NUMERICAL MODELING OF SUBMARINE FLOWSLIDES: MPM AND FEM APPROACH

Mario Martinelli (mario.martinelli@deltares.nl)
Deltares, Delft, The Netherlands.

Francesca Ceccato (francesca.ceccato@deltares.nl)
University of Padua, Italy

Alexander Rohe (alex.rohe@deltares.nl)
Deltares, Delft, The Netherlands & University of Cambridge, UK

ABSTRACT.
In this paper a particular submarine slope, tested in the Deltares laboratory, is studied with two different numerical techniques to simulate its failure: the standard FEM and the Material Point Method.

1. Introduction

The coastal protection is an extremely important challenge for the safety and economic development of in the Netherlands. Zeeland is a region particularly affected by flow slides and liquefaction; it is a province located in the south-west of the country, consisting of a number of islands and a strip bordering Belgium. The security of the environment that lives on this geographic area is directly related to the safety of their coasts affected by the sea.

In fact, several damages due to submarine flow slides have been observed along the shoreline and edges of shelf plates. The observed timescale of failure is significantly short and usually consists of several hours. In practice sliding is often prevented by reinforcement of the whole slope by stone revetments. Stone revetments have proven to be efficient as they prevent further erosion of the slope (in combination with a filter construction) and fix the geometry.

The effect of this solution is basically to increase the effective stress and the soil strength, preventing thereby further erosion of the slope. However, even if this procedure reduces the risk of failure, the global behaviour is not sufficiently understood, especially in the framework of a proper design technique.

The slope under consideration is mainly below sea water level and is representing a common geometry configuration of coastal zones in Zeeland. In fact, slopes in Zeeland got steeper in the past by erosion and the process of deepening/meandering of nearby flow channels. The inclination of the slopes in which flow slides occurred is often in the range 1:7 to 1:2 (dependent on the height) with heights between 10 and 50 m (dependent on the slope angle). The Holocene sand layers are generally loosely packed and the slopes are therefore classified as highly susceptible for liquefaction.

This paper describes only the first preliminary step of this study, in which the submerged slope is analysed without stone revetment. Two numerical techniques are used to simulate the problem: the standard FEM (implemented in PLAXIS software) and the Material Point Method, a code developed at Deltares (named Deltares-MPM for the whole document). Details of the MPM formulation in dry and saturated materials are described in the works of Al-Kafaji I. (2013) and Bandara S. (2013).

2. Geometry, boundary conditions and parameters

The geometry, shown in Figure 1a, is according to the experimental set-up of a series of tests performed in the laboratory of Deltares: the height of the slope is 1.2 m and the thickness of base layer is 0.2 m. The slope is fully saturated and below water level. The loading is represented by an infiltration of water at the bottom of the geometry through the application of a uniformly distributed increase of pore pressure at the lower boundary of the model. The pore pressure increases linearly from 0 to p_{max} in $t_{loading}$ and then is reset to 0 (Figure 1b). A linear elastic soil model associate with the Mohr-Coulomb yield criteria is used in the analyses and the list of parameters is shown in Table 1. The following assumptions are made in first instance: the dilatancy is set equal to zero, and permeability and stiffness are constant in depth and time.

The slope is modelled in PLAXIS assuming two-dimensional plane strain condition (see Figure 2a) with high order triangle elements, 15 nodes and cubic shape function. On the other hand, a quasi plain strain condition is assumed in Deltares-MPM considering only a strip of tridimensional elements, 4 nodes tetrahedrons with linear shape function (see Figure 2b). Figure 2b shows the mesh at the beginning of the calculation that is divided in two parts. The elements filled by dots (Material Points) represents the “active” mesh, in which 4 material points per element are set up. The empty elements are the “non-active” mesh in which no material point is initially assigned. However, any of these elements that initially belong to the “non-active” mesh may become “active” during the calculation if any material point moves into.

In Figure 2a two points (A and B) are selected in which displacements (in point A) and excess pore pressures (in points B) are recorded.

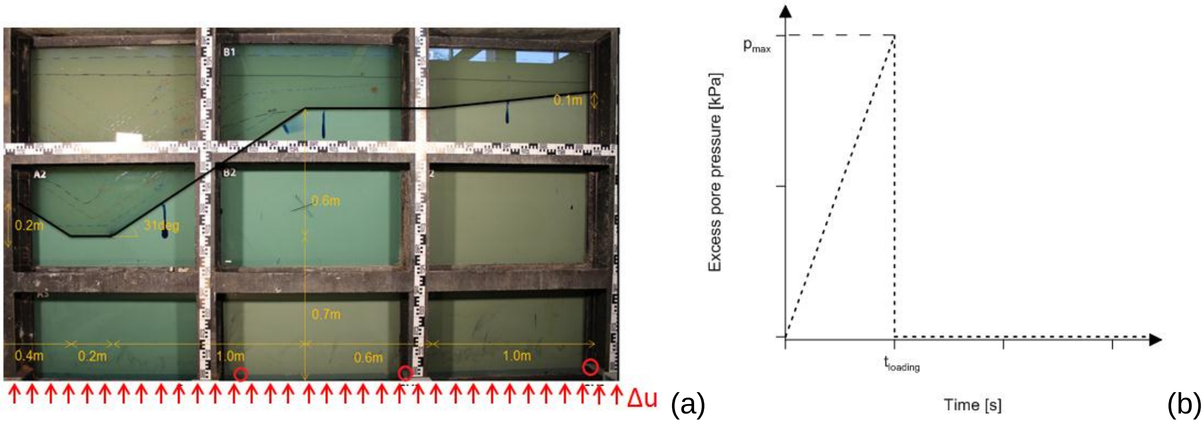


Figure 1: (a) Geometry of the model according to the set-up of the experiment in the Deltares laboratory. (b) Increment of excess pore pressures applied at the bottom of the mesh.

Table 1: Parameters used for the analyses.

Saturated unit weight	γ_{sat}	18.71	kN/m^3
Young modulus	E	5000	kPa
Poisson ration	ν'	0.2	-
porosity	n	0.45	-
Soil permeability	k	10^{-4}	m/s
Cohesion	c'	0	kPa
Friction angle	φ	32	deg
Dilatancy angle	ψ	0	deg

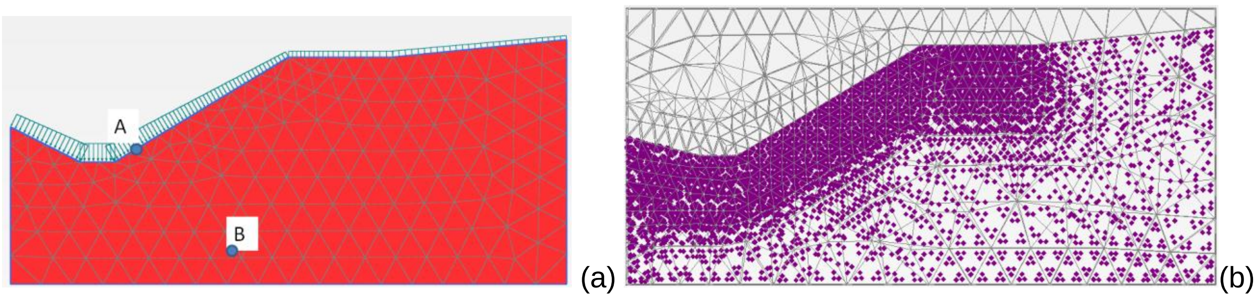


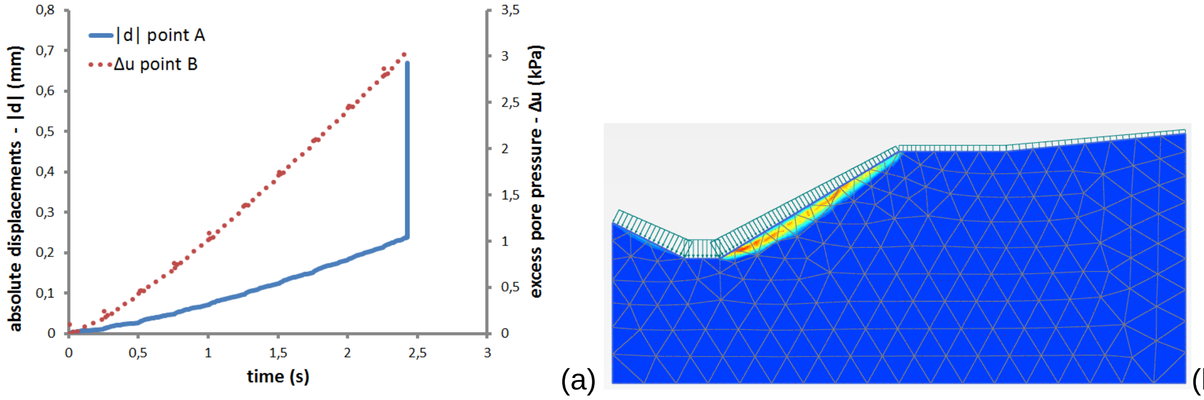
Figure 2: Mesh used in PLAXIS (a) and in Deltares-MPM (b).

3. FEM simulation in PLAXIS

A relatively simple consolidation analysis is performed in PLAXIS, thus no inertia forces are taken into account. The prescribed time-dependent boundary condition (linear pore pressure in time) is not possible to be applied at the base of the model, thus the calculation is set as a sequence of loading steps: the hydraulic head is constant within each step and each increment is equal to 0.25 m from one step to the other.

Figure 3a shows the total displacements of point A located close to the toe of the slope, and the excess pore pressure recorded in point B. It can be observed that as the pore pressure increases into the soil, the slope displacement increases as well, but the quasi-static equilibrium condition is satisfied only up to t = 2.42 s. Right afterwards, the slope starts failing with significant mass acceleration but no equilibrium condition can be satisfied with this type of analysis scheme because it does not take into account the contribution of inertia forces. The deviatoric strain contour is plotted in Figure 3b at time t = 2.42 s, in which can be observed that only the thin superficial part of the soil starts failing, the one with extremely low effective stress.

It is worth noticing that this analysis cannot provide any additional information regarding the evolution of the process, such as the description of the run out and the final static configuration; this kind of information can only be achieved when performing not only dynamic coupled consolidation analyses but also taking into account large deformations updating the mesh (Updated-Lagrangian formulation). However, in some cases, even the large deformation approach is not enough to properly capture the evolution of the landslide; therefore MPM is a suitable approach that can be used for this purpose, because it can describe the problem from the beginning, where the displacements are small, to the very end after extremely large deformation.



(a) (b)

Figure 3: (a) total displacements in A (blue line) and excess pore pressure in B (red line); (b) contour plot of deviatoric strain after 2.42 s.

4. MPM simulation in Deltares-MPM

The MPM is an extension of the Updated-Lagrangian formulation, explicitly formulated to overcome the drawbacks of mesh distortion. The reader can refer to the work of Al-Kafaji (2013) for the details of the formulation used in Deltares-MPM. The following aspects are taken into account for the Deltares-MPM analyses:

- Fully coupled analysis with generation and dissipation of (excess) pore pressures;
- Large deformations, i.e. post-failure behaviour can be assessed.
- Dynamic behaviour (inertia effects).

Figure 4 shows the final configuration of the slope after 16 s. The red line represents the initial shape before the test, whereas the black line represents the final configuration observed in the experiment after failure. It can be observed that the numerical simulation results match the final geometry of the slope in the experiment quite well.

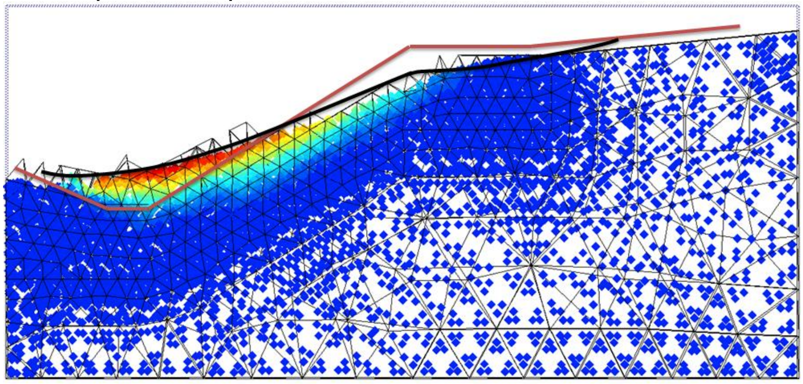


Figure 4: Final configuration of the slope after failure at 16 s in the MPM simulation.

The research leading to these results has received funding from the European Union Seventh Framework Programme (FP7/2007-2013) under grant agreement no. PIAP-GA-2012-324522 “MPM-DREDGE”.

5. References

Al-Kafaji I. (2013). Formulation of a Dynamic Material Point Method (MPM) for Geomechanical Problems. *Ph.D. Thesis.* University of Stuttgart 2013.

Bandara S. (2013). Material Point Method to simulate large deformation problems in fluid-saturated granular medium. *Ph.D. Thesis.* University of Cambridge. 2013.