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GEOTECHNICAL ASSESSMENT OF THE NEMBE - BRASS ROAD, BAYELSA STATE, NIGERIA

By

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ABSTRACT

The Nembe – Brass Road is both significant and important for the development of Nigeria. Although the road has been in successive development plans, it had never been implemented for over 50years. This paper provides a geotechnical assessment of the subsoil along the proposed alignment, contributing data and evaluating the conditions and challenges likely to be encountered. The road which spans 48km, is underlain by very soft and weak clay with moisture content between 50-200%, largely unconsolidated with undrained strength (2-14kPa), liquidity indices that suggest semi-solid soil state and plasticity indicative of diverse origins. Bearing pressures are very low in large segments of the road (in over 70% of the length) with settlement up to 5m in some areas. Ground improvement involving placement of fill or direct excavation/replacement are considered unviable in segments with clay thickness greater that 7.5m and settlement larger than 2.5m because of serious adverse ecological and socio-economic effects. This meant that a combination of ground improvement and piling can be implemented to minimise cost.

Piling option is largely preferred in the 20 – 30km chainnage, characterised by thick clay sequences and river crossings, in spite of its high cost, due to long-term stability and minimal adverse eco-hydrological effects.

INTRODUCTION

The proposed Nembe Town - Brass Island road is a part of the planned Ogbia -Nembe – Brass road. When constructed, it will be the only road link to the Brass Island, with a host of important establishments, such as the Brass LNG, one of Nigeria's four crude oil export terminals, an unused airport of international standard and eco-system with high ecotouristic value (NDES 2000). Since 1957, this road has been part of successive development plans, by various administrations, but without the good fortune of being implemented (RS Dev Plan-1975). A major factor for the lack of will to construct this road derives from the terrain, which consists essentially of diurnally flooded mangrove swamp forests.

It was perceived that a road through such a terrain would be inadvertently expensive.

This paper provides geotechnical data and reviews alternative road design scenarios with the potential for considerable cost minimization.

METHODOLOGY

The proposed road is over 48km in length and traverses both mangrove and freshwater swamp forests (Fig. 1), and five major rivers, varying in width and depth from 25m to 1.8km and 3.5m to 30m respectively.

Geotechnical data of the underlying soils were investigated for the design of a suitable pavement and bridge foundations. The selection of the investigative tools such as drilling rigs, CPT machine duly recognized the challenges of soil investigations in deltaic swamp environments in terms of equipment portability, ease of dismantling as described by George and Abam (1990). The geotechnical investigation determined the stratigraphy of the superficial deposit underlying the site and relevant engineering properties of the deposits.

Field investigations involved the drilling of Eighteen (18No.) boreholes to between 25m and 45m on swamp and bridge abutment areas. In addition, six borings were made over water at the principal river crossings. Eight (8No.) Cone Penetration Tests (CPT) augmented by eight (8No.) shallow Auger borings were carried out at 5km intervals along the proposed alignment.

The sub-grade beneath the proposed road was assessed by evaluating the ultimate net bearing capacity q_n for a shallow foundation using the Terzaghi's relationship (Tomlinson 1999)

(for strip) $q_{nu} = CN_c + P_o (N_q) + 0.5\gamma BN_\gamma$

where: γ = the bulk density of the soil below the foundation level

C = the undrained shear strength of the soil

P_o is the effective overburden pressure at the foundation level

N_c, N_q, N_γ = Terzaghi's bearing capacity factors obtained from charts

The potential total Settlement of the proposed road subgrade on the otherhand, was assessed based on projected net foundation load of $\Delta\sigma$ = 100kPa using the relationship $S = m_v.\Delta\sigma.\ H_o$

Where s = total settlement

Ho = thickness of compressible layer

m_v= H_o coefficient of volume compressibility

$\Delta\sigma$ = increase in vertical stress due to applied pressure

The total settlement was then computed from the summation of the vertical strains of individual soil layers caused by the foundation load as described by Tomlinson (1999).

Axial pile capacities were also computed for each river crossing for driven, single, straight-shafted, close-ended, tubular steel piles using method summarized by the following equations (Bowles 2005):

Ultimate carrying capacity $Q_{ult} = Q_b + Q_s$

Where: Ultimate base resistance in clay

$$Q_{bs} = 9.c_u.A_b$$

Ultimate base resistance in sand

$$Q_{bs} = P^1{}_0.N_q.A_b.$$

Ultimate shaft resistance in clay

$$Q_{sc} = \alpha\ c_u.A_s$$

Ultimate shaft resistance in sand

$$Q_{ss} = K_s.P_0.\tan\delta.A_s$$

Where:

C_u = average undrained cohesion at the pile base

A_b = base area of the pile

P_0 = effective overburden pressure at the pile base

N_q = bearing capacity factor

A_s = exposed area of shaft

K_s= coefficient of lateral earth pressure (K_s/K_0 = 1 to 2), for small displacement piles ratio varies from 0.75 to 1.75, K_0 = 0.6

P_0 = average effective overburden pressure over soil layer

α= pile wall adhesion

δ = effective soil/pile friction angle (smooth surface = 0.5 ϕ to 0.7ϕ)

Undrained strength to which a factor of safety of 1.8 has been applied as recommended by Eurocode $C_{ub} = \frac{Cu}{(Fs = 1.5 - 1.8)}$

Ultimate carrying capacity $Q_{ult} = Q_b + Q_s$

Allowable $Q_a = \frac{Q_b + Q_s}{3}$; $\frac{Q_b}{3} + \frac{Q_s}{2.5}$

Results and Discussions

The data from the soil boring and sampling, standard penetration tests, cone resistance soundings and laboratory tests were carefully evaluated and juxtaposed (Fig. 2) along the proposed alignment for meaningful interpretation of the stratigraphy. This regionalized stratigraphy indicated that the section of the route in the mangrove swamp was underlain by clays beneath which laid a predominantly sandy formation, the upper part of the Benin Formation. The thickness of the top clay layer varied from 6m from the Nembe end to over 40m at the Okpoma river, just before intersecting the beach ridge which consist of fine densely packed sand with intercalating thin clay layers.

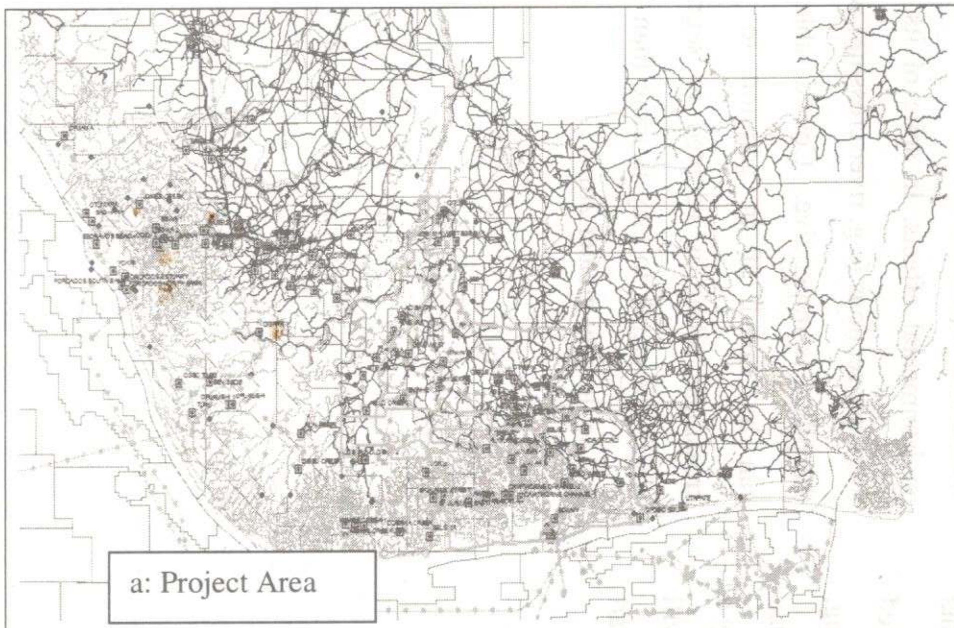
The clay soil exhibited very high moisture content (Fig. 3), mostly between 50% - 200% and low bulk unit weight generally between 12 – 15kN/m^3 (Fig. 4). Soil classification by Casagrande Chart (Fig. 5) indicated a wide range of plasticity suggestive of diverse sources of fine sediments transported and deposited in the area. The depth profile of the plasticity however indicated a narrowing of the range (Fig. 6), which may be attributed to the effects of sediment compaction resulting in the re-alignment of the structural clay.

Assessment of the liquidity indices showed that the clay soils were mostly in semi-solid state, underconsolidated, soft and weak with very low undrained strengths (2kPa – 14kPa) (Fig. 7), and with a tendency for strength enhancement with depth. The calculated ultimate and allowable bearing pressures were similarly low (Fig. 8), particularly in the mangrove swamp within the 20 – 30km span. These conditions predispose the soil to fluidization upon disturbance. The predicted total settlement for this span approximated 5m (Fig. 9), where a single vertical drainage was assumed. The combination of very low bearing capacity and significantly large total settlement in parts of the alignment implied that either ground improvement or piled foundation or a combination of the two was imperative as road design option.

The practicability of implementing ground improvement in this area and the magnitude of the resultant environmental effects are significant issues. Firstly, two forms of ground improvement were considered possible, namely; (a) the removal of the top clay up to 5m and its replacement with granular fill above maximum expected flood level; (b) the placement of preload, in the form of granular fill over and above existing soft and weak clay soil to a significant height in order to induce consolidation and enhance subgrade strength. The implementation of (a) in whatever form, either by jetting or physical excavation/evacuation would generate significant turbidity in the surrounding rivers/creeks (because of the existing ease of fluidization) with the attendant ecological effects. Furthermore, the relative strengths of tidal current of the rivers/creeks suggest that such turbidity although with a net downstream flow, is likely to persist over a fairly long time, possibly months with serious socio-economic consequences.

Due to the very soft and weak consistency of the top clay soil ($\tau < 10$ kPa), slope heights of even 1m can not be supported successfully, without sliding or slumping, thus further exacerbating the adverse ecological effects. Similarly, implementing a preloading soil improvement option will likely result in extensive soil failure, apart from the common challenge of sourcing the sand itself. Another significant challenge with both forms of ground improvement is the alteration of the local hydrology and its ultimate interference with the stability of the structure through gradual erosion of fill materials. Fortunately however, both the surrounding rivers/creeks and the road have an almost north-south alignment (orientation), thereby minimizing the effects of potential adverse hydrological problems.

The option of piling on the otherhand is considered expensive but with less adverse environmental effects. In spite of its cost, there are sections (e.g, river crossing) where the piling is imperative. Considerable cost can be saved, if in addition to river crossings, piling is adopted in segments where the depth to clay is more than 7.5m and settlement greater 2m.



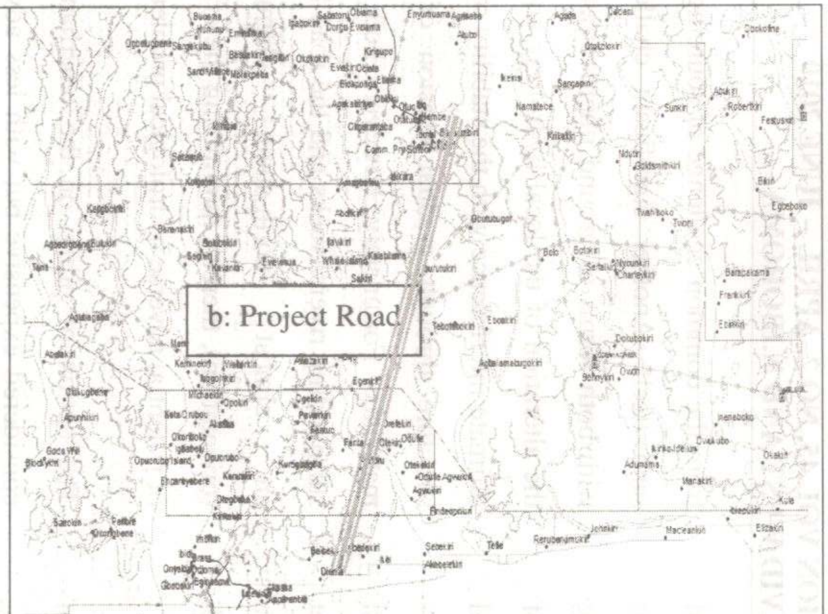


Fig. 1 Location of Project

A: Broad area of Niger Delta, showing of road network to the south

B: Alignment of road line between Nembe-Brass

C: Alignment of road, highlighting vegetation and ecological conditions

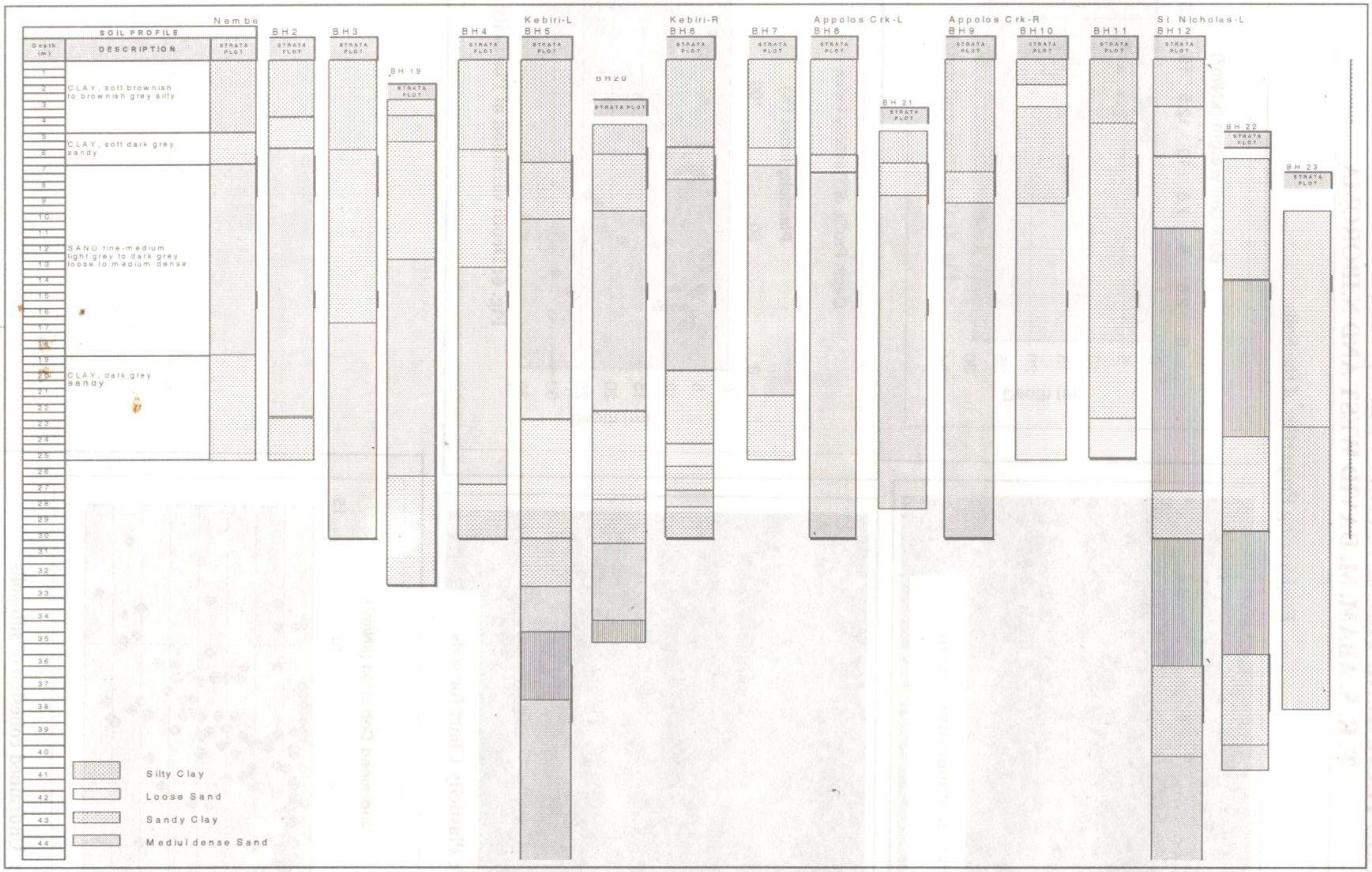


Fig: 2 Soil Stratification along the proposed road alignment

Engineering Properties of the Soils

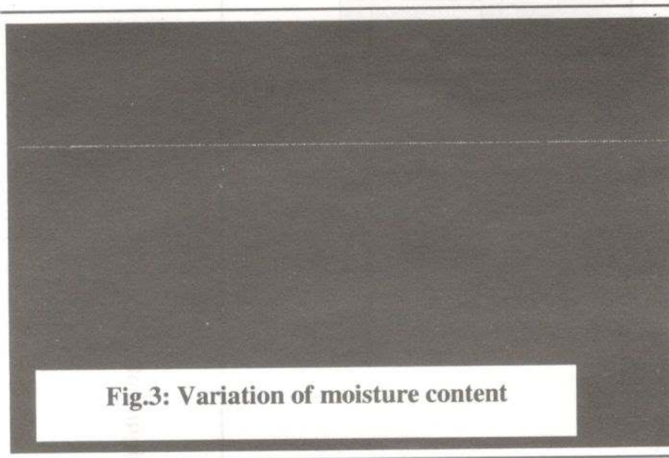


Fig.3: Variation of moisture content

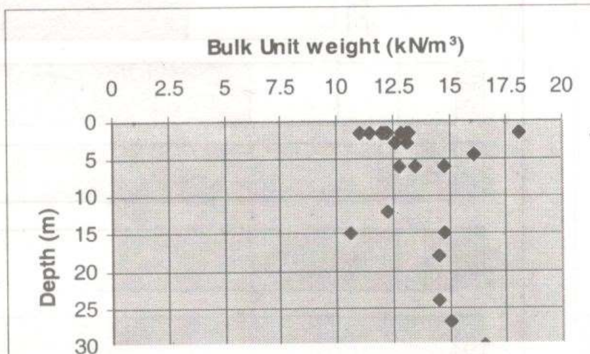


Fig. 4: Variation unit weight

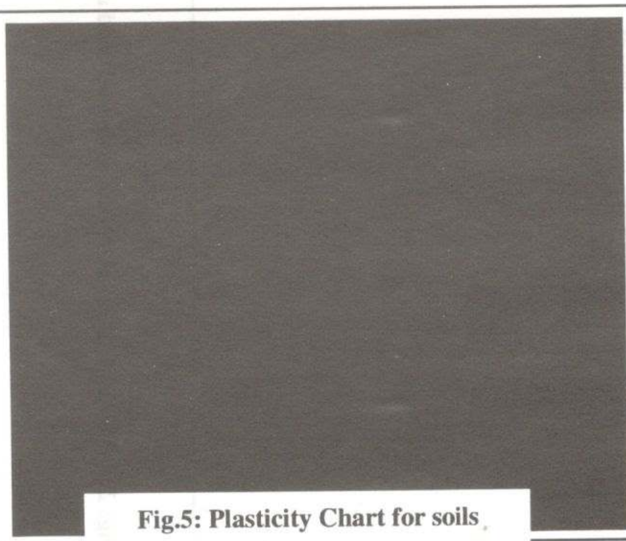


Fig.5: Plasticity Chart for soils

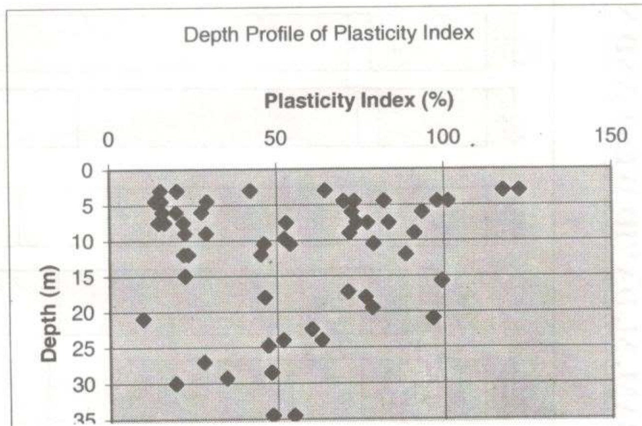


Fig. 6: Depth variation of Plasticity

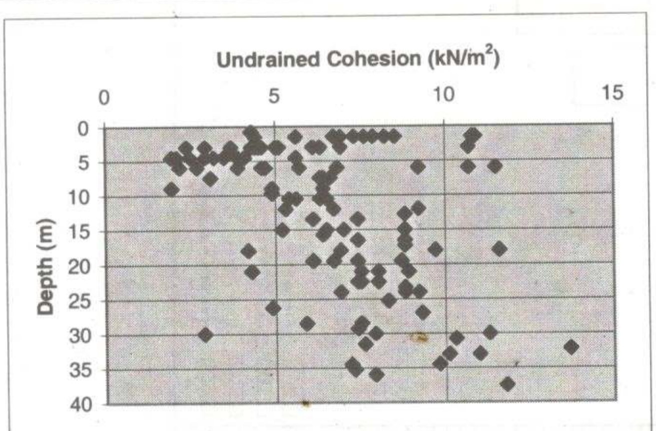


Fig.7: Undrained cohesion of sub-soils

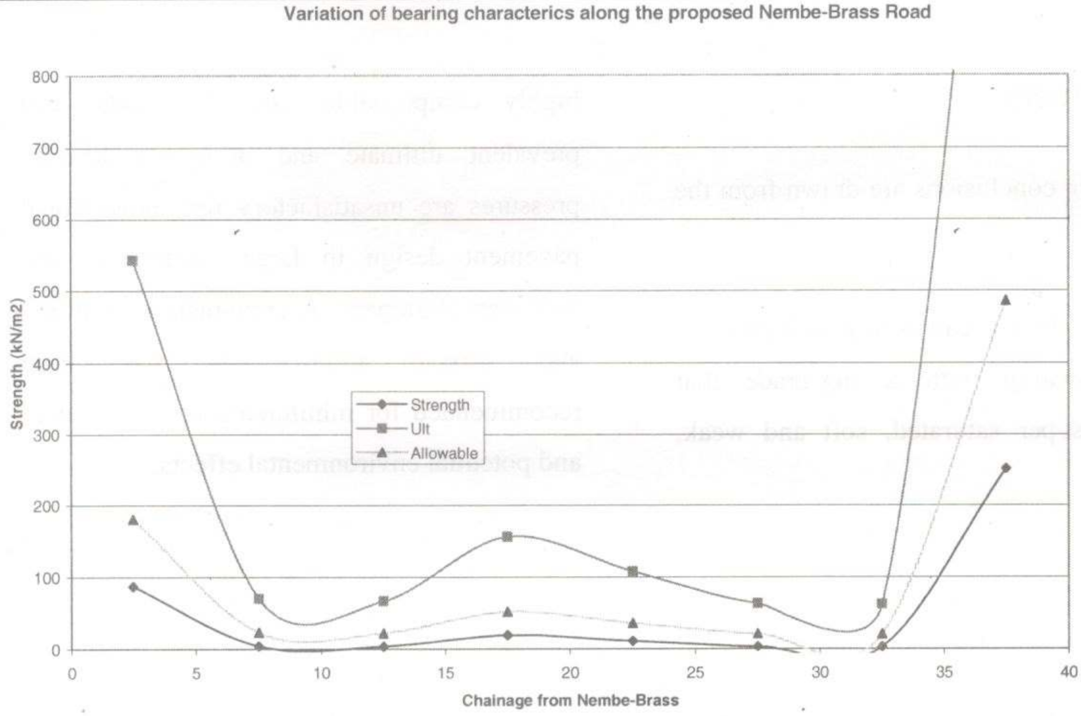


Fig. 8: Variation of undrained strength, ultimate and allowable pressure of sub-grade soils

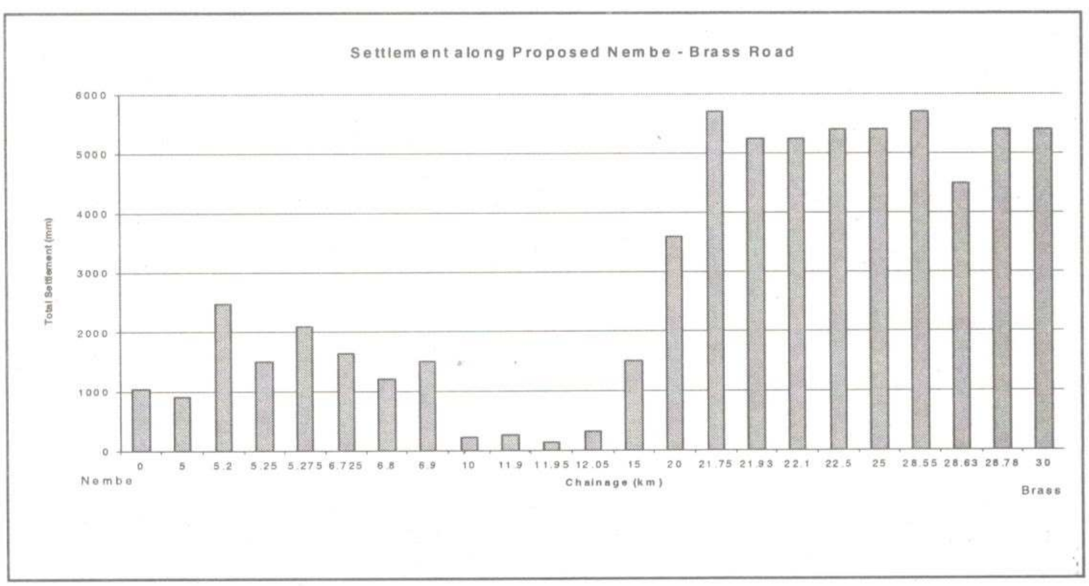


Fig. 9: Predicted settlement along the proposed alignment

CONCLUSIONS

The following conclusions are drawn from the study:

The Nembe – Brass road is largely within the Mangrove swamp with a sug-grade that consists of super saturated, soft and weak, highly compressible silty clay soils. The prevalent ultimate and allowable bearing pressures are unsatisfactory for conventional pavement design in large sections of the proposed alignment. A combination of piling and ground improvement has been recommended for minimizing the cost outlay and potential environmental effects.

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