Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-345

Field-scale validation of a 3D hypoplastic contact model for tension piles in layered soils

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ABSTRACT: Simplified contact models in geotechnical design often fail to capture the complex non-linear behaviour of soil-pile interfaces, particularly in stratified ground. This study presents a comprehensive back-analysis utilising a 3D hypoplastic contact model for full-scale field tensile load tests on driven steel piles in highly stratified soil. The model, calibrated against torsional interface shear tests, demonstrates significantly improved predictive accuracy compared with conventional Coulomb friction approaches. The results indicate that the hypoplastic model accurately replicates the non-linear load-displacement response observed in the field, resulting in a 71 % reduction in prediction error (MAPE). A subsequent parametric analysis quantifies the influence of key physical parameters, such as pile circumference, interface roughness, soil relative density, and in-situ earth pressure. This study demonstrates the importance of physically-based contact models in ensuring the reliable and economical design of pile foundations under challenging geotechnical conditions.

KEYWORDS: Tension piles, hypoplastic contact model, pile-soil interaction, finite element analysis, layered soils.

1 INTRODUCTION

Reliable prediction of the tensile bearing capacity of driven steel piles in layered soils remains a significant challenge in geotechnical engineering. This challenge is particularly pronounced in highly stratified soil profiles, where the interaction between multiple soil layers and the pile interface creates complex stress redistribution patterns. Current design practice relies predominantly on empirical correlations and simplified analytical methods. These conventional approaches often fail to adequately account for the stress-dependent, non-linear nature of soil-pile interface behaviour. This limitation often leads to discrepancies between predicted and actual capacities, highlighting the need for more accurate numerical models (Alkateeb and Grabe, 2022, 2025a, 2025b)

Simplified contact models, such as the Coulomb friction law, are widely employed in numerical simulations (Stapelfeldt et al., 2020; Bienen et al., 2021; Dao et al., 2023). Although capable of estimating ultimate capacities, they are frequently inadequate for complex soil-structure interaction problems (Arnold and Herle, 2006; Stutz et al., 2017; Niebler et al., 2025). A recent Class-A prediction study by Alkateeb and Grabe (2025) demonstrated this limitation, motivating the present study. A Class-A prediction study involves predicting the behaviour of a system before experimental results are known, providing a rigorous test of a model's predictive capabilities.

This study employs an advanced hypoplastic contact formulation, calibrated against dedicated torsional interface shear tests, presenting the first back-analysis implementing a 3D hypoplastic contact model for full-scale tensile load tests on driven steel piles in a highly stratified soil profile. This study directly compares the performance of the hypoplastic model with that of the conventional Coulomb approach, quantifying significant improvements in predictive accuracy through comprehensive parametric studies. Furthermore, comprehensive parametric studies are presented to establish quantitative guidelines for selecting interface parameters, thereby contributing to more reliable and economical pile foundation designs.

2 NUMERICAL MODEL

2.1 *Finite-element set-up*

The numerical analysis was conducted using the finite element software Abaqus/Standard. A quarter-symmetry 3D model was created to simulate the pile and surrounding soil, thereby reducing computational costs (Figure 1). The model domain (10 m × 10 m × 30 m) was discretised with over 74,000 eight-noded continuum elements (C3D8R), featuring a refined mesh near the pile to capture the high stress and strain gradients accurately. A mesh sensitivity analysis confirmed that further refinement resulted in a variation of less than 2% in the results.

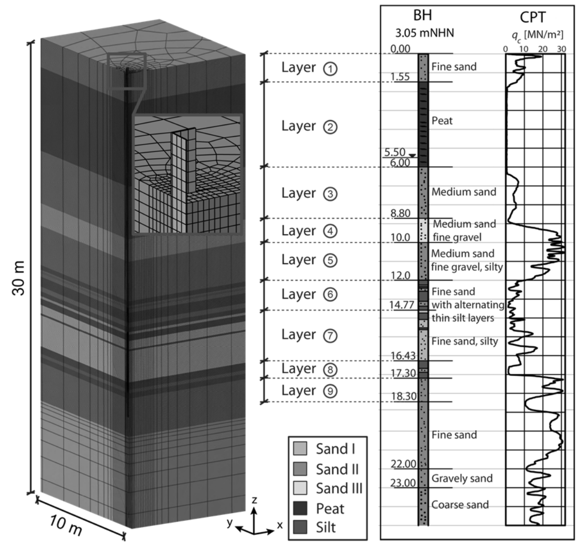


Figure 1. Quarter-symmetry finite element model and soil structure with cone penetration test (CPT) data.

The HP320×88.5 pile was modelled as a linear-elastic material (S355 steel). The pile was installed "wished-in-place", neglecting installation effects to focus on interface behaviour during tension loading. The pile length in the model was 18.8 m, with an embedment depth of 18.3 m, and the pile head extended 0.5 m above ground level.

The simulation sequence involved establishing the initial geostatic stress state of K_0 = 1 - $\sin \varphi'$ using Jaky (1948) coefficients, activating the pile self-weight and contact, and applying the displacement-controlled loading path from the field tests (Alkateeb and Grabe, 2025). The highly stratified subsoil (23 layers) was explicitly implemented with properties grouped into nine representative Layers, as seen in Figure 1. The grain size distribution curves of the main identified sands (Sand I, II and III) are shown in Figure 2.

2.2 *Soil models*

To accurately reflect the site's geological complexity, the 23 distinct soil layers identified in the field exploration were explicitly incorporated into the numerical geometry.

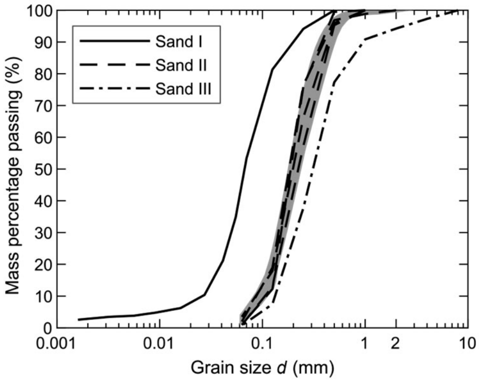
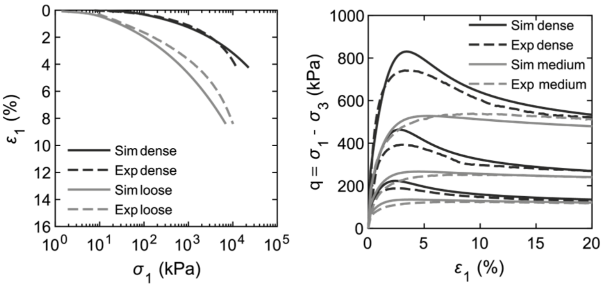


Figure 2. Representative grain size distribution curves of the investigated sands.

The mechanical behaviour of the predominant non-cohesive sand layers was described using the hypoplastic constitutive model with intergranular strain extension, which accurately accounts for pressure- and density-dependent behaviour (von Wolffersdorff, 1996; Niemunis and Herle, 1997). To ensure representative behaviour throughout the profile, model parameters for the three characteristic sand types (see Figure 2) were systematically calibrated against laboratory triaxial and oedometer tests from multiple depths. The simulations show good agreement with the experimental results, as exemplified by the comparison for Sand III in Figure 3. The complete set of calibration results is detailed in Alkateeb et al. (2023) and Alkateeb and Grabe (2025). The final parameter sets for the sands are listed in Table 1.



(a) (b)

Figure 3. Model calibration for Sand III: Comparison of simulations with experimental data for (a) consolidated drained (CD) triaxial tests and (b) constant rate of strain (CRS) oedometric tests under various stress states and densities.

Table 1. Parameters of hypoplasticity with intergranular strain extension for the investigated non-cohesive sands.

Layer	Unit	Sand I	Sand II	Sand III
φ_c	(°)	32.8	31.2	30.1
h_s	(MPa)	1650	2442	2267
n	(-)	0.35	0.255	0.29
e_{d0}	(-)	0.48	0.47	0.37
e_{c0}	(-)	0.98	0.84	0.72
e_{i0}	(-)	1.12	0.97	0.83
α	(-)	0.055	0.09	0.15
β	(-)	1.56	1.65	1.30
m_T	(-)	1.46	1.53	1.17
m_R	(-)	4.68	4.52	4.65
R_{max}	(-)	2.1×10^{-4}	6.7×10^{-4}	2.3×10^{-4}
β_R	(-)	0.16	0.58	0.80
χ	(-)	3.46	4.92	3.06

In contrast, the secondary cohesive layers (peat and silt) were modelled using the Mohr-Coulomb failure criterion. This simplification was justified by sensitivity analyses, which demonstrated that these layers contribute less than 6 % to the total shaft resistance. The corresponding parameters are listed in Table 2.

Table 2. Material parameters for cohesive layers and steel.

Material	E	ν	φ	c'/c_u	ψ
	(kPa)	(-)	(°)	(kPa)	(°)
Peat	1200	0.38	52	25	0
Silt	4000	0.49	0	50	0

2.3 *Interface models*

The pile-soil interaction was modelled using a surface-to-surface contact discretisation with a "finite sliding" formulation. In the normal direction, a "hard" contact behaviour was enforced to prevent penetration. For the tangential behaviour, two distinct formulations were comparatively evaluated:

2.3.1 *Coulomb friction model*

The Coulomb friction model represents the standard elasto-plastic approach. The maximum shear stress is defined as $\tau_{\max} = \sigma_n' \tan\delta$, where σ_n' denotes the effective normal stress and δ is the interface friction angle. This model assumes a linear-elastic response up to a critical displacement threshold u_{limit}, followed by perfectly plastic sliding.

2.3.2 *Hypoplastic contact model*

The hypoplastic contact model represents a physically-based approach derived from the concept that granular soil behaviour within the interface zone is analogous to that of a continuum, with surface roughness acting as the governing state parameter (Arnold and Herle, 2006). A significant advantage of this formulation is parameter consistency: the contact model utilises the same constitutive parameters as the adjacent soil continuum (Table 1). It requires only two additional interface-specific parameters:

- Interface roughness parameter defined as the ratio $\kappa = \frac{\delta}{\varphi}$.
- Virtual shear zone thickness d_s, representing the physical thickness of the localised shear band.

The model is implemented in Abaqus by coupling the FRIC and UMAT user subroutines. In this approach, the FRIC subroutine translates the contact kinematics into stress and strain tensors, which are processed by the UMAT containing the hypoplastic material logic. This method ensures a consistent representation of the soil-structure interaction and uses an explicit forward-Euler integration scheme with adaptive sub-stepping to update the interface stresses.

2.4 *Interface parameter calibration*

To ensure physical validity, the hypoplastic interface parameters were determined via a systematic calibration protocol utilising a combined experimental-numerical approach. The roughness parameter κ was derived from torsional interface shear tests performed on steel-sand specimens using the actual pile steel. These tests yielded κ = 0.66-0.68 for the predominant sandy layers. This value aligns well with established design recommendations for rough, untreated steel surfaces in sand (EAU, 2020). While direct calibration was employed here, alternative approaches can be used to determine these parameters, such as estimation from the literature or back-analysis (Arnold and Herle, 2006; Stutz et al., 2017).

The virtual shear zone thickness was calibrated to d_s= 20 d_{50}, balancing physical representation with numerical efficiency, falling within the range reported in literature (Tejchman, 1989; Tejchman and Wu, 1995; Arnold and Herle, 2006). This parameter represents the actual thickness of the developing shear band at the contact surface and significantly influences the load-displacement response characteristics.

3 RESULTS AND DISCUSSION

3.1 *Comparison of contact models*

The numerical back-analysis demonstrates that the hypoplastic contact model significantly outperforms the conventional Coulomb model in reproducing the field load-displacement behaviour (Figure 4). The hypoplastic model achieves a Mean Absolute Percentage Error (MAPE) of 2.64% and a Root Mean Square Error (RMSE) of 8.83 kN, representing a 71 % reduction in prediction error compared to the Coulomb approach.

This better accuracy of the hypoplastic formulation stems from its inherent ability to capture stress history and non-linear, stress-dependent stiffness, crucial for hysteretic behaviour. The Coulomb model, in contrast, underestimates stiffness and fails to capture hysteresis, exhibiting a simplified linear-elastic-perfectly-plastic response. Although it is computationally more efficient, running approximately 6 times faster than the hypoplastic model simulation.

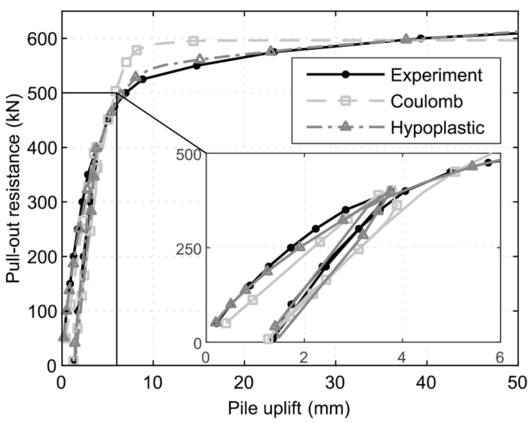


Figure 4. Comparison of load-displacement curves from experimental measurements with numerical calculations using the hypoplastic and Coulomb contact models.

3.2 *Parametric analysis*

Using the validated hypoplastic model as a baseline, a comprehensive parametric study was conducted to quantify the sensitivity of the tensile capacity to key design variables. Five critical parameters were systematically varied, and their impact is summarised in the tornado plot in Figure 5.

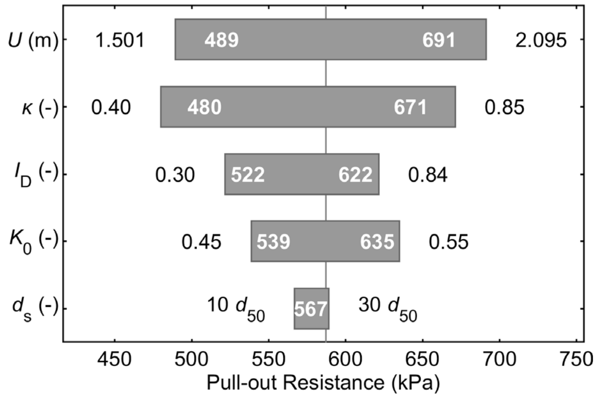


Figure 5. Tornado diagram from the parametric analysis, illustrating the sensitivity of the predicted pull-out resistance to variations in key input parameters relative to the calibrated baseline simulation

Pile circumference: Varying U from 1.501 m to 2.095 m resulted in the pull-out resistance changing from 489 kPa to 691 kPa, respectively. This represents a capacity variation of approximately ±17% relative to the baseline simulation, confirming that the geometric contact area is the dominant scaling factor for capacity.

Interface roughness parameter (κ): Variations in κ from 0.4 to 0.85 resulted in pull-out capacity changes of -18 to +14%, demonstrating the critical importance of accurate interface characterisation.

Relative density (I_D): Relative density variations from 30% to 84% produced capacity variations from the baseline of approximately -11% to +6%, with the hypoplastic model accurately capturing this density-dependent behaviour through its pyknotropy formulation.

Earth pressure coefficient (K_0): Variations from 0.45 to 0.55 affected the ultimate capacity by approximately ±8%, highlighting the significant influence of the initial in-situ stress state.

Shear zone thickness (d_s): The virtual shear zone thickness exhibited a negligible impact on the ultimate capacity. Varying d_s between 10 d_{50} and 30 d_{50} resulted in less than a 4% change in the predicted pull-out resistance, confirming the model's stability with respect to this numerical parameter.

3.3 *Design Recommendations and Practical Implementation*

Based on the quantitative findings, the following recommendations are proposed for engineering practice:

- For projects involving complex pile-soil interaction and loading paths (e.g., cyclic loading, unloading-reloading), advanced contact models are necessary to accurately capture holistic response and non-linear stiffness.
- The roughness parameter κ can be reliably calibrated using interface tests. While this study successfully used torsional shear tests, other methods like interface direct shear tests are also suitable for determining a reliable value for κ.
- Also, employing values from established literature correlations proved for this case to be sufficient. For example, for rough steel surfaces in sand, an interface roughness parameter of κ = 0.67 ± 0.1 is expected to yield predictions within approximately 5% of calibrated results and are suitable for preliminary design.
- For the virtual shear zone thickness, employing values from established literature recommendations provides

sufficient accuracy for the analysis, eliminating the need for complex back-calculation in routine projects.

- It is crucial to invest in detailed site investigations to create a layer-specific, depth-dependent profile for soil relative density (I_{D}) and soil parameters, as simplified uniform assumptions lead to significant errors.
- To ensure convergence in hypoplastic contact simulations, a refined mesh with at least four elements across the pile width, and aspect ratios of ≤ 2 near the interface, is recommended. Furthermore, engineers must consider the limitations of the "wished-in-place" assumption, recognising that real-world installation effects can alter the final pile capacity.

4 CONCLUSIONS

This study presented the first 3D back-analysis of full-scale tension pile tests using a hypoplastic contact model within a highly stratified soil profile. The advanced model, calibrated via torsional interface shear tests, successfully replicated the complex, non-linear field response, including the hysteretic behaviour during unloading sequences. Compared to a conventional Coulomb model, it achieved a 71 % reduction in prediction error (MAPE 2.64 %). The study confirms that a holistic and realistic representation of the pull-out process requires contact models that capture key soil-structure interaction phenomena, such as barotropy and pyknotropy. The Coulomb model, in contrast, proved inadequate even with an advanced hypoplastic soil continuum.

From a practical perspective, the hypoplastic contact model offers a practical advantage, as it utilises the same constitutive parameters as the surrounding soil, requiring only the estimation of the roughness parameter κ. This facilitates reliable Class-A predictions from routine laboratory data without specialised interface testing.

A parametric study quantified the sensitivity of ultimate tensile capacity to key parameters. The analysis confirmed that, beyond the pile circumference U, the interface roughness parameter κ and the soil relative density I_{D} are the most influential factors affecting tensile capacity predictions. Shear band thickness d_{s} showed minimal influence on the ultimate capacity. These results affirm the model's stability and highlight the benefits of physics-informed interface modelling in tension pile design.

The study provides a practical, validated framework for engineers to utilise advanced contact models, with clear guidance on parameter selection and calibration. Future work should address current idealisations, including pile installation effects, cyclic/dynamic loading, and more advanced constitutive descriptions for cohesive soil layers.

5 ACKNOWLEDGEMENTS

This work was conducted as part of the R&D project "Tragfähigkeit von Stahlrammpfählen" in collaboration with the Federal Waterways Engineering and Research Institute (BAW). The authors gratefully acknowledge the BAW for their financial support and technical collaboration.

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