Partially Drained Responses of Dense Sand under Monotonic Simple Shear

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ABSTRACT

Marine structures placed in the shallower seabed can experience pore water drainages with more complexity than those in onshore environments, particularly in coarse-grained soils where drainage is neither purely “drained” nor “undrained,” but Partially Drained (PD). However, current laboratory approaches for characterizing soil behavior are limited to modeling drainage conditions as fully drained or undrained. This paper presents results from a series of confined monotonic saturated simple shear tests under various drainage conditions on reconstituted medium dense to dense Monterey sand specimens to fill this knowledge gap. Although others have performed limited PD element-level tests under triaxial conditions, no documentation exists for tests using a simple monotonic shear configuration. To achieve PD, a special filter was fabricated and connected between the bottom of the specimen and the backpressure controller. The hydraulic filter comprises a series of needle valves to provide various hydraulic impedances. All simple shear tests in this paper were backpressure-saturated. Two different degrees of PD were considered and compared with fully drained and undrained conditions. Results show that the excess pore water pressure generation and measured volumetric changes in the PD tests are bounded between those measured from fully drained and undrained, proving the PD filter provided the hydraulic resistance to achieve PD condition.

INTRODUCTION

For decades, improvements in soil testing have aided the geotechnical engineering community in the goal of ethically serving the public with safe designs and long-service life structures. One of the critical contributions of these improvements is the understanding that the effective stress path of saturated soils, which impacts the stress-strain behavior, is governed by the distribution in space of total stress and pore pressure, as well as their changes over time. At the same time, we understand that drainage boundary conditions play a crucial role in the changes in total stress and pore pressure (Vaid & Eliadorani, 2000). Typical element-level soil testing is often limited to two boundary conditions: drained and undrained. The undrained condition attempts to mimic a loading event in which a lag between stress changes and volumetric changes occurs, resulting in excess pore pressure generation. The drained condition simulates a gradual loading event, leading to simultaneous changes in stress and volume, resulting in no excess pore pressure generation. For simplicity and due to testing limitations, an undrained condition can often be related to a dynamic loading like an earthquake in which dynamic excess pore pressure generation occurs. Since the tests performed in this investigation are under static loading, the changes in pore pressure will be referred to as excess pore pressure. Unfortunately, on-site conditions are more complex, with excess pore pressure generation and volumetric changes co-occurring (Suzuki et al. 2020). In the geotechnical engineering community, this condition is referred to as partially drained. Foundations for marine structures like those for offshore wind turbines are known to be sensitive to drainage conditions and expected to undergo partial drainage conditions during dynamic loading (Shen et al. 2022; Suzuki et al. 2020). In recent years, with the desire to increase sustainable resources, more offshore wind turbines are shifting toward deeper waters, increasing the demand for testing that accurately represents in situ conditions.

In the past two decades, attempts have been made to simulate partial drainage conditions in laboratory settings. Vaid and Eliadorani (2000) used multiple volumetric to axial strain ratios to induce constant water injection/extraction during triaxial testing on Fraser Delta sand to achieve partial drainage conditions, concluding that shear stiffness increases as the injection ratio increases and decreases as the extraction ratio increases in comparison to undrained conditions. Furthermore, Adamidis et al. (2019) performed similar testing on Hostun sand, obtaining similar results. Using available data from drained and undrained simple shear testing on Toyoura sand, Kamai and Boulanger (2012) modified and recalibrated a constitutive numerical model to simulate element testing under partial drainage boundaries, including settings that imitated those used in the tests by Vaid and Eliadorani (2000) obtaining similar trends. Unfortunately, constant water injection and extraction ratios do not directly correlate with field conditions where a mixture of negative and positive volumetric changes can take place. To better simulate field conditions, Suzuki et al. (2020) studied the effects of partially drained conditions on triaxial testing using filter devices to restrict the water flow during shearing, concluding that partially drained samples behave similarly to an undrained test followed by drained conditions once reaching peak shear stress. The flow rate of each filter was determined to influence the sample's behavior successfully, causing it to resemble a drained test at higher flow rates and an undrained test at lower flow rates. Furthermore, the peak shear stress, peak excess pore pressure, and peak volumetric changes were observed simultaneously under partial drainage. Although there have been advancements in soil testing under partially drained conditions, a comprehensive literature review indicates no documentation of simple shear testing under partial drainage boundaries.

This paper aims to determine how partial drainage affects samples tested under confined monotonic simple shear (CMSS). Unlike the coil-based filters used by Suzuki et al. (2020), this research uses a different type of filter assembly made of needle valves (Norwegian Geotechnical Institute 2020 & Yamamoto et al. 2009). Needle valves allow the water flow to be adjusted without the need to interchange parts in the filter. Additionally, the needle valves have precise openings, enabling the tests to be repeated at consistent flow rates. Therefore, the approach presented in this investigation can help provide the geotechnical engineering community with new insights to get a step closer to laboratory testing that accurately reflects in situ conditions.

EQUIPMENT

Confined Multi-Directional Dynamic Cyclic Simple Shear Apparatus

Monotonic simple shear tests were conducted using a Confined Multi-Directional Dynamic Cyclic Simple Shear (CMDSS) device at Cal State LA (Figure 1). The apparatus incorporated two independent electronic shear actuators (x and y-axis) with a travel range of ±20 mm, allowing for unidirectional and bi-directional cyclic and monotonic simple shear testing (Figure 2A). In this study, samples undergo monotonic unidirectional shearing. 1MPa back and cell pressure controllers were attached to the CMDSS device (Figure 1), allowing the testing of saturated sand samples. As shown in Figure 2, a metal chamber provided a closed system, allowing saturated samples to be confined. Cell and pore pressure transducers permitted pressure tracking in the CMDSS chamber and sample with a ±0.15% accuracy. A six-axis load cell placed as close as possible to the specimen's top cap aided in measuring and controlling the normal and shear loads experienced by soil specimens during testing, with a 10kN and 5kN capacity, respectively, with a ±0.1% accuracy. A third actuator (Z-axis) with a travel range of ±25 mm was commanded to maintain a constant normal load during shearing. Linear Variable Differential Transformers (LVDT) in the X, Y, and Z axes with capacities of 9 mm, 9 mm, and 5 mm, respectively, aided in measuring and controlling the displacement during the testing with a 0.1µm resolution.

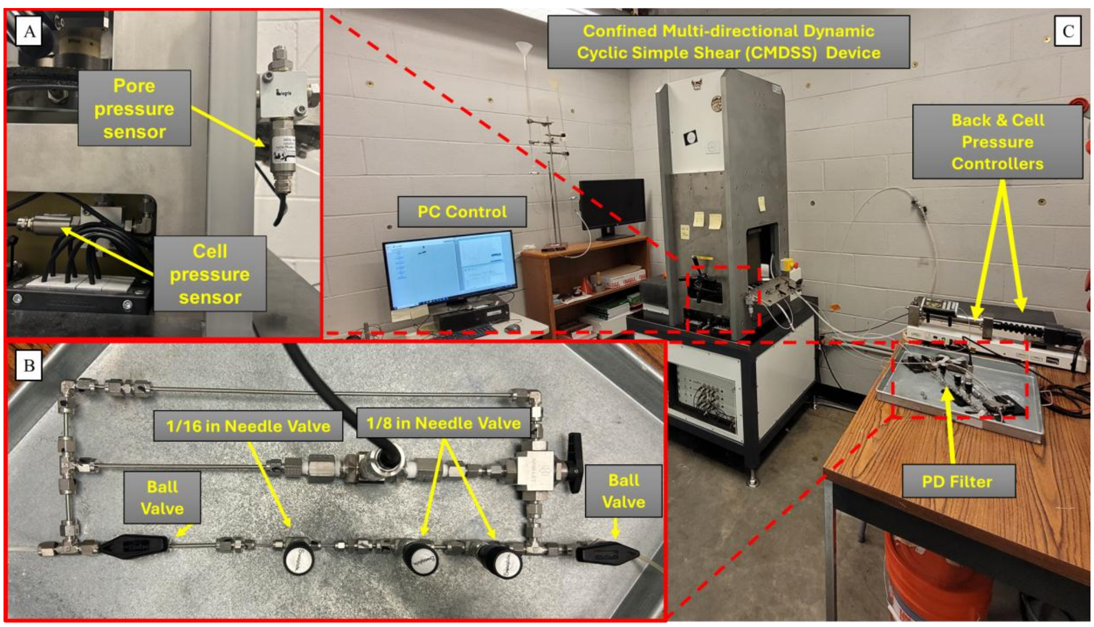

Figure 1. Setup of Cal State LA Confined Multi-Directional Dynamic Simple Shear Device

Partial Drainage Filter Assembly

A custom-made partial drainage (PD) filter was developed using three needle valves to restrict the flow rate into and out of the sample and achieve partially drained DSS testing. As shown in Figure 1B the PD filter was connected in series between the CMDSS device and the back pressure controller. The filter was designed and built after a similar device with one needle valve from the Norwegian Geotechnical Institution (2020). All low-pressure 316 stainless steel needle valves used were from the Swagelock Metering Valves S-Series. Each valve was equipped with a vernier handle, providing gradation accuracy to 1/25th of a turn to ensure repeatable flow adjustments during testing. Iterations on the number of valves needed to reduce the flow rate capacity of the filter were done using 1, 2, and 3 needle valves in series. As shown in Figure 3A, a pressure panel induced and maintained differential pressures across the filter (69 kPa, 138 kPa, 276 kPa, and 414 kPa), generating a flow from the inlet accumulator across the PD filter into the outlet accumulator where volume changes were tracked over time. The filter with one 1/8-inch needle valve (Swagelok #SS-SS2-VH) was tested at needle valve openings of $1/25^{th}$, 2, and 5 turns in the vernier handle to determine how pressure and needle valve opening affected the filter's total flow rate. The flow rate results for the $1/25^{th}$ and 5 turn openings are shown in Figures 3B and 3C, where a linear correlation between the flow rate and the differential pressure can be observed. It can also be noticed that a reduction in the valve opening significantly affected the device's flow rate capacity, indicating the ability to have an adjustable PD filter. A filter with a second 1/8-inch needle valve in a series was also tested, observing similar correlations. In addition, the flow rate capacity was decreased to at least half when the PD filter configuration shifted from 1 valve to 2 valves, with the decrease continuing throughout the different valve openings. The decrease in the flow rate capacity was also observed to increase in magnitude at higher differential pressures. A third 1/16-inch needle valve (Swagelok #SS-SS1-VH) was added in series, and minimal impact was observed on the flow rate. From these observations, it was deduced that the addition of any other valves would have an insignificant effect on the PD filter flow rate capacity.

MATERIAL

Monterey sand samples were used to conduct CDSS tests. Monterey sand was selected for testing because of its grain size. The larger grain size of this poorly graded sand allows for a structure with more volumetric voids for water to leave and/or enter the sample, providing better opportunities to observe the effects of partial drainage. Figure 4 shows the results from the particle size analyses performed according to ASTM D422 for Monterey sand. Material properties such as minimum and maximum unit weight were found according to ASTM D425 and D253. The classification by ASTM D2487 Unified Soil Classification System (USCS) can also be found in Figure 4.

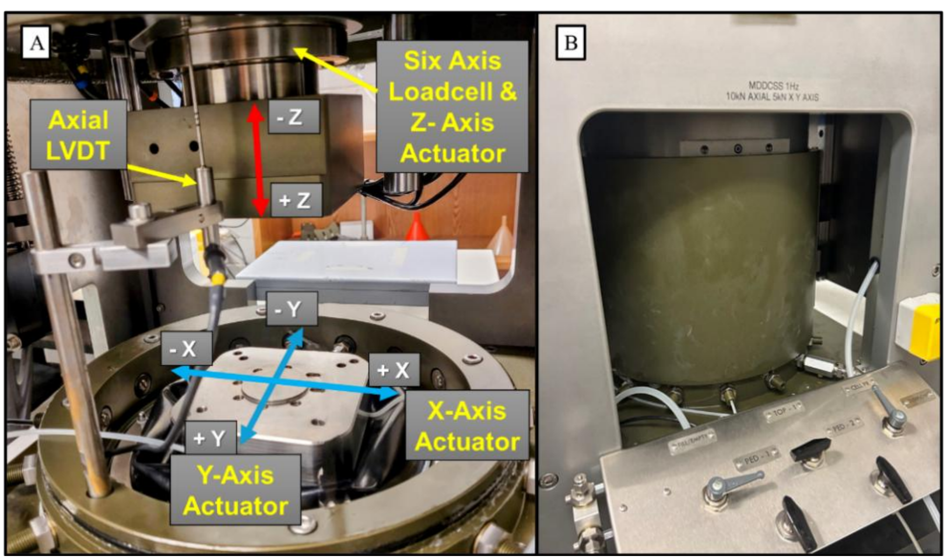

Figure 2. (A) Shake Table Components (B) Confinement Chamber for Back Pressure

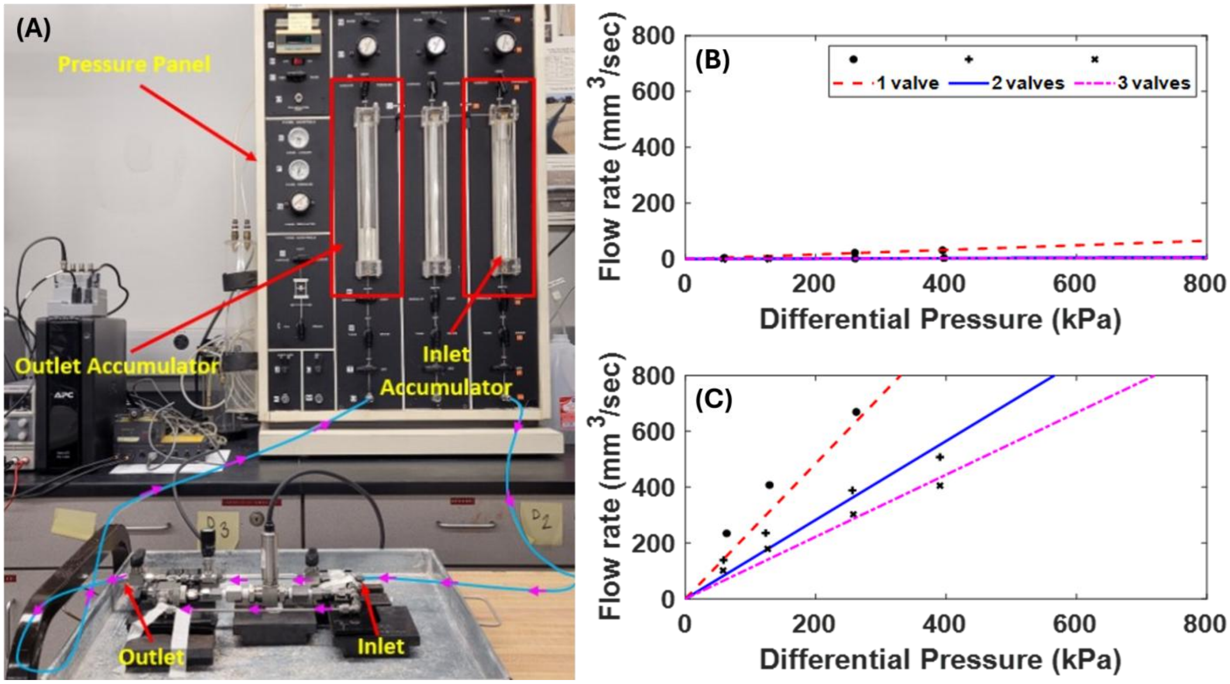

Figure 3. (A) PD Filter Testing (B) PD Filter flow rate capacities at 1/25th turn valve opening (C) PD Filter flow rate capacities at 5-turn valve opening

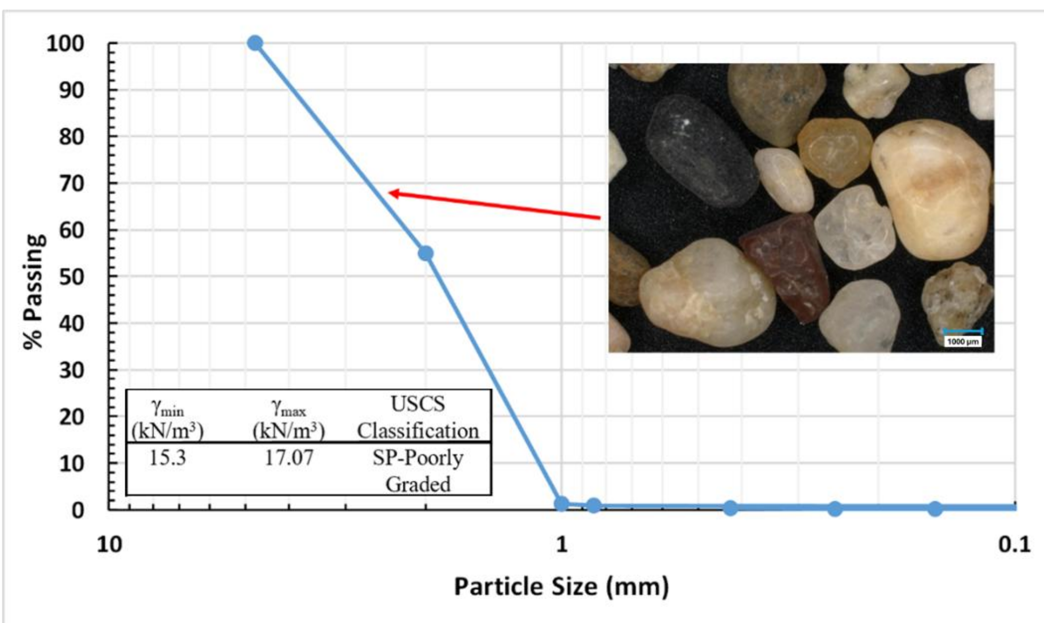

Figure 4. Monterey Sand Grain Size Distribution & Index Properties

SAMPLE PREPARATION AND TESTING PROGRAM

Four medium-dense Monterey samples were reconstituted to relative densities ranging from 56 to 60 percent using the dry pluviation method (Kwan and El Mohtar 2020). The samples had a

100 mm nominal diameter with an average height and weight of approximately 21-22 mm and 286-300 g, respectively. Low-friction, stacked Teflon-coated rings encircled the samples to provide confinement and maintain K_0 conditions as shown in Figure 5. After reconstitution, the samples were flushed with de-aired distilled water to fill air voids and achieve saturated conditions. The samples were put under back pressure saturation: three tests achieved a b-value of 0.95 and one test with 0.9. The four tests were performed in the following order and drain conditions: sample 1 (S1-D) was tested under drained conditions, sample 2 (S2-U) was tested under undrained conditions, and samples 3 (S3-PD1) and 4 (S4-PD2) under partially drained conditions. Figure 6 showcases how the partial drainage filter is able to fulfill the three different drainage conditions. For the partially drained samples, since the beginning of the saturation phase, the PD filter was connected between the soil specimen and the pressure controller for back pressure saturation. Before connecting the filter to the sample, the filter was flushed with de-aired water to remove any air. Once that had been done, the filter was connected to the sample, becoming a single system, and both were put under back pressure to maintain saturation. In S3-PD1 and S4-PD2, the three needle valves in the partial drainage filter were set at opening $1/25^{th}$ of a turn and 0, respectively. Even though the needle valves are at Opening 0, (Figure 6D) water is still able to pass through the ball bearings in the valves (Swagelok 2024).

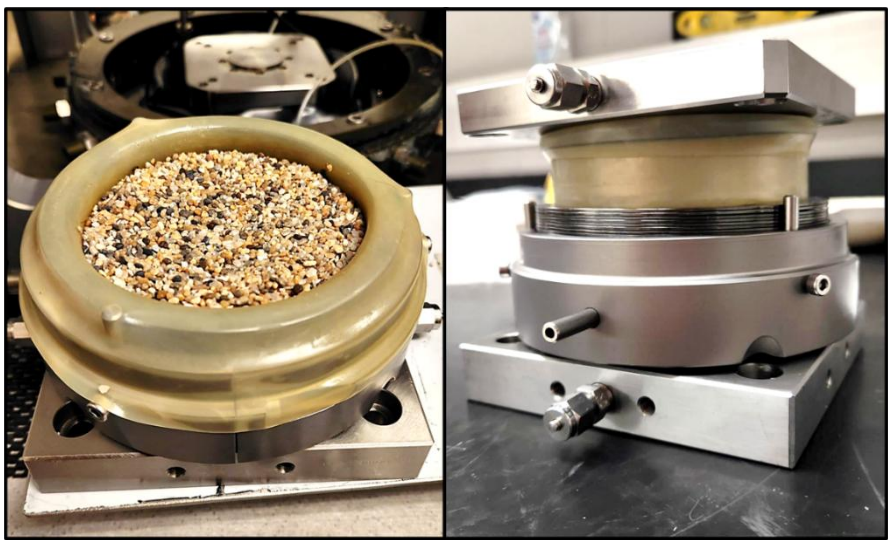

Figure 5. Reconstituted Monterey Sand Sample: Without the Top Cap (Left) & With the Top Cap (Right)

The testing program consisted of two stages: consolidation and shearing. Samples S1-D and S2-U were consolidated to a vertical effective stress of 400 kPa without the use of the PD filter and were consolidated in a single increment of 400 kPa. However, for S3-PD1 and S4-PD2, with the PD filter attachment, consolidation was completed at different rates to minimize the buildup of pore water pressure and allow any accumulated pressure to dissipate. S3-PD1 was consolidated at a rate of 120 kPa/hour and S4-PD2 at 8.3 kPa/hour until they reached the final stress of 400 kPa. The buildup of back pressure confirmed that the partial drainage filter effectively restricted the flow of water. The dissipation of water demonstrated that water could still flow through the partial drainage filter, confirming the partial drainage conditions.

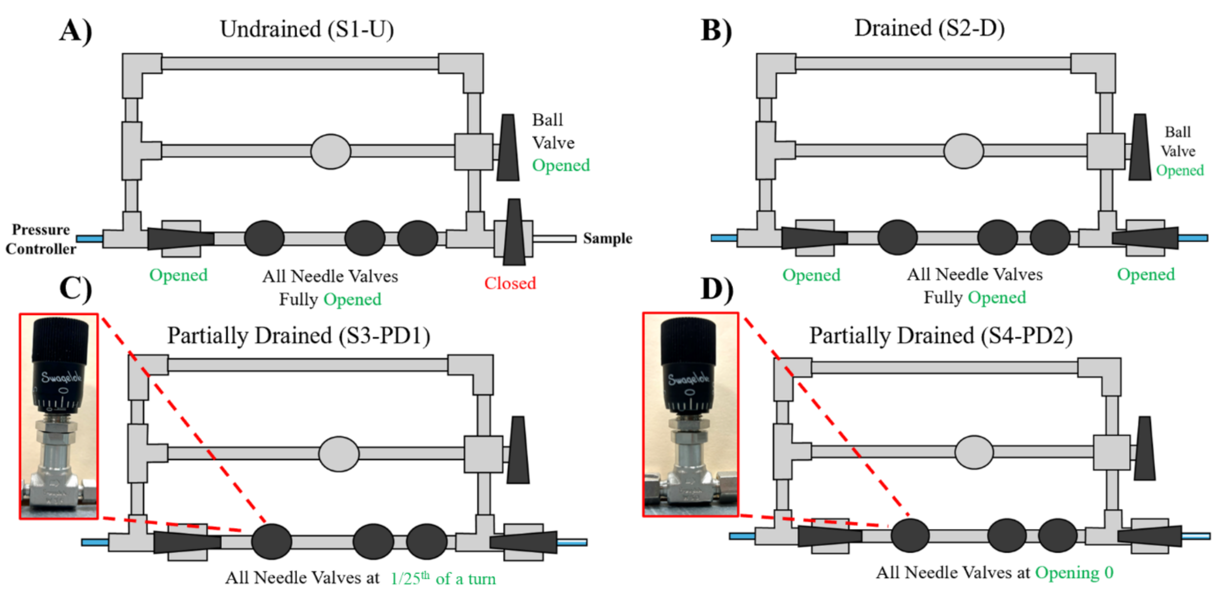

Figure 6. The Four Types of Drainage Conditions: A) Undrained, B) Drained, C) Partially Drained 1, and D) Partially Drained 2

After consolidation, all samples were sheared 8.5 mm along the x-axis at a constant rate of 1 mm per 80 minutes, allowing for the determination of drainage effects on the behavior of Monterey sand during shearing. A slow shear rate was adopted to ensure no excess pore pressure would generate in the fully drained condition. Table 1 summarizes the specimen information, and Figure 7 depicts the test results.

Table 1. Specimen Summary

Test ID	Drainage Type	Relative Density		B Value
		Before Consolidation	After Consolidation	
S1-U	Drained	59.99%	93.63%	0.95
S2-D	Undrained	58.32%	97.37%	0.90
S3-PD1	PD (Opening at 1/25th of a turn)	56.18%	92.39%	0.95
S4-PD2	PD (Opening 0)	56.42%	83.15%	0.95

RESULTS AND DISCUSSION

Shear Stress vs. Shear Strain

The results of the four DSS tests are shown in Figure 7. The shear stress versus shear strain curves (Figure 7A) show that the two partially drained tests follow a path between the drained and undrained tests. These results align with the overall observations from Suzuki et al. (2020). At approximately 25 percent shear strain, the drained and undrained samples switch upper and lower bounds, causing the undrained sample to obtain a greater shear stress than the drained sample. At the shear strain of around 25 percent, the PD samples also overcome the shear stress of the drained sample but are still within the bounds of the drained and undrained staples. These changes seem to correspond to the generation of negative pore pressure shown in Figure 7C, where the undrained sample generates the largest negative pore pressure, leading to the largest shear stress. The pore pressure generation also allowed S4-PD2 to reach a larger shear stress in comparison to S2-D.

Shear Stress vs. Vertical Effective Stress

Figure 7B shows the stress path for all four tests. As expected, for S1-D, the vertical effective stress was able to be maintained at 400 kPa due to no pore water pressure being generated, as shown in Figure 7C, since there are no restrictions on flow. On the contrary, for S2-U, the vertical effective stress decreases as the pore pressure in the sample increases, reaching a minimum value of approximately 269 kPa. However, as the sample transitions from a dilative to a contractive response, vertical effective stress increases, passing the initial applied axial load of 400 kPa. For both samples under partial drainage conditions, the behavior in effective stress is similar to S1-U but not as extreme in magnitude. S3-PD1 had a less restrictive flow, enabling more pore water pressure to dissipate compared to S4-PD2. Resulting in S3-PD1 and S4-PD2 having a lower minimum effective vertical stress of approximately 354 kPa and 384 kPa, respectively.

Excess Pore Pressure vs. Shear Strain

As shown in Figures 7C and 7D, changes in excess pore pressure and volume take place simultaneously throughout tests S3-PD1 and S4-PD2, indicating the achievement of partially drained conditions with the needle valve filter. With pore pressure measured directly at the bottom of the specimen and the volumetric flow measured after the PD filter at the back pressure controller (see Figure 1), a positive rate in the excess pore pressure generation corresponds to a negative volumetric strain rate. Similarly, a negative rate in the excess pore pressure generation corresponds to a positive volumetric strain rate. In excess pore pressure generation, the PD tests have a similar behavior as the undrained test, up to around 25% shear strain, starting with a positive excess pore pressure buildout that later peaks and turns negative. At shear strains larger than 25%, the response of the excess pore pressure deviates from that of the undrained test and transitions into a near-steady state phase similar to the drained test, where no excess pore pressure generation takes place. These observations indicate that a partially drained test behaves like both an undrained and drained sample, depending on the shear strain.

Volumetric Strain vs. Shear Strain

Figure 7D shows how the volumetric strain of the four samples changes with shear strain. Since S2-U was under undrained conditions, water could not enter or leave the sample, so the volume remained constant throughout the test. It is noticeable that both partially drained tests, S3-PD1 and S4-PD2, have a very similar magnitude of volumetric strain in comparison to the drained test, S1-D. S3-PD1 has a larger volumetric strain, which can be due to having a smaller relative density before the shearing stage. Additionally, it is apparent that the volumetric strain for S2-D, S3-PD1, and S4-PD2 exhibits almost linear behavior after reaching their absolute maximum volumetric strain values. This occurs around the same time pore pressure starts to stabilize, as shown in Figure 7C, at approximately 25% shear strain.

To overcome the limitations encountered in achieving partially drained conditions, several potential solutions can be considered. Increasing the shear rate could be one approach, provided it does not induce excess pore pressure during the drained test for clean sand. Alternatively, using a larger sample size, such as a specimen with a greater diameter, might also be effective. However, further testing is necessary to validate these methods.

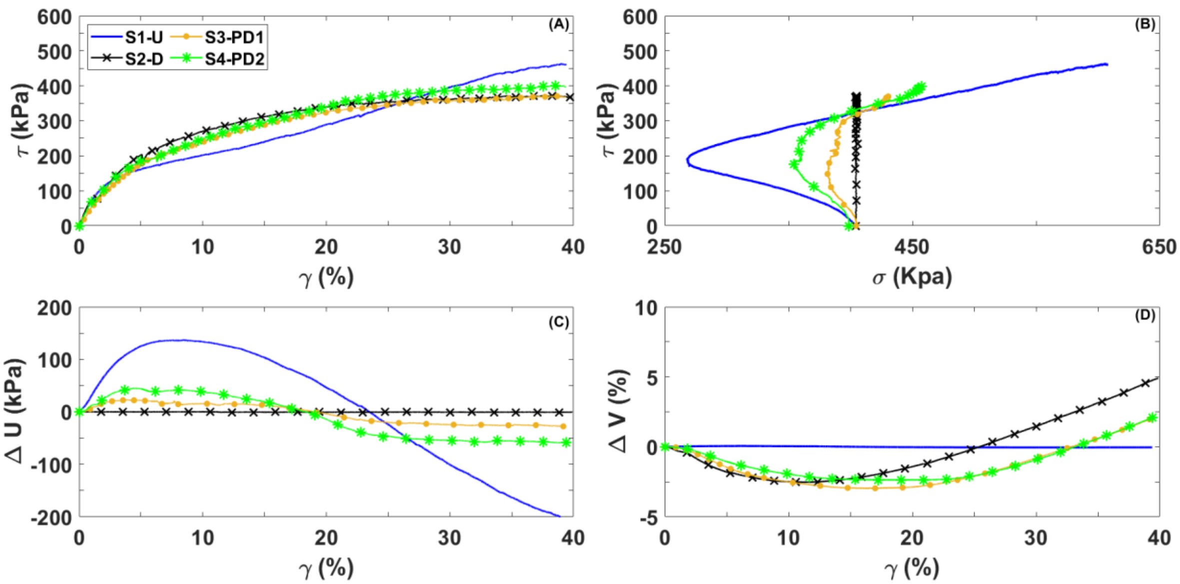

Figure 7. (A) Shear strain vs shear stress (B) Vertical effective stress vs shear stress (C) Shear strain vs excess pore pressure (D) Shear strain vs volumetric strain

CONCLUSION

To study the effects of partial drainage (PD) conditions, Monterey sand samples were tested under unidirectional monotonic direct simple shear (DSS) attached to a PD filter composed of low-pressure needle valves. The results show that the stress-strain curves from the PD DSS tests follow a path between the drained and undrained tests. The excess pore pressure curves from the four tests showed that the PD tests changed from undrained behavior at low strains to drained behavior at higher strains with minimal pore pressure generation. Consequently, the two PD tests showed a decrease in effective vertical stress but not to the degree of the undrained test. Lastly, the volumetric strain for the drained and partially drained samples showed a linear behavior after reaching an absolute maximum value. Overall, the results showcased the effectiveness of the partial drainage filter, confirming that partial drainage conditions were successfully met. Further testing with different densities, sand variations, and other control parameters using the proposed partial drainage filter can lead to a better understanding of soil behavior under partially drained conditions, adding to the design of safer and more reliable structures.

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