7 THE EFFECTS OF GROUND WATER RECHARGE ON THE HYDROLOGICAL MODELLING OF THE SATURATED ZONE OF THE BELINE SLOPE

7.1 Introduction

To establish the stability of a slope, the pore pressure distribution in the slope has to be known. A 2D groundwater model was made of the Beline slope using the Modflow code (McDonalds and Harbaugh, 1988) of release 1996. To study the effects of land use changes on slope stability, one has to quantify the influence of land use changes on the ground water behaviour, in terms of changes in ground water level fluctuations.

The aims of this chapter are:
- Modelling the spatial distribution of pore water pressure for slope stability calculations
- Finding an effective way to determine ground water recharge using precipitation data and unsaturated zone models
- Quantifying the influence of changes in land use or climate on the hydrological behaviour of the slope by means of ground water recharge modelling

In paragraph 7.2.1 the ground water model is described and its geometry and parameterisation is discussed. The ground water model is calibrated with the ground water level data of D1, D2 and D3 of 1996 and is validated with the 1997 data set (§ 7.2.2). The ground water model is tested on its capability to model ground water level fluctuations using hypothetical recharge time series. Then the focus shifts to the estimation of the recharge time series. In general, there is a focus on the schematisation and parameterisation of ground water models while limited attention is paid to the input variable: the recharge time series. These are constructed by means of the bottom flux of the unsaturated zone hydrological model and precipitation series (§ 7.2). To incorporate preferential flow, a methodology is proposed where recharge time series are constructed using a combination of soil moisture content and precipitation (§ 7.3). In paragraph 7.4 the influence of land use change scenarios on the hydrological behaviour of the Beline slope is quantified. The chapter finishes with a summary and discussion in which the aims of this chapter are evaluated.

7.2 The Beline saturated zone model

7.2.1 Model description and parameterisation

In the Hycosi project (Leroi, 1997), several ground water models have been reviewed (van Esch, 1995). This study showed the Modflow software to be one of most suitable hydrological software packages for describing pore pressure fluctuations in slopes. Modflow has been developed by the USGS (McDonald and Harbaugh, 1988) and is one of the most commonly used ground water models in the world. The physical part of the Modflow ground water modelling software is capable of modelling both steady-state and transient conditions in two and three dimensions and includes heterogeneity and
anisotropy of the subsurface layers. The model needs a pre- and postprocessor to help with input and output of the model. The widespread use of the Modflow software has the advantage that a large number of additional software is available, the so-called graphical user interfaces (GUI’s). In this study the PMWIN software (Chiang and Kinzelbach, 1996) is used as a pre- and post-processor. The PMWIN software also supports the automatic calibration program PEST (Doherty et al, 1994). Also the Excel spreadsheet software is used for both pre- and post-processing of the data.

The Beline model uses the digitised 1:25,000 IGN topographical map of the Salins-les-Bains region with a 25 by 25 m² discretisation as the digital elevation model of the surface. The 2D model is a SW-NE orientated transect along the steepest gradient of the slope with a cell length of 25 m (figure 7.1). The surface elevation was interpolated from the digital elevation model of the Beline slope. Along this transect three pore pressure measurement sites are located: D1, D2 and D3 (figure 7.1). The model covers an area, which extends from the river La Furieuse till the plateau of Clucy (see also chapter 4). The part southwest of the river La Furieuse was excluded from the Modflow model as was the plateau of Clucy.

Figure 7.1 Overview of the observation sites and a 2D ground water model cross-section at the Beline slope, Salins-les-Bains, France (see also figure 4.8).
The hydrological model schematisation (figure 7.2) was derived from the geological schematisation as presented in chapter 4. Three layers were identified using borehole information, drilling progress and geophysical surveys:
- a disturbed surface layer, with relatively low bulk density and abundant gravel content and with an average thickness of 5 m;
- a zone of remoulded marls with silty layers, a variable gravel content, an average bulk density and a variation in thickness of 5 m at the bottom of the slope to more than 10 m in the upper part of the slope;
- compact, blue marls, with generally no gravel, but with some limestone fragment concentrations.

The Beline ground water model was built up with these 3 layers only. Instead of the often used horizontal cell geometry with inclined definition of hydrological parameters (like saturated permeability), here the layers have variable elevation definition such that they coincide better with the slope parallel layering. The saturated permeability values of these layers decrease from the surface downwards. The saturated permeability was determined to be on average 0.129 m/d between 40 cm and 1.5 m below surface (see chapter 6). No saturated permeabilities could be obtained from the deeper lithologies. Table 7.1 gives an overview of the initial hydrological model parameters.

The surface was modelled using the drainage package of Modflow. By defining drains in the upper layer near the surface (20 cm below surface) with a high conductance, exfiltration of ground water was modelled. With the drain option water can exfiltrate but not re-infiltrate, so it is withdrawn from the hydrological process. The main advantage of the drain option for surface modelling over other options, like modelling air as a very permeable layer, is its numerical stability. Main disadvantage is the irreversible loss of...
Table 7.1  Initial values of the hydrological parameters of the Beline ground water model.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness [m]</th>
<th>Saturated Permeability [m/day]</th>
<th>Specific Yield [-]</th>
<th>Specific Storage [1/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>5</td>
<td>0.129</td>
<td>0.25</td>
<td>0.001</td>
</tr>
<tr>
<td>Layer 2</td>
<td>5 to 10</td>
<td>0.0129</td>
<td>0.2</td>
<td>0.001</td>
</tr>
<tr>
<td>Layer 3</td>
<td>40</td>
<td>0.00129</td>
<td>0.2</td>
<td>0.001</td>
</tr>
</tbody>
</table>

the seepage water. Re-infiltration, however, has never been observed on the Beline slope. The seepage that was found at the Beline site, was drained in the local sewage system. The average ground water recharge (upper boundary flux) was set at 0.75 mm/day, equal to the average bottom flux in the unsaturated zone model (chapter 6).

Analysis during the construction of the model learned that the horizontal and vertical discretisation (subdivision of a layer into more model layers with the same parameterisation) did not influence the model outcome. However, for the transmissivity of the layers the model is very sensitive. In this model approach the thickness of the layers was fixed and the saturated permeability was calibrated.

7.2.2  Model calibration and validation

The calibration of the ground water model was carried out in two steps. First, the calibration consists of modelling the steady-state ground water level using a transient ground water model with constant upper boundary flux. This first calibration phase leads to a set of permeability values for the three layers of the Beline slope and initial ground water levels for the second part of the calibration. The second calibration phase of the Beline ground water model was to tune the ground water fluctuations by adjusting the porosity related parameters.

The calibration was performed using the ground water level measurements of water pressure devices D1, D2 and D3. As described earlier, the average ground water level at D1 was 3.5 m, at D2 6.5 m and at D3 4.2 m below surface (figure 7.3). The final and steady-state ground water level will be used as initial values for the modelling of the measured ground water level fluctuations.

The calibration starts with an initial, horizontal ground water level equal to the river Furieuse (347 m, see figure 7.2), and stops when the model reaches a steady-state ground water level. Using the average recharge flux of 0.75 mm/day, saturated permeability values of 0.4 m/d for the first layer, 0.1 m/d for the second layer and 0.0075 m/d for the third and lowest layer were found, by method of trial and error. The calculated ground water level provides an average for the three observation points (figure 7.3). After the trial and error calibration the measured and modelled ground water depth are respectively 3.5 m and 3.6 m below surface for observation point D1, 6.4 m and 2.2 m below surface for D2 and 4.1 m and 8.0 m below surface for observation point D3. This parameterisation of the saturated permeability is not a unique solution. However, equivalent ground water levels can be obtained only when the saturated permeabilities do not deviate more than 25 % of the ones described above with the given schematisation and average recharge flux.
These large differences at D2 and D3 have several reasons. First of all, the result of the coarse interpolation of the surface height, which is caused by the 25*25 m², cells. On a small scale, large topographical differences exist at the Beline slope. Secondly, a ground water level error of a few meters is a relatively small error compared to the level change along the slope. Thirdly, there is the aspect of the uncomplicated subsurface schematisation of three continuous layers that results in a smooth water table whereas small-scale heterogeneity has important influence on the local ground water table.

The model’s capability to model ground water level fluctuations was then tested with a hypothetical recharge fluctuation (table 7.2). After reaching a steady-state ground water level, 25 stress periods (model time periods in which the values of the input parameters do not change) of one year followed in which the recharge fluctuates between 0.53 mm/day (30% decrease) and 0.98 mm/day (30% increase) in steps of 10% per model stress period. After this the model continues with monthly stress periods in which the recharge also fluctuates in steps of 10%. The model run ends with a stress period in which the average recharge flux (0.75 mm/day) is used again to check the numerical stability of the ground water model.

Figure 7.4 shows the results of this model run for the three observation boreholes D1, D2 and D3. The results show that the Beline ground water model is capable of modelling transient recharge fluxes, both for longer and shorter stress periods. The final heads are used as initial ground water heads in further analysis.
Table 7.2 List of the stress periods with hypothetical recharge values to test the Beline ground water model on its capability to model ground water level fluctuations.

<table>
<thead>
<tr>
<th>Stress Period</th>
<th>Period length [Days]</th>
<th>Recharge [mm/day]</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20000</td>
<td>0.75</td>
<td>Initiation</td>
</tr>
<tr>
<td>2</td>
<td>20000</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>20000</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>20000</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>360</td>
<td>0.75</td>
<td>Annual fluctuation</td>
</tr>
<tr>
<td>6</td>
<td>360</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>360</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>360</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>360</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>360</td>
<td>0.53</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>360</td>
<td>0.60</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>360</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>360</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>360</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>360</td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>360</td>
<td>0.98</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>360</td>
<td>0.9</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>360</td>
<td>0.83</td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>360</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>360</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>...</td>
<td>360</td>
<td>...</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>360</td>
<td>0.83</td>
<td>Monthly fluctuation</td>
</tr>
<tr>
<td>31</td>
<td>30</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>32</td>
<td>30</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>...</td>
<td>30</td>
<td>...</td>
<td></td>
</tr>
<tr>
<td>599</td>
<td>30</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>30</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>601</td>
<td>10000</td>
<td>0.75</td>
<td>Final average recharge</td>
</tr>
</tbody>
</table>

In the first calibration phase the saturated permeability of the three layers was adjusted. In the second calibration phase the ground water level fluctuation was calibrated by adjusting porosity related parameters (specific yield and specific storage). The 1996 ground water level data of D1 and D2 were used for calibration. The validation data set consists of the ground water level data of 1997.

As recharge input, the ground water model uses the time dependent bottom flux of the Beline unsaturated zone model (chapter 6). This bottom flux of the unsaturated zone model depends slightly on the ground water depth. For reasons of conciseness and clarity, only the bottom fluxes that result from calculation of the unsaturated zone model with the ground water level defined at 4 m below surface are used (see chapter 6). As calibration criteria, the RMSE and the ground water fluctuation range were chosen.
Figure 7.4 Results of modelled ground water level fluctuation (stress period 5-600 from table 7.2) in observation points D1, D2 and D3 (above) and in D1 (detail below) using hypothetically fluctuating long and short term recharge series.

Table 7.3 The final hydrological parameter setting of the 2D Beline ground water model after calibration. The ground water model uses the ground water recharge series from the unsaturated zone model with ground water at 4 m below surface as input series.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness [m]</th>
<th>Saturated Permeability [m/day]</th>
<th>Specific Yield [-]</th>
<th>Specific Storage [1/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>5</td>
<td>0.4</td>
<td>0.375</td>
<td>0.0075</td>
</tr>
<tr>
<td>Layer 2</td>
<td>5 to 10</td>
<td>0.1</td>
<td>0.325</td>
<td>0.0075</td>
</tr>
<tr>
<td>Layer 3</td>
<td>40</td>
<td>0.0075</td>
<td>0.2</td>
<td>0.0075</td>
</tr>
</tbody>
</table>

Figure 7.5 shows the modelled and measured ground water level for 1996 and 1997 for observation points D1 and D2. The fluctuations are plotted against the average ground water level of 1996. The corresponding values of the calibrated porosity related parameters are given in table 7.3. The modelled range of ground water level fluctuation corresponds well with the measurements. The timing of the ground water level maximum
and minimum is not represented, especially in the calibration year 1996. Only the high ground water level in summer 1997 in observation point D1 is well represented. The ground water model fails to represent the individual maxima and minima for observation point D2. Also the velocity of ground water level fall and rise does not match the measurements. Remarkably, the validation period (1997) seems to fit field measurements better than the calibration period (1996).

Figure 7.5 Results of the ground water modelling at the Beline slope for 1996 (calibration) and 1997 (validation) for two recharge time series. The model run used the bottom flux of the unsaturated zone with a ground water table at 4 m below surface as ground water recharge.
The RMSE values are given in table 7.4. Statistic objective functions like RMSE to quantify the goodness of fit between measured and modelled time series are very sensitive for phase shifts of the peak values. This means that if two identical time series are evaluated with a RMSE, while they only differ by a time lag, they will show high RMSE values. Figure 7.5 shows such a shift in peak timing. Therefore especially the range of fluctuations was used as calibration criterion. The RMSE does, however, show that the model performs likewise in the validation and calibration periods.

Although the model represents the general trend of the ground water dynamics, the short-term fast dynamics are not modelled. The explanation for the lack of modelled dynamics and the difficult timing is not hidden in the ground water model (schematisation, parameterisation) but in the relatively smooth recharge time series, which equals the bottom flux of the unsaturated zone model. Figure 7.6 relates the bottom flux of the unsaturated zone model with the measured and modelled ground water results.

Figure 7.6 Comparison of modelled and measured ground water level at D1 with the recharge time series.

<table>
<thead>
<tr>
<th></th>
<th>D1</th>
<th>D2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RMSE [m]</td>
<td>Range [m]</td>
</tr>
<tr>
<td>Measured</td>
<td>1996-1997</td>
<td>0.322</td>
</tr>
<tr>
<td>Modelled</td>
<td>SC (1996)</td>
<td>0.083</td>
</tr>
<tr>
<td></td>
<td>SV (1997)</td>
<td>0.091</td>
</tr>
</tbody>
</table>

The range was determined over 1996 and 1997.

a) The range was determined over 1996 and 1997.
Figure 7.6 learns that the modelled ground water level fluctuations react directly on the recharge input series. The unsaturated zone model retains and thus attenuates the ground water flux considerably but represents the (seasonal) amplitude of the fluctuations reasonably well. The reason for this is that only matric flow was incorporated and that any kind of preferential flow was ignored. Figures 7.5 and 7.6 indicate that the ground water model is calibrated as well as possible with the available data and that the input series need more refinement. The results lead to the question whether there are more suitable recharge time series like the direct use of the precipitation time series.

### 7.2.3 Precipitation as recharge series

Not all precipitation is converted to ground water recharge. Most of the precipitation is buffered in the unsaturated zone and evapotranspiration will take away most of that water. In the Beline area the precipitation was on average 3.1 mm/day. The ground water system was calibrated while receiving on average 0.75 mm/day, equal to 24% of the total precipitation. If only precipitation is used as recharge series, one has to multiply the daily precipitation with 0.24 to obtain the recharge series. The same parameter configuration was used as was given earlier in table 7.3. So no additional calibration of the porosity related parameters was carried out.

![Ground water level modelling results using a fraction of the daily precipitation data as recharge series (SP) for observation sites D1 and D2.](image)

Figure 7.7 Ground water level modelling results using a fraction of the daily precipitation data as recharge series (SP) for observation sites D1 and D2.
Figure 7.7 shows the model results of observation point D1 and D2. The modelled range of the ground water level fluctuations is much smaller than the measurements. However, the timing of the individual peaks is represented better than in figure 7.5. Especially for observation point D2 the results have improved. This suggests that at least a part of the ground water level fluctuations observed in D2 is the result of the direct influence of precipitation. In the D1 case, the general trend of the ground water level fluctuations is modelled well by the unsaturated zone flux as input data (figure 7.5), but the second order fluctuations of D1 are reflected better by the ground water model with direct precipitation input. Additional calibration of the porosity related parameters only resulted in a slightly higher amplitude of the modelled fluctuations when the values of these parameters outranged their normal values.

The model runs can be interpreted as follows. In addition to matric flow through the unsaturated zone, also direct influence of precipitation on the ground water level takes place (see also chapter 5). Ground water modelling using only a constant fraction of the precipitation time series as recharge series does not lead to the desired results because it does not represent the seasonal fluctuations of the unsaturated zone behaviour.

### 7.3 State dependent recharge

As described in section 2.5, in the subsurface especially seasonal and some short-term ground water reactions takes place on precipitation. The seasonality in the recharge comes from the state in which the unsaturated zone is. The ‘state’ of the unsaturated zone refers to the moisture condition in which the soil is. The state-dependency on ground water recharge of the unsaturated zone involves that after dry periods significant amounts of water can be stored in the unsaturated zone and after prolonged wet periods the storage capacity is limited. It is only after the soil is saturated that preferential flow actually transports water downwards (see e.g. Van Asch et al, 2001).

The main consequence for the modelling of ground water level fluctuations is that the recharge time series should be based on state-dependent preferential flow. How to combine matric and preferential flow in one input time series for a ground water model without introducing additional parameters and thus additional uncertainty?

The bottom flux of the unsaturated zone model (chapter 6) is a good indicator whether unsaturated or near-saturated conditions prevail in the upper subsurface. In the case of no downward flux (dry conditions), precipitation is likely to be stored more easily in the unsaturated zone and ground water recharge will be limited. In the case of a large downward flux (wet conditions), a large fraction of the precipitation is likely to recharge the ground water system. At the same time, it should be taken into account that the hydrological model has calibrated the (unknown) saturated permeability values of the subsurface, using a daily average recharge flux of 0.75 mm.

The following conditions are respected when modelling with state-dependent recharge: 1) the average (annual) recharge flux should remain unchanged, and 2) recharge should be derived directly from precipitation but also as a function of the state of the unsaturated zone.
So it is proposed to define the recharge time series as:

\[ R_t = \frac{v_t}{\bar{v}} \cdot f \cdot P_t \]  

(7.1)

\( R_t \) = Recharge time series at time \( t \)  
\( v_t \) = Bottom flux of the unsaturated zone at time \( t \)  
\( \bar{v} \) = Average (annual) bottom flux unsaturated zone  
\( f \) = Optimalisation factor, or factor of proportionality  
\( P_t \) = Precipitation at time \( t \)

The \( f \)-factor has to be introduced to ensure that the water balance of the unsaturated zone is correct. In other words, the \( f \)-factor ensures that the sum of the unsaturated bottom flux \( (\Sigma v_t) \) equals the sum of the newly calculated recharge \( (\Sigma R_t) \).

\( f \) is defined as:

\[
\frac{\sum_{t_1}^{t_2} v_t}{\sum_{t_1}^{t_2} \left( \frac{v_t}{\bar{v}} \cdot P_t \right)} = \frac{\sum_{t_1}^{t_2} R_t}{\sum_{t_1}^{t_2} \left( \frac{v_t}{\bar{v}} \cdot P_t \right)}
\]  

(7.2)

The proposed equation for calculating the state-dependent recharge also has to be restrained to prevent the recharge \( (R_t) \) from exceeding the precipitation \( (P_t) \). Otherwise, in case of a relatively wet unsaturated zone \( f^*(v_t/\bar{v}_{avg}) > 1 \), more recharge could be calculated than has been fallen.

\[
\text{IF } R_t \geq P_t \text{ THEN } R_t = P_t
\]  

(7.3)

Given the precipitation and unsaturated zone model bottom flux time series, the recharge time series \( (R_t) \) and the \( f \)-factor can easily be determined iteratively in a spreadsheet model. Figure 7.8 shows the results of the model run with the ‘state-dependent recharge function’. The ground water model has not been calibrated with this recharge option, and uses the parameter settings as given in table 7.3. The ground water model represents the measurements in observation points D1 and D2 better than in figure 7.5, where the ground water level fluctuations are modelled with a fluent line. Figure 7.8 shows model results, which depict both the seasonal effects and the direct influence of precipitation on recharge.

One deficiency of the model results as shown in figure 7.8 is the time delay between the measured and modelled ground water level rise in October - November 1996. Figure 7.9 visualises the calculation of the ground water recharge as a function of the state of the unsaturated zone. The bottom flux time series of the unsaturated zone model with a lower boundary condition of 4 m below surface is shown. It is shown that in October and November 1996 a significant amount of precipitation has to fall before the unsaturated zone reacts, i.e. larger amounts of precipitation are transported to the ground water system.
The time delay is the direct consequence of the fact that $v_t$ – as calculated by the unsaturated zone model in chapter 6 – is near 0 and thus the ground water recharge is near 0. But the ground water observations show earlier reaction on prolonged precipitation. The proposed model to calculate recharge assumes that only limited rainfall can replenish the ground water system when the upper subsurface is not near saturation ($v_t \to 0$). This assumption does not always have to be true. A particular hydrological system could have some direct contact via (large) preferential flow.

Therefore the proposed equation can be improved easily for this situation in the following way:

$$R_t = \left(f_{\text{dir}} + \frac{V}{V_f} \cdot f\right) P_t$$ (7.4)

With $f_{\text{dir}}$ being the direct recharge fraction, a constant fraction of the precipitation, $f_{\text{dir}} \cdot P_t$ is the amount of precipitation per rain event that is passed directly to the ground water system. Main disadvantage, however, is the introduction of a new -not measurable- parameter.
Figure 7.9
Overview of cumulative precipitation and recharge in relation to the relative bottom flux of the unsaturated zone model.

Figure 7.10 shows the results of the ground water modelling with the extended recharge model (SER) together with the results of the recharge model (SR). In general, the graphs have not changed. The ground water level calculations with the SER are a little attenuated. The extended recharge model did not move the modelled ground water level rise forward in time.

Remarks and discussion

It should be stressed that the results given in this paragraph were calculated without additional calibration. This to make the comparison of the effects of the different recharge models easier. As recharge input, the ground water model uses the time dependent bottom flux of the Beline unsaturated zone model (chapter 6). This bottom flux of the unsaturated zone model depends only slightly on the assumption of the ground water depth. For reasons of conciseness and clarity only the bottom fluxes that results from calculation of the unsaturated zone model with a ground water level defined at 4 below surface are presented. The bottom flux of the unsaturated zone model with a lower boundary of ground water at 5 m below surface has also been used as recharge time series in the ground water model. This does not alter the model results and consequently the insight in the hydrological behaviour of the Beline slope.

The model results have improved significantly when using a more realistic recharge input series. Still, the model results as shown in figures 7.8 and 7.10 show that the ground water level fall in dry periods is significantly faster in the measurements than in the model results. One way to overcome this, is to increase the saturated permeability causing the modelled ground water system to drain more rapidly. However, a set of
higher permeabilities needs a proportionally larger average recharge flux. Higher permeability values also result in a less inclined ground water level within the slope, i.e. downslope a larger area will have ground water seepage and upslope the ground water level will be deeper. Also, the calibrated saturated permeabilities are already relatively high considering the silty clay and marly environment. Another solution is to decrease the porosity, but this also leads to a larger, undesired range of ground water level fluctuations. Also a decrease in porosity would reduce the performance of the unsaturated zone model (chapter 6). The fast decline of ground water in dry periods could also come from 3D effects (diverging flowlines) which can not be taken into account in the 2D set-up as used here.

The state-dependent ground water recharge function is not developed for water transport from the ground water to the unsaturated zone (negative $R_t$). In a clayey slope with a thickness of the unsaturated zone between the 3 and 6 m, in a temperate climate, the unsaturated zone is generally very moist (see e.g. the equivalent soil moisture profiles in figure 6.2). A situation that the hydraulic gradient above the ground water table will be directed upwards, is not likely (see also figure 5.18 with the interpretation of the unsaturated zone behaviour at the Beline slope).

![Ground water model results using state-dependent recharge time series (SR) and its extended version (SER, with $f_D=0.10$) for observation sites D1 and D2.](image)

Figure 7.10
The state-dependent recharge function, as proposed here, assumes the availability of an unsaturated zone model. If this is not so, the state-dependent recharge function can also be approximated with a sinusoidal function (see Bogaard et al., 1998). The recharge function then becomes:

\[
R_t = \left[ f_{\text{avg}} + \alpha \cdot \sin\left( \frac{\text{daynumber} + \text{dayoffset}}{366} \cdot 360 \right) \right] \cdot P_t
\]  (7.5)

with:
- \(f_{\text{avg}}\) = Average recharge fraction
- \(\alpha\) = Amplitude recharge fraction
- \(\text{daynumber}\) = Julian day
- \(\text{dayoffset}\) = Time delay parameter

The average recharge fraction \((f_{\text{avg}})\) is forced to be larger than the amplitude of the recharge fraction \((\alpha)\), otherwise the recharge function becomes negative during some periods. \(f_{\text{avg}} \cdot P_t\) equals the average recharge flux that is used to calibrate the ground water model using its steady-state ground water level. The remaining parameters \(\alpha\) and ‘dayoffset’ should be optimised using ground water level measurements.

### 7.4 A scenario study on the impact of environmental changes on the ground water level

Since the ground water level fluctuations could be modelled well, it is possible to study the effects of changes in the input time series on the ground water system. Changes in land-use and climate can be represented by changes in potential evapotranspiration and fluctuations in precipitation (see also chapter 6). A change in potential evapotranspiration or precipitation (or both) results in a change in unsaturated zone behaviour and thus has an effect on the ground water recharge. This results in changes in the ground water system and finally in slope stability. In chapter 5 different scenarios are introduced. With these the transient effects of changes in boundary condition on the hydrological system can be studied starting with the same initial conditions, i.e. a prolonged wet or dry period, or the effects of land-use change.

For the scenario studies, the reader is referred to table 6.9, where the scenarios are defined. The scenarios can be divided into four groups: 1) changes in potential evapotranspiration, 2) changes in precipitation, 3) changes in both potential evapotranspiration and precipitation with opposing effects and 4) changes in both potential evapotranspiration and precipitation with similar effects.

The changed bottom flux of the unsaturated zone model was used directly for the ground water model. This ensures that the non-linear behaviour of the unsaturated zone is taken into account. The effects of preferential flow in the recharge input time series, as described and discussed in § 7.3, are also studied, by recalculating the recharge series using formulas 7.1 to 7.3.

First of all, figure 7.11 shows that the Beline hydrological system can buffer around one year of changes in input variables. It is after this period that the hydrological system
starts to react. Secondly, the general trend of all the scenario calculations, as depicted in figure 7.11, is that the effects of wetting are larger than the effects of drying. In other words, a decrease of the potential evapotranspiration with 10% has more effect than an increase of potential evapotranspiration with 10%. This can be explained by a high evapotranspiration demand, which does not have to lead to actual evapotranspiration. In case the soil is already dried out the actual evapotranspiration will be low, whereas in wet periods the soil seems to be able to absorb the surplus of precipitation and thus percolation and ground water recharge increase.

Thirdly, the scenarios show that a change in potential evapotranspiration has slightly less effect than a change in precipitation. Scenarios 9 to 12 (changes in precipitation (P) and potential evapotranspiration (PET) with opposing effects) show this effect most clearly. A decrease of rainfall and potential evapotranspiration results in almost no change in ground water behaviour, whereas an increase of both variables results in a ground water level rise.

One of the reasons that no limitation seems to exist for additional precipitation to infiltrate is because all precipitation falling in 1 day is spread over that day and thus results in very low rainfall intensities. In reality this does not have to occur, when
increased rainfall leads to increased rainfall intensities and thus the occurrence of overland flow when rainfall intensities exceed the soil infiltration capacity.

Is it necessary to use the state-dependent recharge model for providing the input of the ground water model when seasonal changes are calculated? Figure 7.12 shows the results of scenarios 1, 3, 5 and 7, calculated with both the recharge input directly from the bottom flux of the unsaturated zone model and the state-dependent recharge model. As shown, for the overall trend of the ground water level fluctuations and the maximum ground water level rise, the choice of input time series does not seem very relevant. However, when the distribution of precipitation changes a lot, the state-dependent recharge model will give more realistic ground water reactions. Secondly, for creep calculations (see chapter 7) the time span that a certain ground water level is sustained, can be of importance.

![Comparison of scenario results using the recharge input series of the unsaturated zone model (thick grey line) and of the state-dependent recharge model (thin grey line).](image)

**Figure 7.12** Comparison of scenario results using the recharge input series of the unsaturated zone model (thick grey line) and of the state-dependent recharge model (thin grey line).

### 7.5 Summary and discussion

This chapter describes the modelling of ground water level and its fluctuations using the Modflow model for clayey slopes. It focuses on the effects of ground water recharge time series. The proposed methodology to study the ground water level fluctuations in clayey slopes consists of modelling the unsaturated and saturated zone separately. The unsaturated zone model results in a bottom flux or ground water recharge flux. Using the annual average value for recharge, the ground water model is calibrated on its steady-state ground water level by adjusting the hydraulic conductivity. The dynamics of the ground water system are modelled by calibrating the porosity related parameters.

The calibration of the saturated permeability is performed on the average, steady-state, ground water level as deduced from only three observation sites. With the used geometry, it was difficult to fit all three observations exactly. The modelled average ground water level is a smooth line through the observations, which resulted from a trial and error approach. The fixed average recharge of 0.75 mm/day also indirectly limits the saturated permeability values for the subsurface. Recharge flux and saturated permeability in a steady-state situation are related.
It was tried to perform this calibration automatically using the PEST program. Two important problems came up using this approach. First of all, the measured depth of the ground water table is too irregular to model with the geometry as described above. Secondly, the automatic calibration program changes the selected parameters within certain boundaries. By automatically selecting an ‘extreme’ parameter value or combination of parameter values, the model becomes numerically unstable. This puts an end to the inverse modelling exercise, resulting in no optimisation. Narrowing the optimisation range for a parameter (e.g. permeability) can prevent this problem from occurring. But tests showed that the range should be narrowed to unpractical small values, leaving no free optimisation. Lastly, the presented results show that it is very hard, if not impossible, to work with an objective function like RMSE if the input time series (ground water recharge) have a phase shift in the time series (measured ground water levels). In such a case, the best statistical fit does not necessarily represent the best model result, and thus remains manual control on the calibration process indispensable.

In this chapter it was shown that the ground water level fluctuations in the clayey Beline slope could neither be explained with matric flow (from the unsaturated zone model) nor with (fractional) precipitation input. The role of the saturation grade of the unsaturated zone is indispensable. The (preferential) recharge flux depends on antecedent moisture conditions in the unsaturated zone. A model is proposed to combine the state of the unsaturated zone and the precipitation input in a simple determination of the recharge time series: the so-called ‘state-dependent recharge function’. It transforms precipitation to recharge by scaling it with soil moisture conditions. As a function for the soil moisture condition the ratio of the matric bottom flux and the average matric bottom flux is used. In this way, the highly non-linear behaviour of the unsaturated zone is incorporated in the ground water recharge time series.

The model results, as shown in figures 7.8 and 7.10, show that the ground water level fall in dry periods is significantly faster when measured than when modelled. So, it is fair to state that the saturated permeability should be set higher in order to make the ground water system drain more rapidly. However, a higher set of permeabilities also results in a less inclined ground water level within the slope, i.e. downslope a larger area will have ground water seepage and upslope the ground water level will be much deeper. Also, the calibrated saturated permeabilities are already relatively high for a silty clay and marly environment.

Besides the mathematical aspects of the model calibration, the spatial uncertainty caused by spatial heterogeneity is also responsible for imperfection of the model results. Other error sources are the absence of on-site actual evaporation, transpiration and rainfall data. In this research it was tried to exclude the effects of spatial variability as much as possible and to focus on the effects of ground water recharge on ground water level fluctuations. It was shown that with the available data a good insight can be reached in the behaviour of the hydrological processes in clayey slopes that are prone to mass movement.

One aim of the ground water modelling was to determine the spatial distribution of the pore water pressure in the Beline slope. This aim is not fulfilled. Using the available data it is not possible to come to a pore pressure distribution in the Beline slope, which improves the ground water level measurements that were the starting point of this study. However, it was possible to quantify the ground water level fluctuations.
The second aim was to determine an effective way to obtain ground water recharge from precipitation data and unsaturated zone models. The recharge and extended recharge models proved to be straightforward and useful methods to determine the important recharge time series that are used as input for ground water modelling.

The next aim was to quantify the influence of changes in ground water recharge on the ground water level fluctuations. The general trend of all the model scenarios was that changes resulting in wetting had more effect than changes resulting in drying. Furthermore, wetting by an increase in precipitation has more effect than by a decrease in potential evapotranspiration. This is mainly caused by the fact that an increase in precipitation is almost totally absorbed by the soil because of the artificially low rainfall intensities caused by modelling on a daily time scale. However, the measured rainfall intensities at the Beline slope and the Arbois Météo France station revealed that the maximum rainfall intensities during a rainfall event were on average between 1 and 2 mm/hour. This in contrast with changes in potential evapotranspiration, which do not automatically lead to changes in actual evapotranspiration. The availability of water in the upper few centimetres of the soil controls the evapotranspiration flux. In clayey soils the unsaturated hydraulic conductivity already becomes very low with a small decrease in soil moisture content, resulting in very slow moisture transport through a dry soil/atmosphere interface layer.
8 SLOPE STABILITY AND DEFORMATION IN RELATION TO GROUND WATER LEVEL FLUCTUATIONS

8.1 Introduction

This chapter aims to quantify the influence of ground water level changes on slope mobility. For this it is necessary to evaluate and quantify the mechanisms of movement at the Beline slope. Slope movement mechanisms can roughly be divided in sudden (plastic) and continuous (visco-plastic) deformations. The former mechanism prescribes rigidity until a certain stress threshold is exceeded. The latter mechanism has a deformation range from a deformation threshold to a failure threshold. This process is generally referred to as (continuous) creep.

This chapter first of all describes the field and laboratory measurements (§ 8.2 and 8.3). In paragraph 8.4 the slope stability is evaluated using the limit equilibrium method for plastic failure with a plane parallel slipsurface (infinite slope model) and circular slipsurface (Bishop’s method). In paragraph 8.5 creep is modelled by means of the Yen creep model.

8.2 Subsurface schematisation and field deformation measurements

For the geotechnical analysis, the same schematisation as for the hydrological modelling was used. This follows from the geological schematisation as given in chapter 4. Three layers were identified using borehole information, drilling progress and geophysical surveys:
- a disturbed surface layer, low density and abundant gravel content, with an average thickness of 5 m;
- a zone of remoulded marls with silty layers, a variable gravel content, an average density and a thickness of 5 m at the bottom of the slope towards more than 10 m in the upper part of the slope;
- compact blue marls, in general no gravel, but with some limestone concentrations.

In order to measure deformation with depth, the Beline slope in Salins-les-Bains was equipped with one inclinometer tube, which was installed at drilling location D3 (see figure 4.8). The flexible tube reached as far as 28 m below surface level. It was manually monitored once a month from January 1996 till December 1997. The displacement was measured in two directions (8-188°N and 98-278°N) of 50 cm vertical intervals. The measurement error of these deformation measurements is in the range of 1-2 mm. With the two measurements both the absolute displacement and the direction can be calculated. Figure 8.1 shows the cumulative displacement in the first 6 m. No displacement was recorded below 6 m depth. During all other months no displacement could be distinguished. Two clear moments of distinctive movement were recorded: July 1996 and November 1996. In 1997 some movement took place, but not as notably as in November 1996. The direction of the movement changes slightly with depth. Near the surface it is in a southerly direction, at 2 m depth in a SW direction and below 3 m the movement has a SSW orientation.
The inclinometer measurements can be interpreted in two ways. At the location of the inclinometer tube a creep zone is situated between 3 and 6 m below surface. Another explanation can be that the deformation takes place in a thin slip surface or zone around 4.5 m below surface. The firmness of the inclinometer tube then spread the (small) displacement over 3 m tube. Another observation from the displacement measurements is that the surface displaces less. An explanation could be that the grass surface cover mobilises additional shear strength and thus holds back the upper part of the profile from displacement.

8.3 Laboratory measurements of geotechnical soil parameters

To determine the strength parameters cohesion and angle of internal friction of the soil (see § 2.2), direct shear and triaxial laboratory tests were executed. For the determination of viscosity and creep threshold an experimental creep test was set up.

In total 37 direct shear tests were performed, 32 with undisturbed samples and 5 with disturbed samples. The direct shear samples were collected in the central part of the slope at depths varying from 20 to 160 cm. The samples were stored at 7 °C before testing. The samples were cut into slices of 6*6 cm with a thickness of 2 cm and saturated for the duration of one week. After this, the samples were consolidated for 2-3 days (while saturated) using an oedometer. The single stage drained direct shear tests were executed with an axial load varying from 50 to 250 kPa and with a deformation velocity of 0.08 to 0.12 mm/hr (2 to 3 mm/day). The tests were stopped when the shear strength reached a constant value or were stopped automatically after ± 9 mm of displacement.
Furthermore 6 triaxial tests were performed. Three triaxial samples were taken from the cored drilling SC1 (see figure 4.8) in the interval of 1.4-2.7 m and three from 21.3-21.6 m below surface. The height and diameter of the shallow samples were 10 and 5 cm respectively and of the samples from 21 m depth 7.6 and 3.8 cm respectively. The drained triaxial tests used an all-round (fluid) pressure of 50, 150 and 400 kPa and a deformation velocity of 0.24 mm/hr.

The results are shown in table 8.1. The cohesion and angle of internal friction are peak strength values for the undisturbed samples and residual strength for the disturbed sample. The soil cohesion ranges from 11 to 20 kPa, and the angle of internal friction ranges from 23 to 28 degrees.

Table 8.1 Results of the strength tests at the Beline slope: n= number of tests, c’=effective cohesion, \( \phi' \)= effective angle of internal friction and \( \rho_d \)=dry bulk density.

<table>
<thead>
<tr>
<th>Test type</th>
<th>Depth [m]</th>
<th>n [-]</th>
<th>c’ [kPa]</th>
<th>( \phi' )[°]</th>
<th>( \rho_d )[g/cm³]</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Direct shear</td>
<td>0.2-0.4</td>
<td>11</td>
<td>20</td>
<td>26</td>
<td>-</td>
<td>Some grass roots</td>
</tr>
<tr>
<td>Direct shear</td>
<td>0.6</td>
<td>6</td>
<td>12</td>
<td>24</td>
<td>1.6</td>
<td></td>
</tr>
<tr>
<td>Direct shear</td>
<td>0.68</td>
<td>5</td>
<td>11</td>
<td>23</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Direct shear</td>
<td>0.96</td>
<td>5</td>
<td>17</td>
<td>28</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Direct shear</td>
<td>1.6</td>
<td>5</td>
<td>13</td>
<td>23</td>
<td>1.7</td>
<td></td>
</tr>
<tr>
<td>Direct shear</td>
<td>1.2</td>
<td>5</td>
<td>14</td>
<td>19</td>
<td>-</td>
<td>Disturbed test samples</td>
</tr>
<tr>
<td>Triaxial</td>
<td>1.4-2.7</td>
<td>3</td>
<td>17</td>
<td>27</td>
<td>1.64</td>
<td>2 tests showed a barreling type of failure</td>
</tr>
<tr>
<td>Triaxial</td>
<td>21.3-21.6</td>
<td>3</td>
<td>0</td>
<td>21</td>
<td>1.58</td>
<td>Unweathered marls</td>
</tr>
</tbody>
</table>

An experiment was set up to measure the viscosity and creep threshold values under laboratory conditions, the simple shear viscosity test. The apparatus, which was designed for viscosity tests, consists of two retention plates of 10 by 20 cm² with 3 cm of soil samples in between. The lower plate is fixed and the upper plate is pulled using a dead weight (figure 8.2). The samples were inundated till the upper retention plate. The retention strips on the retention plates were placed perpendicular to the direction of movement and penetrated the sample 0.5 cm. This leaves 2 cm of soil free to deform. Plastic transparent sides prevented lateral soil displacement.

The normal stress load was applied to the upper retention plate and the shear stress load was hung below the pulley. The displacement rate was measured with a strain gauge behind the upper retention plate. The samples were consolidated on average one week before the viscosity tests were performed. The viscosity test stopped after the displacement rate became constant, which was generally within one week. The average displacement velocity was calculated between 1 and 7 days after the start of the test.
In total 24 tests were performed with 4 different normal loads. The analysis of the viscosity was performed assuming Bingham rheological behaviour of the material (e.g. Nieuwenhuis, 1987, Van Asch and Van Genuchten, 1990). The linear Bingham viscoplastic flow equation is:

$$\tau = \tau_0 + \eta \cdot \frac{du}{dy}$$  \hspace{1cm} (8.1)

with:
- \(\tau\) = shear stress \([\text{kN/m}^2]\)
- \(\tau_0\) = threshold value for creep (yield strength) \([\text{kN/m}^2]\)
- \(\eta\) = dynamic viscosity \([\text{kNs/m}^2]\)
- \(u\) = displacement velocity \([\text{m/s}]\)
- \(y\) = depth \([\text{m}]\)
- \(\frac{du}{dy}\) = shear strain rate \([\text{s}^{-1}]\)

under the assumption that the thickness of the deforming layer does not change in time \((dy/dt=0)\). The shear strain rate is defined as:

$$\frac{du}{dy} = \left(\frac{dx}{dt}\right)$$  \hspace{1cm} (8.2)

where:
- \(dx/dt\) = displacement velocity in x-direction \([\text{m/s}]\)

The displacement velocity can be determined from the straight line in the displacement-time graph. The thickness over which the sample deforms \((dy)\) was taken equal to the sample height between the retention strips, i.e. 2 cm. The shear stresses and corresponding shear strain rates were plotted as shown in the example of figure 8.3. A linear regression results in the creep threshold \((\tau_0)\) and the dynamic viscosity \(\eta\) of the material (eq. 8.1).
Figure 8.3  Example of the determination of viscosity from a laboratory test. First of all the shear strain rate (displacement velocity divided by creep height) is calculated between 1 and 7 days after the test started (first graph). Then the shear strain rates are plotted against their shear stresses. The linear regression through these points gives the creep threshold ($\tau_0$) and the dynamic viscosity ($\eta$).

Figure 8.4  Relationships of the yield strength and dynamic viscosity with the normal stress in the soil, as determined with the laboratory viscosity tests.
The results of the laboratory viscosity tests are shown in figure 8.4. Both yield strength (creep threshold) and dynamic viscosity were plotted against the normal stress that was applied: 15, 20, 25 and 34 kPa. The creep threshold ($\tau_0$) shows to be linearly related to the applied normal stress. The viscosity could not be related with the normal stress.

### 8.4 Static stability model

A general approach to obtain an indication of the stability of a given area is to analyse the stability of potential failure mechanisms. For any mechanism both the maximum resisting force and the driving force are calculated. An instable situation is predicted when the driving force exceeds the maximum resisting force. The ratio of the maximum force and actual driving force can be used to predict the likelihood of failure. This ratio is often referred to as Factor of Safety, FS. A static stability model only evaluates the equilibrium between driving and resisting forces. It does not take into account transient mass movement processes. The aim of the stability calculation in this research is to evaluate changes of slope stability as results of changes in ground water levels within the slope.

In the framework of the Hycosi project several models have been used and developed for slope stability analysis. Boeije and Teunissen (1997) developed a 3D stability model called 3DSTAB, Asté (1997) used the 3D.PENT program. The 3D stability computer codes are capable of evaluating the Factor of Safety of three-dimensional spheres. The above-mentioned researchers performed a detailed analysis of the possibility of using such models for the evaluation of slope stability at the Beline slope. Boeije and Teunissen (1997) worked with a highly simplified schematisation of the slope and concluded: “It is difficult to define or predict the precise geometry of the landslide. There is not sufficient data available for the characterisation of the soil strength in different areas and layers. This limits the quantitative capabilities of the method for 3D stability analysis presented here.” Asté (1997) evaluated and combined the existing data. He analysed three predefined spheres with regard to their stability using the actual topography and one on a hypothetical ancient topography for evaluating the existence of an ancient landslide. He made it plausible that the slope has been subject to an ancient large-scale landslide. For recent landslide analysis his 3D stability calculations are also strongly dependent on input data and pre-information on movement.

For stability analysis one needs information on the strength parameters and on the geometry. For both 2D and 3D models the former information is equally necessary, but a 3D stability model requires much more spatial information on the geometry than 2D models. As outlined above, this geometry information is limited available. It was therefore decided to perform the analysis using 2D slope stability models. Plane parallel failure is studied using an infinite slope model and for circular failure Bishop’s method was used.

**Parameterisation**

The geotechnical parameterisation of the three layers (§ 8.2) is given in table 8.2. Only 3 strength tests were executed on the unweathered marls and none on the intermediate layer. Boeije and Teunissen (1997) and Asté (1997) did not have the triaxial test results of the deeper material when they investigated the slope stability of the Beline site. They
used another parameter configuration (see table 8.2). Boeije and Teunissen (1997) discussed the first assumption that the angle of internal friction was 13° is implausible and that a value of 30° is more reliable. Such a low angle of internal friction for the marls leads to deep-seated slip surface. This is not supported by any field evidence. The shear strength values from the triaxial test (c=0 kPa and φ=21°) on the sample from the Beline slope at 21 m below surface (layer 3) are doubtful.

Table 8.2 The initial parameter setting for the 2D Beline stability analysis.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness [m]</th>
<th>c' [kN/m²]</th>
<th>φ°</th>
<th>γs [kN/m³]</th>
<th>Remarks</th>
<th>c' a) [kN/m²]</th>
<th>φ° a) [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>15</td>
<td>25</td>
<td>20</td>
<td>DS and Triaxial tests</td>
<td>20</td>
<td>26</td>
</tr>
<tr>
<td>2</td>
<td>5 to 10</td>
<td>10</td>
<td>22.5</td>
<td>20</td>
<td>No laboratory tests</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>-</td>
<td>0</td>
<td>21</td>
<td>20</td>
<td>Triaxial tests</td>
<td>40</td>
<td>13</td>
</tr>
</tbody>
</table>

a) In italic the strength values used by Boeije and Teunissen (1997) and Asté (1997).

The geotechnical characteristics of marls have a large range depending on the degree of limestone cementation in the marls. They can behave as a rock with cohesion values in the range of 1000-10000 kPa (Carson and Kirkby, 1972). De Joode (2000) did several strength tests on undisturbed black marls rock from Oxfordian age (“Terres Noires”), unconfined compression tests as well as direct shear test. With the latter, the angle of internal friction was determined with broken material using the contact plane between two slices of Terres Noires as shear plane. A value of 30.5° was determined for the peak strength and a value of 24.5° for the residual strength. Under the assumption of an angle of internal friction of 30.5°, the compressive strength tests showed cohesion of more than 3000 kPa.

Antoine et al (1988), however, find Terres Noires marls with c’=72 kPa and φ=29°. Jibson et al (1998) assigned cohesion values on the basis of available strength tests and expert knowledge of many different geological formations near Los Angeles, California. They come up with cohesion values in the order of 25-40 kPa for geological formations as shale, mud or clay.

Of course every lithology has different characteristics and consequently different strength characteristics. There is, however, severe doubt on the shear strength value of the ‘fresh’ marls from the Beline slope determined by the triaxial tests. Therefore the marls have been assigned a cohesion value of 100 kPa and an angle of internal friction of 30° in the rotational slip analysis.

The ground water level monitoring (chapters 4 and 7) gave ground water levels between 3.4 and 6.4 m depth at the central part of the slope. The maximum ground water level that was measured was at 3.24 m below surface (table 5.7). Furthermore, at the central part of the slope clear signs of deformation have been mapped (figure 4.10). For the stability calculation a ground water level of 3 m below surface will be used as a reference ground water level, a first approximation for the critical ground water level. This is to ease comparison between several stability calculations.
Infinite slope model

The infinite slope model calculates the factor of safety as:

\[
FS = \frac{c' + \left(\gamma^* - h_w \cdot \gamma_w - \cos^2(\beta)\right) \cdot \left(\tan(\phi')\right)}{\gamma^* \cdot z \cdot \sin(\beta) \cdot \cos(\beta)}
\]  \hspace{1cm} (8.3)

with:

\[\gamma^* = (1 - m) \cdot \gamma + m \cdot \gamma_s\]

where:

- \(c'\) = Effective cohesion [kN/m²]
- \(\phi'\) = Angle of internal friction [°]
- \(\beta\) = Slope angle [°]
- \(\gamma^*\) = Average unit weight soil [kN/m³]
- \(\gamma_w\) = Unit weight water [kN/m³]
- \(\gamma_s\) = Unit weight saturated soil [kN/m³]
- \(\gamma_u\) = Unit weight unsaturated soil [kN/m³]
- \(z\) = Thickness soil layer [m]
- \(h_w\) = Height water above slip surface [m]
- \(m\) = fraction layer saturated, \(h_w/z\) [-]

The infinite slope model uses a slipsurface parallel to the surface and it reasons that the interslice forces are cancelled out because of the symmetry. The infinite slope model was used to subject some parameters to sensitivity analysis; parameters that have a large uncertainty or are difficult to determine. Three parameters were studied on their effect on the FS of the slope: the depth of the slipsurface \((z)\), relative height of the water table \((m,\ \text{also written as } r_u, \ \text{the pore pressure ratio})\) and strength parameters \((c' \ \text{and } \phi')\). The slope angle is a very sensitive parameter (see e.g. Mulder, 1991) but relatively accurate to measure. The Beline slope angle measures 17°. Figure 8.5 shows the influence of these three factors on the slope stability. First of all it can be deduced that slope instability cannot occur in the first layer (5 m). All strength parameter combinations result in a stable slope. Instability by a deeper located slipsurface in the second layer is more likely. For the stability at 10 m depth the angle of internal friction is more important than the cohesion due to higher stress levels. On the other hand, the value of the angle of internal friction has almost no influence on the stability of superficial slipsurfaces.

In the situation of a slipsurface at 10 m depth with \(c' = 10 \ \text{kN/m}^2\) and \(\phi' = 22.5^\circ\), the slope becomes unstable for a ground water level of 2.5 m below surface \((m=0.75)\). If a local steepness of 20° is assumed, the slope with a slipsurface at 10 m below surface will become unstable with a ground water level at 5 m below surface \((c' = 10 \ \text{kN/m}^2\) and \(\phi' = 22.5^\circ\)). A slope of 17° with a slipsurface at 5 m below surface, will become unstable with a ground water level at 3 m below surface if the \(c' \ \text{and } \phi'\) reduce to e.g. 5 kPa and 18°. A slipsurface at 7.5 m will need a reduction of \(c' \ \text{and } \phi'\) to 7.5 – 10 kPa and 19 - 20° with the same ground water level (-3 m). Especially the reduction of the cohesion seems quite drastic, when it is compared to the shear strength results of the disturbed samples \((c' = 14 \ \text{kPa} \ \text{and } \phi' = 19^\circ)\), which can be seen as an approximation of the reduced strength of the soil.
Figure 8.5  Relationship between the Factor of Safety (FS) and ratio pore pressure/soil depth (m) with the depth of the slipsurface (z), cohesion (c’) and the angle of internal friction (ϕ’).
Bishop’s method

Rotational slips can also be analysed with limit equilibrium methods by applying the method of slices. The factor of safety is defined as the ratio of the available shear strength to the shear strength, which must be mobilised to maintain a condition of limit equilibrium (Craig, 1992). In several 2D models (Bishop, Fellenius), the slipsurface is assumed to be a circle, along which the shear stresses can be calculated. These stresses result in a resisting moment around the axis of the circle. The gravitational forces on the mass in the soil circle itself create a driving moment around the axis of the circle.

The slope stability calculations of these rotational slips were executed with the SLIDE computer code. This program calculates slope stability according to the method of slices. It is capable of analysing a rotational slipsurface for lowest FS of circular as well as ellipse shape, according to Fellenius, Bishop (ordinary), Janbu (simplified and rigorous), Spencer, Morgenstern and Price (see e.g. Craig, 1992). It can perform a slope analysis if the toe and the scarp of a slipsurface are defined and can work with a non-rotational slipsurface if it is predefined.

Stability calculation along the Beline slope learnt that the part around inclinometer tube (D3) was the most unstable. Further analyses were therefore focussed on this higher part of the slope. Calibration of the slope stability model was strongly limited by data shortage, both of strength parameters and soil and slipsurface geometry. For these analyses the strength parameters of the unweathered marls (layer 3) were set on $c=100$ kPa and $\phi=30^\circ$ while for the intermediate layer $c=10$ kPa and $\phi=22.5^\circ$ was assigned (see table 8.2). The slope around the inclinometer tube showed a slipsurface at maximum 10 m depth (6.2 m below surface at D3) with a FS=1.12 with a ground water level at 3 m below surface (figure 8.6). This is a FS of 0.11 more than when using the infinite slope model. A FS=1.0 can be reached by setting $\phi$ of the second layer at $21^\circ$ instead of $22.5^\circ$.

![Figure 8.6](image_url) Location of the slipsurface of a rotational slide using the Bishop’s method of slices at the Beline slope.
Quantification of the effects of ground water level changes on the slope stability

In chapter 7 changes in land use or climate were modelled by changing the input variable precipitation and potential evapotranspiration with 10 or 20 %. The reaction to these changes is an increase of the ground water level in the order of decimetres. How does this affect the slope stability?

In case of the Beline slope (c=10 kN/m$^2$, \(\varphi=22.5^\circ\) for the second layer), an increase of ground water level from 3 to 2.5 m below surface results in a decrease of FS of 0.035 using the infinite slope model and in case of a rotational slipsurface the FS decreases 0.033. The change in FS seems independent of the model assumptions.

It could be interesting to analyse when the subsurface is most sensitive for ground water level changes. Sensitivity of a slope to ground water level changes is defined as a change in the derivate of FS with water level increase (dFS/dh). This is calculated with the infinite slope model. Figure 8.7 shows the effect of a change in the depth of the slipsurface, cohesion or angle of internal friction on the sensitivity of the slope to changes in pore water pressure. It shows that the deeper the slipsurface is located, the less will the FS decrease when ground water increases with 1 m. This is generally known as the effects of the pore pressure ratio. The same is valid for soils with a low angle of internal friction. Changes in cohesion do not affect the decrease of FS with 1 m ground water level increase. This shows that slopes with a frictional soil and shallow slipsurfaces are most sensitive to changes in pore water pressure. This is the situation, which is encountered at the Beline site. It must be stressed that the 1 m ground water level rise is only a calculation example (unit water level rise), and not expected at the Beline slope.

Figure 8.7 Effects of the depth of the slipsurface and the strength parameters (cohesion and angle of internal friction) on dFS/dh. Depth of the slipsurface runs from 5 m to 12.5 m, cohesion from 15 kPa to 0 kPa and \(\varphi\) from 25° to 5°.

Conclusions

The following conclusions can be drawn from the 2D static stability analyses:
Using the peak strength values determined in the laboratory (table 8.1) and assuming a correct subsurface schematisation, the Beline slope is stable under the ground water conditions encountered in the field.
- On the Beline slope the FS lowers with 0.035 when the ground water level rises from 3 to 2.5 m below surface. This will not cause catastrophic failure.
- The area most prone to landslides is the part around the inclinometer tube. This is also observed in the field and in the inclinometer tube.
- The static stability analysis shows that in the event that a landslide will occur, it can be expected to have a slip surface with a maximum depth of 10 m depth.
- Ground water level changes have a relatively larger effect on the stability of a slope with frictional soils and shallow potential slip surfaces.
- The strength parameters as determined in the laboratory have to be reduced to $c'$ is 5 to 7.5 kPa and $\phi'$ is 18° to 20° in order to come to a FS around 1 when the ground water level is 3 m below surface. This reduction in strength is in agreement with the reduced strength analysed with the viscosity tests (see paragraph 8.4).

8.5 Displacement model

Static slope analysis classifies a slope either as stable or unstable. They do not give information about the movements. This limits their use to relative changes in slope stability and the evaluation of catastrophic mass movements. But accelerating sliding along a slip surface is not the only mechanism of slope movement. An additional movement mechanism is creep. The creep process may be of great importance in the development of landslides because initial movement of the slope may start with creep processes at stress values that lie far below the peak strength of the soil material (Van Asch and Van Genuchten, 1990). These slow creep movements can also lead to accelerated creep and shear failure. The aim of the displacement modelling is to evaluate the slope movement in terms of creep and to quantify the effects of ground water level changes on the slope movement of the Beline slope.

On the basis of the Bingham rheological behaviour of material (Equation 8.1) several so-called creep models were developed. Nieuwenhuis (1987) describes and compares the performance of the models of Ter-Stepanian (1963) and of Yen (1969). The former states that the dynamic viscosity ($\eta$) and the yield strength ($\tau_0$) are a function of the normal stress. Yen (1969) and Suhaydu and Prior (1978) define the yield strength by the residual strength parameters $c'_r$ and $\phi'_r$ of the material. The dynamic viscosity ($\eta$) is constant and independent of the existing isotropic stress conditions in the soil as in the Bingham fluid.

The laboratory viscosity tests show that the yield strength is a function of the normal stress and indicates that the dynamic viscosity is stress independent (figure 8.3). These results support Yen’s creep model and it was therefore decided to further explore the Beline slope using this model.

Yen describes the yield strength ($\tau_0$) as:

$$\tau_0 = c'_r + \sigma \cdot \tan(\phi'_r)$$ (8.4)

$c'_r$ = residual effective cohesion
$\phi'_r$ = residual effective angle of internal friction

Using the laboratory data (figure 8.4) the effective residual strength of the surface material ($c'_r$ and $\phi'_r$) can be determined. The effective residual cohesion is negligible and
\[ \tan(\phi'_{r'}) = 0.32, \text{ thus } \phi'_{r'} = 18^\circ. \] For the dynamic viscosity the average value of the four tests was taken: \(2 \times 10^8\) kNs/m\(^2\). As comparison, Van Asch and Van Genuchten (1990) found a dynamic viscosity for varved clay in the French Alps of \(2.4 \times 10^8\) kNs/m\(^2\). Ter-Stepanian (1965) reports in-situ determined dynamic viscosity values, calculated after 7 year of displacement in Oligocene claystone and sandstone in the Caucasus, of \(1 \times 10^{10}\) kNs/m\(^2\). Yen (1969) calculates in his creep analysis of a silty clay profile near Malibu, California a dynamic viscosity value of \(1.6 \times 10^8\) kNs/m\(^2\).

The creep model of Yen has the basic assumption of the static infinite slope model (Van Genuchten 1989, Van Asch and Van Genuchten 1990). Ground water is assumed to flow parallel to the ground surface. Considering the stress behaviour of the soil, the soil should consist of isotropically consolidated material and there should be no changes in stress distribution due to creep. The viscosity is a time independent material characteristic. Yen’s creep model implicitly assumes that the subsurface, which is subject to creep, is in residual strength condition. As the Beline slope is the result of an ancient landslide (Asté, 1997) and has numerous mass movement features indicating recent displacement, the assumption of residual strength condition of the subsurface seems to be met.

*Calibration of the Yen creep model at the Beline slope*

Given the coordinate system as shown in figure 8.8, the effective normal stress \((\sigma')\) and shear stress \((\tau)\) are given by the following expressions:

\[
\sigma' = \left( (z - z_w) \gamma'_{s} + z_w \gamma_{u} \right) \cos(\beta) + P \quad (8.5)
\]

\[
\tau = \left( (z - z_w) \gamma'_{s} + z_w \gamma_{u} \right) \sin(\beta) + S \quad (8.6)
\]

\[ z = \text{Soil depth measured perpendicular to the soil surface} \quad [\text{m}] \]

\[ z_w = \text{Depth of phreatic surface perpendicular to the slope} \quad [\text{m}] \]

\[ \beta = \text{Slope angle} \quad [^\circ] \]

\[ \gamma_u = \text{Unit weight unsaturated soil} \quad [\text{kN/m}^3] \]

\[ \gamma_s = \text{Unit weight saturated soil} \quad [\text{kN/m}^3] \]

\[ \gamma' = \text{Submerged unit weight of soil (}\gamma_s - \gamma_w) \quad [\text{kN/m}^3] \]

\[ \gamma_w = \text{Unit weight of water} \quad [\text{kN/m}^3] \]

\[ S = \text{Shear stress overburden} \quad [\text{kN/m}^2] \]

\[ P = \text{Normal stress of overburden} \quad [\text{kN/m}^2] \]

*Figure 8.8*  Coordinate system for the creep description.
Table 8.3 Parameterisation of the Yen creep model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta$</td>
<td>17</td>
<td>$[^\circ]$</td>
</tr>
<tr>
<td>$\gamma_u$</td>
<td>16.5</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$\gamma_s$</td>
<td>20</td>
<td>[kN/m$^3$]</td>
</tr>
<tr>
<td>$c_r$</td>
<td>0</td>
<td>[kN/m$^2$]</td>
</tr>
<tr>
<td>$\phi_r$</td>
<td>18</td>
<td>$[^\circ]$</td>
</tr>
<tr>
<td>$\eta$</td>
<td>$2.00 \times 10^8$</td>
<td>[kNs/m$^2$]</td>
</tr>
</tbody>
</table>

The ground water levels were compiled using the average ground water depth of the measurements at D3 (4.1 m below surface) and adding the modelled ground water level fluctuations (see chapter 7). This was necessary because severe doubts had emerged on the registration of ground water level fluctuations at D3. The creep modelling was executed on a monthly time scale for the first 6 m. The parameterisation is given in table 8.3. It is stressed that the exact point in time of occurrence of displacement is disregarded in this analysis. The phase-shift between observed and modelled ground water level fluctuations is the direct consequence of the unsaturated zone modelling and has been discussed in chapters 6 and 7.

The Yen model was calibrated using the inclinometer results of figure 8.1. The optimisation was performed with the total cumulative displacement as objective criterion. This is 14 mm of displacement in 1996 and 1997 (figure 8.1). Consequently, the calculated displacement velocities are integrated over time to come to the cumulative displacement values. The calibration focuses on the determination of the thickness of the creep zone and its absolute depth. This is also studied as a function of the dynamic viscosity values.

As it is generally very hard to deduce a unique parameterisation for a creep model, a sensitivity analysis is performed in order to narrow down the number of possible solutions for the Beline slope. With a creep model, it is necessary to delimit the creep zone, both in absolute depth (compared with the ground water level) and in total thickness. Two extreme interpretations of the inclinometer results can be given with respect to the deformation zone: a) a creep zone between 3 and 6 m, or b) a thin slipsurface or interval around 4.50 m below surface (see also § 8.2).

The depth where the shear stress ($\tau$) exceeds the yield strength ($\tau_0$) is the upper limit of the creep zone. Therefore one needs the ground water level at which movement was initiated. Starting with the parameterisation of table 8.3 and taking a ground water level of 4 m depth, the top of the creep zone then lies at 4.44 m depth. With a ground water level at 4.1 m below surface the subsurface starts to deform at 4.55 m below surface. As the exact ground water level, which initiated the movement, is unknown, in further analysis, the top of the creep zone was set at 4.5 m below surface.

Displacement by creep can occur, in its extreme, by either a combination of a thick creep zone with relatively high viscosity values or a combination of rigid material and a thin slipsurface with a relatively low viscosity value. This implies that either the creep zone thickness or the viscosity value needs to be tuned in such a way that the modelled cumulative displacement in two years equals the measured displacement under the given ground water level data. As a first step in the analysis of the displacement of the Beline
Results of the Yen creep model with different combinations of creep zone thickness and viscosity values. The dynamic viscosity and creep zone thickness are: model 1: \( \eta = 2.0 \times 10^8 \) kNs/m\(^2\), creep zone from 4.50-4.67 m.; model 2: \( \eta = 1.3 \times 10^{10} \) kNs/m\(^2\), creep zone from 4.50-6.00 m.; model 3: \( \eta = 6.0 \times 10^6 \) kNs/m\(^2\), creep zone from 4.50-4.51 m. Other parameterisation is given in table 8.3.

<table>
<thead>
<tr>
<th>Slipsurface</th>
<th>Interval [cm]</th>
<th>Dynamic viscosity [kNs/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 cm</td>
<td>445-455</td>
<td>6.00E+07</td>
</tr>
<tr>
<td></td>
<td>450-460</td>
<td>8.00E+07</td>
</tr>
<tr>
<td></td>
<td>455-465</td>
<td>1.20E+08</td>
</tr>
<tr>
<td></td>
<td>460-470</td>
<td>1.60E+08</td>
</tr>
<tr>
<td>1 cm</td>
<td>440-441</td>
<td>1.20E+06</td>
</tr>
<tr>
<td></td>
<td>445-446</td>
<td>3.50E+06</td>
</tr>
<tr>
<td></td>
<td>450-451</td>
<td>6.00E+06</td>
</tr>
<tr>
<td></td>
<td>455-456</td>
<td>9.00E+06</td>
</tr>
<tr>
<td></td>
<td>460-461</td>
<td>1.40E+07</td>
</tr>
<tr>
<td></td>
<td>465-466</td>
<td>1.80E+07</td>
</tr>
<tr>
<td></td>
<td>470-471</td>
<td>2.50E+07</td>
</tr>
</tbody>
</table>
slope, the laboratory viscosity value (table 8.3) was used. This gives a creep zone of 17 cm thickness. Figure 8.9 (model 1) shows the calculated displacement using the creep model with a deformation layer between 4.50 and 4.67 m depth compared with the measured inclinometer cumulative displacement.

In contrast, the calibration of the creep model can also start assuming that the thickness of the creep zone is known from the inclinometer tube registration: from 4.5 to 6 m depth. The laboratory viscosity results are in that case assumed to be less reliable. Under these assumptions, it is necessary to increase the dynamic viscosity value for the soil almost 100 times to tune the calculated and measured cumulative displacement (figure 8.9, model 2). This results in an almost continuous creep velocity for two years. Model 3 in figure 8.9 assumes a creep zone of only 1 cm, from 4.50 to 4.51 m below surface. In this case the dynamic viscosity has to decrease to $6 \times 10^6$ kNs/m$^2$ to match measured and calculated cumulative displacement.

From the above it is clear that when the dynamic viscosity is allowed to be optimised too, an infinite amount of parameterisation can be obtained to tune measured and calculated displacement. Table 8.4 gives a few examples of combinations of creep zone thickness, absolute depth and dynamic viscosity values.

Table 8.4 shows that creep patterns with a slipsurface of e.g. 1 cm need much lower dynamic viscosities, and are very sensitive for absolute depth. A subsurface with a creep zone of low viscosity is also very sensitive for the height of ground water above the slipsurface. Figure 8.10 shows the resulting displacement in one month when the maximum ground water level would increase with maximum 25 cm. Here also the example models 1, 2 and 3 of figure 8.9 are used.

![Figure 8.10](image.png)

**Figure 8.10** Changes in monthly displacement as a function of an increase of the maximum ground water level (March 1997) for example models 1, 2 and 3 (see figure 8.9).

The displacement at the Beline site that was measured with the inclinometer shows intermittent steps of movement (with a temporal resolution of a month). From the above it can be deduced that the displacement at the Beline site cannot be the result of a creep zone of several meters. In that case continuous creep should have been recorded. On the other hand, a slipsurface of e.g. 1 cm results in discontinue steps of displacement but would show significant acceleration of displacement as a consequence of only minor ground water level rises. This is hard to check while no reliable ground water level time
series at the inclinometer is available. The measured ground water time series of D1 and D2 suggest that high ground water levels hold for one week to more than a month. Such prolonged periods of high ground water points in the direction of a somewhat more gradual creep process.

The combination of a measured viscosity value of $2 \times 10^8$ kNs/m$^2$, the intermittent character of the inclinometer displacement and the measured time span of high ground water levels suggest that the Beline site is subject to a displacement process of creep in a limited creep zone of several cm’s to dm’s. Therefore, the creep model of 17 cm thickness from 4.50 to 4.67 m below surface with a dynamic viscosity equal to the laboratory tests of $2 \times 10^8$ kNs/m$^2$, was chosen to represent the Beline slope and was taken as point of departure for the scenario study.

Quantification of the effects of ground water level changes on dynamic slope movement

As an example the effects of changes in potential evapotranspiration (PET) on creep behaviour of the Beline slope was studied. Figure 8.11 gives the changes in ground water level at location D3 on the changes in potential evapotranspiration. Figure 8.12 gives the resulting changes of monthly displacement. It shows a doubling of the displacement in case of a 10 % decrease of PET and a 4 to 7 times higher displacement velocity in case of a 20 % decrease in PET. An increase in PET will generally stop the creep process in the Beline slope.

![Figure 8.11 Changes in ground water level as a result of changes in potential evapotranspiration.](image-url)
In a two years period, a total displacement of 14 mm was measured whereas a decrease of PET with 10 % results in a total displacement of 30 mm and a PET decrease of 20 % gives 70 mm of displacement. This is the reaction on an increase of ground water level of respectively 20 and 40 cm. These increases in ground water level seem very realistic compared to the ground water level measurements. Clearly, not (only) the absolute ground water level increase but rather the duration of the higher ground water level is important for the total displacement.

Another aspect is well illustrated in figure 8.12. The first year the slope is subject to wetting, the effects are nearly negligible. In this period the system has the resilience to buffer the increased input. After the buffer is full, the ground water levels climb and the displacement accelerates.

**Conclusion for the study of the displacement of the Beline slope.**

A study was performed to analyse the creep behaviour of the Beline slope. The following points should be highlighted:
- The interpretation of the inclinometer data showed that the Beline system exhibits intermittent displacement (on a monthly temporal scale).
- It is interesting to note that the creep tests (simple shear viscosity tests) delivered indeed a residual strength value with nearly zero cohesion and a lower friction angle. Therefore, it can be concluded that the laboratory viscosity tests supported the (theoretical) creep model of Yen. This model assumes viscosity to be independent of normal stress and the creep threshold (yield strength) to be related to the residual strength of the material.
- The direct shear tests of disturbed samples (table 8.1), the sensitivity analysis of the static slope stability failure (§ 8.3) as well as the analysis of the viscosity tests in terms of residual strength, show residual angle of internal friction of around 18-19°.
- Using the parameterisation of table 8.3 it was recognised that the soil deformation behaviour of the Beline slope had to be limited to a small creep zone. The exact thickness of the creep zone could not be determined. As the Beline site showed only limited displacement, but was subject to ground water level fluctuations of decimetres, it can tentatively be concluded that the displacement at the Beline slope is not concentrated in a slipsurface of a few millimetre thick but in a creep zone with cm or dm dimensions. The creep zone is presumably situated between 4.5 and 5 m depth.
- The kinematic sensitivity for ground water level changes is strongly influenced by the thickness of the creep zone. A creep zone of 1.5 m is not sensitive to ground water level changes. Such a system has a continuously slow displacement under its own weight. On the contrary a thin slipsurface is highly sensitive to ground water level fluctuations, especially if it has limited ground water above the slipsurface. A few centimetres of ground water level rise may then lead to a catastrophic acceleration of the displacement.
- The changing ground water conditions as a result of changes in climate or land use have two important consequences for the creep displacement at the Beline slope. Acceleration of the displacement will occur because of an increased ground water level. Probably more significant is the impact of prolonged periods of high ground water level. This prolonged period of movement is for slopes, which encounter creep features, probably the most important factor for catastrophic failure.
- Although the creep process seems to be responsible for the measured displacement at the Beline site, the data limitations make it impossible to unravel the mass movement process beyond doubts. The assumptions of isotropic and time independent material and stress conditions cannot be validated. The lack of well-recorded ground water level data at the inclinometer tube limits strongly the interpretation. Even more important is the absence of information on the depth and the thickness of the slipsurface or creep zone. Every study on the transient behaviour of a slope remains largely hypothetical if the latter information is missing. The analyses described above show that new techniques for displacement observations which follow the deformation profile without constraints are very important.