SIMULATION OF SLOPE FAILURE EXPERIMENT WITH THE MATERIAL POINT METHOD

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Summary

Finite Element methods (FEM) have been widely used in the last decades for a great number of geotechnical problems. However, difficulties arise with large displacement problems such as landslides, pile driving, underground excavations, etc. The Material Point Method (MPM) has been specifically developed to overcome FEM drawbacks in large displacement problems. In MPM the continuum is discretized by material points (MP), which move through a background mesh, thereby following the large deformations of the solid. The MP carry all the properties of the continuum, while the background mesh is used to solve the governing equations at each time step, but does not store any permanent information. In this paper it is shown that with the MPM it is possible to reproduce the large deformations involved in slope failure. Numerical results are compared to a reference laboratory test.

1. Introduction

The Finite Element Method (FEM) has been successfully applied in many fields of engineering and science in the last decades, but shortcomings appear in the field of large displacement problems, mainly because of mesh distortions.

The Material Point Method (MPM) can be considered as an evolution of the updated Lagrangian FEM, specifically developed for large displacements problems. The governing equations are solved at the nodes of the grid, similarly as FEM, but the deformations of the body are simulated by material points moving through a fixed mesh, therefore preventing issues of mesh distortions. A brief overview of the method is provided in Section 2.

Thanks to its ability to easily model large displacements, the method can be applied for the
simulation of problems such as landslides, pile driving, underground excavations etc. In this paper the MPM is used to simulate the failure of a submerged slope.

The stability of submerged slopes is an important issue in many countries. In the Netherlands the problem has a great impact in the province of Zeeland, in the south-east of the country, characterized by numerous islands. The shoreline has been severely damaged by sea attack, and submarine landslides compromise the safety of the area. The phenomenon needs to be deeply investigated in order to enforce the design of mitigation techniques.

A set of small scale laboratory tests has been performed at Deltares, moreover, numerical modelling adds interesting insight in the understanding of the problem. Indeed, the scale effect can be easily investigated and parametric studies can be done.

Section 3 describes the laboratory test which is considered as a reference for the numerical simulation. The MPM model is presented in Section 4, followed by the results in Section 5. The paper ends with conclusions and an outlook on future developments.

1 The Material Point Method

The Material Point Method (MPM) is an advanced numerical method particularly suited to model large deformations. It was firstly developed at the end of the previous century (Sulsky et al., 1994) for solid mechanics and later applied to granular materials by Więckowski (1999, 2004) and Coetzee (2005). It recently found entrance into the field of geotechnical engineering (Beuth, 2012; Alonso & Zabala, 2011; Al-Kafaji, 2013; Bandara, 2013). In the following, the method is briefly described. For more detailed information the reader is referred to Al-Kafaji (2013) and Bandara (2013).

In the MPM the continuum soil body is represented by a cloud of Lagrangian points, called material points (MP). Large deformations are modelled by MP moving through a fixed background mesh, which covers the entire region where the body is expected to move into. The MP carry all the physical properties of the continuum such as density, momentum, material parameters, strains, stresses, as well as external loads, whereas the background mesh is used to solve the governing equations within each time step, but stores no permanent information.

Very often the soil behaviour highly depends on the solid-water interaction. The implementation of the fully coupled two-phase approach follows the v-w-formulation (Zienkiewicz, 1999), i.e. the primary unknowns are the velocity of the water and the velocity of the soil. This formulation, which takes into account all acceleration terms, has proved to be able to capture the physical response of saturated soil under dynamic loading (Van Esch et al., 2011).

The current implementation uses an explicit time integration scheme (Al-Kafaji, 2013). For each time step, the background finite element mesh is used to solve the system of equilibrium equations and to determine the velocity of the soil and the water. Once the strains are determined at the locations of the material points, the mesh is usually reset into its original state. Figure 1 illustrates a single calculation step in MPM.

![Figure 1](image_url)  
Figure 1  
a) Configuration of material points (red) and background mesh at the beginning of a calculation step; b) Deformed mesh after solving the equilibrium equations; c) Background mesh in initial position at the end of a calculation step and new location of material points which transport stresses, strains and material parameters.

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2 The physical model

The slope is completely submerged in a test flume which is 5.4m long, 2.5m high and 0.5m wide. The sand is placed in the flume using a fluidisation system: water is injected through the tubes placed on the bottom of the flume and, due to the upward water flow, the whole sand pack goes into suspension. After stopping the water supply the sand settles in a very loose state.

The slope is built by slowly sucking the sand through a nozzle, it then flows under a natural slope of the embankment.

After the preparation phase, the slope has an inclination of 31° and a height of 0.60 m. Afterwards, in the experiment, the failure is triggered by injection of water under the toe of the slope.

The first macro-scale movement is observed in a superficial layer of about one decimetre sliding downward. This lasted for a few seconds (5 to 10 seconds). The movement continued slowly in a thinner layer. The whole process took about a minute (Stoutjesdijk, 2014).

3 The numerical model

The geometry just before triggering the failure is considered as the initial situation of the numerical analyses, see Figure 2. The discretized domain is shown in Figure 3. The mesh consists of 2 202 elements, 1567 of which are active, i.e. filled with 4 MP each at the beginning of the calculation. The part of the mesh in which most of the deformation is expected is refined to increase the accuracy of the results. The final discretization has been chosen after some sensitivity analysis as result of a good compromise between accuracy and computational efficiency.

At the left and right boundary the displacements are constrained in horizontal direction, while at the bottom no displacements are allowed at all. All boundaries are impermeable for water, except during triggering of the failure at the location where the water-pressure is applied.

An elasto-plastic model with Mohr-Coulomb failure criterion is used for the sand, whose input parameters are shown in Table 1. The input parameters are according to specifications derived from the experiment. A local damping factor of 5% is used for the calculation; this value simulates natural energy dissipation inside the material, which is not taken into account by the constitutive model.

The failure is triggered by applying an excess pore pressure at the bottom of the domain, below the toe of the slope. The pore pressure is increased linearly from 0 to 10 kPa in 5.0 s. After that, the pore pressure is reduced to zero again.

Table 1 Material parameters for the sand

<table>
<thead>
<tr>
<th>Material parameter</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>saturated unit weight of sand</td>
<td>γ&lt;sub&gt;sat&lt;/sub&gt;</td>
<td>18.7</td>
<td>kN/m&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
<tr>
<td>Young modulus of sand</td>
<td>E</td>
<td>5 000</td>
<td>kPa</td>
</tr>
<tr>
<td>Poisson ratio of sand</td>
<td>ν'&lt;sup&gt;'&lt;/sup&gt;</td>
<td>0.2</td>
<td>–</td>
</tr>
<tr>
<td>bulk modulus of water</td>
<td>K&lt;sub&gt;w&lt;/sub&gt;</td>
<td>45 310</td>
<td>kPa</td>
</tr>
<tr>
<td>initial porosity</td>
<td>n</td>
<td>0.45</td>
<td>–</td>
</tr>
<tr>
<td>permeability</td>
<td>k</td>
<td>1.0*10&lt;sup&gt;-4&lt;/sup&gt;</td>
<td>m/s</td>
</tr>
<tr>
<td>cohesion</td>
<td>c'&lt;sup&gt;'&lt;/sup&gt;</td>
<td>0</td>
<td>kPa</td>
</tr>
<tr>
<td>friction angle</td>
<td>φ</td>
<td>32</td>
<td>deg</td>
</tr>
<tr>
<td>dilatancy angle</td>
<td>ψ</td>
<td>0</td>
<td>deg</td>
</tr>
</tbody>
</table>
2. Results

The initial stresses are generated using a gravity loading phase. A quasi-static convergence criterion is applied, which implies that the slope is in static equilibrium and the kinetic energy and the unbalance force are below a limit value. The pore pressure distribution is initially hydrostatic.

After the initialisation phase, the failure is triggered as described in the previous section. The excess pore pressure is applied at the bottom of the mesh below the toe of the slope, and the excess pore pressures propagate upwards. While the excess pore pressures increase, the effective stresses decrease causing instability of the slope. The first clearly visible displacements appear at the crest of the slope after 3.75 s since pore pressure application, and the new equilibrium configuration is reached after approximately 8.5 s. The failure surface localizes at a depth of about 0.15 m.
The final equilibrium state is compared with the experimental results in Figures 4 and 5. It can be concluded that the numerical simulation is in very good agreement with the experiment.

![Figure 4](image1.jpg)

Figure 4 Final state of the slope after failure in the experiment (blue line). The initial state is indicated by the red line.

![Figure 5](image2.jpg)

Figure 5 Comparison of numerical results (MPM) with experimental results. The red line indicates the initial situation and the blue line the final state after failure. Displacements are shown in [mm].

3. Conclusions and future developments

It is demonstrated that with MPM it is possible to model large displacement problems. The failure of a submerged slope triggered by a suddenly increased water pressure at the bottom can be successfully simulated using the fully coupled MPM implementation. The numerical results of the deformed slope in the final equilibrium state are in good agreement with the experimental data (see
The behaviour of sand is complex; the elasto-plastic model with Mohr-Coulomb failure criteria is a very simplified way of describing soil behaviour. Deeper understanding of the failure process could be achieved with more sophisticated material models such as the Hypoplastic model (Niemunis, 1997), the NorSand model (Jefferies, 1993) and the Mohr-Coulomb with strain softening model (Abbo, 1995). The latter proved to be able to capture the progressive failure of the slope (Alonso & Zabala, 2011; Yerro et al., 2014).

It is of great interest to investigate the behaviour of true scale slopes, as found in the region of Zeeland, whose high ranges between 10 and 50 m. Special attention needs to be paid to the stiffness and the density of the sand which is not assumable to be constant with depth, especially for slopes of larger high. Therefore it is expected that slopes with larger scale would behave differently than slopes in model scale. Numerical simulations can indicate safe inclination angle in the natural conditions.

Revetments of various types have been used to prevent erosion and improve the stability of the slope. The effect of a stone revetment will be considered in the future also in the numerical simulations.

4 Bibliography


Bandara S. (2013). Material Point Method to simulate Large Deformation Problems in Fluid-saturated Granular Medium. University of Cambridge, United Kingdom.


