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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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2 **gravel-steel interface from large-scale simple-shear tests**

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

39 **Da-kuo Feng, Jian-min Zhang, and Li-jun Deng**

40 **ABSTRACT**

41 Gravel-structure interfaces are involved in many large-scale geotechnical applications
42 and play a vital role in the performance of soil-structure interaction systems. A large-scale
43 simple-shear device was professionally developed and used to investigate 3-D monotonic
44 and cyclic shear behavior of a gravel-steel interface in two-way beeline, cross and
45 circular shear paths. The soil deformation was measured and was used to determine the
46 interface thickness. The deforming and sliding displacements of the interface were
47 quantified and separated from the total tangential displacement. Results show that the
48 interface thickness is about $(6-7) \cdot D_{50}$, independent of the normal stress and shear path.
49 Under 3-D loading conditions, the shear stress vs. tangential displacement hysteretic
50 response exhibits notably 3-D features. The interface becomes stiffer because of the
51 hardening behavior of the gravel during cyclic shearing. The total normal displacement
52 can be divided into irreversible and reversible components, which present different
53 responses. The peak shear strength is mobilized when the interface dilatancy rate reaches
54 the maximum at the sliding displacement of about $0.5 \cdot D_{50}$, and behaves in an anisotropic
55 manner caused by the shear orientation effect. The normal stress and shear paths have
56 significant influences on the 3-D interface behavior.

57 **Keywords:** gravel-steel interface, 3-D behavior, simple-shear test, shear path, constant
58 normal load

59 INTRODUCTION

60 Gravel-structure interfaces widely occur in many geotechnical practices, such as
61 concrete-faced rock-fill dams, high-speed railways, and pile foundations. The shear
62 response of soil-structure interfaces has a notable influence on the performance of
63 soil-structure interaction systems. Therefore, a good understanding of the monotonic
64 and cyclic shear behavior of interfaces is vital for the design and analysis of
65 soil-structure interaction systems that may be subjected to 3-D static or dynamic loads.

66 Many researchers have investigated the load-deformation characteristics of
67 soil-structure interfaces via laboratory tests. Potyondy (1961), Brandt (1985), Yin et al.
68 (1995), Hu and Pu (2004), Taha and Fall (2013), Chen et al. (2015), and Afzali-Nejad
69 et al. (2017) performed 2-D monotonic interface tests between soil and
70 structure/geotextile, which are the most straightforward loading types. Numerous
71 investigators (Desai et al. 1985; Al-Douri and Poulos 1992; DeJong et al. 2006 and
72 2009; Zhang and Zhang 2006; Miller and Hamid 2007; Mortara et al. 2007; Khoury
73 and Miller 2012; Di Donna et al. 2016; Wang et al. 2016; Boukpeti and White 2017;
74 Farhadi and Lashkari 2017) performed series of 2-D direct-shear tests of interfaces to
75 obtain a sound understanding of the 2-D cyclic behavior of soil-structure interfaces,
76 especially the behavior of sand-structure interfaces.

77 Simple-shear type devices are able to separate the deforming and sliding
78 displacements from the total tangential displacement of an interface. Therefore, several

79 simple-shear tests on sand-steel interfaces were also conducted to investigate the
80 monotonic and cyclic behavior (Uesugi and Kishida 1986a, 1986b and 1989; Kishida
81 and Uesugi 1987; Fakharian 1996; Fakharian and Evgin 1997; Li 2001; Oumarou and
82 Evgin 2005). However, most of the preceding studies, except those of Fakharian (1996)
83 and Fakharian and Evgin (1997), have been limited to the 2-D behavior of
84 sand-structure interfaces because of the limitations in the capabilities of test devices.
85 Few data on the 3-D cyclic behavior of gravel-structure interfaces have been reported
86 in the literature.

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

96 **TEST CONFIGURATIONS**

97 ***3-D simple-shear device***

98 Figure 1 shows the modified 3DMAS device professionally developed for the 3-D

99 monotonic and cyclic tests of soil-structure interfaces. The 3DMAS device has a
100 dimension of 2000 mm long, 2000 mm wide, and 3500 mm high (respectively in the x ,
101 y , and z directions of Fig. 1). The net internal space for the specimen container has a
102 1000 mm length, 1000 mm width, and 800 mm height.

103 Three hydraulic servo actuators controlled by an operational computer with the
104 corresponding software supply the loads in the three orthogonal directions (i.e., x , y ,
105 and z directions in Fig. 1). The hydraulic servo system can independently generate the
106 required loads and displacements, such as normal stress (σ_n) and normal displacement
107 (v) in the vertical direction, shear stresses (τ_x , τ_y) and total tangential displacements (u_x ,
108 u_y) in the tangential directions with high accuracy (Zhang and Feng et al. 2018). The
109 vertical servo actuator is capable of applying different types of normal boundary
110 conditions on the interface (i.e., constant normal load, constant volume, and constant
111 normal stiffness conditions). A maximum of 800 kN normal load (about 11.3 MPa on
112 the specimen with 300 mm diameter) can be applied onto the interface. The two
113 horizontal servo actuators can apply all kinds of shear paths, such as pre-load
114 monotonic, two-way beeline and cross cyclic, one-way and two-way circular and
115 elliptical cyclic, and any other customized shear paths. The shear path could be under
116 displacement control, load control, or a combination of both in the x and y directions.
117 The maximum total tangential displacement is ± 150 mm and the maximum shear load
118 is 400 kN in the two tangential directions.

119 *Simple-shear soil container*

120 Figure 2 shows the simple-shear type soil container used for the interface tests. The
121 ring group was composed of 27 round rings with 300 mm internal diameter and 3 or 4
122 mm thickness. A number of holes were evenly distributed along the 3 mm thick rings
123 to hold the low-frictional steel bearings. Rings with 3 and 4 mm thicknesses were
124 alternately stacked with a 0.5 mm gap. The low-frictional steel bearings ensure little
125 lateral resistance between the rings. The ring group was placed on the lower box, and
126 they were supported by a spring group, as shown in Fig. 2. Therefore, the whole
127 container (including the ring group and the lower box) is capable of freely moving up
128 and down to accommodate the volumetric deformation of the interface during shearing.
129 In addition, a 2 mm thick latex film was attached to the internal wall of the container to
130 prevent the soil particles from sticking into the gap between the rings when the shear
131 deformation of the soil takes place.

132 The relative displacements of the rings were recorded by two cameras installed at
133 the two orthogonal tangential directions, as shown in Fig. 1. The two cameras could
134 synchronously photograph the positions of the marked points on each ring at desired
135 frequency ranging from 0.01 Hz to 1 Hz. The images were then processed to obtain the
136 lateral displacement of the marked points and the tangential displacements of the rings.
137 Details of the image processing technique were described by Zhang and Liang et al.
138 (2006). The measured total tangential displacements using the image processing
139 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 $\varphi = \varphi_0 - \Delta\varphi \cdot \lg(\sigma_3/P_a)$, where $\varphi_0=50^\circ$ and $\Delta\varphi=8^\circ$ are strength parameters.

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

173 *Shear paths*

174 Linear and circular displacement-controlled shear paths were used in the present study.
175 The interface of a linear shear path was sheared along a linear route in three different
176 ways, namely, monotonic, two-way beeline cyclic, and two-way cross cyclic, which
177 are respectively illustrated as the routes of 1, 1-2-3-4-1-..., and 1-2-3-4-5-6-7-8-1-...
178 in Fig. 4(a). On the other hand, the interface of a circular shear path was first sheared
179 along the route of 0 (shown in Fig. 4(b)) to the required displacement amplitude (u_r)
180 and then was repeatedly sheared along a circle in the x - y tangential plane. It included
181 clockwise circular shear path 1-2-3-4-1-... and clockwise-counterclockwise circular

182 shear path 1-2-3-4-5-6-7-8-1-..., respectively referred to as the one-way and two-way
183 circular cyclic shear paths. Either 1-2-3-4 or 5-6-7-8 was regarded as one single shear
184 cycle to facilitate the interpretation of test results. The amplitudes of the total
185 tangential displacement (u_r) were 40 mm for monotonic tests and ± 20 mm for cyclic
186 tests. The normal stress was kept constant for all the interface tests.

187 **MONOTONIC BEHAVIOR**

188 *Total response*

189 The total response of the interface, referring to the interface behavior concerning u , has
190 been the focus of many previous studies (Potyondy 1961, Desai et al. 1985, Hu and Pu
191 2004). Figure 5 provides the test results of the gravel-steel interface subjected to
192 monotonic loads at different normal stress ($\sigma_n=200, 400, 700$, and 1000 kPa). Normal
193 displacement is obviously observed in Fig. 5(c), revealing that the volumetric
194 deformation of the interface takes place due to shearing. Under relatively low σ_n , the
195 interface is compressed first and then dilated obviously. Larger σ_n leads to more
196 compression and less dilatancy. It is indicated that the volumetric deformation of the
197 gravel-steel interface is induced by shear application and is influenced by normal stress.
198 The compression-shear coupling effect is presented in the volumetric deformation of
199 the interface.

200 Figure 5(a) shows that the shear stress (τ) increases gradually with u , reaches the
201 peak shear strength (τ_f), and then remains almost constant, demonstrating that the τ vs.

202 u response exhibits no obvious strain-softening behavior. This behavior is different
203 from the typical dense sand-rough steel interface, which exhibits apparent
204 strain-softening response (Fakharian and Evgin 1997, Uesugi and Kishida 1986a and
205 1986b). Figure 5(b) gives the curves of the stress ratio $\eta (= \tau/\sigma_n)$ vs. u and provides a
206 better overview of the unobvious strain-softening behavior, especially when the normal
207 stress is relatively high (e.g., $\sigma_n = 1\,000$ kPa). The linearity relationship between u/τ
208 and u (depicted in Fig. 5 (d)) reveals that the τ vs. u response behaves in a hyperbola
209 manner. Figures 5(a) and 5(b) reveal that the initial slope of the τ vs. u curves (i.e., the
210 initial shear modulus of the interface) increases with increasing σ_n while the initial
211 slope of the η vs. u curves decreases, indicating that the normal stress greatly
212 influences the shear modulus of the interface.

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

223 *Soil deformation*

224 The soil deformation (δ) and the corresponding u_d were obtained from the lateral
225 displacements of the rings by processing the images captured by the two real-time
226 cameras. Figure 6 provides the profile of soil deformation (δ) along the specimen depth
227 (h) at different σ_n and u . The δ increases with u but degrades with h , and is
228 considerably small when the soil layer is deeper than 62 mm (i.e., $h > 62\text{mm}$); thus, it
229 can be neglected compared with that of the top soil layer near the steel plate. The soil
230 deformation caused by shearing appears limited to a localized shear zone, beyond
231 which the soil is barely affected by the shear application. This thickness of the
232 localized shear zone is defined as the interface thickness. For the tests in the present
233 study, the interface thickness is about 58 mm at $\sigma_n = 400\text{ kPa}$ and about 62 mm at $\sigma_n =$
234 1 000 kPa. The normal stress exerts negligible effects on the interface thickness and on
235 the profile of soil deformation. The results of the monotonic tests reveal that the
236 interface thickness is approximately $(6-7) \cdot D_{50}$ of the gravel. Li (2001) showed that the
237 $(6-7) \cdot D_{50}$ interface thickness also applies to the interface between sand-blasted steel
238 plate and medium-crushed quartz sand at $\sigma_n = 100\text{ kPa}$.

239 The soil deformation directly leads to a component of the tangential displacement
240 of the interface (i.e., deforming displacement, u_d). From the u_d vs. u curves in Fig. 7(a),
241 the u_d is further confirmed to increase with u and σ_n because of the stronger friction
242 constraint of the steel plate. The gravel still deforms after the peak shear strength of the
243 interface is mobilized, as shown in Fig. 7(c). This observation is different from the

244 typical sand-steel interface, of which the sand deformation ceases after the peak shear
245 strength is mobilized (e.g., Fakharian and Evgin 1997). The τ vs. u_d and v vs. u_d curves
246 appear similar to the total response of the interface, except for the larger slope of the τ
247 vs. u_d and v vs. u_d curves in Figs. 7(c) and 7(e).

248 *Sliding displacement*

249 The sliding displacement (u_s) occurs at the contact surface between the gravel and the
250 structural plate. The gravel-steel interface will fail at the contact surface if the sliding
251 displacement is sufficiently large. Figure 7(b) demonstrates that the sliding
252 displacement occurs as long as the shear application is applied. Apparent turning points
253 are observed in the u_s vs. u plots marked as A and B in Fig. 7(b). The u_s increases much
254 faster when u exceeds the turning points, and the peak shear strength of the interface is
255 mobilized at the turning points. These turning points indicate the onset of the interface
256 failure at the contact surface. Before the peak shear strength is mobilized, the gravel
257 particles adjacent to the structural plate may climb and be rearranged caused by the
258 friction constraint of the structural plate. Therefore, the gravel deforms greatly and the
259 deforming displacement accounts for the majority of the total tangential displacement.
260 When the gravel particles have just climbed and began to mainly roll over against each
261 other, the peak shear strength is mobilized. The sliding displacement thus increases
262 faster and the deforming displacement decreases.

263 The τ vs. u_s and v vs. u_s curves of the interface appear similar to the total response,

although the τ 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 σ_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

284 relatively large and the soil deformation is still localized in the shear zone and
285 degrades with the specimen depth. The measured interface thickness from the results of
286 Fig. 8 is about 60 mm during cyclic shearing in both linear and circular shear paths and
287 is the same as that during monotonic shearing. The shear path impacts little effect on
288 the interface thickness. Comparing the soil deformation at different N values, the
289 amplitude of soil deformation inside or outside the shear zone decreases with
290 increasing N , indicating that the gravel becomes hardened gradually. The pattern of soil
291 deformation distribution appears similar at different N values. Figure 9 shows the soil
292 deformations in the x - y tangential plane at different specimen depths for the two-way
293 circular shear test. The curves appear to be circular at different specimen depths,
294 particularly at the first few shear cycles.

295 Figure 10 plots u_d vs. u curves in the x or y direction for the cyclic shear tests. The
296 growth of the deforming displacement is slower than its recovery, and the growth rate
297 decreases with u . Thus, large u_d vs. u cycles are observed, particularly at the first few
298 shear cycles. The amplitude of u_d and the slope of u_d vs. u curves decrease as the cyclic
299 shear application continues in all shear paths. The u_d becomes considerably smaller
300 compared with u when the interface is cyclically sheared to some extent (e.g., $N \geq 15$).
301 As shown in Figs. 8(a), 8(b) and 10(a), the soil deformation and the deforming
302 displacement tend to migrate toward the initial shear direction with increasing N in the
303 two-way beeline cyclic shear path, indicating that the soil deformation and the
304 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 τ vs. u , τ vs. u_d , and τ 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 τ vs. u curves during loading stage are hyperbolic, as shown in Figs. 11(a1) and 11(b1). The hyperbolic τ vs. u curves are similar to those of the monotonic tests. As shear cycle increases, the shear modulus (i.e., the slope of τ vs. u curves) during loading is observed to increase, and the slope of τ vs. u_d curves becomes larger, whereas the slope of τ 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.

325 The two-way cross shear path can be regarded as the extended pattern of the
326 two-way beeline shear path in 3-D interface space. Therefore, the interface presents
327 similar shear stress-displacement hysteretic response in these two linear shear paths,
328 except that the shear stress begins from zero at the start of each shear cycle in the cross
329 shear path. For the same shear cycle, the shear modulus at the start (e.g., $N = 0-0.25$) is
330 larger than that during other loading stages (e.g., $N = 0.25-0.5$ and $0.75-1$). It is also
331 larger than that of the two-way beeline shear test. These findings demonstrates that the
332 shear history of the orthogonal direction affects the shear behavior of the interface. The
333 shear history is disturbed and exerts little influence on the subsequent interface
334 behavior after the shear direction is reversed and the interface is reloaded.

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

344 For the two-way circular shear tests, the τ_y vs. u_y hysteretic curves present similar

response at all N values, but the τ_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 τ_x vs. u_x hysteretic curves exhibit similar response at all N values. Meanwhile, the τ_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 τ vs. u , τ vs. u_d , and τ vs. u_s curves) at different N values do not coincide because of the migration and decreasing amplitudes of τ , 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. τ 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 τ , 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

386 elastic manner with the deforming displacement. The interface shows slight
387 compression at the start of each shear cycle in the two-way cross shear path, because
388 the shear direction changes to the orthogonal and the interface is partially unloaded.

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

397 The curves of v vs. u , v_{re} vs. u , v_{re} vs. u_d and v_{re} vs. τ are different in the x and y
398 directions for the circular shear tests because of the different starting points. They are
399 also different from those in the linear shear paths, indicating that the shear path greatly
400 influences the volumetric behavior of the interface, including the normal displacement
401 amplitude, and the normal displacement vs. tangential displacement and shear stress
402 curves. The v vs. u plots are not closed within a single shear cycle and do not coincide
403 at different N values because of the cumulative v_{ir} , especially at the first few shear
404 cycles. The v_{re} vs. u , v_{re} vs. u_d , v_{re} vs. τ plots appear closed within a single shear cycle
405 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_{re} .

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 (τ) 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 τ_{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., τ_{fx+} , τ_{fx-} , τ_{fy+} and τ_{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 τ_{fd} and the corresponding τ_{fx+} , τ_{fx-} , τ_{fy+} and τ_{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

426 normal stress ($\sigma_n=200, 400, 700$ and $1\,000$ kPa). The τ_{fd} presents linearly proportional
427 response to the applied σ_n , indicating that the τ_{fd} also behaves well in accordance with
428 the Mohr-Coulomb failure criteria that follow the equation $\tau_{fd}=\sigma_n \cdot \tan(\varphi_{fd})$, where φ_{fd} is
429 the cyclic friction angle of the interface. It is also evident from Fig. 15 that the cyclic
430 shear strength and the corresponding cyclic friction angle degrade with increasing N .

431 As shown in Fig. 14, the shear strength in the x or y direction is smaller than the
432 cyclic shear strength for the circular shear tests, proving the necessity of introduction
433 of the resultant shear stress. The shear strength and the corresponding friction angle in
434 the positive tangential direction (i.e., the initial shear direction) are generally smaller
435 than those in the negative direction (i.e., the reload direction), which is also evident in
436 Fig. 13. For two-way beeline cyclic test results shown in Fig. 15, the cyclic friction
437 angle at $N=1$ in positive direction ($\varphi_{fd} = 32.2^\circ$) is smaller than that in negative direction
438 ($\varphi_{fd} = 33.6^\circ$), and the cyclic friction angle at $N=30$ in positive direction ($\varphi_{fd} = 28.7^\circ$) is
439 also smaller than that in negative direction ($\varphi_{fd} = 29.9^\circ$). These findings demonstrate
440 that the shear strength of the gravel-steel interface is anisotropic induced by the shear
441 orientation effect and is influenced by the shear path and shear history. When the
442 interface is sheared in the initial shear direction at the first quarter shear cycle, the
443 gravel particles in the shear zone are rearranged toward this direction, and orientation
444 fabric thus occurs. On the other hand, when the interface is sheared in the reload
445 direction at the subsequent half shear cycle, a higher shear stress is required to weaken
446 or destroy the orientation fabric.

447 Figure 14 also demonstrates that the cyclic shear strength and the shear strength in
448 a single tangential direction for the circular shear tests are slightly smaller than those
449 for the linear shear tests, because the simultaneous shearing in the two orthogonal
450 directions might further loosen the interlocking of the gravel particles. This
451 demonstrates that the shear path plays a significant role in the cyclic shear strength of
452 the interface.

453 CONCLUSIONS

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

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

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

468 cyclic shearing.

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

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

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

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490 REFERENCES

491 Afzali-Nejad, A., Lashkari, A. and Shourijeh, P.T. 2017. Influence of Particle Shape on
492 the Shear Strength and Dilation of Sand-Woven Geotextile Interfaces. *Geotextiles*
493 and Geomembranes, **45**: 54-66.

494 Al-Douri, R.H., and Poulos, H.G. 1992. Static and cyclic direct shear tests on
495 carbonate sands. *Geotechnical Testing Journal*, **15**(2): 138–157.

496 Brandt, J.R.T. 1985. Behavior of soil-concrete interfaces. Ph. D. thesis, University of
497 Alberta, Alberta, Canada.

498 Boukpeti, N., and White, D.J. 2017. Interface shear box tests for assessing axial
499 pipe-soil resistance. *Geotechnique*, **67**(1): 18-30.

500 Chen, X.B., Zhang, J.S., Xiao, Y.J., and Li, J. 2015. Effect of roughness on shear
501 behavior of red clay-concrete interface in large-scale direct shear tests. *Canadian*
502 *Geotechnical Journal*, **52**(8): 1122-1135.

503 Desai, C.S., Drumm, E.C., and Zaman, M.M. 1985. Cyclic testing and modeling of
504 interfaces. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE,
505 **111**(6): 793–815.

506 Di Donna, A., Ferrari, A., and Laloui, L. 2016. Experimental investigations of the soil–
507 concrete interface: Physical mechanisms, cyclic mobilization, and behaviour at

- 508 different temperatures. Canadian Geotechnical Journal, **53**(4): 659-672.
- 509 DeJong, J.T., White, D.J., and Randolph, M.F. 2006. Microscale observation and
510 modeling of soil-structure interface behavior using particle image velocimetry.
511 Soils and Foundations, **46**(1): 15–28.
- 512 DeJong, J.T., and Westgate, Z.J. 2009. Role of initial state, material properties, and
513 confinement condition on local and global soil-structure interface behavior. Journal
514 of Geotechnical and Geoenvironmental Engineering, **135**(1): 1646-1660.
- 515 Fakharian, K. 1996. Three-dimensional monotonic and cyclic behavior of sand-steel
516 interfaces, testing and modeling. Ph.D thesis, Univerisity of Ottawa, Ottawa,
517 Canada.
- 518 Fakharian, K., and Evgin, E. 1997. Cyclic simple-shear behavior of sand-steel
519 interfaces under constant normal stiffness condition. Journal of Geotechnical and
520 Geoenvironmental Engineering, ASCE, **123**(12): 1098–1105.
- 521 Farhadi, B., and Lashkari, A. 2017. Influence of soil inherent anisotropy on the
522 behavior of crushed sand-steel interfaces. Soils and Foundations, **57**(1): 111-125.
- 523 Hu, L.M., and Pu, J.L. 2004. Testing and modeling of soil-structure interface. Journal
524 of Geotechnical and Geoenvironmental Engineering, ASCE, **130**(8): 851–860.
- 525 Khoury C., and Miller G. 2012. Influence of hydraulic hysteresis on the shear strength
526 of unsaturated soils and interfaces. Geotechnical Testing Journal, **35**(1): 1-15.

- 527 Kishida, H., and Uesugi, M. 1987. Tests of the interface between sand and steel in the
528 simple shear apparatus. *Geotechnique*, **37**(1): 45–52.
- 529 Li, B. 2001. An experimental study and numerical simulation of sand-steel interface
530 behavior. M.Sc. thesis, University of Ottawa, Ottawa, Canada.
- 531 Miller, G., and Hamid, T. 2007. Interface direct shear testing of unsaturated soil.
532 *Geotechnical Testing Journal*, **30**(3): 182-191.
- 533 Mortara, G., Mangiola, A., and Ghionna, V. 2007. Cyclic shear stress degradation and
534 post-cyclic behavior from sand-steel interface direct shear tests. *Canadian*
535 *Geotechnical Journal*, **44**: 739-752.
- 536 Oumarou, T.A., and Evgin, E. 2005. Cyclic behaviour of a sand-steel plate interface.
537 *Canadian Geotechnical Journal*, **42**: 1695–1704.
- 538 Potyondy, J.G. 1961. Skin friction between various soils and construction materials.
539 *Geotechnique*, **11**(4): 339–353.
- 540 Taha, A., and Fall, M. 2013. Shear behavior of sensitive marine clay-concrete
541 interfaces. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE,
542 **139**(4): 644-650.
- 543 Uesugi, M., and Kishida, H. 1986a. Influential factors of friction between steel and dry
544 sands. *Soils and Foundations*, **26**(2): 33–46.
- 545 Uesugi, M., and Kishida, H. 1986b. Frictional resistance at yield between dry sand and

- 546 mild steel. *Soils and Foundations*, **26**(4): 139–149.
- 547 Uesugi, M., Kishida, H., and Tsubakihara, Y. 1989. Friction between sand and steel
548 under repeated loading. *Soils and foundations*, **29**(3): 127–137.
- 549 Wang, J., Liu, F.Y., Wang, P., and Cai, Y.Q. 2016. Particle size effects on coarse
550 soil-geogrid interface response in cyclic and post-cyclic direct shear tests.
551 *Geotextiles and Geomembranes*, **44**: 854-861.
- 552 Yin, Z.Z., Zhu, H., and Xu, G.H. 1995. A study of deformation in the interface between
553 soil and concrete. *Computers and Geotechnics*, **17**: 75–92.
- 554 Zhang, G., Liang, D.F., and Zhang, J.M. 2006. Image analysis measurement of soil
555 particle movement during a soil-structure interface test. *Computers and*
556 *Geotechnics*, **33**: 248–259.
- 557 Zhang, G., and Zhang, J.M. 2006. Monotonic and cyclic tests of interface between
558 structure and gravelly soil. *Soils and Foundations*, **46**(4): 505–518.
- 559 Zhang, J.M., Feng, D.K., and Hou, W.J. 2018. An automated large-scale apparatus for
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Graphical abstract of the manuscript