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Shear behaviour of sandy silt treated with lignosulfonate

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Shear behaviour of sandy silt treated with lignosulfonate

Abstract

Chemical stabilizers (e.g., cement, lime, gypsum, and other alkaline admixtures) have been widely used to enhance the strength and compressibility properties of subgrade soils. However, traditional chemical stabilizers are not always acceptable in Australia because they often pose a threat to the surrounding environment. Moreover, traditionally treated soils usually exhibit excessive brittle behaviour, which is often undesirable for transport infrastructure such as rail embankments and airport runways. To establish an alternative stabilizer that could overcome the above problems, this note presents a series of experimental results on the use of lignosulfonate (by-product of timber and paper industry), an environmentally friendly soil stabilizer effective for treating fine sandy silt that formed the bulk of an embankment fill at Penrith, Australia. The effects of lignosulfonate treatment on the shear behaviour of treated soil, including the stress-strain relationships, and the corresponding development of excess pore pressure and volumetric responses under monotonic triaxial testing are discussed.

Disciplines

Engineering | Science and Technology Studies

Publication Details

Chen, Q. & Indraratna, B. (2015). Shear behaviour of sandy silt treated with lignosulfonate. Canadian Geotechnical Journal, 52 (8), 1180-1185.

This journal article is available at Research Online: https://ro.uow.edu.au/eispapers/4657

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TECHNICAL NOTE

Shear Behaviour of Sandy Silt treated with Lignosulfonate

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e-mail**:** indra@uow.edu.au

Words: 2156

Figures: 5

Date of Submission: July 2, 2014

Submitted to: Canadian Geotechnical Journal

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Shear Behaviour of Sandy Silt treated with Lignosulfonate

Qingsheng Chen[1] and Buddhima Indraratna[2]

ABSTRACT: Chemical stabilisers (e.g., cement, lime, gypsum and other alkaline admixtures) have been widely used to enhance the strength and compressibility properties of subgrade soils. However, traditional chemical stabilizers are not always acceptable in Australia because they often pose a threat to the surrounding environment. Moreover, traditionally treated soils usually exhibit excessive brittle behaviour, which is often undesirable for transport infrastructure such as rail embankments and airport runways. To establish an alternative stabiliser that could overcome the above problems, this Technical Note presents a series of experimental results on the use of lignosulfonate (by-product of timber and paper industry), an environmentally friendly soil stabiliser effective for treating fine sandy silt that formed the bulk of an embankment fill at Penrith, Australia. The effects of lignosulfonate treatment on the shear behaviour of treated soil, including the stress-strain relationships, and the corresponding development of excess pore pressure and volumetric responses under monotonic triaxial testing are discussed.

KEYWORDS: laboratory tests; lignosulfonate; sandy silt; soil improvement

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1. Introduction

Vast deposits of unstable soils are widely distributed in rural and regional Australia, and they have posed significant challenges in the design and construction of transport infrastructure. Chemical additives such as cement, lime, flyash have been effectively used to enhance the strength and compressibility properties of subgrade soils. In the past, extensive studies have been carried out on the engineering behaviour of stabilised soils using traditional admixtures (Clough et al. 1981; Indraratna, 1996; Rollings and Burkes, 1999; Horpibulsuk et al. 2004; Marri et al. 2012). However, traditional chemical stabilizers are not always readily acceptable in Australia because of stringent occupational health and safety issues (Indraratna et al, 2013). They also adversely affect the environment by changing pH, which often limits the scope of vegetation and affects the quality of the groundwater (Rollings and Burkes, 1999; Sherwood, 1993). Moreover, cementitious chemical admixtures reduce the capacity of soil to hold water and carry nutrients, thereby affecting its fertility (Kitchen et al. 1996). In addition, excessive use of traditional admixtures to stabilize soil affects the yielding capacity of certain soils (Nalbantoglu and Tuncer, 2001) and often exhibit an excessively brittle performance that influences the stability, especially under cyclic loads in the case of rail embankments and airport runways (Sariosseiri and Muhunthan, 2009).

In order to overcome these problems, it is required to use an alternative soil stabilizer that can improve the strength and durability of the soil, without harming the environment. Recently, a lignin-based chemical, lignosulfonate (LS) has shown some promise in stabilising problematic soil (e.g., Puppala and Hanchanloet, 1999; Tingle

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and Santori, 2003; Indraratna et al, 2008 & 2013). It is statistically reported that over 50 million tons of industrial lignin is produced worldwide annually (Xiao et al. 2001), and when compared to traditional chemical admixtures, LS is cheaper, environmentally friendly, nonhazardous, and does not appreciably change the pH of the soil after treatment (Indraratna et al. 2008).

Although numerous studies have reported that the addition of LS could contribute to a significant increase in soil strength (e.g., Puppala and Hanchanloet, 1999; Tingle and Santori, 2003), most of them have mainly carried out unconfined compression strength (UCS) tests. In this Technical Note, a series of laboratory tests, including UCS tests, isotropic consolidated drained (CID) and consolidated undrained (CIU) shearing tests have been carried out to determine how LS could improve the shear strength of treated sandy silt.

2. Laboratory Investigation

2.1 Type of Soil and Stabilizer Used

The soil selected for this study was a sandy silt that was used extensively as embankment fill at Penrith, New South Wales (NSW), Australia. The soil contains about 49% silt, 42% sand and 9% clay. According to the standard Proctor compaction test, the sandy silt has a maximum dry density (MDD) of 1,920 kg/m^3, corresponding to an optimum moisture content (OMC) of 12.2%, respectively.

Lignosulfonate (LS) is a by-product of paper and timber industry. It is a lignin-based polymer compound that contains a number of hydrophilic groups including sulfonate, phenylic hydroxyl, as well as alcoholic hydroxyl, and hydrophobic groups including the carbon chain (e.g. Chen, 2004). Lignosulfonate used in this study is a light-

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yellow-brown powder that was soluble in deionised water (i.e., it is in the form of liquid).

2.2. Procedure and Test Plan

Treated soil specimens for the current experimental investigation were prepared using five dosages (0.5, 1, 2, 3 and 4%) of LS by dry soil weight. It is mentioned that the authors experimentally found that the results with LS=6% and 8% were very close to the data for LS=4%, therefore, there is no need to present this data in this study. The amount of stabilizer was first added with an additional amount of water needed to achieve the optimum moisture content before mixing with the soil. The authors prepared the specimens using both compaction and vibration methods, and found that the vibration technique not only gave approximately the same OMC and MDD as those obtained using standard compaction method, but also the specimens prepared by vibration were more uniformly compacted. Clear shear band in the specimen was observed during the shearing tests, which was used to verify the homogeneity of the specimen. Segregation did not occur under vibration. In this Technical Note, uniform specimens (38mm diameter and 76mm in height) were prepared by compaction using vibration (via a shaking table) at the optimum moisture content until the maximum dry density was achieved. A top surcharge load of 2kg was used during vibration. The compacted samples were then sealed in a plastic wrap and stored in a humidity controlled room to cure at a constant temperature (20±2^{0}C).

To investigate the shear strength of LS treated soil, UCS, CIU, and CID tests were performed on samples after being cured for 7 days, because the UCS test results for samples cured for 0, 1, 2, 4, 7, 14, and 28 days revealed that the strength and stiffness

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only increased marginally after curing for 7 days. In coastal Australia, most rail embankments are low-lying, and the groundwater table is very close to the surface, hence full saturation is common. Also, fine particles and degraded ballast fines infiltrate groundwater to the bottom of embankment impeding drainage and thereby ensuring full-saturation in most transport corridors in low-lying ground. Also, under heavy rainfall that the Australian coast commonly encounters, the bottom part of embankments is often fully saturated (Chen & Indraratna, 2014). Therefore, all the triaxial tests were carried out on saturated specimens (Skempton's B>0.98) in this study to simulate the field conditions. Prior to testing, the specimens for CID and CIU tests were saturated and consolidated isotropically at a given effective confining pressure. A relatively high backpressure in the vicinity of 500kPa was required to attain near-saturation, i.e. Skempton's B>0.98 at all levels of testing. The triaxial tests attempted to simulate the typical in-situ conditions of railway subgrade soils. Apart from the authors, field measurements have been observed by various other researchers (e.g. Liu 2006; Liu and Xiao, 2010), and relatively small confining pressures typically of 20-60 kPa have been observed. Therefore, initial mean effective confining stresses (σ_3') of 15, 30, and 60kPa were applied to the test specimens under CID and CIU tests to represent an array of typical insitu subgrade conditions. A relatively low shearing rate of 0.2mm/min was used to ensure fully drained conditions (ASTM D7181, 2011). All triaxial results are plotted up to an axial strain (ε_a) of 25%, which was the limit of the load actuator. The deviator peak stress and pore pressure at ε_a =25% are referred to as ultimate or final for simplicity.

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3. Results and Discussion

3.1. Unconfined Compression Test

Strength and Deformation Modulus

Figure 1 shows the unconfined strength and the deformation modulus (E50) determined at 50% of peak axial stress for the sandy silt treated with 0-4% of LS, respectively. The relationship for the axial stress versus axial strain obtained from UCS tests could be referred to Chen et al. (2014). It is observed that the peak axial stress increased significantly as the percentage of LS increased from 0% to 2%, but the UCS decreased slightly when the percentage of LS exceeded 2%. This implies that the optimum percentage of LS required for this soil was approximately 2%. It is observed that the maximum axial stress increased from 149kPa to 285kPa (i.e., the corresponding maximum increment is up to 91%), while E50 increased from 187kPa to 344kPa (i.e., the corresponding maximum increment is up to 84%), as the percentage of LS-treatment increased from 0% to 2%. This verifies the effectiveness of LS treatment of fill (sandy silt) in controlling embankment deformation.

3.2. Undrained Triaxial Compression Test

Stress-Strain and Excess Pore Pressure Characteristics

The left side plots of Figure 2 show the undrained stress-strain behaviour of treated sandy silt (LS=1% to 4%) compared to the untreated counterpart (LS=0%) at increasing effective confining pressure ($\sigma_3^{'}$). It should be noted that all the graphs plotted in Figure 2 are for the experimental data from CIU tests. As expected, when $\sigma_3^{'}$ increased from 15kPa to 60kPa, the peak deviator stress and the initial deformation modulus increased for all samples. It was highlighted by Vinod and

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Indraratna (2011) that LS treatment could significantly reduce the softening behaviour of treated soils after peak deviator stress, which is the main difference between LS treatment and the cement/lime treatment. Therefore, this study was more focused on how LS could improve the ultimate strength (q_{ult}) of treated soil and not just the effect on the peak strength. When the percentage of LS increased, the ultimate deviator stress of treated soil increased significantly from LS=0% to 2%, but it decreased slightly when the LS content exceeded 2%. Similar to the results from the UCS tests, there is little advantage in increasing the LS content beyond 2%. Therefore, the Authors feel that the current experimental data for LS=2% and 4% from CIU tests are sufficient to capture the key aspects of the LS-treated soil behaviour and prove that only marginal returns are expected beyond LS=2%. Therefore, in our view, it is not necessary to repeat the CIU tests on the treated soil with LS=3% as that for UCS tests in this study. For a given amount of LS, the increase in ultimate stress diminished with the increase of σ_3'. For instance, for 2% LS treated specimens, the increase in q_{ult} was about 24 kPa at σ_3'= 15kPa, but less than 14kPa at σ_3'=60kpa. More importantly, as shown in Figure 2, the post-peak ductility of treated soil remained unchanged, unlike the behaviour of soil treated by traditional (alkaline) additives such as lime and cement, where the stress-strain behaviour often transforms from ductile to a brittle response (e.g. Indraratna et al, 1996).

The right side plots of Figure 2 show the responses of excess pore water pressure variation with axial strain for varying levels of LS treatment and increasing confining pressure. At low strains (<0.5-1%) all the specimens showed a peak in excess pore pressure followed by a sudden drop, leading to the development of negative excess

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pore water pressure or suction, for $\sigma_3^{'}$=15 and 30kPa. In contrast, at $\sigma_3^{'}$=60kPa, the excess pore water pressure always remained positive. Moreover, the ultimate excess pore pressure was notably smaller (i.e., higher suction) than its untreated counterpart at all levels of initial $\sigma_3^{'}$. Nevertheless, as LS content was increased to 4%, the specimen behaviour was approaching that of LS=1%, indicating that adding LS beyond 2% is counterproductive.

3.3. Drained Triaxial Compression Test

Deviator Stress-Shear Stress and Volumetric Responses

The left side plots of Figure 3 show the drained stress-strain behaviour of treated sandy silt (LS=1%-4%) compared to the untreated counterpart (LS=0%) at increasing confining pressure. It should also be noted here that all the graphs plotted in Figure 3 are for the experimental data from CID tests. Similar to the observations in the CIU tests, when $\sigma_3^{'}$ increased from 15kPa to 60kPa, the peak deviator stress and the initial deformation modulus increased accordingly. Moreover, the ultimate deviator stress of treated soil increased significantly due to the addition of LS compared to the untreated soil especially when the specimen was subjected to a low confining pressure of 15kPa.

The right plots of Figure 3 show the volumetric responses with axial strain of treated sandy silt (LS=1%-4%) compared to the untreated counterpart (LS=0%) at increasing confining pressure. It can be seen that for all the treated soil examined, dilative behaviour under CID tests was observed for specimens consolidated at a low confining pressure (i.e. 15 and 30kPa), whereas a contractive response was observed

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for the specimens subjected to a higher confining pressure (i.e., 60kPa). As $\sigma_3^{'}$ increased from 15 to 60kPa, the most significant change was the transition from dilation to compression. For 2% LS-treated soil, dilation was greatest at $\sigma_3^{'}$=15kPa, while at $\sigma_3^{'}$ =60kPa, the treated soil showed less compression compared to the untreated specimen. As the LS content was increased to 4%, the sample behaved closer to the LS=1% specimen. This again confirms that for this sandy silt, increasing the LS content beyond 2% is counterproductive.

Figure 4a illustrates the variation of ultimate deviator stress (i.e., at an axial strain at 25%) in absolute values with different percentage of LS treatment. It should be mentioned that, from the deviator stress (q)-strain curves as shown in Figures 2a and Figure3a, q was almost kept constant after axial strain=10%, and the samples could still be assumed to be stable at an axial strain of about 15%. So 25% axial strain is more reasonable to represent failure. Both USC and triaxial (CID & CIU) test results have indicated that the soil strength increased significantly from LS=0% to LS=2%, but hardly any favourable effects were observed when the LS content was increased beyond 2% (see Figures 1, 2 &3). There is no doubt that the use of excessive amount of LS (i.e., beyond LS=2%) is counterproductive, because at these high levels of LS the ultimate deviator stress is decreased. From the results of both CIU and CID tests as shown in Figure 4a, it is also observed that the increase in the ultimate deviator stress (q_{ult}) diminished with the increase in confining pressure. This has been well illustrated in Figure 4b for the case of LS=2%. As shown in Figure 4b, the increase in ultimate deviator stress in absolute values for LS treated specimen up to 2% is more

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significant in the case of CIU testing compared to CID tests for all levels of $\sigma_3^{'}$. For CIU tests, at LS=2%, the gain in q_{ult} at $\sigma_3^{'}$=15kPa is almost 24 kPa (i.e., q_{ult} increases from 83.9 kPa at LS=0% to 108.0 kPa at LS=2%), while at the same level of $\sigma_3^{'}$, the change in q_{ult} for CID specimen is slightly lower (i.e., 9.6 kPa). However, as the value of $\sigma_3^{'}$ increases, the increment in the ultimate deviator stress drops rapidly for CIU specimens in contrast to CID specimens, especially as $\sigma_3^{'}$ increases from 15kPa to 30kPa (Figure 4b).

Critical State Characteristics

The right plots of Figure 3 for the experimental data from CID tests also show that after 25% axial strain, the volumetric strain (ε_v) is nearly constant, implying that the assumption of a critical state is nearly justified. The critical state characteristics can be deduced from both CIU and CID test data corresponding to ε_a=25%, as shown in Figure 5. The corresponding deviator stress represents the "critical/ultimate strength" rather than the "peak strength". The effect of LS-treatment on the critical state line (CSL) is illustrated in Figure 5 through a comparison of untreated and treated soil (LS=2%). It is observed that for untreated soil showed insignificant cohesion intercept; however, the CSL shifted to the left upon LS treatment, i.e., the critical state of the LS-treated soil was located slightly above the untreated soil. In this study, the intercept of CLS with the vertical q-axis has been defined as "apparent cohesion", which in our opinion is attributed to the LS induced bonding in the soil. In our view, the apparent cohesion generated by LS-treatment does play a role in the critical state,

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although only friction angle is considered in classical Cambridge soil mechanics. Similar phenomena have been experimentally verified by past researchers for bonded soils (e.g., Yu et al, 2007; Lee et al, 2004). The role of induced chemical bonding as a form of increased cohesion and tensile strength has also been explained by Lade and Overton (1989), Clough et al. (1981), and Horpibulsuk et al. (2004). As the two CSL are parallel to each other, it may be concluded that LS treatment has insignificant effect on the angle of shearing resistance compared to the definite contribution to enhanced apparent cohesion.

4. Conclusions

The shear behaviour of lignosulfonate (LS)-treated sandy silt was studied, and the following conclusions could be drawn.

1. Due to the addition of LS up to 2%, the unconfined compressive strength (UCS) and deformation modulus (E50) of treated sandy silt increased significantly, with the corresponding maximum increments being up to 136 kPa (i.e., 91.0%) and 157 kPa (i.e., 83.9%), respectively. However, diminishing returns were observed when the LS content exceeded the optimum level of 2%.

2. In both CID and CIU tests, the ultimate deviator stress (i.e. at ε_a=25%) of LS-treated soil increased more than 9 kPa compared to the untreated counterpart, especially under low confining pressures (i.e. σ_3'=15kPa).

3. The post-peak ductility of LS-treated soil remained practically unaltered as that of the untreated soil. Also, the ultimate excess pore pressure was lower in the treated soil than its untreated counterpart at all levels of σ_3'.

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4. From the comparison of volumetric responses of treated and untreated soil, it was evident that the treated soil exhibited a more dilative behaviour than the untreated soil at low confining pressure (i.e. $\sigma_3^{'}$=15 and 30kPa). At higher confining pressure (i.e. $\sigma_3^{'}$ =60kPa), the treated soil was less compressive compared to the untreated specimen.

5. With regard to the soil strength at critical state, the role of LS treatment is reflected by an increase in apparent cohesion, but hardly any effect on the magnitude of apparent friction angle.

Acknowledgements

The authors appreciate the financial support provided by the Queensland Department of Transport and Main Roads, Chemstab Consulting Pty. Ltd, Douglas Partners, and Coffey Geotechnics through an Australian Research Council (ARC) Linkage. The authors express their sincere gratitude to Mr. Mohammad Abadimarand for his laboratory assistance.

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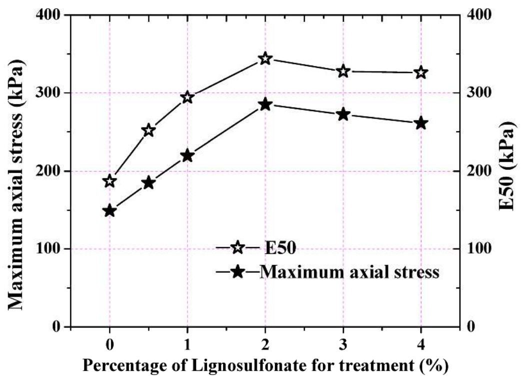

Figure 1 UCS test results of treated sandy silt with different percentage of LS

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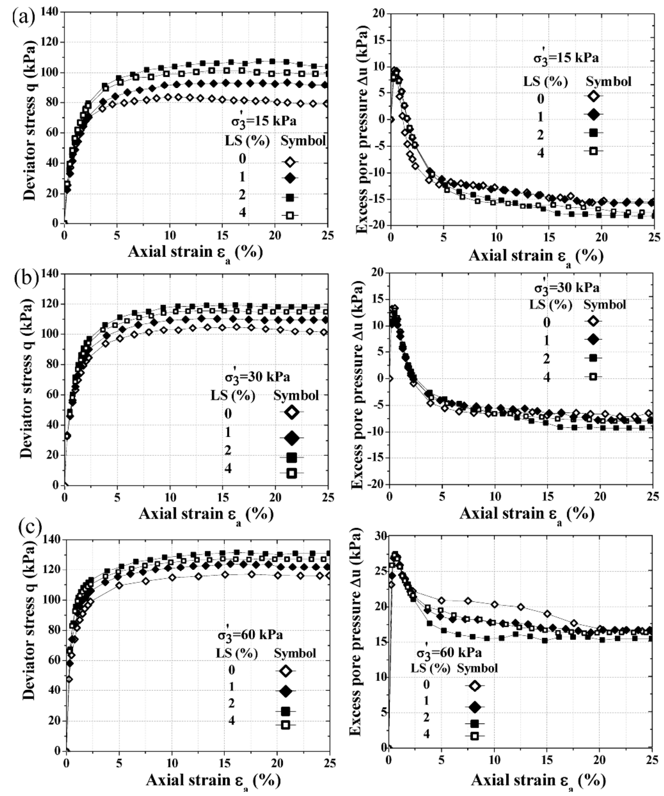

Figure 2 Undrained shear behaviour of treated sandy silt with different percentages of LS at confining pressures of 15kPa, 30kPa, and 60kPa, respectively

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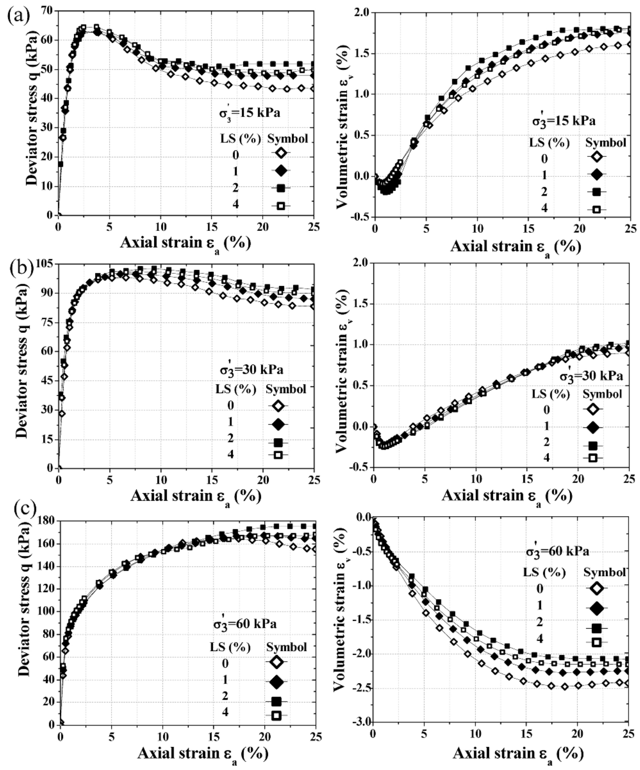

Figure 3 Drained shear behaviour of treated sandy silt with different percentages of LS at confining pressures of 15kPa, 30kPa, and 60kPa, respectively

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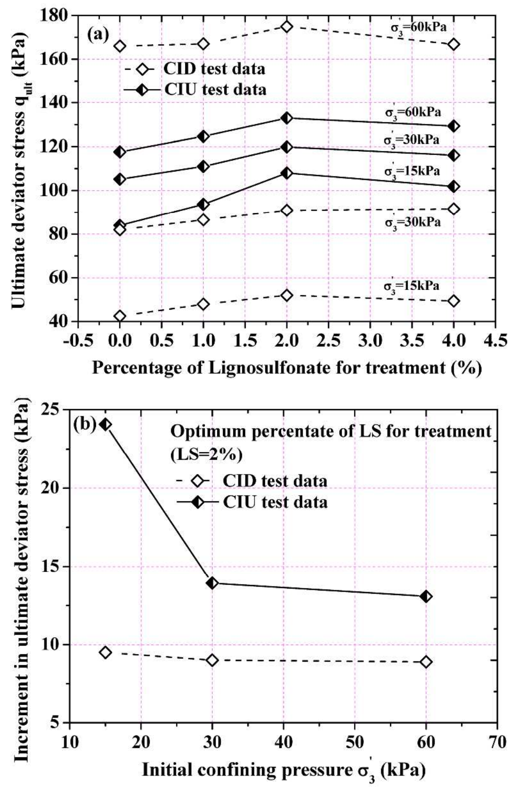

Figure 4 Comparison of the LS-induced improvement in residual drained and undrained strength of treated sandy silt at confining pressures of 15kPa, 30kPa, and 60kPa, respectively

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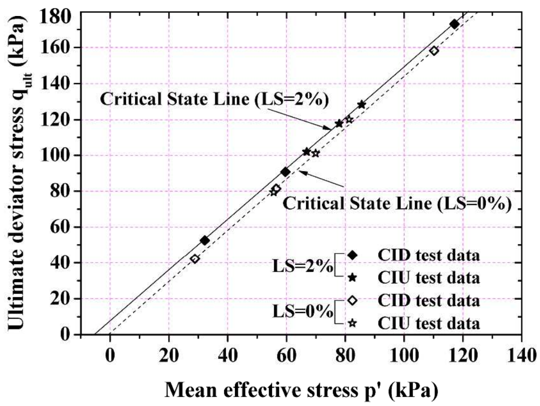

Figure 5 Effect of LS-treatment on critical state characteristics

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