

Calibration of a discrete element model for dry sand and fine grain soil for use in vehicle mobility applications

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Abstract

In this paper, a procedure for calibration of a discrete element (DE) computational soil model for use in prediction of ground vehicle mobility is presented. The procedure relies on using the results of two physical soil experiments: (1) hydrostatic compression; and (2) unconfined shear strength under different levels of normal stress and normal pre-stress. The hydrostatic compression test is used to calibrate the DE bulk density, plastic strain, and elastic strain as a function of hydrostatic stress. The unconfined shear strength test is used to calibrate the DE inter-particle friction coefficient and adhesion stress as a function of the soil plastic strain. The test devices and experimental test procedures are presented in the paper. The DE model calibration procedure is demonstrated in the paper using two types of soil: dry sand-silt and dry silt-clay. The model is then validated by comparing the model's results to experimental confined shear strength test results, pressure - sinkage results for circular disks and cones, as well as single tire drawbar-pull results for those two types of soil.

1 Introduction

Ground vehicles are often required to operate on off-road soft soil terrains in many applications, including, mining, agricultural, construction, military, and recreational applications. Those vehicles include: cars, trucks, motorcycles, ATVs (all-terrain vehicles), snow mobiles, tracked vehicles, earth moving equipment (e.g. bulldozers and backhoes), airplanes (during landing/take-off on unpaved or snow covered runways), and walking robots. The vehicles can be supported and/or propelled on the soil by wheels, tracks, legs and/or skis. In addition, other parts of the vehicle can also interact with the soil such as the vehicle underbody and implements such as tines, blades, and buckets. Soft soils include any combination of gravel, sand, silt, clay, organic matter, and snow with any level of water content. In order to accurately predict the dynamic response (including mobility, stability, and energy consumption) of those vehicles, while driving on soft soil terrains, an accurate coupled model of the vehicles and the soil is needed. The vehicles are generally modeled as multibody systems consisting of rigid and flexible bodies connected using joints (spherical, revolute, and prismatic), contact surfaces, and actuators/motors with control systems [1-3]. In this paper we focus on the soil models. Specifically, the paper presents a procedure for calibration of a discrete element (DE) computational soil model for use in predicting ground vehicle mobility.

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Ideally, a mechanical model of the soil must account for the following effects:

- Soil 3D flow.
- Soil plastic and elastic deformation along with change in bulk density due to the applied stresses.
- Soil shear strength due to internal friction and cohesion. This includes the change of soil shear strength as a function of soil compaction state (plastic deformation). When soil compaction increases the soil cohesion increases and the internal friction also generally increases. This is due to the fact that when soil is compacted soil particles are more tightly in contact with each other.
- Internal soil damping and viscosity.
- Reduction of soil compaction (relaxation) and loss of soil cohesion due to shear and tensile stresses.
- Friction and adhesion as a function of normal stress between the soil and the running gear.

Based on model fidelity, general-purpose soil models for soft-soil vehicle mobility applications range between two main types: One-dimensional (1-D) sinkage-slip models and full three-dimensional (3-D) models. 1-D sinkage-slip models use the normal stress and slip of a wheel or track segment to calculate the sinkage and shear stresses on the tire/track segment. A Bekker-Wong type empirical relation is used to relate the normal stresses to the sinkage [4, 5] and a Janosi-Hanamoto [6] type empirical relation is used to relate the normal stress and slip to the traction shear stresses, and the normal stress and sinkage to the resistance shear stresses. Those shear stresses can in turn be used to calculate the tire/track tangential traction and motion resistance forces, respectively. The main degree-of-freedom in 1-D sinkage-slip models is the vertical deformation (change of height) of the terrain also called sinkage. Those models can be further divided based on the discretization of the terrain into:

- Single sinkage models. Those models use one sinkage value for a tire or track segment and don't keep track of the terrain height and stress state due to tire/track sinkage on the terrain. Many commercial multibody dynamics codes include this fast and simple terramechanics model.
- Height-field models. In those models the 2D terrain surface is discretized into cells with each cell having a height value. In addition, the state of soil stress of the cell can also be stored. The terrain can be discretized at the running gear-scale or at the tread-block scale.
- Height-field models with a soil flow sub-model [7-9] which allows soil to flow (increase the height) from cells which are currently under the running gear to neighboring cells. Those models can be classified as pseudo-3D models.

1-D sinkage-slip models are typically calibrated using circular plate soil sinkage tests and rotational Bevameter soil shear tests [4, 5].

3D soil models include finite element (FE) Lagrangian models, FE ALE (Arbitrary Lagrangian Eulerian) models, and Lagrangian particle-based models. In FE models, the soil deformation is modeled using the motion of the FE nodes which form a Lagrangian finite element mesh [10-18] along with an elasto-viscoplastic constitutive material model such as Drucker-Prager model [19] for modeling soil compaction and plastic deformation. The Lagrangian nature of the finite element mesh means that material which is inside one element remains inside that element since the mass of the element nodes is constant. This means that Lagrangian FE methods are inherently incapable of modeling material flow between elements. In addition, if the soil undergoes large deformation then, in order to avoid excessive element distortion, remeshing is required in FE models. The remeshing step is computationally expensive and degrades solution accuracy since the solution fields (including plastic and elastic deformations) need to be re-interpolated to the new mesh. Some FE studies use the Arbitrary Lagrangian Eulerian (ALE) formulation

[16, 20] which allows mass transfer between elements. However, even with ALE, effects such as soil bulldozing and separation/reattachment still require remeshing. In general, the range of accuracy of FE soil models is limited to small to moderate soil deformation.

In particle-based methods, discrete particles are used to model the soil with inter-particle forces used to model the soil mechanical behavior. In most particle models, a particle has a fixed mass (i.e. is indivisible) and represents a group of soil particles. Particle methods can naturally handle material flow, and surface break-up/merging. Their main disadvantage is their computational cost due to the large number of particles needed in practical vehicle mobility applications. Lagrangian particle methods include the Discrete Element Method (DEM) [3, 21-30], Smoothed Particle Hydrodynamics (SPH) [31-37], and Material Point Method (MPM) [35, 38-40]. SPH and MPM rely on a continuum mechanics formulation, and therefore, they require a continuum mechanics constitutive material model such as the Drucker-Prager model [19]. In the DEM inter-particle forces, including normal contact forces (deflection and/or velocity dependent), attraction forces, and tangential contact forces (friction and viscous) are used to model the soil mechanical behavior. 3D soil models used in ground vehicle mobility applications are typically calibrated using Bevameter, wheel on soil [39], shear cell [36, 37], and plate sinkage [36, 37] experimental tests.

The objective of this paper is to present a procedure to calibrate a high-fidelity DE soil model using small-scale terramechanics experiments for use in ground vehicle mobility simulations. The procedure is based on using the following two experiments:

- Hydrostatic compression. This experiment is used to calibrate the DE normal stress versus plastic strain, elastic strain, and bulk density.
- Unconfined shear. In this experiment the soil shear strength versus current applied normal stress, and normal pre-prestress is obtained. Using those experimental results, the DE plastic strain.

The calibrated model is then validated by comparing the model results to the following experiments:

- Confined shear cell: Soil shear strength versus current applied normal stress, and normal pre-prestress.
- Circular disk penetrometer: Pressure vs sinkage for different disc sizes and penetration speeds and soil pre-stress.
- Cone penetrometer: Pressure vs sinkage for different cone sizes and penetration speeds and soil pre-stress.
- Drawbar-pull of a pneumatic tire on soil.

The calibration procedure and the validation are performed for two types of dry soils: a clay-type fine grain (FG) soil and a coarse-grain sandy soil which is close to 2NS sand (2NS). The paper is organized as follows. In Section 2 an overview of the DE inter-particle force contact model is presented. In Section 3, description of the terramechanics experiments, device components, test procedure, and corresponding computational model is presented. In Section 4, the calibration procedure of the DE model using the hydrostatic compression and the unconfined shear tests is presented and demonstrated for the 2NS and FG soils. In Section 5, the validation of the DE models is presented by comparing the model's prediction for the following experiments: confined shear cell, disk penetrometer, cone penetrometer, and drawbar-pull of a tire on soil. Finally, concluding remarks are presented in Section 6.

2 Overview of the DE soil model

The DE soil model was presented in Refs. [3, 21, 41]. An overview of an improved model where model parameters are independent of particle size is presented here. In this DE model, the inter-particle forces between two particles in contact are divided into normal and tangential forces (Figure 1). In the figure, d is the interparticle penetration distance along the normal contact direction $\vec{n}$, $D_{contact}$ is the current contact diameter of the particle, and D_0 is the undeflected particle diameter. $D_{contact}$ is related to D_0 by:

$$D_{contact} = D_0 - d_{plastic} \quad (1)$$

where $d_{plastic}$ is the particle's plastic deformation. The normal force consists of three types of forces: adhesion, repulsion, and damping forces:

$$F_n = F_{adhesion} + F_{repulsion} + F_{damping} \quad (2)$$

The tangential force consists of two types of forces: friction and viscous forces:

$$F_t = F_{friction} + F_{viscous} \quad (3)$$

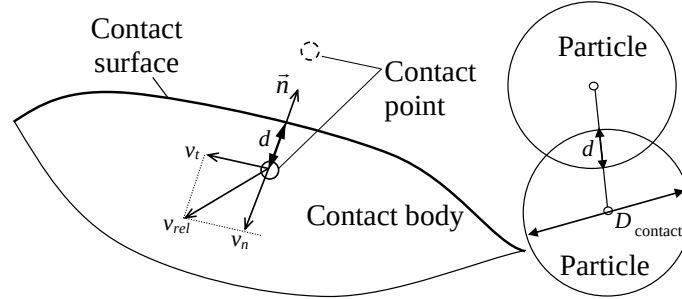


Figure 1: Contact surface and contact point and particle to particle contact. d is the penetration between the two particles.

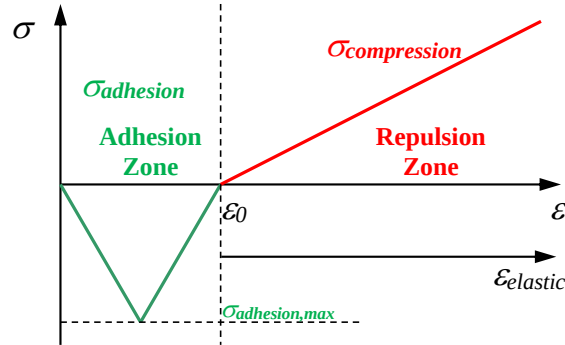


Figure 2: Normal adhesion and repulsion contact stresses.

The total (ϵ), elastic ($\epsilon_{elastic}$), and plastic ($\epsilon_{plastic}$) particle strains are given by:

$$\epsilon = \frac{d}{D_0} \quad \epsilon_{elastic} = \frac{d_{elastic}}{D_0} \quad \epsilon_{plastic} = \frac{d_{plastic}}{D_0} \quad (4)$$

where d is the penetration of the neighboring particle starting from a radius of $D_{contact}/2$, $d_{elastic}$ is the elastic deflection of the particle, and $d_{plastic}$ is the plastic deformation of the particle. The elastic deflection is given by:

$$d_{elastic} = d - d_0 \quad d_0 = \epsilon_0 D_0 \quad d > d_0 \quad (5)$$

The plastic deformation is used to calculate the contact diameter of the particle:

$$D_{contact} = D_0 - d_{plastic} \quad (6)$$

The adhesion ($\sigma_{adhesion}$) and compression ($\sigma_{compression}$) stresses are specified as a function of contact point penetration strain into the contact surface (ϵ) as shown in Figure 2. Up to a penetration strain ϵ_0 corresponding to a penetration distance d_0 the contact stresses are attractive and a stress greater than $\sigma_{adhesion,max}$ is needed to separate the two bodies. If the penetration strain exceeds ϵ_0 , then the contact stresses becomes compressive/repulsive thus opposing further penetration. Note that the adhesion and the repulsion stresses can be a non-linear function of the penetration strain (ϵ). The actual shape of the curve in Figure 2 can be tuned using physical experiments. For example, $\sigma_{compression}$ versus $\epsilon_{elastic}$ can be tuned using the elastic strain versus the applied normal stress for the soil using the hydrostatic compression test described in Section 3.

The normal damping stress ($\sigma_{damping}$) is given by:

$$\sigma_{damping} = \begin{cases} c_{damp} \dot{\epsilon} & \dot{\epsilon} \geq 0 \\ s_{damp} c_{damp} \dot{\epsilon} & \dot{\epsilon} < 0 \end{cases} \quad (7)$$

where $\dot{\epsilon}$ is the interparticle normal strain rate, c_{damp} is the damping coefficient per unit area, and s_{damp} is a separation damping factor (typically between 0 and 1) which reduces normal damping when the two bodies are moving apart. c_{damp} can be specified as a function of both ϵ and $\dot{\epsilon}$. The viscous stress ($\sigma_{viscous}$) is given by:

$$\sigma_{viscous} = c_{viscous} \dot{\epsilon}_t \quad \dot{\epsilon}_t = \frac{v_t}{D_0} \quad (8)$$

where $\dot{\epsilon}_t$ is the interparticle tangential strain rate, v_t is the relative tangential velocity between the particles (Figure 1), and $c_{viscous}$ is the viscosity coefficient per unit area. $F_{adhesion}$, $F_{repulsion}$, and $F_{damping}$ are calculated using:

$$\begin{aligned} F_{adhesion} &= \sigma_{adhesion}/D_0^2 & F_{repulsion} &= \sigma_{compression}/D_0^2 \\ F_{damping} &= \sigma_{damping}/D_0^2 & F_{viscous} &= \sigma_{viscous}/D_0^2 \end{aligned} \quad (9)$$

In order to account for permanent plastic deformation of the soil, the plastic strain ($\epsilon_{plastic}$) is specified as a function of compression stress ($\sigma_{compression}$) (e.g. Figure 3). The plastic deformation is subtracted from the particle radius (Equation 5). $\epsilon_{plastic}$ versus $\sigma_{compression}$ curve can be tuned to match the plastic strain versus hydrostatic pressure curve for the soil obtained from using hydrostatic compression test described in Section 3.

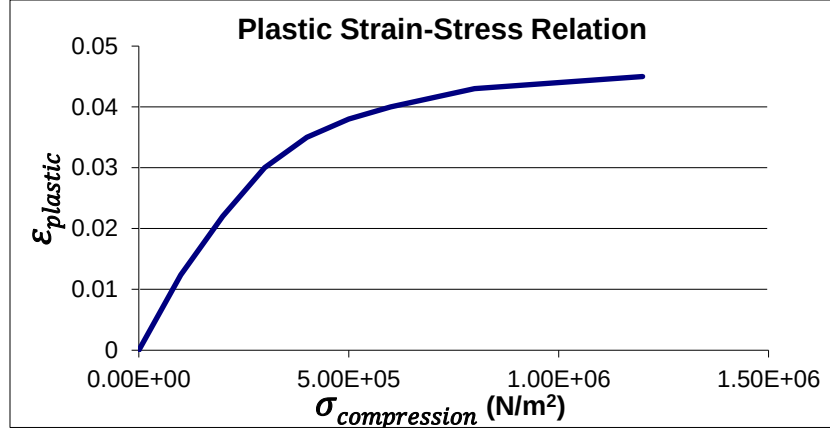


Figure 3: Typical curve of plastic stress-strain relation.

In order to account for the increase in soil cohesive strength due to plastic deformation, the maximum adhesion stress ($\sigma_{adhesion.max}$) in Figure 2 is specified as function of the plastic strain ($\epsilon_{plastic}$) (e.g. Figure 4).

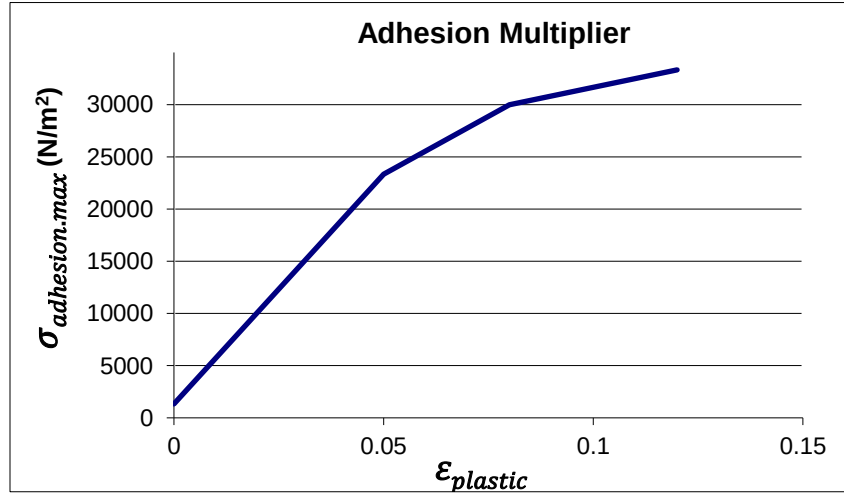


Figure 4: Typical adhesion force as function of plastic deformation.

In order to account for the reduction of soil cohesive strength and soil bulk density due to shearing/tension, a time relaxation is applied to the soil plastic deformation each time step such that the plastic deformation of a particle ($\delta_{plastic}$) is given by:

$$\epsilon_{plastic} = \epsilon_{plastic} - \begin{cases} 0 & \sigma_{repulsion.max} \geq \sigma_{adhesion.max} \\ \dot{\epsilon}_{relax} \Delta t & \sigma_{repulsion.max} < \sigma_{adhesion.max} \end{cases} \quad (10)$$

where $\dot{\epsilon}_{relax}$ is the plastic relaxation strain rate (in 1/ time) and Δt is the solution time step. Thus, if the particle maximum compression stress ($\sigma_{repulsion.max}$) is larger than the maximum adhesion stress ($\sigma_{adhesion.max}$) then the particle plastic deformation is left unchanged. If the particle maximum compression stress is smaller than the maximum adhesion stress then the particle plastic strain is reduced at a speed of $\dot{\epsilon}_{relax}$. The plastic deformation minimum allowable value is zero. The value of V_{relax} must be experimentally tuned.

An asperity friction model is used along with the normal repulsion ($F_{repulsion}$) force to calculate the tangential friction force ($F_{friction}$) [42]. In this model, friction is modeled using a piece-wise linear velocity-dependent approximate Coulomb friction element in parallel with a variable anchor point spring. The model approximates asperity friction where friction forces between two rough surfaces in contact arise due to the interaction of the surface asperities (Figure 5). When two surfaces are in static (stick) contact, the surface asperities act like tangential springs. When a tangential force is applied, the springs elastically deform and pull the surfaces to their original position. If the tangential force is large enough, the surface asperities yield (i.e. the springs break) allowing sliding to occur between the two surfaces. The breakaway force is proportional to the normal repulsion contact force ($F_{repulsion}$). In addition, when the two surfaces are sliding past each other, the asperities provide resistance to the motion that is a function of the sliding velocity and acceleration, and the normal repulsion contact force. Figure 6 shows a schematic diagram of the asperity friction model. It is composed of a simple piece-wise linear velocity-dependent approximate Coulomb friction element in parallel with a variable anchor point spring.

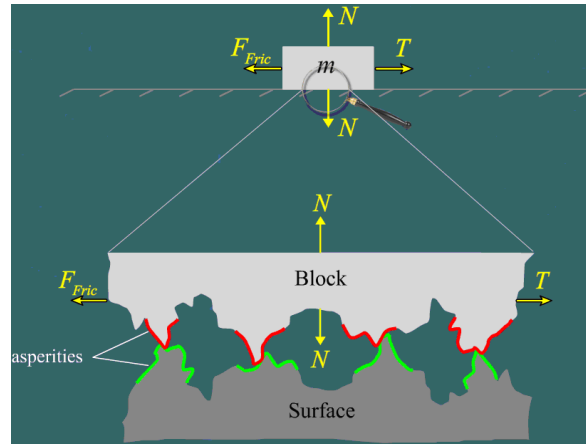


Figure 5: Asperity-based physical interpretation of friction.

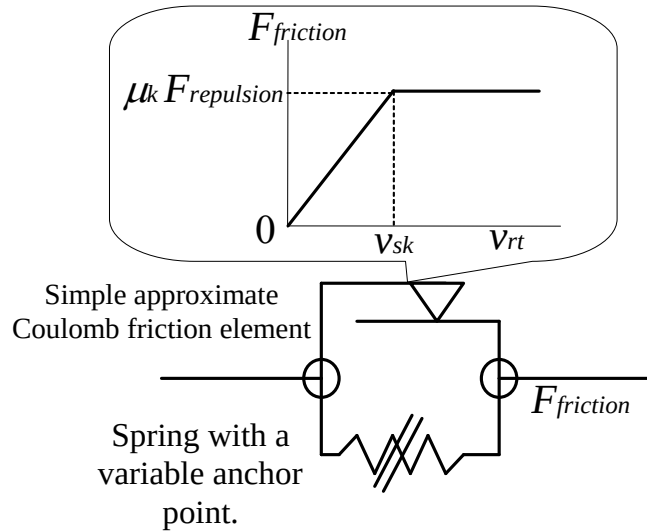


Figure 6: Asperity spring friction model. $F_{friction}$ is the tangential friction force, $F_{repulsion}$ is the normal repulsion force, μ_k is the kinetic friction coefficient, and v_t is the relative tangential velocity between the two points in contact.

Note that in order to connect two points on two bodies using an asperity spring, the model must keep track of which rigid bodies are in contact and of the local position vectors of the asperity spring anchor points on the two contacting bodies. Also, note that the two rigid bodies can be in contact at more than one point, therefore the model must keep track of the corresponding contact points on the two contact bodies.

The friction coefficient (μ_k), viscosity coefficient ($c_{viscous}$) and damping coefficient ($c_{damping}$) can be specified as a function of the plastic strain ($\epsilon_{plastic}$). The curve in Figure 4 along with the friction coefficient (μ_k) can be tuned to match the shear stress versus normal stress for different pre-shear (consolidation) normal stress values obtained using a shear test.

The DE model is integrated into the IVRESS solver [43] (formerly known as DIS) computational mechanics code. In addition to its DE solution capabilities, IVRESS includes multibody dynamics (MBD) and finite element (FE) solution capabilities. IVRESS uses a parallel explicit solution procedure to solve the equations of motion along with the constraint equations [21, 44]. The IVRESS code was used to create the coupled multibody dynamics model of terramechanics test devices and DE soil model and to generate the simulation results in this paper.

3 Test devices description and test procedures

The following terramechanics devices along with the associated test procedures, which are used in the DE model calibration and validation, are presented in this section:

- Hydrostatic compression.
- Unconfined shear.
- Confined shear.
- Disc and cone penetrometer.
- Single tire drawbar-pull test.

A description of the multibody dynamics – DE computational model of each device is also presented.

3.1 Hydrostatic compression

The compression device consists of the following components (Figure 7):

- **A stiff steel soil box** for confining the soil.
- **Compression Plate:** 0.5" thick aluminum plate which is used to apply a vertical hydrostatic pressure on the soil inside the steel box.
- **Vertical actuator** used to move the compression plate and apply the required pressure on the soil.
- **Load cell** to measure the vertical force on the soil.
- **Vertical position sensor** to measure the position of the compression plate.

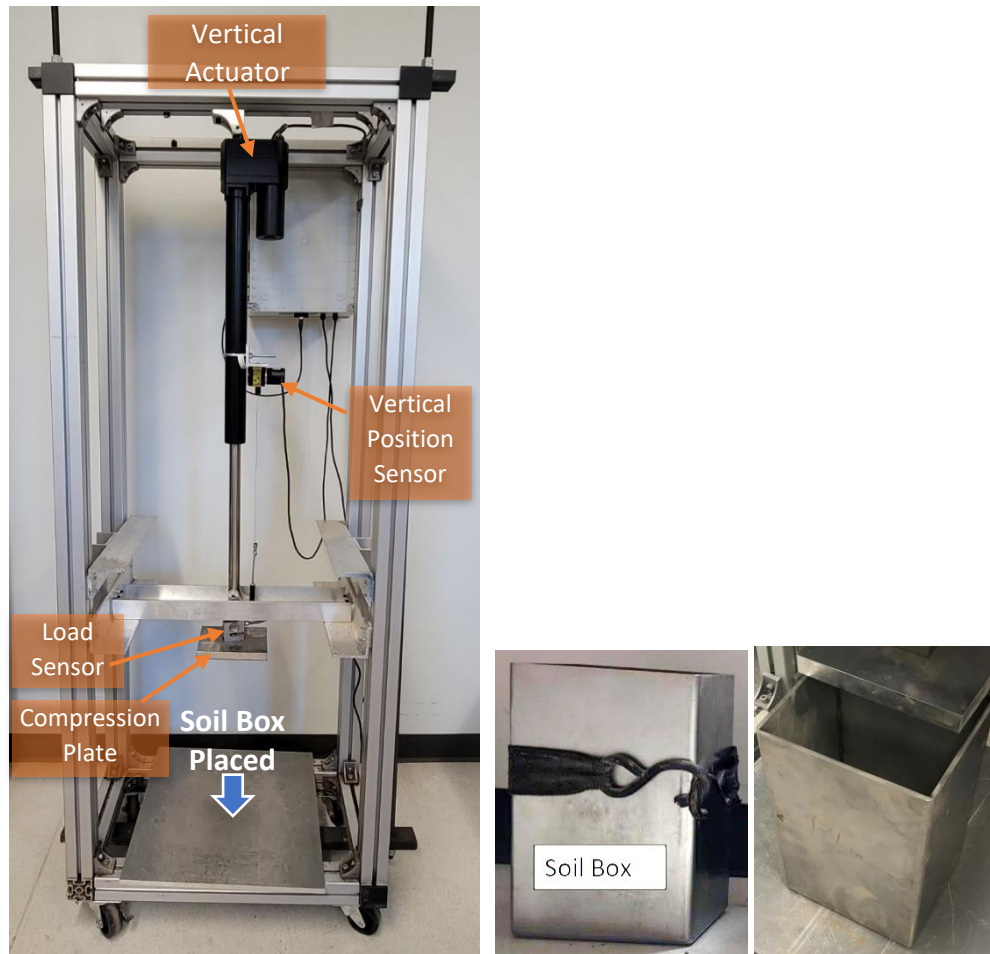


Figure 7: Compression Machine and rectangular steel soil box.

The main purpose of the compression test is to determine the elastic and plastic strains as well as the bulk density of a specific soil as a function of hydrostatic pressure. The test procedure is as follows:

1- Setting the experiment:

- Measure the weight of the empty soil box (M_{Empty}).
- Fully fill the soil box with specific soil. Make sure the top surface of the soil is leveled with the soil box walls.
- Measure the weight of the full box (M_{full}).
- Calculate the weight of the soil (M_{Soil}).

$$M_{Soil} = M_{full} - M_{Empty}$$

- Place the soil box in the test rig and align it under the compression plate.
- Lower the compression plate to be about 5mm from the soil and start the automatic test sequence.

2- Automatic test sequence:

- The actuator advances very slowly until it touches the soil then record the compression plate position from the machine floor as the soil height (H).
- Using the control system, the vertical actuator compresses the soil to set the maximum required hydrostatic pressure (P_n) in “ n ” steps. The control system maintains each pressure step for 45 seconds to allow the soil to settle before taking a height reading h_i , where “ i ” is a

variable equal to the step number (i.e., $i = 1:n$). After reaching the required maximum hydrostatic pressure (P_n) in step n , the control system slowly retracts the compression plate to a hydrostatic pressure of 0 kPa, and record the soil height (h_e). The following parameters are measured at each pressure step (i):

- i. F_i : Vertical force.
- ii. h_i : Compression plate height (after compensating for the machine elastic deformation).

3- Post-Processing:

- a. The following parameters are calculated at each pressure step (i):

- i. P_i : Hydrostatic pressure
$$P_i = \frac{F_i}{\text{Compression Plate Area}} \quad (11)$$

- ii. ϵ_i : Total Strain:
$$\epsilon_i = \frac{H-h_i}{H} \quad (12)$$

- iii. ρ_i : Bulk Density (kg/m^3)
$$\rho_i = \frac{\text{Soil Box Area} \times h_i}{M_{\text{Soil}}} \quad (13)$$

- b. The elastic strain at the final step:

- i. ϵ_e : The elastic strain:
$$\epsilon_e = \frac{h_n - h_e}{H} \quad (14)$$

- ii. E : Soil Young's modulus:
$$E = \frac{P_n}{\epsilon_e} \quad (15)$$

where h_n is the height of the soil at the final hydrostatic pressure step (P_n).

- iii. ϵ_p : The plastic strain:
$$\epsilon_{p_i} = \epsilon_i - E P_i \quad (16)$$

- c. Plot the following graphs (e.g., Figure 8):

- i. Plastic strain (ϵ_{p_i}) versus the vertical pressure (P_i)
- ii. Elastic strain versus the vertical pressure (P_i), assuming the elastic strain as a function of vertical pressure is linear.
- iii. Bulk density (ρ_i) vs vertical pressure (P_i)

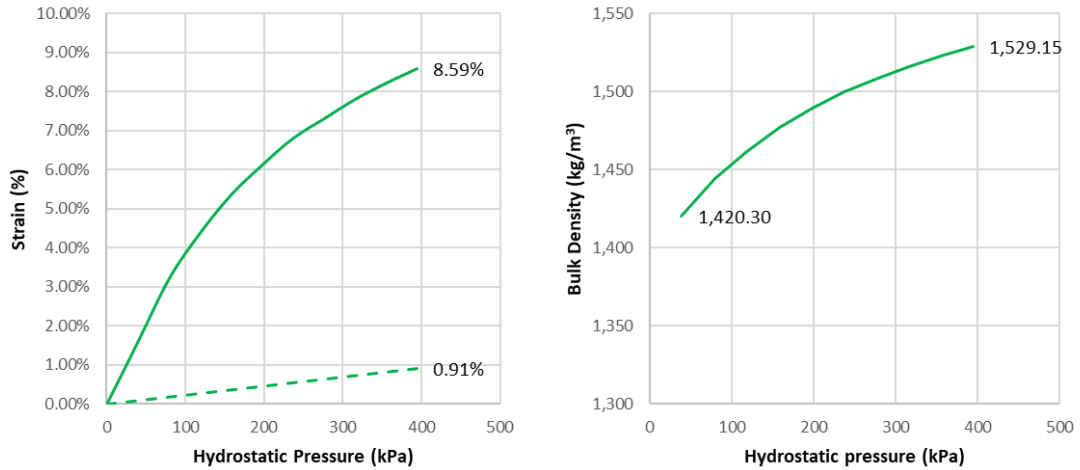


Figure 8: Typical compression test results for fine-grain soil. The left figure shows the plastic strain (solid line) and the elastic strain (dashed line) vs hydrostatic pressure (in kPa). The right figure shows the bulk density (in kg/m^3) vs the hydrostatic pressure (in kPa).

The MBD model of the hydrostatic compression device consists of two rigid bodies: the cylindrical soil container (1 m diameter) and a compression lid (Figure 9). The container is grounded, and the lid is restricted to move vertically using a prismatic joint. A linear actuator is used to apply the compression

force required to achieve the desired pressure on the soil. In all the simulations an unstressed particle diameter of $D_0 = 0.03\text{ m}$ and an explicit integration time step of $1.5\text{E-}5\text{ s}$ are used. We are fixing those two parameters since we plan to use this particle size and time step in the full-vehicle simulations. This particle size is chosen because it is the size of the minimum tire tread block spacing or track grouser spacing. The time step is approximately the maximum stable step size allowed by the explicit time integration solution method used. The total number of particles in the hydrostatic compression simulation is 17,424. The simulation closely follows the steps and post-processing calculations of the experiment, except that the pressure is ramped up linearly from $P_1 = 20\text{ kPa}$ to $P_n = 400\text{ kPa}$ in 10 sec. After 10 sec, P_n is maintained for 2 sec, then released to 0 kPa in 1 sec, in order to measure the elastic strain of the DE soil.

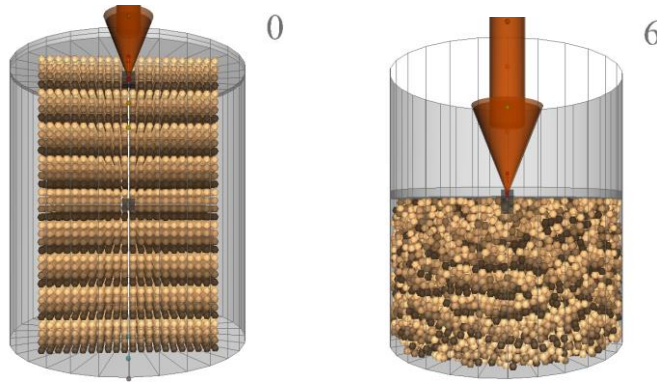


Figure 9: Simulation model of the 2NS sand compression test at time 0 (left) and at time 6 s (right). The compression device model consists of two rigid bodies: a container and a lid.

3.2 Unconfined shear

The unconfined shear device consists of the following components (Figure 10):

- **2 Vertical actuators:** synchronized together to apply vertical loading on the soil.
- **U channel:** A U-shaped aluminum channel that holds the shear assembly and is actuated vertically by the shear actuators.
- **Shear actuator:** Applies shearing force to simulate shearing action to the soil.
- **Vertical Load sensor:** load cell to measure the vertical loading on the soil.
- **Shear Load sensor:** load cell to measure the shear force applied on the soil.
- **2 Vertical position sensors:** monitor the position of the vertical actuators.
- **Shear position sensor:** monitors the position of the shear actuator.
- **Shear Block:** Aluminum block transferring the shear force to the shear plate.
- **Shear plate:** L-shaped stainless-steel plate with 80 mm x 80 mm shear area and 10 mm high back grouser.

The shear block holds the shear plate and can be actuated in two directions: vertical direction (normal load) and horizontal direction (shear). Two vertical actuators are used in parallel in order to allow the shear block to move horizontally without rotation (by resisting the unbalance moment when the normal force is not at the center point between the two actuators) and apply the required normal pressure on the soil. The two actuators are synchronized by the machine controller to keep the shear plate horizontally leveled at all times. This is done using the input from the vertical position sensor of each actuator to evaluate the shear plate inclination and set it to zero as needed using the two vertical actuators.

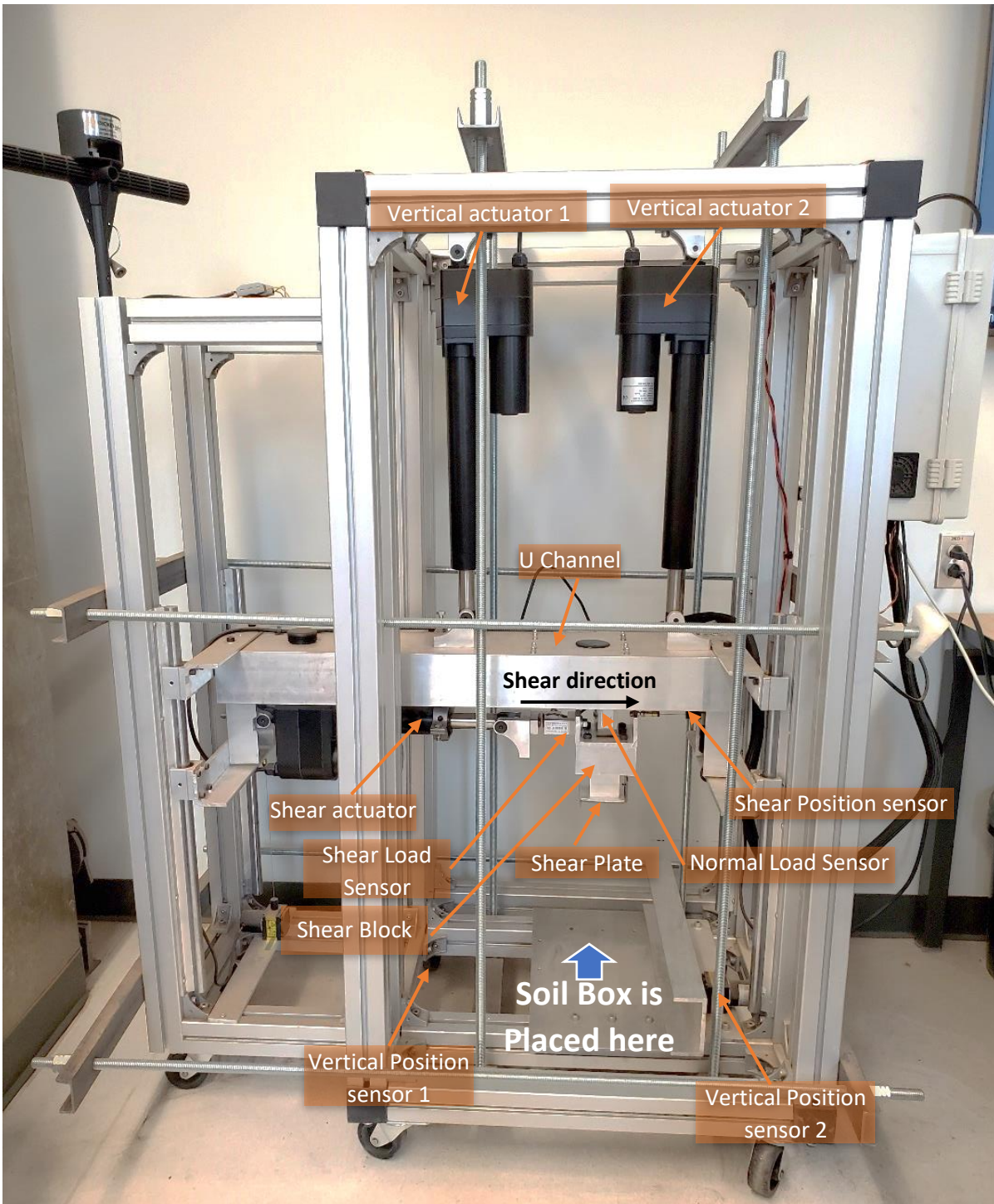


Figure 10: Unconfined soil shear test device.

The main purpose of the test is to determine the shear strength (S) of a specific soil that is prestressed under a specific vertical pressure ($\sigma_{prestress}$) as a function of the applied vertical stress (σ_N). The test procedure is as follows

- 1- Setting the experiment:

- a. Place a soil box with a $\sigma_{prestress}$ prestressed soil in the machine.
 - b. Retract the shear grouser to be about 15mm from the back edge of the box.
 - c. Lower the shear plate to be about 10mm from the soil without touching it and start the automatic test sequence.
- 2- Automatic test sequence:
 - a. Using the control system, the vertical actuators lower the shear plate to set the required vertical loading (σ_N) and maintain it for 45 seconds.
 - b. The shear actuator then pushes the shear plate through the soil for 20 mm, while the control system is maintaining the vertical load throughout the shearing process.
 - c. The following parameters are recorded during the test:
 - i. Vertical Stress.
 - ii. Shear Stress.
 - iii. Vertical Position of the shear grouser from start of shearing.
 - iv. Shear Position: Horizontal Position of the shear grouser from start of shearing.
- 3- Post-processing:
 - a. Plot the vertical stress and shear stress with respect to the shear position as shown in Figure 12.
 - b. Find the maximum shear stress.
 - c. Find the vertical loading corresponding to the maximum shear stress at the same shear position.
 - d. Record the vertical stress and the corresponding maximum shear stress.
 - e. Repeat the test for different values of normal pre-stress and applied normal stress, then plot the shear strength of the soil as a function of those two parameters (e.g. Figure 13).



Figure 11: Snapshot of the unconfined shear test showing the shear plate and soil box.

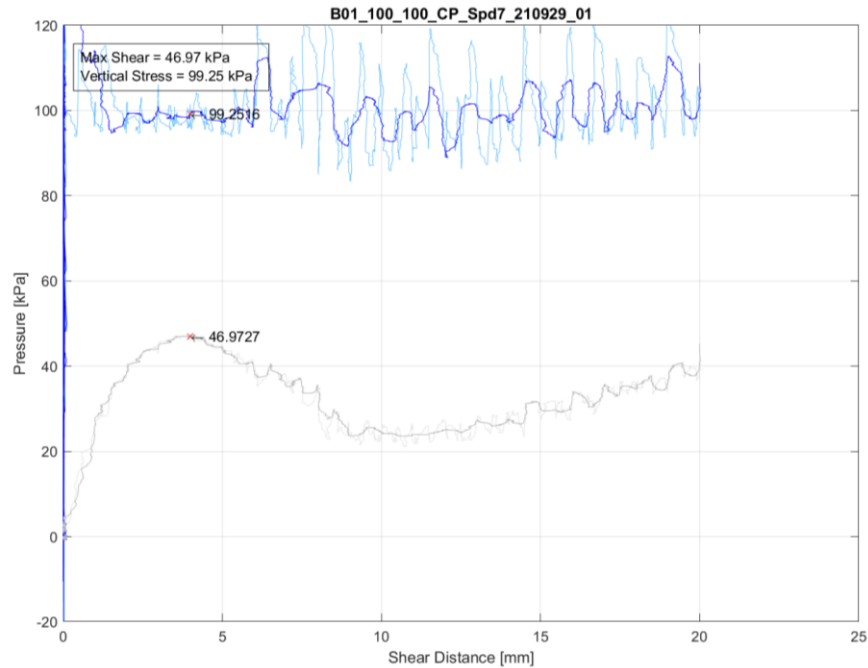


Figure 12 : Stresses as a function of shear position during a typical soil shear test with a target vertical stress of 100 kPa (left). Snapshot of from the test. Showing the shear plate and soil box.

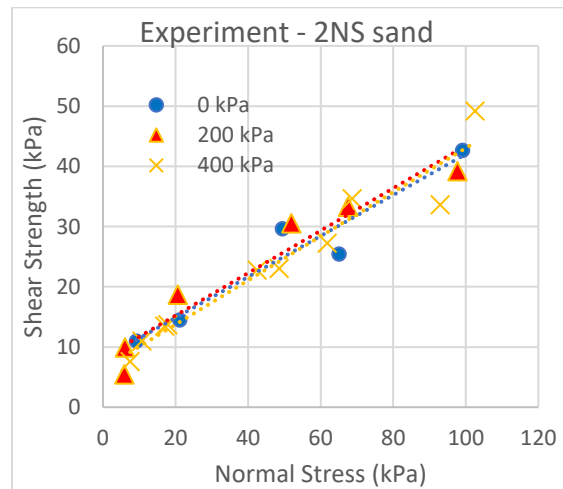


Figure 13: 2NS sand shear summary for 0kPa, 200 kPa and 400 kPa pre-stress at 5kPa, 10kPa, 20kPa, 50kPa, 70kPa, and 100kPa vertical stress

The MBD model of the unconfined shear device consists of four rigid bodies: a rectangular soil container (1.4 x 0.8 m), a compression lid, a horizontal frame (U channel in the device), and a shear plate (0.3 x 0.3 m) with a 0.03 m high back grouser (Figure 14). The container is grounded. The lid and the horizontal frame are restricted to move vertically using prismatic joints. The shear plate is connected to the horizontal frame using a horizontal prismatic joint. Two linear vertical actuators are mounted on the lid and horizontal frame and are used to apply the pre-stress and shear plate compression on the soil, respectively. A horizontal actuator is used mounted on the horizontal frame and used to move the shear plate. The total number of particles is 28,272. The simulation closely follows the steps and post-processing calculations of the experiment. The soil is first compressed by the lid to the desired pre-stress value. Then, the normal pressure of the on the shear plate is ramped linearly in 1 sec to the desired pressure using the

horizontal frame vertical actuator. The pressure is then kept constant for 1 sec. After that the shear actuator is controlled to move at a speed of 0.01 m/s for 0.2 m while the vertical actuator maintains the constant desired normal pressure. The shear stress versus time is recorded for the entire travel distance of the shear plate and the maximum shear stress found. This shear stress is the soil shear strength at the desired normal stress and normal pre-stress.

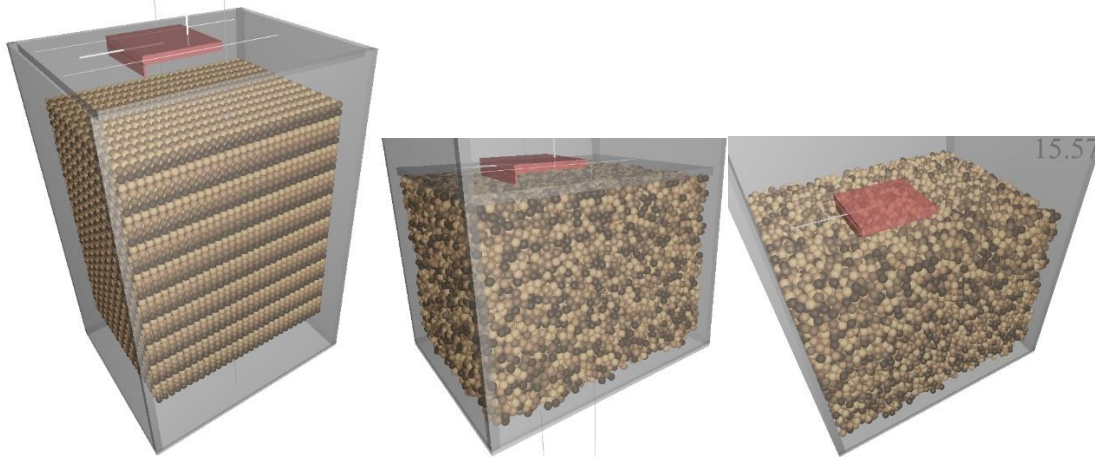


Figure 14: Unconfined shear test simulation model consisting of a rectangular soil container, a rectangular lid for compressing the soil, and a L-shaped shear plate with a back grouser.

3.3 Confined shear

The confined shear cell consists of the following components (Figure 15):

- **Vertical actuator:** to apply the vertical loading on the soil.
- **Shear actuator:** Applies shearing force on the soil.
- **Lower box.**
- **Upper box.**
- **Lid.**
- **Normal load sensor:** load cell to measure the vertical loading on the soil.
- **Shear load sensor:** load cell to measure the shear force applied on the soil.
- **Vertical position sensor:** monitor the position of the vertical actuator.
- **Shear position sensor:** monitors the position of the shear actuator.

The shear cell cross-section shape can be either rectangular or cylindrical. The test procedure is similar to the unconfined shear test procedure presented in Section 3.2. Similar, to the unconfined shear test, the main purpose of the test is to determine the shear strength (S) of a specific soil that is prestressed under a specific vertical pressure ($\sigma_{prestress}$) as a function of the applied vertical stress (σ_N).

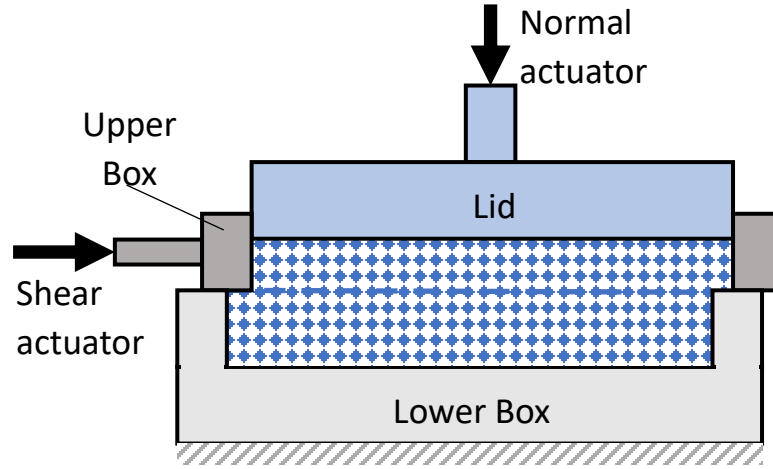


Figure 15: Unconfined soil shear test device.

A cylindrical MBD model of the confined shear cell is used. It consists of three rigid bodies: a lower soil box (1 m diameter), an upper soil box, and a lid (Figure 16). The lower box is grounded. The upper box can slide horizontally using a shear actuator on a prismatic joint. A lid is mounted to the upper box using a vertical prismatic joint and can vertically move using a normal actuator. The total number of particles is 23,276. The simulation closely follows the steps and post-processing calculations of the experiment. The soil is first compressed by the lid to the desired pre-stress value. Then, the normal pressure of the lid is changed to the desired normal pressure. The normal pressure is then kept constant for 1 sec. After that the shear actuator is controlled to move the upper box at a speed of 0.01 m/s for 0.06 m while the vertical actuator maintains the constant desired normal pressure. The shear stress versus time is recorded for the entire travel distance of the upper box and the maximum average shear stress found (e.g. Figure 12). This shear stress is the soil shear strength at the desired normal stress and normal pre-stress.

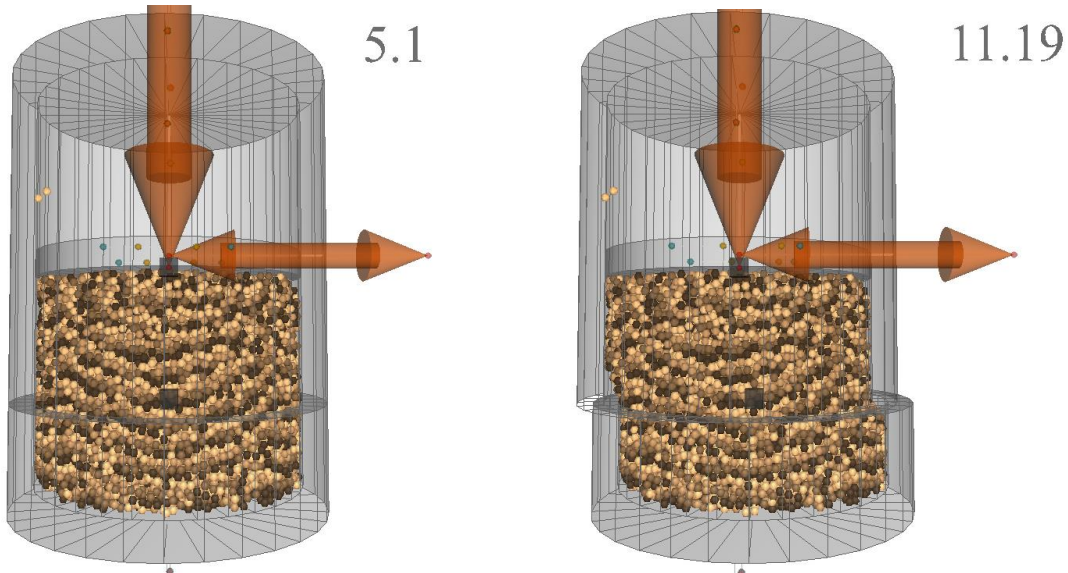


Figure 16: Confined shear cell DE simulation. The left figure shows a snapshot of the shear cell after application of the normal load. The right figure shows a snapshot of the shear cell at the end of the shearing displacement.

3.4 Disc and cone penetrometers

The penetrometer device is the same as the hydrostatic compression device (see Section 3.1) with a penetrometer assembly retrofit as shown in Figure 17. The penetrometer assembly consists of the following:

- **Base Plate:** holds the penetrometer assembly.
- **Vise grips:** clamps the penetrometer assembly to the compression machine.
- **Stem:** Aluminum rod with a smaller diameter than the penetrometer head.
- **Penetrometer:** A cone or disc penetrometer can be attached to the stem. The penetrometers which can be used are (Figure 18):
 1. **C60:** 30 deg. apex angle cone of 60 mm base diameter.
 2. **C30:** 30 deg. apex angle cone of 30 mm base diameter.
 3. **P39:** 7.62 mm (0.3 in) thick circular disc of 39 mm diameter.
 4. **P50:** 7.62 mm (0.3 in) thick circular disc of 50 mm diameter.
 5. **P60:** 7.62 mm (0.3 in) thick circular disc of 60 mm diameter.
 6. **P80:** 12.7 mm (0.5 in) thick circular disc of 80 mm diameter.

The penetrometer assembly is clamped to the compression plate using vise grips. The stem is threaded to the base plate. On the other end of the stem, the various penetrometer cones and discs can be attached (Figure 18).

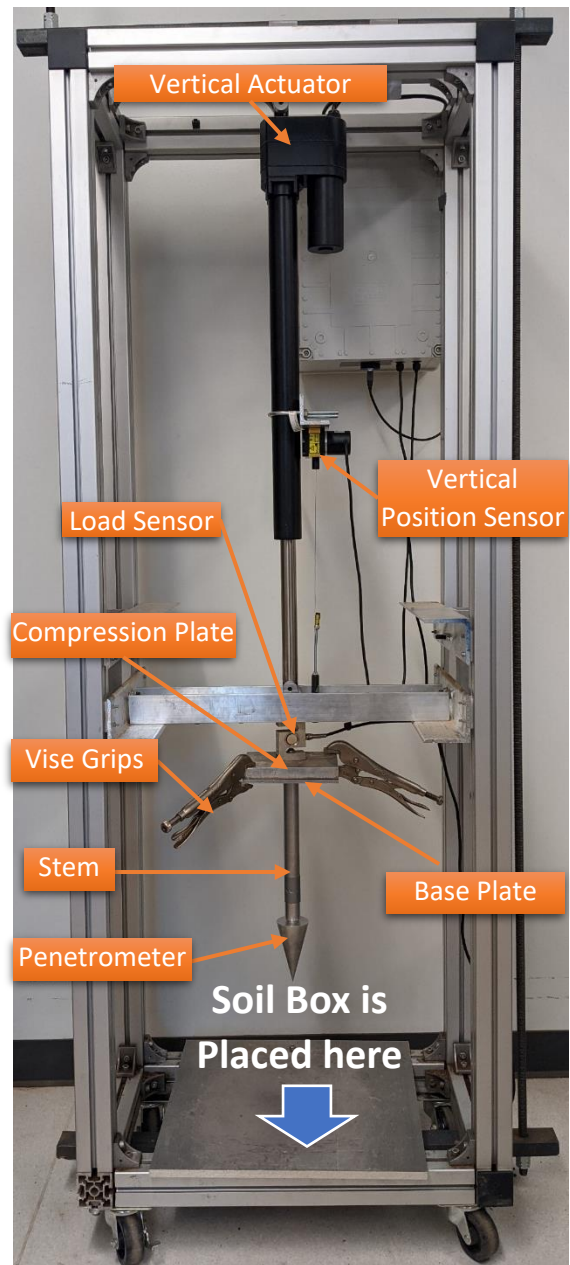


Figure 17: Penetrometer assembly attached to the compression machine

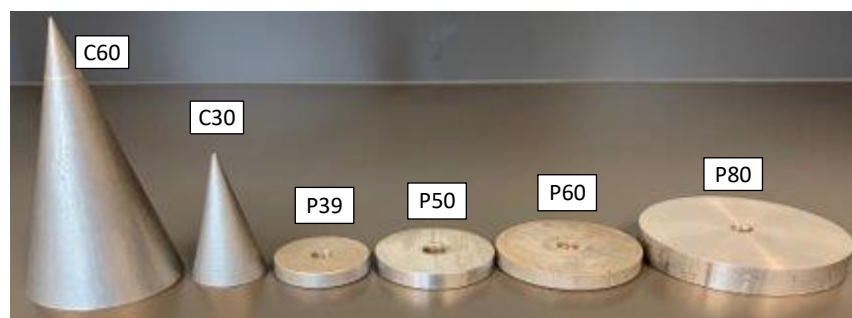


Figure 18: Cone and Disc Penetrometers

The purpose of the test is to obtain the pressure versus sinkage curve for the soil. The test procedures is as follows:

- 1- Setting the experiment:
 - a. Prepare the soil box and place it in the compression test rig.
 - b. Condition the soil by compacting it to the required prestress using a rectangular or circular compression plate of the same size as the soil box.
 - c. Raise the actuator and retrofit the compression plate with the penetrometer assembly.
 - d. Lower the penetrometer to be about 10 mm from the soil surface, so the actuator can accelerate to the desired speed before interacting with the soil.
 - e. Start the automatic sequence.
- 2- The automatic sequence:
 - a. Using the control system, the vertical actuator pushes the penetrometer into the soil at the desired speed up to a desired sinkage (Figure 19).
 - b. The following parameters are measured during the test:
 - i. F : Vertical force
 - ii. Sinkage of the penetrometer from the start of soil penetration
- 3- Post Processing:
 - a. The penetrometer pressure (kPa) is calculated as follows:
$$\text{Penetrometer Pressure} = \frac{F}{\text{Compression Plate or Cone Base Area}} \quad (16)$$
 - b. Plot the penetrometer pressure versus sinkage as shown in Figure 20.



Figure 19: Snapshot of a typical disk penetrometer test.

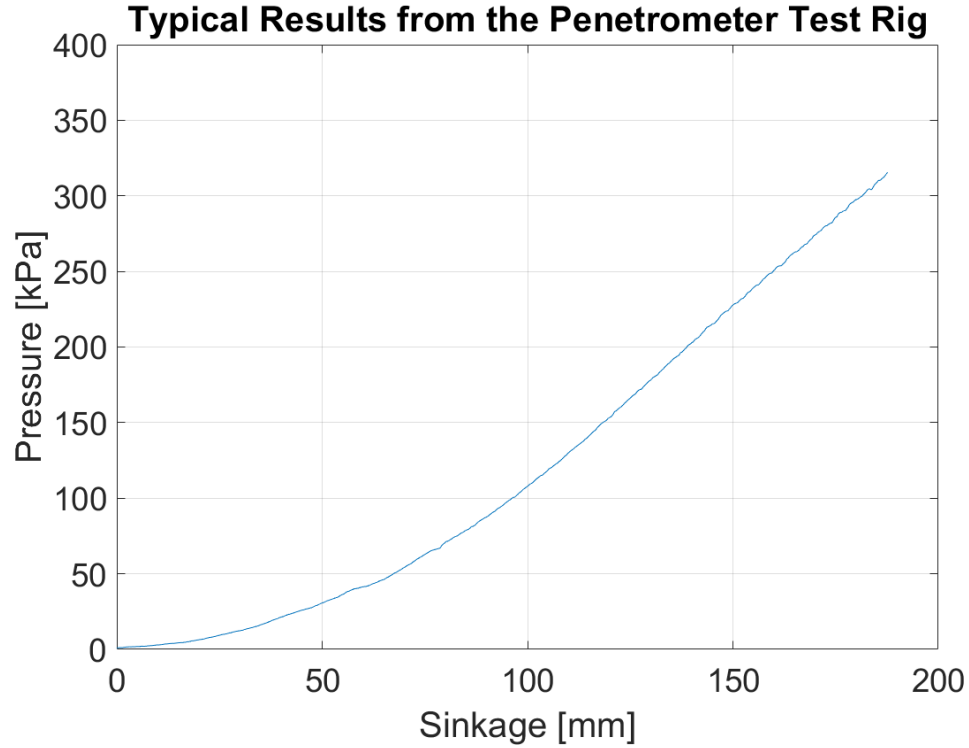


Figure 20: Typical Result of a penetrometer test for 100 kPa prestressed FG soil with 0.5 mm/s penetration speed using a cone with 60 mm base diameter.

The MBD model of the penetrometer test device consists of three rigid bodies: the cylindrical soil container (1 m diameter), the compression lid, and the penetrometer (cone or disk) (Figure 21). The container is grounded, and the lid and penetrometer are restricted to move vertically using prismatic joints. A vertical linear actuator is used to apply the compression force to the lid required to achieve the desired prestress to the soil. Another vertical linear actuator is used to move the penetrometer at the desired speed. The total number of particles is 17,424. The simulation closely follows the steps and post-processing calculations of the experiment (Figure 21).

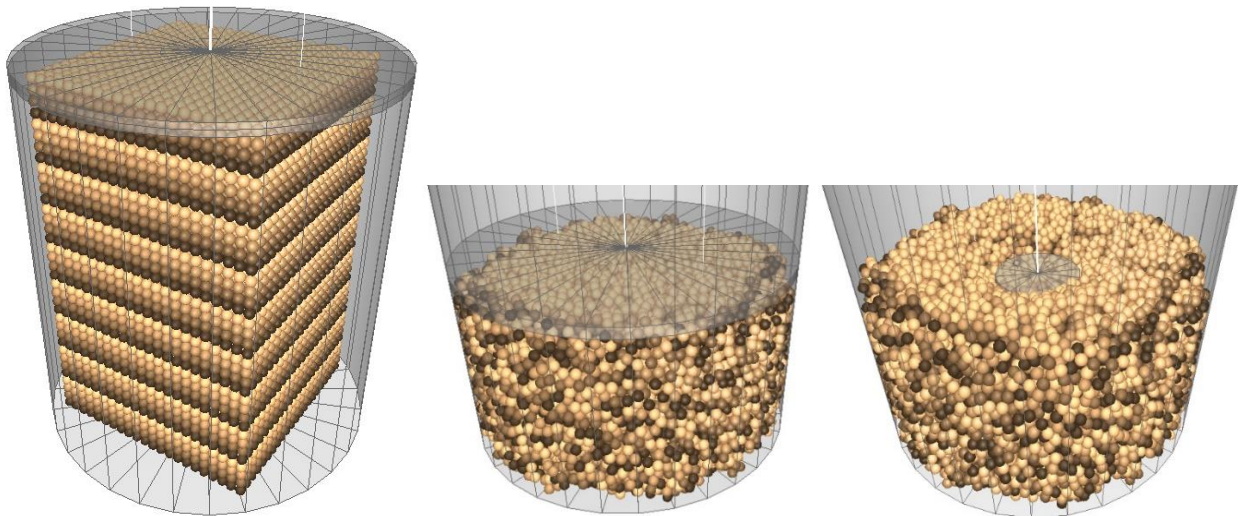


Figure 21: Simulation model of the 2NS sand disk penetrometer test at time 0 (left) and at time 6 s (right). The penetrometer device model consists of three rigid bodies: a container, a lid, and the penetrometer.

3.5 Single Tire Drawbar-Pull Test

The single tire test device consists of a carriage which can move longitudinally at a controlled speed on frictionless rails using a linear actuator. A load cell is used to measure the force of the linear actuator and a length sensor is used to measure the position and speed of the carriage. The wheel is mounted on a C-frame which is mounted using vertical prismatic joints to the carriage. A pneumatic tire is mounted on the wheel. Weights can be added to the C-frame to set the vertical force on the tire. A rotational actuator at the hub of the wheel can be used to rotate the tire at a prescribed angular velocity. A torque sensor is used to measure the torque applied to the tire and a rotational encoder is used to measure the angular velocity of the wheel. The MBD model of the device is shown in Figure 22. The single tire test device allows independent control of the longitudinal speed, normal force, and angular velocity of the tire.

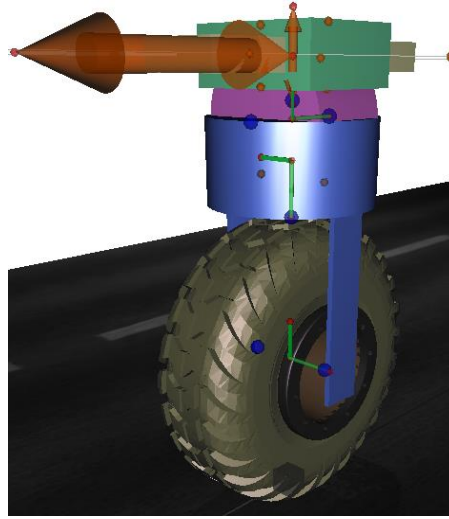


Figure 22 MBD tire test fixture and finite element tire model.

The test starts by setting the normal load on the tire to the desired value. Then the carriage linear speed is ramped up to the desired speed and the angular velocity of the wheel is also ramped up such that tire slip stays at zero where slip (s) is given by:

$$s = \frac{\omega r - v}{\omega r} \quad (17)$$

where ω is the wheel angular velocity, r is the outer radius of the tire, and v is carriage speed. Note that ωr is the theoretical linear speed of the tire if slip is zero. Then slip is increased from 0 to about 50% by linearly increasing the angular velocity of the tire. The main experiment result are: slip versus drawbar-pull force, slip versus torque, and slip versus rut depth (or vertical sinkage of the tire) for the desired tire normal load, the desired nominal carriage speed (v), and the desired tire inflation pressure.

4 Calibration of the DE soil model

The DE soil model parameters were tuned using the iterative procedure shown in Figure 23. The procedure starts by performing the hydrostatic compression and unconfined shear tests for the soil from which the soil responses in Figure 8 and Figure 13 are respectively generated. Then, an initial set of DE model parameters is assumed (see Section 2) and used to perform the hydrostatic compression and unconfined shear simulations. The simulated soil response is then compared with the physical soil response. If the responses do not match then the DE model parameters are adjusted and the simulations are performed

again. This iterative process is repeated until the DE model response matches the physical soil response within a certain desired tolerance.

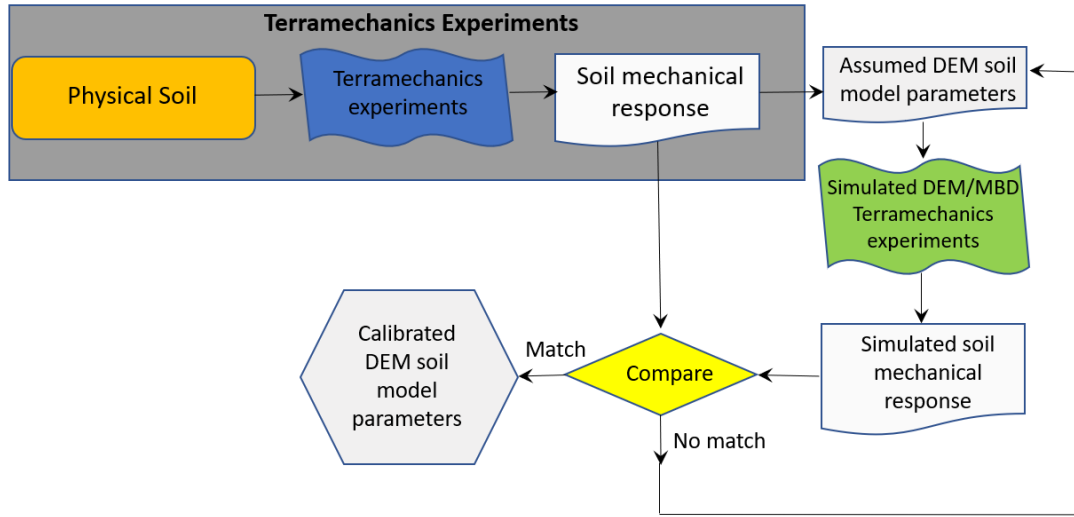


Figure 23: Procedure for calibrating the DE soil model parameters using the hydrostatic compression and the unconfined shear tests.

The calibration procedure and the validation were performed for two types of dry soils: a clay-type fine grain (FG) soil and a coarse-grain sandy soil which is close to 2NS sand (2NS). The results of the calibration procedure are presented in this Section. The results of the validation of the procedure are presented in Section 5. In future papers the effect of moisture content on the mechanical behavior of those soils will be investigated. In addition, the DE model will be calibrated and validated for soils with different moisture content.

4.1 Hydrostatic compression

The experimental hydrostatic compression results in Figure 24 show that the FG soil has a plastic strain of 8.59% and an elastic strain of 0.86% at 400kPa hydrostatic Pressure. The Bulk Density of FG soil is 1417.2 kg/m³ at 50kPa to 1533.5 kg/m³ at 400kPa hydrostatic pressure. In the tuned DE model, the soil plastic strain was about 7.7% and the elastic strain was 2.0% at 400 kPa. Note that the maximum elastic stiffness of the particles using the chosen solution time step of 1.5E-5 s results in the 2.0% elastic strain at 400 kPa. In order to match the 0.86% elastic strain in the experiment this solution time step must be reduced. Figure 24 shows that the simulation FG soil bulk density approximately matches the experimental results.

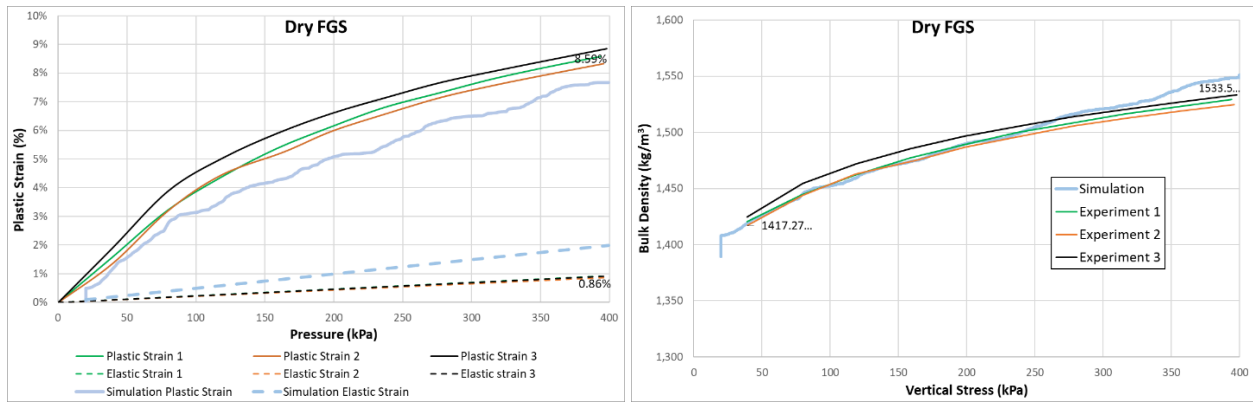


Figure 24: Comparison between experiment and simulation the FG soil @ 0% moisture content compression test. The left figure shows the total strain (solid lines) and the elastic strain (dashed lines) vs hydrostatic pressure. The right figure shows the bulk density versus the hydrostatic pressure.

The experimental hydrostatic compression results in Figure 25 show that the 2NS sand has a plastic strain of about 0.8% and an elastic strain of about 0.7% at 400 kPa hydrostatic pressure. This indicates that 2NS sand is nearly incompressible at up to 400 kPa hydrostatic compression because it undergoes very small plastic and elastic strains. Figure 25 shows that the bulk density of 2NS sand is 1723.6 kg/m³ at 50 kPa to 1749.1 kg/m³ at 400 kPa hydrostatic pressure. As mentioned above, in the DE model, the explicit integration time step was chosen to be 1.5E-5 s. This time step dictated the maximum stiffness of the DE model particles. After setting the inter-particle elastic and plastic stiffness to the maximum value based on this time step, we obtained the simulation results shown in Figure 25. The plastic strain was 1.6% and the elastic strain was 2.0% at 400 kPa hydrostatic pressure. The simulation soil bulk density was about 1723 kg/m³ at 50 kPa and 1777 kg/m³ at 400 kPa hydrostatic pressure. Note that the simulated 2NS sand is still practically nearly incompressible.

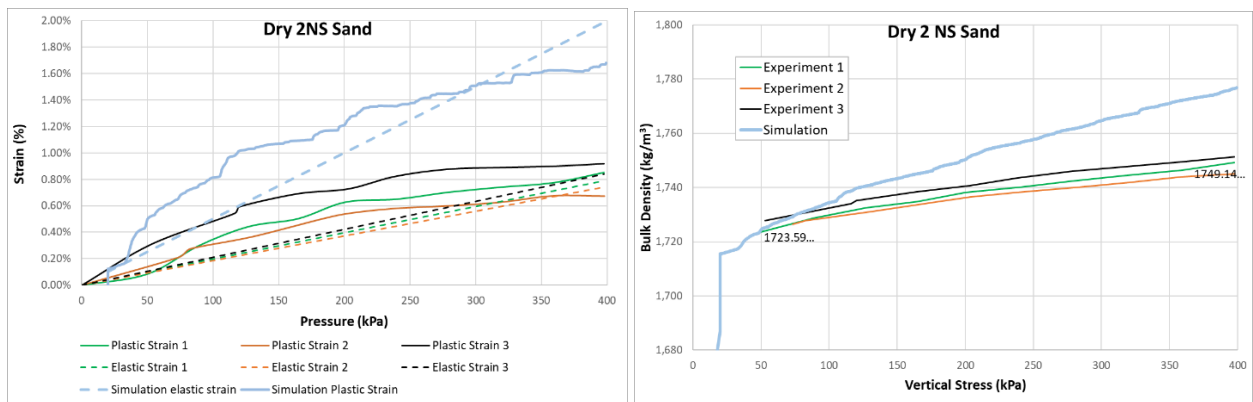


Figure 25: Comparison between experiment and simulation for the 2NS sand @ 0% moisture content compression test. The left figure shows the total strain (solid line) and the elastic strain (dashed line) vs hydrostatic pressure. The right figure shows the bulk density versus the hydrostatic pressure.

4.2 Unconfined shear test

Unconfined Shear tests of pre-stressed 2NS sand and FGS soils are performed at 3 different pre-stress levels: tilled soil, 100kPa, and 400kPa. For each stress level, the soil is sheared under the following applied vertical stress values; 5, 10, 20, 50, 70 and 100 kPa. Each combination of soil prestress and applied vertical stress result in a typical shear stress versus shear displacement plot shown in Figure 12. The result of different loading at a specific prestress are grouped together and fitted linearly using the Coulomb – Terzaghi shear strength equation:

$$S = \sigma_N \mu + C \quad (18)$$

where S is the soil shear strength, C is the soil cohesive strength, μ is the internal soil friction coefficient, and σ_N is the applied normal stress.

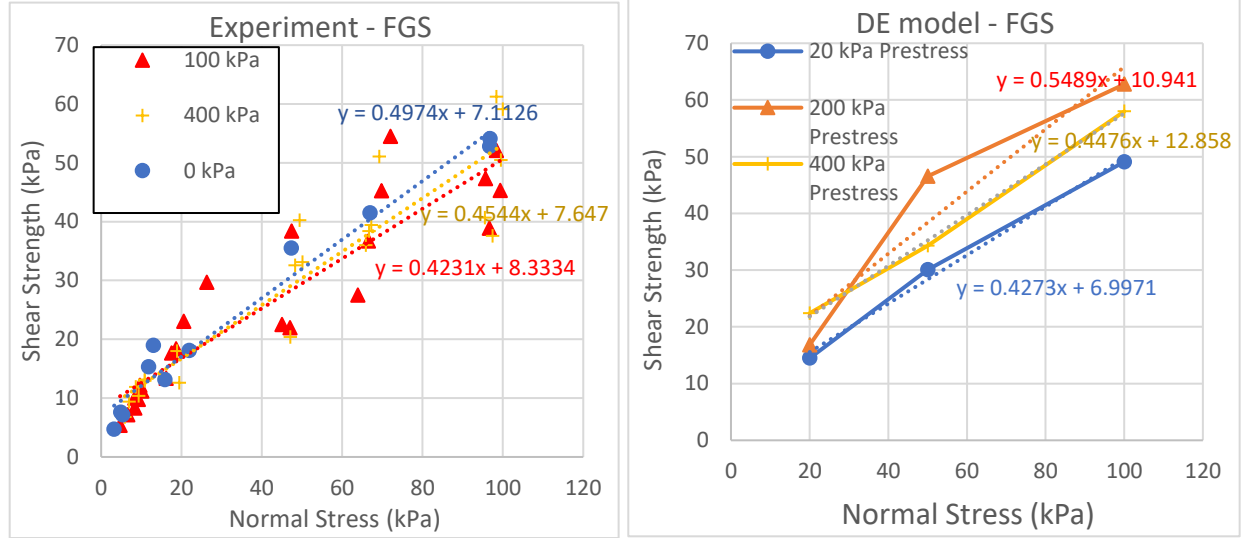


Figure 26: Comparison between FG soil shear strength at different vertical loadings for pre-stressed soil samples at 0, 200 and 400kPa. Experiment results are shown on the left graph and simulation results are shown on the right graph.

The experiment graph in Figure 26 shows the shear results summary for FG soil pre-stressed at 0 kPa (tilled), 200 kPa and 400 kPa. Fitting a line through each set of pre-stress shear strength points we get the following fits:

Tilled soil (0 kPa):	$S = 0.497\sigma_N + 7.63$
Pre-stressed @ 200 kPa:	$S = 0.423\sigma_N + 8.33$
Pre-stressed @ 400 kPa:	$S = 0.454\sigma_N + 7.65$

The experiment results show that FG soil unconfined shear strength is nearly independent of prestress. The average soil internal coefficient of friction is about 0.46 and the average cohesive strength is about 7.9 kPa. The calibrated DE model gives the following results:

Pre-stressed @ 20 kPa:	$S = 0.427\sigma_N + 6.99$
Pre-stressed @ 200 kPa:	$S = 0.549\sigma_N + 10.94$
Pre-stressed @ 400 kPa:	$S = 0.447\sigma_N + 12.86$

Note that the 20 kPa is the minimum pressure needed for leveling the surface of the DE soil. Therefore, we use this pressure as an approximation for tilled soil. The DE soil average soil internal coefficient of friction is about 0.47 and the average cohesive strength is about 10.2 kPa.

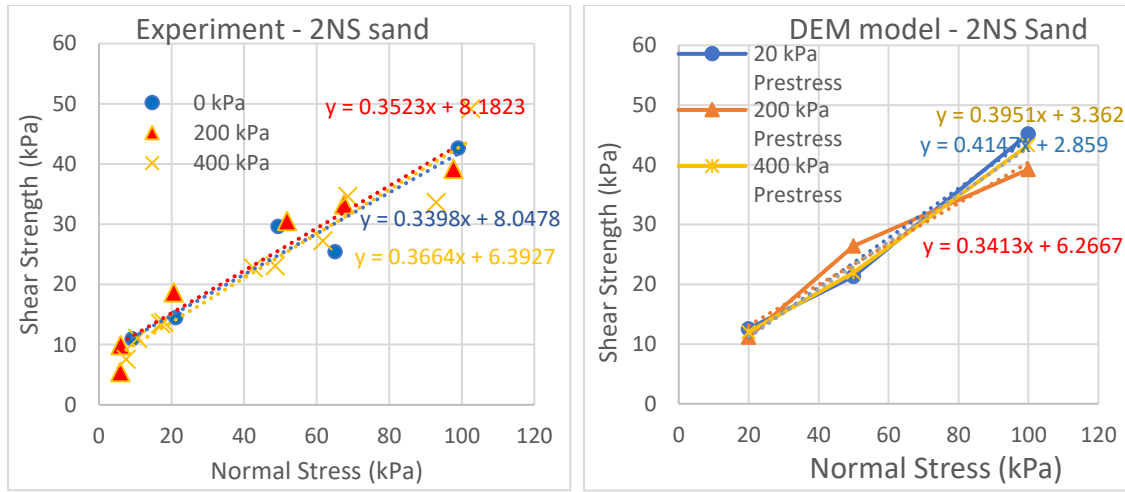


Figure 27: Comparison between experiment and simulation for the 2NS sand @ 0% moisture content unconfined shear test.

The experiment graph in Figure 27 shows the shear results summary for 2NS sand pre-stressed at 0 kPa (tilled), 200 kPa and 400 kPa. Fitting a line through each set of pre-stress shear strength points we get the following fits:

$$\text{Tilled soil (0 kPa):} \quad S = 0.34\sigma_N + 8.04$$

$$\text{Pre-conditioned @ 200 kPa:} \quad S = 0.35\sigma_N + 8.18$$

$$\text{Pre-conditioned @ 400 kPa:} \quad S = 0.37\sigma_N + 6.39$$

The experiment results show that similar to FG soil, 2NS sand unconfined shear strength is also nearly independent of prestress. The average soil internal coefficient of friction is about 0.38 and the average cohesive strength is about 7.5 kPa. The calibrated DE model gives the following results:

$$\text{Pre-stressed @ 20 kPa:} \quad S = 0.41\sigma_N + 2.86$$

$$\text{Pre-stressed @ 200 kPa:} \quad S = 0.34\sigma_N + 6.03$$

$$\text{Pre-stressed @ 400 kPa:} \quad S = 0.39\sigma_N + 3.36$$

The DE model soil average soil internal coefficient of friction is about 0.39 and the average cohesive strength is about 4.08 kPa.

5 Validation of the DE soil model

The following tests are used to validate the calibrated DE soil models for FG soil and 2NS sand:

1. Confined shear.
2. Cone penetrometer.
3. Disc penetrometer.
4. Single tire drawbar-pull.

5.1 Confined Shear Test

The DE model simulations (Figure 16) for the confined shear cell for tilled FG soil and 2NS sand are compared to available experimental results. Typical shear stress versus shear displacement experiment

and simulation results for a normal stress of 50 kPa for FG soil and 2NS sand are shown in Figure 28 and Figure 29, respectively. The shear strength under the specified applied normal stress is the maximum shear stress in the range of the shear cell displacement.

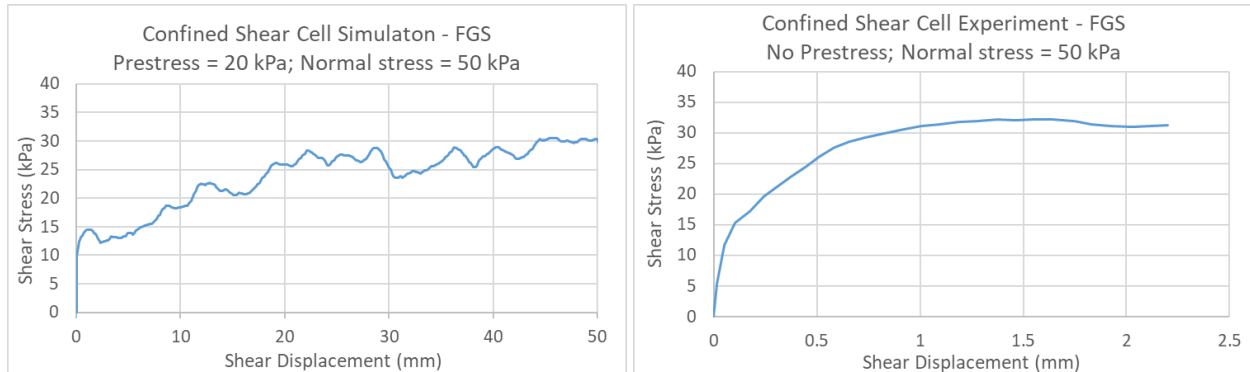


Figure 28: Typical confined shear cell results showing the simulation (left) and experiment (right) shear stress versus shear displacement at 50 kPa normal stress for FG soil.

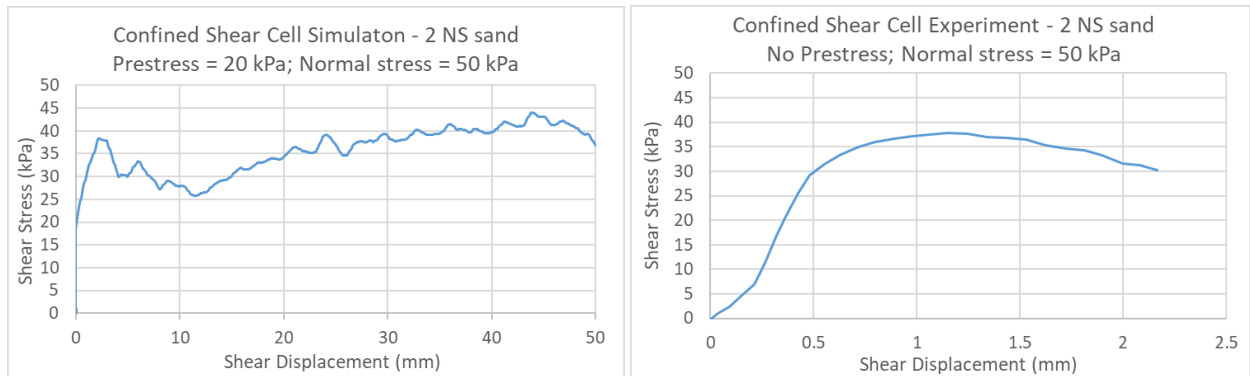


Figure 29: Typical confined shear cell results showing the simulation (left) and experiment (right) shear stress versus shear displacement at 50 kPa normal stress for 2 NS sand.

Figure 30 show a comparison of shear strength versus applied normal stress between simulation and experiment for FG soil. In the experiment the soil was not prestressed (i.e. tilled soil). In the simulation, three prestress values are used 20, 200, and 400 kPa. The 20 kPa simulation shear strength line plot is compared with the experiment line plot. Up to a normal stress of 50 kPa the simulation and experiment results match relatively well. Comparing the unconfined soil shear strength results in Figure 26 to the confined shear cell simulation results in Figure 30, we note the following:

- For tilled soil, the confined and unconfined shear strengths are nearly same at 50 kPa normal stress for the simulation and experiment. The soil cohesion in the experiment is about 1 kPa while in the simulation it is about 10 to 15 kPa. This maybe due to the fact that in the simulation the soil was compacted using a 20 kPa prestress to level the soil surface.
- For the confined shear cell, the prestress increases the soil shear cohesive strength and internal friction coefficient while prestress has no effect on soil shear strength in the unconfined shear test. This increase in cohesive strength and internal friction is due to the compressibility of the FG soil. When the FG is compacted (prestressed) and confined, its shear strength is higher than compacted soil that is not confined.

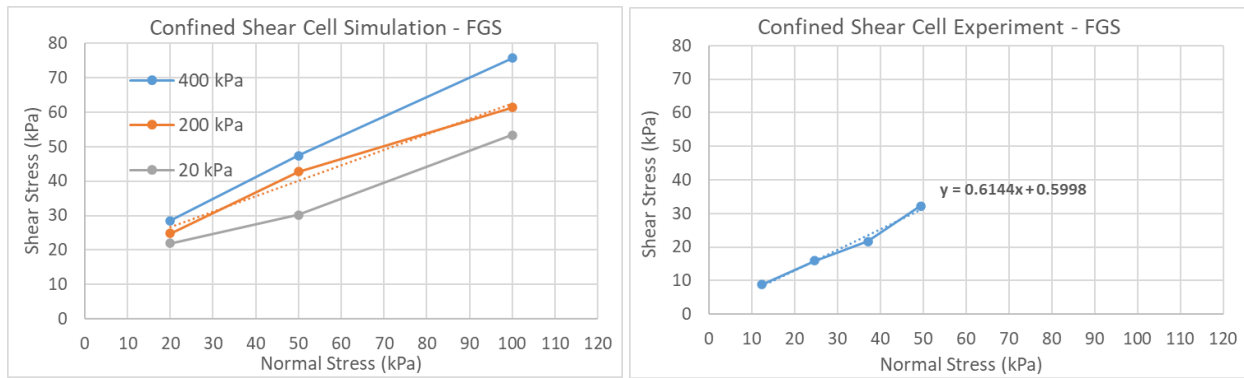


Figure 30: Comparison between the confined shear cell simulation and experiment results showing the shear strength versus the applied normal stress for fine grain soil. The simulation results also show the effect of soil prestress at 20, 200, and 400 kPa.

Figure 31 show a comparison of shear strength versus applied normal stress between simulation and experiment for 2NS sand. In the experiment the soil was not prestressed (i.e. tilled soil). In the simulation, three prestress values are used 20, 200, and 400 kPa. The 20 kPa simulation shear strength line plot is compared with the experiment line plot. Up to a normal stress of 50 kPa the simulation and experiment results match relatively well. Comparing the unconfined soil shear strength results in Figure 27 to the confined shear cell simulation results in Figure 31, we note the following:

- For both confined and unconfined shear tests, the cohesive strength and internal friction of the soil is independent of the initial soil prestress.
- The confined shear test 2NS sand internal friction coefficient is at least 0.64 while the unconfined shear test internal friction coefficient is about 0.39. This is due to the fact that 2NS sand is nearly incompressible, so when it is sheared in a confined shear cell the motion of the soil particles is restricted thus considerably increasing the soil shear strength. Note that in vehicle mobility applications the soil while being shear by the tire or track is nearly unconfined. Thus, using the confined shear strength for incompressible or compacted soils frictional leads to overestimation of the soil shear strength.

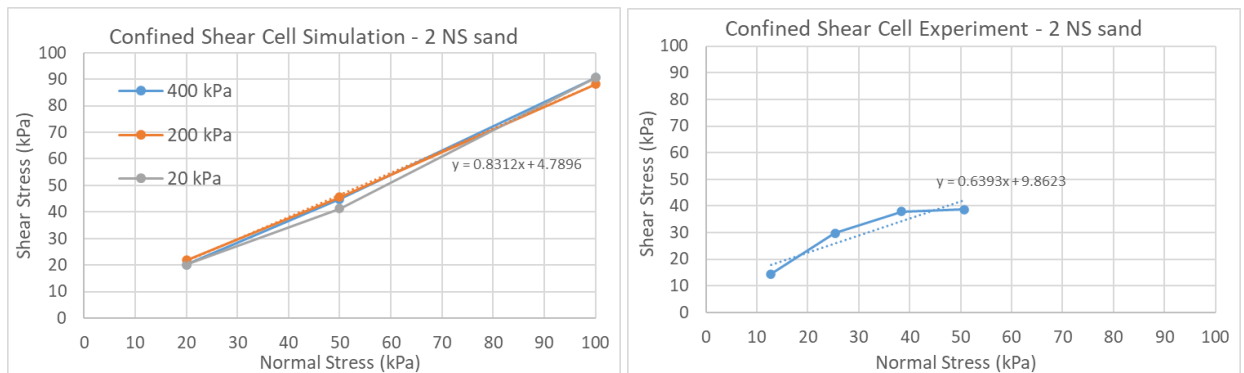


Figure 31: Comparison between the confined shear cell simulation and experiment results showing the shear strength versus the applied normal stress for 2 NS sand. The simulation results also show the effect of soil prestress at 20, 200, and 400 kPa.

The comparison between the unconfined and confined shear test results for both the experiment and simulation (Figure 30 and Figure 31) show that for vehicle mobility applications it is necessary to perform the soil shear test in an unconfined configuration since the confined shear cell grossly overestimates the soil strength if the soil is incompressible and can even lead to an incorrect evaluation of soil internal friction and cohesion if the soil is compressible.

5.2 Disc penetrometer

Disc penetrometer tests are performed using 39, 60, and 80 mm diameter disc penetrometers (Figure 18) at 0.5 mm/s and 6 mm/s vertical speeds on soils pre-stressed at 100 kPa and 400 kPa. The disc penetrometer soil tests are performed in a 300 mm diameter and 285 mm deep cylindrical steel box. Every test was repeated twice to check repeatability of results. The simulations are performed using a 120 mm diameter disc. Since the DE particle size is 30 mm, using a smaller disc diameter will result in a very noisy response due to too few particles under the disc. Each simulation was repeated three times with slightly different particle initial positions in order to find the range of response of the DE model. The two key values that determine the strength of the soil are:

- The initial penetration stress which is the maximum penetration stress in the first 5-10 mm of sinkage.
- The slope of the penetration stress versus sinkage curve.

The disc penetrometer experiment results for FG soil are shown in Figure 32 and the simulation results are shown in Figure 33. From those figures, the following observations can be made:

- The FG soil disc penetrometer curves, especially the penetration curves slopes, are generally not sensitive to penetration speed for the range of chosen (slow) speeds. The initial penetration stress is generally lower for fast penetration speeds.
- The initial stress and curve slope increase with soil prestress.
- For 100 kPa prestressed soil, the results for the 39 mm and 60 mm cones are similar. While for the 80 mm disc the slope of the pressure versus sinkage curve is slightly higher than the 39 mm and 60 mm cone results.
- For 400 kPa prestressed soil, the three disc sizes give a similar pressure versus sinkage curve.
- The simulation results give slightly higher initial stress for both 100 kPa and 400 kPa prestressed soils. The slopes of the experiment and simulation curves are generally similar for both 100 kPa and 400 kPa prestressed soils.

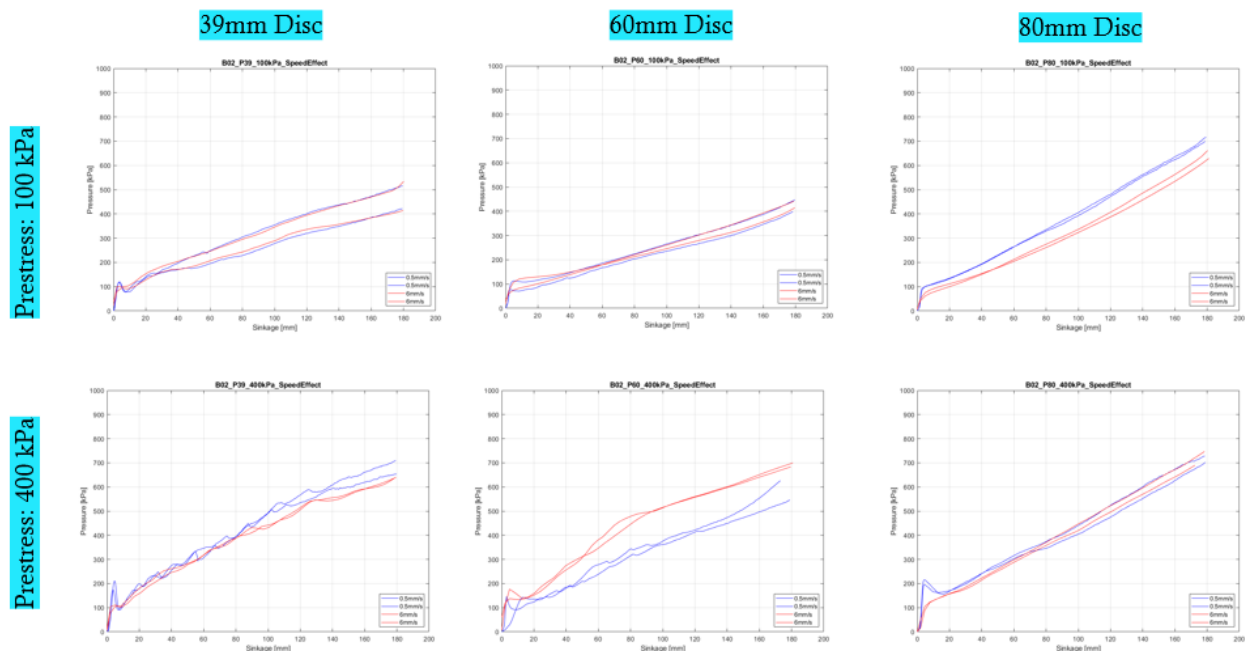


Figure 32: P39, P60 and P80 penetrometer results of FGS soil pre-conditioned at 100 and 400 kPa performed at 0.5mm/s and 6mm/s speeds.

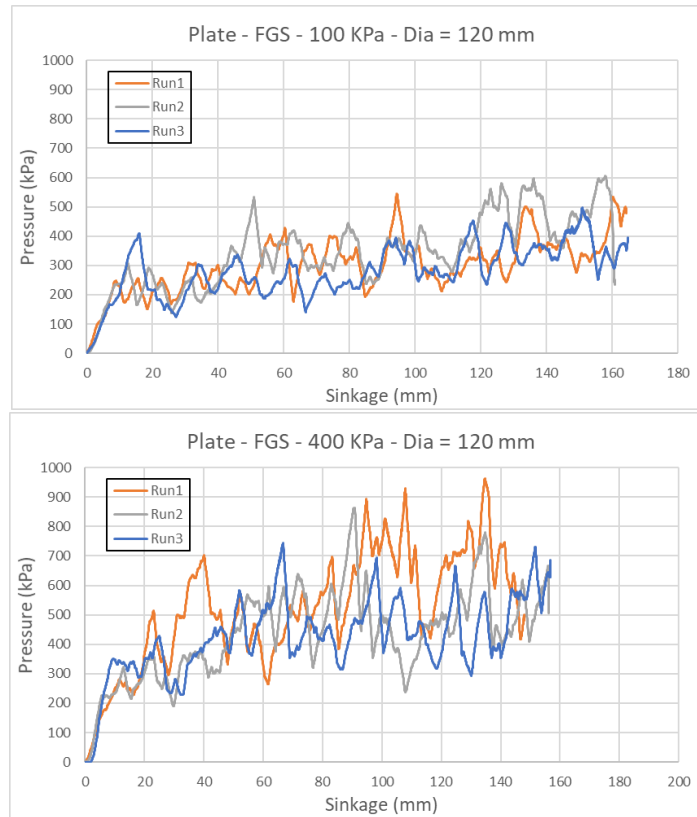


Figure 33: 12 cm disk penetrometer simulations results for FG soil pre-conditioned at 100 and 400 kPa with a penetration speed of 1 cm/s.

The disc penetrometer experiment results for 2NS sand are shown in Figure 34 and the simulation results are shown in Figure 35. From those figures, the following observations can be made:

- The 2NS sand disc penetrometer curves, especially the penetration curves slopes, are generally not sensitive to penetration speed for the range of chosen (slow) speeds. The initial penetration stress is generally slightly lower for fast penetration speeds.
- The initial stress and curve slope are insensitive to soil prestress for both the experiment and the simulation.
- The experiment initial stress and curve slope increase with the disc diameter.
- The simulation initial stress is about the same as the experiment. However, the simulation curves slopes are generally lower than the experiment slopes.

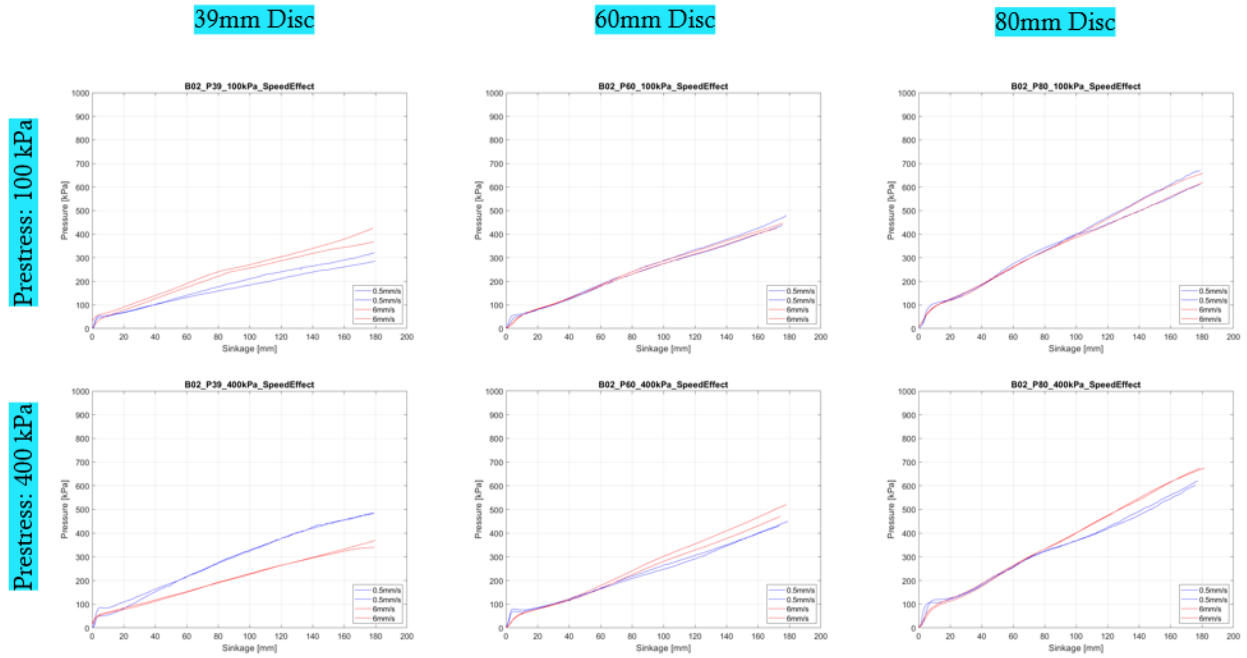


Figure 34: P39, P60 and P80 penetrometer results of 2NS sand pre-conditioned at 100 and 400 kPa performed at 0.5mm/s and 6mm/s speeds

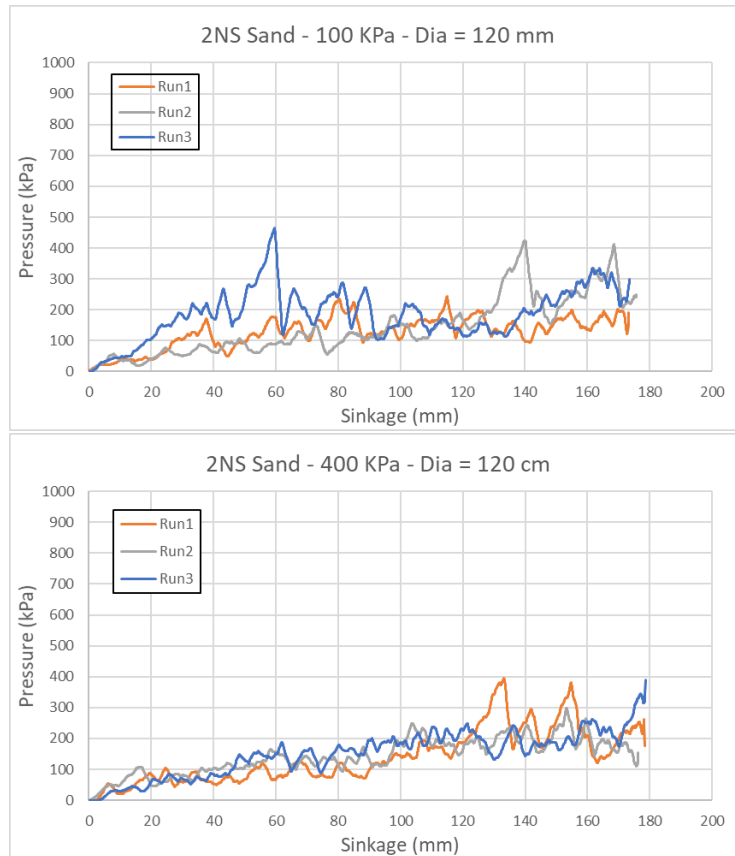


Figure 35: 12 and 16 cm disk penetrometers simulations results for 2NS sand pre-conditioned at 100 and 400 kPa with a penetration speed of 1 cm/s.

5.3 Cone penetrometer

The penetration stress (cone index) as a function of penetration depth is used in the empirical cone index method for predicting vehicle mobility [45, 46]. Cone Penetrometer tests are performed using 30° apex angle cones with 30 mm and 60 mm base diameters (Figure 18) at 2 mm/s speed on soils pre-stressed at 0 (tilled), 100 kPa and 400 kPa. The penetration soil tests are performed in a 300 mm diameter and 285 mm deep cylindrical steel box. Every test was repeated twice to check repeatability of results. The simulations are performed using a 120 mm base diameter cone. Since the DE particle size is 30 mm, using a smaller cone diameter will result in a very noisy response due to too few particles in contact with the cone. Each simulation was repeated three times with slightly different particle initial positions in order to find the range of response of the DE model. The two key values that determine the strength of the soil are:

- The initial penetration stress is the stress when the base of the cone level with the soil's surface. The zero sinkage point in the stress versus sinkage curves is when the base of the cone level with the soil's surface. Therefore, the initial penetration stress is the stress at zero sinkage.
- The slope of the penetration stress versus sinkage curve.

The cone penetrometer results for FG soil are shown in Figure 36 and the simulation results are shown in Figure 37. From those figures, the following observations can be made:

- The experiment initial stress increases with soil prestress. It ranges from 50 to 100 kPa for the 30 mm diameter cone and from 120 to 190 kPa for the 60 mm diameter cone.
- As sinkage increases the stress also increases mostly linearly. There seem to be two distinct slopes for the pressure versus sinkage line plot corresponding to two modes of soil response: a compliant mode and a stiff mode. We plan to investigate those modes in future papers. The stiff mode seems to be the main mode for the 60 mm diameter cone.
- The initial stress increases with prestress especially from 100 kPa to 400 kPa for both the simulation and experiment. However, the curves slopes seem to be independent of soil prestress for both the simulation and experiment.
- The simulation initial stress is slightly higher than the experiment. However, the simulation curves slope is much lower than the experiment slopes. We will further investigate the reason for the slope difference in future papers.

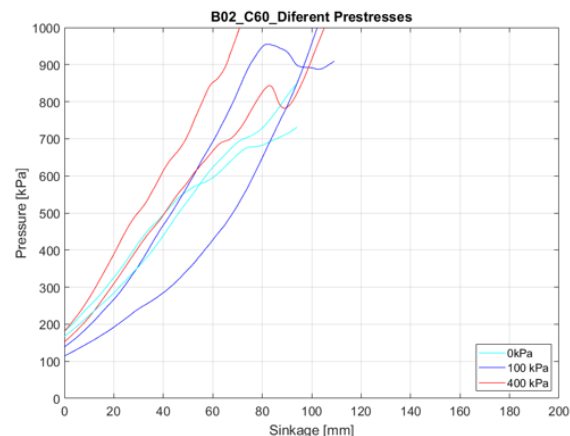
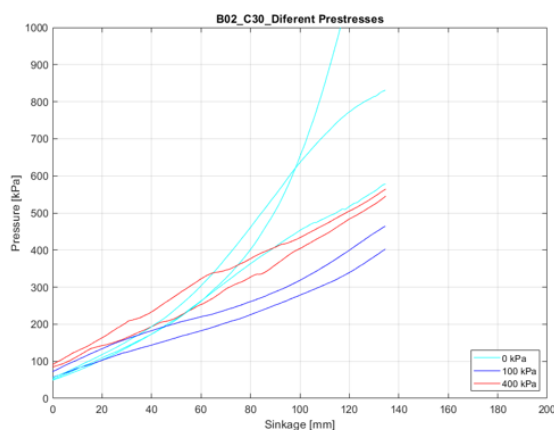


Figure 36: 30mm base cone (left) and 60 mm base cone (right) penetrometer results of FG soil prestressed at 0, 100 and 400 kPa at penetration speed at 2mm/s

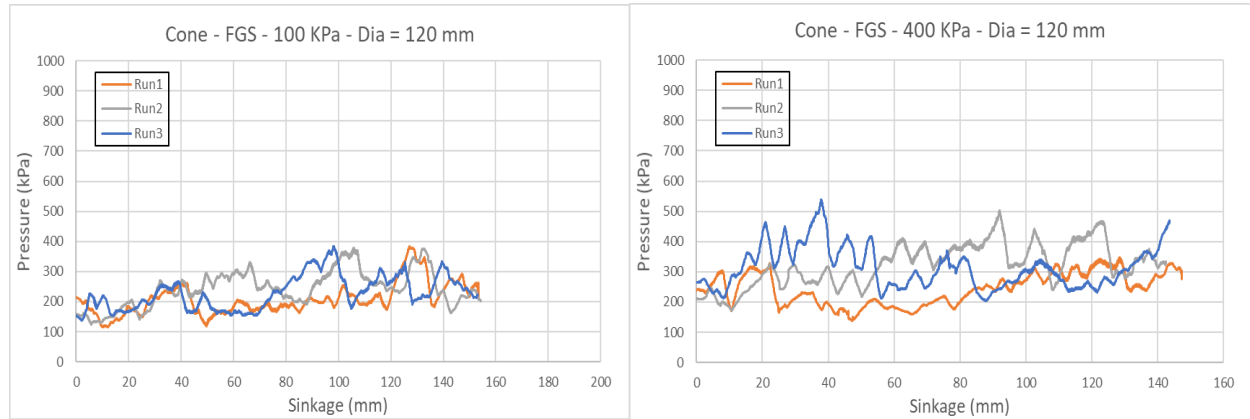


Figure 37: Cone penetrometer simulation results for a 120 mm base diameter cone for FG soil prestressed at 100 (left) and 400 (right) kPa prestress at a penetration speed of 1 cm/s.

The cone penetrometer results for 2NS sand are shown in Figure 38 and the simulation results are shown in Figure 39. From those figures, the following observations can be made:

- The experiment initial stress increases with soil prestress. It is about 50 kPa for the 30 mm diameter cone and from 90 to 100 kPa for the 60 mm diameter cone.
- As sinkage increases the stress also increases mostly linearly. There seem to be two distinct slopes for the pressure versus sinkage line plot corresponding to two modes of soil response: a compliant mode and a stiff mode. We plan to investigate those modes in future papers.
- The initial stress and slope are insensitive to prestress especially for both the simulation and experiment.
- The simulation initial stress is about the same as the experiment's 60 mm diameter cone results. The simulation curves slope are slightly lower than the experiment slopes for the compliant response mode.

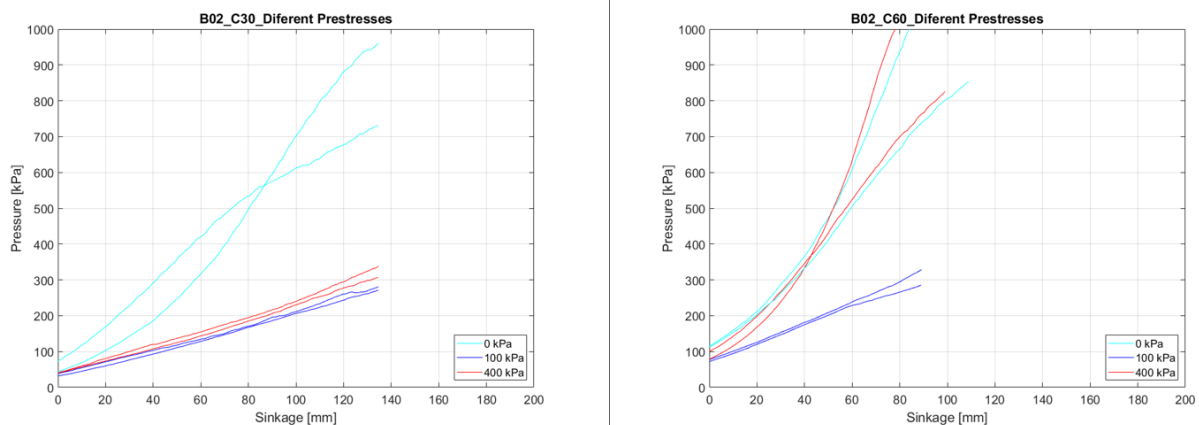


Figure 38: C30 and C60 cone penetrometer test results for 2NS sand pre-conditioned at 0, 100 and 400 kPa at a penetration speed of 2 mm/s.

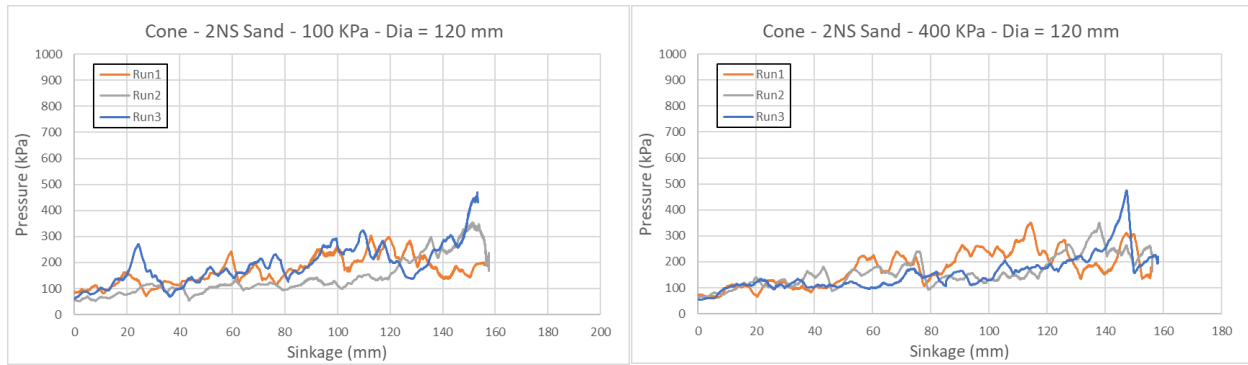
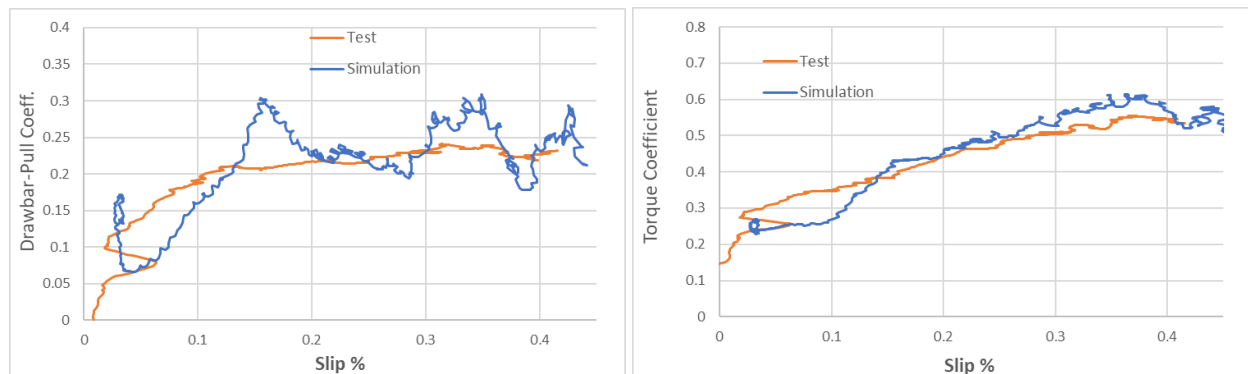


Figure 39: Cone penetrometer simulation results for a 16 cm diameter cone for 2NS sand pre-conditioned at 100 (left) and 400 (right) kPa prestress at a penetration speed of 1 cm/s.

5.4 Single Tire Test

Figure 40 shows a comparison of single tire drawbar-pull normalized results between the experiment and simulation for FG soil. Figure 41 shows a snapshot of the tire simulation. The tire inflation pressure is 35 psi. The tire FE model was presented in Ref. [2], however in this simulation shell element instead of brick elements are used to model the tire's rubber matrix. The tire meridian and circumference reinforcements are modeled using a combination of truss and beam elements. The drawbar-pull coefficient is defined as the ratio of the drawbar-pull force and applied tire normal load. The torque coefficient is defined as the ratio of wheel torque and the product of the nominal tire radius and the applied normal force. The normalized sinkage is the actual tire sinkage (which includes tire deflection and soil sinkage) divided by the tire radius. Figure 40 shows the following:

- The experiment and simulation drawbar-pull coefficient compare well with the simulation drawbar-pull coefficient being slightly higher on average.
- The experiment and simulation torque coefficient approximately the same.
- The simulation tire sinkage is lightly higher than the experiment sinkage.



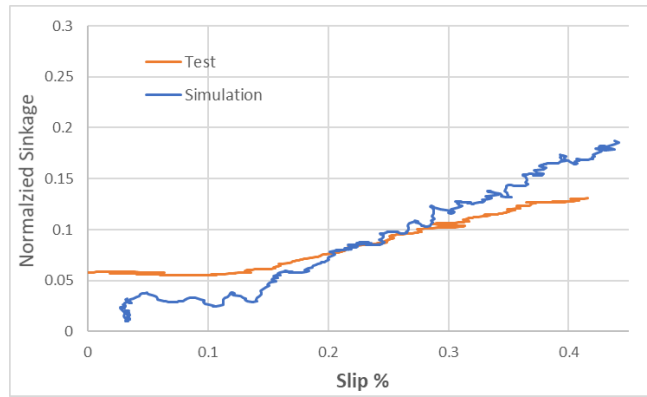


Figure 40: Comparison between simulation and test of drawbar-pull force, torque, and sinkage for the drawbar-pull test on FG soil for a linear speed of 0.9 m/s.

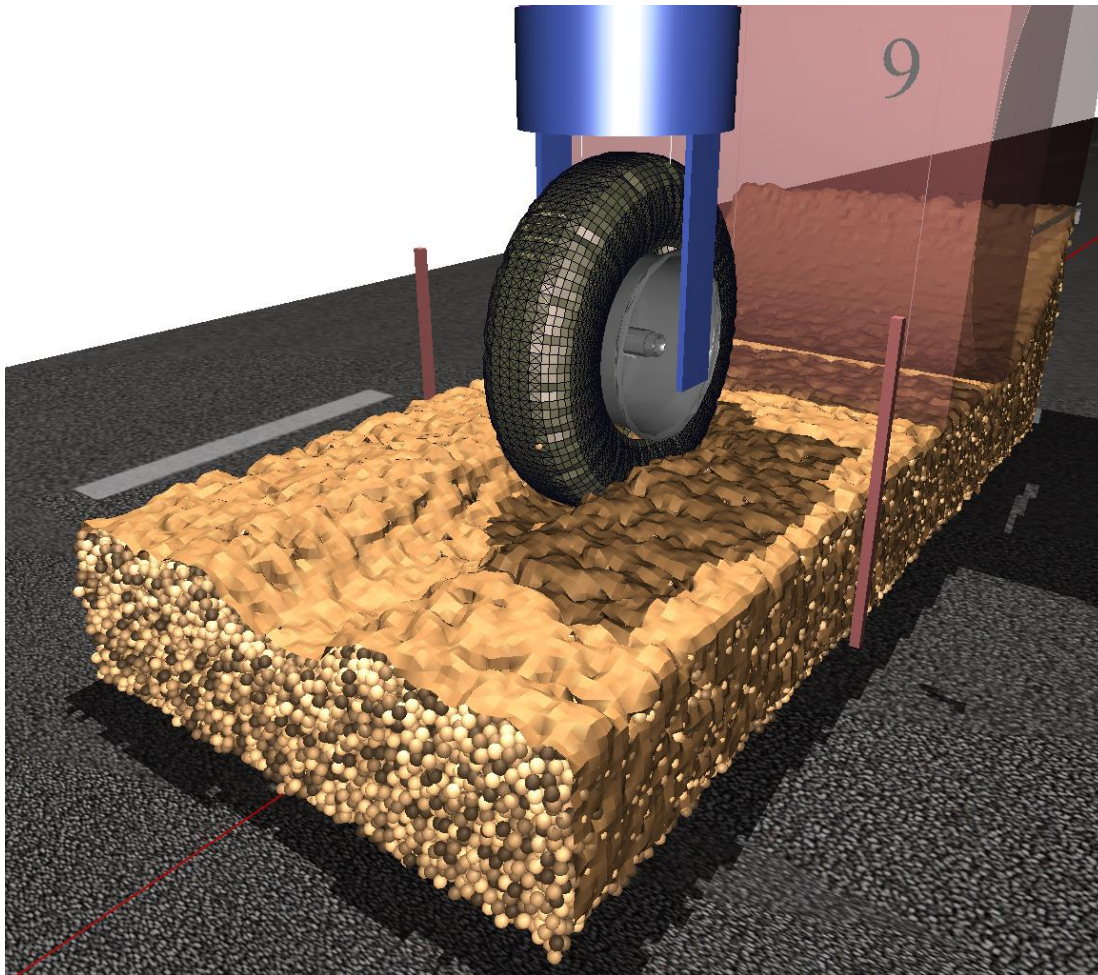


Figure 41: Snapshot of the single tire drawbar-pull simulation on FG soil.

6 Concluding remarks

A method for calibration of a DE computational soil model for use in prediction of ground vehicle mobility was presented. In this method two physical soil experiments are used: (1) hydrostatic compression; and (2) unconfined shear strength under different levels of normal stress and normal pre-stress. The hydrostatic compression test is used to calibrate the DE bulk density, plastic strain, and elastic strain as a

function of hydrostatic stress. The unconfined shear strength test is used to calibrate the DE inter-particle friction coefficient and adhesion stress as a function of the soil plastic strain. The DE model calibration procedure was demonstrated using two types of soil: dry sand-silt and dry silt-clay. The calibrated DE model was then validated using the confined shear, disc penetrometer, cone penetrometer, and single tire test results for those two types of soil. Some of the main conclusions of the paper are:

- FG soil is compressible while 2NS sand is nearly incompressible at up to 400 kPa hydrostatic pressure.
- For FG soil, prestress increases the confined shear strength which includes both the shear cohesive strength and internal soil friction coefficient but does not affect its unconfined shear strength. This increase in cohesive strength and internal friction for confined soil is due to the fact that the soil is compressible. So when FG soil is compacted (prestressed) and confined, its shear strength is higher than compacted soil that is not confined.
- For 2NS sand, the confined shear internal friction coefficient is at least 0.64 while the unconfined shear internal friction coefficient is about 0.39. This is due to the fact that 2NS sand is nearly incompressible, so when it is sheared in a confined shear cell the motion of the soil particles is restricted thus considerably increasing the soil shear strength. In vehicle mobility applications the soil while being sheared by the tire or track is nearly unconfined. Thus, using the confined shear strength for incompressible or compacted soils leads to overestimation of the soil shear strength.
- In penetrometer tests, stress versus penetration is generally independent of penetration speed, as long as the penetration speed is very slow such that inertia and other dynamic effects are negligible.
- For FG soil, the stress vs penetration curve initial stress and slope increase with soil prestress while for 2NS sand the initial stress and slope are insensitive to soil prestress. This again can be explained by the fact that FG soil is compressible while 2NS sand is nearly incompressible.

Future work will focus on the following:

- Studying the effect of moisture on the soil mechanical response.
- Investigating the causes of different soil stiffness modes in cone and disk penetrometer tests.
- Conducting more extensive validation of the DE model using the single tire test including: static pressure-sinkage, drawbar-pull, and rolling resistance as a function of applied normal load, tire pressure, soil type, longitudinal speed, and tire steering (slip) angle.
- Use penetrometer drop tests to calibrate the damping and viscosity of the DE soil model.

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