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Evaluation and application of a constitutive model for frozen and unfrozen soil

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ABSTRACT
Recently, a constitutive model has been developed describing the mechanical behaviour of frozen soil as a function of temperature, all the way to the unfrozen state, and vice versa (Ghoreishian et al., 2016). The model has been implemented as a user-defined soil model 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 our 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, two hypothetical cases are presented. The first one is a chilled pipeline causing a drop in the ground temperature leading to frost heave. The second one is a raft foundation on frozen ground subjected to a thawing period.

RÉSUMÉ
Récemment, une loi de comportement a été développée pour décrire le comportement mécanique des sols gelés en tant que fonction de la température, d'un état complètement gelé à dégelé, et inversement (Ghoreishian et al., 2016). Ce modèle a été implémenté comme un modèle utilisateur dans le code aux éléments finis PLAXIS 2D, et appliqué à des problèmes aux limites concrets de nature thermo-hydro-mécanique.
Un des problèmes lié à l'utilisation de modèles pour les sols gelés et dégelés est que de nombreux paramètres sont nécessaires, dont plusieurs peu communs aux ingénieurs géotechniciens. Ainsi, l'un des objectifs de cette étude est de fournir davantage d'information quant à la signification physique et à la détermination des paramètres du modèle.
Comme exemple d'application du modèle, deux cas hypothétiques sont présentés. Le premier est une canalisation refroidie, provoquant une chute de la température dans le sol entraînant un soulèvement par le gel. Le second est un radier soumis à une période de dégel.

1 INTRODUCTION

The replication of the behaviour of frozen soils has been studied for decades. 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 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 activities to undertake. 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 to tackle the aforementioned 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.

2 THE CONSTIUTIVE MODEL

In this section we provide a short insight in the formulation and capabilities of the model. The constitutive model is fully described in Ghoreishian et al., 2016.

2.1 General description

The constitutive model for frozen and unfrozen soil is a critical-state elasto-plastic time-independent 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:

$$\sigma^* = \sigma - S_{uw}\, p_w \qquad [1]$$

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. 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 effective stress based formulation 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 is able to 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. 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} 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:

$$K = (1 - S_i)\frac{(1+e)p_{y0}^*}{\kappa_0} + \frac{S_i E_f}{3(1-2\nu_f)} \quad [3]$$

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

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, ν is the specific volume and p_{at} is the atmospheric pressure.

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

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

2.3 Yield surface definition

In its unfrozen state, the model becomes a conventional critical state model. In other words, when the value of 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) (Alonso et al., 1990) the so-called loading collapse (LC) yield surface due to the variation of solid phase stress is expressed as:

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

where

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

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

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 is able to take the influence of the unfrozen water saturation S_{uw} into account. The exponent m dictates how much this behaviour wants to be 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:

$$F_2 = s_c - s_{c,seg} \qquad [10]$$

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 is increased.

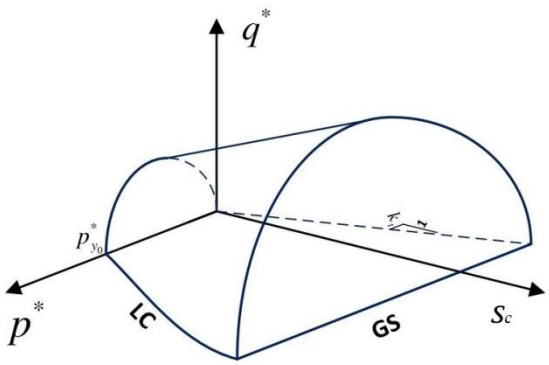

Figure 1. Illustrative three-dimensional yield surface.

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, $E_{f,ref}$, $E_{f,inc}$, ν_f , p_{y0}*, 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).

3 MODEL PARAMETERS AND THEIR DETERMINATION

The Barcelona Basic Model for unsaturated soil is rather employed by researchers than by practitioners. It is the base for the constitutive model for frozen and unfrozen soil. Although it is probably the best known elasto-plastic model for unsaturated soil, the lack of simple and objective methods for selecting parameter values from laboratory tests does not make it attractive for geotechnical engineers. This has been one of the major obstacles to the dissemination of this constitutive model beyond the research context (Wheeler et al., 2002; Gallipoli et al., 2010; D'Onza et al., 2012). Hence, we try to provide a short guideline describing proposed soil tests, possible empirical correlations and iterative calibration methods in order to obtain the necessary input parameters to use the new constitutive model.

Table 1. Parameters of the constitutive model

Parameter	Description	Unit
G_0	Unfrozen soil shear modulus	N/m^2
κ_0	Unfrozen soil elastic compressibility coefficient	-
$E_{f,ref}$	Frozen soils Young's modulus at a reference temperature T_{ref}	N/m^2
$E_{f,inc}$	Rate of change in Young's modulus with temperature	$N/m^2/K$
ν_f	Frozen soil Poisson's ratio	-
m	Yield parameter	-
γ	Plastic potential parameter	-
$(p_{y0}^*)_{in}$	Initial pre-consolidation stress for unfrozen condition	N/m^2
p_c*	Reference stress	N/m^2
λ_0	Elasto-plastic compressibility coefficient for unfrozen state	-
M	Slope of the critical state line	-
$(s_{c,seg})_{in}$	Initial segregation threshold	N/m^2
κ_s	Elastic compressibility coefficient for cryogenic suction variation	-
λ_s	Elasto-plastic compressibility coefficient for cryogenic suction variation	-
k_t	Rate of change in apparent cohesion with cryogenic suction	-
r	Coefficient related to the maximum soil stiffness	-
β	Rate of change in soil stiffness with cryogenic suction	$(N/m^2)^{-1}$

3.1 Elastic parameters

Elastic parameters are generally of minor importance for unfrozen soil, because elastic strains are significantly smaller than plastic strains. However, when we consider frozen soil, it is more likely that elastic strains occur, and the determination of elastic parameters gains importance. The elastic response for partially frozen soil highly depends on temperature and on the availability of unfrozen water: they carry weight on the elastic response. These key parameters are namely $E_{f,ref}$, $E_{f,inc}$ and ν_f (see Eq. [3], [4] and [5]). The pressure-dependent part of the elastic response influenced by κ_0 and G_0 play therefore a minor role. Nevertheless, they characterize the elastic behaviour in an unfrozen state and contribute to the elastic response when phase change occurs.

The elastic compressibility coefficient for cryogenic suction variation, κ_s, is assumed to be a constant parameter and describes the thawing and freezing reversals.

3.2 Strength parameters

The five strength parameters M, k_t, $(s_{c,seg})_{in}$, m and γ include and describe some important soil behaviour. M describes the effect of shear stresses, k_t the increase in the apparent cohesion, i.e. the tensile strength, with the increase in cryogenic suction, $(s_{c,seg})_{in}$ the effect of grain

segregation, m is the yield surface parameter and γ is the dilative behaviour due to ice accumulation.

It is assumed that the slope of the critical state line (CSL) for saturated conditions M is maintained for non-zero cryogenic suction conditions. Furthermore, the CSL also represents the increased strength, i.e. the apparent cohesion, induced by cryogenic suction. The increase in cohesion, and therefore in tensile strength, is assumed to follow a linear relationship with cryogenic suction, represented by the constant slope k_t. This is a simplification: real frozen soil behaviour shows that the increase in tensile strength with cryogenic suction is not linear (Akagawa & Nishisato, 2009; Wu et al., 2010; Azmatch et al., 2011; Zhou et al., 2015).

$(s_{c,seg})_{in}$ is the threshold value for grain segregation. Reaching this value, elastic and plastic strains occur, due to an increase in cryogenic suction. The formation of new ice lenses related to this irrecoverable strain and the separation of the soil skeleton.

The meaning of the yield parameter m is explained in section 2.3. The plastic potential parameter γ is added to have more control on the volumetric behaviour.

3.3 Parameters controlling virgin loading

The parameters controlling the virgin loading under isotropic stress state or due to a change in temperature are the most difficult to determine.

3.3.1 Under isotropic stress state

The five parameters β, λ_0, r, p_c*, together with the initial value of the pre-consolidation stress p_{y0}*, are considered to be the difficult parameters to determine in the general Barcelona Basic Model (BBM) and therefore in its unfrozen/frozen formulation as well. They simultaneously influence many aspects of the soil behaviour under isotropic stress states (Alonso et al., 1990; Wheeler et al., 2002; Gallipoli et al., 2010; D'Onza et al., 2012). Gallipoli et al. (2010) proposed a straightforward sequential calibration procedure where degrees of freedom in the model are progressively eliminated in a specific order, so that the corresponding parameter values are selected one at a time without having to make assumptions about the values of the remaining parameters. This approach can be used to determine the aforementioned parameters for this model.

The approach of Gallipoli requires isotropic testing of soil samples at different constant positive and negative temperatures. Knowing that this type of testing requires sophisticated equipment and is time-consuming, the idea is to use oedometer test results rather than isotropic test results. The temperature-controlled oedometer test requires less sophisticated equipment. Furthermore, a shorter testing period makes it possible to save time and money. No well-established, simple, and objective methods are available on using oedometer test results for constitutive modelling purposes. The research of Zhang et al. (2016) is on the derivation of an explicit formulation of the at-rest coefficient for unsaturated soils. They developed an optimisation approach for simple and objective identification of material parameters in elasto-plastic models for unsaturated soils, like the BBM, using the results from suction-controlled oedometer tests. The same approach, reformulated for the constitutive model for frozen and unfrozen soil, is explained in Aukenthaler (2016).

3.3.2 Under a freezing period

The parameter controlling the elasto-plastic compressibility for cryogenic suction variation, λ_s, is assumed to be a constant value. In a (ν : ln (s_c + p_{at})) – plane, λ_s describes the linear dependence between ν and ln (s_c + p_{at}) in the elasto-plastic range.

3.4 Proposed soil tests

The following laboratory soil tests are proposed to obtain all needed material parameters:

Table 2. Suggested soil tests

1	*Oedometer tests (or isotropic compression tests) in unfrozen and frozen state*
	The (ν : ln σ_1*) - plane provides data to find the initial pre-consolidation stress for unfrozen condition $(p_{y0}*)_{in}$, furthermore the parameters β, κ_0, r and p_c* can be determined by the calibration method mentioned in section 3.3.1. The elasto-plastic compressibility coefficient for unfrozen state under isotropic loading λ_0 can be obtained by taking the compression index C_c of the oedoemeter test in an unfrozen state into account (λ_0 = Cc / ln(10)).
2	*Simple shear test in unfrozen state*
	To obtain the shear modulus G_0 of the soil in the unfrozen state, and also for the slope of the critical state line (CSL), M.
3	*Unconfined axial compression test at an arbitrary reference temperature at a frozen state*
	To determine the Young's modulus of the frozen soil $E_{f,ref}$ and Poisson's ratio of the soil in frozen state, ν_f.
4	*Unconfined axial compression test at a different temperature at a frozen state*
	To determine the rate of change of Young's modulus with temperature of the frozen soil $E_{f,inc}$ and the increase in apparent cohesion, k_t .
5	*Frost heave test (freezing - thawing cycle)*
	To determine the initial segregation threshold value, by finding the temperature at which the frost heave phenomenon starts. Further plotting of the freezing-thawing cycle in the ν : ln(s_c + p_{at}) – plane is required to determine the values of κ_s and λ_s.

3.5 Possible correlations and default values

Correlations and default values should always be seen as first estimations. Laboratory test data, if available, is more reliable and should be preferred.

3.5.1 Young's modulus and change in Young's modulus with temperature

Tsytovich (1975) conducted cyclic compression tests on 200 mm cubes of three different frozen soils having a certain grain size distribution and moisture content. Based on the results of these cyclic compression tests under a confining pressure of 200 kPa and down to a specific temperature, Johnston (1981) published the equations describing the variation of Young's modulus E with temperature. In Table 3 the values are provided in terms of the input parameters of the new constitutive model.

Table 3. Default values of elastic parameters

	Frozen Sand	Frozen Silt	Frozen Clay
$E_{f,ref}$ [MPa]	500	400	500
$E_{f,inc}$ [MPa/K]	2100	1400	230

3.5.2 Poisson's ratio in a frozen state

The Poisson's ratio for the three frozen soil types in Table 3 was found to decrease with decreasing temperatures until all the pore water is frozen and the soil becomes rigid. However, the model assumes a constant Poisson's ratio for soil in frozen state. As a comparison, the Poisson's ratio of ice is about ν_{ice} = 0.31. We propose to use a value for ν_f close to ν_{ice}.

3.5.3 Slope of the critical state line

In Muir Wood (1991) it is suggested that soils are failing in a purely frictional manner at the critical state. After failure, the deformations are so large that the soil is thoroughly churned up. All bonding forces between particles have broken down. No cohesive strength is available any more. Therefore, for triaxial compression (+) and triaxial extension (-), the slope of the CSL can be estimated as:

$$M = \frac{6 \sin \varphi'}{3 \pm \sin \varphi'} \qquad [11]$$

φ' is called the residual or critical angle of friction. Ortiz et al. (1986) provide default values for φ'.

3.5.4 Threshold value for grain segregation

The threshold value is closely linked to the initiation of ice lens growth. Once the temperature drops to the point where the cryogenic suction exceeds this threshold value, plastic strains accumulate and the soil expands. In Rempel et al. (2004), Rempel (2007) and Wettlaufer and Worster (2006), the formation of ice lenses and frost heave is described. Rempel (2007) provides a table for lens temperatures calculated for three different types of porous media at the formation of the first lens, subsequent new lenses and lenses at their maximum extent.

To provide values in terms of cryogenic suction, the given temperatures in Rempel (2007) are transformed by means of the approximation $s_c \approx (|T_{f,bulk} - T| \times \text{MPa/K})$. This approximation provides reasonable values. The proposed default values when no frost heave test has been performed are 0.55 MPa for sand, 1.25 MPa for silt and 3.50 MPa for clay, respectively.

4 APPLICATIONS

In this section we present two simple applications, which represent two main features of the constitutive model, namely the ability to model frost heave and thaw settlements. A chilled pipeline buried in unfrozen ground simulates the frost heave phenomenon. A footing placed on frozen ground subjected to a warming period shows the consequences of ice melting and the associated thaw settlements.

In both simulations the soil is fully saturated, which is essential to use this new constitutive model. The parameter sets given in Table 4 and Table 5 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 is based on the practical approach described in Aukenthaler et al. (2016). The grain size distribution of the used clay and sand are based on the U.S.D.A. default values.

Table 4. Constitutive model parameters for clay and sand

Parameter	Clay	Sand	Unit
G_0	2.22×10^6	6.00×10^6	N/m^2
κ_0	0.08	0.15	-
$E_{f,ref}$	6.00×10^6	20.0×10^6	N/m^2
$E_{f,inc}$	9.50×10^6	100×10^6	$N/m^2/K$
ν_f	0.35	0.30	-
m	1.00	1.00	-
γ	1.00	1.00	-
$(p_{y0}^*)_{in}$	300×10^3	800×10^3	N/m^2
p_c^*	45.0×10^3	100×10^3	N/m^2
λ_0	0.40	0.50	-
M	0.77	1.20	-
$(s_{c,seg})_{in}$	3.50×10^6	0.55×10^6	N/m^2
κ_s	0.005	0.001	-
λ_s	0.80	0.10	-
k_t	0.06	0.08	-
r	0.60	0.60	-
β	0.60×10^{-6}	1.00×10^{-6}	$(N/m^2)^{-1}$

Table 5. 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

4.1 Frost heave simulation – Chilled pipeline

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.
Nishimura et al. (2009) explain frost heave 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.

4.1.1 Geometry and boundary conditions

A pipeline (Ø 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 Nm²/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 cross-section 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 with an assumed surface transfer of 300 W/m² is considered. 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 regime and the boundary conditions are shown in Figure 2.

4.1.2 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. 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. Figure 4 shows the ice saturation and Figure 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 Figure 3). The deformed mesh shows a frost heave of 3.5 cm and is illustrated in Figure 5. The final ice saturation after this cooling period is presented in Figure 4.

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

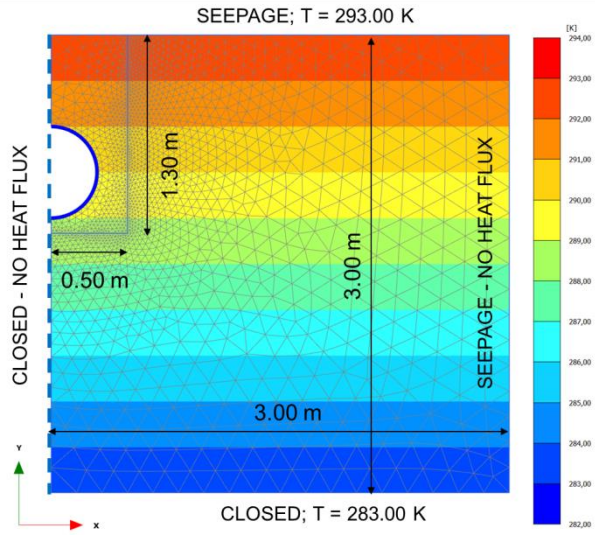

Figure 2. Geometry and boundary conditions.

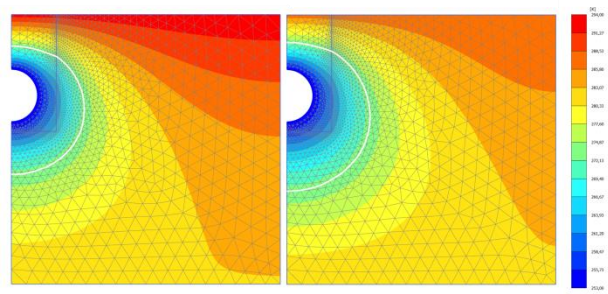

Figure 3. Temperature distribution after 40 days (left) and after the cooling period (right).

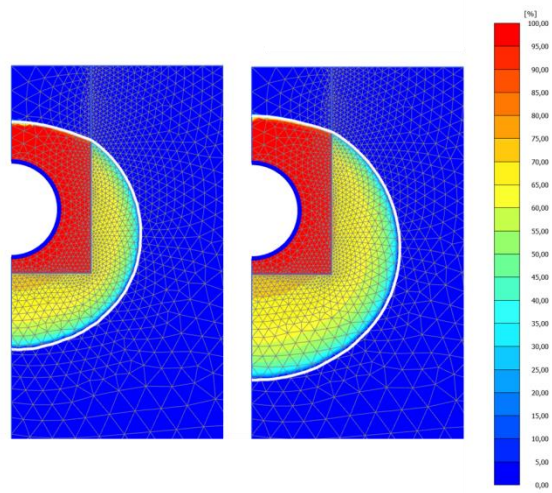

Figure 4. Ice saturation after 40 days (left) and after the cooling period (right).

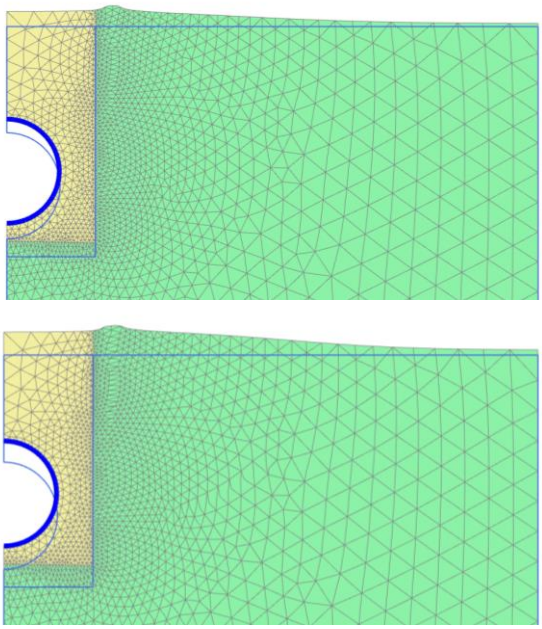

Figure 5. Upper deformed mesh: after 40 days (top) and after the cooling period (bottom).

4.2 Thaw settlement simulation – footing on frozen soil

On thawing, the soil skeleton must adapt to a new equilibrium void ratio. The occurring thaw settlements result from both the phase change from ice to water and flow of excess water out of the soil. They are important e.g. in the design of embankments on permafrost and the design of building foundations.

4.2.1 Geometry and boundary conditions

A raft foundation with a width of 2.00 m is placed on a frozen clay layer of 1.00 m thickness. Under the clay layer there is a sand layer. In this layer phase transition takes place, meaning that part of this layer is in a frozen state whereas the other part of the sand layer is in an unfrozen state. Again, the symmetry of the investigated problem is taken into account; plane strain is considered. The modelled domain is 6.00 m wide and 4.00 m deep. Relative fine meshing is used.

The initial ground temperature regime is set to a constant temperature of 270 K at the surface and 274 K at a depth of 4.00 m. The geometry, temperature distribution and boundary conditions are shown in Figure 6.

Before applying the foundation load, it is important to simulate the freezing period which led to the current temperature distribution. This also implies the correct stress state of the soil. The initial ice saturation is presented in Figure 7 (a).

4.2.2 Simulation and results

Once the soil is loaded with the foundation load of 500kN/m/m, the soil underneath the foundation starts yielding. This outward shift of the Loading-Collapse yield surface results in settlements up to 3.5 cm. Figure 8 shows the deformed mesh.

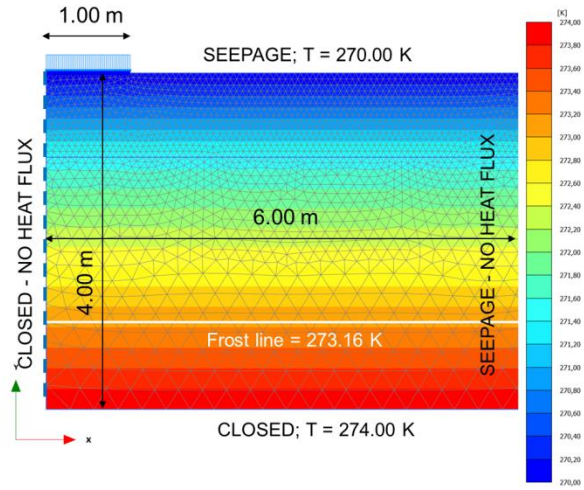

Figure 6. Geometry and boundary conditions.

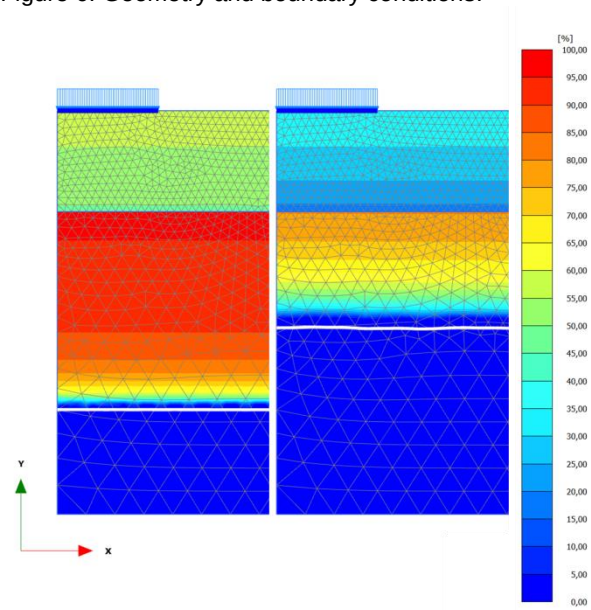

Figure 7. Ice saturation: Initial situation (left) and after the warming period (right).

Furthermore, a linear increase of 2.0 K in surface temperature over a long period of time is assumed. The influence of this increase in temperature can be clearly seen in Figure 7 (c) and Figure 8 (b). Due to the increase in temperature, the cryogenic suction and the ice saturation in both soil layers decrease. The decrease in cryogenic suction and the increase in unfrozen water and therefore also in pore water pressure lead to a new stress state which, at some point, might hit the LC-yield curve again. Once a stress point hits the LC-yield surface, significant thaw settlements are generated. Next to this

mechanism, consolidation and hence the dissipation of excess pore water pressure with time results in thaw settlements.

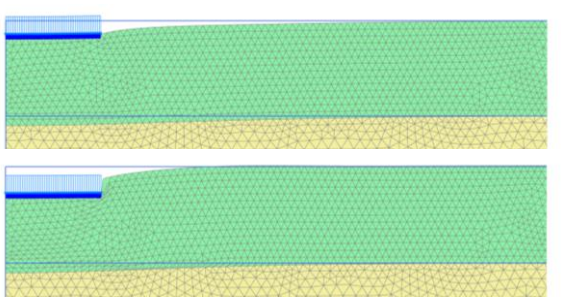

Figure 8. Upper deformed mesh: after applying the foundation loading (top) and after the warming period at constant foundation loading (bottom).

5 CONCLUSION

A new constitutive model from Ghoreishian et al. (2016) 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 soils.

The constitutive model requires several parameters of which quite a few are not very common to geotechnical engineers. Although some correlations and default values can be provided, the number of parameters needed remains high. The calibration methods to determine the BBM parameters mentioned in 3.3.1 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 dependence of stiffness and shear strength on temperature. Furthermore, two main features, namely frost heave and thaw settlements, can be simulated. These phenomena play a key role in designing in, on and with frozen / 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 applications presented in this article have 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.

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