SHAPING THE FUTURE OF TUNNELLING
Innovation, Sustainability and Safety

PROCEEDINGS OF THE SOUTHEASTERN EUROPE TUNNELLING CONFERENCE (SETC-2025)

Papers on Technical Subjects Related to Tunnelling and Underground Space Planning and Engineering

EDITED BY
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ITA SERBIA
Serbian Association for
Tunnels and Underground Structures

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1–3 October 2025, Belgrade, Serbia

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SHAPING THE FUTURE OF TUNNELLING

Innovation, Sustainability and Safety

Shaping the Future of Tunnelling – Innovation, Sustainability and Safety contains the contributions presented at the ITA Awards & SETC-2025, held in Belgrade, Serbia, from 1 to 3 October 2025.

The papers cover a wide range of topics in the fields of tunnelling and underground engineering, including:

1. Advanced construction techniques
2. Use of new materials and machinery
3. Geological investigation and prediction
4. Numerical modelling
5. Instrumentation and monitoring/testing and inspection
6. Digital and information technology in design and construction
7. Strategic planning
8. Operational safety
9. Impact of climate change on tunnel infrastructure

Shaping the Future of Tunnelling – Innovation, Sustainability and Safety aims to provide a useful resource for everyone engaged in tunnelling and underground engineering, from students and young researchers to experienced professionals and engineers.

PREFACE

The ITA Tunnelling Awards and the Southeastern Europe Tunnelling Conference (SETC-2025) were held from the 1st to the 3rd of October 2025 in Belgrade, Serbia.

The Serbian Association for Tunnels and Underground Structures (ITA Serbia) was honoured and proud to host this outstanding event of the international tunnelling community. Bringing together hundreds of distinguished experts, researchers, and industry leaders from across the globe, the event served as a dynamic platform for sharing knowledge, presenting innovations, and advancing scientific and technical excellence in the field of tunnelling and underground construction.

Serbia, with Belgrade as its dynamic capital, is experiencing a period of intensive infrastructure development, particularly in the domain of underground construction and sustainable urban mobility. Landmark projects such as the Belgrade Metro, tunnel connections, and urban underground infrastructure systems are transforming the city's transport network and enhancing its connectivity and sustainability. These projects demonstrate Serbia's growing expertise in modern tunnelling technologies, geotechnical engineering, and integrated urban planning, positioning Belgrade as a regional hub for innovation and progress in underground construction.

The conference proceedings encompass a diverse range of nine thematic areas, reflecting the multidisciplinary nature and technological depth of modern tunnelling. Topics include advanced construction techniques, the use of new materials and machinery, geological investigation and prediction, numerical modelling, instrumentation, monitoring, testing and inspection, the application of digital and information technologies in design and construction, strategic planning, operational safety, and the impact of climate change on tunnel infrastructure. Together, these themes highlight the conference's focus on innovation, sustainability, and resilience in underground construction.

It is our sincere expectation that these proceedings will contribute meaningfully to the professional and scientific community, providing valuable insights for engineers, researchers, and decision-makers engaged in the development of underground infrastructure. The knowledge and experiences shared during SETC-2025 aim to foster innovation, collaboration, and sustainable practices, encouraging the continued advancement of tunnelling and underground construction in the years ahead.

Belgrade, October 2025

Prof. Dr Dejan Divac

Chair of the ITA Awards & SETC-2025 Organising and Scientific Committee
President of the ITA Serbia

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The SETC-2025 Organizing Committee, Scientific Committee, and Editors wish to express their sincere gratitude to the conference sponsors and exhibitors for their generous support and valuable contribution to the success of this Event.

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Parameter determination of the hardening soil model with small strain stiffness: an optimization approach coupling finite element simulations and genetic algorithms

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Abstract: In the scope of Phase I of Line 1 of the Belgrade Metro project, a large laboratory and in-situ testing database of the soils and rock masses of Belgrade was used for determining the parameters of the hardening soil model with small strain stiffness (HSS). Due to the complexity of parameter determination of the HSS model based on conventional laboratory testing results, an optimization approach coupling finite element analyses and genetic algorithms was taken in the parameter determination process with the goal of defining a framework for full, reliable, and unified implementation of the HSS model in all necessary numerical analyses. For the selected geotechnical soil layer, each of the available laboratory tests (direct shear test and CD triaxial test) and in-situ tests (pressuremeter) were numerically simulated and the HSS parameters were calibrated so that each individual simulated test, consisting of multiple test specimens, could achieve its respective experimental recordings as best as possible by minimizing the value of the applied criterion function. The resulting parameters from the calibration process, alongside the analytically derived parameters for laboratory results, were compared and analyzed, and it is realized that while analytically derived parameters provide a reasonable first estimate, numerical calibration is essential for accurately capturing soil stiffness behavior and stress dependency and therefore creating reliable design solutions based on realistic soil properties.

Keywords: hardening soil model; optimization; genetic algorithms; direct shear; triaxial; pressuremeter

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1. Introduction

1.1. Context and focus of the research

Within the upcoming infrastructural development of the City of Belgrade, a large number of underground construction projects of varying purposes have been planned. Among these, in terms of scale, complexity, importance, and current relevance, the most prominent are the Tunnel Connection Project from Karađorđeva Street to the Danube Slope and Phase I of Line 1 of the Belgrade Metro project. In the near future, additional underground infrastructure projects are also planned, such as the Interceptor Project, the Topčider Tunnel, and others.

The planned tunnel projects traverse geologically complex environments along their alignments, characterized by varying lithological compositions, mechanical properties, the presence of groundwater, and other influencing factors. For the purpose of defining the geotechnical properties of the soils and rock masses in which the subject infrastructure will be constructed and considering the high degree of urbanization of the project area, extensive geological investigations have already been carried out, with additional investigations planned for the forthcoming period. Accordingly, there arises

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a need to systematize the large volume of results from experimental geotechnical tests and to enable their application in the development of reliable and robust technical solutions.

The specific requirements of designing and constructing underground facilities – which are in complex interaction with both the surrounding ground and the groundwater regime – necessitate the use of numerical models based on the finite element method (FEM). In the development of such models, a key role is played by the selected constitutive material model used to describe the behavior of the geological medium in which construction is carried out. Consequently, the reliability of determining the parameters of the chosen material model is of critical importance for adequately simulating construction conditions and thereby ensuring the validity of reached conclusions and proposed designs.

Ever since the formulation of the Hardening Soil (HS) material model (Schanz et al., 1999) and its modification to take into consideration the small-strain stiffness of soils (Benz, 2007), the Hardening Soil with Small-Strain stiffness (HSS) material model has been widely used for numerical modelling and design of geotechnical structures. The approach to parameter calibration of the HSS model based on laboratory and/or in-situ test results has been covered by only a few authors. Surarak et al. (2012) calibrated the stiffness and strength parameters of the HSS model of Bangkok clays based on oedometer tests and standard undrained and drained triaxial tests (without an unload-reload loop) while Govindasamy et al. (2019) used discrete FEM models of standard triaxial and pressuremeter tests to calibrate some of the HSS parameters of the Kenny Hill formation by applying regression analysis. Tazakka and Tirta (2024) used a direct field curve fitting method to develop a correlation between SPT tests and stiffness parameters of the HS model, while analytically deriving the strength parameters from triaxial tests. The implementation of genetic algorithms was used by Mendez et al. (2021) for calibrating a hypoplastic sand model based on oedometer and drained triaxial tests, and related work was also done by Mahaček et al. (2022) that implemented Differential Evolution and Particle Swarm Optimization methods.

The planned underground infrastructure development in Belgrade emphasizes the need for dependable numerical analyses supported by accurate constitutive modeling. Although the HSS model is widely applied in geotechnical engineering, existing studies on its parameter calibration during the design phase of a project are relatively limited and often focused on singular test specimens or simplified procedures. Advanced optimization techniques show encouraging potential opportunities, however, comprehensive approaches that integrate multiple laboratory and in-situ tests remain insufficiently explored, indicating opportunities for further development in this area.

1.2. Overview of the Hardening Soil with Small-Strain Stiffness material model

In contrast to ideally elastoplastic models, the Hardening Soil (HS) model accounts for the increase in shear strength (hardening) under dominant deviatoric stresses, as well as the increase in compressive strength under uniaxial and isotropic stress states during plastic behavior of the material. The main characteristics of the HS constitutive model are:

- The failure surface corresponds to the Mohr–Coulomb failure criterion (parameters c, φ, ψ);
- Plastic deformations develop due to primary compression (parameter E_{oed}^{ref}) and primary deviatoric loading (parameter E_{50}^{ref});
- An increase in stiffness occurs during elastic unloading and reloading (parameter E_{ur}^{ref}).

Beyond the advantages provided by the HS constitutive model, the Hardening Soil with Small-Strain Stiffness (HSS) model introduces a nonlinear variation of stiffness at very small strains. Consequently, this model provides an improved description of characteristic shear deformations in the vicinity of geotechnical structures, as well as a more accurate representation of deformations in laboratory tests.

In addition to the parameters required for defining the HS model, the HSS model requires two additional parameters that describe small-strain behavior: the initial small-strain shear modulus G_0^{ref} and the shear strain threshold $\gamma_{0.7}$. Below, all parameters necessary for defining the HSS model are presented, under the assumption that the tensile strength of the soil is $\sigma_t = 0$, that no increase in soil cohesion with depth is considered ($c_{inc} = 0$), and that parameters are defined for the reference stress $p_{ref} = 100$ kPa.

Table 1. Relevant parameters for defining the HSS model.

Parameter	Description	Parameter	Description
c (kN/m^2)	Soil cohesion	m (-)	Exponent governing stress dependency of stiffness
s_u (kN/m^2)	Undrained shear strength (relevant only for undrained tests)	ν_{ur} (-)	Poisson's ratio corresponding to the unloading–reloading branch of the stress–strain curve (default $\nu_{ur} = 0.2$)
φ (°)	Angle of internal friction	K_0^{nc} (-)	Coefficient of earth pressure at rest (for normally consolidated soils, default $K_0^{nc} = 1 - \sin\varphi$)
ψ (°)	Dilatancy angle	R_f (-)	Failure ratio q_f/q_a (default $R_f = 0.9$)
E_{oed}^{ref} (kN/m^2)	Tangent stiffness for primary oedometer loading	G_0^{ref} (kN/m^2)	Initial shear modulus at very small strains ($\varepsilon<10^{-6}$)
E_{50}^{ref} (kN/m^2)	Secant stiffness corresponding to a deviatoric stress equal to 50% of the failure deviatoric stress in a drained triaxial compression test	$\gamma_{0.7}$ (-)	Shear strain threshold at which shear stiffness is redefined as $G_s = 0.722\ G_0^{ref}$
E_{ur}^{ref} (kN/m^2)	Unloading/reloading stiffness		

2. **Methodology**

2.1. General approach to parameter determination

For practical design purposes, the tunnel alignment must be subdivided into a number of geotechnical macro-zones, each characterized by the general behavior of the soil, the physical and mechanical properties of the soil and rock layers, hydrogeological conditions, the thickness of the overburden relative to the structure, and the potential occurrence of faults or other geotechnical hazards. Individual underground structures along the tunnel alignment should then be classified within the corresponding macro-zones according to their location. Subsequently, all relevant stress-strain analyses must be performed for each of the defined macro-zones in order to establish safe and robust engineering solutions for the tunnel and associated underground structures.

Within each geotechnical macro-zone, it is necessary to determine a consistent set of design parameters for every geotechnical layer present in that zone. The parameters thus established are then applied in all required stress-strain analyses of the structures (or structural segments) located within the given macro-zone. The procedure for defining these parameters is illustrated schematically in the following figure and is described in detail in the subsequent text.

For the considered macro-zone, the number of characteristic geotechnical layers is first established. For each geotechnical layer within the macro-zone, the available soil samples belonging to that layer are identified, i.e., the laboratory and field tests that best describe the behavior of each geotechnical layer are determined.

Since the tests are performed on individual test specimens (TS), these specimens are grouped according to the type of test conducted. For each test type within a given geotechnical layer, a test specific calibration procedure is performed. This results in determining a single – unified set of parameters of the HSS model for which the numerical simulation of the entire test (encompassing simultaneously all the test specimens that make up the test) provides the best overall correlation with the corresponding laboratory results.

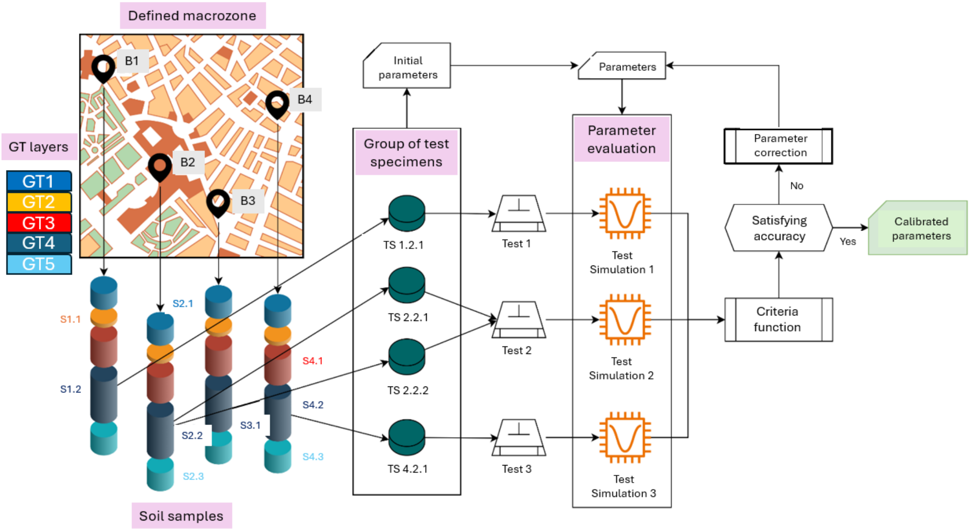

Fig. 1. Workflow for parameter calibration of a specific geotechnical layer in a defined macro-zone.

The correlation between the numerical simulation and the laboratory records is evaluated on the basis of the criterion function defined in Section 2.5, by optimizing it to its minimum value. The parameter sets obtained from the test specific calibration of each test type are then compared with each other and with the analytically derived parameters from the laboratory records.

2.2. Assessed laboratory and in-situ tests

For the purposes of the analysis presented in this study, the macro-zone in the vicinity of the Bele Vode metro station is considered, located on the margin of the Makiš Field area between chainages km 4+070 and km 4+250 of Phase I of Line 1 of the Belgrade Metro project. This macro-zone is of particular significance as it represents the launching zone for the TBM excavation of the tunnel section extending from the Bele Vode station to the Sajam station, with a total excavation length of 5760 m.

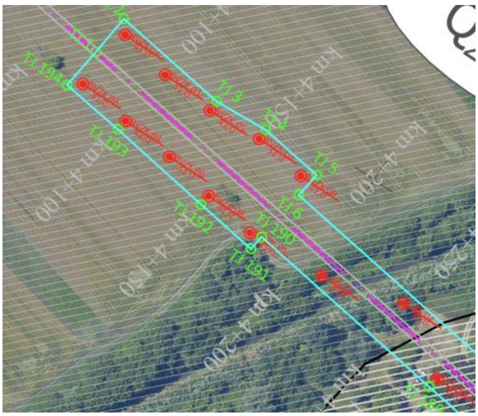

Fig. 2. Analyzed macrozone in the vicinity of metro station Bele Vode with indicated position of boreholes (BVZK and BK).

Within the considered macro-zone, the geotechnical layer of overlapping Neogene limestone–marl sediments, denoted as $M^1{}_3KL$, is analyzed. This layer consists of silty to clayey marlstones/limestones with a heterogeneous carbonate content and exhibits layered textures with irregular thicknesses. This geotechnical layer is of particular importance as its depth extends through the zone of the tunnel, exerting the greatest influence on the stress-strain behavior of the tunnel lining, as well as on the stress-strain response of the diaphragm walls of the Bele Vode metro station during TBM breakthrough.

The following laboratory and in-situ tests are available, which can be reliably attributed to the same analyzed geotechnical layer within the considered macro-zone:

- Drained direct shear test
  - Borehole BVZK-05, sample depth 28.10–28.40 m; Specimen TS 1 (σ_c = 100 kPa)
  - Borehole BVZK-05, sample depth 28.10–28.40 m; Specimen TS 2 (σ_c = 200 kPa)
  - Borehole BVZK-05, sample depth 28.10–28.40 m; Specimen TS 3 (σ_c = 400 kPa)

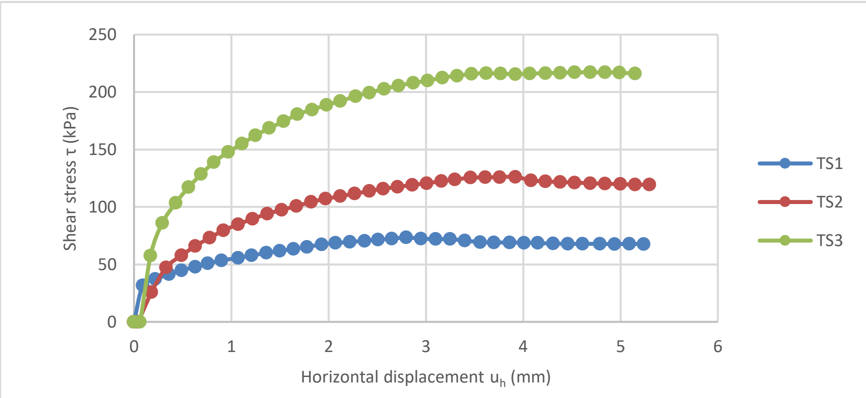

Fig. 3. Test results of the drained direct shear test on test specimens from borehole BVZK-05 (28.10 – 28.40 m).

- Consolidated drained (CD) triaxial compression test
  - Borehole BK-02, sample depth 29.10 – 29.45 m; Specimen TS 1 (σ_c = 100 kPa)
  - Borehole BK-02, sample depth 29.10 – 29.45 m; Specimen TS 2 (σ_c = 200 kPa)
  - Borehole BK-02, sample depth 29.10 – 29.45 m; Specimen TS 3 (σ_c = 400 kPa)

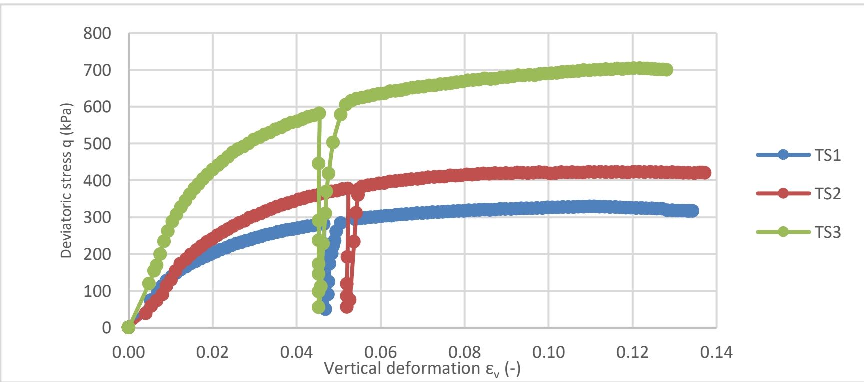

Fig. 4. Test results of the CD triaxial test on test specimens from borehole BK-02 (29.10 – 29.45 m).

- Pressuremeter test
  - Borehole BK-02, sample depth 29.12 – 29.58 m

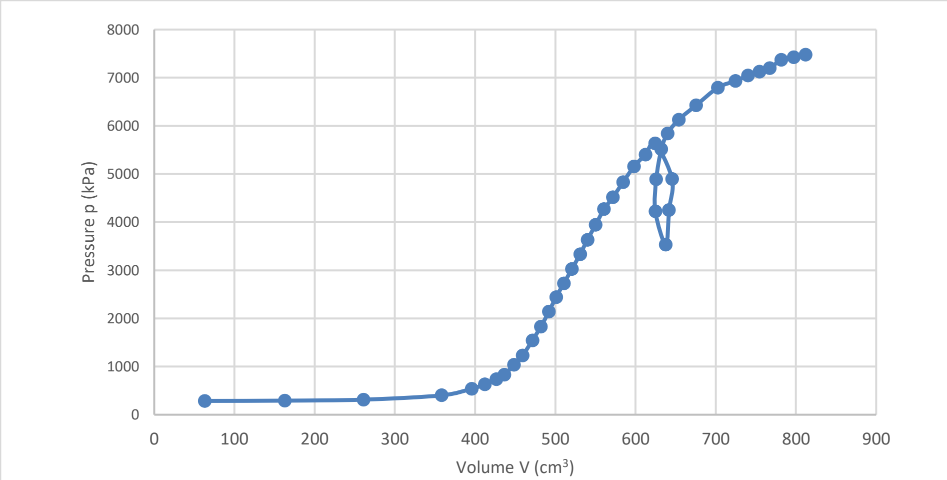

Fig. 5. Test results of the pressuremeter test conducted on test specimen from borehole BK-02 (29.12 – 29.58 m).

It should be noted that the analyzed triaxial compression tests and the pressuremeter test were conducted to include a cycle of unloading and reloading up to failure, and as such, do not represent the "standard" testing procedures for these tests. These tests were intentionally performed in this manner to increase their practical value for defining the parameters of the HSS model, particularly with regard to the unloading–reloading stiffness, which is characterized by the parameter E_{ur}^{ref}.

2.3. Analytical determination of HSS parameters

Certain parameters of the HSS model can be determined analytically from the available laboratory and in-situ tests, while others cannot be determined analytically but can be somewhat reliably estimated for preliminary assessments and the development of basic engineering solutions according to the literature and recommendations of numerical software (e.g., Plaxis). Among these parameters are, for example, the Poisson's ratio, which for elastic states can be assumed as $\nu_{ur} = 0.2$, the coefficient of earth pressure at rest for normally consolidated soils $K_0^n = 1\text{-}\sin\varphi$, the failure ratio $R_f = 0.9$ and others.

For the purposes of this study, the calibrated parameter sets are compared with each other and with analytically derived parameters based on laboratory test results. The analytical determination of parameters from laboratory records follows standard practice, with only certain parameters being reliably derived from specific laboratory and in-situ tests.

From the drained direct shear test, the parameters c, φ, E_{50}^{ref} and m can be somewhat reliably determined analytically. From the CD triaxial test, conducted with an unloading–reloading cycle, the parameters c, φ, E_{50}^{ref}, E_{ur}^{ref}, m, R_f, and G_0^{ref} can be determined analytically with relative reliability. From the pressuremeter test, no parameter can be analytically determined, but the pressuremeter modulus and unload-reload modulus can be used for an approximate estimation of the material stiffness parameters.

2.4. Finite element numerical simulations

All numerical simulations of the laboratory tests will be conducted using Plaxis' SoilTest tool. This tool functions as a virtual laboratory, allowing numerical simulation of laboratory tests on single test specimens without the need to create a complete finite element model.

Within the SoilTest tool, the predefined module for the drained direct shear test is used. In this module, the HSS model parameters are calibrated to achieve the best possible correlation between the $\gamma_{xy} - \tau_{xy}$

curve (shear strain – shear stress) from the numerical simulation and the corresponding curve obtained from laboratory records.

Due to the specific nature of the CD triaxial compression test, which includes an unloading–reloading cycle, the predefined module for this laboratory test cannot be used for numerical simulation. Instead, the test is simulated using the General module. In this case, the curve analyzed to assess the quality of HSS parameter calibration is the ε_1 – q curve (axial strain – deviatoric stress).

Unlike laboratory tests, the in-situ pressuremeter test cannot be numerically simulated using Plaxis' SoilTest tool. For this test, a two-dimensional axisymmetric finite element model must be developed, taking into account all specific conditions of the given test – depth, geotechnical layers, groundwater level, drainage etc. In this approach, the model domain is approximated as a single material layer corresponding to the material in which the test was performed, while the overlying layers above the test location are approximated by adjusting the intensity of the vertical load applied to the top of the model. Both drained and undrained conditions are analyzed. The model simulates all phases of the test in sequence: initial phase, excavation and lining of the investigative borehole, borehole excavation for the pressuremeter, and staged application of pressure steps (test execution).

The curve analyzed for the pressuremeter test in the assessment of parameter calibration quality is the p – V curve (pressure – volume), taking into account the membrane stiffness compensation, probe deformability, and water column between the probe and the instrument pump. Additionally, since a certain amount of pressure must be applied to the probe before it achieves contact with the borehole walls, the calibration of parameters is performed only for the portion of the p – V curve from the step of the test when the probe membrane achieves full contact with the borehole wall to the final step of the test.

2.5. Quality assessment of the calibrated parameter set – application of genetic algorithms

The soil material model parameters were estimated using the JARE optimization service (Ivanovic et al., 2015), which applies a Genetic Algorithm (GA) to fit simulation results to experimental curves. The optimization process relied on evolutionary operators to explore the parameter space efficiently, while the Fréchet distance method (Alt and Godau, 1995) was employed as the goal function to evaluate the similarity between simulated and experimental responses. By minimizing this metric, the service successfully identified parameter sets that ensured close agreement between the model predictions and the experimental data.

The criterion for overall quality assessment is defined as a function of the difference (error) between the numerical and experimental results for all tests incorporated in the calibration process, with the objective of minimizing the value of this criterion function. In general form, taking into consideration all possible test types, the criterion function is defined as follows:

$$K = \left(\sum_{i=1}^{N_{oed}} w_{oed,i} + \sum_{i=1}^{N_{dsd}} w_{dsd,i} + \sum_{i=1}^{N_{dsu}} w_{dsu,i} + \sum_{i=1}^{N_{CD}} w_{CD,i} + \sum_{i=1}^{N_{CU}} w_{CU,i} + \sum_{i=1}^{N_{UU}} w_{UU,i} + \sum_{i=1}^{N_{pres}} w_{pres,i}\right)^{-1} \cdot \left(\sum_{i=1}^{N_{oed}} w_{oed,i} K_{oed,i} + \sum_{i=1}^{N_{dsd}} w_{dsd,i} K_{dsd,i} + \sum_{i=1}^{N_{dsu}} w_{dsu,i} K_{dsu,i} + \sum_{i=1}^{N_{CD}} w_{CD,i} K_{CD,i} + \sum_{i=1}^{N_{CU}} w_{CU,i} K_{CU,i} + \sum_{i=1}^{N_{UU}} w_{UU,i} K_{UU,i} + \sum_{i=1}^{N_{pres}} w_{pres,i} K_{pres,i}\right) \quad (1)$$

In the above formula, the suffixes of the notations used refer to the type of laboratory or in-situ tests, as specified in the following table.

Table 2. Criterion function suffix definitions.

Notation	Test type	Notation	Test type
oed	Oedometer test	CD	Consolidated drained triaxial test
dsd	Drained direct shear test	CU	Consolidated undrained triaxial test
dsu	Undrained direct shear test	UU	Unconsolidated undrained triaxial test
pres	Pressuremeter test		

The notations in the criterion function refer to:

- K – partial criterion function for a specific test type,
- N – number of tests of a specific test type,
- w – weighting factor representing the reliability of the results of tests of a specific test type (for the purposes of this study, w = 1 for all test types)

3. Results and discussion

3.1. Drained direct shear test

For the drained direct shear test, based on a previously conducted sensitivity analyses it was determined that the parameters of greatest significance on the resulting numerical simulation curve are the cohesion c, internal friction angle φ, and secant stiffness E_{50}^{ref}. A relatively good correlation between curves of the laboratory recordings and numerical simulations of all consolidation pressure levels is achieved for the calibrated parameter set (as can be seen in Fig. 6). This is indicated by the low criterion function value of K = 0.041, and by visual verification of the resulting curves. The resulting calibrated parameters are given in Table 4, alongside the values of parameters that can be analytically derived from this type of test.

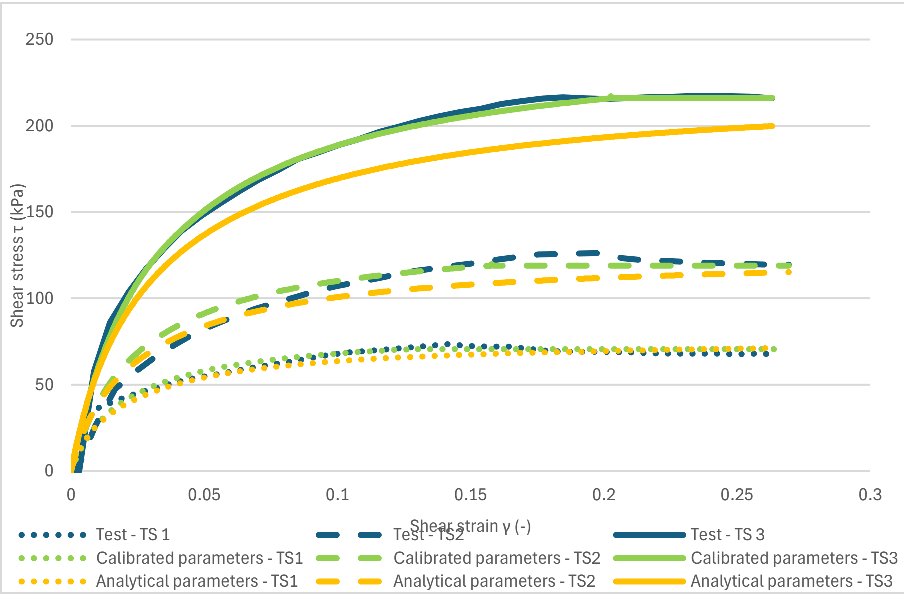

Fig. 6. Comparison of test curves (blue), numerical simulations with calibrated parameter values (green) and numerical simulations with analytical + default parameter values (yellow) for the drained direct shear test.

For comparison, a numerical simulation was also run with a parameter set consisting of the analytically derived parameters, and the default or recommended values of the remaining parameters according to the literature and software recommendations (yellow curves in Figure 6). This simulation resulted in a lower level of correlation with the laboratory results, indicated by a higher criterion value K = 0.074, but with an increasing discrepancy with increasing consolidation stresses.

The small-strain behavior is well captured with the calibrated numerical simulations, and the peak values of the effective strength parameters – soil cohesion c and angle of internal friction φ is in good accordance with the analytically derived values from the laboratory results. The secant stiffness parameter E_{50}^{ref} and exponent m are slightly underestimated via analytical derivation although it should be kept in mind that these parameters are primarily determined based on drained triaxial compression tests, and their analytical derivation based on direct shear tests is for preliminary estimation purposes only.

Table 3. Comparison of calibrated parameters and analytically derived parameters for the drained direct shear test.

Parameter	Analytical derivation (K=0.074)	Test specific calibration (K = 0.041)
c (kN/m^2)*	28.0	23.1
φ (°)*	27.3	27.0
ψ (°)	-	9.89
E_{oed}^{ref} (kN/m^2)	-	16521
E_{50}^{ref} (kN/m^2)*	3223	6666
m (-)	0.405	0.576
ν_{ur} (-)	-	0.358
K_0^{nc} (-)	0.542	0.544
R_f (-)	-	0.803
G_0^{ref} (kN/m^2)	-	54040
$\gamma_{0.7}$ (-)	-	0.0008

* Parameters with greatest influence on resulting numerical curve

3.2. Consolidated drained triaxial test

According to previously conducted sensitivity analyses it was determined that for CD triaxial tests conducted with this testing procedure the parameters of cohesion c, internal friction angle φ, secant stiffness E_{50}^{ref}, unload-reload stiffness E_{ur}^{ref} and failure ratio R_f are of the greatest significance on the resulting numerical simulation curve. For the CD triaxial test, the resulting criterion function value K = 0.050 is obtained for the calibrated parameter set (Table 5) – indicating a relatively good correlation between curves of the laboratory recordings and numerical simulations, although with a visible greater discrepancy for the higher consolidation stresses (Fig. 7). In order to attain a relevant K value, it should be noted that the criterion function was calculated only up to the final point of the numerical simulation or laboratory test (whichever came first), which is justified since the failure deviatoric stress was reached in each simulation.

A numerical simulation was also run with a parameter set consisting of the analytically derived parameters, and the default or recommended values of the remaining parameters according to the literature and software recommendations (yellow curves in Figure 7). This simulation resulted in a comparable level of correlation with the laboratory results, indicated by the criterion value K = 0.124, with the majority of the "error" originating from the lowest consolidation stress, i.e. test specimen TS1. The similarity between the analytically derived parameters and calibrated parameters is to be somewhat expected considering that the HSS material models parameter derivation is based on this type of triaxial test specifically. Samples with even more test specimens could show an even greater similarity between these parameter sets.

Like in the case of the direct shear test, the peak values of the effective strength parameters are in good accordance with the analytically derived values from the laboratory results while the stiffness parameters are slightly less correlated, especially the initial shear modulus G_0^{ref}. The differences between the calibrated parameter sets of the direct shear test and the CD triaxial test, especially the cohesion c and initial shear modulus G_0^{ref}, indicate that the analyzed geotechnical layer exhibits a certain level of heterogeneity depending on the relation of marl and limestone in the engineering-geological complex.

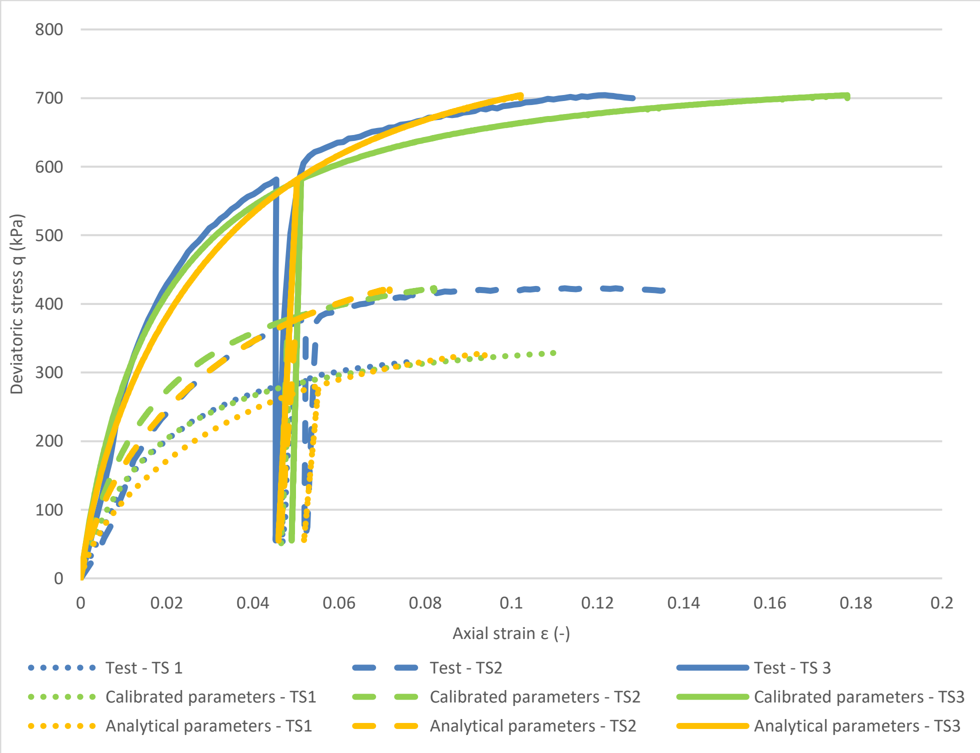

Fig. 7. Comparison of test curves (blue), numerical simulations with calibrated parameter values (green) and numerical simulations with analytical + default parameter values (yellow) for the CD triaxial test.

Table 4. Comparison of calibrated parameters and analytically derived parameters for CD triaxial tests.

Parameter	**Analytical derivation (K=0.124)**	**Test specific calibration (K = 0.050)**
c (kN/m^2)*	72.0	80.2
φ (°)*	22.9	23.0
ψ (°)	-	9.43
E_{oed}^{ref} (kN/m^2)	-	28945
E_{50}^{ref} (kN/m^2)*	12307	11618
E_{ur}^{ref} (kN/m^2)*	66716	121869
m (-)	0.813	0.968
ν_{ur} (-)	-	0.222
K_0^{nc} (-)	0.611	0.477
R_f (-)*	0.865	0.987
G_0^{ref} (kN/m^2)	8788	122733
$\gamma_{0.7}$ (-)	-	0.00001

* Parameters with greatest influence on resulting numerical curve

3.3. Pressuremeter test

Due to the fact that the pressuremeter test represents an in-situ test, the specificities of each testing procedure and the conditions it is executed in have a great influence on the setup of the numerical model with which it is simulated, and in turn, a great influence on the resulting test curves. One of the major effects to consider when calibrating HSS parameters based on the pressuremeter test is the drainage conditions. Depending on the soil medium in which the test is executed, specifically its permeability, but also depending on the time component of the testing procedure, a case-by-case assessment is to be made if the test subjected to numerical simulation is conducted under drained or undrained conditions.

A calibration procedure was carried out on the analyzed pressuremeter test with two approaches – as a drained analysis and as an undrained analysis. The undrained analysis corresponds to the Plaxis' "Undrained B" analysis in which the stiffness parameters are defined with their effective values, and the strength is defined as the undrained shear strength. The corresponding resulting curves are shown in Figure 8, and the values of the calibrated parameter set for both conditions are summarized in Table 6. As can be seen based on the values of the criterion function K, a fairly similar estimate is obtained for the two numerically simulated curves. However, the resulting parameters are quite diverse.

The drained analysis results in a calibrated parameter set consisting of significantly larger values of the internal friction angle φ and the stiffness parameters E_{50}^{ref}, E_{oed}^{ref} and E_{ur}^{ref} in comparison with the values obtained in the calibration process of the CD triaxial test sample from the same borehole and sample depth (Table 5). The undrained analysis results in a numerical curve with a slightly lower criterion function value K, but with significantly lower stiffness parameters E_{50}^{ref} and E_{oed}^{ref}. These attained parameters better correspond to the analytically derived pressuremeter moduli, but it should be kept in mind that the pressuremeter moduli do not represent the actual HSS material model parameters and their physical meaning.

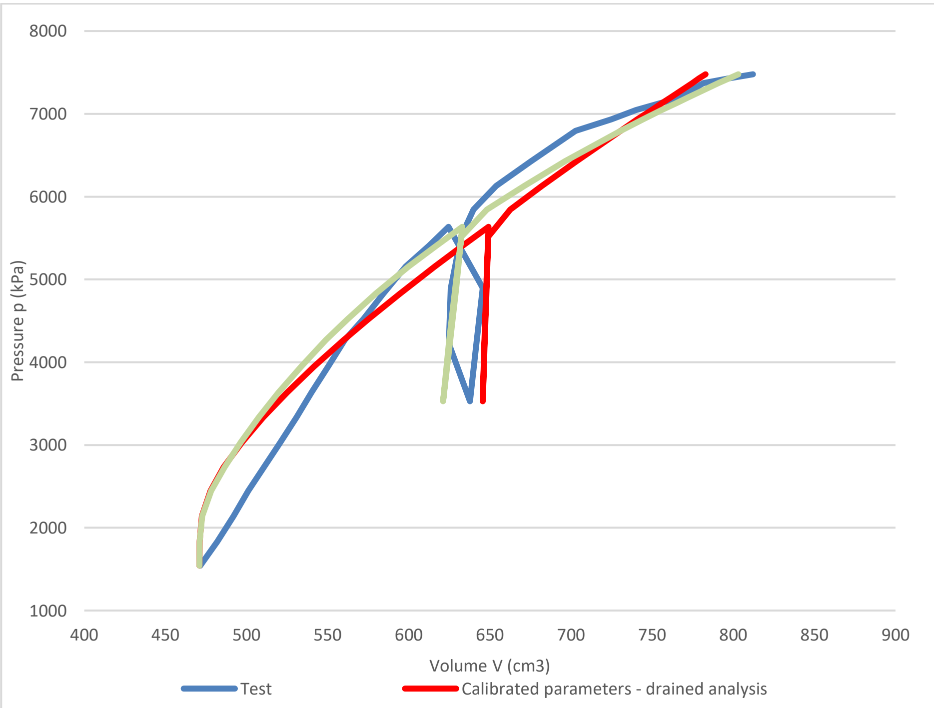

Fig. 8. Comparison of test curve and numerical simulation of pressuremeter tests with calibrated parameter values.

Table 5. Comparison of calibrated parameters and analytically derived parameters for the pressuremeter test.

Parameter	**Analytical derivation**	**Test specific calibration Drained analysis (K = 0.067)**	**Test specific calibration Undrained analysis (K = 0.063)**
c (kN/m^2)	-	76.6	/
φ (°)	-	43.6	/
ψ (°)	-	6.1	/
s_u (kN/m^2)	-	/	2616
E_{oed}^{ref} (kN/m^2)	183667*	334775	100622
E_{50}^{ref} (kN/m^2)		291386	85424
E_{ur}^{ref} (kN/m^2)	774951*	1098971	700560
m (-)	-	0.635	0.401
ν_{ur} (-)	-	0.190	0.327
K_0^{nc} (-)	-	0.383	0.435
R_f (-)	-	0.892	0.760
G_0^{ref} (kN/m^2)	-	671723	564532
$\gamma_{0.7}$ (-)	-	0.00032	0.00067

* Values refer to the pressuremeter moduli, not the specific HSS parameter

4. Conclusions

The conducted calibration of the HSS material model parameters based on drained direct shear, consolidated drained triaxial, and pressuremeter tests has shown that reliable parameter identification requires an approach encompassing a combination of multiple laboratory and in-situ tests. Besides that, careful consideration of the numerical modeling procedure including boundary conditions and drainage assumptions is instrumental in achieving realistic simulations that adequately capture soil behavior under varying stress conditions.

For both the direct shear and triaxial tests, the peak strength parameters – cohesion c and internal friction angle φ – were consistently well reproduced by the calibrated numerical simulations showing good agreement with the analytically derived values from laboratory results. The stiffness-related parameters (E_{50}^{ref}, E_{ur}^{ref}, m, G_0^{ref}) were less consistently matched, with some discrepancy between analytically derived and calibrated values. Besides underlining the importance of correct grouping of soil samples according to their originating soil layer, this highlights the inherent limitation of deriving stiffness parameters solely from singular laboratory test specimens.

The pressuremeter calibration further emphasized the complexity of parameter derivation, especially regarding drainage conditions. Both drained and undrained analyses yielded similarly acceptable levels of correlation with the experimental curve, but the resulting calibrated parameter sets were markedly different. The drained calibration led to unrealistically high stiffness and strength values compared to triaxial test results on specimens of the same borehole and depth, while the undrained calibration produced lower stiffness moduli more consistent with derived values from pressuremeter results – though these do not directly correspond to HSS input parameters. In general, this calls attention to the careful consideration of the drainage conditions of each specific pressuremeter test sample when calibrating parameter values for further design.

The discrepancies of calibration results observed between test types also suggest a degree of material heterogeneity in the analyzed geotechnical layer. This necessitates future research which should focus on calibration strategies that encompass even more specimens and different test types simultaneously.

Overall, the research demonstrates that while analytically derived parameters provide a useful starting point for preliminary design, calibration against numerical simulations is essential for capturing the realistic stress-dependent stiffness behavior of soils and weak rocks, especially for use cases like urban tunnel design where soil deformations play a critical role in the development of technical design solutions. Together with the successful application of genetic algorithms, this research demonstrates the potential of optimization-based calibration methods to enhance the reliability and efficiency of geotechnical numerical modeling.

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