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Discussion

Discussion of "Prediction of the undrained shear strength of remolded soil with non-linear regression, fuzzy logic, and artificial neural network" [J. Mt. Sci. 21(9): 3108–3122]

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Abstract: This opinion article discusses the original research work of Yünkül et al. (the Authors) published in the Journal of Mountain Science 21(9): 3108–3122. Employing non-linear regression, fuzzy logic and artificial neural network modeling techniques, the Authors interrogated a large database assembled from the existing research literature to assess the performance of twelve equation rules in predicting the undrained shear strength (s_u) mobilized for remolded fine-grained soils at different values of liquidity index (I_L) and water content ratio. Based on their analyses, the Authors proposed a simple and reportedly reliable correlation (i.e., Eq. 9 in their paper) for predicting s_u over the I_L range of 0.15 to 3.00. This article describes various shortcomings in the Authors' assembled database (including potentially anomalous data and covering an excessively wide I_L range in relation to routine geotechnical and transportation engineering applications) and their proposed $s_u = f(I_L)$ correlation. Contrary to the Authors' assertions, their proposed correlation is not reliable for fine-grained soils with consistencies in the general firm to stiff range (i.e., for $0.15 < I_L < 0.40$), increasingly overestimating s_u for reducing I_L, and eventually predicting $s_u \rightarrow +\infty$ for $I_L \rightarrow 0.15^+$ (while producing mathematically undefined s_u for $I_L < 0.15$), thus rendering their correlation unconservative and potentially leading to unsafe geotechnical designs. Exponential or regular-power type $s_u = f(I_L)$ models are more suitable when developing correlations that are applicable over the full plastic range (of $0 < I_L < 1$), thereby providing reasonably conservative s_u predictions for use in the preliminary design for routine geotechnical engineering applications.

Keywords: Undrained shear strength; Liquidity index; Soil consistency; Non-linear regression

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Notations

Abb.	Explanation
ANN	Artificial neural network
ASTM	American Society for Testing and Materials
BS	British Standard
FC	Fall cone
FL	Fuzzy logic

NLR	Non-linear regression
SIPF	Shifted inverse power function
a	Fitting parameter for $s_u = f(I_L)$ correlation
I_L	Liquidity index
LL	Liquid limit (%)
MAPE	Mean absolute percentage error (%)
MPE	Mean percentage error (%)
NRMSE	Normalized root mean square error (%)
PL	Atterberg's plastic limit (%)
q_u	Ultimate net bearing capacity (of shallow foundation) (kPa)
R^2	Coefficient of determination
s_u	Undrained shear strength (kPa)
s_{uLL}	Undrained shear strength mobilized at w = LL (i.e., I_L = 1) (kPa)
s_{uPL}	Undrained shear strength mobilized at w = PL (i.e., I_L = 0) (kPa)
w	Water content (%)
WCR	Water content ratio (= w/LL)
x	Fitting parameter for $s_u = f(I_L)$ correlation

1 Introduction

This opinion article presents a discussion of the original research reported in the paper by Yünkül et al. (2024) (hereafter referred to as the Authors) that was published in the Journal of Mountain Science 21(9): 3108–3122. Applying non-linear regression (NLR), fuzzy logic (FL) and artificial neural network (ANN) modeling techniques to a large database assembled from the existing research literature, the Authors investigated twelve equation rules (summarized in Table 1 of their paper), as generalized tools (or functional expressions), in predicting the undrained shear strength (s_u) mobilized for remolded fine-grained soils at different values of liquidity index (I_L) and water content ratio (WCR). Their assembled database consisted of 1306 water content (w), liquid limit (LL), Atterberg plastic limit (PL), I_L, WCR (= w/LL) and s_u results from 230 remolded soil test-settings reported in 21 earlier publications. Based on their NLR, FL and ANN analyses, the Authors reported that of the twelve different $s_u = f(I_L)$ equation rules investigated, the best s_u predictive performance (adjudged for their Eq. 9, and primarily based on its R^2 value) was obtained for the shifted-inverse-power-function (SIPF) rule originally proposed in the paper by Leroueil et al. (1983), and which can be expressed as follows:

$$s_u(\text{kPa}) = \frac{1}{(I_L - x)^a} \tag{1}$$

where a and x are the fitting parameters/ coefficients.

Table 1 of this article presents the values of the a and x fitting parameters for Leroueil et al. (1983)'s correlation and those deduced for Eq. 9 in the Authors' investigation, along with their reported applicable I_L ranges. Note that the different values of the fitting parameters reported for these two studies can be attributed to the fact that they were calibrated using entirely different s_u:I_L datasets (i.e., comprising soils with dissimilar gradation, mineralogical and viscosity/plasticity properties).

This article presents a critique of the Authors' investigation, including their assembled database and proposed $s_u = f(I_L)$ correlation given by Eq. 9 in their paper. As a prerequisite, both of these aspects should relate to the pertinent I_L range applicable for most routine geotechnical and transportation engineering applications. Next, after elucidating some general considerations necessary in the development of reliable $s_u = f(I_L)$ correlations, this article identifies several shortcomings in the Authors' assembled database and their proposed $s_u = f(I_L)$ correlation. In particular, contrary to the Authors' assertions, their Eq. 9 is mostly not suitable for soils with consistencies in the general firm to stiff range [i.e., for $0.15 < I_L < 0.40$, which typically mobilize s_u of 40–150 kPa (Craig 2004)].

2 Pertinent I_L Range for Geotechnical and Transportation Engineering Applications

In considering the remolded-strength case, the s_u predictions can be employed as lower-bound (conservative) estimates for preliminary design purposes in various geotechnical and transportation engineering applications (O'Kelly 2013a). Examples listed by the Authors include the design of shallow/deep foundations, pavements, offshore structures, retaining walls and assessments of the

Table 1 Deduced fitting parameters for the $s_u = f(I_L)$ model given as Eq. 1.

Reference	a	x	I_L Range
Leroueil et al. (1983)	2.000	0.21	0.50–2.50
Authors' Eq. 9	2.280	0.15	0.15–3.00
Authors' Eq. 9 (95% lower confidence bound)	2.258	0.15	
Authors' Eq. 9 (95% upper confidence bound)	2.303	0.15	

stability of embankments/dams and excavations in cohesive (fine-grained) soils. These design applications pertain to the fine-grained soils in their plastic and semi-solid states, with mobilized s_u values of between approximately 20 and 300 kPa (Craig 2004). This informs the relevant I_L range (of between −0.2 and ~0.5) that should be the primary focus for investigations with the stated goal of developing useful $s_u = f(I_L)$ correlations for geotechnical engineering design. Whereas for their compiled database, the Authors included values of s_u mobilized for I_L ranging from 0 up to 3 (see Fig. 2a of the Authors' paper), with the upper bound of this I_L range mobilizing s_u values of as low as 0.1 kPa (i.e., corresponding to a soil suspension), which have no practical relevance for any geotechnical engineering applications.

More importantly, the applicability of the Authors' Eq. 9 (i.e., Eq. 1 in the present article, with a = 2.28 and x = 0.15) is inherently constrained by its mathematical domain of definition, which is limited to $I_L > 0.15$ (i.e., $I_L \rightarrow 0.15^+ \Longrightarrow s_u \rightarrow +\infty$, and for $I_L < 0.15 \Longrightarrow$ undefined s_u). That is, the Authors' proposed $s_u = f(I_L)$ correlation given by their Eq. 9 inherently excludes the lower I_L range (of ≤ 0.15) corresponding to the semi-solid and early-plastic states of fine-grained soils, which are of primary interest in many routine geotechnical design scenarios.

3 General Considerations in Developing $s_u = f(I_L)$ Correlations for Geotechnical and Transportation Engineering Applications

Before commenting in greater depth on the Authors' data-driven models, the Discussers would like to make the following general points in relation to typical experimental s_u versus I_L relationships obtained for fine-grained soils. Firstly, decades of research documented in the geotechnical engineering literature have consistently demonstrated that the plot of $\log s_u$ against I_L can exhibit significant non-linearity over the plastic range (of $0 < I_L < 1$) (e.g., Skempton and Northey 1952; Houston and Mitchell 1969; Wood 1985; Wasti and Bezirci 1986; Stone and Phan 1995).

Secondly, while the Authors define the LL as "...the boundary limit water content value where the shear strength of soil is approximately zero..." (Page 3109 of their paper), the Discussers would point out that the LL state transition (i.e., at I_L = 1), as determined experimentally using the Casagrande-cup or fall cone (FC) testing approaches, has a strength-based definition (Haigh 2012; O'Kelly et al. 2018). Specifically, the Casagrande-cup LL (BS 1377–2 1990; ASTM D4318 2017) is associated with a s_u (i.e., s_{uLL}) value of 1–3 kPa (O'Kelly 2019), while the British Standard (BS) 30°–80 g FC LL method (BS 1377–2 1990) corresponds to a s_{uLL} value of 1.7 kPa (O'Kelly et al. 2018). In other words, assigning a physically meaningful s_{uLL} value (of 1–3 kPa) for I_L = 1 is an important consideration in calibrating $s_u = f(I_L)$ models.

Whereas Atterberg's PL is determined using a ductility test (O'Kelly et al. 2018, 2023; O'Kelly 2024), in which the PL (i.e., ductile–brittle state transition) is defined as the lowest water content for which a uniform fine-grained soil thread first begins to crumble when manually rolled-out on a glass plate to about 3 mm in diameter (BS 1377–2 1990; ASTM D4318 2017). Hence, in considering a diverse range of fine-grained soils, it is well brought out in the research literature that the value of s_u associated with Atterberg's PL (i.e., s_{uPL}) can vary over a wide range (Nagaraj et al. 2012; Haigh et al. 2013; O'Kelly 2013a). For instance, from analyses of measured s_{uPL} results pertaining to 71 different fine-grained soils compiled from seven independent investigations, Haigh et al. (2013) reported their mean s_{uPL} and standard deviation as 152 and 89 kPa, respectively. Accordingly, the value of the s_{uPL}-to-s_{uLL} ratio can vary over a wide range (Nagaraj et al. 2012; Haigh et al. 2013; O'Kelly 2013a; Vardanega and Haigh 2014). For example, considering the data for ten dissimilar fine-grained soils, O'Kelly (2013a) deduced that their s_{uPL}/s_{uLL} ratios ranged between 43 and 128, with a mean of 75. It is important to note that the 'PL' parameter derived for strength-based definitions, e.g., using a 30°–76 g FC at 2-mm penetration (according to the Chinese code), is distinctly different from the almost universally adopted Atterberg PL parameter (Vardanega et al. 2020; O'Kelly 2024); any agreement found between their experimental values for a given fine-grained soil being essentially coincidental (Haigh et al. 2013; O'Kelly 2013a; Sivakumar et al. 2016; O'Kelly et al. 2022).

Consequently, in considering dissimilar fine-grained soils, the relationship between their s_u and I_L is not unique (Houston and Mitchell 1969; O'Kelly 2019); rather, their relationships can be quite different, influenced by the soils' physicochemical attributes (e.g., gradation, plasticity properties and mineralogical activity) and also, to some extent, by the specific measurement methods employed for determining s_u, and the LL and Atterberg's PL (O'Kelly et al. 2018), with I_L = LL − PL. Concerning the latter point for s_u, since different strength testing devices (e.g., FC, vane shear, unconfined compression, triaxial compression and pocket penetrometer) approach s_u measurement in different ways, identical soil samples tested for a given water content using an array of such devices can mobilize varying s_u values, arising from differences in the test-specimen shearing modes, shear rates and confinement pressures (if applied) (O'Kelly 2013b, 2014; O'Kelly and Soltani 2022). Accordingly, $s_u = f(I_L)$ and/or $s_u = f$(WCR) models developed using (and applied for) uniform calibration datasets — i.e., those that exclusively include experimental results obtained from similar s_u, LL and PL testing methods — generally exhibit superior predictive performance.

4 Critique of the Authors' Assembled Database and Their Proposed $s_u = f(I_L)$ Correlation

Before commenting in greater depth on the Authors' adjudged best-performing $s_u = f(I_L)$ correlation (i.e., Eq. 9 in their paper), the Discussers would like to make the following points regarding the Authors' assembled database. Their compiled s_u data (used for calibrating Eq. 9) were obtained using six different strength measurement devices/methods (i.e., FC, vane shear, unconfined compression, triaxial compression, pocket penetrometer and viscometer). However, as discussed in Section 3, the overall predictive performance of data-driven models is dependent on the uniformity of the calibration database from which they are developed. That is, for improved reliability and generalizability of such models, it is preferable to exclusively include s_u and I_L results obtained using similar testing methods in building the calibration database (Vardanega and Haigh 2014). The inclusion of heterogeneously measured s_u data can introduce bias, increase scatter and ultimately compromise the overall validity and practical utility of the resulting correlation. Additionally, the Discussers note that the Authors' database, as depicted in Fig. 2a of their paper, contains some spurious s_u:I_L results. For instance, compared to the accepted s_{uLL} range of typically 1–3 kPa for the Casagrande-cup LL (O'Kelly 2019), and s_{uLL} of 1.7 kPa for the BS FC LL (O'Kelly et al. 2018), Fig. 2a of their paper includes s_{uLL} data points ranging from 0.7 to 12.0 kPa (for I_L = 1). Other spurious data plotted in this figure include super-high s_{uPL} values of up to 530 kPa (for I_L = 0).

Having set the scene, the Discussers begin with a critique of the Authors' assertion that their Eq. 9 produced the best prediction performance (for their assembled database), based on its high R^2 value of 0.911 (obtained for the NLR analysis). It is widely recognized that the R^2 statistic alone (even when it approaches unity) is neither sufficient nor a reliable basis for assessing the predictive accuracy of data-driven models (Vardanega and Haigh 2014; Soltani and O'Kelly 2022). In Table 7 of their paper, the Authors' Eq. 9 is listed as having normalized root mean square error (NRMSE) and mean absolute percentage error (MAPE) values of 12.30% and 14.07%, respectively. These values, which exceed the usual/desirable 5%–10% reference limit, indicate an average offset of up to ~14% associated with the s_u predictions made by Eq. 9 (over its calibration/application domain of $0.15 < I_L < 3.00$). More appropriately, rather than relying solely on these absolute fit-measure indices, which do not capture/explain the direction of bias associated with the predictions (i.e., tendency for over- or under-estimation), the true potentials and/or limitations of Eq. 9 (including its implications when applied in geotechnical design) can be better understood by carefully examining its prediction residuals (Soltani and O'Kelly 2021).

Considering its mathematical domain of definition, the Discussers are concerned with Eq. 9's ability to produce reliable s_u predictions within lower I_L ranges of its calibration/application domain (e.g., $0.15 < I_L < 0.40$), which is of primary relevance to many routine geotechnical design scenarios. In this context, Fig. 1 of the present article illustrates a typical example of prediction residuals (expressed as % error) plotted against I_L for the s_u predictions produced by the Authors' Eq. 9. Note that the experimental s_u:I_L data employed in this analysis were sourced from the studies by Kayabali and Tufenkci (2010) and Spagnoli and Feinendegen (2017), which constitute a representative subset/sample of the Authors' assembled database, comprising 123 vane-shear s_u measurements for 55 different fine-grained soils, with LL = 24.6%–142.5%, PL = 12.2%–49.9% and I_L = −0.02 to 0.99. Much like the s_u:I_L data and trend-

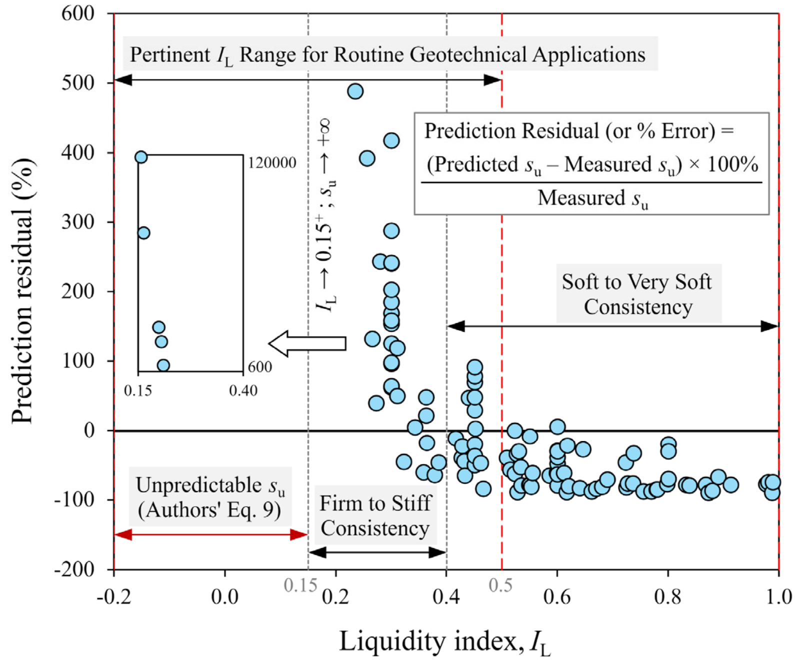

Fig. 1 Prediction residuals (expressed as % error) plotted against I_L for the s_u predictions produced by the Authors' Eq. 9. The experimental s_u:I_L data employed in this analysis were sourced from Kayabali and Tufenkci (2010) and Spagnoli and Feinendegen (2017).

curves plotted in Fig. 2a of the Authors' paper (for their entire assembled database), it is evident that for those fine-grained soils having general firm to stiff consistencies (i.e., $0.15 < I_L < 0.40$), Eq. 9 tends to increasingly overestimate s_u for reducing I_L. The mean percentage error (MPE) for the data points falling within this I_L domain can be calculated as +4022%, implying that Eq. 9 may, on average, overestimate s_u by a factor of ~40. If the prediction domain is limited to $0.3 < I_L < 0.4$, the MPE decreases to +105%, with this improvement attributed to the reduced influence of eliminated data points in the vicinity of Eq. 9's singularity (at $I_L = 0.15$), where predictions are highly sensitive and prone to large errors (i.e., MPE = +13423% for $0.15 < I_L < 0.30$). From a geotechnical engineering design perspective, this remains a critical concern. For example, irrespective of footing geometry and/or load inclination, the ultimate net bearing capacity (q_u) for the short-term (undrained) loading condition of shallow foundations resting on fine-grained soil deposits is directly proportional to their s_u (Budhu 2011) — that is, $q_u = 5.14 \times s_u \times S_c \times D_c \times B_c \times I_c \times G_c$ (where S_c, D_c and B_c = correction factors related to the footing's shape, embedment depth and base inclination, respectively; and I_c and G_c = load and ground inclination factors, respectively). Hence, based on the sample analysis presented in Fig. 1, relying on the Authors' Eq. 9 for soils having $0.15 < I_L < 0.30$ and $0.3 < I_L < 0.4$, respectively, could, on average, result in overestimating q_u by a factor of ~135 and ~2, respectively, thereby leading to (grossly) unsafe design decisions.

When developing $s_u = f(I_L)$ correlations applicable to the full plastic range (i.e., $0 < I_L < 1$), SIPF type expressions (i.e., of the form of Eq. 1) are not suitable. Instead, exponential or regular-power type models (e.g., equation rules given by $s_u = ae^{bI_L}$ and $s_u = aI_L{}^b$, respectively; where a and b are the fitting parameters) are typically adopted (e.g., Koumoto and Houlsby 2001; Vardanega and Haigh 2014; O'Kelly 2024). This is demonstrated in Fig. 2 of the present article, which illustrates the Authors' Eq. 9, alongside their exponential model, given as Eq. 7 in their paper (reproduced in the present article as Eq. 2). Also plotted in this figure is the Leroueil et al. (1983) $s_u = f(I_L)$ correlation (i.e., given by Eq. 1 in the present article, applicable for $0.5 < I_L < 2.5$, with $a = 2.00$ and $x = 0.21$), and, as examples of additional $s_u = f(I_L)$ correlations, those models reported by Whyte (1982) and Spagnoli and Feinendegen (2017), with the latter two being exponential-type prediction models (i.e., given as Eqs. 3 and 4, respectively, in the present article). Note that Eq. 4 was deduced by the Discussers from combining Eqs. 2 and 3 presented in the paper of

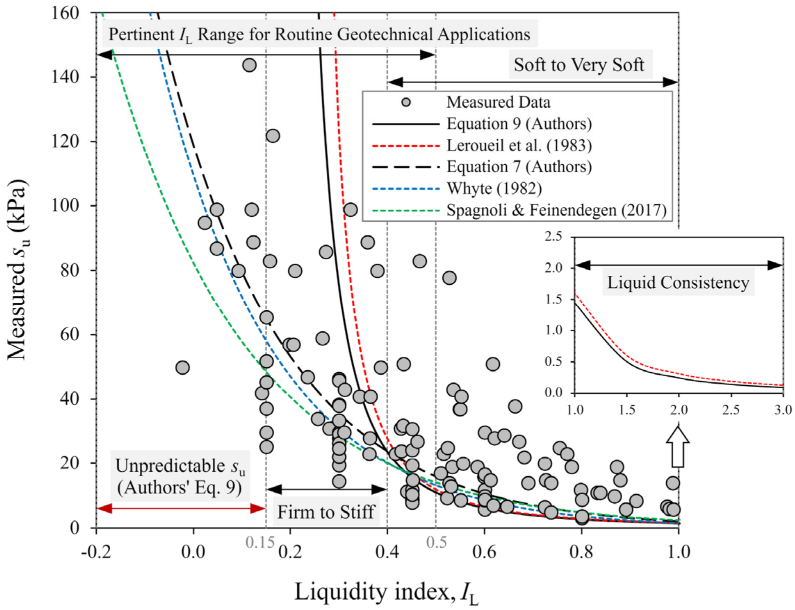

Fig. 2 Comparison of the Authors' Eqs. 7 and 9 with selected $s_u = f(I_L)$ correlations reported in the research literature. The measured s_u:I_L data were sourced from Kayabali and Tufenkci (2010) and Spagnoli and Feinendegen (2017).

Spagnoli and Feinendegen (2017).

$$s_u = 119.283\ e^{-4.020\ I_L} \tag{2}$$

$$s_u = 1.6\ e^{4.23\ (1-I_L)} \tag{3}$$

$$s_u = 597.82\ e^{\frac{-5.131\ (I_L+0.56)}{1.45}} \tag{4}$$

Based on the same data used in Fig. 1, the percentage error associated with the s_u predictions produced by the Authors' exponential model (or their Eq. 7) for soils having $0.15 < I_L < 0.40$ was found to range from −68% to +144%, with mean (i.e., MPE) and median values of ~0 and −6%, respectively. These values are markedly more conservative than those associated with Eq. 9 (i.e., min.=−64%, max.=+111687%, MPE=+4022% and median=+143% considering $0.15<I_L<0.40$; and min.=−64%, max.=+417%, MPE=+105% and median=+97% for $0.3 < I_L < 0.4$). Similarly desirable error values can also be obtained for the Whyte (1982) and Spagnoli and Feinendegen (2017) correlations (i.e., given by Eqs. 3 and 4, respectively), with corresponding min. = −73% and −74%, max. = +112% and +95%, MPE = −14% and −21%, and median = −18% and −26%. Also visually evident in Fig. 2, within the I_L range of 0.4 to 1.0, all five $s_u = f(I_L)$ models exhibit similarly conservative performance, i.e., Eqs. 9 and 7 produce MPE values of −49% and −24%, respectively, while the Leroueil et al. (1983), Whyte (1982) and Spagnoli and Feinendegen (2017) correlations achieve MPE values of −44%, −38% and −30%, respectively.

Considering all the above, it is the Discussers' viewpoint that the Authors' Eq. 9 should be restricted for use with I_L values of greater than 0.4 (rather than 0.15, as reported in their paper). However, since this equation predicts similar s_u values for fine-grained soils of soft and very-soft consistencies as those obtained using Leroueil et al. (1983)'s $s_u = f(I_L)$ correlation (i.e., see the closeness of their two trend-curves for $I_L > 0.4$ in Fig. 2 of this article), it would seem that there is no significant benefit gained in adopting the Authors' Eq. 9 over Leroueil et al. (1983)'s long-established $s_u = f(I_L)$ correlation.

5 Conclusions and Recommendations

When developing data-driven correlations to reliably predict the s_u mobilized by remolded fine-grained soils as a function of their I_L, the following key considerations should be taken into account:

(1) It is essential to recognize the applicable I_L range (i.e., of typically $-0.2 < I_L < \sim0.5$) for most routine geotechnical engineering design applications, hence ensuring the model's applicability and relevance to real-world geoengineering behavior.

(2) A well-curated calibration database must be assembled, with careful attention to uniformity in laboratory testing methods (e.g., for LL, PL and s_u determinations), and the removal of potential outliers that contradict basic soil mechanics principles (e.g.,

with rational s_u confined to the range of 1–3 kPa for $I_L = 1$).

(3) The selection of an appropriate $s_u = f(I_L)$ expression is critical, particularly in terms of its mathematical domain of definition and context-specific applicability. SIPF $s_u = f(I_L)$ models are generally suitable for fine-grained soils of soft to very-soft consistencies (e.g., $I_L > 0.4$ for the Authors' Eq. 9), with their upper applicability bound guided by the calibration database from which they are developed. Exponential-type $s_u = f(I_L)$ models, on the other hand, can generally be applied to cover the full plastic range (i.e., $0 < I_L < 1$), i.e., encompassing fine-grained soils of very soft to (very) stiff consistencies, with their precise upper and lower applicability bounds solely dependent on the database used for their calibration.

(4) Besides assessing routine fit-measure indices, a critical examination of the model's prediction residuals should also be performed, allowing one to better understand the implications of the predictions considering various geotechnical engineering design scenarios (i.e., ensuring that the predictions are reasonably conservative).

Finally, while I_L serves as a valuable index parameter for capturing the consistency-dependent behavior of remolded fine-grained soils, it alone may not be sufficient to account for the range of factors influencing s_u mobilization. For instance, when considering a diverse range of fine-grained soils, it is well established in the research literature (e.g., Nagaraj et al. 2012; Haigh et al. 2013; O'Kelly 2013a) that the s_u mobilized at Atterberg's PL (i.e., for $I_L = 0$) can vary over a wide range of values. Accordingly, to enhance s_u predictive capability, the development of segmented/piecewise $s_u = f(I_L)$ models — tailored to distinct I_L domains and/or informed by soil-specific characteristics, including grain-size distribution and mineralogical activity [e.g., inferred indirectly through sediment volume testing (Prakash and Sridharan 2004, 2010; Soltani et al. 2024)] — is strongly recommended.

Author Contributions

The authors contributed equally to the study conception and design. Both authors read and approved the final manuscript.

Ethics Declaration

Availability of Data/Materials: No new data were created or analyzed in this study. Data sharing is not applicable to this article.

Conflict of Interest: The authors declare no conflicts of interest.

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