Research Article

Consolidated and Undrained Ring Shear Tests on the Sliding Surface of the Hsien-du-shan Landslide in Taiwan

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A new consolidated undrained ring shear test capable of measuring the pore pressures is presented to investigate the initiation mechanism of the Hsien-du-shan rock avalanche, triggered by Typhoon Morakot, in southern Taiwan. The postpeak state of the landslide surface between the Tangenshan sandstone and the remolded landslide gouge is discussed to address the unstable geomorphological precursors observed before the landslide occurred. Experimental results show that the internal friction angle of the high water content sliding surface in the total stress state, between 25.3 and 26.1°, clarifies the reason of the stable slope prior to Typhoon Morakot. In addition, during the ring shear tests, it is observed that the excess pore pressure is generated by the shear contractions of the sliding surface. The remolded landslide gouge, sheared under the high normal stress, rendered results associated with high shear strength, small shear contraction, low hydraulic conductivity, and continuous excess pore pressure. The excess pore pressure feedback at the sliding surface may have accelerated the landslide.

1. Introduction

In 2009, the Hsien-du-shan landslide, triggered by Typhoon Morakot [1, 2], killed more than 400 people at the Hsiaolin village, Kaohsiung, Taiwan. Before this landslide mobilized, the authority announced two potential debris flow torrents (Kaohsiung County DF06 and DF07) (http://246.swcb.gov.tw/allfiles/PDF/98%E5%B9%B4%E8%8E%AB%E6%8B%89%E5%B9%B4%E8%8E%AB%E6%8B%89%E5%85%B8%E9%A2%B1%E9%A2%A8-%E9%AB%98%E9%9B%84%E7%94%9B%E4%BB%99-001-(%E9%80%9F).pdf) as major threats to the local residents in the Hsiaolin village. The debris flow damaged the village during the Kamaegi in 2008. However, the historical record was unavailable to warn the residents that the village is located at a geological site with a large unstable slope.

The Hsien-du-shan landslide had demanded the authorities to respond urgently in developing new technologies for warning the occurrence of a rainfall-induced rock avalanche. In addition to detecting an abnormal geomorphological pattern [3, 4], clarification of the initiation mechanism of a large rock avalanche is another important task to mitigate disasters (Chen and Wu (2018); Hung et al. (2018); [5]). Figure 1 shows the available mathematical models to describe the mechanical behavior of the sliding surface of the Hsien-du-shan landslide. A conventional constant friction coefficient was applied to different numerical methods to simulate the postfailure behavior (dashed line) (Figure 1) [1, 2, 6]. Kuo et al. [7] discovered experimentally slip weakening by the colluvium on the sliding surface that was within the range of natural water contents using high-speed rotary shear tests with maximum shear speed and shear displacement of up to 1.3 m/s and 90 m, respectively (solid lines). Slip weakening is defined as a significant decrease in the friction angle of a sliding surface under high-speed sliding. However, the friction models in Figure 1 did not explicitly consider the pore pressure changes during the Hsien-du-shan landslide. A conventional constant friction coefficient was applied to different numerical methods to simulate the postfailure behavior (dashed line) (Figure 1) [1, 2, 6]. Kuo et al. [7] discovered experimentally slip weakening by the colluvium on the sliding surface that was within the range of natural water contents using high-speed rotary shear tests with maximum shear speed and shear displacement of up to 1.3 m/s and 90 m, respectively (solid lines). Slip weakening is defined as a significant decrease in the friction angle of a sliding surface under high-speed sliding. However, the friction models in Figure 1 did not explicitly consider the pore pressure changes during the Hsien-du-shan landslide. A conventional constant friction coefficient was applied to different numerical methods to simulate the postfailure behavior (dashed line) (Figure 1) [1, 2, 6].
acceleration. Conversely, when shear zone contracts during slope failure. Then, the shear by rain in slide motion occurs while positive pore pressure is supplied granular materials, pore expansion in although dilation reduces the internal friction angle of dry soils [14 – 18]. Moore and Iverson [18] concluded that ring shear tests have focused on sand [12, 13] and clayey of landslide motion and pore pressure feedback. In addition, any magnitude [11].

Figure 1: Different friction model for the initiation of the Hsien-du-shan landslide.

Miller et al. [10] calculated the excess pore pressure of a fault using the porosity reduction mechanism. Iverson [8] proposed that landslide motion is regulated by α, which depends on the dilatancy angle and the intrinsic timescales for pore pressure generation and dissipation. When α < 0, soil in the shear zone contracts during slope failure. Then, the shear zone causes positive pore pressure feedback and runaway acceleration. Conversely, when α > 0, slow and steady landslide motion occurs while positive pore pressure is supplied by rain infiltration.

Currently, knowledge of the shear behavior of rock avalanches is rather limited, especially when sliding occurs at a great depth. Fortunately, the ring shear apparatus has two obvious advantages for investigating the initiation of a rock avalanche: (1) there is no change in the cross-sectional area of the shear plane as the test proceeds; and (2) the sample can be sheared through an uninterrupted displacement of any magnitude [11].

Table 1 shows the available ring shear tests. Developing a CU ring shear test apparatus is crucial to clarify the coupling of landslide motion and pore pressure feedback. In addition, ring shear tests have focused on sand [12, 13] and clayey soils [14–18]. Moore and Iverson [18] concluded that although dilation reduces the internal friction angle of dry granular materials, pore expansion in fluid-saturated materials increases friction and strength by reducing pore-fluid pressure and increasing normal stresses at grain contacts. Atterberg limits, particle shapes, and shearing rates strongly govern stress fluctuations [17], although the particle size distribution is an additional factor for controlling the residual shear strength [19] in ring shear tests.

The maximum depth of the Hsien-du-shan rock avalanche was 85.6 m [20] based on the topographic data before and after the sliding event. In this study, the unit weight of the soils is assumed to be ρ = 27.1 kN/m³. The maximum normal stress may exceed 2000 kPa. In Table 1, although the designed maximum normal stresses of the DPRI-4, DPRI-5, and DPRI-6 exceeded 2000 kPa, a successful case study indicating a maximum normal stress exceeding 2000 kPa was lacking. Although the DPRI-7 achieved a successful maximum normal stress of up to 750 kPa, Sassa et al. [21] raised the issue regarding the poor performance of a servo-control system. Therefore, only the ring shear test devices proposed by Liao et al. [22], ICL-2 [23], and Wu et al. [24] would fulfill the desired high normal stresses that occurred at the Hsien-du-shan rock avalanche. This study presents an improvement in the ring shear device proposed by Liao et al. [22] and Wu et al. [24] to monitor the pore water pressure and the shear behavior of the soil/rock interface of the Hsien-du-shan rock avalanche.

2. The New Ring Shear Apparatus

The ring shear box consists of the upper ring shear box (Figure 2(a)) and the lower shear box (Figure 2(b)). For the ring shear apparatus only for the soils ([15, 25]; https://www.youtube.com/watch?v=R9vTsrE8bSA), the rough interface or vanes must be installed at the top and the bottom shear boxes to ensure that the shear failure occurs in the soils but not the interface of the shear box and the soils. In this study, the apparatus was designed to investigate the interface of two rock samples or of the rock and the soil. Rock sample is adhered to the upper shear box, and the interface between the rock sample and the top shear box is flat (Figure 2(a)). The soils are installed to the lower ring shear box (Figure 2(c)).

This ring shear system consists of an MTS, ring shear box, ring displacement control system, signal generator, and data acquisition system (Figure 3(a)). The MTS applies the normal stresses and measures the normal displacement and torque in the ring shear tests. The axial force and torque capacities of the MTS are 50 kN and 500 N-m, respectively. The ring shear box (Figure 3(b)) comprises the upper and lower boxes and is capable of functioning a maximum normal loading of 100 kN. When investigating the interface of the two rock samples, an epoxy is used to adhere rock specimens to both the upper and lower shear boxes [26]. However, when a soil-rock interface is investigated, the rock sample is adhered to the upper shear box, and the soils are put in the lower shear box. The ring displacement control system below the lower shear box, with a large displacement and rotational speeds ranging from 0.0018–1.8 mm/min, drives the lower shear box during the shearing tests. In each testing, the Square Teflon O-rings are, respectively, attached on the inner and outer sides of the samples, in the upper and lower shear boxes, to prevent extrusions water and soils.

In this study, the inner and outer diameters of the ring-shaped specimen are 11.0 cm and 14.9 cm, respectively. The inner and outer radius ratio is n = 0.74. The designs meet the dimensional requirements for a specimen as suggested by (1) [11] and (2) [27], for obtaining a uniform distribution of stress on the shear plane.

\[
\frac{R_i}{R_o} = \frac{D_i}{D_o} \geq 0.71, \quad (1)
\]

\[
\frac{R_i}{R_o} = \frac{D_i}{D_o} \geq 0.65, \quad (2)
\]

where \(R_o\) is the specimen outer radius, \(R_i\) is the specimen inner radius, \(D_o\) is the specimen outer diameter, \(D_i\) is the
The normal and shear stresses applied to the shear zone of the specimen are represented by (3), (4), and (5):

\[ \sigma_n = \frac{F_n}{\pi (R_o^2 - R_i^2)}, \]  
\[ M = 2\pi \int_{R_i}^{R_o} r^2 \, dr = \frac{2\pi \tau (R_o^3 - R_i^3)}{3}, \]  
\[ \tau = \frac{3M}{2\pi (R_o^3 - R_i^3)}, \]  

where \( \sigma_n \) is normal stress, \( \tau \) is shear stress, \( F_n \) is normal force, \( M \) is torque, and \( R_i \) and \( R_o \) are the inner and outer radii, respectively.

When the ring shear tests start, a stress-controlled normal stress is applied from the upper shear box to consolidate the soil samples, and then the ring displacement control system below the lower shear box drives the lower shear box at a specified shear rate. The shear resistance along the shear plane, the interface of the samples, is measured.

During the ring shear test, the sliding surface must ensure a high water tightness, and the pore pressure cell installation must be adjoining to the sliding surface to accurately measure the changes in sliding-induced pore pressures. In the new ring shear apparatus, a drainage pipeline is drilled in the lower shear box slightly lower than the Teflon square O-ring and is adjoining to the sliding surface (Figure 3(b)). Note that if the pressure cell is directly attached to the drainage pipeline, the rotating ring shear test apparatus under a large shear displacement may tie up the pore pressure cell cable and break the pore pressure cell. To avoid the pressure cell damage, the drainage pipeline is connected to a pore pressure cell by a flexible tube to monitor the pore pressure of the shear plane (Figure 3(c)). The flexible tube is connected to the drainage pipeline after setting rock samples to the ring shear test apparatus, and the filling water to the hollow acrylic cover should fill up the tube and the pipeline (Figure 3(b)). The pore pressure cell should be connected to the flexible tube below the water table and then is attached to the top of the acrylic cover (Figure 3(c)) before the shear test starts. The water tightness of the new ring shear apparatus was assured by Wu [28], in which the pore pressure reduced from its initial state of 125 to 120 kPa within 20 hours. The dilatancy or contraction at the shear plane during shearing can be obtained from changes in the specimen height.

### Table 1: The sample size of ring shear test (modified by [39]).

<table>
<thead>
<tr>
<th>Author</th>
<th>Inner diameter, ( D_i ) (cm)</th>
<th>Outer diameter, ( D_o ) (cm)</th>
<th>( n ) ( (D_i/D_o) )</th>
<th>Sample</th>
<th>Max. normal stress (kPa)</th>
<th>Pore pressure monitoring</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bishop et al.</td>
<td>10.16</td>
<td>15.24</td>
<td>0.67</td>
<td>Clay</td>
<td>980</td>
<td>N</td>
</tr>
<tr>
<td>Hungr and Morgenstern</td>
<td>22.00</td>
<td>30.00</td>
<td>0.73</td>
<td>Coarse sand, sand-rock flour mixtures, and polystyrene beads</td>
<td>200</td>
<td>N</td>
</tr>
<tr>
<td>Tika and Hutchinson [40]</td>
<td>10.16</td>
<td>15.24</td>
<td>0.67</td>
<td>Cohesive soil</td>
<td>980</td>
<td>N</td>
</tr>
<tr>
<td>Garga and Sendano</td>
<td>9.20</td>
<td>13.30</td>
<td>0.69</td>
<td>Sand</td>
<td>660</td>
<td>N</td>
</tr>
<tr>
<td>Sassa (DPRI-3)</td>
<td>21.0</td>
<td>31.0</td>
<td>0.68</td>
<td>Soils</td>
<td>500</td>
<td>Y</td>
</tr>
<tr>
<td>Sassa (DPRI-4)</td>
<td>21.0</td>
<td>29.0</td>
<td>0.72</td>
<td>Coarse grain sandy soil</td>
<td>3000</td>
<td>Y</td>
</tr>
<tr>
<td>Sassa (DPRI-5)</td>
<td>12.0</td>
<td>18.0</td>
<td>0.67</td>
<td>Soils</td>
<td>2000</td>
<td>Y</td>
</tr>
<tr>
<td>Sassa (DPRI-6)</td>
<td>25.0</td>
<td>35.0</td>
<td>0.71</td>
<td>Sands</td>
<td>3000 (designed) 750 (success)*</td>
<td>Y</td>
</tr>
<tr>
<td>Sassa (DPRI-7)</td>
<td>27.0</td>
<td>35.0</td>
<td>0.77</td>
<td>Silica sand</td>
<td>500</td>
<td>Y</td>
</tr>
<tr>
<td>Iverson et al. [16]</td>
<td>48.5</td>
<td>60.0</td>
<td>0.81</td>
<td>Tills</td>
<td>400</td>
<td>N</td>
</tr>
<tr>
<td>Liao et al. [22]</td>
<td>11.0</td>
<td>14.9</td>
<td>0.74</td>
<td>Sandstone</td>
<td>12,600 (designed) 4000 (success)**</td>
<td>N</td>
</tr>
<tr>
<td>Ostric et al. [41]</td>
<td>10.0</td>
<td>14.0</td>
<td>0.71</td>
<td>Marl</td>
<td>1000</td>
<td>Y</td>
</tr>
<tr>
<td>Tomasetta et al. [42]</td>
<td>6.0</td>
<td>12.0</td>
<td>0.50</td>
<td>Fine powders</td>
<td>14,795</td>
<td>N</td>
</tr>
<tr>
<td>Hoyos et al. [15]</td>
<td>9.65</td>
<td>15.24</td>
<td>0.63</td>
<td>Soils</td>
<td>732</td>
<td>Y (suction)</td>
</tr>
<tr>
<td>Sassa et al. [23]</td>
<td>10.0</td>
<td>14.2</td>
<td>0.70</td>
<td>Sands</td>
<td>3000</td>
<td>Y</td>
</tr>
<tr>
<td>Jeong et al. (2014)</td>
<td>11.0</td>
<td>25.0</td>
<td>0.44</td>
<td>Gravelly sandy soil</td>
<td>100</td>
<td>Y</td>
</tr>
<tr>
<td>Wu et al. [24]</td>
<td>11.0</td>
<td>14.9</td>
<td>0.74</td>
<td>Sandstone/remolded shale</td>
<td>4000</td>
<td>N</td>
</tr>
<tr>
<td>This study</td>
<td>11.0</td>
<td>14.9</td>
<td>0.74</td>
<td>Sandstone/remolded shear gouge</td>
<td>2710</td>
<td>Y</td>
</tr>
</tbody>
</table>

*Normal stress servo-control system does not function well. Tests were conducted to 750 kPa by changing the normal stress load cell [21]; ** successful case study of the Tsaoing landslide during the Chi-Chi earthquake [22].
3. The Hsien-du-shan Rock Avalanche

Figure 4 shows the rainfall data during Typhoon Morakot in 2009 recorded at the Chiahsien Station, which is the nearest rainfall station to the Hsien-du-shan landslide site in Kaohsiung County, Taiwan. The cumulative rainfall exceeded 2000 mm. The Hsien-du-shan rock avalanche occurred at 6:09 a.m. on August 9 and resulted in compound disasters, comprised of slope failure, landslide dam, and dam breakage, claiming more than 400 local residents of Hsiaolin village near the toe of the slope in Kaohsiung (Figure 5).

The shear strength parameters of the sliding surface are key factors when investigating a failure mechanism. Studies by Kuo et al. [7] indicated that the static friction as the peak coefficient of friction is between 0.3 (\(\phi = 16.7^\circ\)) and 0.75 (\(\phi = 36.9^\circ\)), with a normal stress = 1.0 MPa, when the water content of local colluvium is between 9 and 25% (Figure 1). When the rotary shear speed reaches 1.3 m/s, significant slip weakening occurs, and the coefficient of friction drops to a range between 0.1 (\(\phi = 5.7^\circ\)) and 0.2 (\(\phi = 11.3^\circ\)). The experimental results of the rotary shear test clarify that the shear strength of the sliding surface decreases remarkably when the landslide accelerates. Wu et al. [2] applied conventional direct shear tests to the high water content remolded soil obtained from the sliding slope of the Hsien-du-shan slope and suggested the normal stresses of 0.5, 1.0, 2, and 3.49 MPa for the experimental testing. The shear strength parameters of the high water content remolded soil are \(c = 0\) kPa and \(\phi = 21.4^\circ\). In addition, Lin et al. [29] concluded that the shear strength parameters of the interface between the Tangenshan sandstone and the remolded Yenshuikeng shale, exhibiting high water contents, were \(c = 0.0\) and \(\phi = 24.3^\circ\) as determined by conventional direct shear tests. Then, the studies validated the stability of the Hsien-du-shan slope under the impact of the Chi-Chi earthquake in 1999 and the Heng-Chun earthquake in 2006. In fact, due to the absence of well-installed in situ groundwater monitoring instruments, the pore pressure change of the Hsien-du-shan sliding surface during its initiation is difficult to be obtained. In addition to field monitoring, Kuo et al. [7] and Wu et al. [2] have also difficulties in monitoring the pore pressure changes in experimental shear tests.

3.1. Geological Outline. The Hsien-du-shan slope is located at the east wing of the Hsiaolin syncline. The Hsiaolin syncline axis is located at the toe of the slope near the Hsiaolin Village and is east of the Chishan River [30–32]. A shear zone was discovered at the south boundary of the Hsien-du-shan rock avalanche [32].

The topographic and elevation differences in the source area before and after the occurrence of a landslide can be used to obtain the volume and maximum thickness of the sliding mass, respectively. The obtained values are carefully considered for maximum normal stress during the shear tests. Chen and Wu [20] calculated that the slid volume of the Hsien-du-shan rock avalanche was c.a. \(27.118 \times 10^6\) m\(^3\)

![Ring shear box](image)
(a) Ring shear test system

(b) Sketch of the ring shear box

(c) Arrangement of the pore pressure cell

**Figure 3:** New pore pressure monitoring ring shear test apparatus.

**Figure 4:** Rainfall data at the Chiahsien station.
and the maximum sliding thickness was 85.6 m using a 5 m × 5 m Digital Elevation Model before and after the landslide. The sliding surface is located at the interface of the fractured and weathered Yenshuikeng shale at the top and the Tangenshan sandstone at the bottom [30]. The water content of the sliding surface was high because the rock avalanche was triggered by heavy rainfall. Gravitational slope deformations were observed as unstable precursors before slope failure [2, 32].

4. The Consolidated Undrained Ring Shear Tests

4.1. Sample Preparation. In 2009 and 2010, the sliding gouges still covered the Tangenshan sandstone formation near the source area post the landslide event. The yellow sliding gouge implies that the gouge was generated as a filling layer at the interface of the Yenshuikeng shale and Tangenshan sandstone prior to the incident of the landslide. In addition, the results of the X-ray diffraction analysis indicated that the minerals in the sliding gouge are very close to those observed in the Yenshuikeng shale [33]. To better understand the shear behavior of the sliding surface of the Hsien-du-shan rock avalanche, the interface between the sliding gouge and the Tangenshan sandstone is also investigated.

4.1.1. The Tangenshan Sandstone Sample. Tangenshan sandstones were obtained at the slope near the east edge of the 590 height (Figure 5), whose elevation is 590 m above the sea level. The physical and mechanical properties of the obtained Tangenshan sandstone are shown in Table 2. The uniaxial compressive strength and Young’s modulus of the sandstone decrease significantly as the water content increases. In addition, the slake-durability index of the sandstone is $I_d^1 = 99.03–99.29$ and $I_d^2 = 98.39–98.76$. Based on the Gamble classification, sandstone is classified as a very high-durability rock [34]. Hollow cylinder samples with an inner diameter of 11.0 cm and an outer diameter of 14.9 cm are drilled from the retrieved rocks. The cylinders are cut with a height of $5 \pm 0.2$ cm to satisfy the size requirements of the ring shear box (Figure 6).

4.1.2. The Remolded Landslide Gouge. Due to the precipitous landform, a sufficient number of the gouges at the sliding surface of the source area were unavailable. In this study, the sandstones and the gouge at the sliding surface were taken in the transitional zone (Figures 5 and 7(a)). Therefore, the gouges are remolded to investigate the postpeak shear behavior because of the unstable topographical precursors before the landslide [2, 32]. In addition, Japanese landslide case studies [35] showed good agreement between the residual friction angles determined from undisturbed samples and remolded specimens using ring shear tests.

In Figure 7(a), only the gouges at the depth adjoining to the underlying Tangenshan sandstone were taken to avoid mixing external debris at the ground surface during slope failure (Figure 7(b)). Table 3 shows the physical properties
of the landslide gouge, which are classified to be CL based on the Unified Soil Classification.

The procedures for preparing the remolded local soils for the consolidated undrained (CU) ring shear tests include the following:

**Step 1.** Soils are passed through a no. 10 sieve to reduce the impacts of large grains on the shear test because the thickness of the sample is 2 cm. The lower shear box is installed in the ring displacement control system (Figure 3). Then, the lower shear box is filled with the dry gouges until the groove for the O-ring.

**Step 2.** At the Hsien-du-shan slope, the landslide gouge is located above the Tangenshan sandstone. However, in the ring shear test, the arrangement of the Tangenshan sandstone sample above the remolded gouges (Figure 8) is opposite to the in situ case on the Hsien-du-shan slope. The upper and the lower shear boxes are integrated into the ring shear apparatus. The specified normal force is applied to the soil sample in dry conditions for 12 hours. Then, the interface of the remolded landslide gouge and sandstone sample is submerged in water for an additional 36 hours for the CU ring shear tests considering that the Hsien-du-shan landslide was triggered by heavy rainfall.

### 4.2. Identification of Test Parameters

The area of the ring shear sample, $A$, is $79.33 \times 10^{-4}$ m² because the outer and inner diameters of the sample are 14.9 and 11.0 cm, respectively. The normal forces used in the tests are calculated by the equation $F_n = \gamma \times D \times A$. In the CU tests, the normal force, $F_n$, to consolidate the remolded landslide gouge is 21.5 kN, considering the maximum depth of the Hsien-du-shan rock avalanche was roughly 100 m based on the topographic data before and after the landslide. Then, three normal forces, $N_{50} = 10.75$ kN, $N_{75} = 16.125$ kN, and $N_{100} = 21.5$ kN, corresponding to depths (Dep) of 50 m, 75 m, and 100 m, respectively, covered the normal force calculated from the maximum sliding thickness of 85.6 m [20] and are applied in the ring shear tests.

Vertical displacement of the sample in each test is monitored using the LVDT of the MTs from the beginning of the consolidation. A pore pressure sensor having a full capacity of 500 kPa, manufactured by Kyowa, Japan, is connected to the drainage pipeline beneath the water table at the lower ring shear box (Figure 3(b)) before the ring shear test starts. The ring displacement control system below the lower shear box (Figure 3(a)) controls the shear rate of 1.5 mm/min (1.3 degree/min) for the CU shear test after the normal stress, converging the vertical displacement, is applied. The normal and shear stresses are computed by incorporating the measured axial force and torque to (3) and (5). Subsequently, the curves of the shear stress-shear displacement, normal displacement-shear displacement, and pore water pressure change-shear displacement are plotted.

### 5. Experimental Results

CU ring shear tests were conducted on the interface of the Tangenshan sandstone and the remolded landslide gouges to clarify the mechanical behavior of this surface under large shear displacement. Figure 9 shows the CU ring shear test results under the three normal forces, $N_{50} = 10.75$ kN, $N_{75} = 16.125$ kN, and $N_{100} = 21.5$ kN. For each ring shear test, the sample was sheared for 3 cycles (rotation angle = 1080°; shear displacement = 1218 mm). The experimental time is approximately 13.5 hours under the shear rate of 1.5 mm/min.

The Mohr-Coulomb failure criterion (Figure 10) is obeyed to interpret the corresponding shear stresses by the three normal stresses, $N_{50}$, $N_{75}$, and $N_{100}$, based on the shear stress-shear displacement curves (Figure 9). Figure 10 shows the minimum, average, and maximum residual strengths of the ring shear tests in each shear circle. The total stress failure criteria of each shear circle show that the cohesion of the sliding interface is 0.0 kPa. In addition, the internal friction angles are $\phi = 25.3°$ (coefficient of friction = 0.480), 26.1° (coefficient of friction = 0.491), and 25.3° (coefficient of friction = 0.473) for shear circles 1, 2, and 3, respectively. The total stress failure criteria changed insignificantly when the shear distance is within 1218 mm, which can be considered as a landslide initiation (Figure 10).
Comparing this experimental data to the available shear strength parameters of the Hsien-du-shan sliding surface (Figure 1), the ring shear test results are close to the data obtained from conventional direct shear tests, $\phi = 21.4^\circ$ [2]. However, this new ring shear test provides higher strengths than the strengths determined by high-speed rotary shear tests with a water content exceeding 20% [7] (coefficient of friction = 0.1 to 0.2), numerical simulations [6] (coefficient of friction = 0.1), or the topographic map [1, 2] (coefficient of friction = 0.23). Studies by Kuo et al. [7] and Lo et al. [6] focused on investigating slip weakening of the sliding surface under high speed and very large shear movements. Rock mass disintegrations and the high sliding speed are key factors in decreasing the friction angle [2, 7]. The increase in pore pressures caused by the shear contraction implies that the effective normal stress and slope stability both decrease as the shear displacement increases after the onset of the sliding. In addition, an increase in the consolidating pressure increases the density but decreases the hydraulic conductivity of the remolded landslide gouge. Therefore, by considering both the long shear displacement of the rock avalanche and the large normal stress, the pore pressure would increase significantly and continuously (Figure 9).

The new CU ring shear tests provide additional remarks to clarify the initial sliding behavior of the Hsien-du-shan rock avalanche:

1. The sliding surface is in the postpeak state because of unstable topographical precursors before the landslide [2, 32].

2. After the landslide starts, the CU ring shear test results indicate that shear contractions would occur.

### Table 3: Physical properties of local landslide gouge.

<table>
<thead>
<tr>
<th>Sample number</th>
<th>Sample 1</th>
<th>Sample 2</th>
<th>Sample 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity ($G_s$)</td>
<td>2.711</td>
<td>2.709</td>
<td>2.696</td>
</tr>
<tr>
<td>Liquid limit (LL)</td>
<td>29.39</td>
<td>28.84</td>
<td>29.51</td>
</tr>
<tr>
<td>Plastic limit (PL)</td>
<td>14.42</td>
<td>15.90</td>
<td>15.22</td>
</tr>
<tr>
<td>Plastic index (PI)</td>
<td>14.97</td>
<td>12.94</td>
<td>14.29</td>
</tr>
<tr>
<td>$D_{10}$ (mm)</td>
<td>0.01247</td>
<td>0.00909</td>
<td>0.01097</td>
</tr>
<tr>
<td>$D_{50}$ (mm)</td>
<td>0.06612</td>
<td>0.06545</td>
<td>0.05478</td>
</tr>
<tr>
<td>$D_{60}$ (mm)</td>
<td>0.07304</td>
<td>0.07254</td>
<td>0.07099</td>
</tr>
<tr>
<td>Uniform coefficient, $C_u$</td>
<td>5.857</td>
<td>7.977</td>
<td>6.469</td>
</tr>
<tr>
<td>Coefficient of gradation, $C_c$</td>
<td>4.798</td>
<td>6.494</td>
<td>3.852</td>
</tr>
<tr>
<td>Unified soil classifications</td>
<td>CL</td>
<td>CL</td>
<td>CL</td>
</tr>
</tbody>
</table>
at the sliding surface leading to a large increase in the pore pressure. Iverson [8] has also supported that the increasing pore pressure would further accelerate the sliding. In addition, a deep sliding surface with a high normal stress results in a landslide gouge with low hydraulic conductivity, which dissipates the pore pressure later.

(3) The ring shear test results with a shearing rate of 1.5 mm/min are close to the data from conventional direct shear tests, $\phi = 21.4^\circ$ [2]. Both shear tests got the shear strength parameters when the landslide was initiated with slow shearing rate but not the whole sliding process. The sliding velocity of the Hsien-du-shan landslide is estimated from 20.4 to 33.7 m/s [32]. Yang et al. [36] indicated that the shear rate controls the shear behavior of a sliding surface. When the sliding surface is accelerated to a high speed, the slip weakening [7] governs the mechanical properties of the sliding surface. In addition, Wu [37] showed that the disintegration of the rock mass during a landslide increases the run-out distance of the sediments. Therefore, the slip weakening at the sliding surface and the disintegration of the blocky rock mass are the two main reasons that the internal friction angle of the sliding surface determined by the ring shear test ($\phi = 25.3^\circ$ to $26.1^\circ$) surpassed the apparent friction angle ($\phi = 13^\circ$), which is defined as the ratio of the vertical height and the horizontal travel distance of a landslide, suggested by the topography [2]. Further verifications can be carried out by a discrete element method with the slip weakening algorithm [38] in the future.

6. Conclusions

In this study, a new ring shear apparatus capable of conducting a CU shear test was developed. The capacity of the new device is ascertained by the successful modelling of the initiation of the heavy rainfall-induced Hsien-du-shan rock avalanche at the sandstone/landslide gouge interface with a maximum normal stress exceeding 2 MPa. Furthermore, the shear behaviors of the sliding surface under the different normal stresses were explored.

The experimental results indicated that the remolded landslide gouge governs the shear behavior of a high water content sliding surface such as the Hsien-du-shan rock avalanche. The internal friction angle of the sliding surface in the total stress state investigated by the ring shear test falls between 25.3 and 26.1$^\circ$, which is larger than the dip angle.
of the slope and thus clarifies the stability of the slope before Typhoon Morakot.

When the movement of the sliding interface initiated, the shear contraction at the sliding surface generates excessive pore pressure feedback and destabilizes the slope. In addition, the low hydraulic conductivity of the landslide gouge/sandstone interface would result in the significant and continuous pore pressure increase during the long shear
displacement when the sliding surface is sheared under large normal stresses. The excessive pore pressure can accelerate the rock avalanche.

**Data Availability**

The data used to support the findings of this study are available from the corresponding author upon request.

**Conflicts of Interest**

The authors declare that they have no conflicts of interest.

**Acknowledgments**

The authors appreciate their colleagues in the rock laboratory of the Department of Civil Engineering, National Cheng Kung University, Taiwan, for their kind help during rock gathering and sample preparations. In addition, thanks are due to the National Science Council of Taiwan (NSC 92-2211-E-426-003) and the Ministry of Science and Technology of Taiwan (MOST 107-2625-M-006-014) for their financial support. Special thanks are due to the reviewers and Prof. Ching Hung for his valuable comments.

**References**


Table 4: Water content of the samples after ring shear test.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth = 100 m</th>
<th>Depth = 75 m</th>
<th>Depth = 50 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Remolded landslide gouge</td>
<td>8.82%</td>
<td>14.49%</td>
<td>10.56%</td>
</tr>
<tr>
<td>Middle part</td>
<td>10.03%</td>
<td>11.83%</td>
<td>10.29%</td>
</tr>
<tr>
<td>Lower part</td>
<td>9.39%</td>
<td>12.00%</td>
<td>9.88%</td>
</tr>
<tr>
<td>Tangenshan rock</td>
<td>1.94%</td>
<td>2.13%</td>
<td>—</td>
</tr>
</tbody>
</table>


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