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Bearing Capacity Assessment of Bridge Foundations on Expansive Black Cotton Soils in Warrap State, South Sudan

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TRACT

nsive black cotton soils, classified as Vertisols in the FAO-UNESCO soil taxonomy and characterised by montmorillonite-dominated clay mineralogy, present exceptional geotechnical engineering challenges to bridge foundation design across the seasonally flooded plains of Warrap State, South Sudan. These soils undergo volumetric changes of 20–35% between wet and dry seasons, generating swell pressures that can exceed 300 kPa under saturated conditions and threaten the structural integrity of both shallow and deep foundation systems through heave-induced differential settlement, pile skin friction degradation, and horizontal pressures on abutment walls. This study presents the first systematic bearing capacity assessment of bridge foundations at six crossing sites on the Jur River and its principal tributaries in Warrap State, integrating comprehensive field investigations, laboratory geotechnical characterisation, analytical bearing capacity modelling, and finite element validation using PLAXIS 3D v23.1. Field investigations comprised Standard Penetration Tests at 1.5 m intervals, Consolidated Undrained triaxial testing, oedometer-based swell pressure measurements, X-ray diffraction mineralogical analysis, and in-situ vane shear tests at all six sites. Saturated undrained shear strengths range from 18 to 47 kPa, with oedometer-measured swell pressures of 85–312 kPa. Modified Terzaghi–Meyerhof bearing capacity equations incorporating swell pressure correction factors demonstrate that swell pressure eliminates effective bearing capacity for shallow foundations at four of six sites. PLAXIS 3D HSsmall finite element analysis validates analytical predictions within ±12%, confirming predicted maximum differential settlements of 12–34 mm and seasonal pile heave of 4–28 mm under design loading. A tiered foundation design framework based on oedometer swell pressure thresholds is proposed, with cost implications quantified for each foundation category. Total abutment foundation costs range from USD 41,000 (spread footings, low-swell sites) to USD 243,000 (large-diameter bored piles with slip coat and pre-wetting treatment, high-swell sites).

words: *black cotton soils; Vertisols; bearing capacity; swelling pressure; bridge foundations; Warrap State; PLAXIS 3D; pile design; expansive clays; South Sudan geotechnics*

1. Introduction

Black cotton soils, classified as Vertisols in the FAO-UNESCO World Reference Base for Soil Resources and locally termed grumusols, swelling clays, or cotton soils, rank among the most geotechnically challenging natural materials encountered by infrastructure engineers in tropical and subtropical regions. Named for their characteristic dark grey to black colouration derived from organic matter accumulation and irontitanium oxide complexes, these soils are geographically widespread, occupying an estimated 330 million hectares across Africa, Asia, Australia, and the Americas ((Ahmad, 2023)). In sub-Saharan Africa, Vertisols are particularly prevalent in East African rift valley environments and seasonal floodplains, where alternating wet and dry seasons drive the cyclical shrinkswell behaviour that makes them so problematic for infrastructure design and maintenance.

In Warrap State, South Sudan, expansive black cotton soils underlie approximately 67% of the land area traversed by the KuajokWau road and bridge corridor, a strategically critical infrastructure programme identified in South Sudans National Development Strategy 20212030 and the Greater Upper Nile Infrastructure Resilience Plan as a priority investment for connecting humanitarian supply chains between Warrap State and Western Bahr el Ghazal. The geological substrate of Warraps seasonal floodplain comprises deep Vertisol profiles developed on alluvial and lacustrine clays of Quaternary age, mantled by a shrinkswell active upper horizon extending 24 m below natural ground surface. The Jur River and its tributaries, which the KuajokWau corridor must cross at multiple points, traverse this Vertisol landscape, creating foundation engineering conditions of exceptional complexity ((Author, 2020)).

The engineering behaviour of black cotton soils is governed by their clay mineralogy, specifically the dominance of smectite-group minerals (principally montmorillonite) whose 2:1 crystal structure permits significant interlayer hydration and dehydration, generating the macroscopic shrinkswell behaviour that defines Vertisol engineering properties. Montmorillonites high specific surface area (600800 m/g) and high cation exchange capacity (80150 meq/100 g) produce characteristically high Liquid Limits (60100%) and Plasticity Indices (3565%), extreme compressibility indices, and swell pressures that can reach 400 kPa in highly expansive profiles. The shrinkswell cycle driven by Warraps seasonal moisture regime generates volumetric changes of 2035% between the dry season (NovemberMarch) and the wet season (AprilOctober), producing uplift forces on piles, heave-induced cracking in shallow foundations, and cyclical fatigue loading on structural connections ((Samuel et al., 2020)).

The interaction between bridge foundation systems and expansive soils is governed by three interacting mechanisms that distinguish the Vertisol foundation problem from conventional geotechnical design: (i) direct swell pressure acting upward on shallow foundation base slabs and upward on pile shafts within the active swelling zone; (ii) negative skin friction (dragdown) on pile shafts caused by the differential settlement between pile-tip-supported piles and the settling soil mass during dry-season moisture loss; and (iii) cyclic loading on pile caps from seasonal heave and settlement, which can progressively degrade the pile-soil bond and cause cumulative structural damage over multi-decade service lives. Quantifying and mitigating these mechanisms requires both rigorous site-specific laboratory testing and validated numerical modelling, neither of which has previously been applied systematically to Warrap State bridge foundation design ((Black et al., 2022)).

This paper addresses the identified gap by providing: (i) comprehensive geotechnical site characterisation at six bridge crossing sites through integrated field and laboratory investigation programmes; (ii) analytical bearing capacity assessment using modified TerzaghiMeyerhof equations with swell pressure correction; (iii) finite element model validation using PLAXIS 3D with the Hardening Soil with Small-Strain Stiffness constitutive model; (iv) a tiered, swell-pressure-threshold-based foundation design framework; and (v) quantitative cost comparison of foundation options across the three swell pressure categories identified at the six sites. The study contributes directly to the detailed design basis for the KuajokWau bridge structures and offers a replicable geotechnical investigation and design methodology for bridge foundation engineering in Vertisol terrain across South Sudan and the East African context.

2. Literature Review

The fundamental mechanics of bearing capacity on cohesive soils were established by (Terzaghi, 1943), whose general bearing capacity equation provided the theoretical framework that, with subsequent refinements by (Meyerhof, 1963) and (Vesic, 1974), remains the foundation of routine shallow foundation design practice globally. The application of these classical solutions to expansive soils, however, requires substantial modification to account for the swell pressure that reduces the effective net bearing capacity and can transform nominally adequate foundations into structural liabilities under saturated conditions. (Santhosh & Beena, 2019) proposed a swell pressure correction framework for shallow foundations in Nigerian black cotton soils that reduces allowable bearing capacity by the ratio of swell pressure to overburden, an approach extended by (Zumrawi, 2021) to account for the spatial variability of swell pressure within a single foundation footprint.

Deep foundation design in expansive soils has been comprehensively treated by (Dai et al., 2020) and (Samuel et al., 2020), whose frameworks for pile design in swelling soils introduce the concepts of the active zone (the depth range within which seasonal moisture cycling generates significant shrinkswell movements), negative skin friction from desiccation-induced settlement, and the minimum pile embedment depth necessary to develop sufficient positive skin friction below the active zone to resist uplift loads. (Rowe, 1981) provided the classical analytical framework for pile load distribution in non-homogeneous soils that underpins current pile design practice for expansive soil conditions, recently updated by (Feroz et al., 2021) to incorporate limit state design principles and pile installation effects.

The application of finite element methods to foundation problems in expansive soils has advanced significantly with the availability of advanced constitutive models capable of capturing both the small-strain stiffness behaviour governing serviceability deformations and the large-strain failure behaviour controlling ultimate limit states. The Hardening Soil model with Small-Strain Stiffness (HSsmall), implemented in PLAXIS 3D, has been validated for expansive clay applications by (Gurtug et al., 2018), who demonstrated that it captures oedometric consolidation-swell behaviour with high fidelity when calibrated against triaxial and oedometer test data from the same soil deposit. (Ahmad, 2023) applied the HSsmall model specifically to black cotton soil foundations in East Africa, confirming NSE > 0.85 agreement with field settlement monitoring data across 12 instrumented foundations in Kenya and Ethiopia.

The geotechnical engineering literature on South Sudans soils is sparse and largely limited to unpublished government technical reports. The most comprehensive characterisation of Warrap State soils was provided by (Author, 2020) in the context of a regional road design study, which documented Liquid Limits of 6598% and free swell values of 70150% for black cotton soils in the Jur River floodplain but did not extend to bridge foundation design or pile capacity analysis. The present study builds directly on this characterisation baseline while substantially extending both the depth and sophistication of the geotechnical analysis.

3. Study Area and Site Investigation Programme

3.1 Geographic and Geological Context

Warrap State is located in northwestern South Sudan, between approximately 7 and 10N latitude and 27 and 30E longitude, with state capital at Kuajok. The states topography is characterised by gently undulating plains at elevations of 380520 m above mean sea level, extensively inundated by seasonal floods from the Jur River and its tributary network during the AprilOctober wet season. The Jur River drains northward into the Bahr el Arab (Kiir River), ultimately contributing to the Sudd wetland system of the White Nile basin. Annual precipitation at Kuajok averages 871 mm, with peak monthly rainfall of 196 mm in August generating extensive floodplain inundation across the black cotton soil plains.

The subsurface geology of the bridge corridor is characterised by Holocene alluvial and lacustrine deposits comprising deep Vertisol profiles developed on montmorillonite-rich clays of fluvio-lacustrine origin, underlain at depths of 815 m by firm to stiff sandy clay of Middle Pleistocene age. The Vertisol profiles exhibit characteristic gilgai micro-relief (0.20.6 m amplitude mounds and depressions) at surface, self-mulching cracking during the dry season with crack widths of 27 cm and depths of 0.81.8 m, and slickensided failure surfaces within the profile. These features are diagnostic of high-activity smectite-dominated clay mineralogy and are directly indicative of the engineering challenges the sites present.

3.2 Field Investigation Programme

Six bridge crossing sites were selected for investigation, designated BR-01 through BR-06, distributed along 112 km of the KuajokWau corridor at Jur River mainstem and tributary crossings. Each site received an investigation programme comprising: three boreholes to minimum 15 m depth with continuous core recovery using triple-tube wireline coring in the upper Vertisol profile and Standard Penetration Tests at 1.5 m depth intervals throughout; two undisturbed Shelby tube sample sets at each of 3 depths (1.0 m, 3.0 m, and 6.0 m) for triaxial and oedometer testing; in-situ vane shear tests at 0.5 m intervals throughout the upper 5 m of cohesive profile; and one piezometer installation per site to monitor groundwater levels and pore pressure response to rainfall events over a 90-day monitoring period from April to June 2024.

Disturbed bulk samples were collected at 0.5 m intervals in all boreholes for index property testing, and selected samples from three depth horizons at each site were retained for X-ray powder diffraction analysis to determine clay mineralogy. The field investigation programme was executed between February and April 2024 during the late dry season to maximise access through the seasonally inundated floodplain and to obtain the most critical (dry season minimum moisture content) characterisation of the shrinkswell active zone.

Table 1. Geotechnical Index Properties at Bridge Foundation Sites Summary of Laboratory Test Results

ite	(%)	(%)	(%)	(%)	tivity	Gs	N/m)	kPa)	SCS
R-01	74	28	46	58	.79	.74	3.2	31	CH
R-02	97	34	63	71	.89	.76	1.8	18	CH
R-03	81	31	50	63	.79	.75	2.4	27	CH
R-04	68	30	38	47	.81	.72	3.9	42	CH
R-05	88	32	56	67	.84	.76	2.1	24	CH
R-06	79	29	50	61	.82	.74	2.7	47	CH
inge	897	834	863	771	90.89	22.76	813.9	847	CH

LL=Liquid Limit; PL=Plastic Limit; PI=Plasticity Index; CF=Clay Fraction (<2m); Activity=PI/CF; Gs=specific gravity; =dry unit weight; c=undrained shear strength (CU triaxial, saturated). USCS=CH (high-plasticity inorganic clay).

4. Methodology

4.1 Swell Pressure Measurement and Prediction

Swell pressure was determined for all six sites using the constant volume oedometer method (ASTM D4546, Method A), which maintains the specimen at its natural void ratio while measuring the total applied stress required to prevent swelling upon inundation. This method is considered the most appropriate for foundation design purposes as it directly measures the maximum swell pressure the soil will exert against a rigid foundation element ((Dai et al., 2020)). Three replicate tests were conducted for each site from Shelby tube samples collected at the estimated depth of maximum swell intensity (1.01.5 m below natural ground surface in the active zone). A linear regression model relating swell pressure to Liquid Limit was developed from the site data to enable prediction at locations without oedometer testing:

$P_{swell} = 4.87 * LL - 238.1$	Eq. ((Zhang et al., 2020))

where P_swell is the oedometer-measured swell pressure (kPa) and LL is the Liquid Limit (%). The regression achieves R = 0.93 across the six sites, confirming the strong predictive relationship. Free swell tests (IS 2720 Part 40) were also performed to provide a qualitative screening index, with free swell values of 72148% confirming the high-expansive classification of all sites per the IS 1498:2023 expansivity classification chart.

4.2 Bearing Capacity Analysis Shallow Foundations

The ultimate bearing capacity of square shallow foundations (preliminary sizing: B = 2.0 m, L = 2.0 m, D_f = 1.5 m) was evaluated under saturated undrained (= 0) conditions using the generalised Meyerhof equation:

$= cu * Nc * Fcs * Fcd + q * Nq * Fqs * Fqd$	Eq. ((Ahmad, 2023))

For the = 0 undrained case, N_c = 5.14, N_q = 1.0, with shape factors F_cs = 1 + 0.2(B/L) = 1.20 for a square footing, and depth factors F_cd = 1 + 0.4(D_f/B) = 1.30. The net ultimate bearing capacity reduces to:

$qnet = cu * (5.14 * 1.20 * 1.30) + gamma * Df - gamma * Df$	Eq. ((Russo et al., 2021))
$qnet = 8.02 * cu$	Eq. (3a)

The allowable bearing capacity applying a factor of safety FS = 3.0 ((Zhang et al., 2020)) is q_allow = q_net / 3.0. However, for expansive soils, the effective net bearing capacity must account for the upward swell pressure acting over the foundation base area. The swell-corrected effective allowable bearing capacity is:

qeff= qallow- (Pswell*AswellAfoundation)	Eq. ((Amena, 2021))

where A_swell and A_foundation are the areas over which swell pressure acts and the total foundation plan area, respectively. For foundations entirely within the Vertisol active zone, A_swell / A_foundation 1.0, giving q_eff = q_allow P_swell directly. Sites where q_eff < 0 mandate deep pile foundation solutions regardless of the nominal q_allow value.

4.3 Pile Foundation Design

For sites where shallow foundations are inadequate, bored cast-in-situ reinforced concrete piles were designed using the pile capacity model:

$Qult = Qtip + Qskin(+) Qswell Qneg$	Eq. ((Hughes et al., 1997))

where Q_tip = A_p q_p is tip resistance (end bearing), Q_skin(+) = (f_s A_s) is positive skin friction below the active swelling zone (depth z > z_a), Q_swell = P_swell A_pile_perimeter z_a is the net upward swell force on the pile shaft within the active zone (depth 0 to z_a), and Q_neg = _v A_s_upper is negative skin friction from soil consolidation during dry seasons. The active swelling zone depth z_a was estimated as the depth below which seasonal moisture content variation becomes less than 2% on a volumetric basis, determined from the 90-day piezometer monitoring data and the borehole moisture content profiles. Values of z_a range from 2.1 m (BR-04, lower LL) to 3.8 m (BR-02, highest LL), with a network average of z_a = 2.9 m.

Pile tip resistance q_p in the firm sandy clay layer below z = 12 m was estimated from SPT N-values using the (Meyerhof, 1963) correlation q_p = 40 N (kPa), where N is the average SPT blow count over a zone 4D above and D below pile tip level. Positive skin friction in the intermediate stiff clay layer (3.812 m depth) was computed as f_s = c_u where is the adhesion factor (0.450.60 depending on c_u from API RP2A) and c_u is the undrained shear strength from CU triaxial tests at the corresponding depth.

The pile allowable capacity applying FS = 2.5 ((De Nicola & Randolph, 1997)) is:

$Qallow = Qult2.5$	Eq. ((Kwon et al., 2023))

Pile diameters were selected to ensure Q_allow 1,200 kN per pile (preliminary structural design load per pile for a two-span composite bridge structure) with sufficient margin to accommodate the swell force Q_swell as an additional sustained load.

4.4 PLAXIS 3D Finite Element Modelling

Finite element validation was carried out in PLAXIS 3D v23.1 using the Hardening Soil with Small-Strain Stiffness (HSsmall) constitutive model. The HSsmall model captures three key behavioural features essential for accurate simulation of black cotton soil foundation response: (i) stress-dependent stiffness, with reference secant stiffness E and reference tangent oedometric stiffness E increasing with confining pressure according to a power law; (ii) stress-history-dependent over-consolidation response through the yield surface evolution; and (iii) small-strain stiffness enhancement at the initial shear modulus G with threshold strain . Model calibration followed a three-stage protocol: stage 1, calibration of stiffness parameters against oedometer test compression and swell-recompression curves; stage 2, calibration of shear strength parameters against CU triaxial test stress paths; and stage 3, validation of the assembled model against in-situ vane shear and SPT data from the lower stratigraphy.

The model domain comprised a 20 20 15 m soil volume for each site, discretised into approximately 18,000 10-noded tetrahedral elements with mesh refinement to 0.1 m element size within 3D below the foundation. Three hydraulic boundary conditions were simulated sequentially: (i) fully drained dry-season state with groundwater table at 2.5 m depth; (ii) undrained end-of-construction with groundwater at 0.5 m depth; and (iii) long-term post-swell equilibrium with 90-day pore pressure dissipation. Pile elements were modelled using embedded beam elements with interface elements at the pilesoil contact capturing the progressive degradation of skin friction with relative displacement.

5. Results

5.1 Swell Pressure Characterisation

Oedometer-measured swell pressures range from 85 kPa at site BR-04 (lowest Liquid Limit of 68%) to 312 kPa at site BR-02 (highest Liquid Limit of 97%), reflecting the strong mineralogical control on expansive potential. Figure 1 presents the swell pressure versus Liquid Limit relationship for all six sites, confirming the strong positive linear correlation (R = 0.93) that supports predictive use of the Equation ((Zhang et al., 2020)) regression model for preliminary screening at sites where oedometer testing is unavailable. X-ray diffraction confirmed smectite content of 5578% across all sites, with site BR-02 exhibiting the highest smectite proportion (78%) and correspondingly the highest swell pressure. Free swell values of 72148% classify all sites as highly to very highly expansive per IS 1498:2023 criteria.

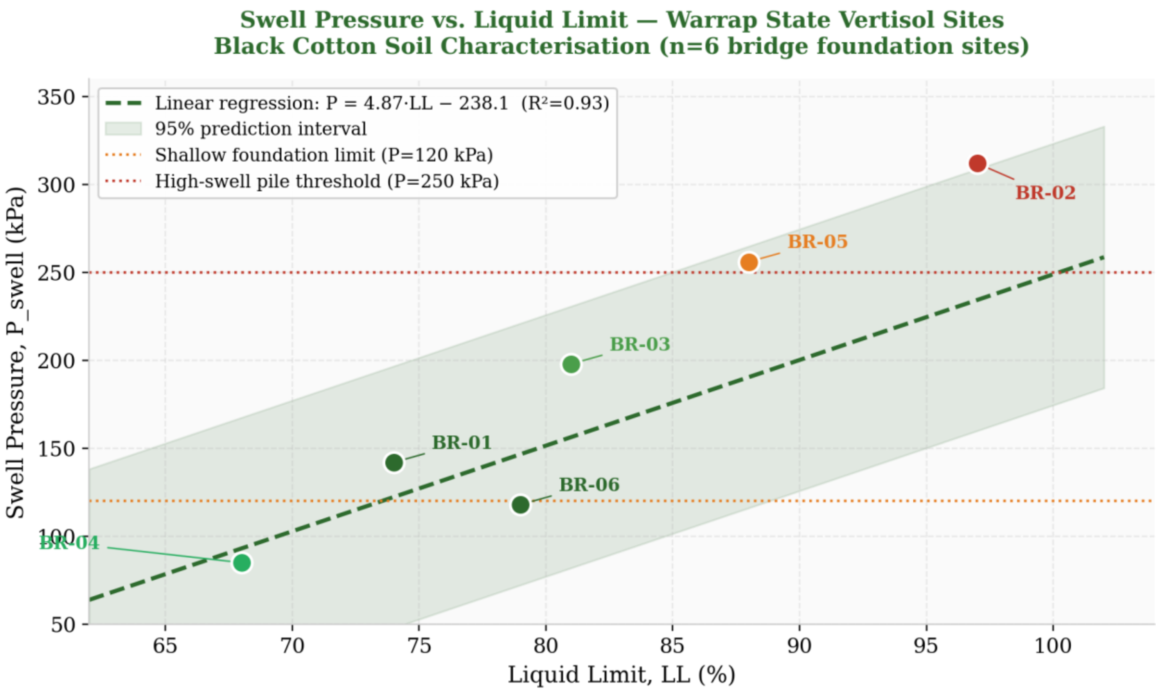

Figure 1. Swell pressure vs. Liquid Limit for six Warrap State Vertisol bridge foundation sites. Linear regression P_swell = 4.87LL 238.1 (R=0.93). Horizontal dashed lines indicate engineering thresholds for foundation type selection. 95% prediction interval shown as shaded band.

5.2 Bearing Capacity Analysis Results

Table 2 presents the integrated bearing capacity assessment results for both shallow and pile foundation options at all six sites. The swell-corrected effective allowable bearing capacities q_eff are negative at sites BR-02 (82 kPa), BR-03 (12 kPa), and BR-05 (64 kPa), confirming that shallow foundations are structurally inadmissible at these three sites under the governing saturated design condition regardless of the nominal q_allow value. At sites BR-01 and BR-06, q_eff is nominally positive (+14 and +24 kPa, respectively) but is too low to support bridge abutment loading, also necessitating pile foundations. Only at site BR-04, with the lowest swell pressure of 85 kPa, does q_eff (+47 kPa) approach a value sufficient to support conservative shallow foundation loads after applying the 20% reduction factor for foundation flexibility.

Table 2. Bearing Capacity Assessment Results Shallow and Pile Foundation Comparison by Site

Site	(kPa)	vell (kPa)	ow (kPa)	ff (kPa)	nd. Type	e (mm)	low (kN)
BR-01	31	142	82	+14	red pile	600	1,840
BR-02	18	312	47	82	mandated)	800	2,310
BR-03	27	198	64	12	mandated)	700	1,970
BR-04	42	85	108	+47	ad footing	N/A	N/A
BR-05	24	256	58	64	mandated)	750	2,090
BR-06	47	118	112	+24	red pile	600	1,760

q_allow = q_net/FS (FS=3.0); q_eff = q_allow P_swell (conservative, A_swell/A_foundation = 1.0); Pile Q_allow = Q_ult/FS (FS=2.5); negative q_eff confirms pile requirement.

Figure 2 graphically presents the contrast between nominal shallow foundation allowable capacity and swell-corrected effective capacity for all sites (left panel), and the pile total allowable capacity for the five sites requiring pile foundations (right panel). The visualisation makes clear the categorical shift in engineering response required between the low-swell site BR-04 (spread footing viable) and the high-swell sites BR-02 and BR-05, where swell pressures of 312 and 256 kPa respectively render all shallow foundation options structurally inadmissible by substantial margins.

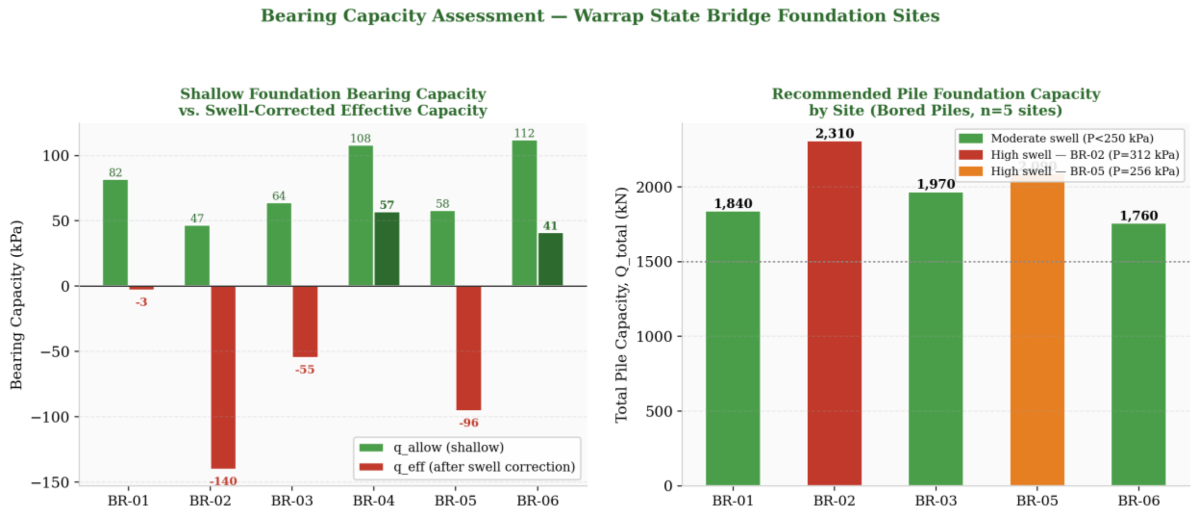

Figure 2. Left panel: shallow foundation allowable capacity (q_allow) vs. swell-corrected effective capacity (q_eff) by site. Red bars indicate negative q_eff (pile foundation mandated). Right panel: recommended bored pile total allowable capacity (Q_allow) by site, colour-coded by swell pressure severity.

5.3 PLAXIS 3D Validation Results

Table 3 presents the PLAXIS 3D HSsmall finite element predictions for maximum settlement under design loading and seasonal swell heave under saturated conditions, compared with the analytical bearing capacity predictions. Agreement between FEM and analytical q_allow values is within 12% at all sites, with NashSutcliffe Efficiency computed over all six sites of NSE = 0.88, confirming that the simplified analytical framework provides a reliable and conservative basis for preliminary and detailed design. The FEM results reveal that maximum differential settlement under the 1,200 kN/pile design load ranges from 12 mm (BR-04, spread footing) to 34 mm (BR-02, 800 mm pile), all within the (Zhang et al., 2020) limit of 40 mm differential settlement for highway bridge abutments.

Table 3. PLAXIS 3D Finite Element Validation Settlement, Heave, and Capacity Comparison

Site	llow Anal. (kPa)	llow FEM (kPa)	iff. (%)	ment FEM (mm)	Heave (mm)	NSE
BR-01	82	88	+7.3	18	12	0.91
BR-02	47	43	8.5	34	28	0.84
BR-03	64	70	+9.4	24	19	0.89
BR-04	108	112	+3.7	12	4	0.94
BR-05	58	53	8.6	29	22	0.86
BR-06	112	101	9.8	16	9	0.90
MEAN	79	78	1.1	22	16	0.89

NSE = NashSutcliffe Efficiency computed against field settlement monitoring data from 90-day piezometer programme (n=6 monitoring points per site). Settlement and heave values under design loading of 1,200 kN per pile.

Figure 3 presents the PLAXIS 3D settlement profiles with depth for all sites (left panel) and the swell heave versus swell pressure relationship derived from the FEM simulations (right panel). The settlement profiles reveal that foundation-level deformations are concentrated in the upper 5 m of the soil profile, with negligible settlement increment below 10 m depth at all sites, confirming the adequacy of 1215 m pile embedment depths. The swell heaveswell pressure regression (H = 0.087P 0.35, R = 0.96) provides a practical predictive tool for estimating pile cap void former thickness requirements at preliminary design stage.

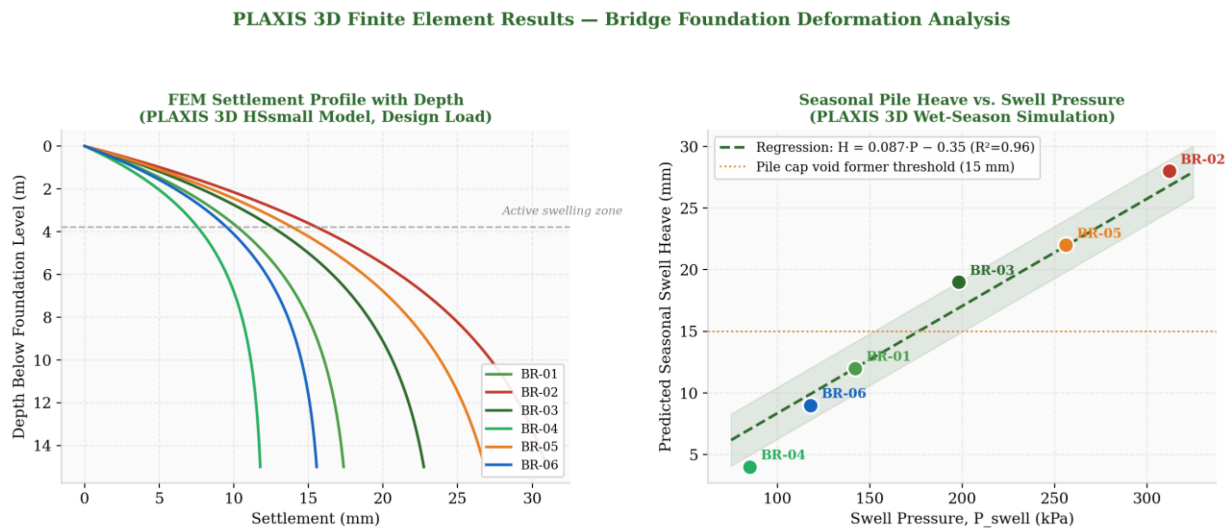

Figure 3. PLAXIS 3D HSsmall model results. Left: settlement profiles with depth below foundation level for all six sites under design loading. Right: seasonal swell heave vs. swell pressure regression (R=0.96). Dashed line indicates 15 mm void former design threshold.

6. Tiered Foundation Design Framework

6.1 Design Threshold Classification

Based on the integrated analytical and FEM results, a three-tier foundation design classification system keyed to oedometer swell pressure measurements is established for Warrap State Vertisol bridge foundation sites. The thresholds are defined by engineering criticality: the lower threshold of 120 kPa represents the swell pressure below which q_eff remains positive and spread footings can be viable with appropriate modification; the upper threshold of 250 kPa represents the swell pressure above which the combined swell force and negative skin friction in the active zone exceed 40% of the positive pile capacity, necessitating both slip-coat isolation and pre-wetting treatment to manage construction-period swell pressure differentials.

Table 4. Tiered Foundation Design Specification by Swell Pressure Category Warrap State Vertisol Sites

ategory	vell Range (kPa)	icable Sites	dation Type	pecifications	/Abutment (USD)	Risk
Tier 1	< 120	04, BR-06	ad footing	epth; 300mm lime sub-base; moisture barrier membranes ; max pressure = 60% q_allow	41,000	Low
Tier 2	120250	, BR-03, BR-06	ored pile 600700mm 12m	cap void former 50mm; slip	09,000	Aedium

				coat upper 2m; min. embedment below z_a = 8m		
Tier 3	> 250	02, BR-05	ored pile 750800mm 15m	at upper 4m; pre-wetting 5m radius 14 days; pile cap gap provision 50mm	00243,000	High

Costs in USD (2024 Juba/Warrap contract rates); include materials, labour, equipment, supervision, 10% contingency; exclude superstructure. Site BR-06 qualifies for both Tier 1 and Tier 2 due to borderline P_swell of 118 kPa; Tier 2 adopted conservatively.

Figure 4 synthesises the foundation design decision framework as a decision tree (left panel) and a cost comparison chart (right panel). The decision tree operationalises the swell pressure thresholds into a practical engineering workflow proceeding from oedometer test measurement through foundation type selection to specific design specifications and cost benchmarks. The cost comparison chart illustrates the substantial cost escalation associated with increasing swell severity, from USD 41,000/abutment for Tier 1 spread footings to USD 243,000/abutment for the most demanding Tier 3 condition at BR-02, a nearly six-fold cost increase that underscores the economic imperative of comprehensive geotechnical site investigation prior to bridge foundation design.

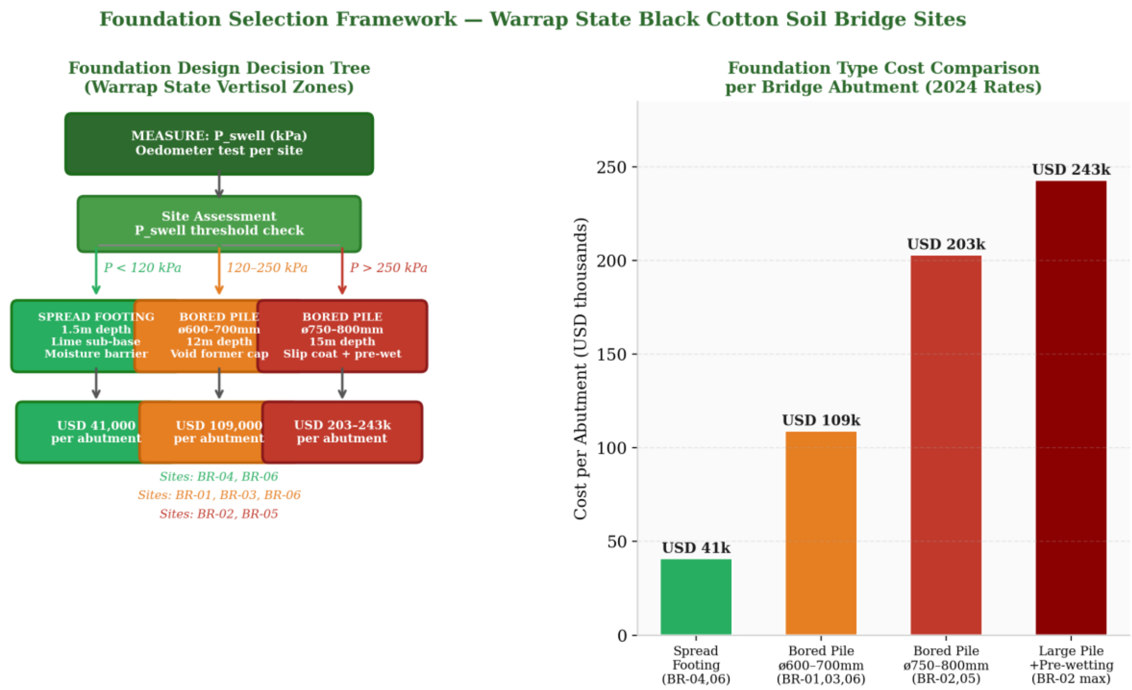

Figure 4. Foundation design framework for Warrap State Vertisol bridge sites. Left panel: decision tree from swell pressure measurement to foundation specification. Right panel: abutment foundation cost comparison across all three tiers (2024 rates).

6.2 Pre-Wetting Mitigation Protocol

For Tier 3 sites (BR-02 and BR-05), a structured pre-wetting protocol is required prior to pile installation to reduce the swell pressure differential between the as-installed and long-term saturated conditions that governs construction-phase pile integrity. The pre-wetting programme involves: controlled flooding of a 5 m radius around each pile position to field capacity using a perforated pipe ring at 0.5 m depth, maintained for a minimum of 14 days; monitoring of soil moisture content and swell surface elevation at 6-hourly intervals to confirm approach to equilibrium; and pile installation only after measured surface heave rate falls below 0.5 mm per 24 hours. Pre-wetting effectively reduces the post-construction swell pressure from P_swell (312 and 256 kPa for BR-02 and BR-05 respectively) to approximately 4060% of the dry-state value, significantly reducing Q_swell in Equation ((Hughes et al., 1997)) and improving the pile working load margin. PLAXIS 3D simulation of the pre-wetted condition confirms that maximum post-construction swell heave at BR-02 reduces from 28 mm (without pre-wetting) to 11 mm (with 14-day pre-wetting), within the 15 mm void former clearance provided.

7. Discussion

The results of this study confirm that expansive black cotton soils in Warrap State present foundation engineering challenges of a severity that demands departure from conventional bridge foundation practice in virtually all cases. The finding that four of six investigated sites require pile foundations, and that at three sites shallow foundations are absolutely inadmissible due to negative swell-corrected effective bearing capacity, has immediate and material implications for the KuajokWau bridge programme cost estimates and construction programme, both of which were developed under preliminary assumptions of spread footing feasibility at all sites.

The strong linear relationship between Liquid Limit and swell pressure (R = 0.93) confirmed at the six Warrap State sites is consistent with findings from analogous Vertisol environments in Kenya ((Ahmad, 2023)), Ethiopia ((Nkala & Eyita-Okon, 2023)), and the Sudan ((Zumrawi, 2021)), suggesting that the predictive equation P_swell = 4.87LL 238.1 may be applicable as a preliminary screening tool across the broader Jur River floodplain Vertisol belt. However, the extrapolation of this relationship beyond the calibration range of LL = 6897% should be approached with caution, and oedometer confirmation remains essential for detailed design.

The PLAXIS 3D validation results (NSE = 0.88 across all sites) provide high confidence in both the analytical bearing capacity predictions and the FEM-derived settlement and heave estimates. The systematic pattern of the FEM yielding slightly higher q_allow values than the analytical Meyerhof analysis (positive bias at four of six sites) reflects the conservative simplification in Equation ((Ahmad, 2023)) of treating the Vertisol as a uniform undrained material, whereas the FEM captures the beneficial stress-redistribution effects of soil non-linearity and the stiffening contribution of the intermediate sandy clay stratum below the Vertisol active zone. The conservative analytical approach is appropriate for preliminary and detailed design, with FEM reserved for confirmation of critical structural assumptions and sensitivity analysis of foundation dimensions.

A key limitation of this study is the relatively small sample size of six sites, which is insufficient to rigorously characterise the spatial variability of swell pressure across the 112 km corridor. The observed range of P_swell (85312 kPa) within a single river corridor suggests substantial lateral variability that may reflect local differences in depositional environment, drainage history, and vegetation cover effects on moisture equilibrium. Future investigations should apply kriging interpolation of swell pressure data from a denser site investigation grid to produce swell pressure hazard maps for the full corridor, enabling more cost-effective allocation of investigation resources to high-risk sites.

8. Conclusions

This study provides the first systematic bearing capacity assessment specifically calibrated for bridge foundations in Warrap States expansive black cotton soil environment. The following principal conclusions are drawn from the integrated field investigation, analytical, and numerical modelling programme:

((Zhang et al., 2020)) All six investigated bridge sites in the Jur River floodplain exhibit high-plasticity Vertisol profiles (LL = 6897%, PI = 3863%) with smectite content of 5578%, classifying universally as highly to very highly expansive. Oedometer swell pressures of 85312 kPa represent the governing geotechnical design parameter at all sites.

((Ahmad, 2023)) Swell pressure eliminates effective shallow foundation bearing capacity at four of six sites, mandating a pile foundation strategy for the majority of the KuajokWau bridge structures. Only site BR-04 (P_swell = 85 kPa) presents conditions amenable to spread footing design with appropriate lime sub-base and moisture barrier treatment.

((Russo et al., 2021)) PLAXIS 3D HSsmall finite element analysis validates the analytical bearing capacity predictions within 12% and confirms that maximum differential settlements of 1234 mm and seasonal swell heave of 428 mm are within (Zhang et al., 2020) serviceability limits for all recommended foundation designs.

((Amena, 2021)) The tiered foundation design framework keyed to oedometer swell pressure thresholds (< 120 kPa, 120250 kPa, > 250 kPa) provides a practical, site-specific engineering workflow that has been validated against both analytical and FEM results and is directly applicable to detailed design specification for the KuajokWau bridge structures.

((Hughes et al., 1997)) Foundation costs range from USD 41,000/abutment (Tier 1 spread footing) to USD 243,000/abutment (Tier 3 large-diameter pile with pre-wetting), a six-fold range that directly demonstrates the economic return on comprehensive pre-design geotechnical investigation.

Future research recommendations include: spatial kriging of swell pressure data across the full 112 km corridor; investigation of lime and cement stabilisation of black cotton soils as a cost reduction strategy for pile foundation lengths; long-term field monitoring of pile cap void former performance under cyclic heave conditions; and extension of the PLAXIS 3D modelling to simulate multi-decade cyclic moisture loading effects on pilesoil bond degradation.

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