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Haxaire, A; Aukenthaler, Manuel; Brinkgreve, R. B.J.

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Application of a Thermo-Hydro-Mechanical Model for Freezing and Thawing

A. Haxaire^{a,*}, M. Aukenthaler^a, R.B.J. Brinkgreve^b

^aPlaxis bv, Delft, The Netherlands

^bDepartment of Geoscience & Engineering – Delft University of Technology, Delft, The Netherlands

Abstract

Recently, a constitutive model has been developed to describe the mechanical behaviour of frozen soil as a function of temperature, all the way to the unfrozen state, and vice versa [1]. The model has been implemented as a user-defined soil model (UDSM) in the geotechnical finite element code PLAXIS 2D and applied in practical thermo-hydro-mechanical boundary value problems. One of the problems with the use of a model for frozen / unfrozen soil is that it involves several parameters of which quite a few are not very common to geotechnical engineers. Hence, one of the goals of this study is to provide more information on the meaning and the determination of the model parameters. As an example of the use of the constitutive model and its implementation, one application is presented: it is a chilled pipeline causing a drop in the ground temperature leading to frost heave. Its results show that the model can simulate frost heave in a qualitative manner, and that the robustness of the numerical implementation is still sensitive to the choice of boundary conditions, temperature gradients, and time.

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Keywords: Thermo-hydro-mechanical couplings; soil freezing, constitutive modelling

1. Introduction

The replication of the behaviour of frozen soils has been studied for decades [2]. Many attempts have been undertaken to either develop new constitutive models or to improve already existing models to simulate the behaviour of frozen geomaterials. To handle the challenges of ground freezing, cold regions engineering and

* Corresponding author. Tel.: +31152517720; fax: +31152573107.

E-mail address: a.haxaire@plaxis.com

periglacial processes, it is vital to understand the mechanical behaviour of frozen soil. Knowing that field studies, large scale laboratory tests and centrifuge modelling offer a good insight, they are however expensive and time consuming. A numerical modelling approach is therefore necessary. The Norwegian University of Science and Technology (NTNU), in collaboration with Plaxis bv, developed a new numerical model [1] to tackle these problems. The aim of this new approach is to provide a reliable design tool to assess the impact of climate change and changes in temperature in general on a variety of engineering problems, in particular ground freezing for mine shaft construction.

2. The constitutive model

In this section we provide a short insight in the formulation and capabilities of the model. The constitutive model is described in [1].

2.1. General description

The constitutive model for frozen and unfrozen soil is a critical-state elasto-plastic mechanical soil model formulated within the framework of two-stress state variables. The stress state variables are the cryogenic suction and the solid phase stress. The latter one is considered as the combined stress of the soil grains and ice and is defined as:

$$\sigma^* = \sigma - S_{uw} p_w \quad (1)$$

where σ^* is the solid phase stress, σ is the net stress, S_{uw} is the unfrozen water saturation and p_w is the pore water pressure. According to this formulation, the saturated frozen soil can be viewed as a porous material consisting of soil grains and ice, in which the pores are filled with water, i.e. the model requires the soil to be fully saturated. Once the phase transition from liquid water to ice takes place, ice is considered as part of the solid phase and contributes to its stress, because it is able to bear shear stresses. This kind of formulation based on effective stress is a Bishop single effective stress, which involves the unfrozen water saturation S_{uw} as the effective stress parameter or Bishop's parameter. The solid phase stress can reflect the effect of unfrozen water on the mechanical behaviour.

The cryogenic suction, s_c , is used as the second state variable. It is the difference between the ice pressure and the pore water pressure and can be obtained using the Clausius-Clapeyron equation [3]. This allows the construction of a complete hydro-mechanical framework. By considering the cryogenic suction, it is possible to take the effects of ice content and temperature variation into account. For completion, s_c is defined as:

$$s_c = p_{ice} - p_w \approx -\rho_{ice} L \ln \frac{T}{T_f} \quad (2)$$

where p_w and p_{ice} indicate the pore water and ice pressure, respectively; ρ_{ice} is the density of ice and L is the latent heat of fusion of water. T represents the current temperature in Kelvin and T_f is the melting/freezing temperature of ice/water for a given soil and pressure.

2.2. Elastic response

The elastic part of strain due to the solid phase stress variation can be calculated from Hooke's law based on the equivalent elastic parameters of the mixture:

$$K = (1 - S_i) \frac{(1+\epsilon)p_{y0}}{\kappa_0} + \frac{S_i E_f}{3(1-2\nu_f)} \quad (3)$$

$$G = (1 - S_i) G_0 + \frac{S_i E_f}{2(1+\nu_f)} \quad (4)$$

where G and K are the equivalent stress-dependent shear modulus and bulk modulus of the mixture, respectively. κ_0 stands for the constant elastic compressibility coefficient and G_0 is the shear modulus of the soil in an unfrozen state. p_{y0}^* is the pre-consolidation stress for unfrozen conditions. E_f and ν_f denote the Young's modulus and the Poisson's ratio of the soil in the fully frozen state, respectively. Finally, S_i is the ice saturation.

Considering the temperature-dependent behaviour of ice, the Young's modulus E_f is defined in Eq. (5), where $E_{f,ref}$ is the value of E_f at a reference temperature T_{ref} and $E_{f,inc}$ is the rate of change in E_f with temperature. Having defined the elastic part of strain due to the solid phase stress variation, the elastic part of strain due to suction variation is given in Eq. (6), where κ_s is the compressibility coefficient due to cryogenic suction variation within the elastic region, v is the specific volume and p_{at} is the atmospheric pressure.

$$E_f = E_{f,ref} - E_{f,inc}(T - T_{ref}) \quad (5)$$

$$d\varepsilon^{se} = \frac{\kappa_s}{v} \times \frac{ds_c}{(s_c + p_{at})} \quad (6)$$

2.3. Yield surface definition

In its unfrozen state, the model becomes a conventional critical state model. In other words, when the value of the cryogenic suction equals zero, the model reduces to a common unfrozen soil model. The simple modified Cam-clay model is adopted for the unfrozen state. Considering the frozen state, two suction-dependent yield functions are applied. Based on the Barcelona Basic Model (BBM) [4] the so-called loading collapse (LC) yield surface due to the variation of solid phase stress is expressed as:

$$F_1 = (p^* + k_t s_c) [(p^* + k_t s_c) S_{uw}^m - (p_y^* + k_t s_c)] + \frac{(q^*)^2}{M^2} \quad (7)$$

where

$$p_y^* = p_c^* \left(\frac{p_{y0}^*}{p_c^*} \right)^{\frac{\lambda_0 - \kappa}{\lambda - \kappa}} \quad (8)$$

$$\lambda = \lambda_0 [(1 - r) \exp(-\beta s_c) + r] \quad (9)$$

In Eq.(7), p^* is the solid phase mean stress, q^* is the solid phase deviatoric stress, M denotes the slope of the critical state line (CSL), k_t is the parameter for describing the increase in apparent cohesion with cryogenic suction, p_c^* indicates the reference stress, and κ denotes the compressibility coefficient of the system within the elastic region. Furthermore, λ_0 is the elasto-plastic compressibility coefficient for unfrozen state along virgin loading, r is a constant related to the maximum stiffness of the soil (for infinite cryogenic suction) and β is the parameter controlling the rate of change in soil stiffness with cryogenic suction.

This formulation can take into account the influence of the unfrozen water saturation S_{uw} . The exponent m dictates the extent to which this behaviour is considered.

An increase in cryogenic suction leads to grain segregation and ice lens formation, and results in the expansion of the soil. This deformation is considered as the part of deformation due to suction variation which induces irrecoverable strains, the plastic part. Therefore, a simple second suction-dependent yield criterion is adopted to capture this phenomenon. The Grain Segregation (GS) yield criterion can be written as follows:

$$F_2 = s_c - s_{c,seg} \quad (10)$$

where $s_{c,seg}$ is the threshold value of suction for ice segregation phenomenon and bounds the transition from the elastic state to the virgin range when cryogenic suction increases. Figure 1 summarises these two yield criteria.

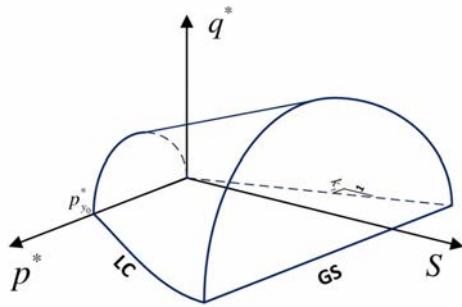


Fig. 1. The three-dimensional yield surface.

2.4. Model parameters

The current model requires seventeen parameters in total. Eleven parameters describe the behaviour under the variation of solid phase stress. These are namely κ_0 , G_0 , $E_{f,ref}$, $E_{f,inc}$, v_f , p_y^* , p_c^* , λ_0 , M , m , and γ . Three parameters describe the behaviour regarding suction-induced strains ($s_{c,seg}$, κ_s and λ_s). Finally, three parameters account for coupling effects between the variation of solid phase stresses and the cryogenic suction (β , r and k_t). These parameters are summarised in Table 1.

3. Application: chilled pipeline

A chilled pipeline is buried in unfrozen ground. Its cold temperature triggers the frost heave phenomenon. Frost heave and ice segregation can potentially cause many engineering problems, like cracking of pavements and fractures of pipelines. It is therefore of particular concern in highway and pipeline engineering. This first simulation provides meaningful insights for modelling the construction of a mine shaft in soft rock, where artificial ground freezing can be used to strengthen the rock to allow the excavation.

In [6], frost heave is explained as the ground expansion caused by water migration and accumulation in the transitional zone just behind a freezing front, where soil is partially frozen. The water migration is driven by the cryogenic suction, but at the same time hindered by the reduced permeability developed in partially frozen soils.

The soil is fully saturated, which is essential to use this new constitutive model. The parameter sets given in Table 1 are used. The determination of the soil freezing characteristic curve (SFCC) and the related unfrozen water saturation, as well as the one of the cryogenic suction and all the hydraulic parameters are based on the practical approach described in [5], where, as here, the grain size distribution of the used clay and sand are based on the U.S.D.A. default values.

3.1. Geometry and boundary conditions

A pipeline ($\varnothing 0.60$ m) is buried in a 1.30 m deep trench at a depth of 1.20 m. The excavated trench in the clay layer is then backfilled with sand. The pipeline has a bending stiffness of $EI = 282,000$ Nm 2 /m. The modelled domain is 3.00 m wide and 3.00 m deep. The symmetry of the investigated problem is taken into account, thus, only half of the pipeline is modelled. Relative fine meshing is used to account for the rapid change in unfrozen water saturation and hydraulic conductivity.

A constant air temperature of 293 K is considered, with an assumed surface transfer of 300 W/m 2 . The temperature at a depth of 3.00 m is set to 283 K. Due to symmetry reasons the left boundary is closed and no heat flux is allowed, whereas seepage is possible at the right boundary of the model. The initial ground temperature field and the boundary conditions are shown in Fig. 2.

Table 1. Constitutive model parameters for clay and sand.

Parameter	Description	Clay	Sand	Unit
G_0	Unfrozen soil shear modulus	2.22×10^6	6.00×10^6	N/m ²
κ_0	Unfrozen soil elastic compressibility coefficient	0.08	0.15	-
$E_{f,ref}$	Frozen soils Young's modulus at a reference temperature T_{ref}	6.00×10^6	20.0×10^6	N/m ²
$E_{f,inc}$	Rate of change in Young's modulus with temperature	9.50×10^6	100×10^6	N/m ²
v_f	Frozen soil Poisson's ratio	0.35	0.30	-
m	Yield parameter	1.00	1.00	-
γ	Plastic potential parameter	1.00	1.00	-
$(p_{y0}^*)_{in}$	Initial pre-consolidation stress for unfrozen condition	300×10^3	800×10^3	N/m ²
p_c^*	Reference stress	45.0×10^3	100×10^3	N/m ²
λ_0	Elasto-plastic compressibility coefficient for unfrozen state	0.40	0.50	-
m	Slope of the critical state line	0.77	1.20	-
$(s_{c,seg})_{in}$	Initial segregation threshold	3.50×10^6	0.55×10^6	N/m ²
κ_s	Elastic compressibility coefficient for cryogenic suction variation	0.005	0.001	-
λ_s	Elasto-plastic compressibility coefficient for cryogenic suction variation	0.80	0.10	-
k_t	Rate of change in apparent cohesion with cryogenic suction	0.06	0.08	-
R	Coefficient related to the maximum soil stiffness	0.60	0.60	-
β	Rate of change in soil stiffness with cryogenic suction	0.60×10^{-6}	1.00×10^{-6}	(N/m ²) ⁻¹

Table 2. Thermal properties for clay and sand.

Parameter	Clay	Sand	Unit
c_s	945	900	J/kg/K
λ_s	1.50	2.50	W/m/K
ρ_s	2700	2650	kg/m ³
$\alpha_{x,y,z}$	5.20×10^{-6}	5.00×10^{-6}	1/K

3.2. Simulation and results

After placing the pipeline and refilling the trench with the backfill material, the cooled fluid in the pipeline causes a decrease of the surrounding temperature. The temperature of the fluid is 253 K. It is assumed that it takes 10 days until the pipeline cools down to 253 K. During another 30 day period the temperature stays constant. Stress points hit the Grain-Segregation yield surface and cause the accumulation of dilative plastic strains. Fig. 4 shows the ice saturation and Fig. 5 the deformed mesh after the total time period of 40 days. A frost heave of about two centimetres takes place. After these 30 days of constant temperature a decrease in air temperature of 5 K over a long period is considered. The temperature at the bottom boundary stays constant. The purpose of this simulation is to demonstrate how frost heave evolves with time and changing temperature regimes. A new temperature distribution is reached (see Fig. 3). The deformed mesh shows a frost heave of 3.5 cm and is illustrated in Fig. 5. The final ice saturation after this cooling period is presented in Fig. 4.

It is visible that clay shows more frost heave than sand and is responsible for the pipeline moving upwards. On the one hand this can be explained due to the choice of differing parameters regarding to cryogenic suction-induced strains ($s_{c,seg}$, λ_s and κ_s). On the other hand, elasto-plastic model parameters, as well as the intrinsic permeability also play a key role in frost heave simulations.

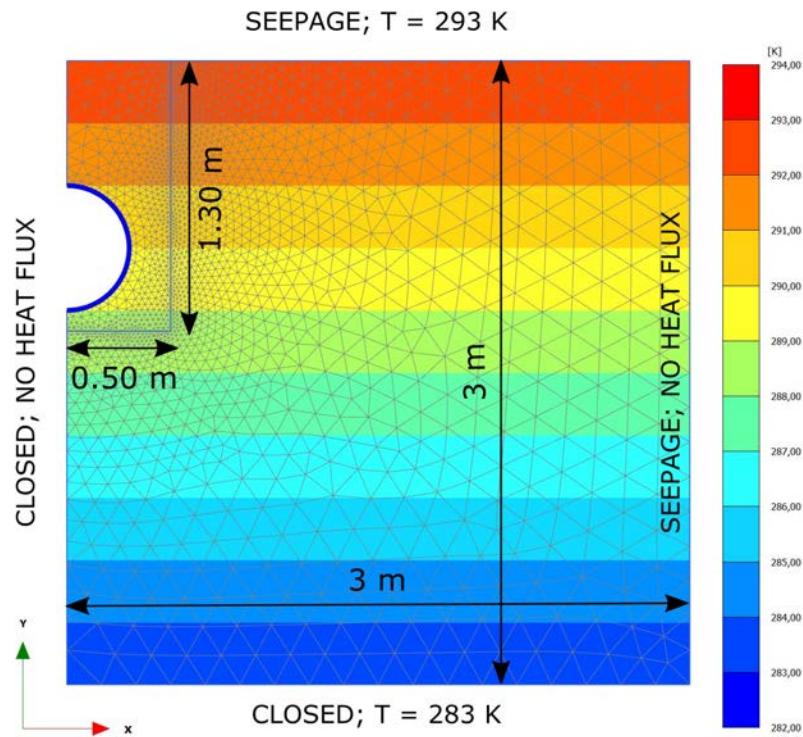


Fig. 2. Geometry and boundary conditions.

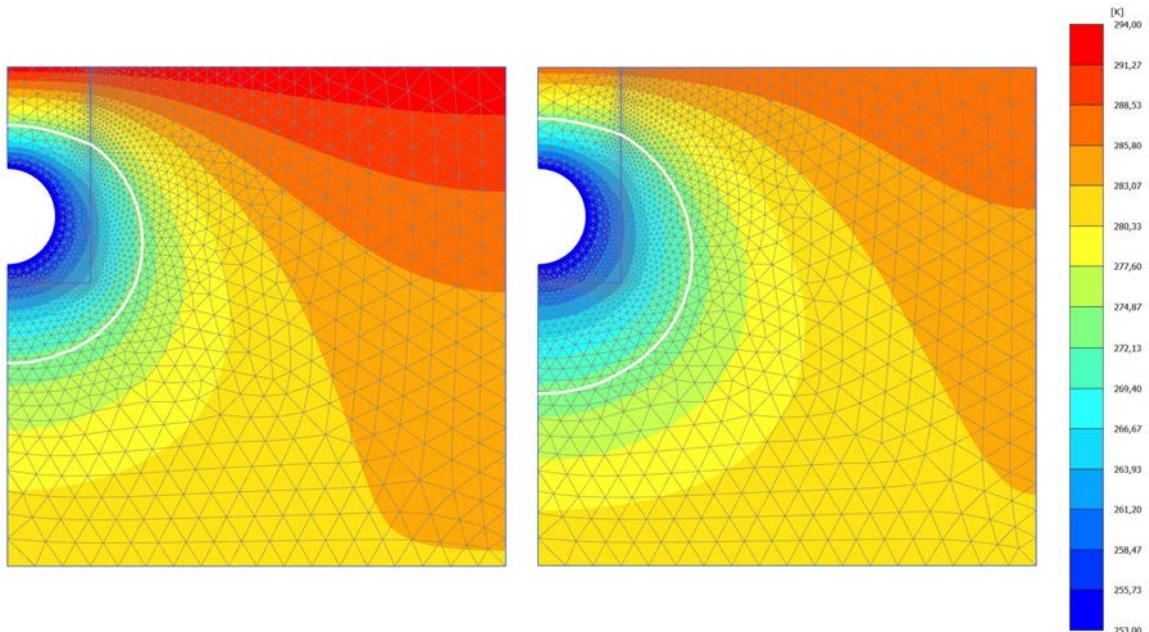


Fig. 3. Temperature distribution after 40 days (left) and after the cooling period (right).

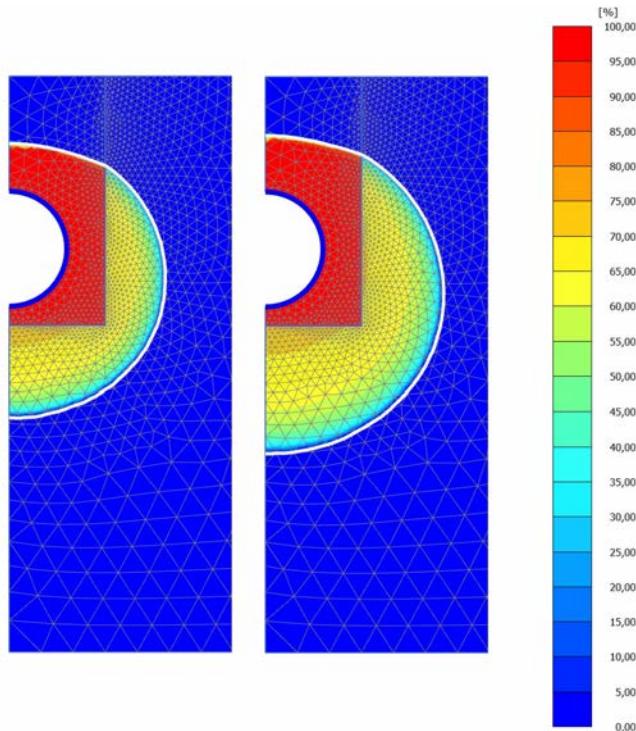


Fig. 4. Ice saturation after 40 days (left) and after the cooling period (right).

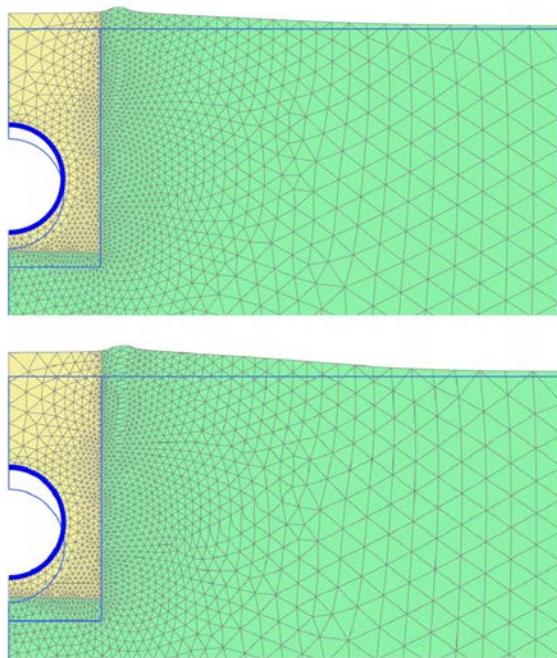


Fig. 5. Upper deformed mesh: after 40 days (top) and after the cooling period (bottom).

4. Conclusion

A new constitutive model [1] describes the mechanical behaviour of frozen soil as a function of temperature, all the way to the unfrozen state, and vice versa. It has been implemented in a fully coupled THM finite element code. This allows the study of a variety of geotechnical issues involving freezing and thawing of soils and soft rocks. The constitutive model requires several parameters, of which quite a few are not very common to geo-engineers. Although some correlations and default values can be provided, the number of parameters to determine remains high. The calibration methods to determine the BBM parameters mentioned in section 3.3 require further development and testing. Many essential features of the mechanical behaviour of frozen and unfrozen soil can be captured with this new constitutive model, like the temperature dependence of stiffness and shear strength. Furthermore, frost heave can be simulated. This phenomenon plays a key role in designing in, on and with frozen and unfrozen soil and may cause significant engineering problems. Cyclic behaviour, as well as time dependent behaviour are not yet implemented in the formulation of this new constitutive model. The application presented in this article has shown that the robustness of the numerical implementation is still sensitive to the choice of boundary conditions, temperature gradients and time. Further research focuses on the reproduction of field test data, in particular the modelling of the construction of a mine shaft where artificial ground freezing is needed.

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