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A theoretical and experimental study on the behaviour of lignosulfonate-treated sandy silt

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ABSTRACT

The effectiveness of an environmentally friendly stabilising agent for soil, namely, lignosulfonate was examined through a series of laboratory tests. A simple bounding surface plasticity model was developed to capture the bonding effects induced by lignosulfonate. One of the appealing aspects of the model is that it can incorporate the mechanical behaviour of the bonded soil during shearing, including the brittle and ductile failure modes. Validity of the model was verified by experimental results of lignosulfonate-treated soils under different stress path conditions. The mechanical behaviour of chemically treated soil was adequately captured by the model.

1. Introduction

Unstable soils with low strength and high compressibility are widely distributed in many parts of Australia, particularly in regional New South Wales, Southeast Queensland, and Western Australia. Unstable soil beneath foundations can easily cause differential settlement, loss of bearing capability and unacceptable lateral movement upon loading, especially under traffic loading if effective ground improvement is not implemented.

Chemical stabilising agents (e.g. cement, gypsum, lime, and other alkaline admixtures) have been commonly used for the construction of highways, rail tracks, and airport runways to enhance the bearing capacity, reduce settlement, control shrinking and swelling, and reduce permeability [18,6,8,23,11,38]. However, such traditional admixtures commonly alter the soil pH, which may have adverse effects on the environment such as limiting vegetation and threatening the quality of the ground water [45]. Traditionally treated soils also exhibit an excessively brittle performance that affects the stability of structures [43,49]. To avoid these problems, lignosulfonate (LS), a by-product of paper and timber industry has been recognised in recent years as a promising stabilising agent for cohesive and non-cohesive soils [40,39,46,24,48,20]. LS is a lignin-based polymer compound documented by researchers (e.g. [30,36]). It consists of both hydrophilic groups including sulfonate, phenylic hydroxyl, alcoholic hydroxyl and hydrophobic groups including the carbon chain (e.g. [10]). Compared to traditional stabilisers, LS is an environmental friendly, non-corrosive and non-toxic chemical that does not alter the soil pH upon treatment [9], while the ductile behaviour of the soil can be maintained while also causing an increase in strength and stiffness [49].

Various studies are available in the past literature dealing with chemical bonding of soft and weak soils (e.g., [15,25,29,12,16,5]), however, most of these studies describe the use of alkaline additives (e.g. cement, lime, gypsum, etc.) which make the treated soil increasingly brittle (formation of new crystals, e.g. ettringite), whereas LS-treated soil retains its ductility attributed to a totally different 'conditioning' process (rejuvenating the clay lattice and interstitial water layer; [21,20,49]). In this context, most of the chemical stabilisation models described in past studies cannot predict the correct stress–strain behaviour of LS-treated soils. Moreover, there are other deficiencies in the existing models, which have been discussed elsewhere by Yu et al. [52] and Yan and Li. [50].

In view of the above, the aim of the current study was to develop a constitutive model for LS-treated soil supported by a series of laboratory studies conducted under triaxial conditions. The proposed model was developed on the basis of bounding surface plasticity theory within a critical state framework. Following the conceptual framework adopted by Yu et al. [52] and Gens and Nova [15], the shape of the bounding surface for the treated

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Notation

B	bulk modulus	q	deviator stress
e_Γ	critical void ratio at a reference pressure of 1kPa	r	parameter for defining destructuration rate of bonding effect
e	current void ratio	R	parameter defining the shape of bounding surface
e_0	initial void ratio	u	excess pore water pressure
e_{cs}	void ratio at critical state	γ	model parameter for hardening
G	shear modulus	ε_1	axial strain
k	destructuration rate of bonding effect	$\varepsilon_v^p, \varepsilon_v^e, \varepsilon_v$	plastic, elastic and total volumetric strain
k_d	model parameter defining stress–dilatancy behaviour	$\varepsilon_q^p, \varepsilon_q^e, \varepsilon_q$,	plastic, elastic and total distortional strain
k_0	initial value of destructuration rate of bonding effect	ε_d^p	damage strain
K_m	shape function controlling the variation of the modulus	η	stress ratio
K_p	elasto-plastic modulus	η^*	modified stress ratio considering bonding effect
M	slope of the critical state line (CSL) in the p–q plane	κ	swelling/recompression index
N	parameter defining the shape of bounding surface	λ	slope of the critical state line (CSL) in e-ln$p$ plane
m_p, m_q	components of plastic flow direction vector	μ	model parameters defining stress–dilatancy behaviour
n_p, n_q	components of unit vectors to bounding surface	v	Poisson's ratio
$\bar{p}_c$	parameter controlling the size of the bounding surface for untreated soil	ξ	state parameter
$\bar{p}_c^{**}$	parameter controlling the size of the bounding surface for treated soil	ψ_c, ψ_t	parameters defining destructuration law for bonding effect
p_t^*	current tensile strength of soil due to LS treatment	ψ_0	parameter accounting for initial bonding effect
p_t^0	initial tensile strength of soil due to LS treatment	p_0	constant mean effective stress
p	mean effective stress		

soil was assumed to be the same as the untreated soil, but it was enlarged in the proposed model to account for the improved properties. In this paper, a new hardening rule together with a destructuration law that can describe the different failure modes of the bonding effects is proposed, adopting a non-associated flow rule that captures the stress–dilatancy relationship. Moreover, an imaging rule with a mobile mapping centre was used to account for the evolution of the bonding effects during shearing. The proposed model was calibrated and validated using the experimental data for different stress paths.

2. Laboratory program

Having an understanding of the characteristics of LS-treated soil is vital in developing a rational stress–strain model. A series of laboratory tests, including unconfined compressive strength (UCS) testing, isotropic triaxial compression, isotropic consolidated drained (CID) and consolidated undrained (CIU) shearing tests were conducted on both treated and untreated soils to determine how LS could improve the soil strength.

2.1. Soil description

The soil selected for this study was a sandy silt that was used extensively as embankment fill in Penrith, Australia. The maximum particle size of the soil was 2.38 mm, and the corresponding particle size distribution is shown in Fig. 1. The particle size test was performed in accordance with AS 1289.3.6.1 [1] and by hydrometer for particles smaller than 0.075 mm (AS 1289.3.6.3 [2]). The soil here was classified as well graded sandy silt with less than 10% clay.

2.2. Test procedure

2.2.1. Preliminary testing phase

To assess the optimum content of LS, the soil samples were mixed with 0%, 0.5%, 1%, 2%, 3% and 4% of LS by weight of dry soil. After mixing with LS, compaction of the material with standard Proctor compaction energy was performed. It was found that all soil samples had very similar optimum moisture content (i.e., OMC = 12.2%) and maximum dry density (i.e., MDD = 19.2 kN/m^3), respectively. 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. In this paper, uniform specimens (38 mm diameter and 76 mm in height) were prepared by compaction using vibration (via a shaking table) at the optimum moisture content until the maximum dry density was achieved, and then cured for 7 days. A top surcharge load of 2 kg was used during vibration. Unconfined compression testing was then carried out on the specimens with a variation of 0–4% of LS by weight.

Fig. 2 shows the UCS test results of sandy silt treated with 0–4% of LS. UCS increased significantly as the percentage of LS increased from 0% to 2%, but the UCS decreased slightly when the percentage of LS exceeded 2%. It can be seen here that the optimum percentage of LS for this soil was approximately 2%, and therefore, only the test

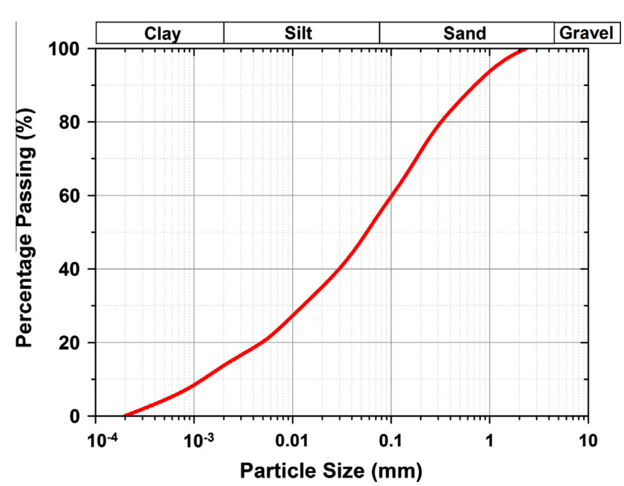

Fig. 1. Particle size distribution of sandy silt.