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POST-CYCLIC BEHAVIOR OF GRANULAR SOIL- STRUCTURE INTERFACE DIRECT SHEAR TESTS

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ABSTRACT: The purpose of this paper is to present one of the most important phases of a series of cyclic direct shear tests on granular soil and rough material interface under constant normal stiffness (CNS) condition. These cyclic interface tests were performed in order to simulate the situation along the pile shaft subjected to a large number of cycles due to environmental or anthropic loadings. This post-cyclic phase can be performed by one single large cycle after the cyclic phase in order to characterize the change of interface resistance. The principal characteristic of interface subjected to cyclic loading is the progressive contraction. This phenomenon leads to the degradation in normal stress acting on the pile shaft and consequently the shear resistance decreases. The influence of relative density of granular soil, initial normal stress, level of stress ratio, cyclic amplitude and imposed normal stiffness on the post-cyclic responses is discussed.

Keywords: Post-Cyclic Loading, Interface Resistance, Constant Normal Stiffness, a Large Number of Cycles

1. INTRODUCTION

When civil engineering structures have to undergo cyclic loading condition, the bearing capacity of structures is often reductive. Designers often have to consider the cyclic bearing capacity and stiffness, as well as the permanent displacements due to cycling and potentially changing patterns of soil. Especially, many offshore oil rig works have to undergo cyclic loading conditions (wind, wave, machine operations, etc.) for a long life service. This reductive phenomenon can also be found in the serviceability of railways and bridges due to traffic loadings. Even though the magnitude of traffic loadings is rather small, a large number of cycles would be crucial. The most recent studies which concern a large number of cycles carried out by Wichtmann [17] can be found. Moreover, the recent developments of renewable installations on shore as well as off shore of energy sources bring the professionals and the researchers to be interested in the effect of very large number of cycles on the soil-structure interactions. Therefore, understanding the interface behavior subjected to cyclic loading is of significant importance. Indeed at the present time, there are not enough methods of reliable calculation of the structure foundations subjected to this stress type and most of the experts adopt the proposed safety factors to take into account the degradations of bearing capacity due to the cycles.

The studies of interface behavior have been specified on the conventional interface experiments with the constant normal load (CNL) condition. In this case, the normal load applied on the interface is kept constant during the process of shearing. Interestingly, when interfaces subjected to cyclic loading, numerous experimental investigations have been reported that the interface responses turn into a progressive densification with increasing number of cycles. This leads to the mobilization of shear strength ([1], [5], [9]).

Boulon and Foray [3] reported that the skin friction of granular soil-pile shaft decreased as a function of number of cycles. This can be attributed that cyclic loadings bring on the contraction of sand adjacent to pile. A decrease in volume of sand leads to a progressive decrease in lateral stress, and consequently a decrease of shear resistance.

A laboratory test of soil-pile interface can simply be interpreted as an interface under constant normal stiffness (CNS) condition (see Fig.1). Considering a pile with radius R_0, embedded in soil with a pressuremetric modulus (E_p), and the thickness of the interface layer (e) mobilized during the large localized shear (where $e << R_0$). The normal stiffness imposed to the interface (k), according to Boulon and Foray [3] resulting from the definition of the pressuremetric modulus (E_p), can be expressed as:

$$E_p = \frac{\Delta\sigma_n}{-2\Delta V / V_0} \cong \frac{\Delta\sigma_n}{-2\Delta[u]}\left(R_0 + e\right) \cong \frac{\Delta\sigma_n}{-2\Delta[u]} R_0 \qquad (1)$$

where $\Delta V/V_0$ is the relative volume change and consequently:

$$k = \frac{2E_p}{R_0} = \frac{\Delta\sigma_n}{-\Delta[u]} \tag{2}$$

where $\Delta\sigma_n$ is the variation of normal stress acting on the interface and $\Delta[u]$ is the variation of normal displacement.

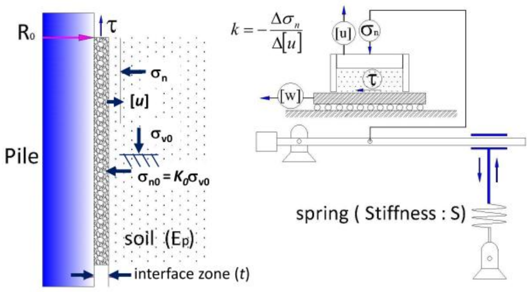

Fig.1 Localized shear zone along pile and a direct shear test with an imposed normal stiffness (after Boulon and Foray [3])

Under CNS condition, the behavior of soil-structure interface exhibits a mobilization of stress state acting on the interface. During shear loading phase, for example, dense sample commonly shows the dilative behavior which leads to an increase in normal stress associated with shear stress. Contrarily, a significant reduction in normal stress as well as shear stress can be found on loose sample. This phenomenon is due to the significant contraction.

The effect of imposed normal stiffness (k) on cyclic interface behavior becomes crucial. The main characteristic of cyclic interfaces under CNS condition performed by either direct shear or simple shear tests is the significant degradation in normal stress acting on the interface as a result of the gradual contraction. Consequently, the degradation of shear stress can be found ([1], [4], [5], [8], [9], [14]). In case of shear stress-controlled tests, the significant degradation of normal stress as a result of the progressive contraction accompanied with N brought the mean cyclic stress ratio (η_{cm}) defined as the ratio between mean cyclic shear stress (τ_{cm}) and mean cyclic normal stress ($\sigma_{n\,cm}$), i.e.,

$$\eta_{cm} = \frac{\tau_{cm}}{\sigma_{n\,cm}} \tag{3}$$

, to the critical state line ([10]-[12])

This paper presents some of experimental observations carried out from a series of direct shear tests of sand and rough surface structure under constant normal stiffness (CNS) condition with a large number of small cycles in terms of shear stress. The responses of post-cyclic phase will be discussed. This work was supported by the SOLCYP (French acronym for National Project in Piles under Cyclic Solicitations) research project.

2. DEVICES AND MATERIALS

With less difficulty in performing the interface test campaigns, the modified direct shear is used in this study as shown in Fig.2. The upper shear box has a diameter of 60 mm containing the specimen with the height of approximately 20 mm. This enables the preparation of sample in various conditions. The lower shear box is replaced by the steel plate on which the surface roughness is made by gluing a mixture of epoxy and Fontainebleau sand.

The surface roughness of structural materials can be quantified as the modified roughness in term of a maximum height R_{max}, which is the relative height between the highest peak and the lowest valley along a surface profile over the gauge length L = 0.20 mm ([15]).The value of roughness (R_{max}) can effectively be quantified by morphology method ([6]) which provides R_{max}= 0.20 mm. When normalizing R_{max} with mean particle diameter D_{50} of soil sample (i.e., $R_n = R_{max}/D_{50}$ [15], [16]), the structural plate used in this study can then represent the rough surface ($R_n \geq 0.10$ would be able to stand for rough surface [7], [15], [16]).

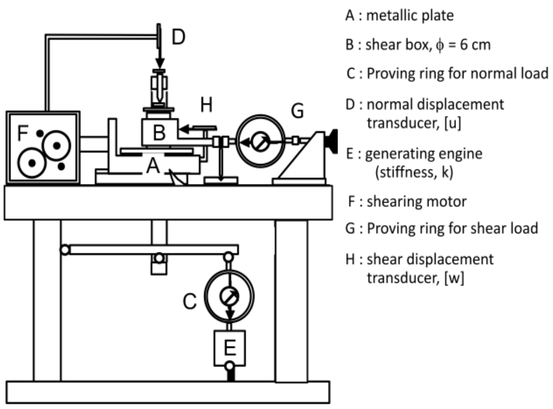

Fig.2 Experimental device

On interface direct shear device, the normal stress (σ_n) is applied vertically via a piston (top cap) by a generating engine in two directions (ensuring loading or unloading of the samples). The operation of this generating engine which is controlled by a computer enables the application of a constant normal stress (k = 0, CNL) and constant normal stiffness ($k \neq 0$, CNS) conditions in accordance with the control set:

$$C = \Delta\sigma - k[\Delta u] = 0 \quad (4)$$

herein $\Delta\sigma$ is the variation of normal stress acting on the interface and $\Delta[u]$ is the variation of normal displacement.

The tangential displacement of the structural surface which can directly be controlled by the computer provides the shear loading. It can be assumed that there is no influence of shear rate on test results by using the shear rate with low range. In this study, the compatible maximum of shear rate of 0.5 mm/min is then used with a sufficient data acquisition. In case of cyclic (shear stress-controlled) tests, two thresholds (high and low, adjustable) of shear stress are prescribed, causing a reversal of shear direction when they are reached.

In this study, the measured and recorded variables are the stress vector applied on the interface (normal, σ_n and shear, τ components) and the relative displacement vector on soil-structure interface (normal, $[u]$ and tangential, $[w]$ components). In this study, it is worth noting that the normal stress and normal relative displacement can be defined as $\sigma_n > 0$ in compression and $[u] > 0$ in dilation.

Two sands were tested in this study, i.e., Loon-Plage (post-glacial Flandrian) and Fontainebleau sands. The first one was tested with saturated specific weight (γ_{sat}) of 18.64 kN/m^3 and water content (w) of 18%. This initial density can be considered as dense ($I_D \approx 75\%$, [2]) condition. This sand was tested to combine the in situ axial pile tests in order to complement and extend the existing data set ([13]). The second one was tested in dry condition with relative density of approximately 90% (dense condition). The grain size distribution of these two sands obtained by sieving method can be shown in Fig. 3. In particular, the cyclic interface test on Loon-Plage sand was performed with N = 5000 cycles and the amplitude of cycles can be attributed to be bigger than those performed on Fontainebleau sand. To achieve the desired samples, two techniques of pouring and tamping were commonly used. In this study, all tests were carried out as completely drained tests.

In addition, to prepare the sample, a spacing of 0.3 mm between the rough plate and the upper shear box was set by a pair of brass foils. This technique is used in order to prevent the direct friction between the shear box and the rough plate. However, this gap would inevitably provide the leakage of fine particle which would take place during shear loading, especially when cyclic tests were performed. A simple inclusive correction can then be performed by considering the loss of sand as the fictitious contraction.

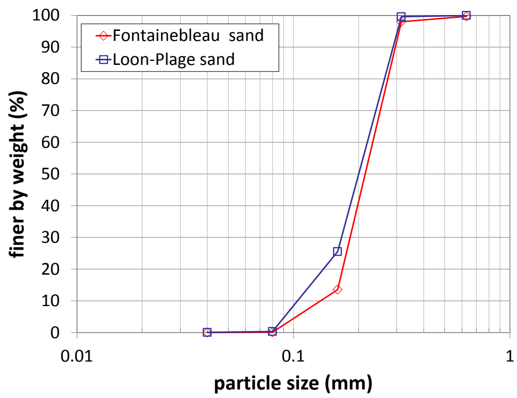

Fig. 3 Grain size distribution of Fontainebleau and Loon-Plage sands used in this study

3. TEST RESULTS

The cyclic tests were performed in order to investigate some of the factors influencing the interface behavior under CNS condition. Prior to performing those tests, monotonic tests were performed in order to evaluate the main variables (Table 1), e.g., the three values of stress ratio defined as $\eta = \tau/\sigma_n$ [11] ;

- peak stress ratio (η_p),
- critical stress ratio (η_{cr})
- characteristic stress ratio (η_{ch}, separating the dilative and contractive domains, where $\eta_{ch} < \eta_{cr} < \eta_p$)

The responses of monotonic tests then provided a series of cyclic tests (e.g., the level of initial mean cyclic stress ratio, η_{cm0} and the amplitude of cycles, $\Delta\tau$). The cyclic tests examined in the present work are summarized on Table 2.

The cyclic test procedure in this study consists of 5 consecutive phases:

- 1st Phase: the application of normal stress since the neutral state until mean cyclic normal stress ($\sigma_{n\,cm}$)
- 2nd Phase: the application of shear loading until mean cyclic shear stress (τ_{cm})
- 3rd Phase: the application of N cycles in terms of shear stress ($\Delta\tau$)
- 4th Phase: one large cycle of shear (after N cycles were reached) to failure

- 5th Phase: discharge in shear stress (τ) and then the normal stress until σ_n = 0, respectively.

In this test campaign, one of the most important phases of the cyclic tests is the post-cyclic phase. This phase can generally be carried out by one single large cycle after the cyclic phase (4th phase) in order to characterize the change of interface resistance (δ_p or η_p) due to cyclic loading. However, under CNS condition this post-cyclic phase could not be carried out when performing the initial mean cyclic stress ratio (η_{cm0}) close to the critical stress ratio (η_{cr}) due to the early termination (i.e., the stress state move towards the critical state line [10], [12]).

Table 1 Main variables from monotonic interface direct shear tests

Variables	Loon-Plage sand	Fontainebleau sand
η_p	0.90	0.79
η_{cr}	0.64	0.566
η_{ch}	0.59	0.555

Table 2 CNS cyclic test program

Condition	Loon-Plage sand	Fontainebleau sand
$\sigma_{n\ cm0}$ (kPa)	100	310
$\Delta\tau$ (kPa)	44	10
η_{cm0} (-)	$1/2\,\eta_p$	$1/2\,\eta_p$
k (kPa/mm)	143	1000
N (-)	$5\text{x}10^3$	$32\text{x}10^3$

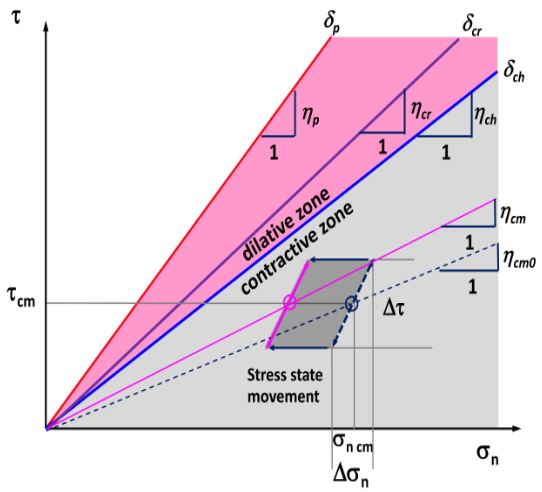

Fig. 4 Typical cyclic CNS paths of soil-structure interface.

As can be deduced from experimental observations, within the range of $\eta_{cm0} < \eta_{ch}$ the principal characteristic of interface subjected to cyclic loading was the progressive contraction. Considering the stress plane in Fig.4, during the interface subjected to cyclic loading under CNS condition, the shear stress was kept constant while the normal stress decreased as a function of N then the mean cyclic stress ratio η_{cm} which started from the beginning (η_{cm0}) increased and then moved to the critical state line.

Fig. 5 shows the post-cyclic behavior, after the application of N = 5000 cycles, for $\sigma_{n\ cm0}$ = 100 kPa on Loon-Plage sand with k = 143 kPa/mm, η_{cm0} = 0.44 (22 < τ < 66 kPa) in comparison with CNS monotonic test. As can be observed in Fig. 5a, the degradation of normal stress as a function of N increased very slowly. This was due to the low value of imposed normal stiffness (k = 143 kPa/mm). At the beginning of post-cyclic phase, the normal stress started at $\sigma_{n\ cm} \approx$ 90 kPa (Fig. 5a, 5d). The values of peak stress ratio after cyclic phase ($\eta_{p\ post} \approx$ 0.96) was obviously higher than that of CNS monotonic test (see Fig. 5b, 5d). Considering the volumetric behavior, [u]−[w] diagram in Fig. 5c, the interface response provided more densification (contraction) during cyclic phase. However, the dilation rate at post-cyclic phase was not different from that of CNS monotonic test.

Indeed, dense sand has high tendency of grain breakage within the localized shear zone. The change in particle size could be attributed to I_{D0}, σ_{n0}, η_{cm} and $\Delta\tau$. The abrasion between the grains and the surface roughness of plate on dense sand therefore provided an increase in crushing and wear of grains. The finer grains resulting from crushing grains during cyclic loading then replaced the void within the interface zone. Uesugi and Kishida [15] concluded that during cyclic loading an increase of crushing particles due to the high intensity of stress state, increasing the normalized the surface roughness (R_n) of the soil-structure interface, then led to the higher coefficient of interface friction.

In case of dense Fontainebleau sand with $\sigma_{n\ cm0}$ = 310 kPa, η_{cm0} = 0.35 (105 < τ < 115 kPa), this test was cyclically performed until N = $32\text{x}10^3$. It was found that the stress state after the application of cyclic loading ($\eta_{cm} \approx$ 0.45) was still so far from the critical line, subsequently the post-cyclic phase was performed instead (Fig. 6). In this case, the degradation of normal stress as a function of N increased very slowly even k = 1000 kPa/mm was applied. This can be described that with high value of $\sigma_{n\ cm0}$ and low level of η_{cm0} the initial stress state was so far from the critical state line, then the stress state was able to evolve further (see Fig. 6d).

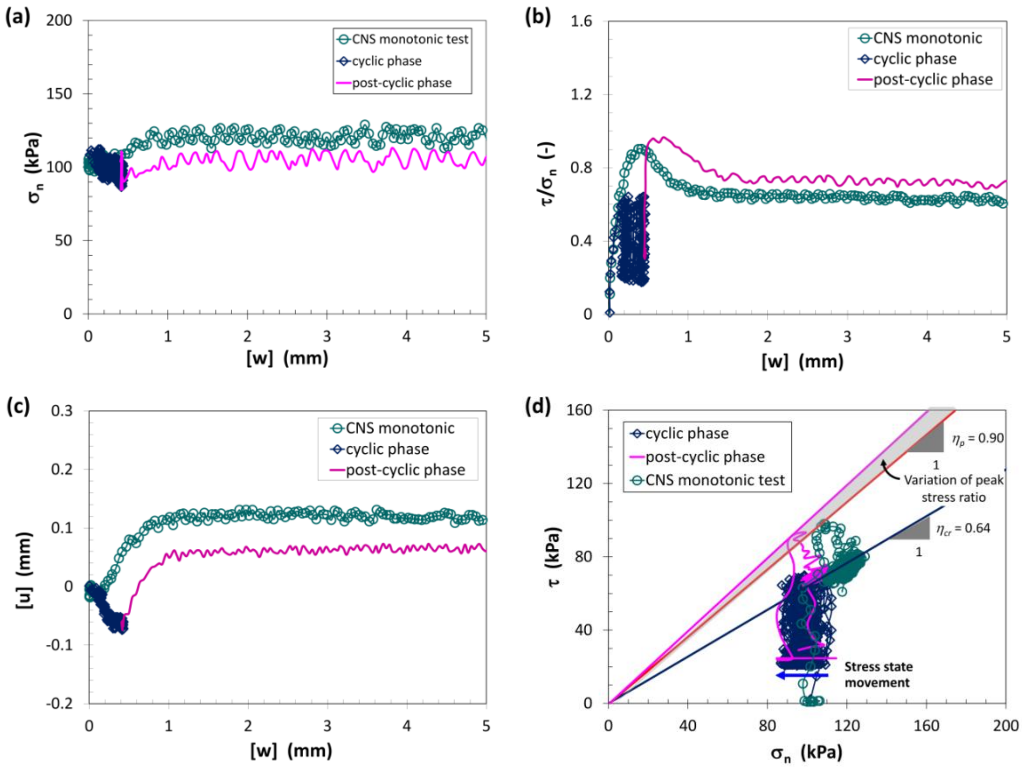

Fig. 5 Post-cyclic phase for $\sigma_{n\ cm0}$ = 100 kPa on dense Loon-Plage sand with k = 143 kPa/mm, η_{cm0} = 0.44 (22 < τ < 66 kPa) in comparison with CNS monotonic test.

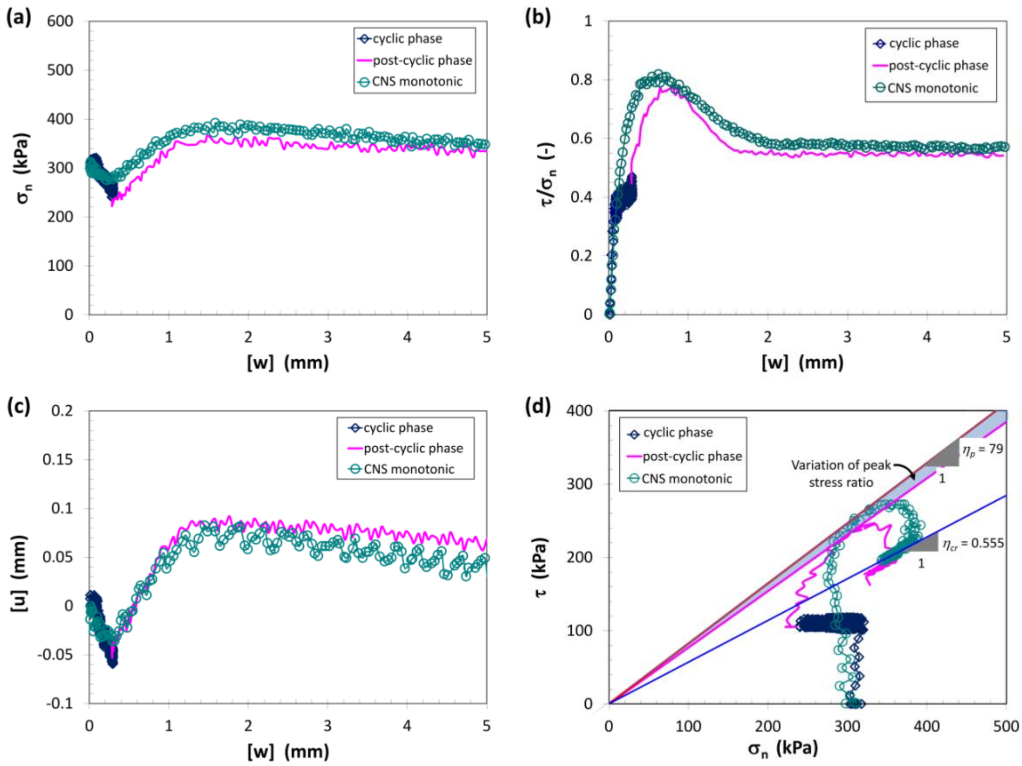

Fig. 6 Post-cyclic phase for $\sigma_{n\ cm0}$ = 310 kPa on dense Fontainebleau sand with k = 1000 kPa/mm, η_{cm0} = 0.35 (105 < τ < 115 kPa) in comparison with CNS monotonic test.

In this case, a large number of cycles were required to reach the critical state.

Considering the post-cyclic phase, this phase started at $\sigma_{n\ cm} \approx$ 240-250 kPa (as shown in Fig. 6a, 6d), the peak shear stress ratio at post-cyclic phase ($\eta_{p\ \ post} \approx 0.77$) was slightly lower than that of monotonic CNS test. Although the cyclic loading induced more densification of interface, there was not a large difference in dilation between the post-cyclic phase and CNS monotonic test (see $[u] - [w]$ diagram in Fig. 6c). This phenomenon might be attributed to the crushing and wear of the grains resulting from an increase of inter-granular particles in the localized shear zone between sand and rough plate during cyclic phase ([1], [14]).

Tabucanon et al. [14] reported that there was a lower stress recovery during post-cyclic response and the loss of strength increased with increasing the number of cycles due to the smaller volume change accompanying shear loading of the interface. Fakharian and Evgin [5] also explained that the shear stress which mobilized to the peak value decreased to a residual stress with a sufficient increase in sliding displacement or slip at interface.

4. CONCLUSION

Based on the post-cyclic phase of interface direct shear tests under CNS condition, the main results presented in this paper can be summarized as follows:

- At post-cyclic phase, the interface exhibited dilative behavior as a result of gradual densification.
- The peak stress ratio at post-cyclic phase could be attributed to the evolution of grain breakage within interface shear zone. With low value of initial normal stress, an increase of crushing particles during cyclic loading increased the normalized the surface roughness (R_n) of the soil-structure interface then led to the higher coefficient of interface friction.
- In case of high value of initial normal stress, a lower stress recovery during post-cyclic response can be found and the loss of strength increased with increasing the number of cycles due to the smaller volume change accompanying shear loading of the interface.

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