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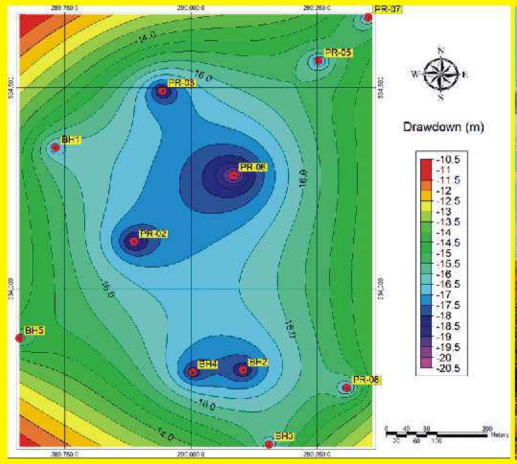

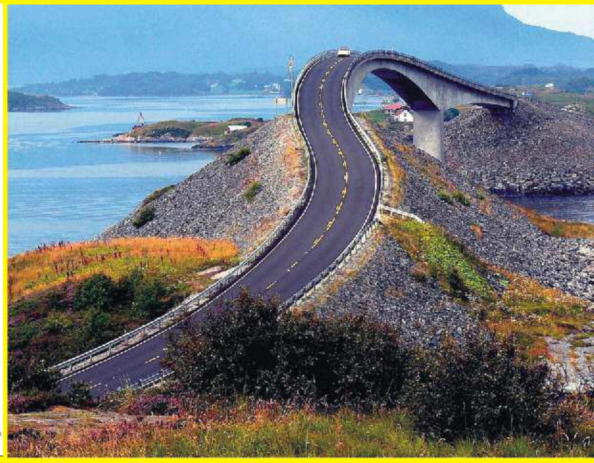

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NIGERIAN ASSOCIATION FOR ENGINEERING GEOLOGY AND THE ENVIRONMENT

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A LEAD PAPER (2018)

Proceedings of the Third International Conference of the Nigerian Association for Engineering Geology and the Environment (NAEGE), Port-Harcourt, Nigeria, 2018 (Vol. 2), 2019.

Sand Mining Threats to Coastal Infrastructure: Case Studies from the Niger Delta, Nigeria

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Abstract

Large scale sand mining from river beds is now common in the Niger delta, due to scarcity of sand for construction and scarcity of useable land, coupled with the necessity of reclaiming land for development purposes. There is currently no regulation as to where sand can be mined in river channels because of the lack of understanding of the risks to coastal infrastructure involved with its abstraction. Two types of instability were distinguished, one relating to the equilibrium slope of the riverbed and the other riverbank instability. This study has shown through computation, supported by examples that a minimum distance of 94m (for sand river beds) from a bridge should be observed to guaranty the safety of bridge foundation. For clay riverbeds, slightly shorter minimum distances can be considered safe. The study further shows that the ability of sand borrowing in river channels to generate bank instability is dependent on the composition and stratigraphy beneath the river bed.

***Keywords**: Sand, Mining, Coastal Infrastructure, River Beds, Niger Delta.*

Introduction

Sand mining from river beds is widespread in the Niger delta with attendant adverse effects. The several potential adverse impacts of indiscriminate sand mining are well documented in literature (Bull and Scott, 1974; Collins et al.; 1990, Lake and Hinch, 1999; Padmalal et al 2008; Anooja et al 2011;). As a summary, these includes: a) bed degradation and consequent effects on channel and bank stability, b) increased sediment loads, decreased water clarity and sedimentation; c) changes in channel morphology and disturbance of ecologically important roughness elements in the river bed; d) ecological effects on bird nesting, fish migration, angling, etc. e) modification of the riparian zone including bank erosion; f) direct destruction from heavy equipment operation; g) discharges from equipment and refueling; h) Reduction in groundwater elevations; i) impacts on structures and access; j) biosecurity and pest risks; k) impacts on coastal processes. On the lowering of the base level of the river, Padmalal et al (2008) reported a case study in which riverbed in the storage zone was lowered at a rate of 7–15 cm y^{-1} over the past two decades. The lowering of base level, in turn, imposed severe damages to the physical and biological environments of the river systems.

The extracted sand which served mostly as construction material as well as fill for land reclamation projects, is persistent across the Niger delta. This sand is also the predominant aquifer in the region. Reclamation remains a veritable source of creating new land for development in the region, where usable land is a premium. This is due to a combination of factors including: relatively low elevation of the region with respect to surface water level and widespread occurrence of compressible sediments. Extensive areas of swamp land are therefore periodically reclaimed by Hydraulic sand-fill, dredged from surrounding rivers and creeks.

Due to lack of understanding of the ground response to the excavation and removal of surrounding sand materials, such river bed and neighboring river banks progress into degradation, beginning with insipient motions of grains (Porto and Gessler, 1999) and in the process threatening the safety and stability of major coastal infrastructures in the area.

The process of incipient motion in natural rivers is closely related to the problem of critical shear stress of sediment mixtures. Shields (1936) proposed his widely accepted criterion for incipient motion of uniformly sized bed material:

$$\frac{\tau_c}{(\gamma_s - \gamma_w)D_s} = f\left(\frac{u_* D_s}{\nu}\right) \quad \text{...(1)}$$

Where τ_c = critical shear stress;
G_s = specific weight of sediment;
G_w = specific weight of the fluid;
D_s = diameter of the grains;
u* = shear velocity; and υ = kinematic viscosity of fluid.

Its applicability is, of course, limited because the criterion was established for uniformly sized bed material. In an attempt to generalize the criterion for sediment mixtures a number of authors have suggested the use of a single "representative" diameter for the mixture. According to Porto and Gessler (1970), better predictions of the ultimate bed slope are achieved using the criterion suggested by Gessler, which is based on the stochastic analysis of incipient motion in non-uniform bed material. In the case of a sediment mixture, Gessler (1967) observed that grains of the same size are partly in movement and partly at rest. Consequently, he treated the incipient motion of sediment mixtures as a probabilistic problem. He assumed that the fluctuations of the bed-shear stress are normally distributed.

According to Gessler (1967) and El-Gamal (1991), the following formula can be used to predict the critical shear stress of the sediment mixture and then the corresponding equilibrium bed slope:

$$\gamma_w h_f S_e = 0.045(\gamma_s - \gamma_w) D_{AVGe} \quad \text{..........................(2)}$$

where
D_{AVG} = average grain size of the armor coat corresponding to $q^{-} = 0.5$.

This equilibrium slope of river beds differs from the stable slope of river banks which have been elaborately discussed by several researchers, including, Thorne (1982), Abam (1993), Abam and Omuso (2000). Consequently, two types of equilibrium slopes are implicated by sand borrow in river channels. This paper investigates the mechanisms of river bed and bank instability triggered by hydraulic sand mining within river channels and reviews case histories to illustrate the dangers posed to coastal infrastructures, especially jetty and bridge foundations and abutments, and to ensure that sand extraction is carried out in a sustainable way to maintain river equilibrium by determining the locations to be extracted

Regional Geology and Site Description

The geological formations in the area consist of the Quaternary sedimentary deposits, and the Tertiary Coastal Plain Sands, generally referred to as Benin Formation. The Quaternary sediments give rise to alluvial plains. The alluvial plains include the estuarine sediments, which are under the influence of tidal brackish waters along the coast and in the estuaries of rivers and creeks.

The general geology of the area therefore reflects the influence of movements of rivers, in the Niger delta and their search for lines of flow to the sea with consequent deposition of transported sediments. The surface deposits in this area comprises silty and sandy-clays. These surface layers are frequently thick (greater than 10m) and would inevitably impact on the road design. The sandy layers underlying the silty-sandy clay are predominantly medium to coarse in grain sizes and found to exist in mostly medium state of compaction. It is this sand that is widely extracted construction and for reclamation.

Method of Investigation

Tidal data was obtained from records of ADCP measurement in nearby creeks. Information on tidal velocity is important not only for predicting the initiation of particle entrainment, but also for the management of transported/buoyant pollutants, which in this case would largely be silt dislodged by the dredging process. Tidal velocities determine the extent of transport of silt particles re-suspended by a dredging operation and assist in the choice of optimum locations for silt curtains to prevent wide spread silt contamination.

Thirteen borings were made over water at three study sites using a workshop fabricated light shell and auger percussion rig mounted on a barge (Figure 1).

Drilling over water on Geotechnical Barge

Fig. 1: Barge mounted rigs for geotechnical exploration

During the boring operations, disturbed samples were regularly collected at depths of 0.75m intervals and also when change of soil type is noticed. All samples recovered from the boreholes were examined, identified and roughly classified in the field and used in the production of lithostratigraphy of sediments beneath the seabed. Particle size distribution analysis was carried out in accordance with the British standards (BS 1377 of 1990) in order to classify the sandy units.

Results

The River system subject to diurnal tidal inundation with Mean Tidal level averaging 1.52m. Tidal velocities vary across the tidal cycle, with peak velocities up to 1.4m/s occurring at mid ebb tide (Figure 2).

The excavation of sand from the river bed created large borrows of 40m diameter and 18m in depth in scattered locations. These pits intercepted and trapped bed load, creating a deficit in the transported sediment and disrupting the sediment transport equilibrium of the river/creek. In a bid to re-establish this equilibrium, the river increases its appetite for erosion, beginning from the most vulnerable areas. Firstly, sub-aqueous slope failures will occur in the near vertical slopes of the pits, altering the bathymetry and creating a steeper basal configuration with comparatively faster flow velocities, with correspondingly higher erosive power and increased capacity for transportation of entrained sediments.

Case Study 1

This case study is centered on the Amadi Creek, a tributary of Bonny River (Fig. 3); where some 2 million m^3 of sand was to be sourced for reclamation of swampland. The river width averages 200m (but with only 100m width at the narrowest point), while its length runs a course of over 3km.

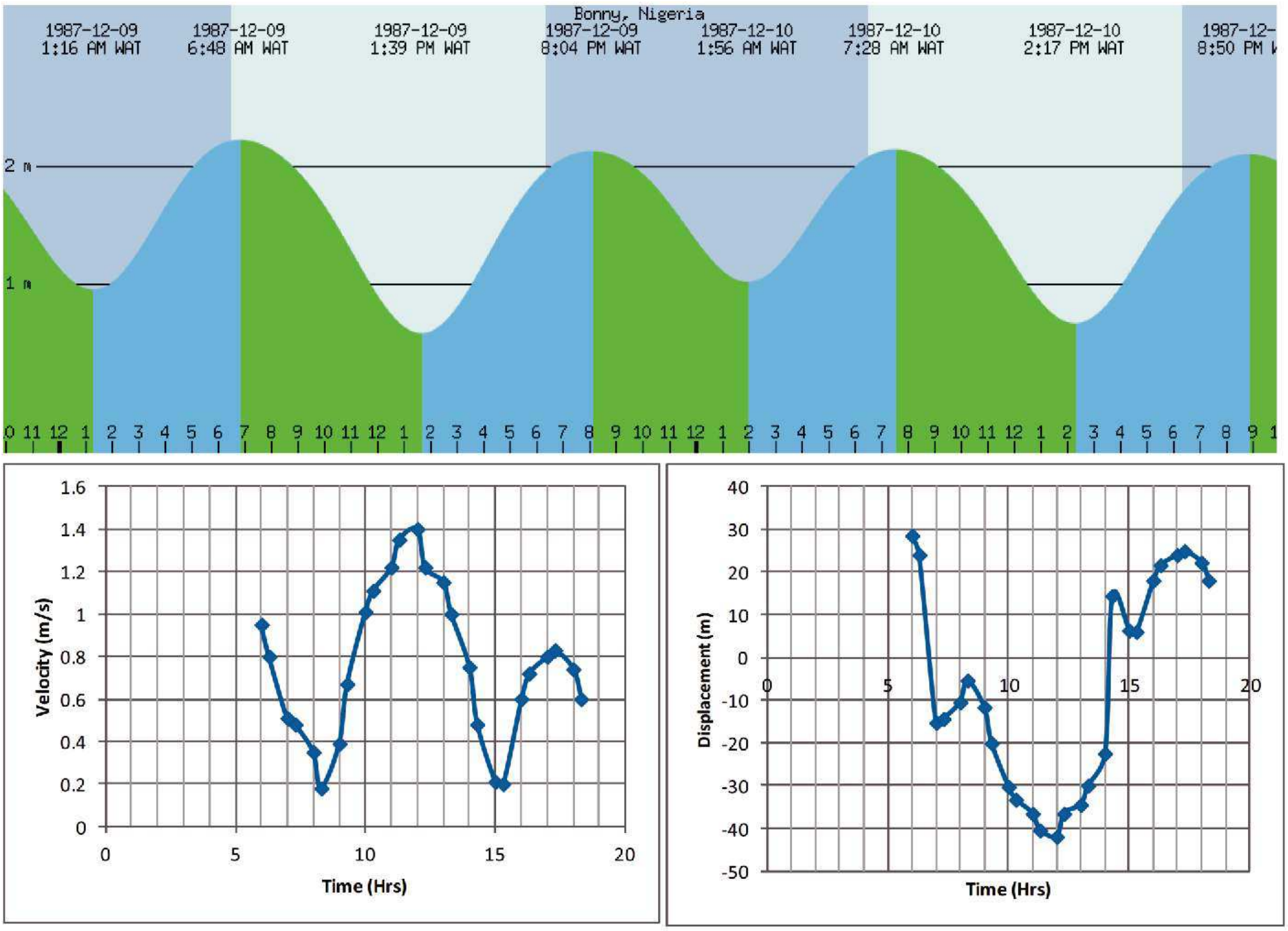

Fig. 2: Tidal regime in study area

S/ No.	BH Ref	Northing	Easting	Design Depth (m)
1	BH1	0281233	0529074	15
2	BH2	0281135	0528595	15
3	BH3	0281282	0528347	15
4	BH4	0281334	0527836	15
5	BH5	0281915	0527430	15

Fig. 3: Site Drawing showing Test Points and coordinates

The composite stratigraphy beneath the river bed (Fig. 4) which shows the vertical stratigraphic succession as well as lateral extent of each layer reveal a top silty clay (3.5m to 6m) thick overlying mostly medium sand that extend well beyond 15m. This composite further indicates the available and dredgeable sand in the area and forms the basis of both bed and bank stability assessment.

It is observed that sand was encountered between 3.5m to 6m below river bed. It is also observed that the sand formation is persistent and laterally extend from BH1 to BH5 locations about a distance of more than 2km. The potential performance of the sand for fill or as construction material or even as filter material can be assessed from its particle size distribution (Table 1). For example, the sand encountered here are mostly within the medium-coarse size fraction. The drainage properties were considered excellent with estimated average permeability of 0.007m/sec. The angles of internal friction averages 32^{o} while Bulk Unit weight averaged 18.8kN/m^{3}.

Since the dredging was aligned principally along the central axis of the river, the distance to each river bank would be less than 50m at the narrowest section. In this circumstance, the average slope from the base of the borrow pit to the base of the riverbank would be 41^{o}, which should present an unstable slope condition, if the bed materials comprised of sand. However, because the bed consists of at least 3.5m thick clay (3.5m to 6m), bank instabilities were not observed, possibly because the dredging did not trigger them. This implies that the ability of sand borrowing in river channels to generate bank instability is dependent on the composition and stratigraphy beneath the river bed, which is much in line with Abam (2004).

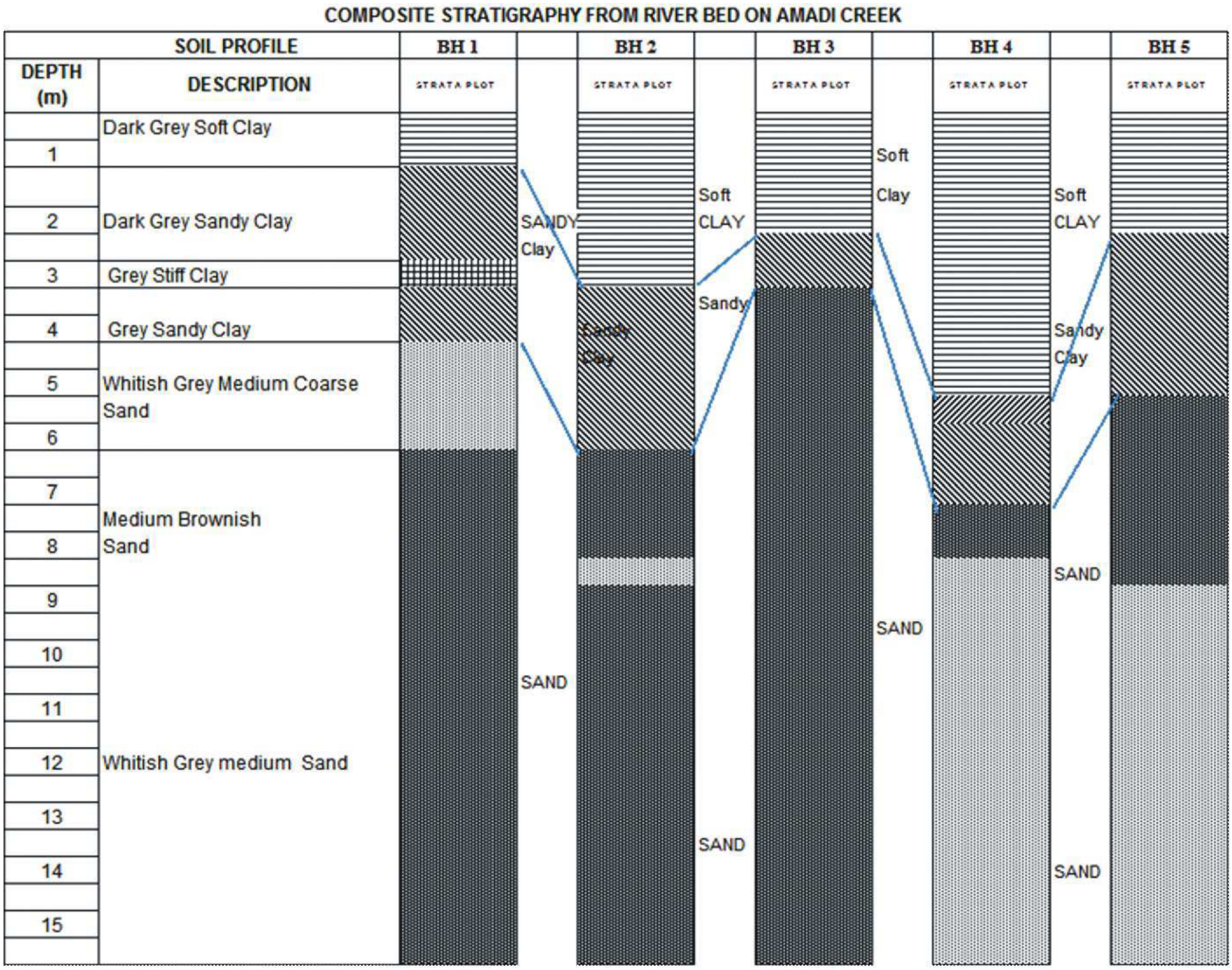

Fig. 4: Lithostratigraphy of the River bed in study area.

Table 1: Particle size distribution statistics

BH NO.	Depth	D_{10}	D_{30}	D_{50}	D_{60}	$Cu=D_{60}/D_{10}$	$Cz=D_{30}^2/(D_{10}*D_{60})$	$K=C*D_{10}^2$
	(m)	(mm)	(mm)	(mm)	(mm)			(m/sec)
	7.5	0.24	0.38	0.53	0.63	2.63	0.96	0.006
1	10.5	0.34	0.47	0.7	1.2	3.53	0.54	0.012
	9	0.28	0.4	0.53	0.64	2.29	0.89	0.008
2	12	0.33	0.44	0.58	0.75	2.27	0.78	0.011
	15	0.23	0.37	0.49	0.56	2.43	1.06	0.005
3	9	0.27	0.37	0.47	0.54	2.00	0.94	0.007
	15	0.33	0.47	0.68	0.87	2.64	0.77	0.01089
4	10.5	0.17	0.34	0.46	0.54	3.18	1.26	0.003
	13.5	0.15	0.24	0.4	0.56	2.15	0.99	0.00169
5	9	0.23	0.39	0.56	0.76	1.69	0.93	0.00529
	12.5	0.23	0.37	0.52	0.65	2.83	0.92	0.005
	15	0.3	0.43	0.58	0.73	2.43	0.84	0.009

Case Study 2

Case history 2 is in the neighboring area of Okrika (Figure 5), where sand was abstracted randomly around 3 connecting bridges for reclamation in Abam-ama, Okrika, Kalio-ama and Oba-ama communities.

The erosion and lowering of the basal level around the abutment within the intertidal zone resulted in its undermining and subsequent detachment and failure of the concrete protection to the abutment. This causes the sand backfill, hitherto confined by it to find an outlet as shown in Fig. (6a). The movement or flow of the sand,

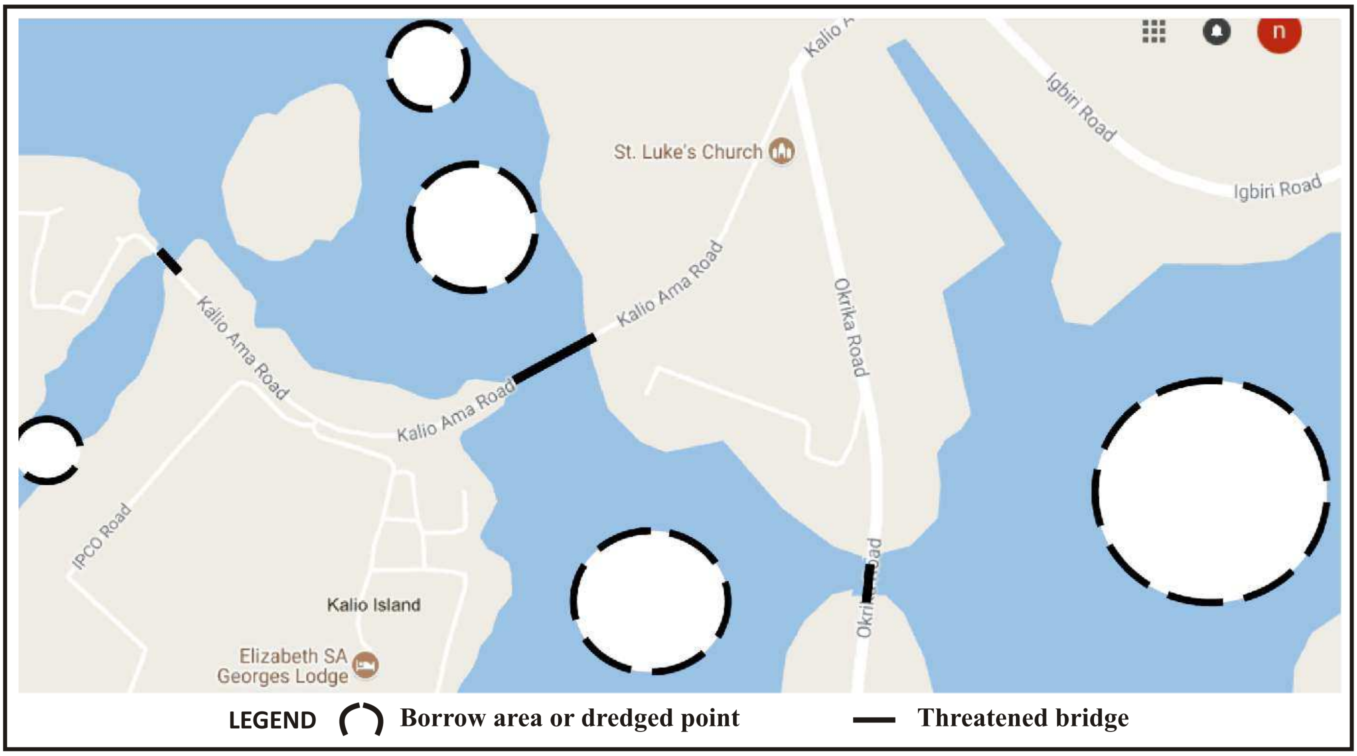

Fig. 5: Map showing dredging locations and threatened bridges

which is exacerbated by infiltration of rainwater and seepage through the abutment, results in the loss of support to the base course and eventual collapse and failure of the asphaltic surface (Fig. 6b). Surprisingly, all three bridge connections in the neighborhood experienced same erosional mechanism and ultimate fate.

Fig. 6: Erosion of Abuttment base and failure of asphaltic surface

Case Study 3

Only recently (2015), the Bridge at Nkpogu-NLNG, which is the upstream section of and feeds into Amadi Creek (case study 1 area) was threatened to the point of failure by accelerated erosion (Fig.7), variously ascribed to an earlier dredging activity downstream of that location.

Fig. 7: Failures due to erosion of bridge abutment at Nkpogu

Unlike in case study 1, the river bed is underlain mostly by medium and coarse sand from top to 30m (Fig. 8). The angles of internal friction varied from 32° to 35° with effective particle size D_{10} averaging 0.15mm and SPT N-values varying from 6 to 31.

In much the same fashion, the dredging lowered the basal level, increased ebb tide flow velocity which eroded the abutment and caused the bridge to fail.

Discussion

Assessment of Threat to Bridge and other Coastal Structures

One major consequence of dredging the Amadi Creek is the deepening of the water channel which has implications on the stability of coastal infrastructure and river banks, besides the dislodgement of aquatic ecosystem and effects on the physical hydrology. In some cases, these potential effects are evaluated during Environmental Impact Assessment.

To further illustrate the effect of channel deepening on bank stability, the width of the channel at the narrowest cross-section was measured at 196m. If sand excavation creates a 100m wide and 18m deep burrow at mid-stream as illustrated (Fig. 9): The maximum angle of repose would be given by $\alpha = Tan^{-1}((18/48) = 41°$.

Since the seabed is underlain by top clay, we expect some degree of vertical slope. A 3m high slope has been

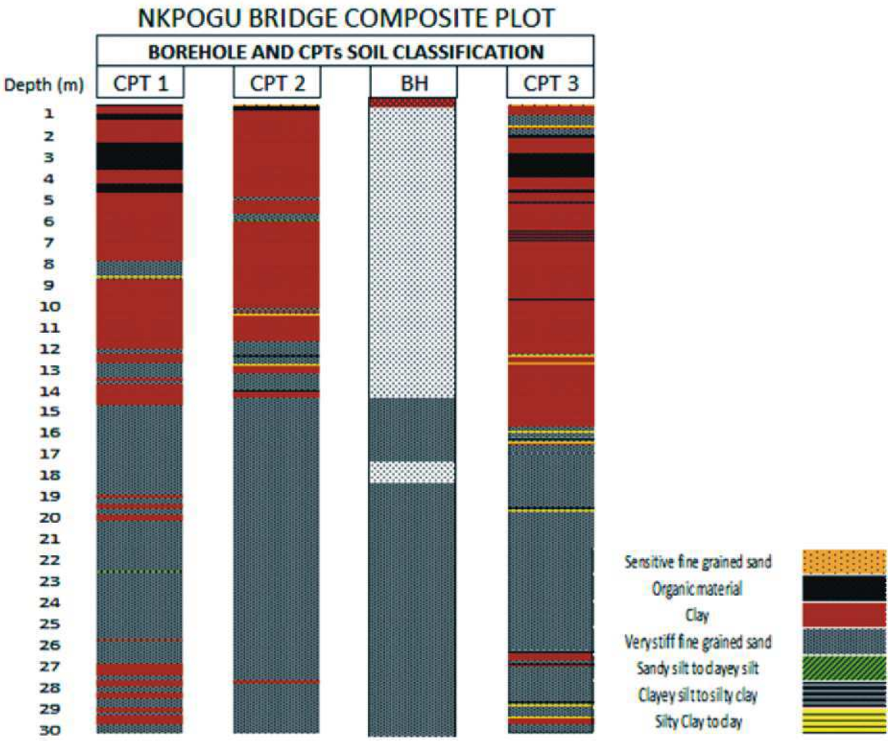

Fig. 8: Lithostratigraphy composed of borehole log and CPT based SBT classification

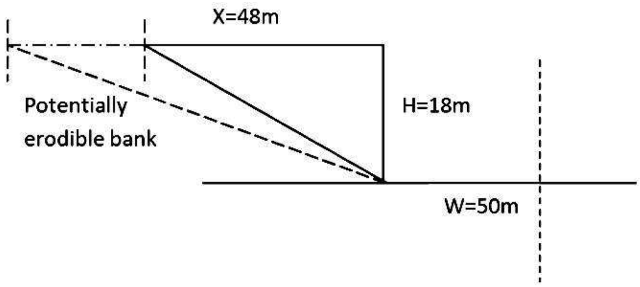

Fig. 9: Dimensions of a section of the channel

assumed based on the properties of this material. This implies that a lower angle of repose should in reality be expected as:

$\beta = Tan^{-1}(15/48) = 34^{o}$ ....(3)

Work by Kleinhans et al 2011 show that angle of repose of submerged sand can vary between 30 and 40 degrees depending on the size of the particles. With a D_{50} averaging 0.5mm, an angle of repose of 36 degrees can be expected.

This computation shows that dredging a burrow that is 100m wide and 18m deep would create near unstable conditions. It is therefore recommended that on no account should the width of the burrow be larger than 100m and deeper than 18m. Where possible the width of the burrow should be reduced to between 60m and 70m to permit a lower angle of repose of between 25 and 29 degrees.

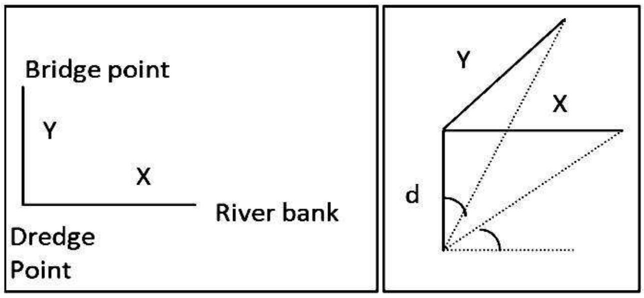

Fig. 10: Schematics of dredged river channel showing angular relationships

In respect of stability of river bank, the angle of declination (α) from river bank:
$\alpha = Tan^{-1}(d/x)$; $X = (d/Tan\ \alpha)$.....................................(4)
By similar reasoning, the angle of declination from river bed at bridge point to dredge depth:
$B = Tan^{-1}(d/y)$..(5)
Where d=dredge depth (m)
X= horizontal distance to river bank
Y= horizontal distance to bridge footing

Thus, the minimum safe distance from river bank to dredge point can be given as: X_{min} = d/Tan (Angle of repose of dredged sand). In general, environmental safety of the river bank requires as α minimum horizontal distance, that, X is greater d/Tan (Angle of repose); ie, the ratio (d/X) should always be less than 0.32 and must not exceed 0.40.

The angle of internal friction in shear test, which approximate the angles of repose, ranged from 26° to 32°. Under submerged conditions, these angles would range from 28° to 33°. Applying α factor of safety FS = 1.5; these angles of repose would then range from 18o to 22°. At FS = 2.5 which takes account of the sensitivity of the coastal infrastructure, the corresponding angles of repose would be 11° to 13°, which would translate to α minimum safe distance of 94m for α dredging operation requiring α dredging arm with α capacity of 18m of dredged depth.

The instability created by the presence of α sub-aqueous borrow pit would extend over an area, the longitudinal extent of which can be estimated by considering the schematics in Figure 11.

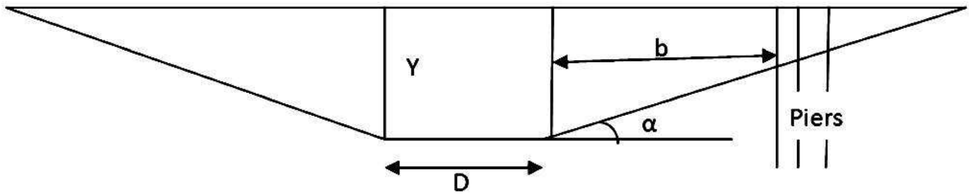

Fig. 11: Schematics for assessing area of influence of sub-aqueous borrow pit
α = angle of repose. D = diameter of borrow pit

Assuming α circular borrow pit of depth Y and diameter D, α Longitudinal Area of Influence (LAI) (which is essentially the length across the channel of flow over which the pit can induce instability on the river bed) can be can be estimated by the equation:

$LAI = D + 2Y/Tan\alpha$ ....(6)

Where α = angle of repose

By the same token, the Area of Influence (AOI) on the river can be estimated by the expression:

$AOI = \int (2Y/Tan\alpha + D)^2/4$ ....(7)

The implication of this is that, if the measured width of α river channel is less than the predicted LAI (Equation 6), then river bank instability induced by dredging is imminent.

The potential for scour for the bridge foundation and abutment was also assessed based on the hydrology of the study area, with tidal velocities peaking at 1.2m/s. The turbidity of the water indicated the movement of substantial suspended load in the flow. Since there is reasonable narrowing of the flow area at bridge crossing location, the river flow velocities are expected to increase significantly. Under these flow conditions, it is highly probable for the development of scour around the bridge piers and abutments. Furthermore, If α bridge pier is within the area of influence of α borrow pit, then it is probable that additional scour arising from bed instability caused by the pit can be introduced. Such scour can be estimated by the expression:

$S_i = (Y - bTan\alpha)$ ....(8)

Where b is the distance from the borrow pit to the bridge pier
For long-term conditions, α = 11°
For medium-term conditions, α = 18°
S_i is to be checked against the depth of pile embedment for assessment of continuing stability of the pile or pier as the case may be.

Conclusions

Sand mining from river beds will continue for the foreseeable future. Bridge foundations as well as abutments can be vulnerable to scour and erosion, if the choice of dredging locations on river channels are not properly guided. There is need to develop regulation for sand mining activities in inland rivers. Based on this study it is shown that α minimum distance of 94m (for sand river beds) from α bridge should be observed to guaranty the safety of bridge foundation. Furthermore, it is concluded that the ability of sand borrowing in river channels to generate bank instability is dependent on the composition and stratigraphy beneath the river bed. This work advocates and provides a basis for the development of river sand mining policies which should include ensuring the conservation of the river equilibrium and its natural environment, avoiding the aggradations at the downstream reach especially those with hydraulic structures such as bridges and jetties as well as ensuring that the rivers are protected from bank and bed erosion beyond its stable profile.

Recommendation

There is a need for the Ministries of Environment to be equipped with the necessary planning and management tools to deal with the problems that arise from river sand mining and this study is an effort in this direction.

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A LEAD PAPER (2018)

Proceedings of the Third International Conference of the Nigerian Association for Engineering Geology and the Environment (NAEGE), Port-Harcourt, Nigeria, 2018 (Vol. 2), 2019.

Integrated Engineering Geology and the Environment in Sustainable Infrastructural Development

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Abstract

Infrastructure does not only include the traditional types (public transport, energy supply, telephones, water and sanitation, etc) but also, the natural infrastructure such as forests, landscapes, watershed protection, etc. Sustainable infrastructural development means ensuring that the infrastructures build are compatible with social and environmental goals. In order to integrate Engineering Geology and the Environment in sustainable infrastructural development, there should be a paradigm shift from the traditional study of planet earth as isolated discrete components (atmosphere, hydrosphere, biosphere, cryosphere, and geosphere) to the concept of integrated whole earth system approach that embraces systems thinking perspective. Engineering geologists should not only focus on the geological factors controlling the structural stability/sustainability of civil engineering structures, but also on the present and future environmental, economic and social issues including decision-making, policy, and governance. A critical requirement is the establishment of a strong and functional communication between engineering geologists and all other stakeholders involved in sustainable infrastructural development.

Keywords*: Integrated Engineering, Geology, Environment, Sustainability, Infrastructure, Development.*

Introduction

Human society is currently facing several complex global challenges which include; exponential growth in population, urbanisation and industrialisation, resources depletion, food, water and energy security, infrastructural facilities, poverty, environmental sustainability, climate change impact, biodiversity loss, natural hazards, etc. These global problems are highly complicated, interconnected and threaten the future of humanity and the integrity of the carrying capacity of the earth's ecosystems. Traditional discipline-based knowledge, methodologies and approaches are no longer sufficiently effective in addressing these complex and interconnected global problems which cut across disciplines, sectors and regions. Therefore, an inclusive, holistic and integrated (or multidisciplinary) approach is required.

Infrastructure is a key pillar of modern society. Infrastructure development is a critical driver for socio-economic progress in developing economies like Nigeria because of its important role in productivity and sustainable economic growth. The current global assessment of the competitiveness of countries' economies by the World Economic Forum, ranked Nigeria at 125th position out of 137 countries assessed in 2017/2018 Global Competitiveness Index (WEF, 2018). National competitiveness is defined as the set of institutions, policies and factors (12 pillars of competitiveness) that determine the level of productivity of a country's economy. In the 2017/2018 Global Competitiveness Index, Rwanda was the highest ranking African country occupying 58th position followed by South Africa at 61st position. In the West African sub-region, Senegal was ranked highest at 106th position followed by Ghana at 111th position. At 125th position, Nigeria's performance on the global scale is very poor considering the country's huge oil and gas resources and a population of about 200 million.

According to the 2017/2018 Global Competitiveness Index, Nigeria's most problematic factor for doing business is underdeveloped infrastructures. The country's position is 132th out of the 137 countries assessed (6th to the last) and the quality of electricity supply in Nigeria is ranked 136th position (second to the last!). Nigeria's overall scores in global economic competitiveness and infrastructure have been falling every year since 2012. The above statistics underscores the importance of this topic to the development of our dear country Nigeria.

Sustainable Infrastructure Development and Economic Growth

Infrastructure has been generally defined as *"the basic socio-economic structures and facilities necessary for a country to function efficiently"*. A comprehensive definition of infrastructure includes both traditional types of infrastructure (public transport - roads, rails, ports, airports-, energy (electricity supply), telephones (fixed & cellular), water and sanitation) and importantly, also natural infrastructure such as forests, landscapes, wetlands and watershed protection (GCEC, 2016) (Fig. 1).

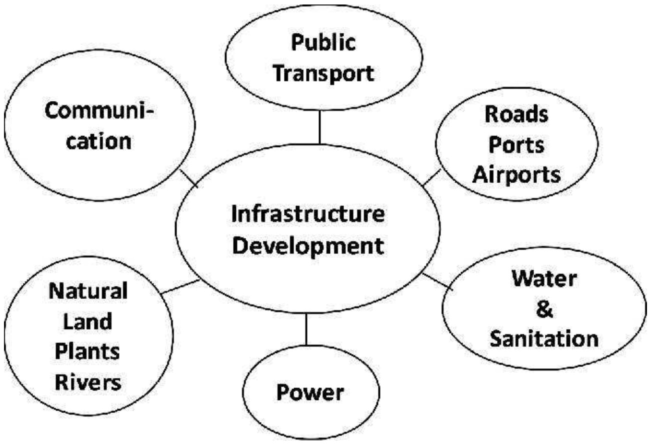

Fig. 1: Components of Infrastructure Development

Sustainable infrastructure development means ensuring that the infrastructure we build is compatible with social and environmental goals. For example, the infrastructure must not endanger human safety, health and well-being, neither should it cause pollution or irreversible changes in the geodynamic natural processes. It should support the conservation and sustainable use of natural resources, and contributes to reducing the impacts of climate change. Sustainable infrastructure refers to the designing, building and operating of these structural elements in ways that **do not** diminish the social, economic and ecological processes required to maintain human equity, diversity, and the functionality of natural systems.

The satisfactory provision and administration of infrastructure make economic development possible, create growth, and enhance competitiveness and productivity thus increasing participation in the global economy. Infrastructure influences growth by improving productivity, reducing production costs, helping to diversify the productive structure, and creating employment through demand for the goods and services used to provide it. As economies attain higher levels of development and their stock of infrastructure grows, the returns on infrastructure also grow.

A new vision and strategy for the infrastructure sector is infrastructure that is planned, built, and maintained in order to support the provision of adequate quality services that promote sustainable and inclusive growth. This new vision of infrastructure rests on the key pillars of environmental, social, and fiscal sustainability. It recognizes the need to expand multisector approaches that allow the synergies among infrastructure sectors to be exploited.

The new vision of infrastructural projects represents a win-win situation for all, as they lead to greater availability and quality of infrastructure services and encourage environmental conservation. For example, expanding the coverage of public transportation systems with buses operating with clean technologies improves access to schools and hospitals for the poor, and cuts commuting times to work by alleviating traffic congestion while reducing emissions. Providing access to improved sanitation leads to less polluted drinking water sources such as lakes and rivers as wastewater is removed by sanitation systems instead of being channelled directly into sources of drinking water. Access to quality electricity services allows businesses to reduce their use of fossil-fuel based generators and associated environmental pollution.

Bad infrastructure, on the other hand, literally kills people by structural failure, cause of deadly respiratory illnesses, exacerbating road accidents and spreading unclean drinking water, and other hazards. It also puts pressure on land and natural resources, creating unsustainable burdens for future generations such as unproductive soils and adverse climate change. Infrastructure development tends to be associated with adverse environmental impacts, which can be minimized in view of the considerable potential for synergies among infrastructure projects, economic growth, and environmental stewardship.

Environment

Although environment has been defined in several ways, it is generally defined *as 'total surroundings of man; land, water, atmosphere, climate, animal and plants (biota), social factors such as economics, social, cultural, political and historical as well of built structures such as buildings, roads, bridges, airports, etc".* It is composed of six major sub-systems, namely:

- Atmosphere,
- Hydrosphere,
- Biosphere,
- Lithosphere,
- Human (population, culture, economics, social, politics, techno),
- Constructed (building, roads, bridges, infrastructure, etc).

We live in a complex dynamic world where everything is connected to something else and all the components of the Earth's environment constantly interact with each other. The components of the Earth's environment are better described as interconnected sub-systems of the

Earth's system. A system has dynamic properties as a whole that exceed the sum of the parts. Systems-thinking focuses on the whole dynamic system and the interactions/ relationships among the parts rather than focusing on properties of the separate components.

Unprecedented global environmental and social awareness emerged in the 1960s following some major events in the US, especially:

- Rachel Carson's landmark environmental book, *Silent Spring, (*1962) stating detrimental effects of use of pesticides;
- Santa Barbara oil spill (Jan. 28, 1969), resulting in blowout and spillage of about 200,000 gallons of oil;
- Oil slick on the Cuyahoga River, Cleveland, Ohio, which caught fire in June 1969;
- Passage of the National Environmental Policy Act by US Congress in 1969.

What could be regarded as the beginning of the global political leadership interest and awareness on environment was the UN Conference on Human Environment in Stockholm in March 1972. It is interesting to note that by this time, no country had a ministry for the Environment. The report of the UN World Commission on Environment and Development (Our Common Future, 1987) formally introduced the concept of sustainable development which stipulates economic and social development without irreversible damage to the earth's natural environment and the depletion of nonrenewable resources. Any development project must satisfy economic, social and environmental objectives (Fig 2). The WCED report defines sustainable development as "Development that meets the needs of the present without compromising the ability of future generations to meet their own needs." It contains two key concepts:

- The concept of needs - in particular, the essential needs of the world's poor, to which overriding priority should be given
- The idea of limitations imposed by the state of technology and the social organization on the environment's ability to meet present and future needs (Oslen et al, 2006).

The concept of sustainable development has experienced huge success since its introduction, and it now pervades the agenda of most governments, corporations and educational/research programs worldwide. The term sustainability describes a state of possessing capacity to continue indefinitely and implies long-term health of the global ecosystem for both present and future generations. Sustainability generally includes a balance of environmental, social, and economic issues. Sustainable development is the process of development from our present unsatisfactory state toward the ideal state of sustainability. It is a process of enhancing the human socioeconomic and ecological well-being currently threatened by human practices that tend to pollute the environment. Sustainability is the destination of the process of sustainable development.

Fig. 2: Interrelations between the three Pillars of Sustainable Development

The Earth Summit that followed in 1992 in Rio de Janeiro, Brazil, adopted a detailed global plan of action (Agenda 21) and commitment of member nations to economic development and human growth without destroying the life-support systems of the earth. The two most important factors that negatively impact the Earth's Environment and natural production capacity are:

- Global Human population
- Western Industrialization and Technology

These two factors demand infrastructures which must be provided in consonance with the principle of sustainable development. In 2015, the United Nations member countries agreed on a set of 17 Sustainable Development Goals (SDGs) to steer global development efforts in decades to come (Table 1).

Attaining these goals requires provision of appropriate sustainable infrastructure.

The world population is projected to reach 9 or 10 billion by 2050. From about the 1950s, approximately 90 million new people are added to the world population yearly (Fig. 3). The greatest population growth occurs in urban centres as well as developing countries in Africa, Asia and Latin America. The unprecedented population growth has caused enormous demand on the productive capacities of Planet earth which is now unable to sustain the demand of the growing human population and infrastructure needs. This has in turn given rise to over-exploitation of virtually all earth resources.

Table 1: The 17 sustainable development goals (SDGs)

GOAL	ISSUES
1	No Poverty
2	Zero Hunger
3	Good Health and Well-being
4	Quality Education
5	Gender Equality
6	Clean Water and Sanitation
7	Affordable and Clean Energy
8	Decent Work and Economic Growth
9	Industry, Innovation and Infrastructure
10	Reduced Inequality
11	Sustainable Cities and Communities
12	Responsible Consumption and Production
13	Climate Action
14	Life Below Water
15	Life on Land
16	Peace and Justice Strong Institutions
17	Partnerships to achieve the Goal

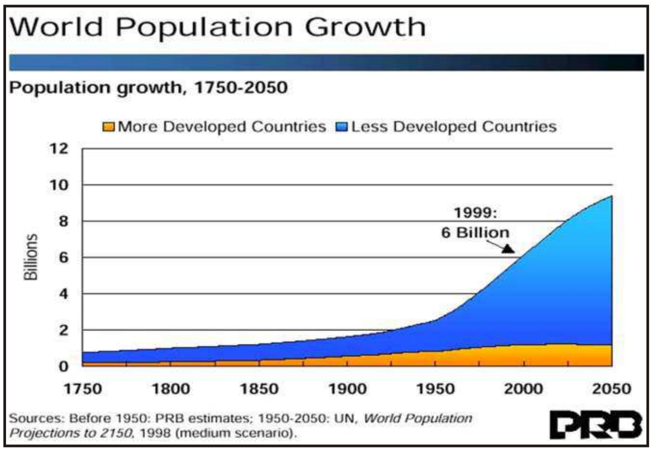

Fig. 3: World Population Growth

Since the early 1970s, humanity has been demanding more from the planet than it can renew (Fig. 4). By 2012, the biocapacity equivalent to 1.6 of planet earth was needed to provide the natural resources and services humanity consumed in that year (Global Footprint Network, 2016). Humans have substantially disrupted hydrological systems through rising consumptive use and impoundment of water (Vörörsmarty and Sahagian, 2000). As a result, streams, wetlands and lakes have dried (Vörösmarty et al., 2010; Davidson, 2014; etc); regional atmospheric vapour flows have shifted and river levels have changed due to increased reservoir storage (Reager et al., 2016). Changing the water cycle affects both the climate and the biosphere.

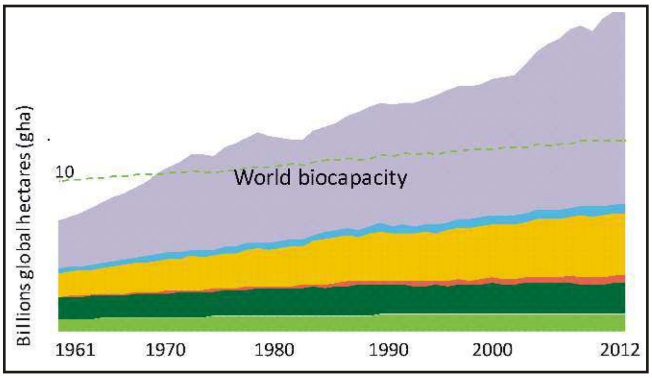

Fig. 4: Global Ecological Footprint by component vs Earth's biocapacity, 1961-2012. *Source:* (Global Footprint Network, 2016).

Engineering Geology

The earth systems have changed more rapidly in the past 60 years than any other times in human history. Geoscientists now have critical additional role to play in this emerging dispensation. In addition to their traditional role in the earth-resource sector, is the new role of applying geological knowledge and skills to broad-ranging engineering, environmental and socio-economic issues which require the integrated earth-science knowledge. This made it imperative for a paradigm shift from the traditional study of planet earth and its' processes as isolated discrete components to the concept of integrated whole system approach (Earth System Science). The concept emphasizes interactions and linkages rather than components, and trends rather than specific contents (Eyles, 1994; Alcamo, 2015). The recent emergence of Engineering Geology and Environmental Geology is in line with this paradigm shift.

Engineering Geology is applied geology that deals with the application of geologic principles and concepts to engineering construction projects such as dams, reservoirs, tunnels, other subsurface structures, roads, highways, airport runways, power plants, waste disposal facilities, and structures to mitigate pollution, flooding, earthquake, coastal erosion, etc. However, as environment became a prominent global issue, engineering geology was redefined to include environmental and hydrological work within the scope of the application to engineering.

The status of the International Association of Engineering Geology and the Environment defined Engineering Geology as:

> "Engineering Geology is the science devoted to the investigation, study and solution of engineering and environmental problems which may arise as the result of the interaction geology and the works or activities of man, as well as the prediction and development of measures for the prevention or remediation of geological hazards" (IAEGE Status, 1992).

The Association of Engineering Geologists gave a similar definition which states; (AEG, 2001)

> "Engineering Geology discipline is the application of geologic data and principles to the study of both, (a) naturally occurring rocks, soils materials, surface and subsurface fluids, and (b) the interaction of introduced materials and processes within the geologic environ. The purpose is to assure that geologic factors affecting the planning, design, construction operation and maintenance of engineering structures and development of groundwater resources are recognized, adequately interpreted, and presented for in engineering practice"

Engineering Geology and Environmental Geology are sub-disciplines of geology that relate directly to human activities, especially their impacts on the natural earth systems. The increasing prominence of environmental issues and Environmental Geology are a sub-discipline prompted several journals to change their names. For example, the *Bulletin of the Association of Engineering Geologists* (first published in 1964) changed its name to the *Bulletin of Engineering and Environmental Geosciences* in 1995, while the *Bulletin of the International Association of Engineering Geology* became known as the *Bulletin of the International Association of Engineering Geology and the Environment* in 1990.

Integrated Engineering Geology and the Environment in Sustainable Infrastructural Development

There is a delicate reciprocal balance between human society and environment. Human activities tend to degrade the environment and alter the land-water ecosystem while environmental hazards result in economic losses to human society. The proper use of geologic principles and engineering practices can minimize both environmental degradation and economic losses. Changes in the design and construction of structures are needed to reduce negative environmental impacts of construction activities and civil engineering structures. Sustainable construction/structure incorporates elements of environmental performance, economic efficiency and social responsibility. Sustainable construction results from the application of sustainable development principles to the global cycle of construction from raw material acquisition, through planning, design, construction and operation to demolition and waste management.

Infrastructures are generally built on or within the dynamic earth system environment which is made up of intimately interrelated sub-systems (atmosphere, water, soil/rock, biota, man-made structures, etc). Therefore, the systems approach should be adopted in infrastructural development. The systems approach highlights the need to encompass all of the sub-systems and sectors in an integrated planning and management process. This approach is not new. Chapter 10 of Agenda 21 of the Rio Summit (1992); "Integrated Approach to the Management of Land Resources" clearly recommends the integrated approach. It states;

> *"Land resources are used for a variety of purposes which interact and may compete with one another; therefore, it is desirable to plan and manage all uses in an integrated manner. Integration should take place at two levels, considering, on the one hand, all environmental, social and economic factors (including, for example, impacts of the various economic and social sectors on the environment and natural resources) and, on the other, all environmental and resource components together (i.e., air, water, biota, land, geological and natural resources). Integrated consideration facilitates appropriate choices and trade-offs, thus*

> *maximizing sustainable productivity and use" (Section 10.3)*

However, the operational approach by most government in the provision of infrastructure and resource management is still strongly based on sectors. For example, there are different ministries for agriculture, water resources, power, works, environment, pollution, etc, that function independently. The current approach in most advanced economies is to group together related sectors into a single ministry. For example, in Switzerland and the Netherlands, there is a single Federal Department of Environment, Transport, Energy and Communications.

The fundamental tool for integrated approach in infrastructure development is integrated physical planning which is a framework outlining the optimal organisation and utilisation of a territory for all sectors simultaneously. Virtually everybody broadly proclaims the need for discipline and sectoral integration but only few really wish to accept it because of lack of political will. Sectoral approaches are still very common due to resistance by sectors to cross-sectoral concept/planning, decision-making and project implementation for fear of being overshadowed by other sectors.

Environmental governance and resource management in developing and emerging economies suffer from segmented approaches to planning and policy implementation. Many developing countries have invested heavily in sector-based infrastructure construction, including large dams, water treatment plants, large irrigation schemes, etc at the expense of investments in maintenance/operation and without explicit focus on service parameters such as affordability, reliability and quality.

Involving Engineering Geologists from Project Conceptualization Stage

According to the AEG definition of Engineering Geology, the "*purpose is to assure that geologic factors affecting the planning, design, construction operation and maintenance of engineering structures and development of groundwater resources are recognized, adequately interpreted, and presented for in engineering practice"* For this purpose to be achieved, the engineering geologist must be involved from the conceptualization stage of any infrastructural project, which is usually not the case! Traditionally, most government administrators, technocrats, planers, engineers and project owners/initiators do not consider the expertise/contribution of the engineering geologist important at the planning stage of project conceptualisation. Generally, the engineering geologist is only consulted whenever apparently unsolvable problems that are perceived to be related to geology are encountered either during project construction, operation, or maintenance. The reality is that not much emphasis is placed on engineering and environmental geologic knowledge and experience in the selection of consultants and contractors for most infrastructural projects. Unfortunately, despite the acknowledged importance of engineering and environmental geology input in sustainability of civil engineering and environmental projects, engineering and environmental geology profession is still perceived as *"service agent"* to be used on *ad-hoc* basis rather than a fundamental component of major infrastructure projects. To reverse this attitude, there must be deliberate efforts to ensure close and functional communication between engineering/ environmental geologists and all other stakeholders involved in achieving sustainable infrastructural development and sustainability. Engineers should change their *"know it all attitude"* when it comes to planning, design construction and maintenance of major engineering infrastructure projects.

The earth and its environment were formed by geological processes that acted in the past and are still active today, according to natural laws and established geologic knowledge and principles. These critical geological processes and controls which include the geological/ depositional history, stratigraphy, depositional environment, current geodynamic hydrological processes and the properties of the sediments which are susceptible to alteration either directly or indirectly by human activities. If these geologic controls are not known, it is impossible to formulate appropriate measures that can effectively address the problems threatening the sustainability and stability of infrastructures and the earth's environment. The recent paradigm shift in favour of a more integrated, holistic, adaptive, and ecologically-based philosophy of system approach, emphasizes the physical components of the Earth System (especially, geology, hydrology, and geomorphology) which have hitherto received little attention in infrastructure development. In addition to the system and integrated approach, there is growing global consensus that collaboration and cooperation which cut across disciplines, sectors and regions in conjunction with good policy and governance are required to effectively address the challenges facing sustainability of infrastructures and the earth's environment.

Examples of Case Studies

Some examples of infrastructural projects whose functionality, stability and sustainability are being threatened due to inadequate input of geologic knowledge and principles are summarized below.

Nkpolu Road Junction and Secondary School in Port Harcourt

The area is part of an extensive low-lying stretch of natural freshwater swamp and stream head waters that extend from Rumuokoro through Nkpolu to University of Port Harcourt. The swamp is crossed by the East-West Highway and several roads which partially subdivided it into several unconnected blocks thereby restricting the free flow of flood water. Several sections of the swamp have also been reclaimed for housing development which greatly reduced the swamp area available for flood waters in the built-up areas. The recent worsening flooding at the East-West Highway-Nkpolu Road Junction is the result of reclamation of old river channel and erection of several buildings around the junction.The flooding problem in this area is due to the fact that the natural local drainage system and hydrology were not considered during the design and construction of the E-W Highway, other roads/streets and swamp reclamation for building construction (Figs. 5(a) - (c)).

Fig. 5(a): Flooded Nkpolu Community Secondary School (2006)

Fig. 5(b): Flooded Nkpolu Road (August 10, 2018)

Fig. 5(c): Flooded UNIPORT bound lane of EAST-WEST HIGHWAY at Nkpolu Road Junction

Gully Erosion sites

Gully erosion has cut several roads into two parts in some locations in Southeastern Nigeria as a result of the collapse of poorly constructed and improperly terminated culverts. The culverts collapsed due to undermining after days of heavy rainfall. In addition, the culverts were constructed without incorporating the soil's erodibility characteristics and the magnitude of run-off resulting from high intensity rainfall (Fig. 6(a)).

Fig. 6(a): Umunnekwu erosion site along Uturu - Isuikwato Road, 1996

Road was cut into two by collapse of a poorly constructed and improperly terminated culvert caused by undermining after two days of heavy rain (Fig. 6(b)).

Fig. 6(b): Erosion site between Abia State University and Nkwonta along Uturu - Isuikwato Road, 2018

Road was cut into two by collapse of a poorly constructed and improperly terminated culvert caused by undermining after days of heavy rain (Fig. 6(c))

Slope failure at Construction Site at the Calabar Port Complex

The construction site consists of unique steep-sided slopes, gullies/ravines and unconsolidated soil materials. Heavy seasonal rainfalls promote active erosion primarily controlled by the availability of groundwater and surface water from rains and springs.

Fig. 6(c): Nkpor-Nnobi Road in Anambra State July 2018

Slope failure and instability occurred in the area following construction activities (excavation, grading, filling, etc) and heavy rainfall events. The construction activities removed the protective natural vegetation cover which resulted in increased surface water infiltration, percolation and soil erosion which in turn elevated the soil water content and ultimate reduction of the soil shear strength. The construction activities impacted the hydrology-hydrogeological system and geotechnical properties of the soils hence the slope failure.

Only limited preliminary geotechnical site investigation (three boreholes) was undertaken prior to site development. A conceptual site model characterizing the soil stratigraphy and geotechnical properties as well as the hydrology-hydrogeology conditions should have been carried out before the design and construction activities.

Conclusion

The unprecedented global population growth has caused enormous demand not only on the productive capacities of Planet earth but also on infrastructure needs. Attaining the 17 Sustainable Development Goals (SDGs) set by the United Nations for post-2015 global development requires designing, building and operating infrastructures that do not diminish the social, economic and ecological processes required to maintain human equity, diversity, and sustainable functionality of natural systems. Global problems are complex and interconnected, cutting across disciplines, sectors and regions. They require inclusive, holistic, interdisciplinary and integrated approach to provide effective solutions

Currently, there is apparent insufficient application and

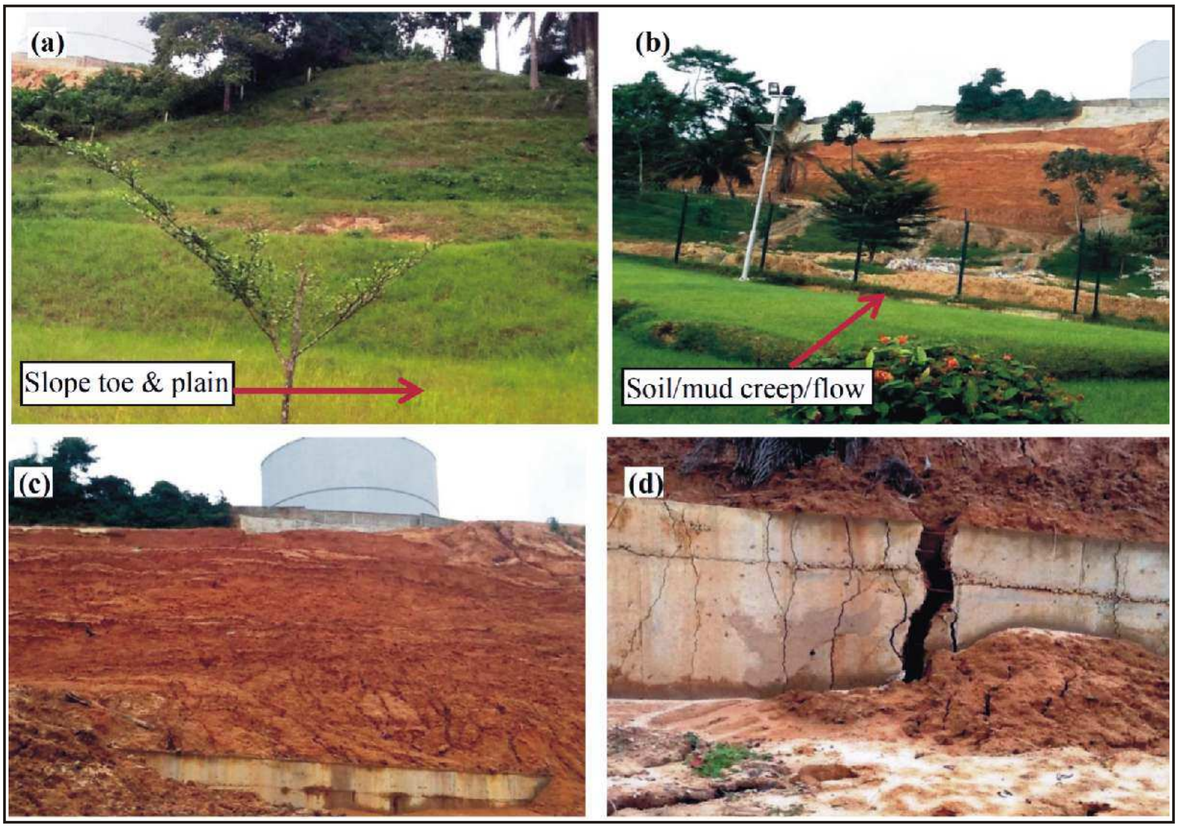

Fig. 7: Photographs showing **(a)** Stable natural and semi landscape river bank slope, **(b)** Failed section of the natural slope caused by excavation, grading and filling operations during construction of the access road, **(c)** section of the concrete retaining wall covered by soil slumping from upper slope section, **(d)** severely cracked and failed retaining wall.

Fig. 7(e): Photographs showing soil plastic/liquid flow and creep causing severe damage to concrete drains, concrete roadside pedestrian walkway and fence

integration of geologic knowledge/ principles and environmental sustainability in infrastructural development process. In order to enhance greater input by engineering and environmental geologists to the process, there should be;

- Close and functional communication between engineering-environmental geologists and all relevant stakeholders, especially experts in engineering, social sciences, biological sciences, decision/policy makers and administrators.
- Strong advocacy for critical policy changes in the conditions for engineering and environment-related contracts to include mandatory input from engineering and environmental geologists.

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The Use of Tracers in calibrating Biostimulation/Bioaugmentation Performance in the Remediation of Oil Contaminated Sites

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Abstract

The successful remediation of an oil contaminated site is dependent on the proper understanding of the geology and hydrogeology of that site; which should be established through detailed site characterisation, prior to the commencement of remediation works. Tracer tests integrated into the site characterisation process elucidates mass transport dynamics and enhances a clearer understanding of the fate and transport of both contaminants and bioremediation cocktails in the subsurface. Bioaugmentation and biostimulatation are becoming increasingly popular in the remediation of contaminated sites around the Niger Delta. Therefore, precise information on mass transfer mechanisms in the vadose and phreatic zones, enhances the efficacy of bio-remediation strategies, in terms of cost, volume of treatment required, rate of application, spatial distribution and depth of inoculation points. Hence, the use of tracers in calibrating bioaugmentation and biostimulation performance in the remediation of oil contaminated sites will aid remediation companies to stay on target, on cost and on time; thus improving the efficiency of the bioremediation strategy.

Keywords*: Tracer, Biostimulation, Bioaugmentation, Inoculant, Injectate, Remediation, Phreatic, Vadose.*

Introduction

Oil spills are increasingly becoming the most common hazard that undermines the environmental quality of the Niger Delta. These spills emanate from oil infrastructure which abound in the region and are caused by either equipment failure or third party interdiction of the oil infrastructure leading to the contamination of soils and groundwater. Several soil remediation projects have been attempted in the past with modest reports of the successful outcomes meeting up target and intervention values as stipulated by the Environmental Guidelines and Standards for the Petroleum Industry in Nigeria (EGASPIN). The information on any successful groundwater remediation project in the region is also sparse to non-existent.

The increasing exploitation of oil and gas resources in most areas of the Niger Delta region, exposes the inhabitants to a myriad of ecological problems arising from the release of petroleum hydrocarbon pollutants into both the aquatic and terrestrial environment. There is an extensive network of over 900 producing oil wells, 100 flow stations and gas plants, over 1,500 kilometres of trunk lines and some 45,000 kilometres of oil and gas flow lines. The Niger Delta recording an average of 221 oil spills per year is therefore prone to oil pollution. This scenario thus imposes the necessity of a comprehensive, well designed remedial action plan for the restoration of oil contaminated sites that dot the entire landscape of the Niger Delta.

Following the release of the UNEP Report on Ogoniland in 2011, there is a renewed wave of remediation projects in the region that attempt to improve on previous remedial strategies. This have necessitated an increase in the use of many bio-remediation techniques in resolving the contaminant profile within contaminated sites in the Niger Delta. Many bioremediation cocktails and their application methods are being developed and tried. A fair knowledge of subsurface mass transfer process is required for the successful inoculation and degradation of the contaminants within soils and groundwater.

Bio-stimulation involves the modification of the environment to stimulate existing bacteria capable of bioremediation. This can be done by addition of various forms of rate limiting nutrients and electron acceptors, such as phosphorus, nitrogen, oxygen, or carbon (e.g. in the form of molasses). Alternatively, remediation of halogenated contaminants in anaerobic environments may be stimulated by adding electron donors (organic substrates), thus allowing indigenous microorganisms to use the halogenated contaminants as electron acceptors. Additives are usually added to the subsurface through injection wells and this requires a knowledge on the advection and dispersion potentials of the soils and aquifers. Bio-augmentation on the other hand, is the addition of archaea or bacterial cultures required to speed up the rate of degradation of petroleum hydrocarbon contaminants.

Tracer testing, however, play an essential role in the experimental investigation of chemical, physical and biological systems. In general, a tracer is a substance or entity that is experimentally measured in a system of interest for the purpose of deducing process information from the tracer signal. Tracers are used when the system of interest is inaccessible by direct measurements. Tracers reflect the outcome of mass transport processes and are useful for characterizing mass transport and mass transfer occurring in the subsurface (Giadom, et al, 2015, Field, 1999, Weight and Sonderagger, 2000).

Soil properties and the unit permeability of many contaminated sites in the Niger delta are quite variable. Therefore, precise information on mass transfer mechanisms within aquifers, will enhance the efficacy of bio-remediation strategies, in terms of cost, volume of treatment required, rate, spatial distribution and depth of inoculation points and the overall performance of the bioremediation strategy. Field research has shown that these porous media properties may change continuously from location to location ("spatial heterogeneity"). Spatial heterogeneity of geologic media may significantly influence flow and transport processes in the vadose and phreatic zones, which leads to uncertainty in predicting the behaviour of fluids, contaminants and inoculants in the subsurface. Estimates of conductivity, hydraulic head, advective velocity and mean concentration enhances the description of the displacement of the inoculant and contaminant plumes during bio-augmentation and bio-stimulation.

Saturated hydraulic conductivity data are valuable and will significantly reduce uncertainties in predicting bioremediation inoculant plume distribution. Bioremediation efforts must cope with these "uncertain" information that has been extrapolated from measurement locations to the surrounding areas, because it is impossible to measure all soil properties at all locations (Harter, 1994). This paper, therefore, posits that the use of tracers, in calibrating the performance of bio-stimulation and bio-augmentation inoculants in soil and groundwater remediation projects will greatly improve the accurate estimation of the quantity and rate of inoculating the contaminated sites.

Tracer Testing, Contaminant Mass Balance and Bio-stimulation

Tracers reflect the outcome of mass transport process, and are useful for characterizing mass transport process processes. Tracer testing involves the design of injection wells and the inoculation of injectates at specific rates and volumes into an aquifer followed by the synoptic evaluation of the tracer advection and dispersion within the observation field (Giadom, *et al*., 2015). The breakthrough curves generated by the snapshots of tracer migration enables the computation of mass balance equations unique to any given aquifer and by extension the contaminated site. Tracer tests are often used to measure or estimate various hydrogeological parameters most commonly the direction and velocity of flow or to identify sources, velocity and direction of movement of contaminants. It reveals the character and direction of migration of the contaminant plume; and provides a continuous characterization of the vertical variation in the hydraulic conductivity/porosity ratio across the full thickness of the aquifer. Hence, tracer experiments may permit the detailed characterization of the hydraulic properties of aquifers (Palmer and Nadon, 1986).

Bio-remediation of contaminated subsurface soil and groundwater media therefore, is a process whose success and economics will depend on a thorough understanding of the hydrogeological dynamics of the contaminated site. It often requires sophisticated in-situ physical, chemical and biological methods to breakdown and degrade organic complexes. Bio-augmentation and bio-stimulation requires the injection of inoculants into the subsurface at predetermined rates and volumes. Understanding the migration patterns of injectates as described from tracer tests and the development of conceptual site models calibrates the mode of application of inoculants particularly in groundwater bio-restoration (Giadom, *et al*., 2015).

The success of the deployment of bioremediation inoculants into the soil and groundwater further depends on a clear understanding of the contaminant-inoculant mass balance dynamics in the subsurface. Several processes determine the interaction of the inoculant and petroleum hydrocarbon contaminants within the soil matrix. These processes can be categorised as:

1. **Physical Process**: Advection, dispersion, evaporation, filtration and degassing.
2. **Geochemical Processes**: Acid-base reactions, adsorption-desorption, ion exchange, oxidation-reduction, precipitation-dissolution, retardation and complexation
3. **Biochemical Processes**: Transpiration, bacterial respiration, decay and cell synthesis.

The inoculant mass occurring in water as ions, molecules or solid particles not only undergo transport but also reactions. These reactions redistribute mass among ions or between solid and liquid phases; this is otherwise known as partitioning. These mass transport processes will define the fate of the inoculant carried by the moving water and ultimately determine how it partitions to contaminants in the subsurface, ultimately leading to the biochemical reactions that will break down and degrade the contaminants to harmless end products.

The transport of bioremediation inoculants through the complex pathways (the micropores within the soil matrix) is dependent on advection, predominantly in the direction of groundwater flow; while dispersion will spread the inoculant in the longitudinal and transverse directions, mostly in the former direction. The most important effect of dispersion is to spread inoculant mass beyond the region than it would normally occupy due to advection alone. Retardation will cause reduction in apparent inoculant velocity, compared to the groundwater velocity, which is caused by linear equilibrium sorption on aquifer materials. Diffusion will result in the spreading of inoculants from areas of high concentration to areas of low concentration, caused by random molecular motion. Note that tracer testing prior to inoculant injection helps to define spatial variabilities inherent in aquifer materials within the contaminated site. It thus, clarifies the uncertainties surrounding the fate and transport of the bioremediation cocktail in the subsurface and enables that evaluation of the progress of bio-augmentation and bio-stimulation.

Tracer Advection and Aquifer Characterisation

In aquifers, groundwater flows through complicated pathways. The use of tracer tests as a predictive tool for evaluating the fate and transport of contaminants is a simplified way to describe a far more complex natural situation (Giadom, et al, 2015; Field, 1999; Domenico and Schwartz, 1990). Tracers are injected into the strata of interest within an aquifer through an injection well. Tracer migration is monitored from observation wells positioned in the direction of groundwater flow at various distances from the injection well.

The mean travel time of the tracer is the difference in elapsed time of the centroids of the tracer breakthrough curves defined upstream and downstream. They may be described in terms of elapsed time after tracer injection according to Kilpatrick and Wilson (1989) as:

$$\mathbf{t_c = T_{c(n+1)} - T_{cn}} \quad \text{...(1)}$$

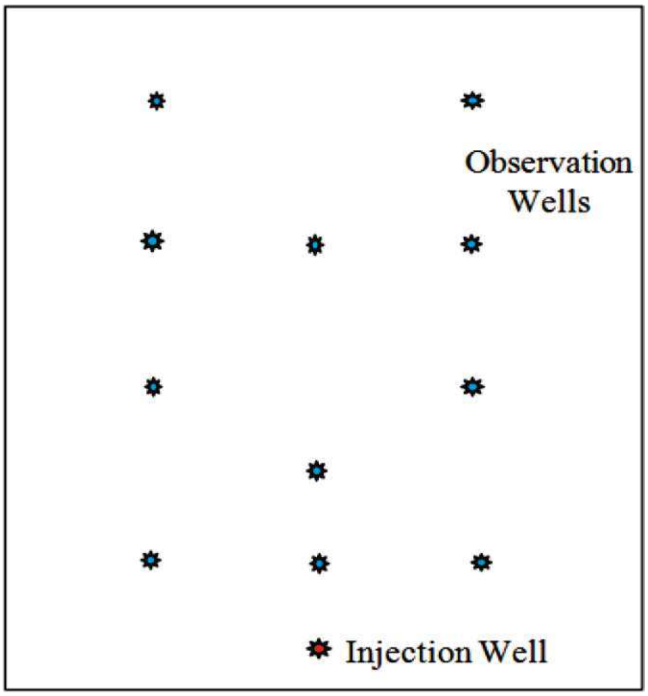

Fig. 1: The orientation injection and observation wells in a natural gradient tracer test field

where: T_{cn} is the elapsed time to the centroid of the breakthrough curve at point n.

The average velocity of the tracer is deduced from the arrival times of the breakthrough moments of the migrating tracer plume from two observation piezometers (see figure 2).

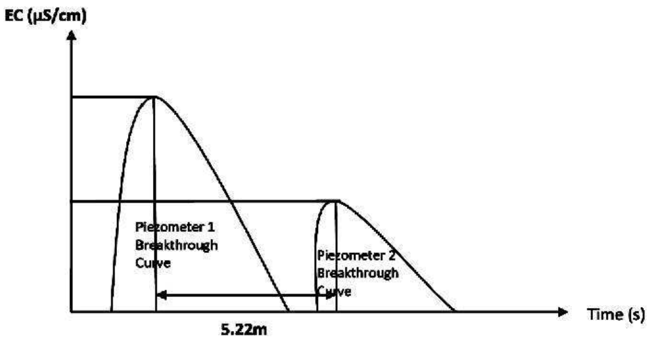

Fig. 2: Centroids of Breakthrough moments from 2 piezometers, spaced 5.22m apart (Giadom, et al, 2015)

The tracer test described in figure 2.2 above was conducted in a contaminated site in Ogale, Eleme, Rivers State; where advection was deduced to be $\boldsymbol{1.403 \times 10^{-3} ms^{-1}}$; which agrees with similar results ($1.26 \times 10^{-3} ms^{-1}$) obtained by Newton and Brian (1995) in Cape Cod Massachusetts. This gave an understanding of the migration dynamics of petroleum hydrocarbon contaminants at the contaminated site in Ogale.

Field observations of tracer velocities reveal them to be quite variable; due to heterogeneities inherent in aquifer materials, even on the micro scale of less than 10 meters. Thus the calculated fluxes, just like the arrival times were variable as well; due to the variation in the magnitude and direction of the migrating tracer from point to point within the test field. The void space may be visualized as an assembly of interconnected tubes with varying diameters; therefore, the velocity will also vary according to the shape and size of the pores. It was also observed that the streamlines may deviate from the mean direction of flow; and as a consequence, any initial tracer cloud will gradually change, and so will the fluid volume occupied by it. As the flow continues, the tracer particles, which originates at the injection point will occupy an ever increasing volume of the flow domain (Bear and Cheng, 2009). Furthermore, as the tracer is transported through the different elements of the conductivity field, asymptotic dispersivity causes the tracer plume to be separated close to the source (Sudicky, et al, 1983; Gelhar, et al, 1979, Domenico and Schwartz, 1990).

Determination of Bioremediation Inoculant Flux Migration in a Contaminated Site.

The driving force behind the migration of any biostimulant or bioaugmentation cocktail is the actual movement of water through the medium. The physical processes responsible for this transport is advection and hydrodynamic dispersion. Other processes operating in this system may cause attenuation but not transport per se. If advection were the only transport mechanism the inoculant would move through the system as a single plug at a rate equal to the average linear groundwater velocity maintaining their initial concentration distribution over the entire flow path. Hydrodynamic dipersion, however, acts a mixing process and will cause the inoculant flux to spread outward from their source and so decrease in maximum concentration. Dispersion results from velocity variation within pores, differences in pore sizes, the tortuosity of the path and from molecular diffusion (Freeze and Cherry, 1979; Domenico and Schwartz, 1990).

Given that the mean velocity of the migrating tracer plume in any given contaminated site can be determined, then it is possible to calculate the bioremediation inoculant flux within any Representative Elementary Volume (REV) in the contaminated site

$$\mathbf{J_i = v_x C_i n} \quad \text{...(2)}$$

Where: Ji is the bioremediation inoculant flux passing through the ith REV, within the contaminated site, Vx the tracer velocity, C_i the concentration of the inoculant and n, the average porosity of the contaminated site.

However it must be stated that as the inoculant plume migrates it tends to expand. This process of spreading observed transverse to the flow direction is attributable to molecular diffusion caused by the random motion of molecules (Brownian motion). This produces an additional flux that is caused by the velocity of the fluid phase causing migration of inoculant from regions of higher concentration to those of lower ones creating a concentration gradient; which tends to equalize the concentration across pores. Hence, hydrodynamic dispersion denotes spreading that result from both mechanical dispersion and molecular diffusion. In field practice, often, the first estimate of the longitudinal dispersivity, prior to actual calibration is taken 1/10 of the size of the domain of interest (Bear and Cheng, 2009). What this means is that if the domain of interest is about 10m for the longitudinal advective front, then horizontal transverse dispersivity will be estimated to be 1m.

Sorption is another important component that will lead to the attenuation of the bioremediation coctail. This is essentially a process where the inoculant is partitioned between the solid and liquid phases within the aquifer by a variety of physical and chemical mechanisms. This partitioning removes a portion of the inoculant from the bioremediation cocktail, thus decreasing the concentration and total amount of the inoculant available for transport further along the flow path. Partitioning will also bind the inoculant to the contaminants and aid the bio-energetic process; ultimately causing a breakdown of the complex organic molecules to simpler non toxic end products. The foregoing is the key to estimating the performance of bioaugmentation and biostimulation cocktails in soils and groundwater.

Conclusion

Soils and aquifer systems in contaminated lands may be visualized as a huge physical-chemical-biological reactor in which many processes occur simultaneously among species present in groundwater and on the solid matrix. Understanding the migration patterns of the inoculant as described from a tracer test, and the development of a conceptual site model will be particularly useful in soil and groundwater in situ biorestoration, surfactant remediation or bioremediation which requires the injection of the bioremediation cocktails.

The interplay of tortuosity of the flow streamlines, dispersivities, partitioning and other processes make the precise prediction of the behaviour of bioaugmentation and biostimulation inoculant to be far from simple. However, tracer tests will enhance the understanding of the flow dynamics and hence the transport of inoculants within the remediation project site. It will aid the location of various inoculation points, depths, volumes and rate of injection of the bioremediation cocktail.

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Pre-Construction Site Investigations for Sustainable Structures

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Introduction

Preconstruction investigations for structures (buildings, dams, roads, bridges and tunnels) form an important aspect of construction of structures. Failure of many structures in Nigeria have been due to lack of comprehensive preconstruction subsoil investigation. Karl Terzaghi (1883-1963) was a Structural Engineer who specialized in the design of dams. Terzaghi who has earlier established soil mechanics, established engineering Geology as a discipline in 1938 when he discovered that most of the dams he designed with members of his team did not last long before they failed. He thus doubles as both the father of soil mechanics and Engineering Geology. Today, in most developed countries of the world, Geoscientists and Geotechnical Engineers work hand in hand to execute preconstruction (subsoil) investigations for structures.

Preconstruction investigation for any structure is a study executed both at the site and in the laboratory for the generation of subsurface model of a site with a view to understanding the nature (geological and geotechnical properties) of foundation soils and/or rocks.

Significance of the Study

1. Data generated from preconstruction investigation can help in knowing the thickness and nature of earth materials below the foundation of a proposed structure.
2. It furnishes information on groundwater condition.
3. It enables the generation of subsurface model.
4. Geological properties such as mineralogy, texture and geochemistry of underlying soils and rocks can help in foundation design for structures.
5. Allowable bearing pressure determined in-situ and/or in the laboratory can be useful in foundation design for structures.
6. Soil compressibility parameters can help in knowing both the amount of settlement and possibility of differential settlement of structures.

Geological and Geotechnical Field Investigations

- An Engineering Geologist must be able to combine his knowledge of Geosciences with soil mechanics and/or rock mechanic for comprehensive preconstruction investigation for any structure.
  Five (5) main stages are usually but not always involved:

Desk Study: this involves comprehensive literature review of the geology of the study area and available geotechnical report. This is also the planning stage.

Reconnaissance Survey: This is the preliminary visit to the proposed site. Observations and studies may call for the modifications of the proposed investigation scheme.

Field Work: This entails geological, geophysical and geotechnical studies of the proposed site.

(a) Geological mapping furnishes data that can give an idea of the geotechnical properties of the rocks and soils at the proposed site.
(b) Surface geophysical surveys are means of generating subsurface model comprehensively, rapidly and cheaply.
(c) Geotechnical Investigation. Results of geophysical surveys will help in isolating locations where in-situ geotechnical investigation should be executed.

- They will also help in deciding where to sink confirmatory geotechnical boreholes.
- The commonest and most reliable in-situ geotechnical test is the penetrometer test.
- It involves forcing a load through a standard cone into the subsoil and reading of penetration resistance every 0.25m

Allowable bearing Pressure in Kpa = 2.7 x Cone Penetration Resistance in Kg/cm^2. The selected penetration resistance is usually the lowest value at a given depth below the corners of a proposed structure (Table 1).

Table 1: Hypothetical Penetrometer resistance at proposed corners and selected penetration resistance from depth to depth.

Depth (m)	Cone Penetration Resistance (kg/cm^2)				Selected Penetration Resistance (kg/cm^2)
	P1	P2	P3	P4	
0.25	25	30	27	15	15
0.50	30	32	25	35	25
0.75	40	33	52	60	33
1.00	60	72	55	80	55
1.25	30	28	60	39	28
1.50	70	82	65	50	50
1.75	80	92	95	100	80
2.00	100	102	75	90	75
2.25	120	130	110	140	110
2.50	140	160	125	150	125
2.75	135	180	140	150	135
3.00	185	160	150	200	150
3.25	210	180	190	220	180
3.50	195	200	230	175	175
3.75	240	210	200	245	200
4.00	260	230	220	250	220

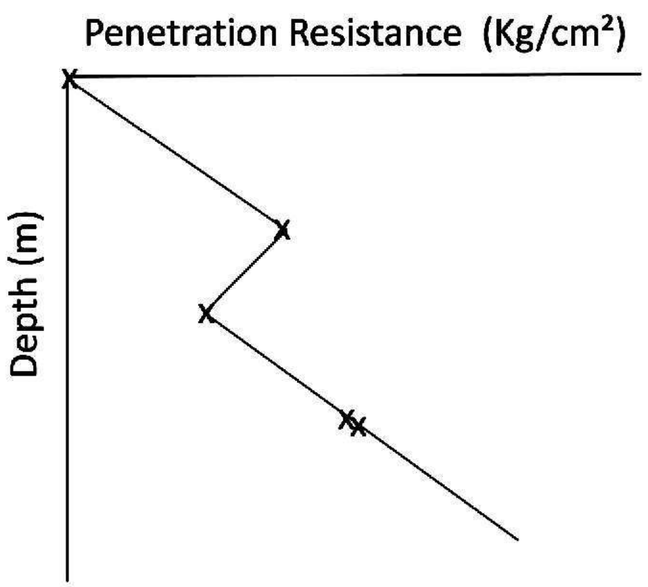


Fig. 1: Plot of depth against penetration resistance

- In sites susceptible to flooding during some period of the year, each of the figures in the last column has to be reduced to 50%.
- Penetrometer machines are manufactured in different capacities. The higher the capacity, the higher will be the depth of investigation and the generation of more reliable data. Soil samples can be collected at regular intervals during the test.
- The machine must be properly anchored to avoid its being pulled out prior to the exhaustion of the maximum capacity of the machine.
- Laterite concretion or large boulders may limit the depth of penetration.

Borings

- ➢ Drilling of Geotechnical Boreholes help in sampling of soils (disturbed and undisturbed).
  - It also helps in determining the penetration resistance (in terms of the number of blows per 300mm of penetration of the SPT spoon or Sampler).
  - Allowable Bearing Pressure can be estimated from SPT blows/300mm.
  - Depth to water table is an important consideration during drilling.
- ➢ Investigation for giant multi-storey residential or factory building and dams may call for rock boring for collection of rock cores.
- ➢ Strengths of rock cores have been found to correlate with Rock Quality Designation (RQD).
- ➢ The higher the value, the stronger being the rock. Low RQD value may indicate the presence of fractures on which a dam must not be founded.

Field Investigation for Roads and Highways

This is often very comprehensive. It includes the following:

i. Geological mapping: across the entire route.

ii. Surface Geophysical surveys: to generate subsurface model from point to point with special emphasis on groundwater condition and determination of fractures in underlying rocks.

iii. Geological, geophysical and geotechnical surveys at proposed bridge sites.

iv. Establishment of trial (test) pits at predetermined locations: Both disturbed and undisturbed samples should be collected. The maximum depth of each pit should coincide with the proposed placement (subgrade) level.

- ➢ Sampling must be comprehensive due to wide variability in properties of lateritic soils even within restricted areas.

v. Investigation at proposed road cuts: Soil and/or rock samples should be collected at hilly locations.

Geological and geotechnical studies of the sampled earth materials will help in slope stability analyses.

vi. **Proposed borrow pits:** Trial pits are often dug for collection of both disturbed and undisturbed samples. Construction materials (sub base soils) are often collected where soils possess desirable properties based on standard specifications

Laboratory Analyses of Rock and Soil Samples

(i) **Rock Samples:** are often subjected to petrographic and geotechnical tests using the American standard for Testing and Materials (ASTM) guidelines.
- Common geotechnical tests are used to determine Rock Quality Designation (RQD), Specific Gravity, Water Absorption Capacity, Aggregate Crushing Value (ACV), Aggregate Impact Value (AIV), Tensile, Shear and Compressive Strength.
- They are important parameters for selection of rocks either for construction or as foundation materials.

(ii) **Soil Samples:** Disturbed samples are often subjected to classification tests such as grain (particle) size distribution and consistency limits tests.
- Lateritic soils require special treatment that would ensure reliable determination of the percentage of each size fraction.
- Such treatment involves soaking of air-dried samples in weak calgon (sodium hexametaphosphate) solution for 24 hours accompanied with regular squeezing with hand. This is to ensure disaggregation of false particles caused by sesquioxide coating.
- This should be followed by wet-sieving which is meant to separate the soil sample into two portions (finer than 63 micrometre and coarser than 63 micro-metre).
- The fine fraction should be subjected to hydrometer analysis with a view to knowing the amount of clay-sized and silt-sized fractions.
- The coarse fraction should be oven dried prior to mechanical sieving.
- The time of sieving should be standardized to avoid excessive breakdown of particles, a phenomenon described by Malomo (1977) as mechanical instability characteristic.

(iii) **Disturbed** soil samples for highway sub-base and sub-grade should also be subjected to **linear shrinkage, California Bearing Ratio and Unconfined Compression tests**. All of which are useful for evaluating road construction materials. Comparison of values of unsoaked and soaked CBR often help in assessing loss in strength likely to be suffered by soils when in contact with water at sites.

(iv) **Permeability test.** Undisturbed soil samples especially those collected from proposed road route need to be subjected to permeability test.
- Although permeability is not a routine test for evaluating road construction materials, experience over the years have shown that use of impervious soils (either as sub-base or subgrade materials for flexible highway pavement often results in development of pot holes.
- These are quick (undrained) triaxial and oedometer consolidation tests. They are used to generate data for determination of values of bearing capacity and settlement respectively.
- **Quick (undrained) triaxial test.** Values of the angle of internal friction (in degrees) are used to determined bearing capacity coefficients which are parameters in Terzaghi's equations. (e.g. $q_u = CN_c + \gamma_z(N_{q-1}) + 0.5\gamma_B N\gamma$ for a strip (rectangular footings).
- Values of cohesion, bulk density, width and depth of foundation are combined with cohesion for estimating Ultimate Bearing Capacity (UBC) which when divided by 3 (factor of safety) gives the Allowable Bearing Pressure (ABP).

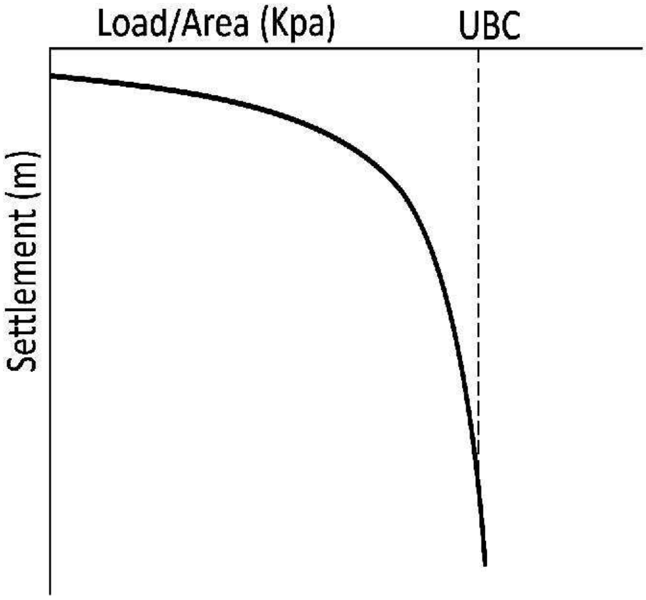


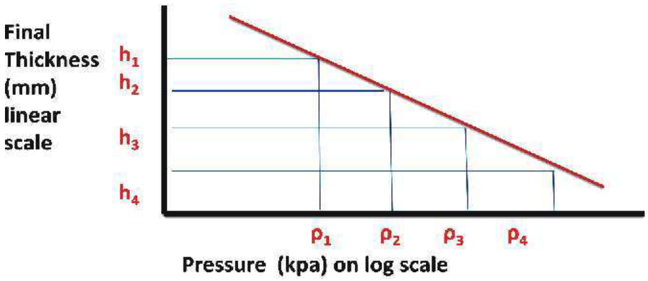
Fig. 2: Ultimate Bearing Capacity (UBC)

- The computed UBC and hence ABP can however be unrealistically low or high if the values of cohesion and angle of internal friction are wrong.

Oedometer Consolidation Test. The common approach in many geotechnical engineering laboratories is to determine the void ratio (which is proportional to moisture content) after completion of compression for each load. This approach takes two days, however.

- A faster method, which unfortunately is not commonly used in laboratories, utilizes the maximum reduction in thickness for each load.
- A plot of final thickness (on linear scale) against pressure (on log scale) gives a straight line with negative slope (Fig. 1)

Coefficient of (volume) compressibility (Mv) is obtained from the equation. Mv = - slope x 1/h. Where **h** is the thickness at which the slope is taken



$$Mv_1 = \frac{-(h_2 - h_1)}{(\rho_2 - \rho_1)} \times \frac{1}{\frac{(h_1 + h_2)}{2}}$$

$$= \frac{(h_1 - h_2)}{(\rho_1 - \rho_2)} \times \frac{1}{\frac{(h_1 + h_2)}{2}}$$

Unit is m^2 / KN or $K\rho_a^{-1}$

The amount of settlement, S is given by

$S = M_v \sigma H$

- Where σ is expected stress from the structure and H is the thickness of the compressible layer below the foundation.
- Value of Mv obtained from final void ratio method have been found to be close (±5%) to those from final thickness method.
- The total settlement at any recommended for a given structure depth should not be higher than the maximum permissible settlement stipulated in the building code of the country.
  - However, such a depth should not be affected by burrowing animals and plants roots.
- Differential settlement which may result from the heterogeneous nature of the foundation soil from one corner of a structure to another should be avoided because it can cause total collapse of the building.

Presentation and Discussions

- Field data in Engineering geology include those obtained from basic geological mapping, structural geological mapping, surface geophysical surveys, in-situ tests (e.g. penetrometer, SPT).
- Systematic and comprehensive testing and sampling are very important.
- Surface geophysical surveys (such as electrical resistivity, refraction seismic) often preceed geotechnical investigations (such as confirmatory in-situ tests and boreholes).
- Field sampling is often complimented with laboratory testing of rocks and soils, such tests can be petrographic, mineralogical, geochemical and geotechnical. Standard sampling and testing techniques are often adopted in order to compare results with standard specifications.

Data Presentation

- Treated raw data are often presented in graphical and tabular forms.
- Geoelectric profiles and borehole logs are presented in standard formats. When results are discussed, it is wrong to repeat what is in the table. Rather the summary can be given.
- The results should be compared with those of previous workers. Comparison with standard specifications is also necessary in order to know the possible utilities of geomaterials.
- Use of descriptive statistics such as analysis of variance (ANOVA) may be necessary.

Application of Data Generated from Field and Laboratory Geotechnical Investigations

- For Soil classifications such as Unified Soil Classification, Casagrande Chart Classification and American Association of State Highways and Transportation Officials (AASHTO) Classification system.
- Visual soil description combined with results of classification tests are often utilized for soil profile description.

- Results of classification tests, CBR, unconfined compression and permeability tests can be used to determine the suitability or otherwise of soils as highway sub-base and subgrade materials.
  Values of Allowable Bearing Pressure determined from penetrometer test are important for foundation design for structures to be founded on both shallow and deep foundations.
- Amounts of settlement determined in-situ or in the laboratory are often utilized to determine depths at which there will be no likelihood of differential settlement.

Construction Materials

These are mostly lateritic soils, sands, gravels and quarry products.

- Qualitative and quantitative evaluation involve use of geological, geophysical and geotechnical investigations.
- They must possess desired properties.
  - Soils from laterite horizon, rocks with high amount of quartz, low amount of feldspars and flaky minerals are good.

Conclusions

- Engineering Geologists and Geotechnical Engineers should work together in order to ensure generation of in-situ and laboratory data of high integrity.
- Activities of technologists and technicians should be diligently monitored to ensure that generated data are reliable.
- Preconstruction investigations which do not often cost more than 15% of contract sum for a structure should be properly executed. This is a sure way to ensure stability of our structures as is the case in developed Countries.

Construction materials should be comprehensively evaluated prior to being used. Not all rocks are granites and not all lateritic soils are good for construction purposes.

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Integration of Geoscience and Engineering Skills for Sustainable Development

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Introduction

More than half of the world's population now live in urban areas and by 2050, two-thirds of all humanity will live in urban areas. In addition, urban expansion in the next few decades will take place in developing world and this rapid urbanization is exerting pressure on fresh water supplies, sewage, the living environment, transportation, public health etc. Thus, the challenges facing our society in the twenty-first century are numerous and inherently complex (United Nations, 2015a) hence, it requires multi-disciplinary approaches to solving them.

In September 2015, the Sustainable Development Goals (United Nations, 2015a) were formally adopted by member states of the United Nations, building on the Millennium Development Goals (Millennium Project, 2006). Their stated aims are to facilitate, sustained an inclusive economic growth, social development and environmental protection, thereby end unsustainable consumption patterns and eradicate global poverty (United Nations, 2015a). The goals are to be achieved, over a 15-year timeframe (2015–2030) through the engagement of the geoscience community and other disciplines.

The societal relevance of geology has long been recognised, with many members of the geology community committed to the practical application of our discipline to address challenges of global hardship, inequality, and vulnerability. (Stow and Laming, 1991; Cordani, 2000; Mora, 2013).

The importance of geoscience investigation prior to environmental and civil engineering works such as, construction of buildings, dams, roads, rails, bridges, tunnels etc. cannot be overemphasized, since it helps in managing and preventing potential shallow subsurface geohazard associated with inaccurate positioning of structures. In addition, remediation of polluted sites and possible restoration to pristine condition, drilling for water and hydrocarbon, location and evaluation of bitumen and mining of solid minerals all requires geoscience investigation for successful and sustainable projects.

Geological research, investigation, monitoring, innovation and engineering continues to drive widespread improvements to wellbeing and quality of life. However, the lack of or failure to incorporate subsurface detailed investigations in project execution will result in non-sustainability of environmental and engineering projects leading to negative impact on the environment and the people.

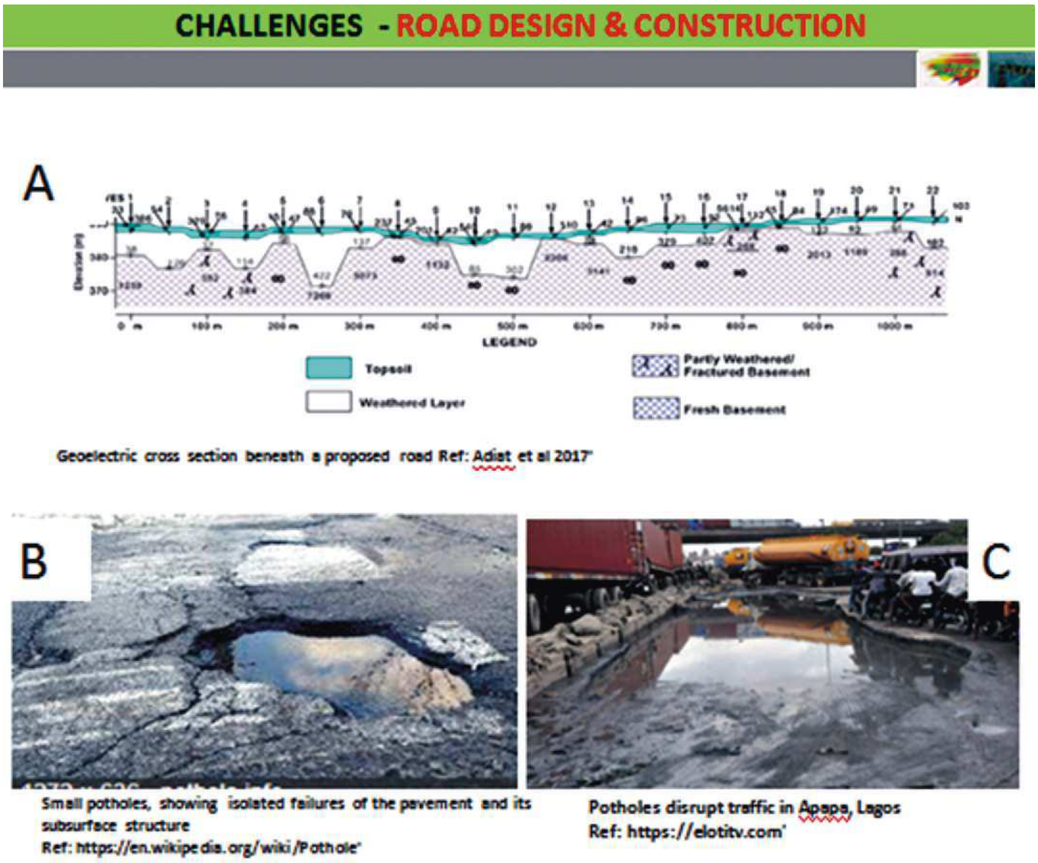
In the past, the construction of railway line initiated and built in 1897, the Niger bridge at Onitsha, the construction of first, second and third mainland bridges in Lagos, the survey and dredging of the Niger/ Benue rivers etc., involved the incorporation of geological studies in the execution of these sustainable projects. Today, technology and techniques continues to change, therefore in achieving sustainability, new technology that unravels location of geological hazards in the subsurface should be introduced in the execution of projects.

Objectives

- To emphasize the importance of integration of various geoscience skills in achieving sustainable projects
- To reduce risk associated with Environmental and Engineering projects by identifying geohazard in project locations thereby increasing confidence and reward.

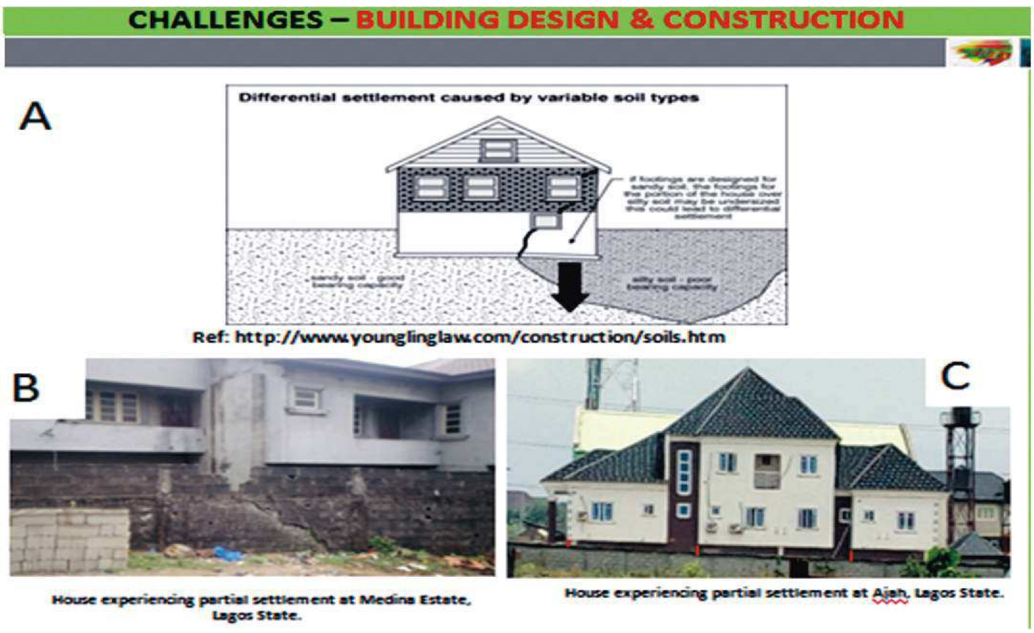
❖ Road Failure could be defined as a discontinuity in a road pavement due *to lateral and vertical heterogeneous nature of the sediment* on which it is constructed.

❖ Failure along Nigerian roads as a result of cracks,

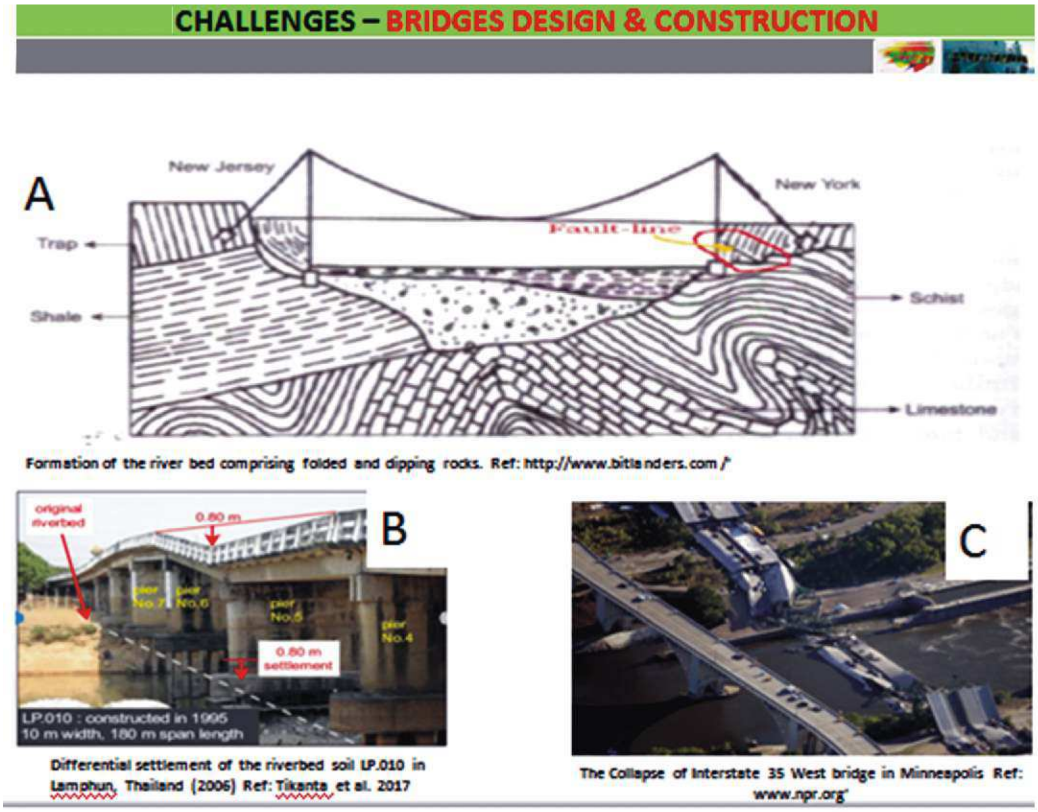


potholes, bulges and depressions affects both the road users and the vehicles.

❖ It represent an increasing safety, environmental and health challenge in an unsustainable manner.

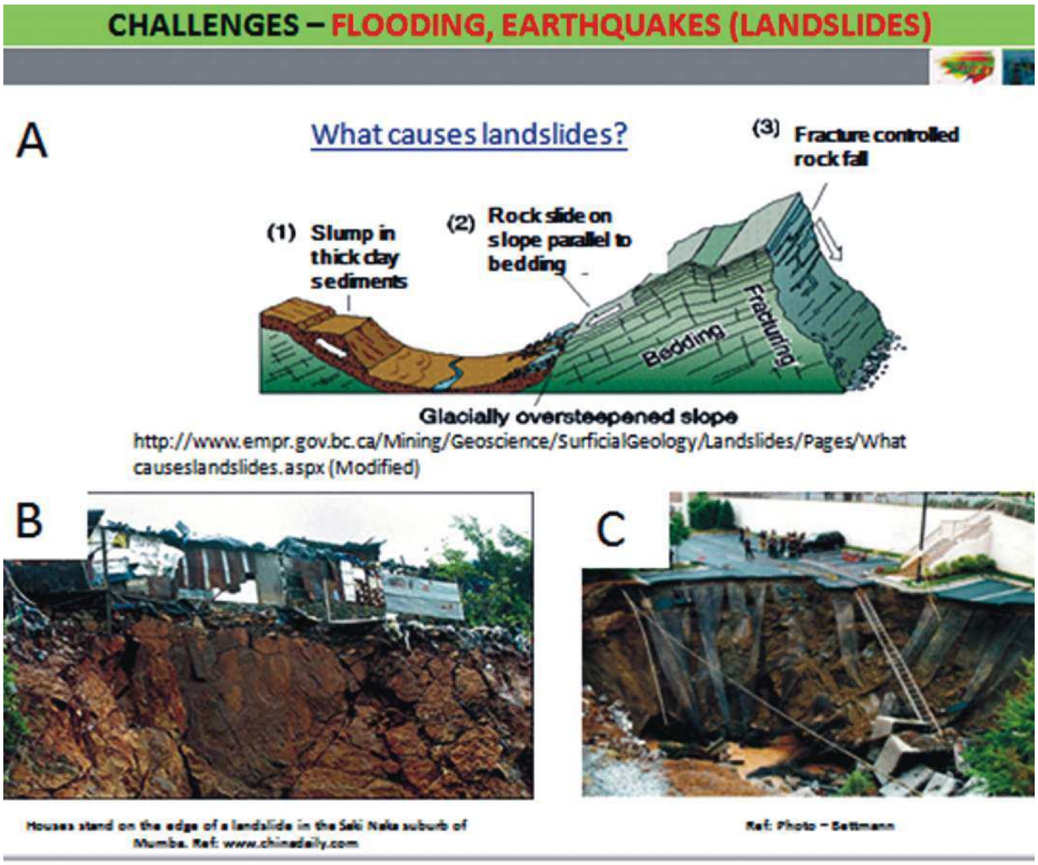


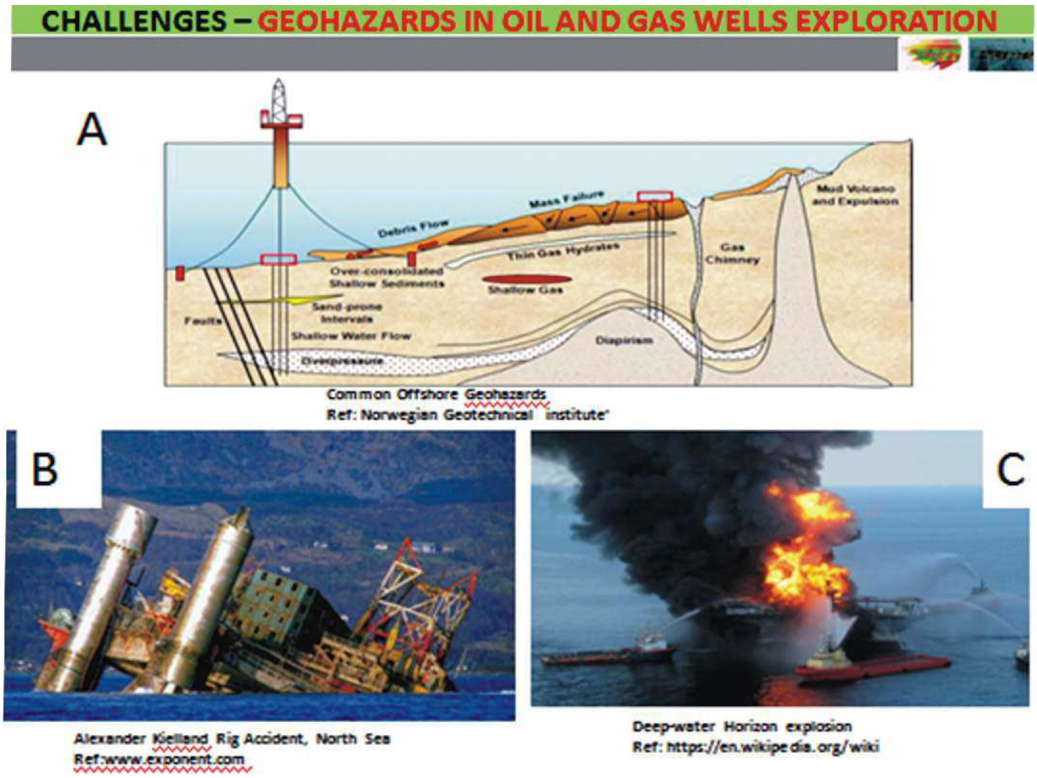
- Generally, every construction work in civil engineering is built on soil or rock and in many instances they are also used as raw materials for construction of infrastructures
- Unstable building foundation occur because of several reasons *including incompetent bedrock such as peat/lignite resulting into partial settling of houses and anchoring bridges on clay.*
- Geotechnical risk affects all those involved in construction, including the client (which may ultimately be government, especially for national infrastructure projects), the designer, the constructor and the general public.
- Therefore, a good understanding of the properties of the bedrock and its behaviour under load before usage is highly essential in civil engineering and highway pavement construction.



- Inadequate geological information on the nature of subsurface conditions, i.e. *strata competence, prior to construction of either bridges or buildings results to structure collapse.*
- In recent times, bridge foundation damage has been a very serious problem that cause unnecessary delay in traffic flow, closure of bridges for integrity checks and repairs, breakdown of vehicle and most significantly, causes road traffic accident.
- The need for bridge foundation studies prior to design/construction therefore becomes necessary so as to prevent loss of valuable lives and properties that always accompany such failures

- Landslides have damaged or destroyed roads, rails, pipelines, dams, airports, building etc. *These are often triggered by natural events such as floods, earthquakes etc*
  - Subsidence on the other hand is caused by a diverse set of human and natural activities such as, groundwater and petroleum mining dewatering of organic soils, liquefaction etc.
  - Colossal loss of life, public and private properties are the results of this natural hazards
  - Identification of landslide area and anticipating landslide events helps to sustain public and private facilities.
  - Mitigation measures as approved by appropriate agencies, is required.


CHALLENGES – FLOODING, EARTHQUAKES (LANDSLIDES)
A
What causes landslides?
(1) Slump in thick clay sediments
(2) Rock slide on slope parallel to bedding
(3) Fracture controlled rock fall
Bedding
Fracturing
Glacially oversteepened slope
http://www.empr.gov.bc.ca/Mining/Geoscience/SurficialGeology/Landslides/Pages/What causeslandslides.aspx (Modified)
B
C
Houses stand on the edge of a landslide in the Saki Naka suburb of Mumba. Ref: www.chinadaily.com
Ref: Photo – Bettmann


CHALLENGES – GEOHAZARDS IN OIL AND GAS WELLS EXPLORATION
A
Mass Failure
Debris Flow
Thin Gas Hydrates
Over-consolidated Shallow Sediments
Shallow Gas
Sand-prone intervals
Faults
Shallow Water Flow
Overpressure
Diapirism
Gas Chimney
Mud Volcano and Expulsion
Common Offshore Geohazards
Ref: Norwegian Geotechnical institute'
B
C
Alexander Kielland Rig Accident, North Sea
Ref:www.exponent.com
Deep-water Horizon explosion
Ref: https://en.wikipedia.org/wiki

- A geohazard is defined as any geological event that poses a threat to people and/or their property.
- When a well accidentally penetrates a shallow gas accumulation in the riserless section of the well, the well is breached to the sea surface causing the collapse of the platform
- The collapse of the platform could be total, leading to loss of life and huge investment to oil companies. Explosion and subsequent fire can result in the sinking of the platforms, oil spill, injuries and death.
- Inadequate geoengineering studies result in this failure.

- Petroleum industry is one of the major industries which release hydrocarbon pollutants through sabotage, mechanical failure etc. to the environment.
- The indiscriminate accumulation of petroleum hydrocarbon pollutants are hazardous to the animals, human life and aquatic biota.
- With over 2000 petrol stations in Lagos with various underground storage tanks poses threat to underground water when they leak.
- Hence, for sustainable environmental and biodiversity management and conservation, environmental and engineering geologist are critical in assessing appropriate geological formations for underground storage of petroleum products

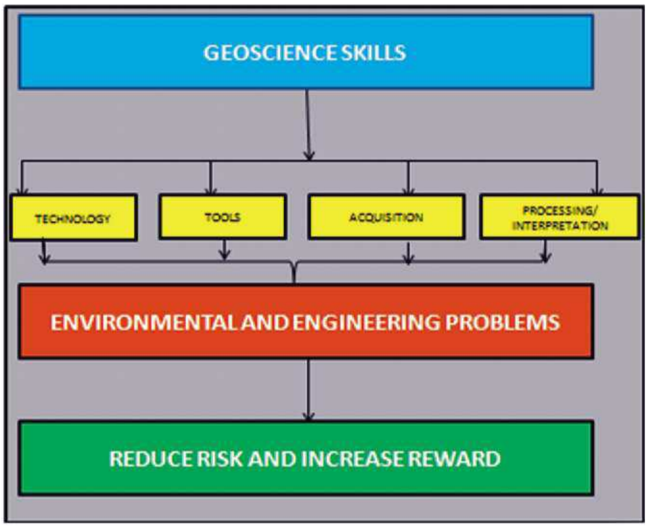
When an abandoned strip mine is converted into a refuse disposal/ dumpsite, the engineering geologic properties and hydrogeological conditions of the site must first be investigated.

Aggregate protective capacity of the underlying lithology, the depths to potential aquifers, the nature and arrangement of the permeable and impermeable materials present in the ground, the direction of ground-water flow, and the direction of surface water drainage must all be determined prior to the use as sanitary landfill or liquid-waste disposal-site.
With this knowledge, the idea of these sites for such refuse can either be abandoned or implemented, and if necessary, adequate site engineering, such as the construction of impermeable liners or artificial leachate drainage systems be implemented for a sustainable landfill use

Geosciences Skills: Environmental and Engineering Projects

- Africa is the world's second largest and second most-populous continent, with over 1.2 billion people as of 2018. It accounts for about 16% of the world's human population. Nigeria with the largest population in Africa has over 140million people in 2006 census. In 2018, over 195million people has been estimated and by 2022, a total of 235million is being estimated. This poses a great challenge on utilizing existing infrastructure and environmental management.
- In developing and sustaining the needed infrastructural development for this growth in population, engineers, engineering geologist and environmental geologist are crucial in identifying safe sites for construction and the exploration of construction materials.
- They would play an active role in identifying land for housing, agriculture and natural resource utilization ensuring that money spent is not wasted on non viable development projects.
- Geosciences skills used in environmental and engineering projects include but not limited to the following
  - Intrusive Methods
  - Non-intrusive Methods
- The application of non-intrusive techniques such as ground penetrating radar, electrical resistivity, echo sounder, side scan sonar, sub bottom profiler , ADCP (Acoustic Doppler Current Profiler), seismic data etc. in assessing geological risks provides a wide range of cutting edge solutions.
- The integration of results from non-intrusive and intrusive (geotechnical data) will provide better knowledge for effective, efficient and improved monitoring of conditions for construction, transportation facilities and remediation processes in contaminated sites.

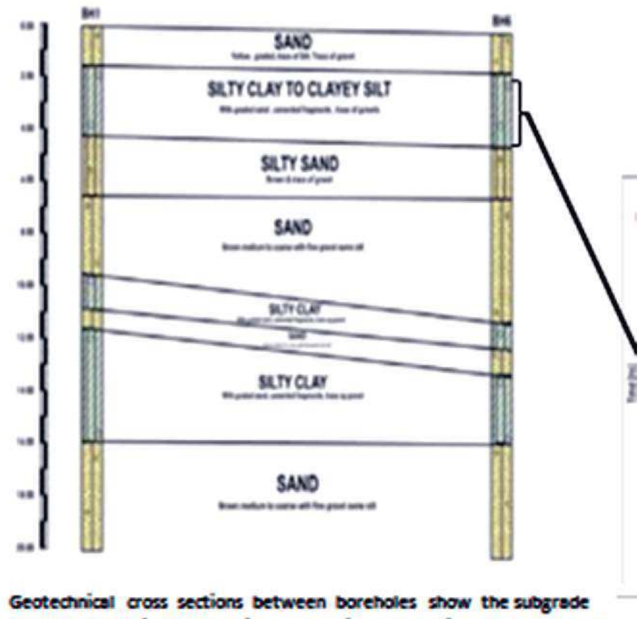
Geosciences Skills Used in Environmental and Engineering Projects



Examples and Experiences – Engineering Projects Using Electrical Resistivity Method in Assessing Road Failure

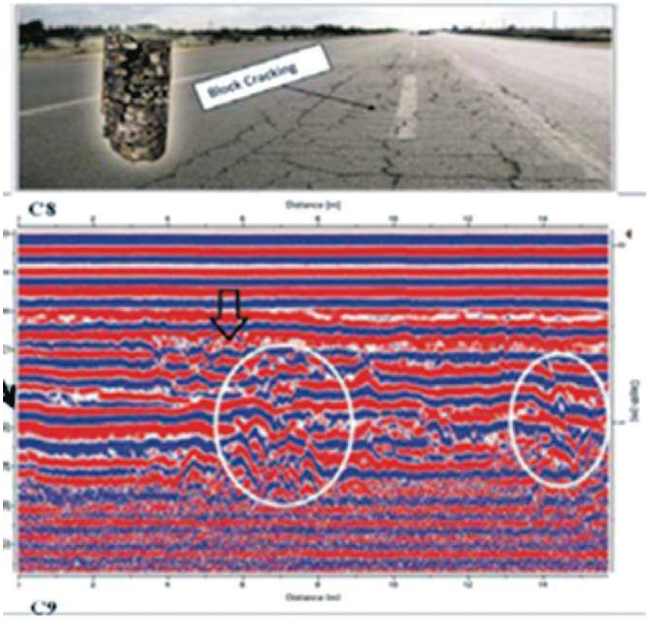
- Traditional approach of performing geotechnical analysis at selected locations and inferring the subsurface lithology along a road path had led to substantial part of the road to be unstable and impassable.
- Today's technology of using non intrusive geophysical applications which include GPR survey, resistivity survey gives both the lateral and vertical lithological and structural variations in 3D .
- Using multidisciplinary approach to execute road engineering projects will lead to sustainable projects with greater rewards.

Geotechnical cross sections between boreholes (Traditional Approach)



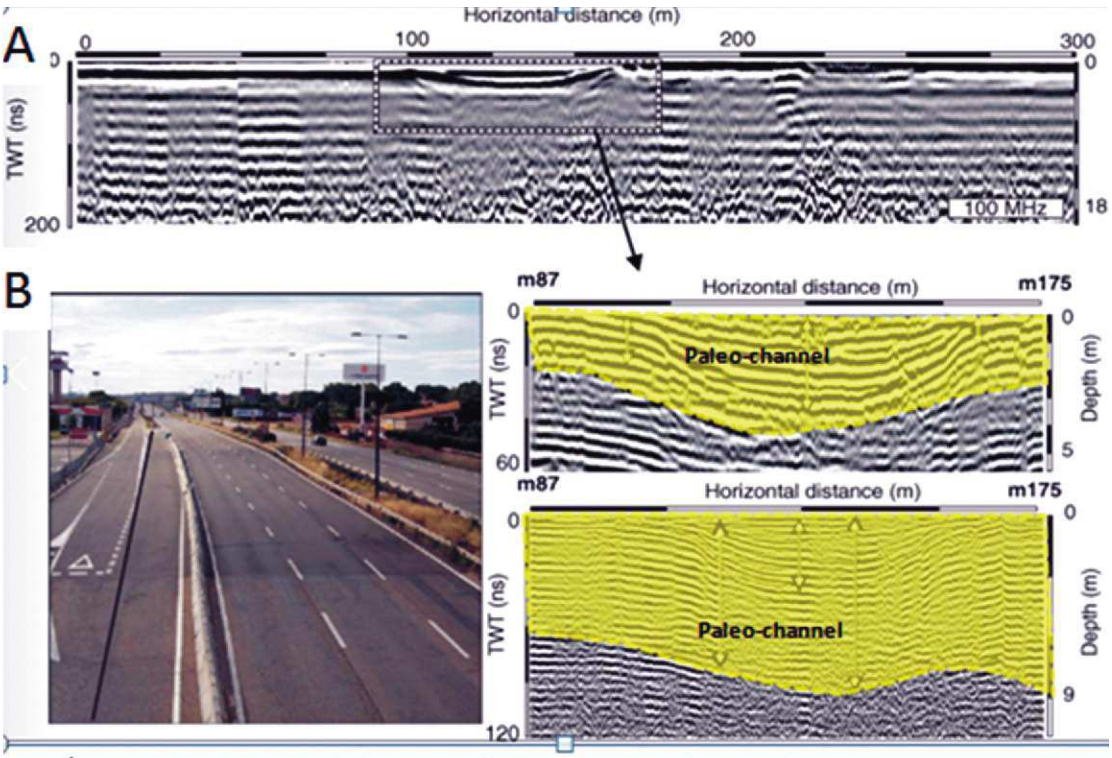
Geotechnical cross sections between boreholes show the subgrade layers of the Cairo-Alexandria desert highway section. Ref: Amer et al 2014 *

Application of non intrusive geophysical method (Ground Penetrating Radar Method)



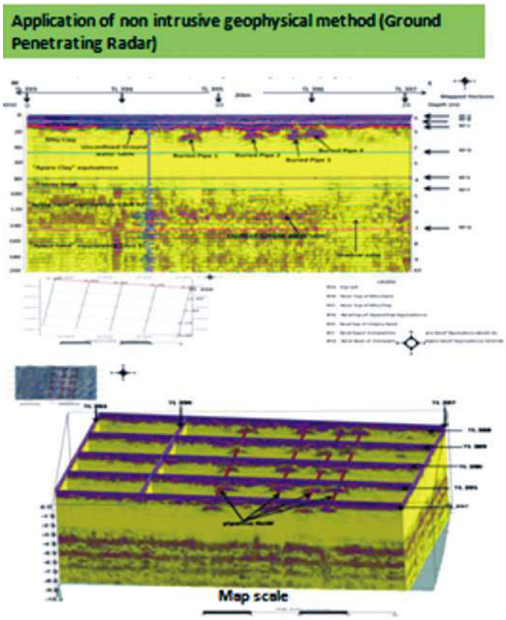
GPR profile carried out at a pavement section with blocking cracks Ref: Amer et al 2014 *

Examples and Experiences – Engineering Projects Using Ground Penetrating Radar in Assessing Road Failure



Ó. Pueyo Anchuela et al. : Characterization of the Karstic Process in an Urban Environment Using GPR Surveys

- ❖ Submerged paleo channels are potential hazards along road paths.
- ❖ Imaging of these features through non intrusive geophysical methods can assist in mitigating against the non-sustainability of constructed highway along this path.
- ❖ Sustainable development can be achieved through integration of geoscience skills and engineering in road construction

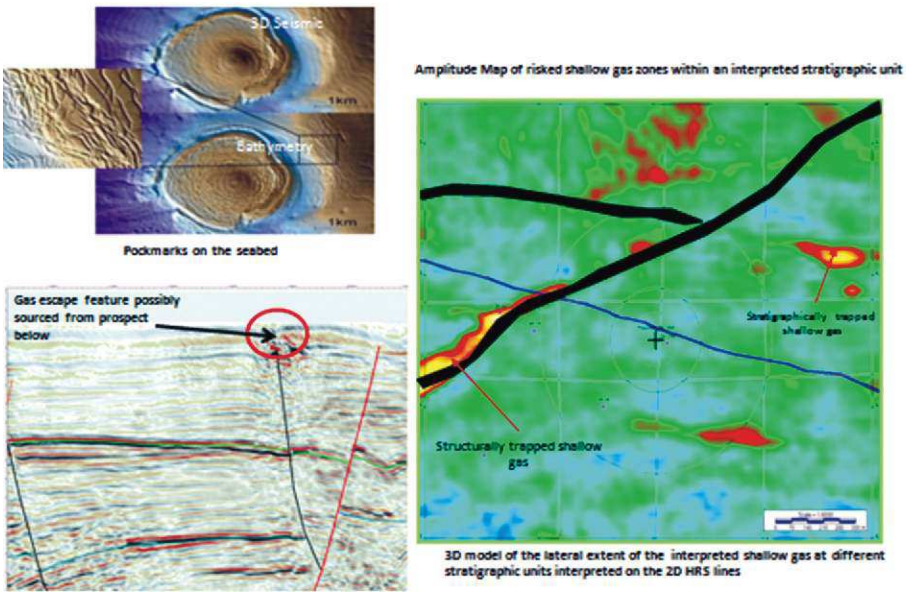


Applications of GPR

- GPR is used for identification of buried infrastructures such as pipelines (both active and abandoned), cables, platforms and maintenance purposes.

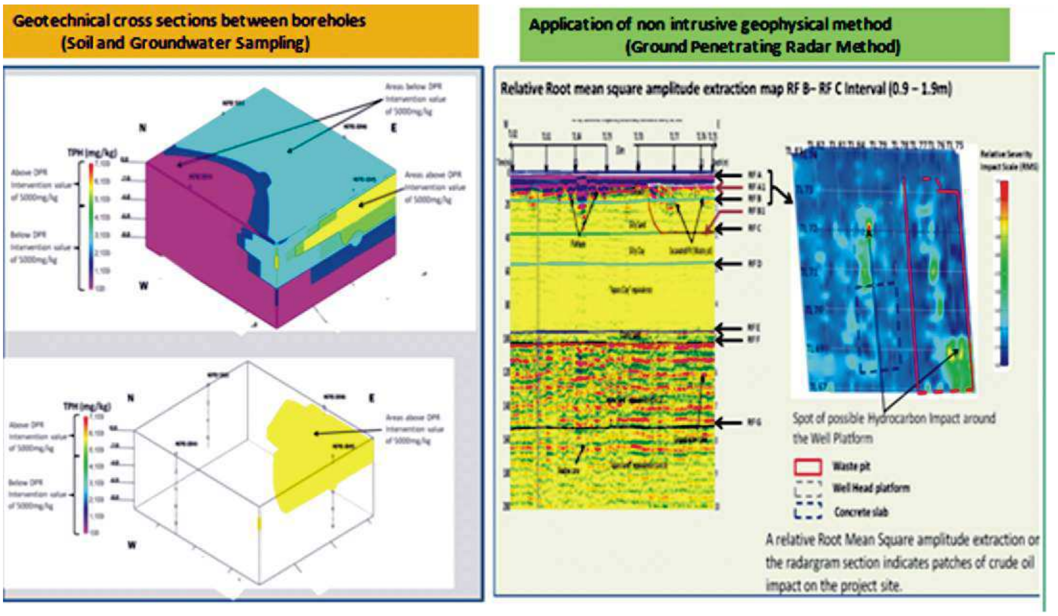
Examples and Experiences – Engineering Projects Using GPR Technique in Locating Buried Subsurface Utilities

Examples and Experiences – Engineering Projects Using Seismic Method in Shallow Gas Geohazard Evaluation and Site Survey for Oil Wells



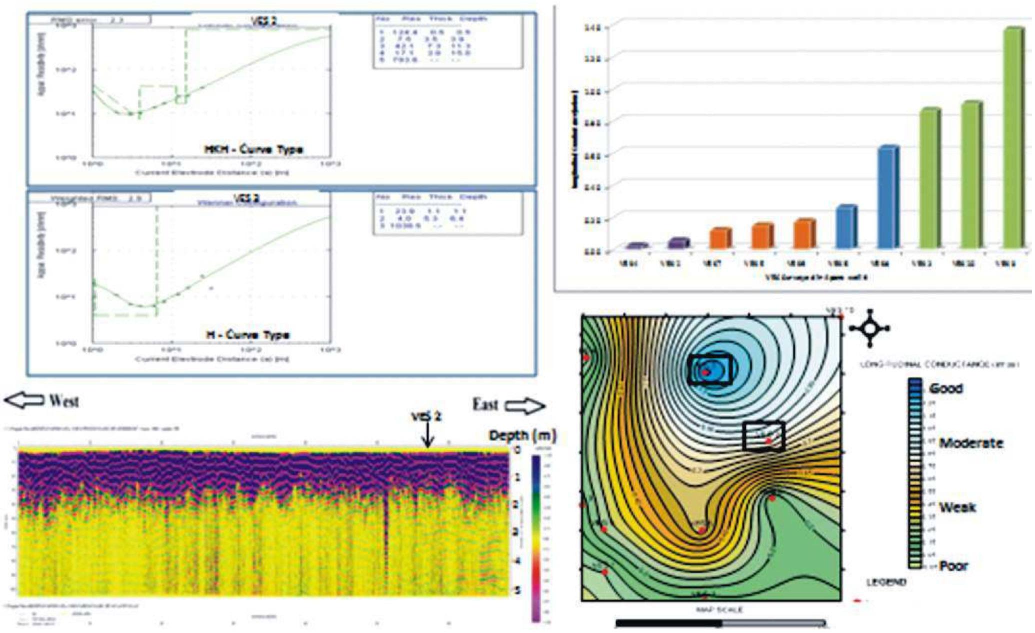
- Shallow gas pose a great threat to the safe extraction of fossil energy.
- Accurate and effective delineation of these shallow gas geohazard mitigates against blow out.
- Understanding the hydrocarbon potential and safe extraction of the resources promote sustainable industrialization.

Examples and Experiences – Environmental Projects Using Intrusive Methods in the Identification of Contaminated Soil and Water for Remediation Purposes



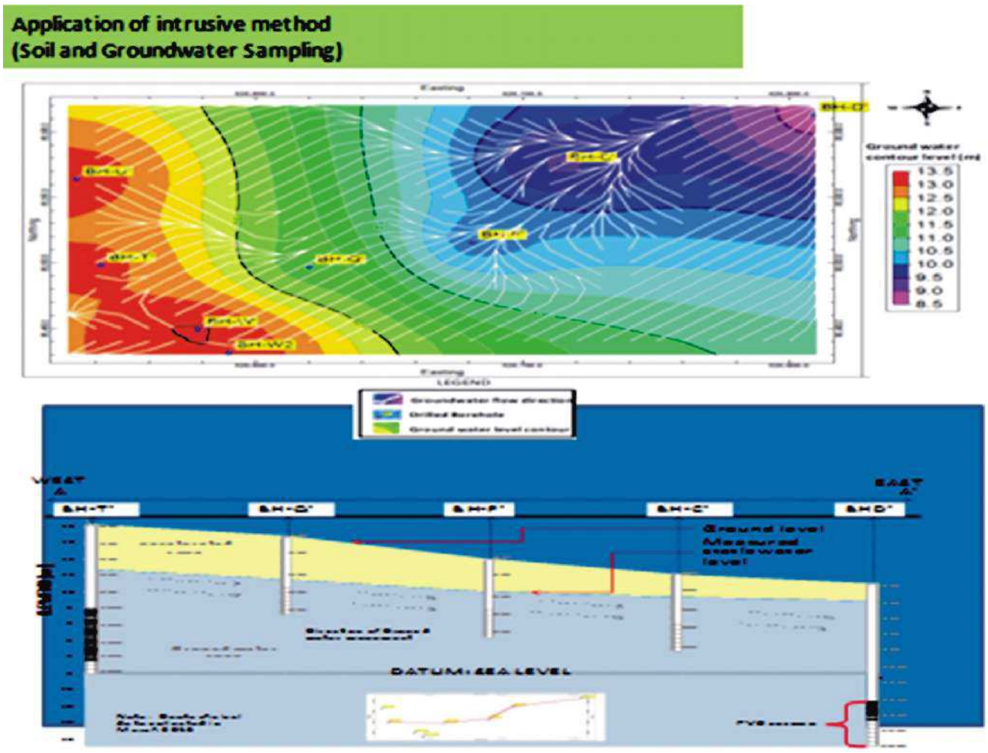
- The use of traditional approach of sampling at selected locations in assessing contaminated sites results in overestimation of impacted sites
- Application of non intrusive accurately determines the impacted areas thereby resulting into a better estimation of impacted soil and allowing robust planning for effective decision making.
- It is also cost effective and time saving

Examples and Experiences – Environmental Projects Using Electrical Resistivity Method in Assessing Geotechnical Properties in the Subsurface



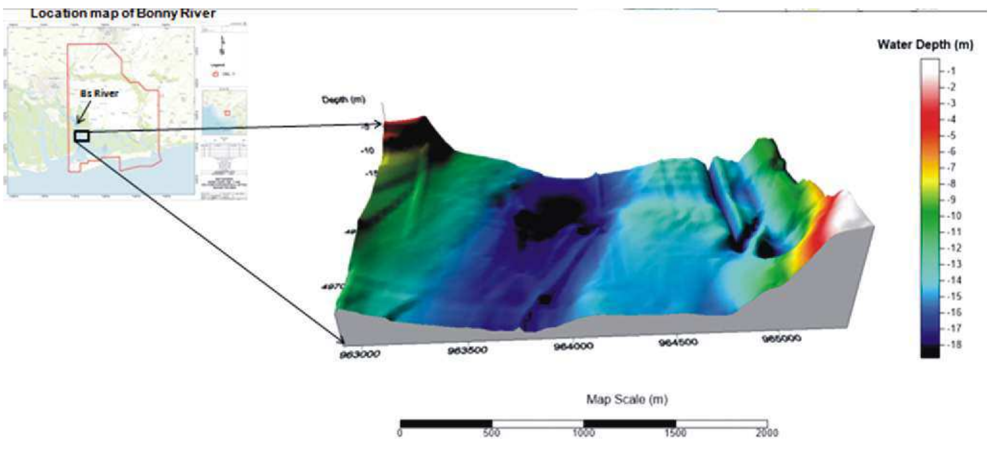
- ❖ High clay content generally corresponds with low resistivity values and low hydraulic conductivities.
- ❖ The protective capacity of the overburden can be considered as being proportional to the longitudinal unit conductance (S) defined as the ratio of the overburden thickness to its resistivity.
- ❖ The higher the overburden longitudinal conductance, the higher is the protective capacity

Examples and Experiences – Environmental Projects Using Intrusive Methods in the Identification of Contaminated Soil and Water for Remediation Purposes



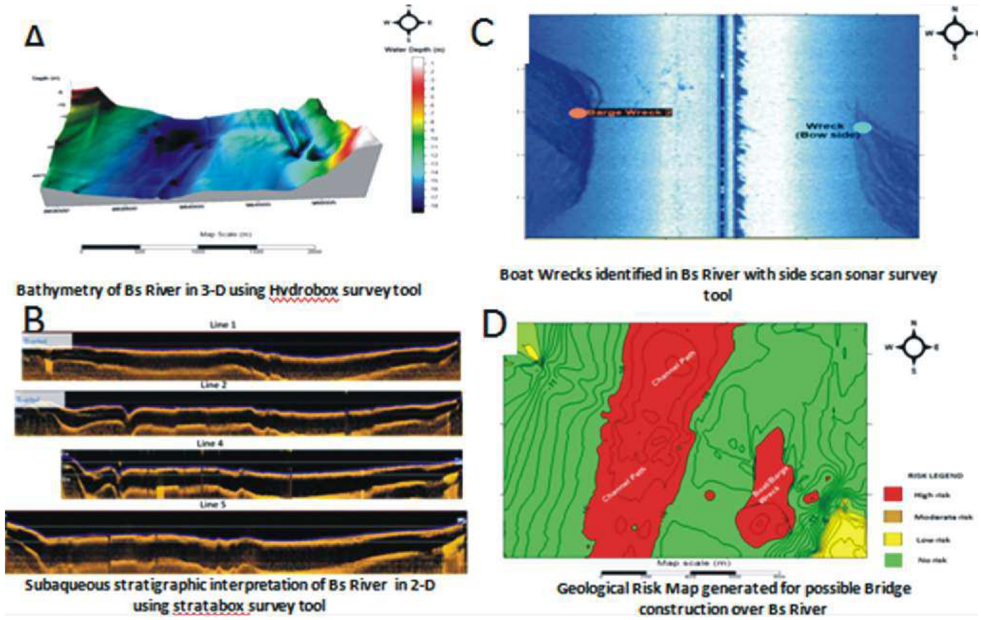
- Using data from bore hole elevation and static water level measurement, groundwater flow direction can be modeled using the appropriate software.
- This is used in planning groundwater remediation projects.
- Used to determine appropriate locations to install injection and recovery wells in groundwater remediation projects.

Examples and Experiences – Hydroengineering Projects Using Hydrological Tools in Detecting and Identifying Geohazard on and Below the Riverbed



Bathymetry of Bs River in 3-D using Hydrobox survey tool

Examples and Experiences – Hydroengineering Projects Using Hydrological Tools in Detecting and Identifying Geohazard on and Below the Riverbed



Bathymetry of Bs River in 3-D using Hydrobox survey tool

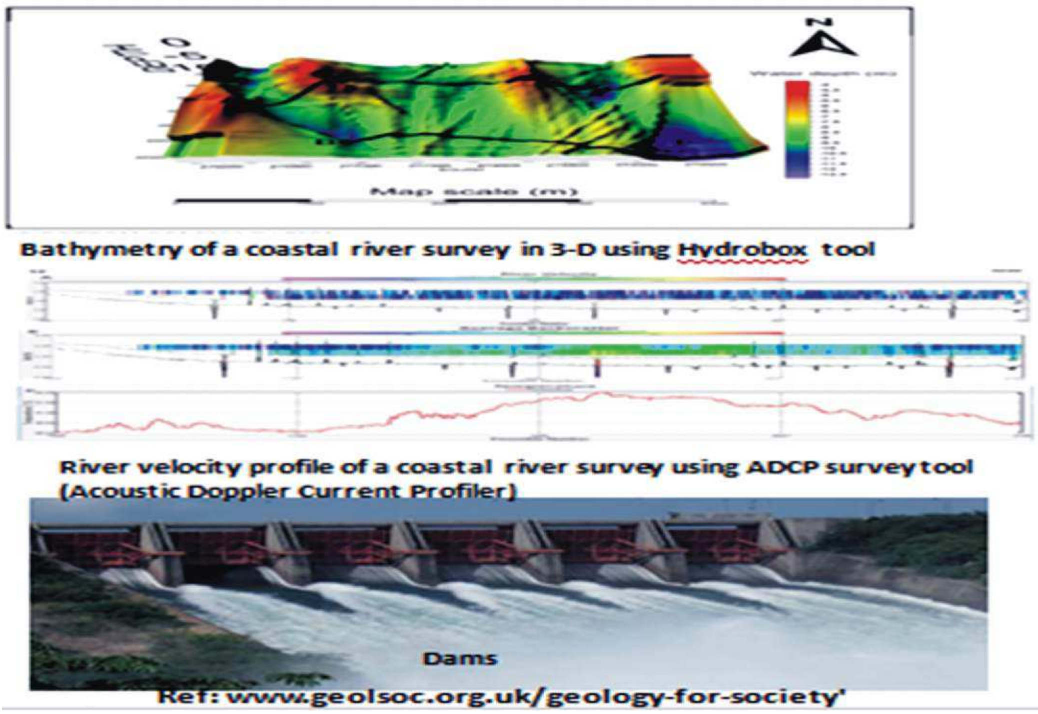
Boat Wrecks identified in Bs River with side scan sonar survey tool

Subaqueous stratigraphic interpretation of Bs River in 2-D using stratabox survey tool

Geological Risk Map generated for possible Bridge construction over Bs River

- Optimal anchor locations for marine platforms in order to avoid subsurface geohazard such as boat/ship wrecks, debris low etc.
- Placement of piles and piers for bridge construction over water bodies.
- In achieving sustainable developmental projects executed over water bodies, geotechnical geophysical survey is often the most cost-effective means for obtaining subsurface information

Examples and Experiences – Hydroengineering Projects Using Hydrological Tools in Detecting and Identifying Potential Power Generation from Rivers



Bathymetry of a coastal river survey in 3-D using Hydrobox tool

River velocity profile of a coastal river survey using ADCP survey tool (Acoustic Doppler Current Profiler)

Dams

Ref: www.geolsoc.org.uk/geology-for-society'

- An understanding of local hydrology is essential to managing water supply and hydroelectric power generation.
- Hydrogeologists and other geoscientists investigate river current using ADCP (Acoustic Doppler Current Profiler) tool in order to model and understand the movement of water, and to quantify and characterize aquifer resources.
- For example, FX river with a depth range of 6m to 20.5m and a total discharge ranging from 21,088.697m3/s to 29,145.380m3/s has the capacity to generate 9, 309.71MW of electricity.
- Similarly, EY river with a maximum depth of 21.7m and a total discharge of 102.96m3/s. has the capacity to generate 19.743MW of electricity
- Finally, NCW river with a maximum depth of 12m and a total discharge of 5,206m3/s has the capacity to generate 552.058 MW of electricity.
- However, hazards associated with dam embankment such as leakages, erosion, slope stability can be prevented by improved prediction and prevention by characterizing rock masses prior to dam construction, thus leading to sustainable executed projects.

Conclusion And Recommendation

- The application of non-intrusive techniques/tools such as ground penetrating radar, electrical resistivity, ADCP (Acoustic Doppler Current Profiler) and seismic data etc. in assessing geological risks provides a wide range of cutting edge solutions. The integration of results from non-intrusive and intrusive will provide a better knowledge for effective, efficient and improved monitoring of conditions for construction, transportation facilities and remediation processes in contaminated sites. Therefore, it is recommended that detailed sub-surface investigations be undertaken while findings and recommendations are taken into account in building of structures and remediation of contaminated sites.
- Nigeria Association for Engineering Geology and Environment (NAEGE) should invite various ministries such as works and housing, transport, environment, mines and solid minerals for NAEGE conferences. Emphasis should be placed on qualified engineering geologist and environmentalist to be staffs of these ministries. These professionals need to work together to achieve the sustainability goals. The lack of or failure to incorporate subsurface detailed investigations in project execution will result in failure of environmental and engineering projects which will lead to negative impact on the environment and the people.

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Geoethics in Engineering Geology and the Environment

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Abstract
The recognition of expected responsibilities of the geoscientists to the society, the environment and the geoscientists themselves, is one of the main purposes of geoethics. Personal and professional attitudes to issues differ. Hence, the peaceful conduct/practice of any science subject including engineering geology requires some standard way or manner of practice that can protect the practitioner, engender public trust in the research findings, satisfy the society and also preserve the environment for future generations. Such a standard is called the code of practice. Other disciplines have put their codes of practice in place. We in engineering geology also need to develop an acceptable way of practicing our discipline. Teaching of geoethics as an interdisciplinary science can help us to do this. Good intra-disciplinary relationship/behaviour between practitioners, adequate training and retraining of geoscientists to prepare them for the needs of a constantly changing world, can promote greater understanding of acceptable and unacceptable professional behaviour in the work environment. Unprofessional issues such as plagiarism, fabrication or falsification of data, and the wrong use of power often lead to mistrust and lack of respect between practitioners in the geosciences. Geoethics can address these cases and cause general public acceptance of geoscientific research results and also rely on the quality of their products.

Keywords*: Ethics, Geoethics, Geoscientist, Engineering Geology, Society, Environment, Inter-disciplinary, Code-Of-Conduct, Relationships.*

Introduction

Every discipline has its code-of-practice. It is important that all the geoscience disciplines develop their own ways of practicing different areas of the subjects in a way that is acceptable to the practitioners, the general society and the environment, so that neither the practitioner's interest nor the society's and eventually the environment is jeopardized. The satisfaction of these three groups is often difficult to achieve because the development of ethics in the geosciences is still in its infancy. In this presentation, it is considered necessary to go through the rudiments of Geoethics starting with the relevant definitions, the importance of this subject, when and where it is needed, and how it is being practiced at the moment. With this approach, it is believed that all attendees at this conference will get some understanding of this important subject and how it is being practiced at the moment. Because almost all that we do in the geosciences affect the environment one way or the other, the subject of environmental sustainability, as conceived and presented by the United Nations may also be considered under this topic.

Definitions of Terms

In other to understand this subject it is important to provide the definitions of certain terms.

Ethics: The dictionary meaning of ethics is given as the study and philosophy of human conduct with emphasis on the determination of right and/or wrong. It is the basic principle of right action especially with reference to a particular person, profession etc. It is a work on treaties or acceptable norms.

GeoEthics: Based on the above definition of ethics, GeoEthics is considered as ethics applied to geological sciences. It is an accepted code of practice by geoscientists in all that they do. It is an emerging field of geosciences which considers many dimensions of the relationship between the geoscientist, the society and the earth. A more detailed definition by the International Association For Promoting Geoethics (IAPG) is provided/reproduced here. "Geoethics consists of the research and reflection on those values upon which to base appropriate behaviours and practices which intersect the geosphere".

Engineering Geology: This is the application of the geologic sciences to engineering practice for the purpose of assuring or ensuring that all geologic factors affecting the location, design, construction, operation and maintenance of engineering works are recognized and provided for. (Dearman, 1974; Esu, 1981; Malomo, 2008). Also, the International Association of Engineering Geology and the Environment (IAEGE) defines Engineering Geology as:

> "The science devoted to the investigation, study and solution of the engineering and environmental problems which may arise

as the result of the interaction of geology and the works or the activities of man, as well as the prediction and development of measures for the prevention or remediation of geological hazards" (IAEGE Status, quoted from Akpokodje, 2016).

Environment: This term may be considered as a containment structure for all geologic studies. It covers the air, water and the solid earth and all they contain. Almost every geologic work is done within the environment. The effect of such activity on the environment can be either positive or negative.

Environmental Sustainability: This involves the exploitation and management of the environment by the present generation in such a way that future generation can still depend on the same environment. This process involves the maintenance of the environment so that no geologic practice will destroy the environment, therefore the generations to come can still depend on that environment. This is the United Nation's concept of environmental sustainability.

Meaning of the Topic

Based on the definitions of the terms above, this topic refers to the conduct of geological work or study in such a way that an accepted (best practice) can be achieved without jeopardizing the interests of professional geoscientists, the society now and the future generations that should depend on the same environment. GeoEthics encompasses the values and professional standards required of geoscientists to responsibly work in the profession and in service to society. The training of scientists in ethics has traditionally been focused on the Responsible Conduct of Research. However, GeoEthics encompasses many more dimensions, including personal and professional behaviours as well as responsibilities to society and in their stewardship of the Earth.

Importance and applications of Geoethics

In other to win the confidence of geoscientific research, teaching of GeoEthics must involve first the education of the general public. In this process, the scientific community as well as our civic communities must be made to have trust in the conduct of the scientists and must also believe that the integrity/quality of scientific products are above reproach. The place of geoethics in providing solution(s) to environmental problems caused by the glutonous attitudes of man's exploitation of the natural environment can be clearly understood when one realizes the strong and rather inevitable dependence of economic and social issues on the environment. There is a delicate reciprocal balance between the human society and the environment. Human activities tend to degrade the environment and alter the land-water ecosystem, while environmentaal hazards result in economic losses to the human society (Akpokodje, 2016).

Geoethics deals with the ethical, social and cultural implications of earth sciences research and practice, providing a point of intersection for Geosciences, Sociology and Philosophy. It provides an opportunity for geoscientists to become more conscious of their social role and responsibilities in conducting their activities. It is also a tool that can influence awareness of the society regarding problems related to geo-resources and the geo-environment.

Based on the definitions of geoethics by the IAPG (2015) and the works of others such as Vasconcelos et al (2016) and Mogk and Bruckner (2017), geoethics:

- Provides a reference and guidelines for behaviour and addresses the consequences of misconduct;
- Points out the social role and responsibilities of geoscientists and the impacts of their work on the society;
- Encourages a critical analysis of the use and management of geo-resources like water and minerals;
- Deals with problems related to risk management and the mitigation of geo-risks;
- Highlights the value and usefulness of geological and geophysical knowledge;
- Encourages proper and correct dissemination of the results of scientific studies and information on risks;
- Determines to improve the relationship between the scientists, the mass media and the general public;
- Promotes the development of geo-parks and geo-tourism, thereby enhancing geo-heritage;
- Encourages the development of effective tools which can promote awareness, values and responsibility among young people.

When and where is Geoehics needed and who needs it?

A close look at the above listings reveal that the practitioner, the society and the environment all need geoethics, all the tme and everywhere geological activity is conducted. Hence, the above listings may be placed in four main areas to address this sub-head. They include:

1. Geoethics and self: this aspect deals with the internal attributes of the geoscientist and the ethical values needed to prepare for and contribute to a career in the geosciences.
2. Geoethics and the geoscience profession: this facet deals with ethical standards expected of geoscientists in the practice of their profession.
3. Geoethics and the society: this facet deals with the responsibilities of the geoscientists regarding effective communication of research results to the society about natural resources utilization, geo-hazards, health, safety and economic security of humanity.
4. Geoethics and the earth: this section concerns the responsibilities of geoscientists regarding their stewardship of the earth based on their knowledge of the earth's composition, architecture, history, dynamic processes and complex systems.

According to Mogk (2017), one of the main contributions of geoethics in professionalism is to help geoscientists to recognize and accept established standards and norms of professional behaviour, recognize unprofessional behaviours as they emerge, prevent these issues from having destructive and irreversible consequences, and the geoscientists must also have tools to act and mitigate/resolve any such undesirable issues promptly.

Other facets of geoethics concern Microethics and Macroethics, which had been discussed by Mogk and Bruckner at the 2014 IAPG Workshop on the topic: "Teaching Geoethics across the Curriculum". To them, microethics concerns personal and professional ethics, which can be tied to responsibilities at personal and intra-professional levels. A typical example is the provision of a client with reliable data by an environmental consultant.

Mogk and Bruckner (2014) also tie macroethics to the personal and professional responsibilities of a professional environmental consultant, whose concern is mainly the preservation and sustainability of the environment, simply because future generations of the society and culture have to depend on the same environment for survival. The above understanding therefore shows the indispensable relationship between microethics and macroethics. This understanding also controls why we think and do things the way we do.

Methodology

Published works based on the experiences of workers in different scientific fields such as physics, mathematics, biology, chemistry and geology etc., tend to show no good relationship between these disciplines. A somewhat stronger relationship is enjoyed between the arts, social sciences, management sciences, law and medical sciences. Geoethics as an emerging inter-disciplinary science has the focus of bridging the gap between the sciences, philosophy, sociology, law, management studies and any other disciplines. The methodology adopted for this paper directly spells out that "everything is related to everything else" (Tobler, 1975). Hence, the focus on relationships between different disciplines which are meant to serve man, the society and the environment advantageously, is that the training programmes for future geoscientists must have a paradigm shift. Hence, we should no longer teach/present different science subjects in rather "tight compartments", as if the subjects have no relationships at all. This technique is employed and emphasized in this write-up. Geoethics as an emerging inter-disciplinary science is presented here as a science that can protect the practitioner, the society and the environment through peaceful conduct/practice of any science subject including engineering geology. This methodology requires the use of a paradigm shift that employs an acceptable Code-of-Practice as well as training programmes that can help the products to function well in a constantly changing society/world.

Geoethics and Engineering Geology

How Is It Practiced?

In Engineering Geology, and other professions as well, it has become very necessary to develop some standard to control the professional practices that are considered reliable and acceptable by informed individuals and societies the world-over. This is what brings about the terms, "code of conduct", code of practice", "best practice" and "standards". Familiar examples abound in the engineering literature (e.g. BS 1377, BS 5930). Following these ideas, we can also develop our code of practice. We can also link up with registered engineers in this country by using and possibly extending what they have already put in place. Let us not forget that all civil engineers everywhere in the world work on geological materials.

Geotechnics can be practiced through practical training of young and amateur geoscientists. This is a type of pupilage which requires inexperienced but qualified geoscientists to work closely with the more experienced specialists in their fields, so that the young ones are supervised/mentored until they can work on their own without close supervision.

Practical Applications of Geoethics in Engineering Geology

1. Use of stepwise processes/guidance to be followed: It is important to lay down certain steps which present and future geoscientists can follow. Examples include engineering-geological classification and mapping guidelines by Dearman (1974), Barton, *et al.* (1974) and Bieniawski (1974, 1980).
2. Data Collection and Interpretation: Although data collection processes are important in geotechnology, interpretation is the key (Roberts, 19770. Charts have been produced for practitioners to follow in different sub-disciplines of geology. Bieniawski (1974), Hook and Bray (1977) and Goodman (1980) have produced guidelines on data collection and interpretation. These could be further modified as part of our code of practice.
3. Technical Report Writing: Every organization has its own way of preparing reportS. These may be different from one discipline to another, depending on the project. The following basic scheme of report writing may prove useful to those who have only little previous experience of report preparation: Title page, Content Page, Introduction, Description of the project, Topography and Geology of the site, Site Investigation, Results of site investigations, Influence of ground conditions on project construction, Discussion, Other aspect of report preparation, Appendices, etc (Esu, 2017).
4. Case histories: work of past researchers on project(s) in an area of current study can usually serve quite well to guide a prospective investigator in the same area. Consultation and acknowledgement of such reports can often be helpful. Expected and unexpected issues can lead to good preparations towards combating issues as they are encountered.
5. Promotion of intra-disciplinary and inter-disciplinary measures: Both professional and anti-professional issues have been reported in many disciplines, even among geoscientific researches. Disrespect and insubordinate behaviours, are common issues which often arise from the wrong use of power and authority. Administrators, faculty staff, students, managers and co-workers may encounter all manner of inter-personal conflicts that may affect the safety, peace and productivity of their work environment, Mogk, (2017). Sexual harassment and bullying are also quite common. All these negative professional behaviours can be curbed or reduced through the introduction of more friendly measures in both intra-disciplinary and inter-disciplinary relationships. Appropriate penalties must be introduced to address unprofessional behaviour. Eventually, public trust in the work of the geoscientist will bring about more public acceptance of the geoscientist products. Hence, patronage would be achieved.

Conclusion and Recommendations

Conclusion

In the practice of engineering geology as a profession in this country, we must understand that 'the journey of a thousand miles starts with one step'. It is not too late to start the development of our profession. Geoethics as an inter-disciplinary field of study, cuts across many areas of human endeavour such as civil and mining engineering, the geosciences, sociology, philosophy and management sciences etc. Ethics, according to Socrates, simply means how we live. All branches of geosciences have ethical, social and cultural implications. Therefore, there is a need to develop an ethical framework for geoscience research and practices that can help geoscientists confronting ethical dilemmas and make them more aware of their responsibility in conducting their activities. Although geoethics spectrum is very broad, this write-up agrees with EGU 2018 General Assembly's views that important areas to consider in detail include the following: self, the geosciences, the society and the earth or the environment.

Geoscientists have a fundamental part to play in addressing many of the most urgent problems affecting our planet and its population. Their technical knowledge and expert advice are vital for informed decision-making, and for ensuring that education at all levels, equips the citizens of the 21st century so that they can participate in meaning debates on challenges. It is believed that geoscientists with greater awareness in relevant areas will be able to put their knowledge and skills to the society, to communicate such effectively and will therefore enhance public trust in the sciences (Peppoloni,2017).

As a discipline, we cannot afford to ignore the causes and consequences of interpersonal conflicts that may arise in the workplace (Mogk, 2017; Vasconcelos et al., 2015; National Academy of Sciences, 2017). Hopefully, this brief write-up will provide incentive for the geoscience practitioner, particularly engineering geologists, to be proactive in their personal and professional lives and to practice every geoscientific activity responsibly, knowing that they are being watched. Hence, we need to hand over a worthy and remarkable legacy to future generations. We must make sure all our colleagues are treated with dignity and respect that they deserve.

Recommendations

1. Geoethics education at all levels will surely create greater awareness on the importance of geoethics in the practice of the geoscience profession. Since geoethics is a relatively young area of science, more education of the society is necessary to promote pulic trust in geoscientific research, the quality of research results and products, and general trust in the relationship between practitioners, the society and environmental conservation.
2. It is very necessary that an acceptable code of practice that can guide and protect work ethics for the geoscience practitioners be provided. This will regulate and advertise the discipline adequately.
3. Proper training of the geoscientist is costly and time-consuming. Hence, responsible professionalism must be adequately rewarded through a well-negotiated and acceptable scale of fees, which must be reviewed with time.
4. Unprofessional behaviour must be punished to protect the profession. This will discourage sexual harassment, discrimination, poor interpersonal interactions between senior colleaques and their subordinates, and irresponsible use of power/authority; and respect would be earned and not forced on others.
5. The curriculum must be reviewed with time, and so geoscientists must undertake specialized training and retraining to update their knowledge and skills, knowing that today's tools and knowledge may be inadequate for tomorrow's job.
6. Conferences like this must be organized regularly to address professional issues, share knowledge and discuss ways and means for protection of present and future generations of geoscientists.

More time should be given for questions and discussion of some key issues raised in papers presented.

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Settlement Prediction in lateritic soils: Comparing Plate load, CPT and Oedometric methods of computation

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Abstract

The paper compares three methods of settlement analysis for a lateritic soil, namely the plate load, CPT and oedometric methods. Two study sites were probed, where the lateritic soil, comprising sandy silty clay, extended for about 10m and 16m was underlain by a continuous deposit of loose to medium sand. The silty - sandy clay layers at both sites classified as medium compressibility with associated moderate bearing capacity values, while groundwater tables were encountered at 8.7m and 23m respectively, below ground surface at the two sites. The Plate load test and CPT, both field based tests, were carried out within 2m radius of the corresponding borehole location to ensure that variability arising from separation was minimised. The results of settlement computation by these methods using the same incremental stress distribution indicated the Plate load test as producing the least settlement (9.98mm to 12.08mm), followed by the method based on the CPT (31mm to 42mm), while that based on laboratory oedometer evaluation of coefficient of volume compressibility produced the highest and most conservative value of settlement (104.5mm to 113.1mm). The results reflect the assumptions embedded in each method and extent of soil interactions taken into account by each method. In the case of the Plate load, the intensity of the reaction load, plate dimensions and soil compressibility near the surface largely determine the recorded settlement. The settlement computed from the oedometer results assume uniform compressibility (m_v) in the sections covered by the test although the m_v may not always be uniform over a significant depth or even the same layer. In essence, the natural variability is not taken into full account by the oedometric method. The CPT method on the other hand, accommodates the natural variability in soil compressibility across the compressible layer, and in this respect, computes a more representative settlement value.

Keywords: *Sand, Settlement, lateritic soils, plate load, CPT, Oedometric.*

Introduction

The design of every major civil structure requires information on the consolidation settlement. Consolidation Settlement is not an intrinsic soil property; rather, it reflects the response to a given load over a given area and depends not only on the deformation characteristics of the soil but also on the size of contact area between load and soil. Settlement can be predicted in a variety of ways. The most direct approach is the placement of preload and the measurement of the resultant settlement (Abam and George 2004).Other methods require field testing with Plate load test, CPT sounding and laboratory oedometer tests of representative soil samples. One of the objectives of this paper is to review these methods for determining settlement of structures founded on cohesive soil deposits in order to check the settlement criteria of foundation design. In the past five to six decades, attempts have been made to propose methods for determining more reliable values of settlement (Shukla, et al; 2013). These methods are critically reviewed in this paper in terms of their assumptions, limitations and practical use. It is found that most of the current methods attempt to fit the characteristic features of average degree of consolidation relationship from the Terzaghi consolidation theory to the observed compression versus time data obtained in the laboratory oedometer test with incremental loading. Other alternative methods such as CPT as elaborated by Sanglerat (1972), Smith and Pole (1980) and Kulhawy and Mayne (1990) are also based on rational approaches and have potential for predicting reliable values, sometimes even better than the values obtained by standard methods.

The plate loading test (PLT) which is a field test used to determine the supporting power of subgrade, subbases, bases and a complete pavement, has in recent times been used as a direct method of predicting settlement. PLT provides a direct measurement of the compressibility and bearing capacity of soil and essentially consists in loading a rigid plate and determining the settlements corresponding to each load increment. When stress is removed from a consolidated soil, the soil will rebound, regaining some of the volume it had lost in the consolidation process. The results of a PLT are presented as applied contact pressure *versus* settlement curves (Fig.1).

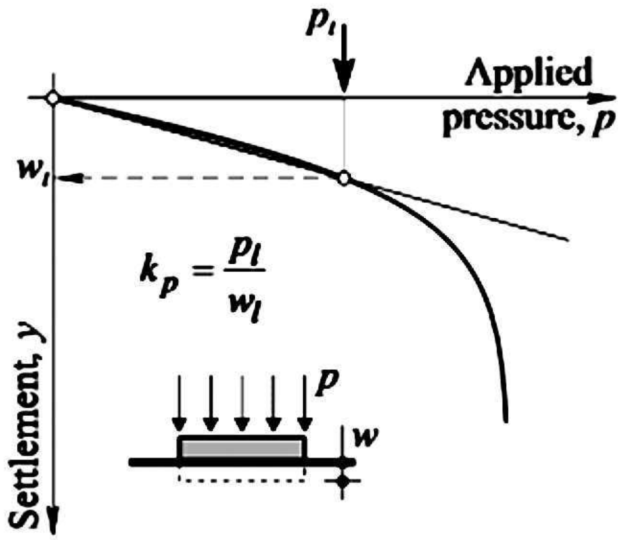

Fig. 1: Typical presentation of results from a PLT.

The interpretation of results (deformation properties) is usually made using isotropic elastic theory because of its convenience (Timoshenko and Goodie 1951; Poulous and Davies 1974). This makes it feasible for geotechnical parameters such as Young's modulus and coefficient of subgrade reaction to be derived, thus making it possible to calculate settlement. Using elastic theory, the settlement of a rigid surface plate of diameter *D*, with uniform load *p* applied on a semi-infinite isotropic soil characterised by Young's modulus *Es* and Poisson's ratio n , is given by (Lana-Bogdan and Toma 2009; Vesic 1973) :

$$w_l = \frac{\pi}{4}\frac{p_l D\left(1 - v^2\right)}{E_s} \qquad (1)$$

Where *E*s and *V* are Young's modulus and Poisson ratio respectively.

The coefficient of subgrade reaction, *ks*, is the initial slope of the curve (Fig. 1) until the limit pressure, *pl*, is reached.

The result of the **Cone Penetration test** has also been used directly to predict the possible settlement of a foundation placed on the soil tested. Prediction of the settlement is made from the summation of the vertical strains caused by the foundation load. The soil beneath the foundation is divided into layers and the coefficient of volume compressibility m_v is obtained for each layer. For clays the m_v value is obtained from the oedometer results. The method of prediction generally used is that proposed by de Beer and Martens (1957), where the Constant of compressibility, is expressed as:

$$C_s = 1.9\,\frac{qc}{Po_1} \qquad (2)$$

and fitted into the Terzaghi One-dimensional consolidation settlement equation.

Where qc = static cone resistance (kN/m^2). Po_1 = effective overburden pressure at point tested.

This means that the Total immediate settlement,

$$p_i = \frac{H}{C_s} Loge \frac{Po_2 + \Delta\sigma_z}{Po_2} \qquad (3)$$

Can be effectively re-written as:

$$p_i = \frac{HPo1}{1.9qc} Loge \frac{Po_2 + \Delta\sigma_z}{Po_2} \qquad (4)$$

Where $\Delta\sigma_z$ = vertical stress increase at the center of the consolidating layer of thickness H.
Po_2 = effective overburden pressure at the center of the layer before any excavation or load application.

Since the work of de Beer and Martens (1957), much thought has been given to the use of CPT in the evaluation of compressibility (Sanglerat, 1972; Kulhawy and Mayne 1990). The method of Sanglerat, 1972 proposed Coefficient of Volume compressibility $M_v = 1/(\alpha\ q_c)$. For low-medium compressibility soils such as those encountered in our site of investigation, where α varies from 3.6 to 6 and typically 4. Where as Kulhawy and Mayne (1990) proposed the $M_v = 1/(8.25(q_c - Po_1)$ for the evaluation Coefficient of Volume compressibility.

Where Po_1 = effective overburden pressure at point tested. Substituting M_v into $S = m_v.Ds.\ H_o$ results in both cases yields settlement estimate. The increase of vertical stress Ds at depth (Z) was based on the assumption that the stress from the foundation load (q_o) spreads out along lines with a 2 vertical to 1 horizontal slope governed by the equation:

$$\Delta\sigma = \frac{q_o \times B \times L}{(B+Z)(L+Z)} \qquad (5)$$

Where B and L are breath and length of foundation.

This vertical stress distribution is graphically represented by the Fig. (2) with the actual values predicted for the study with a load of 100kN/m^2 is presented by the side:
Where B and L are breath and length of foundation.

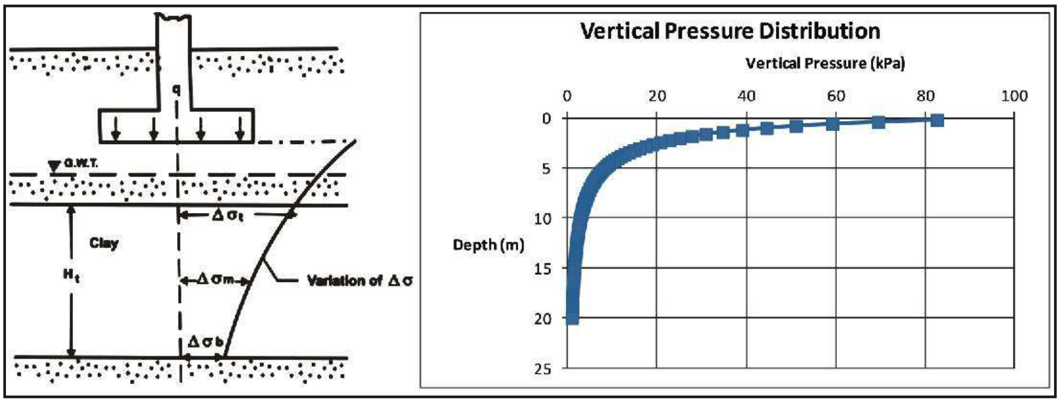

Fig. 2: Vertical stress distribution profile for foundation loading

This vertical stress distribution is graphically represented by the Fig. (2) with the actual values predicted for the study with a load of 100kN/m^2 is presented by the side:

$$S = \int \frac{\gamma dhdh}{1.9qc} \log_e \frac{\gamma(dh+\Delta\sigma)}{\gamma dh} \quad \text{.....................................(6)}$$

Methodology

Cone penetration sounding using a 2.5Ton capacity manual rig and plate load test using a Payloader as reaction load were carried out. The test was carried out at three (3No.) different positions within the first study site. Both Plate load and CPT were carried out within 2m radius of the borehole to minimize the effects of natural soil variability and ensure correlability of results from the various tests. Also, ten borings and ten CPTs were executed within 2m radius at the second study site. These tests yielded results of the parameters for the evaluation of both bearing capacity and settlements.

The Plate Bearing Test involved the loading of a circular steel plate 450mm in diameter and 25mm thick. The test was carried out after stripping off the top soil which was between 30cm and 40cm in depth at the various test positions prior to the loading of the plate. The plate was seated carefully on a leveling course of dry sand. Two dial gauges with sensitivity of 0.01mm were placed on the plate at diametrically opposite ends.

A maximum pressure of 707.92kN/m^2 was applied on the plate providing an equivalent load of 112.6kN (11.26tons). With this load, three increments were applied cumulatively to the plate. The load for each increment is recorded. The settlement under each load increment is recorded commencing with the application of the load.

The Maximum applied pressure q (kPa) beneath the plate in accordance with BS 1377: Part 9: 1990 can be evaluated by the relationship.

$$A(kPa) = \frac{W_e \times 9.81 + P}{1000/A} \quad \text{...(7)}$$

Where:

W_e = total mass of apparatus acting on the plate before adding the applied weight and including mass of the plate (kg)

P = applied forced to cause failure (kN)

A = the area of the base of the plate (m^2)

Estimation of Allowable Bearing Pressure

The allowable bearing pressure from the PLT for the study site was estimated from the relationship in which settlement of the footing of the foundation is related to the settlement of the plate:

$$S_f = Sp \cdot \frac{B}{b} \quad \text{...(8)}$$

Where:
S_f = Settlement of the footing
Sp = settlement of the plate
B = width of footing
b = width of plate

A permissible settlement of the footing of 25mm was assumed in the calculations.

For the oedometric method, undisturbed soil samples were recovered from geotechnical boring using shell and auger percussion rig, for consolidation oedometer testing. One-dimensional consolidation test was carried out under incremental loading. The Oedometer tests, yields the coefficient of volume compressibility which enables Settlement of foundation to be predicted for a net foundation load of $\Delta\sigma$ = 100kPa using the relationships:

$$\mathbf{S = m_v.\Delta\sigma. H_o} \quad (9)$$

Where

S = total settlement

Ho = thickness of compressible layer

m_v = coefficient of volume compressibility

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

Results and Discussion

The borings and CPT provided the geotechnical properties for the characterization of the sites (Table 1 and 2). The water table was 8.5m at site 1 and 23m at site 2. The moisture content at both sites are comparable, so also are the values o undrained strength.

Table 1: Geotechnical Soil properties at study site

BH No.	Depth Sample (m)	Natural Moisture Content (%)	Bulk Unit Weight γ (kN/m³)	Liquid Limit (%)	Plastic Limit (%)	Undrained Cohesion Cu (kN/m²)	Friction Angle φ(Degree)
1	2.5	20.7	20.4	39.0	18.2	80	8
	4.5	21.3	19.6	39.9	18.4	68	11
	9.0	15.6	20.4	39.2	18.1	73	22
	10.5	14.7	20.5	30.3	16.3	120	14
2	2.5	21	20.1	28.6	13.5	80	6
	4.5	19.2	20.3	26.8	13.9	72	6
	6.5	18.9	20.4	39.5	18.2	115	5
	9.0	16.5	20.2	36.2	17.3	105	11
3	1.5	20.4	20.1	38.6	19.2	68	5
	3.5	19	20.6	38.7	18.4	70.5	7
	4.5	17.8	19.8	39.5	19.1	104	6
	8.0	15.9	20.2	39.4	19.4	95	10

Table 2: Geotechnical properties and comparison of total settlement values from Oedometer Test and CPT at Assa, near Owerri, Imo State for a load o 100kPa

Borehole No.	Layer Thickness (m)	Stress Increment (kPa)	Bulk unit wt (kN/m³)	Moisture content (%)	Compressibility index	Voids Ratio	Coeff. Of volume compressibility (mv)	Coeff. Of consolidation (Cv)	Partial Consolidation Settlement (mm)	Total Settlement (mm)	CPT Values
1	12	25.00	19.2	24	0.15	0.642	0.3	66.05	90	92.45	50
	6.5	1.34	17.7	18.2	0.18	0.707	0.28	18.89	2.45		
2	11	18.60	17.3	33.5	0.23	0.877	0.37	29.13	120.47	120.47	100
3	10.5	44.44	19.1	24.5	0.2	0.633	0.22	42.82	48.89	55.21	48
	5	4.21	18.1	19.6	0.16	0.693	0.2	64.27	4.63		
		1.78	20.3	17.9	0.1	0.46	0.19	42.93	1.69		
4	11.5	47.06	17	16.4	0.17	0.729	0.2	50.6	42.35	47.63	48
	4.5	4.00	18	14.5	0.13	0.613	0.16	51.1	4.48		
		1.61	16.5	13	0.143	0.726	0.11	51.47	0.80		
5	12	44.44	15.7	14.8	0.18	0.846	0.2	83.94	44.44	52.93	49
	4.5	2.46	19.9	23.8	0.03	0.547	0.3	28.97	8.49		
6	12	38.10	17	17.8	0.18	0.754	0.33	52.05	81.71	88.74	47
	4.5	2.19	18	21.2	0.15	0.714	0.32	52.19	7.02		
7	12	20.00	18.4	20.6	0.28	0.656	0.33	6.23	105.60	105.60	85
8	11	40.00	17.7	23.4	0.12	0.778	0.15	51.31	36.00	40.25	64
	2	4.00	16.6	20	0.14	0.838	0.2	23.85	3.20		
		2.19	17	12.8	0.15	0.691	0.16	51.2	1.05		
9	12.5	50.00	17.4	25.3	0.1	0.833	0.16	67.31	32.00	40.63	50
	2	3.16	18.6	20	0.208	0.639	0.26	34.88	8.63		
10	15	36.36	18.9	27.6	0.222	0.717	0.28	53.22	71.27	79.40	46
	8	2.04	18.3	19.1	0.28	0.607	0.33	42.12	6.73		
		0.83	18.9	27.6	0.222	0.717	0.28	52.22	1.39		

The coefficients of volume compressibility and coefficients of consolidation determined from oedometer test for samples at site 1 for the different pressure ranges are presented in Table 3.

Table 3: Consolidation parameters derived from oedometer

Borehole No.	Depth (m)	Pressure Range (kPa)	Coefficient of Consolidation Cv (m^2/yr)	Coefficient of volume compressibility Mv 10^{-4}	Coefficient of Permeability K 10^{-8}cm/s
1	4.0	0-25	33.2	3.5	35
		25-50	22	3.8	22.5
		50-100	18.1	1.85	10.3
		100-200	16.2	1.22	8.5
		200-400	12.4	1.12	3.4
		400-800	10.6	0.15	0.8
2	6.0	0-25	22	3.4	25.6
		25-50	21.5	3	17.98
		50-100	18.6	2.65	16.8
		100-200	17.3	1.32	9.22
		200-400	14.2	1.34	5.6
		400-800	8.6	0.75	1.45
3	4.5	0-25	37.6	3	41.5
		25-50	35.2	2.3	25.6
		50-100	22.3	1.78	10.54
		100-200	25.1	1.32	8.94
		200-400	17.6	1.12	5.25
		400-800	14.2	0.65	2.55

The mv and Cv values at 100kN/m^2 to 200kN/m^2 pressure window were used for computation of total settlement using equation (9)

The results of PLT test on the other hand is presented as a graph of bearing pressure against settlement Figure 3. From Fig. (3), settlement values at maximum pressure were extracted and presented in Table (4).

The settlement values derived from Plate load test at a

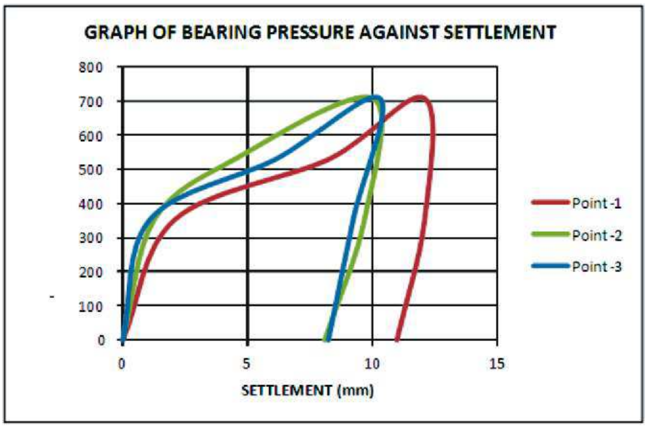

Fig. 3: Bearing pressure versus settlement graph from plate load test

Table 4: Maximum settlement from the Plate load with a load of 112.6kN

Test Point	Pressure on Plate at 112.6kN (kN/m^2)	Settlement (mm)	q(kPa)
1	707.9	12.08	112.73
2	707.9	9.98	112.73
3	707.9	10.28	112.73

loading of 112.7kPa ranged from 9.98 to 12.08mm

Results of settlement computation from the CPT test using equations (4, 5 and 6) are presented in Figure 4 for the 3 test points at site 1.

A vertical projection to the x-axis from the end of the cumulative settlement vs depth graph gives the total immediate settlement. Where the CPT has sufficient penetration, the profile can be represented as a percentage settlement, thereby enabling the contributions to total settlement at designated depths to be determined from the relationship:

$$S = So + \frac{\Sigma_{z=0} (Si + So) \times 100\%}{S_T} \quad ....(10)$$

Where S_T is Total Settlement. So is initial settlement from the first sounding interval. Si subsequent partial settlements at sounding intervals. One major application of this is in making decisions relating to depth of excavation of poor quality soils. In the past, many have been stock to the oedometer test, mainly due to lack of comparisons with field based methods of settlement evaluation. These methods were perceived to demand more data reduction efforts and were therefore considered sophisticated. Consequently, these alternative methods, in spite of their apparent

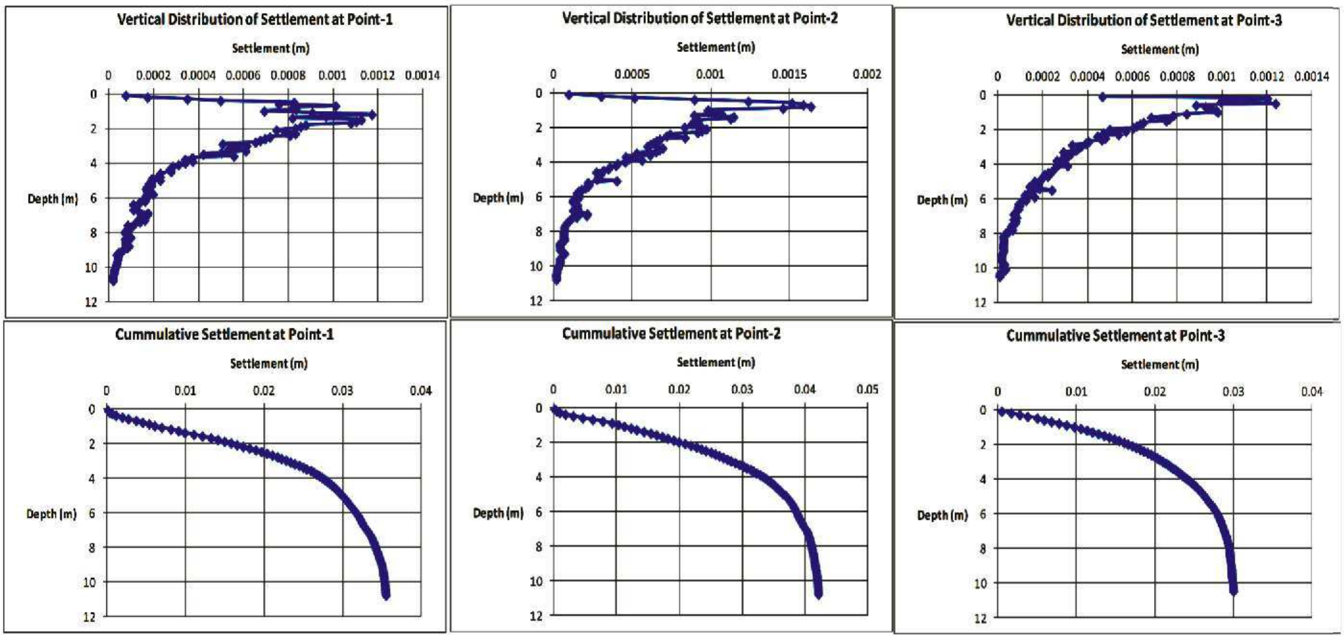

Fig. 4: Vertical distribution of Settlement with depth and cumulative settlement

advantages were considered unsuitable for routine applications.

The CPT's capacity to utilize the contributions of thin layers, (which may escape notice during drilling and sampling), some of which may account for significant fractions of the total settlement is a major advantage. In this respect, the CPT could be considered as sensitive to natural variability of soil properties. Natural variability in soil (Allen 1965) compressibility is common because of the nature of the complex sedimentation processes, especially in deltas. Variability in soil compressibility could also occur within the same layer as judged by drill logs, and as indicated by a plot of compressibility vs depth for the CPT points in site 1. (Fig. 5).

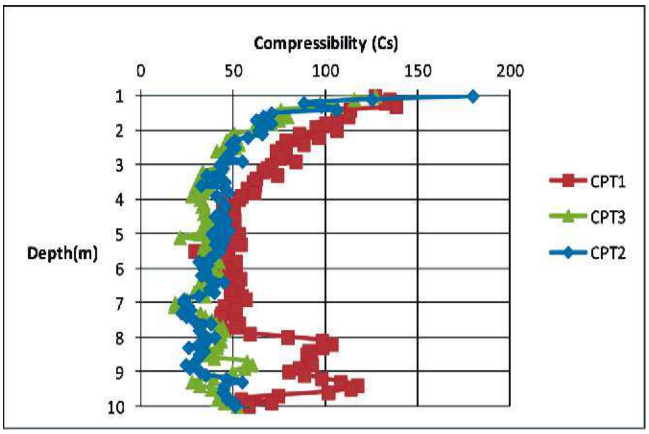

Fig. 5: Variation of Compressibility Index with depth at study site

Compressibility in this case is determined for site 2 from the equation:

$$\mathbf{Cs = 1.9qc/\sigma_v} \quad \text{...(11)}$$

Comparison of total settlement using the plate load, oedometer method and CPT, which results are as shown in Table (5) suggest that the oedometric method may be conservative, as it resulted mostly in higher values of total settlement.

Table 5: Comparison of Settlement values from different methods

BH location	Oedometric Settlement (mm)	Settlement by CPT (mm)	Settlement by Plate load Test (mm)
1	77	36	12.08
2	94	42	9.98
3	78	31	10.28

The prediction of settlement can also be done from the load-settlement curve from the test. Advantages of the Test include the understanding of foundation behavior which will enable the evaluation of foundation bearing capacity and settlement under loading condition, the test is relatively quick and easy to perform. However, it major limitations are that the Plate bearing test can give bearing capacity of subsoil only up to the depth about twice of plate diameter. This limitation is serious, because it excludes the contributions of underlying layers to settlement. Secondly, there is a scale effect due to the comparatively smaller size of testing plate in relation to the actual footing.

Among the methods available for estimating the field values of Settlement, the CPT method is preferred, mainly because of its rational and simple procedures, besides, the apparent advantages that it offers in terms of cost, comparatively short time of execution and ability to capture contributions from thin layers that would otherwise escape observation. A comparison of values obtained from the laboratory data and the field observations are required for assessing the suitability of all the proposed methods, but unfortunately it has not been reported extensively.

Conclusion

The paper compared results of settlement analysis from three methods, namely; plate load, CPT and oedometric, for a lateritic soils from two sites. The results indicated the Plate load test as producing the least settlement, followed by the method based on the CPT, while that based on laboratory oedometer was produced the most conservative value of settlement. It is concluded that the results reflect the assumptions embedded in each method and extent of soil interactions taken into account by each method. Whereas the Plate load test is affected largely by soil compressibility near the surface, the intensity of the reaction load and plate dimensions, the oedometer results assumes uniform compressibility (m_v) in the sections covered by the test although the m_v may not always be uniform over a significant depth or even the same layer. In essence, the natural variability is not taken into full account by the oedometric method. The CPT method on the other hand, accommodates the natural variability in soil compressibility across the compressible layer, and in this respect, computes a more representative settlement value.

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Environmental and Geotechnical Depiction of Foundation Soils in Deltaic Part of Southern Nigeria

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Abstract
The Meander Belt (MB) and Coastal Plain Sand (CPS) are two principal sub-environments in Niger Delta of Southern Nigeria. The influences of the nature of the two environments on geotechnical properties of foundation soils from the area have not featured in the engineering geological literature. In this study, the critical moisture content and cohesive strength in addition to chemistry and clay mineralogical composition of foundation soils in the two principal sub-environments were evaluated. Clay fractions separated from disturbed soils were analyzed for major oxides contents by the lithium melaborate method and atomic absorption spectrophotometry (AAS). Characterisation of clay minerals was conducted by X-ray diffraction (XRD) technique, and scanning electron microscopy (SEM) signals form images for minerals grain interpretation. Other disturbed and undisturbed samples were tested in the laboratory for consistency limits and cohesive strength (C_u) respectively. Results showed kaolinite (18.5%) with higher amounts of Fe_2O_3 (88.3%) and Al_2O_3 (22.7%) in the soils MB area compared to those from the CPS area. Conversely, much higher amount of SiO_2 (83.9%) is recorded in soils from the CPS than those from the MB areas. SEM revealed that quartz grains from the MB range between 5 and 100 µm, while those in the CPS area range between 20 and 100 µm. The Foundation soils from MB are silty clay of low to very high plasticity, while those from the CPS area are silty clay of medium to high plasticity with average C_u of 32.4 and 40.0 kN/m^2 respectively.

***Keywords**: Deltaic soil, kaolinite, plasticity, cohesion, foundation, sub-environment.*

Introduction

Abam and George (1997) and Abam and Okogbue (1997) identified the Meander Belt (MB) and Coastal Plain Sand (CPS), which are principal sub-environments in the Niger Delta, Southern part of Nigeria. Soils that underlain the sedimentary environments serve as foundation and construction materials but are usually problematic (Mohamad,1993; Teme, 2002).

Osammor (2009) and Adeyemi and Osammor (2000) carried out a comprehensive assessment of foundation soils in the area, and obtained a strong correlation between cone resistance and STP number of blows. Tse (2006), Onyebuolise and Akpokodje (2008), Tse and Akpokodje (2010) and Youdeowei and Nwankwoala (2011) evaluated the geotechnical characteristics of the subsoil in the deltaic environment.

However, the general engineering principles governing the design of foundations of structures are quite inadequate for the area underlain by Deltaic soils when considering the role of the principal environments on the stability of foundations.

In this study, the interaction of the major oxides, geochemistry, and clay mineralogy, critical water content and cohesive strength of the soils were considered taking cognizance of the principal environments. This is expected to help in the design of foundations serviceability limit of engineering structures whose failure could lead to huge loss of human and material resources.

Sedimentary Geology

The area under study (Figure 1) developed as a result of sediments accumulating in response to rift Atlantic Ocean during the Miocene-Eocene period (Whiteman, 1982; Obaje, 2019). The geological formations of the Niger Delta are 'Agbada and Akata' of Eocene period. The Recent in age are continuation of Imo-shale (Palaeocene) and the Ameki Formation. The topmost and youngest Benin Formation (Miocene-Recent) is the outcropping formation in the Niger Delta.
In geology and soil classification, clay are adjudged to be less than 2 µm irrespective of their mineralogy. Therefore, the ancient sedimentary processes can be related to the properties of clay-sized particles, which govern the engineering performance of soils (Avwenagha *et al*., 2014).

Methods of Study

A reconnaissance survey was carried out to delineate

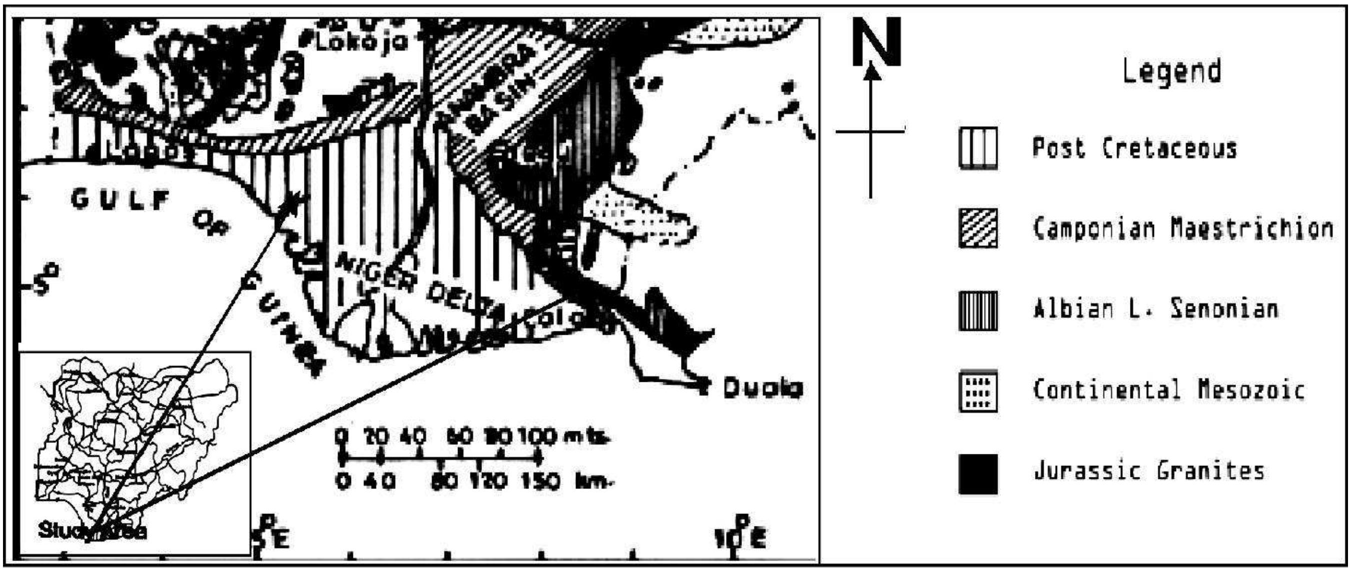

Fig. 1: Geological map of Southern Nigeria showing the Niger Delta Complex

areas that fall into the Coastal Plain Sand (CPS) and Meander Belt (MB) zones. Sixty (60) disturbed, and twenty (20) undisturbed sampling was done from isolated pad footings illustrated in Figure 2. B is the width and H is the height of excavation with water table level ranging between 0.5 to 1.2 m below the natural ground level in the study area. Abundances of major oxides in the sampled soils were determined using X-ray fluorescence (XRF) spectrometric technique under standard analytical conditions. Qualitative and quantitative estimation of respective clay minerals was determined from the X-ray Diffraction (XRD) pattern of an X-ray machine. Photomicrographs of the soils were obtained from scanning electron microscopy (SEM) along a pattern of parallel lines.

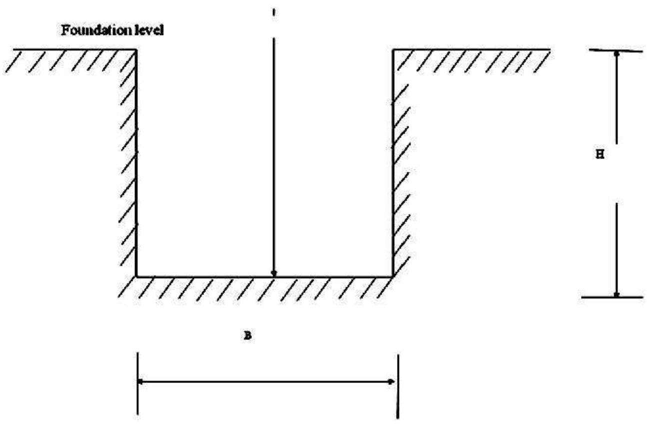

Fig. 2: Typical foundation excavation for soil sampling (B - foundation width, H - foundation height. Drawing, not to scale)

The procedures for the determination of consistency limits, as standardized by the ASTM (1985) designation-2487-92, were such that both liquid limit (L_L) and plastic limit (L_p) were determined using air-dried soil samples which have been passed through the British Standard sieve No. 40 (sieve size 0.425 mm). The difference of which is the plasticity index (I_P) employed for plotting on the Casagrande chart.

Triaxial shear strength testing were conducted in accordance with BS 5930 (1981) procedures on undisturbed samples.

Results and Discussion

From Table 1 the quantities of the major oxides in the soils from the MB contain much higher amounts of Fe_2O_3 (88.3%) and Al_2O_3 (22.7%) than those from the CPS area which contain 3.0% of Fe_2O_3 and 11.8% of Al_2O_3.

Conversely, much higher amount of SiO_2 (83.9%) is recorded in soils from the CPS as against 44.04% for those from the MB area. This shows that the iron and aluminum oxides might have been significantly washed off from the CPS than in the MBS soil which resulted in the dominance of SiO_2 in the MBS.

The X-RD patterns analyzed in Figure 3, and quantitative interpretation provided in Table 2. Kaolinite (30 – 32%) is the most abundant clay mineral

Table 1: A summary of the chemical composition of deltaic soils

ENVIRONMENT	STATISTICS	AMOUNT OF OXIDES PERCENT (%)										
		SiO_2	TiO_2	Al_2O_3	Fe_2O_3	MnO	MgO	CaO	Na_2O	K_2O	P_2O_5	SO_3
MEANDER BELT	Minimum	35.9	0.0	7.9	1.0	0.006	0.05	0.03	0.05	0.07	0.05	0.03
	Maximum	44.04	21.7	22.7	88.3	0.4	1.4	1.8	0.9	5.0	0.2	5.1
	Mean	60.4	1.4	17.8	6.5	0.2	0.9	0.5	0.7	2.5	0.1	0.3
COASTAL PLAIN SAND	Minimum	78.6	0.7	8.3	2.5	0.02	0.04	0.02	0.05	0.06	0.01	0.03
	Maximum	83.9	1.2	11.8	3.0	0.3	0.2	0.04	0.07	0.1	0.2	5.1
	Mean	80.5	1.0	10.3	2.8	0.07	0.1	0.03	0.06	0.08	0.1	0.03

in the foundation soils from both environments, while there are more illite (23%) and chlorite (4%) in the MB soil than in CPS with 2% illite without chlorite. In the CPS area, chlorite and calcite are absent with higher percentage of quartz than the soils from the MB area.

Observation of the scanning electronic microscope (SEM) shown in Figure 4 revealed that quartz grains from the MB range in size between 5 and 100µm, while those from the CPS area range between 20 and 100µm. The soils generally, contain angular grains of quartz and feldspar with variably rounded to sub-rounded detrital grains of goethite and mica. This is indicative of highly matured and weaning sediments. There are voids between the mineral grains especially those of the CPS area where they are largely unsaturated as calcite and chlorite are absent. Sediments show coated grains of clay matrix quartz overgrown with chlorite. Small quartz grains are more coated than large grains with very fine-grained sand stone consisting of quartz, and feldspars extensively coated with diagenetic clay minerals. These appear to be mostly, kaolinite and chlorite with lesser amount of illite in the samples from the MB area.

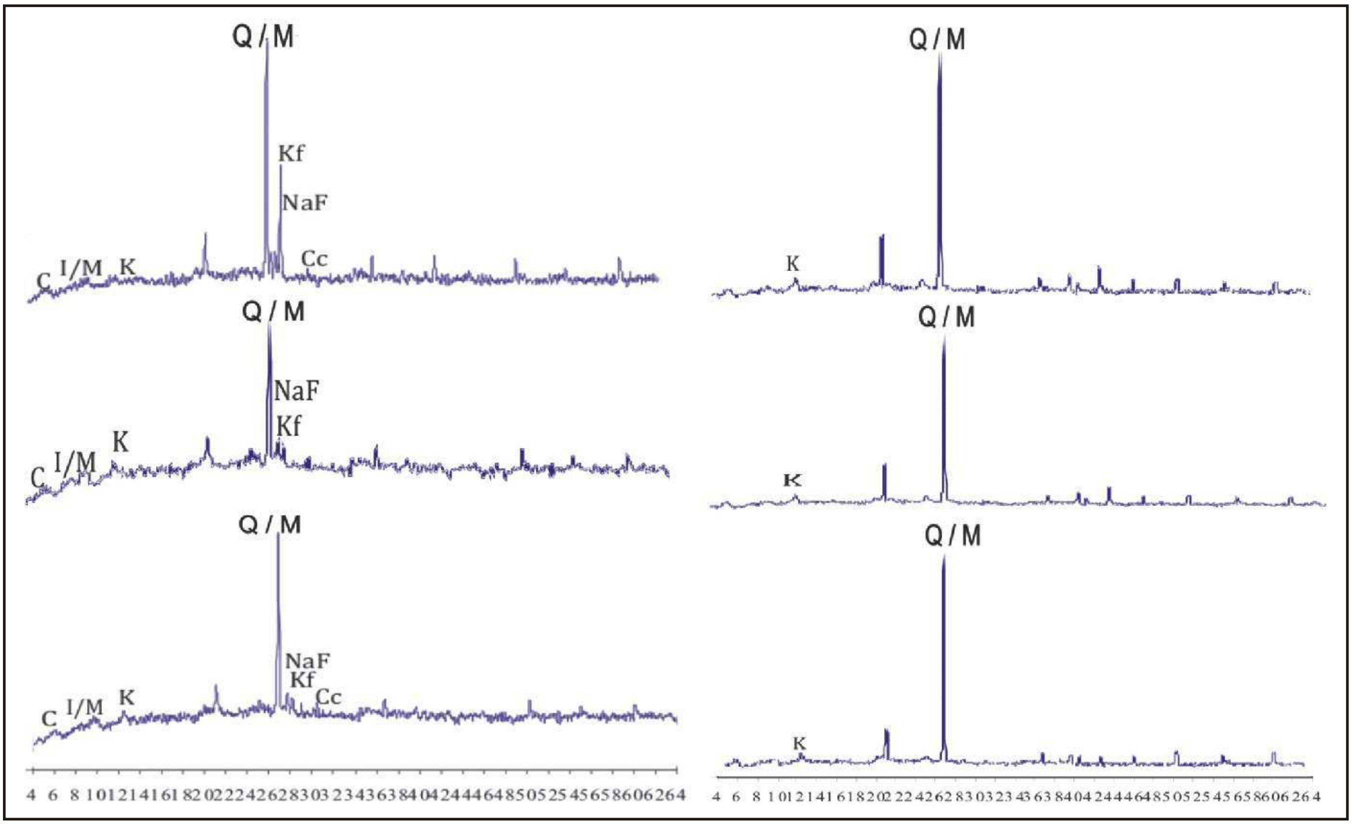

Fig. 3: X-ray diffractograms showing traces of kaolinite (K), muscovite (M), illite (I) and chlorite (C) (a – Meander Belt and b – Coastal Plain Sand)

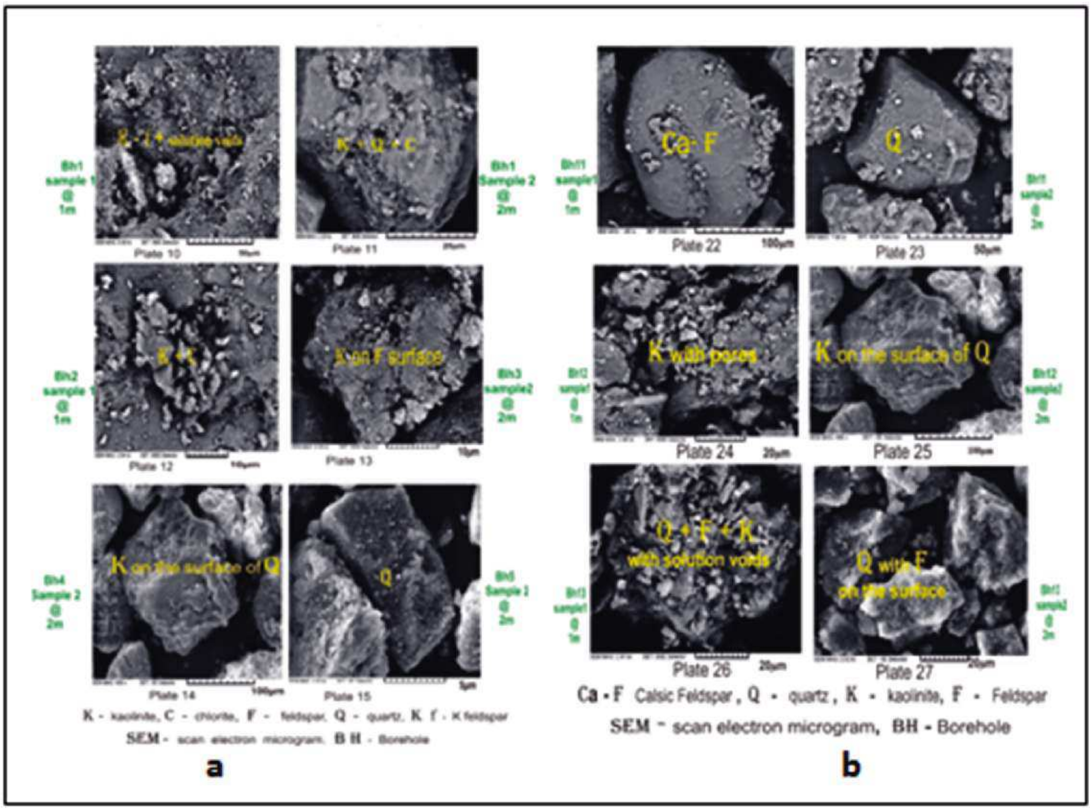

Fig. 4: SEM photomicrographs of clay from deltaic sediments. (a – Meander Belt, b – Coastal Plain Sand)

Table 2: Amount of clay and non-clay minerals (per cent)

ENVIRONMENT	STATISTICS	CLAY MINERALS PERCENT (%)							NON-CLAY MINERALS	
		Kaolinite	Chlorite	Illite	Anatase	Albite	K-feldspar	Geothite	Quartz	Calcite
MEANDER	Minimum	7	1	11	1	2	2	2	29	1
BELT	Maximum	32	4	23	3	8	7	7	56	2
	Mean	18.5	2.8	17.4	1.7	5.4	4.3	5.7	40.2	1.1
COASTAL	Minimum	20	-	1	1	1	1	2	60	-
PLAIN	Maximum	30	-	2	2	1	1	4	75	-
SAND	Mean	28.08	-	1.6	1.2	1.0	1.0	2.8	65.4	-

Soils from the MB environment have slightly higher variation (22.3%) in plasticity index (I_p) compared to the CPS area which *Ip* of 20.5% variation. This is consistent with the established distribution and geotechnical properties of Deltaic soils in parts of Southern Nigeria as reported by Okeke and Okogbue (2010). A maximum C_u of 55 kN/m^2 is recorded for clay from both the CPS and the MB areas. Clays from the MB show much higher C_u variation (48.8%) than those from CPS areas that show 24.3%. In general, average C_u values of 32.4 and 40.0kN/m^2 are recorded for foundation soils from the MB and the CPS areas respectively. On the Casagrade chart (Fig. 5), it can be confirmed that soils from MB are silty clay of low to very high plasticity, while those from the CPS area are silty clay of medium to high plasticity.

Table 3: Consistency and cohesive strength of deltaic soils

ENVIRONMENT	GEOTHECNICAL PROPERTIES	STATISTICS				
		Minimum	Maximum	Average	Standard Deviation	Coefficient of Variation (CV) %
MEANDER BELT	Plasticity Index (Ip) %	20	36	26.4	5.9	22.3
	Undrained Cohesion (C_U) kN / m^2	10	55	32.4	15.8	48.8
COASTAL PLAIN SANDS	Plasticity Index (Ip) %	10	31	22.4	4.6	20.5
	Undrained Cohesion (C_U) kN / m^2	24	55	40.0	9.7	24.3

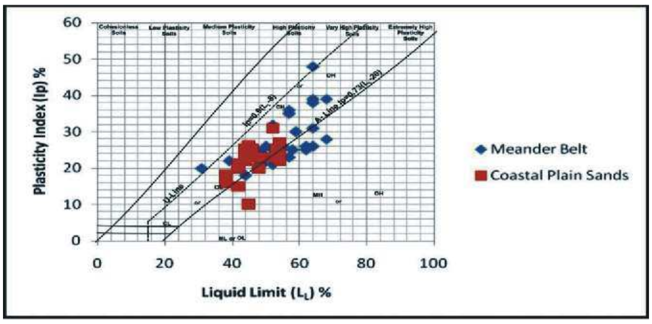

Fig. 5: Casagrade chart for soil classification in the Meander Belt and Coastal Plain Sand areas

A regression plot between kaolinite and alumina for both deltaic soils is shown in Figure 6. Fairly strong correlation (r ≈ 0.7) exists between kaolinite and alumina from both the MB and CPS areas. This implies that clay mineralogical composition increases with increase in chemical composition of the deltaic soils. A regression analysis between C_u and I_p of samples from both the MB soils and CPS areas shown in Figure 8 reveals fairly strong positive correlation (0.54) exists between cohesion and I_P of soils from the CPS area only.

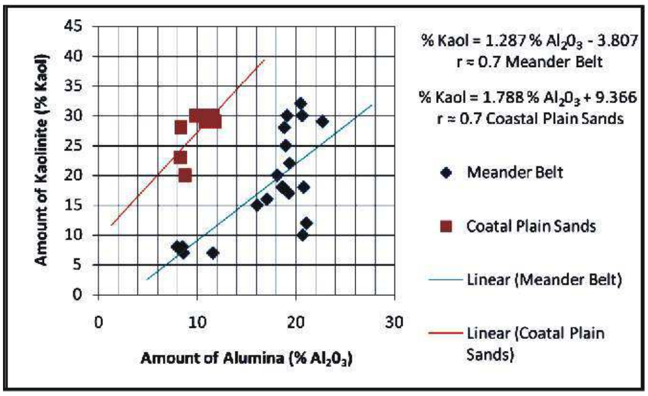

Fig. 6: Regression plot of percentage kaolinite and Al_2O_3 of the foundation soils.

A typical example of such is shown in Figure 8, and it can be observed that this failed portion is at a river across the road. Therefore, due to weak cohesive strength of the clay, the underlying soil might have been eroded over time, which had threatened the foundation of the ring culverts leading to its eventual collapse.

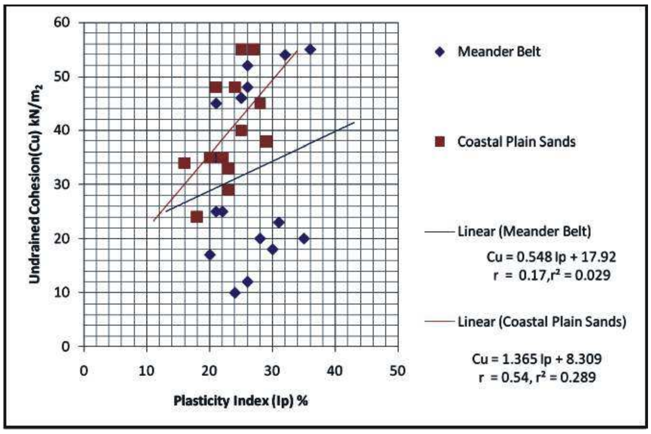

Fig. 7: Regression plots of undrained cohesion against plasticity index of the studied soils

Fig. 8: Typical pavement failure due to poor quality soil

Conclusions

The Deltaic sediments which underlain both the Meander Belt (MB) and Coastal Plain Sand (CPS) areas of Southern Nigeria serve as foundation materials in the area. Geology revealed that the topmost and youngest soil type is the Miocene – Recent Benin Formation.

Quartz grains from the MB have a wider range in size compared to those in the CPS area. Soils from the MB contain much higher amounts of Fe_2O_3 and Al_2O_3 compared to those from the CPS area. Kaolinite is the most abundant clay mineral present in foundation soils from both environments. Soils from the MB area contain more illite and chlorite soils from the CPS have few amount of illite with no chlorite, which significantly affected the plasticity index and cohesive strength of the soils. Cognizance of the sub environments and the clay properties that may affect the stability of foundation is required in executing problem-free design and construction in the Niger Delta area.

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Geotechnical Evaluation of Some Selected Crystalline Basement Complex Rocks From Part of Southwestern Nigeria

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Abstract

Geotechnical properties of rocks determine their suitability for engineering purposes. On this basis, this study was carried out to assess the suitability of some Crystalline Basement rocks in southwestern Nigeria as construction materials. The rocks evaluated were Biotite Granite, Banded Gneiss, Porphyritic Granite, and Amphibolite from four different locations. Thin sections were prepared from representative samples for petrographic analysis. Some samples were subjected to standard geotechnical tests, which include Moisture Content, Water Absorption Capacity (WAC), Specific Gravity (G_s), Aggregate Impact Value (AIV), and Aggregate Crushing Value (ACV). Regression analysis was carried out to assess the relationship between some paired geotechnical parameters of the rocks. Petrographic analysis revealed similarity in compositions of Granite, Gneiss, and Porphyritic granite with Quartz being the dominant mineral. Other minerals include Plagioclase, K-feldspar, Biotite and Muscovite. Amphibolite sample has Plagioclase, Hornblende and Quartz in that order of abundance. Result of geotechnical tests showed average aggregate crushing value (ACV) of 19.7%, 23.1%, 25.9% and 32.8% for Amphibolite, Granite, Gneiss and Porphyritic Granite respectively. Average aggregate impact value (AIV) of 11.2%, 13.1%, 18.7% and 27.0% were obtained for Amphibolite, Granite, Gneiss and Porphyritic Granite, respectively. While water absorption capacity (WAC) and moisture content values were all less than 1. Apparent specific gravity (G_s) values were greater than 2.6. The relationship between the paired parameters showed a fairly significant coefficient of correlation which is reliable enough for estimation purposes. The coefficient of variation (C_v) is lowest in specific gravity due to the similarity in the bulk composition of the samples. All the parameters tested are within the acceptance limits of standards specifications. Therefore, the selected Crystalline Basement Rocks are good for construction purposes. The order of suitability for engineering purpose was found to be Amphibolite, Granite, Gneiss, and Porphyritic Granite. The influence of texture and mineralogy is confirmed to have a profound impact on the geotechnical parameters.

***Keywords**: Geotechnical, Crystalline Basement Complex, Rocks, Southwestern, Engineering, Nigeria..*

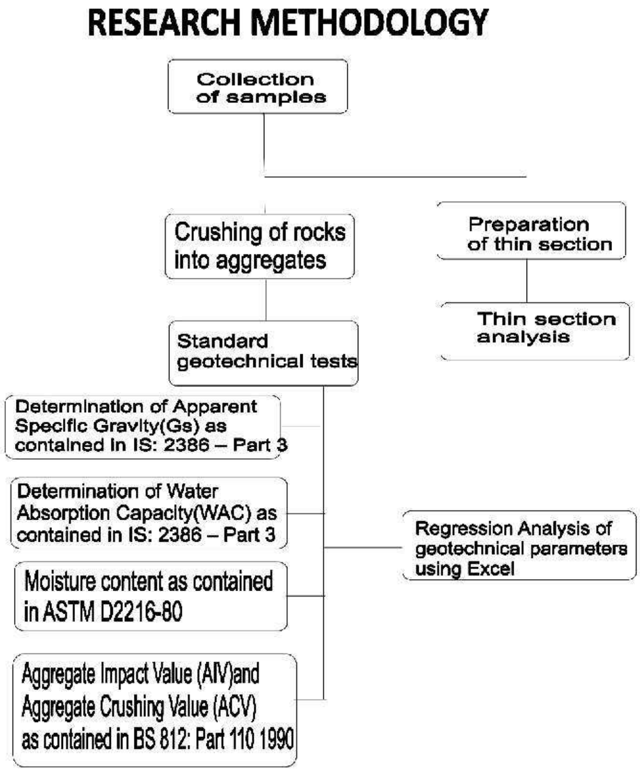
Introduction

Crystalline rocks of both igneous and metamorphic origin form the Precambrian basement complex of Nigeria. These rocks abound in southwestern Nigeria and are being quarried for construction purposes. The rocks are earth materials that are readily available for buildings and reinforcements, highways and bridges. Therefore, the rocks have gained much recognition. However, the rocks are seldomly evaluated and assessed by subjecting them to various test prior their use as construction materials.

Even though similar and generally referred to as granite, these rocks comprise of Gneisses, Amphibolite, Quartzite, Schists, and Granitic rocks (Jones and Hockey, 1964). The rocks are geologically and geotechnically different and there is need to evaluate their characteristics because quarried products (such as crushed stone) used as aggregate require certain desirable characteristics before being used in construction. In the various ways in which aggregate is used it is exposed to a variety of stresses. The response of the structure in which it is used will largely depend upon the properties of the aggregate. It needs to resist heavy loads, high impacts and severe abrasion. It needs to be durable in the prevailing environmental conditions.

The characteristics includes low moisture content, low water absorption capacity (WAC), high specific gravity, high strength, in addition to fine grained texture and little or no flaky minerals. In addition, petrography, aggregate crushing value and aggregate impact value are among the test carried out before a rock can be used for a particular purpose.

The physical property of a rock is an indication of its suitability for civil engineering works (Adebisi, 2012). Therefore, in order to have the full grasp of geotechnical properties of some selected rocks from south-western Nigeria, this research tries to employ field disposition, petrography and more importantly, some basic physical properties of the rock to elucidate further the probable geotechnical complexity that may result due to anisotropy and in homogeneity. The properties of rocks depend upon their mineral composition texture, and structure. Mineral composition controls such properties as hardness and density. Texture and structure comprise the fabric in which the individual components of the rock are arrange. These properties control the properties of the rock as a whole, such as strength, permeability, and durability.



Research Workflow

Results and Discussions

Results

The results include physico-mechanical properties of rocks from geotechnical laboratory analysis upon which regression model was derived and the modal composition of the rocks were carried out from photomicrography analysis.

Modal Composition of Rocks from Southwestern Nigeria

The average modal composition of rocks from southwestern Nigeria is shown in figure 3 while photomicrographs of the thin sections are shown in figure 2. The quality of rock aggregate depends on the presence of lithological variations in the rock deposit (Van de Wall and Ajalu, 1997) and the structural inhomogeneity.

Petrographic study of Granite, Porphyritic Granite and Gneiss showed some similarity in mineral composition, Quartz appeared to be dominant followed by K-feldspar, Plagioclase and Biotite. Muscovite is present in relatively high amounts in Porphyritic Granite compared to Granite and Gneiss. Amphibolite showed a different mineralogical composition from the photomicrograph with less Quartz and high percentage of Plagioclase and Hornblende (Fig. 2). Hornblende is present as an accessory mineral in Biotite Granite, while

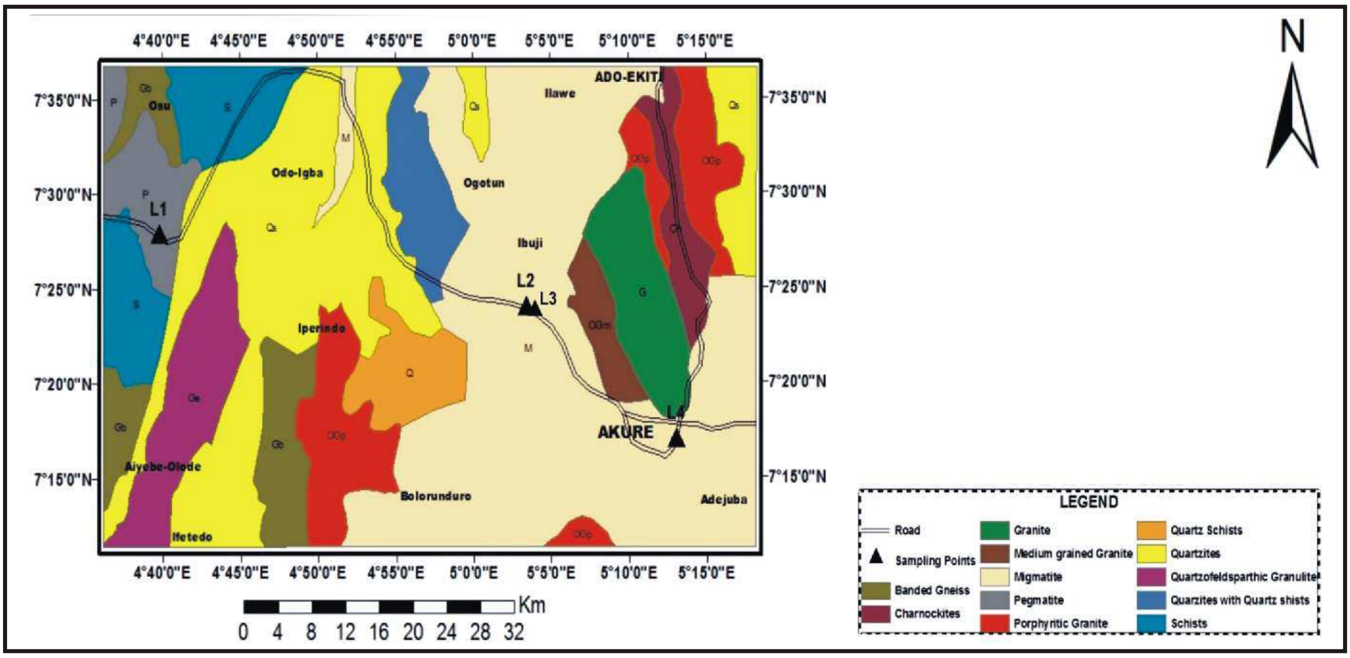


Fig. 1: Map of the study area.

Zircon and Sphene are present as accessory minerals in Porphyritic Granite.

There is little or no variation in the grain sizes of Gneiss and Biotite Granite. These rocks have medium to coarse grain texture. However, Biotite Granite and Amphibolite are massive while Gneiss is foliated. The fine grained Amphibolite is more interlocked than the Biotite Granite. Porphyritic Granite has a coarse grained mineral in finer ground mass.

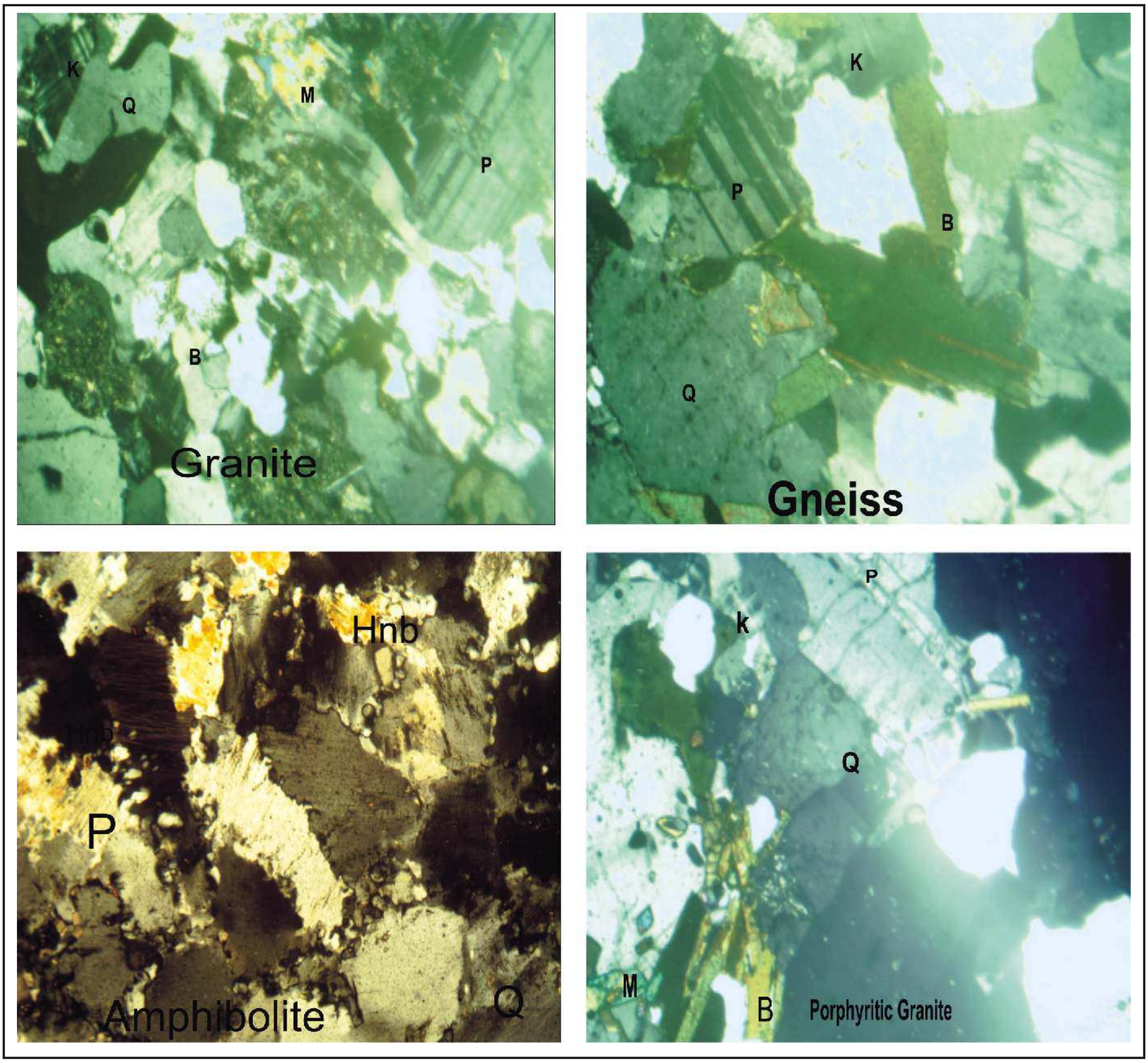


Fig. 2: Photomicrograph of the Four Rocks from Southwestern Nigeria. Magnification (X) = 40.
Biotite(B), Hornblende(Hnb), Granite(G), K-feldspar (K), Quartz (Q), plagioclase (P), and Muscovite (M).

Results of Geotechnical Tests

The Apparent Specific Gravity (Gs)

The apparent specific gravity (Gs) of all the samples tested is within the standard specification. The result ranges from 2.64 (Porphyritic Granite) to 2.71 (Amphibolite). Granite and Gneiss are both similar in composition and fall in-between with average apparent specific gravity of 2.66. Amphibolite has the highest

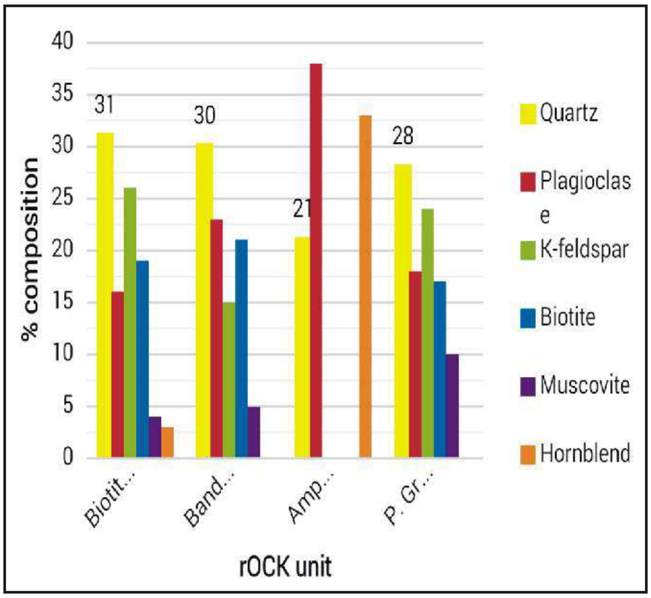


Fig. 3: Average Modal Composition of Rocks from Southwestern Nigeria.

specific gravity of 2.71 due to the mafic mineral present and closely packed mineral grains. This is due to the similarity in dominant mineralogical composition of the rocks except Amphibolite with intermediate composition.

***Water Absorption Capacity of Rock Aggregates* (WAC)**

Generally, the water absorption capacity rate for all the samples are below 1% Porphyritic Granite gives slightly high value than other rocks because porphyritic granite absorbs more water than others which may be due to the high percentage of mica and the presence of porphyry. Mohammad et al (2008; cited in Khan 2004) specified maximum limit of 2% for water absorption capacity (WAC).

Natural Moisture Content

The natural moisture content (w) of the aggregates has the highest percentage of coefficient of variation about 60%. This may be due to the nature and type of rock as layers of weakness, grain sizes and degree of interlocking of crystals as well as degree of weathering can influence the natural moisture content of any rock. Porphyritic granite has the highest M.C of 0.8%, which may be attributed to the flaky minerals, and coarse grain size of the rock when compared with Amphibolite (no mica and well interlocked), which has the lowest M.C of 0.15%.

***Rock Aggregate Impact Value* (AIV)**

The ability of a rock to resist shock or impact is recorded in percentage and the lower the value the better. Aggregate impact values and aggregate crushing values are often numerically very similar and indicate similar aggregate strength properties. Of the four rock samples analysed for aggregate impact strength, all except porphyritic granite are satisfactorily suitable for road surfacing because their AIV are less than 20%. Most roads are more susceptible to crushing than to impact. AIV of the rock aggregate ranges between 11.2% and 27.0%, which are within the acceptable limits. Porphyritic granite has a relatively poor impact value of 27.0%, which may be attributed to the presence of high mica content and the porphyry texture of the rock. The best AIV (i.e. the lowest aggregate impact value) is Amphibolite with 11.2% followed by granite with 13.1% and Gneiss with 18.7%. The IS 283-1970 code however specifies that aggregate impact value shall not exceed 45% by weight for aggregate used for concrete other than wearing surface and 30% by weight for concrete used for wearing surfaces, such as runways roads and pavement.

***Rock Aggregate Crushing Value* (ACV)**

The aggregate crushing value is a value, which indicates the ability of an aggregate to resist crushing. The lower the crushing values the stronger the aggregate and the greater its ability to resist crushing. Crushing strength is related to grain size and degree of interlocking of the finer the grain sizes. The more interlocked the grains are, the lower the crushing value. All the rock samples have aggregate crushing values less than 30%, except porphyritic granite However, the crushing value of aggregate is restricted to 30% for concrete used for roads and pavements but up to 45% may be permitted for other structures (Shetty, 2005).

The comparative study of results from analysis and reference standard specified limit, showed that the parameters fall within the acceptance limits except for ACV of porphyritic granite of 32.8% which is higher than the acceptance limit of 30% for concrete use for roads and pavements but can however support other structures. The values of the specific gravity of the aggregates are from 2.60 to 2.70. These values are within the range for the specific gravity of aggregates from rock fragments (Olanipekun, Olusola, and Ata 2006; Neville, 1995). This further buttresses the point that the aggregates can be used for construction purposes.

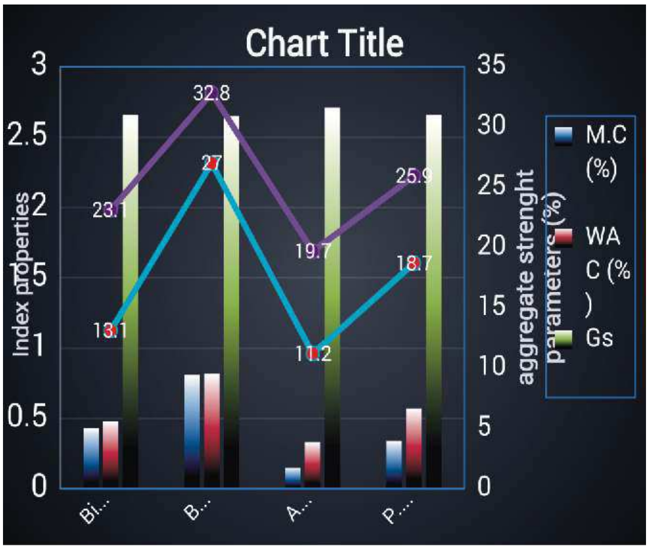


Fig. 4: Average Value of Geotechnical Parameters of Rock Aggregates from Southwestern Nigeria.

Order of Suitability of the Four Rock Types

The properties of rocks depend upon their texture, structure, and composition. Consequently, when these factors were precisely evaluated, the factors have a profound influence on the geotechnical properties of the strength of the rock aggregates (Ersoy and Waller, 1995), (Akpokodje, 1992). Mineralogically, the petrography of the four rock types revealed quartz as the dominant mineral, except in Amphibolite, with granite having the highest quartz content. Quartz has a significant influence on the geotechnical properties of rock; the higher the quartz content, the stronger the rock and the less feldspar and mica it contains, the better its suitability for construction purpose. Texturally, these rocks are medium to coarse grained except porphyritic granite; degree of interlocking of these grains is more in Amphibolite than the three rock types suggesting Amphibolite to yield higher strength than any other rock (Onodera and Asoka-Kumara,1980). Structurally, the rocks are massive except in gneiss where there is foliation.

The result of mineralogical composition favours granite but rendered slightly lower against Amphibolite due to absence of mica. Although, feldspar content is high but there is high degree of interlocking of the grains in Amphibolite than Granite. This further buttresses the fact that composition, texture and structure altogether have a great influence on the geotechnical properties of rock aggregates and hence their suitability for engineering purposes.

The order of suitability for engineering purposes is as follow:

1. Amphibolite.
2. Biotite Granite.
3. Banded Gneiss.
4. Porphyritic Granite.

Linear Regression Model Analysis

Models were developed using the experimental results. Some of these models are presented in table 1. Figure 5 shows some of the Excel package was used for the mathematical model. Model statistics and graphical plots were obtained to explain the adequacy of the linear regression models. This analysis was done at 95% confidence interval.

The coefficient of correlation (r) for ACV and AIV against WAC and M.C shows a very good relationship with values between 0.89 and 0.995 respectively. The closer the value is to 1 the higher the level of accuracy in predicting one parameter from the other. This implies that about 89% to 99.5% of the variability in ACV and AIV is accounted for by the regression model (Montgomery, peck and vining 2001). This result suggests that the developed model is adequate to explain the data. The observed levels of significance for the test on regression coefficient are all less than 0.05. Therefore, the hypothesis that the coefficient of the variables in the model should be zero is rejected because the p-value is very small (less than 0.05); suggesting that at least some of these parameters are nonzero and the terms contribute significantly to the model. Specific gravity (G_s) plot against ACV, AIV, WAC and M.C has the lowest coefficients of correlation of the parameters and are therefore negative.

Table 1: Model Developed from Simple Linear Regression Analysis and Regression Output

Parameters plotted	R^2 Value	r Value	Significance of Regression	Regression Equation (model)
ACV vs M.C	0.883	0.939	0.000520	ACV = 18.872*m.c*+ 17.235
ACV vs WAC	0.991	0.995	0.000000266	ACV = 27.287*wac* + 10.41
AIV vs WAC	0.962	0.981	0.0000179	AIV = 34.173*wac*- 1.2223
AIV vs M.C	0.794	0.891	0.00297	AIV = 22.745*m.c* + 7.7072
AIV vs Gs	0.551	-0.742	0.0351	AIV = 540.1 - 195.9G*s*
ACV vs Gs	0.678	-0.823	0.0120	ACV = 7.707 -22.745*Gs*
WAC vs M.C	0.892	0.944	0.000415	WAC = 0.692*m.c* + 0.250
WAC vs Gs	0.709	-0.842	0.00874	WAC = 17.564 - 6.379*Gs*
M.C vs Gs	0.683	-0.827	0.0114	M.C = 23.242 - 8.5517*Gs*
AIV vs ACV	0.969	0.985	0.00000903	AIV = 1.251*acv* - 14.236

Where, m.c is the moisture content, wac is the water absorption capacity, Gs is the specific gravity, AIV is the aggregate impact value, ACV is the aggregate crushing value and r is the coefficient of correlation and equals square root of R, while R is the coefficient of determination.

Linear relationship is represented by the correlation coefficient (r) between each variable and each strength as indicated. Prediction of aggregates strength, has been an active area of research and a considerable number of studies have been carried out. Many attempts have been made to obtain a suitable mathematical model which is capable of predicting strength of rock aggregates using different index parameters with acceptable level of accuracy.

From table 1, it can be seen that most variables have significant correlation with the predicted strength. This reflects in relationship between the actual value of the strength parameters obtained from the experimental work, and the predicted values obtained from the model. More than 90% of data (AIV and ACV) predicted from the plots with WAC and between ACV and AIV are located on the line of equality (line of best fit) which means that the actual and predicted values are almost identical with each other. This is true because the correlation coefficients are greater than 0.95.

Relationship between the predicted and the actual values decreases with decrease in correlation coefficients as data points shift away from the line of equality and is lowest in specific gravity plots against AIV and ACV, WAC &M.C.

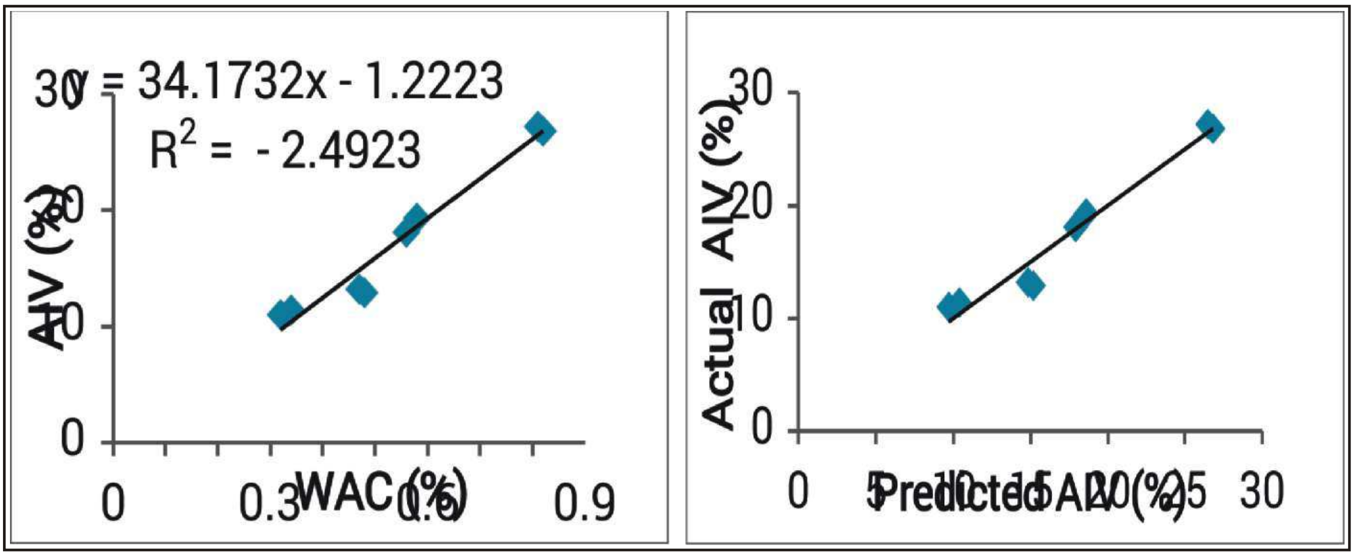


Fig. 5(a): Regression Plot of AIV against WAC and Correlation between Predicted and Experimental (Actual) AIV.

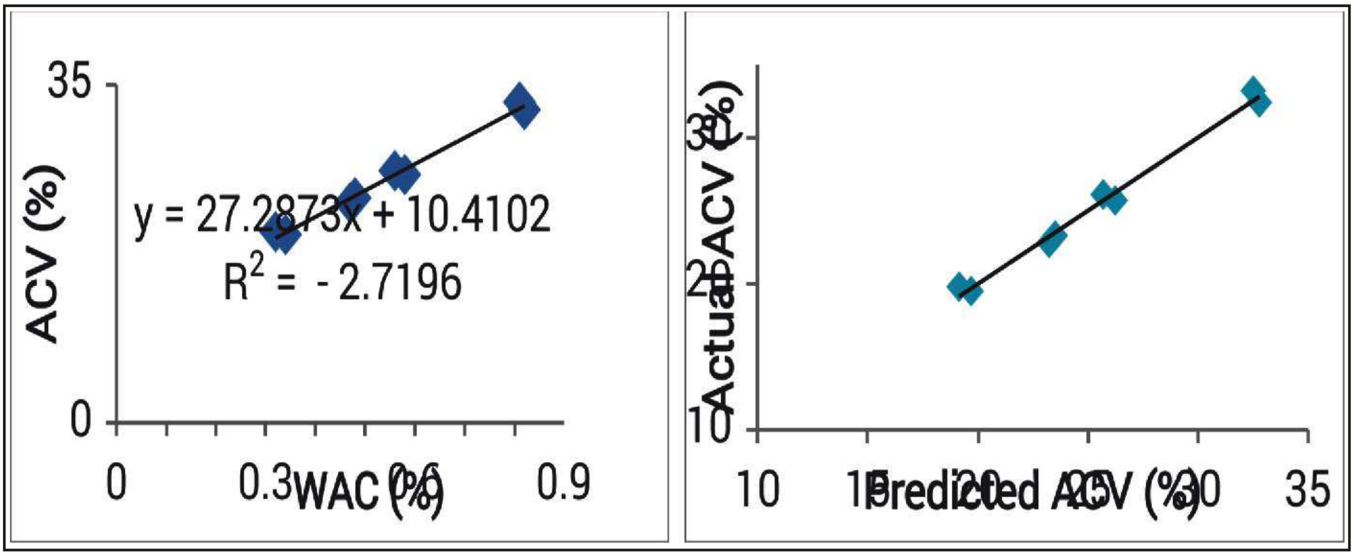


Fig. 5(b): Regression Plot of ACV against WAC and Correlation Between Predicted and Experimental (Actual) ACV.

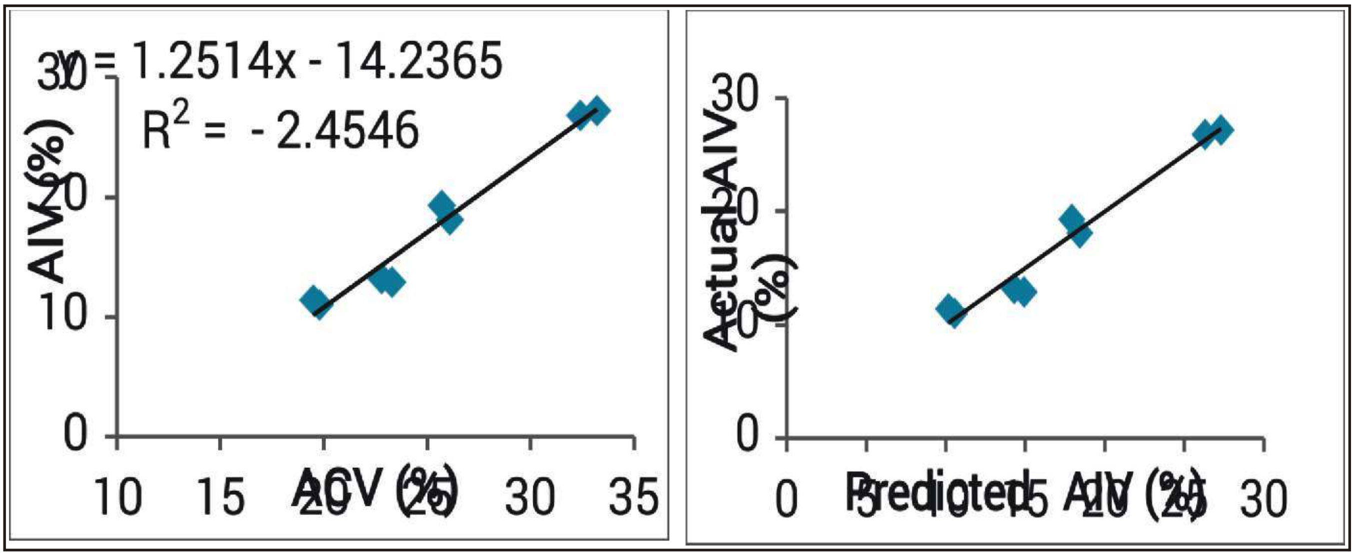


Fig. 5(c): Regression Plot of AIV against ACV and Correlation Between Predicted and Experimental (Actual) AIV.

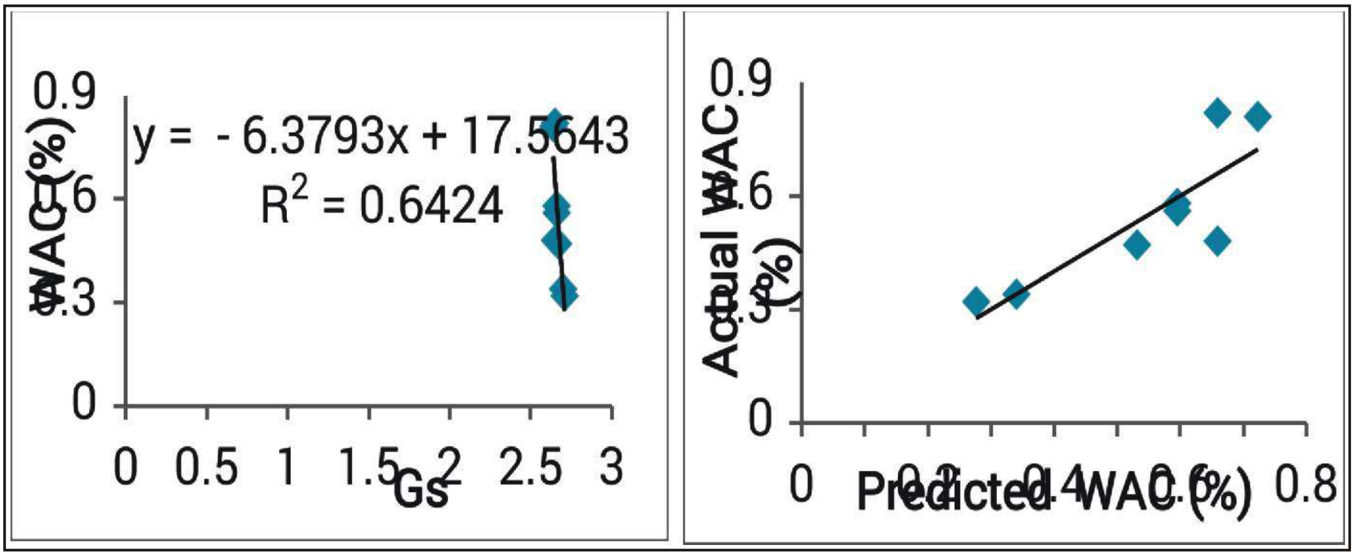


Fig. 5(d): Regression Plot of WAC against Gs and Correlation Between Predicted and Experimental (Actual) WAC.

Conclusions

The complex geotechnical problem which rocks of varying composition may pose is best assessed through petrographic study and geotechnical analysis. In order to achieve this, parameters such as moisture content, water absorption capacity, specific gravity, aggregate impact value and aggregate crushing value were determined. Petrography further helps to reveal Quartz as the dominant mineral, except in Amphibolite. In addition, Plagioclase, K-feldspar, Biotite and Muscovite are the common minerals except in Amphibolite, which is predominantly Plagioclase, Hornblende and Quartz in order of abundance.

The little variation in modal composition from petrographic analysis of the crystalline rocks reflects on the geotechnical parameters. There exists a relationship between the engineering property and the bulk composition of the rocks as well as textural differences. The results of geotechnical tests were used to evaluate the strength and suitability of the rocks for construction purposes.

All parameters tested are within the acceptance limit of AASHTO, ASTM and BS standards specification and show that the samples are good for construction purposes. However, mineral composition and texture have a great influence on the parameters; the high specific gravity of Amphibolite is attributable to the presence of Hornblende, while absence of mica and interlocking of mineral grains ensured it yielded the best result to various tests. Amphibolite is closely followed by Biotite Granite which has a medium to coarse grain size and yielded a better result over Banded Gneiss due to lack of plane of weakness, while Porphyritic Granite has the lowest strength of all the rocks. The order of suitability for engineering purpose was found to be Amphibolite, Biotite Granite, Banded Gneiss, and Porphyritic Granite. This shows greatly the influence of texture and mineralogical composition on geotechnical parameters. In order to investigate the possibility of estimating one geotechnical parameter from another, these parameters were paired and plotted. Significant correlations were generally obtained with simple linear regression equations that can be used to predict one parameter from another without actually performing the tests.

Recommendations

In order to validate this result, further geotechnical parameters such as uniaxial compressive strength (UCS), Los Angeles abrasion value (LAAV) to name but few, should be carried out. In addition, it is recommended that more samples should be analyzed to ensure confidence in the estimation of one parameter from another for decision-making purposes.

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Evaluating the Competence of Subsoil for Foundation Applications at Alagbaka Extension, Akure, Southwestern Nigeria

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Abstract

The subsoil of Alagbaka extension, Akure, was evaluated for its competence as foundation materials. A variety of twenty six (26) disturbed soil samples were collected from different pits at a depth of about 2m. The samples were analyzed for natural moisture content, grain size analysis, Atterberg limits, standard compaction, unconfined compressive strength and consolidation tests. The laboratory tests results revealed that moisture content range from 6.1% to 18.5%, liquid limit from 25.1% to 48.6%, plastic limit from 17.6% to 29.5%, linear shrinkage from 2.9% to 7.7%, and plasticity index from 5.2% to 19.5%. The grain size analysis showed that the amount of fines range from 20.6% to 47.4%. The specific gravity range from 2.65 to 2.8, maximum dry density from 1820Kg/m3 to 1998Kg/m3 and optimum moisture content from 13.2% to 19.7%. The shear strength values varied from 74.1kPa to 103.8kPa and Coefficients of consolidation range from 0.02019 to 0.02944. The superimposition of the competence maps generated from the parameters (compaction, consolidation, Atterberg limits and unconfined compressive strength) segregated the study area into extremely highly competent zone, very highly competent zone, highly competent zone and moderately competent zone. Furthermore, lithology was discovered to be closely related to soil competence.

Keywords*: Shear strength; Subsoil competence; Atterberg limits; onsolidation; Alagbaka extension.*

Introduction

Alagbaka extension in the Akure metropolis, is witnessing various structural and civil engineering developments ranging from the building of five star hotels to high rise buildings, and to road construction. The construction of structures especially high rise buildings, require an adequate knowledge of the subsurface. Information are often needed on the configuration of the subsurface layers, the nature/competence of the subsoil, the bedrock topography and its structural disposition to enable foundation to be designed and located to suit the variable character of the bedrock (Ademeso *et. al* 2016). Where construction works have been carried out without taking cognizance of the sites' geological, geomorphological and groundwater conditions, failure of foundations and by extension the super-structures often occur (Griffiths, 1999; Olorunfemi *et. al,* 2004; Olorunfemi *et. al,* 2005, Ademeso *et. al* 2016). The development of a subsoil competence map for the Alagbaka extension can therefore be a very useful initial discrimination tool for short and long term planning.

Subsoil strength investigation traditionally involves soil boring (auger or cone penetration), and soil sample testing for engineering properties such as grain size distribution, natural moisture content, specific gravity, hydrometer analysis, plasticity characteristics, bearing capacities and consolidation/compressibility characteristic determinations (Adeyemi and Oyediran, 2004).

Akure Metropolis lies within latitudes 07° 09' N and 07° 19' N and longitudes 05° 07' E and 05° 17' E (Fig. 1) while the study area (Alagbaka extension) lies within latitudes 07° 14' 12" N and 07° 14' 50" N and longitudes 05° 13' 18" E and 05° 14' 33" E covering an area extent of about 9.2 km^2. The study area is located on a gently undulating terrain.

Geology

General Geology

The Nigerian Basement Complex (NBC) lies within the remobilized zone of the West African craton. The major rock types of the NBC as classified by Adekoya *et. al* (2003) are (a) the migmatite-gneiss-quartzite complex; (b) the schist belts which are low to medium grade supracrustal and meta-igneous rocks; (c) the Pan African granitoids (Older Granites) and other related rocks such as charnockitic rocks and syenites and (d) minor felsic and mafic intrusives. Among these, the following lithologies namely (i) migmatite, (ii) gneiss, (iii) quartzite, (iv) porphyritic biotite granite, (v) charnockitic rocks and (vi) other minor rock types are represented in Akure. The rocks listed (iv) to (vi) have been found to have intruded the migmatite-gneiss-quartzite complex. The geological map of Akure as presented by Ademeso (2009) is as shown in Figure 2.

Tectonic joints have been proved to be quantitative and directional manifestations of operative forces that can give a clue to possible stress distribution in a deformed

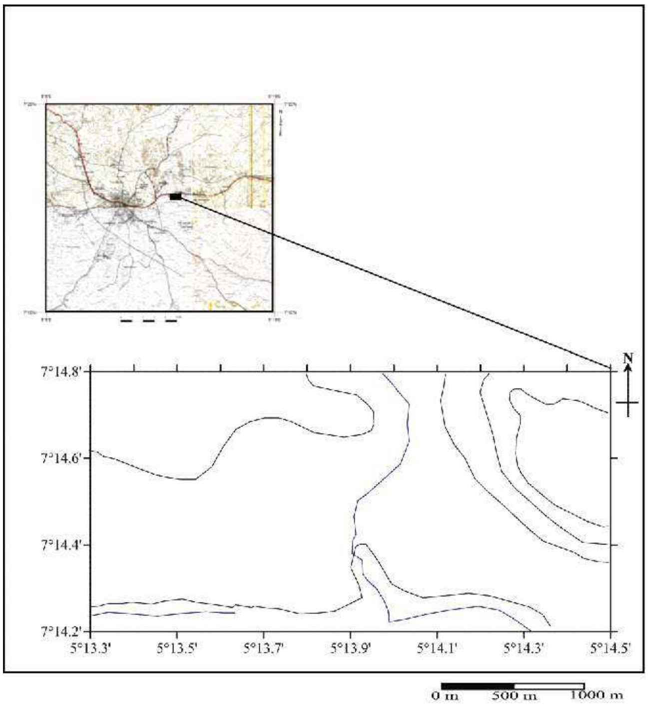

Fig. 1: Base Map of Alagbaka Extension. (Digitalized from an Excerpt of Akure S.W Topographical Map Sheet 265 [Federal Surveys Department, 1969]).

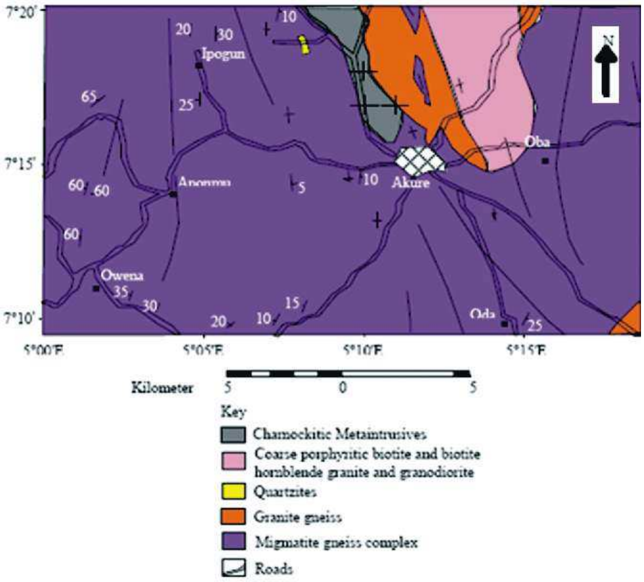

Fig. 2: Geological Map of Akure (Adapted from the Geological Map of Nigeria (Ademeso, 2009)

rock (Adekoya, 1977). The general N-S trend of the joints in the rocks of southwestern Nigeria is therefore indicative of the Pan-African orogeny. Workers that reported on the age/origin of the charnockitic rocks of the Precambrian Basement Complex (PBC) of the Southwestern and Southeastern Nigeria include Rahaman (1976), Olarewaju (1988), Ekwueme and Kroner (2006), Oyinloye and Obasi (2006), Olarewaju (2006). It is suggested that the medium-coarse-grained hypersthene-granodiorite of Eastern Hebei Province, China is the product of crystallization of anatectic magmas of the same composition. The description the three types of charnockitic rocks in Akure area on the basis of their textural characteristics as (1) coarse-grained as exemplified by the Akure body, (2) massive fine-grained which form along the margins of the granitic bodies as seen in Ijare, Uro and Edemo-Idemo and (3) the gneissic fine-grained types which were recognized within the bodies of the gneisses in Ilara and Iju was reported by Ademeso (2010). The charnockitic rocks of Akure-Ikerre-Ado Ekiti have earlier been described as an association by Olarewaju (2006). He further concluded that these charnockitic rocks were contemporaneously emplaced with the associated granitic rocks. However, considering the field and petrographic relationship of the charnockitic and the associated granitic rocks, Ademeso (2010) concluded that the charnockitic rocks of Akure were later emplacements that intruded the porpyritic granite, assimilated it thereby contaminating itself and subsequently emplacing the hybrid.

Local Geology

The geological mapping and other related studies of the area around the Akure Metropolis have been carried out by several workers amongst whom are Rahaman (1976), Olarewaju (1988, 2006). The area around the Akure Metropolis is underlain by four of the six lithological units of the Basement Complex of Southwestern Nigeria identified by Rahaman (1976). These are the migmatite-gneiss-quartzite complex, older granites, charnockitic/dioritic rocks and unmetamorphosed dolerite dykes (Fig. 2). The basement rocks exhibit varieties of structures such as foliation, folds, faults, joints and fractures. Generally, the structural trends in the study area are NNW-SSE and NNE-SSW. The geology of Alagbaka extension consists mainly of migmatite-gneiss, quartzite and porphyritic granite. The rocks generally occur as low lying outcrops except for areas underlain by the quartzites which have high reliefs.

Soils

The top soil in the study area is characterized by dark grey colour and is believed to contain organic materials. The subsoil (regolith) occurs as a result of in-situ weathering of the underlying crystalline rocks under tropical conditions. It is composed of reddish-brown loose, medium to coarse grained mineral materials with some clayey components. The soil is essentially lateritic (Fig. 3).

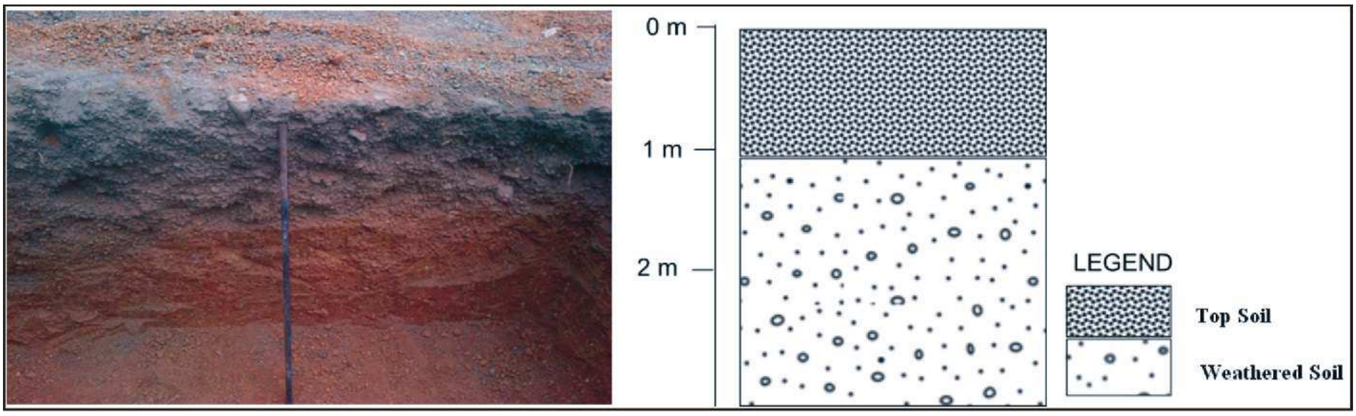

Fig. 3: Soil Profile of the Study Area.

Methodology

Soil samples were collected from twenty six (26) geo-referenced locations from the study area. The samples were analyzed for natural moisture content, specific gravity, grain size analysis, hydrometer analysis, Atterberg limits, standard compaction, consolidation test and unconfined compressive strength. The geotechnical parameters can be used as indices of subsoil competence evaluation as shown in Table 1 (Adeyemi and Oyediran, 2004).

Table 1: Ratings Adopted for Consolidation, Atterberg limits, unconfined compressive strength (CAU) (Geotechnical) Parameter (Adeyemi and Oyediran, 2004).

Parameter	Index Range	Competence Classification	Index Rating
Consolidation	> 0.15	Low Competence	0.25
	0.15-0.075	Moderate Competence	0.50
	< 0.075	High Competence	0.75
Atterberg Limits (Plasticity Index)	> 20	Low Competence	0.25
	10-20	Moderate Competence	0.50
	< 10	High Competence	0.75
Unconfined Compressive Strength	< 38	Low Competence	0.25
	38-71	Moderate Competence	0.50
	> 71	High Competence	0.75

This study therefore intends to integrate the competence maps generated from the results of compaction, plasticity index, consolidation and shear strength tests to generate subsoil competence map for Alagbaka extension. The Unified Soil Classification System (USCS) was adopted for soil classification.

Base map of the study area were pre-processed for geometric correction, haze reduction and re-sampling. The sampling points was super-imposed on the geological map of the area (Fig. 4).

Results and Discussion

Engineering Geotechnical Investigation

The moisture content obtained for the soil samples range from 6.1% to 18.5% (Table 2). The specific gravity values of the tested soils range from 2.65 to 2.80 (Table 3). An increase in specific gravity has been found to be associated with a decrease in voids ratio (Olorunfemi *et. al,* 2004, Ademeso *et. al* 2012). The tested soil samples have varying amounts of fines ranging between 26.1% and 47.4% (Table 4). These show that the soils are generally matured residual materials with sufficient binders for the coarse constituents to attain high shear strength. The curve of the plot indicates that the soils are generally well-graded (Fig. 5). The plastic limit values range from 17.6% to 29.5% while the plasticity index range from 5.2% to 19.5%. Results of consistency limit tests are as shown in Table 5. They generally fall within the values obtained for soils within the Basement Complex and can be therefore be used as foundation materials (FMWH, 1972). Since the USCS employs the grain size distribution as well as the consistency limits for the classification of soils, they are therefore classified as in Tables 4 and 5.

The OMC ranges between 13.2% and 19.7% while the MDD ranges between 1820kg/m^3 and 1995kg/m^3 (Table 6). The best soils are those with high MDD at low OMC.

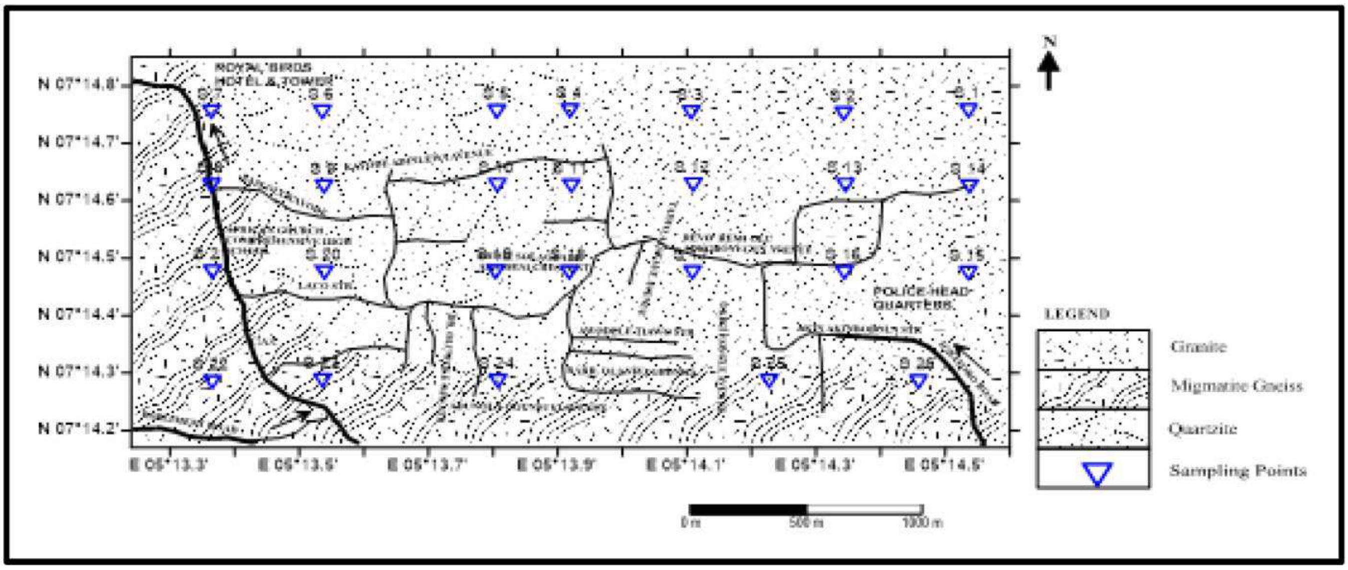

Fig. 4: Geological Map of the Study Area Showing the Soil Sampling Points

The moisture-density relationship of the soils as indicated shows that the MDD's are relatively moderate.

Table 2: Result of Moisture Content Test on the soils of the Study Area.

Sample No.	Test 1 (%)	Test 2 (%)	Average (%)
1	15.0	15.5	15.3
2	18.6	18.2	18.4
3	9.2	8.8	9.0
4	6.4	6.4	6.4
5	8.4	8.2	8.3
6	8.4	8.7	8.6
7	12.8	13.2	13.0
8	9.1	9.5	9.3
9	8.7	8.9	8.8
10	9.5	9.9	9.7
11	8.2	8.2	8.2
12	7.9	7.8	7.8
13	9.7	9.5	9.6
14	10.6	10.4	10.5
15	12.0	13.5	12.8
16	13.4	13.2	13.3
17	23.0	14	18.5
18	10.0	10.3	10.2
19	9.7	10.1	9.9
20	13.7	14.1	13.9
21	11.3	11.7	11.5
22	12.3	12.7	12.5
23	15.2	15.9	15.6
24	16.2	16.3	16.3
25	13.8	13.4	13.6
26	6.2	6.0	6.1
Mean	11.5	11.3	11.4

Table 3: Result of Specific Gravity Test on the soils of the Study Area.

Sample No.	Average Specific Gravity
1	2.66
2	2.65
3	2.66
4	2.80
5	2.75
6	2.80
7	2.65
8	2.65
9	2.75
10	2.80
11	2.75
12	2.65
13	2.65
14	2.65
15	2.65
16	2.66
17	2.65
18	2.78
19	2.75
20	2.65
21	2.65
22	2.66
23	2.66
24	2.66
25	2.65
26	2.66
Mean	2.69

Table 4: Result of Grain Size Analysis of the soils of the Study Area.

Sample No.	% Clay	% Silt	% Sand	% Gravel	USCS Soil Group
1	22.8	9.5	64.6	5.1	Clayey Sand
2	31.0	8.1	58.3	2.6	Clayey Sand
3	20.9	5.8	58.8	14.5	Clayey Sand
4	24.9	7.6	56.7	10.8	Laterite
5	24.0	9.1	56.1	10.7	Laterite
6	22.0	10.3	58.0	9.7	Laterite
7	27.6	7.4	47.9	3.3	Clayey Sand
8	38.7	8.7	51.0	1.7	Clayey Sand
9	21.7	10.4	58.6	9.3	Laterite
10	23.4	9.6	60.4	6.7	Laterite
11	23.9	8.5	58.5	9.2	Laterite
12	18.3	7.7	59.5	14.5	Clayey Sand
13	27.4	11.8	49.9	10.9	Clayey Sand
14	19.9	7.9	60.8	11.4	Clayey Sand
15	20.6	8.0	57.2	10.6	Clayey Sand
16	25.5	8.1	52.5	9.2	Clayey Sand
17	20.6	8.0	57.2	3.9	Clayey Sand
18	24.5	9.2	56.6	9.8	Laterite
19	22.4	7.9	60.8	8.9	Laterite
20	38.5	7.0	51.3	3.2	Clayey Sand
21	39.6	7.0	51.3	2.0	Clayey Sand
22	30.1	9.0	57.0	3.4	Clayey Sand
23	36.2	11.2	50.1	3.9	Clayey Sand
24	35.6	11.4	51.0	4.1	Clayey Sand
25	28.8	9.8	46.7	5.6	Clayey Sand
26	27.4	11.8	49.9	10.9	Clayey Sand

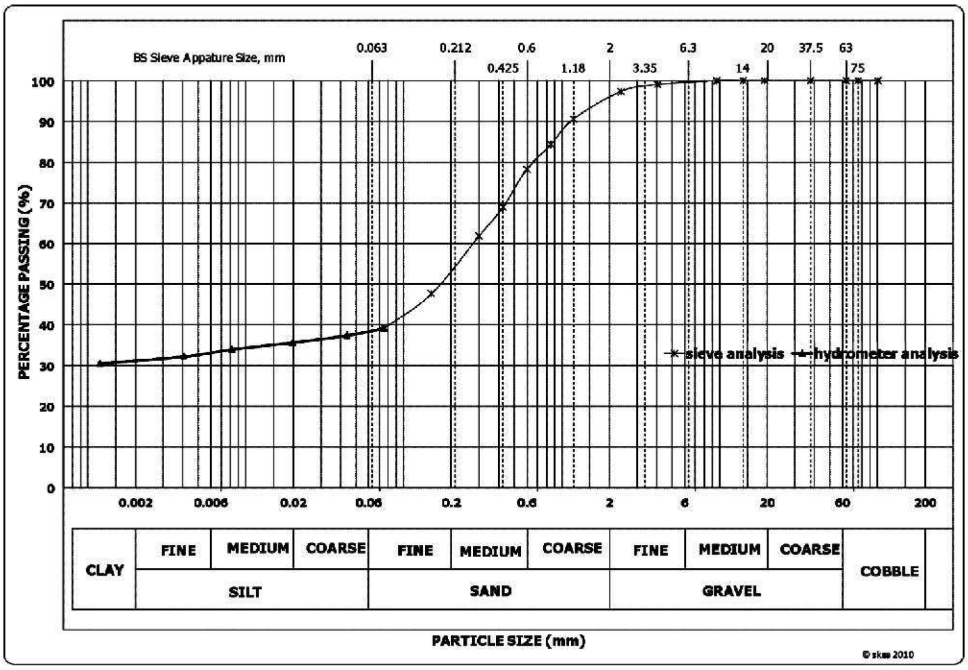

Fig. 5: Grain Size Distribution Curve of the soils of the Study Area.

Table 5: Results of Consistency Limit tests of the Study Area.

Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Linear Shrinkage	USCS Group Symbol
1	32.1	20.8	11.3	5.3	CI
2	35.6	24.4	11.2	5.2	CI
3	25.2	17.6	7.6	4.2	CL
4	27.9	22.7	5.2	3.9	CL
5	30.0	23.2	6.8	2.9	CL
6	28.7	22.0	6.7	3.6	CL
7	35.2	24.9	10.3	5.1	CI
8	47.6	29.5	18.1	7.5	CI
9	28.9	23.9	7.0	3.1	CL
10	29.5	23.1	6.4	3.1	CL
11	31.2	23.7	8.5	4.6	CL
12	25.1	17.8	7.3	4.1	CL
13	37.2	26.1	11.1	5.2	CI
14	26.2	18.3	7.9	4.3	CL
15	29.3	21.2	8.1	4.5	CL
16	31.8	23.1	8.7	4.6	CL
17	48.6	29.1	19.5	7.7	CI
18	30.7	23.6	7.1	3.5	CL
19	29.7	21.5	8.2	4.5	CL
20	48.1	29.0	19.1	7.7	CI
21	48.1	28.9	19.2	7.7	CI
22	38.6	26.9	11.7	5.4	CI
23	47.1	29.0	18.1	7.5	CI
24	46.6	29.1	17.3	7.5	CI
25	36.6	24.0	12.6	5.3	CI
26	28.3	20.3	8.0	4.5	CL

Table 6: Compaction Characteristics of the Soil Samples of the Study Area.

Sample No.	Maximum Dry Density (Kg/m^3)	Optimum Moisture Content (%)
1	1857	13.2
2	1868	19.2
3	1899	18.2
4	1989	15.3
5	1986	15.4
6	1998	15.0
7	1820	14.0
8	1853	19.7
9	1995	15.1
10	1977	15.7
11	1971	15.9
12	1909	17.9
13	1905	18.0
14	1856	19.6
15	1915	17.7
16	1896	18.3
17	1859	19.5
18	1974	15.8
19	1967	16.0
20	1840	20.1
21	1918	17.6
22	1905	18.0
23	1902	18.1
24	1893	18.4
25	1881	18.8
26	1865	19.3

purposes. It is hereby concluded that the subsoil of the area have been found to be generally competent with the competence map revealing a zonation as in Fig. 11. Furthermore, the zones tend to show an alignment with the lithology as the moderately competent zone tend to fall in the area underlane by migmatitic rocks, highly competent fall in area underlane by granitic rocks while the very highly competent and extremely highly competent fall in the area underlane by the quartzites (Fig. 10 & 11).

Table 7: Results of Consolidation Tests of the Study Area.

Sample No.	Coefficient of Compressibility (A_v)	Coefficient of Vol. Compressibility (M_v) (MPa^{-1})	Compression Index (C_c)	Swelling Index (C_s)	Coefficient of Consolidation (C_v) (m^2/min)
1	0.33648	0.24911	0.0456	0.01919	0.02684
2	0.51812	0.32256	0.0699	0.03159	0.02413
3	0.29904	0.18978	0.0409	0.01543	0.02919
4	0.37069	0.25124	0.0503	0.02119	0.02676
5	0.36620	0.24804	0.0497	0.02086	0.02688
6	0.36468	0.24911	0.0495	0.02080	0.02684
7	0.38034	0.27808	0.0515	0.02238	0.02575
8	0.69695	0.43127	0.0936	0.04484	0.02051
9	0.36234	0.24697	0.0492	0.02061	0.02692
10	0.38016	0.25552	0.0515	0.02184	0.02660
11	0.37180	0.24911	0.0504	0.02120	0.02684
12	0.28722	0.18349	0.0393	0.01460	0.02944
13	0.49869	0.31929	0.0673	0.03034	0.02425
14	0.32693	0.20135	0.0446	0.01730	0.02872
15	0.32454	0.20874	0.0442	0.01742	0.02842
16	0.41274	0.26195	0.0559	0.02389	0.02635
17	0.70993	0.44142	0.0953	0.04584	0.02019
18	0.39359	0.26410	0.0533	0.02283	0.02627
19	0.34087	0.22780	0.0463	0.01889	0.02767
20	0.69511	0.42676	0.0934	0.04465	0.02065
21	0.65582	0.42564	0.0881	0.04211	0.02068
22	0.50331	0.32256	0.0679	0.03069	0.02413
23	0.64421	0.41329	0.0866	0.04117	0.02108
24	0.64364	0.40992	0.0865	0.04108	0.02119
25	0.50448	0.31711	0.0681	0.03064	0.02433
26	0.52418	0.32365	0.0707	0.03198	0.02410

Table 8: Unconfined Comprehensive and Shear Strengths Result of the Study Area.

Sample No.	UCS (kPa)	Shear Strength (kPa)
1	169.8	84.9
2	174.1	87.0
3	161.3	80.7
4	165.6	82.8
5	186.4	93.2
6	207.1	103.6
7	190.8	95.4
8	160.6	80.3
9	199.1	99.6
10	203.3	101.7
11	207.6	103.8
12	199.1	99.6
13	186.4	93.2
14	178.3	89.2
15	174.1	87.0
16	169.8	84.9
17	152.8	76.4
18	198.7	99.3
19	202.9	101.4
20	160.6	80.3
21	156.4	78.2
22	156.7	78.4
23	148.3	74.1
24	161.0	80.5
25	165.2	82.6
26	165.2	82.6

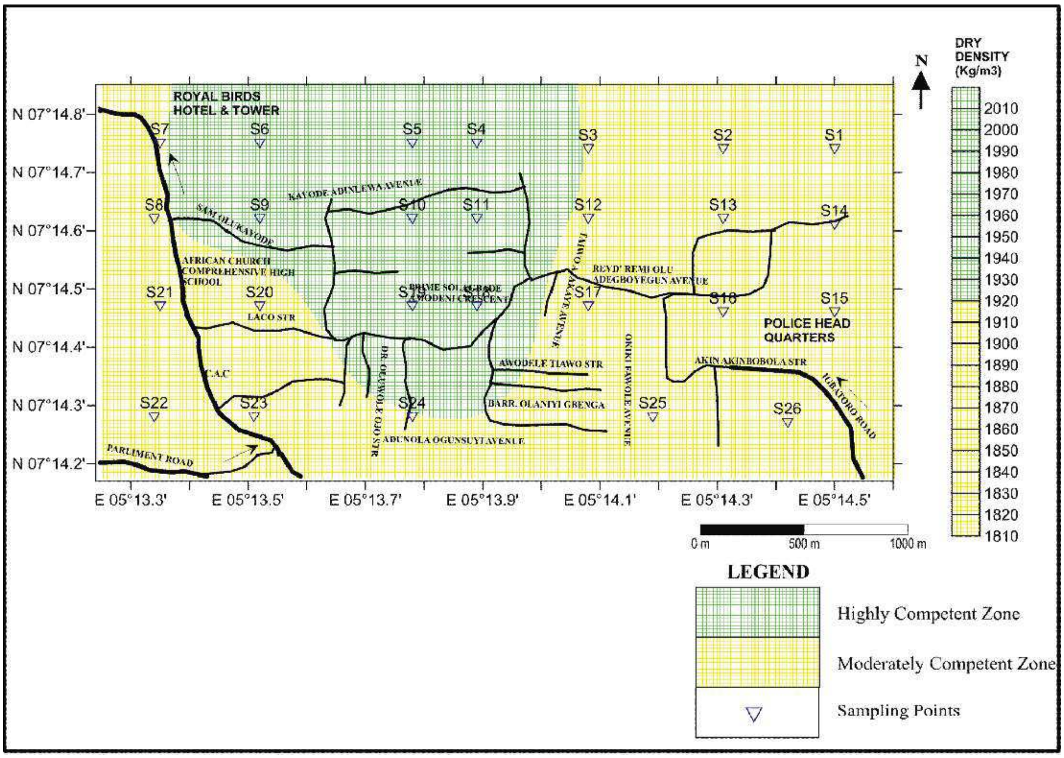

Fig. 6: Competence Map Generated from Maximum Dry Density Values of the Study Area.

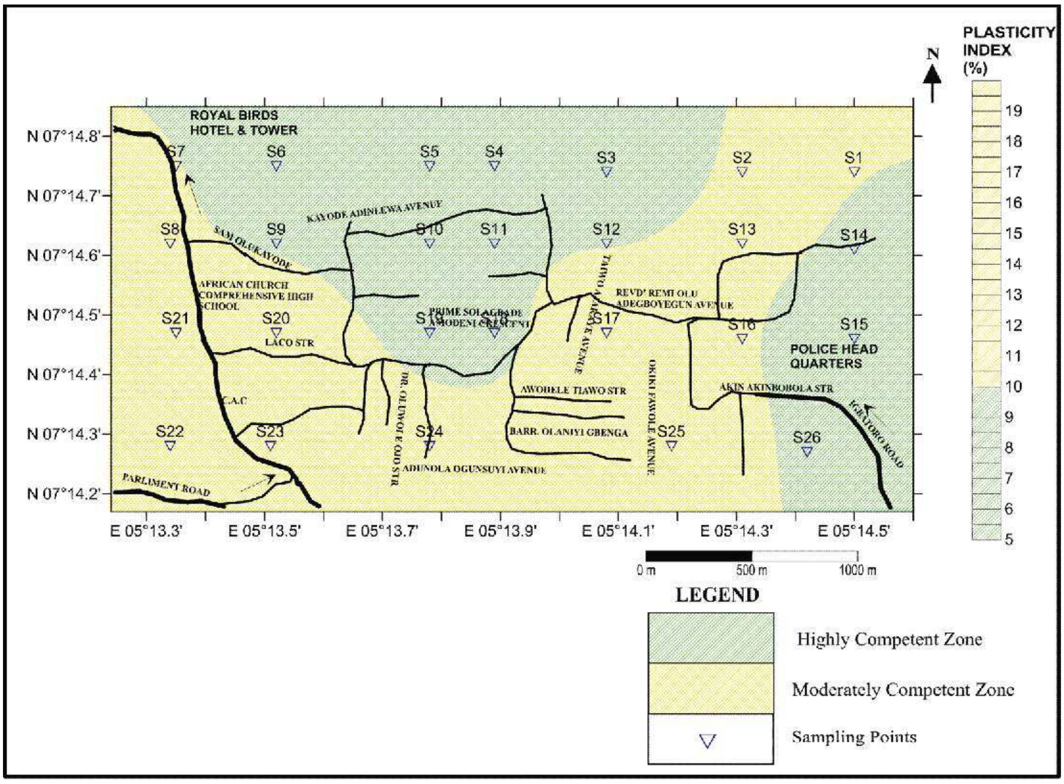

Fig. 7: Competence Map Generated from Plasticity Index Values of the Study Area.

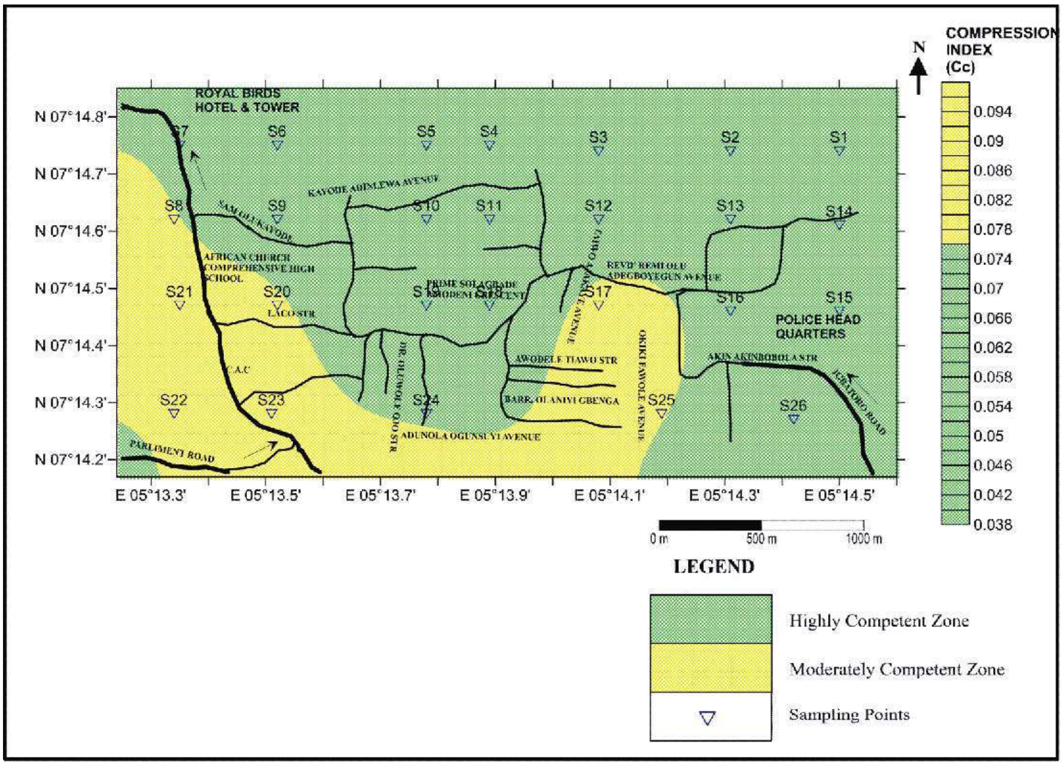

Fig. 8: Competence Map Generated from Compression Index Values of the Study Area.

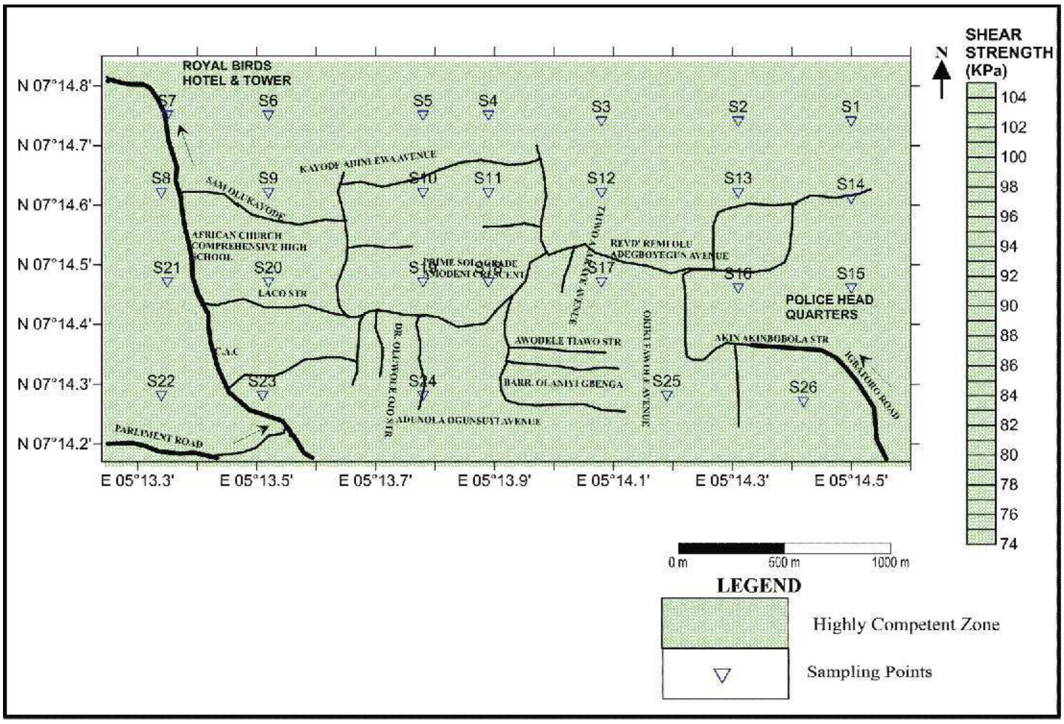

Fig. 9: Competence Map Generated from Shear Strength Values of the Study Area.

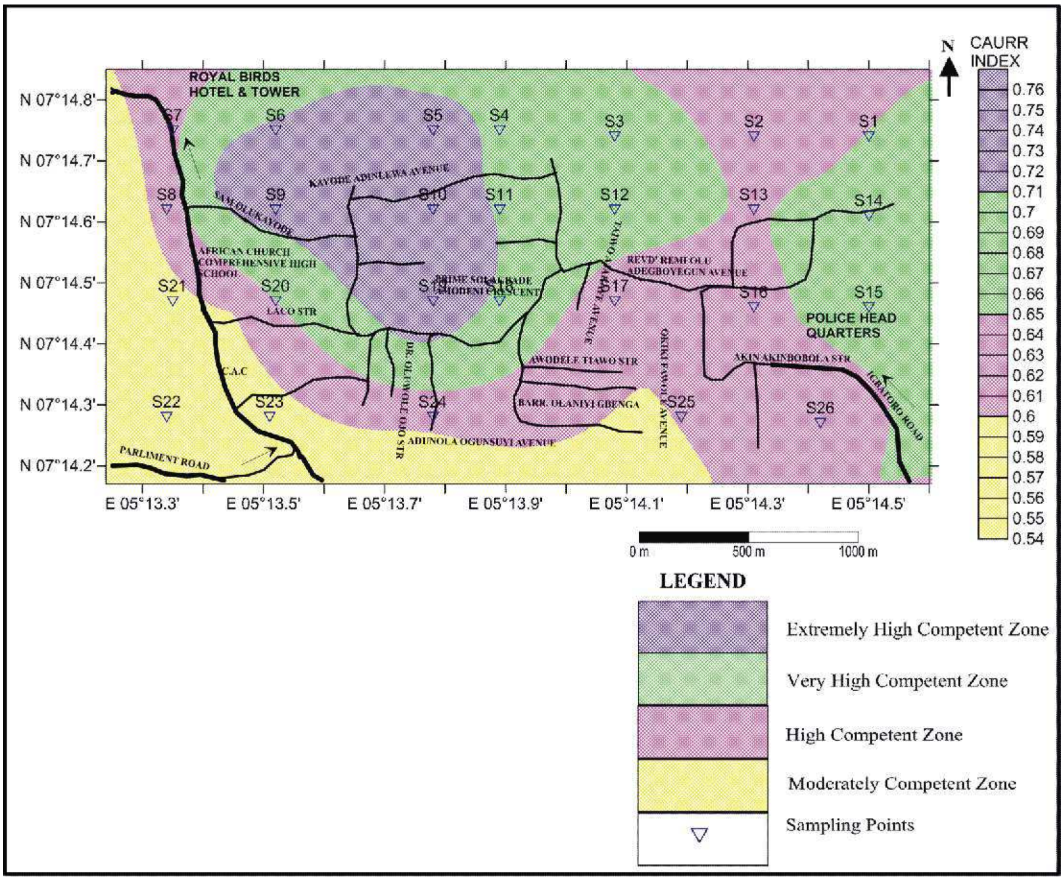

Fig. 10: Competence Layer Map of the Study Area

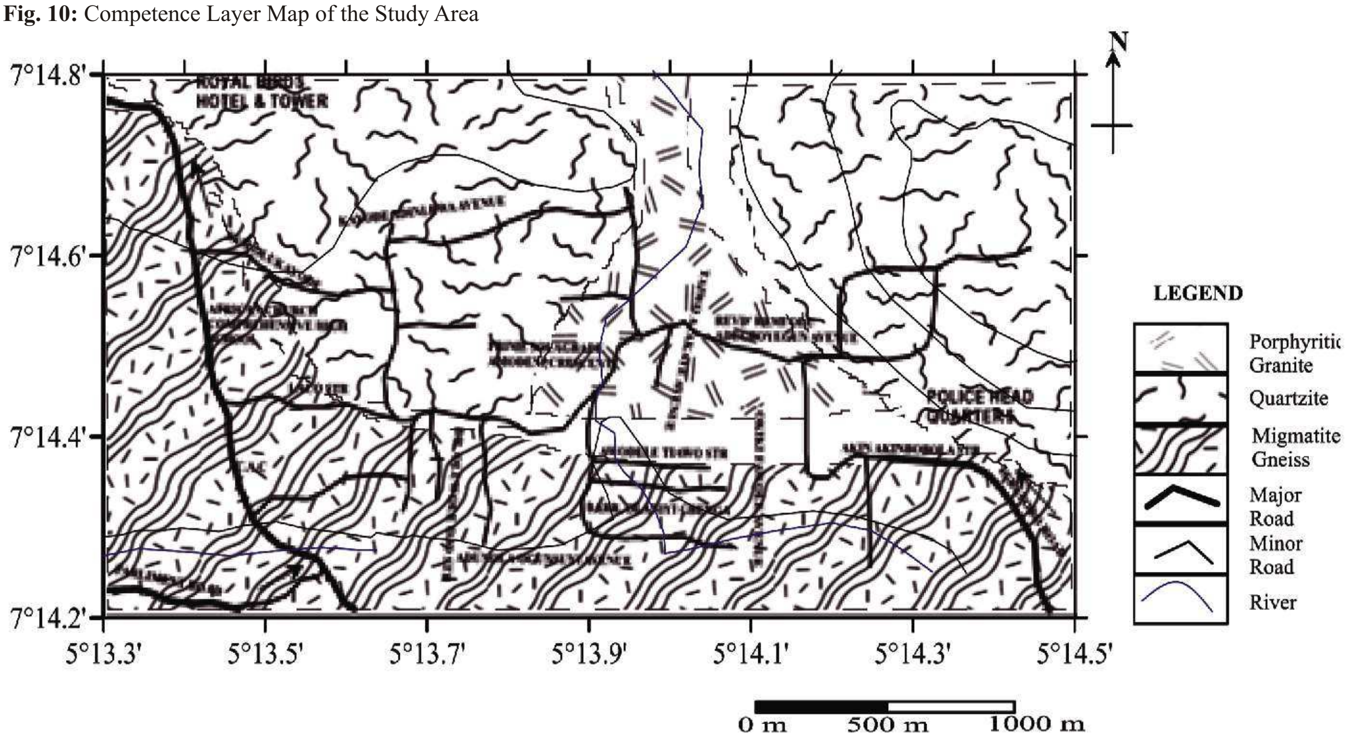

Fig. 11: Geological Map of the Study Area.

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Subgrade Strength Investigation Using Vertical Electrical Sounding

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Abstract

Near surface geophysics as a potential tool is gaining ground in engineering sub-surface investigation for project designs. Twenty six (26) soil samples were collected along Okpella-Auchi-Igarra-Ibillo-OkeneRoad Network in southern part of Nigeria. Vertical Electrical Soundings (VES) were carried out on respective sample collection points. Road sections with California Bearing Ratio (CBR) ranging between 9 to 32 and plasticity index of 0 to 15 are having subgrade strength between S4 to S6. Resistivity values at these stations ranges between 47ohm-m to 1120 ohm-m and classified as "good" to "excellent" subgrades. The geoelectrical curve-types in these stations are mainly A-type and Q-type. Sections of the road with subgrade strength of S1 to S3 show a "poor" to "fair" subgrades. These stations are characterised by low CBR of 1 to 8 and intermediate plasticity index between 0 to 42. Their resistivity at 1m depth ranges between 27ohm-m to 832ohm-m. There is a relationship between VES curve-types and subgrade strengths in the Basement Complex terrain of the study area. It is noteworthy to state that failed portion of the roads investigated have poor to fair subgrades strength with most of their geo-electrical sections revealing K-curve and H-curve respectively. On the other hand, the stable portions of the roads have good to excellent subgrade strength with A-curve and Q-curve types respectively.

***Keywords**: Subgrade strength, VES, GeoelectricCurve-type, Resistivity, California Bearing Ratio*

Introduction

A lot of reasons have been suggested for the incessant failure of roads in Nigeria. These include presence of expansive clays such as montmorillonite, chlorite,halloysite, etc. (Mesida 1987), heterogeneity of the subgrade materials (Adeleye, 2005; Mesida, 1987), presence of undetected linear features, such as joints, fractures and rock boundaries. In the past two decades, the field of geophysics has proven quite relevant in highway site investigations (Nelson and Haigh, 1990). However, in an attempt to unravel causes of persistent failure of roads across the country, various researchers have identified chiefly the underlying geologic conditions among other factors to be responsible for this mishap (Momoh et al., 2008; Oladapo et al., 2008; Adiat et al., 2009). It therefore becomes imperative to investigate the subsurface geology upon which a road structure is to be founded rather than having recourse to a post-construction investigation and remedies.Fahad and Syed (2012) admitted that the conventional methods for determination of engineering properties are invasive, costly and time-consuming. Electrical resistivity survey is an attractive tool for delineating subsurface properties without soil disturbance. Reliable correlations between electrical resistivity and other soil properties will enable us to characterize the subsurface soil without borehole sampling. This paper presents the preliminary results of an ongoing research on prediction of subgrade strength using geoelectrical information, for road design in a tropical environment. Soil investigation and field electrical resistivity survey (VES) were conductedalong Auchi - Igarra - Ibilo - OkeneRoad networks in Edo and KogiStates, Southern Nigeria. From the data analysis, we observed significant correlations between resistivity and subgrade strength in the sedimentary sections of the pavements investigated. There is also relationship between geoelectric curve type and soil strength in the basement and sedimentary sections of the area of investigation.

Location and Geology of the Study Area

The study area extends from longitude 5°55'E to 6°12'E and latitude 7°10'N to 7°28'N, summing a total area of 1,848km^2. It is located between Edo and Kogi States, southern Nigeria. The road networks of interest cuts across Auchi - Okpella-Okene road. Thisis an important road which links the oil rich Niger Delta to the Federal Capital Territory. Contract for the dualisation of the road has been awarded by the Federal Government of Nigeria. Auchi-Okenesegment A is about 68km long (Fig. 1), profiling NS and its parallel to River Niger with an offset of 40km.

The Geology of the study area is divided into two parts; The Northern Precambrian crystalline Basement complex and the Southern Cretaceous to Tertiary sedimentary rocks of the Niger Delta Basin (Fig. 2). The Northern part consists of thePan-African Older Granite, Meta-Sediment, Meta-Volcanic and Migmatite Gneiss complex. The southern region consists of the mid

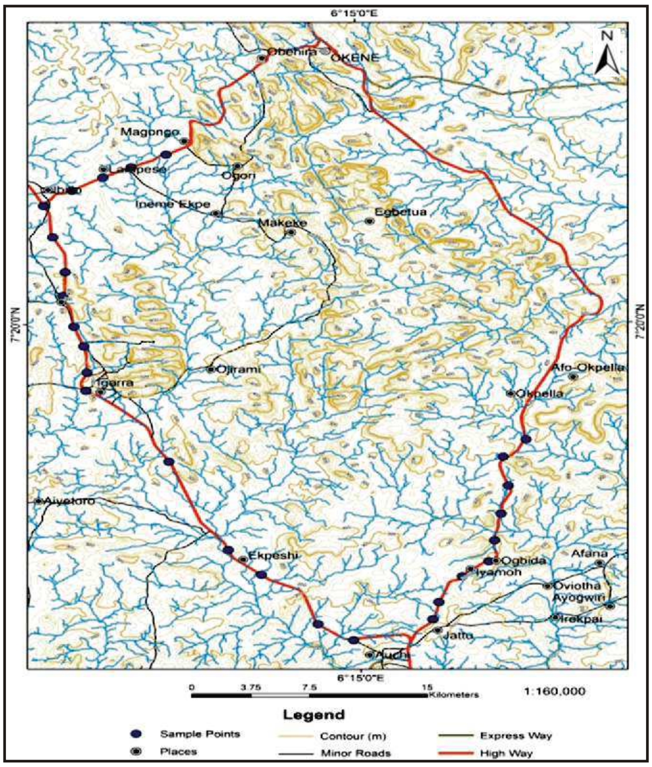

Fig. 1: Location map of the study area showing Auchi - Okpela - Okene - Ibillo - Igarra - Ekpeshiroad network

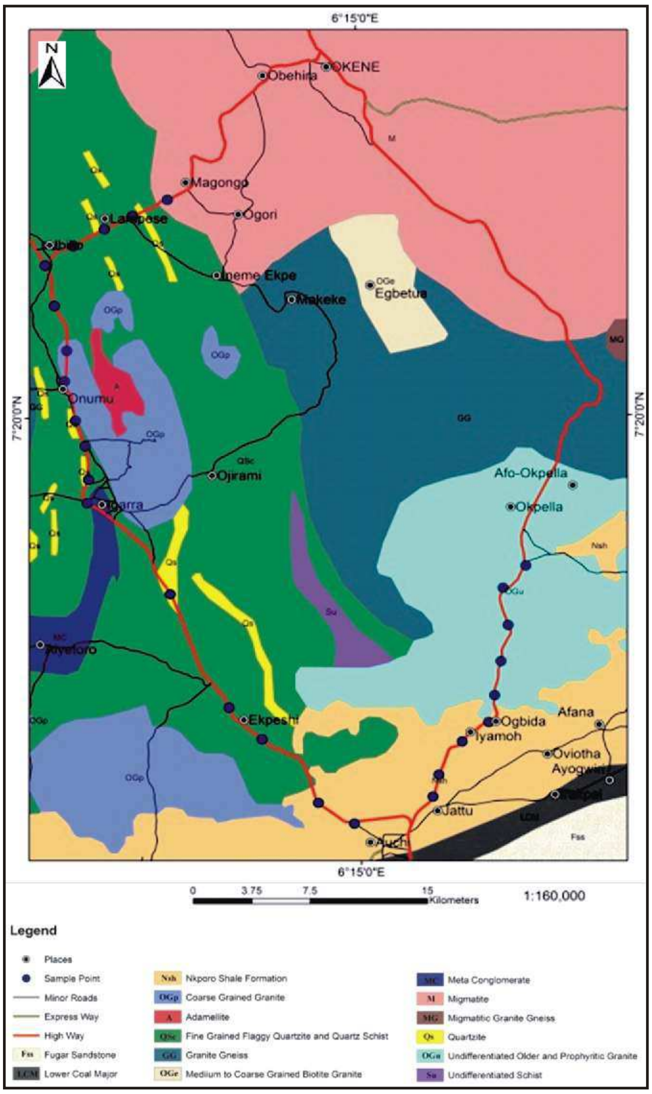

Fig. 2: Geological map of the study area showing the sample points (digitised from the mineral and geological maps of Edo and Kogi States by NGSA2006)

Fig. 3: Plate Showing distressed portion of Station C3 along Igarra- Ibillo Road.

Senonian -Nkporo Shale Group, upper Senonian, Lower Coal Formation, Mastrichian False bedded sandstone/coal Formation, Eocene Imo Shale Formation, Upper Eocene Ameki Formation, and the Oligocene Miocene Lignite (NGSA, 2006).

Materials and Methods

Twenty six (26) soil samples were collected along the pavement sections A,B,C,Dfor geotechnical testing. Geophysical investigation using Schlumberger Vertical Electrical Sounding (VES) was carried out at each samplecollectionpoint. The collected soil samples were taken to Auchi Polytechnic, Civil Engineering Department Soil Lab for geotechnical analysis.

Vertical Electrical Sounding Investigation

The survey was carried out offset of the road network using ABEM Terrameter SAS 1000 with Schlumberger array.This equipment displays apparent resistivity values digitally as computed from ohm's law. Applying two outer currentelectrodes (A and B) into the ground, and are adjusted to vary the distance. The inner(potential) electrodes (M and N) remain fixed. The resultant potential difference (V)measured between the potentials M and N electrodes; the distance between the potentialelectrodes (MN) is small compared to the distance between current electrodes (AB) and ABN 5MN. The spacing is adjusted when it is needed because of decreasing sensitivity ofmeasurement. The centre point of the electrode array remains fixed but the spacing of theelectrodes was increased so as to obtain the information about the stratification and deeperpenetration of the ground.When the ratio of the distance between the current electrodes to that between the potentialelectrodes become too large, the potential electrodes must also be displaced outwardsotherwise the potential difference becomes too small to be measured with sufficient accuracy.

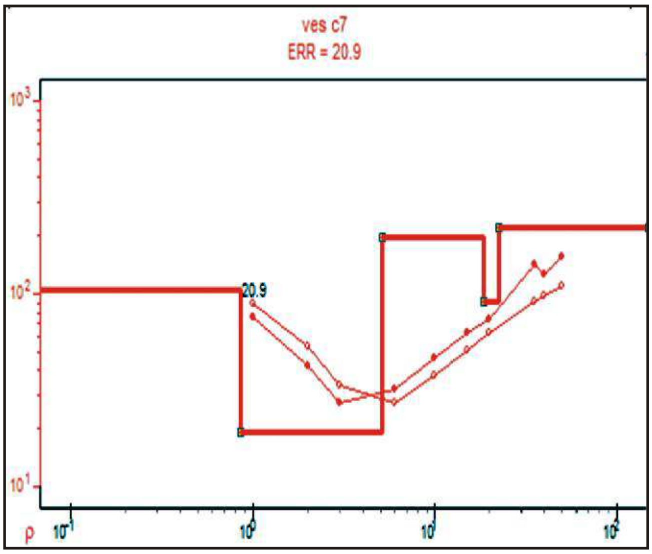

Fig. 4: Geoelectrical Section for Station C7 showing H type Curve

Geotechnical Methods

Different geotechnical tests were performed in the laboratory such as sieve analysis, hydrometer analysis, natural moisturecontent, specific gravity, Atterberglimits, compaction and California Bearing Ratio (CBR) to determine the soil engineering properties.These laboratory tests were carried out according to British Standard Methods of test for soils for engineering purposes (BS 1377: 1990). The grain size distribution analysis wasconducted according to BS 1377: Part 2:1990 Section 9. The hydrometer analysiswas performed using 50g of material finer than0.075 mm. The moisture content test was carried out as stated in BS 1377: Part 2 1990 Section 3. The specificgravity (Gs) was determined according to the BS 1377:Part 2: 1990 Section 7. Atterberg limits test (liquid and plastic limits)was conducted according to the BS Standard Test Method (Part 2: 1990 Sections 4 and 5). The determination of dry density/moisture content relationship was carried out using BS 1377: Part 4: 1990 section 3 test principles and procedures while CBR test was done according to BS 1377 Part 4 : 1990 Section 7.

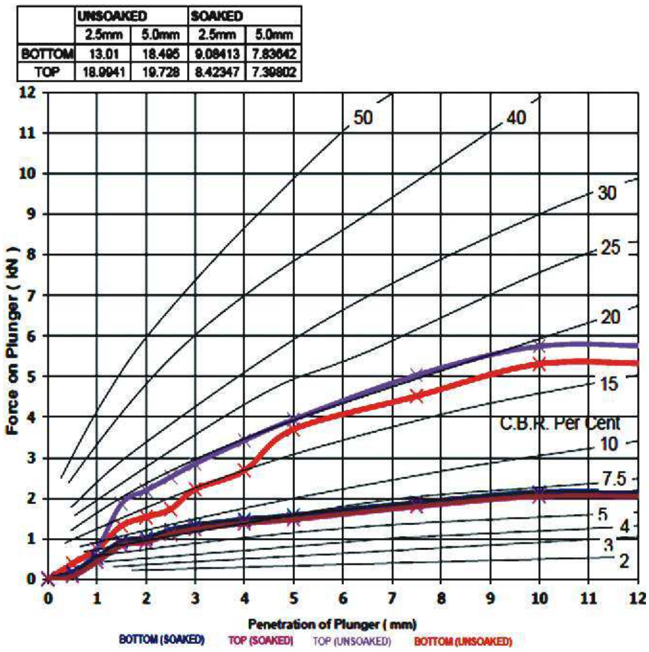

Fig. 5: Sample C1 CBR analysis for soaked and Unsoaked

Results and Discussions

The results of the geotechnical and geophysical analyses are summarized in Table 1.The studied roads have varying soil percentage passing No. 200 (0.075mm) ranging between 16 - 71%, 0 – 90.3%, 17 – 91.7% and 0 – 90% for road sections A,B,C,D respectively. Based on AASHTO classification, the soil is rated as poor to excellent subgrade materials and USCS classification as inorganic clays of low to medium plasticity with gravelly sand. The Road Note 31 classifies Subgrade into Six classes(S1 –S6) base on their CBR values. S1 has CBR value of 2% and S6 has CBR of 30% or more. The Four major curve types(A, H, Q and K)which represent the first two layers of the geoelectric sections were used to delineated the subgrade strength.

Section A: Auchi – Okpella Highway

Road section A is on a transition zone, underlain.by gravelly and sandy soils with less amount of clay (<20%). The fine soil mixtures from locations A6 and A8 contained clay with higher liquid limit and plasticity index values than soil mixtures from other locations

Table 1: Results of the Geotechnical and Geophysical properties the soils

LOCATION	% Gravel	% Sand	% Silt	% Clay	% Fines	% LL	%PL	%PI	RESISTIVITY	CURVE TYPE	CBR	FMW	AASHTO	SUBGRADE
A7	22.3	71	6.7	0	3.4	13	-	-	47	A	32	S6	A-1-b	EXCELLENT
A4	9.1	69.9	1	20.1	10.6	17	14	3	213	A	16	S5	A-2-4	GOOD
A3	24.1	38.8	2.9	17.6	10.3	10	5	5	303	Q	22	S5	A-2-4	GOOD
A1	0	16.9	73.1	7	40.1	5	-	-	832	A	38	S6	A-4	FAIR
A2	0	43.2	27.7	23	25.4	20	15	5	246	H	16	S5	A-4	FAIR
A5	0	53.6	40.4	6	23.2	13	-	-	27	H	5	S3	A-4	FAIR
A9	0	49.4	40.1	1.4	20.8	7	-	-	51	H	7	S3	A-4	FAIR
A6	0	43.4	9.3	40.8	25.1	43	23	20	51	K	3	S2	A-7-6	POOR
A8	0	41.5	15	35.5	25.3	35	14	21	49	K	3	S2	A-6	POOR
B6	80.4	9.6	0	10	5	10	4	6	164	A	22	S5	A-1-a	EXCELLENT
B2	0	90.3	9.7	0	4.85	7	-	-	82	A	34	S6	A-3	EXCELLENT
B4	0	0	100	0	50	3	-	-	97	H	7	S3	A-4	FAIR
B1	10.2	39.8	40.3	9.7	25	9	-	-	259	K	16	S5	A-4	FAIR
B5	10.2	39.8	42.1	7.9	25	38	17	21	227	K	4	S2	A-6	POOR
C2	72.4	17.6	0	10	5	12	5	7	150	A	25	S5	A-1-a	EXCELLENT
C5	0	91.7	8.3	0	4.2	8	-	-	144	A	15	S5	A-1-a	EXCELLENT
C6	0	85	8.5	6.5	7.5	3	-	-	57	A	18	S5	A-1-a	EXCELLENT
C8	13.8	52.2	20.4	13.6	17	27	12	15	161	Q	16	S5	A-2-6	GOOD
C1	39.8	31.2	21.3	8.7	15	18	10	8	172	Q	8	S4	A-2-4	GOOD
C7	0	17	62.1	20.9	41.5	18	9	9	19	H	7	S3	A-4	FAIR
C4	40	22.3	6.7	31	18.9	35	14	21	133	K	4	S2	A-6	POOR
C3	0	20	8.8	71.2	40	65	23	42	266	K	1	S0	A-7-6	POOR
D2	0	90	10	0	5	5	-	-	80	A	42	S6	A-1-b	EXCELLENT
D4	74	0	10.7	15.3	13	18	7	11	1120	Q	14	S4	A-2-6	GOOD
D1	10	29.5	0	60.5	30.3	53	22	31	64	K	3	S2	A-7-6	POOR
D3	0	49.5	0	50.5	25.3	45	22	23	596	K	4	S2	A-7-5	POOR

along the road section. Therefore, the soil is characterised intermediate plastic.The CBR values ranges from 3 – 38%. Resistivity value is relatively low except A1, A2 indicating highly compacted soil.Based on FMW&H (1997) standard, the quantity of fines is more than the required 15% maximum value except for A3, A4 and A7 which therefore rated the subgrade as poorto excellent subgrade materials. Based on AASHTO classification, soils are grouped under A-1-b, A-2-4, A-4, A-6, and A-7-6.Location A7 and A4 is sited in basement and sedimentary terrain respectively. Their Resistivity at 1.2m is 47 ohm-m and 213 ohm-m respectively. The first two layers of theirgeoelectrical section depict an A type curve. A7 is an excellent subgrade a CBR value of 32% and it has Subgrade strength of S6.(AASHTO 1978, Overseas Roadnote 31 1993). A6 and A8 have K type curve. Their resistivity is 51 ohm-m and 49 ohm-m respectively. They are classified as poor Subgrade. A6 and A8 has CBR of 3% and Subgrade strength class S2.

Section B: Auchi – Igarra Highway

This section of the road equally has mixtures of gravel and sand with less amount of clay (<10%). The liquid limit ranges from 3 – 38%.The soils have high sands, low CBR (3 – 22%) and arenon-plastic except B5.The less quantity of fines for B2 and B6 is within the required 15% maximum value. The soils have an AHK-curve type, resistivity range (82 - 256Ωm) indicative of weathered layer or a highly conductive material. Similarly, the soil rating is poor to excellent subgrade.AASHTO classification of soils in this area falls into A-1-a, A-3, A-4, and A-6. Location B6 and B2 have A type curve. Their resistivity at 1.2m, CBR and Subgrade strength are:164 ohm-m and 82 ohm-m, 22% and 34%, S5 and S6 respectively. B6 and B2 are located in Basement complex and sedimentary terrain respectively.

Section C: Igarra – Ibillo Highway

Soils in this area contained gravelly and sandy soil mixture with moderate amount of clay (<20%). Locations C3 and C4havehigh clay content (>30%), high liquid limit (35 - 65%) and plasticity index (21 - 42%). Excluding these two locations, C1 – C7 are non - plastic. The soils have low resistivity (19 – 266Ωm), A,Q,H and K-curve types, CBR (1 – 25%) and are grouped based on AASHTO as A-1-a, A-2-4, A-2-6, A-4, A-6, and A-7-6. Location C8 and C1 have Subgrade strength and resistivity S5 and S4, 161 and 172 respectively. Their curve type show a Q type curve. C3 on the other hand has the lowest CBR(1%). It is characterised by high clay content(71%) and a K type curve

Section D: Ibillo – Okene Highway

The fine grained flaggy quartzitic soils contained high amount of clay (<60%). Location D2 with zero clay however has high sand content (90%) and 42% CBR. The liquid and plastic limits are also high ranging from 5 – 53% and 0 – 22% respectively. Locations D1 & D3 soils are highly plastic and have poor subgrade rating. The quantity of fines in D2 & D4 is within acceptable limit of <15% maximum value and highly resistive (1120Ωm) suggesting an excellent subgrade rating. Soils are grouped as A-1-b, A-2-6, A-7-6 based on AASHTO classification.

Summary and Conclusion

There is a nil correlation between subgrade strength (CBR) and resistivity. However, the geoelectrical curve type of the first two layers of the VESshows a positive relationship with the subgrade strengths. Most of the A and Q type curves are associated with high CBR values and of course higher subgrade strength. It is also observed that most of the K and H type curves are associated with low CBR values. Other electrical (Induced Polarisation and Self Potential) methods will also be investigated in the continuation of the ongoing research. In conclusion, resistivity geophysical method using Schlumberger array system is a potential tool predicting subgrade strength.

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Performance of In-Situ Cement-StabilizedWeak Subgrade for Highway Embankment Construction in South-eastern Nigeria Analysed Using the Finite Element Analysis Method

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Abstract

Weak soils of southern Nigeria are characteristically composed of highly compressible weak clay, which are prone to intolerable settlements and shear failures. Most road projects in southern Nigeria are usually preceded by the construction of embankments that are subsequently exposed to stability challenges due to shear failure. These are associated with bearing capacity failure and or poor construction practices. This research evaluates the deformation and stability of weak embankment foundation stabilized in-situ with 6 and 10% Portland cement using the Finite Element Method which based on elastic theory utilizing both shear stress and elastic parameters. The deformation and stability of the embankment was analysed for un-stabilized subgrade and then for cement-stabilized subgrade.The results showed that the untreated thick soft clayey silt (embankment subgrade) recorded maximum Cu of 17kPa and after cement stabilization, the Cu improved to 154 and 208kPa for 6 and 10% respectively. Short term vertical settlement of 363mm and horizontal displacement of 175mm were obtained for a 4m high embankment while a Factor of Safety (*Fs*) of 1.17 was obtained prior to stabilization. Consequent upon stabilization, vertical settlement reduced to 141mm after 5m of in-situ 6% cement stabilization, further reducing to 134mm after 5m of in-situ 10% cement stabilization, while horizontal displacements were reduced to 70mm and 67mm for 6% and 10% cement stabilization respectively. *Fs* was also observed to improve to 2.50 and 2.67 after 6% and 10% cement stabilization respectively. The displacement and stability conditions were observed to better improve as depth of mass stabilization and cement content increased indicating a linear relationship between cement content, stabilization depths, displacement and stability conditions.

Keywords: *Embankment, Stability, Factor of Safety, Deformation, Cement, Shear strength*

Introduction

Various weak, expansive, poor strength soils have been identified by several authors in Nigeria e.g. Abdulfatai *et. al.* (2014), Adesunloye (1987), Chukwueze (1991), Bolarinwa & Ola (2016), Ekeocha & Akpokodje (2014), Tse & Eyang (2016). Bolarinwa & Ola (2016) observed that most subgrade material in Nigeria are predominantly clay having clay minerals susceptible to expansions and shrinkage in the presence and absence of moisture. These soils which are mostly organic soils predominantly underlay southern Nigeria and are poor construction materials. Such soils are intrinsically weak and have very poor foundation carrying characteristics (George & Abam, 1990). Civil infrastructural constructions on these soils are prone to distress caused by their poor mechanical and unfavourable compressibility characteristics (Salem *et. al*., 2013).

Embankments are commonly used in southern Nigeria for various infrastructural developments; these embankments are typically designed to satisfy safety and stability requirements measured as the embankment's factor of safety (*Fs*) against shear failure and degree of permissible deformation with respect to displacement. Typical embankment construction involves the quick loading of usually soft soil foundations; these give rise to changes in the prevailing equilibrium stress regimes as a result of the self-weight of the embankment (Briggs *et. al*, 2017). This ultimately leads to an increase in pore-water pressure, and the dissipation of the excess pore-water pressure generated by the weight of the embankment is generally slow in expansive soils (Okeke & Wang, 2014). Consequently, undrained loading tends to occur in soft soil foundations due to the inability of the soil to gain strength from the dissipation of the generated excess pore-water pressures as a result of the poor drainage characteristics of expansive soft soils. This implies that embankments constructed on soft soil foundations are prone to cumulative increases in pore-water pressures which would ultimately lead to internal stress redistribution, differential settlement, and failure (Wang, Dai, & Okeke, 2018).Huat, (1998) observed that due to quick undrained loading of weak soil experienced with the construction of highway embankment, distress of the embankment and foundation will occur resulting in failure mostly during the construction stage (short term).

The delivery date and cost of constructing of a highway in the study location have been severely undermined by engineering problems associated with soft soil foundation failures under repeated traffic loading. Partial and complete failure of some sections of the already constructed highway embankment includes some of the challenges witnessed in the area together with the failure of road culverts along their centreline (**Plate 1a & b**). Remedial works such as topping up the embankment with fill to achieve level grade have only exacerbated the situation (**Plate 1c**). It becomes necessary to develop a method that would improve the expansive soil to be able to address the critical issue of foundation failure if the highway is to be safely completed.

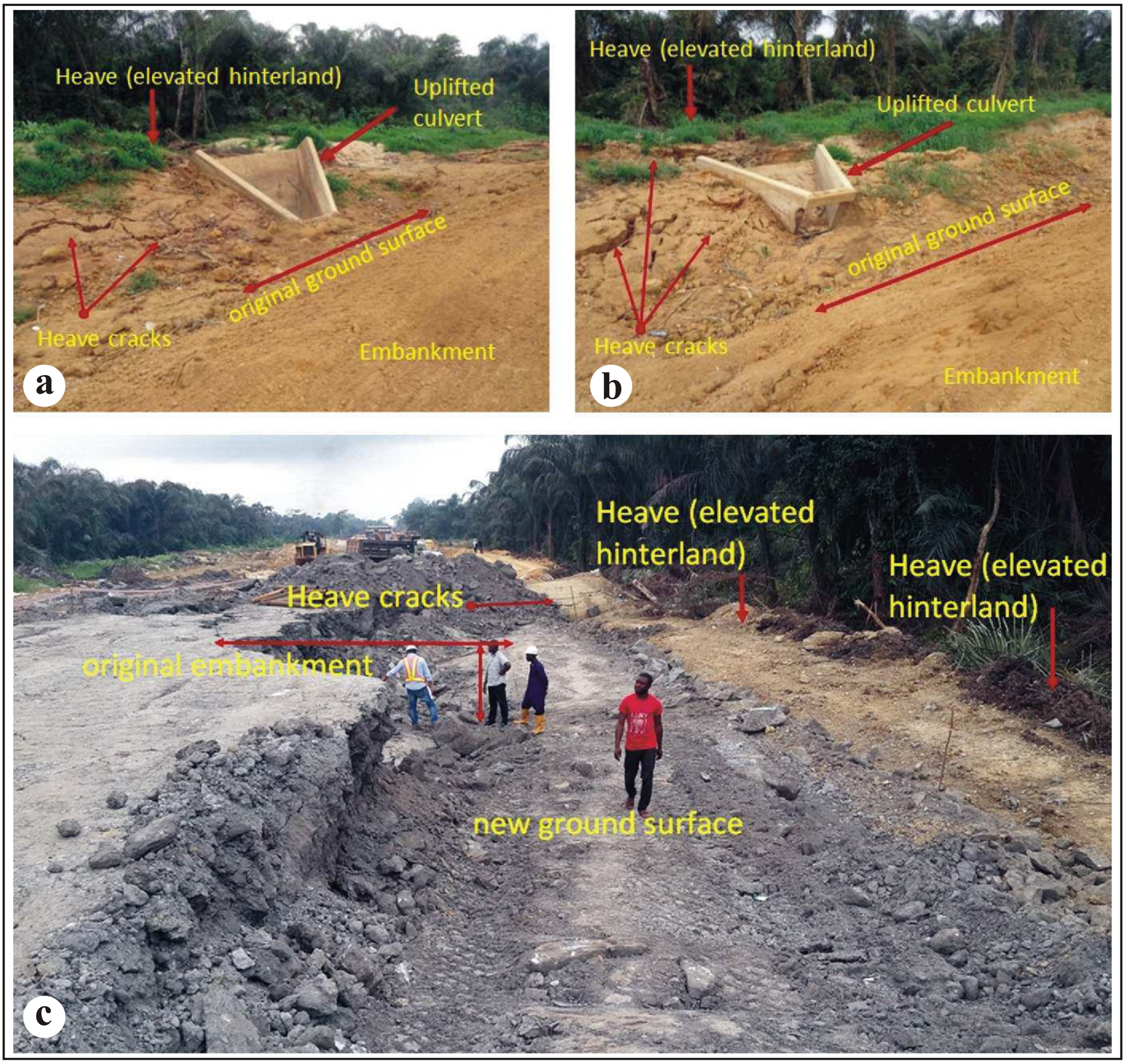

Plate 1: (a, b) Differential settlements and failure of a road culvert at a section of the highway alignment. (c) Shear-failure of the soft soil foundation due to overburden (fill) pressure

Deep in-situ improvement of weak soil using chemical additives such as cement or chemical components to reduce settlement and improve stability for embankments has a wide application and acceptance in many European countries (Euro SoilStab, 2002). Its application being either by adding of wet or dry binders to weak soils of embankment subgrade through mixed in place columns or by stabilization of the entire soil volume under the embankment (Euro SoilStab, 2002).

This research reports the performance of highway embankment subgrade stabilized by mass in-situ mixing using cement. The performance was analysed by using the Finite Element Model (FEM) software Plaxis to model the embankment on weak subgrade stabilized by mass mixing with 6% and 10% cement.

Location, Site Condition and Geology

The study site is on a section of a proposed highway situated about 20km north-east of Calabar, Cross River State and accessed through the Calabar-Ikom road. The road lies between Latitude 5°4'43.46" - 5°9'1.10" N and Longitude 8°30'.02" - 8°20'54.43" E (**Fig. 1**) and was under construction at the time of this study. The study location is seasonally flooded, with complete submersion of low lying portions of the highway occurring during months of extended rainfall. The seasonal wetting and drying are potential sources of concern for weak soils as most of these soils lose 80% of their strength on contact with water.

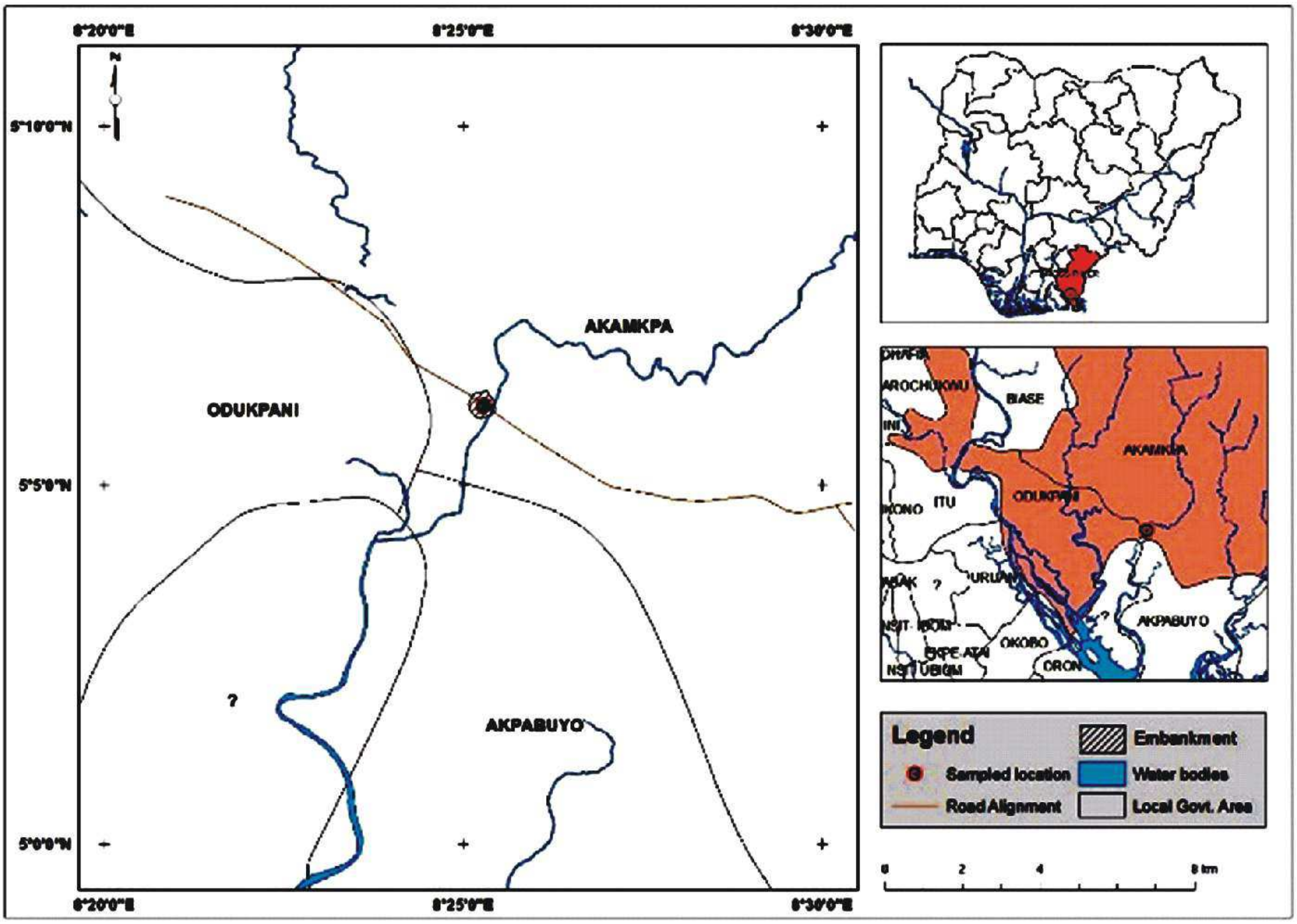

Fig. 1: Location Map of the Study Area showing Sampled Points

Geologically, the study area lies within the Cretaceous Calabar Flank sedimentary basin of southern Nigeria. The Calabar Flank forms part of the lower Benue Trough (**Fig. 2**) (Obaje, 2009) and is bounded by the Oban Massif to the north and the Calabar Hinge line down south (Nyong, 1995).The basin comprises of the Awi formation, Mfamosing Limestone, Ekenkpon Shale, New Etim Marl, Nkporo Formation and the Benin Formation (Adeleye & Fayose, 1978, Reijers & Petters, 1987, Petters, *et. al.,* 1995, Ekwueme *et. al.*, 1995).The Awi Formation sits unconformable on the Oban Massif marked by basal conglorometrates (Tse & Eyang, 2016).

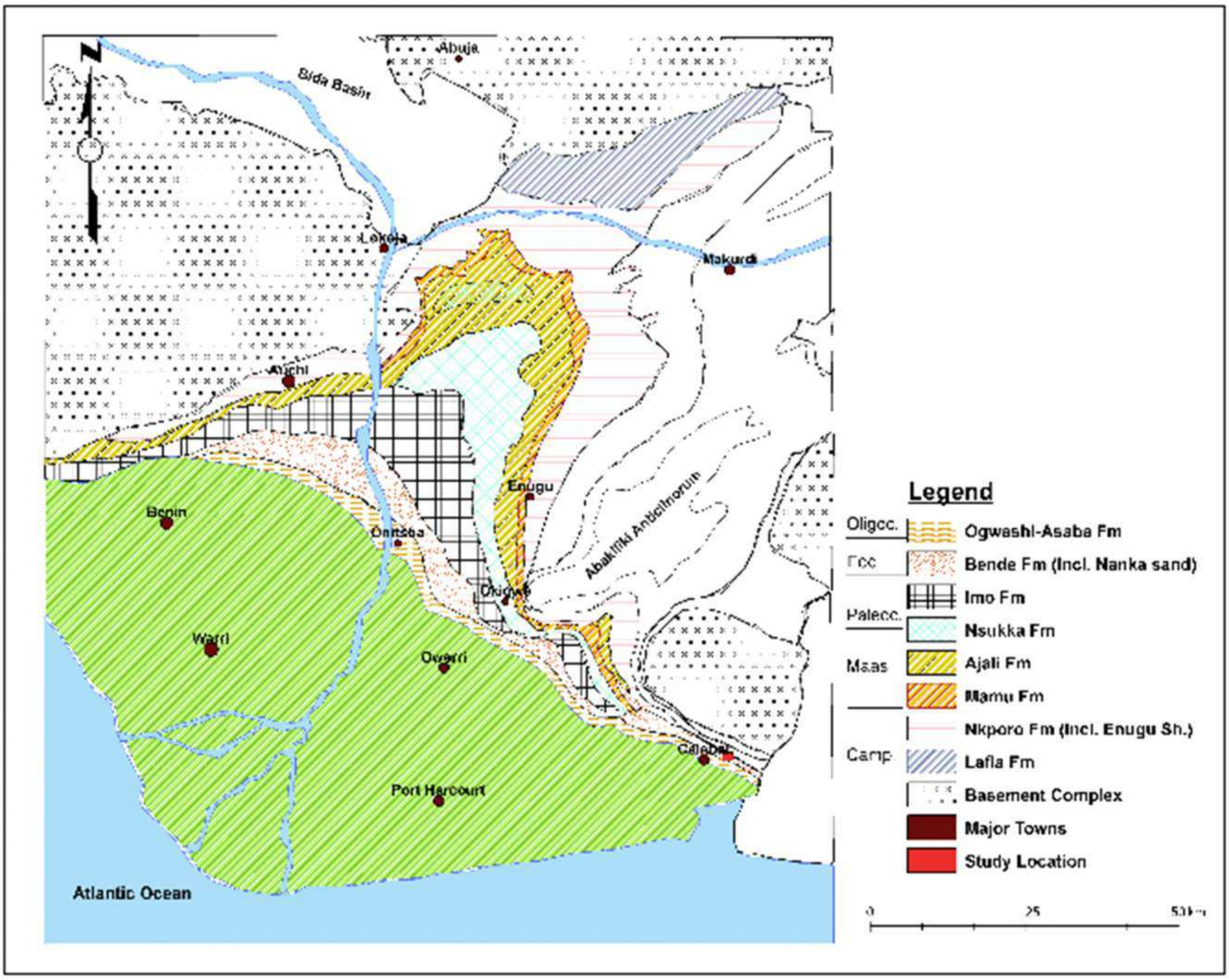

Fig. 2: Geological map of the study area (modified from Obaje, 2009)

Materials and Methods

Three geotechnical borings were used to assess the subsoil profile at the study area. Representative soil samples were obtained and analysed in the laboratory according to BS and ASTM standards. The laboratory test was designed to obtain index and strength soil parameters for unstabilized subsoil (USS) and Cement Stabilized Subsoil (CSS). Appropriate sample quantity was obtained and mixed at natural moisture content with 6% and 10% by weight of ordinary Portland cement. The mixing was achieved using a standard laboratory mixer, the CSS was subsequently placed in a mould and allowed to cure for 28 days, according to Euro SoilStab, (2002) recommended approach.

The strength characteristics of the weak soil was obtained from undrained triaxial test while that of the CSS were determined from unconfined compressive strength test. Cement as a stabilization agent is a hydraulic binder having the capacity to achieve stabilization alone and is not dependent on soil mineral grains (Euro SoilStab, 2002) with its reaction with present soil moisture being a key role in stabilization. Cement will bind with the soil by a hydration process to improve strength but will not alter the structure of the soil (Euro SoilStab, 2002). MacLaren & White (2003) noted that cement hydration involves series of complex and relatively unknown chemical which affects the hydration process, however Makusa (2012) suggested a range of factors which include presence of impurities, the water to cement ratio, the curing temperature, inclusions of additives and the specific surface of the cement-soil mixture. These will influence both the setting time of the mix as well as the gain in strength. Al-Tabbaa and Perera (2005), Euro SoilStab, (2002) both identified Calcium silicates, CaS04 as the principal cementitious component of Portland cement giving it the capacity for strength improvement. Calcium hydroxide; a hydration product of Portland cement also produces cementitious material when it combines with pozzolanic materials available in stabilized soils (Sherwood 1993). The stability of a 4 m embankment was investigated with respect of FoS for the un-stabilized soil (USS) and Cement stabilized soil (CSS).

Two methods for assessing the stability of embankments are available, namely the classical simple Limit Equilibrium Method (LEM) (Craig, 2004) also known as the method of slices for instance Bishop's method (1955) etc. and the Finite Element Method (FEM) (Clough & Woodward, 1967). Finite Element Modelling (FEM) of embankment is a useful tool to determine and predict failure mechanism before construction. FEM analysis is deformation based elastic theory relying on shear stress and elastic parameters (Townsend *et. al.*, 2001). The Finite Element Analysis of geotechnical problems relies on the discretization of a body into a number of elements that are attached at nodal points. The result of the division progression is a set of simultaneous equations which are solved to produce nodal displacements. This displacement of the nodes is then used to calculate the strains and stresses within each element.

FEM enables for stability analysis removing the pre-assumptions of probable slip surface (Duncan, 1996). Several commercial software suites have been developed over the years to perform stability analysis and modelling using FEM. Plaxis 2D; a FEM software was used in this work.FEM is suitable alternative to the LEM (Khan & Abbas, 2014). FEM has the advantage of high realism being able to model complex geometry, loading scenario, material properties, reinforcements, pore fluid behaviours and complex soil behaviours with high accuracy and better visualization of deformations (Matthews *et. al.*, 2014).

Results and Discussion

The results of the investigation showed the site to consist of the following layers. The deepest ground water level observed was at 0.5m.

Layer 1 (embankment fill)

This layer comprises of a firm reddish brown lateritic sandy CLAY of medium plasticity from the existing ground level to the depth of 11m. The layer forms the embankment.

Layer 2 (subgrade)

This consists of a soft dark grey clayey SILT of medium to high plasticity. Decayed remains of plant matter were occasionally observed in the thick cohesive layer. The material extended from 11.0m depth to 27.0m depth in the deepest drilled well. The occurrence of this material at shallower depths in boreholes drilled on the embankment hinterland indicates this material occurred at the original ground level before the embankment construction. Fines content of 87 to 95% and high moisture content range of 49.2 to 108.6% were obtained for the soil, the soil liquid limit (LL) ranged from 63 to 149%, plastic limit (PL) ranged of 33 to 75% while PI ranged from 30 to 74 %, the soil plots below the 'A' Line on the Casagrande plasticity chart and is classified as high plasticity silt (MH). AASHTO classifies the soil as A-7-6 soil and is unsuitable for construction purposes. Having undrained shear strength (Cu) range of 10 to 17kPa and high plasticity results, this material fell short of the Nigerian Federal Ministry of Works and Housing (FMWH) (1997) LL requirements of < 30% and PI requirement < 13% for highway subgrade. The test results are summarized in **Table 1**.

Table 1: Summarized Results and Geotechnical parameter for soils

Results		Fill	Subsoil	Stabilized Soil	
		Range	Range	6%-CSS	10%-CSS
Unit weight (kN/m^2)	Bulk	19.8-20.6	13.8-16.4	14.1	14.4
	Dry	16.5-16.6	6.8-11	8.3	9.0
Moisture content; Mc (%)		19.2-24.9	49.2-108.6	-	-
Liquid Limit; LL (%)		43-58	63-149	-	-
Plastic Limits; PL (%)		22-27	33-75	-	-
Plasticity Index; I_P (%)		21-26	30-74	-	-
Grain size (%)	Sand	55-75	5-13	-	-
	Fines (silt/clay)	25-45	87-95	-	-
Undrained shear strength (S_u) kPa		34-66	10-17	154	208
Friction Angle (^{0})		7-10	1-3	-	-
USCS		CL	MI - ME	-	-
ASSHTO		A-7-6	A-7-5	-	-

Unconfined Compressive Strength

Unconfined Compressive strength (UCS), Triaxial and CBR are effective ways of determining strength improvement of stabilized soil (Euro SoilStab, 2002 & Ingles & Metcalf, 1972). **Figure 3a** shows the stress-strain curves of the soft soil foundations treated with various proportions of cement, while Figure **3b** shows the results obtained from unconfined compression tests carried out on the soft soil foundations treated with various proportions of cement and cured for a period of 28 days. The result shows that the compressive strength of the stabilized soils increased with an increase in cement content. The increment was in the order: 15, 154 and 208 kPa for cement contents of 0, 6 and 10%, respectively. The result further shows that the compressive strength values of the 0 and 10% cement contents revealed a 1287% increase in strength. The increase in compressive strength of the various soil-cement mixtures could be ascribed to the hydration process which produces bonded soil mass or skeletal hardened soil (Mitchell & Jardine, 2002).It should be noted that field strength of chemically stabilized soil is significantly lower than those obtained from laboratory test (Euro SoilStab, 2002).

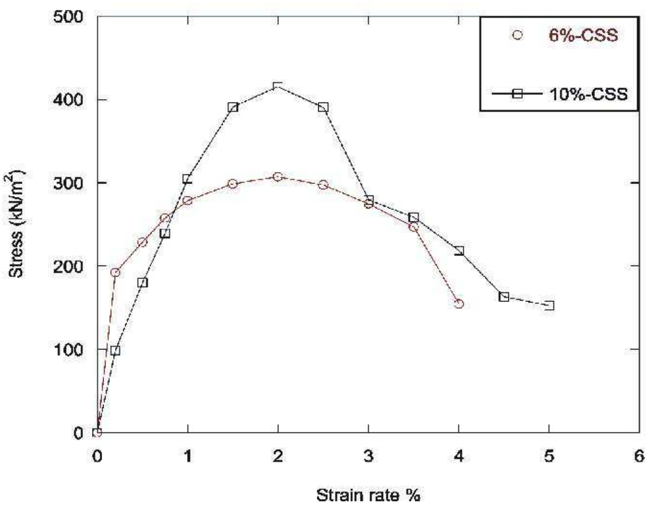

Fig. 3(a): Stress-Strain Curve

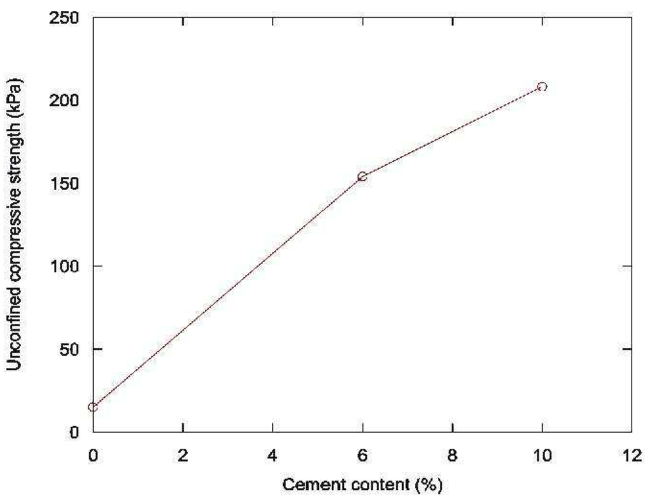

Fig 3(b): UCS vs Cement % content

Stability Analysis

Stability analysis was conducted on the unstabilized soil (*USS*) considering a 1V:2H embankment of 4m embankment height (Figure **4a**), the analysis involves simulating a traffic surcharge load of 20 kPa applied on top of the embankment. Mass stabilization of the top 1 to 5 m of the soft soil foundation was analysed for embankment constructed over the Cement-stabilized soil (*CSS*) having improved shear strength parameters (Figure **4b**) (Euro SoilStab, 2002). The embankment will be considered safe when it satisfies a short term maximum displacement of 100mm with a *FoS* value of 1.3 and 1.5 on a long-term basis (Balasubramaniam, *et. al*, 2010), (Duncan, *et. al*, 1987). The stability analysis was carried out using parameters summarized in **Table 2**.

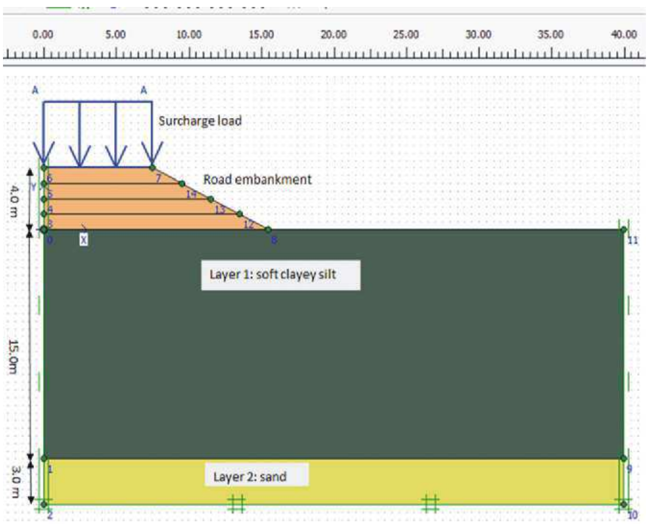

Fig. 4(a): Cross section of the Embankment on soft soil

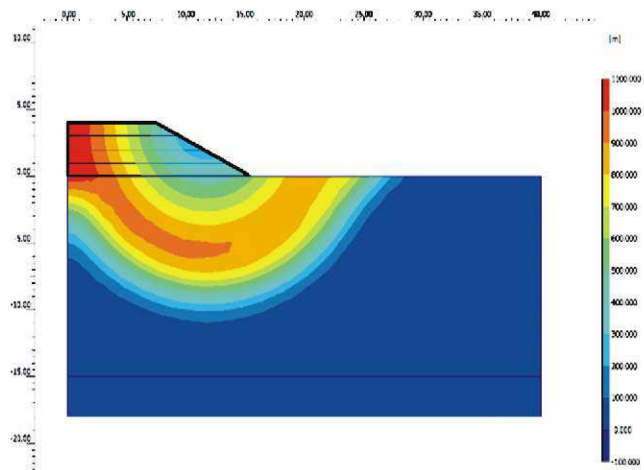

Fig. 4(b): FE Analysis Geometry of the Study location

Fig. 4: Attergberg limit of the Trial Pit Samples

Table 2: Summarized Parameters for Stability Analysis

Parameter	Symbol	Fill	USS	Sand	CSS-6%	CSS-10%	Unit
Binder	-	-	-	-	Cement	Cement	-
composition	-	-	-	-	6	10	%
Dry unit weight	γ_{unsat}	17	8	18	8.3	9.0	kN/m^3
Bulk Unit weight	γ_{sat}	20	15	18	14.1	14.4	kN/m^3
Undrained shear strength	c_u	50	15	1	154	208	kPa
Friction angle	ϕ	9	3	35	-	-	°

Results obtained from the stability analysis using the FEM show that the vertical displacement for a 4 m embankment constructed on the unstabilized (natural) soil was 363mm for short term (Figure **5a**) and 504mm for long term while the *FoS* was 1.17 and 1.19 for short and long term respectively at a traffic load of 20kPa (Figure **5b**). The displacement and *FoS* values determined from the analysis indicate that the short and long-term stability of the highway embankment constructed over the unstabilized soft soil foundation was below the required safety standards and is subject to potential progressive failure over time. However, treating the top 1 m of the soft soil foundation with 6% cement indicated a 30% decrease in the vertical displacement and then a 61% reduction in displacement for a 5m stabilization while also showing improved results for 10% cement content (Figure **5c**). The results show that displacement reduced and *FoS* increased with an increase in the depth of stabilization (Table **2**) indicating that there is a linear correlation between the stability of highway embankments and the depth of stabilization of the foundation(Figure **5c & d**). The overall trend of the results suggests that the major factor influencing the stability of embankments constructed on cement-stabilized soft soil foundations is the stabilization depth (volume of the subsoil) compared to the amount of cement used to improve the soil mechanical properties. (Saadeldin*et. al*, 2011) (Salem, *et. al*, 2013).

Conclusions

This study investigated the underlying factors affecting the short and long-term stability and performance of a highway embankment founded on a soft soil foundation. Safe completion of the 20-km highway project has been marred by shear failures and small to large-scale deformations which occur as a result of

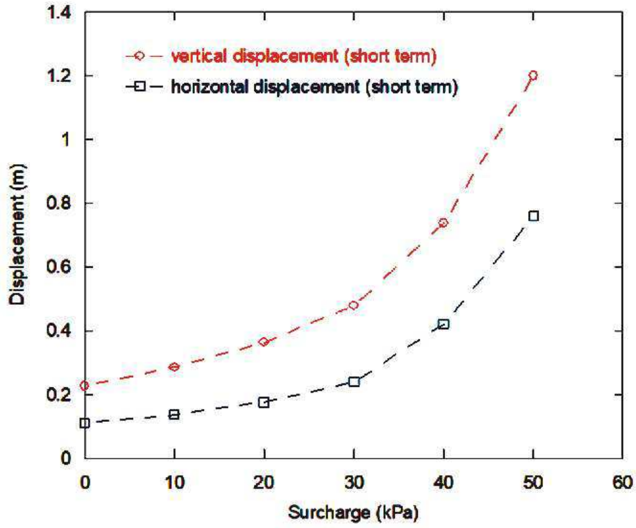

Fig. 5(a): USSShort Term Displacement for various surcharge

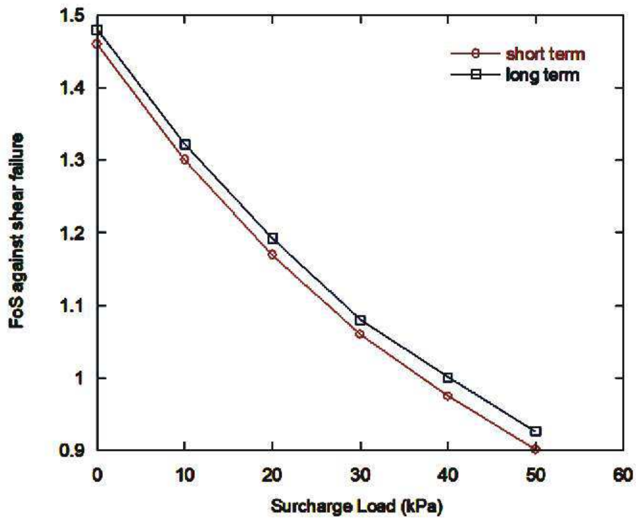

Fig 5(b): FoS of Embankment on USS

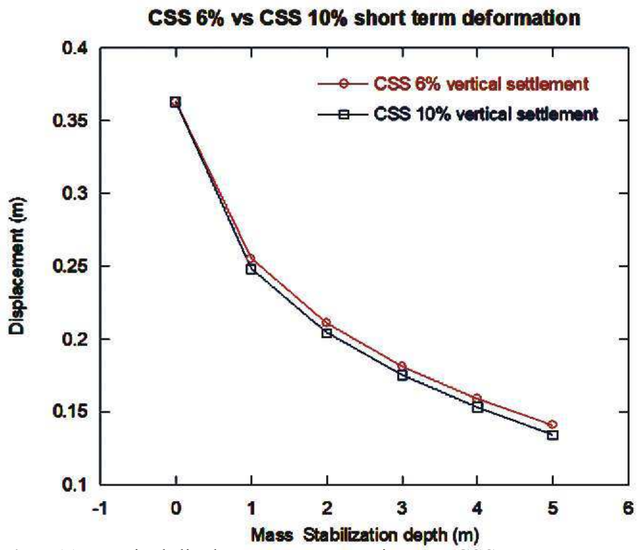

Fig. 5(c): Vertical displacements comparison on CSS

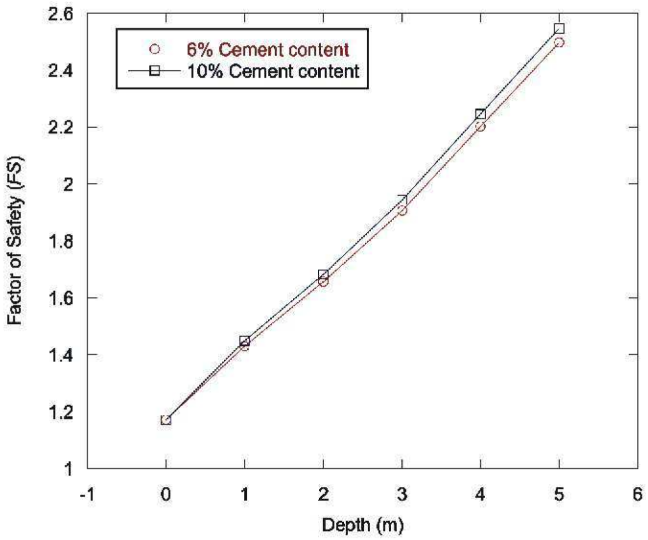

Fig. 5(d): FS improvement with stabilization depth

Table 3: Comparison between USS, CSS 6% and CSS 10%

Stabilization depth (m)	CSS 6%		CSS 10%	
	Short term displacement (*m*)	Factor of Safety (*Fs*)	Short term displacement (*m*)	Factor of Safety (*Fs*)
0	0.363	1.170	0.363	1.170
1	0.255	1.430	0.248	1.449
2	0.211	1.657	0.204	1.682
3	0.181	1.906	0.175	1.944
4	0.159	2.201	0.153	2.246
5	0.141	2.497	0.134	2.546

volumetric changes triggered by variations in the natural moisture content of the soft soil foundation. Combination of geotechnical boreholes drilled to a depth of 30 m and laboratory tests were used to characterize the dominant lithologic units underlying some sections of the highway alignment. The result showed that the plasticity index and natural moisture content of the soft soil foundation ranged from 30 to 74% and 49.2 to 108.6%, respectively. These poor physical properties, together with the high amount of fines and low undrained shear strength of the soil which ranged from 10 to 17 kPa all contributed to the diverse engineering problems encountered in the study area. To overcome these challenges, the present study adopted chemical stabilization of the top 5 m of the soft soil foundation using cement of varying proportions and analysed the overall stability of the embankment and the cement-stabilized soft soil foundation considering different thicknesses of the overburden. Based on the results, the following major conclusions were drawn:

- This study explored the use of FEM analysis in place of conventional LEM analysis. The use of the FEM approach is recommended for safety and stability analysis as it removes the pre-assumptions of probable slip surface and the attending errors of these assumptions.
- Large vertical and horizontal displacements occurred for a 4 m embankment on a soft unstabilized soil resulting in a partial and complete failure of embankment or other structures.
- The FoS of the unstabilized weak subgrade were below short term requirement of 1.3 and long term requirement of 1.5 and therefore unsafe. Rotational slip failure is the failure pattern with the slip surface deep in the soft subsoil.
- The undrained shear strength of the cement-stabilized soils which were cured for a period of 28 days increased from 154 kPa for 6% cement content to 2098 kPa for 10% cement content. This strength gain signifies an improvement in the engineering properties of the soft soil foundation which can be correlated to a significant reduction in plasticity index and swelling potential.
- Mass soil stabilized technique was the most significant contribution to the safety and stability of the embankment. The embankment stability linearly improved with the increase in depth of mass stabilization irrespective of the binder content This technique is more attractive compared to the soil replacement techniques.

Considering the economic implications of cement stabilization and the overall stability of the highway project, this study recommends stabilizing the soft soil foundation with 10% cement by weight of the wet soil to a maximum depth of 3 m.

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State-of-the-Art in Remediation of Hydrocarbon Contaminated Soil: The Nigerian Experience

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Abstract

Remediation of hydrocarbon contaminated soils in Nigeria began in 1999 in a few facilities belonging to Shell Petroleum Development Company, Nigeria Limited in Port Harcourt, Rivers State. Prior to this time, clean-up of hydrocarbon contaminated soil was limited to containment and recovery of free phase oil. The method deployed for remediation in 1999, was a bioremediation techniques named Remediation by Enhanced Natural Attenuation (RENA). It involved creating a large surface area in the impacted soil by tilling and homogenization in order to allow sufficient aeration in the soil matrix to provide oxygen for microbial degradation of the hydrocarbon compounds. Over the years the process has been improved by addition of nutrient amendments, and large number of sites have been remediated and certified at intervention level criteria. Other methods that have been implemented include thermal desorption of oily sludge followed by fixation and solidification of burnt inert ashes and sand. Choice of remediation method begins with an assessment of the impacted site. This provides information on the nature and type of contamination, as well as the risk of contaminants migration to nearby receptors. The closeout criteria has been essential to reduce the contaminant concentration to intervention levels prescribed in EGASPIN, which were adopted from the Dutch system. The prescribed target levels in EGASPIN for closure also adopted from the Dutch system are adjudged as near impossible to achieve. There is need to have a pragmatic close out criteria that is based on risk analysis in terms of harm to humans and the environment. This concept is where the world is heading to.

Keywords*: Remediation, Hydrocarbon, Contamination, Soil, Nigeria.*

Introduction

In Nigeria, major sources of soil contamination are in the handling of hazardous substances within industrial processes, indiscriminate disposal of domestic and commercial wastes and wrong application of fertilizers and pesticides in the farms, as well as oil spills. In view of the extensive use of chemical substances, there is hardly any industrial sector without the possibility of soil or groundwater contamination Remediation of contaminated soil in Nigeria has been limited to sites impacted by hydrocarbon spill incidents, though legislation covers other sources of contamination. The examples in this presentation are around the remediation of soils impacted by hydrocarbon spills.

Types and Sources of Environmental Contamination

The type of contaminants in the environment is closely related to its source or origin. Industries: Industries have been polluting our environment especially since the beginning of the industrial revolution, as mentioned above, notably due to the increasing use of fossil fuels. Though pollution by industries mainly causes air pollution, soil and water contamination can also occur. Transportation: Ever since men abandoned animal power to travel, pollution of the environment has become higher and higher. Similar to industries, pollution caused by transport can mainly be attributed to fossil fuels. Agricultural Activities: Agriculture is mainly responsible for the contamination of water and soil. This is caused by the increased use of pesticides, and application of chemical fertilizers.

Oil Exploration and Production.

Nigeria is the largest producer of petroleum in Africa and the 8th among the Organization of Petroleum Exporting Countries (OPEC). Oil spills from operational causes and more dominantly from crude theft and illegal refining, has resulted to significant environmental pollution as well as economic loss to the nation. In the first quarter of the year 2013, Joint Task Force (JTF) destroyed 3,778 illegal refineries and seized eight vessels, 120 barges, 878 boats, 178 fuel pumps, 5238 surface tanks, 606 pumping machines and 626 outbound engines allegedly belonging to oil thieves.

Process Steps for Remediation Technique Selection for Oil Spill Contaminated land.

- The selection of a remedial or suit of remediation techniques for contaminated soils, begins with an environmental site assessment. This is underpinned by risk consideration of the exposure of receptors to the contaminants.
- Santos divides environmental pollutants into

Plate 1: Photos of Crude theft and Illegal Refining Activity

biodegradable and non-biodegradable ones, and describes them as follows.

- o Biodegradable pollutants are the ones that can be broken down and processed by living organisms, including organic waste products, hydrocarbon compounds, phosphates, and inorganic salts
- o Non-biodegradable pollutants are the ones that cannot be decomposed by living organisms and therefore persist in the ecosphere for extremely long periods of time.

Application of Risk-Based Corrective Action (RBCA) Process

- RBCA is a framework developed by ASTM in which exposure and risk assessment practices are integrated with traditional components of the corrective action process.
- RBCA, starts with development of Conceptual Site Model (CSM. This process identifies, sources of contamination, pathways along which contaminants could migrate and receptors that could be negatively impacted by the contaminants.
- A risk is said to exist, when there is a link, between source – pathway – receptor.

RBCA Goals are :

- protection of human health and environment
- consistent and technically-defensible
- appropriate and resource-efficient remedies are selected
- optimal allocation of limited resources
- practical and resource-efficient approach
- allow corrective action and redevelopment to proceed together

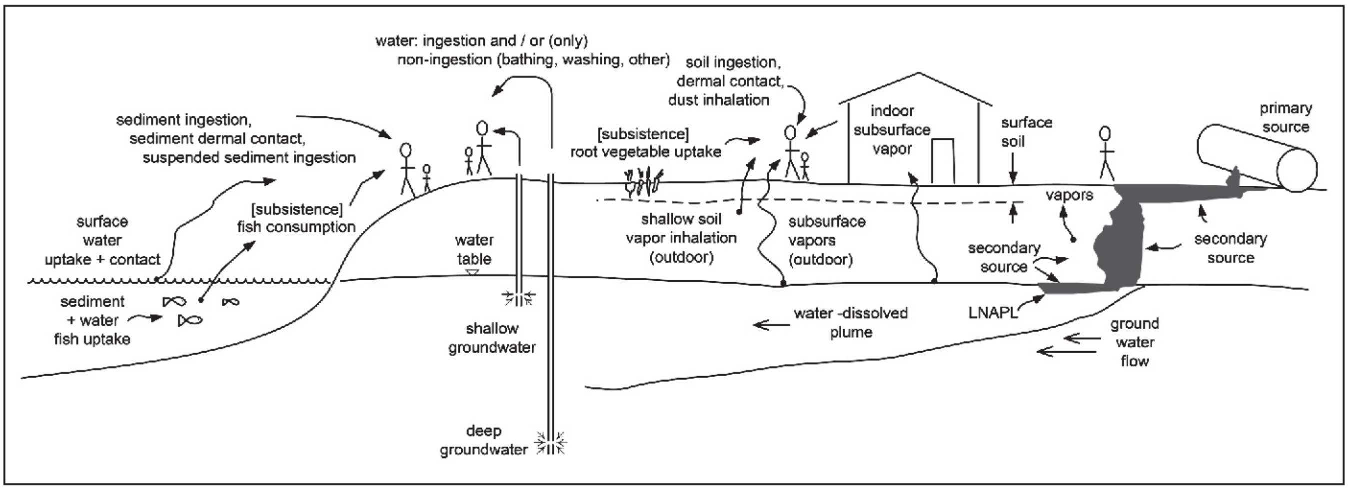


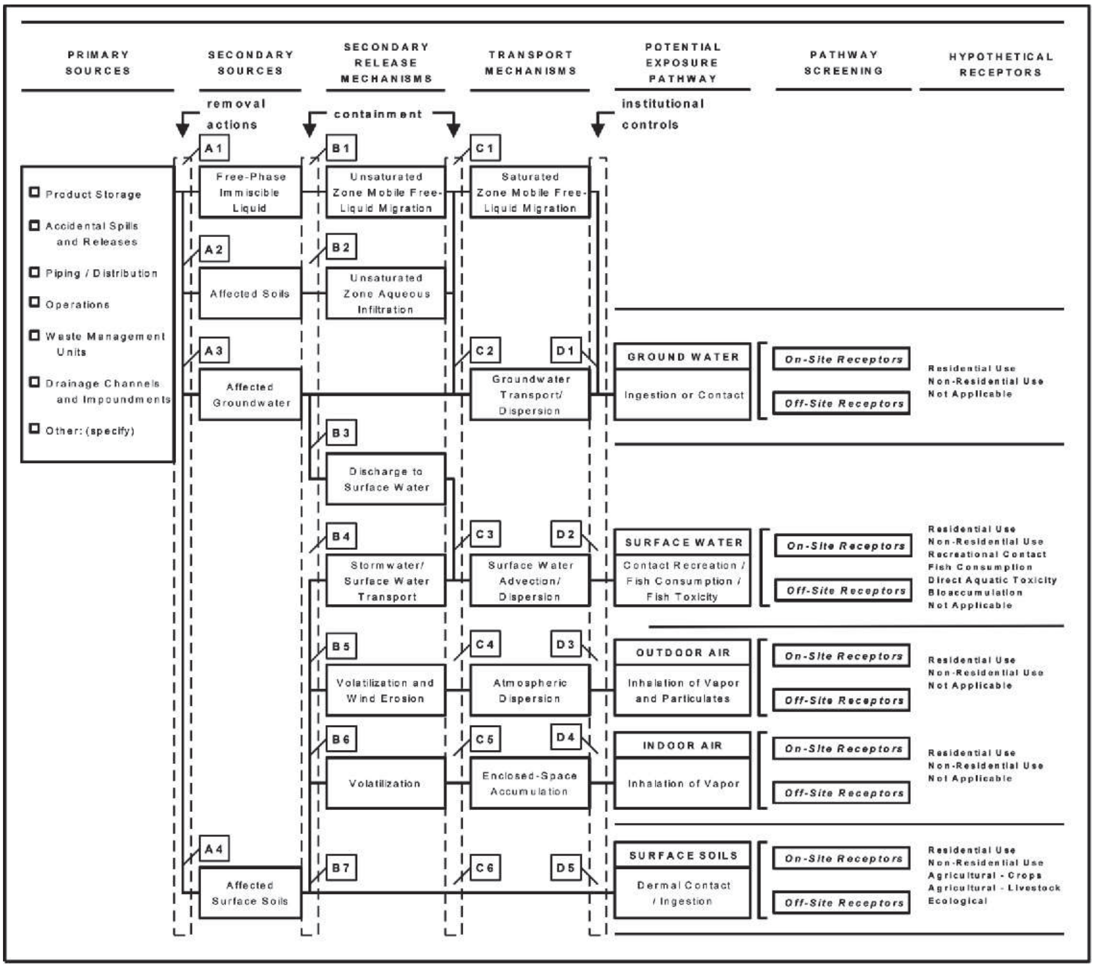
Fig. 1: Example of Conceptual Site Model (CSM)

Process Steps for Remediation Technique Selection for Oil Spill Contaminated land

- The selection of a remedial or suit of remediation techniques for contaminated soils, begins with an environmental site assessment. This is underpinned by risk consideration of the exposure of receptors to the contaminants.
- Santos divides environmental pollutants into biodegradable and non-biodegradable ones, and describes them as follows.
  - o Biodegradable pollutants are the ones that can be broken down and processed by living organisms, including organic waste products, hydrocarbon compounds, phosphates, and inorganic salts
  - o Non-biodegradable pollutants are the ones that cannot be decomposed by living organisms and therefore persist in the ecosphere for extremely long periods of time.

➨ Clean-up and Remediation techniques can be broadly classified into four types:

- ➨ Biological
- ➨ Chemical
- ➨ Physical
- ➨ Thermal



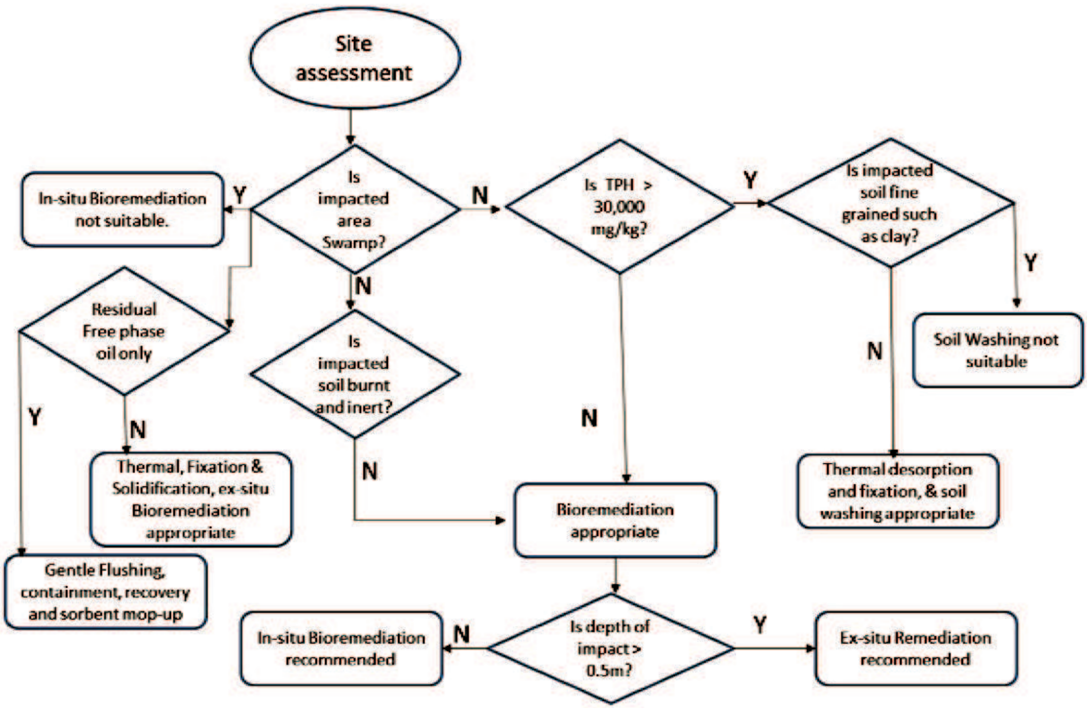


Fig. 2: Guide for Selection of Remedial Option

- These methods could be deployed individually or in combination depending on the type, nature, degree, extent of contamination engineering feasibility and environmental impact.
- The selection of a remedial or suit of remediation techniques begins with a site assessment, which is underpinned by risk consideration of the exposure of receptors to the contaminants.

Oil Containment and Recovery Processes

- Containment is the first process in responding to oil spill incident. It is a process of using suitable barriers to prevent the further migration of spilled oil from the vicinity of spill incident. This could be on water or land.
- On water body, the spill is contained using booms, after which the oil is removed using bailers or skimmer equipment. Booms are used either to surround and isolate a slick of oil, or to block the passage of a slick to vulnerable areas.
- On Land, dikes are built across the path of flow to contain spills before removal by bailers or transfer pumps.
- After containment barriers have been deployed, residual oil is flushed into the containment structures for removal by bailers. This is known as oil recovery.
- Containment and recovery of spilled oil are necessary to prevent further escalation into the surrounding environment

Fig. 3: Oil Spill Containment by River Boom Deployment an Water and Sand-bag Dike Used to Contain Spilled Oil on Land

Fig. 4: Oil Spill Flushing, Containment and Recovery

Thermal and Physical Techniques

- Thermal process involves the use of low temperature thermal desorption unit which separates contaminants from impacted soil. Soil is heated in a chamber in which water, organic contaminants and certain metals are vaporised. A gas or vacuum system transports vaporised water and contaminants to an off-gas treatment system. Thermal desorption temperatures range, generally between 90 – 320°C, and organic components in the soil may be damaged. This method is suitable for recalcitrant hydrocarbon contaminants.

- The residual sandy ash is transformed leach proof cement- concrete by fixation and stabilization processes.
- Fixation by Solidification: Stabilization by concrete fixation is applicable for carbonized crude residue that has undergone deliberate or accidental incineration. Such residues are in practical terms immobile and inert.
- The carbonized residue is crushed to gravel-size particles and used as coarse aggregate in a cement concrete mixture, thus fixing the residue against mobility in the environment. Products of fixation include blocks and slabs, which could be used for construction work.

Chemical Methods

This method makes use of chemicals to flush, extract and transform contaminants from the impacted media. One the commonest method known as Chemical Oxidation, involves the application of liquid oxidants such as hydrogen peroxide, modified Fenton's reagent, activated sodium persulfate, potassium permanganate, as well as a combination of potassium permanganate and hydrogen peroxide, and a combination of activated sodium persulfate and hydrogen peroxide. This method is suitable for recalcitrant compounds and for groundwater remediation.

Biological Techniques

Bioremediation is a process used to treat contaminated media, including water, soil and subsurface material, by altering environmental conditions to stimulate growth of microorganisms and degrade the target pollutants. In many cases, bioremediation is less expensive and more sustainable than other remediation alternatives.

Most bioremediation processes involves oxidation-reduction reactions where either an electron acceptor (commonly oxygen) is added to stimulate oxidation of a reduced pollutant (e.g. hydrocarbons) or an electron donor (commonly an organic substrate) is added to reduce oxidized pollutants (nitrate, perchlorate, oxidized metals, chlorinated solvents, explosives and propellants). In both these approaches, additional nutrients, vitamins, minerals, and pH buffers may be added to optimize conditions for the microorganisms.

In some cases, specialized microbial cultures are added (bioaugmentation) to further enhance biodegradation. Some examples of bioremediation related technologies are: phytoremediation, mycoremediation, bioventing, bioleaching, landfarming, bioreactor , composting, bioaugmentation, rhizofiltration, and biostimulation.

During the redox process, the electron donor is said to be oxidized while the electron acceptor is reduced. Common electron acceptors in bioremediation processes include oxygen, nitrate, manganese (III and IV), iron (III), sulfate, carbon dioxide and some pollutants (chlorinated solvents, explosives, oxidized metals, and radionuclides). Electron donors include sugars, fats, alcohols, natural organic material, fuel hydrocarbons and a variety of reduced organic pollutants.

Common approaches for providing oxygen above the water table include landfarming, composting and bioventing. During landfarming, contaminated soils, sediments, or sludges are incorporated into the soil surface and periodically turned over (tilled) using conventional agricultural equipment to aerate the mixture The redox potential for common biotransformation reactions is shown in Table 1.

Table 1: Types of Bio-chemical Processes for Biodegration of Hydrocarbon Compound

Process	Reaction	Redox Potential (E_h in mV)
acrobic	$O_2 + 4e^- + 4H^- \rightarrow 2H_2O$	600 ~ 400
Aero	O 2 − 4e− + 4H+ → 2H2O	600 – 400
Denitrification	$2NO_3^- + 10e^- + 12H^+ \rightarrow N_2 + 6H_2O$	500 ~ 200
Manganese IV reduction	$MnO_2 + 2e^- + 4H^+ \rightarrow Mn^{2+} - 2H_2O$	400 ~ 200
Iron III reduction	$Fe(OH)_3 + e^- + 3H^+ \rightarrow Fe^{2+} - 3H_2O$	300 ~ 100
Sulfate reduction	$SO_4^{2-} + 8e^- + 10H^+ \rightarrow H_2S + 4H_2O$	0 ~ −150
Fermentation	$2CH_2O \rightarrow CO_2 + CH_4$	−150 ~ −220

- Composting accelerates pollutant biodegradation by mixing the waste to be treated with a bulking agent, forming into piles, and periodically mixed to increase oxygen transfer. Bioventing is a process that increases the oxygen or air flow into the unsaturated zone of the soil which increases the rate of natural in situ degradation of the targeted hydrocarbon contaminant.

Plate 5: Ex-situ Landfarming/Tilling of Hydrocarbon Contaminated Soil in an Engineered Biocell

Landfarming could be an ex-situ waste treatment process that is performed in the upper soil zone or in biotreatment cells, or an in-situ treatment process for shallow impacted soils that are not deeper than the length of the tilling equipment.

- Ex-situ landfarming will start with the design and construction of an engineered biocell, as shown below. Contaminated soils, sediments, or sludge are transported to the landfarming site, incorporated into the soil surface and periodically turned over (tilled) to aerate the mixture. Landfarming commonly uses a clay or composite liner to intercept leaching contaminants and prevent groundwater pollution, however, a liner is not a universal requirement. Modern landfills generally require a layer of compacted clay with a minimum required thickness and a maximum allowable hydraulic conductivity, overlaid by a high-density polyethylene geo-membrane.
- This technique has been used for years in the management and disposal of drill cuttings, oily sludge and other petroleum refinery wastes. The equipment employed in land farming is typical of that used in agricultural operations. These land farming activities
- cultivate and enhance microbial degradation of hazardous compounds. As a rule of thumb, the higher the molecular weight (i.e., the more rings within a polycyclic aromatic hydrocarbon), the slower the degradation rate. Also, the more chlorinated or nitrated the compound, the more difficult it is to degrade.

Plate 6: Various Stages of Fixation and Stabilization of Burnt Carbonized Materials

Limitations/Challenges of Landfarming Practices

- Factors that may limit the applicability and effectiveness of the process include: large space requirements the conditions advantageous for biological degradation of contaminants are largely uncontrolled, which increases the required length of time until complete degradation, particularly for recalcitrant compounds
- inorganic contaminants are not biodegraded

- the potential of large amounts of particulate matter released by operations.
- the presence of metal ions may be toxic to microbes and may leach from the contaminated soil into the ground.
- Hydrocarbon compounds that have been identified as being not readily de graded by land farming include creo sote, pentachlorophenol (PCP), and bunker C oil.
- Biodegradation of hydrocarbon contaminated clayey soils through landfarming process is slow, because of the poor permeability of the clay which reduces aerobic redox reactions and consequently microbial activities.

Pictures of excavation and In-situ Landfarming / Tilling of Hydrocarbon Contaminated Soil

Before Remediation

NOVEMBER 1999

After Remediation

JUNE 2002

Waste Pit of Agbada Well 56

Before Remediation

36'' NKPOKU – OGALE T/L SPILL SITE DURING REMEDIATION 2008

After Remediation

36'' NKPOKU – OGALE T/L SPILL AFTER REMEDIATION 2008

Before Remediation

After Remediation

Old Tank Farm at Rumuekpe Flow Station - 2001

Before Remediation

After Remediation

AGBADA W-52 TEMPRORARY RETENTION PIT [TRP] – 2002.

Before Remediation **After Remediation**

Oil spill impacted area at Km 42 Oroigwe along 28inch Rumuekpke – Nkpogu Trunk Line – 2003.

Challenges and Conclusions

- The conditions advantageous for biological degradation of contaminants are largely uncontrolled, which increases the required length of time until complete degradation, particularly for recalcitrant compounds
- Inorganic contaminants are not biodegraded
- The presence of metal ions may be toxic to microbes and may leach from the contaminated soil into the ground.
- Hydrocarbon compounds in sludge at illegal refining sites will take a much longer time to degrade by land farming.
- Biodegradation of hydrocarbon contaminated clayey soils through landfarming process is slow, because of the poor permeability of the clay which reduces aerobic redox reactions and consequently microbial activities.
- High level of skill is required in handling oxygen releasing chemical agents.
- Concept of RBCA is yet to be adopted in remediation approach as a sustainable development option.
- There is urgent need for scientific research co-sponsored by the Federal Government and International Oil Companies to determine the site specific target level for closure of remediation processes at hydrocarbon impacted sites.

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An Evaluation of the Suitability of Rock Aggregates for Civil Engineering Construction

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Abstract

Experiences over time have shown that civil constructions such as highway pavements and foundations generally do not live through their engineering lifespan. Designs alone cannot address these problems as construction materials such as cement, coarse and fine aggregates also play significant roles. Job owners walk into quarries and buy granitic rock aggregates of different sizes without knowing the physical and engineering characteristics of such materials. This paper aims at giving information on the properties of the rock aggregates used for road pavements and foundations. A comparative study of nineteen rock samples which have been crushed into aggregates and tested for their physical and engineering attributes regarding their suitability for civil construction is considered here. Ranges of the results of tests performed include: Specific Gravity, 2.60 – 2.72; Water Absorption, 0.30 – 0.96%; Aggregate Impact Value, 12 – 30%; Aggregate Crushing Value, 12 – 33%; Flakiness and Elongation, 14 - 29%; Uniaxial Compressive Strength, 190 – 209 NM/m2; and the Los Angeles Abrasion Test, 14 – 41%. These tests have been conducted on samples from three (3) different quarries in Akamkpa LGA of Cross River State. Petrographic studies of the samples from the three quarries have also been carried out. Based on acceptable engineering specifications, these rock aggregates are certified suitable for civil constructions. It is expected that the interpretation and ranking of the test's results will provide a suitable guide to prospective clients of quarry operators, as they can now have some scientific basis for selecting materials to buy for pavement or foundation purposes. Quarry operators are encouraged to test and advertise their products for public consumption.

Keywords*: Aggregates, basement complex, construction materials, engineering properties.*

Introduction

The primary intent of civil engineers as they put down engineering structures is that it lives through their engineering life-spans. This however is not the case as experience overtime has shown that civil constructions such as highway pavements and foundations wear out within too short a time frame. While careful design is very necessary for the safety of structures as it will try to close most loop holes through which the structures may fail, this however has not solved the growing cases of the failure of structures in our environment today.

Therefore both foundation and construction materials need to be carefully analyzed as they play a very crucial role to the safety of any engineering structure (Bell 1993). Construction materials such as fills (materials for compaction), cement, steel rods and even water have their associated challenges especially soils which may contain swelling and shrinking clays. Structures placed on soils which contain active clays respond in a way that may not be easily predicted and this has led to in-depth studies of soils as construction materials and how to overcome their general challenges (Punmia, *et al*., 2005).

Cement and steel rods used in concrete mix pose fewer problems as they are artificially made, hence, behavior under certain structural loads can be easily modeled in the laboratory and the results applied in the field. Less attention has been paid to rock materials used for construction purposes. This stems from the fact that since the basement rocks used for construction purposes "appear strong" in the hand specimen, laymen generally believe that they should withstand any stress placed on them. This has also crawled into civil engineering practices where civil engineers tend to treat rocks as other construction materials such as steel rods and cement which are purely engineering in nature rather than geologic materials which often come with structural imperfections such as planes of weaknesses, fractures and preferred mineral orientations, all of which affect their overall performances (Deere and Miller, 1966).

This paper focuses on rock aggregates used in civil construction, their physical and engineering attributes such as abrasion under certain static and dynamic loads, their petrography and how they affect the rock behavior. The overall purpose of the paper is to characterize rock aggregates based on their mechanical attributes and present them in the form of a working guide to civil construction engineers and other aggregate users during civil constructions so that the longevity of civil structures can be increased. Ndukauba and Akaha (2012) noted that more than 90% of asphalt pavements and 80% of concretes consist of construction aggregate, the remainder being binders such as cement, asphalt and lateritic soil.

Aggregates for pavement designs should withstand impact and abrasive loads (Waltham, 1998) hence rock aggregates should be fresh and possess high strength. (BSI, 1983).

A comparative study was carried out using nineteen (19) rock samples from three quarry sites in Akamkpa Local Government Area of Cross River State. The rock aggregates were crushed and tested for their physical and engineering properties such as Specific Gravity, Water Absorption, Elongation and Flakiness and Elongation index, Uniaxial Compressive Strength (UCS), Aggregate Impact Value (AIV), Aggregate Crushing Value (ACV) and Los Angeles Abrasion Test (LAT). The results of these tests have been used to characterize and compare these aggregates based on acceptable standards for engineering practices.

Location and Geology of the area

Rock aggregates used for this study were obtained from three quarry sites. Two are operational quarries in Njahasang while the third sets of samples come from a proposed quarry in Okom-Ita all in Akamkpa Local Government Area, Cross River State. The area forms part of the Oban Massif of south eastern Nigeria and lies between latitudes 05^013'N and 05^022'N of the Equator and longitudes 008^016'E and 008^024'E of the Greenwich Meridian (Figure 1). It is covered by the thick equatorial rainforest; hence, it is characterized by heavy rainfall which is evenly distributed (Adeleke and Leong, 1980). The temperature of the area ranges from 24^0C to 28^0C and rarely exceeds 35^0C (Iloeje, 1981).

The geology of Oban Massif, south eastern Nigeria of which the study area is an integral part, is typically a basement complex (Figure 2) (Ekwueme and Onyeaokocha, 1985). It consists of banded gneiss, granite and biotite gneiss intruded by pegmatite, granites and granodiorite in some places (Rahman *et al.*, 1981, Ekwueme, 1990, 1995). Typical minerals include quartz, feldspars, microcline, biotite, muscovite and some ferromagnesian minerals. The drainage of the area is structurally controlled. The rocks are moderately weathered especially at the surface with the overburden profile reaching up to 40-50m in places (Okereke *et al.*, 1998). However, fresh samples were collected from the quarry faces and quarry floor and were used for this study. Rock sample for these studies from Quarries 1 and 2 (Figure 2) occur as intrusions within the Asu River Group. The scale of the map (Figure 2) does not allow these exposures to manifest on the surface.

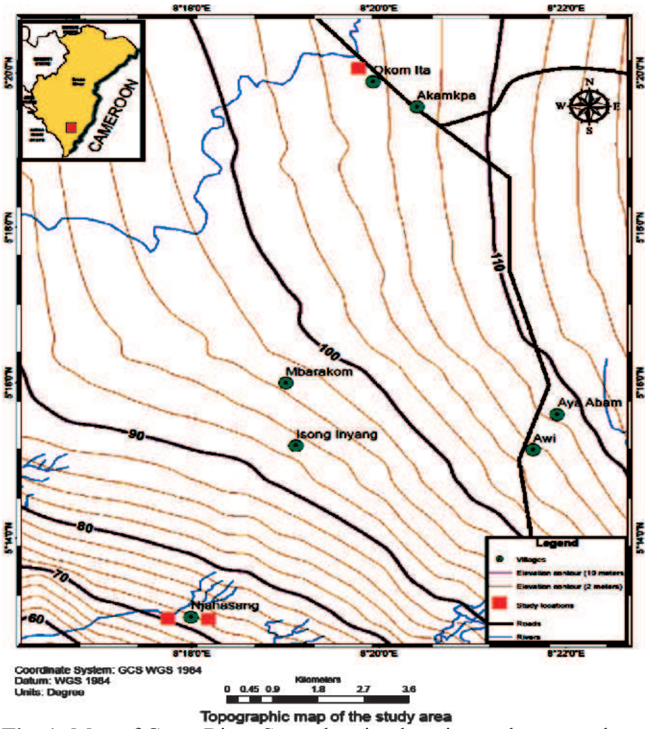


Fig. 1: Map of Cross River State showing location and topography of the area

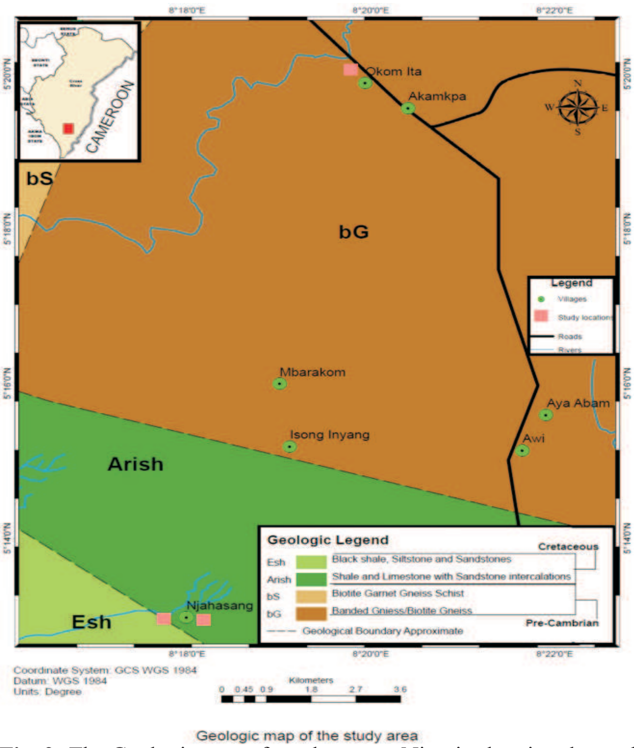


Fig. 2: The Geologic map of south eastern Nigeria showing the study area (Ekwueme, *et al.*, 1995).

Materials and Method

Samples for this work come from three quarry sites. The samples were obtained from the quarry floor, quarry face and from surface exposures at the proposed quarry site. The sites were accurately surveyed using Garmin 76 Global Positioning System (GPS) to obtain coordinates (latitudes and longitudes) and the relative elevation of the study area. Samples were collected with hammer, properly labeled (with masking tape and a permanent marker) and stored in a calico bag. Petrographic studies of thin sections prepared from the representative aggregates from the three sites indicate that Quarry 1 and 2 have granites, while Quarry 3 is made up of granodiorites. The physical and engineering properties tests which were performed on rock aggregates from the three quarry sites are reported below;

Fig. 3: Rock fragment in Quarry 1

Fig. 4: Rock fragment in Quarry 3

Fig. 5: GPS reading at Quarry 2

Fig. 6: Rock aggregates prepared/processed for Mechanical testing

Los Angeles Abrasion Test

To determine the Aggregate abrasion value, the ASTM C131 grade B procedure was used. This grade utilizes 11 steel balls of approximately 48mm in diameter as an abrasive charge, 5kg crushed aggregates, 2.5kg passing sieve 20mm and retained on sieve 12.5mm while the other 2.5kg passes sieve 12.5mm and retained on sieve 10mm. Other apparatus include a balance of capacity 5kg, drying oven, a steel rotating drum and trays. The procedure involved oven drying the sample at 105^{0}C, then placing the aggregates and the abrasive charge on the cylindrical drum and the cover fixed. The machine was then rotated at a speed of 30 revolutions per minute for 500 revolutions. The machine was then stopped and the materials discharged into tray. The entire stone dust was placed on 1.70mm BS sieve and the coarser fraction (retained) on the 1.7mm sieve weighed. Los Angeles abrasion value (LAAV) was determined using the relationship;

$$LAAV = \frac{W1-W2}{W1} \quad ......(1)$$

Where LAAV= Los Angeles abrasion value
W_1 = Original mass of aggregate
W_2 = Mass of aggregate retained
LAAV is expressed in percentage

Aggregate impact value

This was carried out with 5kg of aggregates passing BS sieve 12.5mm and retained on sieve 10mm. Apparatus used for the determination of aggregate impact value include a balance of capacity 5kg, cylindrical steel cup of internal diameter 10.2mm and length of 5mm attached to a metal base of the impact testing machine and a hammer of 14kg by weight. The steel cup was filled in three layers with each layer tamped for 25 numbers of blows. The metal hammer was arranged to drop with a free fall through a vertical height of 38.0cm on the test material and the specimen was subjected to 15 numbers of blows. The crushed aggregates were sieved through a BS sieve of size 2.36mm and the material passing weighed and recorded .Aggregate impact value was determined from the relationship;

$$AIV = \frac{W1 \times 100}{W2} \quad ......(2)$$

Where; W_1 = weight of sample passing 2.36 sieve
W_2= Total weight of sample.

Aggregate impact value is expressed in percentage

Aggregate Crushing Value

This was carried out with aggregate 3kg of crushed aggregates passing BS sieve 14mm and retained on sieve 10mm. Apparatus used involved, a steel cylinder with open ends and internal diameter of 152mm, square base plate, a plunger hanging on the piston of diameter 150mm with a hole provided across the stem of the plunger so that a rod can be inserted for lifting or placing the plunger in the cylinder. A cylindrical measure with internal diameter of 115mm and height of 180mm, a tamping rod with one rounded end, having a diameter of 16mm and length of 450mm, a balance of capacity 3kg and a compression testing machine were used. The procedure involved filling the cylindrical measure (mould) with the test samples in 3 layers of approximately equal height with each layer tamped using the rod for 25 number of blows. The weight of the aggregate was measured and recorded as 'A'. The surface of the aggregate was leveled in the mould and the leveled samples were inserted in a compression testing machine. The machine was loaded at a uniform rate of 4tonnes per minute for approximately 10 minutes to achieve 40tonnes (400KN) in 10 minutes. After 10 minutes, the mould was released and the material emptied into a dry pan. The crushed sample was then sieved mechanically through a sieved size of 2.36mm and the fraction passing recorded as 'B'.

Aggregate crushing value was determined from the relationship;

$$ACV = \frac{B \times 100}{A} \quad ......(3)$$

Where; B = Weight passing sieve size 2.36mm
A = Total weight of sample

Aggregate crushing value is expressed in percentage.

Uniaxial Compressive Strength

The apparatus used to carry out the uniaxial compressive strength includes the compressive machine, shaping tools, weighing balance, calipers. To run test, the rock samples were first shaped into standard cubes with dimension of 40mm × 40mm. The test area was relatively flat to give a uniform loading. The cube was place in a compressive machine and loaded axially to failure. The pressure at failure was read on the dial gauge of the compression machine.

Uniaxial compressive strength of the sample was obtained from the relationship:

$$\sigma = \frac{P}{A} \quad ......(4)$$

Where; σ = Uniaxial compressive strength in MN/m^2
P = Maximum load in MN
A = Area of cube in m^2.

Flakiness and Elongation Index

Samples for the elongation test weighed 5kg. It was mechanically shaken in a set of BS sieve ranging from 63mm to 6.3mm. Samples passing sieve 6.3mm were rejected. The apparatus used include; BS set of sieves, a balance of capacity 5kg and a gauge. The procedure includes mechanically shaking the samples in a set of BS sieve and the mass passing or retained at each sieve was recorded. A maximum of about 200 pieces of each fraction of the samples was weighed in a gauge for thickness and samples passing each slot were weighed.

$$\text{Flakiness} = \frac{Total\ mass\ passing \times 100}{Total\ mass\ of\ aggregate} \quad ......(5)$$

Flakiness is expressed in percentage.

Water Absorption

This was determined using aggregates passing BS sieve 65mm and retained on sieve 6.3mm. The following apparatus were used; wire mesh basket, oven, and balance of capacity 1kg, trays and dry absorbent clothes. The steps used include;

- The aggregates weighed approximately 1kg and was thoroughly washed and dried to free them of clay particles.
- The aggregates were soaked in wire mesh basket and submerged in water for 24hours
- After 24hours, the samples were dried with dry absorbent cloth and weighed and recorded as m_1 (saturated surface dry)
- The dry samples were then placed in a well ventilated oven at a constant temperature of 105^0C for about 1hour
- After 1hour, the samples were removed, weighed and recorded as m_2 (oven dried sample)

Water absorption was determined from the relationship;

$$\text{Water absorption} = \frac{m1-m2 \times 100}{m1} \qquad (6)$$

It is express as a percentage.
Where; m_1 = weight of saturated surface dried sample
m_2 = weight of oven dried sample

Specific Gravity

This is also called particle density. Specific gravity was conducted with 3kg of aggregates which were less than 10mm in diameter. Apparatus used in the determination of specific gravity include, a balance which has the capacity not less 3kg, a thermostatically controlled oven that can maintain a temperature range of 105^0C-110^0C, a pykonometer and trays. Specific gravity was determined using density bottle (pykonometer). The following steps were taken;

- Density bottle was weighed and recorded as m_1
- Density bottle was filled with dry sample and weighed as m_2
- Water was added to density bottle filled with sample and weighed as m_3
- Density bottled was filled with water alone then weighed and recorded as m_4

Specific gravity (G_s) was then determined from the relationship:

$$G_s = \frac{m2-m1}{[(m4-m1)-(m3-m2)]}$$

G_s has no unit since it is the ratio of weight to weight

Results and Discussion

Comparative studies of aggregates suitability for civil constructions entail running petrographic, physical and engineering properties tests on the aggregates. The results of petrographic studies of the aggregates presented in Table 1 below significantly explains the differences in the empirical deductions obtained in the cross-plots presented in Figures 10, 11 and 12. These figures show that properties of granodiorites cannot correlate well with those of the granites.

Results of Petrographic Analysis

Table 1: Average modal composition of aggregate from study area

Minerals	Quarry 1	Quarry 2	Quarry 3
Quartz	38	40	31
Microcline (alkali plagioclase)	25	24	24
Oligioclase (sodic plagioclase)	28	26	30
Hornblende	2	3	3
Muscovite	2	2	6
Biotite	3	3	4
Accessory minerals	2	2	2
Total	**100**	**100**	**100**
Rock name	GRANITE	GRANITE	GRANODIORITE

Table 1 indicates that the rock types in Quarries 1 and 2 are granites. In Quarry 3 granodiorites are found (Figures 7, 8 and 9)

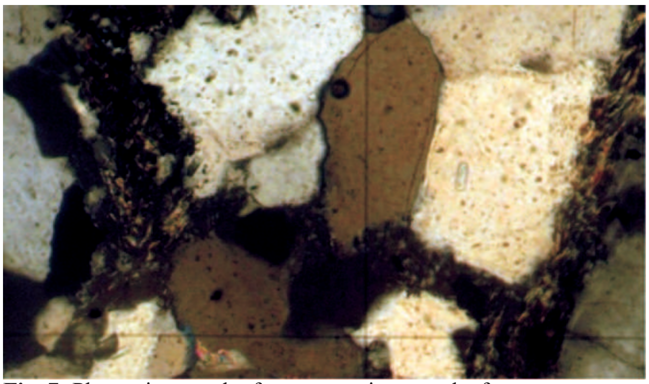


Fig. 7: Photomicrograph of representative sample from Quarry 1 (M × 100)

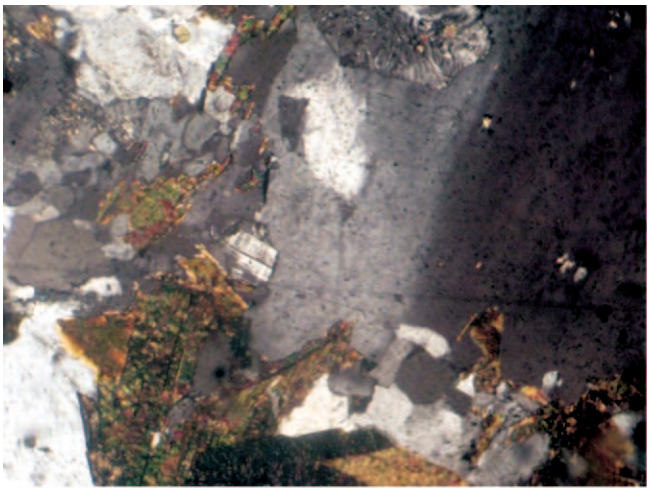


Fig. 8: Photomicrograph of representative sample from Quarry 2 (M × 100)

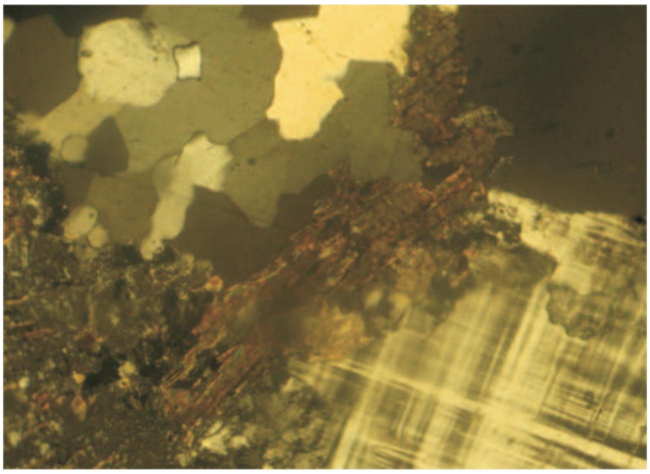


Fig. 9: Photomicrograph of representative sample from Quarry 3 (M × 100)

Table 2: Physical and engineering properties of aggregates from the three quarries in the study area

	QUARRY 1 (operational)			QUARRY 2 (operational)			QUARRY 3 (proposed)		
Mechanical properties	MIN	MAX	AVER	MIN	MAX	AVER	MIN	MAX	AVER
Water Absorption (%)	0.3	0.96	0.66	0.43	1.01	0.73	0.87	0.92	0.90
Specific Gravity	2.60	2.65	2.63	2.60	2.66	2.64	2.66	2.72	2.69
Aggregate impact Value (%)	14	30	21	12	28	19	25	29	27
Aggregate Crushing Value (%)	12	31	19	12	21	16	26	33	29
Uniaxial Compressive Strength MN/m^2	190	206	201	190	209	203	192	208	202
Flakiness and Elongation Index (%)	14	22	19	21	28	23	15	29	21
Los Angeles Abrasion Test (%)	14	29	20	12	31	18	39	42	41

Physical and Engineering properties

The results of the physical and engineering properties of the rock aggregates are shown in Table 2. Table 3 indicates the average values of physical properties against acceptable specifications used in judging the aggregates suitability.

Specific Gravity

Specific gravity rages from 2.60 to 2.65 in Quarry 1, 2.60 to 2.66 in Quarry 2 and 2.66 to 2.72 in Quarry 3. Specific tends to increase with increasing rock strength.

Water Absorption

The values range from 0.66% in Quarry1, 0.73% in Quarry 2 to 0.9% in Quarry 3. There is an inverse relationship between water absorption and the rock strength which increases with depth. Samples from Quarry 3 showed higher water absorption (Table 2). This is due to the fact that they were taken on the surface as the intensity of weathering to which they were subjected to is high. This is as the result of socio-economic activities of the inhabitants of the area such as bush burning from farming practice which must have opened more cracks in these samples, hence, the higher water absorption value. However, their values are less than 1.0% which makes it acceptable for civil engineering construction.

Aggregate Impact Value (AIV)

The value ranges from 12 to 21% in Quarry 1, 12 to 28%

Fig. 4: Attergberg limit of the Trial Pit Samples

in Quarry 2 and 25 to 25 in Quarry 3. The maximum allowable value of AIV for any construction work is 45% (Table 3) hence, the aggregates are judged suitable for civil engineering construction.

Aggregate Crushing Value (ACV)

This ranges from 12 to 31% in Quarry 1, 12 to 21% in Quarry 2 and 26 to 33% in Quarry 3. It measures resistance to crushing load. BS 812 specifies a range of 35 to 45% ACV for all construction works. The lower the ACV, the better the material will be for civil construction as such materials (low ACV) will resist crushing imposed by loads. Using that specification, the aggregates are considered suitable for civil construction.

Uniaxial Compressive Strength

The aggregates from the three quarries show high compressive strength ranging from 190 to 208 MN/m^2. Using ASTM D 2988-71a specification, the rock is judged to be suitable for engineering construction.

Flakiness and Elongation Index

As far as construction is concerned, flaky aggregates should be avoided as much as possible. Flakiness values show a range less than 30% for the study area. The test results fall within the allowable limits for civil construction as indicated in Table 3.

Los Angeles Abrasion Value

This gives resistance to abrasive wear. The result ranges from 18% in Quarry 2, 20% in Quarry 1 to 41 in Quarry 3. The high abrasion value in Quarry 3 can be attributed to weathering since sample from Quarry 3 showed higher degrees of weathering as these surface samples are more susceptible to weathering.

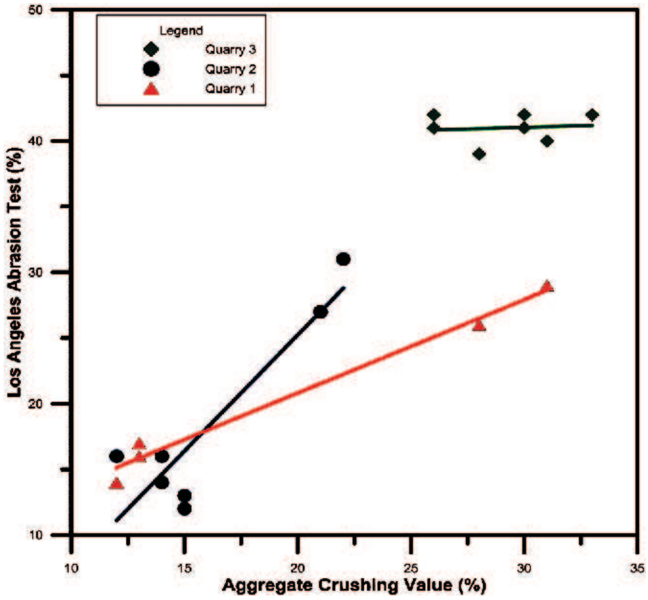
Table 3: Average values of physical properties of aggregates against acceptable specifications

Mechanical Properties	Quarry 1	Quarry 2	Quarry 3	Specifications	Test Designation	Engineering Judgment
Water absorption (%)	0.66	0.73	0.90	(0.1-1.0)%	BS 812, part 110, Clause 2005	Accepted
Specific gravity	2.63	2.64	2.69	2.50-2.90	ASTM C 20-46	Accepted
Aggregate impact value (%)	21	19	27	(27-49)% Granite	BS 812 part 110	Accepted
Aggregate crushing value (%)	19	16	29	35% (max=45%)	BS 812, Clause 2003	Accepted
Uniaxial compressive strength (MN/m^2)	201	203	202	>190MN/m^2	ASTM D-71a	Accepted
Flakiness and elongation index (%)	19	23	21	30%	BS 812, part 110, Clause 2005	Accepted
Los Angeles abrasion test (%)	20	18	41	(27-49)%	ASTM C131	Accepted

Correlation Diagrams

Figure 10 shows a linear relationship between Los Angeles abrasion value and the aggregate crushing value. Since larger samples, higher cost and a more complex procedure is involved in running Los Angeles abrasion test compared to aggregate crushing test, aggregate crushing value can be used to estimated the Los Angeles abrasion value. This implies that few Los Angeles test can be run to produce a correlation graph for the site under study. More ACV tests run for the site can assist in the estimation of accurate Los Angeles abrasion test data. For quarry 3, ACV actually is a good replacement for the Los Angeles abrasion test.

Figure 11 shows a relationship between water absorption and specific gravity. The plot shows a general increase in water absorption rate with decreasing particle density (specific gravity) (Williamson, 2005). This indicates that very compact rock types have little pores in between. Hence, they do not have higher ability to absorb water making them more suitable for construction.



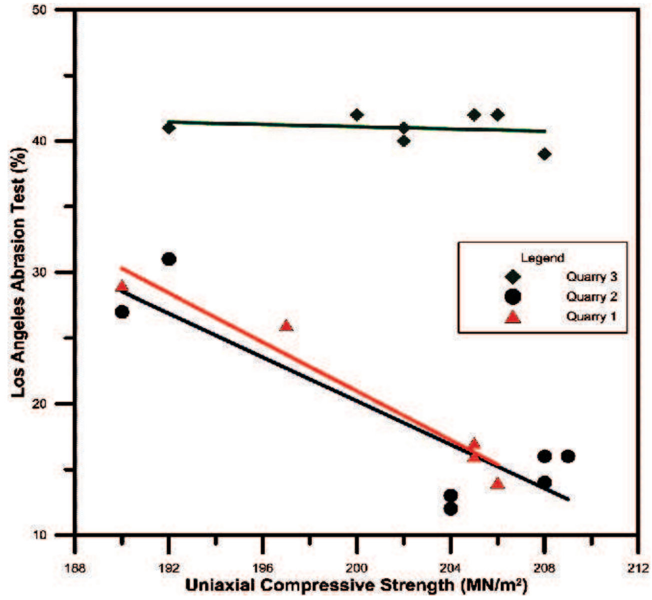


Fig. 10: Los Angeles abrasion value vs Aggregate crushing value.

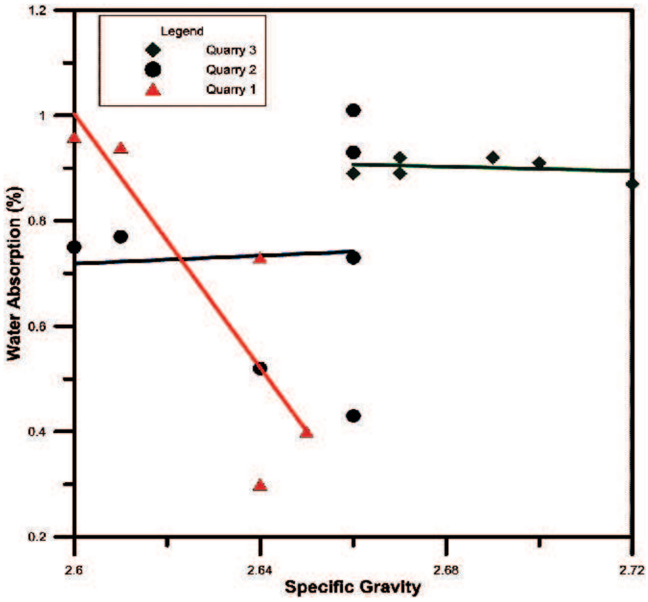


Fig. 11: Water absorption vs Specific gravity

Figure 12 shows an inverse relationship between the uniaxial compressive strength and the Los Angeles abrasion value. This suggests that aggregates with higher uniaxial compressive strength will have low Los Angeles abrasion value. This will give it a higher resistance to abrasive wear, making it suitable for pavement and other civil constructions (Kazi and Al-Mansour, 1980a). For Quarry 3, UCS can serve as a good replacement to Los Angeles abrasion test but it is advisable to investigate each quarry site in its own right to avoid blind substitution of one parameter for another.

Applications

It is expected that aggregates from the quarries under studies will be used in asphalt and concrete mix design for the construction of pavements and foundations as well as other civil construction works. The high value of uniaxial compressive strength, specific gravity and high resistance to crushing and impact load as well as the ability to withstand abrasion make the aggregates excellent materials for civil construction from engineering point of view.

Ranking of the Aggregates for the three Quarries

Though aggregates from the Third Quarry in Okom-Ita show slightly lower quality compared to aggregates from Njahasang, this is attributed to greater impact of weathering on these surface samples. It is believed that when the weathered overburden is removed at the commencement of quarrying in Okom-Ita, the aggregate properties would improve with depth. However, the materials are suitable for construction purposes since they fall within the ranges of acceptable standards in engineering construction.

Conclusions and Recommendations

Conclusions

a. On the basis of the assessments, (Table 3), aggregates from Njahasang and Okom-ita which are part of Oban Massifs are judged to be suitable for civil engineering construction be it a pavement or a foundation.

b. Cross-plotting the parameters (Figures 10, 11 and 12) have produced useful and significant empirical relationships which help to reduce number of expensive tests. However, it is important to note that treating each site by running the required physical and engineering properties tests would avoid blind substitution of one test for another.
c. It is expected that sample properties from Quarry 3 at Okom-Ita will appreciate with depth. For now, on the basis of the test data, Quarries 1 and 2 at Njahasang are better construction materials based on the engineering specification judgment (Table 3)
d. The averages of the test result for all three quarries are suitable for pavement and for foundation works.

Recommendations

1. All quarries which produce rock aggregates should run the necessary geotechnical engineering tests on their products and display the test results on their bill boards and if possible in the print media for public consumption.
2. The general public should request for the results of the physical and engineering properties test prior to the purchase of rock aggregates for civil construction purposes.
3. All granitic rocks are not the same. They should be tested and ranked for civil construction purposes.
4. Government should bring appropriate legislation on the necessary physical and engineering properties tests and ensure compliance by quarry operators.

It is anticipated that with proper designs of pavements and foundations and real supervision of civil works by geotechnically-informed personnels, characterization of rock aggregates employed for such works will surely increase the longevity of civil construction work in the country and elsewhere.

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Engineering Properties of Flexible Pavement and Subgrade Soil along Papalanto – Sagamu Road, Sagamu, Southwestern Nigeria

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Abstract

The Papalanto-Sagamu Road is approximately 43km south of Abeokuta and almost 80km through Logbara-Shagamu Interchange, Ogun State, within the tropical rainforest belt of Nigeria. The worksite consists of an existing 43.5km roadway connecting the Lagos-Abeokuta via Papalanto Junction, Ilaro-Papalanto Road with the Shagamu Interchange. The study area lies within Ogun State, which is bounded in the west by Benin Republic, in the south by Lagos State, in the north by Oyo and Osun States, and in the east by Ondo State. It transverses between Papalanto Junction through several villages and settlement to Shagamu Interchange. The road is host by two/three local government Ewekoro, Ifo and Shagamu/Ado Odo Ota and lies between 7°00 N 3°35′E. The investigation of the pavement and subgrade materials of the existing roadway was carried out to; determine the physical properties, composition, characteristics and quality of the pavement, subgrade materials, perform engineering analyses to evaluate the existing pavement section, summarizing the field, laboratory and engineering recommendations. The defined work scope and executed work scope based on field conditions and available resources for both the supplementary geotechnical and pavement materials investigations are listed below: excavate twenty one (21) test pits of 1.2m x 1.0m x 0.5m (L x B x D) on the pavement; excavate test pits of 1.0m x 1.5m x 1.0m (L x B x D) on the borrow pit sites; carry out stipulated sampling and profiling in the test pits; describe pavement materials and perform laboratory test on selected soil. **Natural Moisture Content**: The range is as low as 1% in TP8A and high as 12% in sample TP 16. **Sieve Analysis:** The samples are mainly sandy clayey soil with percentage sand lowest in sample TP 10B with 44.7% and as high as 93.4 % in sample TP 16. Gravels ranges from 51.2% highest in TP 10B and lowest in 5.6 % in TP 16. Fines as low as 0.3 % in TP 2B and highest with 6.3% in TP 15.**Atterberg Limit:** Plastic Index lowest in TP 10B and TP 13with 6 and highest in TP 14A with 28. Liquid Limit highest in TP 12B with 41 and lowest in TP5A with 23.**California Bearing Ratio:** The strength of the materials in TP 7B is lowest with 1.039 % and highest in TP 14 with 84.98 %. Generally, the samples are mostly gap-graded, poorly graded and clayey sand with gravels. TP 14A is the only sample from those analyzed having a high plasticity, others are either low or medium plasticity; generally having low to medium clay content. The samples generally have high natural moisture content. It is advised to ensure that the section with poor bearing capacity after raising its bearing capacity, be only used as subgrade as a more resistance materials such as crushed stones be used as sub base. The Geo-section shows varying lithology and varieties of materials, a dynamic cone penetration test is therefore recommended to be carried during construction phase so as to determine the strength and field California bearing ratio of each pavement in-situ, compared with laboratory California Bearing Ratio before intervention at each section due to moisture as a result of seasonal variation.

Keywords: *Papalanto-Sagamu Road, Pavement, Subgrade Material, CBR, Physical properties, Composition Characteristics*

Introduction

The project site through Papalanto Junction is approximately 43km south of Abeokuta and almost 80km through Logbara-Shagamu Interchange, Ogun State, within the tropical rainforest belt of Nigeria. The worksite consists of an existing 43.5km roadway connecting the Lagos-Abeokuta via Papalanto Junction, Ilaro-Papalanto Road with the Shagamu Interchange. Pavement materials exhibit different kinds of behaviour. In modelling of materials in flexible pavements, two major categories exist: the bound and unbound layers. Among the bound layers, asphalt concrete shows rate dependency and creep behaviour, and the viscous behaviour is accounted in the constitutive models. For the unbound layers, i.e. subgrade and aggregates, elasto-plastic models are commonly used to formulate the load associated deformation.The roadway was constructed across most relatively stable and some predominantly low-lying areas, and has influx of water into the carriage way during wet season, the asphaltic course has been washed off in most area of the road exposing the sub-base and subgrade material -Laterite defined as soils as all product of tropical weathering with reddish, brown color with or without nodules or concretion but not exclusively found below hardened ferruginous crust of hardpan. Osula (1984) defined laterite as a highly weathered tropical soil; rich in secondary oxides of combination of iron, aluminum and manganese. Laterite (also known as "red soils") is used to cover all tropically weathered soil that has been involved in the accumulation of oxides of iron, aluminum or silica (Malomo, 1977). In other words, red soil is a highly weathered material rich in secondary oxides of iron, aluminum, or both. According to Alexander and Candy (1962) it is nearly devoid of bases and primary silicate, but it may contain large amount of quartz and kaolite. It is either hard or capable of hardening on exposure to wetting and drying (Agbede, 1992) and where it is not, cracks-crocodile like figure. The areas adjacent to the road are subject to flooding in the rainy season. Also, bridges along the road subgrade, which is obtained through investigation and accumulation of test results. In this way the risks regarding the subgrade can be estimated. Variation in the subgrade directly below the pavement cannot be prevented so it is important not to overload the subgrade at any place. The primary structural task of the pavement is to prevent too high stresses in the subgrade.

The behaviour of fine-grained soils such as silt and clay is highly affected by the amount water in soil pores. Particularly, subgrade soils are mostly prone to moisture variation in pavements. A slight change in the level of moisture in the subgrade soils is determinant on the overall behaviour, both on the stiffness and deformation properties. So, the characterization of subgrade soils must comprise both load related and environmental factors. The theoretical aspects of pavement design calculated using mechanics are crucial, but do not stand alone to design pavements realistically. They should be combined with the empirical experience for a better understanding of the pavement materials. In the implementation of a mechanistic-empirical pavement design approach, substantial knowledge about pavement materials should be accumulated. The structural and functional performances of flexible pavement sections are highly dependent on the subsoil conditions.

Objectives

The main objective of this work is to determine the physical properties, composition, character tics and quality of pavement, determining the CBR value of compacted subgrade beneath the existing road pavement and comparison of results between findings and standard design guides.

The Study Area

The study area lies within Ogun State, which is bounded in the west by Benin Republic, in the south by Lagos State, in the north by Oyo and Osun States, and in the east by Ondo State. It transverses between Papalanto Junction through several villages and settlement to Shagamu Interchange. The road is host by two/three local government Ewekoro, Ifo and Shagamu/Ado Odo Ota and lies between 7°00 N 3°35´E (Figure 1).

Fig. 1: Map Source GPS of the Location of the Route Alignment

The study area is located in the sedimentary area of southwestern Nigeria. Ewekoro formation belongs to tertiary-formed Palaeocene and Eocene; and the greater part of the depression is a potential artesian basin where ground water can be sourced. Adegoke et al. (1976) outlined the Albran and younger Palaeographic history of Nigeria and summarized the nature and extent of transgressive, regressive phases as well as the nature of the sediment.

The geology of Ogun State comprises sedimentary and basement complex rocks, which underlie the remaining surface area of the state. It also consists of intercalations of argillaceous sediment. The rock is soft and friable but in some places cement by ferruginous and siliceous materials. The sedimentary rock of Ogun State consists of Abeokuta formation lying directly above the basement complex (Figure 2). Ewekoro, Oshosun and Ilaro formations in turn overlie this, which are all overlain by the coastal plain sands (Benin formation).

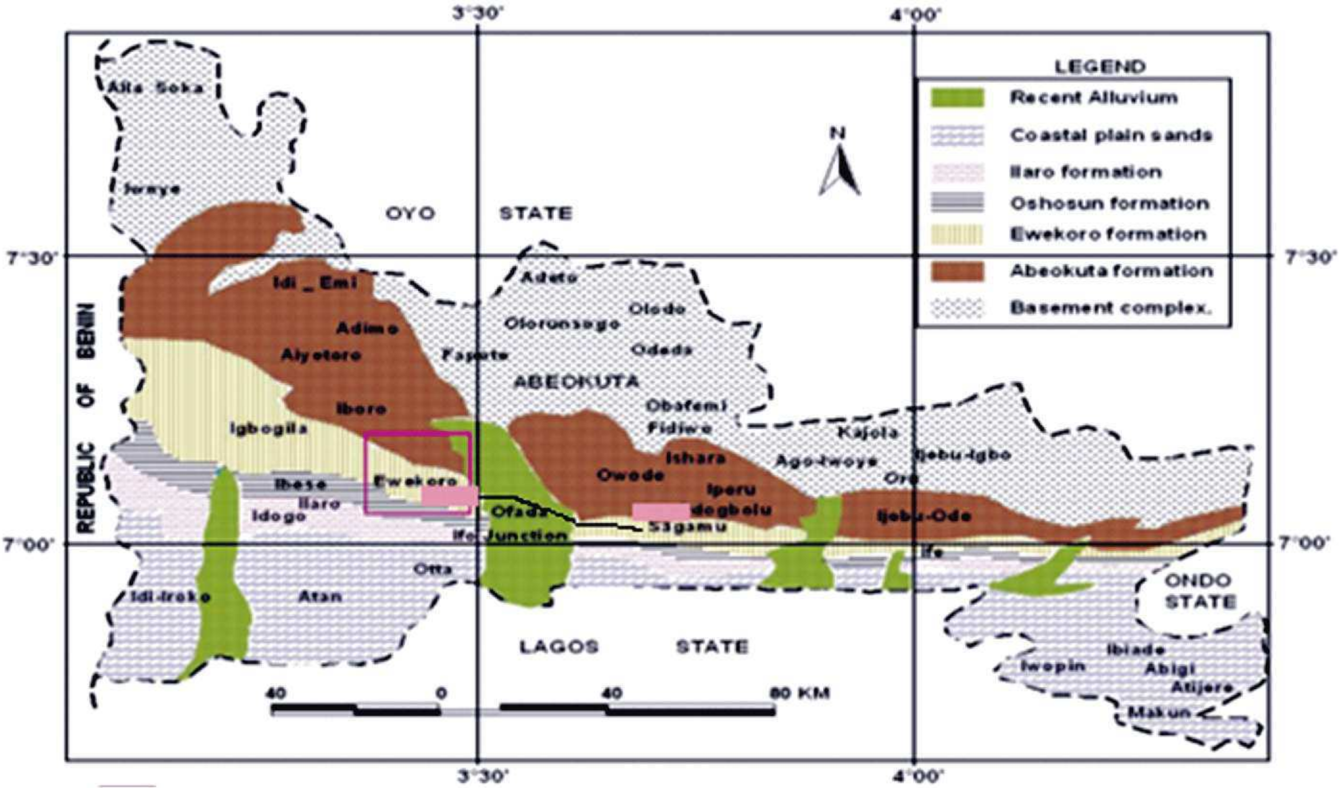


Fig. 2: Geological map of Ogun State showing the study area

Materials and Methods

Materials

Excavate twenty one (19) test pits of 1.2m x 1.0m x 0.5m (L x B x D) on the pavement; excavate test pits of 1.0m x 1.5m x 1.0m (L x B x D) on the borrow pit sites; carry out stipulated sampling and profiling in the test pits; describe pavement materials,perform laboratory test on selected soil; and compare with standards.

Field Investigation

Test pits of 1.2 m x 1.0 m x 0.5 m (L x W x D) were excavated in the pavement. In a zigzag manner to have various representative samples of differing sections of the road-Center, Right, Left. In most cases, a section on the shoulder but closer and part of the carriage way or already washed off asphalt in carriage way were marked. After the portion of the pavement surface to be excavated was clearly marked, virtually all test points are devoid of Asphalt due to the provision for replacement and to avoid additional damage to the road, safety and health implication if material is not adequately replaced and compacted in each of the marked portion, while manual digging using digger is employed was used to open the cut portion down to the pavement base before commencement of the test pit digging. The excavation was carried out with due care, so that each layer/type of material encountered could be sampled separately. The excavation was carried out neatly, by hand, with the sides of the test pit being vertical. The test pits were careful dug, profiling clearly visible, the layers were distinct and recorded on the test pit logs. (see table 1) After the excavation, sampling and profiling, the pits were filled.Each material encountered while digging the test point pits are carefully described visually using color, touch-material/particle size and moisture-dampness.

Table 1: Trial Pit along the Road Alignment

Test Pit No	Easting	Northing	ID on Map
TP1	3.1950767	6.884075	A
TP2	3.2024117	6.8816383	B
TP3	3.21063	6.8750617	C
TP4	3.2174267	6.8691433	D
TP5	3.2228783	6.86715	E
TP6	3.24568	6.8645383	F
TP7	3.2582233	6.866355	G
TP8	3.2782667	6.8691583	H
TP9	3.2929283	6.8736917	I
TP10	3.2928933	6.873695	J
TP11	3.30569	6.87889	K
TP12	3.353298	6.878932	L
TP13	3.3290183	6.8774833	M
TP14	3.3802417	6.8818283	N
TP15	3.41589	6.882105	O
TP16	3.4511183	6.8825867	P
TP17	3.496115	6.88679	Q
TP18	3.5203233	6.8870333	R
TP19	3.5458033	6.8907717	S
BTP1	3.3897267	6.8837933	T
BTP2	3.190065	6.865818	U
BTP3	3.188625	6.893312	V

Laboratory Tests

The soil samples collected were subjected to laboratory test in accordance with BS 1377 (1990), AASHTO specifications (AASHTO, 2004) and ASTM D4318-00 (ASTM, 2000) test methods to characterize the engineering properties of the soils. Sieve, Atterberg limit test, water content and density were determined using the above standards. Visual examination, soil classification testswere carried out on the samples. Compaction test and California Bearing Ratio (CBR) test carried out.

Data Analysis

California Bearing Ratio test (CBR) is a load test applied to the surface and use in soil investigation as an aid to the design of pavement. It is used to measure the pressure required to penetrate a soil sample with a plunger of standard area. The test was performed by measuring the pressure required to penetrate a soil with a plunger of standard area. The measured pressure is then divided by the pressure required to achieve an equal penetration on a standard crushed rock material. The test was conducted in the laboratory first by compacting the

Fig. 2: Photomi

specimen samples on which CBR is to be determined in a standard mould by standardized compactive energy to its maximum dry density and its optimum moisture contents.

Results and Discussion

Laboratory Test Results

From the data analyzed and presented above, various civil engineering parameters such as Natural Moisture Content, percentage passing in sets of British sieves- Co-efficient of Conformity, Co-Efficient of uniformity, Haven permeability, Plastic Index, Optimum Moisture content, Maximum Dry density as well as the California Bearing Ratios of some selected samples has been completed, presented and reference drawn from it. Using the Unified System of Classification of Soil and Burmister Classification for Clayey materials with Relevant Excerpts from Federal Ministry of Works Specification. **Natural Moisture Content**: The range is as low as 1% in TP8A and high as 12% in sample TP 16, **Sieve Analysis:** The samples are mainly sandy clayey soil with percentage sand lowest in sample TP 10B with 44.7% and as high as 93.4 % in sample TP 16. Gravels ranges from 51.2% highest in TP 10B and lowest in 5.6

% in TP 16. Fines as low as 0.3 % in TP 2B and highest with 6.3% in TP 15,**Atterberg Limit:** Plastic Index lowest in TP 10B and TP 13with 6 and highest in TP 14A with 28. Liquid Limit highest in TP 12B with 41 and lowest in TP5A with 23, **California Bearing Ratio:** The strength of the materials in TP 7B is lowest with 1.039 % and highest in TP 14 with 84.98 %. See table 2, 3 &4.

Table 2: Plasticity Index value of the Trial Pit Samples

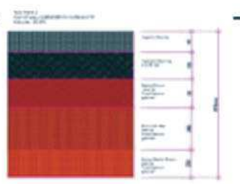
S/N	SAMPLE NUMBER	PI Value	Interpretation	Remarks
1	TP 14A	28	High plasticity	Good
2	TP 3A,6B,8B,12A,12B,15,16, 17	19,18,15,10,12,12,10,10	Medium Plasticity	Very Good
3	TP 2B,3B, 4B,5A,10A,10B,13	9.3,9,7,7,8, 6,6	Low Plasticity	Very Good

Generally, the samples are mostly gap-graded, poorly graded and clayey sand with gravels. TP 14A is the only sample from those analyzed having a high plasticity, others are either low or medium plasticity; generally having low to medium clay content.The samples generally have high natural moisture content maximum

Table 3: Summary of trial pit CBR value

S/N	SAMPLE NUMBER	CBR Value	Interpretation	Remarks
1	TP 7B	1.1	Poor material should be excavated.	Poor
2	TP 4B,16A,17	5.2, 4.6, 6.5	Fair Material for subgrade	Subgrade
3	TP 1B, 6A, 8B,	12.7, 12.5, 15	Good material for subgrade	Sub grade
4	TP 2C,14, 15	32.6, 85,80.9	Good material for sub base	Sub Base

Table 4: Pavement analyses of trial pit profile

PROJECT DEPARTMENT-ROADS					
QUALITY CONTROL/ASSURANCE DEPARTMENT					
PAVEMENT ANALYSIS-TEST PIT PROFILE				Geo-Section of the Test Pits	Pictures of the Test pits Profile
PAPALANTO-SHAGAMU INTERCHANGE ROAD					
S/N	DEPTH (mm)	SAMPLE TYPE	DESCRIPTION		
1	60	Aspahlt			
2	480	Laterite	Reddish Brown laterite		
3	100	Laterite	Dark Brownish Red Laterite		
1	40	Asphalt	Aspalt Overlay		
2	100	Aspahlt	Two layer-Binder and wearing		
3	20	Laterite	Reddish Brown Laterite-Fine-medium grained		
4	200	Laterite	Brownish Red Gravelly Laterite		
5	250	Laterite	Damp Reddish Brown Laterite		
1	230	Laterite	Brownish Red Laterite fine-Medium Grained		
2	70	Asphaltic Laterite	Asphalt with influx of laterite (Looks Reworked)		
3	200	Laterite	Damp Brownish Red Laterite Fine-Medium Grained		

value. Plastic Limit (PL) is between 13.8-14.8%, Liquid Limit (LL) falls between 32.9 -43.4% and plasticity index (PI) ranges from 18.1-28.8%. The presence of clay soil in foundation materials is capable of causing road surfaces failure when in contact with moisture during wet season.

Compaction Test

Figure 5 shows the highest maximum dry density (MDD) of 1.88g/cm^{3}value in TP 12A and lowest with a value of 1.65g/cm^{3}of TP 14. OMC from 8.1 minimum TP3A highest value of 20.9 in TP 17. .

California Bearing Ratio: The strength of the materials in TP 7B is lowest with 1.039 % and highest in TP 14 with 84.98 %.

Generally, the samples are mostly gap-graded, poorly graded and clayey sand with gravels.
TP 14A is the only sample from those analyzed having a high plasticity, others are either low or medium

TEST			PLASTIC LIMIT				LIQUID LIMIT			
Variable	NO Var.	NO Units	1	2	3	4	1	2	3	4
Number of Blows	N	blows					10	13	12	16
Can Number	---	---	3	16	20	19	9	10	19	25
Mass of Empty Can	M_C	(g)	NP				17.00	17.00	18.00	18.00
Mass Can & Soil (Wet)	M_{CMS}	(g)					34.60	31.50	29.20	31.50
Mass Can & Soil (Dry)	M_{CDS}	(g)					31.80	29.10	27.50	29.10
Mass of Soil	M_S	(g)	######	0.00	0.00	0.00	14.80	12.10	9.50	11.10
Mass of Water	M_W	(g)	0.00	0.00	0.00	0.00	2.80	2.40	1.70	2.40
Water Content	w	(%)	######	#DIV/0!	#DIV/0!	#DIV/0!	18.9	19.8	17.9	21.6

Liquid Limit (LL or w_L) (%):	20
Plastic Limit (PL or w_P) (%):	#VALUE!
Plasticity Index (PI) (%):	#VALUE!
USCS Classification:	

PI at "A" Line = 0.73(LL-20)

One Point Liquid Limit Calculation:

$$LL = w_n (N/25)^{0.12}$$

PROCEDURE USED

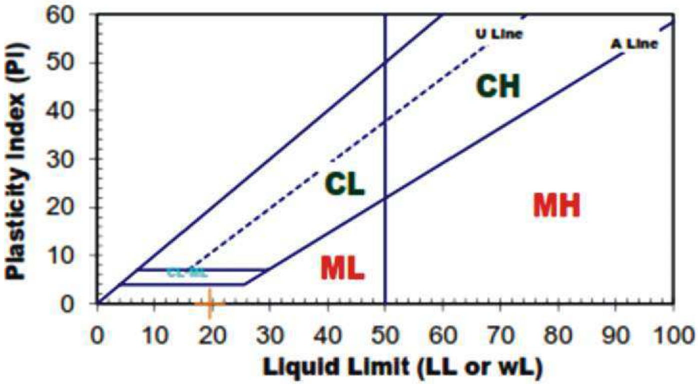


Fig. 4: Attergberg limit of the Trial Pit Samples

plasticity; generally having low to medium clay content. The samples generally have high natural moisture content.

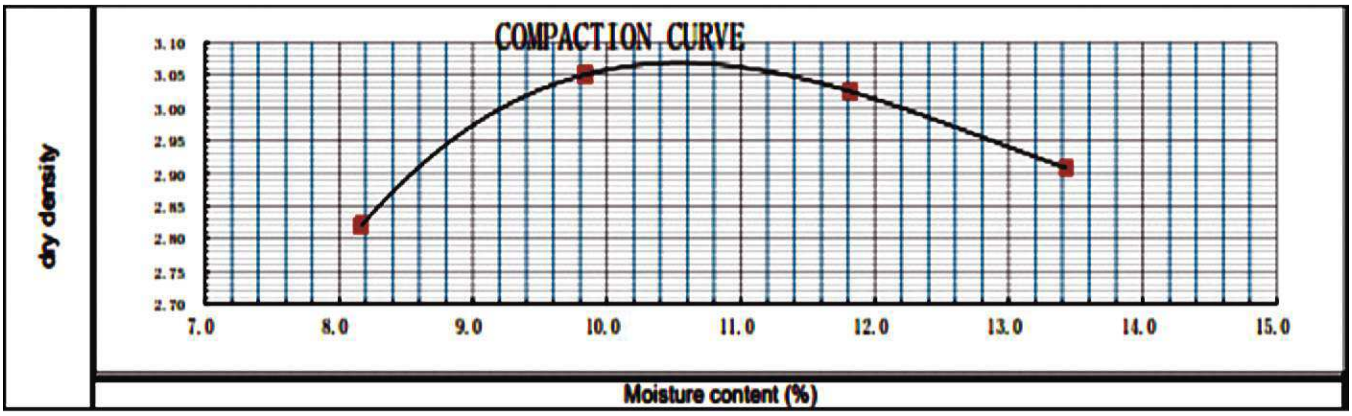


Fig. 4: Attergberg limit of the Trial Pit Samples

California Bearing Ratio: The strength of the materials in TP 7B is lowest with 1.039 % , 12.7% for sample TP1B in fig 6 and highest in TP 14 with 84.98 %. Generally, the samples are mostly gap-graded, poorly graded and clayey sand with gravels.

TP 14A is the only sample from those analyzed having a high plasticity, others are either low or medium plasticity; generally having low to medium clay content.The samples generally have high natural moisture content.

			CALIFORNIA BEARING RATIO TEST				
	Project title: PAPALANTO-SAGAMU ROAD ASSESSMENT PROJECT						
Soil Type: Laterite Compaction: B.S. 56 Blows Location and depth: TP 1B							
Operated by Festus Date: 20/04/2018							
Density Detemination	Before soak	After soak	Water content detemination	Before soak		after soak	
Wt Wet sample mould(g)	9139		container No.	15	4	8	7
Wt of mould (g)	4289		wet soil+ container	153.46	157.34		
Wt Wet sample (g)	4850	0	dry soil+ container	145.57	148.87		
Volume of sample (cm^3)	2109	2177	dry soil alone	102.64	105.96	-16.48	-16.8
Wet density (g/cm^3)	2.30	0.00	container alone	42.93	42.91	16.48	16.80
Moisture content %	7.8	0.0	moisture content	7.69	7.99	0.00	0.00
Dry density (g/cm^3)	2.13	0.00	AV moisture content	7.8		0.0	

Fig 6: California Bearing ratio (for Sample TP1B)

PEN mm	BASE		TOP	
	DIAL	LOAD(KN)	DIAL	LOAD(KN)
0.5	3	0.13		
1.0	6	0.26		
1.5	9	0.39		
2.0	12	0.51		
2.5	18	0.77		
3.0	24	1.03		
3.5	32	1.37		
4.0	40	1.71		
4.5	49	2.10		
5.0	61	2.61		
5.5	74	3.17		
6.0	88	3.77		

CORRECTED LOAD(KN)		
mm	BASE	TOP
2.5	0.8	/
5	2.6	/
CORRECTED CBR %		
mm	BASE	TOP
2.5	5.6	/
5	12.7	/

CBR= 12.7 %

CBR Calculations:

*CBRat2.5mm=Load/13.7 **CBRat5.0mm=Load/20.6

Moisture content at compaction 10.40%

Moisture content after soaking . 13.80%

Prov.Ring factor 0.0428

Expansion after soak			
Initial reading			
Final reading			
Expansion Inch			
Expansion %			
Period of soaking 24 hrs			

Expansion after soak			
Initial reading			
Final reading			
Expansion Inch			
Expansion %			
Period of soaking hrs			

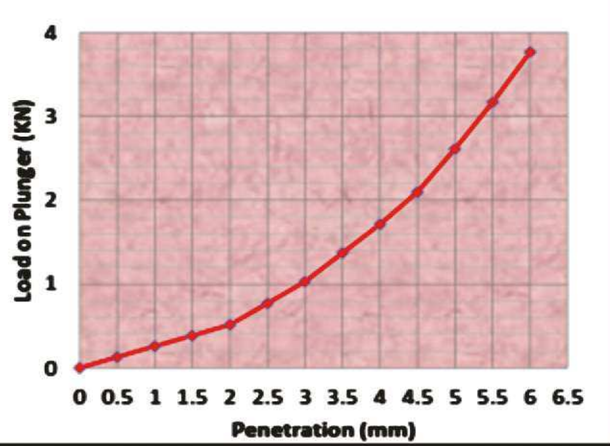


Fig. 7: CBR Result along Traverse One with base, top and corrected load

The lateriticclay, which occurs preponderantly from the ground surface to the end of the investigated depths is appreciably sandy and exhibits a firm to stiff becoming very stiff consistency. One zones of the clay were encountered within the first 1.25 m from the ground surface. Fig 8 shows summary of the investigation findings with picture plate in appendix A-D.

PAPALANTO-SHAGAMU ROAD PAVEMENT AND GEOTECHNICAL MATERIALS TEST RESULT

LABORATORY TEST

S/N	LOCATION ID	TEST ID	Coordinates	Depth	CBR	Compaction		NMC	% SIEVE PASSING								ATTERBERG			Percentage Content			Coefficient of Gradavity Cc	Coefficient of Uniformity Cu	Hazen Permeability K	USCS	Remarks
						MDD	OMC		19	10	4.75	2.36	1.18	0.6	0.3	0.15	LL	PL	PI	GRAVEL	SAND	SILT/CLAY					
1	TP1	TP1A	A					7.83%	100	100	84.5	69.6	65.9	49.5	20.2	2.1				15.5	82.4	2.1	0.85	4.29	4.41		
2		TP1B	A		12.67	3.07	10.6	3.04%	100	100	87.3	77.5	75.2	65.7	36.6	4.8	20	NP	NP	12.7	82.5	4.8	0.86429	2.85714	3.0625	CL-ML	
3	TP2	TP2A	B					8.14%																			
4		TP2B	B					7.54%	100	100	64.73	40.14	30.96	15.49	3.41	0.32	33.3	24	9.3	35.1	64.1	0.3	0.82	9.44	20.25	GP-CL	
6		TP2C	C		32.62	2.85	13.0	6.36%	100	100	83.7	71.8	69.2	58.9	29.7	3.3	30	19	11	16.3	80.5	3.2	0.76	3.29	3.61	GP-GC	
7	TP3	TP3A	C		75.00	2.65	8.1	4.24%	100	100	63.9	48.9	45.6	33.6	13.9	1	29	10	19	36.1	62.9	1	0.26	14.81	7.29	GP-GC	
8		TP3B	C					6.44%	100	100	76.85	63.55	45.43	21.92	4.83	1.34	33	24	9	23.1	71.1	1.3	2.00	5.33	4.41	GP-GC	
9																											
10	TP4	TP4A	D					8.22%	100	100	76.1	47.5	42.1	27.6	10.8	0.3				23.9	75.9	0.3	0.49	10.83	9		
11		TP4B	D		5.19	3.225	13.3	5.88%	100	100	59.4	35.7	32.1	22.4	10.6	2.2	29	22	7	40.6	57.2	2.2	4.76	2.33	9	GP-GC	
12																											
13	TP5	TP5A	E					4.78%	100	100	86.6	77.5	75.2	60.6	25.9	2.5	23	16	7	13.4	84.1	2.5	0.65	3.08	3.8025	GP-GC	
15																											
16	TP6	TP6A	F		12.466	3.025	9.4	2.83%	100	100	88	58.2	48.5	25.5	7.2	0.9				12	87.1	0.9	0.56	7.14	12.25		
17		TP6B	F					5.42%	100	100	90.4	72	64.6	36.5	9.7	0.7	39	21	18	9.6	89.7	0.6	0.88	3.50	9	GP-GC	
18		TP6C	F					11.20%	100	100	88.8	79.4	76.8	65.4	35.2	4.3				11.2	84.6	4.3	0.88	3.00	3.0625		
19																											
20	TP7	TP7A	G					3.26%																			
21		TP7B	G		1.0388	3.05	11	1.94%	100	100	98.9	91.8	87.1	58.3	9.9	1.2				1.1	97.6	1.2	0.94	2.08	9		
22																											
23	TP8	TP8A	H					0.67%																			
24		TP8B	H		14.959	2.75	18.5	7.57%	100	100	86.8	74.7	71.2	56.2	24.9	3.7	42	27	15	13.2	83.1	3.6	0.90	3.59	3.8025	GP-GC	
		8C	H					2.57%																			
25																											
26	TP9	TP9A	I					5.32%	100	100	94	83.9	79.3	58	21.8	3				6	91	3	1.08	3.25	4		
27																											
28																											
29	TP10	TP10A	J					6.51%	100	100	90.3	79.7	74.5	23.5	7.5	4.5	30	22	8	9.7	85.8	4.7	1.07	2.47	14.82	GP-GC	
30		TP10B	J					8.91%	100	100	48.8	34.6	31.9	13.5	8.3	4.2	29	23	6	51.2	44.7	3.8	-	0.00	23.52	GP-GC	
31																											
32	TP11	TP11A	K					8.57%																			
34		TP11B	K					7.71%	100	100	95.6	80.5	73.6	46.7	15.2	1				4.4	94.6	1	0.90	3.20	6.25		
35																											
36	TP12	TP12A	L			3.3	10.1	7.59%	100	100	91.9	77.9	73	51.3	23.2	6.3	34	24	10	8.1	85.6	6.2	0.96	4.27	3.4225	GP-GC	
37		TP12B	L					7.88%									41	29	12								
38																											
39	TP13	TP13	M					9.31%	100	100	89.7	79.8	56.4	26.9	8.9	4.2	35	29	6	10.3	85.5	2	0.93	4.31	10.56	GW-GC	
40																											
41	TP14	TP14	N		84.975	2.535	12.2	9.05%	100	100	84.7	80.9	79.9	72.4	37.9	3.4	38	10	28	15.3	81.3	3.4	0.76	3.29	3.61	GP-GC	
42																											
43																											
44	TP15	TP15	O		80.914	2.625	15.75	10.46%	100	100	88.1	81.7	77.2	70.5	41.7	6	32	20	12	11.9	81.5	6.3	0.83	2.36	3.24	GP-GC	
47																											
[redacted]	TP16	TP16A	P		4.57	2.8	17	12.11%	100	100	94.4	81.7	75	45.1	12.8	1	29	19	10	5.6	93.4	0.9	0.07	3.00	7.5625	GP-GC	
49																											
51																											
52	TP17	TP17	Q		6.44	2.75	20.9	10.86%	100	100	88.7	74.3	71.1	35.5	7.3	0.7	40	30	10	11.3	88	0.7	7.47	1.01	11.90	GP-GC	
55																											
56	TP18	TP18	R					11.13%	100	100	86.2	83.4	82.8	76.5	44	9.3	28	18	10	13.8	76.9	9.3	0.02	2.52	2.7225	GP-GC	

Fig. 8: Summary of Findings of the Results

Relevant Excerpts from Federal Ministry of Works Specification

Clause(s)	Specification	Remarks
6251 6252	Liquid Limit LL n.g.t 30% and PI n.g.t 6% Continuously graded aggregates falling into Grading Envelope A-E % Passing BS #200 n.g.t 1/3 of % Passing BS #36	
6201	Materials Suitable for Sub Base % Passing Sieve #200 /≥ 35 % Liquid Limit LL /≥ 30 % Plastic Index /≥ 12 % Relative Compaction /≥ 95 %	
6201 & 6122	Materials Suitable for Sub Base % Passing Sieve #200 /≥ 35 % Liquid Limit LL /≥ 50 % Plastic Index /≥ 30 % Relative Compaction /≥ 90 %	Specification is Silent on CBR. But 5% minimum is recommended

Conclusion and Recommendations

The Project is an assessment of the pavement and geotechnical analysis of Papalanto - Sagamu Road in Ogun state. When fully rehabilitated the road would increase the socio-economic activities of Sagamu, Papalanto and Ewekoro including communities and towns in the corridor of the project. It would also provide ease of access to market to Industries within the area such as Lafarge Africa and Dangote Plc. The road is in a critical condition, moderately trafficked with pronounced heavy vehicles during the course of investigation.

With the high moisture content despite storage of materials of about 8.5% on average after a week of demobilizing before materials were received in the laboratory. It is advised that the materials be stabilized to reduce the moisture and increase its strength to meet the volume of traffic projected on the road. The strength of the material can be alleviated using quarry fines and chippings to reduce plasticity where necessary, also sandy materials imported from dredging sites meeting the FMW specifications for sand can also be used with appropriate stabilization conforming to the nature and type of materials at each section. It is advised to ensure that the section with poor bearing capacity after raising its bearing capacity, be only used as subgrade as a more resistance materials such as crushed stones be used as sub base.

The Geo-section shows varying lithology and varieties of materials, a dynamic cone penetration test is therefore recommended to be carried during construction phase so as to determine the strength and field California bearing ratio of each pavement in-situ, compared with laboratory California Bearing Ratio before intervention at each section due to moisture as a result of seasonal variation.

Further design recommendations will be made by the Road Structural Engineer after proper analysis and interpolation with traffic studies and strength of pavement materials.

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APPENDIXES

Appendix A: Picture plate for Trial pit

Appendix C: Picture plate for Lab works

Appendix B: Picture plate for Trial pit TP5

Appendix D: Picture plate for Trial pit lining with sampling depth

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Geoethics: Scope and Dynamics

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Abstract

Geoethics is the union of the prefix "geo" and the word "ethics". This means responsibility towards the Earth, an ethics for the planet. Geoethics involves research and reflection on the values which underpin appropriate behaviours and practices, wherever human activities interact with the Earth system. Geoethics deals with the ethical, social and cultural implications of geoscience knowledge, education, research, practice and communication, and with the social role and responsibility of geoscientists in conducting their activities. Geoethics recognizes that human beings are a geological force capable of acting on natural environments, and in virtue of this prerogative assigns to them an ethical responsibility towards the Earth system. Studying and managing the Earth system, exploiting its geo-resources, intervening in natural processes are actions that involve great responsibilities towards society and the environment are the exclusive reserve of geoscientists. Only by increasing the awareness of this responsibility, can we work with wisdom and foresight, and respect the balances that exist in nature while guaranteeing a sustainable development for future generations. Promoting Geoethics articulates the responsibilities of geoscientists to improve both the quality of professional work and the credibility of geoscientists, to foster excellence in geosciences, to assure sustainable benefits for communities, as well as to protect local and global environments; all with the aim of creating and maintaining the conditions for the healthy and prosperous development of future generations. Equally as important to the success of the scientific enterprise are the personal attributes required of being a scientist and the responsible conduct of scientists in their personal interactions with colleagues and the public. The paper therefore provides an overview the scope and dynamics of the emerging field of geoethics, by showing the trajectory that has led to the current point of development of geoethics and suggesting some cues for thought for further advancements of ethical thinking in geosciences.

Keywords: *Geoethics, geo-resources, professionalism, ethical requirements, integrity, professional standards*

Introduction

The professional duties of geoscientists go beyond scientific and technological knowledge and skills, due to the fact that ethics must be an important part of the professional responsibility of geologists. (Martinez-Frias, *et al*., 2009). Ethics is part of our professional responsibility. Without attempting to be exhaustive, ethics is defined as: (i) "the philosophical study of the moral value of human conduct, and of the rules or principles that ought to govern it"; (ii) "a code of behavior considered correct, especially that of a particular group, profession, or individual" and (iii) "the moral fitness of a decision, course of action, etc" (Collins English Dictionary). Generally, Ethics is the field of knowledge that deals with the principles that govern how people behave and conduct activities. Professional ethics on the other hand, refers to "those principles that are intended to define the rights and responsibilities of scientists in their relationship with each other and with other parties including employers, research subjects, clients, students, etc.", (Chalk *et al.,* 1980).

Ethics is well established as being of relevance to other scientific disciplines (e.g., medical ethics, bioethics). Given the multiple interfaces of geoscience with society, it is appropriate that the social role and responsibilities are considered – geoethics. This is not just a niche area of research, but extends to all geoscientists irrespective of their field (e.g., volcanology, engineering geology, hydrogeology, metamorphic petrology) and employment sector (e.g., industry, academia, public sector). Geoethics is particularly concerned with the way humans relate to the geosphere (Matteucci *et al*., 2014). At the same time, Geoethics focuses on how geologists develop their academic and professional work which impacts in sustainability (Almeida and Vasconcelos, 2014). Therefore, geoethics is a subject that discusses principles which guide scientists on how to deal with the non-living part of the planet.

Geoethics was born in 1991 at the junction of *Ethics* and *geology* (Nemec, 2005a; 2007) despite the term being later used with various meanings which are not directly related to Geology and Geosciences (Rothblatt, 2003, Cascio 2005, Pumain, 2009, Brennetot, 2010). Broadly, it derives from 1996, when a group of geographers from North America and the UK envisioned exploring more thoroughly the relationship of geography and moral philosophy (Proctor, 1996), and a specific listserv named "Geo-Ethics" was made on geography, ethics and justice (Lynn,1996)

Geoethics was born to define a conceptual substratum of categories, useful as framework of reference for geoscientists, to help them develop a new way of thinking and interacting with the Earth system (Di Capua, 2017). Geoethics provides a framework for reflecting on the shared values that underpin the work of geoscientists, and how these values shape the professional actions, and interactions with colleagues, society and the natural environment.

The International Association for Promoting Geoethics and the 'Cape Town Statement on Geoethics (CTSG)' defines geoethics as:

"Research and reflection on the values which underpin appropriate behaviours and practices, wherever human activities interact with the Earth system. Geoethics deals with the ethical, social and cultural implications of geoscience knowledge, education, research, practice and communication, and with the social role and responsibility of geoscientists in conducting their activities."

The "Cape Town Statement" statements provide clear references to the eminent position of geoethics as an under-pinning principle for geoscientists and geo-scientific information and knowledge to follow while addressing societal challenges and the SDGs. Geoscientists have specific knowledge and skills, which are required to investigate, manage and intervene in various components of the Earth system to support human life and well-being, to defend people against geohazards and to ensure natural resources are managed and used sustainably. As such, geological services include geoethics values as they are aimed at safeguarding the exploration and safe development of natural resources and subsurface capacities based on impartial and scientifically validated information and knowledge, always having the interests of society as a core value. This entails serious ethical obligations. Therefore, geoscientists must embrace ethical values in order to serve the public good.

The fundamental values of geoethics (Di Capua *et al*., 2017), involves the following:

(a) Ensuring sustainability of economic and social activities in order to assure future gen-erations' supply of energy and other natural resources.
(b) Sharing knowledge at all levels as a valuable activity, which implies communicating science and results, while taking into account intrinsic limitations such as probabilities and uncertainties.
(c) Verifying the sources of information and data, and applying objective, unbiased peer-review processes to technical and scientific publications.

Geosciences have major impacts on the functioning and knowledge-base of modern societies. Humans are recognized as a "geological force", capable of modifying natural environments, and in virtue of this prerogative they have an ethical responsibility towards the planet. By studying and managing the Earth system, exploiting its geo-resources, intervening in natural processes are actions that involve great responsibilities towards society and the environment, of which perhaps geoscientists, are not sufficiently aware. Only by increasing the awareness of this responsibility, can geoscientists work with wisdom and foresight, and respect the balances that exist in nature while guaranteeing a sustainable development for future generations.

Geoethics as an emerging subject promotes a way of thinking and practicing geosciences, within the wider context of the roles of geoscientists interacting with colleagues, society and the planet Martinez-Frias, *et al*., 2011). Only by guaranteeing the intellectual freedom of researchers and practitioners to explore and discover in the Earth system, is it possible for geoscientists to follow ethical approaches in their work. Likewise, only by increasing re-searchers' and practitioners' awareness of the ethical implications of their work is it possible to develop excellent geoscience to serve society and to reduce the human impact on the environment. This paper therefore, summarizes the fundamentals of geoethics, higlights its institutionalization and current development and emphasizes the significance of geoethics, providing primary information about its innovation and progress as well as the current and future developments in geosciences.

Geoethics: Historical Overview

The word "geoethics," as used starting from the early 1990's (Savolainen 1992; Cronin 1993), signifies the duty of mankind to behave responsibly and become the natural consciousness of the planet. Geoethics was born in 1991, and it was established as an independent scientific field in 1992, in the context of the symposium "The Mining Pøíbram in science and technique". Dr. Vaclav Nemec (since 2004 Vice-president for Europe of the Association of Geoscientists for International Development - AGID, Head of the AGID Working

Group for Geoethics) is considered the father of this discipline. As Nemec stated "he was inspired by the field of business ethics, where his wife, Lidmila Nemcova, had been engaged, as represented by the prestigious French Professor Jean Moussé" (Moussé, 2001), to start to investigate problems of ethics applied to the Earth sciences.

Geoethics recognizes the contingency of human evolution on the planet. Pievani, (2009, 2012) identifies *Homo sapiens* as geological force acting on the geological and biological environments, and assigns to humans an ethical responsibility that arises from the consciousness of being a modifier of Earth systems. There had been attempts to date formulation of Geoethics in 1973, when Antonio Stoppani, Italian geologist and palaeontologist proposed an idea of introducing the anthropologic era into the geochronological scale - an era of domination of Homo sapiens that significantly affected to the natural environment. In 1980's, this idea was captured by Eugene Stoermer, American ecologist, and in 2000, it was popularised by Paul Crutzen, Nobel Prize winner for chemistry as a proposal of the Commission for Stratigraphy of the Geological Survey of London to use the term "anthropocene" that indicates the geological epoch with the level of human activity that plays a significant role in the Earth ecosystem (Crutzen and Stoermer, 2000). The advanced statements did not mean formulation of Geoethics in the rank of a scientific discipline. This was more occurrence of ecological way of thinking. While formulation of ecological ethics was based on awareness of significance of the impact of human activity to natural systems and crust of the planet, together with this awareness, Geoethics was originated by the following assumptions:

(i) accumulation of geological knowledge that has facilitated understanding of geographic irregularity of distribution of mineral deposits, their limitation in volume/size, exhaustibility, non-renewability, potential for high economic, environmental and social risks that are associated with mining;

(ii) occurrence of ethical problems like fair distribution of income from mining of minerals, the minerals belonging not to contemporary, but also future generations, responsible (irresponsible) subsoil use, acceptability (unacceptability) of destruction and disappearance of geological objects and systems that are classified as non-renewable resources, ethical collisions that arise in prognosticating geological calamity processes (eruptions, earthquakes, landslides, floods) etc.

Thus, determination of Geoethics as a science, classification of Geoethics into an independent philosophic discipline owes to Vaclav Nemec. He and his associated and followers from different countries specified the objectives of Geoethics, objects and targets of its studies. The international institutionalisation of geoethics was established in 2004, by forming a working group for Geoethics with the backing of the Association of Geoscientists for International Development (AGID). Therefore in 2008 Geoethics was for the first time incorporated in the official programme of the 33rd International Geological Congress under the auspices of AGID in Oslo (Nìmec and Nìmcová, 2008), whereas the previous symposia to this object in previous Congresses were mostly based on a "private" initiation of Vaclav Nemec, Lidmila Nemcova and once also of Professor W.S. Fyfe (former IUGS President).

However, despite the fact that more and more scientists have to some extent considered geoethical issues in their research works, Geoethics still looked a little-known scientific discipline. At its initial stage of development, Geoethics as a new scientific trend, it was important to formulate the notion "Geoethics" itself. Geoethics combines a complex of ethical problems, associated with geological scientific studies, practical geological exploration works, mining and use of mineral-raw resources, being one of the most important components of the natural environment, by preserving the geo-diversity and geo-heritage, by development and implementation into practice of professional codes of conduct. One way or another, but today all researchers agree with the fact that Geoethics is a notion that includes moral principalities in relation to the Earth as a geological body, and to social and economic objects in all their diversity (GSIED, 2006).

The incorporation, through Geoethics, of new key questions associated to the "abiotic world" is, besides widening the classical concept of Planetary Protection, giving an additional dimension to the geological research of the solar system (including the study of meteorites, asteroids, comets, planets and moons) (Martinez-Frias, *et al*., 2009). The life in which these astronomical and geological processes culminate is still more impressive, but it is of a piece with the whole projective system (Rolston, 1988). Thus, inanimate objects and the abiotic processes of nature also possess objective value under Rolston's ethic (Sheppard, 2000). To this end, the new planetary facet of Geoethics

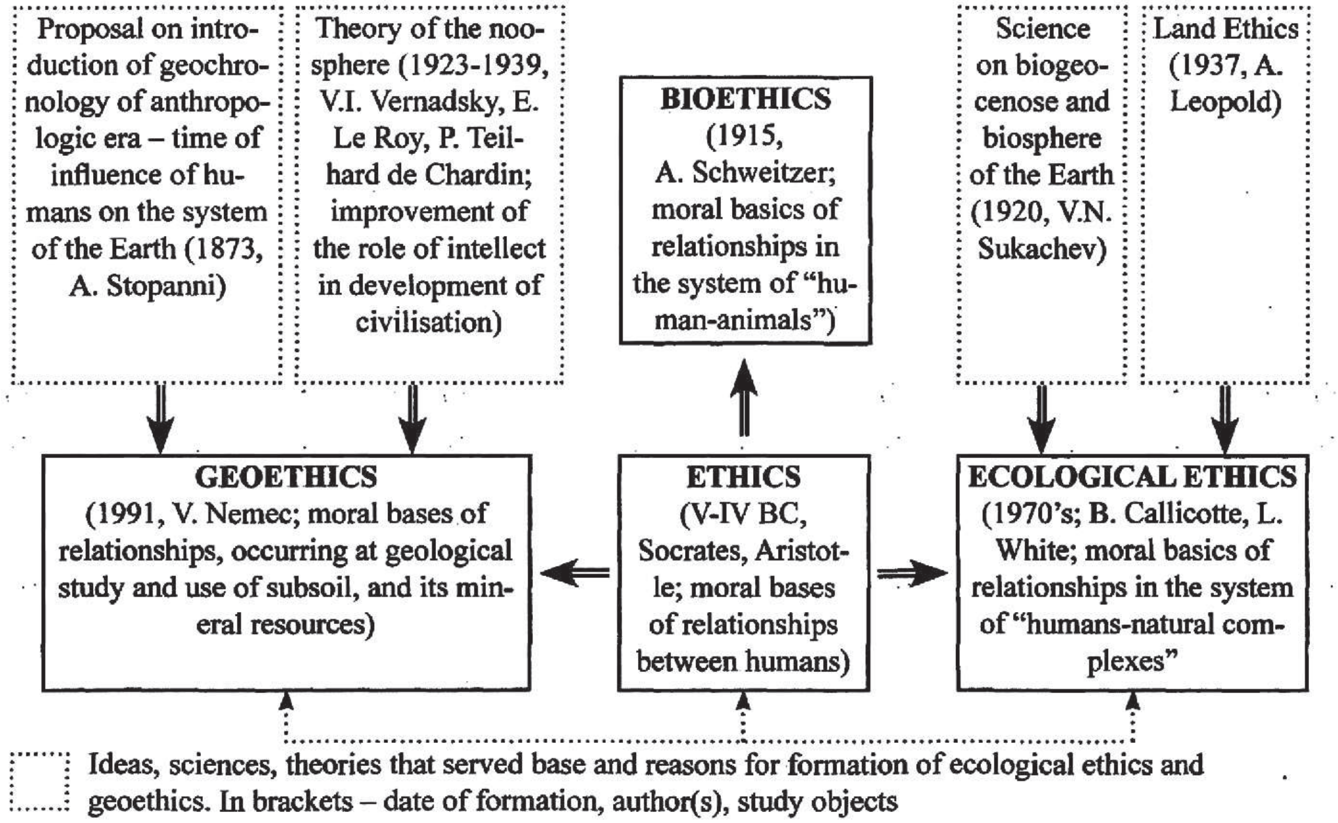

Fig. 1: Basics and history of formation of geoethics (Source: Nikitina, 2016).

involves a new paradigmatic use of the term, extending the scope of the definition of Geoethics beyond the Earth (although maintaining the original Nemec's foundational spirit) (Nemec, 1992). Taking into account this additional perspective, the following formal definition of Geoethics is proposed (Martinez-Frias, 2008): Geoethics is a key discipline in the field of Earth and Planetary Sciences, which involves scientific, technological, methodological and social-cultural aspects (e.g. sustainability, development, museology), but also the necessity of considering appropriate protocols, scientific integrity issues and a code of good practice, regarding the study of the abiotic world. Studies on planetary geology (*sensu lato*) and astrobiology also require a geoethical approach.

The Earth System Relationships

Stoppani, (1871-1973 and 1873) conceived the earth as a *"great machine governed by a Supreme Principle"* that exists through a continuous and simultaneous competition including antagonism from endogenous agents, physical forces, chemical and mechanical properties, that *"are called to entertain what you can call globe life"*, but which also contribute to *"the biological forces order to maintain that wonderful circle in motion, that noble balance, so there is variety together with units.* The force that maintains the balance in the world, considering the complexity and the relational system that characterizes the Earth, is the continuous antagonism of natural agents.

Stoppani approaches the more recent Gaia hypothesis by James Lovelock and Lynn Margulis, that conceives the Earth as a single organism in which living and non-living things are connected to each other by negative retroactive processes that tend to maintain the stability of the main parameters that allow life on the planet (Lovelock, 1979). Stoppani, albeit in the simplicity of his language and based on the knowledge of his time, seems to have launched the foundations for a modern approach to the study of the Earth system from a perspective of dynamic relationships between the various components of the ecosystem. This vision is now indispensable for environmental protection, proper management of geo-resources, evaluation and risk mitigation, all closely related with geo-ethical issues and common good (Figure 2).

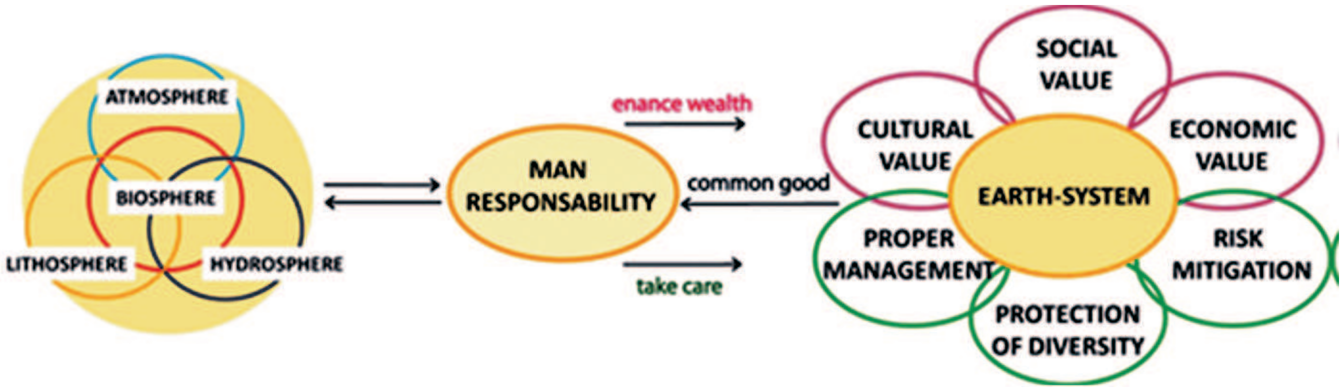

Fig. 2: Sketch of the dynamic relations inside the Earth system and their interconnection and relationship with human's responsibility (Source: Lucchesi, 2017).

Foundations of Geoethics and Geoethical Mission

The mission of Geoethics is in implementation of the values approach, values criteria in practice of geological exploration and mining activities, use of mineral resources and preservation of objects of inorganic nature (geo-heritage) as opposed to self-interest and (individual, corporate, state) mercantilism (Nikitina, 2016). The object of study of Geoethics is morals in the field of study of subsoil of the Earth and other planets that contain mineral-raw resources, in the field of reproduction of the mineral-raw base, mining and use of mineral-raw resources and useful properties of subsoil, while the subject of its study are pragmatic sciences for starting from and surpassing the latter. Geoethics can fulfil the noble role of regulating the behaviour of people in the system of "human-inanimate nature". As a science about morals, Geoethics studies the process of motivation of behaviour, general orientation of relationships in the said system, justifies the necessity and most expedient form of the rules of joint existence of this system, which humans are prepared to accept and fulfill based on voluntary intention. Position and relation of Geoethics with other sciences is shown in Figure 3.

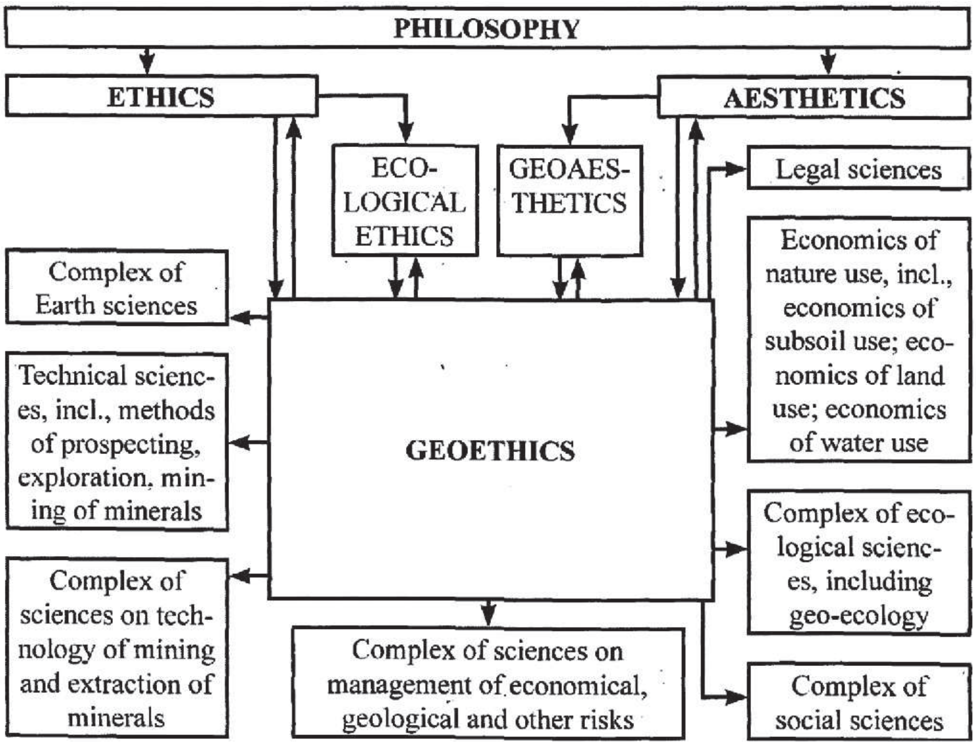

Fig. 3: Interrelation of Geoethics with other sciences (Source: Nikitina, 2016)

Morals in the field of study of subsoil of the Earth and other planets, reproduction of mineral resources and their use as it is, occurs in the history of the society when there is a freedom of choice, possibility of fulfilling these processes in a different way, by preferring this or that system of valuables. Such choice is only possible in accordance with some ideas, on the basis of contrapositioning of "true" and "false" targets owing to establishing of understanding of the true mission of man by way of realising the position and role of humans in the nature system of the planet Earth (Nikitina, 2016). For the period of its existence, being a short time for a science, there are several practical justifications for expansion of the moral field to all objects of inorganic nature and all spheres of the Earth and other celestial bodies: lithosphere, hydrosphere, atmosphere, relief, landscapes, and the circumplanetary space. The subject of study of Geoethics is morals in the field of study and use of maximum large conglomerate of geological and geographical environments and their systems that cover any planet (and not only the Earth) as a single unit and that are combination of various parameters of inorganic nature, which are in close indissoluble connection, while on the Earth they are involved in the globalisation process (Nikitina, 2016).

At the initial stage of formation of Geoethics as a scientific discipline (1992-2012), in the process of formulation of definitions, specification of objectives, purpose, objects and subjects of these categories, many scientists tried to maximise the extent of the list of each category, often, possibly, by incidentally including some objects and subjects of studies, purpose and objectives of ecological ethics. There existed another extremity. Some philosophers did not see any problems that could be resolved using already existing ecological ethics and directly refused Geoethics in its right for existence. It is possible that in near future all applied ethical disciplines, related with study and use of organic and inorganic systems of the earth, will be combined into a single science – something like the Ethics of the Earth (Nikitina, 2016; Lucchesi, 2017).

The American futurologist, Jamais Caascio, is known for his works on prognostics and development of moral norms of future life. He defines the ethics of the Earth as *"a set of guideline principles, which should determine human behaviour and deeds that deal with large planetary systems, including atmospheric, oceanic, geological and ecosystems of flora and fauna. These guideline principles are especially necessary, if human behaviour and deeds may lead to long lasting, large scale and/or difficult to repair changes in planetary systems; but even local and surface changes should be considered through the prism of the Ethics of the Earth. The principles of the Ethics of the earth do not ban long term, large scale transformations, but require mandatory prognostication and accounting of consequences, including so called "secondary order effects", in other words undeliberate consequences, that are the results of interaction of the changed system with other connected systems" [Cascio, 2007].*

Geoethics is primarily based on perception of the planet Earth, its geological spheres, its subsoil, and all geological objects as the base of the life of humanity, on acknowledgement of equality and equivalence of inorganic matters, and on limitation of the rights of people in relation to inorganic nature. Within the framework of these new global ethical assumptions, humanity is trying to rethink the main issues of the entire complex of earth sciences. Combination of geoscientific problems (geographic unevenness of distribution of mineral deposits on the planet, exhaustion of mineral resources, constant growth of costs for discovery of such, natural and commercial risks for development, increase of the coverage area of protected natural territories etc.), main ethical achievement (responsibilities, rights and justice, responsibility of generation, religious beliefs in secular societies, etc.) and possibilities of such practical instruments like local and global geological knowledge, prognostics, scientific expertise of various projects and participation of citizens in decision making, allow formulating the following main geoethical postulates:

(a) natural, including mineral resources have specific internal properties that do not allow reflecting certain elements of their value in market prices or in any other similar utilitarian units of measure of value (Wilson, 1994);
(b) geographic unevenness of distribution of mineral deposits on the planet requires using principally new global approaches to management and use of mineral resources, and to distribution of waste from development of such;
(c) exhaustion of mineral resources, limited volume and finiteness of such cause the issue of access, rights of currently living and future generations for mineral resources;
(d) the geography of world mineral resource mining is expanding: it at least depends on availability of mineable mineral deposits in a given territory, and it to larger extent is determined by social conditions and requirement of nature protection legislation of

the given territory; moving mining centres to poorly developed counties has become a tendency;

(e) sustainable development assumes priority use of secondary resources, re-processing of which does not cause a destructive effect to all spheres of the Earth, which happens at initial (primary) extraction and processing of minerals.

(f) the nature, landscapes, biological diversity of species, subsoil should be treated not simply as objects of protection in the territory of mining and processing of minerals, they are primarily the objects of heritage for future generations [Abel and Varet, 2007].

The Facets of GeoEthics

GeoEthics has many important facets and explores four important dimensions:

(i) **GeoEthics and self**: What are the internal attributes of a geoscientist that establish the ethical values required to successfully prepare for and contribute to a career in the geosciences?

(ii) **GeoEthics and the geoscience profession**: What are the ethical standards expected of geoscientists if they are to contribute responsibly to the community of practice expected of the profession?

(iii) **GeoEthics and society**: What are the responsibilities of geoscientists to effectively and responsibly communicate the results of geoscience research to inform society about issues ranging from geohazards to natural resource utilization in order to protect the health, safety, and economic security of humanity?

(iv) **GeoEthics and Earth**: What are the responsibilities of geoscientists to provide good stewardship of Earth based on their knowledge of Earth's composition, architecture, history, dynamic processes, and complex systems?

Microethics Versus Macroethics

***Micro**ethics* deals with personal and professional ethics and can be tied to responsibilities at the personal and intra-professional level (e.g. an environmental consultant's ethical responsibility to providing their client with reliable data).

***Macro**ethics* deals with the ethics of a society or culture and can be tied to personal and professional responsibilities towards society (e.g. environmental consultants' responsibilities - as a profession - to ensure environmental stewardship in their professional conduct).

The understanding micro and macroethics and the interplays between them illuminates the roots of ethical thinking and behavior (why do we view things the way we do) and can help establish guidelines for ethical standards. The understanding of the interplay can help shift thinking and behaviors by getting to the roots of why we think and act the way we do. For example, taking action to reduce one's carbon footprint is tied to both microethics and macroethics. At a microethical level, our beliefs about the impact of humans on climate will influence our perception of responsibilities and stewardship and will guide us on whether or not to take actions such as driving and consuming less to lessen our footprint (Grigorev, 2005). At a macroethical level, our professional responsibilities to be stewards to the Earth and abide by the high ethical standards set by society will influence how we conduct research and report data. The subject of study of Geoethics includes geoethical situations, geoethical problems and geoethical dilemmas.

Geoethical Situations: This occurs when there are two different points of view in relation to the issue of what is acceptable or inacceptable in a specific situation. For instance, as a whole, geoethical situations occur every time when a decision has to be made on commercial developing of a mineral deposit, if there are two equivalent objects, there are two(or more) options of its development methods. A fair decision in such a case would be based on a complex analysis of existing geological, economic, environmental and other information, on assessment of the objectiveness, reliability and completeness of information, drawing of conclusions on the basis of the above to facilitate a correct choice.

Geoethcal Problems: These are more sophisticated than geo-ethical situations for they assume the presence of several possible ethical decisions. For determination of content and decision of the problem, it is necessary to have time and collective common sense to determine the best option out of all available decisions for all interested parties.

Geoethical Dilemmas: This occurs when, in any case, upon making any decision one of the sides incurs losses. For instance, for various reasons when local population acts against mining of mineral resources in the territory of their habitat. In this case, it is necessary to choose the least of several evils, for no decision would be good for all. Often, dilemmas are caused in crisis situations, for instance, during natural calamities.

Geoethical Principles and Imperatives

In moral geoethics system the main element is represented by the principles that determine the strategy of moral behaviour and its unconditional moral orientation in its general terms. The principles were formulated in different years by different authors mostly for allied sciences (ecological ethics, global ethics) and later introduced into geoethics, but all of these are based on the essential properties of mineral resources – deficiency (limitation), exhaustibility, non-renewability and belonging not only to currently living but also to future generations: *the planet Earth is primarily considered to be the absolute value of life*, and not as an object of industrial impact (Jonas, 1984);

- ***principle of sympathy:*** It is necessary to treat the problems of organic and inorganic nature from the point of view of "its interests" – normal existence of the natural, including geological environment, and humans, by avoiding egoistic or lucrative approaches (Groenfeldt, 2013; Moiseev, 1990, 1994);

- ***inter-relations principle*:** No geosystems, planetary or local, do exist in isolines, and any change in any of these will inevitably lead to changes in another system of the same or higher level (Cascio, 2007);

- ***principles of harmony and balance of interests*:** The necessity of liaising/harmonising interests of all social groups, related with use of mineral resources and useful properties of subsoil, by intruding into the geological environment, development of the mechanism of social accessibility of resources (Groenfeldt, 2013)

- ***principle of responsibility in front of future generations and increasing variability*:** Any development should satisfy the needs of currently existing generation without any threat to the needs of other generations, and any taken decision for implementation of geoethical situations, dilemmas and problems should increase the possibilities/opportunities of currently living and future generations, and not degradation of such (Cascio, 2007; Gray, 2014);

- ***principle of forecasting*:** Analysis of possible changes should take into account not only the velocity of the processes of development of human civilisation, but also the velocity of the processes of geological evolution (Cascio, 2007);

- ***precautionary principle*:** Any threat from any possible danger of natural, including geological, catastrophes upon taking management decisions should be taken into account as a really existing danger, even if such risk is of a preliminary scientific hypothetic nature(Nemec and Nemcova, 2005; Nemec, 2005b);

- ***principle of reversibility*:** The changes in geosystems of all levels, in the process of their performance must leave a possibility for taking a different geoethical decision in case of occurrence of unforeseen consequences (Cascio, 2007);

- ***principle of integration*:** The norms of ethical approach to inorganic nature should be introduced in laws, standards and rules of conduct of nations of the world. For comparison purposes, we shall demonstrate the main principles of ecological ethics that are established in the Rio-de-Janeiro Declaration on Environment and Development (Rio Declaration) signed in 1992 at a UNO conference:

- ***principle of respect to all life forms***, that affirms the value of each living creature: "any form of life should be respected irrespective of its usefulness for humans", "each organism, whether human or else, whether it has a capability of feeling or not, safe for humans or not, is a values itself"

- ***biodiversity principle***, that affirms the value of biodiversity and necessity in its preservation;

- ***principle of maintaining sustainability of biosphere*** that is the basics of sustainable development;

- ***principle of ecological justice*** states equal distribution of the rights for ecological safety between humans; and everybody is imposed responsibility for its preservations;

- ***precautionary principle***, according to which, it is necessary to primarily take into account most dangerous possible development of events while developing a policy that directly or indirectly impacts to ecology;

- ***principle of general ownership to natural resources*** expresses the understanding of the Earth as an integral unity; according to this principle, people carry equal responsibility for natural resources.

Moiseev (1994) introduced the term "ecological imperative". The scientific circles immediately started discussions that are still ongoing, about valuable-normative bases of the ecological imperative and fields of its application. Despite wide use of this term, its content is not yet fully developed. Its use often occurs in the context of general calls of ecological alarmists ("do not cause damage to the nature") and does not carry any moral-ethical content. Moiseev (1994) defined it as "a system of limitations, violation of which may cause irreversible consequences
for further existence of humanity and the entire surrounding world"

Grigoryev (2005) introduced *the principle of moderateness* (*the principle of "do not damage"*) in geoethics based on the ecological imperative: actions in relation to geological objects and geological systems of any level should by all means avoid causing damage. However, this direct borrowing from a discipline that is "allied" to geoethics is still within "alarmism" ideas Grigoryev (2005).

Geoethics as an Important Area of Research

Geoethics as an important area of research and reflection has an overall aim that all geoscientists work should be integrated into their education and continued professional development. The *'10 fundamental values'* expressed in the Cape Town Statement on Geo-ethics (CTSG) help articulate what it is that we as geoscientists could (and should) be doing if our professional engagement with one another and society is to be considered 'ethical' according to Fundamental Values of Geoethics, Cape Town Statement on Geoethics (CTSG):

1) Honesty, integrity, transparency and reliability of the geoscientist, including strict adherence to scientific methods;
2) Competence, including regular training and life-long learning;
3) Sharing knowledge at all levels as a valuable activity, which implies communicating science and results, while taking into account intrinsic limitations such as probabilities and uncertainties;
4) Verifying the sources of information and data, and applying objective, unbiased peer-review processes to technical and scientific publications;
5) Working with a spirit of cooperation and reciprocity, which involves understanding and respect for different ideas and hypotheses;
6) Respecting natural processes and phenomena, where possible, when planning and implementing interventions in the environment;
7) Protecting geodiversity as an essential aspect of the development of life and biodiversity, cultural and social diversity, and the sustainable development of communities;
8) Enhancing geoheritage, which brings together scientific and cultural factors that have intrinsic social and economic value, to strengthen the sense of belonging of people for their environment;
9) Ensuring sustainability of economic and social activities in order to assure future generations' supply of energy and other natural resources.
10) Promoting geo-education and outreach for all, to further sustainable economic development, geohazard prevention and mitigation, environmental protection, and increased societal resilience and well-being.

(Source: Fundamental Values of Geoethics, Cape Town Statement on Geoethics)

Conclusions

Ethical behavior is essential to science, whose purpose is to develop reliable knowledge about the physical world based on reproducible observation and development of testable explanations. There is no science without honesty, and truth telling is a fundamental ethical virtue. There is an ethical element present in even the most basic scientific observations (Cascio, 2007). The interest by geoscientists in (geo) ethical aspects of geoscience knowledge, education, research, practice and communication has grown considerably. Today the topic of geoethics has gained a significant/tremendous visibility within the scientific community. The International Association for Promoting Geoethics (IAPG) founded in 2012, has worked to widen the discussion and create awareness about issues of ethics as applied to the geosciences. Thanks to continuous voluntary work, the respectful exchange, and fruitful sharing of ideas, the IAPG community has produced a conceptual substratum on which to base the future development of geoethics, by clarifying the meaning of the word "geoethics", formalizing its definition, and better identifying a framework of reference values on which the geoscience community can base more effective codes of conduct and guidance.

The IAPG considers the 35th IGC as the scientific event that opened a new phase for furthering the concept of geoethics. Also, the Cape Town Statement on Geoethics (CTSG), released officially on October 2016, is a document that defines a conceptual framework for the study of geoethics, and provides a first step to exploring whether geoethics could evolve into a new discipline, either within geo-sciences or within ethical sciences. The true development of geoethics is, above all, a responsibility of all of us as Earth and Planetary Scientists. Geoethics is a discipline in full growth and in recent years many ideas have been planted in hopes that they would sprout. The results obtained up to now are encouraging. To this end, with the progress of science and technology and with questions arising on globalization, ethical issues, in particular geoethics, concerns us all. More importantly, teaching geoethics could help students to understand the ethical dilemmas of geosciences and to develop strategies to address sustainability issues (Vasconcelos, 2012; Vasconcelos *et al.*, 2016). In this way, early immersions in the learning of geological abilities linked with other transversal disciplines can outline long-term attitudes toward the interdisciplinary, beyond the mere geological work. Indeed, the geoethics arena may be the space in which we can discuss and share those values that will help to develop a healthier relationship between humankind and the planet. More importantly, Geoethics is an orientation tool for geoscientists, able to provide geoscientists with the ethical dimension of their actions. To this end, geoscientists must be able to face the enormous challenge of reconciling geoethical values with the practice of geosciences. With this aim in mind, geoscientists must be able to function without making compromises in their work, undertake the pursuit of the common good, and ensure the right balance between sustainable living conditions while respecting Earth processes.

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Application of Electrical Resistivity Tomography in the Investigation of Causes of Road Failure in a Typical Basement Complex of Southwestern Nigeria

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Abstract

The usefulness of road as route of commercial activity and also for other uses has necessitate the construction and maintenance of road, hence to avoid the failure of this road before and after construction there is need to understand the properties of soil and rock underlying the site of construction. Hence, this study is aimed among other thing to assess the factor responsible for the failure of the Ogunmakin Highway in South Western Nigeria, which falls within the Basement Complex of Nigeria with Longitude 3^0 46" 00' E - 3^0 48" 00' E and Latitude 7^0 8" 00' N and 7^0 10" 00'N. Three Vertical Electrical Sounding (VES) and 1 Horizontal Profiling was carried out in both the failed and stable sections of the road using Schlumberger and Wenner array respectively. The result obtained were interpreted quantitatively and qualitatively using partial curve matching technique and computer iteration (WINRESIS Software and Diprofwin Software). The VES interpreted result was able to delineate 3-4 geoelectric layers which include Topsoil, Clayey, Sandy and Fresh Basement. The topsoil resistivity values ranges from 118.8-835.0Ωm with layer thickness between 1.2-2.0m., weathered layer (clayey/sandy layer) has resistivity values that ranged from 86.4-518.0Ωm with thickness of between 1.7-7.1m and the fresh basement resistivity values that ranged from 1461.3-2513.2Ωm. The 2-D resistivity imaging generated for profiling one on the failed section reveals amount of Clayey Layer at the top surface to a depth of 0.5m at electrode position of 25-30m, 65-83m, 95-135m, and 150-165m which means the competency rating of this first profile is poor. Hence cause of road failure, while the second profile on the stable section shows the occurrence of competency material in virtually all the length of the profile, hence the stability of this section of the road. Based on the results obtained in the study area, it could be concluded that 2-D imaging gives a better view of the subsurface layer and that the presence of clay material close to the surface is the main factor responsible for the failure of the road in the study area.

Keywords: *Tomography, Resistivity, Failure, Investigation, Basement complex*

Introduction

The performance of the Nigerian roads sector has not been satisfactory despite its enormous potentials for growth and development. Traditionally, the poor transport facilities and infrastructure have severely delayed economic development which weakened Transport infrastructure and contributed negatively to attempts to alleviate poverty in the country. Yet John F. Kennedy was once quoted as saying: **"It is not strong economies that give good roads; but rather that it is good roads that give rise to strong economies!"**(Danladi, 2013).The increase in Population has led to increase in urbanization which has brought about the construction of several roads in Nigeria while existing roads has experienced more usage and this brings about reconstruction of several roads. But yet there is still incessant failure of highways on most Nigeria Highway which leads to loss of Life during accident on these roads. This Road failures which is a very common features on Nigeria Major Highways because most of these roads are known to fail few years after construction and well before their design age hence, this makes this road to be continuously reconstructed or rehabilitated without any effort made to identify factor(s) responsible for their repeated failure. Research has therefore show that road failures can either be functional (Surface Pavement Failure) or Structural (Deep-Seated Pavement Failure).Hence other factors such as poor construction materials, bad design, usage factor, poor drainage network are some of the factors considered as responsible for these failures.

Geological factors are rarely considered as causes of road failure even though the highway pavement is founded on the geology. This is due to non-appreciation of the fact that proper design of highway requires adequate knowledge of subsurface conditions beneath the highway route. The non-recognition of this fact has led to loss of integrity of many highway routes and other engineering structures across the country as observed by Olorunfemi *et al.,* 2000, and Olorunfemi *et al.,* 2005.The geological factors influencing road failures include the nature of soils (mostly Laterite) and the near surface geological sequences, existence of geological structures such as fractures and faults, presences of cavities, existences of ancient stream channels and shear zones. The collapse of concealed subsurface geological structure and other zones of weakness controlled by regional fractures and joint system along silica leaching which has led to rock deficiency are known to contribute to failure of highways and rail tracks (Nelson and Haigh, 1990).

Ogunmakin, Southwestern Nigeria (study area) road is a very important feeder road in the area for commercial purposes, hence the road has experienced lots of failure which make us to focus on the possible geological factors that could be responsible for the several failure observed on this road in this research work. Hence the main aim is to investigate the possible causes of road failure along this commercial road using the geophysical method. Geophysical investigation of the interior of the earth involves taking measurement at or near the surface of the earth that are influenced by the internal distribution of physical properties. Analysis of these measurements can reveal how the physical property of the earth's interior varies vertically or laterally. By working at different scale, geophysical methods may be applied to a wide range of investigation from studies of the entire earth (global geophysics; Kearey and Brooks, 1984) to exploration of a localized region of the upper crust, probing the soil/subsoil and subsurface for any engineering construction activities. The deduced soil characteristics are used as preliminary crucial step is omitted, concealed geologic features within the subsurface may precipitate excessive total or differential settlement leading to failure or collapse of civil structures (Adebisi, *et al*., 2016).

In this study, the electrical resistivity method was adopted for data collection because it provides reasonable information on the nature of the underline materials (Adiat,et.al,2009 and Adeyemo,2004). The geotechnical interpretation of the resistivity data in the study area will be used to identity and describe the nature of the subsurface lithology which will be able to give an insight to the roof causes of the failure of this road.

Site Description and Geology

The study area Ogunmakin lies within the South Western Nigeria basement complex which is located between Longitude 3^0 46" 00' E - 3^0 48" 00' E and Latitude 7^0 8" 00' N and 7^0 10" 00'N of the equator. The study area lies within the Southwestern part of the Nigerian Precambrian Basement Complex. Rocks found in the study area are; Migmatite, Pegmatite, Porphyritic Granite and Granite (Fig. 1).

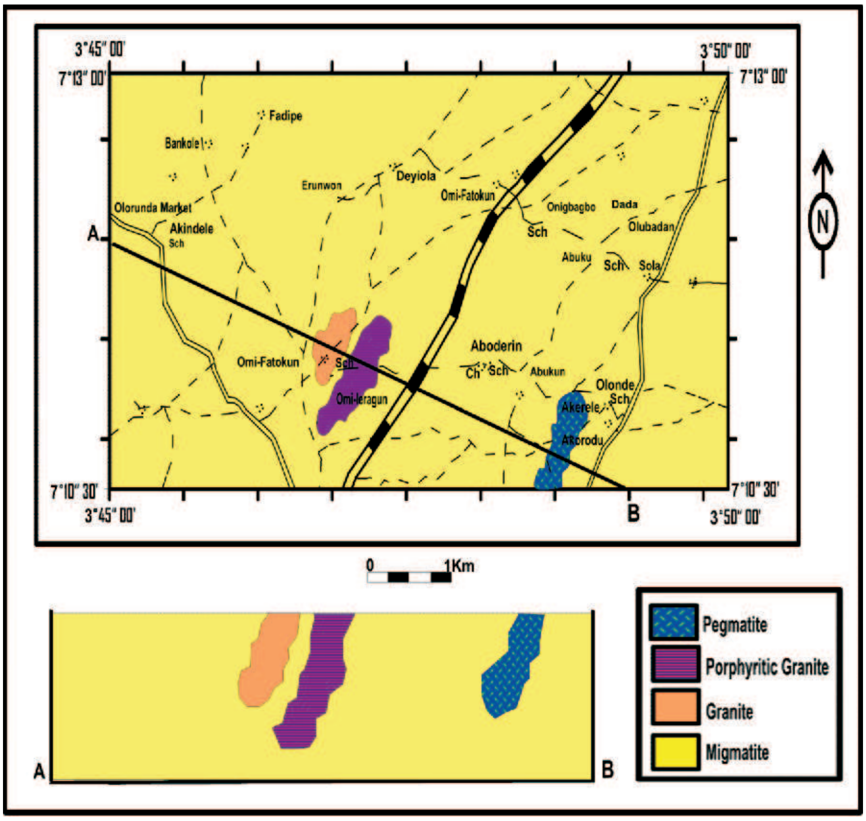

Fig. 1: Geological Map of the Study Area.

Materials and Methods

In this work Electrical Resistivity method were adopted via Vertical electrical sounding (VES) and Horizontal Profiling (HP), collectively called Electrical Resistivity Tomography (ERT). Basically, the electrical resistivity method involves the passage of electric current (using D.C or low frequency A.C current) into the subsurface, through two electrodes (the current electrodes). The potential difference is measured between another pair of electrode, which may or not be within the current electrodes depending on the electrode array in use. Actual resistivity of subsurface layer is determined from ground apparent resistivity, which is computed from the measurement of current and potential difference between the electrodes pair placed on the surface.

Vertical Electrical Sounding (VES) also known as electrical drilling or 'expansion probe, is a method of resistivity employed in investigating resistivity variation with depth .VES furnishes detailed information on the vertical succession and their thicknesses. The techniques is based on the fact that the fractions of the electric current put into the ground will penetrate to any particular depth, increases with an increased separation of the current electrodes. While **Horizontal Profiling (HP)** is used to determine lateral variations of resistivity. The current and potential electrodes are maintained at a fixed separation and progressively moved along a profile. It is used in geotechnical surveys to determine variations in bedrock depth and the presence steep discontinuities. Two-dimensional multi-electrode arrays provide a two-dimensional vertical picture of the sounding medium. Six VES stations and two HP were occupied in the both stable and unstable portions of the road in the study area.

The data obtained with interpreted both quantitatively and qualitatively using curve matching techniques and appropriate soft ware to generate results that were later used to achieved the aim of the work.

Results and Discussion

The interpretation of six (6) Schlumberger sounding and (2) Horizontal Profiling conducted in the study area indicated that the lithological layers vary from 3-4 layers. The subsurface layers within this study area include Topsoil, Clayey, Sandy and Fresh Basement based on their corresponding resistivity values obtained during the interpretation. The summary of the lithological parameter which includes Thickness, Depth and Resistivity are shown in Table 1.

Table 1: Qualitative and Quantitative Interpretation of VES Curves

VES NO.	LAYER	P (Ωm)	DEPTH	THICKNESS	LITHOLOGY
1	1	472.4	1.2	1.2	Topsoil
	2	224.0	6.5	1.7	Sandy
	3	1461.3			Fresh Basement
2	1	118.8	1.0	1.0	Topsoil
	2	86.4	4.9	5.9	Clayey
	3	1828.7			Fresh Basement
3	1	835.0	1.0	1.0	Topsoil
	2	518.0	4.5	5.4	Sandy
	3	2143.0			Fresh Basement
4	1	329.4	1.0	0.7	Topsoil
	2	189.6	5.2	6.2	Sandy
	3	1760.9			Fresh Basement
5	1	552.6	1.0	1.0	Topsoil
	2	327.7	5.1	6.1	Sandy
	3	2513.2			Fresh Basement
6	1	417.0	2.0	2.0	Topsoil
	2	141.5	5.1	7.1	Sandy
	3	1632.9			Fresh Basement

Geo-Electric Section

The geoelectric section is a geophysical tools that is used to view the subsurface lithological section and also the geoelectric sections delineate two major geologic units.This give a pictorial view of the various residual soil wih the depth and thickness of the subsurface lithology (Fig.2).

The geo-electric section reveals 4 lithological layers which include Topsoil, Sandy, Clayey and Fresh Basement. The resistivity of the topsoil from ranges from 118.8 Ωm to 552.6 Ωm and depth of 1.0m - 2.0m.The second layer is Sandy with resistivity values range of 141.5 Ωm - 518 Ωm and depth of 4.5m -6.5m. But in VES 2 there is presence of a Clayey Layer with a resistivity value of 86.4 Ωm at depth of 4.9m. Fresh Basement occur as the Last Layer with resistivity value of 1461.3 Ωm - 2513.2Ωm.

Geophysical Rating of Subsoil Competency for Investigation of Highway Failure

The sub-grade soil beneath a stable highway pavement

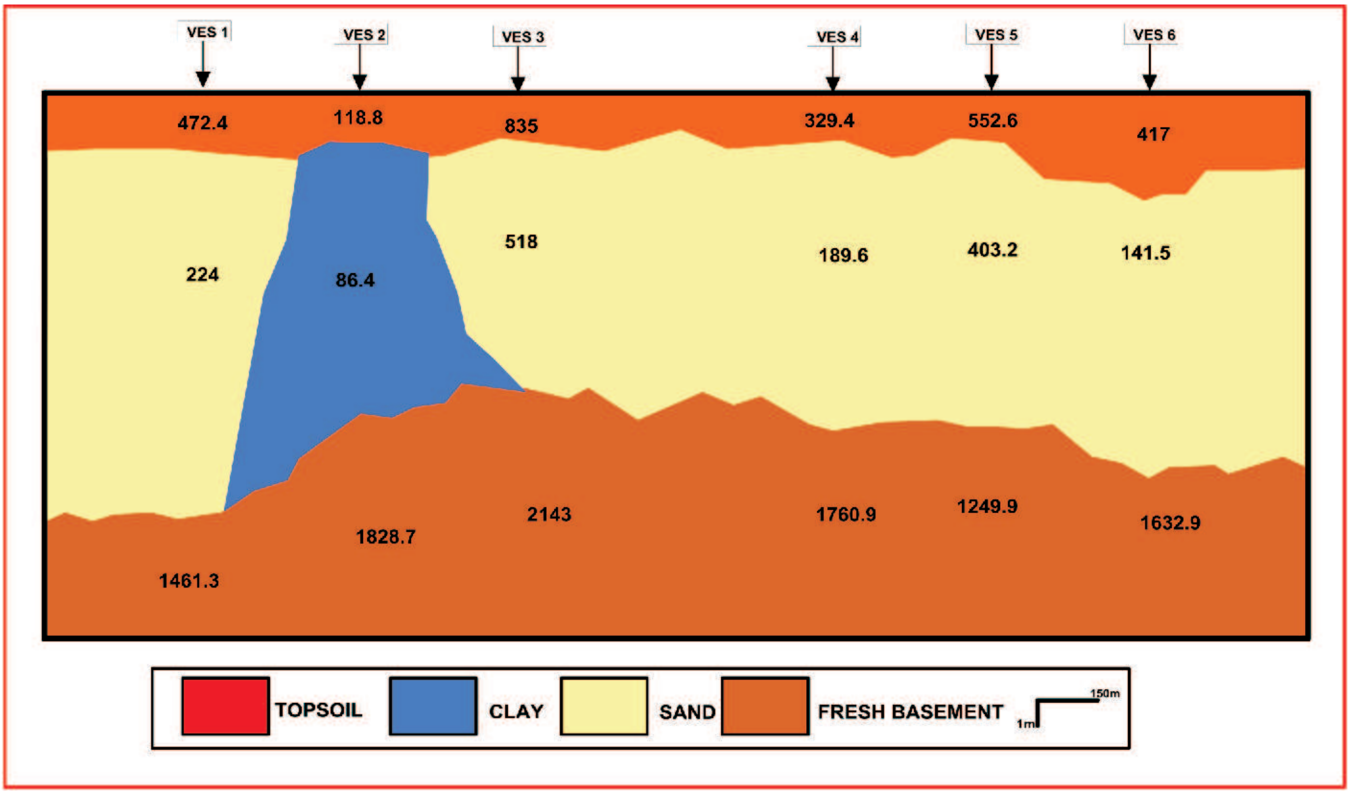

Fig. 2: Geoelectric Section of the Transverse

is expected to possess sufficient strength to support the structure or wheel load imposed on it. It must not swell or shrink excessively and must have proper permeability and drainage characteristics (Adeleye, 2005 and Oladapo, 1998). Unfortunately, due to the heterogeneous nature of the tropical soil and subsurface geological structures, the above conditions are rarely met and hence the strength of the sub-grade decreases and eventually, the pavement on it fails.

Residual soils such as laterites, resulting from the weathering and decomposition of the Precambrian Basement rocks cut across the various rock types in the study area above. The highway pavement was constructed with/or on the lateritic materials derived from either the in-situ weathering of the underlying lithology or from somewhere else (Momoh, et.al., 2008).The rating of subsoil competence using resistivity values after Momoh, *et.al.*, 2008 in Table 2 &3.

Competency Rating of the Transerve

The competency rating as it is shown in Fig. 3 shows that topsoil within the study area is competent for all the VES point except at VES 2 where it shows a fairly competency. The second layer is competent at VES 1, VES 3 and VES 5. The VES 4 and VES 6 shows a fairly competency. But clayey layer occur at VES 2 which is incompetent for engineering construction and the Last layer is competent.

Table 2: Rating of subsoil competence using Resistivity

App. resistivity range (ohm-m)	Lithology	Competence rating
< 100	Clay	Incompetent
100 – 200	Sandy/clayey/weathered layer	Fairly competent
>200	Sandy/weathered layer/bedrock	Competent

From the two profiling occupied in the study area, two pseudo sections were presented as Fig.4 and 5. Fig. 4 show the presence of 3-4 Lithological Layers which include Clay(Blue), Sand (Green), Weathered Basement (Yellow) and Purple (Fresh Basement). The Clayey Layer occur between the top surface to a depth of 0.5m at electrode position of 25-30m, 65-83m, 95-135m, 150-165m. The presence of Clay in the topsoil shows poor competency but its effect may be minimal because it occurs to a shallow depth. The presence of

Table 3: Competency Rating Of the Resistivity Values

VES NO.	NO OF LAYER	RESISTIVITY	LITHOLOGY	Competency Rating
1	3	472.4	Topsoil	Competent
		224.0	Sandy	Competent
		1461.3	Fresh basement	Competent
2	3	118.8	Topsoil	Fairly competent
		86.4	Clayey	Incompetent
		1828.7	Fresh basement	Competent
3	3	835.0	Topsoil	Competent
		518.0	Sandy	Competent
		2143.0	Fresh basement	Competent
4	3	329.4	Topsoil	Competent
		189.6	Sandy	Fairly competent
		1760.9	Fresh basement	Competent
5	3	552.6	Topsoil	Competent
		327.7	Sandy	Competent
		2513.2	Fresh basement	Competent
6	3	417.0	Topsoil	Competent
		141.5	Sandy	Fairly competent
		1632.9	Fresh basement	Competent

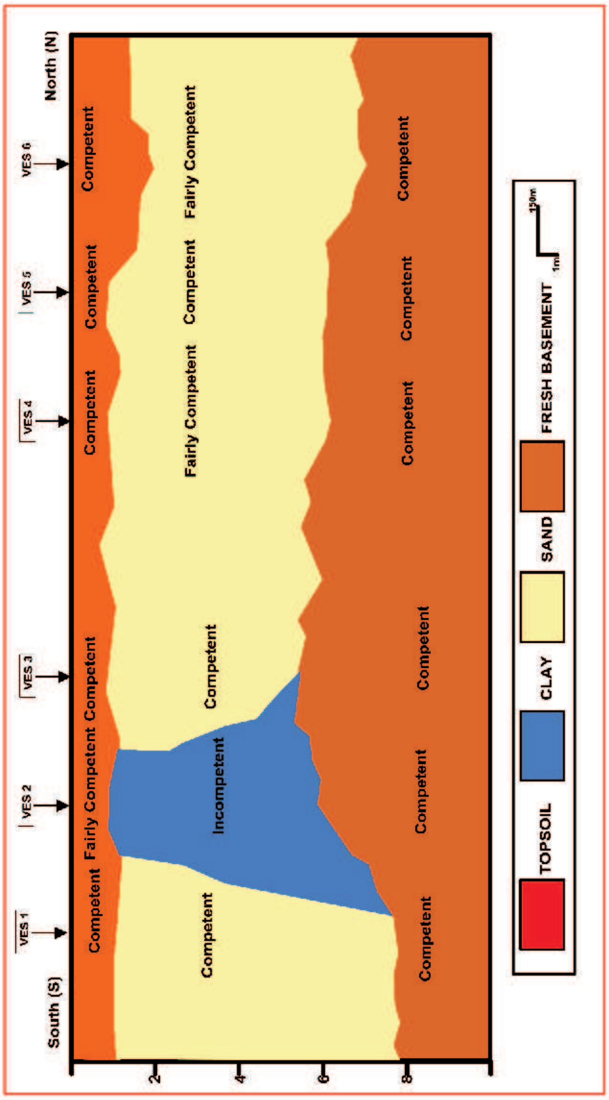

Fig. 3: Competency Rating of the Transverse.

sand and weathered basement from some part of the topsoil to a depth of 2.5m is an indication that this layer is moderate to competent. The occurrences of basement at depth of 2.5m from the surface is an indication that the basement is closer to the surface and also since high resistivity shows great competency .Hence the competency of this Pseudo section can be rate as fairly to competent, and it is therefore advisable that the clay on the topsoil should be excavated before any construction should be cited in this region of the pseudosection.

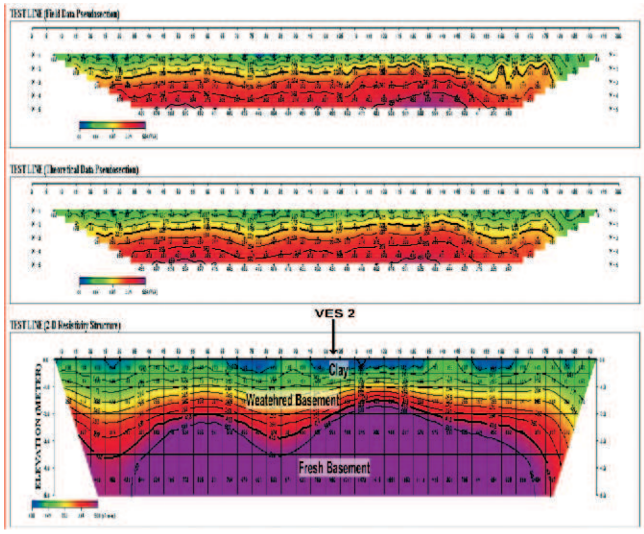

Fig. 4: The Inversion Model Sections for Traverse 1; (A) Field Data Pseudosection, (B) Theoretical Data Pseudosection, and (C) 2D Resistivity Structure.

While Pseudosectionof Fig. 5 shows the presence of 3-4 Lithological Layers which include Clay (Blue), Sand (Green and Yellow), and Weathered Basement (Yellow). The pseudosection shows the presence of Clay at electrode spacing 0-40m at depth from the surface to 1.8m.Other part of the study area is occupied by sand down the transverse from an electrode position of 45-120m but a layer of Fresh Basement occur between electrode position of 130-185m with a resistivity value ranging between 584-647Ωm .Due to the presence of sand within the study area, the competency of this area is fairly competent but areas with little clay shows poor competency.

Conclusion and Recommendation

Based on the results obtained in the study area, it could be concluded that 2-D imaging gives a better view of the subsurface layer and that the presence of clay material close to the surface is the main factor responsible for the failure of the road in the study area. It is hereby

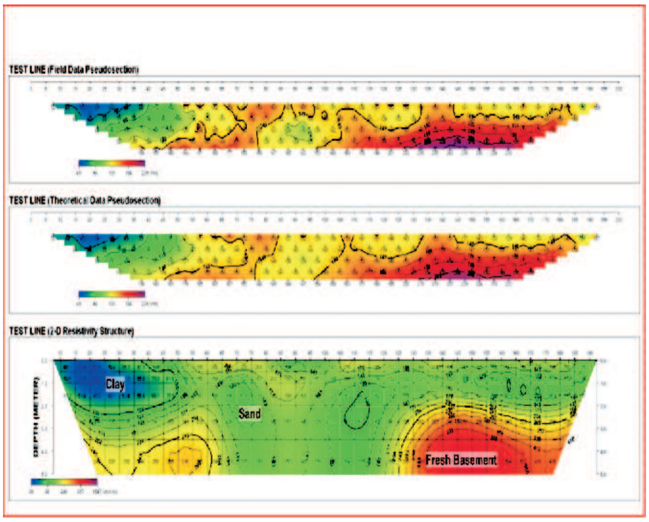

Fig. 5: The Inversion Model Sections for Traverse 2; (A) Field Data Pseudosection, (B) Theoretical Data Pseudosection, and (C) 2D Resistivity Structure.

recommended that the clayey layer should be excavated whenever it is found along the route during road construction surface because Clayey topsoil/sub-grade soils have tendency of absorbing water which makes them swell and collapse under imposed wheel load stress which subsequently lead to road failure.

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Hydro-Geoenvironmental Effects of Organic Contaminants from Dumpsite Leachates in the Enugu Metropolis (Southeastern Nigeria)

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Abstract
The organic chemistry of leachates from a dumpsite within the Enugu Metropolis (southeastern Nigeria) and water (surface and groundwater) samples within the dumpsite influence zone were assessed to determine the impact of the dumpsite leachates on the water sources in the area. Results of laboratory analyses indicated that the organic contaminants in the leachates and water samples varied as follows: diethyl phthalate 0.18 – 0.22 mg/l and 0.04 – 0.17 mg/l; total organic halogen 1.705 – 3.988 mg/l and 0.571 – 2.650 mg/l; 2,4-dichlorophenol 0.97 – 1.11 mg/l and 0.05 – 0.34 mg/l; methyl ethyl phthalate 0.08 – 3.12 mg/l and 0.00 – 0.31 mg/l; borneol 0.036 – 0.045 mg/l and 0.034 – 0.44 mg/l; total organic carbon 1.73 – 10.99 mg/land 0.52 – 2.05 mg/l; total Kjeldahl 0.70 – 1.68 mg/l and 0.14 – 0.40 mg/l; and phosphate 29.16 – 30.86 mg/l and 0.28 – 3.04 mg/l, respectively. Dilution, filtration and flow direction are some of the factors likely to be responsible for the variation in concentrations in the tested water samples. Water quality assessment, based on organic contaminant concentration revealed that the water sources rated as fairly to marginally suitable for drinking purpose. This may indicate that the dumpsite system is in the meantime only slightly harmful to the water sources. Based on statistical projections, however, negative impacts from the dumpsite is to be expected in the future. Hence there is need to replace the dumpsite with sanitary landfill.

***Keywords**: Drinking purpose; Dumpsite; Leachate; Organic contaminants; Water quality.*

Introduction

In the Enugu Metropolis (the capital territory of Enugu State, southeastern Nigeria, Fig. 1), dumpsite system is currently being used for the disposal of varied types of wastes (domestic, agricultural, industrial and municipal). Reasons for this may include government nonchalance or scarcity of fund for provision of sanitary landfill, which is one of the most ideal techniques for waste disposal. The use of dumpsite system in the area has exposed the environment within the metropolis to one of the major challenges associated with the disposal method, which is leachates.

Previous researchers had identified leachate as having the capacity to pollute or contaminate both surface and groundwater quality in an area within its influence zone. Yasuhara et al. (1997), Agirtas et al. (1999), and Slack et al. (2014), had pointed out that leachates from the dumpsites contain harmful inorganic and organic elements and compounds that contaminate natural waters, through infiltration and migration from the site to nearby natural water sources. Some of these organic compounds are toxic and/or carcinogenic and could cause diseases and illnesses that range from cancer, heart diseases to liver damage (Awomeso, et al. 2010; World Health Organization, 2011).

This work aims at highlighting the impact of the organic compounds and contaminants on the quality of water resources. The outcome of this research would assist in waste and water resources planning and management in the area, in particular and other areas of the world with similar waste generation and disposal potentials, in general.

Location and description of dumpsite area

Ugwuaji Nike is within the Enugu Metropolis, the capital of Enugu State (southeastern Nigeria). The metropolis lies within latitudes 6° 23′N and 6° 29′ N and longitudes 7° 30′E and 7° 36′ E. The Ugwuaji dumpsite is about 1.6 kilometers off Enugu – Port Harcourt expressway (see Fig. 1), close to the Independence Layout junction. The dumpsite, on a local scale, slopes gently downwards in all directions from its centre (Fig. 2), and is presently the final destination for all kinds of wastes. The statistics of the waste is given in Fig. 3. The dumpsite measures about by 0.8 sq.km, and currently with an average total dump thickness of about 6 m high. Field investigations revealed that the dumpsite do not have bottom-liner nor was the bottom compacted prior to use. This situation exposes the site and its environs to all forms of pollution and contamination, especially from leachates, of which about 200 L is produced on a daily basis.

Topography and drainage

The most striking feature within the Enugu Metropolis

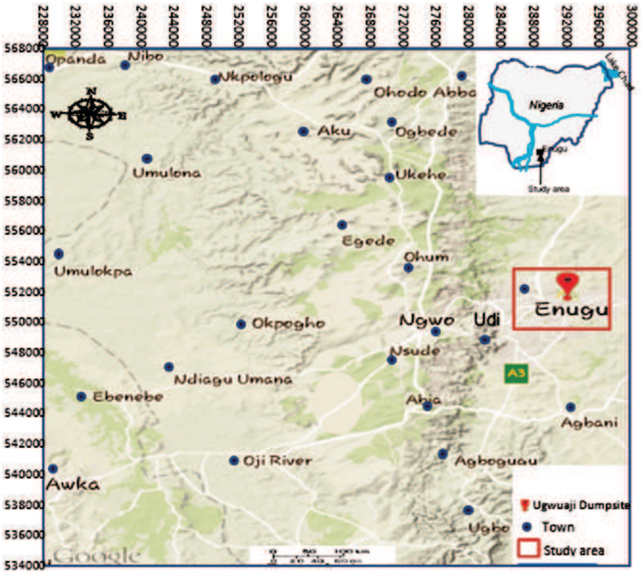

Fig. 1: Map of Nigeria showing the location of Enugu Metropolis

and environs is the Enugu – Udi Escarpment. This escarpment is part of the Nsukka – Okigwe Cuesta, which was formed by the resistant sandstones of the lower part of the Ajali Sandstone and Mamu Formation (Reyment 1965). The major rivers that drainthe area are the Ekulu, Inyaba, Nyo and Asata Rivers. These rivers and other rivulets give rise to a dendritic drainage pattern (Fig. 4), usually developed by a combination of topography and rock type factors (Egboka 1993). Rainfall is the major source of recharge in the Enugu Metropolis. The nature and short span of the surface water bodies in the area shows that none of them has influent tributaries in the drainage basin, due to low porosity and reduced permeability of the shaley units close to the surface.

Climate

The Enugu area falls within the tropical zone, with characteristic hot and humid climatic conditions. The day-time temperature generally ranges from 27 to 32 °C, while night-time temperature valuesare between 17 and 28 °C. Two main climatic seasons occur in the area: the dry season, which lasts from early November to March; and the rainy season, which lasts from April to October (Inyang 1974). The mean annual rainfall value is over 1500 mm.

Geology and hydrogeology

Geologically, the Enugu Metropolis is underlain by two formations; namely, the Coniacian Agbani Sandstone and the Campanian Enugu Shale (Fig.5). The dumpsite spot is, however, underlain by the Enugu Shale. The Agbani Sandstone consists of medium to coarse grained, moderately consolidated sandstones, lenticular shales and beds of grits and pebbles. The formation dips at an amount between 3 and 12 °, in the west – northwest direction, and conformably overlain by the Enugu Shale. The Enugu Shale comprises predominantly dark grey shales and alternating sequences of thin layer sandstones and sandy shales, with occurrence of mudstone and iron nodules in some locations. At the dumpsite area, the formation has dip amount that ranges between 4 and 8 °, with the dip direction tending towards west – southwest.

Two major aquifer systems exist in the area; confined and unconfined aquifer systems. The unconfined aquifers occur essentially in the exposed sandstone member of the Agbani Formation and places where the Enugu Shale had undergone very deep weathering, while the other areas overlain by the predominantly shaley Enugu Shale constitutes the confined aquifer system. The sandy shales, shales and mudstones of both formations, however, constitute low productive aquifers and aquitards in the dumpsite area. Geo-electrical resistivity surveys, carried out by Aka (1983), indicate that watertable in the area lies at an average depth of about 20 m.

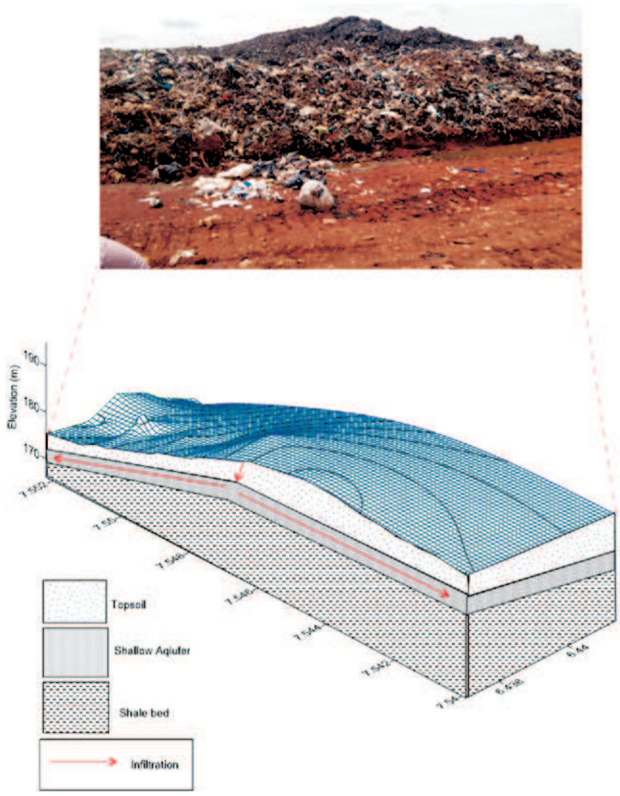

Fig. 2: Schematic diagram showing the topography of the Ugwuaji dumpsite and the leachate flow pattern

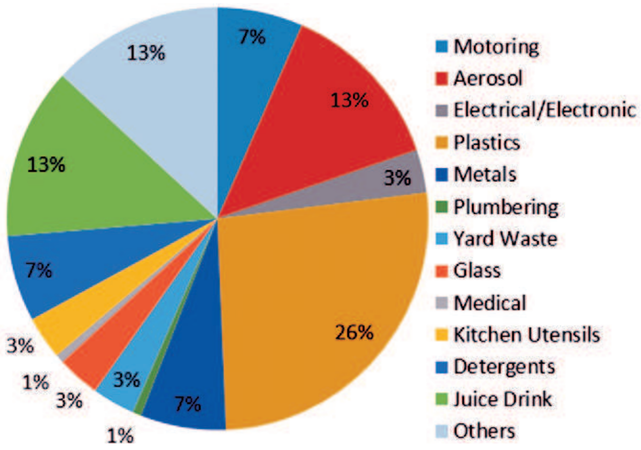

Fig. 3: Pie chart showing the percentage composition of wastes at Ugwuaji dumpsite (Source: Enugu State Waste Management Agency)

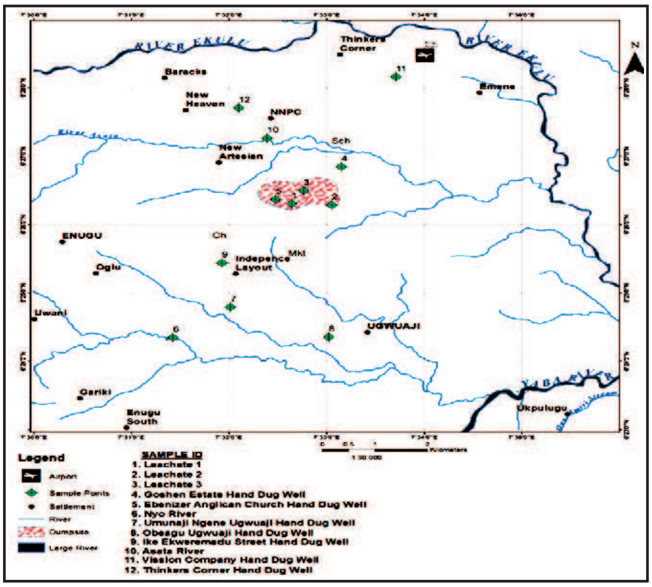

Fig. 4: Drainage map of Enugu Metropolis showing the sample locations

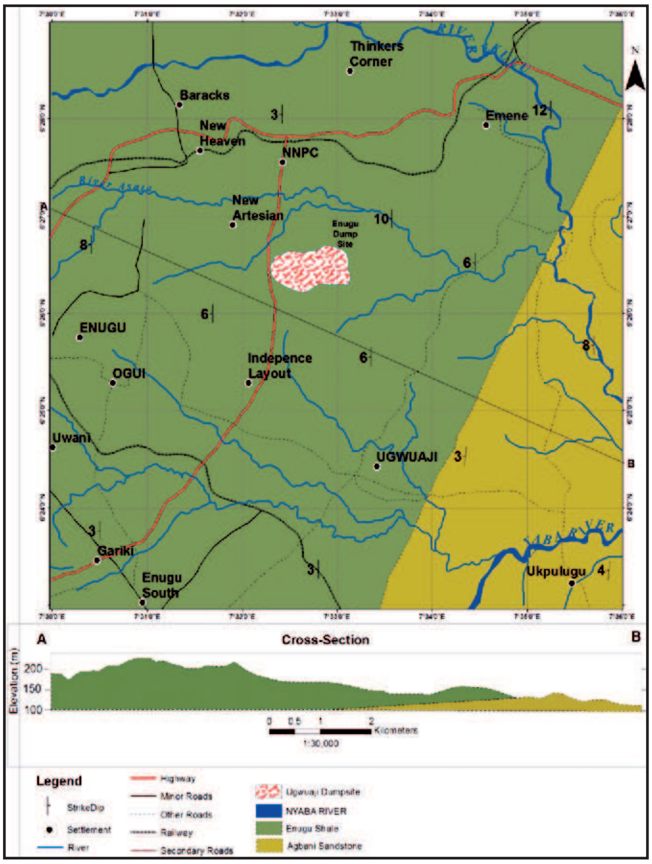

Fig. 5: The Geological map of Enugu Metropolis showing the Ugwuaji Dumpsite

The average linear groundwater flow and hydraulic conductivity of the area, calculated from the electrical conductivity data, are 4 x 10^{-3} ms^{-1} and 2 x 10^{-4} ms^{-1}, respectively (Aka 1983). Onyekuru et al. (2010) had noted that the general groundwater flow direction in the Enugu area is northeast – southwest. This is flow direction, according to Ananaba (1991), is structurally controlled. At the dumpsite area, however, the infiltration pattern is radial owning to site topographic outline (see Fig. 2), with static water level and hydraulic head values that range between 0.9 and 4.9 m, and 162.2 and 196.1 m, respectively (see Table 1).

Materials and Methods

Sampling

A total of 12 samples (3 leachates, 7 hand dugwells and 2 streams) were collected andsubjected to organo-chemical analyses for organic compounds. These samples were all from spots within the 2 km radius of the Ugwuaji Dumpsite. This radium is proposed to be the influence zone, within which the leachates from the dumpsite have the maximum potential to contaminate the natural water sources. The coordinates of the sample locations, distance from the dumpsite and sample type description have been summarized in Table 1. The samples are herein coded DSL for leachates, HDW for hand dug wells and RWS for rivers. The two rivers sampled are also located up (RWS 1 - Nyo River) and down (RWS 2 – Asata River) the base flow gradient of the Ugwuaji Dumpsite.

Laboratory analysis

The compounds tested for included diethylphthlate, total organic halogen, 2,4-dichlorophenol, nonyl-phenol ethoxylate, methyl ethylphthalate, borneol, total organic carbon, total Kjeldahl, ammonium-nitrogen, and nitrate-nitrogen. Other pollution indicators analyzed for were nitrate and phosphate. Details of the procedure are outlined in Yasuhara et al. (1997) and American Public Health Association guideline (APHA, 2012).

Principal component and correlation analyses

Principal component analysis and Pearson correlation were carried out on the generated data following guidelines outlined by Olabaniyi and Owoyemi (2006). Computer software tools used for the data analyses was the XLSTAT (version 2017.01.41150). ArcGIS 9 (version 9.3), Surfer 12, and Corel Draw (version 11) were used for graphical presentation and contouring of the datasets.

Results and Discussion

Organic contaminants

The results of the organo-chemical analyses are summarized in Table 2. Reinhard et al. (1984) had noted the sources of DEP, borneol and majority of the organic compounds found in leachates to include decomposing of cellulose and hemicelluloses in vegetation and degradation products of natural materials, as well as from consumable plastics, insecticides, drugs, cosmetic wastes. Table 3 presents the possible sources of common organic contaminants in the Ugwuaji Dumpsite leachates. As expected, the highest concentrations of DEP and borneol were observed in leachate collected from a dug hole within the dumpsite (sample no. DSL 3).

The concentration attenuation in the DEP and borneol values of the samples some distances away from the dumpsite could be brought about by dilution, filtration or by degradation over time and distance. Reinhard et al. (1984)had pointed out organic compounds, such as phenols, terpenes and phthalates, have tendency to degrade as any leachate plume migrates over some distance.The increase in DEP in 2 samples (nos. HDW 2 and HDW 9) farther away from the dumpsite would suggest influence from roadside waste disposal or borneol influence. Kjeldsen et al. (2002) reported that the solubility of DEP is controlled by co-precipitation, decomposition oradsorption by borneol. Thus, DEP may have been co-precipitated, decomposed or adsorbed by borneol. Dissolution of borneol could have led to high DEP and low borneol recorded on water samples from hand dug wells at Goshen Estate and Ekweremadu Street (sample nos. HDW 2 and HDW 9).

The concentrations of TOH and 2,4-dichlorophenol measured in the leachate samples were generally higher (highest in the sample collected within the dumpsite) compared to shallow hand dug wells and surface water samples. Only in samples shallow hand dug wells at Ebenezer Anglican Church and Goshen Estate (samples nos. HDW 3 and HDW 2, respectively) were concentrations of TOH very significant. This buttresses an earlier observation in the concentration of DEP; that organo-chemical compounds in the leachates might have attenuated down the flow gradient and in the surface water sources. Of the 2 rivers tested, the 2,4-dichlorophenol concentration of the up-slope river was surprisingly higher than that of the down-slope river. Possible explanation to this could be that organic effluents from other sources than the dumpsite may have contaminated Nyo River.

The highest values of the two organic compounds were both recorded in leachate samples. Sources of both organic compounds are likely to be from the decomposition of organic matters, detergents, wetting or dispersing agents and emulsifier, as well as solvent in perfumes and pesticides containers dumped at the site. Their spatial concentrations follow an earlier observed pattern, which had suggested a general attenuation of the organo-chemical contaminants down the flow gradient, especially the base flow. That NPE was below detection in a sample from hand dug well from Thinkers' Corner (sample no. HDW 6), 1000 m away from the dumpsite, and MEP below detection in samples from Nyo River and hand dug well from Thinkers' Corner, both also 1000 m away from the dumpsite, strongly support the fact filtration, dilution and precipitation may be playing some attenuation roles in the dumpsite environs. It is very likely, from the findings in this work,that the zone of influence of the leachate organic contaminants is well below 2 km radius of the dumpsite area.

As shown in Table 3, the TOC and total Kjeldahl, like the other organic compounds, were appreciably higher in the leachate samples than in the hand dug well and surface water samples. TOC is one of the most important composite parameters used in detecting contaminants in drinking water, cooling water, and waste water used in semiconductor manufacturing and pharmaceutical industries, while total Kjeldahl indicates presence of putrescible wastes in dumpsites.

A general reduction in TOC amount was expectedly recorded on the tested hand dug well and surface water samples. The hand dug well sample (HDW 6) and surface water sample, farthest in both cases from the

Table 1: Description of sample types, sampling locations and their coordinates

Serial No.	Sample location	Sample code	Sample type	Water source	Coordinates		Elevatio	Distance*	Borehole Data (m)	
					Latitude (N)	Longititude (E)			SWL	Hyrdaulic head
1	Ugwuaji Dumpsite	DSL 1	Leachate		6° 26' 18.2"	7° 32' 37.6"	194	5		
2	Ugwuaji Dumpsite	DSL 2	Leachate		6° 26' 17.3"	7° 33' 02.2"	193	6		
3	Ugwuaji Dumpsite	DSL 3	Leachate		6° 26' 12.3"	7° 33' 20.2"	195	0		
4	Goshen Estate	HDW 2	Hand dug well	GW	6° 26' 50.9"	7° 33' 08.8"	184	200	0.9	183.1
5	Ebenezer Ang. Chr.	HDW 3	Hand dug well	GW	6° 26' 21.5"	7° 32' 28.8"	196	20	2.5	193.5
6	Nyo River	RWS 1	Stream	SW	6° 24' 22.3"	7° 31' 27.0"	181	1000		
7	Umunaji Ngene	HDW 1	Hand dug well	GW	6° 24' 48.3"	7° 32' 01.0"	194	950	2.5	191.5
8	Obeagu Ugwuaji	HDW 4	Hand dug well	GW	6° 24' 19.6"	7° 32' 59.2"	194	970	3.8	162.2
9	Ike Ekweremadu Str.	HDW 9	Hand dug well	GW	6° 25' 26.4"	7° 31' 54.6"	201	900	4.9	196.1
10	Asata River	RWS 2	Stream	SW	6° 27' 18.7"	7° 32' 22.4"	185	300		
11	Vision Company	HDW 7	Hand dug well	GW	6° 28' 9.57"	7° 33' 42.4"	199	500	1.8	183.2
12	Thinkers' Corner	HDW 6	Hand dug well	GW	6° 27' 43.1"	7° 32' 06.1"	175	1000	2.2	172.8

*Distance (in metre) of sampling point from Ugwuaji Dumpsite; SWL-Static water level; GW-Groundwater; SW-Surface water

Table 2: Results of organic chemistry and pollution indicator analyses on samples from the Ugwaji dumpsite area

S/No	Sample location	Sample code	Organic parameter (mg/l)											
			Diethyl phthalate	TOH	2,4-dichloro phenol	Nonylphenol ethoxylate	Methyl ethyl phathalat	Borneol	TOC	Total Kjeldahl	NH_3-N	NO_3^-	NO_3-N	Phosphate
1	Ugwuaji Dumpsite	DSL 1	0.18	1.704	0.97	0.44	0.08	0.041	1.73	0.70	0.014	374.11	84.51	30.86
2	Ugwuaji Dumpsite	DSL 2	0.19	1.705	1.07	0.48	0.08	0.045	2.61	1.14	0.028	287.52	64.95	29.61
3	Ugwuaji Dumpsite	DSL 3	0.22	3.988	1.11	0.36	3.12	0.036	10.99	1.68	0.036	286.61	64.75	30.48
4	Goshen Estate	HDW 2	0.16	1.002	0.08	0.36	0.31	0.036	1.91	0.14	0.028	111.03	25.08	3.04
5	Ebenezer Ang. Chr.	HDW 3	0.17	2.650	0.05	0.35	0.06	0.035	2.05	0.14	0.028	123.93	27.99	1.02
6	Nyo River	RWS 1	0.09	0.841	0.34	0.26	0.00	0.034	1.83	0.14	0.028	87.02	19.66	0.46
7	Umunaji Ngene	HDW 1	0.06	0.620	0.08	0.21	0.10	0.040	0.52	0.14	0.028	12.03	2.72	0.28
8	Obeagu Ugwuaji	HDW 4	0.04	0.628	0.10	0.16	0.20	0.038	1.63	0.14	0.003	28.87	6.52	0.73
9	Ike Ekweremadu Str.	HDW 9	0.15	0.908	0.20	0.18	0.25	0.036	1.91	0.28	0.028	19.88	4.49	0.73
10	Asata River	RWS 2	0.11	0.636	0.22	0.11	0.06	0.035	1.64	0.14	0.028	63.29	14.29	1.75
11	Vision Company	HDW 7	0.10	0.980	0.25	0.12	0.00	0.044	1.94	0.28	0.003	42.22	9.54	1.38
12	Thinkers' Corner	HDW 6	0.12	0.571	0.21	0.00	0.17	0.034	1.57	0.40	0.028	20.19	4.56	1.94

Table 3: Common organo-chemical compounds in leachates and their probable sources

Organic compound/ Contaminant	Category	Composition
Nonylphenol ethoxylate	Spent Motoring Waste	Old tyres, used engine oil can, used lubricant, automated battery
2-4, dichlorophenol and Borneol	Aerosol Waste	Disinfectant, hair spray, room deodorizers, body spray, furniture polish, oven cleaners, pesticides, herbicides, tile cleaners.
Diethyl phthalates	Plastic Waste Medical Waste	Gallons, buckets, water can, butter cans, chairs, and baskets. Drip cans
Nonyl phenol ethoxylate	Detergent Waste	Morning fresh, Jik, Hypo and Dettol.

Ugwuaji Dumpsite, however, did not record the lowest TOC and total Kjeldahl. In actual fact, the highest total Kjeldahl in the water source sample category was recorded in the hand dug well from Thinkers' Corner (sample HDW 6), with farthest distance from the dumpsite. This situation suggests caution, as there might be other factors at play in the organic compound distribution in the area yet to be fully understood.

The leachate samples also recorded the highest amount of NH_3-N, NO_3^- and NO_3-N in the three set of sample categories. Similar to the uneven distribution pattern noted in the total Kjeldahl data, no spatial distribution trend could also be established NH_3-N for the hand dug well and surface water samples. NH_3-N is a parameter that measures the amount of ammonia, an obnoxious gas very often found in landfill and dumpsite leachates. It commonly results from the biological degradation of amino acids and other nitrogenous organic matters (Kruempelbeck et al. 1999).

NO_3^- and NO_3-N both indicate the presence of nitrogen in the leachates. High concentration of nitrogen infers organic pollution and encourage growth of algae and other organisms which may produce undesirable taste and odour in water. Apart from the 2 samples closest to the dumpsite, the concentration of NO_3^- and NO_3-N were marginally higher in the surface water samples than in the hand dug well samples. Probable explanation to this situation is that the attenuation phenomenon, as noticed in the distribution of the other organic compounds, is strongly influenced by distance and filtration capacity of the geological formation.

High content of NO_3^- measured within the leachate samples is an indication of decomposition of aerobic and organic wastes such as sewage, spoilt foods and plant materials which are being disposed of at the site. According to Fatta et al.(1999), nitrates are conservative contaminants as they are not affected by biochemical processes and natural decontamination processes taking place inside the dumpsite and landfill as well as their infiltration into the vadose zone.

As shown in Table 3, the highest amount of PO_4^- was on a leachate sample. TP is considered a good indicator of a leachate nutrient status, as its levels remain more stablethan soluble reactive phosphorus. The sources ofTP include soluble phosphorus and the phosphorus in plant and animal fragments suspended in leachate water. PO_4^- value of the hand dug well samples range between 0.28 and 3.04 mg/l, and for the Nyo and Asata River samples 0.150 and 0.571 mg/l, respectively. When converted to TP, in µg/l, the hand dug well samples would record values that range from 0.91 to 9.92 µg/l, while the Nyo and Asata Rivers would record TP values of 1.50 and 5.71 µg/l, respectively.

An irregular pattern is observed when the spatial distribution of the PO_4^- is considered. The diversity in the source of PO_4^- could be responsible for this; it could originate from human and animal wastes, run off from farmlands and soil erosion of geological formations. Lillie and Mason (1983) had used TP to classify of water for drinking purpose. This approach was adopted, in this study, in classifying water samples from the Ugwuaji Dumpsite and environs (see Tables 3 and 4). Major sources of PO_4^- are likely to be from human and animal wastes, detergents, septic systems and runoff from farmlands or lawns.

Table 4: Quality of the water sources in the Ugwuaji area of Enugu Metropolis for drinking purpose

Parameter (mg/l)	Maximum permissible limit (USEPA, 1984)	Average value	
		Hand dug well	Surface water
Diethyl phthalate	0.003	0.08	0.1
Borneol	0.02	0.04	0.03
TOH	0.45	1.05	0.74
Nonylphenol ethoxylate	3.0	0.20	0.19
TOC	5	1.64	1.74

Quality of natural water sources in the Ugwuaji dumpsite area

A comparison of the organo-chemical quality of the natural water sources in the study area with standard specifications recommended by the United States Environmental Protection Agency (USEPA 1984) is presented in Table 4. As revealed in the table, the samples from hand dug wells and surface water in the area are not potable for drinking, when untreated. This is due to the fact that 3 parameters (TOH, diethyl phthalate, and borneol) out of the 5 parameters, considered in this work, have their concentrations above maximum permissible limits. High concentration of nitrogen was inferred from appreciable amounts organic nitrogen compounds recorded in the water. This compounds would impart produce undesirable taste and odour in water.

That 2 parameters (TOH and TOC) out of the 5 organic contaminants were quite below the maximum permissible limits may likely mean that treatment would improve the quality of the water sources for this purpose. However, continued disposal of substances like detergents, antiseptics and spent motor oils in the dumpsite could pose future risks. Awomeso et al. (2010) had noted that detergents and antiseptics contains trichloroethylene, trichloroethane, methylene chloride, nitrobene chemicals which cause liver damage if inhaled or swallowed in drinking or agricultural produce. Organic contaminants in dumpsite could also lead to cancer, heart diseases and abnormalities due to groundwater contamination, according to the World Health Organization (WHO, 2011).

Principal component

The number of principal components, the loading of variables on each component and percentage of data variance in the dataset are summarized in Table 5. Five components which accounted for 98.364 % of the total variance in the dataset were considered. However, only two components, which have significant number of variables (parameters) with factor loading ≥0.800, were used for the interpretation.

Principal component 1: This component has high loading factors of TOH, 2,4-dichloro phenol, total Kjeldahl, NO_3^-, NO_3^-N and phosphate. These high loading factors account for total variance of 61.917 % of the dataset. These compounds and parameters are related to microbial activities in the dumpsite leachates. Principal component 1 can consequently be attributed to

Fig. 4: Attergberg limit of the Trial Pit Samples

the microbial actions on the organic matters and phenol compounds.

Principal component 2: This component accounts for 19.917 % of total variance. Borneol is the only loading factor considered. It is likely to have originated from plastic wastes disposedof at the dumpsite and other roadside dumps. Field investigation revealed that varieties of plastic wastes combined to constitute 26 % of the entire waste disposed at the dumpsite.

Pearson correlation

As shown in Table 6, a very strong positive relationship exists between 2,4-dichloro phenol and phosphate (with correlation coefficient, r^2, of 0.972), methyl ethyl phathalate and TOC (r^2 = 0.978), NO_3^- and phosphate and NO_3^-N and phosphate. This statistical relationship confirms the earlier observation in the results of principal component analysis that 2,4-dichloro phenol, the organic nitrogen compounds and phosphate have direct positive relationship. A good of organic compounds do not correlate significantly with each other ($r^2 = \pm 0.600$), thus, buttresses the fact that some of the factors that influenced the origin and spatial distributions of the organic parameters in the water sources are still unclear.

Table 5: Results of principal component analysis of samples from Ugwuaji Dumpsite area

Variable	Principal component				
	1	2	3	4	5
TOH	**0.838**	-0.325	-0.021	-0.343	0.142
2,4-dichloro phenol	**0.931**	0.196	-0.067	0.244	-0.125
Nonylphenol ethoxylate	0.741	0.314	0.444	-0.273	0.216
Methyl ethyl phathalate	0.698	-0.610	-0.326	-0.058	0.036
Borneol	0.272	**0.801**	-0.338	0.130	0.386
TOC	0.771	-0.539	-0.312	-0.077	0.032
Total Kjeldahl	**0.935**	-0.115	-0.221	0.224	0.041
NH_3-N	0.300	-0.628	0.583	0.356	0.213
NO_3^-	**0.915**	0.296	0.190	-0.068	-0.172
NO_3-N	**0.915**	0.296	0.190	-0.068	-0.172
Phosphate	**0.948**	0.247	-0.006	0.142	-0.110
Eigenvalue	6.811	2.191	0.982	0.484	0.353
Variability (%)	61.917	19.917	8.926	4.399	3.206
Cumulative %	61.917	81.833	90.759	95.158	98.364

Bold values indicate scores considered

Table 6: Correlation matrix of samples from Ugwuaji Dumpsite area

Variable (mg/l)	TOH	2,4-dichloro phenol	Nonylphenol ethoxylate	Methyl ethyl phathalate	Borneol	TOC	TK[a]	NH_3-N	NO_3^-	NO_3-N	P[b]
TOH	1										
2,4-dichloro phenol	0.618	1									
Nonylphenol ethoxylate	0.595	0.632	1								
Methyl ethyl phathalate	0.780	0.525	0.224	1							
Borneol	-0.010	0.413	0.342	-0.182	1						
TOC	0.844	0.616	0.302	0.978	-0.115	1					
TK	0.762	0.911	0.506	0.773	0.276	0.832	1				
NH_3-N	0.359	0.176	0.224	0.387	-0.487	0.366	0.310	1			
NO_3^-	0.670	0.898	0.829	0.394	0.350	0.485	0.753	0.142	1		
NO_3-N	0.670	0.898	0.829	0.394	0.350	0.485	0.754	0.142	1.000	1	
P[b]	0.649	0.972	0.714	0.507	0.434	0.579	0.888	0.153	0.949	0.949	1

[a]**Total Kjeldahl;** [b]**Phosphate**

Conclusions

1. Disposal of a variety of wastes using a dumpsite method in the Ugwuaji area of Enugu Metropolis yields leachates with appreciable organic contaminants
2. The organic compounds from the leachates have impacts on the hand dug well and surface water sources in the area. However, the quality of the natural water sources in the area would be poor to fair for drinking purpose, judging from the fact that they failed 3 out of the organic contaminant indictors for this purpose considered in this work.
3. Use of sanitary landfill is advocated as this study has shown that long term use of the dumpsite method for waste disposal in the area could have impact on the natural water sources that serve the need of the metropolis.

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Geoacoustic and Geotchnical Investigations of the Sea Floor in Pobes Oil Field Western Niger Delta Nigeri: Implications on Subsea Facilities

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Abstract

The understanding of the sediments and sedimentary processes and geotechnical index properties of the sea floor is very integral to subsea facilities development and safety within offshore province. This study of the "Pobes" Field in the shelf environment of western Niger Delta Nigeria, with combined results from measurements using high fidelity onboard geoacoustic instrumentation and data from well cuttings have helped in determining the implications of sedimentary processes and geotechnical index properties on subsea facilities in the Field. The geoacoustic equipment (sub bottom profiler, side scan sonar, echo sounder and magnetometer) use the principles of sound reflectivity and refraction. Bathymetry ranges between 5m-31m, water depth with a deepening trend from the north to south. Topography generally is undulating in the north to gently flat in the south caused by ripple currents and storm processes .Sediments from the sea bed scan vary from silty clayey sand through clayey silty sand. Strong seismostratigraphic interface(competent bed) depth below the seabed ranges 20m-25m as corroborated by the geotechnical index properties analyses of the well cuttings which show the presence of sandstone at 25m.Gas charged sediments about15m-20m thick occurs between the seabed and the lithified layer. The sea floor scan also shows existence of genetically related depressions and surrounding rings of sand called pock marks which vary between 0.5m-9m in diameter. Essentially, the site consists of silty clayey sand on the surface that is soft in consistency and weak in shear strength. However, the consistency and the strength of the clay improve down depth from 14.50m where it becomes soft- firm. At 30.00 m depth, the clay becomes very firm as some shell fragments (mostly calcareous shells of gastropods and mollusks) occurred within it. Below this clay unit at about 25.00 m depth, a dense to very dense Sand unit of about 10.00m -12.00m thick occurred. The sand is poorly to moderately graded. Below this sand unit at about 57.0m depth, another firm clay unit occurs. This clay unit becomes intercalated with sand at 60.50m and continues to about 62.50m where it grades into firm clay and continues to the termination depth at 66.00m. Based on the findings, a number of recommendations have been formulated for the safe and cost effective development and safety of subsea facilities of this Oil Field.

Keywords*: Sediments, Competent bed, Shear Strength, Sea Floor, Geohazards.*

Introduction

The failures of offshore structures and subsea facilities, grounding of sea-going vessels, pipeline interferences and concomitant oil spillages within the offshore oil field province have increased in recent timesChuku,Odigi,Ibe and Ideozu(2018). The necessity for seafloor characterization for operations purposes has therefore become very vital so as to prevent loss of valuable lives and properties that always accompany such failures offshore. The seafloor sediments and sedimentary processes studies and geotechnical index properties are high-fidelity information tools used to identify the geological state of the seabed Chuku and Ibe (2015). Geotechnical characterization usually provide subsurface information that assists in the design of structures Al-Khafaji and Andersland (1992). The acoustic equipment used are sub bottom profiler, side scan sonar and echo sounder, whose principle is sound reflectivity. The geotechnical index properties were revealed through the standard penetration tests (SPT), Piezocone penetration tests (PCPT) and undrained triaxial test Sridharan(1999) . The study area is geographically located at south-western Niger Delta.

Study Location

The study area is about 10.2 square kilometers. It is situated 20.0meters to 28.0kilometers off the coast line from the south western end of Benin and Escravos Rivers in western Niger Delta of Nigeria.

Methodology

From 2nd January, 2015 to 22nd May 2015, 30 km of along track side scan sonar data, subbottom profiler, echo sounder and magnetometer data were collected from 'Pobes' field within a 1 km long of the field in the Western Niger Delta for the purpose of determining the Geoacoustic properties of sediments and sedimentary processes of the sea floor and their implications on subsea facilities installations/development and safety. The total area of geoacoustic data coverage was 8.93

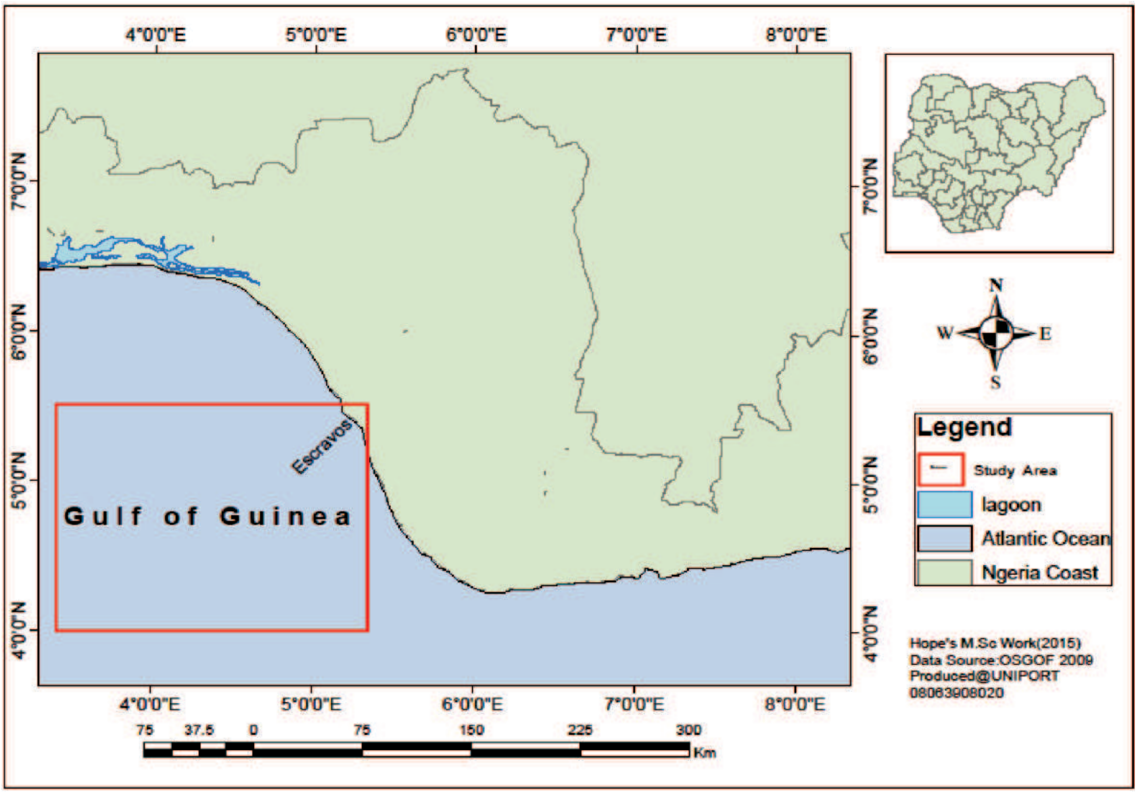


Fig. 1: Map of the Study Area, offshore Western Niger Delta Nigeria (modified after Chuku and Ibe 2015).

km^2.Sidescan sonar and seismic subbottom profiler data were collected simultaneously in a dual display. The instruments were towed at speeds of 3kts. Surveys took place at high tide and ended as the tide was receding. The water depths during the survey ranged from 0.5–6m.

Vessel Data

The vessel used for this research survey was the MV Rektar, a towing and anchor handling tug. She is 40.3m long, with a draft of 3.2m and beam of 11.3m, capable of a top speed of 15 knots.

Digital Global Positioning System (DGPS)

Positioning was accomplished by means of a Kongsberg Seapath 330 DGPS/DGLONASS, NR 203 MDGPS and Trimble 5700 with 0.5-1m accuracy.

Navigation Data

Navigation data was logged (with bathymetric signal) through Eiva Navipack software to a computer hard disk.

Side Scan Sonar (SSS) Data

The survey used side-scan sonar to map the seabed, elucidating bedforms that are suggestive of sediments and sediments mobility, sedimentary structures and inferring sediment types. The GeoAcoustic dual frequency side scan sonar provided mapping of the seabed and consist of a pair of dual frequency (100 kHz to 500 kHz). The unit includes standard controls such as: Gain, Time varying Gain (TVG), automatic Gain Control (AGC) with duplicated control for Port and Starboard transducer.

Sub Bottom Profiler (SBP) Data

The GeoPulse Model 5210A Receiver functions as a transceiver in a GeoPulse "pinger" profiling system provides a powerful transmit pulse at an operator selected frequency which then converts the electrical pulse to an outgoing acoustic signal. This acoustic signal is reflected off sub-bottom layers. The returning echoes are converted by the transducers to an electrical signal which is inturn passed on to the GeoPulse Model 5210A Receiver to provide stratigraphic data, processed with Coda Octopus software.

Echo Sounder (ES) Data

Bathymetric data was collected using a single-beam echo sounder Model 400 (710 kHz) echo sounder. Online survey guidance, and simultaneous logging of bathymetric and positioning data were handled through a PC running the Coastal Oceanographic Eiva survey program.

Conductivity, Temperature and Depth (CTD) Probe Data

Vale port CTD probe profiler was used to measure the speed of sound in the water column to provide calibration data for the echo sounder.

Magnetometer Data

The G-880 and G-881 marine Cesium magnetometer was used to provide reliable detection of geological features and manmade objects (cables and metals) by magnetic anomalies.

Tide Data

Tidal levels were recorded at Ogidigbe Port with a time correction of –10 minutes applied for data reduction at the working site.

Borehole Drilling/Sampling

Borehole drilling and sampling together constituted major activities during the field work. Borehole drilling was carried out using the conventional light cable percussion rig.(later down the hole, rotary drilling rig was used). Shell and Auger were used in retrieving the soil samples. Disturbed samples were collected at regular intervals of 0.75m and also at the changes in sediment type. Undisturbed samples were obtained with the conventional open tube sampler, which measures 75mm in diameter and 450mm long. All the samples were examined and classified in the field, properly stored and transported to the laboratory for necessary tests. Standard penetration tests (SPT) were performed in the soils to assess the relative density of soil strata. In the test a 50mm diameter spoon sampler was driven 450mm into the soil with a 63.5Kg hammer falling through a height of 760mm. The initial 150mm penetration is the test drive. The number of blows required to effect the remaining 300mm penetration was recorded as the SPT (N) value. Borehole characteristics including the SPT blow counts are illustrated in Borehole log Day(2001).

Insitu Tests

To complement laboratory tests, insitu tests were also carried out on the samples recovered from the borehole. Field tests such as Piezocone penetration tests were also carried.

Piezocone Pe*netrometer Test*

A piezocone penetration tests was conducted to refusal below mudline. The plots of the tip, sleeve and cone with depth are presented.

Standard Penetration Tests

The Standard Penetration Tests (SPT) were performed solely on sand strata. Standard penetration tests can be used to assess the relative density of soil strata. In the test a 50mm diameter spoon sampler was driven 450mm into the soil with a 63.5Kg hammer falling through a height of 760mm. The initial 150mm penetration is the test drive. The number of blows required to effect the remaining 300mm penetration was recorded as the SPT (N) value Al-Khafaji and Andersland (1992).

Laboratory Tests

a series of classification, strength and compressibility tests were carried out in the laboratory. The tests were performed in accordance with British and ASTM standards. Detail of the type of tests are given below.

Moisture Content

The water content was determined by drying selected moist/wet soil material for at least 18hours to a constant mass in a 110°C drying oven. The difference in mass before and after drying was used as the mass of the water in the test material. The mass of material remaining after drying was used as the mass of the solid particles. The ratio of the mass of water to the measured mass of solid particles was the water content of the material. This ratio can exceed 1 (or 100%). Reference test standard: BS 1377: Part 2:1990 Sridharan (1999).

Unit Weight

The unit weights were determined from measurements of mass and volume of the soil. For the cohesive soils, a specimen was obtained from a standard steel cylinder with cutting edge, which was pushed manually into the extruded soil sample.The dry unit weight γd, was determined from the mass of oven-dried soil and the initial volume. Reference test standard: BS 1377: Part2: 1990 Bowles (1984).

Particle Size Analysis

Particle size analyses were performed by means of sieving readings. Sieving was carried out for particles that would be retained on a 0.063 mm sieve, Dry sieving was carried out by passing the soil sample over a set of standard sieve sizes and then shakes the entire units for few minutes with sieve shaker (machine) Chuku,et al(2018**)**.

Results and Discussions

These consist basically of five sets of data: bathymetric data, sub bottom data, side scan data, magnetometer data, well cutting data and geotechnical index properties with respect to sedimentary processes, sediments, sea floor topography, subsurface stratigraphy, observed features, environment of deposition, shear strenght and sedimentary structures.

Bathymetric Sounding Data

The seabed slopes gently from north towards the south of the study location. The range of water depths within each alignment sheet are listed in the table below:

Table 1: Showing water depth within the study location in kilometer poles

From KP to KP	Water Depth Range(m)
KP 9.50 to KP 12.00	5 - 13
KP 12.00 to KP 14.50	13 - 31

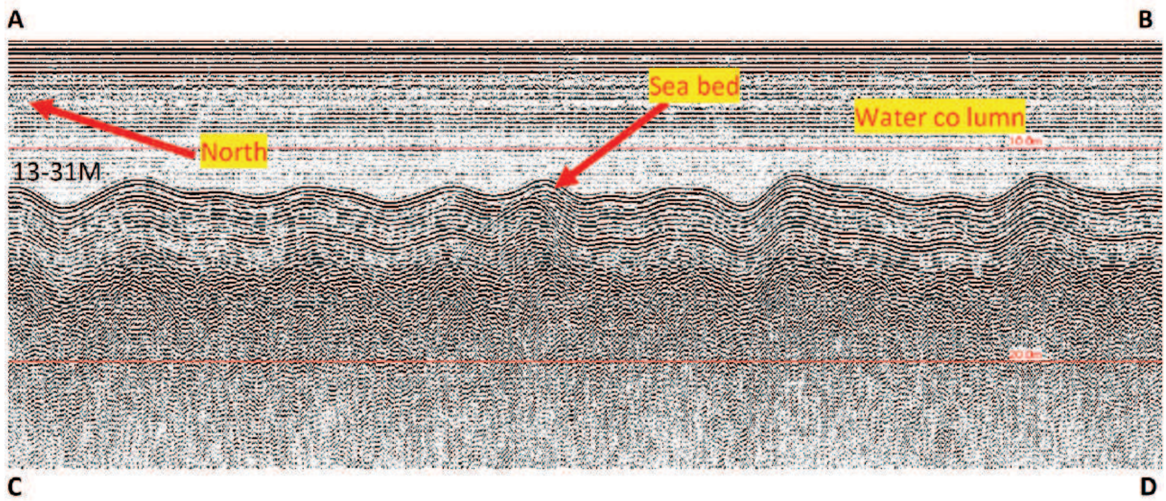


Fig. 2: Section E-F:Undulating and gentle dipping sea bed. Water Depth ranging between 13-31m. Drawing Drawing Scale 1:1000

Sea Floor Sediments

The field is characterized by the presence of low to moderate reflective sediments interpreted as sandy silty clay. However scattered patches of high reflective sediments interpreted as clayey silty sand were also observed within the field.

Sub Bottom Profile Data (Shallow Geology)

The seismic profiles of the surveyed area suggest the presence of strong seimostratigraphic interface at about 25m below the seabed. The acoustic backscatter properties of this layer indicate that the layer is composed of lithified sediments or calc-arenite formations. The acoustic penetration was found to be limited at the top of this layer.Intermittently occurring

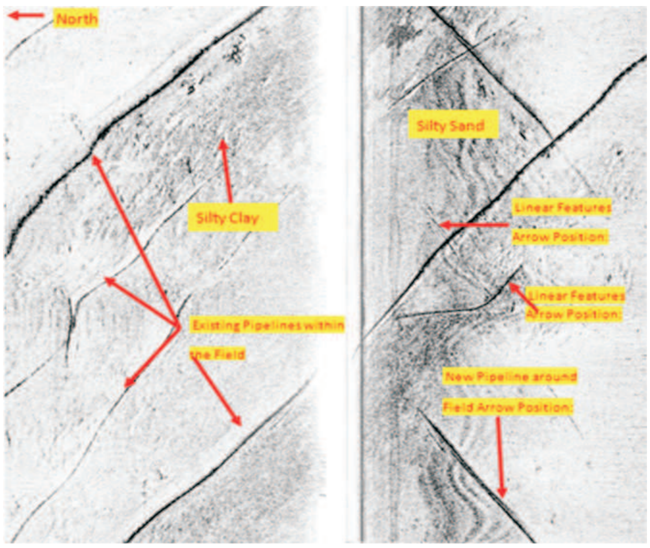


Fig. 3: SSS Data Extract showing existing subsea facilities and sediment type Northeast of the field

weak seismostratigraphic layers were also observed just below the seabed and the area between seabed and strong seismostratigraphic interface.

Well 1 Cutting Data For Pobes Field

Sea Bed Features

Gas charged sediments are found between lithified layer and the sea floor. They occur as a result of the activities of gases associated with petroleum and water in the reservoir during exploration and exploitation of petroleum,looking for avenue to escape to the surface thereby charging the sediments. They are precursor to the existence of pock marks Chuku et al (2018).. Many

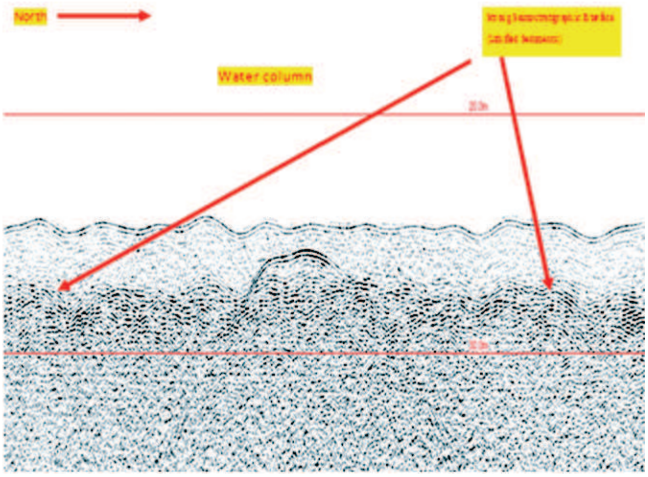


Fig. 4: SBP Data Extract showing buried pipeline in the Field

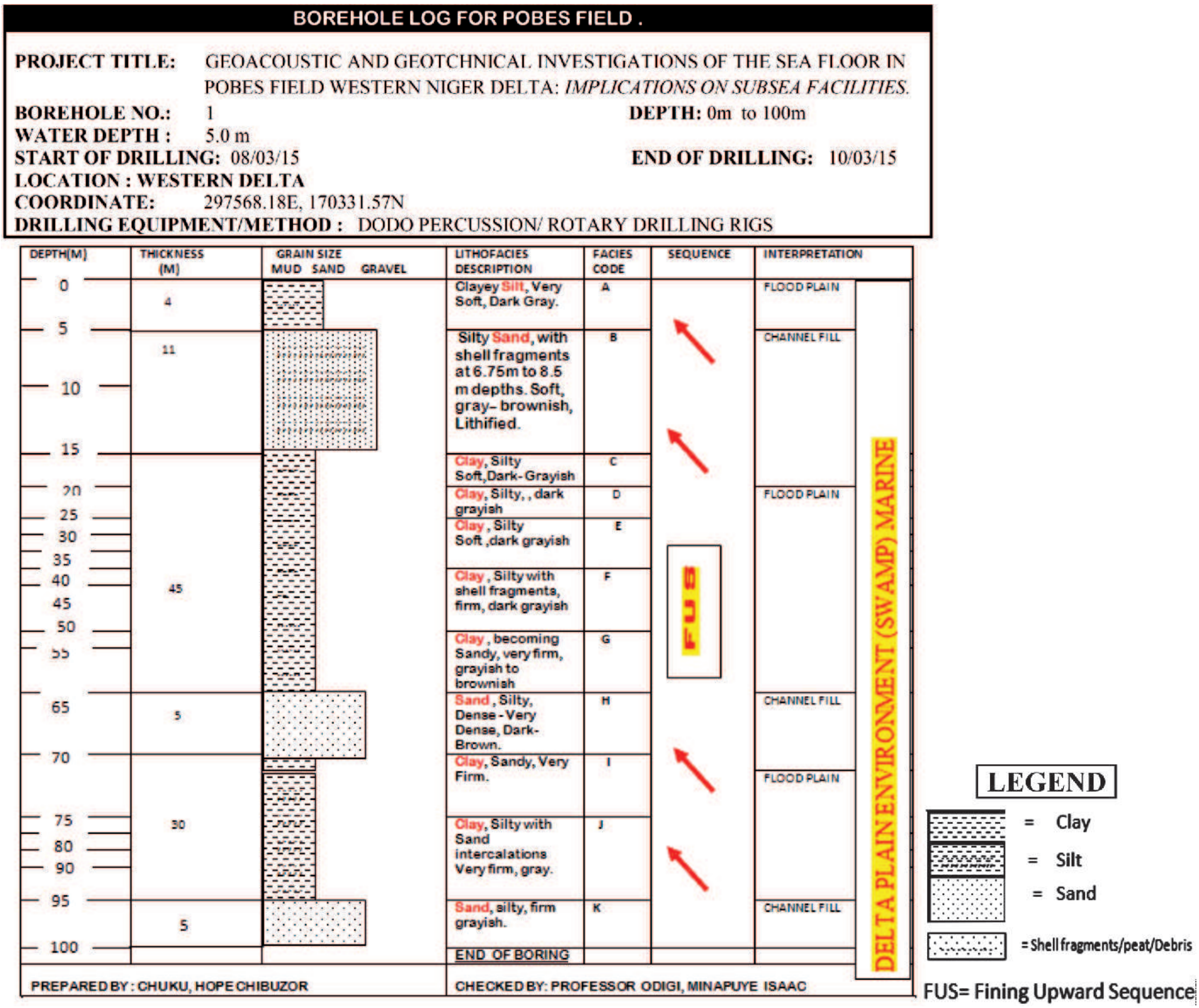


Fig. 5: Graphic sedimentary Log of well cuttings at the pobes Field. (Modified after chuku and Ibe 2015).

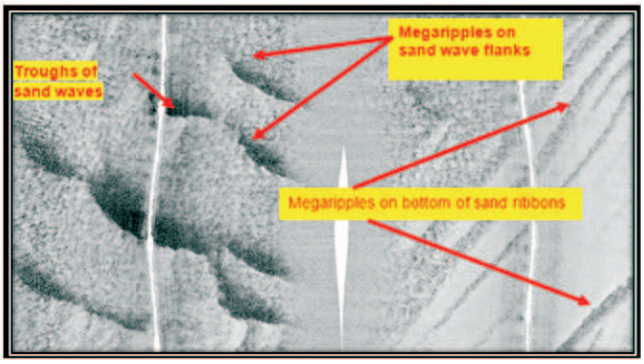


Fig. 6: SSS Data extract showing sand waves, ripples and ribbons in the Field..

Pockmarks of varied diameters were observed within the surveyed corridor.The pockmarks may pose threats to the mechanical integrity of the existing subsea facilities since they are actually shallow gas vents. Numerous scar marks, interpreted to be associated with fishing and offshore activities were observed all over the study area Sridharan (1999).

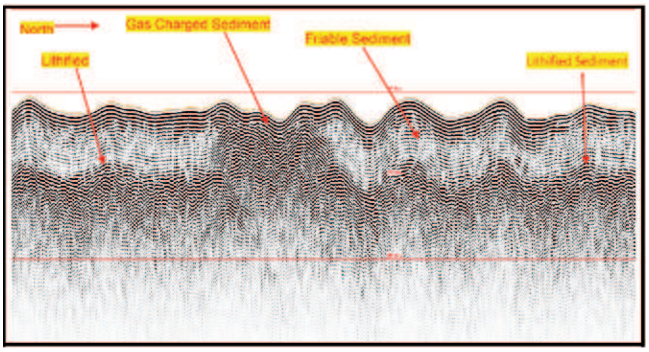


Fig. 7: SBP Data extract indicating Gas charged sediments.

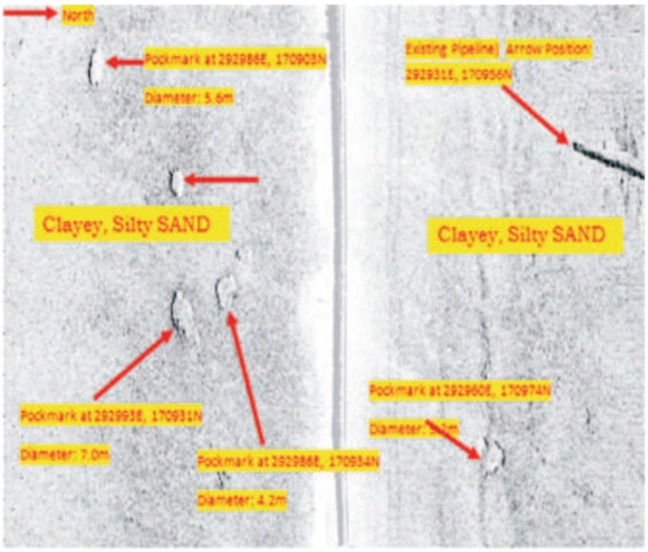


Fig. 8: SSS Data Extract showing several pockmarks at 2km Southeast of the Field.

Magnetic Anomalies

These anomalies were determined by the survey lines that are running perpendicular to the Line. The localized magnetic anomalies could not be discerned for the survey lines running parallel to the Line since the magnetic record of these survey lines were found to be dominated by the magnetic field of the study location. The acoustic equipment detects the existence of cables and metals on or below the sea bed by the spike in figures below:

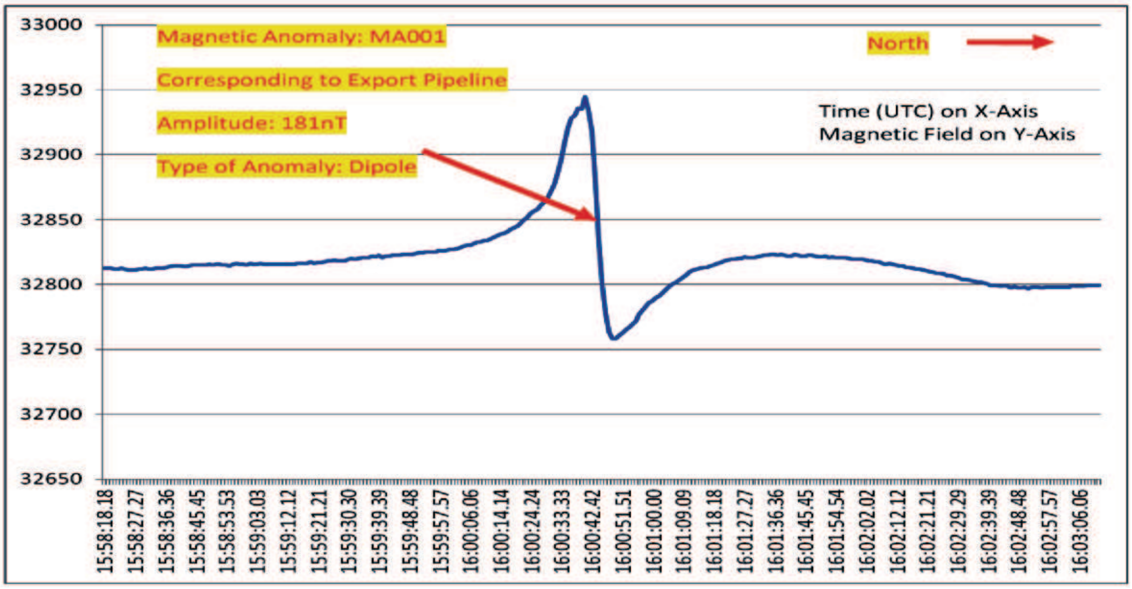


Fig. 9: Magnetometer Data Extract showing magnetic anomaly corresponding to Export Line in E-Field

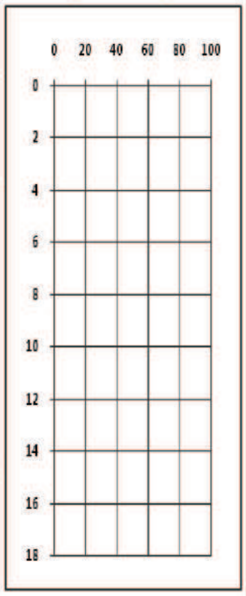
Borehole Drilling/Sampling

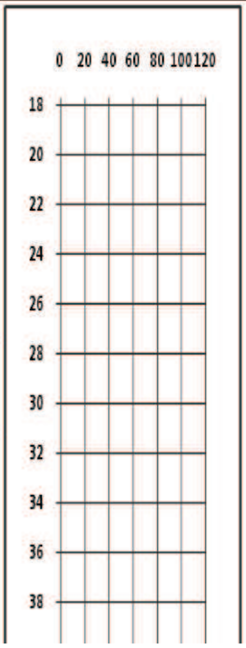
Borehole drilling and sampling together constituted major activities during field work.Borehole drilling was carried out using the conventional light cable percussion rig.(later down the hole, rotary drilling rig was used).

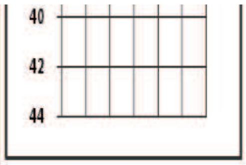
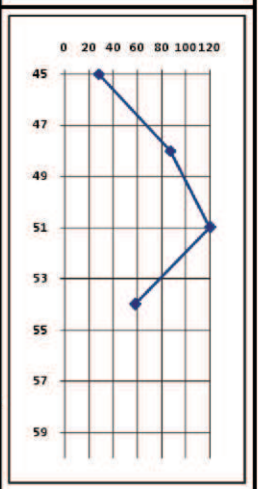
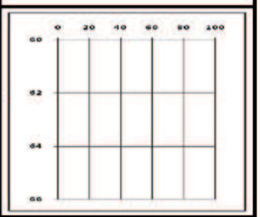
BOREHOLE LOG

PROJECT TITLE: GEOACOUSTIC AND GEOTCHNICAL INVESTIGATIONS OF THE SEA FLOOR IN POBES FIELD WESTERN NIGER DELTA: *IMPLICATIONS ON SUBSEA FACILITIES.*
BOREHOLE NO.: 1 **DEPTH:** 0m to 66m **WATER DEPTH :** 5.0m
START OF DRILLING: 02/12/15 **END OF DRILLING:** 10/12/15
LOCATION : WESTERN DELTA **COORDINATE:** 297568.18E, 170331.57N
DRILLING EQUIPMENT/METHOD: DODO PERCUSSION/ ROTARY DRILLING RIGS

	Sample type	Lithologic Description	Strata Log	Undrained Cohesion (Kpa)	Friction Angles, φ (^{0})	SPT N (blows/ 0.3m)
0.00 -0.75	D	CLAY, very soft , dark grayish.				
0.75 - 1.50	D					
0.50 - 2.25	UD					
2.25 - 3.00	D	CLAY, Silty,with shell fragments at 3.75m to 4.5 m depths. Soft,grayish – brownish,		13	2	
3.00 - 3.75	D					
3.75 - 4.50	D					
4.50 - 5.25	D					
5.25 - 6.00	UD	CLAY, Silty ,Soft ,dark grayish				
6.00 - 6.75	D			12	2	
6.75 - 7.50	D					
7.50 - 8.25	D					
8.25 - 9.00	D					
9.00 - 9.75	D					
9.75 -10.50	D					
10.50-11.25	D					
11.25-12.00	UD					
12.00-12.75	D			14	2	
12.75-13.50	D					
13.50-14.25	D	CLAY, Silty, , dark grayish .				
14.25-15.00	D					
15.00-15.75	D					
15.75-16.50	D					
16.50-17.25	UD			9.5	1	
17.25-18.00	D					
18.00-18.75	D					
18.75-19.50	D					
19.50-20.25	D					
20.25-21.00	D					
21.00-21.75	D					
21.75-22.50	UD	CLAY , Silty Soft ,dark grayish				
22.50-23.25	D			9	2	
23.25-24.00	D					
24.00-24.75	D					
24.75-25.50	D					
25.50-26.25	D					
26.25-27.00	D	CLAY ,Silty with shell fragments, firm, dark grayish				
27.00-27.75	D					
27.75-28.50	UD					
28.50-29.25	D			15	3	
29.25-30.00	D	CLAY, Silty with shell fragments, firm, dark grayish				
30.00-30.75	D					
30.75-31.50	D					
31.50-32.25	D					
32.25-33.00	D	CLAY , becoming Sandy, very firm, grayish to brownish				
33.00-33.75	D					
33.75-34.50	UD			12	2	
34.50-35.25	D					
35.25-36.00	UD					
36.00-36.75	D					
36.75-37.50	D			19.5	3	
37.50-38.25	D					
38.25-39.00	D					



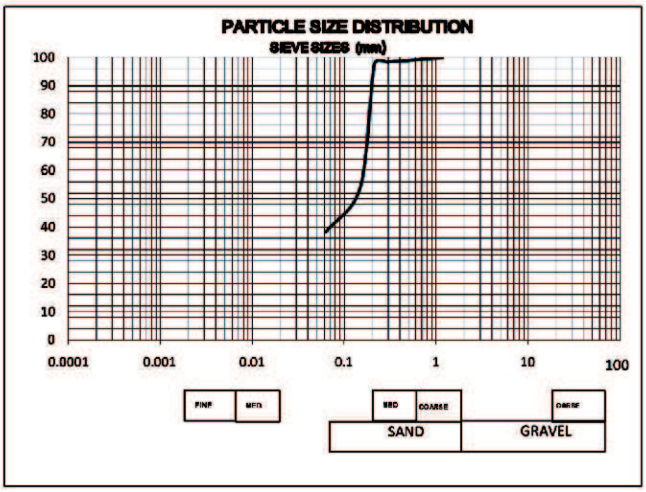


39.00-39.75 39.75-40.50 40.50-41.25 41.25-42.00 42.00-42.75 42.75-43.50 43.50-44.25	D D D D D D D			20.5	4	
44.25-45.00 45.00-45.75 45.75-46.50 46.50-47.25 47.25-48.00 48.00-48.75 48.75-49.50 49.50-50.25 50.25-51.00 51.00-51.75 51.75-52.50 52.50-53.25 53.25-54.00 54.00-54.75 54.75-55.50 55.50-56.25 56.25-57.00	D D D D D D D D D D D D D D D D D	SAND ,Silty,dense - Very dense, dark brownish				
57.00-57.75 57.75-58.50 58.50-59.25 59.25-60.00	D D D D	CLAY,Sandy, very firm, grayish				
60.00-60.75 60.75-61.50 61.50-62.25 62.25-63.00	D D D D	CLAY, Silty with Sand intercalations , very firm, grayish.				
63.00-63.75 63.75-64.50 64.50- 65.2565.25- 66.00	D D DD	CLAY ,silty, firm grayish . END OF BORING				
PREPARED BY: CHUKU, HOPE CHIBUZOR			**CHECKED BY:** PROFESSOR ODIGI MINAPUYE ISAAC			

Particle Size Analysis

Particle size analyses were performed by means of sieving and/or hydrometer readings Chuku, *et al.* (2018).

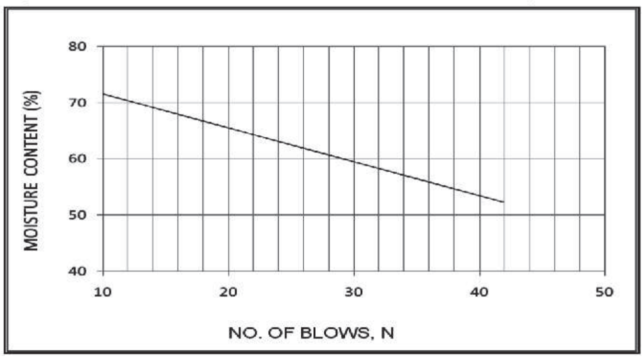
Borehole No.: 1
Location: Western Niger Delta (289667E, 168669N) **Sample Depth:** 20.5m



Atterberg Limits

Atterberg limits were determined on soil specimens with a particle size of less than 0.425 mm.The Atterberg limits refer to arbitrary defined boundaries between the liquid limit and plasticstates (Liquid Limit, WL), and between the plastic and brittle states (Plastic Limit, Wp) of fine grained sediments Chuku et al (2018).

ATTERBERG LIMITS DETERMINTION
COORDINTE: 289667E, 168669N
DESCRIPTION OF SOIL: Very Soft Grayish CLAY **BOREHOL NO.:** 1
DEPTH OF SAMPLE: 20.5 m
Liquid Limit = 61.8% **Plastic Limit** = 27.7% **Plasticity Index** = 34.1%



Unconsolidated Undrained Triaxial (UU)

This test was performed on undisturbed samples of cohesive soils.

UNCONSOLIDATED UNDRINED TRIXIAL TEST

Location: Western Niger Delta: (289667E, 168669N)

Borehole No. : 1 Depth :20.5m

RESULT		
Undrained Cohesion(KN/m^2)	Angle Of Int. Friction(Φ)	Moisture Content (%)
13	2	53.8

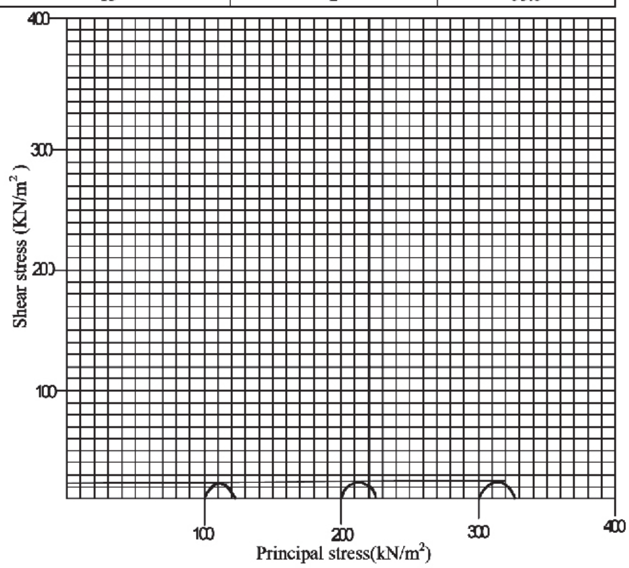


Table 2: Summary of Index and Engineering Parameters

SANDY CLAY			
PARAMETERS	Min.	Max.	Ave.
Natural moisture content (%)	20.3	94.0	57.15
Liquid limit (%)	61.5	117.0	89.25
Plastic limit (%)	25.0	31.3	28.15
Plasticity index (%)	35.5	85.7	60.6
Bulk unit weight (kN/m^3)	14.8	17.5	16.15
Dry unit weight (kN/m^3)	9.0	12.4	10.7
Specific Gravity	2.50	2.64	2.57
Undrained cohesion ((KN/m^2)	9	20.5	14.75
Undrained angle of internal friction (deg)	1	4	2.5

Conclusion

The sea bed sediments from the scan in the study area support a composition consistent with sand, silt and clay deposits. The current ripples are located in the north western part while the wave ripples occur in the south western part. Bathymetry: water depth ranges between 5m-31m. Topography generally is undulating in the north to gently flat in the south caused by ripple currents and storm processes The strong seismostratigraphic interface(lithified sediments-competent bed) depth below the seabed ranges:4.5m-5.5m as corroborated by the sediment analysis of the well cuttings which show the presence of sandstone at 2m-6m. The sea floor scan also shows existence of genetically related depressions and surrounding rings of sand called pock marks which vary between 5m-10m in diameter. The long scar features found on sonar records in all survey location is indicative of disturbances within the survey area that are believed to result from anthropogenic activities.

The location of existing facilities within the "Emobs" fields was based on subbottom and magnetometer data. The sediments investigated within the boring and the depth explored revealed that the site is characterized by near-surface deposit very soft clay to soft silty clay that is of low to medium plasticity .The clay unit is conformably underlain by sand of poor to moderate grading. Another clay unit underlies the sand unit and continues to the termination depth, the geotechnical characteristics of the sediment were determined from the laboratory and field work.

A careful assessment of the field and laboratory test results revealed that the sand unit is fine to coarse-grained, loose to medium dense in relative density, grayish to dark brown in colour. The deposit overlies sandy clay unit and underlies the soft clay/ soft silty clay unit.The soil investigation was conducted with the main aim to determining the prevailing subsoil conditions at the proposed project site and to provide geotechnical data, for the evaluation of the type of foundation suitable for the intended structure. The feasibility of adopting a shallow foundation for the proposed structures may not be possible due to environmental conditions surrounding the project site. Pile foundation is therefore recommended for the project. In the entire depth of the borehole, the SPT N-values within the sandy deposit ranges between 28-120, while the undrained cohesion for the clay were low (9.0-20.5). The sulphate content of the soil is high and the P^H tends to be slightly alkaline. Other parameters as summarized in the engineering properties, are as indicated in tables and charts attached to this report.

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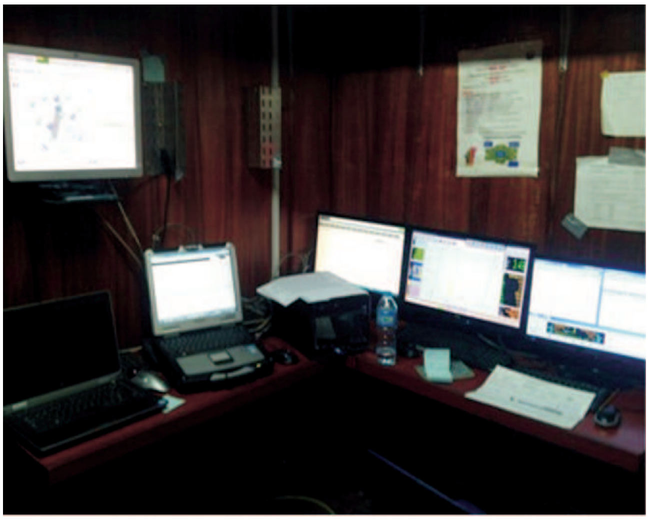
APPENDICES

Borehole Drill site 1A at Pobes Field

Borehole Drill Site 1B at Pobes Field

Data acquisition/Survey Vessel

Geoacoustic Equipment Calibration



Survey Room Set Up

Sea floor features identification/Recording

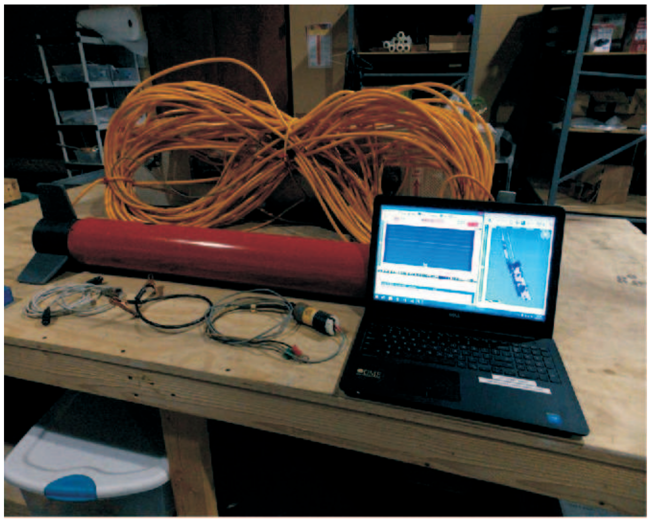
Vessel Navigation/Positioning

Preliminary on board Charting

Echo Sounder (ES)

Side Scan Sonar (SSS)

Sub Bottom Profiler (SBP).



Magnetometer (MAGGY).

CODA Geophysical Acquisition System

ISBN: 978-978-975-787-9

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Engineering Geology in Sustainable Minerals Development

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Abstract
The mineral sector is a complex system that requires the contribution of many specialists in order to drive it forward. One such specialist area is engineering geology. In this presentation, the author has attempted to show the role of the engineering geologist in different aspects of mineral exploitation, such as mine development, drilling and blasting, haulage, tailings dam construction, underground tunneling, stope operation and long wall advancing. It is hoped that this attempt will help the engineering geologist discover his appropriate place and role in a mining organization.

Keywords: *Engineering, Geology, Sustainable, Minerals, Development, Exploitation, Mining.*

Introduction

Before going into details about the role of the engineering geologist in mineral development, first, let us have a look at 4 categories of professionals whose jobs are similar and interwoven; namely, the rock mechanics engineer, the geomechanics engineer, the geotechnical engineer and last but not the least, the engineering geologist. The first 3 deal with rock behaviour when there is a disturbance in rock mass equilibrium, while the fourth one, engineering geologist is concerned with the properties of rocks that lead to these behaviours and the characterization of rock mass.

Rock mechanics deals with the behaviours of rocks when a disturbance occurs to dislodge the natural equilibrium of rock mass, but without a major interest in other geologic factors like the influence of underground water, electric and magnetic fields in rock mass. Geomechanics on the other hand, is wider in scope considering other geologic factors than just the behaviour of rocks in themselves. When it comes to assessing rock stability, you must consider the influence of underground water in rock strength and failure, and that is why geomechanics is gaining more acceptance and a wider spread than rock mechanics.

The geotechnical engineer is the engineer that considers the influence of external bodies like equipment and structures in the behaviour of rocks, and attempts to design a stable rock mass in view of the interaction between the external bodies and rock mass. The engineering geologist does not need to study rock behavioural mechanisms as these belong to rock mechanics, geomechanics and geotechnical engineering.

The Role of the Engineering Geologist in Mineral Exploration

Apart from exploration of metallic deposits and industrial minerals, it is equally very important to explore for construction materials, building and ornamental stones. The engineering geologist has a crucial role in preparing the engineering geological map of Nigeria to ease the exploration for granite deposits and other granitic rocks suitable for road construction, tiles production and preparation of ornamental stones.

The Role of the Engineering geologist in Pit Development

The engineering geologist has a role to play in virtually all aspects of mineral exploitation. Mineral exploitation starts with mine development. Before developing a mine, the topography and terrain have to be studied with respect to equipment needed for mine development especially when surface mining is involved. A crawler mounted dozer can work in all kinds of terrain, soil and rock types. However, a wheel dozer that can do the same job will be more preferable because of cost, provided engineering geological studies can show that a wheel dozer can work in such terrain. A swampy, flowing rock formation is a problem for a wheel dozer and reduces its efficiency.

The Role of the Engineering Geologist in Pit Benching

In benching, the design of stable and optimum bench slope is critical. It is the duty of the engineering geologist in an open-pit mine to collect rock samples and analyze them to provide the uniaxial compressive strength, the angle of internal friction. He also conducts the general rock mass rating (RMR) that helps the mine planner to design the optimum bench height and working slope.

The Role of the Engineering Geologist in Drilling and Blasting

Even at the level of drilling and blasting when an optimum blast hole diameter is to be determined, the rock class factor which is a component of blast hole diameter equation will have to be determined. The engineering geologist has a role to play. After drilling, it is the job of a geotechnical engineer / engineering geologist to assess the stability of slopes for the safety of the loading crew. Unstable bench slope can be very dangerous for the loading crew and drilling crew.

The Role of the Engineering Geologist in Material Handling

In mine haulage, the role of the engineering geologist in haul road track selection cannot be over emphasized especially when rail haulage is involved in large open-pits. The engineering geologist studies the soil / rocks and gives a useful advice to the railway engineering for safe and stable track layout. In mines where waste material are weathered or soft, as in Itakpe open-pit mine, the waste materials are usually pushed down slope. Overtime, these materials become consolidated and become operational platform for further pushing. Here again, the engineering geologist will have to ascertain the degree of consolidation to avoid equipment toppling. The same applies in tailings dam where similar technology is used.

The Role of the Engineering Geologist in Process Plant Construction

In beneficiation plant, where the gangue is removed to enrich the ore, the civil works for the beneficiation plant also involves the analysis of the strength of rock forming the foundation for the plant.

The Role of the Engineering Geologist in Underground Shaft Sinking and Tunneling and Underground Mine Exploitation

In underground mining, the role of the engineering geologist is not diminished. In shaft sinking and tunnel driving for underground mine construction, the engineering geologist in conjunction with geomechanics engineer carries out rock mass rating (RMR) of the exposed roof to help the geotechnical engineer design the appropriate roof support system that will prevent roof failure. The same applied at stope and long wall.

Definitely, this paper cannot be exhaustive on the role of engineering geologist in driving the mineral sector forward. This is just what I think is prominent at this point in time.

Conclusion

In conclusion, it is the role of the engineering geologist to study the properties of rocks and characterize the rocks to give the geotechnical engineer adequate information to design a stable, safe structure that can support mineral exploration. The engineering geologist can also play a crucial role in producing the engineering geological map of Nigeria for ornamental and construction stones. The data produced by the engineering geologist can assist the mine planner in decision making for equipment selection. The engineering geologist is a vital professional in many aspects of mineral exploration and exploitation.

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Assessment of Impact of Municipal Wastes on the Water Quality Around Rumuola Borrow Pit, Southern Nigeria

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Abstract
The impact of municipal wastes dumped into the Rumuola Borrow Pit on the surface and groundwater quality of its environment was assessed by analyzing the physicochemical parameters, polycyclic aromatic hydrocarbons (PAHs) and heavy metals from adjoining boreholes and surface water from the borrow pit. The deposition and decomposition of municipal wastes led to an increase in acidity, BOD_5 concentrations (4.9mg/l), Turbidity (13 NTU), NO_3-N values [<0.05 mg/l – 0.15 mg/l (mean 0.085mg/l)] and Dissolved Oxygen depletion downstream. The presence of 16 PAHs was screened by Gas Chromatography with flame ionization detector (GC/FID). PAHs components recorded in surface water ranged between 112 µg/l and 133 µg/l. PAHs were not detected in the boreholes implying that they are not being impacted.Fish tissues of Tilapia fish spp. and African Snake Head fish spp. were screened for the presence of PAHs and Heavy Metals. Total PAH concentrations found in both fish species exceeded the European Commission's acceptable limit (2 µg/kg Benzo(a)pyrene)for fish considered safe for human consumption. Carcinogenic PAH compounds comprised over 98% of the total PAH components found in the fishes. Traces of the metals assessed (Lead, Cadmium, Chromium, Nickel and Zinc) were found in these two fish species and could constitute a health hazard if inhabitants continue its consumption.

Keywords*: Water Quality, Fish Quality, Rumuola Borrow Pit, Heavy Metals and Polycyclic Aromatic Hydrocarbons (PAH).*

Introduction

Municipal wastes refers to all categories of wastes including garbage (food waste), rubbish, yard wastes, street sweeping, ashes, industrial wastes, dead animal and man, abandoned vehicles, hazardous wastes, demolition debris, hospital disposables, construction wastes, garage crankcase oil and acids (Onwughara, 2010) disposed of by a municipality. The sources of these wastes are households, commercial businesses, industry, demolition sites and factories (Ukpong, 2015; Ayotamuno, 2004).

The commonest sources of groundwater pollution are leachates from waste dumpsites and sewage from septic and surface tanks (Owuama, 2004). The major components of dumpsites are degradable organic solids (Ukpong, 2015) and plastics while steel and Aluminium cans are also common in urban wastes. When dumped indiscriminately, solid wastes have negative aesthetic impacts (Ngah, 1993) and the decay of organic solids invariably leads to contamination and eutrophication (Isirimah, 2000).

Clean and unpolluted water is essential for life in order to maintain human health and the quality of the environment. Despite its importance, water is the most poorly managed resource in the world (Lateef, 2012). The status of surface and ground water are not commonly known and are located around municipal areas where they can be easily impacted by indiscriminately dumped refuse.

Rumuola Borrow pit is located in a high density area and the inhabitants put the large body of water to several uses such as washing, bathing, swimming and fishing. It is on the basis of the health implication that this study is carried out to evaluate the surface water quality and how it impacts the adjoining ground water.

Statement of Problem

Rapid urbanization and lack of sound waste management systems, in the developing world, has led to the indiscriminate dumping of organic and inorganic materials especially into open spaces (ISWA, 2015; Ugbaja, 2004) such as the Rumuola borrow pit. These wastes may have some consequences on the surface and ground water quality with attendant health implications. It is on this basis, that the groundwater and surface water was monitored to ascertain their status.

Aim / Objectives of the Study

The aim of this study was to examine the possible impact of municipal wastes dumped into the borrow pit on the surface water and adjoining ground water quality. The objectives of the study are to determine the impact of municipal wastes discharged and make appropriate recommendations on the amelioration of any negative effects on the water quality; determine the physico-chemical characteristics, concentrations of specified trace metals, Polycyclic Aromatic Hydrocarbons (PAHs) on the surface water and adjacent boreholes; provide data and information on the water quality around the Rumuola Borrow Pit used to monitor future changes in the system and finally determine the possible sources/origin of contaminants.

Description of the Study Area

The Rumuola Borrow Pit is located at the vicinity of an Oil Location Road, Rumuola in Port Harcourt, Rivers State. It is situated between longitudes E 007° 00' 01.0" to E 007° 03' 09.7" and latitudes N 04° 50' 08.5" to N 04° 50' 14.2" as indicated in Figure 1. The climate is typical of the Equatorial Tropical Latitude. Temperature ranges from a minimum of 21.2 - 23.2 °C and a maximum of 28.7 - 33.4 °C. Annual rainfall is 2,480 mm while average precipitation ranges between 22.2 mm and 367.1 mm. The soil was mined for civil construction in 1982 (32 years ago) and that gave rise to this large borrow pit covering an area about 135,000 m^2 with a depth of about 7.68 m high. Depth to the ground water table is 8 ft 5 inches. The area is a lowland area now developed by civil engineering structures such as buildings and bridges. Inhabitants use the water from the borrow pit for washing clothes and cars, fishing, bathing and a section as fish ponds.

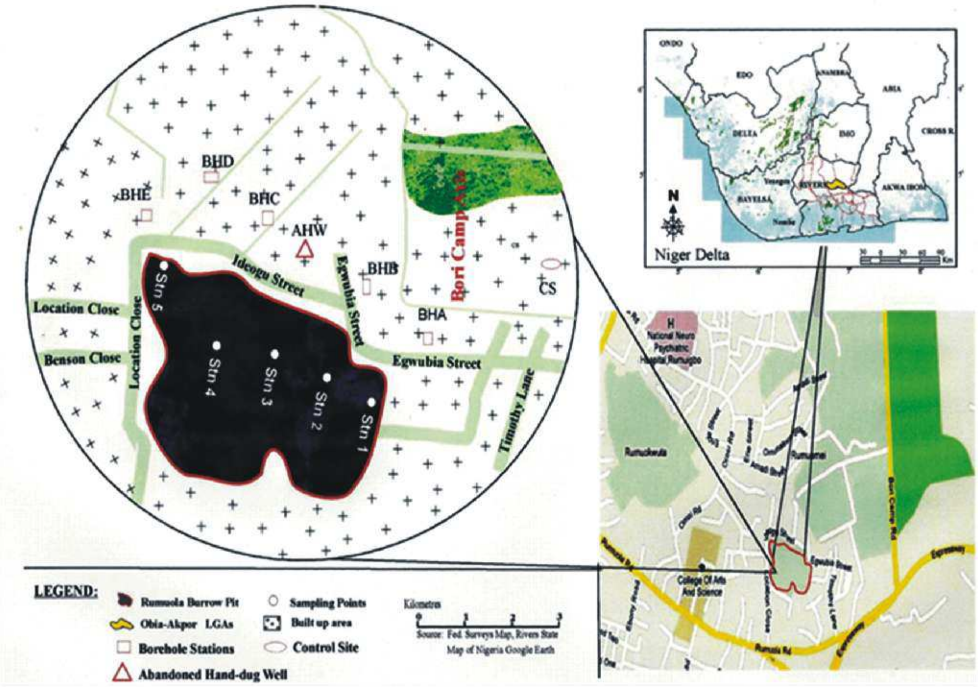

Fig. 1: Map of the study area (Rumuola Borrow Pit)

Plate 1: Sewage pipes channeled directly into the Borrow Pit

Plate 2: Scavengers at work around the Rumuola Borrow Pit

Plate 3: *Parachanna Obscura* (African Snake Head) *spp.* caught from pond

Plate 4: *Tilapia guineensis spp.* caught from pond

Literature Review

Solid waste results from human activities. It is generated by domestic, commercial, industrial, healthcare, agriculture and mineral extraction activities which eventually accumulate in streets and public places (Isirimah, 2000; Gobo, 2002). Population explosion in coastal towns like Port Harcourt has resulted in an increase, by several magnitudes, of waste generated (Ayotamuno, 2004). Unfortunately, waste management in Nigeria and most underdeveloped nations has not received the desired attention. In these nations, disposal of solid waste in open dumps is most common (ISWA, 2015). Indiscriminate dumping of municipal wastes can be linked to environmental pollution through the introduction of chemical substances above threshold limit into the environment (Etusim, 2013); introduction of additional heavy metal into surrounding soil and groundwater (Arimieari, 2014); encourages the growth of populations of insect vectors and rodents that transmit various kinds of diseases (ISWA, 2015); generation of unhealthy odours by the uncontrolled release of methane and carbon dioxide (biogas) from decaying organic substances or anaerobic decomposition (Ngah, 1993); devaluation of property and infrastructure (Wokekoro, 2014); reduces aesthetic beauty of the environment (Ukpong, 2015) and initiates road accidents especially of commercial motorcycles popularly called okada (Ukpong, 2015).Conclusively, Municipal waste disposal into open dumps is a major cause of public health problems.

Data Analysis Technique

A reconnaissance visit was made to the borrow pit and five sampling stations established within the water body (BPA – BPE). Five water borehole stations were chosen adjacent to the borrow pit points (BHA – BHE). Water sample was collected from an abandoned hand dug well adjacent to Borrow Pit Station C. A control station was established 300 m upstream from dumpsite Table 1.Five sampling stations were established within the Rumuola borrow pit water body coded BPA – BPE. Two fish species – African Snake Head spp. and Tilapia spp. (Plate 3 and Plate 4) were caught from the pond. The study commenced in August 2010 and was undertaken to cover the wet and dry seasons. Standard methods were adopted for field and laboratory studies (APHA, 1998). Physico-chemical parameters; Heavy Metals and Polycyclic Aromatic Hydrocarbons (PAHs) were analyzed in Surface Water, Ground Water (Table 2) and Fish species.

Table 1: Study Sites and their Coordinates

S/No.	Code	Coordinates (determined with a GARMIN GPS 72) *Northing*	*Easting*
1	BPA	04°50'08.5"	007°00'01.0"
2	BPB	04°50'13.8"	007°00'02.1"
3	BPC	04°50'12.2"	007°00'06.4"
4	BPD	04°50'10.2"	007°00'09.7"
5	BPE	04°50.172'	007°00'01.0"
6	BHA	04°50'06.8"	006°59'59.6"
7	BHB	04°50'15.7"	007°00'02.6"
8	BHC	04°50'14.2"	007°00'07.7"
9	BHD	04°50'10.0"	007°00'13.1"
10	BHE	04°50.166'	007°00.219'
11	CS	04°50.164'	007°00.180"
12	AHW	04°50'13.8"	007°00'10.2"

*B P - Borrow Pit Station; BH - Borehole; CS - Control Station; AHW – Abandoned Hand Dug Well.

All samples were collected in appropriate containers, labeled and transported to Institute of Pollution Studies (IPS) laboratory, Rivers State University of Science and

Table 2: Field and laboratory methods for analyzing pollution indication parameters

Parameter	Field and Laboratory Method
Temperature	Mercury-in-glass thermometer
pH value	Horiba multiprobe U-10 water checker
Dissolved Oxygen (DO)	Winkler method (Stirling, 1999)
Biochemical Oxygen Demand (BOD_5)	Winkler's method
Sulphate	Spectronic Spectrophotometer 21D – Turbidimetric method
Phosphate	Spectronic Spectrophotometer 21D – Stannous Chloride method
Nitrate-Nitrogen	Brucine method
Heavy Metal	Atomic absorption spectroscopy (AAS)
Polycyclic Aromatic Hydrocarbon (PAH)	Gas Chromatographic (GC) method using Flame Ionization Detector (FID)

Technology, Port Harcourt for analysis. Standard analytical methods were adopted for all the physicochemical analysis (Ukpong, 2015).

Data Analysis

Statistical tools such as mean, standard deviations and variance were used to analyze the data obtained to ascertain how representative and close the data obtained were. Two-way analysis of variance (ANOVA) and Students T-test analysis was used to show temporal variations and also to determine significant difference between the stations and months of sampling. Bar charts and multiple bar charts were employed to illustrate. Correlation coefficient analysis was carried out to highlight the relationship between the parameters.

Results and Discussions

Results

Surface Water

Temperature values were higher in the dry seasons 26.90°C - 30.7°C (mean 27.78°C). pH values were slightly acidic in the dry season 6.4 – 6.7 (mean 6.58) and neutral to slightly alkaline 7.1 – 7.5 (mean 7.30) in the wet season. Borrow Pit Station E recorded the highest value for Turbidity (13 NTU) in the rainy season. Dissolved Oxygen (DO) concentration was higher in the dry season with values between 2.8mg/l and 7.3mg/l (mean 4.89 mg/l). Biochemical Oxygen Demand (BOD_5) test results ranged from 0.8mg/l – 4.9mg/l (mean 2.38 mg/l) in the rainy season. The Hardness levels were higher in the wet season at 46.1 mg/l as $CaCO_3$ – 76.8 mg/l as $CaCO_3$ (mean 66.74 mg/l as $CaCO_3$). Sulphate had the highest concentrations amongst the nutrient variables during the dry season and ranged from 3.8 mg/l – 14.7 mg/l (mean 6.24 mg/l). Phosphate concentrations were higher (<0.05 mg/l – 0.05 mg/l) in the wet season and Borrow pit station E had the highest values. Nitrate –Nitrogen (NO_3-N) had higher concentrations <0.05 mg/l – 0.15 mg/l (mean 0.085mg/l) in the wet season.

Cadmium concentrations ranged between <0.01mg/l – 0.008 mg/l (mean 0.029 mg/l) and <0.003 mg/l – 0.004 mg/l (mean 0.0005 mg/l) for the dry and wet season respectively. Mean concentrations of Iron detected were above the permissible limits. Lead concentrations were higher in the wet season < 0.001 mg/l – 0.230 mg/l (mean 0.044 mg/l) than in the dry season < 0.001mg/l for all stations. Most values exceeded the permissible limits of 0.01 mg/l. The concentrations of total Polycyclic Aromatic Hydrocarbons (Σ PAHs) ranged from 112 – 133 µg/l (mean 122.830 µg/l). Σ PAH concentration of the seven carcinogenic PAH compounds ranged from 73.703 – 128.222 µg/l (mean 108.270 µg/l). Benzo(b)Fluorantheneand Indeno (1,2,3,cd) Pyrene had the highest concentrations. They are carcinogenic PAHs.

Ground Water

Temperature values ranged from 26.9°C – 29.6°C (mean 27.6°C) and 22.6°C to 28°C (mean 25.12°C) for the dry and wet seasons respectively. Hydrogen ion (pH) ranges in the dry and wet seasons were 3.82 – 6.7 (mean 4.49) and 4.55–7.9 (mean 5.09) respectively. Mean Conductivity levels for the dry season was higher [16.8 µS/cm – 585 µS/cm (mean 125.88 µS/cm)] than in the wet season (49 µS/cm - 278 µS/cm (mean 113.16 µS/cm)].

The highest levels of Cadmium concentrations were recorded in Bore Hole Station E (0.007mg/l) and above the W.H.O. permissible limits of potable water (0.003 mg/l). Cadmium showed non-significance with other variables. Chromium concentration was <0.003mg/l for all stations and were within the permissible limits of potable water. Lead concentrations were higher in the wet season. Concentrations of Iron found in most of the boreholes were within the permissible limits of potable water.

The total Polycyclic Aromatic Hydrocarbons (Σ PAHs) concentrations in the water sample collected from the abandoned hand dug well was 4.365 µg/l. Σ PAH concentration of the seven carcinogenic PAH compounds was 4.337 µg/l.No PAH component was detected in the borehole samples.

Fish Species

Zinc concentrations ranged from 35.98 µg/kg in African snake head species. to 36.8 µg/kg in Tilapia species, with a mean of 36.41 ± µg/kg.In general, Zinc metal had the highest concentration when compared to the other metal concentrations studied, with Tilapia having the higher concentration. This was followed by Lead, Nickel, Chromium and Cadmium. Apart from Zinc and Nickel, African snake head fish species had higher metal concentrations than Tilapia fish species.

Discussion

Surface Water

The surface water temperature values were generally high. The dry season recorded higher temperatures (26.90°C - 30.7°C) typical of the Niger Delta region. This was attributed to its location at the equatorial latitude where ambient temperature is consistently high all year round (Egborge, 1986). At leachate inflow points, lower pH values were observed. These lower pH values resulted from high acidic potential substances dumped and subsequently hydrolyzed. This indicated that the dump site was impacting increased acidity in the borrow pit. Organisms living in the habitat would be adversely affected and biodiversity disrupted as a result of the altered pH (Umunnakwe, 2009). Conductivity concentrations were higher in the wet season (mean range 233.50 *µS/cm* – 252.50 *µS/cm*) when compared with the dry season. Similar information was gathered by Chindah, *et al.*, 2005 for Agbonchia and Ntawogba streams in Port Harcourt. Most Turbidity levels recorded during the rainy season exceeded the permissible limit of 5NTU (W.H.O., 2006). This could be as a result of run-off through the dumpsite into the borrow pit. The BOD_5 concentration serves as a measure of organic pollution in water bodies. Based on this criterion, Moore and Moore (1976) (Moore, 1976) ranking (Table 3) revealed the state of the surface water as doubtful (Borrow Pit Station E measured 4.9mg/l). In both seasons, the BOD_5 concentrations increased downstream.

Table 3: Biochemical Oxygen Demand Levels and the State of the Water Body (Moore and Moore Ranking, 1976)

S/No.	BOD levels (mg/l)	State of Water body
1	1.0 – 2.0	Clean
2	< 3.0	Fairly Clean
3	< 5.0	Doubtful
4	10.0	Bad and Polluted

Nutrient parameters (Sulphate, Phosphate and Nitrate-Nitrogen) were below W.H.O., 2006 permissible limits. However, higher concentrations of Phosphate at the borrow pit station E may be as a result of the impact of the waste discharges but organisms in tropical water bodies quickly use up these nutrients under high temperature conditions (Chindah, 2005). Nitrate –Nitrogen (NO_3-N) had higher concentrations above the EPA permissible limits of 0.1mg/l. This high level showed the presence of organic pollution (from human or animal wastes). A positive significance was observed with Turbidity (r^2 = 0.54), Sulphate (r^2 = 0.98), Phosphate (r^2 = 1.00) and Zinc (r^2 = 0.96) in the wet season. All nutrient variables displayed similar trends in correlation significance for both seasons.

Chromium had a positive significance with pH (r^2 = 0.88), Conductivity (r^2 = 0.98), DO (r^2 = 0.54), BOD_5 (r^2 = 0.63), Hardness (r^2 = 0.78), Cadmium (r^2 = 0.57) and Iron (r^2 = 0.57) in the dry season. Cadmium concentrations recorded in the dry season (Borrow Pit Station A - 0.004 mg/l and Borrow Pit Station E - 0.008 mg/l) exceeded the permissible limit of 0.003 mg/l (W.H.O., 2006) and could have a possible effect on the kidneys of human beings. In the dry season, Cadmium showed negative significance with most variables with exception of pH (r^2 = 0.52), Conductivity (r^2 = 0.62) and Chromium (r^2 = 0.57). Iron concentration was higher than the W.H.O. (2006) permissible limit. These could have resulted from external contamination or breakdown of crustal material rich in Iron content. It showed similar trends in significance like Chromium with exception of DO and Cadmium that were not significant during the dry season. Lead concentrations were higher in the wet seasons for most stations compared to the set W.H.O. limit (0.01 mg/l). Borrow Pit Station C had the highest value of 0.230mg/l. These high levels were attributed to anthropogenic enrichments arising from domestic effluents and wastes from the mechanic garage as well as surface run-off from dumpsites in the area. Lead is the most toxic of the heavy metals. Its inorganic forms are absorbed through ingestion by food, water and inhalation. In humans, exposure to Lead can result in a wide range of effects depending on the level and duration of exposure. High levels of exposure may result in cancer; interfere with Vitamin D metabolism; problems in the synthesis of haemoglobin; effects on the kidneys, gastro-intestinal tract, joints, reproductive system and acute or chronic damage to the nervous system (NIS, 2007, Duruibe, 2007). Zinc concentrations were higher in the dry seasons. It exhibited a trend different from that of the other metals assessed.

Borrow Pit Station A had the highest total PAH concentration (Σ PAHs[c] 132.951 µg/l; Σ PAHs[b] 128.222 µg/l). About 96% of the PAH concentration was comprised of the carcinogenic compounds. Benzo(b)Fluoranthene, a carcinogenic component - harmful to humans, had the highest concentration. Benzo(a)pyrene had the highest concentration in Borrow Pit Station B (1.790 µg/l) exceeding the W.H.O. recommended value of 0.70 µg/l for drinking water. This corresponded to an excess lifetime cancer risk of 10^{-6} documented by (Anyakora, 2004). Most of the PAH components exhibited strong positive significance with each other.

Ground Water

Temperature values were generally high as characteristic of the equatorial tropical latitude. Water was acidic in the dry season to neutral in the wet season. The concentration of Total Hardness in samples collected from Bore Hole Station E exceeded the WHO desirable limit of 150mg/l as $CaCO_3$ meaning that the water is hard. This indicates that he water might not be suitable for laundry as it may need more soap to lather. Borehole station A showed conductivity levels of 585µS/cm that exceeds the EPA permissible limits but with no health implications. Based on the Moore and Moore (1976) ranking, BOD_5 concentrations revealed the state of the ground water as fairly clean. It is advised that Borehole Station A and B with Nitrate-Nitrogen concentration of 0.76mg/l and 1.32 mg/l respectively should be checked periodically by local or state health authorities. Concentrations as low as 10 to 20 mg/1 have caused illness and even death among infants under six months of age. It results in oxygen starvation called methemoglobinemia, or more commonly, the "blue baby" condition (Husaini, 2007). All nutrient parameters had values above the W.H.O. permissible limits of 0.1mg/l indicating the presence of organic pollution from human or animal wastes.

Borehole E recorded a Cadmium level of 0.004 mg/l above the permissible limits of 0.003 mg/l (W.H.O., 2006). This concentration could possibly affect the kidneys of human beings (Ogbonna, 2006). The high proportion of Lead in the sampled boreholes indicates Lead contamination. No PAH component was detected in the borehole samples indicating that the water is safe for consumption, based on this criterion.

Fish Species

Almost all metals present in the environment have been biogeochemically cycled since the formation of the Earth. However, human activity has introduced additional processes that have increased the rate of redistribution of metals between environmental compartments, particularly since the industrial revolution (Khim,1999). The high level of Zinc, indeed all metals in fish is due to environmental level in the pond. Zinc is part of nature. Most rocks and many minerals contain zinc in varying amounts and zinc exists naturally in air, water and soil. The average natural level of zinc in the earth's crust is 70 mg/kg (dry weight), ranging between 10 and 300 mg/k (Malle,1992).

Although zinc occurs naturally, most zinc finds its way into the environment because of human activities. Mining, smelting metals (like zinc, lead and cadmium) and steel production, as well as burning coal and certain wastes can release zinc into the environment.

Studies on the different parts of fish revealed higher concentrations of 4.00 µg/g Ni in the head of the Tilapia fish and 2.40 µg/g Ni in the intestine of African Snake Head fish. The highest concentration of 1.95 µg/g Zn was detected in the head of the African Snake head fish while the lowest concentration with a value of 0.16 µg/g was recorded in Tilapia fish. The concentration of Zn in the water is within the limits permitted by the Lagos State Environmental Protection Agency (LASEPA) of 1.0 mg/L Zn set for water. None of the trace metals investigated was above the maximum permissible level set by world health organization (WHO) (Olowu, 2010).

The accumulation of metals in an aquatic environment has direct consequences to man and to the ecosystem. Interest in metals like Zn and Cu which are required for metabolic activity in organisms, lies in the narrow "window" between their essentiality and toxicity (Skidmore,1964; Spear, 1981). Others like Al, Cd and Pb exhibit extreme toxicity even at trace levels (Merian,1991; Fatoki, 2002).

Extensive literature on the aquatic toxicity of Zn and especially its toxicity to fishes has been reviewed by (Alabaster, 1980) and by (Malle, 1992). Zinc is unusual in that it has low toxicity to man, but relatively high toxicity to fish (Alabaster, 1980; Fatoki, 2002).

Higher Polycyclic Aromatic Hydrocarbons concentrations found in the fish tissue of Tilapia fish spp. showed its ability to accumulate PAHs more readily than the African Snake Head fish spp. Naphthalene was not detected in both fish samples analyzed and could be related to the fish species inability to absorb it irrespective of its presence in the water and sediments. Total PAH concentrations found in both fish species exceeded the acceptable limit set by the European Commission (2 µg/kg) for fish considered safe for human consumption. Carcinogenic PAH compounds comprised over 98% of the total PAH components found in Tilapia species.

Molecular Ratios as Determinants of Sources of PAHs

Molecular indices based on ratios of individual PAH levels in water can be used to distinguish PAHs from pyrolytic/pyrogenetic (e.g. fossil fuel combustion and vegetation fires) and petrogenic/petroleum (e.g. oil spill and petroleum products) input. In this study, two ratios were used to indicate the source of PAHs.

1. Phenanthrene (Phe) / Anthracene (Ant)
2. Fluoranthene (Flu) / Pyrene (Pyr)\

For petrogenic origin ; Phe/Ant>10 and Flu/Pyr<1
Pyrolytic origin; Phe/Ant<10 and Flu/Pyr>1

From the study, it can be concluded that the PAHs are of both pyrolytic and petrogenic origin since Phe/Ant<10 (mean value 1.660) and Flu/Pyr<1 (mean value 0.030) in surface water for all stations. The study shows that the total PAH in Tilapia was higher than the range found in African snake head species. The PAH profiles revealed that three, four, five and six rings PAH dominated in the two fish samples.

The ratios of Phe/Ant, Flu/Pyr, Flu/(Flu + Pyr) and BaA/(BaA + Chyr) in both species indicated both petrogenic and pyrolytic sources of PAH. Alternant PAHs which are high molecular weight PAHs (4, 5, 6 rings) such as benzo(ghi) perelyne , dizenz(a,h) anthrancene and indeno (1,2,3-cd) pyrene were detected and had high concentrations. Carcinogenic PAHs were generally higher in concentration than the non-carcinogenic ones.

Conclusion and Recommendations

Conclusion

Depletion of the Dissolved Oxygen downstream in surface and ground water are clear indications of the impact of these municipal wastes dumped indiscriminately into the Rumuola Borrow Pit. PAHs found in the Rumuola Borrow Pit are of pyrolytic (e.g. fossil fuel combustion and vegetation fires) and petrogenic (e.g. oil spill and petroleum products) origin as indicated by the molecular indices – Phenanthrene /Anthracene and Fluoranthrene / Pyrene. Concentrations of the low molecular weight (LMW) PAHs such as Naphthalene, Acenaphthylene, Acenaphthene, Pyrene, Fluoranthene, Fluorene, Phenanthrene, Chrysene, Benzo(a)Anthracene and Anthracene were found to be lower than that of the high molecular weight (HMW) PAHs which includes Benzo(k)Fluoranthene, Benzo(b)Fluoranthene, DiBenzo(a,h)Anthracene, Benzo(g,h,i)Perylene, Indeno(1,2,3-cd)Pyrene and Benzo(a)Pyrene. Benzo(b)Fluoranthene and Indeno(1,2,3-cd)Pyrene had the highest concentrations. Most of the PAHs like Benzo[a]Anthracene, Chrysene, Benzo[b]Fluoranthene, Benzo(g,h,i)Perylene have properties that are carcinogenic hence dangerous to health.

The fact that PAHs were not detected in the borehole samples infers that these boreholes are not being impacted by the water quality of the Rumuola Borrow Pit and its water is safe for consumption. The findings of this study showed that there is relatively high level of PAHs in the environment. This suggests significant risk of cancer to the people of this environment. The study of the presence of polycyclic aromatic hydrocarbons (PAHs) and heavy metals in the aquacultural environment is of global importance because they play an important role in fish bioaccumulation of transfer of heavy metals to humans after consumption.

Recommendations

From the study carried out in the borrow pit and adjacent boreholes in the Rumuola vicinity, I strongly suggest continuous public enlightenment programs for the inhabitants, the dire need to hire the services of qualified borehole drilling contractors with requisite knowledge of boreholes, its effective treatment methods and routine treatment plan. The borehole water should be treated before consumption and good hygiene observed by inhabitants. Human activities, surface water and groundwater should be monitored by the relevant government agencies to ensure a reduction in level of pollution from the borrow pit. Government agencies should ensure that the standards set, to ensure that the concentrations of these elements do not exceed the set permissible level, are adhered to strictly.

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The Subsurface Stratigraphy and Foundation Quality of Soils Underlying Uyo Town, Southeastern Nigeria

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Introduction

Aim of Study

This study delineates the various subsoil horizons, their lateral extent and determines their engineering properties in order to ascertain their quality as foundation materials.

Objectives of Study

- To determine the subsoil profile Uyo town and its lateral extent
- To determine the engineering properties of the subsoil
- To determine the bearing capacity of the subsoil

Scope of the Study

- ✓ Drilling of eleven (11) boreholes to a maximum depth of 20m each beneath the ground surface at different locations in Uyo Town
- ✓ Sampling and description of the subsoil
- ✓ In situ and laboratory determination of the engineering properties of the soil
- ✓ Bearing capacity analysis of the soil

Fig. 1: Ariel view of Itam Bridge, Uyo, Akwa-Ibom (Source: Thisday Newspaper, 26 August, 2016).

Method of Study

Field Studies

- ✓ Drilling of eleven (11) boreholes to a maximum depth of 20m each using the Shell and Auger Percussion drilling rig.
- ✓ Sampling and description of the soil samples
- ✓ In situ Standard penetration test and Cone

Penetration tests using a 2.5 ton CPT machine.

Laboratory Studies

The laboratory analysis captured the moisture content, specific gravity, atterberg limits and particle size distribution. Test Procedures are in accordance with British Standards BS 1377 (1990)

Fig. 2: CPT Analysis BH 3, Nwaniba Road, Uyo.

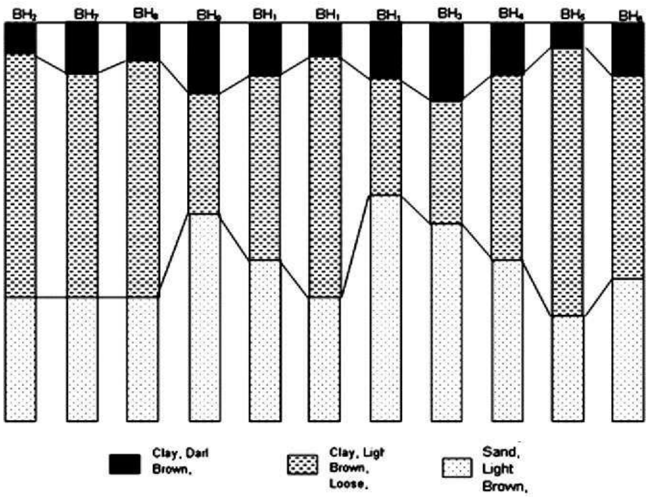

Fig. 3: Subsoil profile of Uyo Town

Table 1: Summary of Soil Stratigraphy

LAYER	DEPTH INTERVAL (m) MIN	MAX	DESCRIPTION OF SOIL	AVERAGE THICKNESS OF SOIL (m)
I	0	3	CLAY, DARK BROWN, LOOSE, SANDY, SILTY	3
II	3	15	CLAY, LIGHT BROWN, LOOSE, SILTY, SANDY	12
III	10	20	SAND, LIGHT BROWN, FINE-MEDIUM-COARSE	10

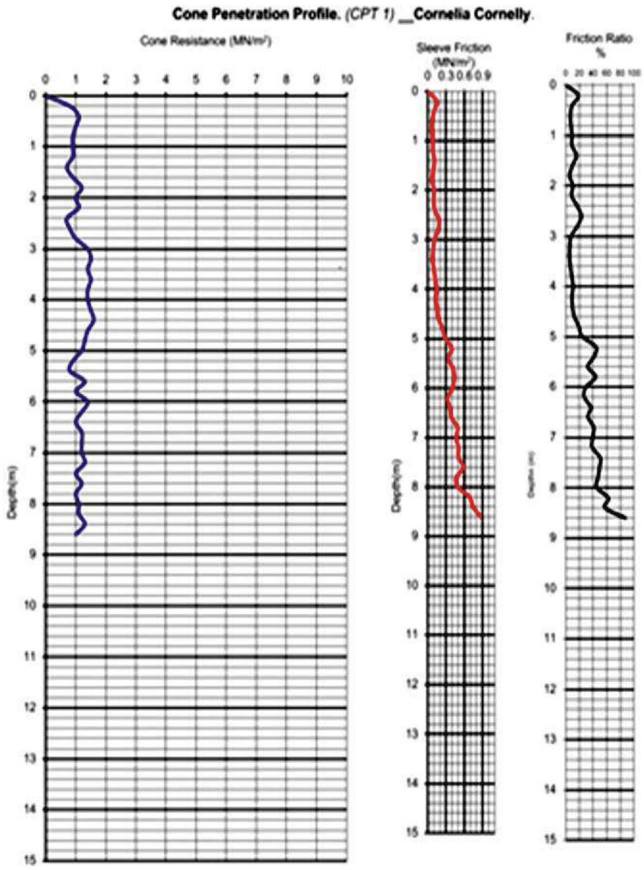

Fig. 4: A typical CPT Plot

Table 2: Water Table Depths

BOREHOLE NUMBER	BH1	BH2	BH3	BH4	BH5	BH6	BH7	BH8	BH9	BH10	BH11
DEPTH TO WATER TABLE	12.8	16.7	13.6	15.5	15.4	16.6	16.9	16.6	15.8	15.5	15.3

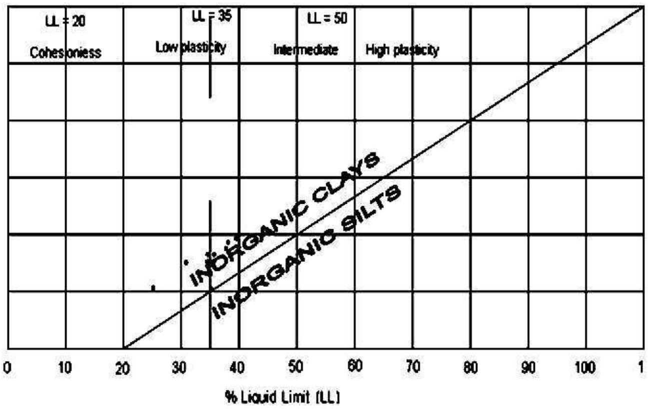

Fig. 5: Plasticity plots of the clays

Table 3: Summary of the engineering properties of the clays

BOREHOLE NUMBER	DEPTH OF SAMPLE (m) MIN	MAX	AVERAGE VALUES OF MOISTURE CONTENT (%)	AVERAGE VALUES OF SPECIFIC GRAVITY	AVERAGE VALUES OF LIQUID LIMIT (%)	AVERAGE VALUES OF PLASTICITY INDEX (%)
BH1	0	9	23.0	2.62	34	15
BH2	0	14	23.0	2.59	34	18
BH3	0	10	23.8	2.62	39	25
BH4	0	12	25.0	2.59	37	20
BH5	0	14	24.2	2.71	38	21
BH6	0	13	23.1	2.61	35	17
BH7	0	14	19.6	2.65	35	22
BH8	0	14	24.3	2.60	25	20
BH9	0	10	23.3	2.72	35	19
BH10	0	12	23.0	2.65	35	23
BH11	0	14	20.6	2.68	32	15

L L : 25%(BH8) -37%(BH3) M.C: 19.6% (BH7) – 25% (BH4)
P I : 15% (BH1 & B11) -25% (BH3) S.G : 2.59 (BH2 & BH4) – 2.72 (BH9)

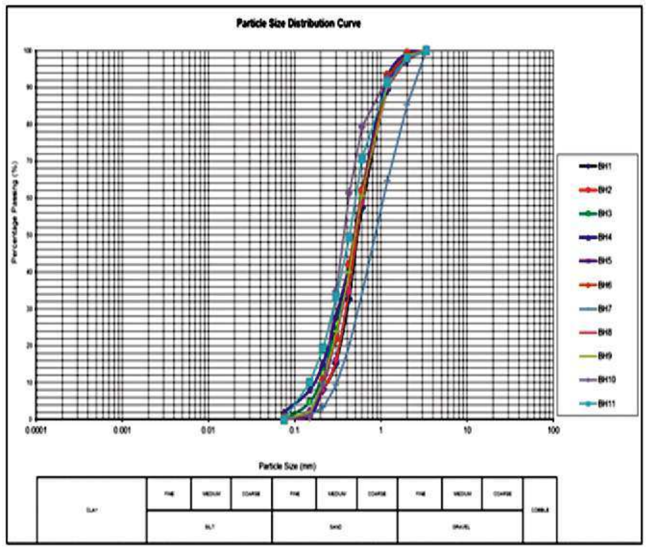

Fig. 6: Particle size distribution plots of the sand

Conclusion

- ✓ This study reveals that Uyo town is underlain by two major soil types; clay (0-15 meters) and sand (10 – 20meters).
- ✓ The firm to stiff clays are inorganic and of low plasticity. While the sand is poorly graded and of medium density.
- ✓ Analysis of the foundation potentials using CPT results revealed the clay layer may be suitable for medium structures, while both CPT and SPT results revealed that the underlying sand layer constitutes the best foundation substratum for medium to heavy structures.

Results of this study have provided data that can be used as a guide and preliminary data for planning/design of future structures.

Borehole Number	Average values of parameters from Particle Size Distribution Analysis											
	Depth of Sample (m) Min	Max	A_0 (mm)	D_{50} (mm)	Cu	K (CD_{10}) (m/sec)	Fine Sand Fraction	Medium Sand Fraction	Coarse Sand Fraction	Total Sand Fraction	SPT N – Values Corrected	Description of Soil using USCS
BH_1	10 -	20	0.26	0.61	2.4	6.47×10^{-2}	3	54	40	97	22	SP
BH_2	15 -	20	0.25	0.61	2.5	6.25×10^{-2}	4	52	41	97	13	SP
BH_3	11 -	20	0.21	0.60	2.9	4.41×10^{-2}	8	52	38	98	21	SP
BH_4	11 -	20	0.23	0.61	2.7	5.29×10^{-2}	5	52	41	98	21	SP
BH_5	15 -	20	0.30	1.18	3.9	9.00×10^{-2}	3	30	50	83	20	SP
BH_6	15 -	20	0.24	0.52	2.3	5.76×10^{-2}	8	61	28	97	20	SP
BH_7	15 -	20	0.31	1.20	3.8	9.61×10^{-2}	3	33	50	86	20	SP
BH_8	15 -	20	0.21	0.41	2.0	4.41×10^{-2}	7	72	18	97	20	SP
BH_9	11 -	20	0.26	0.80	3.1	6.76×10^{-2}	7	43	41	91	21	SP
BH_{10}	14 -	20	0.23	0.42	2.0	5.29×10^{-2}	8	72	18	98	20	SP
BH_{11}	15 -	20	0.23	0.63	2.1	5.29×10^{-2}	10	50	37	97	20	SP

%FINE SAND(7) %MEDIUM SAND (51%) %COARSE SAND (34) K 3.4×10^{-2} - 9.6×10^{-2} cm/sec M.C 14% (BH2) – 21.4% (BH11)
MED-COARSE L-BH5(30-50%) H-BH1(54-40%) SPT N –VALUES 13-22 S.G 2.59 (BH4) - 2.67 (BH6)