STABILISATION OF NIGER DELTA FAT CLAY WITH BLEND OF BINDERS FOR SUBGRADE APPLICATION (PART 3) - TRIAL STABILISED ROAD SECTIONS IN SAMPOU, NIGER DELTA

Alayaki, F. M.[1*], Al-Tabbaa, A.[2], Ayotamuno, M. J.[3]

[1]Civil Engineering Department, Federal University of Agriculture Abeokuta, Nigeria
[2]Engineering Department, University of Cambridge, United Kingdom
[3]Faculty of Engineering, Rivers State University of Science and Technology, Nigeria
[1]alayaafunmi@yahoo.com, [2]aa22@cam.ac.uk, [3]mjayotamuno@hotmail.com
*Correspondence author (+234-805-651-1993)

ABSTRACT

The field performance of subgrade fat clay stabilised with additives and cement (PC) was evaluate by constructing trial sections along Odoni-Agbere road in Sampou, Bayelsa state. The blends mixed with the subgrade used in the study were, Drill Cuttings Ash (DCA)-PC (1:1), sand-PC (4:1), and lateralite-PC (2:1). Comparison of the dry density of cored stabilised samples with the laboratory samples showed that 100% compaction was achieved in the field for all the sections. Dynamic Cone Penetrometer (DCP) and field plate loading tests were used to determine the in-situ California Bearing Ratio (CBR) and settlement respectively. The in-situ CBR of the unstabilised and stabilised sections exceeded the targeted 15%. However, the in-situ CBR of the unstabilised section could be compromised if the soil is excessively manipulated and excess water is present. The percentage increase in settlement after two cyclic loading operations for the stabilised section ranging from -17 to 7%. This study revealed that the stability of the stabilised sections could be ensured if they are sealed on or before the 45th day after construction.

Keywords: Fat clay; Subgrade; Stabilisation; Trial road; Performance

Introduction

Many factors are contributing to the poor state of roads in the Niger Delta, and paramount amongst these factors are the terrain, the intense climatic conditions, and the geology. The deltaic rivers and creeks usually overflow during the raining season while the undraining terrain and erosion cause constant transportation of the soil. Reports given by Olorunfemi (1984); Akpokodje (1986, 1987); Essien and Udosen (2000); Ibemesim (2010) revealed that the underlying geology profile consists of thick layer of clayey soil and peat. Both the derived in-situ soil from rocks and the transported soils were also reported to be expansive, gap or uniformly graded, contain high moisture content, saline, and acidic.

Furthermore, the region has limited laterite outcrops which can be used as fill materials, and in fact Sampou town is completely devoid of them. The usual practice is to dredge sand from the rivers and creeks and use same as a replacement for the excavated poor soil or as a surcharge on it; a practice which is both environmental and economic unsustainable. Also, placing of the sand as a surcharge load without stabilising the underlying poorly in-situ subgrade might be creating excessive longtime consolidation stresses, thereby causing the massive road deterioration and failure which is visible in many places in the region. Hence, the need to improve the in-situ subgrade soil using locally available additives and binders that would give structurally effective and affordable pavement structure has become critical.

This paper presents the fieldwork carried out in Sampou, Bayelsa state, Nigeria involving the construction of trial sections and evaluation of the field performance of the formulated stabilising composites used to stabilise the site soil.

Site Description

Visual inspection suggested that the Sampou which is located in the suburb of Bayelsa State, Nigeria is within the swamp zone containing organic peaty clays, sands and composites of these, devoid of any red soils (laterites) occurring residually or as an outcrop. The community is less dense populated but the completion of Odoni-Agbere road is expected to attract increase traffic especially for those that would use it as a link road to other communities for commercial purposes. Consequently, the traffic on this road was estimated to be between 150 - 300 vehicles per day and a total of about 1 Million Equivalent Standard Axles over the design life relatively put at 10 – 15 years.

This site (shown in Plate 1) investigation was carried out at the beginning of the raining season thus, the ground was relatively firm to walk on in places which have not been worked on with the bulldozer, but water level was appreciably high, almost to the ground surface in some places on the site. Furthermore, the soil was very muddy and slippery in places where the soil had been disturbed especially where the terrain was flat and there was poor natural drainage. Information obtained from the site engineer revealed that the site could be almost impassable during the heavy rains. At the time of this site visitation construction work was in progress such as bush clearing, excavation of bad-spots, and filling was done using sand sourced locally from borrow-pit or dredged, and there was no improvement or stabilisation of the in-situ subgrade soil.

The construction practise on the site involved the placing and compacting of fine-grained sand on the clayey soil. The compaction was done with the sheep-foot rammer after every 150mm thickness of sand-fill, up to a compacted thickness of 1 metre or more in most sections of the road. Thereafter the top 100mm of the fill is stabilised with 2% cement to make the sub-base layer. The base layer is made using a mixture of granite stone, stone-dust, and about 8% cement, and the pavement is finished off with asphalt wearing course of about 60mm thickness. Side drains, in-built drains (in certain locations), and culverts are constructed at specified locations. All these loads are placed of a clayey soil with more than 85% of its particle sizes passing sieve 63µm. There was no chemical

Plate 1: Map showing location of the study area
Source: Google

stabilisation nor any particle size improvement of the subgrade.

Methodology

(i) Preliminary and Site Preparations

A portion of 5m width by 30m length along Odoni – Agbere road in Sampou, Bayelsa state, Nigeria was used for the pilot scale testing of the stabilising blends, and the construction was carried out on the 2nd of March 2015. The study area as recommended by the contractor for investigations had issues of problematic soil and the challenges faced in the construction operations. The fieldwork involved the construction of trial sections and evaluation of the field performance of the formulated stabilising composites used to stabilise the site soil. The laboratory investigations of the site soil and the stabilised variants are presented in Alayaki, et al. (2017).

The construction operation commenced with in-situ testing of the pH and salinity of the soil-water, and the natural moisture content of the soil. The in-situ pH ranged between 6.5 and 6.8, the salinity (total salt compounds) was less than 10%. Also, the natural moisture content ranged between 15 – 20%, and this was about the same range of the Optimum Moisture Content (OMC) obtained in the laboratory for all the composites. Based on the observation in the laboratory, that the soil tended to be difficult to mix and compact at water content above the OMC, no additional water was added during the mixing process. The topsoil was scarified and the subgrade formation level was established. Thereafter the 30m length of the portion to be stabilised was divided into 3 sections of 10m length each. An unstabilised portion of the road was used as the control.

Subsequent to the laboratory investigations reported in Alayaki, et al. (2017) which was the Part 1 of this paper, the blends (including the control section) presented in Table 1 were adopted for the field trials. The quantity of these additives and cement prescribed here were soil specific. These stabilising blends were dry-mixed with the subgrade at each sections. The analysis and discussion presented in the following sections were related to these composition, thus the tag name was used when reference was made to them.

Table 1: Compositions of the stabilised and unstabilised trial sections

Sections	Compositions	Tag
Section 1	Soil + 5% lateralite + 3% PC	Soil-lateralite-PC
Section 2	Soil + 20% sand + 5% PC	Soil-sand-PC
Section 3	Soil + 5% DCA + 4.5% PC	Soil-DCA-PC
Section 4	Unstabilised	Fat clay subgrade

(ii) Calculations of Actual Additives and Cement Quantities

DCA was bagged in 1 tonne jute-bag, both cement and lateralite were bagged 50kg, while the required sand was weighed using the pail-loader bucket. Equation 1 according to MRS07B (2013) was used to calculate the actual quantity of the additives and cement required in the field.

$$\mathrm{M} = (\mathrm{A} \times \mathrm{T} \times \mathrm{D} \times \mathrm{Co})/100000 \quad (1)$$

Where

M is Mass of stabilising agent in tonnes

A is Surface area of pavement to be in-situ stabilised in $m^2 = (5.5 \times 10)\ m^2$

T is Target Depth of stabilisation in mm (150 mm)

D is Dry density of a representative sample of material to be stabilised that has been compacted to 100% of standard compaction, in tonnes per m^3

Co = Ordered content of stabilising agent in percent (%).

Based on the calculations the quantities of additive and cement used were as follow:

- Section 1 - 0.62 tonne of lateralite (12 bags) plus 0.37 tonne of cement (7 bags)
- Section 2 – 2.77 tonne of sand plus 0.69 tonne of cement (~ 14 bags)
- Section 3 - 0.63 tonne of DCA plus 0.57 tonne of cement (~ 11 bags)

(iii) Construction of Trial Sections

Manual application of the additives and cement was done. For section 1, lateralite was spread on grid widths of 1.1m/1bag, the sand was spread manually as uniformly as possible while bulk bag of DCA was placed in the middle of the 10 metres section and spread uniformly also using rakes and shovels. All the sections were thereafter divided into grids so as to ensure even distribution of the cement. Section 1 was divided into grid width of 1.4m/ 1 bag of cement, section 2 was divided into grid width of 1.3m/1 bag of cement while sections 3 was divided into transverse grids of ~ 1m/ 1 bag of cement. The bags were placed centrally and spread uniformly (Plate 2 a- f).

Plate 2: Application of dry additives – (a): Measurement of the in-situ natural moisture content; (b): DCA brought in jute-bag; (c): Manual spreading of additives on scarified section; (d): All the additives applied on the subgrade; (e): Laying of bags of cement within grids; (f): Cement added on top of the additives

A depth of about 150mm of the in-situ subgrade soil was mixed with the composites using the rake-mixer machine, many passes were done until uniform colour and consistency were achieved. Care was taken by the operator to ensure that the stabilised soils were not transferred from one section to another. As stated earlier no additional water was added, in order to ensure adequate compaction of the stabilised composites. Three passes of the vibrating steel-drum roller was used to compact the stabilised soil, then water was sprinkled on the compacted surface to prevent cracking. A layer of sand about 50mm thick was placed on the compacted stabilised subgrade. Water was sprinkled at intervals throughout the curing period, to prevent the drying-out of the stabilised soil and to aid the pozzolanic reaction of the additives and cement. Core samples were taken from the sections to determine the quality of the field compaction. Furthermore, the in-situ strength was determined using the DCP and field quality of the field compaction. Furthermore, the in-situ strength was determined using the DCP and field plate loading test (Plate 3 a -f).

(iv) Evaluation of the Level of Compaction on Site

The effectiveness of the vibrating steel drum roller in the compaction of the stabilised composites was evaluated immediately after the process by using the core-cutter method. A sharp-edge split mould of 100mm diameter by 110mm height was used (Plate 3d). Two cored samples were taken by section and the average weight calculated. The bulk density (ρ_b) was obtained from equation 2:

Plate 3: Mixing and Compaction – (a): In-situ mixing of soil, additives and cement by the rake mixer; (b): Compaction using vibrating steel-drum roller; (c): Stabilised subgrade; (d): Maually coring of stabilised soil; (e): DCP testing of section; (f): Field plate loading test

$$\rho b = \frac{\text{weight of soil}}{\text{Volume of soil (or volume of the mould)}} \quad (2)$$

Then with the in-situ moisture content (m%), the dry density on site (ρ_d) was also calculated using equation 3, while relative compaction was obtained from equation 4

$$\rho d = \frac{100}{100+ \text{m\%}} \text{ x } \rho_b \quad (3)$$

$$\text{Therefore, RC} = \frac{\text{Dry density on site}}{\text{Dry density in lab}} \text{ x } 100 \quad (4)$$

Where RC is relative compaction (%)

(v) DCP and Field Plate Loading Tests

The dynamic cone penetrometer (DCP) test and field plate loading test were used to evaluate the in-situ strength of the unstabilised and constructed stabilised sections. The DCP test (Plate 3e) was carried out at three points per stabilised section and the penetration was taken down to 500mm depth. The unstabilised section was also tested at three points having different natural moisture levels and penetration taken down to 1 metre depth; this was done to know the variation of the in-situ CBR. The UK DCP version 3.1 software was used to generate the in-situ subgrade CBR at different depths and to obtain the adjusted Structural Number (SNP), which was used to define the pavement strength and determine the thickness of the overlying layer required. With the 60º cone used and other site conditions in-put, the TRL formula was chosen in the software to generated the CBR values.

Repetitive field plate loading test (two cycles) was done per section. A 750mm diameter plate with a hydraulic jack capable to lift a truck carrying ~20 tons of sand was set-up. The load was applied on the subgrade through the plate and using the jack to lift the loaded truck. The reaction on the jack was taken by the cross beam, which were anchored on both ends. The settlement of the plate was measured using 3 LVDT dial gauges of sensitivity 0.02mm, and which were placed at 120º apart. The gauges were fixed to independent supports and were not disturbed, except by their reactions to the lifting actions. The load was applied at incremental rate of 50 kN/min (Plate 3f).

Results and Discussion

Comparison of Laboratory and Site Compaction

The compaction level achievable on site relative to that obtained in the lab for all the stabilised composites were calculated. An average site moisture content of 18% was used and duplicate samples were cored per section. Table 2 shows that a commensurate compaction level was achieved on the site for all the section with an approximate relative compaction of 100% based on the calculated MDD. This revealed that the vibratory steel drum roller was able to densify the composites to the same level as with the West African compaction.

Table 2: Laboratory and site relative compaction

Sections	Average compacted weight (kg)	MDD in lab (kg/m^3)	MDD on site (kg/m^3)	Relative compaction (%)
Soil-lateralite-PC	1.67	1501	1634	109
Soil-sand-PC	1.72	1678	1687	101
Soil-DCA-PC	1.67	1530	1634	107

Evaluation of the Stabilised Sections

The evaluation of the strength index of the stabilised sections was performed between the 15th and 16th of April 2015, and this was about 45 days after the construction. By the 20th day the traffic in the community had started using the stabilised portion, because it appeared to be more stable. There were intense but short duration of thunderstorms during this period, with at least one occurrence in 2 weeks, and the temperature ranged between 28 – 38ºC. The site falls within the zone of the Niger Delta where rainfall occurs throughout the year. The only feature that could be established to differentiate the dry from the wet season are the lesser occurrence and shorter duration of rainfall, and the more intense temperature. Therefore, these weather conditions were expected to have some effect on the unstabilised and stabilised sections.

DCP Test

Table 3 presents the summary of the in-situ CBR, and the calculated overlying pavement thickness of the unstabilised and stabilised sections. The in-situ CBR values versus depth profile generated from the UK DCP 3.1 software are presented in Figure 1. Three points on the natural subgrade with different moisture contents were tested in order to have a non-biased in-situ CBR value, while a dry surface was noted for each of the stabilised sections. The average Structural Number (SNP) of each section which indicated the strength of the subgrade layer from the surface up to the final penetration depth, and this was used by the software to calculate the thickness of the overlying layer. An in-situ CBR value of 15% was targeted in this evaluation.

Table 3: Summary of Results of Laboratory unsoaked and soaked CBR, in-situ CBR, and the calculated overlying pavement thickness

Test Point	Sections	In-situ moisture (%)	Laboratory CBRu (%)	Laboratory CBRs (%)	Average In-situ CBR (%)	Average SNP	Pavement thickness (mm)
1		40 – 50					
2	Unstabilised	38 - 40	0.56	0.38	21	1.66	422
3		0 - 5					
4							
5	Soil-later- PC	Dry	10.32	16.53	16	1.53	389
6							
7							
8	Soil- sand- PC	Dry	15.53	10.02	23	1.77	450
9							
10							
11	Soil- DCA- PC	Dry	11.15	16.16	18	1.64	417
12							

CBRu = unsoaked CBR; CBRs = soaked CBR which was taken after 24hrs for the unstabilised soil and after 96hrs for the stabilised blends; later = lateralite; PC = cement

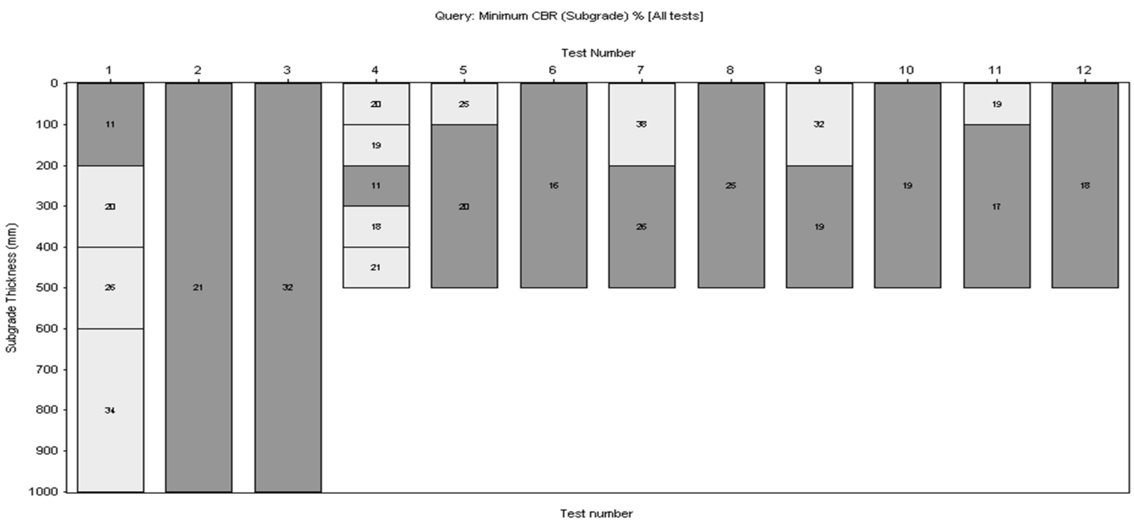


Figure 1: Subgrade in-situ CBR versus depth

(i) Unstabilised Section

The unstabilised subgrade section was characterised as fat clay soil with ~ 85% of its fine particles passing sieve 63 µm. Figure 1 shows that the test point 1 on the unstabilised section with 40 – 50% moisture content had the least in-situ CBR value of 11% of the first 200mm depth, and it increased to 34% down to the 1 metre depth. The point with moisture content range of 38 – 40% had in-situ CBR of 21%, while the point with the least moisture content range of 0 - 5% had in-situ CBR of 32%. The CBR of these latter two points were uniform from the surface down to the 1 metre depth. On the average, the unstabilised section had an in-situ CBR of 21%. These values were in complete variance to the unsoaked and soaked CBR values (0.56% and 0.38% respectively) obtained in the laboratory.

This high in-situ CBR value could be attributed to the light scarification applied to the subgrade during the clearing operation. This minimal disturbance of the in-situ fat clay and the relatively dry weather reduced its tendency to absorb water, hence the high strength exhibited. However, this enhanced strength of the subgrade during the dry season is transitional, because the contrast was observed with the soil during the wet season. Furthermore, the process of obtaining the disturbed soil sample, and the working on the soil during laboratory tests had shown that the soil responded well to the destabilising effect of water, and this was also reflected in the reported low CBR values of the remolded soil. These results and observations agreed with the reports in Akpokodje (1987); Omotosho and Ogboin (2009). The later authors reported 0% and 4% laboratory soaked CBR values for A-6 and A-7 Niger Delta soils.

(ii) Soil-Lateralite-PC Stabilised Section

The DCP test result of the first point (test point 4) on this section revealed that there were five boundaries with different in-situ CBR values at each 100mm depth down to the 500mm depth, and the least in-situ CBR value was 11% (Figure 1). Two boundaries were discovered at the second point with the top 50mm CBR value of 25%, and thereafter the value reduced to 20% down to the 500mm depth, test point 6 had a constant value of 16%. On the average the in-situ CBR value for the section was 16% which was higher than the targeted in-situ CBR value of 15%, and slightly lower than the 96hrs soaked CBR of 16.53% obtained in the laboratory. Hence, it can be hypothesized that the stability of the fat clay subgrade stabilised with lateralite-PC composite could be ensured when it is sealed not later than the 45th day after construction.

(iii) Soil-Sand-PC Stabilised Section

The soil-sand-PC had two layer boundaries at test points 7 and 9 with in-situ CBR values of 26% and 19% respectively while point 8 had uniform stratum with 25% CBR. The average in-situ CBR for this section was 23% which was higher than the targeted field value of 15%. It can be hypothesized that fat clay subgrade stabilised with blend of sand-PC composite could be expected to remain stable if it is adequately protected from water or sealed up by the 45th day after construction.

(iv) Soil-DCA-PC Stabilised Section

The DCP results of the test points 10 and 12 on this section revealed uniform in-situ CBR of 19% and 18% respectively, down to the 500mm depth. Test point 11 had 2 layer boundaries, and the in-situ CBR of the first 100mm depth was 19%, which decreased to 17% down to the 500mm depth (Figure 1). The average in-situ CBR for this section was 18% which was also higher than the targeted field value of 15%. Similar hypothesis proposed for other stabilised sections was also applicable for this section.

In summary, the unstabilised fat clay appeared to have suitable in-situ CBR value for subgrade application, when this strength index was measured under less intense weather condition and there was minimal disturbance of the top-layer. In contrast, the disturbed clayey soil could not be judged to be useful for such purpose without stabilisation. Furthermore, all the stabilised fat clay had in-situ CBR above the targeted value of 15% as at the 45th day after construction.

(v) Effect of the Stabilised Subgrade on Pavement Thickness

The structural number (SNP) generated from the software depicted the structural strength of the pavement, which in this scenario was obtained from the subgrade only. Thus, the overlying layer (combined sub-base and base) thickness on the subgrade was calculated by the UK DCP 3.1 software with Equation 5 as used by Pavement Interactive (2014):

$$SNP = a_i D_i \qquad (5)$$

Where: a_i = structural coefficient for each, and the value for the engineered material used to construct the upper layer was taken as 0.10 (based on standard design policy)

D_i = upper layer thickness in inches, which was converted to mm.

The section stabilised with lateralite-PC required the least upper layer thickness of 389mm, while other sections required upper layer thicknesses above 400mm. These values clearly defined the pavement thickness that should be placed on the stabilised sections, apart from the filling thickness to take care of the flood level. However, the 422mm thickness for the natural subgrade section was taken as improbable, based on the observations and discussion given earlier. Thus, the stabilised subgrade sections are expected to result into a huge saving in the construction cost, compared with the current practice on the site of sand filling the soft ground to a minimum thickness of 1m.

Field Plate Loading Test

Table 4 and Figure 2 present the summary of the results of the repetitive field plate loading test performed on the stabilised subgrade sections. The response to the loading of the stabilised surface in conjunction with the underlying stratum was revealed in the measured immediate settlement. This was just a fraction of the total settlement that is likely to occur over the service life of the pavement, and was used to predict the possible long-term settlement of the sections.

Table 4: Subgrade Settlement and Modulus of Subgrade Reaction

Sections	1st cyclic loading		2nd cyclic loading		Non-recoverable Settlement	% increase in maximum settlement
	S_1	R_1	S_2	R_2	$R_1 - R_2$	
Unstabilised	1.95	3.02	4.64	2.15	0.87	138
Soil-later-PC	2.72	0.91	2.27	2.27	-1.36	-17
Soil-sand-PC	3.36	0.81	3.39	1.33	-0.52	1
Soil-DCA-PC	2.84	0.92	3.03	1.16	-0.24	7

S = Maximum settlement during pressure (mm)
R = Rebound depth at zero load (mm)

(i) Unstabilised Section

Figure 2a shows that the first loading pressure on the untreated clayey soil caused an immediate settlement of 1.95mm at the maximum bearing pressure of 200 kN/m^2, and the unloading caused more deflection of the subgrade down to 3.02mm. The second loading process also caused further deflection down to 4.64mm, while a rebound depth of 2.15mm from the ground surface was obtained when the pressure was released. The natural subgrade recovered a settlement depth of 0.87mm after the whole process. It was noted that substantial increase in deflection/settlement of 138% occurred in the natural subgrade and this was the reflection of the nature of the clayey soil. Furthermore, this settlement is likely to be more during the peak of the raining season when the water in the environment would be extensive and the rate of evaporation would be lower.

(ii) Soil-Lateralite-PC Stabilised Section

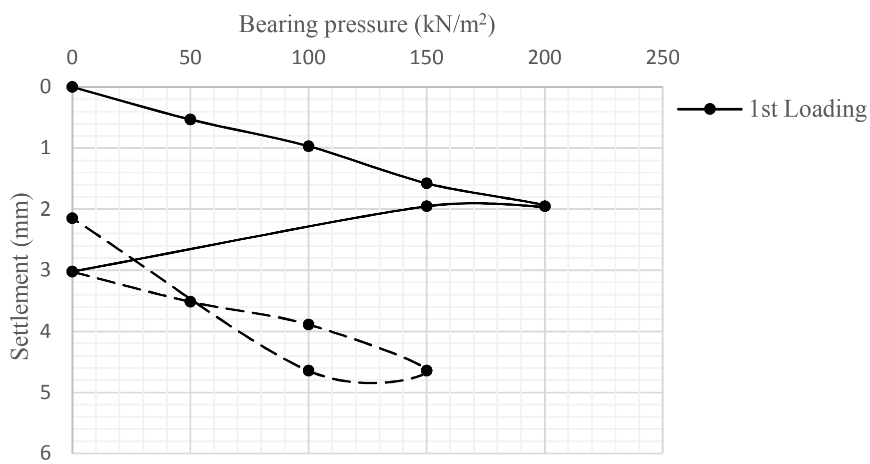
Figure 2b shows that the soil-lateralite-PC stabilised subgrade had a maximum settlement of 2.72mm during the first cyclic loading, and rebound to a depth of 0.91mm when the load was released. However, a reduction in maximum settlement depth of 2.27mm was obtained during the second cyclic loading, no further change or rebound in settlement was observed after the pressure was released. A percentage decrease of 17% was obtained between the first and second settlement with a non-recoverable depth of 1.36mm. It is thus evident that this section has reached a stability state by the second cyclic loading.

(iii) Soil-Sand-PC Stabilised Section

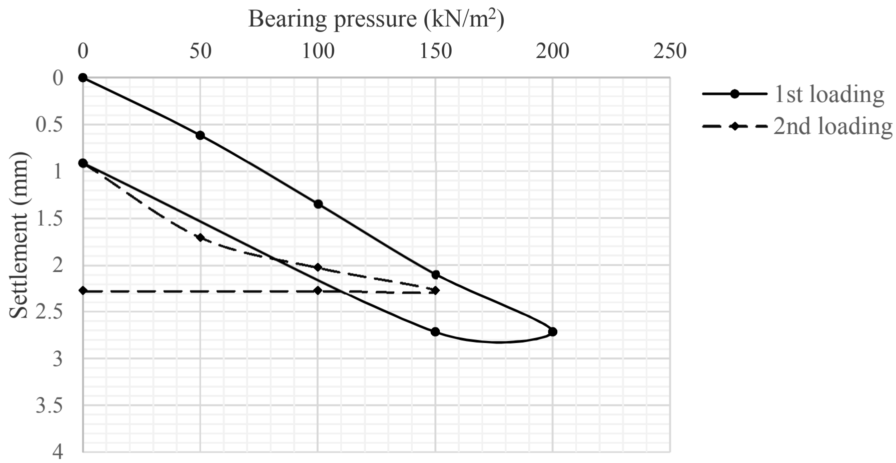
Figure 2c shows that the soil-sand-PC stabilised section had a maximum settlement of 3.36mm during the first loading, and a rebound depth of 0.81mm was obtained when the pressure was released. The second loading caused a maximum settlement of 3.39mm, which was a very minimal increase of 1%. A non-recoverable settlement of 0.52mm was calculated after the process. Also, slight buckling of the section could be seen from the shape of the graph.

(iv) Soil-DCA-PC Stabilised Section

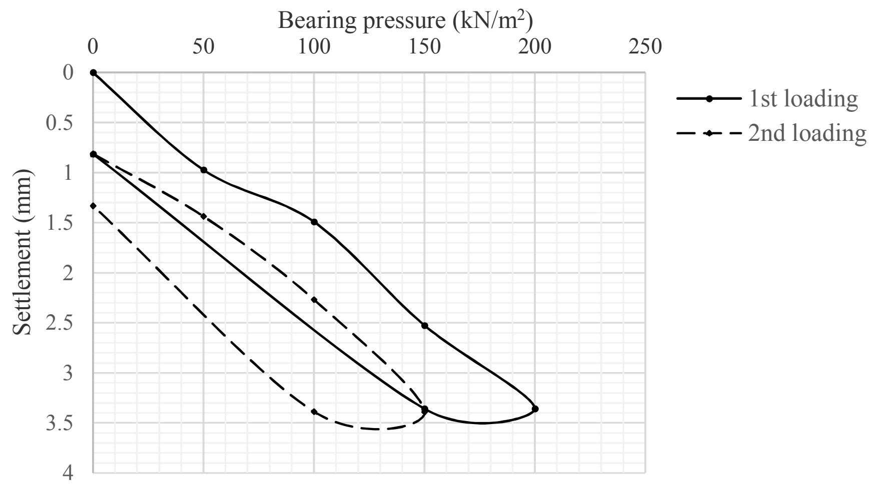
Figure 2d shows that the first bearing pressure caused a maximum settlement of 2.84mm in the soil-DCA-PC stabilised subgrade while a rebound depth of 0.92mm was obtained when the pressure was removed. The second bearing pressure caused a maximum settlement of 3.03mm with a rebound depth of 1.16mm. A non-recoverable settlement depth of 0.24mm was obtained after the whole process. An increase of 7% in maximum settlement was also calculated. The graph revealed that the stabilised section had the capacity to rebound when the loading pressure was removed. However, the shape of the curve shows that the loading pressure appeared to effect a buckling of the stabilised subgrade.



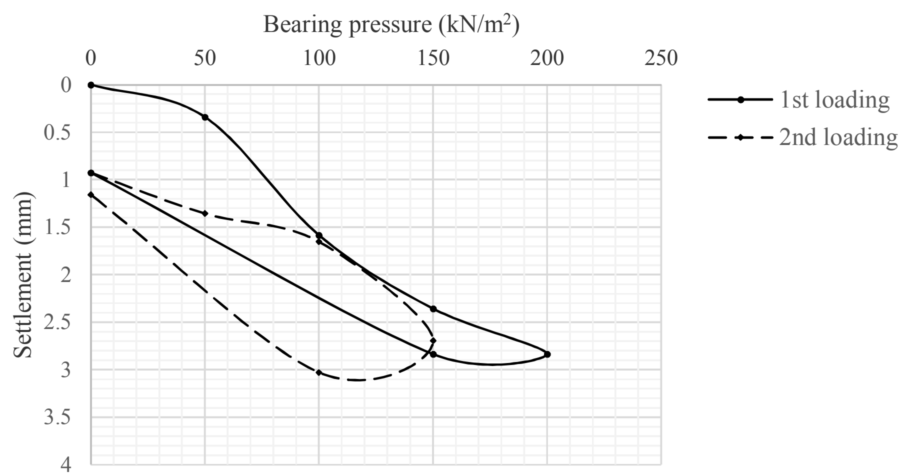
(a)



(b)



(c)



(d)

Figure 2: Bearing pressure versus Total settlement – (a): Unstabilised section; (b): Soil-Lateralite-PC Section; (c): Soil-Sand-PC Section; (d): Soil-DCA-PC Section

Conclusion

The laboratory and field analyses of Sampou subgrade revealed that the soil is not suitable for use in its natural state as subgrade material because of its susceptibility to the destabilising effect of water. The addition of blends lateralite-PC, sand-PC, and DCA-PC gave commendable improvement on the workability (based on laboratory observation), water resistant, and ultimately the bearing capacity of the soil. The field work showed that the stability of the stabilised subgrade sections could be ensured when they are sealed not later than 45th day after construction. The study therefore offers improved adaption of the fat clay subgrade in the construction of pavement in the Niger Delta.

References

Akpokodje, E. G. (1986): A Method of Reducing the Cement Content of Two Stabilized Niger Delta Soils. *Quarterly Journal of Engineering Geology, London.* Vol. 19, pp. 359 – 363.

Akpokodje, E. G. (1987): The Engineering – Geological Characteristics and Classification of the Major Superficial Soils of the Niger Delta. *Eng. Geol., 23:* pp. 193 – 211. Elsevier Science Publishers. B. V., Amsterdam.

Alayaki, F. M., Al-Tabbaa, A. and Ayotamuno, M. J (2017): Stabilisation of Niger Delta Fat Clay with Blend of Binders for Subgrade Application – Part 1. *Nigerian Journal of Technology (NIJOTECH)*, Vol. 36, No. 3, pp. 740 -748.

Department of Transport and Main Roads (2013): Main Roads Specifications and Technical Standards, MRS07B Insitu Stabilised Pavements using Cement or Cementitious Blends. State of Queensland.

Essien, J. P. and Udosen, E. D. (2000): Distribution of Actinomycetes in Oil Contaminated Ultisols of the Niger Delta Nigeria. *Journal of the Environmental Sciences.* Vol. 12 (3) pp. 296 – 302.

Ibemesim, R. I. (2010): Effect of Salinity and Wytch Farm Crude Oil on Paspalum Conjugatum Bergius (Sour Grass). *Journal of Biological Sciences 10 (2):* pp. 122 – 130.

Olorunfemi, B. N. (1984): Mineralogical and Physico-chemical Properties of Niger Delta Soils in Relation with Geotechnical Problems. *Journal of African Earth Sciences.* Vol. 2 (3) pp. 259 – 266.

Omotosho, O. and Ogboin, A. S. (2009): Active Soils of Niger Delta in Road Pavement Design and Construction. *Geotechnical and Geology Engineering 27,* pp. 81 – 88.

Pavement Design (2014): What's My Structural Number?" 2 June 2014. http://www.pavementinteractive.org.

Transport Research Laboratory (TRL) Limited (2004): UK DCP version 3.1 software. Funded by UK Department for International Development (DFID) for the benefit of developing countries.