

A numerical framework for modelling the soil-structure-interface using advanced constitutive models

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ABSTRACT: The behaviour of the interface plays a crucial role in soil-structure interaction (SSI) under shear deformation and is, therefore, essential for the numerical analysis of geotechnical applications involving structures. To capture this interaction realistically, many widely adopted approaches in the literature use the constitutive model of the surrounding soil to describe the interface behaviour. However, these approaches face two significant limitations: they assume a sticking contact, failing thus to capture the stick-slip transition observed in experiments. In addition, they focus only on the tangential interface behaviour, neglecting the volumetric strain in the shear zone and its influence on the normal stress when confined by the surrounding soil.

This contribution addresses these shortcomings by introducing a shear strain evolution relationship that can accurately reproduce the stick-slip transition, considering the roughness of the surface. Additionally, the interface model couples volumetric and shear behaviour in the interface, capturing stress or changes in relative density – jamming or unjamming – when the surrounding soil impedes the volumetric strains. Unlike conventional approaches that assume a free volume change, the proposed framework calculates the evolution of normal stresses from strains within and adjacent to the interface using the constitutive equations of the surrounding soil and vice versa. Validation against experimental monotonic interface shear tests on dry granular soils using a hypoplastic constitutive model demonstrates the framework's capability to reproduce the experimentally measured shear stress, soil-structure relative displacements, and variations in normal stress. The proposed framework significantly enhances the predictive capacity of interface models, providing a reliable numerical tool for analyzing complex SSIs under shear deformation in geotechnical applications.

KEYWORDS: Finite element modelling, granular materials, interfaces, soil–structure interaction (SSI).

1 INTRODUCTION

Shear deformation in soil–structure interfaces involves mutual influence in the transfer of shear and normal forces. The resulting stresses and deformations in both the structure and the soil do not only depend on their individual mechanical properties, but are also influenced by the behaviour of the interface. Various experimental studies have been conducted to better understand the interface behaviour. These studies demonstrate that the development of contact stresses is affected by several factors, including surface roughness, grain shape and mineralogy, particle size, density, confining pressure, and loading history, similar to the behaviour of the adjacent soil (Boulon & Nova, 1990; DeJong & Westgate, 2009; Potyondy, 1961).

Surface-based contact algorithms are typically employed in boundary-value problems involving soil–structure interaction (SSI) under large relative displacements. While complex constitutive models are commonly used to represent the surrounding soil – capturing variations in stiffness and strength due to changes in stress level and density (Arnold & Herle, 2006) – modelling the contact interface itself is often highly simplified. Many approaches either neglect shear transmission entirely – assuming frictionless contact – or apply a linear elastic–perfectly plastic friction model based on the Coulomb failure criterion. However, these approaches are limited: the former transfer only normal stresses, while the latter fails to capture the state-dependent behaviour of the soil in the shear zone.

More advanced contact models have been proposed to overcome these limitations, such as those based on hypoplasticity, incorporating a state-dependent friction angle (Arnold & Herle, 2006; Staubach et al., 2022; Stutz, 2016). These models enable the consideration of essential features of soil behaviour at the interface and avoid additional interface parameters calibration by linking contact behaviour directly to the surrounding soil properties and two additional parameters: the surface roughness, κ , and the initial shear zone thickness, d_{s0} .

In these models, sliding, defined as relative displacement without a corresponding increase in the shear stress, occurs only after the critical state of the interface soil is reached. Consequently, modelling interfaces with varying wall friction angles or surface roughness requires adjusting the critical friction angle of the soil model itself, which control the shear strength. This raises a conceptual issue: changes in surface roughness should not alter the constitutive behaviour of the adjacent soil.

Experimental evidence supports this concern. For very rough surfaces, sliding occurs at or beyond the critical state. However, sliding may begin for smooth interfaces before the soil reaches a critical state (Uesugi & Kishida, 1986). Existing contact models do not differentiate between localisation within the soil (as for rough interfaces) and along the interface (as for smooth interfaces), and cannot capture this fundamental distinction.

Additionally, considering the interface behaviour in the normal direction, the shearing of granular soil under drained conditions leads to volumetric changes in the shear zone. Due to the mechanical confinement imposed by the structure and the surrounding soil, such volumetric changes are often partially constrained and thus lead to an increase in the normal stress acting at the interface. This process which is linked to increase of grain-to-grain forces and elastic energy of the skeleton is known as jamming. Conversely, in soils in a contractive state, unjamming is associated with a release of elastic energy, which may result in loosening of the soil or a reduction in the mean effective stress. At the soil–structure interface, the partial suppression of volume reduction in this case leads to a decrease in the normal stress (Rebstock, 2011). The existing numerical implementations of the advanced contact models mentioned previously cannot capture the changes in normal stress caused by either jamming or unjamming effects in boundary value problems.

This paper proposes an extension of existing contact formulations to model interface behaviour using the same model parameters for the soil inside and outside the shear zone. Furthermore, the proposed approach accounts for the normal stress component arising from volumetric constraints, thus

enabling the simulation of both jamming and unjamming phenomena. This contribution focuses on describing the underlying mechanisms of localisation and the conceptual framework of the proposed model extension.

2 INTERFACE MODEL

2.1 Tangential interface behaviour

Experimental studies have shown that two distinct limit states can occur at soil–structure interfaces during monotonic shearing, depending on factors such as relative density, stress state, and, most significantly, surface roughness (Potyondy, 1961; Uesugi and Kishida, 1986). For rough interfaces, the soil reaches a critical state within the shear zone. Beyond this point, the shear stress remains constant despite ongoing deformation, i.e. $\dot{\tau}_{xz} \approx 0$ while $\dot{\gamma}_{xz} \neq 0$. In contrast, smooth interfaces prevent the development of a critical state of the soil within the shear zone. Instead, ideal sliding occurs due to particle movement along the structure, leading to a vanishing shear strain rate $\dot{\gamma}_{xz} \approx 0$, and, consequently, no further mobilisation of shear resistance, $\dot{\tau}_{xz} \approx 0$.

To represent this behaviour, Niebler et al. (2025) proposed a framework that incorporates the relative motion at the contact interface, which is primarily influenced by surface roughness. As schematically illustrated in Figure 1, the model distinguishes two contact phases and the smooth transition between them:

- sticking, where the structural velocity v_{str} equals the soil velocity at the interface v_{s_0} and no relative movement occurs; $v_{str} = v_{s_0} = \frac{\delta u_{str}}{\delta t}$, see Figure 1 (a)

- a transition phase with partial relative motion; $|v_{str}| > 0 \wedge |v_{s_0}| > 0 \wedge |v_{str}| > |v_{s_0}|$, and
- sliding, where the soil ceases to deform and only the structure continues to move; $|v_{str}| > 0 \wedge |v_{s_0}| = 0$, see Figure 1 (b).

The transition between these phases is described by a smooth relationship between the actual shear strain rate $\dot{\gamma}_{xz}$ and a theoretical strain rate under full sticking conditions $\dot{\gamma}_{xz,st}$:

$$\dot{\gamma}_{xz} = \frac{\gamma_{sliding}^2 \cdot \dot{\gamma}_{xz,st}}{(\gamma_{sliding} + \gamma_{xz,st})^2} \quad (1)$$

This relationship is illustrated in Figure 1 (c), which shows how the actual shear strain γ_{xz} evolves as a function of the strain under full sticking condition $\gamma_{xz,st}$. The strain rate under sticking $\dot{\gamma}_{xz,st}$ is defined as:

$$\dot{\gamma}_{xz,st} = \frac{1}{2} \cdot \frac{v_{str}}{d_s} \quad (2)$$

using the structural velocity v_{str} and the current thickness of the shear zone d_s . The shear strain γ_{xz} approaches the limiting strain $\gamma_{sliding}$ as the interface transitions toward ideal sliding. The limiting strain determines this transition behaviour and depends on the interface roughness. It can either be obtained experimentally or derived from a calibrated constitutive model, as illustrated by Niebler et al. (2025).

2.2 Normal interface behaviour

In order to model the soil-structure-interface and consider the jamming and unjamming effects, some approaches model the

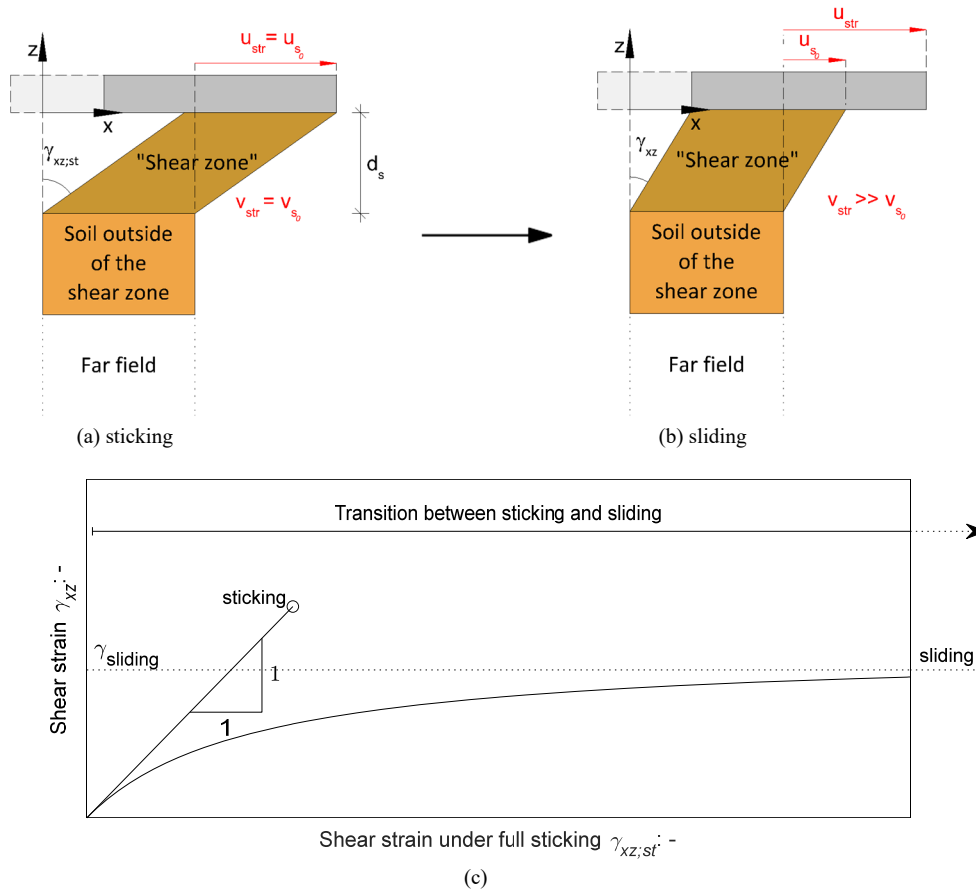


Figure 1. Shear strain increment in the shear zone for (a) sticking contact and (b) sliding at the soil and structure contact and (c) continuous transition between contact sticking and sliding according to Equation (1).

shear zone as a continuum or interface element which provides a consistent description of the shear behavior and the development of normal stress. However, these approaches are restricted to small sliding displacements between the two interacting surfaces (Dassault Systèmes, 2017; Griffiths, 1988; Rebstock, 2011). Therefore, these formulations can cause severe element distortions and consequent numerical instabilities when combined with large deformations.

To accommodate finite sliding – i.e., arbitrary relative separation – the soil–structure interaction is instead modelled as a surface contact, implemented as a zero-thickness element following Arnold & Herle, 2006; Staubach et al., 2022; Stutz, 2016. In this framework, the constitutive model considering the far-field soil behaviour also describes the shear behaviour within the interfacial shear zone, while normal stresses are prescribed through dedicated contact formulations (such as “soft” or “hard” normal-direction contacts) rather than being derived from the constitutive model of the soil. The normal stress σ_n thus serves as an input, enabling only the estimation of interface stress and strain under the assumption of free volumetric change.

Consequently, these models cannot capture the experimentally observed increase in normal stress due to partially prevented dilation (jamming) or the decrease due to constrained contraction (unjamming) within the shear zone (Rebstock, 2011).

To address this limitation, we couple the normal strain ε_n of the soil in the shear zone with the stiffness of the soil at the interface, allowing σ_n and the tangential stresses τ_{xz} and τ_{yz} to evolve consistently with ε_n , γ_{xz} and γ_{yz} as prescribed by the far-field soil constitutive model. This approach eliminates the free volume change assumption of existing approaches, enabling the consideration of jamming and unjamming effects in the interface behaviour. The normal strain in the shear zone first causes an overclosure, respectively a gap between the soil and the structure. To eliminate the overclosure / gap, the normal stress in the shear zone must be increased or decrease.

Figure 2 schematically illustrates this process. Assuming a rigid structure and surrounding soil with its “stiffness” k (here depicted with the spring support / foundation k), an incremental tangential displacement Δu_{str} is applied (Figure 2 (a) and (b)). The normal stress at the interface is initially computed assuming a free volume change (i.e. $k = 0$), leading to CNL (Constant Normal Load, $\dot{\sigma}_n = 0$) conditions (Figure 2 (c)) and to a change in thickness of the shear zone. However, for dilative soil, this underestimates σ_n since the adjacent soil’s “stiffness” k introduces an additional stress rate $\dot{\sigma}_n = \dot{\varepsilon}_n \cdot d_s \cdot k$ based on the normal strain rate $\dot{\varepsilon}_n$ and the shear zone thickness d_s .

If σ_n cannot satisfy the Neumann boundary conditions (Figure 2 (d)), the adjacent soil deforms further in the subsequent iteration increment, increasing σ_n iteratively until equilibrium is reached (Figure 2 (e)). The same mechanism applies to contractive soils, where unjamming reduces σ_n in subsequent steps. Thus, jamming and unjamming are consistently captured within the interface by iteratively updating σ_n based on the volumetric behaviour of the soil in the shear zone, the boundary conditions and the stiffness of the surrounding soil and structure.

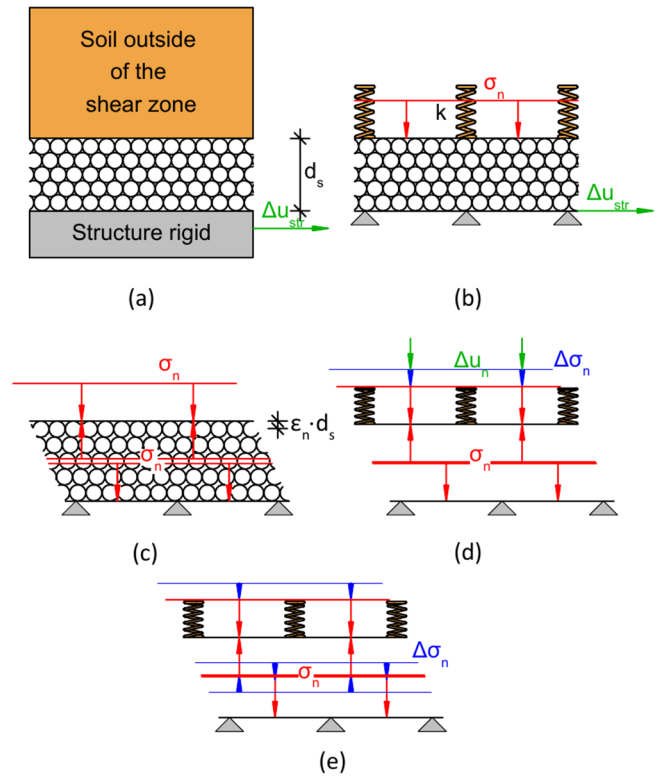


Figure 2. Schematic illustration of the subsequent calculation increments to consider un- and jamming. (Note: For the sake of simplicity only the contact stresses in normal direction are depicted).

2.3 Implementation

The presented framework is implemented within the ABAQUS user subroutine UINTER, which allows the user to define the mechanical behaviour of the interface in both the normal and the tangential directions (Dassault Systèmes, 2017). This capability eliminates the common assumption of unconstrained volumetric deformation typically made in existing approaches which assume free volume change, i.e. CNL (Constant Normal Load, $\dot{\sigma}_n = 0$). The input to the subroutine consists of displacement increments, which are converted into a strain rate tensor to compute the corresponding stress rate tensor using the constitutive model of the surrounding soil. In addition to the stress rate tensor, the subroutine returns the interface stiffness matrix derived from the Jacobian matrix of the constitutive model.

The implementation follows the adapted scheme proposed by Niebler et al. (2025), which is based on the approach of Stutz (2016). Therefore, and for the sake of brevity, the specific implementation details are not further elaborated in this contribution. For more information, the reader is referred to the cited literature.

3 VALIDATION

Using the proposed concept and the derived model extension, the evolution of normal and shear stresses at the interface can be modelled as a function of the tangential displacement, incorporating the constitutive behaviour of the adjacent soil. To validate the potential of the extended interface model, the interface shear experiments by Fakharian & Vafaei (2021) are recalculated in this section as a quasi-element test. The experiments were conducted with Firuzkuh sand ($d_{50} = 0.71 \text{ mm}$) in a simple shear interface apparatus, schematically depicted in Figure 3 (a). In this setup, the soil specimen is laterally confined by 2 mm-thick Teflon-coated rings with a

rectangular base shape and internal dimensions of 10 cm × 10 cm. The stiffness of the confining spring k , which limits the specimen's volume change, was varied in the experiments. Figure 3 (b) shows the corresponding numerical model, where the interaction between the soil and structure surfaces represents the interface and is modelled using the proposed approach. For simplicity, the numerical model is idealised as a one-dimensional soil column by coupling the degrees of freedom at the lateral boundaries of the soil. The initial height of the model $h_{0,sim} = 12.9 \text{ mm}$ corresponds to the experimental setup height $h_0 = 20 \text{ mm}$, reduced by the initial shear zone thickness d_{s0} .

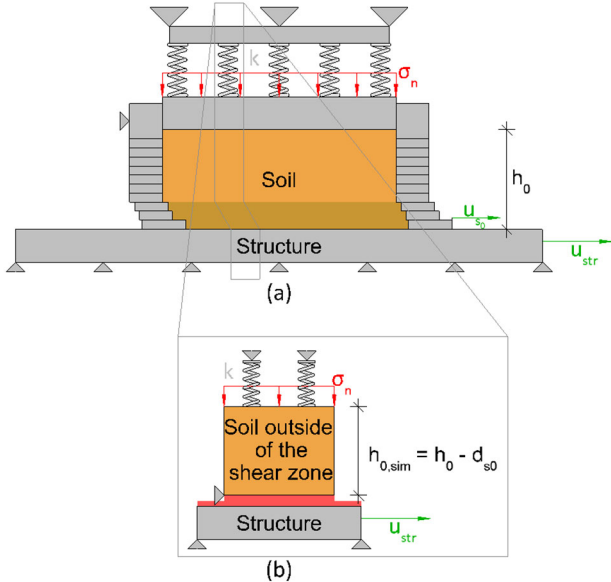


Figure 3. Schematic sketch of an interface shear test apparatus (Fakharian & Vafaei, 2021) and the numerical quasi-element test used for the recalculation.

The structure, i.e. the steel plate below the soil sample, is modelled as rigid. The hypoplastic constitutive model by von Wolffersdorff (1996) is applied to describe the soil behaviour in the shear-zone and the attached soil. Since the hypoplastic material parameters for Firuzkuh sand are not available in the literature, the hypoplastic parameters for Karlsruhe sand, featuring a similar grain size distribution, are used, see Table 1.

Table 1. Constitutive parameters of Karlsruhe sand used for the recalculation. Parameters from Rebstock (2011).

φ_c [°]	h_s [MP]	n	e_{d0}	e_{c0}	e_{i0}	α	β
30	5800	0.28	0.53	0.84	1.00	0.13	1.05

The initial shear zone thickness d_{s0} is assumed to be ten times the mean grain diameter of the sand d_{50} , i.e. $d_{s0} = 10 \cdot d_{50} = 7.1 \text{ mm}$, as it was not measured in the experiments. The limit strain $\gamma_{sliding}$ required to describe the tangential contact behaviour is determined following Niebler et al. (2025) based on the normalised surface roughness R_n , defined as $R_n = \frac{R_{max}(L_G)}{d_{50}}$ by Uesugi and Kishida (1986). Here, R_{max} is the peak-to-valley distance over a length $L_G = d_{50}$ (Yoshimi and Kishida, 1981). For the surface used in the experiments, with $R_{max} = 32.7 \mu\text{m}$, the surface roughness κ is calculated following Gutjahr (2003):

$$\begin{aligned} \kappa &= 0.25 \cdot \log(R_n) + 1.05 \\ &= 0.25 \cdot \log\left(\frac{32.7 \mu\text{m}}{0.71 \text{ mm}}\right) + 1.05 = 0.725 \leq 1 \end{aligned} \quad (3)$$

Based on this, the limit strain $\gamma_{sliding}$ can be determined using the relationship $\gamma_{sliding} = \gamma_{sliding,rough} \cdot \kappa^{f_\kappa} = 2 \cdot 0.725^7 = 0.21$ derived by Niebler et al. (2025) based on the evaluation of experimental results from the literature.

Figure 4 compares the measured and predicted normal stress σ_n and shear stress τ_{xz} as a function of the tangential displacement u_{str} and relative displacement between structure and soil $u_r = u_{str} - u_{s0}$, where u_{s0} is the deformation of the soil directly adjacent to the structure, see Figure 3 (a). Simulations and Experiments are compared for different confining stiffness values k , tested with dense sand specimens with an initial relative density of 85 % under an initial normal stress of $\sigma_{n0} = 100 \text{ kPa}$. The agreement between experimental and numerical results is satisfactory. The model captures the relative displacement u_r at the end of the test and demonstrates that the interface model via Equation (1) realistically reproduces the shear deformation within the shear zone and thus the relative movement between soil and structure.

In both the experiments and the simulations, the relative displacement is independent of the stiffness of the confining spring k . For a further and more detailed validation of the tangential interface behaviour, one is referred to Niebler et al. (2025). The numerical model also reproduces the qualitative evolution of the normal stress σ_n , which features initial unjamming due to the partially suppressed contractive behaviour, followed by jamming caused by partially suppressed dilatancy (see Figure 4 (c)). The initial reduction in normal stress from $\sigma_{n0} = 100 \text{ kPa}$ is more pronounced with increasing normal stiffness k . Similarly, the magnitude of stress increase due to jamming is proportional to k .

However, the model underestimates the initial reduction in normal stress, leading to an overestimation of the initial shear stress τ_{xz} , see Figure 4 (a) and (c). One possible explanation for this discrepancy is that the described interface shear test is not exactly a true element test. As a result, the numerical reproduction of the test – based on the assumption of a homogeneous stress and strain distribution in the soil outside the shear zone – constitutes a strong simplification. The variation in normal stress acting on the interface also affects the mobilised shear stress τ_{xz} , resulting in higher shear stresses at the end of shearing as the confining stiffness k increases.

To illustrate the influence of jamming and unjamming to the interface behaviour, additional simulations are depicted in black in Figure 4, representing results independent from the normal stiffness obtained when these interaction is neglected. As shown in Figure 4 (c), the normal stress acting on the interface remains constant, indicating that the evolution of normal stress is independent of the confining stiffness. Consequently, the mobilised shear stress plotted in Figure 4 (a) and (b) also becomes independent of stiffness and underestimates the actual values due to the absence of jamming effects. These experiments and the numerical variation without considering the interface normal behaviour highlight that neglecting jamming and unjamming, i.e. assuming $\dot{\sigma}_n = 0$,

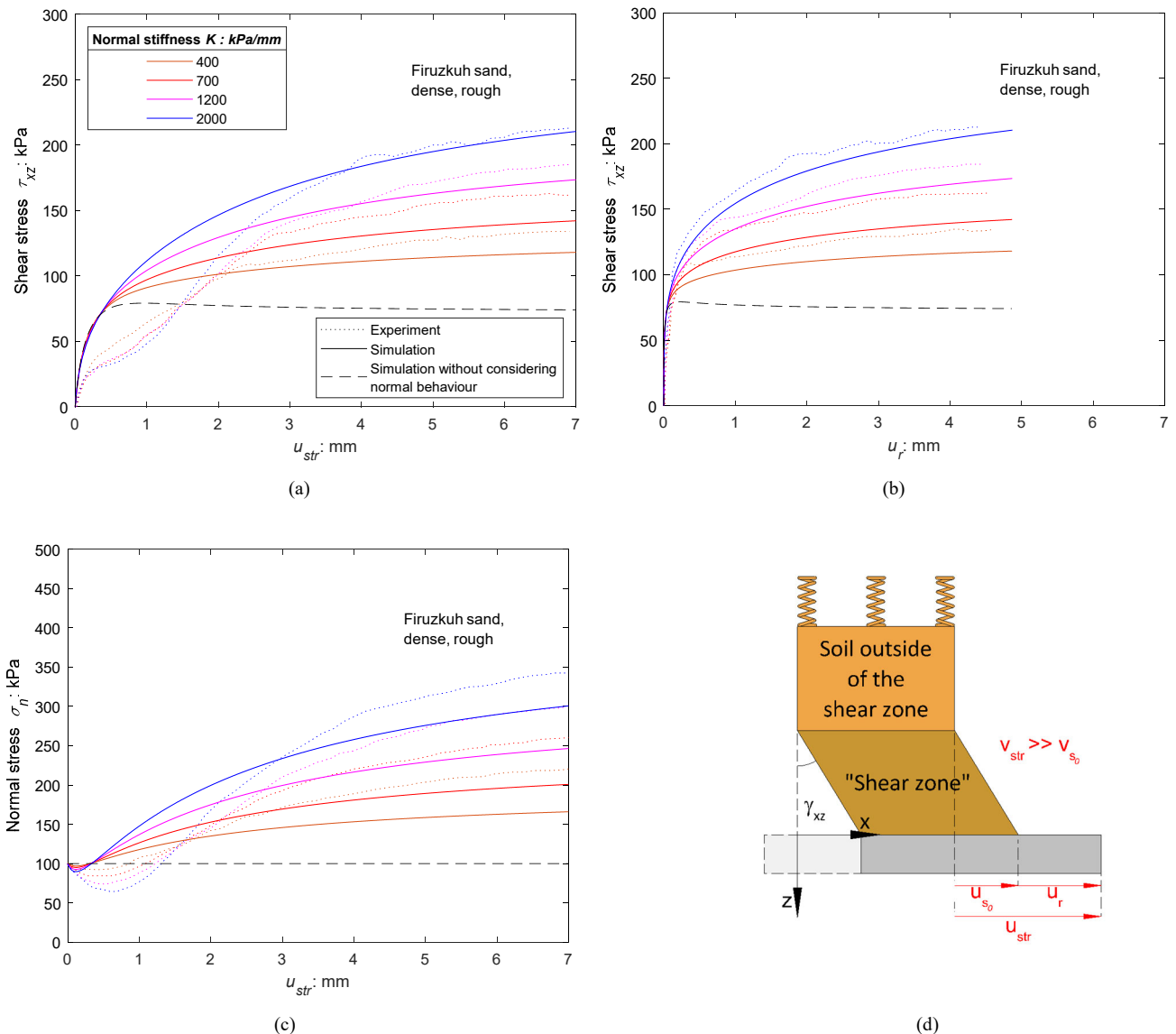


Figure 4. Experimental (dotted-line) and numerical results (solid line) of the interface simple shear test under CNS-conditions with dense Firuzkuh sand and $\sigma_{n0} = 100$ kPa: (a) $u_{str} - \tau_{xz}$, (b) $u_r - \tau_{xz}$ and (c) $u_{str} - \sigma_n$ -curves. (Experimental data from Fakharian & Vafaei (2021)). The black line represents the numerical results if the influence of the interface normal behaviour is neglected and is therefore independent of the normal stiffness. (d) Schematic illustration of the displacements occurring at the soil-structure interface.

leads to a significant underestimation of the normal and shear stresses acting at the interface and thus highlights the importance of accounting for these effects.

4 CONCLUSION AND OUTLOOK

This contribution has presented an extension to existing contact models that enables numerical simulation of tangential and normal behaviour at the soil-structure interface as observed in experiments. In the tangential direction, the proposed framework captures the two fundamental localisation mechanisms that influence the maximum transferable shear stress – either slip along the interface or attain the soil’s critical state within the shear zone. In the normal direction, it reproduces both jamming and unjamming effects, i.e. the associated decrease or increase in the mobilised shear strength due to constrained volumetric changes.

The interface response is driven solely by the constitutive model and parameters of the neighbour soil, and two additional interface parameters: the initial shear-zone thickness d_{s0} and

the limit strain $\gamma_{sliding}$. The initial shear zone thickness d_{s0} could be easily estimated without further experiments as it has a physical meaning. The limit strain $\gamma_{sliding}$ can either be obtained experimentally or derived from a calibrated constitutive model. Consequently, the framework accurately represents soil-structure interaction without requiring further calibration of interface-specific parameters or dedicated interface tests and allows the use of any constitutive model to describe the behaviour of the soil within the shear zone.

For validation, simple-shear interface tests reported in the literature were simulated, in which the relative displacement between soil and structure and the evolution of shear and normal stresses at the interface were measured. The comparison of experimental and numerical results demonstrates that the complex interface behaviour can be realistically captured with the proposed approach. However, the comparison with published interface tests represents a first validation step. A comprehensive validation and an investigation of the influence of additional state variables, such as grain shape, grain size, and interface dilatancy behaviour, will require a systematic

experimental database. To this end, extensive simple-shear interface tests with Fujian Sand are currently being conducted at our Institute as part of the DFG-funded project “TUNLIQ: Soil–structure interaction of shallow tunnels in liquefiable soils during strong earthquakes.” Furthermore, the present numerical implementation is limited to one-dimensional contact deformations. To enable the application of this contact model extension to a broader range of geotechnical boundary-value problems, future work is needed in order to generalise the framework to arbitrary load paths, including cyclic loading and two-dimensional contact deformations.

In future research, the proposed interface model will be applied to boundary value problems where interface behaviour is critical. A particular focus is on investigating the influence of competing shear zones. In such boundary value problems, a shear zone may develop directly at the soil–structure interface, which the current model captures. However, shear zones can also form within the adjacent soil, near the interface. Capturing this shift in the localisation of the shear-zone from the interface into the soil is challenging in finite element simulations, as the occurrence and location of strain and stress concentrations are strongly influenced by the discretisation and mesh density. Therefore, further research is needed to realistically model this transition of the shear zone. One promising approach is the use of regularisation techniques, which enable mesh-independent simulation of shear band formation and evolution (Maier, 2002; Slominski, 2007).

Additional, large-scale experiments have revealed significant material transport within the shear zone, characterised by distinct particle movements (Rebstock, 2011). Discrete element methods (DEM) are particularly well-suited to capture such phenomena, as they explicitly model individual particle interactions and their trajectories. Consequently, applying DEM to represent the shear zone is a logical extension of the current work. This motivates further research into the coupling of DEM and FEM, allowing for a more comprehensive and physically realistic simulation of interface behaviour that incorporates both continuum and discrete representations of soil deformation.

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