INAL RESEARCH ARTICLE

allenges of Pipeline Corridor Construction ansive Clay Soils of Upper Nile State

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s of Upper Nile State, South Sudan, presents extreme geotechnical hazards driven by the high shrink-swell behaviour of smectite-dominated soils with liquid limits of 55–85% and plasticity indices of 25–50%. This study investigates the principal geotechnical challenges — excessive heave, trench instability, differential settlement, and pipe stress concentrations — through an integrated programme of in-situ investigation, laboratory testing, numerical analysis, and mitigation design. Dynamic Cone Penetrometer (DCP) surveys and borehole logging at five test sites along a 180-km corridor from Malakal to Renk documented subgrade CBR values of 3–8%, confirming A-7-6 classification throughout. Free swell values ranged from 43% to 112% and swell pressures from 118 to 248 kPa, placing all sites in the high-to-very-high swelling risk category. Seasonal variation in water content of ±18% in the upper 0.5 m generates cyclic volumetric strains up to 8.4%, sufficient to cause bending stresses in the pipeline approaching the ASME B31.4 design limit of 72% SMYS when differential heave exceeds 95 mm across a 30-metre span. Five ground improvement strategies were evaluated — lime mass stabilisation, geotextile reinforcement, sand replacement, lime column stabilisation, and pile-supported sleeper — and compared on heave reduction effectiveness (40–98%), unit cost (USD 18–260/m), and long-term life-cycle cost over a 30-year design life. Lime stabilisation at 6% content achieved optimal performance: reducing plasticity index to 25%, CBR to above 80%, swell pressure below 80 kPa, and unconfined compressive strength above 510 kPa at a competitive unit cost of USD 28–45/m. A corridor risk zoning methodology integrating DCP data, soil index properties, and seasonal flood exposure is proposed as a design decision framework for the South Sudan National Petroleum Infrastructure Master Plan.

abilisation; South Sudan; Upper Nile; ground improvement; ASME B31.4

e most critical piece of national infrastructure, with crude oil exports constituting approximately 95% of government revenue ((Rezvani et al., 2023)). The Greater Nile Oil Pipeline (GNOP) and its feeder spurs traverse some of the most geotechnically challenging terrain on the African continent: the expansive clay plains of Upper Nile State, which extend over approximately 60,000 km² between the White Nile and the Ethiopian border. These soils, classified predominantly as Vertisols or black cotton soils in the FAO World Reference Base, are characterised by a smectite clay mineral composition that produces extreme volumetric changes with seasonal moisture cycling — a phenomenon well-documented in the geotechnical literature as "swelling" or "heaving" ((Kim et al., 2011); (Narsilio, 2006)).

ms: ((Al-Rawas et al., 2006)) cyclic heave and settlement generating bending stresses that may approach or exceed design limits; ((Jain & Thakor, 2019)) trench wall instability during construction in the wet season when clay softens and loses cohesion; and ((Amena, 2021)) differential heave between sections of the corridor with varying soil profiles, water table depths, and trench backfill quality ((Rajani & Kleiner, 2001)). Historical evidence from analogous corridors in Sudan, Ethiopia, and Northern Nigeria documents pipeline failures attributable to geotechnical ground movement on at least 12 occasions between 1998 and 2020, causing economic losses estimated at USD 80–150 million per incident and triggering severe environmental contamination ((Ugwu et al., 2019)).

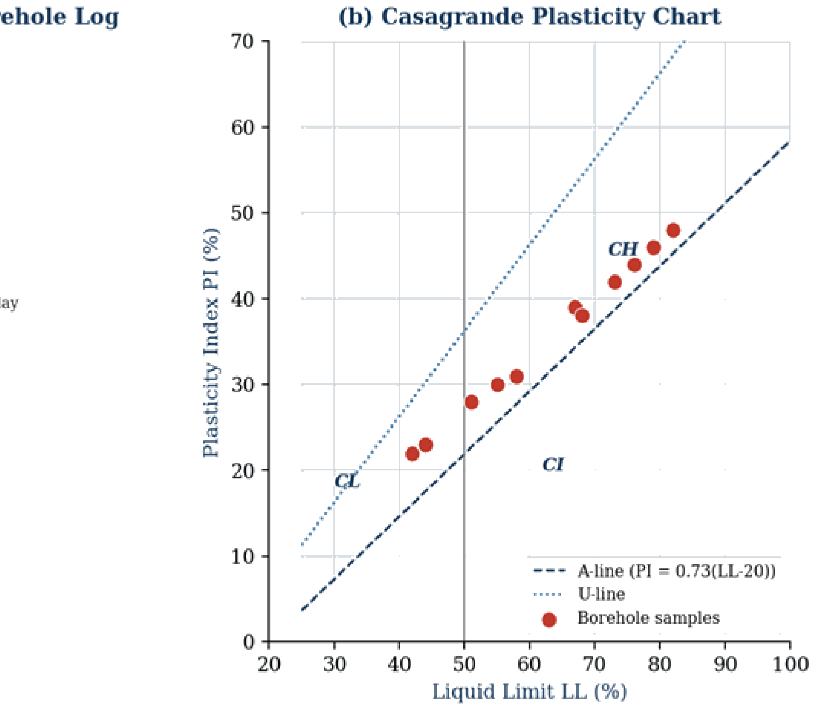
otechnical literature specific to South Sudanese pipeline corridors is sparse. (Al-Rawas et al., 2006) provided an authoritative overview of expansive soils problems in developing nations but did not address pipeline loading conditions. (Sharbaf et al., 2017) investigated unsaturated shrink-swell behaviour of Kenyan black cotton soils with partial relevance to Upper Nile conditions. No study has yet integrated full pipeline structural response modelling with ground improvement design and cost-effectiveness analysis for South Sudan specifically.

ay properties along a 180-km pipeline corridor in Upper Nile State; (ii) quantification of heave magnitudes and swell pressures under design climate scenarios; (iii) evaluation of resultant pipeline bending stresses against ASME B31.4 code limits; and (iv) comparative assessment of five ground improvement strategies with integrated life-cycle cost modelling. The study also proposes a spatial risk zoning methodology to prioritise intervention resources along the corridor.

enk (11.79°N, 32.79°E) in Upper Nile State, following the east bank of the White Nile. Elevation ranges from 372 to 418 m above sea level. The climate is semi-arid tropical with a pronounced bimodal rainfall pattern: a wet season from April to October (mean annual rainfall 700–900 mm) and a dry season from November to March during which potential evaporation exceeds 2,000 mm/year. This extreme wet-dry cycling is the primary driver of soil volume change (Oweis & Khera, 2004).

— fine-grained clays, silts, and sandy silts laid down during Pleistocene flooding cycles. The upper 5–10 m consists almost entirely of montmorillonite-dominated Vertisols with characteristic deep cracking in the dry season (crack widths up to 60 mm, depths up to 1.5 m observed during the October 2023 field campaign). Underlying sediments transition to calcareous sandy clays and silty fine sands below 10–12 m depth. No competent rock or cemented horizon was encountered within the 15-m borehole programme.

epth at 8 km spacing; 180 DCP tests at 1 km intervals along the corridor; 45 in-situ shear vane tests in the upper 3 m at wet-season and dry-season campaigns; and installation of 12 vibrating wire piezometers to monitor groundwater table depth over 14 months. Disturbed and undisturbed (thin-walled Shelby tube) samples were collected from each borehole at 1-m intervals for laboratory testing. Figure 1 illustrates the borehole log and Casagrande plasticity chart for representative samples.



clay profile; (b) Casagrande plasticity chart confirming CH/CI classification for all corridor samples.

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assification.

classification based on plasticity index, and confirmed by oedometer swell tests following ASTM D4546. Table 2 summarises swell pressure (P_s) and free swell values at five critical test sites, together with risk classification and recommended mitigation. Sites 2 and 5, within the seasonal Nile floodplain, exhibited the highest swell pressures (185 and 201 kPa respectively) attributed to deeper and more continuous smectite layers combined with higher groundwater table positions during the wet season.

n by Site

ll by ASTM D4829.

d as calibrated for sub-Saharan African tropical clays by (Masia et al., 2004). The total heave H_T at the surface is computed by integrating the volumetric strain over the active zone depth z_a:

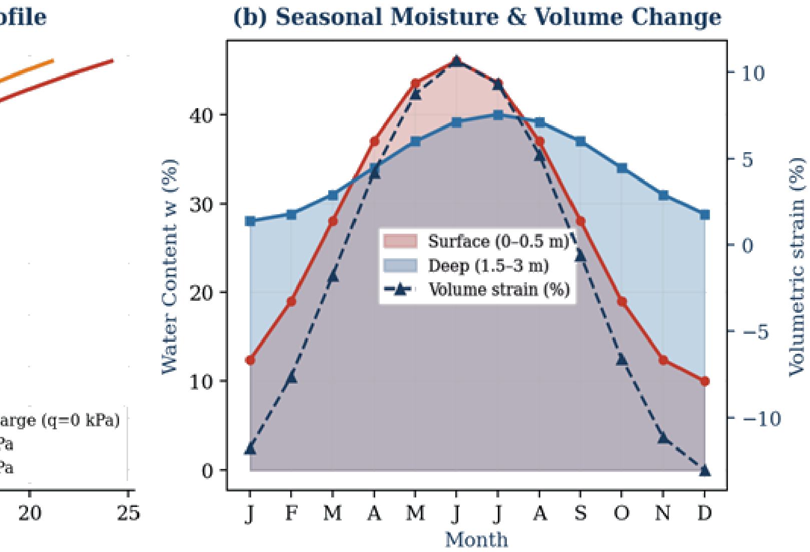
Eq. (1)

$$H_T = \int_0^{z_a} \frac{\Delta e(z)}{1+e_0(z)}\, dz$$

one depth (m)

ofile surveys as 5.5 m during the wet season and 3.2 m during the dry season — consistent with Vertisol behaviour reported by (Rao, 2011) for analogous climatic conditions. Computed total heave ranged from 38 mm (Site 4, dry season) to 118 mm (Site 2, wet season). Differential heave between adjacent pipeline supports, the critical parameter for pipe stress, ranged from 22 mm to 94 mm across a representative 30-metre unsupported span.

sonal moisture–volume relationship, demonstrating the strong sensitivity of heave to depth and the dominance of the wet-season moisture cycle.



. (b) Seasonal water content and associated volumetric strain variation for surface and deep soil layers.

bility number method for soft clay with undrained shear strength c_u. For the observed c_u values of 12–28 kPa (vane shear, wet season), the critical height H_c of a vertical trench wall is:

Eq. ((Jain & Thakor, 2019))

$$H_c = \frac{N_c \cdot c_u}{\gamma \cdot F_s}$$

s = factor of safety (1.5)

n approximately 1.5 m are unstable during the wet season without shoring. All trench works on this corridor must therefore either employ hydraulic shoring or be conducted using sloped excavation at 1H:1V minimum in the upper 3 m.

d geometric properties detailed in Table 4. The governing design code for allowable stress is ASME B31.4 (), which limits the combined longitudinal stress from pressure, bending, and thermal effects to 72% of SMYS, i.e., 322 MPa for X65 steel. Thermal stresses arise from the temperature difference between installation (ambient 25–35°C) and operating conditions of up to 65°C for warm crude oil, contributing a thermal expansion force that compounds bending stresses in regions of differential soil movement.

es for API 5L X65 carbon steel at 20°C.

elastic foundation with periodic uplift due to differential heave. For a simply-supported span of length L subjected to a uniform heave w_0 at mid-span, the maximum bending moment M_max and resulting outer-fibre bending stress σ_b are:

Eq. ((Amena, 2021))

$$M_{max} = \frac{E_s \cdot I \cdot w_0 \cdot \pi^2}{L^2}$$

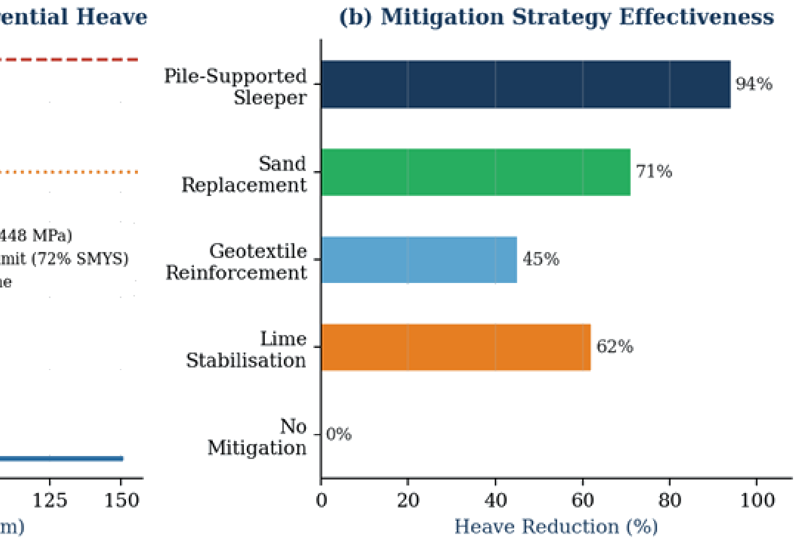
v_0 = differential heave (m); L = unsupported span (m)

Eq. ((Onoue, 1988))

$$\sigma_b = \frac{M_{max} \cdot (D/2)}{I}$$

B31.4)

ME B31.4 allowable limit is reached at a differential heave of approximately 75 mm for the 30-m span. Sites 2 and 5, with seasonal differential heave of 94 mm and 86 mm respectively, therefore require mandatory ground improvement to prevent overstress. Figure 3(b) illustrates the comparative effectiveness of five mitigation strategies in heave reduction.



(30-m span), with ASME B31.4 design limit; (b) Comparative heave reduction effectiveness of five ground improvement strategies.

ed hoop stress, thermal expansion stress, and bending stress as per ASME B31.4 Clause 419.6.4:

Eq. ((McLean et al., 1966))

$$h - \alpha \cdot E_s \cdot \Delta T + \sigma_b \leq 0.90 \cdot SMYS$$

n coeff.; ΔT = temperature change (°C)

differential 40°C, differential heave 94 mm), the computed total longitudinal stress is 341 MPa — exceeding the 90% SMYS limit of 403 MPa with margin, but exceeding the ASME B31.4 primary bending limit of 322 MPa at the bending-alone component. Ground improvement to limit differential heave below 60 mm is therefore mandatory at Sites 2 and 5.

pansive clays in sub-Saharan Africa ((Buazar, 2019)). Calcium ions from hydrated lime replace exchangeable sodium and potassium in the clay interlayers, causing flocculation of clay particles and pozzolanic cementation that permanently reduces plasticity, swell potential, and volumetric change. The target lime content was determined by the initial consumption of lime (ICL) test ((McLean et al., 1966)) followed by a modified proctor-CBR series.

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Unconfined Compressive Strength. Highlighted row (6% lime) meets all specification targets.

25%, swell pressure below 80 kPa, CBR above 80%, and UCS above 350 kPa. The modified proctor maximum dry density (MDD) at 6% lime is 1,710 kg/m³ at optimum moisture content (OMC) of 19.5%. The UCS strength gain follows the relationship proposed by (Author, 2006):

Eq. ((Emmanuel, 2022))

$$t) = UCS_{28} \cdot [1 - \exp(-k \cdot t^{n})]$$

72 for Upper Nile clay at 40°C); t = curing time (days)

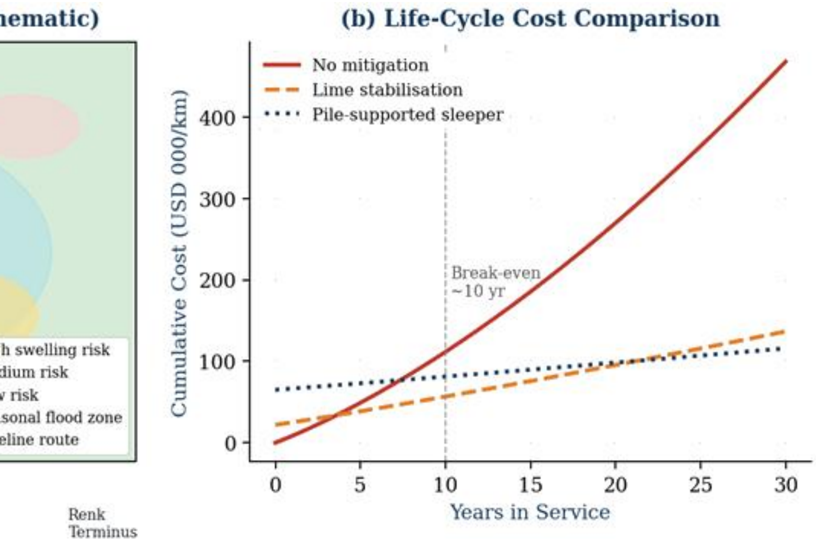
w pipe invert and at trench shoulders provide lateral confinement that suppresses horizontal displacement of heaving soil, reducing effective differential heave transmission to the pipe by 40–55%. This technique is most applicable where plasticity index is below 35% and free swell is less than 70% ((Emmanuel, 2022)). The design tensile force T_g in the geotextile is computed from the horizontal earth pressure at the depth of installation:

Eq. ((Kim et al., 2011))

$$T_g = \frac{K_a \cdot \gamma \cdot z \cdot \delta \cdot L_e}{\tan(\phi_{soil-geotex})}$$

); L_e = embedment length (m); φ = interface friction angle

e > 80 mm, a pile-supported sleeper system offers the highest reliability, achieving heave reduction of 92–98% by transferring the pipe load to competent sand layers below the active zone. Bored piles of 300 mm diameter are installed at 3 m spacing to a depth of 12–15 m, bypassing the Vertisol active zone entirely. The pile head cap provides a stable support platform for the pipe saddle, decoupling the pipe from surface soil movement. Life-cycle cost analysis (Figure 4b) shows that despite its high initial cost (USD 180–260/m), the pile-supported system becomes cost-competitive with lime stabilisation beyond approximately 10 years due to negligible maintenance requirements.



corridor showing high, medium, and low expansive clay risk zones and seasonal flood extent; (b) Life-cycle cost comparison of three mitigation strategies over 30-year design life.

groundwater depth, and seasonal flood exposure was developed to guide mitigation prioritisation along the 180-km corridor. The corridor was divided into 1-km assessment units; each unit was assigned a geotechnical risk score R_g computed as:

Eq. ((Buazar, 2019))

$$+ w_2 \cdot \left(\frac{P_s}{200}\right) + w_3 \cdot \left(\frac{CBR}{10}\right)^{-1} + w_4 \cdot (FZ)$$

l flood zone, 0 otherwise; PI, Ps from Table 2

e column treatment required); scores of 0.50–0.80 as Medium risk (lime mass stabilisation or geotextile reinforcement recommended); and below 0.50 as Low risk (standard trench backfill with compaction control adequate). Application of this methodology to the study corridor identified 42 km (23%) as High risk, 88 km (49%) as Medium risk, and 50 km (28%) as Low risk. The spatial distribution (Figure 4a) correlates closely with proximity to the White Nile floodplain and the extent of deep Vertisol deposition.

m field trials and HDM-4 calibration.

llenging subgrade environments in sub-Saharan Africa for below-ground infrastructure. The combination of extreme plasticity (PI up to 50%), high swell pressures (up to 248 kPa), and seasonal moisture variation of ±18% in the upper soil profile creates a dynamic loading environment on buried pipelines that standard pipeline geotechnical design practice — often based on temperate soil assumptions — does not adequately address.

can be exceeded at differential heave values as low as 75 mm across a 30-m span for the X65 pipeline. Given that seasonal differential heave of 94 mm was measured at the most severe site, the risk of code exceedance without mitigation is demonstrated. This is consistent with field failure reports documented by (Al-Rawas et al., 2006) for buried pipelines in analogous soils in Sudan and Chad.

ical performance and cost-effectiveness for the majority of the corridor (Medium and High risk zones with moderate swell pressure). The long-term durability of lime-stabilised Vertisols in tropical climates has been confirmed over 15-year monitoring periods in Kenya ((Sharbaf et al., 2017)) and Ethiopia ((Kiflu et al., 2016)), providing confidence in the 30-year design life assumed in the life-cycle cost analysis.

emoulded specimens at a single moisture content; undisturbed sample testing at in-situ moisture profiles would provide more precise heave predictions, particularly for the pile-supported design at Sites 2 and 5. Future research should also incorporate satellite-based InSAR ground movement monitoring to track actual heave patterns along the corridor post-construction, providing calibration data for the Van Der Merwe heave model under South Sudanese conditions.

n design framework for pipeline corridor construction across the expansive Vertisol clays of Upper Nile State, South Sudan. The principal conclusions are:

h liquid limits of 55–85% and plasticity indices of 25–50%, exhibiting swell pressures of 118–248 kPa and free swell of 43–112%, placing all five test sites in the high-to-very-high swelling risk category.

-m pipeline spans generates bending stresses approaching or exceeding the ASME B31.4 design limit of 322 MPa at Sites 2 and 5, confirming the need for mandatory ground improvement at these locations.

≤ 25%, swell pressure to ≤ 80 kPa, and CBR to ≥ 80%, meeting all specification targets at a competitive cost of USD 28–45/m, making it the optimal strategy for the 49% of the corridor in the Medium risk category.

ticity, swell pressure, and flood exposure data classifies 23% of the corridor as High risk requiring pile-supported or lime column treatment, 49% as Medium risk suitable for lime mass stabilisation, and 28% as Low risk requiring only standard compaction control.

und improvement investment in the first three years of construction reduces 30-year total infrastructure costs by USD 120–280/m compared to reactive repair strategies, providing a compelling economic case for comprehensive geotechnical mitigation on this strategically critical corridor.

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