

Effect of Shear Strain Rate on Undrained Shearing Resistance of a Clean Silica Sand Measured in Direct Simple Shear Tests

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ABSTRACT: A number of studies examining the effect of shear strain rate on undrained shearing resistance of clean sands using conventional laboratory tests have been reported. Constant-volume direct simple shear laboratory tests were performed to model flow displacements at various shearing rates after triggering flow liquefaction. Tests on clean sand (Ottawa F-65 sand) at shear strain rates of 0.1%/min and 10%/min illustrated that the undrained yield (peak) shear strength at a shear strain rate of 10%/min was about 14% greater than that measured at a shear strain rate of 0.1%/min, while the undrained critical-state shear strength is relatively independent of shear strain rate for the shear strain rates (0.1%/min and 10 %/min), sand type, and initial state considered in this study.

1 INTRODUCTION

Whether the undrained shearing resistance of a cohesionless soil is affected by the shear strain rate is a subject of significant engineering importance. A significant number of conventional laboratory tests have been reported in the literature that explored this topic. Among laboratory tests, results from triaxial compression tests generally suggest that the strain rate has an effect on the undrained shearing resistance, especially undrained peak shear strength, of non-plastic, coarse-grained soils (Casagrande and Shannon 1948, Seed and Lundgren 1954, Nash and Dixon 1961, Whitman and Healy 1962, Reeves et al. 1967, Lee et al. 1969, Yamamuro and Lade 1993, Yamamuro et al. 2011, Suescun-Florez 2016). Table 1 summarizes undrained triaxial compression test results found in the literature, which cover shear strain rates ranging from 0.004%/min to 60,000%/min. Although the effect of shear strain rate on undrained peak shear strength varies greatly (Table 1), the overall trend appears to be that the undrained peak shear strength for a cohesionless soil increases with increasing shear strain rate. The increase is generally on the order of around +5% to +20% for each tenfold increase in shear strain rate (Seed and Lundgren 1954, Yamamuro and Lade 1993, Whitman and Healy 1962), but may reach of about 40% especially when the shear strain rate is very large, over 10^3 %/min (Reeves et al. 1967). Differences in drainage conditions, initial void ratio, and consolidation stress also result in variations in the increase in peak shear strength. However, because the tests compiled in Table 1 are triaxial compression tests, the observations related to increases in shearing resistance with shear strain rate are limited to relatively small shear (or axial) strains.

Table 1. Summary of the literature on the strain rate effect investigated using triaxial compression tests at small strains

Reference	Soil type	Saturation condition	Drainage condition	Loose or dense	Relative density or void ratio	Strain rate (%/min)	Consolidation stress (kPa)	Shear strain rate effect on peak shear strength ¹
Seed and Lundgren (1954)	Sacramento river sand	Saturated	Undrained	Dense and loose	0.59 to 0.83	0.7 to 60,000	200	Up to +10% to +15%
Nash and Dixon (1961)	Leighton Buzzard sand	Saturated	Undrained	-	-	2 to 8,000	700	-20%
Whitman and Healy (1962)	20-30 Ottawa sand	Saturated	Undrained	Dense	0.49	18 to 27,000 ²	275	No effect
		Saturated	Undrained	Loose	0.65	120 to 27,000 ²	70 to 350	+40%
Reeves et al. (1967)	20-30 Ottawa sand	Saturated	Undrained	Dense	0.5 and 0.6	2,900 to 12,750	200	Up to +38%
	Victoria sand	Saturated	Undrained	Dense	0.56	3,300 to 8,500	100	Up to +54%
	Arkansas sand	Saturated	Undrained	Dense	0.55	3,300 to 8,400	100	Up to +32%
Yamamoto and Lade (1993)	Uniform Cambria sand	Saturated	Undrained	Dense	89.5%	0.004 to 0.74	3400	+7%

Note: ¹With the increase of strain rate, + for increase and - for decrease; ²Calculated using a specimen height of 4 inches according to the original investigators.

Because of the capability of imposing large shear displacement, extensive research using the ring shear device have been conducted on evaluating the shear strain rate effect on the critical state shear strength or the shear strength at large strains of non-plastic, coarse-grained soils. For example, based on the results from constant-volume ring shear tests on Ottawa 20/40 sand, Sadrekarimi (2009) concluded that the imposed strain rate (4.7 to 111.6 cm/min, which corresponds to approximate shear strain rates from 180%/min to 4,300%/min considering that the sample was 2.6 cm in height) did not influence the shearing resistance and effective normal stress at the critical state. Other constant-volume ring shear tests (e.g., Novosad 1964; Scarlett and Todd 1969; Savage 1982; Sassa 1984, 1985; Hungr and Morgenstern 1984; Lemos 1986; Fukuoka 1991; Tika et al. 1996; Infante-Sedano 1998) reported similar observations.

In this paper, the authors explore the effect of shear strain rate on the undrained shearing resistance of a loose, clean sand (Ottawa F-65 sand) at relatively large shear strain while the sand is liquefied using constant volume direct simple shear (DSS) test. Unlike conventional laboratory tests, the constant-volume DSS tests reported here were performed in two stages. First, the specimens were loaded cyclically until they liquefied. Following initial liquefaction, the second stage involved undrained monotonic loading at either 0.1%/min or 10%/min shear strain rate. Companion monotonic tests also were performed on specimens prepared to the same initial state (density and consolidation stress) and sheared at the same shear strain rates. To limit any differences caused by other test variables, all tests described here were performed on Ottawa F-65 sand at 10% relative density after Ko-consolidation, D_{rc} , to an effective vertical stress, σ'_{vc} , of 200 kPa in constant-volume condition.

2 TESTING MATERIAL AND SAMPLE PREPARATION METHOD

Ottawa F-65 sand is commercially available from U.S. Silica. It is a white, inert, uniformly graded, silica sand with rounded to subrounded particles. Silica content is 99.7% (U.S. Silica 2016) and fines content (weight of particles passing No. 200 sieve) is typically very small (around 0.05%). Sand gradations from batch to batch. Figure 1 illustrates one representative grain size distribution. The specific gravity of the sand was 2.65 (ASTM D854-14). The maximum and minimum void ratio were determined to be 0.839 and 0.538 using the method proposed by Lade et al. (1998). Table 2 summarizes the sand index properties, which closely match those reported by Parra Bastidas (2016). Test specimen in this study were prepared by moist tamping because of its capability of forming very loose to loose reconstituted sand specimens (Ishihara 1993).

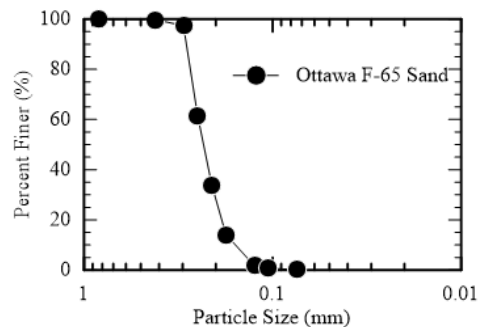


Figure 1. Grain size distribution of Ottawa F-65 Sand

Table 2. Index properties of the Ottawa F-65 sand

USCS	D_{50} (mm)	C_U	C_C	FC (%)	G_s	e_{max}	e_{min}
SP	0.23	1.56	1.0	< 1	2.65	0.839	0.538

Notes: USCS – Unified Soil Classification System; D_{50} – median particle size; C_U – coefficient of uniformity; C_C – coefficient of curvature; FC – fines content; G_s – specific gravity; e_{max} – maximum void ratio; e_{min} – minimum void ratio.

3 TESTING EQUIPMENT

Tests were carried out using the University of Illinois multidirectional direct simple shear (I-mcDSS) device (Bhaumik et al. 2017). This device allows load application in three independent axes, the vertical direction (z) and the two mutually orthogonal directions (x and y) in the horizontal plane (Figure 2). The control system allows both stress- and strain-controlled testing. Multi-stage tests also can be performed by setting various stress- or strain-controlled testing stages in the desired order.

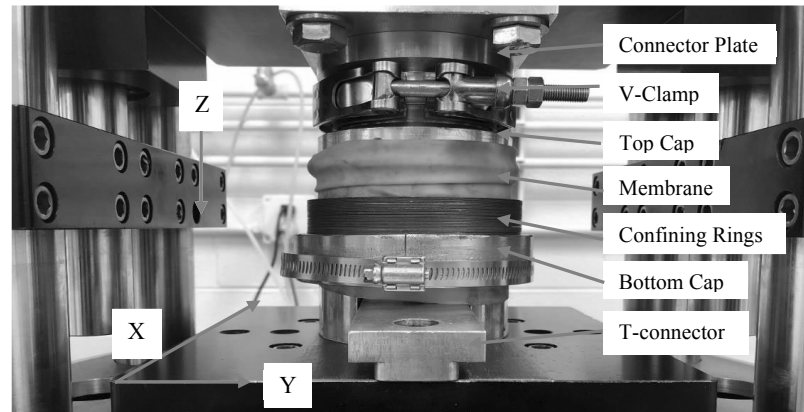


Figure 2. Specimen assembly on the University of Illinois multidirectional cyclic direct simple shear (I-mcDSS) device

4 DIRECT SIMPLE SHEAR TESTS

4.1 Initial condition

Two sets of DSS tests were completed in this study and test parameters are summarized in Table 3. As mentioned in the previous section, each set of DSS tests consisted of a two-stage test, a cyclic stage followed by a monotonic stage. A companion monotonic test was performed for each two-stage test. All two-stage and monotonic tests were performed under strain-controlled conditions and at identical shear strain rates. For Ottawa F-65 sand, a relative density of around 10% with an effective vertical stress of 200 kPa (Ko-consolidation) exhibited contractive behavior throughout constant volume monotonic DSS tests. Critical state conditions were defined when the shearing resistance and the inferred excess porewater pressure (interpreted using the change in the normal stress from constant volume DSS tests performed on dry specimens) became constant. These conditions suggest that the tendency for volume change was exhausted and a critical state was achieved.

Table 3. Summary of DSS tests performed in this study

Test No.	Test type	Monotonic shearing rate (%/min)	Drainage condition	Relative density after consolidation (%)	Consolidation stress (kPa)
1	Cyclic+Monotonic	10	Constant Volume	10.90	200
2	Monotonic			12.12	
3	Cyclic+Monotonic	0.1		13.75	
4	Monotonic			10.65	

4.2 Constant volume testing

In the current study, constant volume conditions were applied to mimic undrained conditions. Saturation is not needed in a constant volume test, and therefore, this protocol can greatly accelerate the testing process. In constant volume DSS testing, it is assumed that changes in applied vertical stress, as the specimen height is maintained constant during shearing, equals the excess pore pressure that would have been measured in an undrained test on saturated specimen with constant total vertical stress (Bjerrum and Landva 1966). The shear stress and the inferred porewater pressure response of the specimen during a constant volume DSS test were the same as those in the undrained saturated DSS tests (Iversen 1977, Dyvik et al. 1987). During the test, the vertical actuator was set to be fixed to maintain a constant specimen height. As the specimen was confined by the stacked rings, constant volume condition was achieved.

4.3 Cyclic and monotonic phases

The two-stage test started with a cyclic shearing stage and followed by a monotonic shearing stage. The cyclic stage was intended to liquefy the specimen and the following monotonic stage was aimed to evaluate the effect of strain rate on shearing resistance of the specimen when it was liquefied. In this study, the liquefaction caused by cyclic loading is defined as the condition that the inferred excess porewater pressure exceeds 95% of the initial effective vertical stress (σ'_{vc} in this case). For cyclic stages, trials proved that six sinusoidal cycles with a peak amplitude of 2% horizontal shear strain and a frequency of 0.1 Hz were sufficient to trigger liquefaction.

5 TEST RESULTS AND INTERPRETATION

Shear stress-shear strain and inferred porewater pressure responses for tests with different shear strain rates during the monotonic loading stages are illustrated in Figures 3 and 4. Stress paths for these tests are presented in Figure 5. In general, the Ottawa F-65 sand, Ko-consolidated to a relative density of around 10% under an effective vertical stress of 200 kPa, exhibits contractive behavior through the entire monotonic test. With that initial condition, the critical state was achieved within the displacement limit of the I-mcDSS device. The excess porewater pressures all become a constant value of around 167.3 kPa at large horizontal shear strains, confirming that there is no tendency for volume change. In addition, assuming the horizontal plane to be the plane of maximum stress obliquity at large shear strains (Wijewickreme et al. 2013), the mobilized friction angles at the end of the tests are all calculated to be around 32° (indicated in Figure 5), which is a typical value for the constant volume friction angle, ϕ'_{cv} , (friction angle mobilized at critical state, ϕ'_{cs}) for silica sand. This further confirms that a critical state was achieved in all tests.

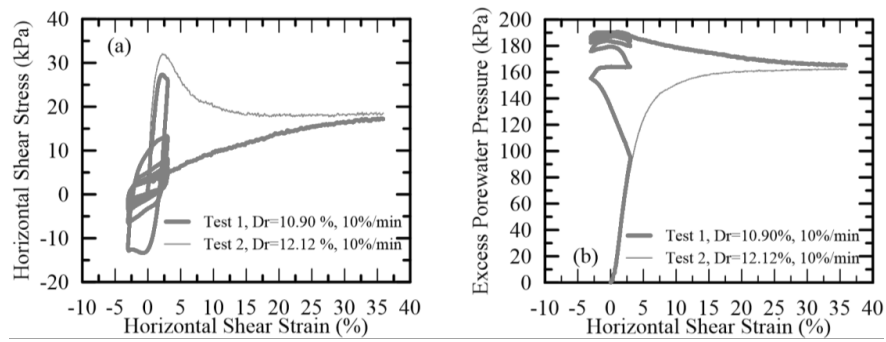


Figure 3. (a) Stress-strain response and (b) excess porewater pressure response for tests with 10%/min shearing rate.

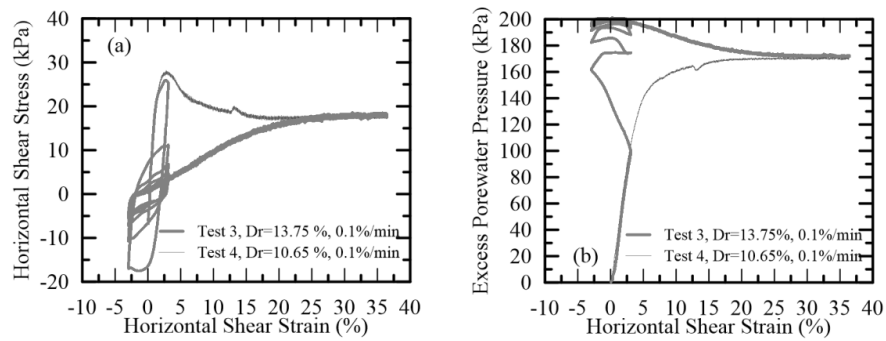


Figure 4. (a) Stress-strain response and (b) excess porewater pressure response for tests with 0.1%/min shearing rate.

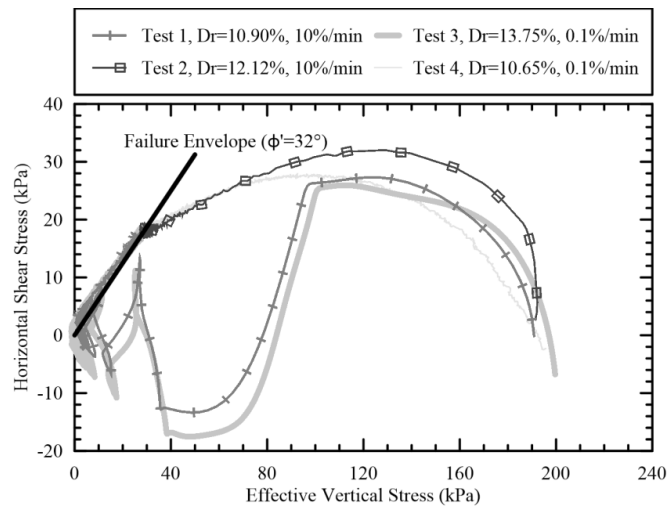


Figure 5. Stress paths for the DSS tests sheared at different strain rates

In the current study, differences in the shear strain rate indeed exhibited differences in yield shear strength (peak deviatoric stress), $s_u(\text{yield})$, under the initial state (density and consolidation stress) in constant volume DSS tests. Within the shear strain rates of 0.1%/min (Test 4) to 10%/min (Test 2), $s_u(\text{yield})$ increased by 14% from 28.1 kPa to 32.0 kPa. This increase agrees with results reported by Seed and Lundgren (1954) and Yamamuro and Lade (1993) for clean sands tested under undrained conditions.

Under a certain shear strain rate in a two-stage test, the monotonic stage (which followed the cyclic stage that liquefied the soil) exhibited the same critical state shear strength, $s_u(\text{critical})$, as that measured in the companion monotonic test (see Figures 3a and 4a). Similarly, the excess porewater pressures coincided in these tests (see Figures 3b and 4b). During the cyclic loading stage, as the cyclic loading proceeded, the effective stress path moved rapidly to the left as excess porewater pressure increased. After four cycles, the effective stress path started to oscillate along the effective stress failure envelope (ϕ'_{cs}), during which the effective stress path would pass through the origin. At the origin, the specimen is in an instantaneous state of zero effective stress, referred to as initial liquefaction by Seed and Lee (1966). If monotonic loading is applied at this condition, the specimen would dilate to its critical state shear strength, as indicated in Figures 3(a) and 4(a). This observation agrees well with tests reported by Ishihara et al. (1991).

In addition, the shearing resistance in all tests leveled off at about 18.3 kPa (Figures 3a and 4a). For practical purposes, the shearing resistance at the critical state for all tests (Tests 1, 2, 3, and 4) was the same. This value is quite consistent with DSS or Rotational Shear (RS) tests data reported by Olson and Mattson (2008) for contractive soils with σ'_{vc} of 200 kPa. The critical state (liquefied) shear strength ratio [$s_u(\text{critical})/\sigma'_{vc}$] for these tests is computed to be 0.09. This value is within the range of 0.01 to 0.16 reported by Olson and Mattson (2008) for DSS or RS tests. The results of these tests suggest that $s_u(\text{critical})$ of Ko-consolidated clean Ottawa F-65 sand ($D_{rc} \sim 10\%$, $\sigma'_{vc} = 200$ kPa) is independent of shear strain rate for shear strain rates of 0.1%/min and 10%/min in constant volume DSS tests. This corroborates the work of Novosad (1964); Scarlett and Todd (1969); Savage (1982); Sassa (1984, 1985); Hungr and Morgenstern (1984); Lemos (1986); Been et al. (1991); Fukuoka (1991); Tika et al. (1996); Infante-Sedano (1998) and Sadrekarimi (2009) who reported that shear strain rate does not influence the shearing resistance at the critical state.

As pointed out by Whitman and Healy (1962), sources of shear strain-rate effects consist of two candidate mechanisms under undrained conditions: (1) the friction angle is shear strain rate-dependent; or (2) the tendency to create excess porewater pressures is shear strain rate-dependent. The critical state of soil is defined as a state during which the mass of soil undergoes continued distortion at constant shear stress and constant volume (Schofield and Wroth 1968). This condition implies that there would be no tendency to create excess porewater pressure. And therefore, the tendency to create excess porewater pressure at the critical state would be rate independent. In addition, the friction angle at the critical state, ϕ'_{cs} , also would remain the same at different strain rates. The friction angle, ϕ' , consists of two components: (1) inter-particle sliding friction, ϕ'_μ ; and (2) geometrical interference, ϕ'_g . The geometrical interference can be expressed as the sum of ϕ'_d , which is produced by dilation or particle climbing, and ϕ'_p , caused by particle pushing and rearrangement (Rowe 1962, Rowe et al. 1964, Lee and Seed 1966, and Terzaghi et al. 1996). As the volumetric strain levels off at the critical state, the friction angle at the critical state, ϕ'_{cs} , depends only on ϕ'_μ and ϕ'_p because net dilation has ceased. The value of ϕ'_μ depends exclusively on surface micro-roughness and ϕ'_p typically is about 5° to 6° (Terzaghi et al. 1996). Therefore, for a certain material, ϕ'_{cs} is a constant, even the critical state was reached by shearing at different rates. As a conclusion, it is reasonable to expect that the $s_u(\text{critical})$ is independent of shear strain rate under controlled laboratory testing using specimens with the same initial states.

6 CONCLUSIONS

This paper describes laboratory tests conducted to evaluate the effect of shear strain rate on the shearing resistance of a liquefied clean sand. Results from two sets of constant volume DSS tests performed on loose Ottawa F-65 sand, each consisting of one two-stage test (a cyclic stage to trigger liquefaction followed by a monotonic stage) and one companion monotonic test are presented. The shear strain rate during the monotonic stages was varied to evaluate its influence on the critical state shear strength. These tests were designed to mimic flow displacements at

various shearing rates after triggering flow liquefaction. For clean sands with a relative density of about 10% after Ko-consolidation to an effective vertical stress of 200 kPa, the following conclusions can be drawn.

- (1) The specimens exhibited contractive behavior during all monotonic tests despite the difference in shear strain rates (0.1%/min and 10%/min).
- (2) The yield (peak) shear strength at the shear strain rate of 10%/min was about 14% greater than that at the shear strain rate of 0.1%/min.
- (3) Specimens reached critical state in both the two-stage tests and the monotonic tests.
- (4) Critical state shear strength given by the monotonic stage performed after liquefaction in a two-stage test was essentially identical to that measured on specimens at the same initial state tested in monotonic loading.
- (5) The critical state shear strength appears independent of shear strain rate for the shear strain rates (0.1%/min and 10 %/min), sand type, and initial state considered in this study.

It is acknowledged that the observations reported here that the critical state shear strength is independent of the shear strain rate for liquefied specimens applies only to the gradation used in this study (Ottawa F-65 sand) Ko-consolidated to a single effective vertical stress (200 kPa) in constant volume DSS tests sheared at two strain rates (0.1%/min and 10 %/min). Further evidence, especially with more gradations and more strain rates, is needed to confirm these findings.

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