Numerical Simulation of Saturated and Unsaturated Consolidation Behaviour of Marl Residual Soil

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SUMMARY

Consolidation tests were performed on reconstituted marl samples under saturated and unsaturated conditions in the geomechanical laboratory at the Faculty of Civil Engineering, Architecture and Geodesy (FCEAG). Reconstituted marl samples were used to simulate the behaviour of marl residual soil as a final product of weathering. The experimental results were implemented in the Plaxis 2D finite element software, and saturated and unsaturated consolidation tests were simulated using hydro-mechanical coupling analysis. Material parameters for the Mohr-Coulomb and Hardening Soil models were calculated from the obtained experimental results. The validity of above mentioned constitutive models for marl residual soil was evaluated using the consolidation tests results and considering the values of axial displacement and void ratio as deformation indicators for saturated and unsaturated conditions. The validity of the input parameters for unsaturated analysis (i.e. the obtained SWCC fitting parameters) and the applied boundary conditions were evaluated considering the achieved degree of saturation value for each calculation phase.

KEY WORDS: *marl; residual soil; saturated and unsaturated consolidation testing; Plaxis 2D; hydro-mechanical coupling.*

1.   INTRODUCTION

Problems associated with soft rock behaviour have been widely recognized in geotechnical engineering practice. Soft rocks can be defined as transitional material between hard rock and soil. They are generally considered a critical geomaterial due to their low strength and susceptibility to weathering which leads to disaggregation and crumbling [1]. Marls, which are a part of the Split area flysch formations, can be defined as soft rock. The weathering process in this area is usually a result of precipitation effects, i.e. physical weathering due to wetting and drying cycles. The alternation between fully saturated and dry conditions in the wetting–drying cycles implies the development of suction (the negative pore pressures) in the material pores. The effect of suction and unsaturated hydraulic conductivity on the weathering of soft rocks has been recognized by various authors [2–5] and the emphasis in the latest research

trends is on the determination of unsaturated soil properties. The result of the weathering process is a transformation from a soft rock into a fine-grained material [6]. Therefore, to use this material in engineering practice, it is necessary to determine the properties of marl as an intact rock (pre-weathering material and/or early stages of weathering), coarse-grained material (the intermediate stage of weathering), and fine-grained material (marl residual soil as the final product of the weathering process). It is also necessary to specify the adequate constitutive model for numerical modelling of material behaviour since finite element software programs are widely used by geotechnical engineers.

The scope of this paper is the implementation of experimental results obtained by laboratory tests on marl residual soil in Plaxis 2D finite element software. Saturated and unsaturated oedometer tests were simulated using consolidation and fully-coupled analysis in Plaxis 2D. Since the behaviour of unsaturated soil is non-linear, new constitutive models, such as Barcelona Basic Model (BBM) [7], were developed to describe unsaturated behaviour. Although BBM has been implemented in Plaxis finite element code and it has been proven successful in the simulation of unsaturated behaviour [8], it is still not widely used in engineering practice due to its complexity. Therefore, Mohr-Coulomb and Hardening Soil models were verified against saturated and unsaturated oedometer test results.

2. EXPERIMENTAL TESTS

Experimental tests were conducted on a marl sample with a calcium carbonate content of *55.02%*, collected at the location of Žnjan (Split, Croatia) and labelled as sample 01/19. Intact marl samples are crushed to a silt-sized material to simulate the residual soil behaviour. All tests on reconstituted samples are performed according to the suggested testing methods and standards [9–13].

Fig. 1 *Marl sample 01/19*

2.1 SATURATED CONSOLIDATION TESTS

One-dimensional consolidation test on saturated reconstituted marl sample is performed in compliance with HRN EN ISO 17892-5:2017 standard [12] and using standard (incremental loading) oedometer test apparatus. The sample is prepared by hand compacting the silt-sized material in the oedometer ring at *18%* water content (≈ plastic limit) to the desired density. Thereby, the dry density of *1.53 g/cm³* is achieved. The preparation method is according to the HRN EN ISO 17892-5:2017 sample preparation method for recompacted specimens. The oedometer ring has a diameter of *69.50 mm* and a height of *19.50 mm*.

After preparation, the sample is placed in the consolidation cell, the cell is filled with water, and initial vertical stress of *15 kPa* is applied. The test procedure consists of six loading

increments (*15 kPa*, *25 kPa*, *50 kPa*, *100 kPa*, *200 kPa*, and *400 kPa*), followed by the unloading phase. Sample compressibility characteristics are determined based on the recorded axial displacement values. The test results for the loading phases are presented in Table 1. Figure 2 shows the sample compressibility curve (void ratio *e* vs vertical effective stress $log\sigma_v$').
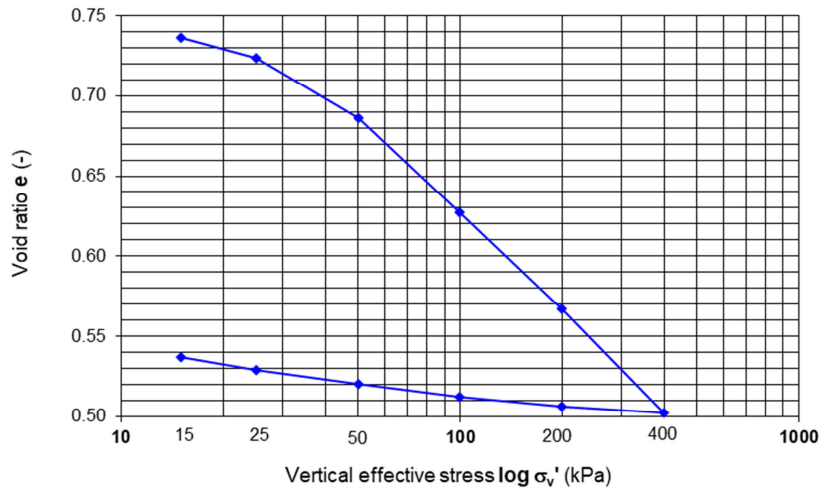
The value of the coefficient of permeability is determined based on the known value of coefficient of the consolidation, according to Eq. (1):

$$k_v = \frac{c_v \cdot \gamma_w}{E_{oed}}$$

(1)

Where: k_v – coefficient of permeability (hydraulic conductivity) (*m/s*), c_v – coefficient of consolidation (*m²/s*), γ_w – unit weight of water (*kN/m³*), E_{oed} – oedometer modulus (*kPa*).

Table 1 *Oedometer test results for loading phases*

No. Increment	σ_v kPa	e -	E_{oed} kPa	c_v m²/s	k_v m/s
0	0	0.751	-	-	-
1	15	0.736	1740	7.76 x 10 $^{-8}$	4.38 x 10 $^{-10}$
2	25	0.724	1400	3.84 x 10 $^{-8}$	2.69 x 10 $^{-10}$
3	50	0.686	1150	7.42 x 10 $^{-8}$	6.35 x 10 $^{-10}$
4	100	0.627	1430	1.40 x 10 $^{-8}$	9.60 x 10 $^{-10}$
5	200	0.567	2710	1.30 x 10 $^{-8}$	4.70 x 10 $^{-10}$
6	400	0.502	4830	1.34 x 10 $^{-8}$	2.72 x 10 $^{-10}$

Fig. 2 *Compressibility curve for the reconstituted marl sample 01/19 for saturated oedometer test*

2.2 UNSATURATED CONSOLIDATION TEST

Unsaturated consolidation test is conducted using a CRS (*Constant Rate of Strain)* type system for unsaturated consolidation testing by GDS Instruments. The testing system consists of several main components: load frame, LVDT (*Linear Variable Differential Transducer*) as displacement transducer, CRS cell, HAEPD (*High Air-Entry Porous Disc*), pneumatic controller, and STDDPC (*Standard Digital Pressure Controller*). HAEPD used for this test has an air-entry value of *1500 kPa*, which is the limit point for the imposed suction values in the system. The test is automated and controlled by the GDSLab software. The testing procedure is based on

the axis translation technique [14]. In this technique, the environmental conditions in the unsaturated zone are simulated in the laboratory by subjecting the sample to certain values of pore air and pore water pressures. The achieved value of matric suction in the sample pores, after equilibration of the applied pressures, is equal to the differential between the pore air pressure (u_a) and the pore water pressure (u_w). The principles of the axis translation technique are explained in detail in the literature [14–17].

At FCEAG geotechnical laboratory, the axis translation technique was primarily used to obtain the Soil-Water Characteristic Curve (SWCC) of reconstituted marl samples. Therefore, the tests were performed by applying small constant stress (*15 kPa*) in the first test stage and then six suction increments (*25 kPa*, *50 kPa*, *100 kPa*, *200 kPa*, *400 kPa*, *600 kPa*). Additional consolidation tests considering the effect of vertical stress on SWCC by applying different load increments under different suction conditions are currently in progress. The axis translation technique in these tests is used to control the suction in the sample, which is achieved, e.g. for the first test stage with a suction increment of *25 kPa*, by imposing *55 kPa* of air pressure in the cell using a pneumatic controller and *30 kPa* of water ('back') pressure through HAEPD using STDDPC. STDDPC is also used to monitor water drainage from the sample. When equilibrium conditions are reached, i.e. the back volume of water drained from the sample reaches a constant value, a new suction increment is set by increasing the value of air pressure in the cell while keeping the water pressure constant.

The reconstituted marl sample is prepared from the crushed silt-sized and clay-sized material by compacting the material in the oedometer ring (*d=50 mm*, h_0*=22 mm*) at an initial water content of *16.04 %*. The dry density of the prepared sample is *1.57 g/cm³*. The sample is then placed in the CRS cell, on top of the HEAPD, and saturated for *24* hours. The saturation phase is followed by a consolidation phase with a vertical load of *15 kPa* for the next *24* hours. In the following phases, suction increments are imposed and the duration of a single phase depends on the water drainage rate, which can range from *3* days to *15* days for higher suction values.

The performed tests resulted in axial displacement of the sample and back volume of water drained from the sample during the testing. Water content and degree of saturation for different suction values are calculated from the recorded back volume for the individual test stage, taking into account the sample volume change. Since radial deformation is not possible, the total volume change is calculated from the axial displacement values. Obtained water content values are used to define SWCC for *15 kPa* vertical stress level (Figure 3). SWCC, which represents the relationship between water content and suction, can be modelled using a variety of empirical equations. The water content can be expressed as gravimetric water content *w*, volumetric water content *θ* or degree of saturation *S*, whereas suction can be expressed as the pressure in *kPa* or as pressure head in m. In this case, van Genuchten [18] equation (Eq. (2)) is selected as the SWCC model since Plaxis 2D input for the unsaturated groundwater flow is based on this equation.

The equation is expressed in terms of the degree of saturation versus pressure head to conform with Plaxis 2D input.

$$\Theta = \left[\frac{1}{1+(\alpha \cdot h)^n}\right]^m \tag{2}$$

$$\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r} \tag{3}$$

Where: Θ – dimensionless water content according to the Eq. (3), h – pressure head (m) or suction (kPa), $α, m, n$ – fitting parameters, $θ$ –water content at given suction value, $θ_r$ – residual water content, $θ_s$ – saturated water content.

The values of the fitting parameters for the van Genuchten model in Table 2, as well as for the van Genuchten – Mualem model (i.e. van Genuchten model in Plaxis 2D) in Table 3, are calculated using experimental results for sample 01/19 and regression analysis (the least squares method).

Table 2 *Fitting parameters for van Genuchten model for reconstituted marl sample 01/19*

S_s	S_r	$α$	m	n
1.00	0.06509	0.00794	0.64805	1.41026
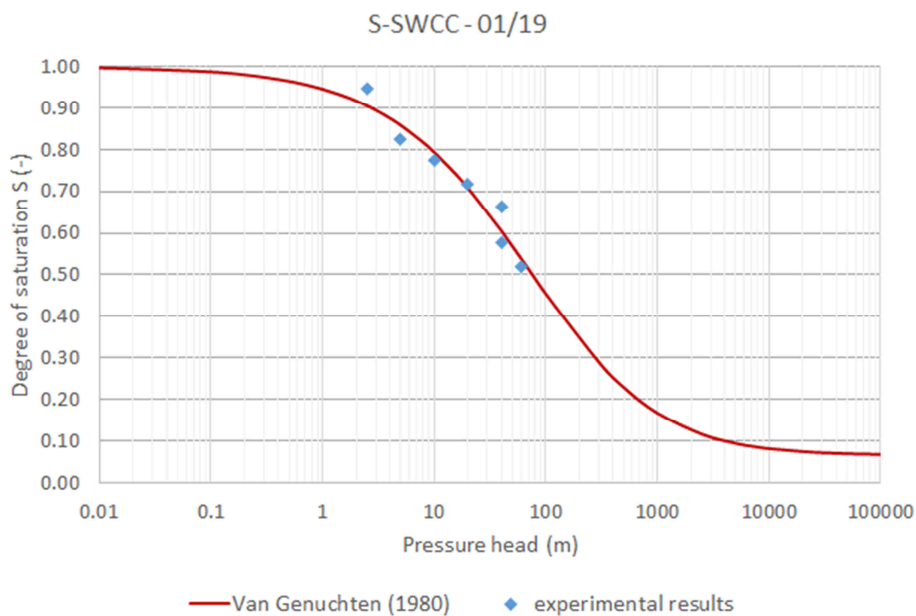
Fig. 3 *Van Genuchten model of soil-water characteristic curve for reconstituted sample 01/19 (degree of saturation (-) vs pressure head (m))*

3. NUMERICAL SIMULATION

Numerical simulation of saturated and unsaturated compressibility tests was performed in Plaxis 2D. Plaxis 2D is a two-dimensional finite element software developed for deformation, stability, and flow analysis in geotechnical engineering [19]. Proper analysis of the mechanical behaviour of saturated and partially saturated soils must take both deformation and groundwater flow into account [20]. This approach, in which displacement and pore pressure equations are solved simultaneously, is defined as coupled hydro-mechanical analysis. The coupled hydro-mechanical approach is based on the continuity equations for deformable media which are coupled with the mechanical equilibrium relations through the volumetric strain rate and the relationships between the stresses and pore pressures [21]. Both consolidation and fully-coupled analysis used in this research operate on the principles of hydro-mechanical coupling.

The effective stress calculation in Plaxis 2D is based on Terzaghi's principle for saturated and dry soils, whereas Bishop's equation (Eq. (4)) is used in case of unsaturated conditions. Bishop [22] defined the effective stress as a function of net normal stress and matric suction by extending Terzaghi's equation. The use of Bishop's effective stress equation as a single stress state variable to describe the unsaturated soil behaviour instead of using the net normal stress and matric suction as two independent stress state variables is considered the main disadvantage of unsaturated calculations in Plaxis 2D.

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \tag{4}$$

Where: $(\sigma - u_a)$ – net normal stress, $(u_a - u_w)$ – matric suction, σ – total stress, u_a – pore air pressure, u_w – pore water pressure, χ – Bishop's parameter.

Bishop's parameter χ or the effective stress parameter is a factor related to the degree of saturation and it is used to decrease the effect of suction in the effective stress equation. The value of Bishop's parameter is 0 for dry soil state and 1 for fully saturated soil state. As for the unsaturated state, the value of Bishop's parameter should be determined experimentally. However, Plaxis 2D simplifies this problem by assuming the value of this parameter equal to the effective saturation (Eq. (7)) [20]. The Bishop's effective stress calculation procedure for unsaturated soil state in Plaxis 2D takes the form of Eqs. (5–9), where Eq. (9) is the final simplified form of the Bishop's effective stress [19]. It should be noted that in Plaxis 2D all compressive stresses and forces are taken to be negative, whereas all tensile stresses and forces are taken to be positive, pore pressures included [19].

$$\sigma' = \sigma - (\chi \cdot p_{water} + (1 - \chi) \cdot p_a) \tag{5}$$

$$p_{water} = p_{steady} + p_{excess} \tag{6}$$

$$\chi = S_{eff} = \frac{S - S_r}{S_s - S_r} \tag{7}$$

$$p_{active} = S_{eff} \cdot p_{water} \tag{8}$$

$$\sigma' = \sigma - p_{active} \tag{9}$$

Where: p_{water} – pore water pressure, p_a – atmospheric pressure (in Plaxis 2D it is taken as the zero reference level), p_{steady} – steady-state pore pressure, p_{excess} – excess pore pressure, S_{eff} – effective degree of saturation, S – degree of saturation, S_r – residual degree of saturation, S_s – saturated degree of saturation, p_{active} – active pore pressure.

In the unsaturated zone, the degree of saturation S (Eq. (7)) is suction dependent, and the dependency is defined by inputting the soil-water characteristic curve (SWCC) in the Plaxis 2D material data set. The hydraulic models available in Plaxis 2D are van Genuchten, linearised van Genuchten, spline, and saturated. Van Genuchten model, used in this research, has the simplified van Genuchten – Mualem form, where the three-parameter van Genuchten equation (Eq. (10)) is converted to a two-parameter equation by assuming the value of the third fitting parameter according to Eq. (11):

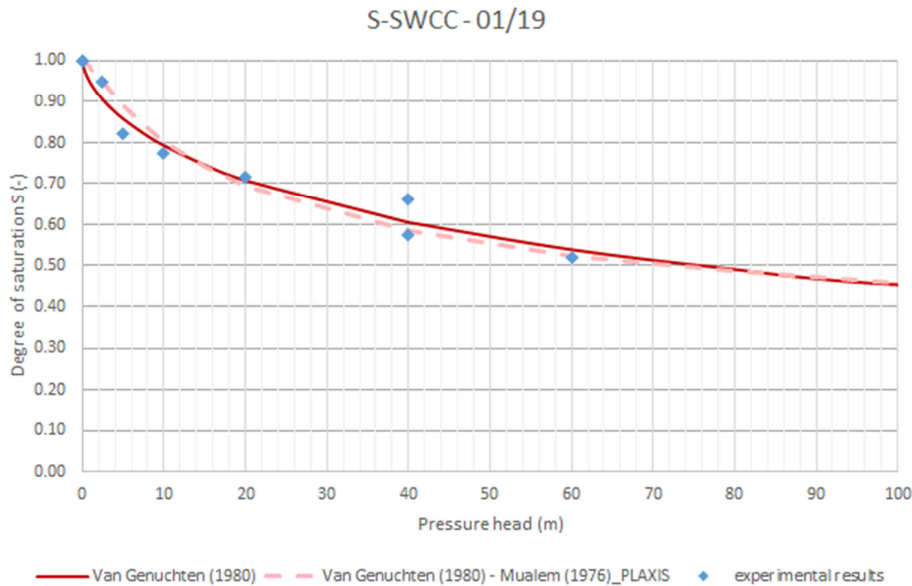$$S(\psi) = S_r + (S_s - S_r)[1 - (g_a|\psi|)^{g_n}]^{g_c} \tag{10}$$

$$g_c = \frac{1 - g_n}{g_n} \tag{11}$$

Where: ψ – suction defined as pressure head in m ($\psi = -\frac{p_w}{\gamma_w}$), g_a, g_n, g_c – fitting parameters.

Plaxis 2D input SWCC for reconstituted sample 01/19 is shown in Figure 4. Curve parameters are listed in Table 3.

Table 3 *Input parameters for van Genuchten model in Plaxis 2D for reconstituted marl sample 01/19*

S_s	S_r	g_a	g_n
1.00	0.06509	0.14634	1.31997

Fig. 4 *SWCC for reconstituted sample 01/19 prepared for Plaxis 2D input*

Relative permeability in the unsaturated zone is suction dependent as well. In Plaxis 2D, it is calculated using the effective saturation value, according to Eq. (12) [23] based on van Genuchten - Mualem expression for relative hydraulic conductivity [18, 24]:

$$k_{rel}(S) = max\left[(S_{eff})^{g_l}\left(1 - \left(1 - S_{eff}^{\left(\frac{g_n}{g_n-1}\right)}\right)^{\left(\frac{g_n-1}{g_n}\right)}\right)^2, 10^{-4}\right] \tag{12}$$

Where: g_l – fitting parameter.

Fitting parameter g_l accounts for the correlation between pores and for the flow path tortuosity, and it contributes to the flexibility of Eq. (12) [24]. The value of the fitting parameter g_l depends on the soil type and should be determined experimentally. Since the value of this parameter for marl residual soil has not been laboratory determined nor known in the literature, the value of *0.50* has been accepted. According to [18, 24], the value of *0.50* is determined as the value for which the square deviation, averaged over a great number of soils, takes a minimum value.

3.1 MATERIAL MODELS

Material models are used to define the mechanical behaviour of the soil. They are described by a set of mathematical equations that give the relationship between stress and strain [23]. Plaxis 2D offers a wide range of different material models. For this research, Mohr-Coulomb and Hardening Soil models were selected for numerical verification of the conducted consolidation tests.

The Mohr-Coulomb (MC) model is a simple linear elastic perfectly plastic model where the linear elastic part is based on Hook's law of isotropic elasticity, whereas the perfectly plastic part is based on the Mohr-Coulomb failure criterion [23]. Plaxis 2D requires four basic input parameters for the MC model: E – Young's modulus, ν – Poisson's ratio, c – cohesion, φ – friction angle, and ψ – dilatancy angle.

The values of Young's modulus are calculated from the known Poisson's ratio and oedometer modulus values, according to Eq. (13). Thereby, the value of Young's modulus is obtained for different stress increments in the oedometer test. For numerical simulation of saturated consolidation test with a unique value of E' for all calculation phases, an average value of Young's modulus is taken as a reference.

$$E' = E'_{oed} \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \tag{13}$$

The values of cohesion and friction angle are derived from direct shear test results, according to the HRN EN ISO 17892-11:2019 standard [13]. Since cohesive soils exhibit a small amount of dilatancy, the dilatancy angle value is taken to be $\psi \approx 0°$. The values of input parameters for the MC model are listed in Table 4.

Table 4 *Mohr-Coulomb model input parameters for reconstituted marl sample 01/19*

E' * (kN/m^2)						$\bar{E}'$ **	ν'	c'_{ref}	φ'	ψ'
15 kPa	25 kPa	50 kPa	100 kPa	200 kPa	400 kPa	(kN/m^2)	(-)	(kN/m^2)	(°)	(°)
1292.60	1040	854	1062	2013	3588	1641.58	0.30	11.30	27.70	0.00

* Young's modulus for different stress increments; ** average value of Young's modulus

The Hardening Soil (HS) model is an advanced soil model based on the hyperbolic relationship between the vertical strain and the deviatoric stress in primary triaxial loading, developed by Schanz et al. [25]. HS model includes two types of hardening – shear hardening (used to model irreversible strains due to primary deviatoric loading) and compression hardening (used to model irreversible plastic strains due to primary compression in oedometer and isotropic loading) [23]. Apart from the parameters necessary for MC model input (such as c', φ' and ψ'), the HS model in Plaxis 2D requires the input of additional basic and advanced parameters: E_{50}^{ref} – secant stiffness in standard drained triaxial test, E_{oed}^{ref} – tangent stiffness for primary oedometer loading, E_{ur}^{ref} – unloading/reloading stiffness, m – power for the stress-level dependency of stiffness, ν_{ur} – Poisson's ratio for unloading-reloading, p^{ref} – reference stress for stiffness, K_0^{NC} – K_0-value for normal consolidation, and R_f – failure ratio.

The values of E_{50}^{ref} and E_{ur}^{ref} are derived from triaxial test results, where E_{50}^{ref} is determined from a triaxial stress-strain curve for mobilization of *50%* of the maximum shear strength q_f, and E_{ur}^{ref} is determined as the reference Young's modulus for unloading-reloading path corresponding to the reference pressure p^{ref} [25]. Since the HS model does not involve a fixed relationship between the drained triaxial stiffness E_{50} and the oedometer stiffness E_{oed}, the value of E_{oed}^{ref} should be determined independently from the oedometer test results [25]. Therefore, E_{oed}^{ref} represents the value of tangent stiffness at vertical stress equal to p^{ref}.

Since the results of triaxial testing were not available at the time of this research, we were unable to calculate the exact values of E_{50}^{ref} and E_{ur}^{ref} for residual marl soil. Instead, the stiffness parameters of the HS model were calculated from the alternative parameters, i.e. compression index C_c, swelling index C_s and initial void ratio e_{init}, known from the oedometer test results. These relationships are given with Eqs. (14–16) [23]:

$$C_c = \frac{2.3 \ (1+e_{init})p^{ref}}{E_{oed}^{ref}}$$ **(14)**

$$C_s \approx \frac{2.3(1+e_{init})(1+\nu_{ur})(1-2\nu_{ur})p^{ref}}{(1-\nu_{ur})E_{ur}^{ref}K_0}$$ **(15)**

$$E_{50}^{ref} = 1.25E_{oed}^{ref}$$ **(16)**

When inputting the alternative parameters, the value of parameter *m* is automatically set to 1. The values of input parameters for the HS model are listed in Table 5.

Table 5 *Hardening Soil input parameters for reconstituted marl sample 01/19*

C_c	C_s	e_{init}	ν_{ur}	p^{ref}	K_0^{NC}	R_f
(-)	(-)	(-)	(-)	(kN/m²)	(-)	(-)
0.1996	0.0206	0.7510	0.20	100.00	0.5352	0.90
			(default value)	(default value)	($K_0^{NC} = 1 - sin\varphi$)	(default value)

3.2 SATURATED CONSOLIDATION TEST MODEL

A simple axisymmetric model with dimensions *34.75 x 19.50 mm* (radius x height of the sample) is selected to model the saturated oedometer test sample. The model dimensions are set following the dimensions of the testing equipment, i.e. the dimensions of the consolidation ring for the saturated oedometer test. 15-noded triangular mesh elements with very fine element distribution are used for finite element mesh generation. The sample model with generated mesh is shown in Figure 5. Displacement boundary conditions are set as normally fixed for side boundaries, fully fixed for the bottom boundary, and free for the sample top boundary. As for groundwater flow boundaries, they are set as closed for side boundaries and open to seepage for top and bottom boundary. Given that the saturated consolidation test is simulated, water conditions are set as the global water level on top of the sample. The groundwater flow through the sample is conditioned by the coefficient of permeability value given in the material data set. The inputted value of the coefficient of permeability is taken as an average of calculated k_v values shown in Table 1.
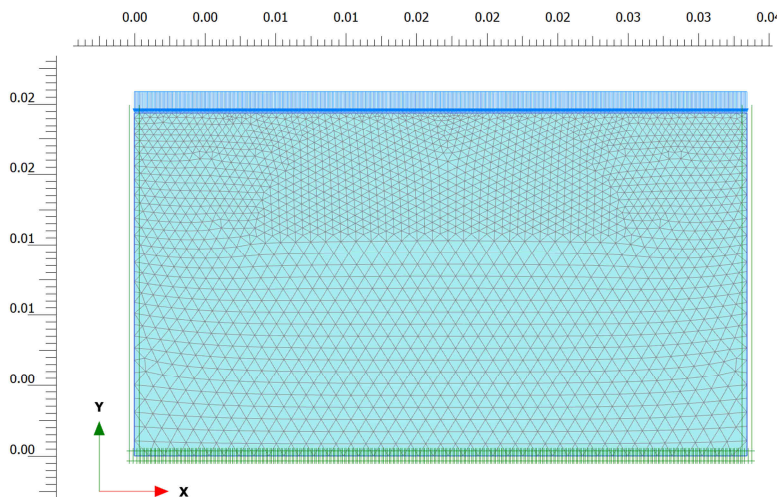
Fig. 5 *Saturated consolidation test model*

The loading is defined as a line load on the top boundary, and the value of the line load is changed through the calculation phases, according to the given test loading increment. The

calculation procedure consists of initial phase with K_0 procedure, 12 loading phases (each loading increment modelled as a plastic phase for load input and a consolidation phase for time-dependent consolidation behaviour with time interval equal to the duration of experiment test phase), and 5 unloading phases (modelled as consolidation analysis with corresponding time intervals). The calculation procedure is shown in Table 6.

Table 6 *Saturated consolidation test calculation procedure*

No. Phase	Phase name	Calculation type	Calculation time interval
1	Initial phase	K0 procedure	-
2	Loading 15 kPa	Plastic	-
3	Loading 15 kPa_consolidation	Consolidation	7 200 s
4	Loading 25 kPa	Plastic	-
5	Loading 25 kPa_consolidation	Consolidation	86 400 s
6	Loading 50 kPa	Plastic	-
7	Loading 50 kPa_consolidation	Consolidation	86 400 s
8	Loading 100 kPa	Plastic	-
9	Loading 100 kPa_consolidation	Consolidation	86 400 s
10	Loading 200 kPa	Plastic	-
11	Loading 200 kPa_consolidation	Consolidation	172 800 s
12	Loading 400 kPa	Plastic	-
13	Loading 400 kPa_consolidation	Consolidation	86 400 s
14	Unloading 200 kPa	Consolidation	4 200 s
15	Unloading 100 kPa	Consolidation	5 400 s
16	Unloading 50 kPa	Consolidation	4 800 s
17	Unloading 25 kPa	Consolidation	5 100 s
18	Unloading 15 kPa	Consolidation	72 000 s

3.3 UNSATURATED CONSOLIDATION TEST MODEL

The unsaturated oedometer test sample is modelled using an axisymmetric model with dimensions *25 x 22 mm* (radius x height of the sample). The model dimensions are set following the dimensions of the consolidation ring for the unsaturated oedometer test. The finite element mesh and displacement boundary conditions are set equivalent to the saturated oedometer test model (Figure 6). The groundwater flow boundary conditions are set as closed for side boundaries and open for seepage for a top boundary as well. However, since the permeability of *1500 kPa* HAEPD is lower than the permeability of the sample, in this model the bottom sample boundary is considered closed. The suction in the sample is achieved by imposing a desired value of suction as a pressure head on the sample model top boundary.

The loading is defined as line load on the top boundary with a constant value of *15 kPa* for all calculation phases. The calculation procedure for the unsaturated consolidation test consists of the initial phase with K_0 procedure, consolidation phase in saturated conditions with *15 kPa* load value, and 6 suction calculation phases modelled as fully-coupled analysis with corresponding time intervals. The calculation procedure is shown in Table 7.
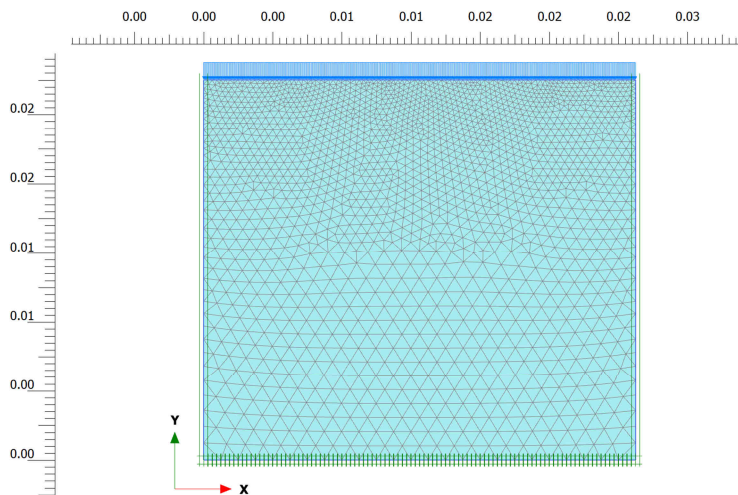
Fig. 6 *Unsaturated consolidation test model*

Table 7 *Unsaturated consolidation test calculation procedure*

| No. Phase	Phase name	Calculation type	Calculation time interval
1	Initial phase	K0 procedure	-
2	Consolidation 15 kPa	Consolidation	70 580 s
3	Suction 25 kPa	Fully coupled flow-deformation	345 000 s
4	Suction 50 kPa	Fully coupled flow-deformation	298 300 s
5	Suction 100 kPa	Fully coupled flow-deformation	219 300 s
6	Suction 200 kPa	Fully coupled flow-deformation	273 700 s
7	Suction 400 kPa	Fully coupled flow-deformation	1 395 000 s
8	Suction 600 kPa	Fully coupled flow-deformation	256 400 s

4. RESULTS AND DISCUSSION

The validity of the saturated consolidation test numerical models is verified by comparing the values of axial displacement in the numerical simulations with different soil models and the results of the oedometer test performed in the laboratory (Table 8). Numerically and experimentally obtained compressibility curves are compared as well (Figure 7).

Values of axial displacement obtained using the MC model in Plaxis 2D differ significantly from the experimental results, as shown in Table 8. The average error value is around *5.00 %*, with a maximum value of *11.50 %* for unloading phases. The same can be observed in Figure 7a, where the deformation is described graphically in terms of void ratio. When using a constant value of Young's modulus for all loading/unloading increments, the loading and unloading paths of the compressibility curve coincide, meaning that the MC model cannot correctly reproduce the relaxation of the material in the unloading stages. Compared with the experimentally obtained curve, the loading path of the curve for the model generally shows higher void ratio values for the same loading increment, whereby the increase in loading increases the deviation. If the corresponding value of Young's modulus is entered for each loading increment (calculated from the experimentally obtained E_{oed}), the error values decrease and the numerically obtained curve follows the experimental compressibility curve, as shown in Figure 7a (*Plaxis MC (E=stress dependent)* curve). However, it is neither practical nor advisable to change the value of Young's modulus for each loading phase. Therefore, the

inability of the MC model to reproduce stress-dependent stiffness is its main disadvantage. Consequently, it cannot be used to simulate the consolidation behaviour of marl residual soil.

On the other hand, the results of the HS model in Plaxis 2D are satisfactory. As shown in Figure 7b, the HS model curve and the experimental compressibility curve coincide very well, with minor deviation for lower stress increments (up to *50 kPa*) and the unloading path. The average error for axial displacement is app. *0.60%.*
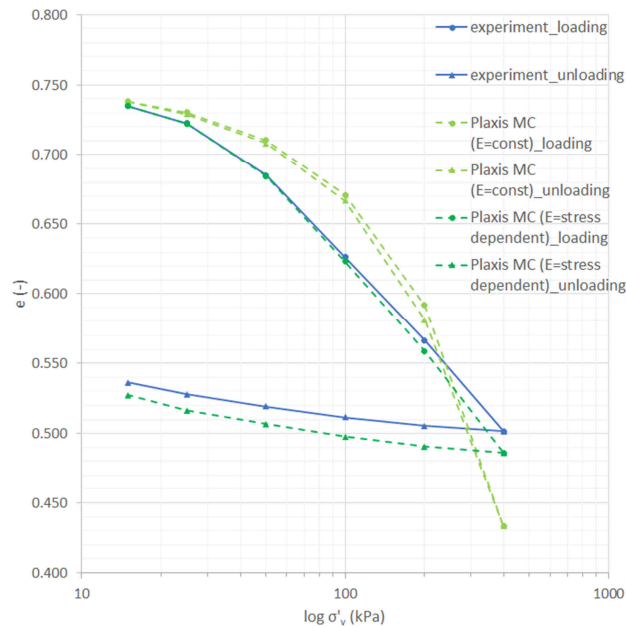
a)
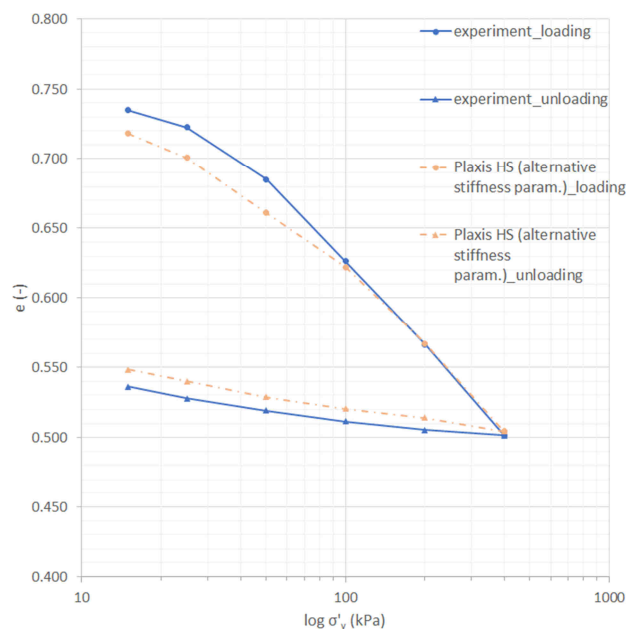
b)

Fig. 7 *Compressibility curves e – log σᵥ' for saturated consolidation test: a) experiment vs Mohr-Coulomb model (Plaxis 2D), b) experiment vs Hardening Soil model (Plaxis 2D)*

Table 8 *Values of axial displacement for experiment and numerical simulations in Plaxis 2D for saturated consolidation test*

	EXPERIMENT		MOHR-COULOMB MODEL (PLAXIS)		HARDENING SOIL MODEL (PLAXIS)	
σ_v (kPa)	u_y (mm)	Pu_y (mm)	u_y (mm)	Pu_y (mm)	u_y (mm)	Pu_y (mm)
15	-0.168	-0.168	-0.132	-0.132	-0.356	-0.356
25	-0.306	-0.138	-0.221	-0.089	-0.549	-0.193
50	-0.724	-0.418	-0.441	-0.220	-0.923	-0.374
100	-1.379	-0.655	-0.882	-0.441	-1.426	-0.503
200	-2.048	-0.669	-1.765	-0.883	-2.040	-0.614
400	-2.770	-0.722	-3.530	-1.765	-2.738	-0.698
200	-2.727	0.043	-1.883	1.647	-2.630	0.108
100	-2.658	0.069	-0.929	0.954	-2.557	0.073
50	-2.572	0.086	-0.467	0.462	-2.465	0.092
25	-2.476	0.096	-0.233	0.234	-2.338	0.127
15	-2.381	0.095	-0.133	0.100	-2.247	0.091

σ_v – vertical loading, u_y – total displacement, Pu_y – phase displacement

For unsaturated consolidation test numerical models, the main problem is to reach the set suction value in the calculation phase. When the experimental phase time interval is used, the value of suction in the numerical models is not uniformly distributed in the sample, which is especially noted in phases with higher suction values (Figure 8). Therefore, the value of the time interval for Plaxis 2D fully-coupled analysis should be determined by iterating the time interval until the target suction value is reached (Figure 9). Comparing the values of the axial displacement (Table 9) and the values of the degree of saturation for each suction increment (Table 10), the validity of the model is checked.

Table 9 *Values of axial displacement for experiment and numerical simulations in Plaxis 2D for unsaturated consolidation test*

	EXPERIMENT		MOHR-COULOMB MODEL (PLAXIS)		HARDENING SOIL MODEL (PLAXIS)	
s (kPa)	u_y (mm)	Pu_y (mm)	u_y (mm)	Pu_y (mm)	u_y (mm)	Pu_y (mm)
0.01 (saturated consolidation)	-0.491	-0.491	-0.188	-0.188	-0.429	-0.429
25	-0.586	-0.095	-0.489	-0.301	-0.907	-0.478
50	-0.636	-0.050	-0.749	-0.260	-1.210	-0.303
100	-0.684	-0.048	-1.187	-0.438	-1.599	-0.389
200	-0.705	-0.021	-1.889	-0.702	-2.049	-0.450
400	-0.728	-0.023	-3.000	-1.111	-2.543	-0.494
600	-0.739	-0.011	-3.925	-0.925	-2.847	-0.304

s – suction, u_y – total displacement, Pu_y – phase displacement

Table 10 *Values of degree of saturation for experiment and numerical simulations in Plaxis 2D for unsaturated consolidation test*

s (kPa)	0.01	25	50	100	200	400	600
S (%) – experiment	100.00	94.72	82.32	77.32	71.66	57.52	52.03
S (%) – numerical model	100.00	94.85	89.20	80.34	69.46	58.45	52.55

s – suction, S – degree of saturation
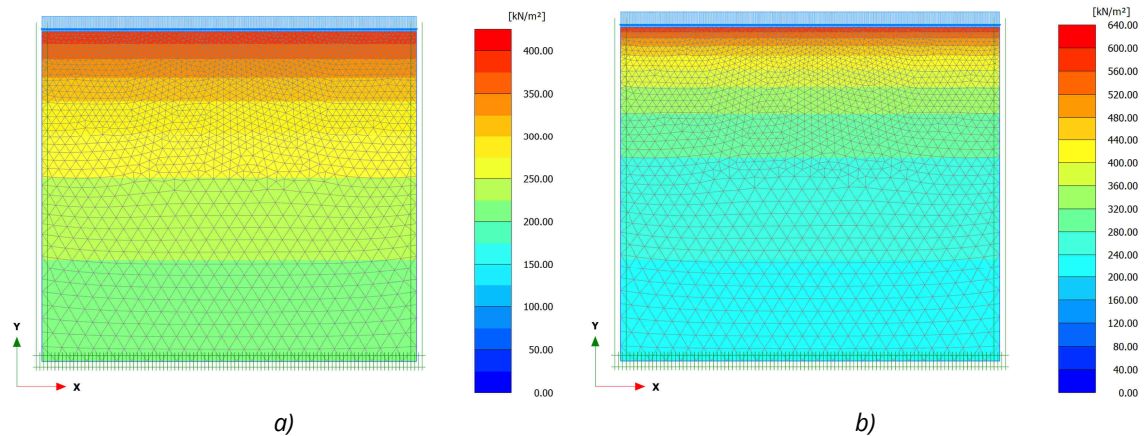
Fig. 8 *Distribution of suction pressure for experimental time interval: a) 400 kPa suction phase, b) 600 kPa suction phase*
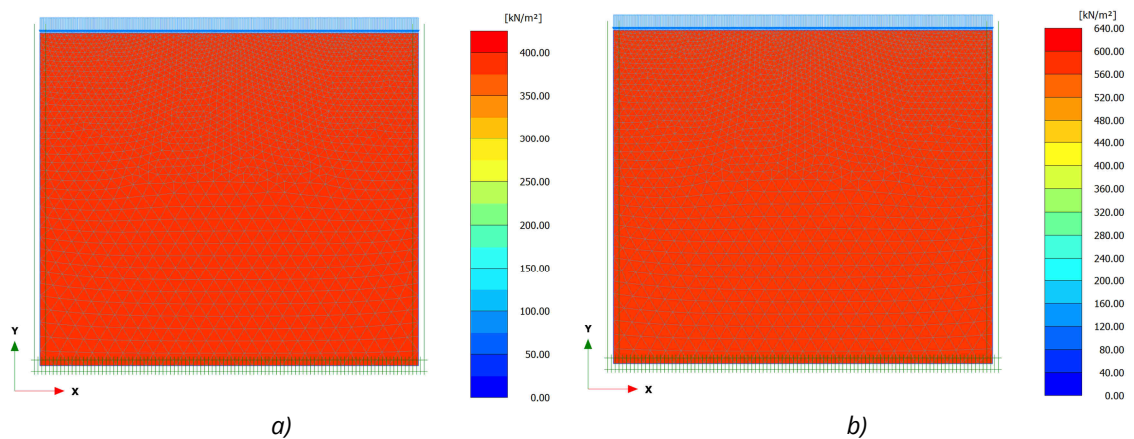
Fig. 9 *Distribution of suction pressure for iterated time interval: a) 400 kPa suction phase, b) 600 kPa suction phase*

As can be seen in Figure 8, for suction values greater than *25 kPa*, the equilibrium of suction pressure throughout the sample is not achieved in the experimental time interval. Uniform distribution of suction pressure is achieved by increasing the calculation time interval. For each suction phase, the value of the degree of saturation is compared with the experimental values. Since the value of the degree of saturation does not depend on the material model used, but on the groundwater flow conditions and the inputted SWCC (which is the same for both observed models), the values obtained for MC and HS model numerical simulations are equal.

These values do not significantly differ from the experimentally obtained results (Table 10), which confirms the validity of the set boundary conditions and the inputted SWCC parameters.

The validity of the MC and HS soil model is checked for unsaturated consolidation conditions as well. Since the unsaturated consolidation test is simulated for one loading level, the values of MC stiffness parameters are adopted for the loading increment of *15 kPa*. The values of axial displacement as a deformability indicator for numerical simulations deviate significantly from the experimental test results, as shown in Table 9. The average error for both MC and HS models is app. *5.60 %* and has an increasing tendency with the increase of suction level.

Hence, the conclusion is that none of the material models tested has satisfactory accuracy regarding the simulation of unsaturated consolidation test results.

5. CONCLUSIONS

One-dimensional consolidation tests (i.e. oedometer tests) were conducted under saturated and unsaturated conditions on reconstituted marl samples. Intact marl samples were crushed to a silt-sized material to simulate the residual soil behaviour. Apart from saturated and unsaturated oedometer tests, additional laboratory tests (e.g. direct shear test) were performed to obtain necessary material parameters.

The experimental results were then implemented in Plaxis 2D finite element software to simulate the consolidation behaviour of residual marl soil in saturated and unsaturated conditions. The simulations were performed using Mohr-Coulomb (MC) and Hardening Soil (HS) constitutive models in combination with consolidation and fully-coupled analysis as forms of hydro-mechanical coupling. The results of numerical simulations were compared with the experimental results to verify the validity of inputted material parameters as well as model boundary conditions.

It has been proved that the application of the MC material model for the consolidation behaviour of residual marl soil is unable to accurately replicate the experimental results under both saturated and unsaturated conditions. The main disadvantage of this material model is its inability to reproduce the stress-dependent stiffness and material hardening in the unloading phase. On the other hand, the HS material model gives satisfactory results for the saturated consolidation test due to its implemented stress-dependent stiffness, but it does not meet the expectations in unsaturated consolidation simulations. Since the HS model input was based on alternative stiffness input parameters, i.e. compression index C_c, swelling index C_s, and initial void ratio e_{init}, known from the oedometer test results, triaxial tests should be performed in the future to obtain the values of E_{50}^{ref}, E_{ur}^{ref}, and m for residual marl soil.

The hydraulic behaviour of the material in unsaturated conditions is defined by the inputted soil-water characteristic curve (SWCC) parameters, and it does not depend on the material model used. Therefore, the validity of the inputted SWCC parameters and the set boundary conditions for unsaturated consolidation test simulation was evaluated comparing the values of the degree of saturation achieved in numerical simulations for different suction levels and the experimental results. The simulations have shown that the values of the degree of saturation in numerical models coincide with the experimental results if the extended calculation time interval is applied. This result can be considered an example of successful implementation of experimental results in unsaturated numerical models and therefore used as a guideline in future simulations.

The results of the conducted research show that the problem of unsaturated soil behaviour cannot be simplified and that the combination of "simpler" constitutive models and Bishop's effective stress equation implemented in Plaxis 2D is not sufficient to describe the complexity of unsaturated soil behaviour.

6. ACKNOWLEDGEMENTS

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