soil model with small strain stiffness

Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne

- Assistance in parameter selection

August 29-31 2011  Introduction to ZSOIL PC, short course, at EPFL, Lausanne, Switzerland
September 1-2 2011  NUMERICS IN GEOTECHNICS AND STRUCTURES, seminar & ZSOIL special lectures, at EPFL, Lausanne, Switzerland.


The Hardening Soil model with small strain stiffness

Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne

Initialization menu

Empty field reserved for numeric data (basic soil properties, specific soil properties, \textit{in situ} test data) in Zsoil v2012

- Assistance in parameter selection
Initialization menu

Non-cohesive soil setup

ZSoil + Structures

INITIALIZATION

Basic soil setup

Soil Behaviour Type:
- Unknown
- Gravel
- Sand
- Silt
- Clay

Stress History:
- Unknown
- Normally consolidated
- Lightly consolidated
- Overconsolidated
- Heavily overconsolidated

Soil Density:
- Unknown
- Very loose
- Loose
- Medium
- Dense
- Very dense

Gradation:
- Unknown
- Well-graded
- Poorly graded
- Silty
- Clayey

Particle shape:
- Unknown
- Angular
- Subangular
- Subrounded
- Rounded

-Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
Initialization menu

Cohesive soil setup

- Assistance in parameter selection

The Hardening Soil model with small strian stiffness
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The Hardening Soil model with small strain stiffness

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- Assistance in parameter selection

Initialization menu

Parameter selection assistance is based on statistical data and empirical correlations. It is the user's responsibility to verify the suitability of parameters for a given purpose, in particular by verifying reproducibility of available experimental results and by adjustment of parameters.

Basic parameter selection uses parameter correlations which are summarized in the report:
The Hardening Soil model with small strian stiffness - Assistance in parameter selection
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
The Hardening Soil model with small strain stiffness
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne

- Assistance in parameter selection
Parameter selection – secant modulus $E_{ur}$

Typical values of the stiffness modulus which are provided in most textbooks are referred to as the "static" stiffness modulus $E_s$.

Typical values for the "static" modulus $E_s$ [MPa] (compiled from Kezdi, 1974; Prat et al., 1995).

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Loose Min</th>
<th>Loose Max</th>
<th>Medium Min</th>
<th>Medium Max</th>
<th>Dense Min</th>
<th>Dense Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravels/Sand well-graded</td>
<td>30</td>
<td>80</td>
<td>80</td>
<td>100</td>
<td>100</td>
<td>200</td>
</tr>
<tr>
<td>Sand, uniform</td>
<td>10</td>
<td>30</td>
<td>30</td>
<td>50</td>
<td>50</td>
<td>80</td>
</tr>
<tr>
<td>Sand/Gravel silty</td>
<td>7</td>
<td>12</td>
<td>12</td>
<td>20</td>
<td>20</td>
<td>30</td>
</tr>
</tbody>
</table>

It can be assumed that they represent the values of the unloading/reloading modulus $E_{ur}$.

In the basic parameter selection, typical $E_{ur}$ is taken as $E_s$ and the reference pressure is assumed $\sigma_{ref}=100kPa$. 
Parameter selection – secant modulus $E_{50}$

In most practical cases:

$$\frac{E_{ur}^{\text{ref}}}{E_{50}^{\text{ref}}} \approx 3$$

In toolbox - ranges:

$$\frac{E_{ur}^{\text{ref}}}{E_{50}^{\text{ref}}} = 2 \text{ to } 4$$

*Table 3.8: Relationship between stiffness moduli derived from drained and undrained triaxial tests for three lacustrine clays in Germany, from Kempfert (2006).*

<table>
<thead>
<tr>
<th>Drainage conditions</th>
<th>$E_i/E_{50}$</th>
<th>$E_{ur}/E_i$</th>
<th>$E_{ur}/E_{50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>drained</td>
<td>2.02</td>
<td>3.20</td>
<td>5.93</td>
</tr>
<tr>
<td>undrained</td>
<td>1.48</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>drained</td>
<td>2.17</td>
<td>3.10</td>
<td>6.72</td>
</tr>
<tr>
<td>undrained</td>
<td>1.84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil 3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>drained</td>
<td>1.94</td>
<td>6.55</td>
<td>12.66</td>
</tr>
<tr>
<td>undrained</td>
<td>3.02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>drained</td>
<td>2.04</td>
<td>4.28</td>
<td>8.43</td>
</tr>
<tr>
<td>undrained</td>
<td>2.11</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$E_i$ was derived from the initial slope of the triaxial curve $\varepsilon_1 - q$
Parameter selection - oedometric modulus $E_{oed}$

- $E_{oed}^{ref} \approx E_{50}^{ref}$
- $E_{oed}^{ref}$
- $E_{50}^{ref}$
- $\sigma_{oed}^{ref} = \sigma^{ref} / K_0^{NC}$
- $\sigma^{ref}$

**Table 3.7:** Relationship between triaxial stiffness moduli and oedometric moduli for three lacustrine clays in Germany, from Kempiert (2006).

<table>
<thead>
<tr>
<th></th>
<th>$E_s / E_{oed}$</th>
<th>$E_{50} / E_{oed}$</th>
<th>$E_{ur} / E_{oed,ur}$</th>
<th>$E_{oed,ur} / E_{oed}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil 1</td>
<td>2.08</td>
<td>1.03</td>
<td>2.33 - 2.52</td>
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<td>0.77</td>
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</tr>
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<td>Soil 3</td>
<td>2.82</td>
<td>1.45</td>
<td>1.32 - 2.51</td>
<td>6.65</td>
</tr>
<tr>
<td>Average</td>
<td>2.17</td>
<td>1.08</td>
<td></td>
<td>4.29</td>
</tr>
</tbody>
</table>

$E_s$ was derived from the initial slope of the triaxial curve $\varepsilon_1 - q$

$E_{oed,ur}$ denotes unloading/reloading oedometer modulus

**- Assistance in parameter selection**

**The Hardening Soil model with small strian stiffness**

Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
Parameter selection – friction angle $\phi$

Multi-criterion selection depending on quantity of provided input data (“soil keywords”)

Keyword: Density

<table>
<thead>
<tr>
<th>State of compaction</th>
<th>Relative density $D_r$ [%]</th>
<th>$\phi$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>0-15</td>
<td>&lt; 25</td>
</tr>
<tr>
<td>Loose</td>
<td>15-35</td>
<td>25-30</td>
</tr>
<tr>
<td>Medium</td>
<td>35-65</td>
<td>30-37</td>
</tr>
<tr>
<td>Dense</td>
<td>65-85</td>
<td>37-43</td>
</tr>
<tr>
<td>Very dense</td>
<td>85-100</td>
<td>&gt; 42</td>
</tr>
</tbody>
</table>

*Table 3.7: Representative values of $\phi$ observed in sands (after Schmertmann, 1978).*

Keyword: Soil type, Density, Gradation
Content

- Introduction
- Hardening Soil - SmallStrain
- Hardening Soil – Standard
- Model Parameters
- Parameter Identification
- Assistance in Parameter Selection
- Practical applications
Practical applications – Excavation in Berlin sand

Engineering draft and the sequence of excavation
Practical applications – Excavation in Berlin sand

Seepage Elements + Fluid head BC

Interface wall-soil

Fictitious interface to model impermeable barrier

90m

30m 120m
Practical applications – Excavation in Berlin sand
Practical applications – Excavation in Berlin sand

Berlin sand excavation

Standard Mohr-Coulomb

Hardening-Soil

The Hardening Soil model with small strian stiffness - Practical applications
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
Practical applications – Excavation in Berlin sand

Berlin sand excavation

![Graph of excavation parameters](image)

- Practical applications
Practical applications – Shallow footing

Texas sand benchmark

- Practical applications
Practical applications – Shallow footing Texas sand benchmark

1. DMT data

2. OCR interpreted form DMT data

Variable profile
Practical applications – Shallow footing  Texas sand benchmark

3. Selecting $q^{POP}$

4. OCR based on $q^{POP}$

- **Practical applications**

- **Variable profile**
**Practical applications** – Shallow footing  Texas sand benchmark

5. Interpreting and Setting $K_0$

![Graph showing the relationship between $K_0$ and depth with data from DMT-1, DMT-2, and FE Model.]

**Variable $K_0$ can be introduced merely through Initial Stress option**

---

**The Hardening Soil model with small strian stiffness** - Practical applications
Rafal Obrzud, GeoMod SA
30.08.2011, Lausanne
Practical applications – Shallow footing
Texas sand benchmark

![Graph showing load vs. settlement for different models]

- Experiential
- Hardening Soil Small Strain
- Standard Mohr-Coulomb

Settlement [m] vs. Load [kN]
Summary

Hardening Soil-SmallStrain model:

- correctly reproduces a strong reduction of soil stiffness with increasing shear strain amplitudes
- is recommended for Serviceability Limit State analyses as it generally predicts soil behavior and field measurements more precisely than basic linear-elasticity models
- is essential for the engineering problems with unloading/reloading modes such as excavations, tunneling
- accounts for stress stress history which is crucial for problems such as footing, consolidation, water fluctuation
- is applicable to most soils as it accounts for pre-failure nonlinearities for both sand and clay type materials regardless of the overconsolidation state

The Hardening Soil model is not as difficult in practical use as you may think. When you run a simulation with the M-C model, as an exercise, try to run the Hardening Soil model as well.

Practical applications