II. SYNTHESES OF EXPERIMENTAL BEHAVIOUR OF SOILS AND INTERFACES

II.1. SOIL BEHAVIOUR UNDER A MONOTONIC LOADING

The behaviour of the soil that may observed during a monotonic triaxial stress test is composed of several phenomena, amongst which the contractancy, dilatancy, and shear stress softening phenomena.

The contractancy is a phenomenon in which the volumetric deformation of the soil decreases during a shear stress test, and on the contrary, the dilatancy is a contractancy in which the volumetric deformation of the soil increases. In contrast to the contractancy and dilatancy which are both linked to the phenomena of volumetric deformation, and the softening are rather related to a variation of shear strength.

The observed behaviour varies depending on several parameters, such as sollicitation path and initial relative density. Figure 1 presents typical results of tests made with a drained triaxial compression on a saturated sand with two different density indices, corresponding to a loose sand and a dense sand. The shear strain curve of a loose sand shows a continuous increase followed by a stabilization to a level of perfect plasticity for large deformations. Regarding the dense sand, the initial response is more rigid and reaches a more loosely determined maximum. If both tests were carried out with the same initial confinement, the shear strain curve of these two soil reaches the same plastic level. In terms of volumetric deformation, loose sand contracts continuously until reaching a level which corresponds to a state of null volumetric deformation. On the contrary, the dense sand shrinks slightly at the beginning of the experiment, and then expands to reach the state of null volumetric deformation.

Figure 2 shows typical results obtained during a non-drained test with a triaxial compression and the same soil density. For the dense soil, the shear strain curve shows a strong contracting behaviour, and the evolution of the interstitial presssure, first positive and then negative, reflects the strong dilatancy of the material. For the loose soil, the shear strain curve shows a characteristic peak for a low level of deformation, followed by a rapid decline of the resistance to a stabilized level for large deformations.

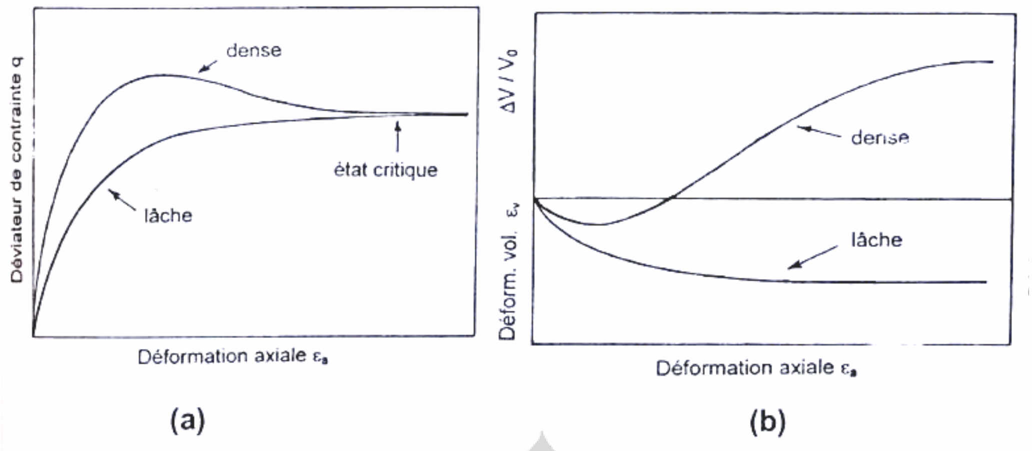


Figure 1. Different behaviours of soil, depending on its initial density during a drain direct shear test

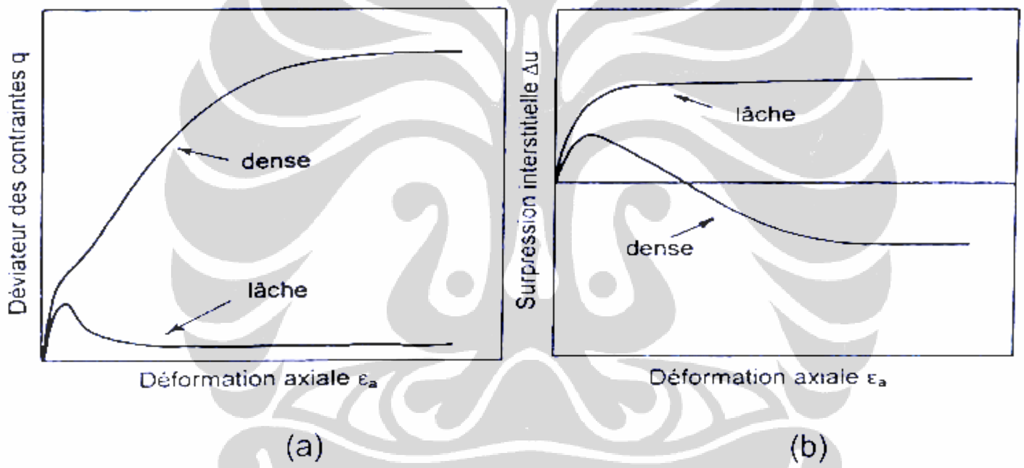


Figure 2. Different behaviours of soil, depending on its initial density during an undrain direct shear test

To better describe the behaviour of the soil, several reference states of the stress – deformation curve are introduced below:

- ***Characteristic state***

  The concept of characteristic state, introduced by Luong (1978.1980) for the sand, used to define, in terms of deformation, an area where the soil is contracting and an area where it is expanding for a deviating solicitation. This condition can typically be observed in the case of dense soils (see point A in Figure 3). This state also corresponds to a null variation of the sample's volume.

- ***Maximum resistance state***

  This maximum is the peak of the load curve (see B). For soils without cohesion, it can not be defined by the friction coefficient between granular particles. The angle of friction of this state for sand can be presented as a contribution of three variables (Rowe, 1962): the friction angle of contact between particles, the phenomenon of particle arrangement, and the soil's expansion.

  The materials, which are dense or confined at low, show a well marked maximum (peak). Loose materials, or materials whith a higher confinement, tend to present a monotonic load curve. In the later case, the maximum deformation is more difficult to estimate. This maximum, in the displacement curve, also corresponds to the maximum slope of the dilatancy.

- ***Critical state pf soil***

  This third condition, also called state of residual stresses, corresponds to a large deformation of the soil and is defined as the final state (the constant plate at point C) of any sample regardless of the initial normal stress and the initial density of the sample in drained condition. In the case of a sand, a constant vacuum index is reached, because the granular material flows at constant volume. This state is a state of perfect plasticity and represents a condition of constant volume and of constant anisotropic constitution.

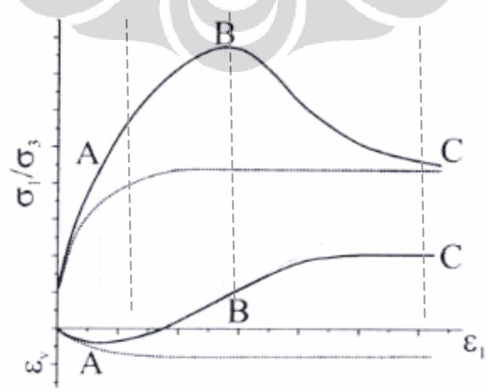


Figure 3. Global behavor of an interface

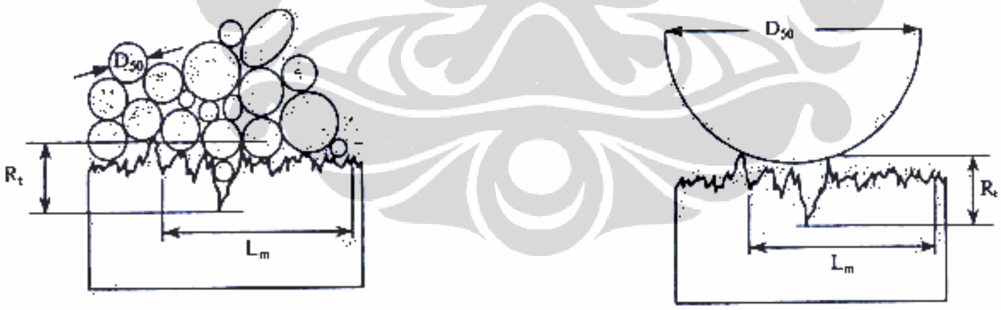
II.2. DEFINITION OF AN INTERFACE

The interface between the soil and the structure is defined as a thin layer of sand or soil located in the contact area. The thickness of this layer depends on the interface's roughness. This value ranges from 2 to 5 times D_{50} for a smooth surface, and 10 to 15 times D_{50} for a rough surface, D_{50} being the average size (mass) of soil particles (Wernick, 1978; Yoshimi and Kishida, 1981; Bolt, 1989; Hoteit, 1990).

Uesugi and Kishida (1986) defined the interface's roughness as the standard roughness of the structure, and therefore as the ratio between maximum roughness of the structure (R_t) and the average size of soil particles D_{50}.

$$R_n = {R_t}/{D_{50}} \qquad (2.2.1)$$

With R_t the roughness or the maximum surface roughness structure measured from peak to peak over a length of surface L_m = 0.2 to 2.5 mm. R_t is considered as relevant because it depends on the L_m chosen, and represents the roughness of the structure with respect to the average particle size of the ground. Figure 4 shows the different definitions of the interface according to R_t and D_{50}



Rough Interface *Smooth Interface*

Figure 4. Definition of the maximum roughness

To identify the type of interface, a critical value Rcri can be defined. If R_n is smaller than this critical value, the interface is described as smooth and if the value of R_n exceeds this value, the interface is described as rough. To define the critical value of R_n, Hu and Pu (2004) have carried out direct shear tests on a soil-steel interface with Yongdinghe sand. They have made changes on the value of R_n, with a relative density Dr= 90% (dense sand), and then set a critical value of R_n, R_{cri} = 0.1 in their study. If Rn is greater than Rcri, the behaviour of the interface between the sand and the steel structure begins to show a **softening**, which is most significant compared to the tests with R_n less than R_{cri} (see Figure 5). The test results with a R_n less than R_{cri} determine smooth interfaces, and the state of maximum resistance is in this case almost elastic-perfectly plastic.

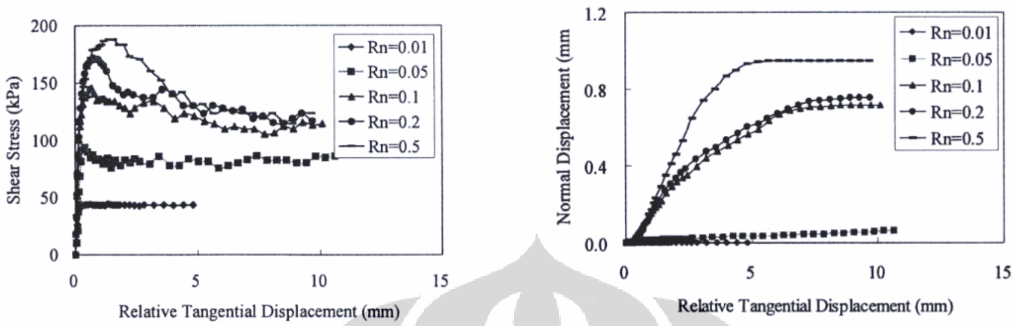


Figure 5. Shear tests with differents values of R_n. (ID = 90%, σ_n = 200 Kpa)

Several other experiments were made to define the critical value R_n (Yoshimi and Kishida, 1981, Jardine et al. 1993; Foray et al. 1995, Garnier and Konig, 1998 and Lehane and White, 2005). We can summarize their studies in the following manner:

- If $R_n < 0.02$, the interface is smooth.
- If $R_n > 0.02$, the interface is considered moderatly rough.
- If $R_n > 0.1$, the interface is very rough.

The soil-structure interface can be modeled by a system made of a rigid body, a intense shear zone and a ground represented by elastic springs, as shown in Figure 6. This model correctly expresses the phenomena taking place during a classic shear test between piles and soil as observed in reality as well as in the laboratory.

In the shear tests realized, the normal stress on the shear surface varies sometimes linearly with normal displacement. This variation will be expressed by a stiffness constant, K.

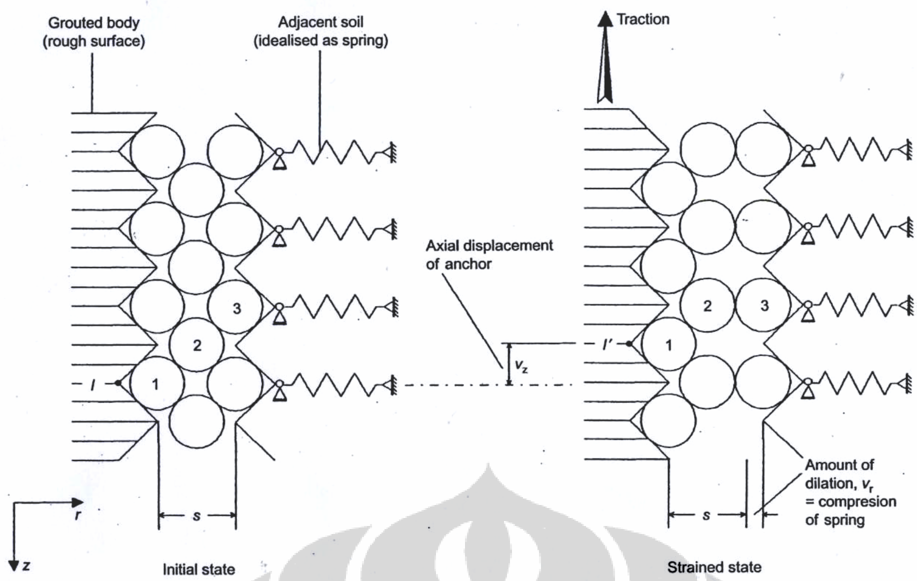


Figure 6. Interface system

II.3. DIRECT SHEAR TEST

To study the behaviour of the interface, a test often used by researchers is the direct shear test. The results of such a test are comparable to the behaviour of the soil (found from a triaxial shear test) as shown in a study by Bolt and Nova (1990) who observed the equivalency between this test and triaxial shear tests on sand.

A direct shear test is a laboratory test used by geotechnical engineers to find the shear strength parameters of soil. In France the standard who define how the test should be performed is NF EN ISO 12957-1 (*Géosynthétiques - Détermination des caractéristiques de frottement - Partie 1 : essai de cisaillement direct*); and in the U.S., the standard is ASTM D 3080.

The test is performed on three or four specimens from a relatively undisturbed soil sample (cf. picture 7). A specimen is placed in a *shear box,* which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A *confining stress* is applied vertically to the specimen, and the upper ring is pulled horizontally until the sample fails, or through a specified displacement. The load applied and the strain induced is recorded at certain intervals to determine a stress-strain curve for the confining stress.

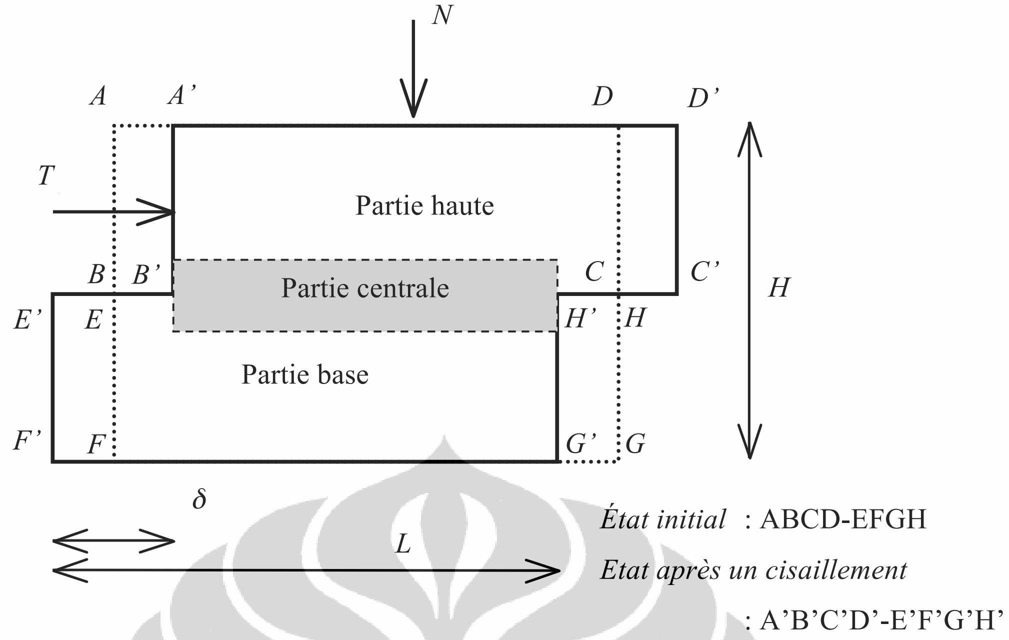


Figure 7. Schematic diagram of the direct shear test (Zhang, 2007)

In the direct shear test, the top half of the sample is translated relative to the bottom half of the specimen (cf. figure 7) in order to create a shear band/plane across the mid-height of the specimen. This test is used to measure the flow properties (in particle technology) and the shear strength (in geotechnical engineering) of granular material (Zhang, 2007). In the geotechnical application, the apparatus that is usually used is a Casagrande shear box (square cross section). In both cases the externally applied vertical and horizontal forces are measured, and the ratio of shear stress to normal stress are recorded and thereby provides a direct measure of the internal friction angle. The shaded area shown in Figure 7, is an approximation to the anticipated shear zone that will develop at the mid-height of the specimen.

By adding all the forces at the particle/wall contacts, the resolved reaction forces may be calculated. Figure 8 shows the wall reaction forces for the upper part of the shear box. From equilibrium, the following relationship may be obtained:

$$T = N_1 - N_2 + T_3 \qquad (2.3.1)$$

$$N = T_1 - T_2 + N_3 \qquad (2.3.2)$$

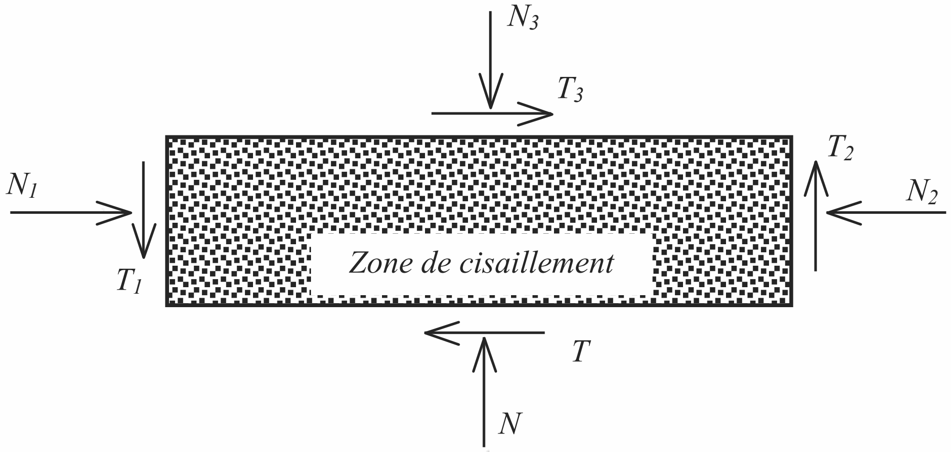


Figure 8. Forces acting on the top half of the specimen

Direct Shear Test can be performed under several conditions. The sample is normally saturated before the test is run, but can be run at the in-situ moisture content. The rate of strain can vary to create a test of *undrained* or *drained* conditions, depending whether the strain is applied slowly enough for water drainage in the sample to prevent pore-water pressure buildup. Several specimens are tested at varying confining stresses to determine the shear strength parameters, the soil cohesion (c) and the angle of internal friction (commonly *friction angle*) (φ). The results of the tests on each specimen are plotted on a graph with the peak (or residual) stress on the y-axis and the confining stress on the x-axis. The y-intercept of the curve that fits the test results is the cohesion, and the slope of the line or curve is the friction angle.

Direct-shear test imposes stress conditions on the soil that forces the failure plane to occur at a predetermined location (on the plane that separates the two halves of the box). On this plane, there are two forces (or stresses) acting, a normal stress (σ_n) due to an applied vertical load P_v and a shearing stress (τ) due to the applied horizontal load P_h. These stresses are computed by using these next formulas:

$$\sigma_n = {P_v}/{A} \qquad (2.3.3)$$

$$\tau = {P_h}/{A} \qquad (2.3.4)$$

Where A is the nominal area of the specimen (or of the shear box). It is usually not corrected for the change in sample area caused by the lateral displacement of the sample under the shear load P_h. These stresses should satisfy Coulomb's criterion:

$$\tau = c + \sigma_n \tan\phi \qquad (2.3.5)$$

As there are two unknown quantities (c and ϕ) in the above equation, two values, as a minimum, of normal stress and shear stress will be required to obtain a solution. Since the shear stress and normal stress have the same significance as when used in a Mohr's circle construction, rather than solving a series of simultaneous equations for c and tan ϕ, one may plot on a set of coordinate axes the values of τ versus σ_n from several tests (generally with τ on the ordinate), draw a line through the resulting locii of points, or the average locii of points, and establish the slope of the line as (tan ϕ) and the τ-axis intercept as the c parameter. This is commonly known as the *Mohr-Coulomb Failure Envelope*. For cohesion-less soils like sand, the intercept is usually negligible, and Coulomb's criterion simplifies to :

$$\tau = \sigma_n \tan\phi \qquad (2.3.6)$$

Test inaccuracies and surface-tension effects of damp cohesion-less materials may give a small value of c, called the "apparent" cohesion. This should be neglected unless it is more than 7 or 14 kPa. If c value is large and the soil is a cohesion-less material, the reason for the large value of c should be investigated. The direct shear test was formerly quite popular, but with the development of the triaxial test which is much more flexible, it has become less popular in recent years. The advantages of the direct shear test are :

1. Cheap, fast and simple - especially for sands.
2. Failure occurs along a single surface, which approximates observed slips or shear type failures in natural soils.

And the disadvantages of the test are:

1. Difficult or impossible to control drainage, especially for fine-grained soils.
2. Failure plane is forced, may not be the weakest or most critical plane in the field.
3. Non-uniform stress conditions exist in the specimen.
4. The principal stresses rotate during shear, and the rotation cannot be controlled. Principal stresses are not directly measured.

Because the drainage conditions during all stages of the test markedly influence the shear strength of soils, the direct shear test is only applicable for relatively clean sands, which are free draining during shear. For clay soils, some unknown amount of consolidation could occur during shear, which would give a larger shear strength than actual. Therefore, the test is not generally recommended for cohesive soils.

Several experimental methods of Direct Shear Test that have usually been used to study the strength of the interface are Constant Normal Load (CNL), Constant Normal Stiffness (CNS), and Constant Normal Volume (CNV). The CNL and CNV tests correspond respectively to a drained and undrained condition tests, and CNS correspond to an intermediary condition, where the confinement provided by the surrounding soil is simulated by a group of elastic spring, with a stiffness *K*. The fact that the normal stress of the interface is not fixed and that the expansion is only limited by the rigidity of the mass of soil, showed that CNS condition is the test that simulated the best the reel soil-structure interface.

II.3.1. Constant Normal Loading test (CNL)

The interface is confined by a constant normal stress at the upper part of the system and measurement is conducted under increasing shear stress. If the joints tend to dilate, then the apparatus, by proportionality constant, serves to maintain a constant normal load on the systems.

The proportionality constant *K*, must satisfy the following relationship,

$$K = {d\sigma_n}/{du_n} \qquad (2.3.1.1)$$

In CNL test, the upper part is free in normal direction so that $d\sigma_n = 0$, which leads to $K = 0$. The model of constant normal load boundary condition is shown in Figure 9.

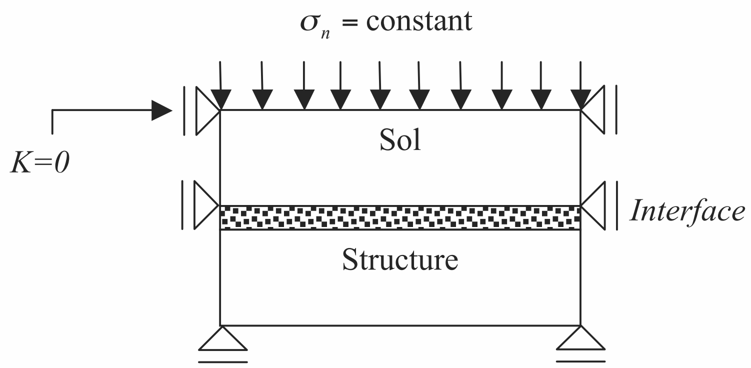


Figure 9. Constant Normal Stress Boundary Condition.

II.3.2. Constant Normal Volume tests (CNV)

In this test, the joint is confined by an infinite rigidity (stiffness). No displacement is allowed at the upper boundary of the interface in the direction normal to the interface plane as illustrated in figure 10. In the upper part of the soil, we fix the boundary condition so that $d\sigma_n \neq 0$ et $du_n = 0$, which leads to $K = \infty$.

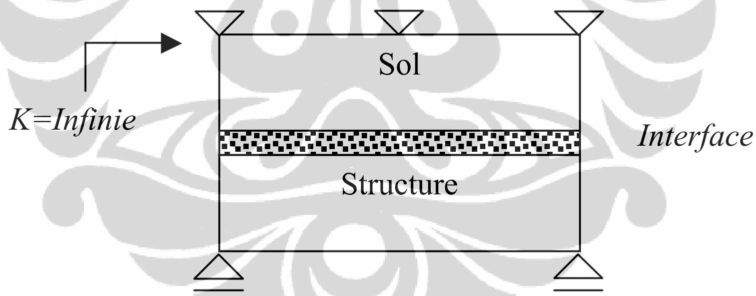


Figure 10. Constant Normal Volume Boundary Condition

II.3.3. Constant Normal Stiffness tests (CNS)

The CNS is an appropriate method for the direct shear test device used by many researchers (including Boulon et al. 1986, Johnston et al. 1987; Tubucanon et al., 1995). This test is a good experimental method to study the complexity of the load transfer mechanisms involved in the friction between soil and structures.

In CNS test, the systems is is confined by a prescribed stiffness in the upperpart of the system (Figure 11), in which normal stress and normal displacement vary proportionally

If the joint has a tendency to dilate, the normal stress will increase as the surrounding apparatus responds. Normally the joint is subjected to an initial normal stress.

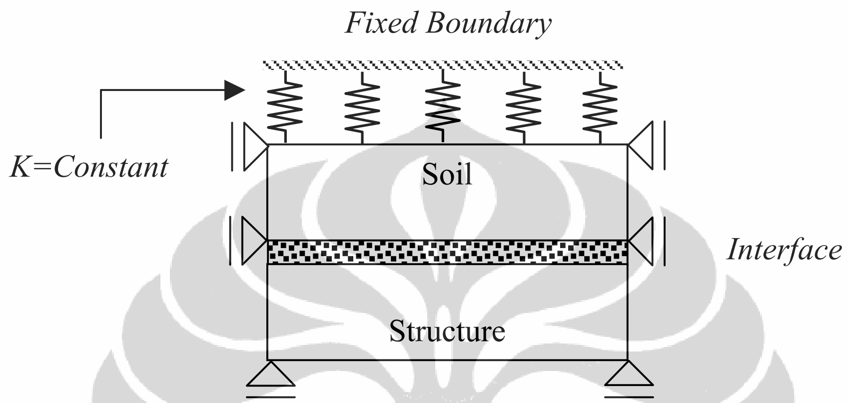


Figure 11. Constant Normal Stiffness Boundary Condition

The rate of cyclic degradation ($d\tau_{max}/dN_{cycles}$) is primarily dependent on the confinement condition of the soil-structure interface, which can be modeled as a constant normal stiffness (CNS) condition:

$$\sigma_{ni} = \sigma_{n0} + u_n K = \sigma_{n0} + u_n\left(4\, {k_{s0}}/{d}\right) \quad (2.3.1.2)$$

where σ_{ni} = current normal stress, σ_{n0} = initial normal stress, u_n = sample deformation normal to the interface, K = spring stiffness, k_{s0} = soil small strain shear stiffness, and d = pile diameter (Boulon and Foray, 1987). In most of the testing devices, a simply supported reaction beam is used to imposed the normal stiffness and the desired stiffness is modelled by varying the span or moment inertia of the beam (Fakharian,1997).

II.4. *INTERFACE BEHAVIOUR UNDER A MONOTONIC LOADING*

To understand the interface mechanisms under cyclic loading, one first needs to have a good understanding of the behaviour observed during a monotonic solicitation. In the following paragraphs, the reference states of the interface will be described, followed by the different types of behaviour observed for the method of direct shear tests used. The different behaviours depending on the initial density are explained, before concluding by the influence of various parameters on the interface.

II.4.1. Reference states of the interface's behaviour

There are reports of benchmarks for the interface in the paths of stresses and strains that are equivalent to the ground, in particular the concepts of state and characteristic of great strength. The definitions of these are in fact identical to the soil and the interface. However, the concept of critical state soil is only valid in the case of a rough interface.

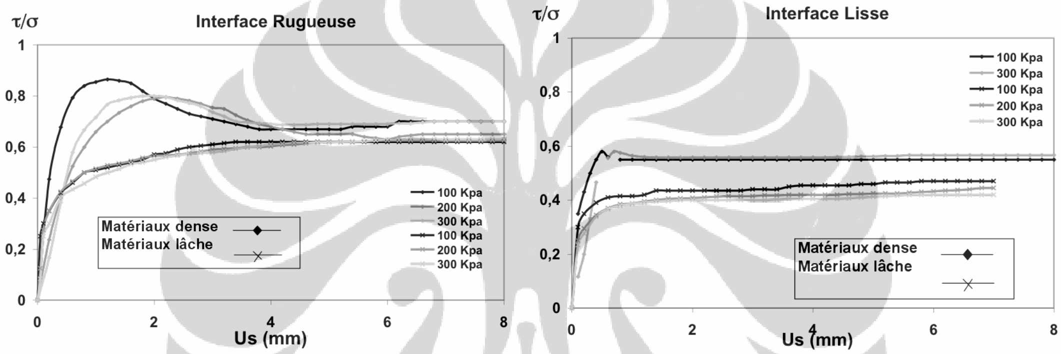
There exists several reference states of the shear strain and stress curves for the interface, which are equivalent to those of the ground, in particular the concepts of characteristic state and maximum resistance. The definitions of these are in fact identical for the soil and the interface. However, the concept of critical state of the soil is only valid in the case of a rough interface.

Liu et al. (2006) confirmed in their study the equivalence between the behaviour of the soil and a rough interface. They note that the concept of critical condition of the soil is indeed valid for a rough interface but not for a smooth interface. There is however for a smooth interface a state of large deformation which is independent of the initial confinement, but which depends heavily on the initial density.

The analogy between the behaviour of the sand and the behaviour of a rough interface between a sandy soil and the structure was also discussed by Boulon and Nova (1990). It was demonstrated during a shear test for a rough interface that the phenomena of dilatancy, contractancy and softening exist for a dense sand. From the experimental results with an interface with different roughness indexes, it was noted that the interface reaches large shear deformation state, in which the stress ratio remains constant and the deformation continues without any expansion nor contraction. These characteristics are similar to those of a sand.

For the rest of this report, in order to limit the number of parameters, we will treat the interface parameter that corresponds to the state of maximum deformation for a smooth interface, and the parameter corresponding to the critical state for a rough interface as a same parameter marked as ϕ_{res} (ϕ residual). The parameters corresponding to the maximum state will be designated in the same manner than for a soil: $\phi_{\max}$ **and** ϕ_{car} **.**

Figures 12 (a) and (b) present the results of a forced shear CNL test according to a study by Shahrour and Rezaie (1997). They have conducted studies on two soils of different density: *ID* = 90% (considered as a dense sand) and *ID* = 15% (considered as a loose sand), two different types of interface: rough and smooth. These figures show for different initial densities and different levels of roughness, a plateau of perfect plasticity for large deformations.



a) b)

Figure 12. Shear stress curve [Shahrour et Rezaie, 1997]

For the rough soil, the high strain plateau is almost identical for a same initial density regardless of values of the initial confinement strain and the initial density. The offsets of the curves for dense sand when reaching the plateau are due mainly to the difficulties in reaching large deformations while remaining within the framework of continuous material mechanics and the theoretical assumptions made on the homogeneity of the material. However, for a smooth interface, materials of different initial densities reach different plateaux of large deformations. The concept of critical state is not valid, which confirms the theory of Liu et al. (2006).

II.4.2. Behaviour according to different shear strain curves

Different behaviours can be observed under different shear strain curves (Figure 13). During a drained test or CNL, the main phenomena that can be observed are **softening**, hardening, and the evolution of normal displacement (dilatancy or contractancy). The evolution of the shear strain curve does not involve any change in normal stress.

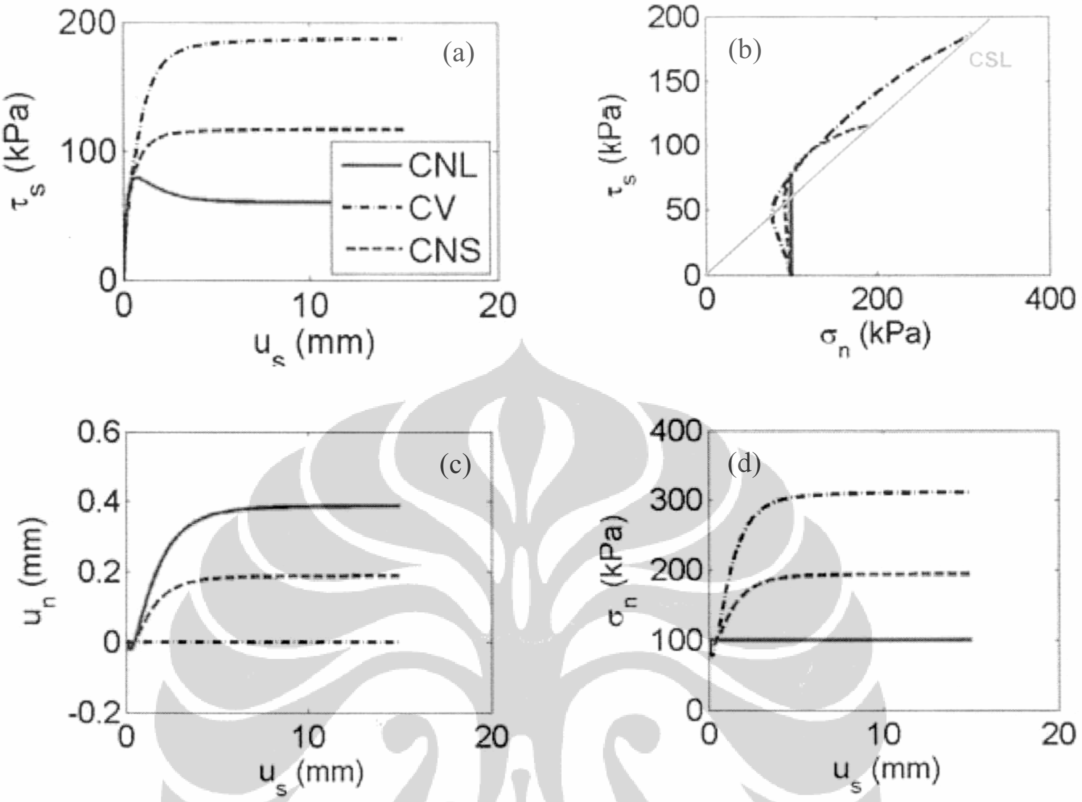


Figure 13. Typical results for different types of direct shear test: (a) Shear stress curve ; (b) Relation between shear stress and normal stress ; (c) Normal displacement curve; (d) Normal stress curve

On the contrary, during a non-drained test or CNV, the results show a degradation of the interface properties due to the dissipation of excessive interstitial pressure. This test is actually a test with a null normal displacement in the upper part of the interface element. Thus, during shearing, the mobilization of the shear strain is followed by the evolution of the normal stress. This latter will increase or decrease depending on the contracting or expanding trend of the interface.

The CNS test, which is the intermediary boundary condition, imposes a normal stiffness on the interface without blocking normal movements, so that the variation of normal stress and of normal displacement of the interface is possible and proportional. The variation in normal displacement for a contraction or an expansion is followed by changes in the normal stress. From these observations, it is also possible to note that increasing the rigidity of the ground leads to an increase in the shear strain mobilization (see Figure 13.a).

II.4.3. Behaviour according to different initial densities

Figures 14 and 15 present the results of a constant normal stress test carried out by Shahrour and Rezaie in 1997. On the shear strain curves of the interface we can see that a dense sand is subject to a hardening and a degradation of its shear strain resistance during the solicitation. On the other hand, for a loose sand, the hardening is gradual until it reaches a plateau for large deformations.

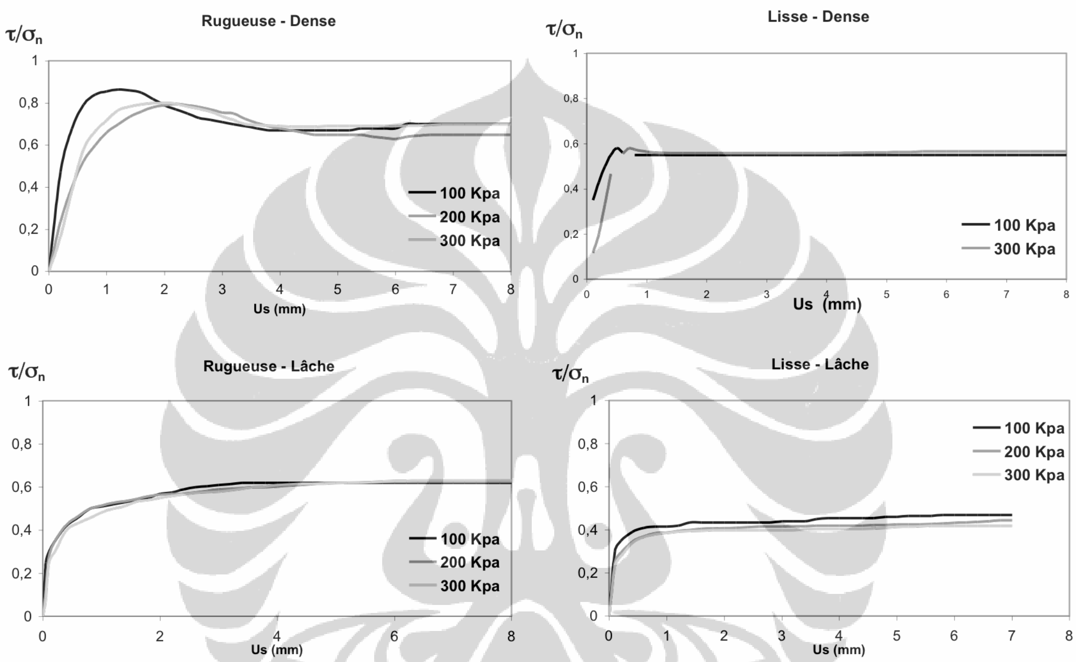


Figure 14. Stress ration for CNL tests, for loose sand with *ID*=15% and dense sand with *ID*=90% [Shahrour and Rezaie, 1997]

From figure 12, which shows the shear strain ratios for the same tests, both for the dense sand and the loose sand, it can be concluded that dense sands tend to be more rigid for small deformation compared to loose sands.

With regard to the normal movement mobilized for the interface (Fig. 15), it may be noted that a loose sand, regardless of the roughness of its interface, suffers purely contractions. However, a dense sand contracts slightly at the beginning of the solicitation, followed by a much larger dilatancy.

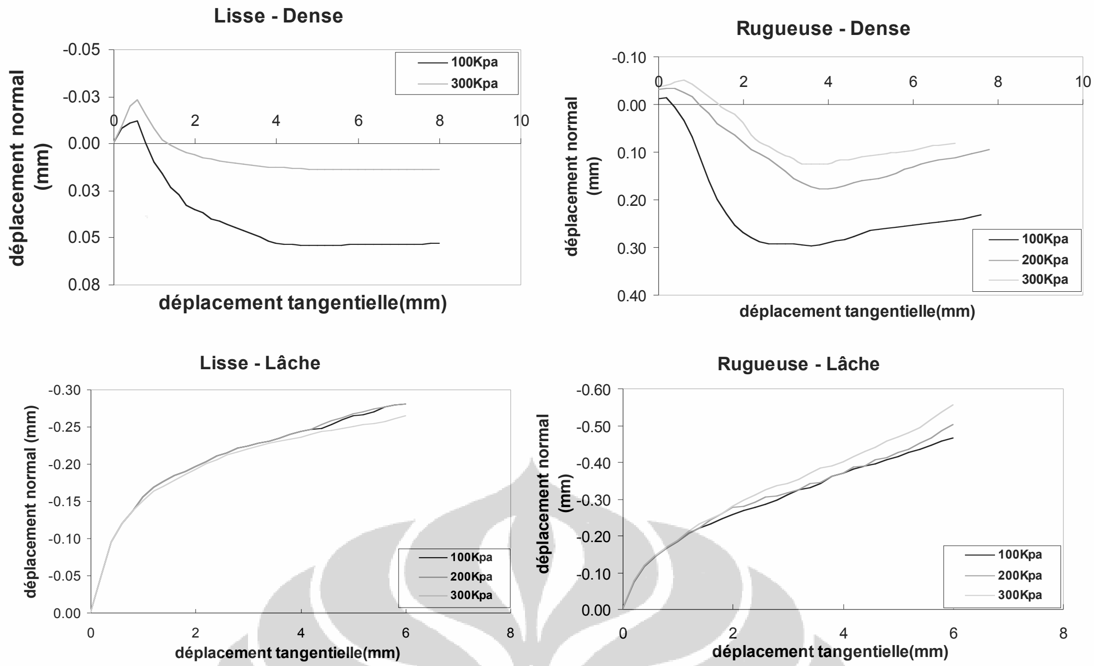


Figure 15. Normal displacement for CNL tests, for loose sand with *ID*=15% and dense sand with *ID*=90% [Shahrour et Rezaie, 1997]

II.4.4. Influence of differents parameters

Some studies have revealed the existence of some key parameters that influence the behaviour of the interface, amongst them: the interface roughness, as was discussed in chapter II.2, which divides the interface into two types: rough and smooth ; the average particle size of the ground D_{50} ; the initial states of the system in terms of relative initial density *ID* ; and the effective normal stress σ_n.

a) *Interface's roughness*

The magnitude of this variable, in addition to characterizing different types of interface according to its roughness, also plays heavily on the failure mode of the interface and the friction angle mobilized.

Several studies have been carried out on this subject, which share the same view that when the interface is described as smooth, the soil particles slide along the length of the interface. If the interface is rough, the fracture occurs mainly in the soil and the friction angle of the interface is up close to the maximum friction angle of the soil.

Regarding the friction angle, Fioravante (2002) conducted a test with a constant shear stiffness (CNS), and showed that the angle of friction of an interface is higher if the interface is rougher. Figure 16 illustrates this hypothesis, with the parameter *tanδ* being the average friction angle of the interface.

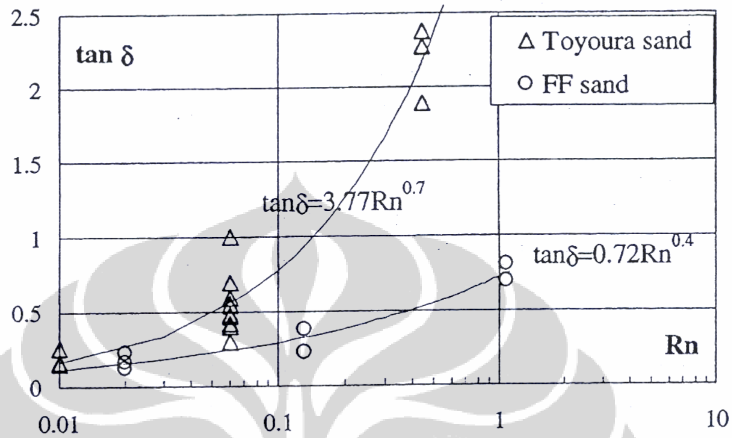


Figure 16. Influence of the inteface's roughness [Fioravante, 2002]

Similarly, according to the shear stress curves in a CNL test by Shahrour and Rezaie (Fig. 12), the angle of friction for large deformations is higher ($\tan\phi_{res} = \pm\ 0.7$) for a rougher interface, ie for a value of R_n more important, compared to a smooth interface ($\tan\phi_{res} = \pm\ 0.6$) with a lower value of R_n.

b) Average diameter of the soil particules

The magnitude of D_{50} influences the angle of friction of the interface. This concept is shown in Figure 17. This figure shows two particles of soil with different average size (D_j and D_i) and a structure with a roughness R_t. The soil particle with a greater average size (D_j) has a friction angle θ_j while the particle with a smaller average size (D_i) has a friction angle θ_i with $\theta_j < \theta_i$. It shows that for a same surface roughness, a bigger average particle size leads to a smaller angle of friction of the interface.

This confirms the idea of Uesugi and Kishida (1986) at the beginning of this chapter, which have defined the interface roughness as the roughness of the normalized structure divided by the average size of soil particles.

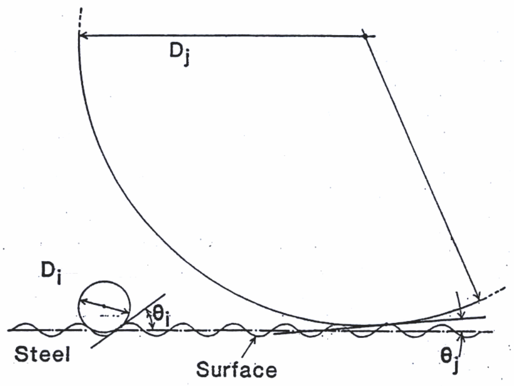


Figure 17. Friction angle of the inteface as a function of D_{50}

c) *Initial relative density*

As discussed above, the initial relative density of the soil has a major influence on the general behaviour of the interface. A soil with a relatively high density (dense) is the place of softening phenomena and dilatancy. To the contrary, a soil with a lower density (loose) creates only contractancy phenomena even with a rougher interface. This latter means that for an interface with loose sand, the characteristic angle of friction, the friction angle for large deformations and the maximum angle of friction are the same.

d) *Initial normal stress*

- Influence on the maximum of shear resistance of the interface

Most authors (Evgin and Fakharian 1996, Hu and Pu, 2004; Shahrour and Rezaie, 1997; Ghione and Mortara, 2002) found that the shear strength increases with an increase of the normal stress imposed. (See Figure 18 and figures of the test results by CNL Shahrour Rezaie and in Appendix A). Through the results of Shahrour and Rezaie, for dense materials, we can see a resurgence of resistance for displacements of 6 mm approximatively.

Similarly, as regards to the behaviour of the displacement curve, these authors unanimously agree that the displacement curve increases when the normal stress imposed decreases. This is valid only for a dense sand (cf. figure 15 and figure 19).

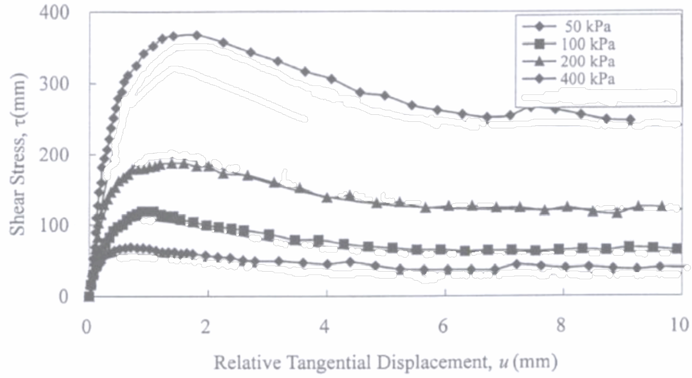


Figure 18. Shear stress curve for a CNL test for an interface of R_n=0.5 and dense sand of *ID*=90% [Hu et Pu, 2003]

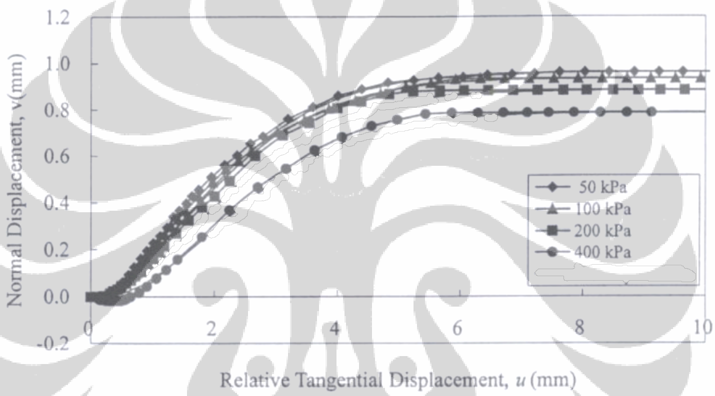


Figure 19. Normal displacement curve for an interface of R_n=0.5 and dense sand of *ID*=90% [Hu et Pu, 2003]

- Influence on the maximum resistance state (stress ratio)

The normal stress also influences the initial friction angle of the interface at the state of maximum resistance (and therefore influences the maximum stress ratio). Evgin and Fakharian (1996) have proved this relation from tests on a rough interface (R_t = 25μm) with dry sand of initial density 84% (considered as a dense sand). Figure 20 present the result of a CNL and CNS test respectively .

Figure 20.a shows an increase in the stress ratio for a lower initial normal stress. Figure 14 (results of Shahrour and Rezaie) draws the same conclusions for tests with initial normal stresses equal to 100 kPa and 300kPa. Based on these two results (Shahrour and Rezaie, and Evgin and Fakharian), it may be noted that this increase in stress ratio are valid only for a rough interface with a dense sand.

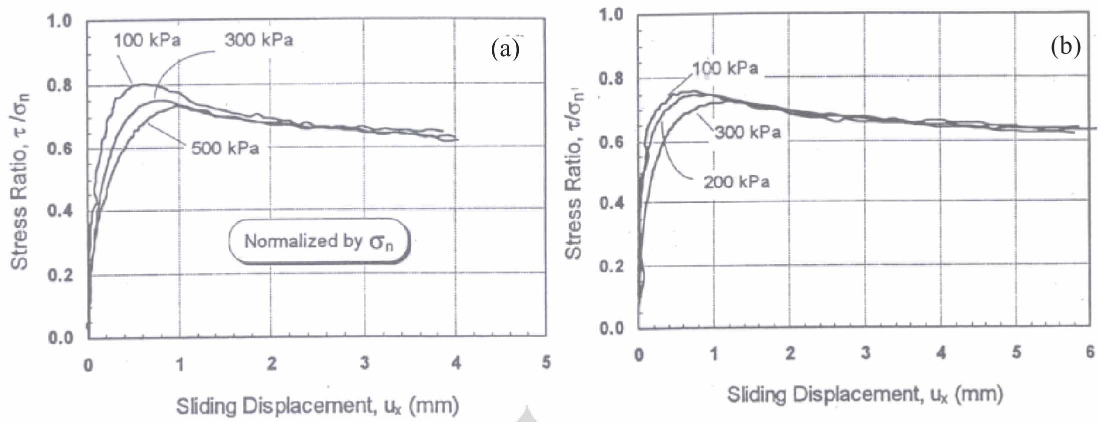


Figure 20. Stress ration for a rough interface with a dense sand (ID=84%) for differents values of initial normale stress : a) CNL test; b) CNS test (k = 800 kPa/mm) [Evgin et Fakharian, 1996]

Similarly, the results of Evgin and Fakharian, when tested CNS (Figure 20.b) shows that the relationship between the stress ratio and the tangential displacement remains the same even for different types of test. This observation remains valid despite the fact that various types of direct shear test inevitably induce a change in the behaviour of the shear strain curve. A similar observation was made by Leichnitz (1985) for a sand interface.

- <u>Influence on the residual state</u>

Contrary to the value of the stress ratio for the maximum resistance, the residual value is not influenced by the normal stress imposed. To confirm this, the results of tests conducted by Shahrour and Rezaie were drawn in the stress ratio (τ/σ_n) - shear displacement (u_s) plan in Figure 11. It can be easily noticed that the coefficient of friction (τ/σ_n) for large deformations does not change regardless of the initial normal stress for each type of interface and each initial density used.

II.5. CYCLIC BEHAVIOUR OF INTERFACE

In the case of a drained test, the cyclic densification of the ground in the vicinity of the interface can lead to a softening or hardening of the properties of the interface according to the boundary conditions of the system. In the case of a semi infinite plan, there is generally a decrease in the level of effective stresses in the vicinity of the structure, resulting in a weakening of the interface (softening); However, in shear test boxes, where the normal component of the strain is maintained constant (drained test), there will be a tightening.

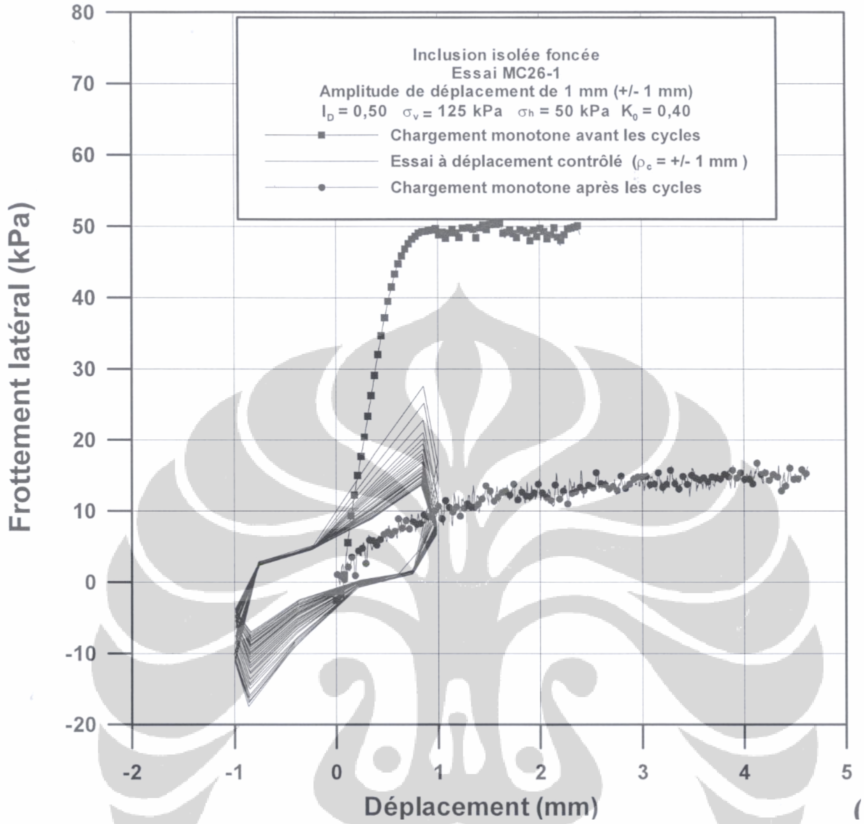


Figure 21. Degradation of the interface's properties discovered from an instrumented probe test on a *fontainebleau* sand, tested in a calibration room [Le Kouby, 2003]

Similarly, for a non-drained shear test, the progressive development of excessive interstitial pressure at a constant volume leads to lower effective stresses and a weakening of the interface, regardless of the boundary conditions of the test (hardening phenomena are not possible in these conditions). The dissipation of excess interstitial pressures during the consolidation will then lead to a re-increase of effective stresses and renewed resistance.

In conclusion, the global behaviour of the interface during a cyclic solicitation corresponds to a deterioration of its properties. Figure 21 shows the degradation obtained under cyclic sollicitations with controlled displacement in a *Fontainebleau* sand tested in calibration room with an instrumented probe (Le Kouby, 2003)

In the next chapter, it will be presented the results obtained using different types of shear tests.

II.5.1. Constant normal stress test

Figure 22 presents the results obtained in a direct shear box during an alternating cyclic solicitation with a controlled shear displacement (-1mm < u_s <1mm). The tests are made on a rough surface with loose and dense sands by Fakharian Evgin (1993). In both cases, the cyclical contractancy phenomenon is clearly highlighted. This phenomenon is even more pronounced when the sand is initially loose. In terms of stress-displacement ratio, these results show that the phenomenon of accommodation with cyclical hardening, typical of the observations made with shear test boxes, appears during the first cycle.

Similarly, Figure 23 shows the results for the same types of tests (with σ_n = 100 kPa) made by Shahrour and Rezaie (1997). It may be noted that the cyclic loading induces softening phenomena (decrease of the maximum amount of shear) and hardening (increase of the maximum amount of shear) only during the first few times.

The curves obtained for a rough interface with a dense sand show trend which differs from the results of these two authors. While one shows a hardening phenomenon, the other reveals rather a softening phenomenon. This contradiction suggests that the origin of these phenomena is questionable. They are probably more related to a poor adjustment of the initial state, which was not perfectly homogeneous and isotropic.

Regarding the evolution of the normal displacement, the interface undergoes a contractancy at each load reversal, followed by a dilatancy. The overall behaviour of the interface is first contracting and the normal displacement accumulated during the cycle then decreases during loading.

Influence of the initial density under cyclic loading

As discussed above, there are different behaviours depending on the initial density of the sand (see Figure 23). The interface with a loose sand shows first a contracting behaviour followed by a slight dilatancy (phase of stabilization of the normal displacement), whereas with a dense sand, it is contracting at each reversal of the loading followed by a larger dilatancy compared to that of a loose sand. After 20 cycles, the accumulation of normal displacement for a loose sand is around 1.7 mm while for a denser sand, it is 1.2 mm.

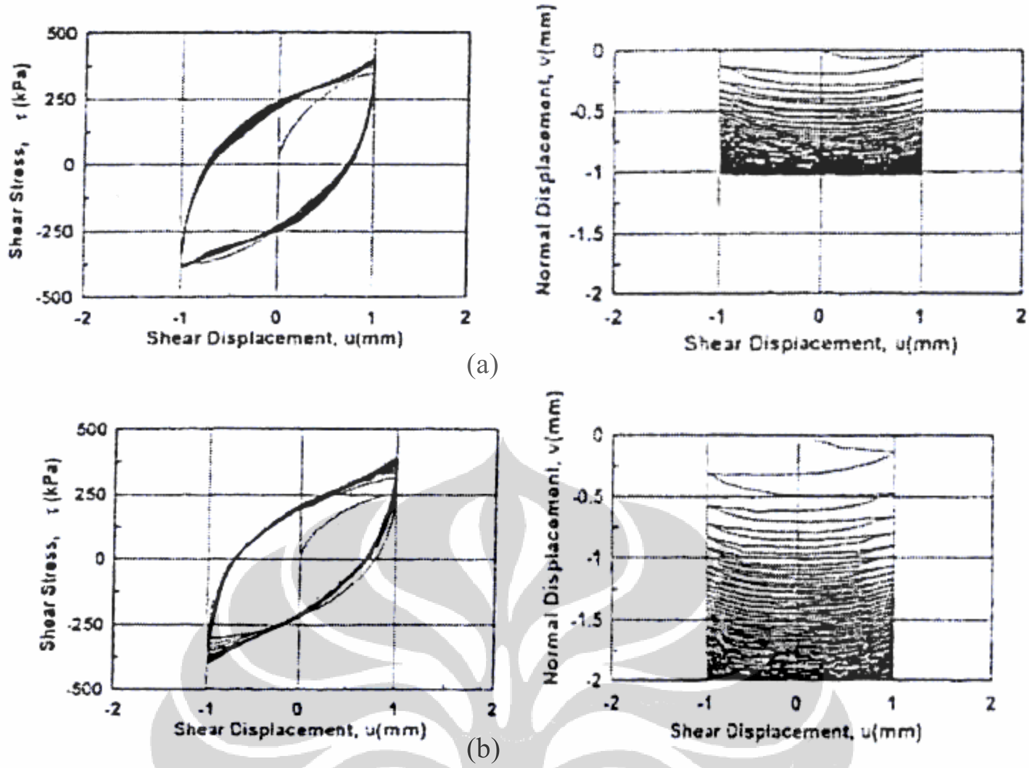


Figure 22. Interface behaviour (rough case, σ_n = 500 kPa) studied on a direct shear box for a: (a) dense sand ; (b) loose sand [Fakharian and Evgin, 1993]

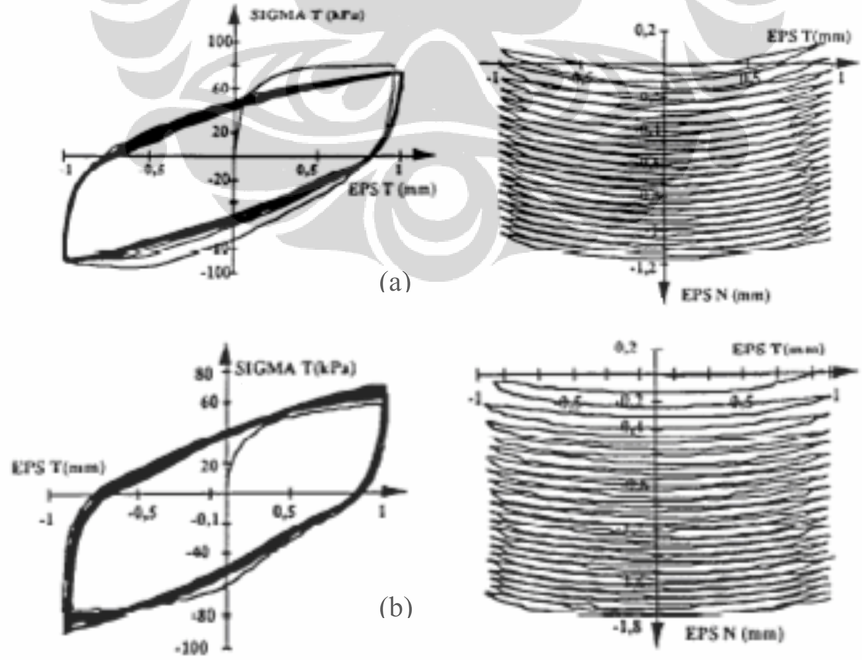


Figure 23. Interface behaviour under CNL cyclic loading for a rough interface: (a) dense sand (ID=90%); (b) loose sand (ID=15%). [Shahrour and Rezaie, 1997]

<u>*Influence of the interface's roughness under cyclic loading*</u>

Figure 24 presents the results of two tests with a smooth interface on dense sand and loose sand. It may be noted that the overall behaviour of a smooth interface is different compared to the results of a test on a rough interface. Only contractions can be noticed, especially for a loose sand. For a dense sand, the displacement curve also shows only a contraction followed by a stabilization, and not a dilatancy as observed during tests on rough interfaces. This result proves that for a smooth interface with a constant normal stress, the angle of friction is identical between the characteristic state and the state for large deformations.

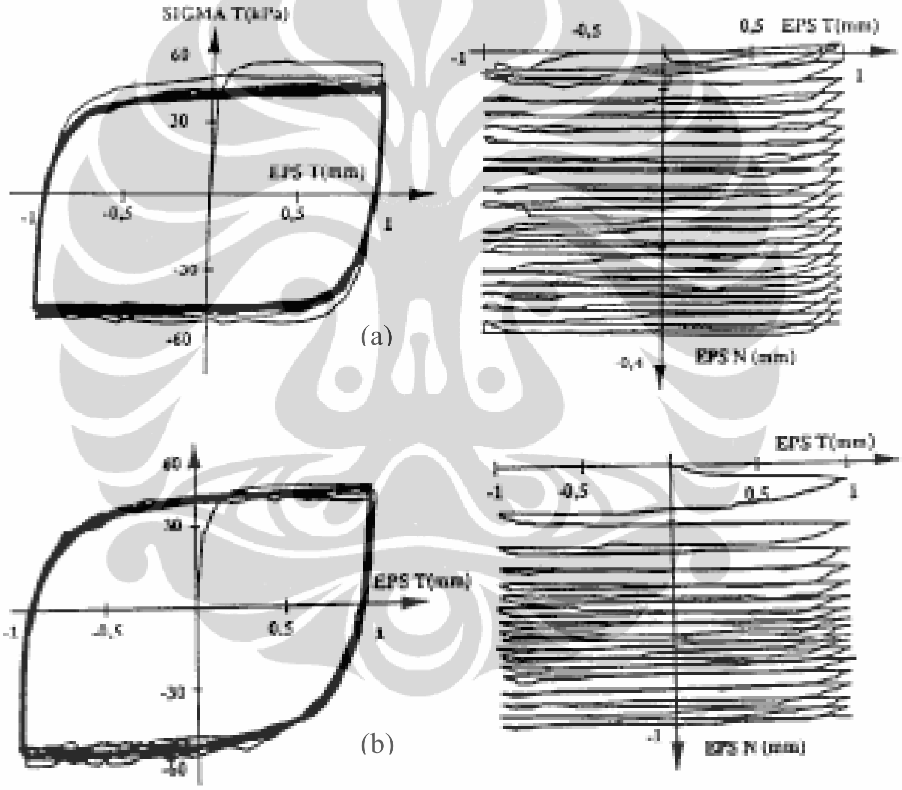


Figure 24. Interface behaviour under a CNL cyclic loading for a smooth interface:(a) dense sand (ID=90%); (b) loose sand (ID=15%) [Shahrour and Rezaie, 1997]

Another effect of the influence of the roughness of the interface, which can be observed is its influence on the normal displacement after *n* normal cycle. The normal displacement is much lower for a smooth interface than for a rough interface. There is for example in Figures 22 and 23, after 20 cycles, a normal displacement accumulated for a smooth interface of about 0.35 mm with dense sand (1 mm for a loose sand), while it reaches 1.2 mm for a rough interface (1.7 mm for a loose sand).

Regarding the shear stress ratio, it may be noted that the surface of the hysteresis loop for a smooth interface is higher compared to a rough interface.

II.5.2. Null normal displacement test

Figure 25 shows the results of a alternating cyclic shear test with a controlled shear displacement (-1mm < u_s <1mm) with a null normal displacement for a rough interface. For the dense sand, it can be observed that the cyclic loading induces a significant change in the normal stress. At each reversal of loading, the contractancy behaviour of the interface gives rise to a decrease in normal stress (σ_n). This phase is followed by an increase of σ_n because of the soil's expansion.

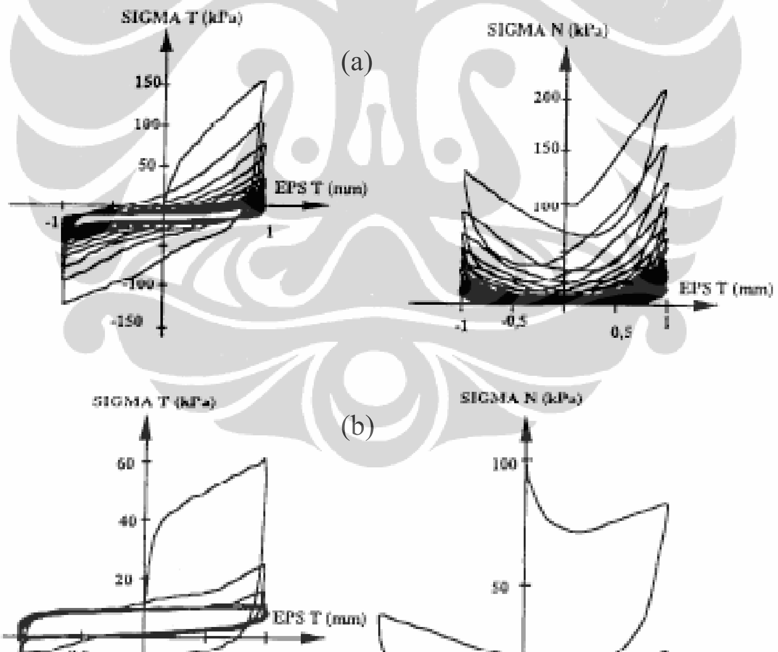


Figure 25. Interface behaviour under cyclic loading at a zero normal displacement (CNV) for rough interface: (a) dense sand (ID=90%); (b) loose sand (ID=15%) [Shahrour and Rezaie, 1997]

The overall contractancy behaviour of the interface induces a significant reduction in normal stress and, therefore, a significant decrease in the maximum shear strain, particularly in the early stage of the test. As the first loading induces a significant decrease of σ_n, the residual hysteresis shows no symmetry with respect to the axis of shear displacement. With a loose sand, there is also a decrease in normal stress at a higher rate compared to a dense sand.

Figure 26 presents the results of a CNV test for a smooth interface. There is an overall decrease of normal stress and a decrease in the resistance to shear forces during cyclic loading, especially in the first cycles. With regard to the displacement curve, we see that the normal stress first decreases at the beginning of the reversal of the load before increasing due to expansion in the case of dense and loose sands. These phenomena are less important than for a rough surface. With this observation, it can be concluded that , for this type of test, the characteristic friction angle is not identical to the residual friction angle for a smooth interface. This remark suggests that the characteristic angle might also depend on the normal stress increments during each cycle.

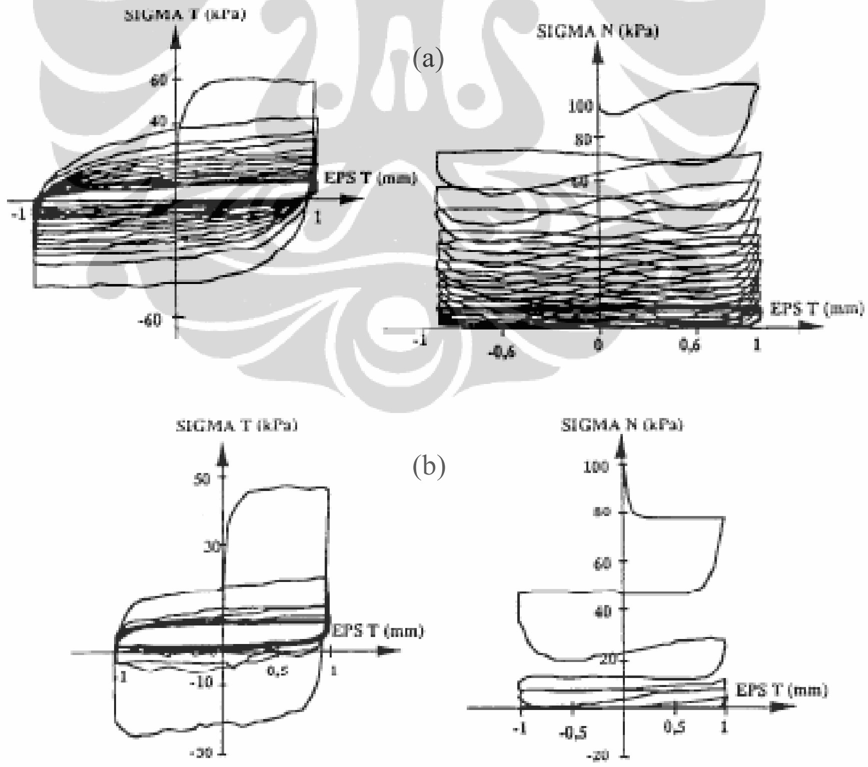


Figure 26. Interface behaviour under cyclic loading at a zero normal displacement (CNV) for smooth interface: (a) dense sand (ID=90%); (b) loose sand (ID=15%) [Shahrour and Rezaie, 1997]

Another interesting observation is that the displacement curve for a loose sand shows a dilatancy during loading and not during unloading.

II.6. PARTIAL CONCLUSION

Physically, the soil-structure interface is a thin layer of soil consisting of granular particles. When the interface is rough, its behaviour is virtually identical to this of the ground. For a smooth interface on the other hand, different behaviours were observed, particularly concerning the concept of critical state, which is no longer valid for a monotonic solicitation. A smooth interface is therefore a thin boundary layer whose behaviour is almost not influenced by the soil.

Most of these observations being based on tests with constant normal stress, the influence of the variation of normal stress on non-drained tests is often ignored. The results of a cyclical CNV test show that the friction angle feature can also depend on the increments of the interstitial pressure.