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Reconciling laboratory shear parameters and SPT-based bearing capacity in a loose saturated alluvial sand: a forensic case study of low-stress direct shear artefacts at the Orle River, Auchi, Nigeria

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Abstract

Site investigation reports frequently derive the shear strength of sands from small-box direct shear tests and carry those parameters straight into bearing capacity calculations. This study shows how such tests can yield physically implausible strength parameters and how those parameters propagate into misleading design values, using a 2026 investigation of a loose, saturated, erosion-derived alluvial sand at the Orle River, Auchi, Nigeria, as a forensic case. The reported allowable bearing pressures (38 values) were first reproduced to identify the

calculation method. The raw shear-box records were then examined, and effective friction angles were derived independently from 28 Standard Penetration Test (SPT) results corrected for hammer energy, rod length and overburden, using two published correlations and energy ratios of 45–75%. Bearing capacities were recomputed like-for-like under effective stress for a shallow pad and at depth, and compared with SPT-based pile toe resistance. The reported values were reproduced exactly by Meyerhof's equation for a 1 m square footing placed at the sample depth, with total unit weight and a factor of safety of 3. The shear-box tests on non-plastic sands gave friction angles of only 6.8–11.1°; they were run at normal stresses of 4.9–14.7 kPa, against 71–262 kPa in situ, and the shear stress was still rising when each test ended. The corrected SPT data (normalised blow counts of 7.7–14.7; relative density 0.36–0.49) indicate effective friction angles of 31–38°, and the gap survives every plausible energy-ratio assumption. For a 1 m pad at 1.5 m depth, the reported parameters give 34–40 kPa, against 159–463 kPa with friction angles of 28–32° under effective stress, an understatement of 5 to 12 times. The large reported capacities at depth (786–2357 kPa) arise from treating the sample depth as the embedment depth, and their agreement with SPT-based pile toe resistance (1067–2400 kPa) is coincidental. Direct shear tests on sands must be run to a defined failure at normal stresses that bracket field conditions and must be checked against in situ estimates; bearing calculations must use effective stress and the actual foundation geometry. Five simple checks are proposed that would catch this class of error in routine practice.

Keywords: Direct shear test; Standard penetration test; Friction angle; Bearing capacity; Effective stress; Alluvial sand; Site investigation quality assurance; Nigeria

Introduction

The drained friction angle, φ', of a clean sand is one of the better-constrained parameters in geotechnical engineering. Depending on mineralogy, grading, particle shape and density, its peak value typically lies between about 28° and 45°, and at low confining stress dilatancy raises the peak value further [1–3]. Because φ' controls every term of the bearing capacity equation for a cohesionless soil, small errors in φ' produce large errors in design capacity: the factor N_q roughly doubles for every 5° increase in φ' in the range of practical interest [4–7].

In routine practice φ' is obtained in two broad ways. The first is from in situ tests, chiefly the Standard Penetration Test (SPT), through empirical correlations that link the corrected blow count to relative density and friction angle [8–11]. The second is from laboratory tests, chiefly the small shear box [12–14]. Each route has known weaknesses. SPT results depend strongly on the energy delivered to the rods, the rod length, the borehole condition and the overburden stress, so that raw blow counts must be corrected before use [15–18]. The direct shear test imposes a non-uniform and partly undefined strain field, and its interpretation depends on the stress level, the rate of displacement and how far the test is carried [19–22]. Specimen size, apparatus friction and the self-weight of the loading system also matter, particularly at low normal stress [23–25]. When both routes are applied correctly to the same clean sand, they should agree within a few degrees; when they do not, the discrepancy usually reveals a procedural problem rather than a property of the soil [26, 27].

Site investigation reports in Nigeria commonly present direct shear parameters for sands together with allowable bearing pressures computed from them, often alongside SPT-based assessments of the same ground [28–31]. Where the shear-box procedure is unsuitable, for example because normal stresses are unrealistically low, the shearing rate is not controlled, or the test is stopped before peak, the error passes directly into the design values. Reviewers and

clients rarely have access to the raw test sheets, so such errors can go undetected and may influence foundation selection, cost and safety. Although the individual mechanisms are well known in the literature, published case histories that trace a specific set of implausible laboratory parameters through to the reported design values, and that quantify the consequences against independent in situ evidence, remain scarce, particularly for West African practice.

This paper presents such a case. A 2026 geotechnical investigation of the erosion-derived sand mass that is silting the Orle River at Auchi, Edo State, reported friction angles of 6.8–11.1° for clean, non-plastic sands. Allowable bearing pressures computed from these parameters ranged from under 50 kPa near the surface to over 2300 kPa at 30 m. The objectives of the study are to: (i) reproduce the reported values and identify the calculation method; (ii) diagnose the cause of the low friction angles from the raw test records; (iii) derive independent friction angles from the SPT data; (iv) quantify how the choice of parameters and method changes the design values; and (v) propose simple checks that would catch such problems in routine practice. The study is intended constructively, as a lesson about test procedure and interpretation that is applicable well beyond the present site.

Methods

Site, ground conditions and data

The site lies at the outlet of the Water Tanker Road gully in the Orle River valley, Auchi, Edo State, Nigeria (7.054–7.057° N, 6.237–6.239° E). Sediment eroded from the gully has been deposited as a loose sand fill within and alongside the river channel. Seven boreholes (BH1–BH7) were advanced to 24–30 m by manual light cable percussion, and groundwater stood at 0.5–3.0 m below ground level. The data examined here were obtained from an unpublished 2026 site investigation report and its raw laboratory sheets; the stratigraphy follows a

companion characterisation study by the authors (unpublished data). Three units are recognised (Table 1).

Table 1 Summary of ground units at the Orle River site

Unit	Depth range (m)	Description	USCS class	Mean bulk unit weight (kN/m^3)	SPT N (blows/300 mm)
I	0 to 3.5–7.0	Loose, non-plastic silty fine sand (recent fill)	SM	18.0	Not tested
II	0–13 m thick	Soft organic clay	OL/OH	16.1	7 (2 tests)
III	To at least 30	Uniform, non-plastic fine to coarse sand	SP, SP-SM, SM	18.4	8–18 (28 tests)

SPTs were performed with a 63.5 kg hammer and a 750 mm drop in general accordance with BS EN 1997-2 and ISO 22476-3 [32–34], at 28 depths in Unit III and two depths in Unit II. Hammer energy was not measured [35].

The report presents cohesion, c, and friction angle, φ, for 38 samples: 33 from small shear-box tests on 100 mm × 100 mm specimens (area 0.01 m^2) and 5 from unconsolidated undrained (UU) triaxial tests on cohesive samples. For the non-plastic sands of Units I and III, the shear box gave c = 0.45–2.0 kPa and φ = 6.8–11.1°. From these parameters the report computed allowable bearing pressures, q_{all}, of 46–54 kPa at 2 m, 179–422 kPa at 6–10 m and 1594–2357 kPa at 24–30 m. The raw shear-box sheets record normal loads of 5, 10 and 15 kg, giving normal stresses of 4.91, 9.81 and 14.72 kPa; a proving-ring constant of 1.21 kg per division; and readings at fixed elapsed times from 5 s to 10 min, with no horizontal displacement recorded.

The overall procedure followed in this study is summarised in Fig. 1.

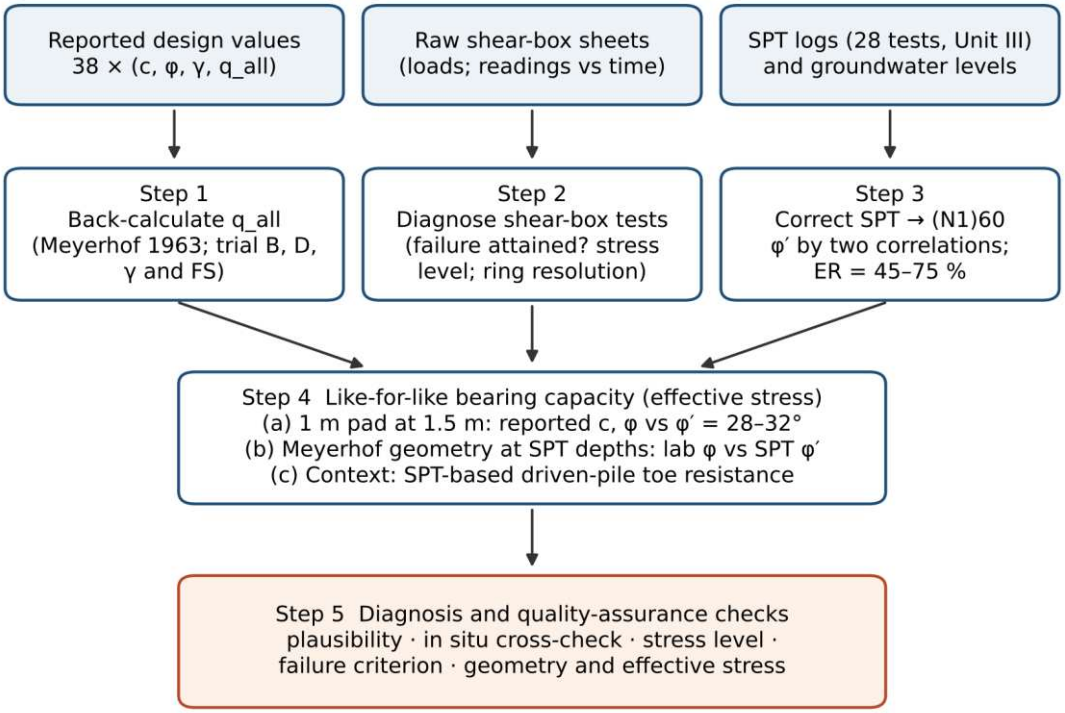


Fig. 1 Workflow adopted to reconcile the laboratory and SPT-based parameters. The five steps correspond to the methods in this section and the checks proposed in the Discussion.

Reproducing the reported bearing pressures

The reported q_{all} values were recomputed with the general bearing capacity equation of Meyerhof [4], assigning to each the c, φ and bulk unit weight, γ, of its sample:

$$q_{ult} = c\,N_c\,s_c\,d_c + \gamma D\,N_q\,s_q\,d_q + 0.5\,\gamma B\,N_\gamma\,s_\gamma\,d_\gamma \qquad (1)$$

The bearing capacity factors are

$$N_q = e^{\pi \tan\phi}\,K_p, \qquad K_p = \tan^2\left(45^\circ + \frac{\phi}{2}\right) \qquad (2)$$

$$N_c = \left(N_q - 1\right)\cot\phi \qquad (3)$$

$$N_\gamma = \left(N_q - 1\right)\tan(1.4\phi) \qquad (4)$$

and the shape and depth factors are

$$s_c = 1 + 0.2\,K_p\,\frac{B}{L}, \qquad s_q = s_\gamma = 1 + 0.1\,K_p\,\frac{B}{L} \qquad (5)$$

$$d_c = 1 + 0.2\sqrt{K_p}\,\frac{D}{B}, \qquad d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\,\frac{D}{B} \qquad (6)$$

where c is cohesion (kPa), φ is the friction angle, γ is unit weight (kN/m^3), B and L are the footing width and length (m) and D is the footing depth (m). The allowable pressure is $q_{all} = q_{ult}/FS$. A trial combination of B = L = 1.0 m, D equal to the sample depth, total unit weight throughout and FS = 3 was tested against all 38 reported values. Other formulations [5, 36, 37] were also tried for comparison.

Diagnosis of the shear-box tests

The time series of shear stress for the specimens from BH4 and BH7 at 8 m were digitised from the raw sheets by converting each proving-ring reading to force with the stated ring constant and dividing by the specimen area. For each specimen, the slope of the curve at the final reading was examined to establish whether failure, defined as a peak or a plateau in shear stress [12, 13, 38], had been reached. The secant friction angle at each normal stress, $\arctan(\tau/\sigma_n)$, was computed, and the number of ring divisions corresponding to the final reading was used to judge measurement resolution. The tested normal-stress range was compared with the in situ vertical effective stress at the depths of the samples and SPTs.

SPT corrections and correlations

Field N values were corrected following Skempton [15]:

$$N_{60} = N\,\frac{E_R}{60}\,C_B\,C_S\,C_R \qquad (7)$$

$$(N_1)_{60} = C_N\, N_{60}, \qquad C_N = \left(\frac{p_a}{\sigma'_v}\right)^{0.5} \le 1.7 \qquad (8)$$

The energy ratio E_R was taken as 60% for the manually released free-drop hammer, with a sensitivity range of 45–75% to reflect the spread reported for such equipment [16–18]. The borehole-diameter and sampler corrections, C_B and C_S, were both taken as 1.0. The rod-length correction, C_R, was taken as 0.75 for rod lengths below 4 m, 0.85 for 4–6 m, 0.95 for 6–10 m and 1.0 above 10 m [39], with rod length taken as depth plus 1 m. The overburden correction C_N follows Liao and Whitman [40], with p_a = 101.3 kPa. The vertical effective stress, σ'_v, was integrated through the units of Table 1 using their mean bulk unit weights and hydrostatic pore pressure below the measured groundwater level.

Relative density was estimated from $(N_1)_{60}/D_r^2 \approx 60$ [15]. Two friction-angle correlations were used:

$$\phi'_{HU} = \sqrt{20\,(N_1)_{60}} + 20 \qquad (9)$$

$$\phi'_{KM} = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\,(\sigma'_v/p_a)}\right]^{0.34} \qquad (10)$$

Eq. (9) is from Hatanaka and Uchida [10] and Eq. (10) from Kulhawy and Mayne [9]. Both are widely used and have been tested against independent data sets [11]; their typical scatter is about ±3–4°.

Like-for-like bearing capacity

Three comparisons were made, each isolating one source of difference:

1. **Shallow pad.** A 1.0 m square pad at 1.5 m depth in Unit I at BH2, BH3, BH4, BH6 and BH7 was analysed using (a) the reported c and φ at 2 m with total unit weight, as in the report; and (b) $c' = 0$ with $\varphi' = 28°$, 30° and 32°, with effective overburden and

the buoyant unit weight $\gamma' = \gamma - \gamma_w$ below the water table. No SPT was made in Unit I, so the φ' range was chosen to bracket a loose fine sand [1, 8, 9] and lies below the values derived for Unit III.

2. **At each SPT depth.** The report's method with the nearest laboratory c and φ was compared with the same Meyerhof geometry using the SPT-derived φ' (Eq. 9) under effective stress. This comparison holds the geometry fixed and isolates the effect of the strength parameters.

3. **Deep foundations, for context.** The base resistance of a driven pile was estimated from Meyerhof [41] as $q_b = 40\ N_{60}\ (L/B) \leq 400\ N_{60}$ (kPa) and divided by FS = 3. For embedment ratios $L/B > 10$ the limiting value $400\ N_{60}$ governs.

All calculations were scripted in Python 3 with NumPy and Matplotlib, and every reported value was recomputed individually.

Results

Identification of the reported method

The trial method reproduced every one of the 38 reported q_{all} values to within rounding: the ratio of the computed q_{ult} to the reported q_{all} ranged from 2.98 to 3.03, consistent with FS = 3. None of the alternative formulations reproduced the full set. Two features of the identified method are critical. First, the footing depth D was set equal to the sample depth, so that a 1 m pad was notionally founded at 20 or 30 m, and the overburden term $\gamma\ D\ N_q\ s_q\ d_q$ dominates at depth. Second, total unit weight was used throughout, although the water table lies only 0.5–3.0 m below ground. Below the water table this overstates the overburden stress by a factor of about $\gamma/\gamma' \approx 18.4/8.6 \approx 2.1$.

Why the shear-box angles are low

The shear-box tests did not reach failure (Fig. 2a). In every specimen examined, the shear stress was still rising at the final 10 min reading; for example, at σ_n = 14.72 kPa in BH4 at 8 m it rose from 3.12 to 3.22 kPa over the last minute. No peak or plateau was recorded, and no horizontal displacement was logged, so the displacement attained when the test was stopped is unknown.

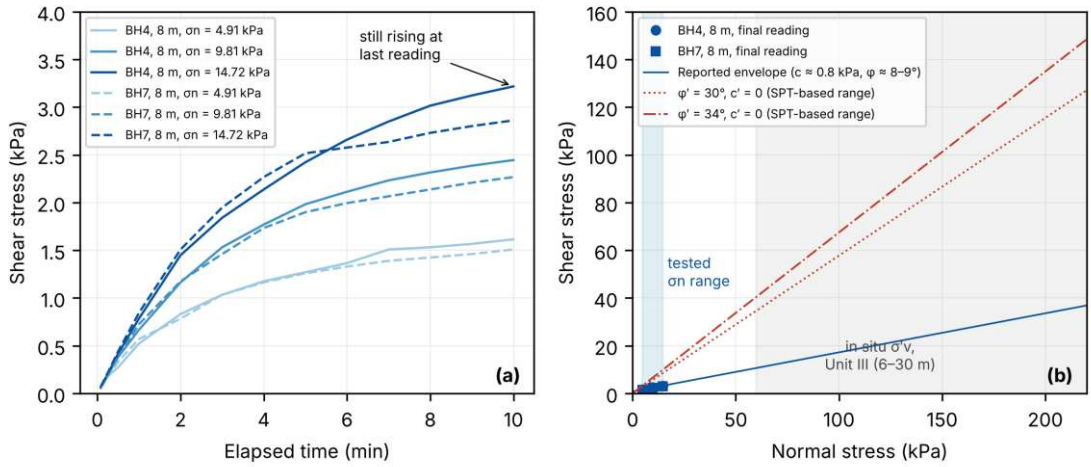


Fig. 2 Shear-box records for BH4 and BH7 at 8 m. **a** Shear stress against elapsed time at three normal stresses, showing that no specimen reached a peak or plateau. **b** Final readings with the reported envelope, compared with envelopes for φ′ = 30° and 34°, the tested normal-stress range and the in situ stress range of Unit III.

Table 2 summarises the test conditions. The secant angles at the highest normal stress were only 11.0–12.3°, and the corresponding stress ratios τ/σ_n of about 0.2 compare with values well above tan 30° ≈ 0.58 expected for a sand sheared to peak at such low stress, where dilatancy is strongly expressed [1, 25, 42]. The tested normal stresses (4.9–14.7 kPa) are also about 5–53 times lower than the in situ vertical effective stress in Unit III (71–262 kPa at the SPT depths). At these loads, seating errors, friction between the box halves and proving-ring

resolution form a large share of the measured force [23, 38]: with a ring constant of 1.21 kg per division, the final readings correspond to only 1.3–2.7 divisions.

Table 2 Shear-box test conditions and final readings (BH4 and BH7, 8 m) compared with in situ stress

Normal load (kg)	σ_n (kPa)	Final τ, BH4 (kPa)	Final τ, BH7 (kPa)	Secant angle (°)	Ring divisions at final reading	In situ σ'_v, Unit III (kPa)
5	4.91	1.62	1.51	17.1–18.3	1.3–1.4	71–262
10	9.81	2.45	2.27	13.0–14.0	1.9–2.1	71–262
15	14.72	3.22	2.86	11.0–12.3	2.4–2.7	71–262

The secant angle falls as normal stress increases, which is the pattern expected when a roughly constant, displacement-limited resistance is divided by a growing normal stress, not the pattern of a frictional material at failure. Linear regression through such points necessarily returns a small apparent cohesion and a very low friction angle, which is exactly what the report presents.

SPT-based parameters

The corrected SPT data indicate a loose to medium-dense sand with φ′ of about 31–38° (Table 3, Fig. 3).

Table 3 SPT-derived parameters for Unit III compared with the reported direct shear values

Parameter (Unit III, n = 28)	Range	Mean
Field N (blows/300 mm)	8–18	13.5
σ'_v at test depth (kPa)	71–262	–
N_{60} (E_R = 60%)	8.0–18.0	13.5
$(N_1)_{60}$	7.7–14.7	10.5
Relative density, D_r	0.36–0.49	0.42
φ′, Hatanaka–Uchida, Eq. (9) (°)	32.4–37.1	34.4
φ′, Kulhawy–Mayne, Eq. (10) (°)	31.5–37.7	33.4

Parameter (Unit III, n = 28)	Range	Mean
φ′, Hatanaka–Uchida, E_R = 45–75% (°)	30.8–39.2	–
φ, direct shear, reported (Units I and III) (°)	6.8–11.1	9.4

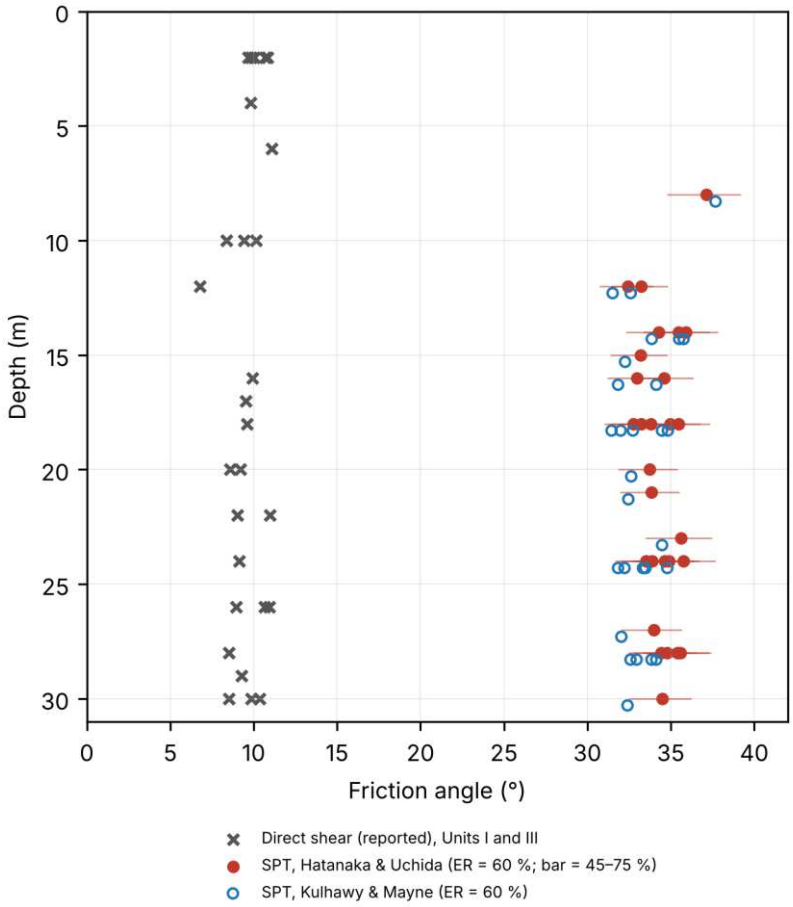


Fig. 3 Reported direct shear friction angles compared with SPT-derived φ′ from the Hatanaka–Uchida and Kulhawy–Mayne correlations, plotted against depth. Horizontal bars show the effect of energy ratios from 45% to 75%.

Even at the lowest energy ratio considered, the smallest SPT-derived angle (30.8°) is almost three times the largest direct shear value (11.1°). The two correlations agree within about 2°, and the ranking of the two data sets is insensitive to every assumption in the correction chain, including the energy ratio, the rod-length correction and the choice of overburden normalisation.

221 ***Effect on bearing capacity***

222 Table 4 shows why the parameters matter so much. At $\varphi = 9°$ the factor N_q is 2.25, against
223 18.4–29.4 at $\varphi' = 30–34°$, an order of magnitude difference; N_γ differs by about two orders of
224 magnitude.

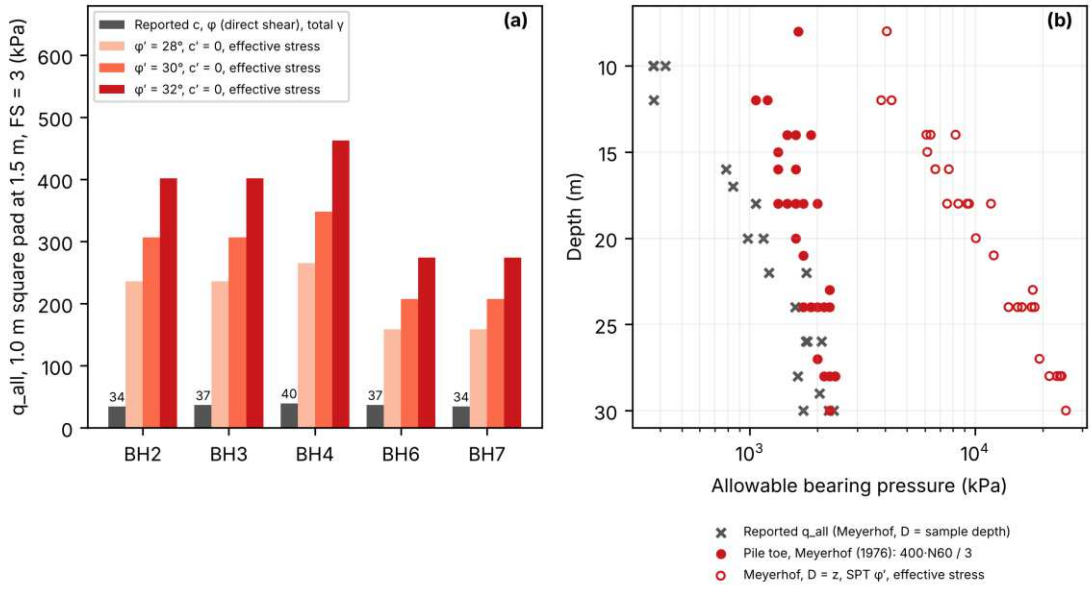
225 **Table 4** Meyerhof bearing capacity factors at the reported and SPT-based friction angles

φ (°)	K_p	N_c	N_q	N_γ	Basis
9	1.37	7.92	2.25	0.28	Reported direct shear (mean)
11	1.47	8.80	2.71	0.47	Reported direct shear (maximum)
28	2.77	25.80	14.72	11.19	Lower bound, Unit I (assumed)
30	3.00	30.14	18.40	15.67	Central value, Unit I (assumed)
32	3.25	35.49	23.18	22.02	Upper bound, Unit I (assumed)
34	3.54	42.16	29.44	31.15	Mean SPT-based, Unit III
37	4.02	55.63	42.92	53.27	Upper SPT-based, Unit III

226 **Shallow pad.** For a 1.0 m square pad at 1.5 m depth, the reported parameters give q_{all} = 34–40
227 kPa. With $c' = 0$ and $\varphi' = 28–32°$ under effective stress, q_{all} is 159–463 kPa, that is, 4.7–11.6
228 times higher (Fig. 4a). The lowest values occur at BH6 and BH7, where the water table is at
229 0.5 m and the buoyant unit weight governs both the overburden and the self-weight terms.

230 **At depth.** With the same Meyerhof geometry (D equal to sample depth), replacing the
231 laboratory parameters by the SPT-derived φ' under effective stress raises q_{all} by a factor of
232 6.7–24 (median 10.4). Neither set of numbers is a meaningful design value, because a 1 m pad
233 cannot be founded at 20 m; the comparison is presented only to isolate the effect of the
234 parameters.

235 **Deep foundations.** Meyerhof's [41] toe resistance for a driven pile in Unit III gives allowable
236 values of 1067–2400 kPa, close to the reported q_{all} of 786–2357 kPa at 16–30 m (Fig. 4b).



237

238 **Fig. 4** Bearing capacity comparisons. **a** Allowable pressure for a 1.0 m square pad at 1.5 m

239 depth from the reported parameters and from φ′ = 28°, 30° and 32° under effective stress. **b**

240 Reported q_{all} against depth compared with SPT-based driven-pile toe resistance and with the

241 like-for-like Meyerhof result using SPT-derived φ′.

242 Table 5 brings the comparisons together.

243 **Table 5** Summary of bearing capacity comparisons (FS = 3)

Case	Reported method and parameters (kPa)	Re-analysis (kPa)	Ratio, re-analysis/reported
1.0 m pad at 1.5 m, Unit I	34–40	159–463 (φ′ = 28–32°, effective stress)	4.7–11.6
Meyerhof geometry at SPT depths, Unit III	179–2357	SPT-derived φ′, effective stress	6.7–24 (median 10.4)
Driven-pile toe, Unit III, 16–30 m	786–2357	1067–2400 (400 N_{60}/3)	Apparent agreement only

Discussion

Two errors that partly cancel

The reported bearing values combine two errors that act in opposite directions. The first is that the strength parameters are far too low. Friction angles of about 9° make the sand behave in the calculation almost like a soft clay, so that N_q and N_γ are respectively one and two orders of magnitude too small (Table 4). The second is that the geometry is too favourable. Placing the footing at the sample depth and using total unit weight makes the overburden term grow linearly with depth, at about twice the effective-stress rate below the water table. At shallow depth the first error dominates and capacity is understated 5–12-fold. At depth the second error grows until it outweighs the first, producing values of 1500–2400 kPa.

The resemblance between those deep values and SPT-based pile toe resistances (Fig. 4b) is therefore coincidental. A designer who accepted the reported values at depth because they appeared reasonable would be right for the wrong reasons, would understate shallow capacity badly, and would have no rational basis for shaft resistance, settlement, or the negative skin friction (downdrag) that consolidation of the soft Unit II clay will impose on piles [43, 44]. This illustrates a broader point: agreement between a design value and an independent estimate is reassuring only when both are obtained for the same foundation, the same stress state and the same mechanism [45].

Why small-box tests on sand go wrong

Three procedural factors explain the low angles, and each has been documented separately in the literature.

1. **Tests stopped before failure.** The shear stress was still rising after 10 min. Without displacement records it is impossible to tell whether the box had travelled far enough

to mobilise peak resistance, which for a loose to medium-dense sand typically requires several millimetres of relative displacement in a 60–100 mm box [21, 22, 46]. Time-based readings without a controlled displacement rate cannot establish that failure was reached.

2. **Very low normal stress.** At 5–15 kPa the applied forces are only a few kilograms. Friction between the box halves, the weight of the top platen and loading hanger, seating, and proving-ring resolution then become comparable to the soil resistance [23, 24]. At such stresses a correctly run test on sand should show high, not low, stress ratios because dilatancy is unsuppressed [1, 25, 42], but only if the upper half can move and rise freely and the test is carried to peak. The present results, with stress ratios of about 0.2, are the opposite of this expectation.

3. **Unrealistic stress level.** Even a correctly run test at 5–15 kPa characterises the soil at a stress level 5–53 times below the field stress. Because the peak friction angle of sand decreases with increasing mean effective stress [1, 2, 6], the envelope cannot simply be extrapolated to the stresses that govern design.

BS 1377-7, ASTM D3080 and ISO 17892-10 all require shearing at a controlled displacement rate to a defined failure or displacement limit, with the horizontal displacement and vertical movement recorded, at normal stresses chosen to cover the design stress range [12–14]. The Auchi tests met neither the failure nor the stress-range requirement.

Practical checks for site investigation reports

The case suggests five checks that reviewers and clients can apply, most of them without access to the raw data (Table 6).

Table 6 Proposed quality-assurance checks for strength parameters and bearing capacity in site investigation reports

No.	Check	Criterion or action	Would have flagged this case?
1	Plausibility	Question any φ below about 25° for a non-plastic sand (plasticity index 0, fines < 35%) unless explained	Yes (φ = 6.8–11.1°)
2	Cross-check with in situ data	Compute φ′ from SPT or CPT data; investigate differences greater than about 5°	Yes (difference about 20–30°)
3	Stress level	Report normal stresses used and confirm they bracket in situ effective stress	Yes (4.9–14.7 kPa against 71–262 kPa)
4	Failure	Report stress–displacement curves and the failure criterion adopted	Yes (no peak, no displacement record)
5	Geometry and effective stress	Compute capacity for the actual foundation width and depth, with γ′ below the water table; do not tabulate q_{all} against sample depth	Yes (D = sample depth, total γ)

Checks 1 and 2 require only the reported parameters and SPT logs, which are present in almost every Nigerian site investigation report [28–30]. Check 5 requires only that the foundation geometry be stated. Together they would have caught every element of the present problem before it reached design. Incorporating such checks into the review of investigation reports is consistent with the principles of Eurocode 7, which requires derived values to be compared with relevant published data and local experience [32, 47], and with the wider emphasis on characterising geotechnical variability and uncertainty explicitly [45, 48].

Implications for the Orle River works

For shallow foundations of light structures on Unit I, bearing capacity calculated with realistic φ′ (159–463 kPa) is unlikely to govern. Settlement will govern instead, both within the loose sand and, more importantly, from consolidation of the soft Unit II clay beneath, which is 2.5–13 m thick. The report's recommended allowable pressure of 75–100 kPa may therefore be

reasonable, but on settlement grounds rather than the bearing grounds it cites; settlement should be assessed explicitly [49–51]. For deep foundations, piles should be designed with SPT-based shaft and base resistances in Unit III, ignoring positive shaft support in Units I and II and checking for downdrag [41, 43, 52]. Static load tests should be used to calibrate the design, as the report rightly recommends.

Limitations

Only two shear-box records (six specimens) were examined in full. The remaining tests followed the same procedure and stress levels, but should be checked individually. No SPT was made in Unit I, so the shallow comparison relies on an assumed φ' range; this range is deliberately conservative relative to Unit III. The SPT energy ratio was not measured, which is why a range of 45–75% was used; energy measurement to ASTM D4633 [35] would remove this uncertainty. The correlations are empirical and not site-specific, and their scatter of about ±3–4° is small compared with the 20–30° discrepancy at issue. Finally, independent laboratory tests at appropriate stresses (drained direct shear or triaxial tests on reconstituted specimens at the field density) were not available and are recommended to confirm the SPT-based values.

Conclusions

1. The reported allowable bearing pressures at the Orle River site were reproduced exactly. The method was Meyerhof's equation for a 1 m square footing placed at the sample depth, with total unit weight and FS = 3.

2. Small-box direct shear tests on non-plastic sands gave φ = 6.8–11.1°. The tests were run at 4.9–14.7 kPa, 5–53 times below in situ effective stress, and stopped while the shear stress was still rising, so failure was never reached.

3. Corrected SPT data ($(N_1)_{60}$ = 7.7–14.7; $D_r \approx$ 0.36–0.49) give φ' = 31–38° by two independent correlations. The difference from the direct shear values survives every plausible energy-ratio assumption.

4. For a shallow pad, the reported parameters understate allowable pressure 5–12-fold. At depth, the geometry of the method inflates values until they resemble SPT-based pile toe resistance, but only by coincidence.

5. Five routine checks are proposed: plausibility, in situ cross-check, stress level, failure criterion, and geometry with effective stress. Together they would catch this class of error before it reaches design, and they can be applied by reviewers and clients using information already present in standard reports.

Abbreviations

ASTM: American Society for Testing and Materials; BH: Borehole; BS: British Standard; CPT: Cone penetration test; E_R: Energy ratio; FS: Factor of safety; ISO: International Organization for Standardization; SPT: Standard penetration test; USCS: Unified Soil Classification System; UU: Unconsolidated undrained.

Declarations

Availability of data and materials

The datasets supporting the conclusions of this article, including the digitised shear-box records, SPT logs and calculation scripts, are available from the corresponding author on reasonable request.

Competing interests

The 2026 site investigation report whose design values are re-examined in this study was prepared by E. Onwubolu, N. Uwaila, G. Evbaru-Okhuaihesuyi, J. Odemerho and S. Ilavbare. The re-analysis was undertaken constructively to improve practice, and the authors declare that they have no competing interests.

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Authors' contributions

AAI conceived the study, performed the re-analysis and wrote the manuscript. PAA contributed to the site investigation data and interpretation of the field results. OEC contributed to the laboratory data, the review of test procedures and the revision of the manuscript. All authors read and approved the final manuscript.

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