

# Numerical and Experimental Studies of Sand-Clay Interface

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**Abstract.** A recent experimental work of Frikha et al. (2015) demonstrated that when the number of granular columns increases at a fixed area replacement ratio “ $\eta$ ”, the contact surface between the reinforcing columns and the clayey soil increases and results in an increase in the soil-column interface friction, which enhances the strength properties of the reinforced soft clayey soil. Granular Columns/soil interface strength can play then an important role in the bearing capacity of reinforced soil. However, it would appear that none of the methods proposed of design of granular column to date are able to reliably predict performance to a satisfactory level of accuracy due to the intrinsic characteristics of interface properties Columns/soil.

The present paper try to carry out into the shear behavior of sand/clayey soil interface using a simple direct shear test. In the shear test a shearing surface is produced and the resistance to shearing is defined as the shear resistance. The frictional resistance is, however, determined on an existing joint surface. The frictional behavior at the soil – other materials interface is usually and commonly obtained from direct shear tests. A numerical modelling analysis of direct shearing box test using FLAC3D was performed in order to define the interface parameter by comparing the experimental results with the numerical one. The principal purpose of this work is to define the properties of the interface for a future numerical simulation of behavior of clayey soil reinforced by granular column.

## 1 Introduction

Modeling of soil-structure interaction is very important in geotechnical engineering including hydraulic structures. It is relevant to a wide range of project problems, such as numerical analysis of concrete-faced rock fill dams, retaining walls, shallow foundations, piles, tunnels, reinforced earthworks, and geosynthetic liners. The relationship between shear stress and shear displacement of the soil-structure interface plays a major role in modeling soil-structure interactions (Wu et al. 2011).

Several investigations were conducted for determining the bearing capacity of an isolated column by considering several methods (Bouassida and Hadhri 1995) including the state of stress (Aboshi et al. 1979), a failure mechanism combined with a state of stress (Greenwood 1970; Hughes et al. 1975; Datye 1982; Balaam and Booker 1985; Frikha and Bouassida 2015), or a failure mechanism only is used (Bouassida and Jellali 2002).

Hu (1995) built a laboratory-scale model to examine the behavior of a cohesive soft soil reinforced by a group of granular columns supporting a rigid footing. He found that reinforcement by a group of columns is more effective than that by an isolated column. The interaction between the columns and the soft clay was found to efficiently contribute to the enhancement of the load bearing capacity and to provide a wider transfer of loading. Bachus and Barksdale (1984) concluded that there is only a slight increase in the ultimate load bearing capacity per column when the number of columns increases. However, in their tests, the effect of lateral confinement was more significant since the reinforcing columns were set close to the borders of the testing box. Wood et al. (2000) commented on the tests performed by Hu (1995) and confirmed that the mode of failure for each column, in the case of group-column reinforcement, depends on its location within the group, its length, and the type of loading. Their results showed that the pre-failure mechanisms and the failure modes of a granular-column group are different from those observed for a single column. They reported that the area replacement ratio affects the extent of the columns' interaction and the load transferred to the soft clay in between the columns. This research concluded that a significant improvement in the bearing capacity depends on a minimum area replacement ratio of 25%.

A recent experimental work performed by Frikha et al. (2015) demonstrate that when the number of columns increases at a fixed area replacement ratio " $\eta$ ", the contact surface between the reinforcing columns and the clayey soil increases and results in an increase in the soil-column interface friction, which enhances the strength properties of the reinforced soft clayey soil.

Granular columns/soil interface strength can play then an important role in the bearing capacity of reinforced soil. However, it would appear that none of the methods proposed of design of granular column to date are able to reliably predict performance to a satisfactory level of accuracy due to the intrinsic characteristics of interface properties granular columns/soil.

The present paper tries to carry out into the shear behavior of granular Column/Clayey soil interface using a simple direct shear test (Fig. 1). In the shear test a shearing surface is produced and the resistance to shearing is defined as the shear resistance. The frictional resistance is, however, determined on an existing joint surface. The frictional behavior at the soil– other materials (e.g. geosynthetic, geogrid, tire shreds, rubber chips, geofam, polymer, etc.) interface is usually and commonly obtained from direct shear tests (O'Rourke et al. 1990; Bernal et al. 1997; Lee and Manjunath 2000; Xenaki and Athanasopoulos 2001; Bergado et al. 2006; Liu et al. 2009; Palmeira 1988; Anubhav and Basudhar 2010).

The principal purpose of this work is to define the properties of the interface for a future numerical simulation of behavior of clayey soil reinforced by granular column. There are in fact several instances in geomechanics in which it is desirable to represent planes on which sliding or separation can occur. Include for example, the cases of joint, fault or bedding planes in a geologic medium; an interface between a foundation and

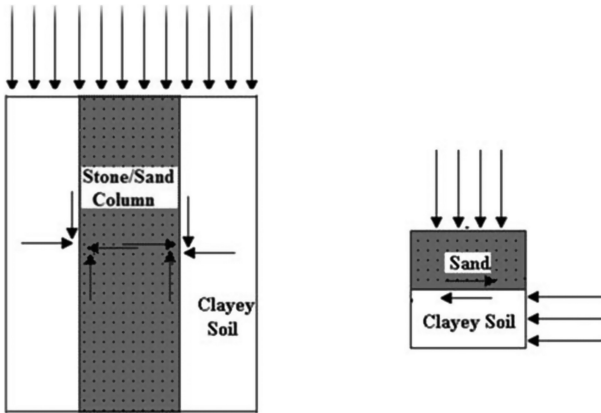


Fig. 1. Proposal model

the soil; a contact plane between a bin or chute and the material that it contains; a contact between two colliding objects; and a planar “barrier” in space, which represents a fixed, non-deformable boundary at an arbitrary position and orientation.

The sample preparation, test method, and test results of an experimental program of direct shear testing in the interface between two layers of sand/clayey are undertaken. A simulation of performed model using finite-difference code (Itasca FLAC3D) is also conducted and interpreted.

## 2 Studied Soil

The tested soft soil is grey in color, contains shell debris, and has a characteristic smell. The properties of Tunis soft soil are a liquid limit of  $\omega_l = 66.5\%$ , a plastic limit of  $\omega_p = 23.9\%$ , a plasticity index of  $IP = 42.6\%$ , a natural water content of  $\omega = 74\%$ , a compression index of  $C_c = 0.64$ , and an oedometer modulus of  $E = 1700$  kPa. According to the French Standard XP P 94-011 classification, the tested soft soil is classified as a highly plastic clay.

The material used is siliceous sand classified as CEN in accordance with the European standards EN 196-1. The sand particles are generally isometric and rounded in shape. The coefficient of uniformity of the CEN sand is  $C_u = 6$  and its dry unit weight is  $\gamma_d = 16$  kN/m<sup>3</sup>.

The grain size distributions of Tunis soft soil and CEN Standard sand are depicted in Fig. 1. It is noticed that 60% of the particles of Tunis soft soil are smaller than  $2 \mu\text{m}$  and 98.3% are smaller than  $80 \mu\text{m}$ . As seen in Fig. 2, the mean particle diameter,  $D_{50}$ , of the CEN Standard sand is 0.7 mm.

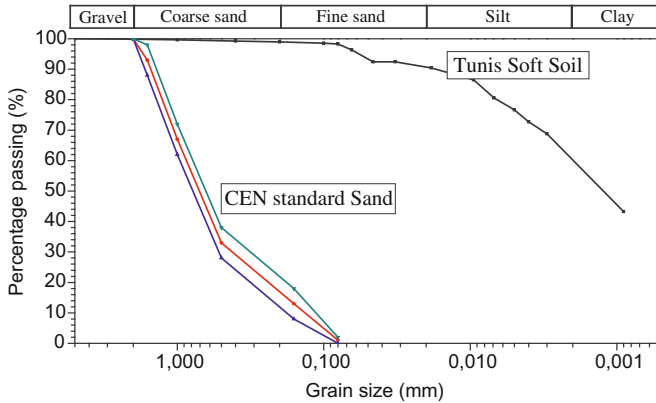


Fig. 2. Grain size distribution of used clayey and sandy soil

### 3 Experimental Procedure

The first step consists of wet sieving the natural soft soil through a 100- $\mu\text{m}$  sieve. Then, the sieved soft soil is air-dried until its moisture content reaches 100%, which represents approximately 1.5  $\omega$ .

The second step is to subject the slurry material to a vertical pre-consolidation pressure of  $K_0$  in a special mould (consolidometer container). The consolidometer is a metallic box, 6  $\times$  6 cm in surface and 20 mm in height (Fig. 3).

The inner surface of the consolidometer is lubricated with silicone grease prior to the slurry placement in order to reduce the effects of roughness, and therefore, allow an easy unloading of the specimens. The inner surface of the consolidometer is also lined with filter paper to accelerate the consolidation. The slurry is then carefully placed into the mould, using a spoon, where it is frequently tamped with a plastic tamper to avoid the formation of air during the filling phase. When the predetermined specimen height

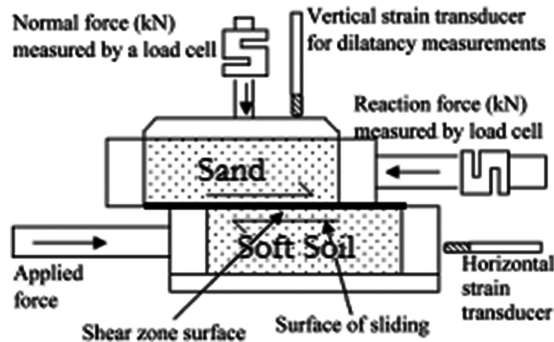


Fig. 3. Used shear box

is attained, the consolidation loading cap is placed to start the loading phase. The specimens are consolidated under a vertical stress of 54 kPa for at least seven days. At the end of the consolidation phase, the specimen height is 1,2 cm. During the pre-consolidation phase, drainage is allowed at the top and bottom of the specimen through the installed upper and lower porous stones. A dial gauge on the loading cap is used to measure the axial deformation of the tested specimen. The consolidation is considered to be complete when quasi-constant axial deformation is recorded. The pressure is then reduced to zero and the loading cap is removed. The soft clay sample is placed in the shear lower half-box.

The sample is then cut at the shear plane of lower half-box by means of a steel wire to obtain a final height of 1 cm. the upper half-box is then fixed so as to secure the two half boxes and is filled with a dry sand in small layers (20 gr of sand per layer), in order to attain the final height of sand layer (10 mm). Each layer is compacted by a plastic tamper. The same weight (20 gr) of dense compacted sand is considered in all tests.

The obtained sample composed of two layers is then subjected to direct shear stress in a shear box equipment (Fig. 3) under a velocity of  $V = 0.5$  mm/min and a normal stresses of  $\sigma_N = 54.8$  kPa, 67.4 kPa and 80.0 kPa.

## 4 Experimental Results

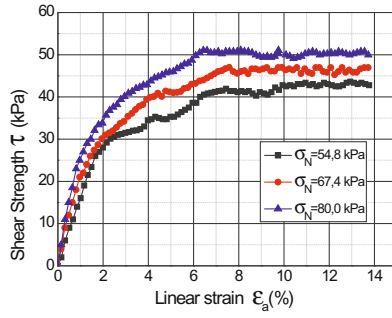
Figures 4 present typical results of the direct shear box tests carried out on (i) reconsolidated soft soil, (ii) normalized sand and (iii) sample composed from two layers (Sand and Clay) subjected to different normal stresses. These figures show the variations in shear stress,  $\tau = F/S$ , with respect to the calculated linear strain under undrained conditions. Linear strain  $\epsilon_a$  is calculated as the measured displacement divided by the specimen's initial length  $\epsilon_a = \delta l/l$  (%).

For the case of the Tunis soft soil, shear stresses are quasi-independent of normal stresses.

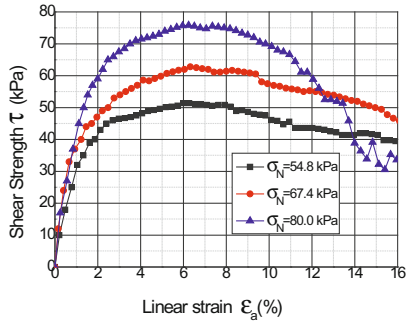
Figure 5 show the obtained results of shear stress *vs* a function of applied Normal stress for (i) Clay-Clay (ii) Sand-Sand and (ii) Clay-Sand. In the three cases the shear stress is evolution quasi-linear with the normal stress. The obtained cohesion “*c*” is 34.0, 23.7 and 0 for Clay-Clay, Sand-Sand and Clay-Sand respectively. Whereas the friction angle “ $\varphi$ ” is 9.44°, 27.6° and 43.1° for Clay-Clay, Sand-Sand and Clay-Sand respectively. Both the cohesion and the friction angle derived from performed test on interface clay-sand are between those obtained for clay-clay and sand-sand interfaces.

## 5 Numerical Modelling and Analysis

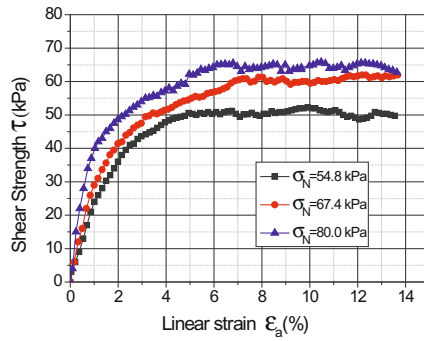
FLAC3D provides interfaces that are characterized by Coulomb sliding and/or tensile and shear bonding. Interfaces have the properties of friction, cohesion, dilation, normal and shear stiffness, and tensile and shear bond strength. FLAC3D represents interfaces as collections of triangular elements (interface elements), each of which is defined by three nodes (interface nodes) (Figs. 6, 7 and 8).



(a)



(b)



(c)

**Fig. 4.** Results of shear tests performed on (a) Clay-Clay (b) Sand-Sand and (c) Clay-Sand

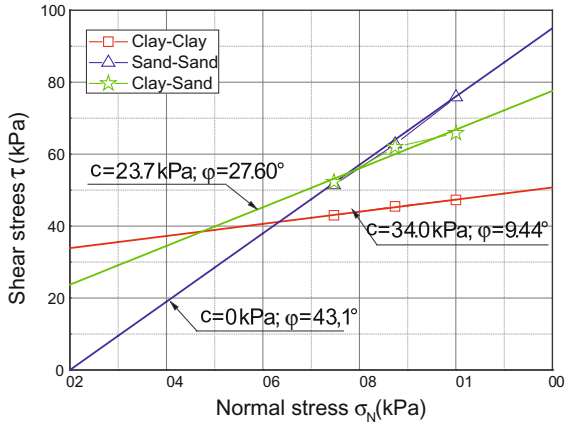


Fig. 5. Shear stress versus normal stress for (i) Clay-Clay (ii) Sand-Sand and (ii) Clay-Sand

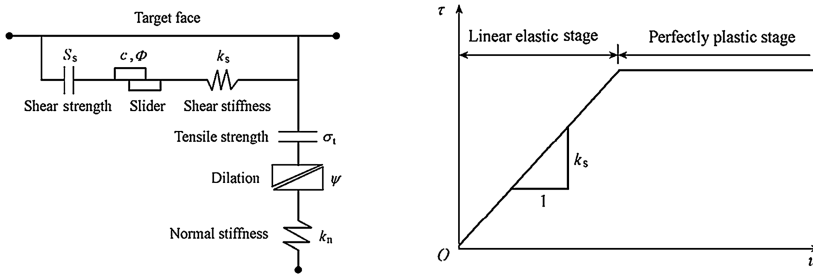


Fig. 6. Theory of interface element implemented in FLAC3D

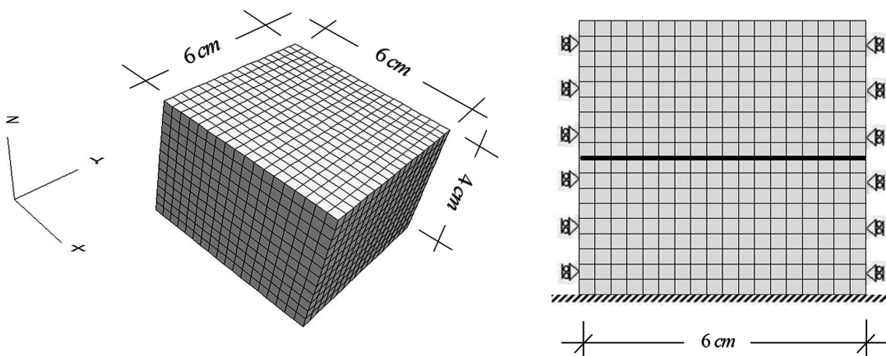
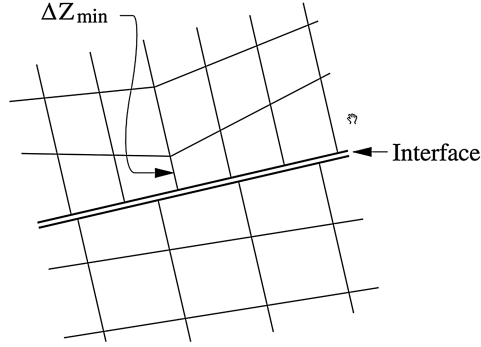


Fig. 7. Grid of used numerical model



**Fig. 8.** Zone dimensions used in stiffness calculation

The normal and shear forces that describe the elastic interface stage response are determined at calculation time  $(t + \Delta t)$  the following equations provided by FLAC3D (FLAC3D Online Manual 2005):

$$F_n^{(t+\Delta t)} = (k_n u_n + \sigma_{n0})A, F_{si}^{(t+\Delta t)} = \left( F_{si}^{(t)} + k_s \cdot \Delta u_{si}^{(t+\Delta t/2)} + \sigma_{si} \right) A \quad (1)$$

Where  $F_n^{(t+\Delta t)}$  is the normal force at the calculation time  $t + \Delta t$ ;  $k_n$  is the normal stiffness of interface element;  $u_n$  is the normal penetration of the interface node into the target face;  $\sigma_{n0}$  is the normal effective stress added due to interface stress initialization;  $A$  is the representative area associated with the interface node;  $F_{si}^{(t+\Delta t)}$  and  $F_{si}^{(t)}$  are the shear forces at calculation times  $t + \Delta t$  and  $t$  respectively;  $k_s$  is the shear stiffness of interface element, which is a constant in the elastic stage;  $\Delta u_{si}^{(t+\Delta t/2)}$  is the incremental shear displacement between  $t + \Delta t$  and  $t$ ; and  $\sigma_{si}$  is the vector of additional shear stress due to interface stress initialization (FLAC3D Online Manual 2005).

According to the Mohr-Coulomb criteria, the yield relationships in the shear and normal directions are:

$$F_{Smax} = CA + \text{Tan}\varphi(F_n - pA), F_n = \sigma_t \quad (2)$$

where  $F_s \text{ max}$  is the maximum shear strength,  $c$  is the cohesion of the interface,  $\varphi$  is the friction angle of the interface,  $F_n$  is the normal force,  $p$  is the pore pressure, and  $\sigma_t$  is the normal tensile strength of the interface.

If the interface does not have dilation characteristics, the forces are corrected as follows:

$$F_{si} = F_{s \text{ max}} \quad (3)$$



If the interface has dilation characteristics, the forces are corrected as follows (FLAC3D Online Manual 2005):

$$F_n = F_n + \frac{(|F_s|_0 - F_{smax})}{Ak_s} \tan\psi k_n, F_{si} = F_{smax} \tag{4}$$

Where  $|F_s|_0$  is the shear force before the above corrections are made, and  $\psi$  is the dilation angle of the interface.

A series of numerical analysis has been conducted to simulate the direct shear tests using FLAC3D code by ITASCA. The model geometry is shown in Fig. 4. The numerical model of the test is composed of two parts: each part is a shear box with soil in it and discretized 5832 brick-shaped mesh elements. Soil strengths are defined by the Mohr-Coulomb failure criterion.

The interface between the dissimilar materials is modeled as linear spring-slider system with interface shear strength defined by a linear-elastic perfectly plastic model with a Mohr-Coulomb failure criterion. The relative interface movement is controlled by interface normal stiffness ( $k_n$ ) and shear stiffness ( $k_s$ ). A recommended thumb rule is that  $k_s$  and  $k_n$  be set to ten times the equivalent stiffness of the stiffest neighboring zone. The maximum stiffness value is given by as:

$$k_n = k_s = 10 \times \max \left[ \frac{K + \frac{4}{3}G}{(\Delta z)_{min}} \right] \tag{5}$$

Where the parameters  $(\Delta z)_{min}$ ,  $K$  and  $G$  are the smallest dimensions in normal direction, bulk modulus and shear modulus continuum zone adjacent to the interface respectively. This approach gives the preliminary values of the interface stiffness components, and these can be adjusted to avoid intrusion to adjacent zone and to prevent excessive computation time. The interface behavior of clay-Sand is represented numerically at each node by a rigid attachment in normal direction and spring-slider in the tangent plane to the interface surface. The required input parameters for interaction between interface grids are: (i) coupling spring cohesion (ii) coupling spring friction (iii) coupling spring stiffness. Table 1 summarizes the geotechnical parameters of each studied soil.

**Table 1.** Material properties used in numerical simulation

Material	Cohesion c (kPa)	Friction $\phi$ (°)	Bulk modulus $K$ (MPa)	Shear modulus $G$ (MPa)
Sand	0	43.38	0.29167	0.14615
Clay	34.35	10	0.23167	0.024138

**Table 2.** Interface properties used in numerical simulation

Interface	$k_n = k_s$ (N/m)	Friction $\phi$ (°)	Cohesion c (kPa)	Tensile (kPa)
Sand-Sand	424.04 $10^7$	43.38	0	$10^{10}$
Sand-Clay	424.04 $10^7$	27.6	23.75	$10^{10}$
Clay-Clay	237.47 $10^7$	10	34.35	$10^{10}$

In accordance with Eq. 5, parameters of different interfaces are listed in Table 2.

The analysis was carried out considering first step, only normal stress was applied on the top surface of the model and in the second step, shear stress was applied. All of the analyses were performed using normal stress of 54.8, 67.4 and 80.0 kPa respectively.

Figure 9 shows the boundary condition model. The whole model is restrained in the y direction while the bottom part is also restrained in z directions at its base.

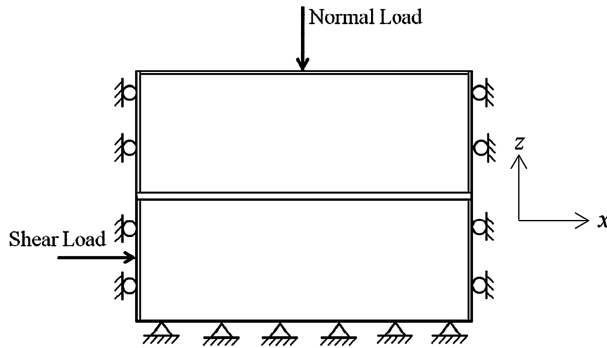


Fig. 9. Boundary conditions in xz plane

In the initial step, a normal load is applied at the top of the model making a set of stresses representing soil initial state installed in the grid, and then FLAC3D is run under elastic assumptions until an equilibrium state is obtained. The initial horizontal stress is related to the initial vertical stress by at rest lateral earth pressure coefficient ( $K_0$ ) which equals to  $(1 - \sin \varphi)$ , where  $\varphi$  is the angle of internal friction of the soil. In the second step a horizontal velocity of  $5 \times 10^{-7}$  m/s is applied to the bottom box in x direction.

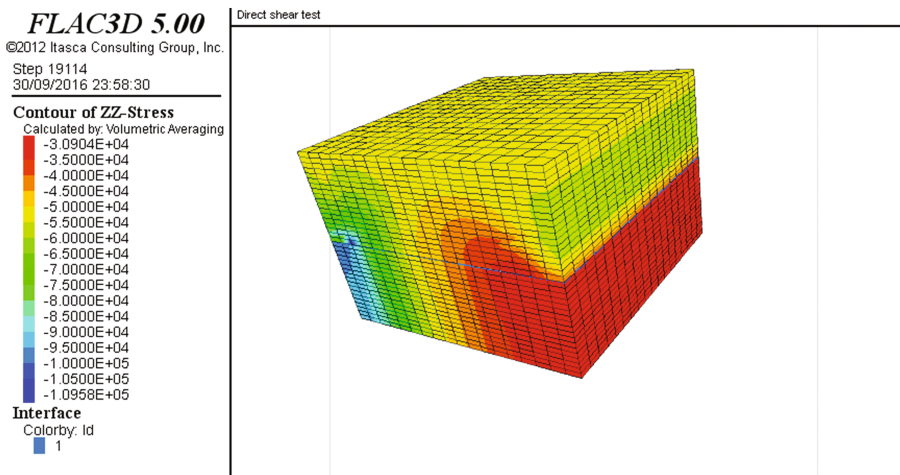
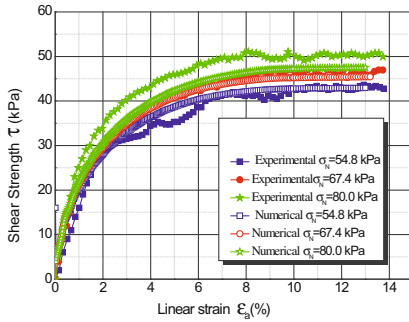


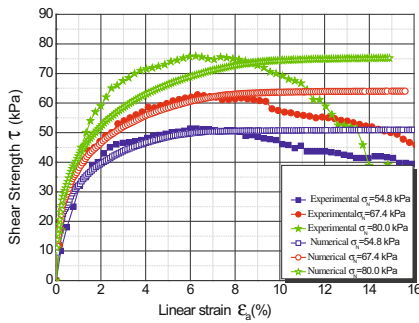
Fig. 10. Vertical stress obtained at the end of simulation

In this present simple analysis of the last section, the friction in the interface is assumed to be fully mobilized. In fact, the degree of mobilization will depend on the shear displacement.

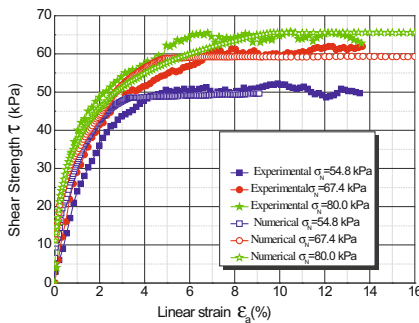
Figure 10 presents an example of obtained vertical stress at the end of simulation numerical modelling. Numerical results are shown in Figs. 11 together with the experimental results. It can be seen that the model is able to reproduce the behavior of soil-soil interface with different normal stresses.



(a)



(b)



(c)

**Fig. 11.** Comparison of model predictions with experimental results in the case of (a) Clay-Clay, (b) Sand-sand and Clay-Sand

## 6 Conclusion

The shear behavior of sand/clayey soil interface using a simple direct shear test is studied. In the shear test a shearing surface is produced and the resistance to shearing is defined as the shear resistance. The frictional resistance is, however, determined on an existing joint surface. The frictional behavior at the soil– other materials interface is usually and commonly obtained from direct shear tests.

The built-in interface element in FLAC3D can simulate the soil-soil interfaces, i.e. relationship between shear stress and shear displacement, according to the linear elastic-perfectly plastic model. The used input parameters are derived and calibrated from experimental results. It can be seen that the numerical model is able to reproduce the behavior of soil-soil interface with different normal stresses. Some elaborated models can be also used according to nonlinear strain-softening interface behavior (Wu et al. 2011).

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