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Three-dimensional monotonic and cyclic behavior of a gravel-steel interface from large-scale simple-shear tests

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Three-dimensional monotonic and cyclic behavior of a gravel-steel interface from large-scale simple-shear tests

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ABSTRACT

Gravel-structure interfaces are involved in many large-scale geotechnical applications and play a vital role in the performance of soil-structure interaction systems. A large-scale simple-shear device was professionally developed and used to investigate 3-D monotonic and cyclic shear behavior of a gravel-steel interface in two-way beeline, cross and circular shear paths. The soil deformation was measured and was used to determine the interface thickness. The deforming and sliding displacements of the interface were quantified and separated from the total tangential displacement. Results show that the interface thickness is about \((6–7) \cdot D_{50}\), independent of the normal stress and shear path.

Under 3-D loading conditions, the shear stress vs. tangential displacement hysteretic response exhibits notably 3-D features. The interface becomes stiffer because of the hardening behavior of the gravel during cyclic shearing. The total normal displacement can be divided into irreversible and reversible components, which present different responses. The peak shear strength is mobilized when the interface dilatancy rate reaches the maximum at the sliding displacement of about \(0.5 \cdot D_{50}\), and behaves in an anisotropic manner caused by the shear orientation effect. The normal stress and shear paths have significant influences on the 3-D interface behavior.

Keywords: gravel-steel interface, 3-D behavior, simple-shear test, shear path, constant normal load
Gravel-structure interfaces widely occur in many geotechnical practices, such as concrete-faced rock-fill dams, high-speed railways, and pile foundations. The shear response of soil-structure interfaces has a notable influence on the performance of soil-structure interaction systems. Therefore, a good understanding of the monotonic and cyclic shear behavior of interfaces is vital for the design and analysis of soil-structure interaction systems that may be subjected to 3-D static or dynamic loads.

Many researchers have investigated the load-deformation characteristics of soil-structure interfaces via laboratory tests. Potyondy (1961), Brandt (1985), Yin et al. (1995), Hu and Pu (2004), Taha and Fall (2013), Chen et al. (2015), and Afzali-Nejad et al. (2017) performed 2-D monotonic interface tests between soil and structure/geotextile, which are the most straightforward loading types. Numerous investigators (Desai et al. 1985; Al-Douri and Poulos 1992; DeJong et al. 2006 and 2009; Zhang and Zhang 2006; Miller and Hamid 2007; Mortara et al. 2007; Khoury and Miller 2012; Di Donna et al. 2016; Wang et al. 2016; Boukpeti and White 2017; Farhadi and Lashkari 2017) performed series of 2-D direct-shear tests of interfaces to obtain a sound understanding of the 2-D cyclic behavior of soil-structure interfaces, especially the behavior of sand-structure interfaces.

Simple-shear type devices are able to separate the deforming and sliding displacements from the total tangential displacement of an interface. Therefore, several
simple-shear tests on sand-steel interfaces were also conducted to investigate the monotonic and cyclic behavior (Uesugi and Kishida 1986a, 1986b and 1989; Kishida and Uesugi 1987; Fakharian 1996; Fakharian and Evgin 1997; Li 2001; Oumarou and Evgin 2005). However, most of the preceding studies, except those of Fakharian (1996) and Fakharian and Evgin (1997), have been limited to the 2-D behavior of sand-structure interfaces because of the limitations in the capabilities of test devices. Few data on the 3-D cyclic behavior of gravel-structure interfaces have been reported in the literature.

The present study focuses on the monotonic and 3-D cyclic behavior of a gravel-structure interface by performing a series of simple-shear tests using the modified 3DMAS device. This paper introduces the test device, describes the test configurations, and presents the results of the monotonic and 3-D cyclic tests of the interface. The soil deformation was measured, and the deforming and sliding displacements of the interface were quantified and separated from the total tangential displacement. The newly observed simple-shear behavior of the gravel-steel interface is described in detail and the influences of the normal stress magnitude and shear path are also discussed.

TEST CONFIGURATIONS

3-D simple-shear device

Figure 1 shows the modified 3DMAS device professionally developed for the 3-D
monotonic and cyclic tests of soil-structure interfaces. The 3DMAS device has a
dimension of 2000 mm long, 2000 mm wide, and 3500 mm high (respectively in the x,
y, and z directions of Fig. 1). The net internal space for the specimen container has a
1000 mm length, 1000 mm width, and 800 mm height.

Three hydraulic servo actuators controlled by an operational computer with the
corresponding software supply the loads in the three orthogonal directions (i.e., x, y,
and z directions in Fig. 1). The hydraulic servo system can independently generate the
required loads and displacements, such as normal stress ($\sigma_n$) and normal displacement
($v$) in the vertical direction, shear stresses ($\tau_x$, $\tau_y$) and total tangential displacements ($u_x$,
$u_y$) in the tangential directions with high accuracy (Zhang and Feng et al. 2018). The
vertical servo actuator is capable of applying different types of normal boundary
conditions on the interface (i.e., constant normal load, constant volume, and constant
normal stiffness conditions). A maximum of 800 kN normal load (about 11.3 MPa on
the specimen with 300 mm diameter) can be applied onto the interface. The two
horizontal servo actuators can apply all kinds of shear paths, such as pre-load
monotonic, two-way beeline and cross cyclic, one-way and two-way circular and
elliptical cyclic, and any other customized shear paths. The shear path could be under
displacement control, load control, or a combination of both in the x and y directions.
The maximum total tangential displacement is ±150 mm and the maximum shear load
is 400 kN in the two tangential directions.
Figure 2 shows the simple-shear type soil container used for the interface tests. The ring group was composed of 27 round rings with 300 mm internal diameter and 3 or 4 mm thickness. A number of holes were evenly distributed along the 3 mm thick rings to hold the low-frictional steel bearings. Rings with 3 and 4 mm thicknesses were alternately stacked with a 0.5 mm gap. The low-frictional steel bearings ensure little lateral resistance between the rings. The ring group was placed on the lower box, and they were supported by a spring group, as shown in Fig. 2. Therefore, the whole container (including the ring group and the lower box) is capable of freely moving up and down to accommodate the volumetric deformation of the interface during shearing. In addition, a 2 mm thick latex film was attached to the internal wall of the container to prevent the soil particles from sticking into the gap between the rings when the shear deformation of the soil takes place.

The relative displacements of the rings were recorded by two cameras installed at the two orthogonal tangential directions, as shown in Fig. 1. The two cameras could synchronously photograph the positions of the marked points on each ring at desired frequency ranging from 0.01 Hz to 1 Hz. The images were then processed to obtain the lateral displacement of the marked points and the tangential displacements of the rings. Details of the image processing technique were described by Zhang and Liang et al. (2006). The measured total tangential displacements using the image processing technique are observed to be consistent with the results using the LVDTs (displacement
transducers) in all tests.

Three tangential displacements of the interface (illustrated in Fig. 2) that will be reiterated in the subsequent sections need to be defined. The total tangential displacement \( u \) is the tangential displacement of the structural plate relative to the lower box of the container. The deforming displacement \( u_d \) is the tangential displacement caused by the shear deformation of the soil, and can be obtained from the displacement of the top ring adjacent to the structural plate relative to the lower box. The sliding displacement \( u_s \) is the tangential displacement of the top ring slipping relatively to the structural plate, and can be obtained from the equation \( u_s = u - u_d \).

**Test materials**

The tested interface consists of a 3-D rough steel plate and dry limestone gravel (shown in Fig. 3(a)). The grain size of the gravel ranged from 5 mm to 16 mm, with mean grain size \( D_{50} \) of 9.0 mm \( (G_s=2.65) \), and has a poorly-graded distribution (Fig. 3(b), \( C_u = 1.8, C_c = 0.9 \)). The gravel was tamped into the container in five layers to obtain 175 mm high specimen with dry density of 1.78 g/cm\(^3\). This specimen height, consisting of 108 mm ring group and 67 mm lower box, ensures that the shear deformation of the soil occurs in the ring group and that the normal displacement of the interface takes place freely during shearing. Based on conventional triaxial compression tests, the gravel presents similar behavior to other gravels (Zhang and Zhang 2006), and its friction angle can be expressed using the equation

\[
\varphi = \varphi_0 - \Delta \varphi \log(\sigma_3/P_a),
\]

where \( \varphi_0 = 50^\circ \) and \( \Delta \varphi = 8^\circ \) are strength parameters.
A 3-D artificial isotropic steel plate was used as the structural plate for the interface tests. The steel surface was notched uniformly with repeating patterns of standard quadrangular frustum pyramids that were 5 mm wide at the base, 1 mm wide at the top, and 2 mm high. The surface roughness (i.e., $R=2\text{mm}$) was maintained constant throughout shearing. Therefore, the normalized roughness of the tested interface is $R_n=0.222$ (Uesugi and Kishida 1986b). The area of the steel plate was 800 mm × 800 mm, which is suitably larger than the gravel specimen to ensure that the area of the contact surface between the steel plate and the gravel remains constant during sliding. Therefore, the normal displacement can represent the volumetric deformation of the interface. The interface dilatancy and compression are respectively denoted as the negative and positive normal displacements in all the figures of the paper.

**Shear paths**

Linear and circular displacement-controlled shear paths were used in the present study. The interface of a linear shear path was sheared along a linear route in three different ways, namely, monotonic, two-way beeline cyclic, and two-way cross cyclic, which are respectively illustrated as the routes of 1, 1-2-3-4-1-..., and 1-2-3-4-5-6-7-8-1-... in Fig. 4(a). On the other hand, the interface of a circular shear path was first sheared along the route of 0 (shown in Fig. 4(b)) to the required displacement amplitude ($u_r$) and then was repeatedly sheared along a circle in the $x$-$y$ tangential plane. It included clockwise circular shear path 1-2-3-4-1-... and clockwise-counterclockwise circular
shear path 1-2-3-4-5-6-7-8-1-..., respectively referred to as the one-way and two-way circular cyclic shear paths. Either 1-2-3-4 or 5-6-7-8 was regarded as one single shear cycle to facilitate the interpretation of test results. The amplitudes of the total tangential displacement \( u_t \) were 40 mm for monotonic tests and ±20 mm for cyclic tests. The normal stress was kept constant for all the interface tests.

**MONOTONIC BEHAVIOR**

**Total response**

The total response of the interface, referring to the interface behavior concerning \( u \), has been the focus of many previous studies (Potyondy 1961, Desai et al. 1985, Hu and Pu 2004). Figure 5 provides the test results of the gravel-steel interface subjected to monotonic loads at different normal stress \( \sigma_n = 200, 400, 700, \text{ and } 1000 \text{ kPa} \). Normal displacement is obviously observed in Fig. 5(c), revealing that the volumetric deformation of the interface takes place due to shearing. Under relatively low \( \sigma_n \), the interface is compressed first and then dilated obviously. Larger \( \sigma_n \) leads to more compression and less dilatancy. It is indicated that the volumetric deformation of the gravel-steel interface is induced by shear application and is influenced by normal stress. The compression-shear coupling effect is presented in the volumetric deformation of the interface.

Figure 5(a) shows that the shear stress \( \tau \) increases gradually with \( u \), reaches the peak shear strength \( \tau_f \), and then remains almost constant, demonstrating that the \( \tau \) vs.
u response exhibits no obvious strain-softening behavior. This behavior is different from the typical dense sand-rough steel interface, which exhibits apparent strain-softening response (Fakharian and Evgin 1997, Uesugi and Kishida 1986a and 1986b). Figure 5(b) gives the curves of the stress ratio \( \eta (= \tau / \sigma_n) \) vs. \( u \) and provides a better overview of the unobvious strain-softening behavior, especially when the normal stress is relatively high (e.g., \( \sigma_n = 1000 \text{ kPa} \)). The linearity relationship between \( u/\tau \) and \( u \) (depicted in Fig. 5(d)) reveals that the \( \tau \) vs. \( u \) response behaves in a hyperbola manner. Figures 5(a) and 5(b) reveal that the initial slope of the \( \tau \) vs. \( u \) curves (i.e., the initial shear modulus of the interface) increases with increasing \( \sigma_n \) while the initial slope of the \( \eta \) vs. \( u \) curves decreases, indicating that the normal stress greatly influences the shear modulus of the interface.

Figure 5(e) provides the normal displacement slope with total tangential displacement (i.e., \( dv/du \) vs. \( u \) plot). The peak shear strength is mobilized when the interface dilatancy rate reaches the maximum (i.e., the peak magnitude of \( dv/du \) during dilatancy). From the peak shear strength line given in Fig. 5(f), the mobilized \( \tau_i \) increases almost linearly proportional to the applied \( \sigma_n \), demonstrating that the gravel-steel interface behaves well in accordance with the Mohr-Coulomb failure criteria for cohesionless soils that follows the equation \( \tau_i = \sigma_n \tan(\phi_p) \), where \( \phi_p = 32.5^\circ \) is the peak monotonic friction angle of the interface. The \( \tau \) vs. \( \sigma_n \) curves before the peak shear strength (i.e., the yield surface) exhibit nonlinearity characteristics because smaller \( \sigma_n \) leads to earlier mobilization of \( \tau_i \).
Soil deformation

The soil deformation ($\delta$) and the corresponding $u_d$ were obtained from the lateral displacements of the rings by processing the images captured by the two real-time cameras. Figure 6 provides the profile of soil deformation ($\delta$) along the specimen depth ($h$) at different $\sigma_n$ and $u$. The $\delta$ increases with $u$ but degrades with $h$, and is considerably small when the soil layer is deeper than 62 mm (i.e., $h>62\text{mm}$); thus, it can be neglected compared with that of the top soil layer near the steel plate. The soil deformation caused by shearing appears limited to a localized shear zone, beyond which the soil is barely affected by the shear application. This thickness of the localized shear zone is defined as the interface thickness. For the tests in the present study, the interface thickness is about 58 mm at $\sigma_n = 400$ kPa and about 62 mm at $\sigma_n = 1000$ kPa. The normal stress exerts negligible effects on the interface thickness and on the profile of soil deformation. The results of the monotonic tests reveal that the interface thickness is approximately $(6-7)\cdot D_{50}$ of the gravel. Li (2001) showed that the $(6-7)\cdot D_{50}$ interface thickness also applies to the interface between sand-blasted steel plate and medium-crushed quartz sand at $\sigma_n = 100$ kPa.

The soil deformation directly leads to a component of the tangential displacement of the interface (i.e., deforming displacement, $u_d$). From the $u_d$ vs. $u$ curves in Fig. 7(a), the $u_d$ is further confirmed to increase with $u$ and $\sigma_n$ because of the stronger friction constraint of the steel plate. The gravel still deforms after the peak shear strength of the interface is mobilized, as shown in Fig. 7(c). This observation is different from the
typical sand-steel interface, of which the sand deformation ceases after the peak shear
strength is mobilized (e.g., Fakharian and Evgin 1997). The $\tau$ vs. $u_d$ and $v$ vs. $u_d$ curves
appear similar to the total response of the interface, except for the larger slope of the $\tau$
vs. $u_d$ and $v$ vs. $u_d$ curves in Figs. 7(c) and 7(e).

*Sliding displacement*

The sliding displacement ($u_s$) occurs at the contact surface between the gravel and the
structural plate. The gravel-steel interface will fail at the contact surface if the sliding
displacement is sufficiently large. Figure 7(b) demonstrates that the sliding
displacement occurs as long as the shear application is applied. Apparent turning points
are observed in the $u_s$ vs. $u$ plots marked as A and B in Fig. 7(b). The $u_s$ increases much
faster when $u$ exceeds the turning points, and the peak shear strength of the interface is
mobilized at the turning points. These turning points indicate the onset of the interface
failure at the contact surface. Before the peak shear strength is mobilized, the gravel
particles adjacent to the structural plate may climb and be rearranged caused by the
friction constraint of the structural plate. Therefore, the gravel deforms greatly and the
deforming displacement accounts for the majority of the total tangential displacement.

When the gravel particles have just climbed and began to mainly roll over against each
other, the peak shear strength is mobilized. The sliding displacement thus increases
faster and the deforming displacement decreases.

The $\tau$ vs. $u_s$ and $v$ vs. $u_s$ curves of the interface appear similar to the total response,
although the $\tau$ vs. $u_s$ and $v$ vs. $u_s$ curves show a larger slope in Figs. 7(d) and 7(f). The sliding displacement at which the peak shear strength is mobilized does not change too much under different $\sigma_n$, as shown in Fig. 7(d). The sliding displacement corresponding to the peak shear strength ranges from 4.2 mm to 6.2 mm ($\approx 0.5 \cdot D_{50}$), which is approximately the distance that a gravel particle climbs to override another. This finding is consistent with the preceding observation and interpretation in Fig. 7(b) and demonstrates that the mobilization of the peak shear strength is governed by the sliding displacement of the interface. It is also evident from Fig. 7(f) that the interface dilatancy rate reaches the maximum when $u_d$ is about $0.5 \cdot D_{50}$. This reveals that the interface is dilated slower after the gravel particle climbs to override each other and further conforms the close relation between the peak shear strength and the dilatancy of the interface.

**CYCLIC BEHAVIOR**

This section presents the 2-D and 3-D cyclic behavior of the gravel-steel interface from series of linear and circular cyclic shear tests. The symbol $N$ is introduced for interpretation of cyclic test results, and denotes the number of shear cycles that follow the cyclic shear paths in Fig. 4.

**Soil deformation**

Figure 8 provides the profiles of the soil deformation in the $x$ direction subjected to cyclic shearing at $\sigma_n=400$ kPa. The gravel presents large deformation when $u$ is
relatively large and the soil deformation is still localized in the shear zone and degrades with the specimen depth. The measured interface thickness from the results of Fig. 8 is about 60 mm during cyclic shearing in both linear and circular shear paths and is the same as that during monotonic shearing. The shear path impacts little effect on the interface thickness. Comparing the soil deformation at different $N$ values, the amplitude of soil deformation inside or outside the shear zone decreases with increasing $N$, indicating that the gravel becomes hardened gradually. The pattern of soil deformation distribution appears similar at different $N$ values. Figure 9 shows the soil deformations in the $x$-$y$ tangential plane at different specimen depths for the two-way circular shear test. The curves appear to be circular at different specimen depths, particularly at the first few shear cycles.

Figure 10 plots $u_d$ vs. $u$ curves in the $x$ or $y$ direction for the cyclic shear tests. The growth of the deforming displacement is slower than its recovery, and the growth rate decreases with $u$. Thus, large $u_d$ vs. $u$ cycles are observed, particularly at the first few shear cycles. The amplitude of $u_d$ and the slope of $u_d$ vs. $u$ curves decrease as the cyclic shear application continues in all shear paths. The $u_d$ becomes considerably smaller compared with $u$ when the interface is cyclically sheared to some extent (e.g., $N \geq 15$). As shown in Figs. 8(a), 8(b) and 10(a), the soil deformation and the deforming displacement tend to migrate toward the initial shear direction with increasing $N$ in the two-way beeline cyclic shear path, indicating that the soil deformation and the deforming displacement present anisotropic response during cyclic shearing. The
anisotropic response is also evident for the deforming displacement in the $y$ direction
of the circular shear tests (Figs. 10(d) and 10(f)), while not obvious for the two-way
cross shear test (Figs. 10(b)). This anisotropic response may be attributed to the shear
orientation effect, referring to the orientation rearrangement of gravel particles and the
plastic work induced by the initial shear application, which has been discovered by
previous research (Zhang and Zhang 2006). The $u_d$ vs. $u$ curves of the two circular
shear tests are different from those of the linear shear tests after comparing Figs. 10(c)
and 10(e) to Fig. 10(a) and 10(b). The observations above indicate that the shear path
governs the soil deformation and the $u_d$ vs. $u$ response of the interface.

Shear stress vs. displacement

Figure 11 plots the hysteretic curves of $\tau$ vs. $u$, $\tau$ vs. $u_d$, and $\tau$ vs. $u_s$ of the interface
in $x$ or $y$ direction subjected to different shear paths at selected $N$ values.

For the two-way beeline and cross cyclic shear paths, the $\tau$ vs. $u$ curves during
loading stage are hyperbolic, as shown in Figs. 11(a1) and 11(b1). The hyperbolic $\tau$ vs.
$u$ curves are similar to those of the monotonic tests. As shear cycle increases, the shear
modulus (i.e., the slope of $\tau$ vs. $u$ curves) during loading is observed to increase, and
the slope of $\tau$ vs. $u_d$ curves becomes larger, whereas the slope of $\tau$ vs. $u_s$ curves
becomes smaller, as illustrated in Figs. 10(a2), 10(a3), 10(b2) and 10(b3), indicating
that the interface becomes stiffer because of the hardening behavior of the gravel
caused by cyclic shearing.
The two-way cross shear path can be regarded as the extended pattern of the
two-way beeline shear path in 3-D interface space. Therefore, the interface presents
similar shear stress-displacement hysteretic response in these two linear shear paths,
except that the shear stress begins from zero at the start of each shear cycle in the cross
shear path. For the same shear cycle, the shear modulus at the start (e.g., \(N = 0–0.25\)) is
larger than that during other loading stages (e.g., \(N = 0.25–0.5\) and \(0.75–1\)). It is also
larger than that of the two-way beeline shear test. These findings demonstrates that the
shear history of the orthogonal direction affects the shear behavior of the interface. The
shear history is disturbed and exerts little influence on the subsequent interface
behavior after the shear direction is reversed and the interface is reloaded.

The shear stress vs. tangential displacement hysteretic curves appear to be
elliptical for the circular shear test results, and are distinctly different from those in the
linear shear paths. This indicates the 3-D features of the interface behavior and the
significant effect of the 3-D shear path on the shear stress-displacement hysteretic
response. A roundish transition is observed when the tangential displacement reaches
the amplitude, as illustrated in Figs. 11(c), 11(d) and 11(e). The shear stress almost
drops to zero when the sliding displacement magnitude reaches the maximum because
the interface is not being sheared in this tangential direction, but rather being sheared
in the orthogonal direction.

For the two-way circular shear tests, the \(\tau_y\) vs. \(u_y\) hysteretic curves present similar
response at all $N$ values, but the $\tau_x$ vs. $u_x$ hysteretic curves are different at the neighboring shear cycles if the $N = 1$ plots are compared with the $N = 30$ plots in Fig. 11(c). For the one-way circular shear tests, the $\tau_x$ vs. $u_x$ hysteretic curves exhibit similar response at all $N$ values. Meanwhile, the $\tau_y$ vs. $u_y$ hysteretic curves for one-way circular shear test are similar to those for two-way circular shear test, and are thus omitted in this paper.

From Fig. 11, the peak shear stress degrades with increasing $N$ for all cyclic shear tests. Thus, the shear stress vs. tangential displacement hysteretic curves (including $\tau$ vs. $u$, $\tau$ vs. $u_d$, and $\tau$ vs. $u_s$ curves) at different $N$ values do not coincide because of the migration and decreasing amplitudes of $\tau$, $u_d$, and $u_s$.

### Normal displacement

The total normal displacement ($v$) of the interface can be divided into irreversible and reversible components. The irreversible normal displacement ($v_{ir}$) is defined as the peak magnitudes of the total normal displacement at each shear cycle, and the reversible normal displacement ($v_{re}$) can be obtained from the equation $v_{re}=v-v_{ir}$. The $v_{ir}$ shows the general trend of the volumetric deformation caused by cyclic shearing and the $v_{re}$ shows the “recoverable” volumetric deformation that could grow and recover during each shear cycle.

Figure 12 provides the curves of $v$ vs. $u$, $v_{re}$ vs. $u$, $v_{re}$ vs. $u_d$ and $v_{re}$ vs. $\tau$ at selected $N$ values. The normal displacements (i.e., $v$, $v_{ir}$ and $v_{re}$) are obviously observed, and
they are primarily caused by the shear application since the normal stress is kept constant throughout tests. The interface is generally compressed due to cyclic shearing and the $v_{ir}$ accumulates and increases with $N$. The $v_{ir}$ at $N = 30$ for the two-way beeline, cross, circular and one-way circular cyclic shear tests is approximately 12.67 mm, 13.15 mm, 16.72 mm and 16.02 mm respectively. This demonstrates that the shear path greatly impacts the irreversible normal displacement, which is primarily attributed to crushing of the gravel particles caused by cyclic shearing and refilling of the large voids by crushed smaller gravel particles. Post-test gravel particle distribution analysis (illustrated in Fig. 3) confirmed that the crushing during shearing produces smaller gravel particles and significantly changes the grain size distribution of the gravel. On the other hand, the tendency of the rearrangement of the gravel particles to reduce the void ratio secondarily leads to the irreversible volumetric deformation.

The normal displacements of the interface presents similar responses in the two-way beeline and cross cyclic shear paths which are both linear, as illustrated in Figs. 12(a) and 12(b) respectively. The interface presents volumetric dilatancy during loading and compression during unloading, and reaches peak $v_{re}$ at limiting $u$. The dilatancy rates of $v$ and $v_{re}$ decrease with $u$, and are much smaller than the compression rate. However, the dilatancy rates of $v_{re}$ increase with $\tau$, and are much bigger than the compression rate. Interestingly, the dilatancy rate of $v_{re}$ with $u_d$ is similar to the corresponding compression rate, particularly when the interface is cyclically sheared to some extent, indicating that the reversible normal displacement behaves in a similarly
elastic manner with the deforming displacement. The interface shows slight compression at the start of each shear cycle in the two-way cross shear path, because the shear direction changes to the orthogonal and the interface is partially unloaded.

The peak $v_{re}$ is similar at the first shear cycle and decreases with $N$ in all two-way shear paths. It decreases from about 2.5 mm at $N = 1$ to about 1 mm at $N = 30$, corresponding to 19.0% and 7.6% of $v$ for two-way cross shear test and 14.9% and 6.0% for two-way circular shear test, respectively. This decrease is mainly attributed to the crushing and reducing size of gravel particles adjacent to the structural plate. However, the interface exhibits insignificant reversible normal displacement in one-way circular shear path except at the first shear cycle, because the interface is always monotonically sheared along a circular route without any reversal of the shear direction.

The curves of $v$ vs. $u$, $v_{re}$ vs. $u$, $v_{re}$ vs. $u_d$ and $v_{re}$ vs. $\tau$ are different in the $x$ and $y$ directions for the circular shear tests because of the different starting points. They are also different from those in the linear shear paths, indicating that the shear path greatly influences the volumetric behavior of the interface, including the normal displacement amplitude, and the normal displacement vs. tangential displacement and shear stress curves. The $v$ vs. $u$ plots are not closed within a single shear cycle and do not coincide at different $N$ values because of the cumulative $v_{ir}$, especially at the first few shear cycles. The $v_{re}$ vs. $u$, $v_{re}$ vs. $u_d$, $v_{re}$ vs. $\tau$ plots appear closed within a single shear cycle with the absence of $v_{ir}$ except for the first shear cycle, but they still do not coincide at
different $N$ for the reduction of the peak of $v_{ye}$.

**Shear strength**

Under 3-D loading conditions, the interface may be simultaneously sheared in $x$ and $y$ directions, for example, in the circular shear paths. Thus, the peak shear stresses in $x$ or $y$ direction are no longer appropriately regarded as the shear strength of the interface.

The resultant shear stress ($\tau$) is introduced and defined as the root mean square of the shear stresses in $x$ and $y$ directions (i.e., $\tau = \sqrt{\tau_x^2 + \tau_y^2}$). The peak resultant shear stress at each shear cycle is considered as the cyclic shear strength of the interface (denoted as $\tau_{fd}$). In linear shear paths, the interface is only sheared in $x$ or $y$ direction within a single shear cycle. Therefore, the peak shear stress in $x$ or $y$ direction is the peak resultant shear stress and thus is the shear strength of the interface. Meanwhile, the shear strength in a single tangential direction (i.e., $\tau_{fx+}$, $\tau_{fx-}$, $\tau_{fy+}$, and $\tau_{fy-}$, respectively in the positive $x$, negative $x$, positive $y$ and negative $y$ directions, as depicted in Fig. 13) is also introduced and defined as the maximum resultant shear stress in this tangential direction at each shear cycle.

Figure 14 provides the cyclic histories of the $\tau_{fd}$ and the corresponding $\tau_{fx+}$, $\tau_{fx-}$, $\tau_{fy+}$ and $\tau_{fy-}$ with $N$ in different shear paths. As observed in Figs. 11, 13 and 14, the shear strength of the interface reaches the peak at $N = 1$ in all cyclic shear paths, degrades with increasing $N$ and then becomes gradually stabilized. Figure 15 gives the cyclic shear strength at $N=1$ and $N=30$ for two-way beeline cyclic test under different
normal stress ($\sigma_n=200, 400, 700$ and $1000$ kPa). The $\tau_{fd}$ presents linearly proportional
response to the applied $\sigma_n$, indicating that the $\tau_{fd}$ also behaves well in accordance with
the Mohr-Coulomb failure criteria that follow the equation $\tau_{fd}=\sigma_n \cdot \tan(\phi_{id})$, where $\phi_{id}$ is
the cyclic friction angle of the interface. It is also evident from Fig. 15 that the cyclic
shear strength and the corresponding cyclic friction angle degrade with increasing $N$.

As shown in Fig. 14, the shear strength in the $x$ or $y$ direction is smaller than the
cyclic shear strength for the circular shear tests, proving the necessity of introduction
of the resultant shear stress. The shear strength and the corresponding friction angle in
the positive tangential direction (i.e., the initial shear direction) are generally smaller
than those in the negative direction (i.e., the reload direction), which is also evident in
Fig. 13. For two-way beeline cyclic test results shown in Fig. 15, the cyclic friction
angle at $N=1$ in positive direction ($\phi_{id} = 32.2^\circ$) is smaller than that in negative direction
($\phi_{id} = 33.6^\circ$), and the cyclic friction angle at $N=30$ in positive direction ($\phi_{id} = 28.7^\circ$) is
also smaller than that in negative direction ($\phi_{id} = 29.9^\circ$). These findings demonstrate
that the shear strength of the gravel-steel interface is anisotropic induced by the shear
orientation effect and is influenced by the shear path and shear history. When the
interface is sheared in the initial shear direction at the first quarter shear cycle, the
gravel particles in the shear zone are rearranged toward this direction, and orientation
fabric thus occurs. On the other hand, when the interface is sheared in the reload
direction at the subsequent half shear cycle, a higher shear stress is required to weaken
or destroy the orientation fabric.
Figure 14 also demonstrates that the cyclic shear strength and the shear strength in a single tangential direction for the circular shear tests are slightly smaller than those for the linear shear tests, because the simultaneous shearing in the two orthogonal directions might further loosen the interlocking of the gravel particles. This demonstrates that the shear path plays a significant role in the cyclic shear strength of the interface.

CONCLUSIONS

A new large-scale apparatus was modified to investigate 3-D monotonic and cyclic simple-shear behavior of the interface between uniform gravel and a 3-D isotropic steel plate. The shear deformation of the gravel was measured and the deforming and sliding displacements of the interface were differentiated. The 3-D monotonic and cyclic behaviors of the interface are summarized as follows:

(1) Sliding displacement begins at the onset of shearing. Soil deformation degrades along the specimen depth and is limited to a localized shear zone. The soil deformation and the corresponding deforming displacement presents an anisotropy response due to the shear orientation effect. The interface thickness is \((6–7)·D_{50}\), independent of shear path, normal stress and the number of shear cycles.

(2) The hysteretic curves of shear stress vs. total tangential displacement, deforming displacement, and sliding displacement exhibit apparently 3-D features and appear to be elliptical in circular shear paths and hyperbolic in linear shear paths. The interface becomes stiffer because of the hardening response of the gravel caused by
cyclic shearing.

(3) The normal displacement can be divided into irreversible component and reversible component, which is insignificant in one-way circular shear path. The irreversible normal displacement accumulates primarily because of particle crushing, whereas the peak of the reversible normal displacement reduces with increasing shear cycles.

(4) The peak shear strength is mobilized when the interface dilatancy rate reaches the maximum at the sliding displacement of about $0.5 \cdot D_{50}$, irrespective of normal stress. It presents anisotropic characteristics owing to the shear orientation effect, behaves in accordance with Mohr-Coulomb failure criteria, and degrades with shear cycles.

(5) The normal stress and shear path play a vital role in the interface behavior. Large normal stress leads to the increase of soil deformation, deforming displacement, initial shear modulus, irreversible normal displacement, and shear strength. The shear path mainly affects the normal displacement, shear strength, as well as the response of shear stress vs. displacement, normal vs. tangential displacement and normal displacement vs. shear stress.

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