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Preliminary communication

Fall cone test calibration for slope stability analysis

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Abstract

Undrained shear strength is considered as an essential factor to assess the stability of natural slopes and embankments. Traditionally, this parameter is determined through laboratory tests, which are often costly and time-consuming. Recently, geotechnical research has shifted toward using the Fall Cone Test (FCT) for short-term strength estimation. However, prior studies focusing on pure clay soils have often overlooked the combined impact of soil texture and stress conditions on the reliability of FCT-derived values. To address this gap, undrained shear tests were conducted using both the FCT and pocket vane shear devices on samples exhibiting various degrees of consolidation and different textures. The findings demonstrate that applying a constant cone factor of 0.8 in the FCT yields inconsistent results compared to vane shear tests, as this factor fails to reflect the soil's consolidation state and the texture effect. To overcome this limitation, a new empirical expression for the cone factor, based on the overconsolidation ratio and fines content, is suggested. The results indicate that this approach significantly improves the accuracy of undrained resistance calculations by the FCT, thus providing a refined methodology for better risk management related to slope and embankments instability.

Keywords:
fall cone test, pocket shear vane test, overconsolidation ratio, fines content, undrained shear strength

1. Introduction

Fall cone test (FCT) has emerged as a prominent tool to evaluate the Atterberg limits of clayey materials (**Spagnoli, 2012; Nini, 2014; Niazi et al., 2019; Vardanega et al., 2022; Marušić & Jagodnik, 2023; Malki & Abed, 2024; Marušić & Jagodnik, 2025**). This method provides accuracy comparable to the Casagrande test while offering a key advantage by minimizing the human error associated with subjective blow count measurements (**Shimobe & Spagnoli, 2020**). Since the late 1950s, the FCT has evolved to serve as a valuable tool for estimating undrained shear strength (s_u) and quantifying sensitivity of both disturbed and undisturbed cohesive soils (**Wood, 1985; Tanaka et al., 2012**). The s_u values derived from the FCT align well with those obtained from direct shear tests conducted at low moisture content, as well as from laboratory vane shear tests (**Canelas et al., 2018**).

The efficacy of the FCT performance is intrinsically linked to its physical setup. Factors such as shape and weight of the cone are rigorously studied and defined by established standards (**Shimobe & Spagnoli, 2019**). For example, the commonly used 30° cone weighing 80 grams is prevalent in the United Kingdom, New Zealand, Australia and France, whereas the 60° cone weighing 60 grams is more frequently employed in Scandinavian nations (**Shimobe, 2010**). Beyond these physical specifications, the accurate estimation of s_u relies heavily on the cone factor (K), which is known to be significantly influenced by additional parameters (**Hansbo, 1957; Karlsson, 1961; Wroth & Wood, 1978; Houlsby, 1982; Koumoto & Houlsby, 2001**). Research has demonstrated that these factors include; the shearing rate and clay sensitivity, as well as the cone's surface roughness and the intrinsic cone angle influence, which is explained through fundamental energy conservation principles (**Wood, 1982; Wood, 1985; Sivakumar et al., 2015; Farias & Llano-Serna, 2016; Vardanega et al., 2018; Llano-Serna & Contreras, 2020; Zeng et al., 2020**).

Despite extensive research, the literature presents a wide and inconsistent range of K values (**Karlsson, 1961; Medhat & Whyte, 1986; Brown & Huxley, 1996; Sharma & Bora, 2003; Kyambadde, 2010**). A value of 0.8 is currently recommended by (**EN ISO 17892-6, 2017**). This variability is largely attributed to

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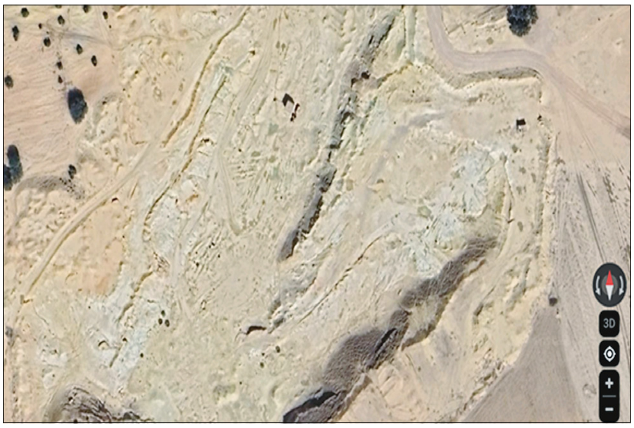

Figure 1. Bouihi quarry (North West of Algeria) – Google Map (34.42166, -1.690772)

Figure 2. Clay deposit (North West of Algeria) – Google Map (35.11198, -1.427161)

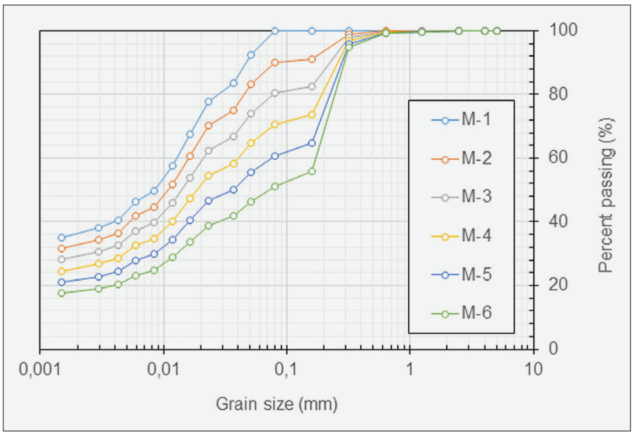

Figure 3. Particle size analysis for all mixtures

Table 1. Geotechnical characteristics of clay and sand

	Clay	Sand
Liquidity limit ω_L (%)	49.75	-
Plasticity limit ω_P (%)	27.30	-
Plasticity index I_P (%)	22.45	-
Specific weight γ_S(kN.m^{-3})	26.30	26.50
Passing 2 mm (%)	100	100
Passing 80 µm (%)	100	1.54
Passing 2 µm (%)	36	/
Coefficient of uniformity Cu	-	2.14
Curvature coefficient Cc	-	0.95

the inexistence of a coherent and standardised methodology of cone factor calibration (**Llano-Serna et al., 2018**). Compounding this issue, the applicability of the Hansbo relationship (**Hansbo, 1957**) is itself questionable for highly consolidated samples (**Stróżyk & Tankiewicz, 2013**), with significant uncertainties remaining in the estimation of s_u for fine-grained soils (**Llano-Serna & Contreras, 2020**), even with proposed modifications to account for strain rate effects (**Hazell, 2008**).

A critical limitation in much of the existing research is the tendency to focus primarily on idealised pure clay samples. This approach fails to capture the complexity of in situ reality, where natural soils often contain varying amounts of silt and sand. This granulometric variability can markedly affect their geotechnical properties (**Terzaghi, 1925; Gilboy, 1928; Lees, 1964; Olson & Mesri, 1970; Abbireddy et al., 2009; Clayton et al., 2009; Göktepe & Sezer, 2010; Cabalar et al., 2013a; Cabalar & Hasan, 2013; Cabalar, 2018**), a challenge particularly evident in studies examining the impact of clay-sand mixtures on s_u (**Cabalar et al., 2020**). Given the research gaps, this investigation intends to examine the combined influence of soil texture (i.e., the fines content) and overconsolidation ratio on cone factor, and their implications for the accuracy of undrained shear strength estimation.

2. Investigated materials and methodology

2.1. *Origin and identification*

This geotechnical investigation primarily utilised two materials which are identical and reported previously (**Chalabi et al., 2024**). Sand, characterized by its granularity and absence of cohesion, is commonly integrated as a plasticity modulator in geotechnical engineering in line with various established study practices (**Louafi & Bahar, 2012; Amri et al., 2019; Bhardwaj & Sharma, 2020; Meddah et al., 2022; Qusai et al., 2025**). Clay, which is a cohesive, fine-grained material capable of undergoing significant consolidation. The combination of these two types of materials can effectively represent a mixture of clay, silt, and sand as described by (**Gökalp, 2009**).

The sandy material was procured from the Bouihi quarry located North West of Algeria (see **Figure 1**). This poorly graded sand (according to USCS (**ASTM D2487-17e1, 2017**)) is widely employed in construction applications, specifically for wall plastering. Meanwhile, The clayey soil utilised was collected adjacent to a ceramic tile manufacturing facility North West of Algeria (see **Figure 2**). This low plasticity clay (according to USCS (**ASTM D2487-17e1, 2017**)) locally designated as "blue clay," constitutes a base material of ceramic wall tiles. **Figure 3** and **Table 1**, provide a summary of the comprehensive geotechnical properties of the employed materials.

2.2. Specimen formulations

This study employed six distinct mixtures for the experimental investigations in accordance with the protocol defined in **Chalabi et al. (2024)**; M-1 (100% Clay + 0% Sand), M-2 (90% Clay + 10% Sand), M-3 (80% Clay + 20% Sand), M-4 (70% Clay + 30% Sand), M-5 (60% Clay + 40% Sand), and M-6 (50% Clay + 50% Sand). The investigation involved testing clay-sand mixtures at varying water contents (ω %) to pinpoint the optimal moisture level that achieves the required physical properties.

Specimens were meticulously fabricated by gradually incorporating water, subsequently statically compacting the resulting mixture into an oedometer ring with a predefined volume of 7.914×10^{-5} m³ (see **Figure 7**). Later analyses were conducted to assess key characteristics as illustrated in **Figures 4a** to **4c**, which demonstrate the relationship between moisture content and material behaviour.

The findings illustrated in **Figure 4a** and **Figure 4b** reveal that a moisture content ranging from 15% to 20% is optimal for achieving peak density in the various formulations, underscoring the role of a specific amount of moisture in attaining the desired density within these mixtures. Conversely, **Figure 4c** shows that a moisture content of 25% induces full saturation in every specimen. Therefore, this consistent moisture content was selected and applied for the constitution of the testing samples. This decision supports the validation of the consolidation criterion based on Terzaghi's postulate (**Terzaghi & Peck, 1967**), enabling a more accurate observation and analysis of the samples' consolidation behaviour.

2.3. Stress history and consolidation protocols

Following the defined mixture moisture specification, cylindrical samples with a volume of 7.914×10^{-5} m³ were prepared (see **Figure 5**) and subjected to a range of pressure conditions. The procedure implemented as described by (**Chalabi et al., 2020**) comprised imposing a consolidation pressure (σ'_c) to the samples, subsequently releasing this pressure to achieve a targeted vertical effective pressure (σ'_v) corresponding to the desired overconsolidation ratio (*OCR*).

In this regard, consolidation cells were employed (see **Figure 6**), designed with mechanical linkages that magnify the applied load tenfold. The overconsolidation ratios OCR selected were 1, 2, 4, and 8. Their corresponding stress paths, defined by (σ'_c [kPa], σ'_v [kPa]), were set as follows; (25,25), (50,25), (100,25), (200,25) as previously established by (**Chalabi et al., 2024**). Throughout the consolidation stage, the imposed pressure exerted upon the soil specimens was systematically incremented by a factor of two at every loading step. This consistent consolidation stress escalation facilitated the meticulous monitoring and evaluation of the soil's behaviour under elevated stress states.

Each transition between loading and unloading increments within the consolidation cells was allotted a duration of 24 hours. This timeframe was crucial for allowing the samples to stabilise and achieve an equilibrium

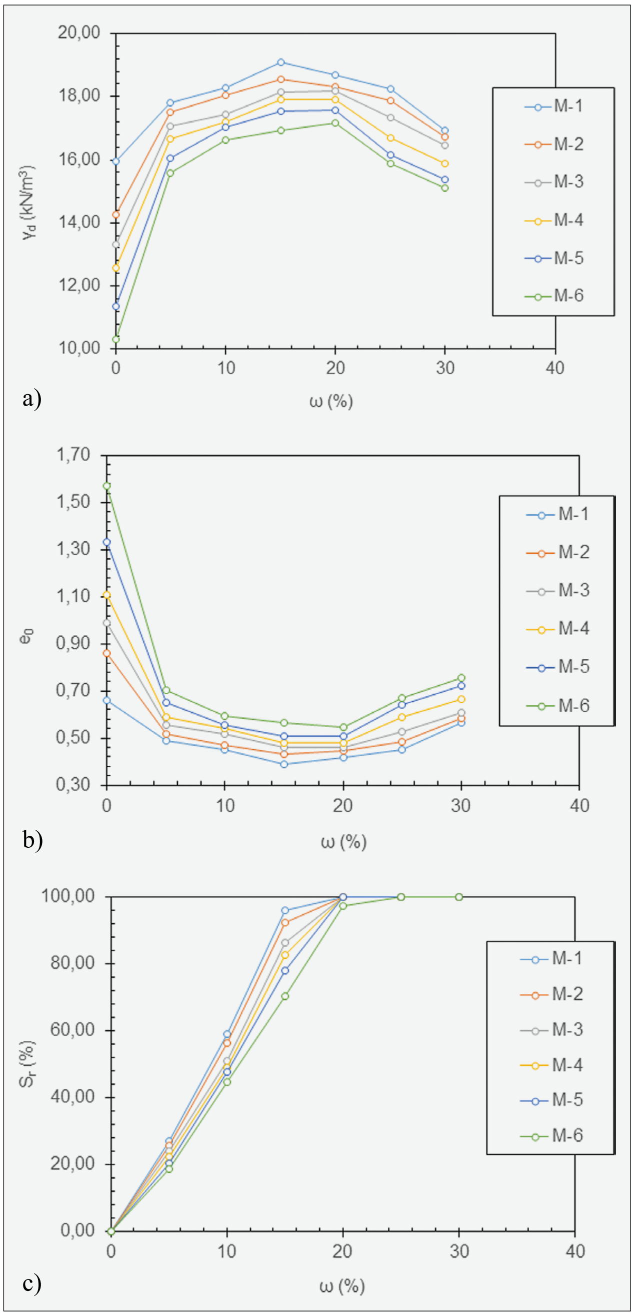

Figure 4. Variation of mixtures behaviour as function of moisture content a) Dry unit weight b) Void ratio c) Degree of saturation.

Figure 5. Soil filled in the cylindrical mold

Figure 6. Sample under consolidation

condition under the newly imposed load. Throughout this 24-hour period, the specimens experienced consolidation mechanisms, characterised by a reduction in voids between particles and a reorganisation of particles to accommodate the new pressure conditions.

3. Results and discussion

The samples of the six consolidated mixtures were subsequently subjected to two types of laboratory experiments; the pocket shear vane (see **Figure 7**) and the fall cone test (see **Figure 8**). These tests are primarily employed to assess s_u, that is to be considered and further examined in subsequent sections of this paper.

3.1. Influence of overconsolidation ratio and fines content on cone penetration

The FCT (see **Figure 8**) is performed by releasing the standardised steel cone, which then penetrates the soil sample by gravitational force alone. Resulting vertical displacement is measured using an integrated dial gauge. The cone used for this test has a 30° angle and a mass of 80 g.

Figure 7. Pocket shear vane test

Figure 8. Fall cone test

As shown in **Figure 9**, cone penetration decreases with an increase in the overconsolidation ratio, irrespective of the mixture composition. This relationship is found to follow a power law, with greater penetration observed in mixtures containing a lower fine content. Furthermore, the results presented in **Figure 10** corroborate that cone penetration is higher for mixtures with a lower fine content, regardless of the overconsolidation ratio. Specifically, an inverse power-law relationship is demonstrated between cone penetration and the fine content, confirming that increasing the proportion of fines leads to a reduction in penetration.

3.2. Comparison of s_u from pocket shear vane and fall cone tests

The pocket shear vane (see **Figure 7**) is a portable instrument equipped with different-sized vanes designed to adapt various soil consistencies. During testing, the

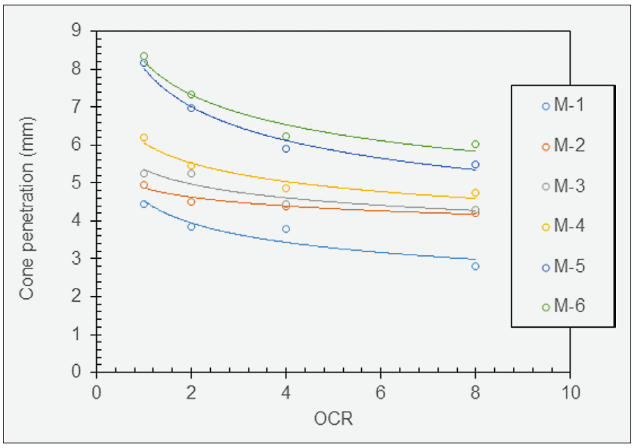

Figure 9. Variation of cone penetration with respect to overconsolidation ratio

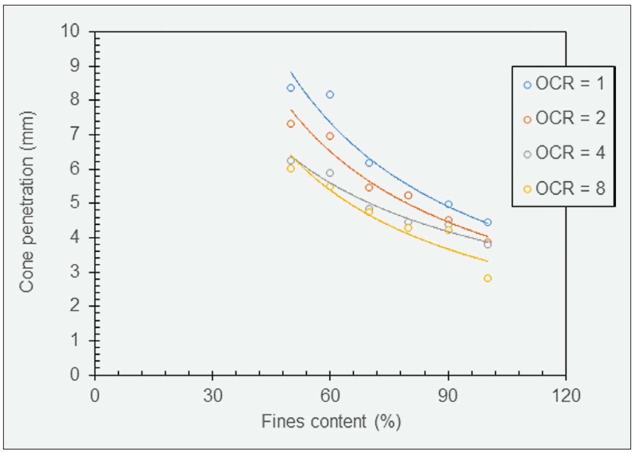

Figure 10. Variation of cone penetration with respect to fines content

selected vane is inserted perpendicularly within the specimen, then subsequently rotated slowly at a uniform speed until failure takes place. The torque at the point of failure is displayed on a dial, and this measurement is then converted to undrained shear strength using a calibration chart.

On the other hand, undrained shear strength by FCT is calculated using the empirical formula (**Equation 1**) proposed by (**Hansbo, 1957**), which correlates it with the penetration depth as follows :

$$s_u = K\, m\, g / H^2 \quad (1)$$

With:

g – Acceleration due to gravity (9.81 $m.s^{-2}$),
m – Cone masse (80 grams),
H – Penetration (in millimeters),
K – Cone factor.

In this study, K is set to 0.8 as recommended by (**EN ISO 17892-6, 2017**).

Figure 11 demonstrates that the undrained shear strength values derived out of FCT are significantly important compared to the measurements yielded by pocket shear vane test. Assuming that the pocket shear vane test provides more appropriate measurements, the RMSE (Root Mean Square Error, as assessed using **Equation 2**) of 1.6 indicates that the estimated values are significantly high, particularly given the narrow range of undrained shear strength (1 to 33 kPa). This discrepancy as quantified by NRMSE (Normalized Root Mean Square Error,estimated using **Equation 3**) of 10%, is attributed to the cone factor K, which is typically set at 0.8 – a value deemed inappropriate for overconsolidated soils with varied textures.

$$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(A_i - B_i)^2} \quad (2)$$

Where:

A_i – i^th^ value of derived s_u by FCT,
B_i – i^th^ value of estimated s_u by pocket shear vane test,
n – Total of measurements.

$$NRMSE = \frac{RMSE}{Y_{max} - Y_{min}} \quad (3)$$

Where:

Y_{max} – Maximum value of s_u by pocket shear vane test,
Y_{min} – Minimum value of s_u by pocket shear vane test.

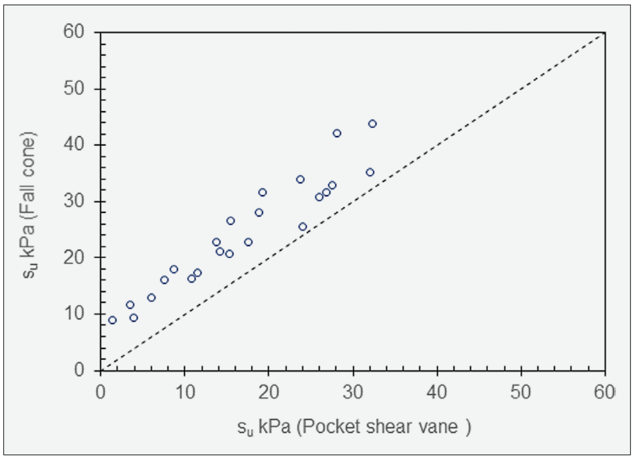

Figure 11. s_u derived by FCT versus pocket shear vane tests.

3.3. *Correlation of cone factor with OCR and fines content*

As previously observed, FCT significantly overestimates s_u values when compared to the pocket shear vane test. To achieve a reliable estimation of this strength, a back-calculation for K factor (see **Equation 6**) is performed for all mixtures, assuming that s_u values estimated by both FCT and pocket shear vane test are equal :

$$s_u(Pocket\ shear\ vane) = s_u(Fall\ cone) \quad (4)$$

$$s_u(Pocket\ shear\ vane) = K\, m\, g / H^2 \quad (5)$$

$$K = s_u(Pocket\ shear\ vane)\, H^2 / m\, g \quad (6)$$

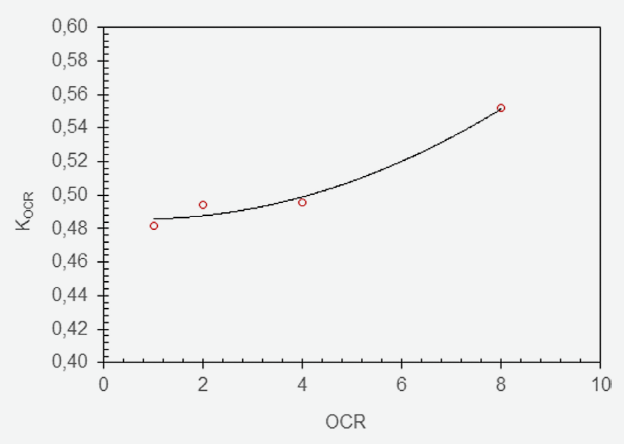

Figure 12. Cone factor expressed by overconsolidation ratio

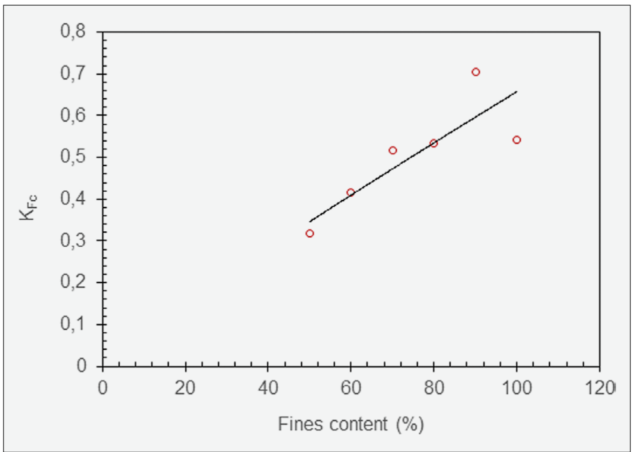

Figure 13. Cone factor expressed by fines content (%)

The back-calculation yielded highly dispersed values, as there are six different cone factor values for each overconsolidation ratio depending on the mixture, making interpretation challenging. Therefore, two specific cone factors are defined. On one hand, the average of the cone factors for all mixtures sharing the same *OCR* is calculated, denoted as K_{OCR}. On the other hand, the average of the cone factors across the entire *OCR* range is calculated for a given mixture, denoted as K_{Fc}.

K_{OCR} is plotted against *OCR* in **Figure 12**. A strong quadratic correlation is observed across the paired parameters, achieving a very good R^2 (coefficient of determination) corresponding to 0.977. The expression for the cone factor as a function of the *OCR* is represented in **Equation 7**. K_{Fc} is plotted against the fines content in **Figure 13**. A good power law correlation is observed among the last enumerated parameters, yielding a R^2 value of 0.782. The resulting empirical expression for the cone factor as a function of fines content is given in **Equation 8**.

$$K_{OCR} = 0.0012\ OCR^2 - 0.0019\ OCR + 0.4864 \quad (7)$$

$$K_{Fc} = 0.0092\ Fc\ (\%)^{0.926} \quad (8)$$

3.4. Application of the proposed cone factors

The evaluation methodology for calculating the undrained shear strength via FCT employed three distinct approaches, incorporating the combinations of K_{OCR} and K_{Fc}; the minimum (*K_Min*), the maximum (*K_Max*), and the mean (*K_Mean*) of the two factors. The FCT calculations for each approach were performed according to **Equation 1** for other cone penetration measurements. The results of this comparative assessment are presented in **Figure 14**, which demonstrates a significant improvement in s_u estimation compared to the use of the conventional constant factor ($K = 0.8$).

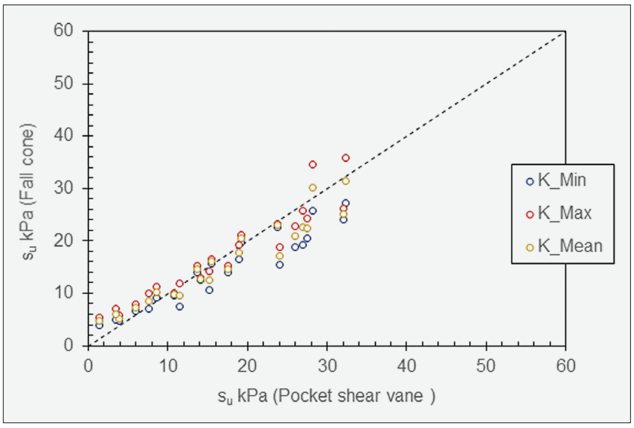

Figure 14. Undrained shear strength by fall cone using combined cone factors versus pocket shear vane tests

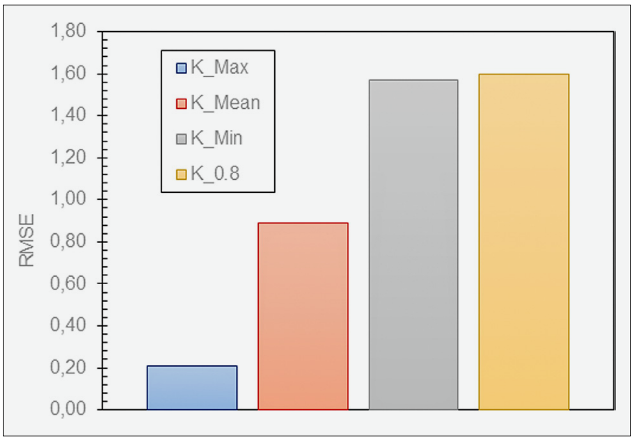

Figure 15. Root Mean Square Error comparison implementing different cone factors

Figure 15 shows that the combination utilising the maximum of the two factors (*K_Max*) yields the lowest RMSE among the other combinations with a value of 0.21. This finding is further validated by the NRMSE, which decreased significantly from 10% for the conventional factor to 1.2%. This outcome clearly validates the reliability and significant predictive accuracy of this combined approach.

4. Conclusions

FCT is commonly employed to evaluate the consistency of clayey materials. However, since the late 1950s, scientific studies have increasingly focused on its application to estimate s_u for stability analysis. A notable gap in previous existing literature is rooted in the limited research on estimating s_u for consolidated soils with varied textures. This study aims to address this by proposing a simple and rapid method using the fall cone for the estimation of short-term strength parameters for slope stability analysis. To achieve this, a testing program was conducted on specimens composed of clay and sand, consolidated at various overconsolidation ratios. The subsequent inferences were developed based on the observed findings:

- A decrease in cone penetration is observed with both an increase in the fine content and a rise in the overconsolidation ratio. This reduction occurs irrespective of the soil's consolidation state in the former case and its texture in the latter.
- A quadratic correlation was observed across *OCR* and cone factor. This factor is expressed as a second-degree polynomial equation achieving a very good R^2 corresponding to 0.977.
- A good power law correlation was observed between the fines content and the cone factor with a coefficient of determination of 0.782.
- The proposed cone factors (*K_Max*), derived based on *OCR* and fines content, demonstrably improved estimated s_u values by the FCT method. This enhancement is quantified by a significant reduction in both RMSE (from 1.6 to 0.21) and NRMSE (from 10% to 1.2%) metrics when compared to the conventional approach utilising a constant cone factor of 0.8.

This study did not investigate the effects of unloading or, more broadly, changes in the soil's confinement state. Therefore, future research should focus on exploring the influence of soil sensitivity within this framework. Furthermore, while the pocket shear vane test was used to calibrate the cone factor in FCT approach for undrained shear strength estimation, the implementation of CPTU (Cone Penetration Test with pore water pressure measurement) inside the scope of this analysis, is highly recommended for rigorous geotechnical studies. The *K_Max* combination suggested in this study could be used with reasonable certainty to evaluate s_u for slope stability analysis.

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SAŽETAK

Kalibracija ispitivanja pomoću padajućega šiljka za analizu stabilnosti kosina

Nedrenirana posmična čvrstoća jedan je od ključnih parametara za procjenu stabilnosti prirodnih kosina i nasipa. Uobičajeno se ovaj parametar određuje laboratorijskim ispitivanjima koja su često skupa i vremenski zahtjevna. U novije vrijeme geotehnička istraživanja sve se više usmjeravaju na primjenu ispitivanja pomoću padajućega šiljka (*Fall Cone Test* – FCT) za procjenu kratkotrajne nedrenirane posmične čvrstoće tla. Međutim, dosadašnja istraživanja, uglavnom provedena na čistim glinovitim tlima, često zanemaruju zajednički utjecaj teksture tla, stanja naprezanja i stanja konsolidacije na pouzdanost vrijednosti nedrenirane posmične čvrstoće dobivenih FCT-om. Kako bi se premostio navedeni nedostatak, provedena su ispitivanja nedrenirane posmične čvrstoće primjenom padajućega šiljka i ispitivanja posmične čvrstoće terenskom krilnom sondom, koja se u normativnoj praksi koristi kao referentna metoda. Ispitivanja su provedena na uzorcima tla različitoga stupnja konsolidacije i različitoga granulometrijskog sastava. Rezultati pokazuju da primjena konstantnoga koeficijenta korelacije vrijednosti 0,8 kod ispitivanja pomoću padajućega šiljka dovodi do nepodudarnih rezultata u odnosu na ispitivanje terenskom krilnom sondom jer takav koeficijent ne odražava stanje konsolidacije tla niti utjecaj granulometrijskoga sastava. Radi prevladavanja navedenoga ograničenja predložen je novi empirijski izraz za koeficijent korelacije koji se temelji na omjeru prekonsolidacije (OCR) i udjelu sitnih čestica u tlu. Dobiveni rezultati pokazuju da takav pristup znatno poboljšava točnost određivanja nedrenirane posmične čvrstoće primjenom ispitivanja pomoću padajućega šiljka, čime se osigurava pouzdanija metodologija za analizu stabilnosti kosina i nasipa te učinkovitije upravljanje geotehničkim rizicima u kratkotrajnim (nedreniranim) uvjetima.

Ključne riječi:
padajući šiljak, terenska krilna sonda, koeficijent prekonsolidacije, udio sitnih čestica, nedrenirana posmična čvrstoća

Author's contribution

Youssouf Chalabi (PhD, Associate Professor) defined the overarching research objectives through conceptualisation and designed the specific experimental and analytical framework, alongside conducting the laboratory testing and interpretation. **Soufyane Aissaoui** (PhD, Associate Professor) provided essential equipment and materials, contributed to investigation through meticulous sample preparation, and provided critical intellectual feedback during writing – review & editing. **Abdeldjalil Zadjaoui** (Full Professor) was responsible for data curation, performed the formal analysis of the experimental results, and contributed to the critical review process. **Sidi Mohammed Aissa Mamoune** (Full Professor) managed secondary data curation and generated the initial manuscript draft.
All authors have read and agreed to the published version of the manuscript.