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# Shearing Rate Effects on Dense Sand and Compacted Clay

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**ABSTRACT**
This paper reports the findings of a series of triaxial compression tests performed on saturated specimens of compacted clay and dense sand at axial strain rates ranging from 0.01 to 800 %/minute. The purpose of these tests is to provide a baseline for the shear strength of near surface soils during buried explosions, and to evaluate the transition from conventional quasi-static experimental soil mechanics to faster rate testing. The behavior of the saturated clay and sand specimens under undrained loading is examined in detail using pore water pressure measurements at the boundary, which permits evaluation of the impact of the rate of loading on the effective stress paths during shearing.

**KEYWORDS:** Axisymmetric triaxial testing; Soil shear strength; Strain rate, Blast loading, Experimental soil mechanics

## INTRODUCTION
The impact of high axial compression rates on the shear strength of soils has been a topic of interest for many years. This information is important to understand and simulate dynamic loading of soils in events such as explosions and earthquakes, as well as in rapid construction loading. Thus, a thorough understanding of the behavior of soil subject to high strain rates is an important factor. However, the effects of high strain rates on the shear strength and excess pore water pressure of compacted clay and dense sand under saturated conditions have not been thoroughly investigated in terms of effective stress. The effect of loading rate was investigated in this study by performing consolidated undrained (CU) triaxial tests on two soils under the same effective confining stress. The specimens were sheared using conventional axial strain rates depending on the rate of consolidation (0.01 to 0.1 %/min), as well as under axial strain rates up to 800 %/min.

## BACKGROUND
It has been well established in several classic studies that the shear strength of remolded clays is dependent upon the axial strain rate during axisymmetric triaxial compression testing (Casagrande and Shannon [7], Richardson and Whitman [12]; Olson and Parola [10]; Lefebvre and LeBoeuf [9]; Zhu and Yin [15]). Specifically, the shear strength of clay at failure increases with increasing strain rate. The average rate of increase for soil under normally consolidated stress state is 10% per log cycle of the time to failure, where failure is often defined as 15% axial strain. Casagrande and Shannon [7] were the first to observe an increase in shear strength at higher strain rates. Richardson and Whitman [12] expanded this research by incorporating pore water pressure measurements at the boundaries of the specimens. They verified the hypothesis that the increase in shear strength with increasing strain rate was due to a lower magnitude of pore water pressure at failure. Lefebvre and LeBoeuf [9] investigated the effects of changing the initial stress state by conducting tests on both intact overconsolidated samples and remolded normally consolidated specimens of the same clay. Zhu and Yin [15] looked at changes in soil behavior with increasing strain rate as the over consolidation ratio changed. Olson and Parola [10] were one of the only researchers who investigated the behavior of compacted clay soils. However, they only considered the effect of strain rate on shear strength when varying the initial compaction water content, and did not consider saturated conditions. They were not able to monitor generation of excess pore water pressure during shearing. Accordingly, the impact of the excess pore water pressures on the behavior of saturated compacted clays subject to increased strain rates have not yet been thoroughly addressed in the literature.

Most of the classic papers on the impact of axial strain rate on the shear strength of sands focused on dry conditions, and it is not until recent years that saturated conditions have been considered. The shear strength response of sands subject to increased loading rates is quite variable depending upon the saturation conditions as well as the density of the test material. Casagrande and Shannon [7] and Dayal and Allen [8] looked at the effect of strain rate on the shear strength of dry sands. Both studies observed that the shear strength of dry sands is relatively insensitive to the axial strain rate (or penetration rate). Yamamuro and Abrantes [14] investigated the effect of strain rate on saturated medium dense sand under drained triaxial compression, and observed that the shear strength of the sand increased by nearly 50% from a strain rate of 0.0024 %/second to 1764 %/second. Whitman [13] evaluated the shear strength of loose saturated sand, and observed an increase in shear strength of up to 200% due to decreases in excess pore water pressure. This indicates that the faster shearing rates lead to dilation. Shearing of loose sands under slow axial strain rates typically leads to contraction and positive pore water pressure, which means that a transition in behavior of saturated sand behavior occurs with faster loading rates. The mechanisms of this transition are discussed in detail by Omidvar et al. [11]. Despite the previous studies on saturated loose and medium dense sands, the shear strength behavior and excess pore water pressure generation of saturated dense sands during high strain rate triaxial compression tests has not been fully investigated in the literature and is a focal point of this paper.

## MATERIALS

Two natural soils, a sand and clay obtained from the region around Boulder, Colorado were chosen as test materials for this study. The clay was obtained from a stockpile of soil from a construction site on the University of Colorado Boulder campus, and is referred to as Boulder clay. The clay was processed after collection to remove all particles greater than the #10 sieve, which provided a more homogeneous and consistent material for experimental testing. The sand was purchased from a quarry in Longmont, Colorado (Colorado Materials), and is referred to as Mason sand.

Soil classification tests were performed on Mason sand to measure the grain size distribution (ASTM D422 [1]), specific gravity (ASTM D854 [3]), maximum dry density, and minimum dry density. Relevant grain size distribution parameters ($D_{10}$, $D_{30}$, $D_{50}$, $C_u$, $C_z$) and are listed in Table 1. The sand has negligible fines content. Based on the grain size distribution curve, Mason sand is classified as poorly graded sand (SP) according to the Unified Soil Classification System (USCS). The specific gravity $G_s$ was measured to be 2.62. The minimum void ratio of 0.50 corresponds to a maximum dry density of 1.74 kg/m$^3$, while the maximum void ratio of 0.78 corresponds to minimum dry density of 1.47 kg/m$^3$. Soil classification tests were also performed on Boulder clay to measure the grain size distribution (ASTM D422 [1]), Atterberg limits (ASTM D4318 [5]), specific gravity (ASTM D854 [3]), standard Proctor compaction curve (ASTM D698 [2]), and compression curve and consolidation characteristics (ASTM D2435 [4]). Relevant grain size distribution parameters ($D_{10}$, $D_{30}$, $D_{50}$, percent fines) and soil index properties (liquid limit LL and plasticity index PI) are listed in Table 1. Boulder clay is classified as a low plasticity clay (CL) according to the Unified Soil Classification System (USCS). The specific gravity $G_s$ was measured to be 2.70. From the standard Proctor compaction test, an optimal water content of 18% corresponds to the maximum dry unit weight of 17.5 kN/m$^3$. An oedometer test was performed on a specimen of Boulder clay compacted to at optimal conditions then soaked in the oedometer to reach as close to saturated conditions as possible. The results from the test indicate that the clay has an apparent preconsolidation stress of 300 kPa, a compression index $c_c$ of 0.23, and a recompression index $c_r$ of 0.041.

**Table 1: Index properties of Mason sand and Boulder clay**

| Mason Sand | | | Boulder Clay | | |
|---|---|---|---|---|---|
| Property | Value | Units | Property | Value | Units |
| $D_{10}$ | 0.20 | mm | $D_{10}$ | $< 1.7\times10^{-4}$ | mm |
| $D_{30}$ | 0.44 | mm | $D_{30}$ | < 0.001 | mm |
| $D_{60}$ | 0.90 | mm | $D_{50}$ | 0.001 | mm |
| $C_u$ | 4.5 | - | % Fines | 100 | % |
| $C_z$ | 1.1 | - | LL | 43 | - |
| $G_s$ | 2.62 | - | PI | 22 | - |
| $e_{min}$ | 0.50 | - | $G_s$ | 2.70 | - |
| $\rho_{max}$ | 1.74 | kg/m$^3$ | $\gamma_{d\ max}$ | 17.5 | kN/m$^3$ |
| $e_{max}$ | 0.78 | - | $w_{opt}$ | 18 | % |
| $\rho_{min}$ | 1.47 | kg/m$^3$ | | | |

## CONVENTIONAL TRIAXIAL TESTING

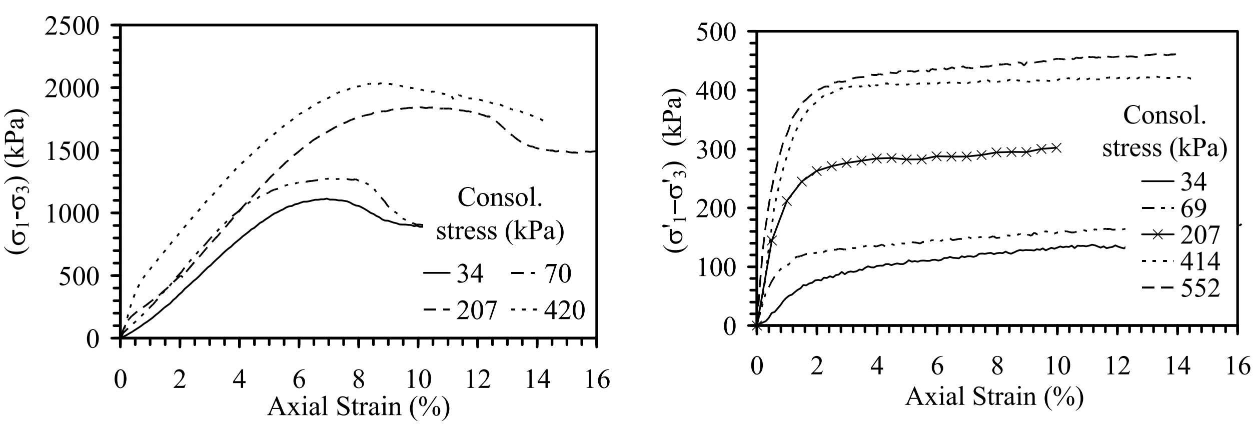
Five consolidated undrained (CU) triaxial tests were conducted in accordance to ASTM D4764 [7] to determine the effective shear strength parameters of Boulder clay. Similarly, a series of four consolidated undrained (CU) triaxial compression tests were performed on Mason sand using the same testing standard. Each clay specimen was compacted using a mechanical press in a cylindrical mold having a height of 71.1 mm and a diameter of 35.6 mm. To ensure uniformity throughout the sample, each specimen was compacted using five lifts of equal mass. This method of compaction is referred to as "static compaction". The target dry unit weight and water content for each specimen were 17 kN/m$^3$ (target void ratio of 0.51) and 17.5% respectively. These values correspond to 0.6% of the maximum standard Proctor dry density and 10% dry of the standard Proctor optimum water content, respectively. After the specimen was prepared and the cell was assembled, the specimen was placed under a vacuum of 80 kPa and the cell filled with water. The dense sand specimen was prepared by placing the bottom platen of the triaxial cell on a shaking table, and pouring sand into the latex membrane fitted on the inside of a split mold. The dense sand samples were prepared using a mechanical vibrator, and the sand was vibrated in three lifts within the split mold until reaching a target void ratio of 0.54. The relative density corresponding to this target void ratio can be calculated as follows:

$$I_D = \frac{(e_{\max} - e)}{(e_{\max} - e_{\min})} \tag{1}$$

A value of $I_D$ of 0.88 corresponds to the target void ratio, indicating that the sand is dense. After vibration, the top cap was then placed on the sand specimen, the membrane was attached, and the specimen was placed under a vacuum of 80 kPa.

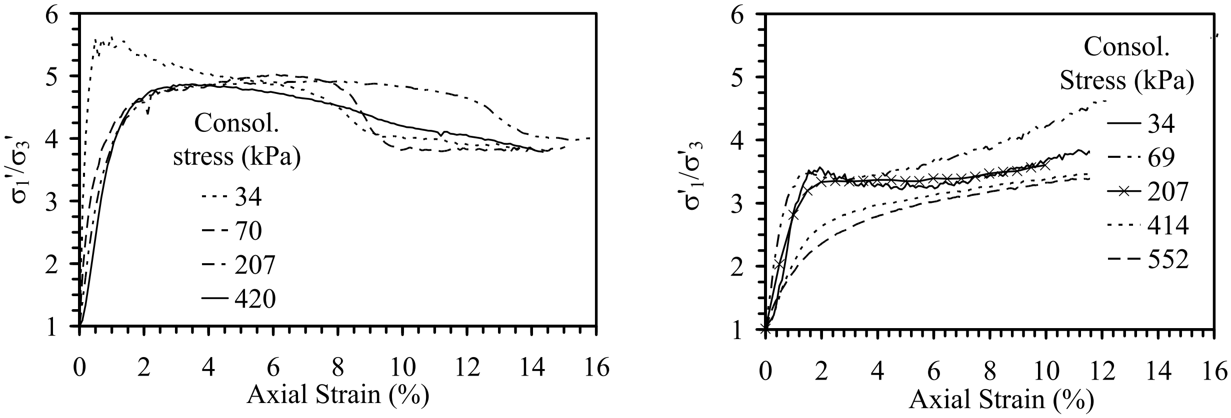
The saturation procedures for both soil types were similar. After specimen preparation and filling of the triaxial cell with water, the unsaturated specimens were permitted to de-air under vacuum for at least 1 hour. A seating cell pressure of 20 kPa was then applied to the cell water. Next, de-aired water was permitted to flush through the specimen from the bottom while vacuum was maintained on the top. After water was observed to exit from the top of the specimen, the water vapor was flushed from the top of the specimen and the sample was placed under backpressure. Specifically, the cell pressure and the pressure applied to the water within the specimen were increased in stages until reaching a measured value of Skempton's B parameter of 0.9, or when the B parameter remained constant with additional increases in backpressure. In most tests, a cell pressure of 483 kPa and a backpressure of 276 kPa were applied at the end of saturation.

After saturation of each specimen, a specific effective stress was applied and consolidation was permitted to occur until the volume change inferred from the cell water level and backpressure water levels reached equilibrium. Upon completion of consolidation, the specimens of Boulder clay were sheared to an axial strain of 15% in 150 minutes (an axial strain rate of 0.0686 mm/min), and the specimens of Mason sand were sheared to an axial strain of 15% in 20 minutes (an axial strain rate of 1.28 mm/min). These times to failure (and corresponding shearing rates) were defined using the value of $t_{50}$ for the soil specimens calculated from the consolidation data (ASTM D2435 [7]). For all Mason sand tests, the load was applied using a hydraulic press. The load for the Boulder clay was applied using an electrically driven motor for slower tests and the hydraulic press for higher rates. All sand and clay tests were run under strain-controlled conditions. During shearing, the variables measured include the pore water pressure at the bottom of the specimen, the axial load, and the axial displacement. From this data, the shear strength (in terms of principal stress difference), principal stress ratio, and excess pore water pressure were calculated. The shear strength as a function of axial strain is shown in Figure 1(a) for Mason sand and in Figure 1(b) for Boulder clay, the principal stress ratio as a function of axial strain is shown in Figure 2(a) for Mason sand and Figure 2(b) for Boulder clay, and the excess pore water pressure as a function of axial strain is shown in Figure 3(a) for Mason sand and Figure 3(b) for Boulder clay.



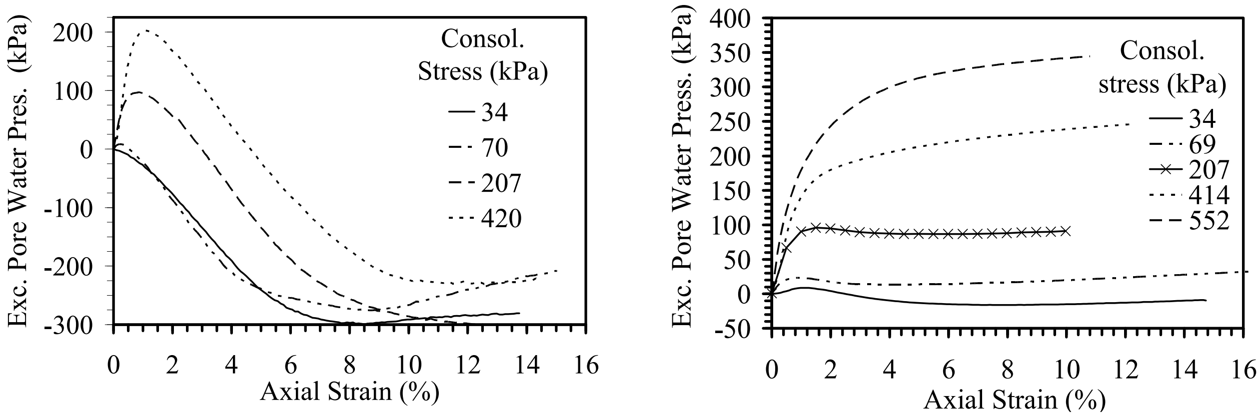

(a) (b)

**Fig. 1 Principal stress difference versus axial strain for conventional loading rate: (a) Mason sand; (b) Boulder clay**




(a) (b)

**Fig. 2 Principal stress ratio versus axial strain under conventional loading rate: (a) Mason sand; (b) Boulder clay**




(a) (b)

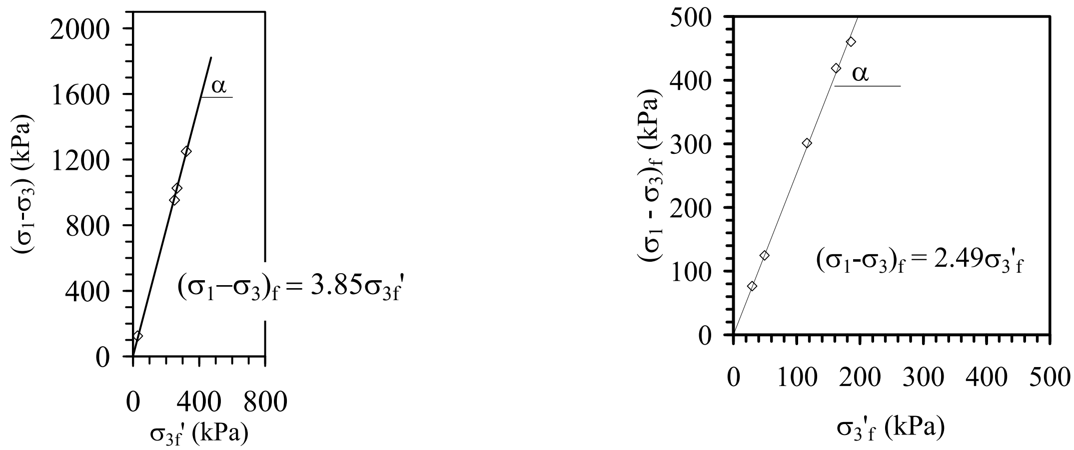
**Fig. 3 Excess pore water pressure versus axial strain for conventional loading rates: (a) Mason sand; (b) Boulder clay**

In each of the tests, shear failure of the specimen was defined as the point where the maximum value of internal friction is mobilized, which is referred to as the stress path tangency failure criterion. In a consolidated undrained triaxial compression test, the point where the maximum friction is mobilized occurs at the maximum value of the principal stress ratio, $\sigma_1'/\sigma_3'$. Examination of a Mohr circle at failure indicates that the principal stress ratio is directly proportional to the friction angle [$\sigma_1'/\sigma_3' = \tan^2(45+\phi'/2)$]. The points of failure defined using the stress path tangency failure criterion in a modified Mohr-Coulomb stress space (triaxial stress space: effective confining stress versus principal stress difference) for specimens consolidated to different initial effective consolidation stresses are shown in Figure 4(a) for Mason sand and 4(b) for Boulder clay. The failure envelopes in triaxial stress space can be defined by fitting a line through the failure points. The angles of inclination of the lines shown in the modified Mohr-Coulomb diagrams in Figure 4 correspond to the tangents of the transformed friction angle, α, while the y-intercepts correspond to the transformed apparent cohesion values, d. The following equations were used to convert the modified Mohr-Coulomb parameters of the failure envelope to the conventional Mohr-Coulomb failure envelop parameters, as follows:

$$\phi' = \sin^{-1}\left(\frac{\tan(\alpha)}{2+\tan(\alpha)}\right) \tag{2}$$

$$c' = \frac{d\left[1-\sin(\phi')\right]}{2\cos(\phi')} \tag{3}$$

where ϕ′ is the effective friction angle and c′ is the apparent cohesion. The values of α and d calculated from the data in Figure 4 as well as the Mohr-Coulomb parameters calculated using Equations 2 and 3 are summarized in Table 2.




(a) (b)

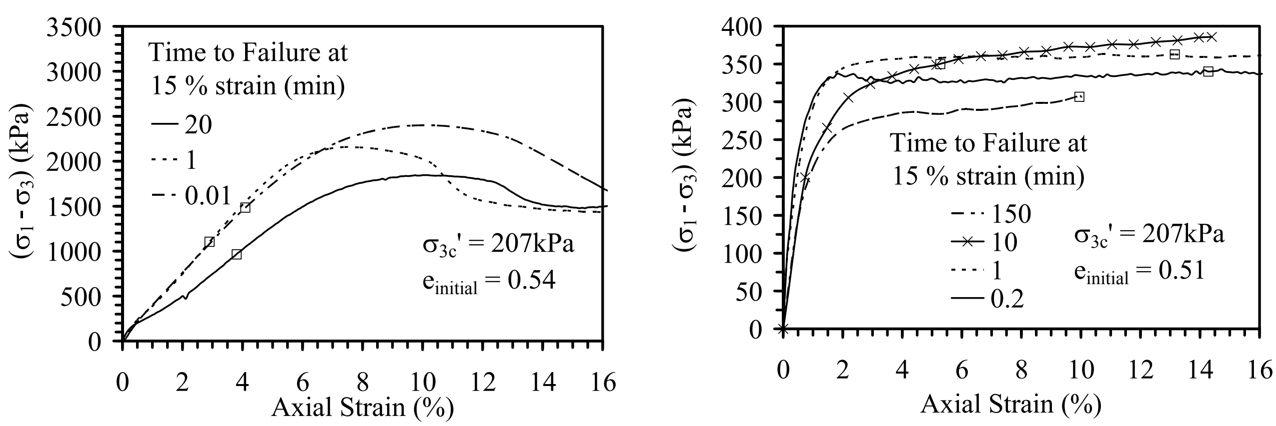
**Fig. 4 Failure envelopes defined using the stress path tangency failure criterion: (a) Mason sand; (b) Boulder clay**

**Table 2 Modified and conventional Mohr-Coulomb failure envelope parameters for Mason sand and Boulder clay**

| Mason Sand | | | Boulder Clay | | |
|---|---|---|---|---|---|
| Parameter | Value | Units | Parameter | Value | Units |
| α | 75.3 | º | α | 68.5 | º |
| d | 0.0 | kPa | d | 0.0 | kPa |
| ϕ' | 41.0 | º | ϕ' | 34.0 | º |
| c' | 0.0 | kPa | c' | 0.0 | kPa |

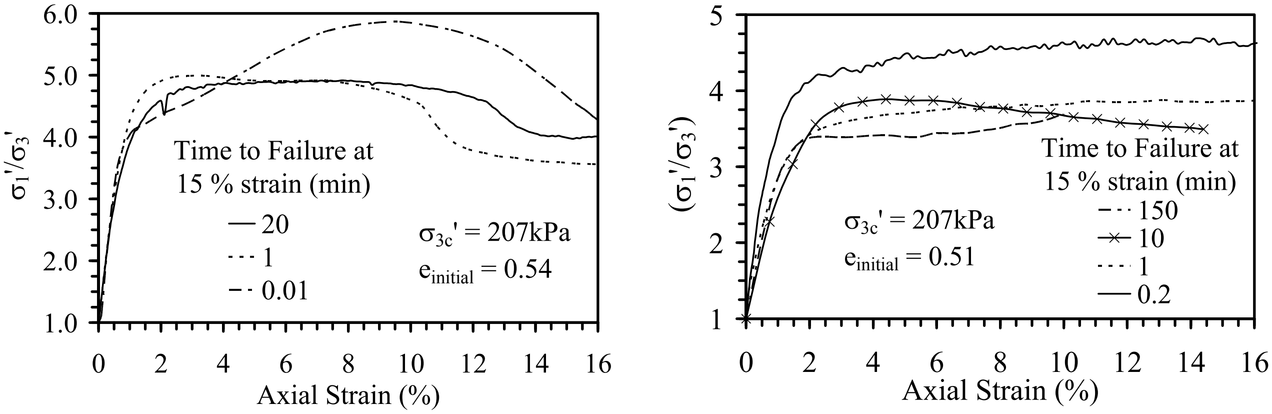
**RATE EFFECTS**

To investigate the effects of strain rate on the shear strength of Boulder clay and Mason sand, additional consolidated undrained triaxial tests were preformed at increased loading rates. Tests were performed on Boulder clay at four different axial strain rates: 0.1, 1.5, 14 and 100 %/minute, and three tests were preformed on Mason sand at different axial strain rates: 0.9, 16, and 800 %/minute. Each specimen was prepared and saturated using the same procedures discussed in the previous section for conventional consolidated undrained triaxial tests. All specimens were sheared until reaching an axial strain of 15%, so the shearing rates correspond to times to failure of 150, 10, 1, and 0.2 minutes for Boulder clay and times to failure of 20, 1, and 0.01 minutes for Mason sand. Similar to the standard tests, the variables measured during shearing include the excess pore water pressure at the bottom of the specimen, axial load, and axial displacement. The variation of shear strength (in terms of the principal stress difference), principal stress ratio and excess pore water pressure with axial strain for the Mason sand are shown in Figures 5(a), 6(a), and 7(a), respectively, while those for Boulder clay are shown in Figures 5(b), 6(b), and 7(b), respectively. The shear strength at failure was defined using stress path tangency criterion, and the points of failure are indicated in Figures 5 through 7 with hollow squares.



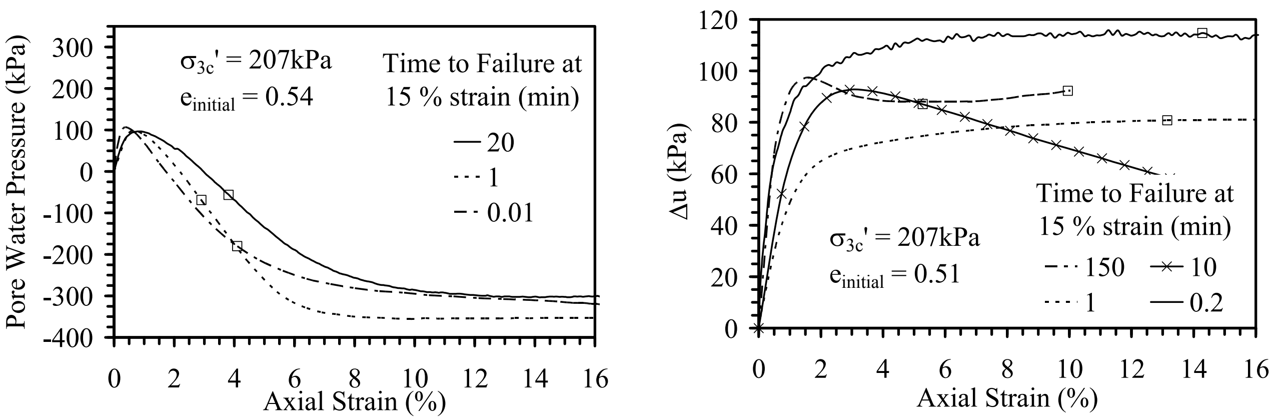

(a) (b)

**Fig. 5 Principal stress difference with axial strain for different times to failure: (a) Mason sand; Boulder clay**




(a) (b)

**Fig. 6 Principal stress ratio with axial strain for different times to failure: (a) Mason sand; (b) Boulder clay**




(a) (b)

**Fig. 7 Excess pore water pressure with axial strain for different times to failure: (a) Mason sand; (b) Boulder clay**

**ANALYSIS**

The shear strength (principal stress difference) at failure defined by the stress path tangency failure criterion for both soils is plotted versus logarithm of strain rate in Figure 8(a). The shear strength of Mason sand during undrained shearing increases nonlinearly, but for low rates of shearing the rate of increase is approximately 15% per log cycle of time to failure. Compared to the results from previous studies (Yamamuro and Abrantes [14]; Whitman [13]), it appears that the increase of shear strength with strain rate of saturated dense sand is not as pronounced for dense sand as it is for medium dense and loose sand. With exception of the test on Boulder clay having a time to failure of 0.2 minutes, it is clear that the shear strength for this soil increases log-linearly with increasing strain rate. The percent increase in shear strength for the Boulder clay is approximately 9% per log cycle of time to failure. This rate of increase in shear strength is consistent with previous studies conducted on normally consolidated, remolded clays by Casagrande and Shannon [7], Richardson and Whitman [12] and Lefebvre and LeBoeuf [9].

The values of excess pore water pressure at the point of failure defined by the stress path tangency failure criterion for both soils under different rates of shearing are shown in Figure 8(b). The Mason sand data indicates a negative excess pore water pressure that decreases with increasing strain rate. This suggests that the increase in shear strength of the Mason sand is due to increasing amounts of dilation. With exception of the fastest test, the excess pore water pressure at failure for Boulder clay is positive, and decreases with increasing axial strain rate. This observation is consistent with studies conducted by Richardson and Whitman [12] and Lefebvre and Leboef [9]. Both studies found that for normally consolidated, remolded clay the shear strength increased with increasing strain rate, and this strength increase was accompanied by a decrease in pore water pressure. Thus, it is believed that the increase in shear strength of the Boulder clay with increasing strain rate is due to an increase in effective stress caused by the decrease in pore water pressure.

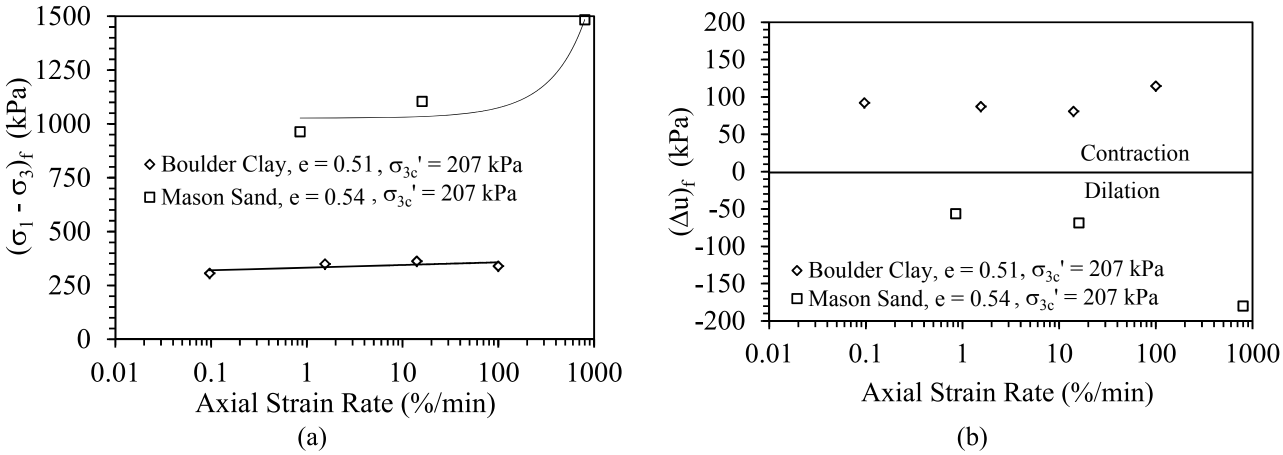



**Fig. 8 Impact of strain rates on Mason sand and Boulder clay: (a) Shear strength; (b) Excess pore water pressure**

## CONCLUSION

The results presented in this study emphasize the importance of monitoring the pore water pressure response during triaxial compression tests on soils under high rates of strain. Both sands and clays were observed to have an increase in shear strength with increasing axial strain rates, although the rate of increase in the shear strength of sand was greater due to the negative excess pore water pressure induced by dilation during shearing.

## ACKNOWLEDGEMENTS

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