*Original Article*

# Laboratory investigations of sand-smooth steel interface under monotonic and cyclic loadings

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## Abstract

Frictional characteristics of the interface between a structure and surrounding soil, which can commonly be found in many geotechnical works, play an important role in evaluating the bearing capacity. The objective of this paper is to present some results of an experimental series on sand – smooth steel interface under constant normal load (CNL) and constant normal stiffness (CNS) conditions. First, the monotonic interface direct shear tests were performed in order to determine the peak shear stress ratio or peak friction angel of sand – smooth steel interface. Then the cyclic test campaigns were designed to perform. When subjecting to cyclic loading, the interface evidently showed the contractive behavior. Under CNS condition, this contraction led to the degradation in normal stress acting on the interface and then the stress state moved towards the peak (or residual) stress ratio line in the stress plane.

**Keywords:** sand-steel interface, direct shear test, shear stress – controlled tests, mean cyclic path, cyclic degradation

## 1. Introduction

The response of soil – structure systems such as deep and shallow foundations, tunnels, retaining walls, reinforced earth and joints in rocks is generally influenced by the characteristics of the interfaces. The behavior of the interface, which is distinct from those of the granular material and the structure, may change from one system to another, depending on the nature of soil as well as the surface roughness of structure. For defining the soil – structure interface thickness, it is very delicate and requires the high - quality measuring equipment. Many experimental observations have shown that the interface thickness is mainly influenced by grain size, surface roughness and initial density of soil. In a general manner, the approximation of 7-14 $D_{50}$ can be used to represent the interface thickness.

Developments of accurate understanding of the mechanical response of interfaces have widely been performed from appropriate laboratories and field tests in order to properly describe the interface behavior. To contribute a better understanding, a number of refinements and modifications of many devices have continuously been carried out. For example, an interface simple shear device which consists of a stack of plates confining the sample was designed to measure separately the shear deformation and shear displacement of the sample (Fakkarian & Evgin, 1995; Kishida & Uesugi, 1987). Obviously, many laboratory interface shear tests have been carried out by using a modified version of direct shear device as a result of less technical difficulties (e.g., Mortara, 2001; Mortara, Mangiola, & Ghionna, 2007; Pra-ai, 2013; Pra-ai & Boulon, 2017; Tabucanon, Airey, & Poulos, 1995). A ring shear device, which is a powerful device for investigating the soil – structure interface behavior (Kelly, 2001; Yoshimi & Kishida, 1987), is also mentioned. On the ring shear device, the sample is ring shape which has no change in the cross sectional area of the shear plane and can be sheared through an uninterrupted displacement. This device also provides the homogeneity of stress state as the test

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proceeds. However, it is difficult to perform the tests as a result of operational reasons.

In general, the problems involving the retaining wall and slope stability can be described by the interface behavior under constant normal load (CNL) condition. However, the investigation of pull – out tests of model pile embedded in sand showed that the variations of shaft friction coefficient mainly involved the volumetric behavior of sand adjacent to the pile. Boulon and Foray (1986) performed a laboratory test of pile – soil interface which could be interpreted as an interface shear test under constant normal stiffness (CNS) condition. In this approach, the soil adjacent to the pile induces a normal stiffness ($k$), depending on the shear modulus of soil ($G$) and the diameter of pile ($D$), which can be expressed as;

$$k = \frac{4G}{D} \tag{1}$$

In this boundary condition, the normal stiffness imposed to the interface ($k$) can then be given by;

$$k = \frac{\Delta\sigma_n}{\Delta[u]} \tag{2}$$

where $\Delta\sigma_n$ is the variation of normal stress and $\Delta[u]$ is the variation of normal displacement. This definition can be considered in three different conditions: constant normal stress (CNL, $k = 0$), constant volume (CV, $k \to \infty$) and constant normal stiffness (CNS, $0 < k < \infty$).

Interestingly, at CNS conditions, the normal stress acting on the interface mobilizes during shear loading. When the soil has a tendency to dilate, an increase in the normal stress associated with shear stress can be found. On the other hand, a significant degradation in normal stress can be found when the soil behaves contractively. The effect of normal stiffness becomes more crucial when the interface is subjected to cyclic loading. In practice, various cases of foundation structures under cyclic loading with a large number of cycles are commonly encountered. The environmental loads of waves and currents result in the significant degradation in pile shaft resistance of off – shore foundations. Likewise, the traffic loadings which are rather small might lead to the loss of serviceability of railways and bridges in the long term.

Experimental investigations of geomaterials under cyclic loading with a large number of cycles are relatively rare. In this approach, the principal citation works are relevant to the very complete cycle of cyclic triaxial tests (Wichtmann, Niemunis, & Triantafyllidis, 2005; 2006). In the context of soil-structure interfaces under cyclic loading, some results concerning experimentation and modeling are available (e.g., Dejong, Randolph, & White, 2003; Fakharian & Evgin, 1997; Mortara, Boulon, & Ghionna, 2002; Mortara *et al.*, 2007; Shahrour & Rezaie, 2002). However, these investigations described the low number of cycles (typically, $N < 50$ where $N$ is the number of cycles) with its classic characters.

Pra-ai and Boulon (2017) performed a series of cyclic direct shear tests with a large number of cycles on Fontainebleau sand – rough material interfaces under CNL and CNS conditions. In case of cyclic loading under CNS condition, the effect of medium and high values of imposed normal stiffness was investigated. They highlighted that the CNS cyclic paths behaved contractively and this contraction consequently led to a drop of normal stress associated with shear stress. This phenomenon is often called friction degradation. Nevertheless, the studies mentioned above only focused on the rough surface structure and medium to high values of imposed normal stiffness. The cyclic interface behavior with smooth surface structure, low values of imposed normal stiffness, and different cyclic amplitudes and stress ratios is still an open question and then appeals to additional research works.

This paper describes some of experimental results carried out from a series of direct shear tests on sand – smooth steel interface. First, the experimental parametric investigation under monotonic loading is reported. Subsequently, the results of cyclic shear stress – controlled tests are described. During cyclic loading, the mean cyclic values (i.e., the middle of each cycle) were considered rather than the description of the detail of each cycle. The effect of low value of imposed normal stiffness is also discussed.

## 2. Experimental Device and Materials

### 2.1 Interface direct shear device

A wide variety of devices has been used to investigate the behavior of soil – structure interface in laboratory scale. The necessary requirement for laboratory testing is to simulate the field conditions as closely as possible. With less technical difficulty, a modified version of direct shear device has widely been used. In this study, a modified direct shear device, on which the boundary of CNL and CNS conditions can be applied, was used (Figure 1). The upper shear box which contains the sample has a diameter of 63.5 mm and the lower shear box was replaced by the steel plate with a dimension of 120 x 120 x 20 mm. This steel plate represents a smooth surface roughness. In CNL condition, the normal load was applied to the sample by a scissor jack acting on the level arm. A load cell was placed between the scissor jack and the level arm to measure the normal load. When performing a test under CNS condition, a spring acting between the scissor jack and the load cell was used to supply an imposed normal stiffness. A variety of imposed normal stiffness can be executed by changing the spring. According to Pra-ai and Boulon (2017), the high value of imposed normal stiffness ($k$ = 5000 kPa/mm) can be attributed to an approximately constant volume condition. In this study, the low values of imposed normal stiffness ($k$ = 212 and 628 kPa/mm) were investigated. The application of shear loading can be done by a motor which can directly be controlled by a computer. A shearing speed of 0.5 mm/min was set in this study (both monotonic and cyclic tests). From experimental point of view, the slow speed of shearing (0.20 – 0.6 mm/min, Dejong *et al.*, 2003; Hu, & Pu, 2003; Mortara *et al.*, 2007; Tabucanon, Airey, & Poulos, 1995) has no influence on the test results.

During testing, the variables of stresses applied to the interface (normal, $\sigma_n$, and shear, $\tau$, components) and displacement vectors (normal, $[u]$, and shear, $[w]$, components) were measured; Figure 2. Note that the define-
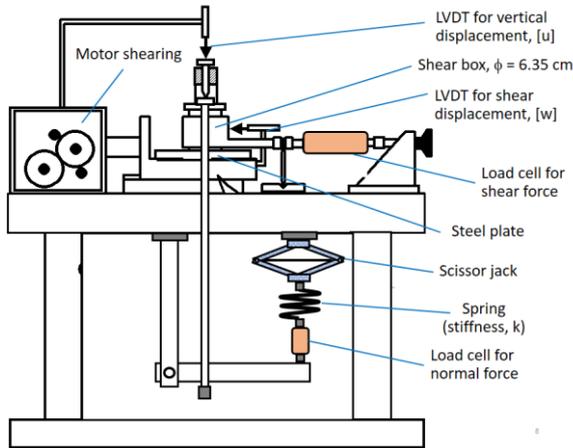
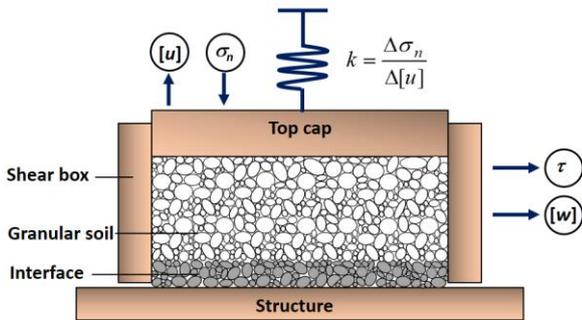Figure 1.　Schematic view of the interface direct shear device.



Figure 2.　Interface direct shear variables.

tion of normal component was given as $\sigma_n > 0$ in compression and $[u] > 0$ in dilatancy. In cyclic test series, the cycles in terms of shear stress $(\Delta\tau)$ were applied. Two thresholds of shear stress (maximum and minimum) were carried out by a computer.

## 2.2 Materials

　　　　Based on laboratory investigations, it has been found that one of the main effects on interface test responses can be attributed to a surface roughness of structure (Lu & Pu, 2003; Mortara *et al*, 2007; Uesugi & Kishida, 1986). Generally, the surface roughness of structure can be measured in terms of a maximum height, $R_{max}$ (the relative height between the highest peak and the lowest valley along a surface profile over a 2.5 mm gauge length, Uesugi & Kishida, 1986). Later, the diameter of sand particle was incorporated in instead of 2.5 mm gauge length in order to correlate the surface roughness with the interface friction coefficient (Uesugi, Kishida, & Yasunori, 1989). A normalized roughness $(R_n = R_{max}/D_{50})$ was then defined to evaluate the surface roughness. The polished steel plate used in this study represented the smooth surface structure.

　　　　This experimental investigation was performed by using cleaned sand derived from Chiang Rai province in the north of Thailand. Figure 3 shows a grain size distribution of
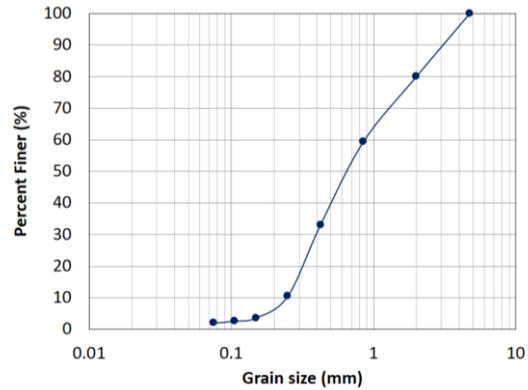


Figure 3.　Grain-size distribution of tested sand.

this sand which has a mean grain diameter ($D_{50}$) of 0.65 and a coefficient of uniformity ($C_u = D_{60}/D_{10}$) of 0.29. The maximum ($\gamma_{d\,\max}$) and minimum ($\gamma_{d\,\min}$) dry densities are 17.6 and 14.9 kN/m³, respectively. The samples were prepared with an average height of 20 mm in dry condition. Two distinct relative densities, i.e., $D_r = 85\%$ for dense sample and $D_r = 35\%$ for loose sample were investigated. To achieve the required density, several techniques were applied. A simple pluviation was used for loose sample while tamping and vibration techniques were used for achieving dense sample. For the purpose of the sample preparation, the bottom of shear box was coated by a thin layer of silicone grease in order to prevent the direct friction between shear box and the steel plate. This technique can also prevent the leakage of fine particles of sand from the gap between the shear box and the steel plate. If the leakage occurs during shear loading, the fictitious contraction for correcting the normal displacement has to be taken into account (Pra-ai & Boulon, 2017).

## 3. Experimental Results

### 3.1 Stress variables

　　　　In terms of a large number of cycles ($N > 50$, according to Wichtmann *et al.*, 2005; 2006), the first cycle is found to be very different from the subsequent ones. The so called regular cycles ($N > 2$) were then considered for determining the strain accumulation. In this approach, the term of mean cyclic path (Pra-ai & Boulon, 2017) was considered rather than the detail of each cycle in such a way that it indicated either an increase or decrease of state variables. To clarify the characterization of mean cyclic paths, Figure 4 illustrates the stress variables in stress plane. The relationship of peak stress ratio ($\eta_p$) and the peak friction angle ($\delta_p$) of soil – structure interface can be expressed as:

$$\eta_p \;=\; \tan\delta_p \;=\; \frac{\tau_p}{\sigma_n} \tag{3}$$

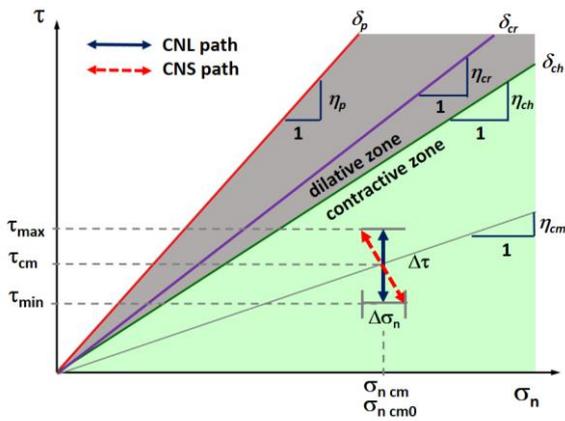where $\tau_p$ is the peak shear stress. The critical value of stress ratio and friction angle can respectively be given as $\eta_{cr}$ and

Figure 4.　Characterization of cyclic variables in stress plane (after Pra-ai & Boulon, 2017).

$\delta_{cr}$. The characteristic stress ratio ($\eta_{ch}$ corresponding to the friction angle, $\delta_{ch}$, separating the dilative and contractive domains according to Luong, 1976) can be expressed in the range of $\eta_{ch} < \eta_{cr} < \eta_p$. The mean cyclic stress ratio ($\eta_{cm}$) is

$$\eta_{cm} \quad = \quad \frac{\tau_{cm}}{\sigma_{ncm}} \tag{4}$$

where $\tau_{cm}$ is the mean cyclic shear stress (or the middle of cycle). $\sigma_{ncm}$ is the mean cyclic normal stress. Note that $\sigma_{ncm0}$ is the initial value in CNS condition. The normalized cyclic amplitude can alternatively be used as

$$\Delta\eta \quad = \quad \frac{\Delta\tau}{\sigma_{ncm}} \tag{5}$$

where $\Delta\tau$ is the cyclic amplitude in terms of shear stress.

### 3.2 Monotonic interface direct shear tests

Prior to a series of cyclic test campaign, monotonic tests were firstly performed in order to evaluate the main variables. To design the cyclic tests, a determination of peak and critical stress ratios was necessary. For the purpose of determining the peak stress ratio in dependence on the relative density, CNL and CNS monotonic tests with initial stress ($\sigma_n$) of 100, 200 and 300 kPa were performed. Pra-ai and Boulon (2017) proposed the concept of interpretation of interface direct shear tests. Following their hypothesis, a soil – structure interface composes of two part: active lower part (very thin layer) and passive upper part (the rest of the sample). The relationship of the normal displacement measured at the top of the sample ($[u]_s$) and at the interface ($[u]_i$) can be expressed as:

$$\frac{[u]_s}{h} \quad = \quad \frac{[u]_i}{t} \tag{6}$$

where $h$ is the height of entire sample and $t$ is the interface thickness.

After finishing the sample preparation, once the normal stress is applied, the normal displacement measured at the top of the sample can expressed as $[u]_{son}$. When considering the evolution of normal displacement variables as a function of time (or number of cycles), the additional displacement at the top of the sample coincides with the additional relative displacement of interface, but these two displacements are not the same. This means that any measurable change of the sample ($\Delta[u]_s$) provides the corresponding change of interface ($\Delta[u]_i$):

$$\Delta[u]_s \quad = \quad \Delta[u]_i \tag{7}$$

In general, $[u]$ is used to stand for the normal displacement of the interface for the sake of simplicity.

Figure 5 shows the typical monotonic interface results with $\sigma_n$ = 200 kPa on loose and dense samples. It was found that under CNL and CNS conditions the same aspect of shear stress could be observed. The shear stress increased as a function of shear displacement ($[w]$) until reaching a peak value and then continued to a residual (or critical) value without showing any softening phase whether on dense or loose sample (Figure 5a and 5d).

The imposed normal stiffness provided several important aspects concerning the mobilization of normal stress acting on the interface. On loose sample, the degradation in normal stress was found as a result of the gradual contraction. The rate of normal stress degradation obviously increased as a result of an increase in normal stiffness (Figure 5b). It was found that with $k$ = 628 kPa/mm the normal stress dropped from 200 kPa to 123 kPa at $[w]$ = 5 mm. On dense sample, the dilative behavior which could insignificantly be observed, depending on $\sigma_n$ and $k$, provided a slight variation in normal stress associated with shear stress. In the case of $k$ = 212 and 628 kPa/mm, at the beginning of shear loading, the normal stress slightly decreased as a result of contractive behavior and afterward it slightly increased due to the dilatancy. Consequently, the shear stress evolved until reaching the residual (or critical) value without showing softening phase. The CNS tests on dense sample showed that the normal stress increased from 200 kPa up to 212 kPa and 228 kPa for $k$ = 212 and 628 kPa/mm, respectively (Figure 5e).

When considering the volumetric behavior (Figure 5c and 5f), the contraction was obviously found during shear loading on loose sample while the dilatancy phase could be observed on dense sample. However, this dilatancy was not significant, $[u]$ = 0.04-0.05 mm. This is due to the slippage occurring along the contact surface between granular soil and smooth plate. Contrarily, the intense shear localization generally occurs between the granular soil and rough structure (Mortara *et al*, 2007; Uesugi & Kishida, 1986).

Figure 6a and 6b respectively show the stress paths in $\tau - \sigma_n$ plane of monotonic tests indicating the effect of imposed normal stiffness on loose and dense samples. The
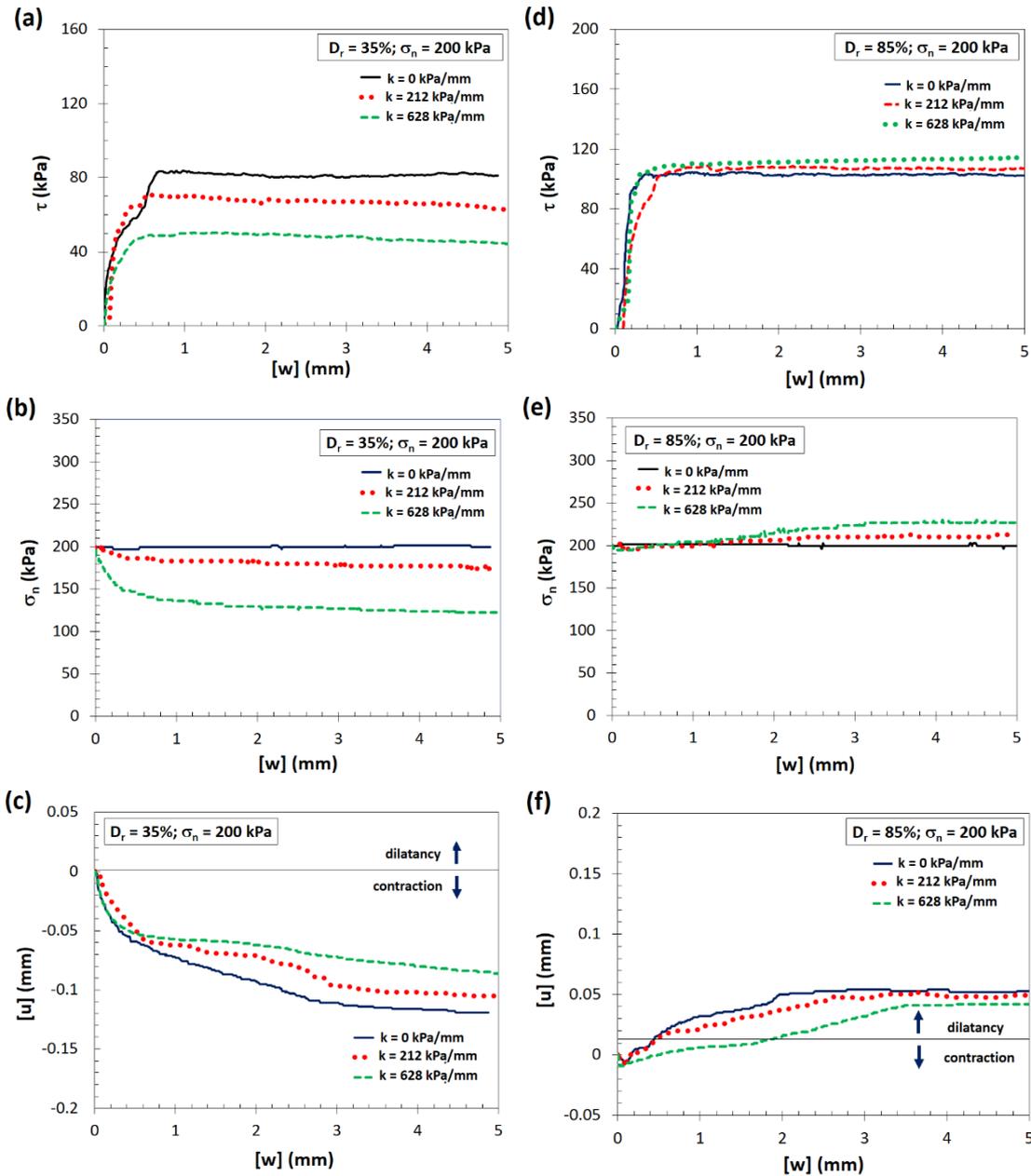
**(a)**



**(d)**



**(b)**



**(e)**



**(c)**



**(f)**



Figure 5.    Monotonic interface tests with three different values of normal stiffness ( $k$ ) for $\sigma_{n0}$ = 200 kPa: (a), (b) and (c) on loose sand ( $D_r$ = 35 %); (d), (e) and (c) on dense sand ( $D_r$ = 85 %).

peak stress ratio $(\eta_p = \tau_p / \sigma_n)$ slightly decreased with an increase in normal stress. The range of $\eta_p$ = 0.35 – 0.5 was determined on loose sample while dense sample provided $\eta_p$ = 0.47 – 0.60. Since the responses of interface between sand and smooth plate did not show the softening phase of shear stress, the peak value coincided with the residual one. It was found that the effect of imposed normal stiffness had no influence on the friction of sand – steel interface under monotonic loading condition. For each density, when plotting the peak shear stress against the corresponding normal stress, the determination of peak (or residual) stress ratios can be

achieved by the fitting of a linear function through the origin. Then, the peak stress ratios $\eta_p$ = 0.50 and $\eta_p$ = 0.38 were held for dense and loose samples, respectively.

In order to verify the regularity of the CNS shear loading path, Figure 7 shows the $\sigma_n - [u]$ diagram of the tests with $\sigma_{n0}$ = 200 kPa and $k$ = 628 kPa/mm allowing for the relationship of $k = \Delta\sigma_n / \Delta[u]$. Starting from the initial normal stress $\sigma_{n0}$ = 200 kPa, the variation of normal stress and normal displacement obviously followed equation (2) during shearing phase. Even though scattered data still existed, the
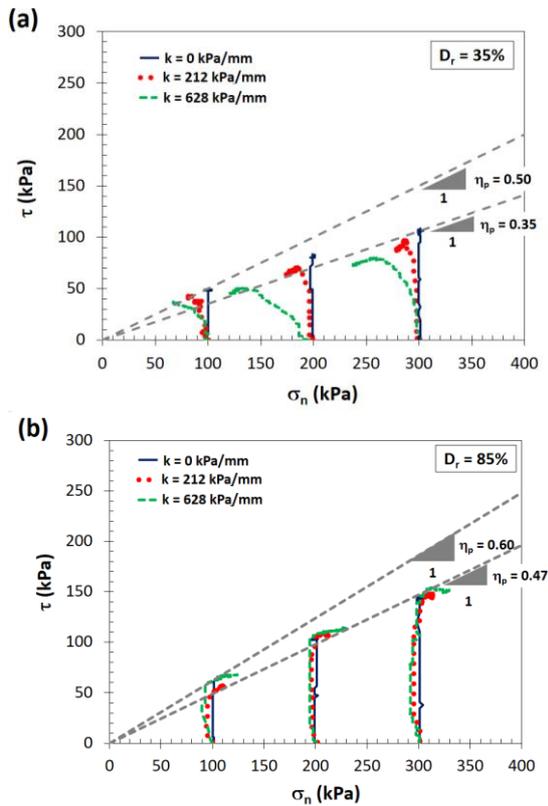
Figure 6.   Stress paths of monotonic interface direct shear tests with $k$ = 0, 212 and 628 kPa/mm: (a) on loose sand ($D_r$ = 35 %); (b) on dense sand ($D_r$ = 85 %).
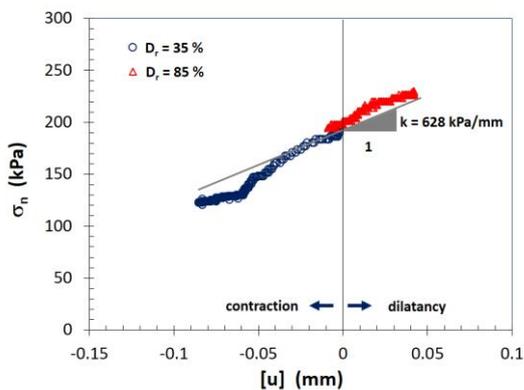


Figure 7.   Relationship of imposed stiffness during shear loading on loose and dense samples with $\sigma_{n0}$ = 200 kPa and $k$ = 628 kPa/mm.

trend fitted through the data could be satisfactory. From experimental point of view, it is worth noting that loose samples are less stable than dense samples as a result of the occurrence of oscillation during shearing phase.

### 3.3 Cyclic interface direct shear tests

Since the peak stress ratios on both densities were achieved from monotonic tests, the cyclic test campaign could

then be decided to perform. The purpose of cyclic tests was to describe the basis aspect of granular soil – smooth interface behavior. In this part, preliminary test results are discussed. Table.1 summarizes the cyclic tests examined in the present work.

To identify the main characteristics of cyclic response of sand – steel interface, Figure 8 shows a typical $\tau - [w]$ diagram of a CNS cyclic test (CNS.D2) performed with $\sigma_{ncm0}$ = 200 kPa, $\Delta\tau$ = 40 kPa, $\tau_{cm}$ = 40 kPa and $k$ = 628 kPa/mm. It was found that the first cycle was very different in shear displacement from the subsequent ones and the shear displacement rate decreased as a function of number of cycles. Since a large number of cycles in terms of shear stress were applied, the ambiguity of cyclic responses would arise. Cyclic responses were the irreversible relative displace-ments and, in addition, the change in normal stress for CNS tests. In this paper, for the sake of simplicity, the mean cyclic paths of ten consecutive cycles were then representative (i.e., red circles in Figure 8). It means that $N$ = 2, 12, 25, 50, 100, 150 and 200 can be used to express the mean cyclic path of $N$ = 2-11, 12-21, 25-34, 50-59, 100-109, 150-159 and 191-200, respec-tively.

Many experimental observations have shown that the gradual densification can be regarded as the main characteristics of cyclic granular soil – structure interface behavior. This gradual densification becomes crucial when the imposed normal stiffness is involved. Figure 9 shows the evolution of mean cyclic variables as a function of number of cycles ($N$) on both densities. On loose sample, CNS.L2, the test was performed with a cyclic amplitude of $\Delta\tau$ = 36 kPa ($4 < \tau_{cm} < 40$ kPa) and $k$ = 628 kPa/mm. The cyclic stress ratio was set at $\eta_{cm0}$ = 0.11 which could be considered to be relatively far from $\eta_p$ = 0.38. This signified that the cycles were set in very contractive zone. Obviously, the cyclic responses on loose sample showed a prominent contraction as well as the shear displacement in dependence of $N$ (Figure 9a and 9b). A substantial degradation in normal stress could be found during the first fifty cycles (Figure 9c). The mobilization of mean cyclic normal stress, which was initially set at 200 kPa before applying the cyclic loading, dropped to 176
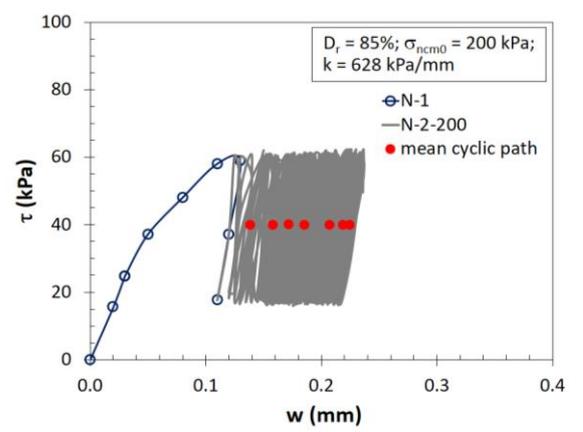


Figure 8.   Typical cyclic test result on dense sand – steel plate with $\sigma_{n0}$ = 200 kPa, $20 < \tau_{cm} < 60$ kPa and $k$ = 628 kPa/mm.

Table 1.　Cyclic test program.

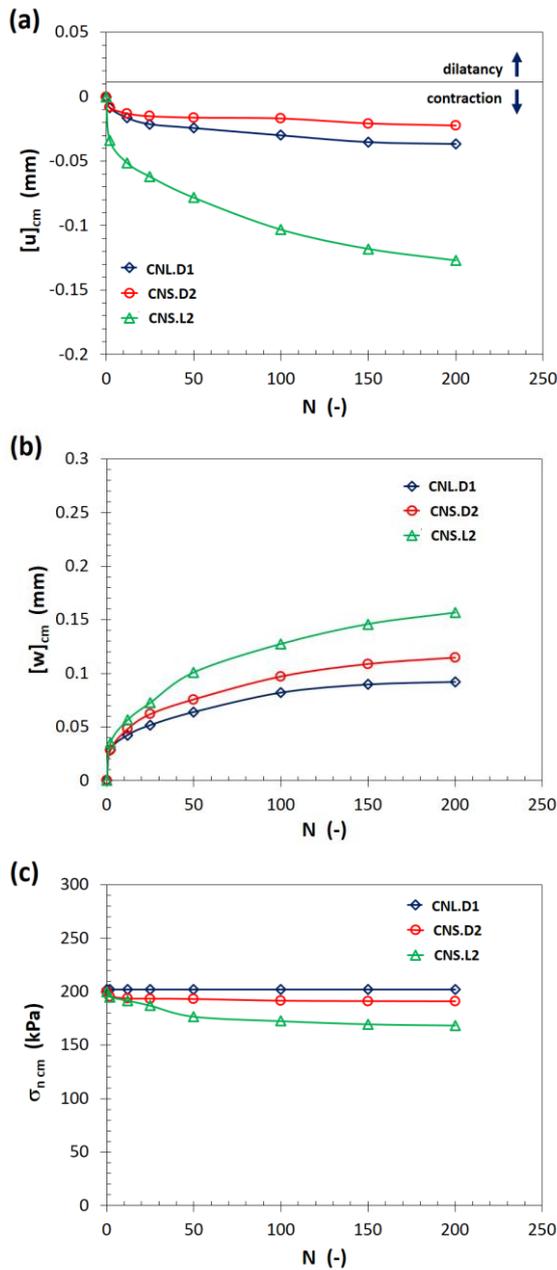| Tests | $D_r$ (%) | $\sigma_{ncm0}$, $\sigma_{ncm}$ (kPa) | $\tau_{cm}$ (kPa) | $\eta_{cm}$ (kPa) | $\Delta\tau$ (kPa) | $N$ (-) | $k$ (kPa/mm) |
|-------|-----------|------------------------|-----------|-----------|-----------|--------|-----------|
| CNL.D1 | 85 | 200 | 40 | 0.2 | 40 | 200 | 0 |
| CNS.D2 | 85 | 200 | 40 | 0.2 | 40 | 200 | 628 |
| CNS.L1 | 35 | 200 | 22 | 0.11 | 36 | 200 | 628 |



Figure 9.　Evolution of mean cyclic variables as a function of $N$ on sand – steel interface with $\sigma_{n0} = 200$ kPa; (a) $[u]_{cm}(N)$; (b) $[w]_{cm}(N)$; )c( $\sigma_{ncm}(N)$.

kPa at $N = 50$ and then continued slowly until reaching $\sigma_{ncm}$ = 168 kPa at $N = 200$.

On the other hand, dense sample provided a slight degradation in normal stress under CNS condition due to a very low attitude to contraction. The mean cyclic normal stress dropped from 200 kPa to 190 kPa during 200 cycles. Considering the volumetric behavior between CNL.D1 and CNS.D2, it was found that CNS.D2 with $k = 628$ kPa/mm showed less contraction as a result of the effect of boundary condition in the direction normal to the interface. Figure 9b also shows that the shear displacement of CNS.D2 was greater than that of CNL.D1 due to its lower normal stress acting on the interface.

Figure 10a and 10b shows respectively the stress paths of the cyclic interface tests under CNS condition describing the gradual degradation in normal stress on loose and dense samples. In this study, the cycles in terms of shear stress were applied. This indicated that two thresholds of shear stress were kept constant while the normal stress decreased asa function of $N$. Since the degradation of normal stress increased continuously, the stress state movement had a tendency to approach the peak (or residual) stress ratio line. It was found that the evolution of mean cyclic stress ratio as a function of $N$ on loose sample which started from $\eta_{cm0} = 0.11$ reached $\eta_{cm} = 0.13$ at $N = 200$ (Figure 10a). On dense sample, since a slight degradation in normal stress could be observed, the mean cyclic stress ratio which was $\eta_{cm0} = 0.20$ mobilized to $\eta_{cm} = 0.21$ at $N = 200$ (Figure 10b). When performing the test with the low value of $k$ as well as $\eta_{cm0}$, the stress state evolved slowly to the peak (or residual) stress ratio line on both densities. This indicated that a large number of cycles were required in order that the state stress could reach the peak (or residual) stress ratio line.

## 4. Conclusions

This study mainly focuses on the interface direct shear tests on dry sand and smooth steel plate under constant normal load (CNL) and imposed normal stiffness (CNS) conditions. A modified direct shear device was used, allowing for the investigation of interface shear behavior under several conditions. Two distinct densities of sand ($D_r = 25\%$ and 85%) were investigated. Firstly, monotonic tests were performed in order to characterize the basic feature of interface direct shear behavior both under CNL and CNS conditions. It was found that the effect of the imposed normal stiffness had no influence on the frictional behavior of sand – steel interface under monotonic loading condition. From the fitting of a linear function through the origin in stress plane, the peak shear stress ratios ($\eta_p$) of 0.38 and 0.5 can be achieved for
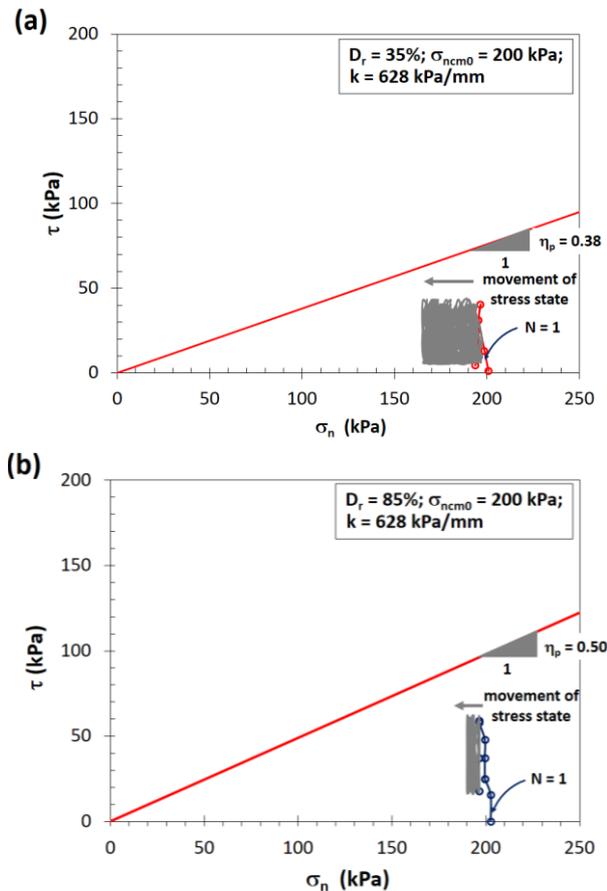
Figure 10. Stress path in plane of CNS cyclic interface direct shear tests with $\sigma_{ncm0}$ = 200 kPa and $k$ = 628 kPa/mm; (a) on loose sand ($D_r$ = 35 %) with $\Delta\tau$ = 36 kPa and $4 < \tau_{cm} < 40$ kPa; (b) on dense sand ($D_r$ = 85 %) with $\Delta\tau$ = 40 kPa, $20 < \tau_{cm} < 60$ kPa.

loose and dense samples, respectively. For sand – steel interface, the volumetric behavior was not prominent. This could be attributed to the slippage along the contact surface. The dilatancy and the contraction were found to be affected by the imposed normal stiffness. An increase of $k$ resulted in a reduction in dilatancy and contraction on dense and loose samples, respectively.

From monotonic results, the program of cyclic tests could then be decided to perform. These tests were performed by applying the cycles in terms of shear stress. The irreversible relative displacements and, in addition, the change in normal stress for CNS tests were measured. When $\eta_{cm}$ or $\eta_{cm0}$ was set to be relatively far from $\eta_p$ (i.e., $\eta_{cm0}$ and $\eta_{cm0} < \frac{1}{2}\eta_p$), the main characteristic of cyclic interface behavior was the gradual contraction under both CNL and CNS conditions. During an application of cyclic loading, the salient effect of normal stiffness and initial relative density could be observed. With $k$ = 628 kPa/mm, dense sample

showed a slight degradation in mean cyclic normal stress as a result of the insignificant contraction. Contrarily, the cyclic responses on loose sample showed a prominent contraction as well as shear displacement in dependence of *N*. This phenomenon led to a rapid degradation in mean cyclic normal stress and then the stress state had a tendency to move forwards the peak (or residual) stress ratio line in stress plane. When performing the CNS cyclic interface shear test with low value of $k$ and $\eta_{cm0}$, a large number of cycles were expected in order that the state stress could reach the peak (or residual) stress ratio line.

## Acknowledgements

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