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/ICSMGE2026-266

Calibration of Interface Parameters in Soil–Structure Interaction Problems: An Experimental and Numerical Approach

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ABSTRACT: The proper calibration of interface parameters is important in the prediction of soil-structure interaction, specifically for the interface between the soil and shotcrete lining in tunnel structures. Inaccurate representation of the interface in the calculation model is likely to generate large errors for the predicted stress and strain distributions. A hybrid approach involving experimental and numerical methods is adopted in this study to realize adhesion and friction of the soil-shotcrete interface which are important dynamics and static interactions between soil and structure. In this research, a series of controlled laboratory experiment are conducted to validate the finite element modeling approach. A range of finite element techniques is employed to simulate the mechanical properties of the soil-shotcrete interface, integrating constitutive models that iteratively refine interface parameters to achieve precise alignment with experimental data. This comprehensive approach facilitates robust calibration of soil-shotcrete interactions, significantly enhancing model accuracy and reliability.

KEYWORDS: Interface Calibration, FEM, Direct Shear Test, Soil-Structure Interaction.

1 INTRODUCTION

The direct shear test remains one of the most widely used experimental techniques for determining the shear strength parameters of geomaterials, including soils and soil-structure interfaces. Despite its simplicity, the test is affected by non-uniform stress distributions, particularly in the shear plane, leading to uncertainties in interpreting shear strength parameters such as friction angle φ, cohesion c, and dilatancy angle ψ. Dirgėlienė et al. (2017) emphasized the non-uniformity of stress distribution within the specimen during direct shear testing, demonstrating through both experimental and finite element analysis (FEA) that not all applied vertical stress is transmitted directly to the shear plane, especially under constant vertical load conditions. Recent studies have incorporated advanced numerical techniques, such as the FEA, to better understand and replicate the mechanical behaviour observed in laboratory shear tests. Anwar et al. (2012) employed FEA with a modified Drucker–Prager model to simulate large-scale direct shear tests on sandy clay, validating numerical outcomes against experimental data and exploring the sensitivity of shear strength to changes in cohesion, friction angle, and Young's modulus.

Several numerical strategies have been proposed to simulate these interfaces effectively. Damians et al. (2022) explored the equivalency between continuum and zero-thickness interface elements for soil–facing interactions, highlighting that, with appropriate calibration, both approaches can yield comparable stress distributions and load transfers in finite element analyses. Similarly, Yu et al. (2015) conducted a comparative study of interface models in FLAC and PLAXIS, demonstrating how variations in interface stiffness, cohesion, and structural element types (e.g., beam vs. geogrid) significantly influence the numerical prediction of reinforced soil-structure behaviour. Zhu et al. (1995) emphasized the importance of capturing uneven shear displacements along soil–concrete interfaces, proposing a new thick-interface element that better reflects real deformation patterns than traditional zero-thickness elements. Complementing these works, Stastny and Tschuchnigg (2023) investigated cyclic soil–structure interaction using zero-thickness interface elements and found that such models may suffer from interpenetration effects, especially under large lateral loading, which can compromise earth pressure estimations. These insights collectively underscore the critical role of selecting and calibrating appropriate interface models in numerical simulations to ensure realistic soil–structure interaction behaviour.

2 METHODOLOGY

This research employs an integrated approach combining laboratory direct shear tests and numerical modeling in PLAXIS to characterize and calibrate the interface parameters between concrete and sand. The main objective of this paper was to investigate how sand—and, more broadly, soil properties—influence the behavior of the soil–structure interface. The methodology consists of three main phases: experimental testing, parameter determination, and numerical calibration.

2.1 *Laboratory experiments*

A series of direct shear tests were conducted on concrete-sand interfaces under varying normal stresses to obtain shear stress-displacement and normal stress-displacement relationships. The tests were performed in a controlled environment to minimize boundary effects, ensuring a precise assessment of interface strength parameters. Data acquisition included shear displacement u_t, normal displacement u_n, shear force F_t, and normal force F_n, allowing for a detailed analysis of shear stiffness K_t and normal stiffness K_n.

Four low-displacement (0.05 mm per minute) rate direct shear tests, with 4 normal stress levels each, were performed on interface specimens comprising 28-day cured concrete and Ottendorf-Okrilla sand to systematically investigate the mechanical behaviour of the interface under varying normal stresses. The material properties are detailed in Table 1. Ottendorf-Okrilla Sand is a commercially available quartz Siligran from the deposit Ottendorf-Okrilla in south-east Saxonia, Germany. Ottendorf-Okrilla sand is available in the mineral size fractions 0.1 - 0.5 mm and 0.1 -2.0 mm. In this study, sand with a particle size range of 0.1 to 2.0 mm was utilized.

Table 1. General Properties of Ottendorf-Okrilla Sand 0.1 - 2.0 mm.

Parameter	Symbol	Value	Unit
Grain Size	d_{10}	0.19	mm
	d_{30}	0.30	mm
	d_{60}	0.69	mm
	d_{90}	1.49	mm
Mean Grain Size	d_{50}	0.58	mm
Specific Weight	γ	2.635	gr/cm
Mineral Composition	Quartz	97	%
Max.Void Ratios	$e_{\max}$	0.75	-
Min. Void Ratios	$e_{\min}$	0.42	-

The shear box dimensions were 100 mm × 100 mm, conforming to standardized testing protocols for direct shear testing of interfaces (Fig. 1). The tests were conducted under monotonic loading conditions, with normal stresses varying from 100 kPa to 400 kPa, simulating a range of confinement conditions typically encountered in geotechnical and tunneling applications. A constant normal stress (CNS) condition was imposed during the tests, ensuring that vertical stress remained unchanged while shear displacement was applied incrementally. The shear loading rate was selected to minimize pressure buildup and allow for quasi-static conditions, ensuring that the interface response was governed predominantly by frictional and cohesive mechanisms rather than rate-dependent effects.

These laboratory experiments provide fundamental data for the subsequent numerical calibration and validation of interface constitutive models in PLAXIS, ensuring a robust and physics-based representation of concrete-sand interactions in computational simulations.

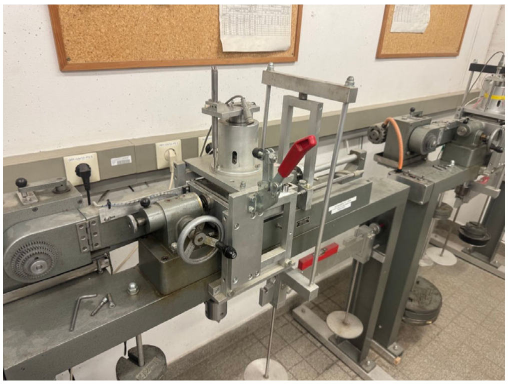

Figure 1. Direct shear device.

2.2 *Concrete mix design*

The concrete mix design follows a water-to-cement *w/c* ratio of 0.65, ensuring an optimal balance between strength and workability. The mixture includes 400 kg/m³ of cement, acting as the primary binding material, while 160 kg/m³ of water is incorporated to facilitate hydration and maintain the desired consistency. Additionally, 1200 kg/m³ of fine aggregate is used to enhance the mixture's density and provide structural stability. This mix design was selected based on common industry practice; although specific additives were not included in the formulation, their absence is not expected to significantly affect the validity or reliability of the results.

2.3 *Parameter Validation*

The interface shear strength parameters were extracted from the laboratory results using a Mohr-Coulomb (MC) failure envelope, where friction angle φ and cohesion c were determined through linear regression of shear stress versus normal stress data. The stiffness properties were obtained from the initial elastic region of the shear stress - displacement and normal stress - displacement curves. Specifically, K_t was derived as the slope of the initial portion of the shear stress versus shear displacement curve, and K_n was obtained as the slope of the normal stress versus normal displacement response.

The peak friction angle for the sand was selected based on the results of 13 consolidated drained triaxial tests performed on reconstituted samples. The strength parameter selection is grounded in the application of the MC failure criterion, assuming drained conditions and zero cohesion - typical for clean, coarse granular soils. As shown in Figure 2, the distribution of test results indicates a concentration of peak friction angles between 36° and 37°, which corresponds to the modal range of the data (Figure 2). Also, the results of the experimental programs is presented inTable 2.

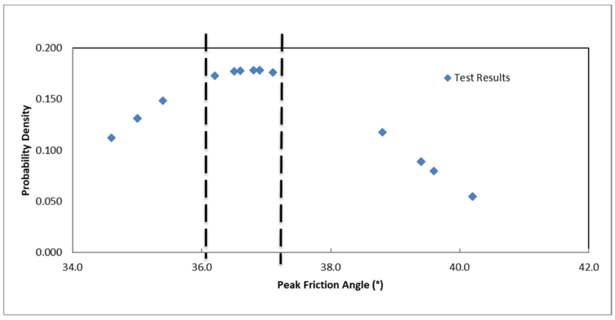

Figure 2. Distribution of Peak friction angle in triaxial tests (adopted from Bathaeian 2014).

The peak deviatoric strength q_{peak} was estimated using a simplified Mohr's circle representation based on the effective principal stress differences. This approach provides a conservative yet realistic characterization of the material behaviour for use in numerical modeling and design calculations (Equation 1).

$$q_{\text{peak}} = 2 \cdot \sigma_r \cdot \tan(\varphi_p) \qquad (1)$$

q_{peak}: Peak deviatoric stress
σ_r: Radial or confining stress
φ_p: Peak effective friction angle

Table 2. The corresponding input parameters of Ottendorf-Okrilla

Parameter	Symbol	Value	Unit
cohesion	C'	0	kpa
Friction angle	ϕ'	36.5	°
Dilation angle	ψ	6.3	°
Reference stiffness	E_{50}^{ref}	13,000	kpa
Oedometer stiffness	E_{oed}	39,000	kpa
Unloading stiffness	E_u^f	39,000	kpa

3 SIMULATION

3.1 *FEA Model*

A FEA of the direct shear test was developed in PLAXIS 2D, incorporating interface elements to simulate the behaviour of concrete-sand contact. As illustrated inFigure 3, the soil layer extends to a depth of 4 cm, with 10 cm width in accordance with the experimental set up. Vertical stress is applied to replicate the initial normal stress conditions at the beginning of the test. To simulate the shear test, a horizontal displacement of up to 15 mm is applied to the upper part of the model. The bottom boundary of the model is fixed in both horizontal and vertical directions. Between upper and lower part of the model an interface is applied. In PLAXIS, the soil-structure interface can be simulated using interface elements that allow relative displacement and shear transfer between the structural surface and the surrounding soil, with properties calibrated to reflect adhesion and friction behavior.

The soil domain is discretized using 15-node second-order triangular elements under plane strain conditions. To enhance the accuracy of the numerical results, a refined mesh zoning strategy is employed. This approach involves locally increasing the mesh density in areas of interest while maintaining a coarser mesh elsewhere (Figure 3).

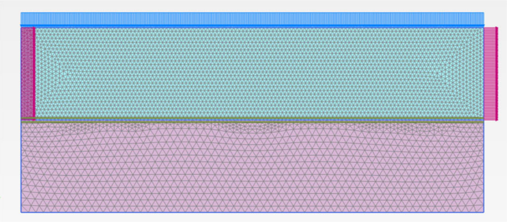

Figure 3. Schematic of numerical model.

3.2 *Constitutive models*

Both the MC and HS constitutive models were employed to simulate the mechanical response of the sand in PLAXIS. The strength and stiffness parameters for these models were derived from the results tests performed on Ottendorf–Okrilla sand (Table 2).

3.3 *Interface behaviour in FEM media*

In FEA, soil–structure interfaces are commonly represented using specialized elements known as zero-thickness interface elements (e.g., Goodman et al., 1968; Day & Potts, 1994), or alternatively, by employing thin layers of continuum elements with modified strength and stiffness parameters (e.g., Desai et al., 1984). The benefits and constraints of these modeling approaches have been extensively examined in previous studies (e.g., Sharma & Desai, 1992; Potts & Zdravković, 2001). Zero-thickness elements are composed of node pairs - each assigned to either the structural or soil domain - connected by elastic-perfectly plastic springs. These springs independently regulate displacement discontinuities, such as separation or penetration in the normal direction and sliding along the interface.

In numerical modeling, two primary approaches are commonly employed to simulate the interface behaviour between soil and structural surfaces. The first approach is the Strength Scaling Method, in which the interface shear strength parameters are considered as a proportion of those of the adjacent ground material. The second approach involves defining the elastic stiffness of the interface in both the normal and shear directions in addition to the strength parameters.

In Plaxis (Bentley, 2022), the transition between elastic and plastic interface behaviour is governed by the Coulomb failure criterion, with permanent slip occurring once this threshold is exceeded:

$$c_i = R_i . c_{\text{Soil}} \tag{2}$$

$$tan\ \varphi_i = R_i . tan\ \varphi_{\text{soil}} \tag{3}$$

The interface strength behaviour is defined using the normal stress, the shear stress, the interface friction angle φ_i, and the interface cohesion c_i. To account for differences between the interface and the adjacent soil, a scaling factor $R_i \leq 1$ is applied. This factor modifies the strength characteristics of the interface, either based on the default relationship with the surrounding soil properties (as done in eq. 2 and 3) or as specified directly by the user through input parameters.

Regarding second approach, the elastic stiffness of the interface in the normal and shear directions is defined based on the material's mechanical properties. In total 3 different interface representations have been investigated in this contribution. One set up with interface strength resulting from the MC model, one set up with interface strength resulting from the Hardening Soil (HS) model and one set up with interface representation by stiffness.

4 RESULTS AND DISCUSSIONS

4.1 *Strength scaling based on Mohr-Coulomb (MC) Model*

In this part of study, the mechanical behaviour of the interface subjected to different normal stress is evaluated using numerical simulations based on MC in PLAXIS. A key focus is placed on the effect of the interface strength scaling factor R_i, which governs the mobilization of shear strength along the contact surfaces. The study presents simulation results for three levels of R_i: 65%, 75%, and 85%, each compared against physical test data obtained under four axial loads: 100 kPa, 200 kPa, 300 kPa, and 400 kPa. The results are presented in terms of sheer force–displacement behaviour (Figure 4).

At R_i = 65%, the simulated curves underestimate the peak shear strength across all normal stress. This is most evident in the 400 kPa prestress case, where the test shows a peak shear stress of over 330 kPa, while the FEM response plateaus around 260 kPa. The discrepancy becomes more pronounced at lower normal stress; for instance, at 100 kPa, the numerical model only reaches a peak shear of approximately 55 kPa, compared to nearly 85 kPa in the experiment. The post-peak or residual response in all cases for this R_i value is essentially flat, reflecting the limitation of the MC model in reproducing strain-softening behaviour. Generally, a R_i of 65% leads to a conservative estimate. Although this may be preferred in safety-critical designs, it clearly fails to reflect the actual load-transfer capacity observed in the tests, especially in the peak region

When R_i is increased to 75%, the agreement between numerical and experimental peak values improves significantly. In the 400 kPa stress case, the FEM results now reach close to the experimental peak, albeit slightly overshooting it before levelling off. For intermediate stress (200 kPa and 300 kPa), the simulated peak shear stress is also much closer to the test results, with differences reduced to within 10–15%. Notably, the post-peak portion continues to plateau, reflecting the inherent constraint of the M-C model in representing progressive failure or dilatancy effects. Nonetheless, this level of scaling factor appears to offer a good compromise between safety conservatism and fidelity to experimental trends

At a scaling factor of Ri = 85%, the numerical simulation shows a marked improvement in reproducing the experimental shear–displacement behaviour, particularly under medium to high normal stresses. For the 400 kPa case, the numerical model slightly overestimates the peak shear strength by approximately 19% (405 kPa in FEA versus 340 kPa in the experiment), while maintaining a consistent trend with the observed rapid rise and plateau in the experimental curve. Under 300 kPa confinement, the simulated peak shear force exceeds the experimental value by roughly 16%, demonstrating a satisfactory match in both magnitude and initial stiffness. At lower normal stresses, the discrepancies are less pronounced. For example, at 100 kPa, the numerical response overpredicts the experimental peak by only 11%, and at 200 kPa, the deviation remains within 13%.

Despite the improvement in peak force predictions, the post-peak behaviour remains inadequately captured. All FEA curves exhibit flat plateaus after reaching their peak, with no noticeable strength degradation. In contrast, the experimental results show a softening trend, reflecting typical interface behaviour under displacement-controlled shear.

4.2 *Strength scaling based on Hardening Soil (HS) Model*

In MC model assumes linear-elastic perfectly plastic behaviour for soil and the interface follows a Coulomb friction law, and no stiffness variation occurs along the loading path. The stiffness is implicitly defined by the surrounding soil elements and the numerical formulation of the interface.

The main difference in MC and HS results lies in the surrounding soil's stiffness response. On the other hand, in HS model follow stress-dependent stiffness and non-linear strain-hardening/softening behaviour.

The FEA curves based on the HS model demonstrate a significantly improved match with the experimental data, especially when compared to the results generated using the MC model MC for the same R_i value. This improvement is

particularly evident in the initial stiffness (slope) of the shear-displacement response, the mobilization of shear strength, and the representation of softening behaviour.

The Figure 5 presents the shear force–displacement response obtained from numerical simulations using the HS with an interface scaling factor of 0.75.

The stiffness of the interface is not explicitly defined in PLAXIS; rather, it is implicitly governed by the stiffness of the adjacent soil elements and the numerical formulation of the interface elements. A notable strength of the HS model is its capability to simulate non-linear stiffness degradation with strain and stress path dependency, making it inherently more suited to capturing the real behaviour of soil-structure interfaces under loading.

The experimental curves exhibit a rapid initial increase in shear force followed by a distinct peak and gradual softening. While the HS model offers improved representation of stiffness characteristics, particularly under 300 kPa and 400 kPa stress conditions, it is also limited in

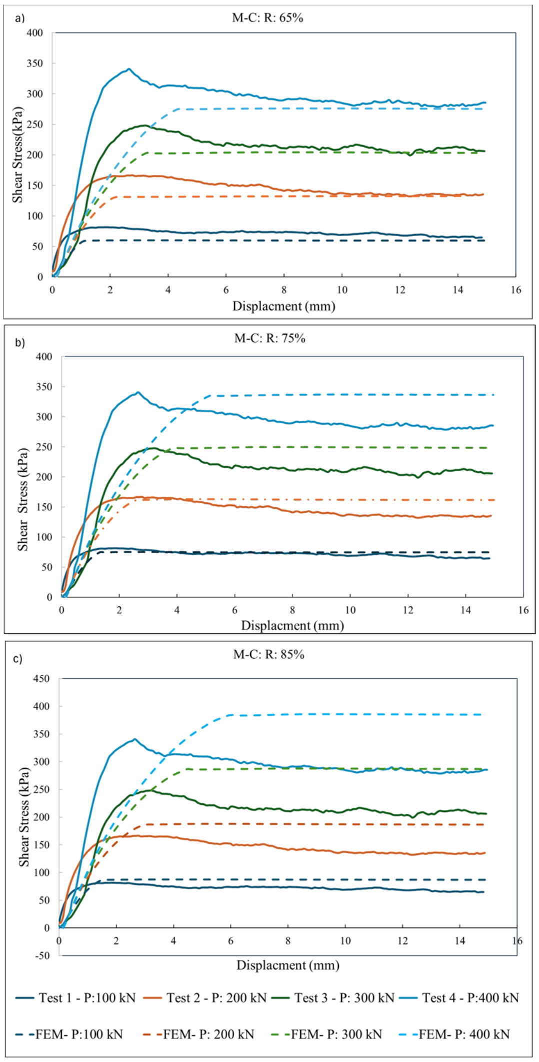

Figure 4. Shear force–displacement response (MC) in different R_i values, via experimental results.

The slopes of the ascending branches in the FEA results closely follow the experimental trends, indicating that the HS model more accurately reflects the stiffness evolution of the interface as displacement increases. This is a critical improvement because the slope (i.e., tangent stiffness) governs not only the stress-displacement prediction but also energy dissipation, which is essential in both static and seismic interface behaviour assessments.

This suggests that the HS model, when supplemented with carefully tuned interface parameters, can closely replicate the mobilized shear capacities of the system under high confinement. The ability to capture strain hardening followed by softening, a limitation in MC, is particularly beneficial in reproducing the real response of geotechnical interfaces, especially where dilation and stiffness degradation occur.

The HS formulation, which incorporates both stress-dependent stiffness and plastic strain accumulation, aligns more closely with observed experimental behaviour. These advantages underscore the HS model's suitability for advanced modelling of soil-structure interaction where interface behaviour under varying normal loads plays a crucial role.

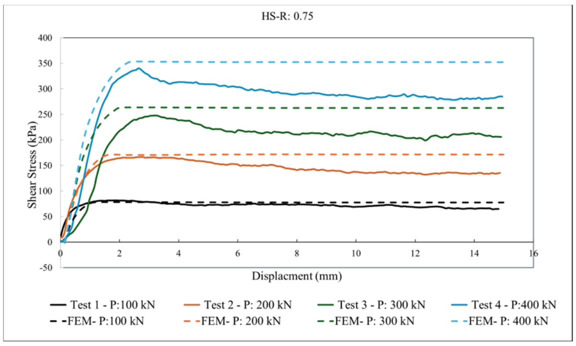

Figure 5. Shear force–displacement response (HS), via experimental results.

Since shear strength scaling approach, provide more accurate results, a sensitivity analysis was conducted to evaluate the influence of the interface scaling factor Ri on the accuracy of the simulation results. The comparative graph between HS and MC are demonstrated in Figure 6 to provide the predictive capability of both constitutive frameworks. Notably, HS demonstrates a closer alignment with the experimental data across all scaling factors, particularly at higher normal stress (300–400 kPa). In contrast, MC exhibits a more linear trend with uniform spacing between scaling factor curves, which reflects its simplified elastic–perfectly plastic nature and lack stress-dependent stiffness features.

As the scaling factor increases from 45% to 85%, both models predict lower shear forces, as expected. However, the progression is more realistic and curved in HS. The experimental results mostly fall within the range defined by R_i = 65% to 85% in both models, but HS provides a better fit in magnitude and non-linearity trend. It is evident that the relationship between shear stress and normal stress is predominantly linear and differs slightly from the trend observed between shear stress and deformation which is non-linear.

4.3 *Stiffness approach*

An alternative approach was adopted to simulate the interface behaviour based on the normal and tangential stiffness parameters, K_n and K_t, respectively.

To estimate interface stiffness parameters, the initial linear portion of the force–displacement curve is used, since it represents the elastic behaviour of the interface. Stiffness, as a material property, is defined as the ratio of force to displacement at small strains, where the response remains linear and reversible (Eq. 4). At this stage, nonlinear effects such as slip, dilation, or particle crushing have not yet been activated, allowing for a reliable estimation of the interface's elastic stiffness. Therefore, the initial slope of the curve provides a meaningful approximation of K_n and K_t, which are critical inputs for numerical modeling.

$$K = \frac{\Delta \text{Force}}{\Delta \text{ Displacement}} \tag{4}$$

The normal stiffness K_n is calculated as $K_n = E_{\text{oed}}/t_i$, where E_{oed} (initial value: 25,000 kPa) is the one-dimensional compression modulus of the interface and t_i is the interface thickness. Similarly, the shear stiffness K_s is given by $K_s = G_i/t_i$, with G_i representing the shear modulus of the interface material. These stiffness parameters govern the elastic response of the interface prior to yielding.

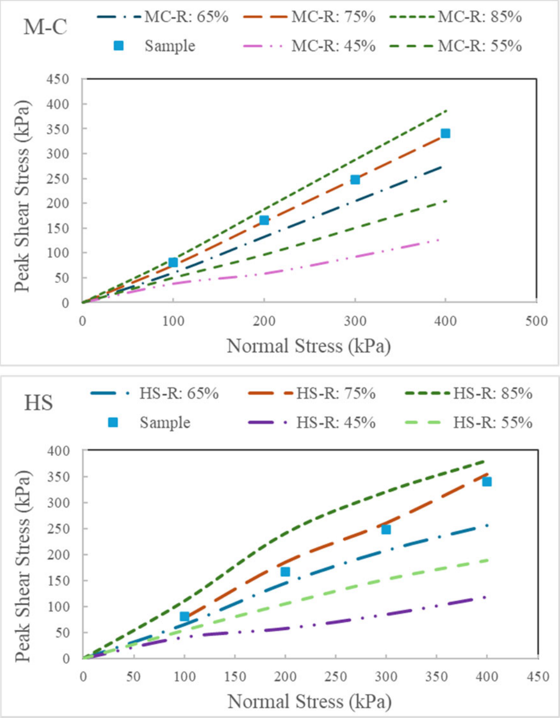

Figure 6. Effect of the interface scaling factor R_i on interface shear strength for MC and HS.

Test No. 3 (normal stress of 300 kPa) was selected as the reference case for validating this method. Parametric analyses revealed that variations in K_t significantly influenced the shear force–displacement response, whereas changes in K_n had comparatively minimal impact. This observation highlights the dominant role of tangential stiffness in controlling the mobilized shear resistance along the interface, particularly under conditions where slip or relative displacement governs the deformation mechanism.

Despite extensive calibration of the normal and tangential stiffness parameters, the numerical simulations failed to replicate the shear force–displacement response observed in the experimental results (Figure 7). The experimental curve is characterized by an initial increase in shear force, reaching a approximately 250 kPa at a displacement of around 2.8 mm, followed by a pronounced stabilization near 200 kPa. In contrast, all numerical results exhibited a monotonically increasing shear force without any indication of peak strength or softening behaviour, even under significantly reduced K_t values (e.g., 0.05). This discrepancy indicates that the divergence cannot be attributed solely to the choice of stiffness magnitudes.

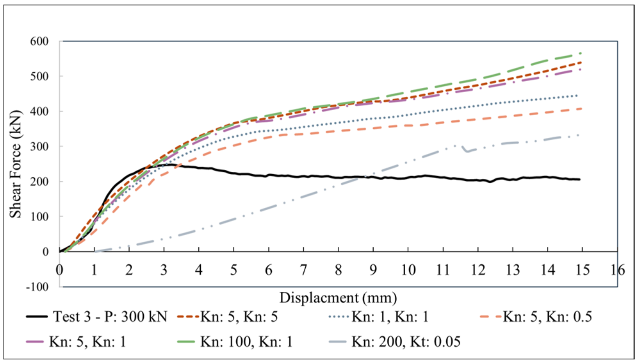

Figure 7. Shear Force–displacement response (Stiffness approach), via experimental results.

5 CONCLUSIONS

The primary aim of this study was to examine the influence of sand, and more generally soil characteristics, on the behavior of the soil–structure interface.

In results show that, the variation of R_i significantly affects both the peak and residual shear force predictions in Mohr-Coulomb-based numerical modeling. R_i of 65% provides a conservative estimate but under predict the shear strength values of the interface. R_i = 75% offers a reasonable balance between safety and realism, particularly when combined with calibrated stiffness parameters, yielding a close match in peak forces and improved agreement in the displacement range. R_i = 85%, while optimal for capturing peak shear strength, tends to overestimate residual capacity due to the model's inability to simulate strain softening. These findings underline the importance of careful calibration of R_i and consideration of constitutive model limitations when interpreting or relying on FEM results, especially for interface-dominated problems.

To accurately reproduce this behaviour, the numerical model must incorporate an elasto-plastic interface formulation, such as a HS model with appropriate strength parameters and, ideally, a displacement-dependent softening law.

When using the MC model, the constant and relatively low stiffness of the surrounding soil led to an underestimation of interface stiffness, particularly at small strain levels. In contrast, the HS model, with its stress-dependent stiffness and non-linear strain behavior, provides a more realistic representation of soil response. This, in turn, enhances the accuracy of the interface simulation, resulting in shear–displacement curves that more closely replicate experimental trends.

The numerical simulations incorporating the shear strength scaling technique demonstrated also a strong ability to replicate the elastic stiffness observed in the early stages of the direct shear tests. Specifically, in simulations with the HS model, within the low deformation range, up to approximately 2 mm of displacement, the numerical results closely followed the experimental curves in both slope and magnitude. Since this early phase is governed primarily by elastic deformations prior to shear stress, the model's capacity to accurately simulate this range confirms the suitability of using scaling -based calibration for the pre-peak elastic behaviour in practical interface modeling.

Despite these advancements, the stiffness-based approach, relying solely on elastic normal and tangential stiffness parameters (K_n and K_t), was insufficient to replicate experimental trends. This limitation underscores the necessity of adopting elasto-plastic interface formulations to account for irreversible deformation and interface degradation mechanisms observed in physical tests.

Beside the choice adequate constitutive models, the calibration of interface stiffness parameters still requires further investigation to enhance simulation reliability. In particular, variations in shear stiffness K_t have shown a more pronounced impact on the shear force–displacement response than changes in normal stiffness K_n. This is likely due to the fact that shear deformation governs the mobilization of interface strength during sliding, making K_t a critical parameter for capturing both the initial stiffness and post-peak behaviour. In contrast, K_n primarily affects the normal contact pressure and has a relatively smaller effect on the tangential load transfer.

Therefore, achieving accurate representation of interface behaviour depends strongly on the proper selection and calibration of Kt, especially in cases involving significant shear displacement. Nevertheless, integrating both approaches may offer a more comprehensive perspective and should be investigated further in future research.

The findings in this study could be applications to real-world such as shotcrete lining performance in NATM tunnels, or soil-structure interaction in retaining walls with ample cautions due to limited experimental results and soil types as well as use of various material models that can capture the behavior of soil-shotcrete interface, especially post peak shear strength and dilation behavior.

In this study sand was selected to provide a simple, well-characterized medium for model calibration and to isolate the mechanical behavior of the constitutive models. The results should therefore be viewed as a methodological benchmark, with future applications extending to more representative tunnel ground materials such as cohesive or mixed soils.

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