PLAXIS implementation of hypoplasticity

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# Contents

1 HYPOPLASTICITY ........................................... 2
  1.1 Hypoplastic model for granular materials .................. 3
  1.2 Hypoplastic model for clays ................................ 6
  1.3 Intergranular strain concept ................................ 6

2 TIME INTEGRATION ........................................... 9

3 PLAXIS INPUT .............................................. 11
  3.1 Hypoplastic model for granular materials .................. 11
  3.2 Hypoplastic model for clays ................................ 13

4 TROUBLESHOOTING .......................................... 18

5 EVALUATION ................................................ 19
  5.1 Drained triaxial test ....................................... 19
  5.2 Submerged construction of an excavation .................... 19
  5.3 Settlement due to tunnel construction ....................... 22
  5.4 Building subjected to an earthquake ......................... 24

6 ACKNOWLEDGEMENT .......................................... 26
Chapter 1

HYPOPLASTICITY

Hypoplasticity is a particular class of incrementally non-linear constitutive models, developed specifically to predict the behaviour of soils. The basic structure of the hypoplastic models has been developed during 1990’s at the University of Karlsruhe. In hypoplasticity, unlike in elasto-plasticity, the strain rate is not decomposed into elastic and plastic parts, and the models do not use explicitly the notions of the yield surface and plastic potential surface. Still, the models are capable of predicting the important features of the soil behaviour, such as the critical state, dependency of the peak strength on soil density, non-linear behaviour in the small and large strain range, dependency of the soil stiffness on the loading direction, etc.

This is achieved by the hypoplastic equation being non-linear in the stretching tensor \( D \). The basic hypoplastic equation may be written as

\[
\dot{T} = L : D + N \|D\|, \tag{1.1}
\]

where \( \dot{T} \) is the objective (Jaumann) stress rate, \( D \) is the Euler’s stretching tensor and \( L \) and \( N \) are fourth- and second order constitutive tensors, respectively. The early hypoplastic models were developed by trial and error, by choosing suitable candidate functions from the most general form of isotropic tensor-valued functions of two tensorial arguments (Kolymbas [6]). An important step forward in developing the hypoplastic model was the implementation of the critical state concept. Gudehus [3] proposed a modification of Equation (1.1) to include the influence of the stress level (barotropy) and the influence of density (pyknostropy). The modified equation reads

\[
\dot{T} = f_s L : D + f_s f_d N \|D\|. \tag{1.2}
\]

Here \( f_s \) and \( f_d \) are scalar factors expressing the influence of barotropy and pyknostropy. The model by Gudehus [3] was later refined by von Wolffersdorff [11]
1.1. Hypoplastic model for granular materials

Chapter 1. HYPOPLASTICITY

to incorporate Matsuoka-Nakai critical state stress condition. This model is nowa-
days considered as a standard hypoplastic model for granular materials, and this
version is also implemented in PLAXIS.

Later developments focused on the development of hypoplasticity for fine grained
low for lower friction angles and independent calibration of bulk and shear stiffnesses.
Based on this model, and the ”generalised hypoplasticity” principle by Niemunis [9],
Mašín [7] developed a model for clays characterised by a simple calibration procedure
and capability of correct predicting the very small strain behaviour (in combination
with the ”intergranular strain concept” described later). This clay version is incor-
porated in PLAXIS.

The basic hypoplastic models characterised by Eq. (1.2) predict successfully the soil
behaviour in the medium to large strain range. However, in the small strain range
and upon cyclic loading they fail in predicting the high quasi-elastic soil stiffness.
To overcome this problem, Niemunis and Herle [10] proposed an extension of the
hypoplastic equation by considering additional state variable ”intergranular strain”
determine the direction of the previous loading. This modification, often denoted as
the ”intergranular strain concept”, is implemented in PLAXIS and can be used with
both the model for granular materials and the model for clays.

The rate formulation of the enhanced model is given by

\[
\dot{T} = \mathbf{M} : \mathbf{D}
\]  

(1.3)

where \(\mathbf{M}\) is the fourth-order tangent stiffness tensor of the material. The total strain
can be thought of as the sum of a component related to the deformation of interface
layers at intergranular contacts, quantified by the intergranular strain tensor \(\delta\); and
a component related to the rearrangement of the soil skeleton. For reverse loading
conditions and neutral loading conditions the observed overall strain is related only
to the deformation of the intergranular interface layer and the soil behaviour is
hypoelastic, whereas in continuous loading conditions the observed overall response
is also affected by particle rearrangement in the soil skeleton and the soil behaviour
is hypoplastic.

1.1 Hypoplastic model for granular materials

The hypoplastic model for granular materials has eight material parameters - \(\phi_c\),
\(h_s\), \(n\), \(e_{d0}\), \(e_{c0}\), \(e_{i0}\), \(\alpha\) and \(\beta\). Their calibration procedure has been detailed by
Figure 1.1: Influence of \( n \) (a) and \( h_s \) (b) on oedometric curves (Herle and Gudehus [4]).

Herle and Gudehus [4]. A somewhat simplified calibration procedure is described in the following. The critical state friction angle \( \phi_c \) can be obtained directly by the measurement of the angle of repose. The next two parameters \( h_s \) and \( n \) can be directly computed from oedometric loading curves. The parameter \( n \) controls the curvature of oedometric curve and \( h_s \) controls the overall slope of oedometric curve as is shown in Figs 1.1 (a) and (b). Having two states at the oedometric curve (Fig. 1.1), the parameter \( n \) can be calculated from

\[
\begin{align*}
n &= \frac{\ln(e_{p1}C_{c2}/e_{p2}C_{c1})}{\ln(p_{s2}/p_{s1})} \\
\end{align*}
\]

where mean stresses \( p_{s1} \) and \( p_{s2} \) can be calculated from axial stresses using the Jáky formula \( K_0 = 1 - \sin \phi_c \), and \( e_{p1} \) and \( e_{p2} \) are the void ratios corresponding to the stresses \( p_{s1} \) and \( p_{s2} \). Tangent compression indices corresponding to the limit values of the interval \( p_{s1} \) and \( p_{s2} \) (\( C_{c1} \) and \( C_{c2} \)) can be approximated by secant moduli between loading steps preceding and following the steps \( p_{s1} \) and \( p_{s2} \). The parameter \( h_s \) can then be obtained from

\[
\begin{align*}
h_s &= 3p_s \left( \frac{n e_p}{C_c} \right)^{1/n} \\
\end{align*}
\]

where \( C_c \) is a secant compression index calculated from limit values of the calibration interval \( p_{s1} \) and \( p_{s2} \); \( p_s \) and \( e_p \) are averages of the limit values of \( p \) and \( e \) for this interval.

The next three model parameters are the reference void ratios \( e_{d0}, e_{c0} \) and \( e_{i0} \), corresponding to the densest, critical state and loosest particle packing at the zero mean
stress. The reference void ratios $e_d$, $e_c$ and $e_i$ corresponding to the non-zero stress depend on the mean stress by formula due to Bauer [1]:

$$e_c = e_{c0} \exp \left[- \left( \frac{3p}{h_s} \right)^n \right]$$  \hspace{1cm} (1.6)

The dependency of the reference void ratios on the mean stress is demonstrated in Fig. 1.2.

Following Herle and Gudehus [4], initial void ratio $e_{max}$ of a loose oedometric specimen can be considered equal to the critical state void ratio at zero pressure $e_{c0}$. Void ratios $e_{d0}$ and $e_{i0}$, which are the next two parameters, can approximately be obtained from empirical relations. The physical meaning of $e_{d0}$ is void ratio at maximum density, void ratio $e_{i0}$ represents intercept of the isotropic normal compression line with $p = 0$ axis. Void ratio $e_{i0}$ can be obtained by multiplication $e_{c0}$ by a factor $1.2$. The ratio $e_{i0}/e_{c0} \approx 1.2$ was derived by Herle and Gudehus [4] considering skeleton consisting of ideal spherical particles.

The minimum void ratio $e_{d0}$ should be obtained by densification of a granular material by means of cyclic shearing with small amplitude under constant pressure. If such a test is not available, it can be approximated using an empirical relation, with $e_{d0}/e_{c0} \approx 0.4$.

The last two parameters $\alpha$ and $\beta$ should be calibrated by means of single-element simulations of the drained triaxial tests. The two parameters control independently different aspects of soil behaviour, namely the parameter $\beta$ controls the shear stiffness and $\alpha$ controls the peak friction angle.
1.2 Hypoplastic model for clays

The basic model [7] requires five parameters \((N, \lambda^*, \kappa^*, \varphi_c, \text{and } r)\). The parameters have the same physical interpretation as parameters of the Modified Cam clay model, and they are thus easy to calibrate based on standard laboratory experiments. The model parameters \(N\) and \(\lambda^*\) define the position and the slope of the isotropic normal compression line in the \(\ln(1 + e)\) vs. \(\ln p\) plane:

\[
\ln(1 + e) = N - \lambda^* \ln \frac{p}{p_r},
\]

where \(p_r = 1\) kPa is a reference stress; parameter \(\kappa^*\) defines the slope of the isotropic unloading line in the same plane. Their definition is demonstrated in Fig. 1.3 and their calibration using isotropic loading and unloading tests on reconstituted London Clay specimens by Gasparre [2] is demonstrated in Fig. 1.4 [8]. The last two parameters are the critical state friction angle \(\varphi_c\) and the parameter \(r\) that controls the shear stiffness. Due to the non-linear nature of the model, the parameter \(r\) needs to be calibrated by simulation of the laboratory experiments. With decreasing value of \(r\) the shear stiffness is increasing (Fig. 1.4b).

1.3 The intergranular strain concept (small strain behaviour)

The intergranular strain concept requires five additional parameters: \(R\) controlling the size of the elastic range, \(\beta_r\) and \(\chi\) controlling the rate of stiffness degradation,
1.3. Intergranular strain concept

Chapter 1. HYPOPLASTICITY

Figure 1.4: (a) Calibration of parameters $N$, $\lambda^*$ and $\kappa^*$ using isotropic tests on reconstituted London clay, (b) calibration of the parameter $r$ using undrained shear test on reconstituted London clay (exp. data from Gasparre [2]).

$m_R$ controlling the initial shear stiffness for the initial and reverse loading conditions and $m_T$ controlling the stiffness upon neutral loading conditions. When combined with the hypoplastic model for clays, the initial very-small-strain shear stiffness $G_0$ may be calculated from

$$G_0 \simeq \frac{m_R}{r \lambda^*} \bar{p}$$

(1.8)

The parameters should be calibrated by simulating the laboratory experiments with measurements of the small strain stiffness using local strain transducers and with measurements of the very-small-strain stiffness using dynamic methods (such as bender elements). The influence of the parameter $m_R$ and $\beta_r$ on the predicted small-strain shear stiffness curves is demonstrated in Fig. 1.5.
1.3. Intergranular strain concept

Figure 1.5: The influence of the parameters $m_R$ and $\beta_r$ on the predicted small-strain-stiffness behaviour (experimental data from Gasparre [2]).
Chapter 2

TIME INTEGRATION

Constitutive models are integrated using explicit adaptive integration scheme with local substepping. The constitutive model forms an ordinary differential equation of the form

\[
\frac{dy}{dt} = f(t, y)
\]

The equation is for finite time step size \( \Delta t \) solved using the Runge-Kutta method. Solutions that correspond to the second- and third- order accuracy of Taylor series expansion are given by

\[
y^{(2)}_{(t+\Delta t)} = y(t) + k_2
\]

\[
y^{(3)}_{(t+\Delta t)} = y(t) + \frac{1}{6} (k_1 + 4k_2 + k_3)
\]

where

\[
k_1 = \Delta t \ f \left(t, y(t)\right)
\]

\[
k_2 = \Delta t \ f \left(t + \frac{\Delta t}{2}, y(t) + \frac{k_1}{2}\right)
\]

\[
k_3 = \Delta t \ f \left(t + \Delta t, y(t) - k_1 + 2k_2\right)
\]

The accuracy of the solution is estimated following Fehlberg as the difference between the second- and third- order solutions. The time step size \( \Delta t \) is accepted, if

\[
err = \left\| y^{(3)}_{(t+\Delta t)} - y^{(2)}_{(t+\Delta t)} \right\| < TOL
\]
where $TOL$ is a prescribed error tolerance. If the step-size $\Delta t$ is accepted, $y^{(3)}_{(t+\Delta t)}$ is considered as a solution for the given time step and the new time step size $\Delta t^n$ is estimated according to Hull

$$\Delta t^n = \min \left[ 4\Delta t, 0.9\Delta t \left( \frac{TOL}{err} \right)^{1/3} \right]$$

If the step-size $\Delta t$ is not accepted, the step is re-computed with new time step size

$$\Delta t^n = \max \left[ \frac{\Delta t}{4}, 0.9\Delta t \left( \frac{TOL}{err} \right)^{1/3} \right]$$

In the case the prescribed minimum time step size or the prescribed maximum number of time substeps is reached, the finite element program is asked to reject the current step and to decrease the size of the global time step.
Chapter 3

INPUT OF PARAMETERS AND STATE VARIABLES IN PLAXIS

3.1 Hypoplastic model for granular materials

Parameters are specified in the PLAXIS input in the following order:

- Parameter 1 – critical state friction angle $\varphi_c$
- Parameter 2 – $p_t$ – shift of the mean stress due to cohesion. For the basic hypoplastic model set $p_t = 0$, but non-zero value of $p_t$ is needed to overcome problems with stress-free state.
- Parameters 3-9 – parameters of the basic hypoplastic model for granular materials $h_s, n, e_{d0}, e_{c0}, e_{d0}, \alpha, \beta$.
- Parameters 10-14 – the intergranular strain concept parameters ($m_R, m_T, R, \beta_r, \chi$). If $m_R = 0$ the intergranular strain concept is switched off and the problem is simulated using the basic hypoplastic model.
- Parameter 15 – not used.
- Parameter 16 – initial void ratio corresponding to the zero mean stress $e_0$ or initial void ratio $e$. If $Par(16) < 10$, then $e$ is calculated from the mean stress $p$ and from $e_0 = Par(16)$ using Bauer [1] formula. If $Par(16) > 10$, then $e = Par(16) - 10$. 
3.1. Hypoplastic model for granular materials

- Parameters 17-22 – initial values of the intergranular strain tensor \( \delta \) in Voigt notation \((\delta_{11}, \delta_{22}, \delta_{33}, 2\delta_{12}, 2\delta_{13}, 2\delta_{23})\).

**State variables:**

The routine uses 14 state variables:

- State v. 1-6 – intergranular strain tensor \( \delta \) in Voigt notation \((\delta_{11}, \delta_{22}, \delta_{33}, 2\delta_{12}, 2\delta_{13}, 2\delta_{23})\).
- State v. 7 – void ratio \( e \).
- State v. 8 – not used.
- State v. 9 – Effective mean stress.
- State v. 10 – Number of evaluation of the constitutive model in one global time step (for postprocessing only).
- State v. 11 – Mobilised friction angle \( \varphi_{mob} \) in degrees (for postprocessing only).
- State v. 12 – Normalised length \( \rho \) of the intergranular strain tensor \( \delta \) (for postprocessing only).
- State v. 13 – Suggested size of the first time substep (for calculation control).
- State v. 14 – free.

The hypoplastic model for granular materials is implemented via user defined subroutine usermod.dll. To use the model in PLAXIS, copy the file UsrMod.dll into the PLAXIS installation directory. Then, select "user-defined model" from the Material model combo box in the General tab sheet (Fig. 3.1). After selecting the user-defined model, correct user-defined dynamic library (typically UsrMod.dll) needs to be selected in the "Available DLL’s" combo box under "Parameters" tab sheet. In the "Models in DLL" combo box, model with ID 1 (Hypoplas. - sand) must be selected. The parameters can then be input into the parameter table (Fig. 3.2).

Parameters of the sand hypoplastic model for different soils have been evaluated by Herle and Gudehus [4]. They are given in Table 3.1. Parameters of the intergranular strain concept for granular materials are in Tab. 3.2.
3.2 Hypoplastic model for clays

Parameters are specified in the PLAXIS input in the following order:

Parameters:

- Parameter 1 – critical state friction angle $\varphi_c$
- Parameter 2 – $p_t$ – shift of the mean stress due to cohesion. For the basic hypoplastic model set $p_t = 0$, but non-zero value of $p_t$ is needed to overcome problems with stress-free state.
- Parameters 3-6 – parameters of the basic hypoplastic model for clays $\lambda^*, \kappa^*, N, r$.
- Parameters 7-9 – free.
- Parameters 10-14 – the intergranular strain concept parameters ($m_R, m_T, R, \beta_r, \chi$). If $m_R = 0$ the intergranular strain concept is switched off and the problem is simulated using the basic hypoplastic model.
### 3.2. Hypoplastic model for clays

**Chapter 3. PLAXIS INPUT**

- **Parameter 15** – bulk modulus of water $K_w$ for undrained analysis using the penalty approach with user-defined value of $K_w$. In drained analysis, consolidation analysis, and undrained analysis using PLAXIS option undrained $K_w$ should be set to 0.

- **Parameter 16** – initial void ratio $e$ or overconsolidation ratio $OCR$. If $Par(16) < 10$, then $e = Par(16)$. If $Par(16) > 10$, then $OCR = Par(16) - 10$.

- **Parameters 17-22** – initial values of the intergranular strain tensor $\delta$ in Voigt notation ($\delta_{11}, \delta_{22}, \delta_{33}, 2\delta_{12}, 2\delta_{13}, 2\delta_{23}$).

**State variables:**

The routine uses 14 state variables:

- **State v. 1-6** – intergranular strain tensor $\delta$ in Voigt notation ($\delta_{11}, \delta_{22}, \delta_{33}, 2\delta_{12}, 2\delta_{13}, 2\delta_{23}$).

- **State v. 7** – void ratio $e$. 


---

Figure 3.2: Selecting sand hypoplasticity model in the "Parameters" tab sheet.
### 3.2. Hypoplastic model for clays

#### Table 3.1: Typical parameters of the hypoplastic model for granular materials (Herle and Gudehus [4])

<table>
<thead>
<tr>
<th>Material</th>
<th>$\varphi_c$</th>
<th>$h_s$</th>
<th>$n$</th>
<th>$e_{d0}$</th>
<th>$e_{e0}$</th>
<th>$e_{i0}$</th>
<th>$\alpha$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hochstetten gravel</td>
<td>36°</td>
<td>$3.2 \times 10^6$ kPa</td>
<td>0.18</td>
<td>0.26</td>
<td>0.45</td>
<td>0.5</td>
<td>0.1</td>
<td>1.9</td>
</tr>
<tr>
<td>Hochstetten sand</td>
<td>33°</td>
<td>$1.5 \times 10^6$ kPa</td>
<td>0.28</td>
<td>0.55</td>
<td>0.95</td>
<td>1.05</td>
<td>0.25</td>
<td>1.5</td>
</tr>
<tr>
<td>Hostun sand</td>
<td>31°</td>
<td>$1.0 \times 10^6$ kPa</td>
<td>0.29</td>
<td>0.61</td>
<td>0.96</td>
<td>1.09</td>
<td>0.13</td>
<td>2</td>
</tr>
<tr>
<td>Karlsruhe sand</td>
<td>30°</td>
<td>$5.8 \times 10^6$ kPa</td>
<td>0.28</td>
<td>0.53</td>
<td>0.84</td>
<td>1</td>
<td>0.13</td>
<td>1</td>
</tr>
<tr>
<td>Lausitz sand</td>
<td>33°</td>
<td>$1.6 \times 10^6$ kPa</td>
<td>0.19</td>
<td>0.44</td>
<td>0.85</td>
<td>1</td>
<td>0.25</td>
<td>1</td>
</tr>
<tr>
<td>Toyoura sand</td>
<td>30°</td>
<td>$2.6 \times 10^6$ kPa</td>
<td>0.27</td>
<td>0.61</td>
<td>0.98</td>
<td>1.1</td>
<td>0.18</td>
<td>1.1</td>
</tr>
</tbody>
</table>

#### Table 3.2: Parameters of the intergranular strain concept for sandy soils (Niemunis and Herle [10])

<table>
<thead>
<tr>
<th></th>
<th>$R$</th>
<th>$m_R$</th>
<th>$m_T$</th>
<th>$\beta_r$</th>
<th>$\chi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hochstetten sand</td>
<td>1.e-4</td>
<td>5.0</td>
<td>2.0</td>
<td>0.5</td>
<td>6</td>
</tr>
</tbody>
</table>

- State v. 8 – Excess pore pressure $u$ for undrained analysis using user-defined value of $K_w$. In undrained analysis using PLAXIS option *undrained* this variable is equal to 0 and excess pore pressure may be found in standard PLAXIS menu.

- State v. 9 – Effective mean stress.

- State v. 10 – Number of evaluation of the constitutive model in one global time step (for postprocessing only).

- State v. 11 – Mobilised friction angle $\varphi_{mob}$ in degrees (for postprocessing only).

- State v. 12 – Normalised length $\rho$ of the intergranular strain tensor $\delta$ (for postprocessing only).

- State v. 13 – Suggested size of the first time substep (for calculation control).

- State v. 14 – Free.

The hypoplastic model for granular materials is implemented via user-defined subroutine usermod.dll. To use the model in PLAXIS, copy the file UsrMod.dll into the PLAXIS installation directory. Then, select "user-defined model" from the Material model combo box in the General tab sheet (Fig. 3.1).
3.2. Hypoplastic model for clays

After selecting the user-defined model, correct user-defined dynamic library (typically UsrMod.dll) needs to be selected in the "Available DLL’s" combo box under "Parameters" tab sheet. In the "Models in DLL” combo box, model with ID 2 (Hypoplas. - clay) must be selected. The parameters can then be input into the parameter table (Fig. 3.3).

![User-defined model - Hypoplasticity - stiff clay - undrained](Image)

Figure 3.3: Selecting clay hypoplasticity model in the "Parameters" tab sheet.

Parameters of the clay hypoplastic model for different soils have been evaluated by Mašín and co-workers. They are given in Table 3.3. Typical parameters of the intergranular strain concept for fine-grained soils are in Tab. 3.4.
3.2. Hypoplastic model for clays

<table>
<thead>
<tr>
<th>Material</th>
<th>(\varphi_c)</th>
<th>(\lambda^*)</th>
<th>(\kappa^*)</th>
<th>(N)</th>
<th>(r)</th>
</tr>
</thead>
<tbody>
<tr>
<td>London clay</td>
<td>22.6°</td>
<td>0.11</td>
<td>0.016</td>
<td>1.375</td>
<td>0.4</td>
</tr>
<tr>
<td>Brno clay</td>
<td>19.9°</td>
<td>0.13</td>
<td>0.01</td>
<td>1.51</td>
<td>0.45</td>
</tr>
<tr>
<td>Fujinomori clay</td>
<td>34°</td>
<td>0.045</td>
<td>0.011</td>
<td>0.887</td>
<td>1.3</td>
</tr>
<tr>
<td>Bothkennar clay</td>
<td>35°</td>
<td>0.12</td>
<td>0.01</td>
<td>1.34</td>
<td>0.07</td>
</tr>
<tr>
<td>Pisa clay</td>
<td>21.9°</td>
<td>0.14</td>
<td>0.01</td>
<td>1.56</td>
<td>0.3</td>
</tr>
<tr>
<td>Beaucaire clay</td>
<td>33°</td>
<td>0.06</td>
<td>0.01</td>
<td>0.85</td>
<td>0.4</td>
</tr>
<tr>
<td>Kaolin</td>
<td>27.5°</td>
<td>0.11</td>
<td>0.01</td>
<td>1.32</td>
<td>0.45</td>
</tr>
<tr>
<td>London clay (data Gasparre)</td>
<td>21.9°</td>
<td>0.1</td>
<td>0.02</td>
<td>1.26</td>
<td>0.5</td>
</tr>
<tr>
<td>Kaolin</td>
<td>27.5°</td>
<td>0.07</td>
<td>0.01</td>
<td>0.92</td>
<td>0.67</td>
</tr>
<tr>
<td>Trmice clay</td>
<td>18.7°</td>
<td>0.09</td>
<td>0.01</td>
<td>1.09</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 3.3: Typical parameters of the hypoplastic model for clays

<table>
<thead>
<tr>
<th>Material</th>
<th>(R)</th>
<th>(m_R)</th>
<th>(m_T)</th>
<th>(\beta_r)</th>
<th>(\chi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>London clay [7]</td>
<td>1.e-4</td>
<td>4.5</td>
<td>4.5</td>
<td>0.2</td>
<td>6</td>
</tr>
<tr>
<td>London clay (data Gasparre)</td>
<td>5.e-5</td>
<td>9</td>
<td>9</td>
<td>0.1</td>
<td>1</td>
</tr>
<tr>
<td>Brno clay (nat.)</td>
<td>1.e-4</td>
<td>16.75</td>
<td>16.75</td>
<td>0.2</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Table 3.4: Typical parameters of the intergranular strain concept for clays
Chapter 4

TROUBLESHOOTING

Although the implementation has been tested to be accurate and robust, the hypoplastic models are highly non-linear which may cause problems during solving complex boundary value problems. When encountering problems, the following steps may help to improve the overall performance:

1. In the "Iterative procedure" settings under "Parameters" Tab sheet, set "Manual settings" and do not use the arc-length control.

2. Try to modify the iterative procedure by decreasing the "Desired minimum" and "Desired maximum" number of iterations (for example, 3 and 5 respectively).

3. When used in combination with structural elements using interface elements, try to decrease the interface stiffness in the "Interface" Tab sheet under "Material models" window.

4. The hypoplastic models are undefined in the tensile stress region, which can cause integration problems in the vicinity of the free surface and in the case of staged construction starting from the stress-free state. For this reason, artificial cohesion is introduced in the model implementation through the parameter $p_t$. The user can specify any $p_t$ value; in the case $p_t=0$ kPa, however, the program replaces it by a default value $p_t=10$ kPa, which is sufficient to overcome most problems due to the zero stress state.
Chapter 5

EVALUATION

5.1 Drained triaxial test

PLAXIS implementation of the hypoplastic models has been evaluated by means of simulations of a drained triaxial test. Fig. 5.1 shows the stress-strain curves of the drained triaxial test, compared with "exact" results obtained by accurate time integration of the model using a single-element program. The curves generated by the PLAXIS implementation practically coincide with the "exact" solution.

5.2 Submerged construction of an excavation

In this case, submerged construction of an excavation close to the river is simulated. The upper 20 m of the subsoil consist of soft soil layers, which are modelled as a single homogeneous clay layer using the clay hypoplastic model. Underneath this clay layer there is a stiffer sand layer, modelled using the sand hypoplasticity model. As in this case the displacement field is significantly influenced by the small-strain behaviour, hypoplastic models with the intergranular strain concept are used.

Soil parameters of the sand layer correspond to the Hochstetten sand from Tab. 3.1; the intergranular strain parameters used are in Tab. 3.2. The sand is in a medium dense state with $e_0=0.7$. The clay layer is simulated with London clay parameters (Tab. 3.3 for parameters of the basic model; Tab 3.4. for the intergranular strain parameters), with the initial value of OCR equal to 2.

Figure 5.2 shows the displacement field and Fig. 5.3 shows the normalised length.
of the intergranular strain tensor. The normalised length of the intergranular strain tensor varies between 0, which indicates the soil being inside the elastic range, and 1, corresponding to the state swept-out of the small-strain memory. In the case of the normalised length of the intergranular strain being equal to 1 the soil behaviour is governed by the basic hypoplastic model. Indeed, Fig. 5.3 shows that this is the case of the submerged construction of an excavation, where the soil is below the bottom of the excavation and behind the wall outside the small-strain-stiffness range.
5.2. Submerged construction of an excavation

Chapter 5. EVALUATION

Figure 5.2: Submerged construction of an excavation - total displacement.

Figure 5.3: Submerged construction of an excavation - normalised length of the intergranular strain tensor.
5.3 Settlement due to tunnel construction

In the next example, a shield tunnel excavated partly in soft clay and partly in medium dense sand is simulated, and its influence on a pile foundation is evaluated. The tunnel has a diameter of 5 m and is located at an average depth of 20 m. The soil profile indicates four distinct layers: The upper 13 m consists of soft clay type soil. Under the clay layer there is a 2 m thick sand layer, which is used as a foundation layer for the piles. Below this sand layer, there is another clay layer followed by a deep sand deposit.

As in the previous example, soil parameters of the sand layer correspond to the Hochstetten sand from Tab. 3.1; the intergranular strain parameters used are in Tab. 3.2. The sand is in a medium dense state with $e_0=0.7$. The clay layer is simulated with London clay parameters (Tab. 3.3 for parameters of the basic model; Tab 3.4. for the intergranular strain parameters), with the initial value of OCR equal to 2.

Figure 5.2 shows the displacement field and Fig. 5.3 shows the normalised length of the intergranular strain tensor. Again, the normalised length of the intergranular strain tensor in Fig. 5.3 demonstrates how the small-strain stiffness is activated in different parts of the modelled geometry. The soil is in "hypoplastic" state in the vicinity of the tunnel, above the tunnel and also next to the pile foundations, which bear the building weight. Further from the tunnel, the value of the normalised length of the intergranular strain tensor is low and at these places the soil remains elastic with high shear and bulk stiffnesses.
5.3. Settlement due to tunnel construction  

Chapter 5. EVALUATION

Figure 5.4: Settlement due to tunnel construction - total displacement.

Figure 5.5: Settlement due to tunnel construction - normalised length of the intergranular strain tensor.
5.4 Building subjected to an earthquake

This example demonstrates capabilities of a hypoplastic model in dynamic analysis of the earthquake impact on existing infrastructure. A real accelerogram of an earthquake recorded by USGS in 1989 is used for the analysis.

The building consists of 4 floors and a basement. It is 6 m wide and 25 m high. The subsoil consists of a sand with water level reaching the surface. The soil behaviour during the earthquake is considered as undrained. Two cases were simulated. In one case, the soil is in a loose state \( e_0 = e_{c0} \), in the second case the soil is in a dense state \( e_0 \) is close to \( e_{d0} \). Hypoplastic model parameters of the Hochstetten sand from Tab. 3.1 and the intergranular strain parameters from Tab. 3.2 are adopted.

Overall displacements of the top of the building are shown in Fig. 5.6. The soil response to the earthquake depends significantly on the soil state. The loose soil liquefies after 3-4 s of the earthquake, leading to the failure. The analysis cannot continue and fails. The displacements are much lower in this case of dense soil. Although some displacements occur also in this case, the soil retains some bearing capacity sufficient to overcome the failure.

![Figure 5.6: Displacement of the top of the building during the earthquake.](image)

Figure 5.7 shows total displacements after approx. 4 s of earthquake for the loose

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Figure 5.7 shows total displacements after approx. 4 s of earthquake for the loose
soil case, whereas Figure 5.8 shows the displacements at the same time (and in the same scale) for the dense soil. The figures indicate foundation failure for the loose soil case and relatively low displacements for the dense soil case.

Figure 5.7: Total displacements after 4 s of earthquake - loose soil.

Figure 5.8: Total displacements after 4 s of earthquake - dense soil.
Chapter 6

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