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Sustainability of Structures and Environment: Challenges for Engineering Geology Profession in the 21st Century

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Abstract

The concepts of sustainable development and sustainability stipulate that human economic and social development should take place without irreversible damages to the carrying capacity of the Earth's natural environment and significant depletion of non-renewable resources. The sustainable management of the Earth's finite natural resources in the face of unprecedented global population growth and climate change is one of the most profound challenges facing human society in the 21st century. Although an integrated and holistic resource management approach encompassing all disciplines has been recognized and recommended, sectoral and disciplinary approaches are still prevalent in practice. In order to enable engineering geologists play a more active role in the trans-disciplinary resource management approach and contribute meaningfully to the sustainability of engineering structures and the environment, a paradigm shift from the traditional study of planet earth as isolated discrete components to the concept of integrated whole earth system approach (Earth System Science) is imperative. This involves knowledge of the interactions between all the components of the earth system (atmosphere, hydrosphere, biosphere, cryosphere, and geosphere) and embraces a systems thinking approach. Engineering geologists should not only focus on the geological factors controlling the structural stability 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 close and functional communication between the engineering geologists and all other stakeholders involved in the pursuit of sustainable development and sustainability. This paper discusses the challenges of the sustainability of civil engineering structures and global environment in the 21st century and makes suggestions on how engineering geologists can play a more pivotal role in achieving a sustainable built and natural environment for both present and future generations.

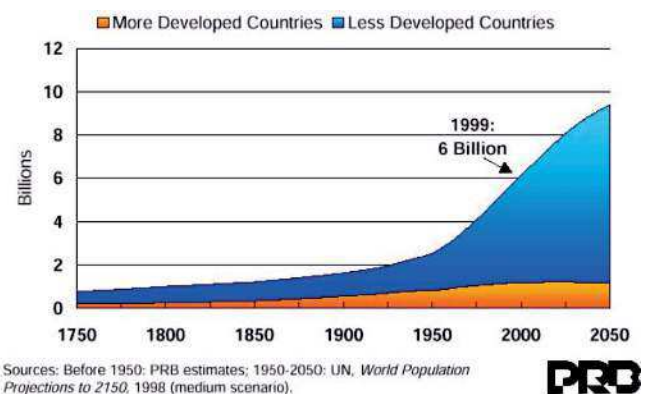
Keywords: Sustainability, structures, environment, environmental geology, integrated approach, climate change, population growth, transdisciplinary.

Introduction

Human society is currently facing several unprecedented global challenges. These include; exponential growth in population, urbanisation and industrialisation, resources depletion, food, water and energy security, poverty, environmental sustainability, climate change impact, biodiversity loss, natural hazards, etc. These challenges threaten the future of humanity and the integrity of the carrying capacity of the earth's ecosystems.

The world is currently experiencing the largest increase in population and urban growth (Fig. 1). From about the 1950s, approximately 90 million new people are added to the world population yearly. 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. This has in turn given rise to over exploitation of virtually all earth resources. Therefore, meeting the needs of more and more people (higher production & consumption) without causing

irreversible damage to the productivity capacity of the natural global environment is at the heart of the "global change" problem.



Note

- 90 million new people added each year
- Nigeria is 7th most populous nation in 2015 and will be 3rd by 2050

Fig. 1: Projected Population of the World

The report of the UN World Commission on Environment and Development (*Our Common Future*, 1978) 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 (Fig. 2). The flexibility of the definition makes room for different disciplines to develop feasible solutions that are communicated across various disciplines (Daly, 1990) with the ultimate goal of obtaining a solution that is acceptable environmentally, economically and socially. 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.

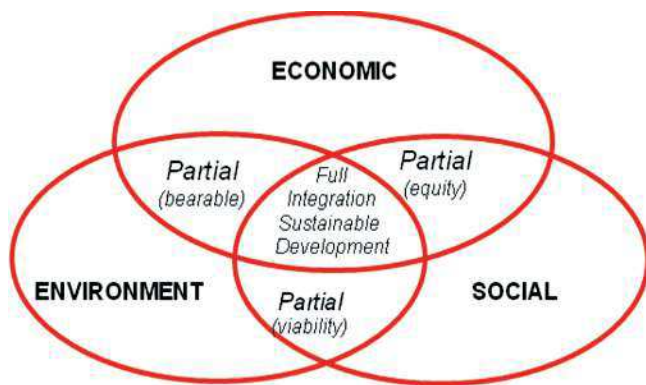


Fig. 2: Interrelations between the three Pillars of Sustainable Development

Sustainability, Engineering Geology and Environmental Geology

The global problems are highly complicated and interconnected. 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 interdisciplinary (or multidisciplinary) approach is required. Global environmental concerns emerged in the late 1960s because of the great difficulties in meeting the needs of the rapidly increasing human population and the corresponding increasing demands on the finite and depleting resources of the earth. Sustainability Science, Environmental Science, Engineering Geology and Environmental Geology have central roles to play in the formulation and implementation of appropriate sustainable solutions to global problems. These disciplines show considerable congruence between them (Fig. 3) (Akpokodje, 2016, *in press*). For example, they (i) focus on the understanding of Earth Systems, (past, present & future) and (ii) utilize multidisciplinary and interdisciplinary approaches.

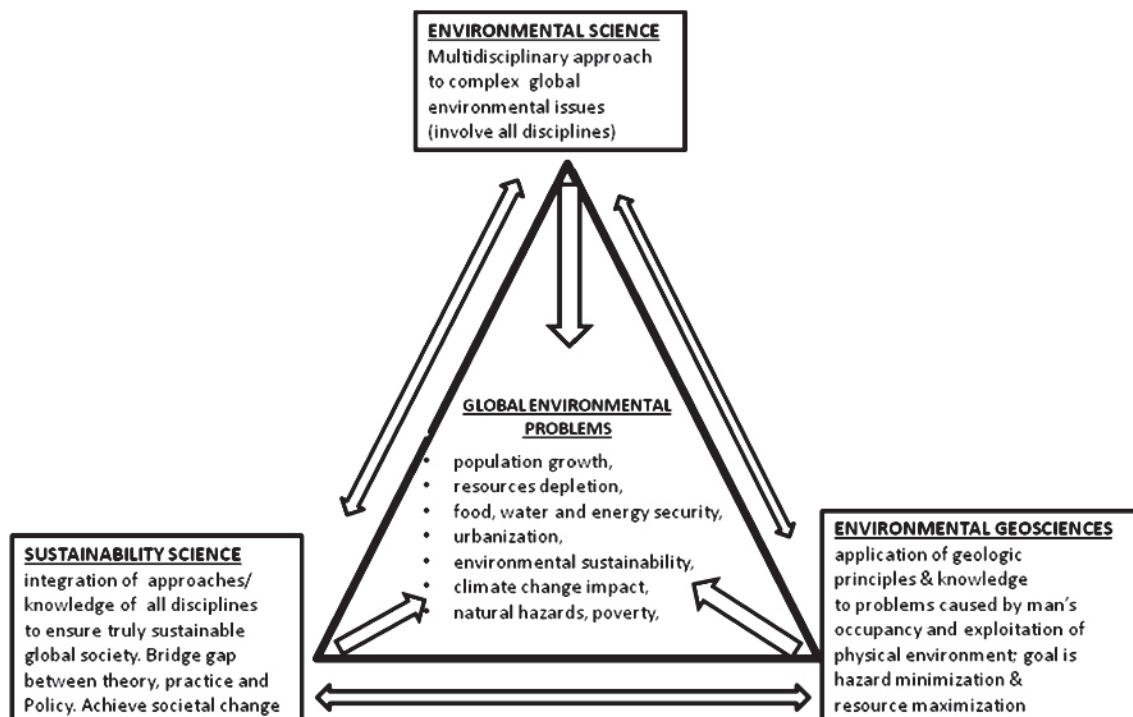


Fig. 3: Interrelations between Environment-Sustainability Science and Environmental Geosciences

Environmental Science and Sustainability Science

Environmental Science can be simply defined as the study of the biological and physical components of the natural environment (plants, animals, soils/rocks, air, and water) as well as the socio-political organizations, institutions and structures created by humankind using science and technology. In other words, it is an interdisciplinary study of how humankind affects other living and non-living physical systems of the earth's environment. It is a relatively new and highly multidisciplinary, inclusive and holistic field which incorporates knowledge from all disciplines (biological, physical, and social sciences) to address environmental issues. It gained prominence as an applied science discipline in the 1970s because of the increasing public awareness for definite actions to address environmental problems and the need to use a multi-disciplinary approach in analyzing complex environmental problems.

“**Sustainability Science**” was formally introduced as a new academic field in 2001 at the World Congress in Amsterdam, “*Challenges of a Changing Earth 2001*”, jointly organized by the International Council for Science (ICSU), the International Geosphere-Biosphere Programme (IGBP), the International Human Dimensions Programme on Global Environmental Change, and the World Climate Research Programme (WCRP). It is an interdisciplinary approach of achieving a truly sustainable global society by integrating all disciplines and policy. The website of the Proceedings of National Academy of Science of the United States of America (PNAS) defined “Sustainability Science as:

“an emerging field of research dealing with the interactions between natural and social systems, and with how those interactions affect the challenge of sustainability: meeting the needs of present and future generations while substantially reducing poverty and conserving the planet's life support systems”.

The term “sustainability” describes a state of possessing capacity to continue indefinitely and implies long-term health of global ecosystem for both present and future generations. Sustainability generally includes a balance of environmental, social and economic issues. It is sometimes incorrectly defined only in terms of general continuation or maintenance of a process. Sustainable Development is process of development from our present state towards that ideal state. It is a process of

enhancing human social-economic and ecological well-being currently threatened by our current practices which pollute the environment. In contrast, *sustainability is the destination of the process.*

The word “Sustainability” is widely used in Environmental Science, especially in terms of natural resources. According to Botkin and Keller (2003), sustainability has two scientific meanings; (i) Sustainable resource harvest means that the same quantity of resource can be harvested annually and (ii) A sustainable ecosystem is an ecosystem from which a resource can be harvested and the ecosystem is still able to maintain its essential functions and properties. Okiwelu and Noutcha, (2016) opined that sustainability is one of three themes in Environmental Science; the other two are human population and global warming.

Engineering Geology and Environmental Geology

Geology, the study of the earth, became recognized as a branch of the Physical Sciences in 1786. Engineering Geology and Environmental Geology are relatively new sub-disciplines that were not generally integral parts of traditional textbooks or curricula of Geology. Other sub-disciplines within this so-called “non-traditional geology” include Forensic Geology, Medical Geology, Geoindicators, etc.

The earth systems have changed more rapidly in the past 60 years than any other times in human history. Geoscientists (including Geologists) have a new and critical 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 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 consonance with this paradigm shift.

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 of 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 prepared 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 American Geological Institute (AGI, 1981) defined Environmental Geology as:

“Application of geologic principles and knowledge to problems caused by man's occupancy and exploitation of the physical environment. It involves problems concerned with construction of building and transportation facilities, safe disposal of liquid and solid wastes, management of water resources, evaluation and mapping of rocks and mineral resources, and long-range physical planning and the development of the most efficient and beneficial use of the land” (Bates & Jackson, 1987).

Although this definition has remained essentially the same since then, there is increasing emphasis on hazard mitigation and environmental restoration in the practice of the profession. According to Hasan, (1993), the overarching goal of Environmental Geology is hazard minimization and resource maximization. Environmental Geology encompasses a wide range of geoscientific fields that include interactions between the four components of the earth (geosphere, hydrosphere, atmosphere and biosphere) as well as the activities of humankind and their impacts on the natural

earth systems (Montgomery, 1994).

Initially, Engineering Geology and Environmental Geology were regarded as the same discipline (John Irey, 1975). The very close connection between Engineering Geology and Environmental Geology is clearly demonstrated in their definitions. However, Hasan (1993, 1996, 2002) and most contemporary researchers regarded both Engineering Geology, Environmental Geology as separate disciplines, a position that is still held by the majority of geoscientists in the field today. In addition to all the topics covered in Engineering Geology, Environmental Geology also covers waste disposal, land use planning, environmental health, pollution prevention and environmental law. Furthermore, Environmental Geologists also interact with more professionals from other disciplines.

Sustainability of Structures and the Environment: Need for Geologic Input

The onset of the Industrial Revolution around 1760 led to extensive excavation (many associated with mineral mining) and construction of large civil engineering structures (such as dams, power plants, etc). However, the danger to the earth and its environment as a result of increased industrialization, urbanization and gluttonous exploitation of resources was only realized in the 20th century. The issue attained global priority attention with emphasis on the need for environmental protection and restoration by the end of the 20th century.

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 alteration of the land-water ecosystem is a unique problem in deltas more than any other region in the world. The proper use of geologic principles and engineering practices can minimize both environmental degradation and economic losses.

Over half of world population now lives in urban areas with increasing demand for buildings and infrastructures. Changes in 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 (*Chrisna du Plessis –Agenda 21, Sustainable construction in Developing countries*)

The current boom in infrastructure and reclamation projects in waterfronts and fragile ecological areas globally, including Lagos, provides a unique opportunity to expand the role and scope of engineering and environmental geology in context of sustainability and sustainable development. The question to be asked is - how much emphasis is placed on engineering and environmental geology knowledge and experience in the selection of consultants and contractors for these mega projects? Unfortunately, despite the 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 these mega projects. To reverse this attitude, there must be deliberate efforts to ensure close and functional communication between engineering and environmental geologists and all other stakeholders involved in achieving sustainable development and sustainability which has hitherto received little attention. A recent global survey on the participation of geologists on sustainability issues revealed only about 5% contribution in comparison to the social and biological sciences that contribute 34% and 23% respectively (Fig. 4).

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, and the properties of the sediments 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 structures and the earth's environment.

To apply geologic knowledge and principles to providing sustainable solutions to challenges facing structures and the earth's environment, there must be close and functional communication between environmental geologists and all other stakeholders involved in delta management. In figure 5, this is represented by the two lower arrows pointing to the left. Traditional management approaches have significant adverse impact on the sustainability and stability of structures and the earth's environment partly because of either none or insufficient application/integration of geologic knowledge and principles. 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 addition to the system and integrated approach, there is growing global consensus that collaboration and cooperation which cuts across disciplines, sectors, regions and also incorporating good policy and governance are required to effectively address the challenges facing sustainability of structures and the earth's environment.

Concluding Remarks

The human society is currently facing several unprecedented global challenges that 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. An inclusive, holistic and interdisciplinary (or multidisciplinary) approach is required. Currently, there is apparent

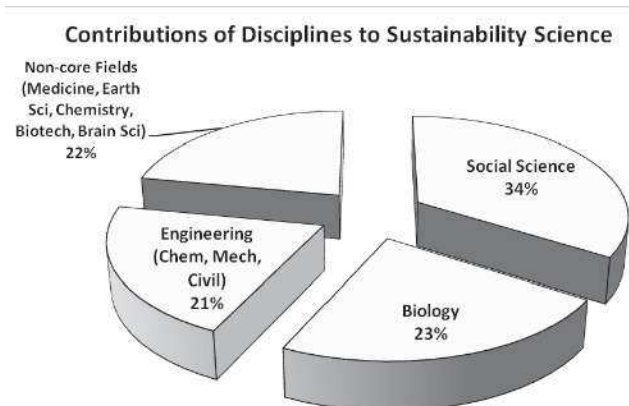


Fig. 4: Contributions of Disciplines to Sustainability Science.

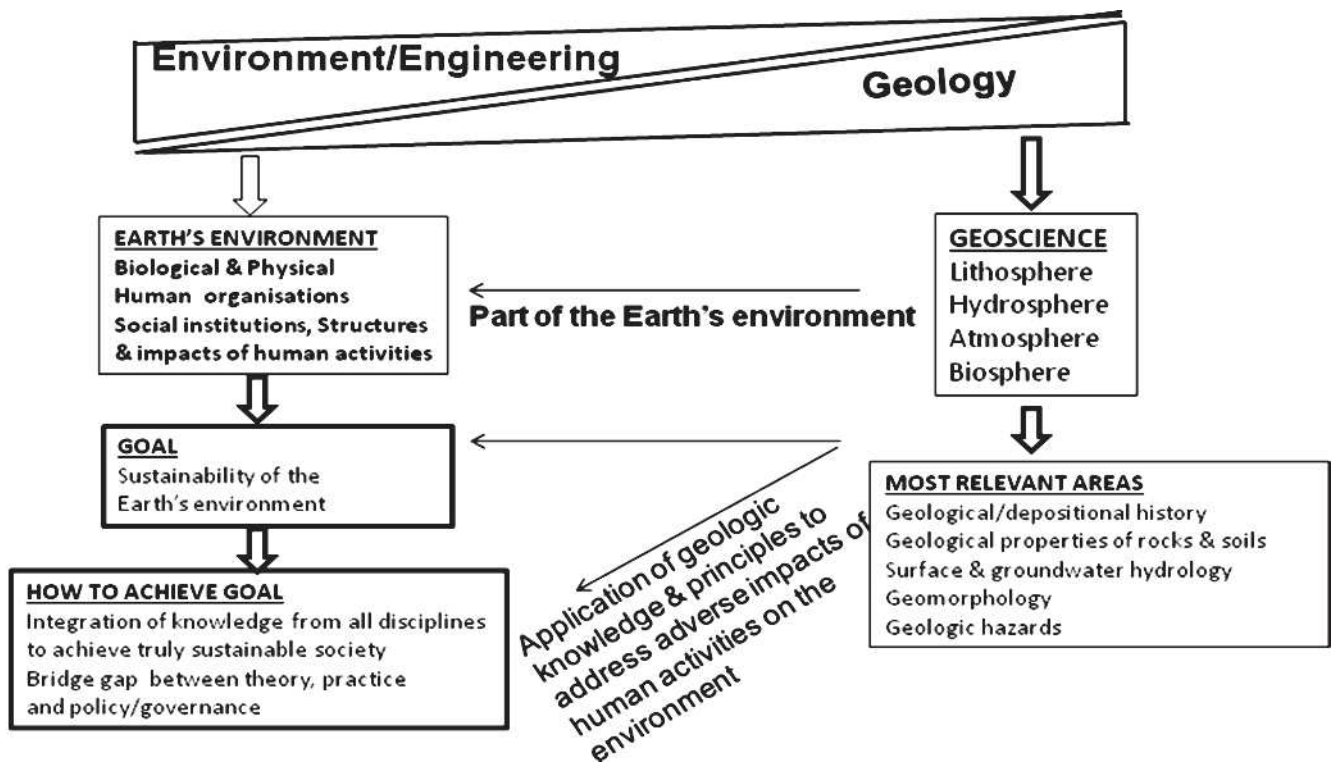


Fig. 5: Environmental Geosciences Sustainability of Structures and the Earth's Environment

insufficient application and integration of geologic knowledge and principles in engineering and environmental sustainability issues.

In order to enhance greater input by engineering and environmental geologists towards the goal of attaining sustainability of the earth's environment and infrastructures in Nigeria, the following steps should be undertaken:

- Improve the academic training of would-be practitioners, to include practical experiences and suitable mentoring by experienced proficient practitioners;
- Upgrade practitioners' fundamental knowledge of Sustainability Science, Environmental Science and other relevant non-geology disciplines;
- Establish a professional body (like COMEG, COREN, etc) for the registration of Engineering geologists and Environmental geologists as a platform for professional registration and advocacy for required policy changes;
- Advocate for critical policy changes in the conditions of engineering and environment-related contracts to include specified and mandatory input from engineering and environmental geologists;
- Ensure close and functional communication with all relevant stakeholders, especially experts from the core sustainability science disciplines (social sciences, biology, engineering) as well as decision/policy makers and administrators.

Organize regular conferences, seminars and workshops including those on major infrastructural and environmental projects to promote the much needed wider public interest in the engineering and environmental geology profession.

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Engineering Geophysics: A Veritable Tool in Geotechnical Investigation

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Abstract

Engineering Geophysics is a branch of near surface Geophysics employed in Geotechnical investigation. In developed countries, it has gained wide publicity in subsoil investigation for structures and evaluation of naturally occurring construction materials. It is usually not possible to execute purely pre-construction comprehensive geotechnical investigation for structures such as highways, dams, buildings, bridges and tunnels due to time constraint and financial consideration. Execution of surface geophysical investigations can be followed by sinking of few confirmatory boreholes and execution of few penetrometer tests. The generated mathematical models can be used to estimate values of geotechnical parameters from those of geophysics. The resulting comprehensive investigation can prevent differential settlement which often leads to collapse of structures.

Keywords: *Engineering Geophysics, Geotechnical Investigation, Construction, Buildings, Differential settlement.*

Introduction

Engineering Geophysics is a branch of near surface Geophysics that relates geophysical data with engineering properties of soils and rocks. It is not usually possible to execute geotechnical investigation (in-situ test & sinking of boreholes) everywhere at a proposed site due to time and cost constraints. Geophysical surveys (Electrical Resistivity and Seismic refraction) can be cheaply and rapidly executed to generate subsurface models. Few in-situ geotechnical tests and confirmatory boreholes can then be executed. This approach is particularly useful at sites with complex geology that can result in inhomogeneity in foundation soils and rocks. Comprehensive investigation of the subsurface can prevent differential settlement that often results from heterogeneous earth materials below the foundation.

Engineering geophysical investigation is also important in characterising the subsurface and ground-water condition of proposed highway routes. Depth to thickness of and quality of naturally occurring construction materials such as gravels, sands, clay, lateritic soils and rocks can also be cheaply and rapidly determined with near surface geophysical surveys.

Subsoil Investigation for Buildings

Important geotechnical data for foundation design are Allowable Bearing Pressure and settlement. Although the building code of a country specifies the maximum permissible settlement for a given structure, differential settlement is a more crucial consideration. A purely

geotechnical investigation cannot generate data large enough to adequately characterise foundation soils. This is often the case where geology is complex. Detailed subsurface exploration is required to generate reliable subsurface model at proposed site for any building.

Olorunfemi *et al* (2005) employed electrical geophysical techniques to generate sub-surface model around a distressed building at a factory site in Asaba, Southern Nigeria. They established that the foundation soils were highly heterogeneous lithologically. The soils were found to vary from clay, through sandy clay to sand from one corner of the building to another. They concluded that their findings could have been responsible for differential settlement. A purely geotechnical approach cannot furnish such a detailed information.

Our recent integrated geophysical and geotechnical studies at a hypo-thetical proposed building site in the Teaching and research farm, University of Ibadan, were utilized to establish mathematical relationship between some geotechnical and geophysical parameters. Negative correlation was established between plasticity index (and hence compressibility) and electrical resistivity of soils. Daodu(2013). The negative correlation are also established between cohesion and layer resistivity is due to the fact that clayey soils are known to exhibit high cohesion and low resistivity (Fig. 23b). Bulk density was also shown to have positive correlation with resistivity. The above mathematical models can help in rapid, cheap and comprehensive subsoil investigation for building.

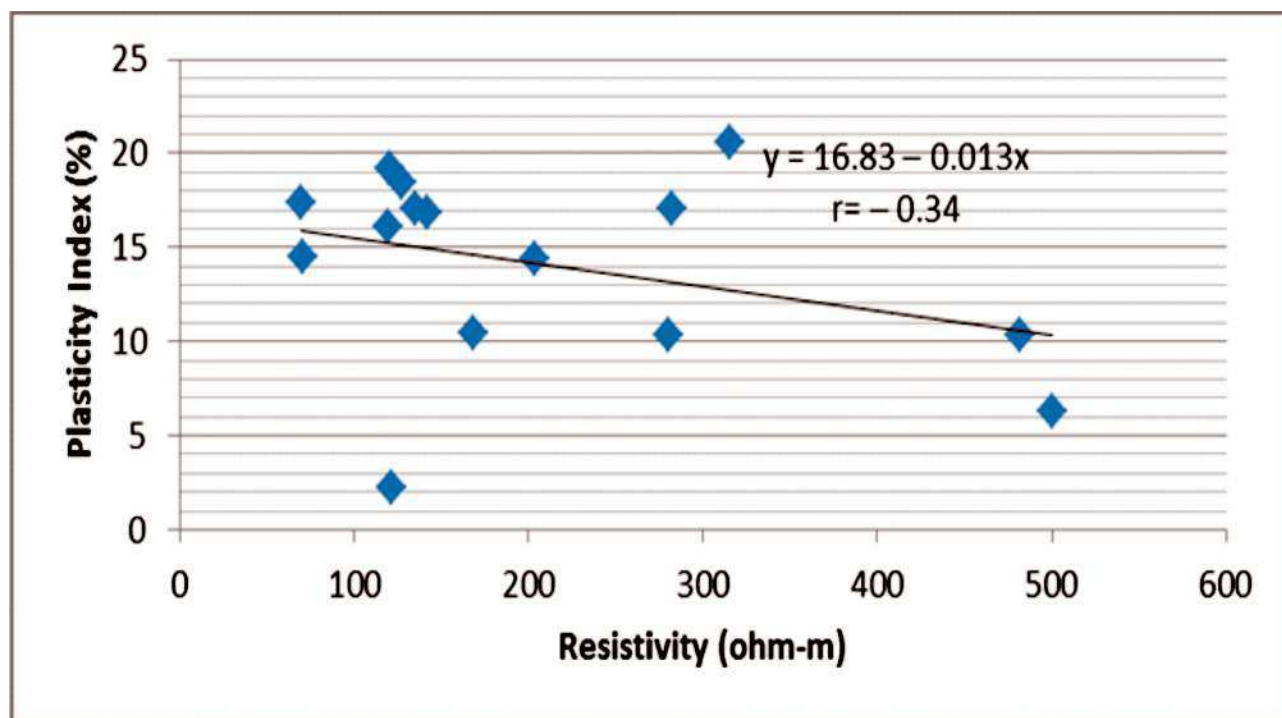


Fig. 1: Regression plot of Plasticity index against Resistivity of soils at the Teaching and Research Farm, University of Ibadan (After Daodu, 2013).

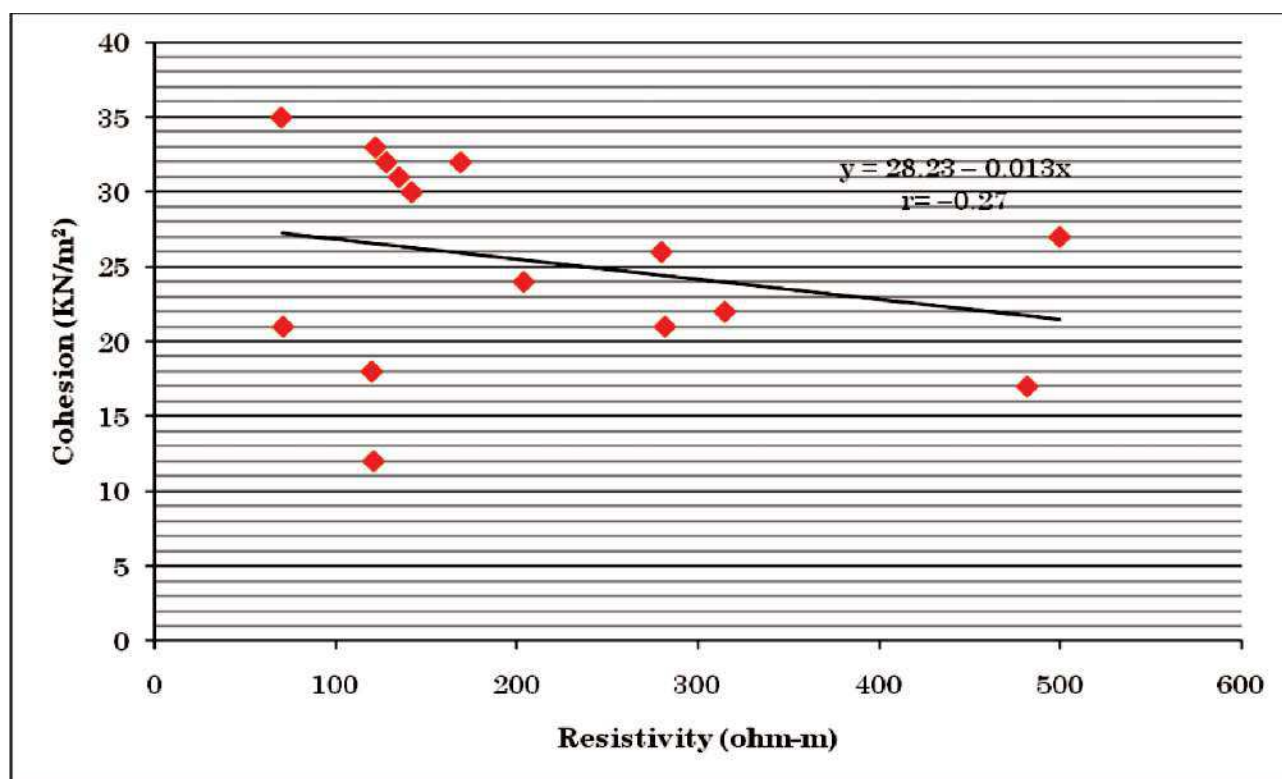


Fig. 2: Regression plot of Cohesion against Resistivity of soils at the Teaching and Research Farm, University of Ibadan (After Daodu, 2013).

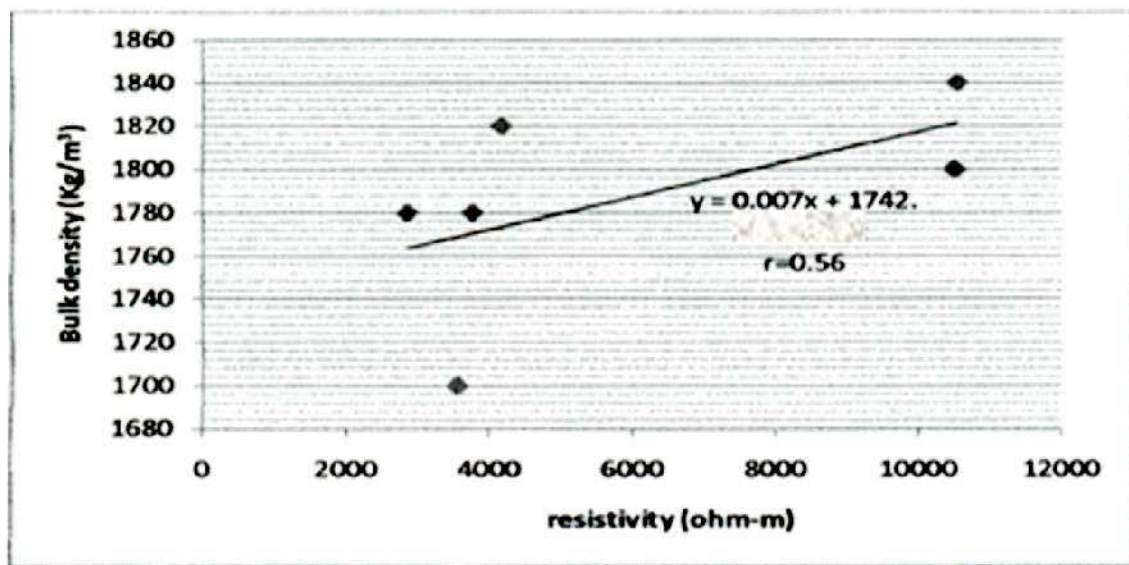


Fig. 3: Regression plot of Bulk density against Resistivity of soils at the Teaching and Research Farm, University of Ibadan (After Daodu, 2013).

Subsoil Investigation for Dams.

The routine geotechnical investigations are in-situ test such as dutch cone penetrometer test and sinking of boreholes. Although the results obtained from both the field and laboratory tests are invaluable in foundation design, the test points are often inadequate due to time and financial constraints. Utility of geophysical investigations alone may not be useful to foundation engineers who are not familiar with geophysical data. However geophysical surveys may generate preliminary data that may aid the probably reduce the quantum of in-situ geotechnical studies. Aina *et al*(1995) for instance employed both aeromagnetic and electrical resistivity techniques to determine the nature of overburden soil, depth to bedrock and presence or absence of fractures during pre-construction investigation for a dam on Katsina Ala river. They noticed no discontinuity and thus recommended for geotechnical investigation.

Ako (1976) Executed integrated geological and geophysical studies at the proposed Opa dam within the campus of the then University of Ife. He established strong correlation between the actual depth to bedrock and what was estimated from geophysical survey. The integrated geophysical and geotechnical investigation around the proposed dam site across river Ewawa near Okada Village Edo state (Akinlabi and Adeyemi. 2014)

was used to estimate strong correlation between the actual depth to lithological interface and the figure estimated from the geophysical survey. (Fig. 25a). They also established positive correlation between SPT blows/200mm and Electrical resistivity of soils. (Fig 24)

Roads and Highways

Comprehensive preconstruction investigations are often necessary for road and highways, including proposed bridge sites. Surface geophysical surveys such as electrical resistivity techniques can be rapidly used to investigate the sub-base soils and rocks, and ground-water. This can be followed confirmatory in-situ tests, boreholes and trial pits. Olayinka and Oyedele (2001) established strong positive correlations between actual depth to bedrock obtained from the boreholes and those estimated from the electrical resistivity survey (fig 71, page 102).

Construction Materials

Surface geophysical surveys such as refraction seismic and electrical resistivity have been used in many parts of the world to;

- i. Determined the thickness of overburden
- ii. The thickness of clays, sands and gravel.
- iii. The quality of the construction materials.

Table 1: Showing cost effectiveness of geophysical survey over geotechnical survey in borehole drilling

Exploratory Tool	Extent/Scope of Exploration	Cost Justification		Total Cost (₦)
Geophysical Survey	Comprehensive Survey involving Ves + VLF + resistivity imaging plus data analysis, computer interpretation and report writing	Lump Sum		100,000.00
Borehole Drilling	2 No Boreholes at a site	10m of overburden @ N10,000.00/m per borehole	100,000.00 per borehole	200,000.00
		3m of competent bedrock @ N30,000.00/m per borehole	90,000.00 per borehole	180,000.00
		TOTAL		380,000.00

(Source: various consultant geophysicists and borehole drilling companies).

Note: The prices quoted are valid as at January, 2010.

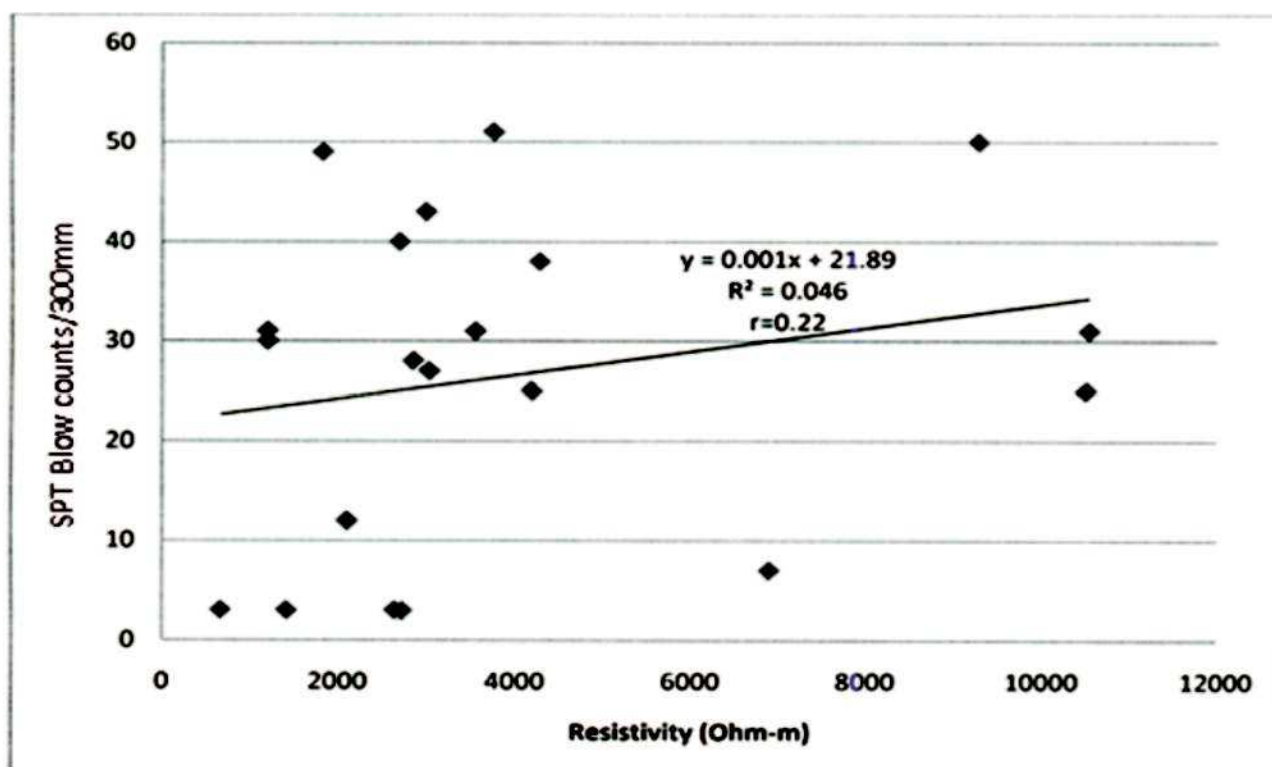


Fig. 1: Regression plot of Plasticity index against Resistivity of soils at the Teaching and Research Farm, University of Ibadan (After Daodu, 2013).

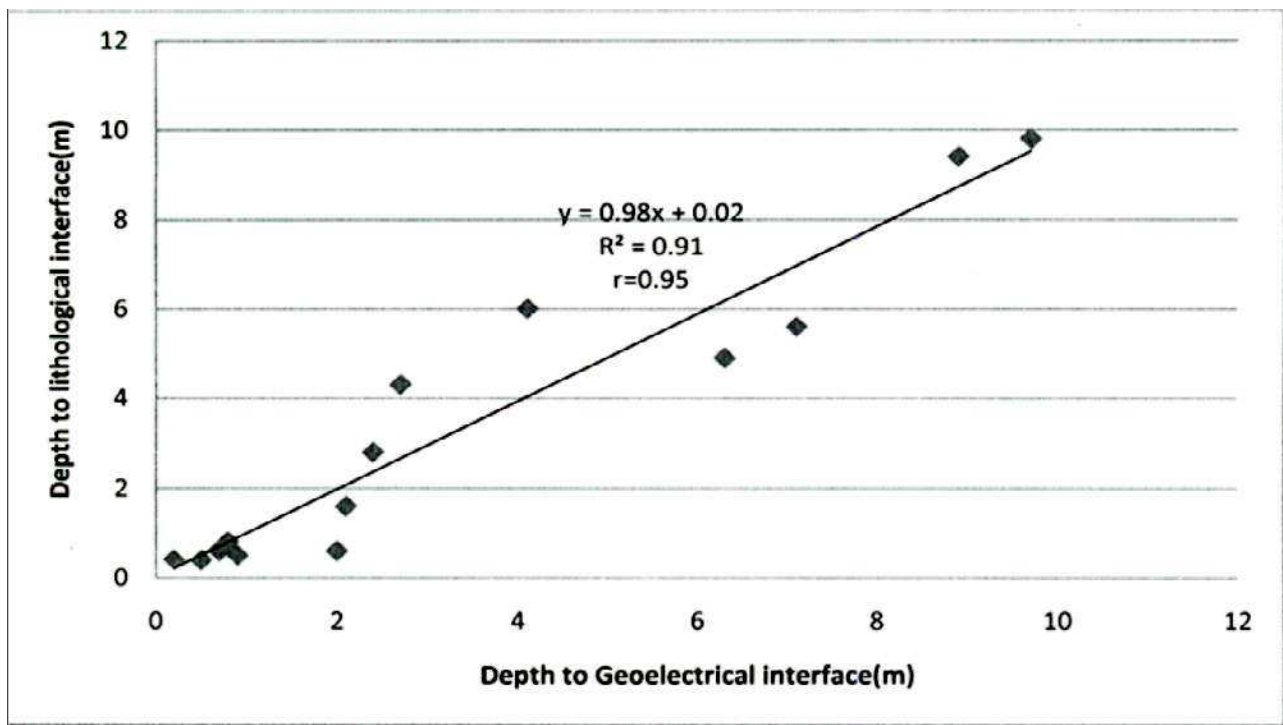


Fig. 4: Actual depth to lithological interface against depth to geoelectric interface around a proposed dam site across river Ewawa, near Okada Village (After Akinlabi, 2011).

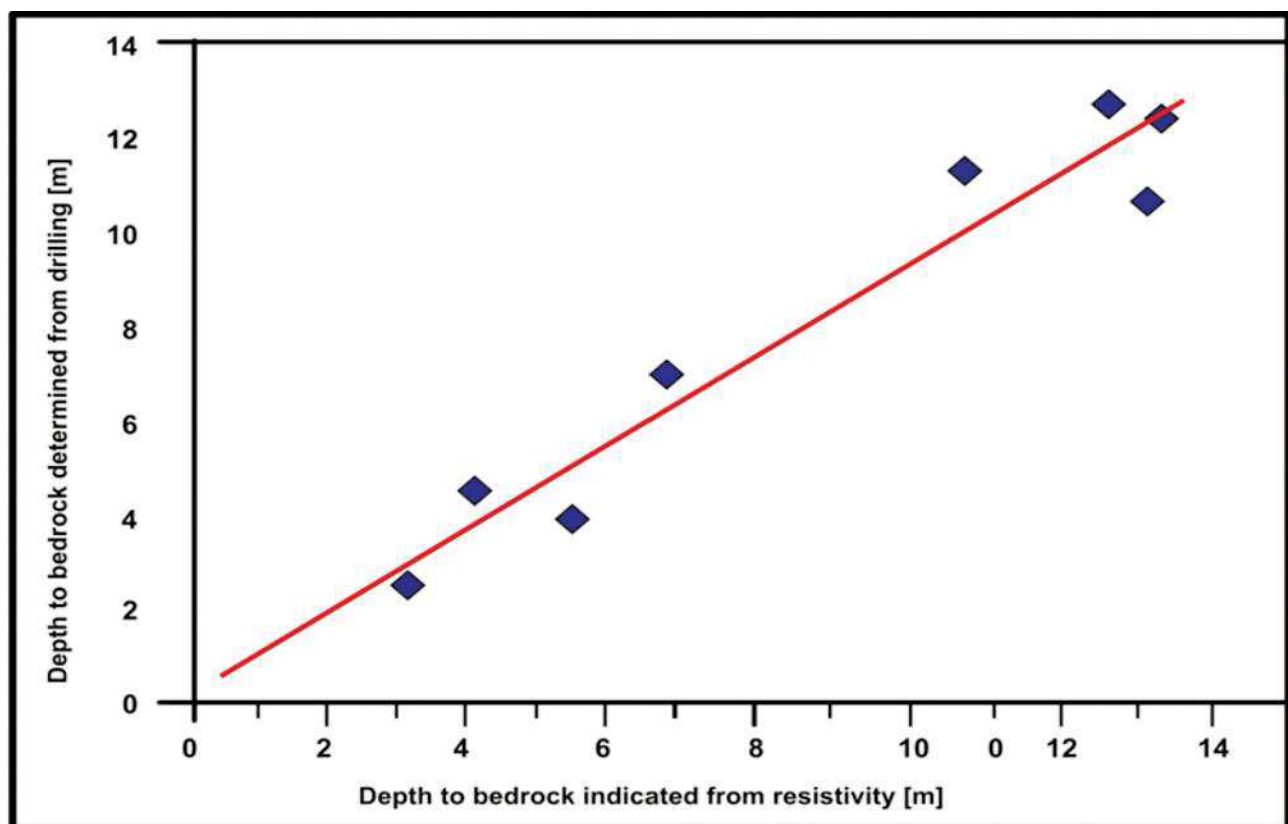


Fig. 5: Correlation between actual depth from boreholes and those estimated from resistivity survey (After Olayinka & Oyedele, 2001).

Seismic refraction method has been shown to give more reliable information than the electrical resistivity survey. Proposed quarry sites can also be rapidly and cheaply investigated through geophysical surveys. Few confirmatory boreholes, should however be drilled as control for data garnered from surface geophysical surveys.

Conclusions

The following conclusions can be drawn.

- i. Near Surface geophysics is of wide application in subsoil investigations for structures.
- ii. Integrated geophysical and geotechnical investigations have been found to furnish comprehensive information about the subsurface. This can prevent differential settlement.

- iii. Groundwater condition such as depth to water table can be rapidly determined from engineering geophysical investigation.
- iv. The thickness and nature of naturally occurring construction materials can be rapidly, cheaply and reliably determined from integrated geophysical and geotechnical investigations.
- v. Engineering geophysical investigation is not only fast but also cheap.

Recommendations

Integrated geological, geophysical and geotechnical approach has for many years been adopted in many developed countries. This has prevented distress and collapse of structures in such countries. It is high time we adopted this approach in Nigeria (the giant of Africa) to prevent incessant collapse of our structures.

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The Place of Engineering Geology and Geotechnics in the Design of Building Foundations in the Nigerian Niger Delta Sub-Region (A Lead paper presented at the 2017 NAEGE Conference)

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Corresponding E-mail: cliffordteme@yahoo.com**Abstract**

The Niger Delta sub-region of Nigeria is the arcuate structure situated at the southernmost section of the Nigerian coastline and lies between *Latitudes* 4° 15' 00" and 6° 30' 00" North of the Equator and between *Longitudes* 6° 37' 42" and 7° 30' 00" East of the Greenwich Meridian. It is the major petroleum province of the Country. Geographically and geologically, it is defined *sensu stricto*, as that area extending from a northern apex situated at Aboh, bound on the east by the Imo River, on the west by the Benue River and on the south by the Atlantic Ocean. The Niger delta basin serves as the receptor of the water and sediments generated upstream of the Rivers Niger and Benue catchments. These excess water and sediments generated during the annual floods are released through an anastomosing network of rivers, creeks, rivulets and in some cases canals into the Atlantic Ocean. The sediments upon reaching the mouths of the rivers are acted upon by both the waves and Long Shore drifts to form the various barrier beaches and islands such as Bonny, Brass, Akassa and Forcados in the present-day delta. Thus, *sensu lato*, the Niger Delta incorporates the Eastern Lowlands which comprise the present day Akwa Ibom, Cross River and Imo states and the Western Lowlands which comprise the present day central and western Edo, Ondo and even Lagos states. The Niger Delta had been subdivided into three hydro-meteorological zones namely: Freshwater, Transition (or Mangrove) and the Coastal zones. The Coastal or Lower Niger Delta zone is principally subjected to diurnal ebb and flow regimes with tidal ranges reaching up to or more than 10 meters in places, depending on the coastal slope angles while the Transition or Mangrove Zone is characterized by soft, fibrous clays (chicoco muds) and silty-clays underlain in places by poorly-graded sands and gravels at depths. The freshwater zone towards the north of the sub-region is characterized by c-φ soils inundated annually by 1-3.5m high floods for over two-to-three months of the year. Due to the characteristics of the sub-soils of the various zones, designs and constructions of building foundations are controlled largely by engineering geologic and geotechnical properties of the subsoils. This Lead Paper presents briefly the engineering geology of the various zones, as revealed by almost two decades of subsurface investigations in the Niger Delta sub-region, types of typical foundations found appropriate for buildings in the various sub-zones, some behaviour patterns of soil/structure interactions found in the different sub-zones as revealed by selected case histories from sections of the sub-zones of the sub-region.

Keywords: Zones of the Niger Delta; engineering geology; foundation systems; behaviour patterns; soil/structure interactions; case histories.

Introduction

Definitions: Some terms need to be defined in order to have a better understanding of the discussions that will follow in this paper.

Engineering Geology and Geotechnics: Engineering geology which is a course in Geology Department and Geotechnical Engineering which is a course in Civil engineering department are intricately related. Geotechnics is the combined efforts of Engineering Geology and Geotechnical Engineering. The existing relationship between the two important disciplines of Engineering geology and Geotechnical engineering is as shown in Figure 1 below by Coduto (2007).

Several authors have described Engineering geology. West, T.R. (1995) defined "Engineering geology as the application of the geological sciences to engineering practice for the purpose of assuring that the geologic factors affecting the location, design, construction, operation and maintenance of engineering works are recognized and adequately provided for". Attewell &

Farmer (1976) in tandem with the American Geological Institute has defined Engineering geology as "the application of the geological sciences to engineering practice for the purpose of assuring that the geological factors affecting the location, design, construction, operation and maintenance of engineering works are recognized and adequately provided for". W.R. Judd (1957) defined Engineering geology as "the application of education and experience in geology and other geosciences to solve geological problems posed by civil engineering structures". The engineering geologist thus supplies geologic input for design purposes and ensures that geologic factors are properly considered throughout all the construction phases and in some special cases, depending on the training programmes in postgraduate schools, take part in the design functions of the civil engineering projects.

The Niger Delta Sub-Region

The Niger Delta sub-region of Nigeria is the arcuate structure situated at the southernmost section of the Nigerian coastline and lies between *Latitudes* 4° 15' 00"

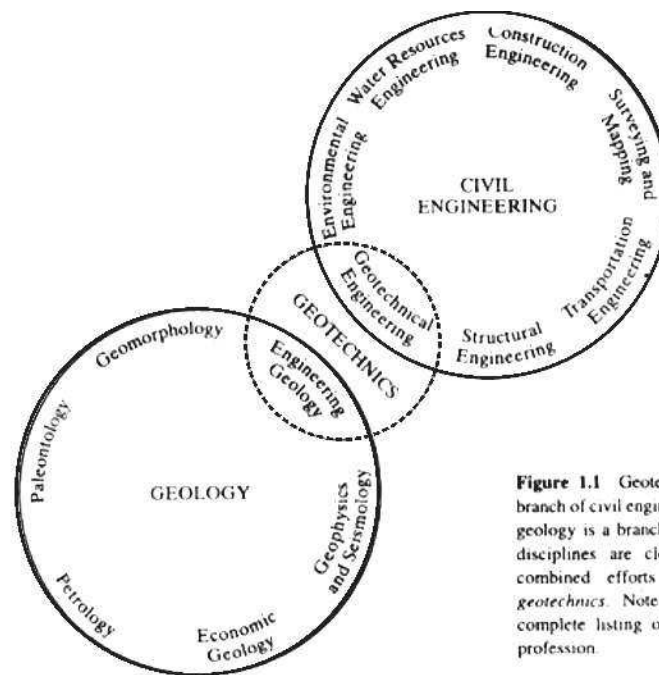


Figure 1.1 Geotechnical engineering is a branch of civil engineering, while engineering geology is a branch of geology. These two disciplines are closely related, and their combined efforts are sometimes called *geotechnics*. Note: This illustration is not a complete listing of the branches of either profession.

Figure 1: Relationship between Engineering geology and Geotechnical Engineering. (after Coduto, 2007)

and $6^{\circ} 30' 00''$ North of the Equator and between Longitudes $6^{\circ} 37' 42''$ and $7^{\circ} 30' 00''$ East of the Greenwich Meridian.

It is the major petroleum province of the country. Geographically and geologically, it is defined *sensu stricto*, as that area extending from a northern apex situated at Aboh, bound on the east by the Imo River; on

the west by the Benue River and on the south by the Atlantic Ocean. The Niger delta basin serves as the receptor of the water and sediments generated upstream of the Rivers Niger and Benue catchments. These excess water and sediments generated during the annual floods are released through an anastomosing network of rivers, creeks, rivulets and in some cases canals into the Atlantic Ocean. The sediments upon reaching the mouths of the rivers are acted upon by both the waves and Long Shore drifts to form the various barrier beaches and islands such as Bonny, Brass, Akassa and Forcados in the present day delta. Thus, *sensu lato*, the Niger Delta incorporates the Eastern Lowlands which comprise the present day Akwa Ibom, Cross River and Imo states and the Western Lowlands which comprise the present day central and western Edo, Ondo and even Lagos states.

Hydro-Meteorological Sub-Divisions of the Niger Delta Sub-region

Fubara, Teme et al (1988) sub-divided the Nigerian Niger Delta sub-region into three hydro-meteorological zones as:

- (i) Coastal zone,
- (ii) "Transition" or Mangrove zone, and
- (iii) Freshwater zone.

These three zones are briefly described below.

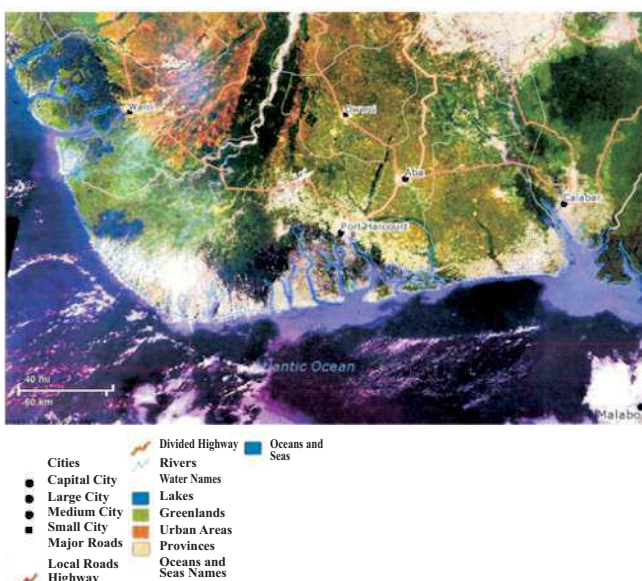


Fig. 2a: A satellite image of the Nigerian Niger Delta sub-region.

Coastal Zone

This zone consists mainly of sand bars and ridges and the water bodies here are saline in nature. The area is subjected to diurnal ebb and flow tides and thus not prone to the annual floods of the Freshwater zone. The sub-soils here consist of *sand, silts and highly plastic clays* in some inlet areas. The vegetation of this zone is basically mangrove trees with a preponderance of nippa palms (a recent development in this zone). Some freshwater trees such as palm trees are also found within the few elevated beach ridges within this zone.

The climate of the zone is basically that of tropical monsoon with rainfall occurring almost all through the year except the months of December, January and February which are not completely free from rainfall in some years. Rainfall within the zone reaches highs of over 4000mm at Akassa, Bonny and Forcados during the peak of the wet seasons (*Institute of Geosciences and Space technology Meteorology division records 1988-2000*). Towns within this zone are Bonny, Akassa, Brass, Bekinkiri, Koluama I (wiped out by wave erosion), Koluama II, Oyorokoto.

Morphological Characteristics of the Coastal Zone

Morphologically, this zone is characterized by intermittent Sand Bars, especially at the mouths of the several rivers that empty into the Atlantic Ocean. These vary in widths from as low as 1.50m to as high as over 20-50m, especially around Bonny, Forcados during the low tides.

"Transition" or Mangrove Zone

This area coincides with the Mangrove brackish water zone immediately north of the Coastal Zone. The vegetation within the 'Transition' Zone comprises basically mangrove trees especially within the saline swamps and along the banks of the numerous rivers, rivulets and creeks. Nippa palms are the present day invasive vegetation patterns predominant in this zone. The climate of the zone is basically that of tropical monsoon with rainfall occurring almost all through the year except the months of December, January and February which are not completely free from rainfall in some years. Rainfall within the zone reaches highs of over 600mm at Port Harcourt, Opobo and Warri during the peak of the wet seasons (*Institute of Geosciences and Space Technology Meteorology*

division records 1988-2000. Just like the Coastal Zone, this zone experiences *diurnal ebb-and-flow of the tides* with maximum values obtained during the once-a-year *Spring tides*.

Morphological Characteristics of the "Transition" Zone.

Morphologically, the Transition or Mangrove zone is characterized by its numerous anastomosing network of creeks, rivulets, rivers, inter-tidal flats and various species of the mangrove vegetation. The sub-soils here are typically *fibrous, pervious clayey mud that exhibits large values of compressibility and consolidation*. Usually beneath these fibrous layers are *silty sands which, most often, grade into poorly-graded sands and further downwards into well-graded sands and gravels*. *Lateritic clays also occur in certain 'old' residual deposits* on which are usually located densely populated towns such as Bakana, Buguma, Opobo and Abonnema in Rivers State and Sapele in Delta State.

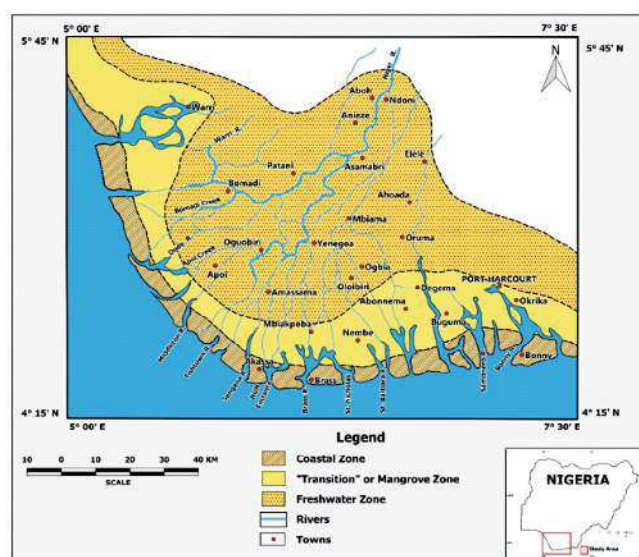


Fig. 2b: The Niger Delta sub-region showing the three hydro-meteorological zones. (after Fubara, Teme et al, 1988)

Freshwater Zone

This zone which is the Upper Niger Delta comprises the remaining northern portion of the Niger Delta sub-region. This covers the predominantly fresh water rivers, creeks and ephemeral depressions. The vegetation within the Freshwater zone comprises Palm trees, broad-leaved trees with very thick undergrowth characterized by creepers and climbing varieties. Tall

grasses are not uncommon within the zone. Climatically, rainfall occurs in over nine months of the year, especially during the annual flood periods of May through October and ranges in intensity between 200 and 600mm in towns such as Ahoada and Omoku in Rivers State (*Institute of Geosciences and Space technology Meteorology division records 1988-2000*).

Morphological Characteristics of the Freshwater Zone

The zone is characterized predominantly by soil profiles comprise of a top lateritic clay layer usually underlain by silty clays and silty sands which are further underlain by poorly-graded sands and sands and gravels. Notable towns in this zone include Agbere- Odoni, Isampou, Asamabiri, Yenegoa and Amasoma all in Bayelsa State, Patani in Delta State and Ndoni in Rivers State. Annual floods that sometimes exceed 3.0 – 5.0m above the normal ground levels are common occurrences in this zone. During the intervening dry seasons the water levels in the large rivers are so low that the river banks become very steep that steps are usually constructed to access potable water from these river beds, while the smaller rivers usually dry up completely.

Brief Geology of the Niger Delta Sub-Region

The geology of the area comprises the Quarternary to Recent Sediments of the Sandy Beaches, Mangrove swamps and the Niger Flood Plains that overlies and form parts of the Benin Formation. Beneath the Benin Formation is the Agbada Formation which forms the reservoirs for the crude oil in the Niger Delta. Beneath

the Agbada Formation is the Akata Formation where the crude oil is formed.

Engineering Geology of the Various Zones

The Coastal Zone consists predominantly of cohesion-less clastics (Sands, gravels), cohesive clays and the intermediate ($c - \Phi$) materials. The “Transition” Zone comprises mostly the very Plastic and cohesive Muds such as the “Chicoco” swelling muds with very low friction angles between $<4^\circ - 6^\circ$. Usually below these very soft compressible layers are soils ranging from poorly – well graded sands at depths of between 10.00 – 15.00m. In exceptional places, the cohesive clays may extend to depths of between 35.00m to 42.00m. These materials have low load- bearing capacities which in most cases require deep foundations such as Pile and Piers to support infrastructural loads. The Freshwater zone has both cohesion-less soils especially along the river beds and banks. In between the river systems are usually ($c - \Phi$) soils, predominantly Laterites and various forms of Lateritic soils (Gidigas, 1976). The upper sections of this Zone are usually characterized by the presence of acidic clay-soils typical of which is Laterites.

Sub-Surface Engineering Characteristics

The subsurface engineering characteristics of each of the three hydro-meteorological zones discussed in this paper will be unveiled during the discussions of the Foundation types in each of the zones during soil investigations.

Coastal Zone

Since this zone consists predominantly of cohesion-less clastics (Sands, gravels), cohesive clays and the intermediate ($C-\Phi$) materials usual engineering characteristics of these materials cut across a whole range of properties, from cohesion-less ($c \sim 0.00$ kPa) through ($c \geq 55$ kPa for clays) field investigations should be thorough since there may be many pockets of hidden soft materials. Foundations here have to be determined by the type, weight of structures and the thickness of clay horizons. Note that usual “Rules of Thumbs” may not apply but detailed soil investigations.

“Transition” or Mangrove Zone

The “Transition” Zone comprises mostly the very Plastic and cohesive Muds such as the “Chicoco”

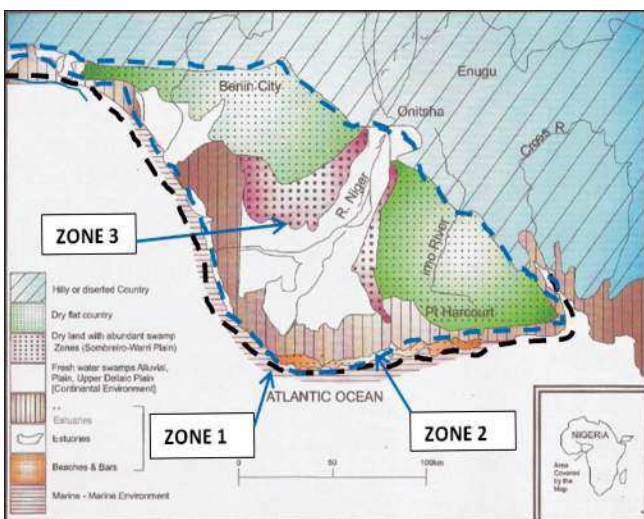


Fig. 2(c): Approximate Location of the study site in the morphological setting of the Niger Delta.

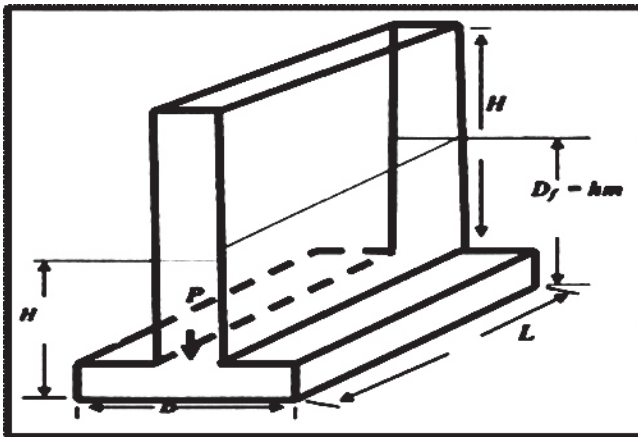


Fig. 4(A): Sideview of the Continuous Strip Footing.

W = width of the base of the footing
 L = Length of the foundation footing
 D_f = Depth of the foundation below ground surface
 P = Net Pressure on Footing
 BL = Foundation Footing Area
 $P/(BL)$ = Stress on soil

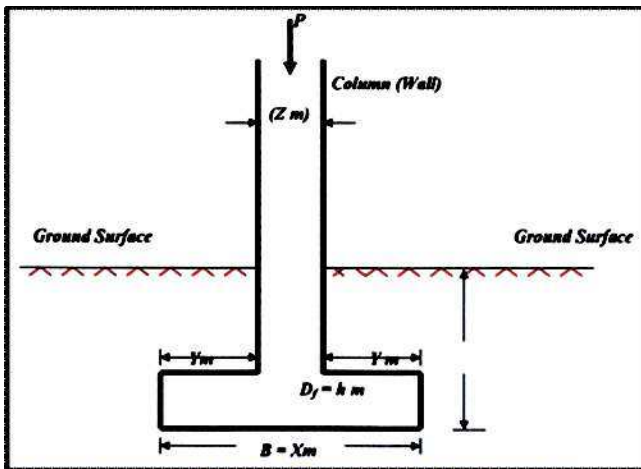


Fig. 4(B): Cross-section of the Continuous Strip Footing

Calculation Of Ultimate Bearing Capacities For The Continuous Strip Footing Foundations At the Site

Assuming $D_f = 1.75 \text{ m}$

$B = X \text{ m}$ (as shall be directed by the client as advised by the site engineer)

$$.q'_d = cN_c + D_f \quad \dots \quad \dots \quad \dots \quad \dots (x)$$

where:

$$.q'_d = \text{ultimate bearing capacity (kPa)}$$

$$C = \text{Cohesion intercept} = 0.00 \text{ kPa}$$

$$N_c = \text{Cohesion Terzaghi factor (from table)} = 32.00$$

$$\gamma = \text{Unit weight of soil at site} = 2.265 \text{ kN/m}^3$$

$$D_f = \text{depth of foundation} = 1.75 \text{ m}$$

From equation (x) above we have,

$$.q'_d = (0.00) [32.00] + (2.265) [1.75] = 39.6 \text{ kPa}$$

Factor of Safety (F.S) = 3.00

Therefore,

$$\begin{aligned} \text{Allowable Bearing pressure } q_{(allow)} &= .q'_d / 3.00 \\ &= 13.20 \text{ kPa} \end{aligned}$$

Raft Foundations [Floating or Bouyant Footings]

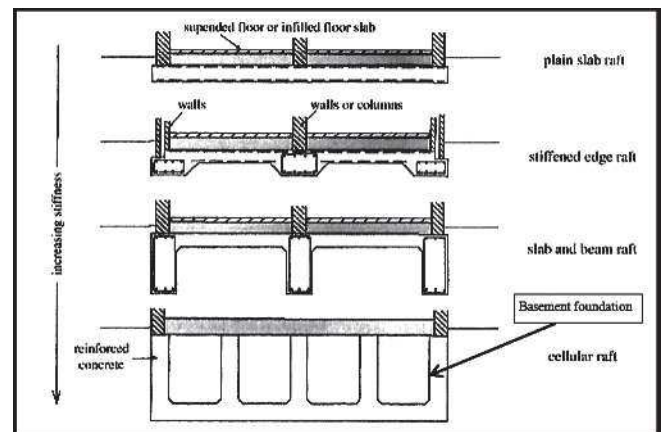


Fig. 5(A-D): Raft Foundations. (after Barnes 2000)

Pad Foundations

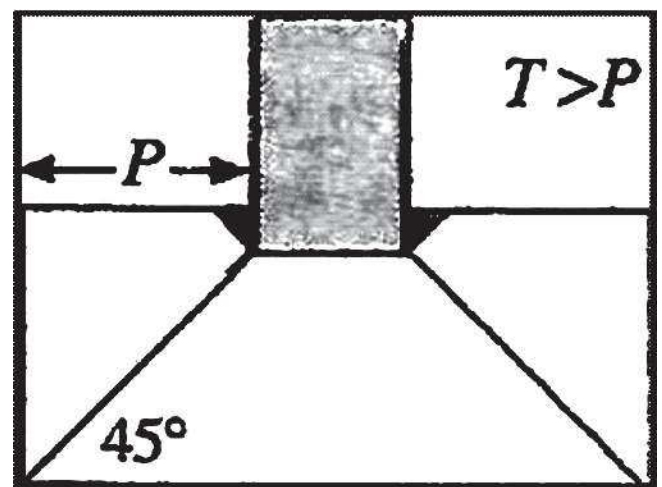


Fig. 6(A): Unreinforced Pad Foundation (after Barnes 2000)

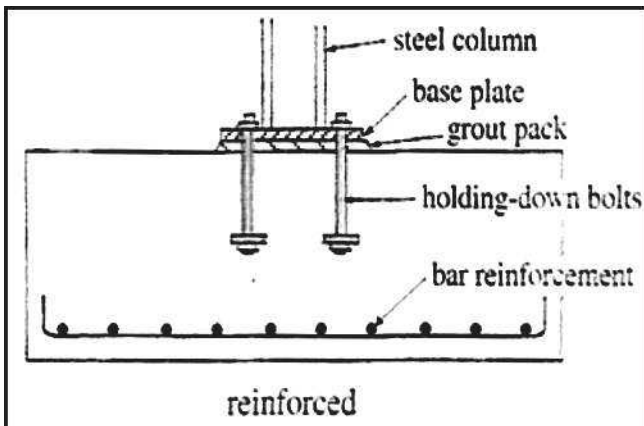


Fig. 6(B): Reinforced Pad Foundation (after Barnes 2000)

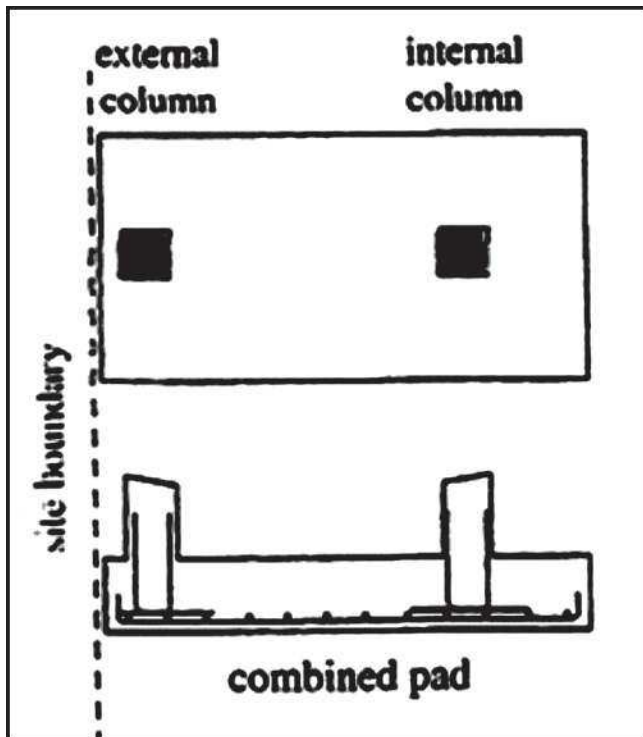


Fig. 6(C): Combined Pad Foundations (after Barnes 2000)

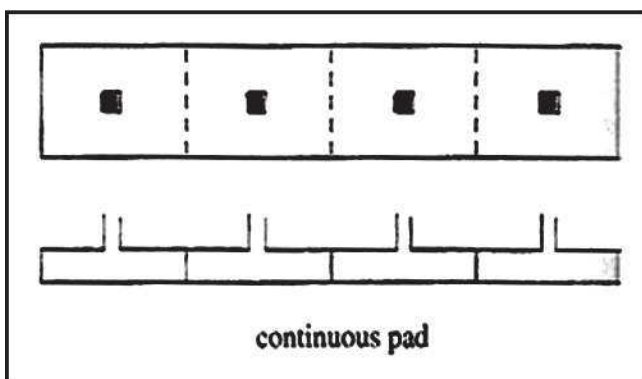


Fig. 6(D): Continuous Pad Foundation (after Barnes 2000)

Deep Foundations

Piles and Types

Driven Piles: These consist of

- (i) Timber [round square sections];
- (ii) Precast concrete [solid or hollow sections];
- (iii) Pre-stressed concrete [solid or hollow sections];
- (iv) Steel H-section, box and tubes.

Driven and Cast-In-Place piles: These consist of

- (i) Withdrawable steel drive tube, end closed by detachable point,
- (ii) Steel shells driven by withdrawable mandrel or drive tube,
- (iii) Precast concrete shells driven by withdrawable mandrel.

Note: Both **Driven Pile** and **Driven and Cast-in-Place piles** are known as “**Displacement Piles**”.

Bored Piles: These consist of

- (i) Augered Piles,
- (ii) Cable Percussion drilling,
- (iii) Large-diameter under-reamed,
- (iv) Types incorporating precast units,
- (v) Micro- or Mini-Piles

Note: Bored Piles are also known as “**Replacement Piles**”

Reinforced Concrete (Raymond) Piles

Pile Properties

Material = Reinforced Concrete

$$K_s \text{ value} = 1.0 \{ \text{for low relative density} \}$$

$$2.0 \{ \text{for high relative density} \} B = 0.68m$$

Angle of Wall friction of Pile (δ)

$$= 2/3\phi$$

[where: ϕ = friction angle of the sub-soil]

$$L = 26 - 27m$$

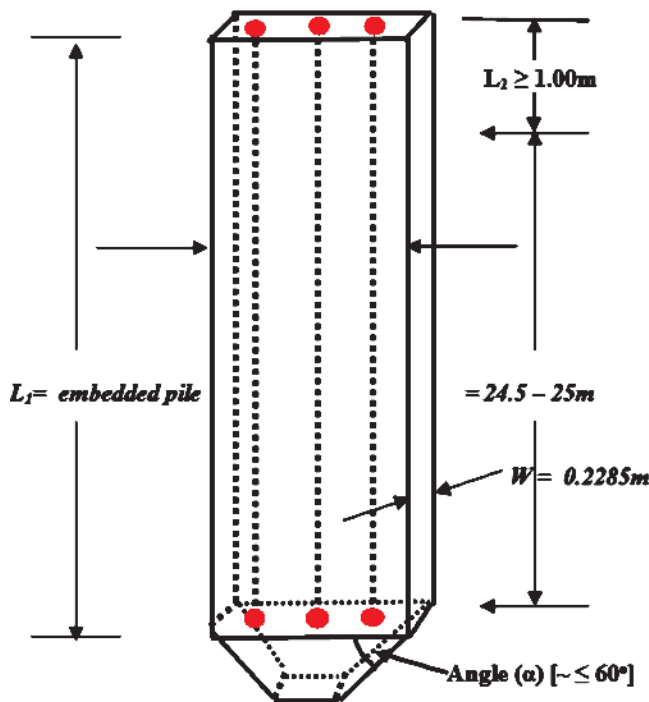


Fig. 7(A): Schematic Diagram of an Option of a Proposed Reinforced Concrete Pile to be used at the X Company Ltd. Project Site (Teme, 2017)

Reinforced Hollow Cylindrical Piles

Pile Properties

Material = Reinforced Concrete

$K_s \text{ value} = 1.0 \text{ \{for low relative density\}}$
 $2.0 \text{ \{for high relative density\}}$

Angle of Wall friction of Pile (δ)
 $= 2/3\phi$

[where: ϕ = friction angle of the sub-soil]
 $L = 25 - 30m$

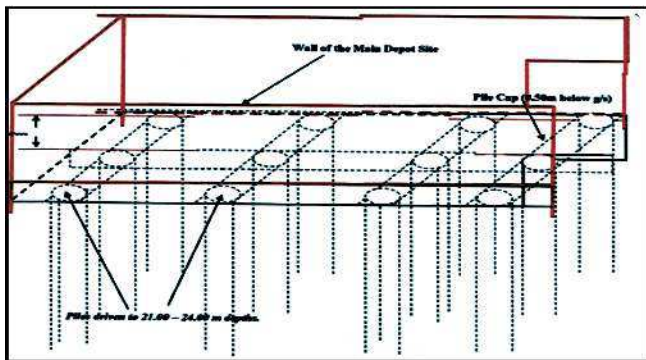


Fig. 8: Raft-Pile-Foundation type (Teme 2015.)

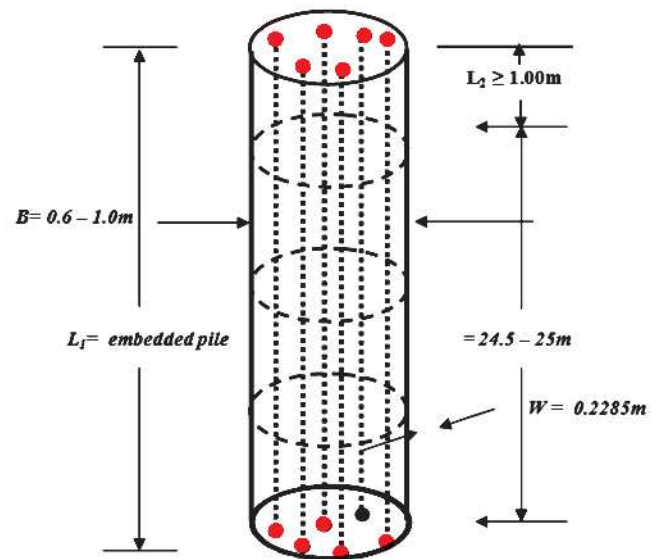


Fig. 7(B): Schematic Diagram of another Option of a Proposed Reinforced Concrete Pile to be used at the X Company Ltd. Project Site (Teme, 2017).

Behaviour Patterns of Soil/Structure Interactions in the Various Zones of the Niger Delta Sub-Region

When structures such as building foundations are emplaced in the ground, there is usually a reaction between the in-situ materials and the structures that are emplaced. If the emplaced materials are lighter than the maximum or ultimate loads the in-situ materials can carry then there will be no problems but when these structures are much heavier than maximum strength of these in-situ materials then there will be settlements occurring. These settlements are either uniform or differential in nature depending on the mode of the loading of the structures.

Shallow Foundations

Building foundations are said to be shallow when the Depth of emplacement (D_f) is less than the Width (B) of the building, i.e. $D_f < B$, in normal parlance, when the depths of the foundations are constructed within the reach of normal excavation plant, i.e. $D_f < \sim 3m$ and $D_f/D \sim 3m$. when the excavated grounds are left open, they may be treated as either "Basement" foundations or "Bouyant" foundations.

Settlements

Settlements are vertical movements of buildings on the ground produced by the applied stress from the building. These movements can be in any of the following forms:

- (i) Immediate settlements ρ_i ,
- (ii) Consolidation settlements ρ_c , and
- (iii) Secondary compression ρ_s

Thus, the Total Settlement $\rho_T = \rho_i + \rho_c + \rho_s$

Consolidation settlements can be determined by the use or application of the Undrained Modulus correlation of Jamiolkowski *et al*, (1979) shown in the graph below.

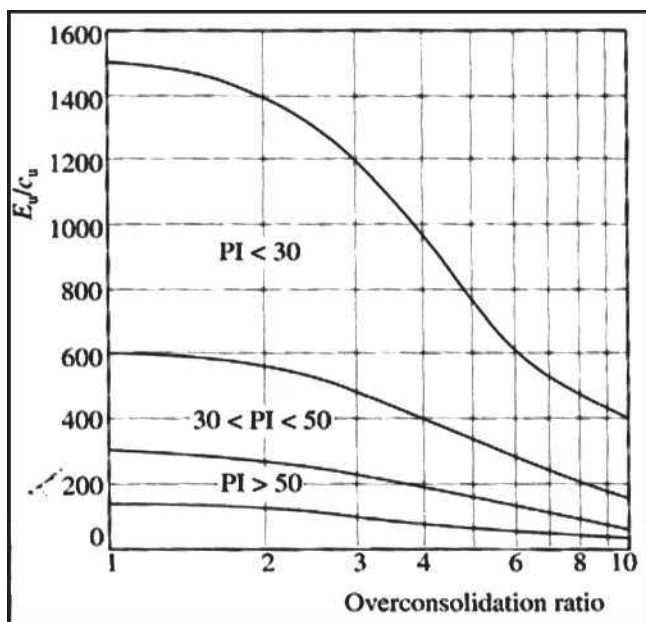


Fig. 9: Jamiolkowski *et al* 1979 method of Compression index C_c for computation of consolidation Settlement in lightly over-consolidated clays

Uniform Settlements

These occur when settlements are the same throughout the building plan.

Non-Uniform Settlements

These occur when the settlements are not uniform throughout the building plan but different loads are applied from one section to another.

Deep Foundations

Deep foundations are those whose depths are much greater than the widths of the structures. In the Niger Delta region, due to the soft soils that often occur to great depths in places, Deep foundations are preferable as bearing media for building structures in excess of two to three floors and more or less for vibratory machine foundations.

CPTs with Calculations [eg. PH Gardens Eliozu PHC.]

In investigating for deep foundations, usually two common methods are used and these are the Standard Penetration Test (SPT) N-value method and the Dutch Cone Penetration Test (CPT) method. Both methods are to determine the safe bearing capacities and adequate depths to which the piles are to be emplaced in order to sustain a safe bearing capacity. One way of doing this is to use the CPT to determine the bearing capacities of the subsurface s shown in the figure below.

Variations of Pile Loads Against Pile Diameters Versus Depths of Pile Emplacement

Behavior patterns of Soil-structure interactions in the various zones of the Niger Delta sub-region can be shown and demonstrated by the behaviour of driven pile loads against various Pile diameters versus depths of Pile emplacements. Engineering geology practices over two-and half decades in the Nigerian Niger Delta has shown that both the various diameters of piles used, the various pile loads applied during installation, and the various depths of Pile emplacements have inter-dependent relationships with the soil types encountered in the various sub-zones of the Niger Delta. A few recent examples from the “Transition” and “Transition/Freshwater boundaries” examples [case of parts of Port Harcourt] will be used to show these relationships in this Lead paper.

Select Case Histories

Just a few select cases will be used to demonstrate the influence of Engineering geology and geotechnics as both inputs and in real designs of Building Foundations in the various zones of the Niger Delta sub-region over the last two and half decades of personal involvement in the practice of this field.

Mangrove Zone

Pre-design Site Investigation for an Oil and Gas Facility Terminal in the Marginal Landsof the Nigerian Niger Delta Sub-region.

Dateline: 2015 - 2016

Purpose: A pre-design site investigation for Oil and Gas

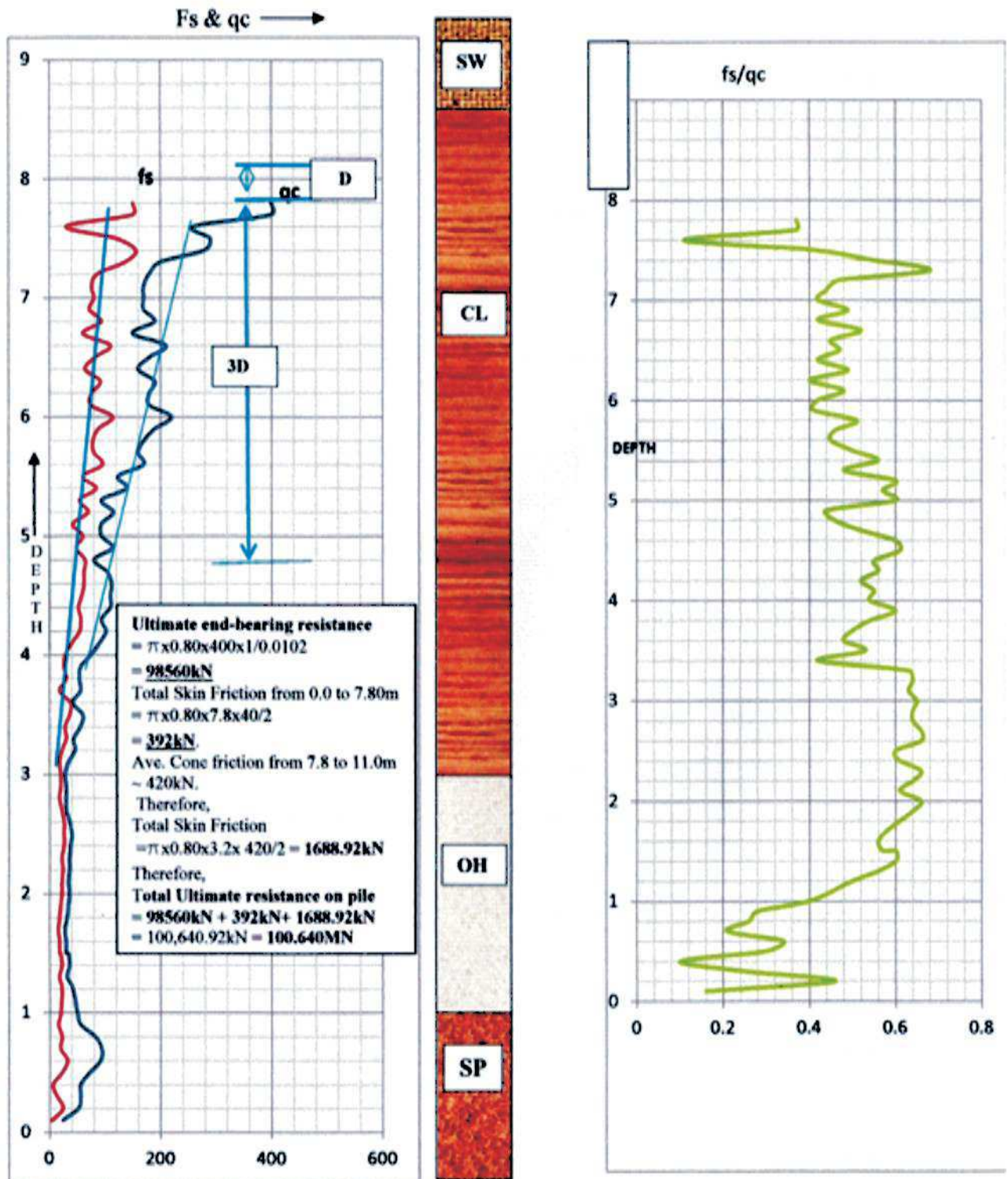


Fig. 10: Use of CPT with calculations to determine the depths of building foundations embedment in the Freshwater zone of the Niger Delta (Teme 2016)

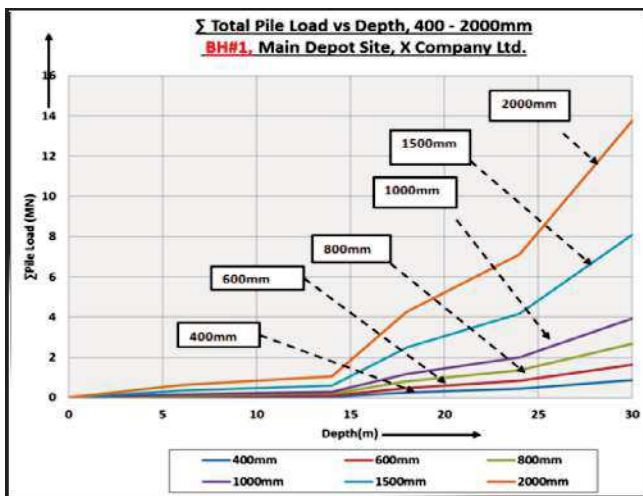


Fig. 11(A): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta] – (Teme 2017)

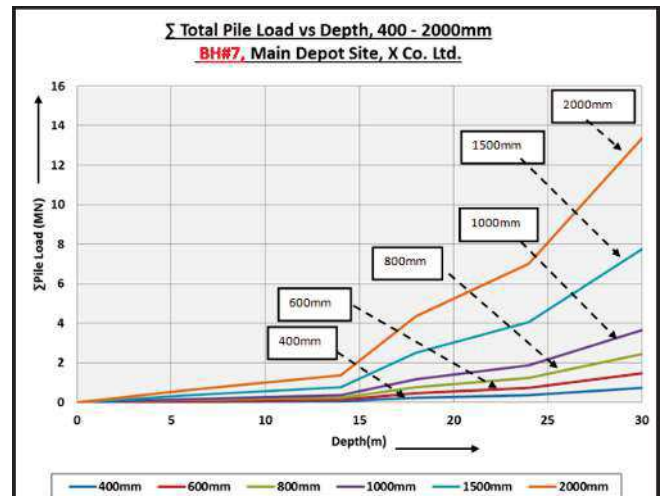


Fig. 11(D): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta]

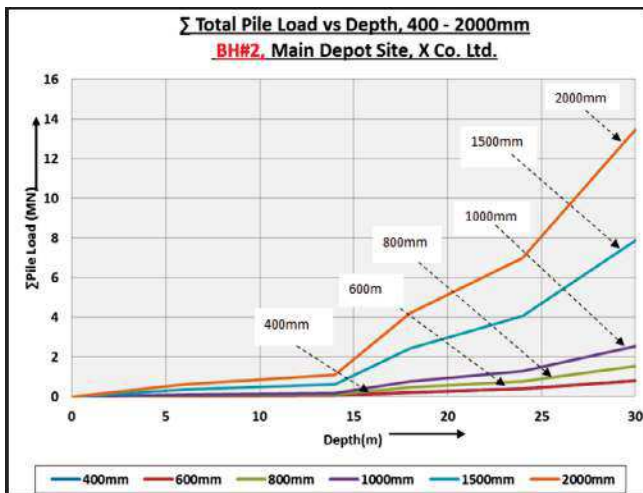


Fig. 11(B): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta] - (Teme 2017)

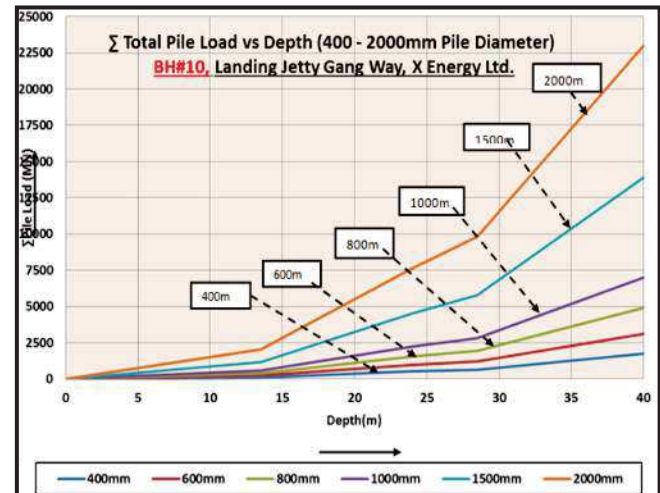


Fig. 11(E): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta]

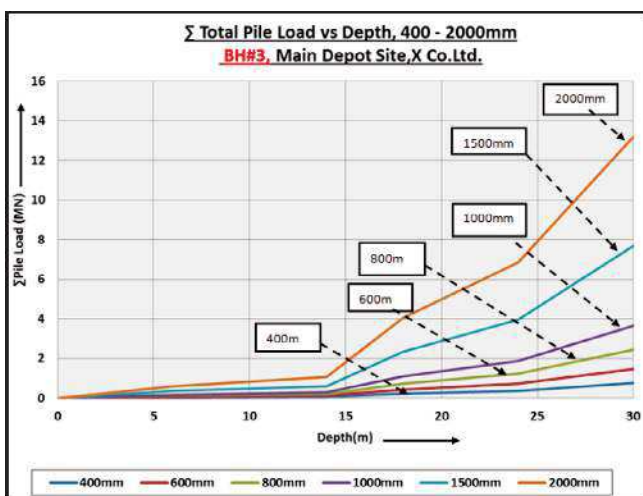


Fig. 11(C): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta]

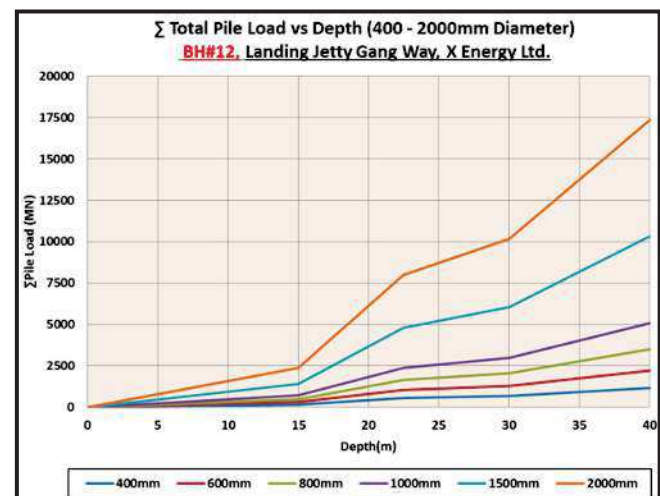


Fig. 11(F): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta]

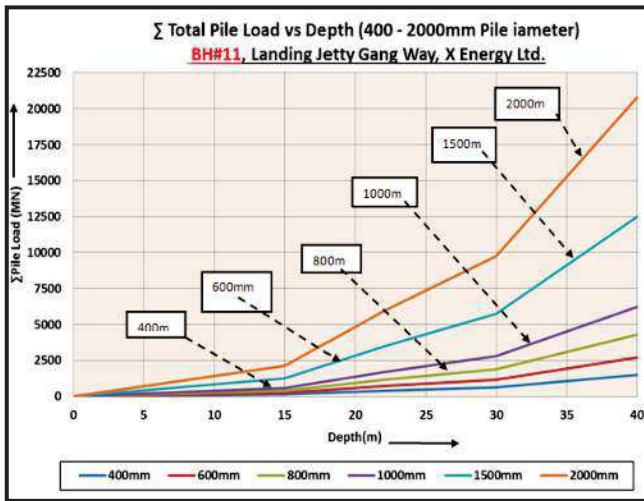


Fig. 11(G): Pile-Soil Interactions [Pile Load Vs Pile Diameters Vs Depth of Emplacement [in the “Transition Zone of the Niger Delta]

Storage Tanks to be located at a project site between Latitudes N04° 47'32.2" to N04° 47'49.5" and Longitudes E006° 55' 35.9" to E006° 56' 35.9" within the Marginal Lands of the Niger Delta sub-region has been carried out for an Oil & Gas Company in Nigeria for purpose of providing a Quay Wall for anchoring marine vessels, acquiring data for Foundations for Tank Farms, an Oil Refinery and a future Bitumen Plant.



Fig. 6(1): Satellite Imagery showing Location of Quay Wall Line at project site.

Results of Investigation: The Quay Wall at this site is almost 80% completed and it is being constructed with reinforced concrete tubular steel piles which are embedded at depths of between 30.00 to 40.00 m with computed average Total Pile Loads $[(Q_b + Q_s)] = q_{ult}$ varying from 323.25MN to 1044.50 MN, [where: Q_b = total Pile base load and Q_s = Pile Skin friction].

Foundation Systems for the Quay Walls

On the basis of the subsurface soil types encountered at the project site and bearing in mind the proposed project for the sites, it was recommended that only Deep Foundations in the form of Quays Walls be used for the proposed shore line protection. In employing deep foundations, *Pre-cast Reinforced Concrete Piles* were recommended and used due to their durability in saline water environments and relative ease of installation with appropriate installation methods. Based on the detailed field investigations carried out at the project site, the design of the *Pre-cast Reinforced Concrete Pile Foundation* took the following into considerations.

- (i) Depth of embedment of the reinforced concrete piles (D_f).

From boring and sounding records obtained from the *Seven (7)* On-Shore borings along the Quay Wall line, the depths of pile embedment were determined to be at about 25.0 meters below ground level.

Soil Bearing Pressures. Based on soundings from field investigations and laboratory tests results on soil samples, the safe bearing loads obtained, based on the methods of Meyerhof (1953) and Bowles (1998), range from 62.90 to 210.50 MN for 25.00m long piles and between 90.60 to 338.65MN for 30.00m long piles. These values were found to be comparable values obtained in similar soil conditions elsewhere within the Niger Delta sub-region and adjoining flanks (*Teme, 1989, 1991a & b, 1992a, 1992b; 1993a & b, 1994a, & b; 2002; 2004, 2007, 2008, 2012, 2014a & b*).

Ultimate Carrying Capacities of the reinforced Concrete Piles at the project site.

Based on the *Meyerhof (1953) Method*, the Bearing Capacity of a reinforced concrete pile in a soil possessing both cohesion and friction and using Berezantsev et al. (1961) equations were taken as:

$$P_u = Q_s + Q_b = A_s \{c_a + K_s \gamma D/2 \tan \delta\} + A_b \{c N_c + K_s \gamma D N_q + \gamma B/2 N_\gamma\} \quad \dots \quad \dots \quad \dots \quad \dots \quad \dots (1)$$

where:

$$\text{Pile Base Load } (Q_b) \text{ is given as: } A_b \{c N_c + K_s \gamma D N_q + \gamma B/2 N_\gamma\} \dots \quad \dots \quad \dots \quad \dots \quad \dots \quad \dots (1a)$$

Pile Shaft Load (Q_s) is given as: $A_s \{c_u + K_s \gamma D / 2 \tan \delta\}$
 (1b)

K_s = the coefficient of earth pressure on the shaft within the failure zone (varying from 0.50 for loose soils to 1.00 for dense soils)

N_c, N_γ = Terzaghi bearing capacity factors while N_q were those of Berezantsev et al. (1961) and the embedment Ratio (L/B)

where:

W = Width of Pile base;

B = Breadth of Pile

L = Length of reinforced concrete pile = 25.00m

A_s = Area of shaft of pile = $\{(2B + 2W) \times L\} = 45.425 \text{ m}^2$

A_b = x-sectional Area of base of pile = $\{(B \times W)\} = 0.1554 \text{ m}^2$

c_a = the adhesion per unit area of the pile

δ = the angle of friction of the soil on the shaft of the pile [$\sim 2/3$ of soil friction angle (Φ)];

Note that for clays: $\delta = 0^\circ$,

γ = unit weight of soil at foundation level.

Base Resistance Of Piles

Net unit base resistance $q_{ult} = q_f = p = p_d (N_q - 1) = A_b \{cN_c + K_s \gamma L N_q + \gamma B / 2 N_\gamma\}$ (3)

where:

P_d = effective overburden pressure at pile base level (4)

The proposed Pile depth (L_p) for fore-shore was given 35-40m deep.

Shaft Resistance {Unit Skin Friction} of Piles

The total Ultimate Skin Friction on pile shaft was taken as:

$f = K_s \cdot p_d \cdot \tan \delta = A_s \{c_a + K_s \gamma L / 2 \tan \delta\}$ (5)

The various Pile Shaft Loads (Q_s) estimated for the Quay Wall Shore Protection studied in the course of this investigation are given in the Pile design Table 2 below.
Oil & Gas Terminal Facility in the Marginal Lands of the Nigerian Niger Delta Sub-region

Dateline: 2017 – Date.

Purpose: An Oil and Gas Storage Tanks facility with provision for a 40-m long Jetty Gangway to the Crude Loading point. Location is not too far away from the first case, both within the “Transition Zone of the Niger Delta sub-region



Fig. 6(2): Case B: 1 Site of Project

Results of Investigation: Preliminary detailed Sub-surface Investigations and Report have been produced to the clients with many novel soil-structure interactions observed and accordingly reported. **NOTE:** the Relationships observed in Figures 11 a-g above are some few outcomes from this project site which is in the early stages of design with inputs from this detailed study. The design engineers and the Geotechnical investigation teams are collaborating very closely due to the intricacies of some of the results of the investigation

Conclusions and Recommendations

From the above few examples it is very obvious that the Engineering geologist (properly trained) can be very useful in the design team for building and other infrastructural structures in a complex environment such as the Niger Delta sub-region.

**THANK YOU FOR LISTENING
AND I WISH YOU ALL SUCCESS IN THE PRACTICE OF
ENGINEERING GEOLOGY & GEOTECHNICS!!!**

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Safe Disposal of Solid Waste in Nigeria (A Lead paper presented at the 2017 NAEGE Conference)

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Abstract

Huge volume of waste is generated on daily basis in most part of Nigeria. However, proper disposal is seldom done, a situation that often leads to pollution of groundwater in the vicinity of dumpsites. Comprehensive, geophysical and geotechnical investigation can guarantee siting of a landfill that will ensure that groundwater is not polluted. The paper also discusses possible generation of wealth from waste.

Keywords: Waste, Pollution, Geophysical, Geotechnica, Landfill.

Introduction

At its Council meeting in Vancouver Canada, in 1997, the Association's name, "International Association of Engineering Geologists", was changed to "International Association for Engineering Geology and the Environment" in view of the importance of our environment. Rapid population growth and industrialization have resulted in generation of huge volume of waste especially in major cities in Nigeria. Waste is also associated with exploration and exploitation of oil and gas, and mining of solid minerals.

Municipal waste is often dumped indiscriminately without any consideration for the environment. In Ibadan, for instance, the space between carriage ways of the Lagos-Ibadan highway has been turned to illegal waste dumps, comprising faeces and other domestic waste. Despite the fact that the Oyo state Government has agency that clears the waste, the effect is hardly felt. Heaps of waste often give out repulsive odour and may block free movement of water. Heavy flood often takes some waste to rivers, a situation that can enhance flooding when the river channel can no longer cope with its contents.

Impact on Groundwater

Infiltration of leachates from waste often results in pollution of groundwater. Studies of the chemical composition of water from hand dug wells in the downslope section of illegal waste dumps have confirmed improvement in the quality of groundwater with distance from highways. The concentrations of nitrate, (an index of faecal pollution) were found to decrease with distance from waste dumps.

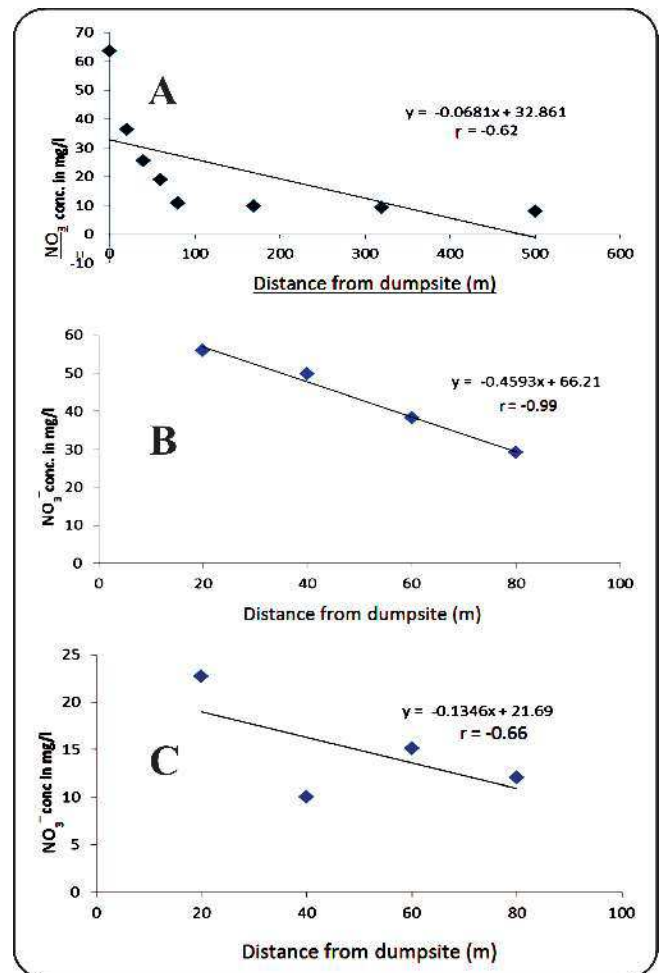


Fig. 1 (a), (b) and (c): Regression plots showing nitrate concentrations against distances from downslope sections (axes 1, 2 and 3 respectively) from Lapite dumpsite (After Akintola, 2014).

Integrated geophysical, geotechnical and hydrogeological investigation around Lapite landfill,

the largest in Oyo state also revealed improvement in the quality of groundwater in the downslope section with distance from the landfill. (Figs. 1a, 1b & 1c). The concentrations of Nitrate and Ca^{2+} were used to map pollution plume around the active landfill. (Oguntayo, 2013).

Statistical treatment of the concentrations of some cations and anions in water samples from 20 hand dug wells in Ogbomoso revealed that anthropogenic factor had stronger influence than geogenic factor on the composition of groundwater. (Bamigbala & Oginni, 2010). Coliform counts, an indication of faecal pollution were also found to decrease with both the distance from waste dump and depth of well.

Disposal of Solid Waste

There are many means of disposing solid waste such as

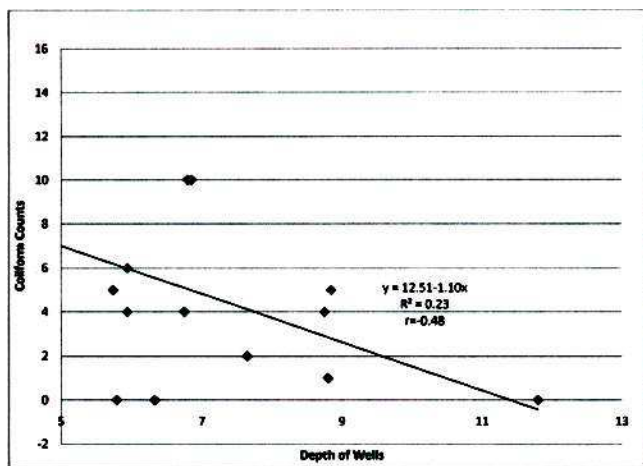


Fig. 2(a): Plot of Coliform Counts in water samples against Depth of Wells in parts of Ogbomoso, Southwestern Nigeria (After Bamigbala & Oginni, 2010).

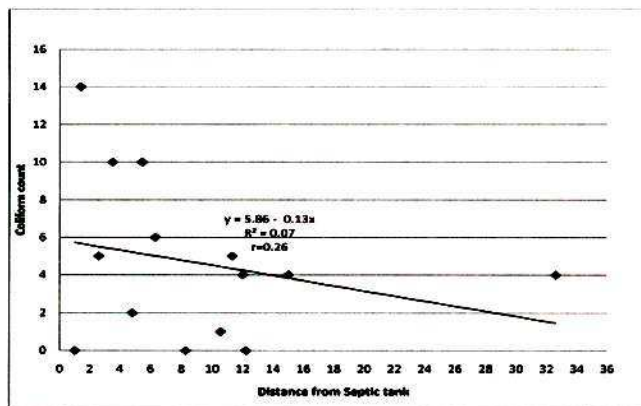


Fig. 2b: Plot of Coliform Counts in water samples against Distance of Wells from septic tanks in part of Ogbomoso, Southwestern Nigeria (After Bamigbala & Oginni, 2010).

use of incinerator, injection holes and landfills. The smoke that emanates from an incinerator during burning can pollute the environment. Injection holes are abandoned oil wells. They are often utilised in advanced Countries for disposal of solid waste associated with exploration and exploitation of hydrocarbons. Their use necessitates a comprehensive knowledge of the hydrogeology of the area. The commonest means of disposing solid waste worldwide is landfills. The 5 main steps involved in the evaluation of any location for its suitability or otherwise for a landfill are:

- (i) Introduction
- (ii) Reconnaissance surveys
- (iii) Field investigations
- (iv) Laboratory analyses of soil samples
- (v) Discussions of results, conclusions and recommendations.

Introduction: This involves consultation of existing geological and geotechnical reports. Topographical and geological maps can also be studied while air photographs and radar imageries can also be examined. This is also the planning stage.

Reconnaissance Survey: This is preliminary visit to the proposed site. Such a site must be extensive with flat topography. It must be remote from major highways, rivers and habitable areas.

Field Investigation: Topographical survey should be followed by comprehensive geological mapping (basic and structural). The next stage is surface geophysical surveys which can be integrated. Pitting or drilling helps in detailed study of the subsurface. Disturbed and undisturbed soil samples are to be collected while depth to water table should be noted.

Laboratory Analyses of rocks, soil and water samples will furnish detailed information. Mineralogical and geochemical studies of selected soil samples have to be executed. Common geotechnical tests on soil samples include particle size distribution analyses, consistency limits and permeability tests. Oedometer consolidation tests can also be carried out.

Discussions of Results: Integration of data garnered in the field with those from the laboratory can help in:

- (i) Generating subsurface models.
- (ii) Comparing results with standard specifications for sanitary landfills.

The model generated can then be compared with the standard one. The standard model stipulates impervious (or low permeability, less than 10^{-9} m/s) silty clay for landfill seal, (Clayton & Huie, 1973), there are also standards for amount of fines (7 to 10%) and plasticity index (Daniel, 1993b). The soil must also possess good attenuation property while the depth to water table must be appreciable to the base of the landfill. This will ensure natural filtration of leachates before they infiltrate to the water table.

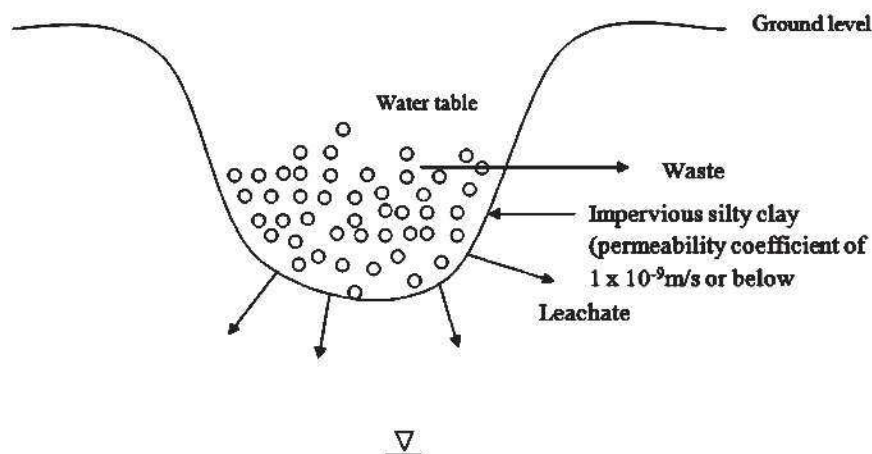


Fig. 3: Model of a sanitary landfill.

Recommendations

In-situ compaction will reduce both the permeability and compressibility of the landfill seal. Location where good seal can be won may be recommended. Lowering of groundwater level using appropriate state-of-the-art technology can also be recommended.

A good sanitary landfill should prevent pollution of groundwater.

Waste to wealth

- (i) Waste can be treated and converted to organic fertiliser. The Lagos State Waste Management Authority (LAWMA) has been producing and exporting bags of such fertiliser for quite some time.
- (ii) Metallic scraps, exposed or buried can be exploited and recycled. Surface geophysical surveys can reveal the depth of occurrence of such scraps. Some advanced countries of the

world have been "mining" metallic scraps which they recycle. Such scraps have been found to be richer in metals than many naturally occurring ores.

Conclusions

Waste is dumped in most cities and towns in Nigeria with little or no regard for the environment. This practice results in pollution of groundwater, which majority depend upon. Comprehensive Engineering Geological investigations must be executed at any proposed landfill site.

Nigeria should start waste to wealth practice, which Lagos state has pioneered. Geoscientific studies are necessary not only during preconstruction of a landfill but also in assessing environmental impact of waste.

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Exploring Ground Conditions for Sustainable Building Foundation Design

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Abstract

Building collapse has become routine news in Nigeria. Complex buildings are erected without understanding whether or not the soil over which the building is to stand has the necessary attributes to effectively support it. Incidences of building collapse are usually followed by news frenzy, calling for enquiries and prosecution of contractors. The emphasis has never been on due diligence on the assessment of ground conditions for sustainable building foundation design. The goals of the paper therefore are to make a case for soil investigation for all civil engineering projects and to provide clarity on the processes on soil investigation, in order to improve accuracy and reliability of results. The paper overviews the basic methodologies and their challenges, introduces some developments in the applications of the CPT and ultimately hopes to provide some insight into the future of Soil Investigation in Nigeria.

Keywords: *Exploring, Ground Conditions, Building Foundations, Building Collapse, Soil Investigation.*

Introduction

One of the greatest causes of building collapse is foundation failure. One of the greatest causes of foundation failure is insufficient knowledge of ground conditions. This is why it is imperative to explore ground conditions for sustainable building foundation. Building collapse cases are widespread in Nigeria, particularly Lagos state, which recorded 139 cases between 1978 and 2013 (Table 1). These are so widespread that nearly every local government area registered a building collapse case (Fig. 1). In the most general and simplified sense, the major problem during construction is the "Ground" behaving differently than anticipated. Predictions based on inaccurate or insufficient information can easily result in a partial to total difference between expected and encountered conditions. SI should provide the basis for anticipating in a reliable and specific way the behavior of the ground.

The correct Interpretation and Application of Test Results is a frequently neglected topic. It is however only briefly covered in civil engineering and engineering geology universities curricula. The danger of using results directly without interpretation can be of overwhelming consequence. One of the most important decision facing design engineers is the decision on choice of values of soil parameters for design. By conducting a geotechnical investigation and properly interpreting the results, you will be able to design a solid building plan, anticipate any possible problems and avoid failure and any additional expenses. If you ignore SI, you will still pay the price for it and perhaps more!

Purpose and Importance of Geotechnical Investigation

Geotechnical investigation is an integral part of the construction process. It is a procedure that determines the stratigraphy and relevant physical properties of the soil underlying the site. This is done to ensure that the super-structure (ie, the building), which the soil supports is safe and enduring. All civil engineering projects, (big or small), need precise geotechnical investigation to be conducted. It involves sampling and testing the soil strata to establish its characteristics and influence on the construction project. Because soil characteristics can vary, people may be misled into thinking that if the neighbouring areas conducted a geotechnical investigation and finished their buildings, you don't have to go through the same process because of the vicinity. Finally, soil investigation provides you with a certain level of predictability and stability when it comes to possible construction expenses. By conducting soil investigation, you'll be able to anticipate any possible problems and plan accordingly. Otherwise, you might invest so much in your building project only to realize that the soil is inappropriate for it.

Methods of soil investigation

The process of Soil Investigation entails different steps and methods and may vary from project to project. The Steps in soil investigation include:

- Field Reconnaissance and Inspection – This involves examination of the site, adjacent buildings and their foundations and possibly nearby areas.

Table 1: Trends in building collapse cases in Lagos State, Nigeria

Class Interval	No of Cases within Interval	Annual Average (%)	Aggregate Percentage	Highest	Lowest
1978-1986	17	2.45	12.23	7	1
1987-1992	19	3.8	13.67	7	2
1993-1998	19	3.8	13.67	8	1
1999-2003	36	7.2	25.89	9	5
2004-2008	30	6	21.59	14	1
2009-2013	18	3.6	12.95	5	2
	139		100		

**Fig. 1:** Distribution of building collapse incidences in Lagos, Nigeria

- Test pits/ Trenches– subsoil inspection, collecting samples,
- Boring – Different types may be used but the

common intent is to provide deeper insight into the underlying soil.

- Probing or Soundings- This covers use of Cone Penetrometers (CPT) and Geophysics (Seismic Refraction and Reflection, Resistivity, Ground Penetrating Radar (GPR)). These techniques also provide deeper insight into the underlying strata, soil properties, and ground waters, but may not be as much the drilling programme.
- Environmental projects may require [environmental approaches to drilling operations](#)

Some of the routine equipments and site conditions are captured in Fig. (2).

**Fig. 2:** Some equipment used for soil investigation and site conditions for work

Guidelines for Subsurface Investigation

Although the extent of soil exploration varies with the nature of the project, general standards apply to all investigation programs: It requires that Preliminary exploration depths should be estimated from data obtained during field reconnaissance, existing data, and local experience. The borings should penetrate unsuitable founding materials (organic soils, soft clays, loose sands, etc.) and terminate in competent material. All borings shall be extended below the estimated depths of influence /induced pressure. Each boring, sounding, and test pit should be given a unique identification number, location coordinates, elevation for easy reference. Offshore borings should be referenced to Mean Sea Level with the aid of a tide gauge. A sufficient number of samples, suitable for the types of testing intended, should be obtained within each layer of material. Water table observation should be carried out within each boring or test pit. Other groundwater observations (artesian pressure, etc.) should also be recorded. Unless serving as an observation well, each borehole, sounding, and test pit should be backfilled or grouted

Minimum requirements for Structures

The purpose of borings for structures is to provide sufficient information about the subsurface materials to permit design of the foundations and related geotechnical construction. All borings in this case shall include Standard Penetration Testing (SPT) at regular intervals. Extend borings sufficiently below the foundation tip elevations (to determine both bearing capacity and the long term settlement behavior of the foundation) and until all unsuitable foundation materials have been penetrated and the predicted stress from the foundation loading is less than 10% of the original overburden pressure or until at least 6m of bedrock or N-values of 50 or greater) is encountered.

Determining the depth of probe based on the foundation loading demands understanding of the Vertical Stress Distribution which is given in a broad sense by equation (1) and graphically by (Fig.3).

$$\Delta P = q_0 \left\{ 1 - \frac{1}{1 + \left(\frac{B^2}{(2z)^2} \right)^{3/2}} \right\}$$

where, q_0 = load per leg of jack-up
 B = diameter of spudcan
 Z = depth of embedment(1)

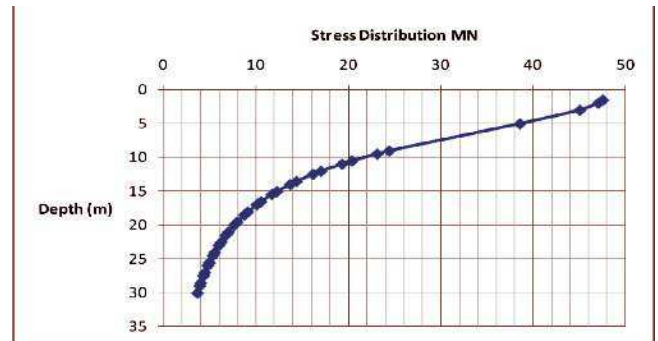


Fig. 3: Graphical representation of Vertical Stress Distribution

In practice however, the increase of vertical stress $\Delta\sigma$ at depth (Z) was based on the assumption that the stress from the foundation load (q_0) spreads out along lines with a 2 vertical to 1 horizontal slope governed by the equation:

$$\Delta\sigma = \frac{q_0 \times B \times L}{(B+Z)(L+Z)} \dots \dots \dots (2)$$

Where B and L are breadth and length of foundation.

Considerations in the design of any foundation:

There are three important considerations in the design of any foundation, namely: (i) Stability analysis-which aims at removing the possibility of failure of foundation by tilting, overturning, uprooting and sliding due to load intensity imposed on soil by foundation being in excess of the ultimate capacity of the soil. (ii) The next most important consideration is Total and Different Settlement and (iii) The stability of foundations should be checked for the following aspects: bearing capacity, uplift resistance, side thrust, over-turning and sliding.

Relating induced stress and bearing capacity with depth

The vertical profile of the bearing capacity at factors of safety $FS= 1.5, 2.5$ and 3.0 are plotted against the profile for vertical stress increment corresponding to 100kN/m^2 , 150kN/m^2 and 250kN/m^2 (Fig.4). The bearing capacity profiles reveal a significant degree of desiccation of the top soil up to 1.4m depth, effectively defining the zone of rapid temperature changes. Further analysis of the interaction of the profiles suggest that whereas a bungalow can safely be founded close to the surface, a one-storey building exerting surface load of 150kN/m^2 needs a least 1.4m depth of foundation to be safe from shear failure of foundation.

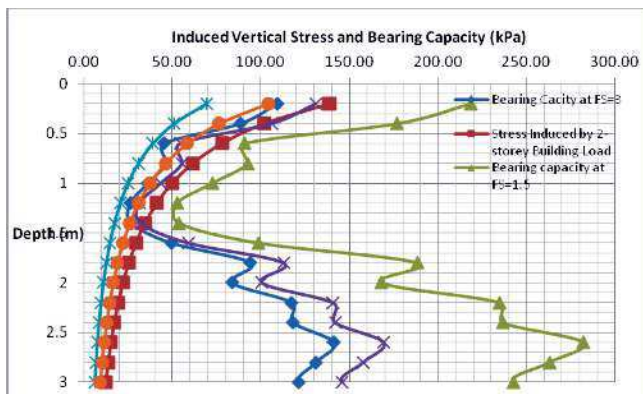


Fig. 4: Relating induced stress and bearing capacity with depth

In-situ Testing

Standard Penetration Test (SPT)

SPT is probably the most widely used field test. The reasons are: It is simple, has a wide variety of correlations for its data, and can obtain a sample during test. However, recorded SPT(N-value) may not be accurate for many reasons as shown in Table (2)

Factors affecting SPT (N) values

x Inadequate cleaning of borehole	⊠ N, sludge trapped in sampler
x casing driven bottom of borehole	↑ N in sand
x damage tip of sampling spoons	↓ N in clay
x loose joints on connecting rods	↑ N
x not using guide rod	↑ N
x water level in borehole below ground water level	↑ N, eccentric blows
	↓ N esp sand at bottom of borehole, piping effect

The SPT (N-values) obtained in the field may need to be corrected for overburden pressure and dilatancy (Terzaghi and Peck, 1969). In general, the N-values greater than 15 are corrected for dilatancy and overburden pressure as follows:

$$N^1 = 15 + 0.5(N_o - 15) \dots \text{for dilatancy effect} \dots (3)$$

$$N = 4 * (N^1) / (2.25 + 0.015 P_o^1) \dots \text{for overburden pressure} \dots (4)$$

Where P_o^1 = effective overburden pressure.

Effectively both corrections can be applied simultaneously by combining the relationships, which yield the expression:

$$N = 2(15 + N_o) / (0.015 P_o^1 + 2.25) \dots (5)$$

Comparison of Boring and Cone Penetration Sounding (CPT)

Boring obtains life soil samples, which is then tested to obtain results. Boring can change the physical characteristics of the sample itself and is a more expensive operation. CPT gaining rapid acceptance among the engineering community because it addresses the disadvantages and more. It is more efficient in terms of time, energy and resources. CPT gives instantaneous results and can generate by far (up to 7 soil parameters in native conditions) more information on soil conditions, but penetration can be a limitation. Geotechnical Boring lacks the accuracy and efficiency that CPT offers Cone Penetrometer Test (CPT).

The CPT is a **quasi-static** penetration test in which a cylindrical rod with a conical point (Cone) is advanced through the soil at a constant rate and the resistance to penetration is measured. Several types in use, (mechanical mantle) cone, electric cone, electric friction-cone, piezocone, seismic cone). CPTs measure the resistance to penetration at the tip, or the end-bearing component of resistance. All cones (Fig.5) have 60° tip angle and a 10 cm² projected end area. Penetration rates between 0.4 to 0.8 in/sec. Tests performed in accordance with ASTM D 3441



Fig. 5: Some locally fabricated Single and double Cones

CPT data is usually plotted (6) showing the end-bearing resistance, the friction resistance and the friction ratio vs. depth. Pore pressures, if measured, can also be plotted with depth. The friction ratio plot can be analyzed to determine soil type. Many correlations of the cone test results to other soil parameters have been made, and design methods are available for spread footings and piles. The piezocone can also be used to measure the dissipation rate of the excessive pore water pressure. The Plot of pore pressure versus log-time can be used directly to calculate the pore water pressure dissipation rate or rate of settlement of the soil.

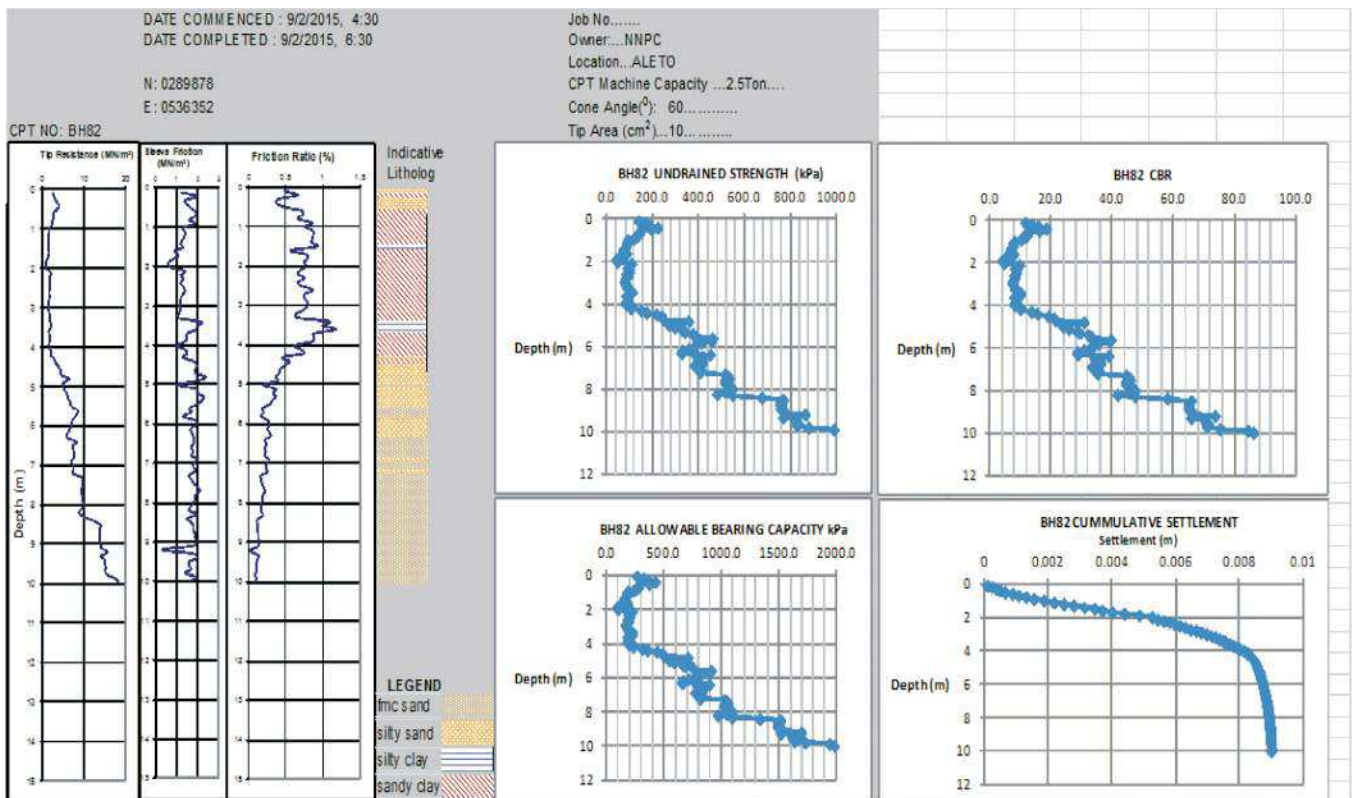


Fig. 6(a): Tip and sleeve friction profiling and lithology prediction using the locally fabricated cone penetrometer

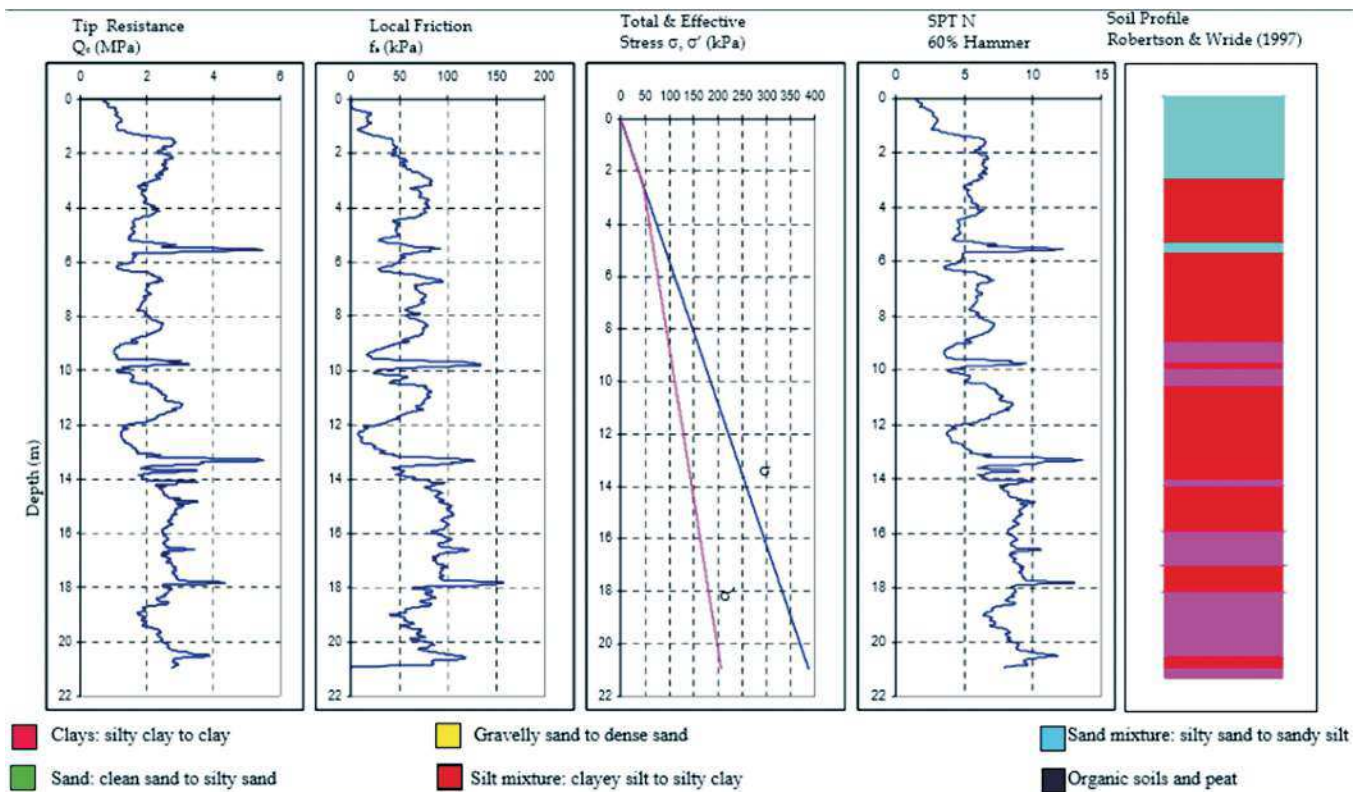


Fig. 6(a): Tip and sleeve friction profiling and lithology prediction using the locally fabricated cone penetrometer

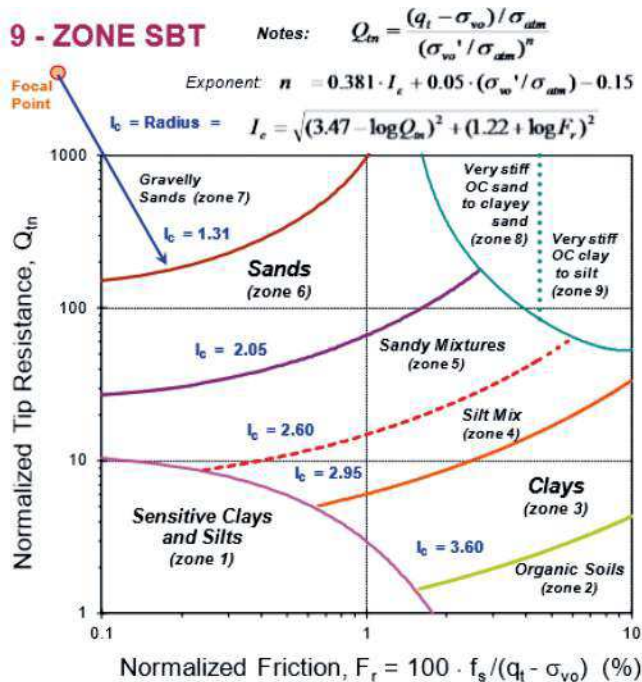


Fig. 7: Sol behavior type classification (After Robertson 2009)

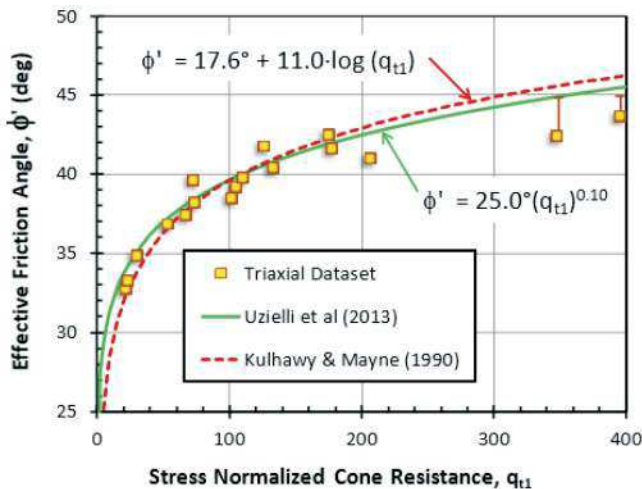


Fig. 8: Direct relationship between peak friction angle of quartz silica sand and normalized cone resistance

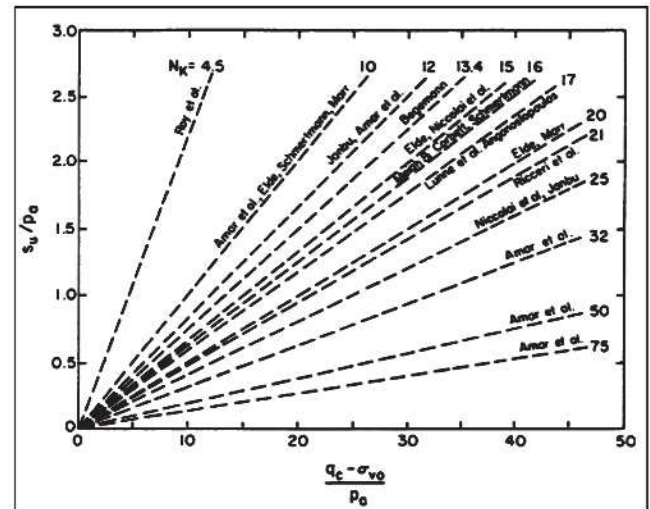
Correlation with Undrained Shear

Shear (s_u)

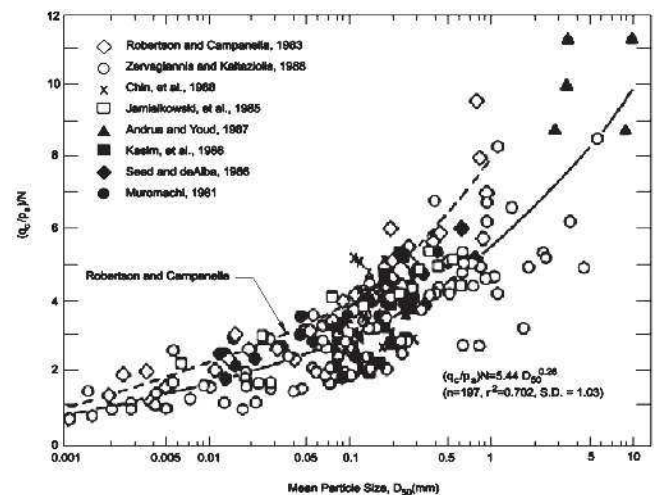
$$s_u = \frac{q_t - \sigma_{vo}}{N_k} \quad N_k = 11-19 \quad \text{Lunne \& Kleven (1981)}$$

or

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \quad N_{kt} = 15 \quad \text{Gue \& Tan (2000)} \quad \dots\dots\dots(6)$$

Fig. 9: N_k factors for CPT data (after Djoenaidi, 1985 cited in Kulhawy & Mayne, 1990)

In spite of its immense value, the interpretation of CPT results must proceed with caution. This is largely due to the assumption of undrained conditions during the test. It is understood that the 2cm/min rate of penetration of the cone is in excess of typical permeability or pore pressure dissipation rate in sandy soils. This means that truly undrained conditions are hardly met by the CPT. Furthermore; the tip resistance of the CPT is increased with higher mean particle size (Fig. 10) at the same overburden pressure. This implies that the cone factor need not be a constant even at the same test point.

Fig. 10: Variation of Q_c/N with mean grain size distribution

The use of CPT for settlement analysis relies on the relationship between the cone tip and soil compressibility. The method of prediction generally used is that proposed by de Beer and Martens (1957) in which:

$$\text{Constant of compressibility, } C_s = 1.9 \frac{C_r}{Po_1} \dots \dots \dots (7)$$

Where C_r = static cone resistance (kN/m^2)

Po_1 = effective overburden pressure at point tested.

$$\text{Total immediate settlement, } Sp_i = \frac{H}{C_s} \text{Loge} \frac{Po_2 + Ds_z}{Po_1} \dots (8)$$

Where

Ds_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.

The great advantage of CPT is that it captures the compressibility response of thin layers on a near continuous basis. These responses when integrated can result in an Incremental Settlement Evaluation, which indicates the contribution to settlement of individual layers or groups of layers. This makes it easy not only to determine total settlement, but also the depth of excavation for soil replacement purposes in order to improve both bearing capacity and settlement.

Field Vane Test

This Test consists of advancing a four-bladed vane into

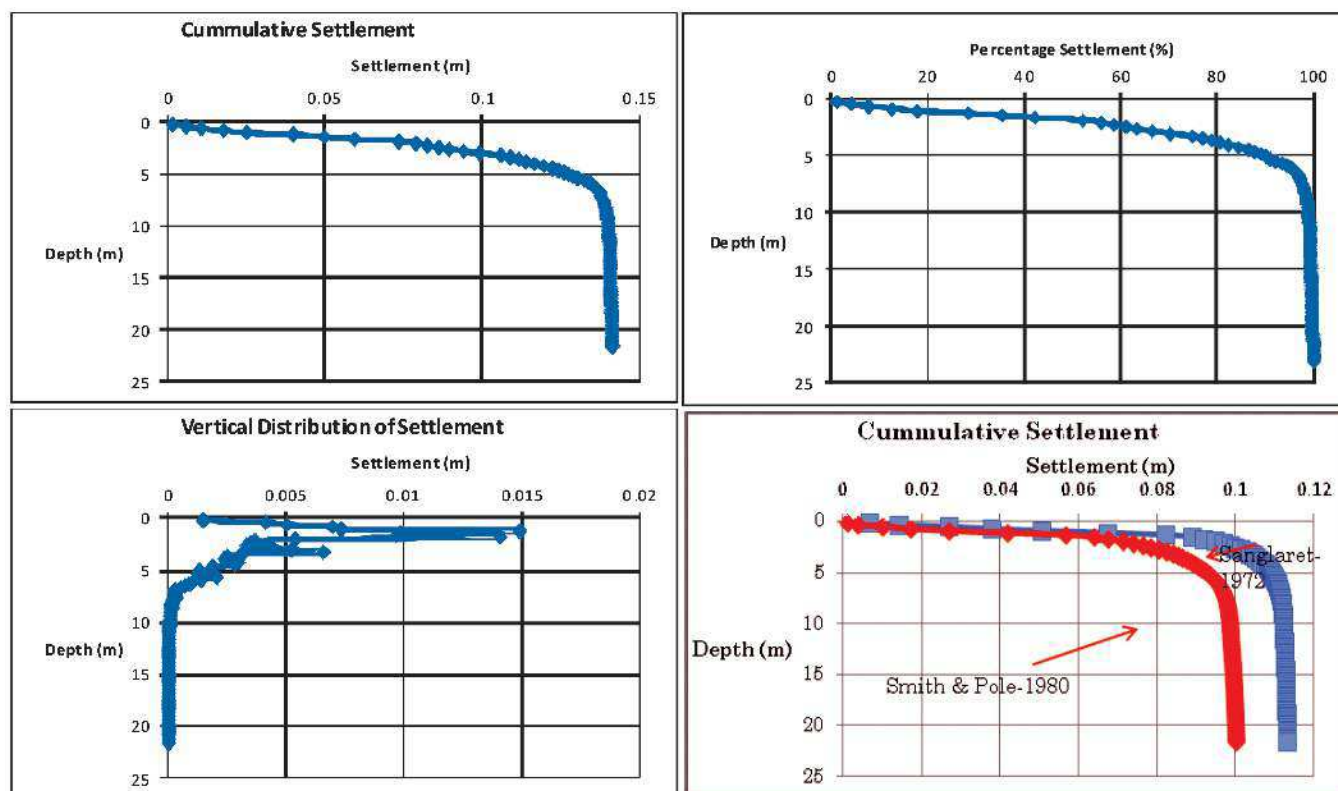


Fig. 11: Settlement profiling using the cone penetrometer

cohesive soil to the desired depth and applying a measured torque at a constant rate until the soil fails in shear along a cylindrical surface. The torque measured at failure provides the undrained shear strength of the soil. A second test run immediately after remolding at the same depth provides the remolded strength of the soil (soil sensitivity and Anisotropy). The Method is commonly used for measuring shear strength in soft clays and organic deposits. It should not be used in stiff and hard clays. Results correlated with undrained shear

strength, remolded undrained shear strength and sensitivity.

Laboratory Tests

A variety of laboratory tests can be performed on soils to measure a variety of soil properties. Some soil properties are intrinsic to the composition and therefore not affected by sample disturbance, while other properties depend on the structure of the soil as well as

its composition, and can only be effectively tested on relatively undisturbed samples. Some soil tests measure direct properties of the soil, while others measure "index properties" which provide useful information about the soil without directly measuring the property desired. Laboratory testing must be intelligently planned in advance but flexible enough to be modified based on test results. For example- Selecting test horizons based on lithostratigraphy. An ideal laboratory program will provide the engineer with sufficient data to complete an economical design.

Key Laboratory Tests

There may be several laboratory tests, but the key remains strength, compressibility and permeability. The shear strength (UU, CU, CD) of a soil is the maximum shearing stress the soil structure can resist before failure. Soils generally derive their strength from friction between particles (expressed as the angle of internal friction, ϕ), or cohesion between particles. The laboratory test procedure for C- ϕ soils require the use of the triaxial equipment (Fig.12) and results plotted in the Mohr-coulomb to derive the strength parameters of Angle of internal friction and cohesion.

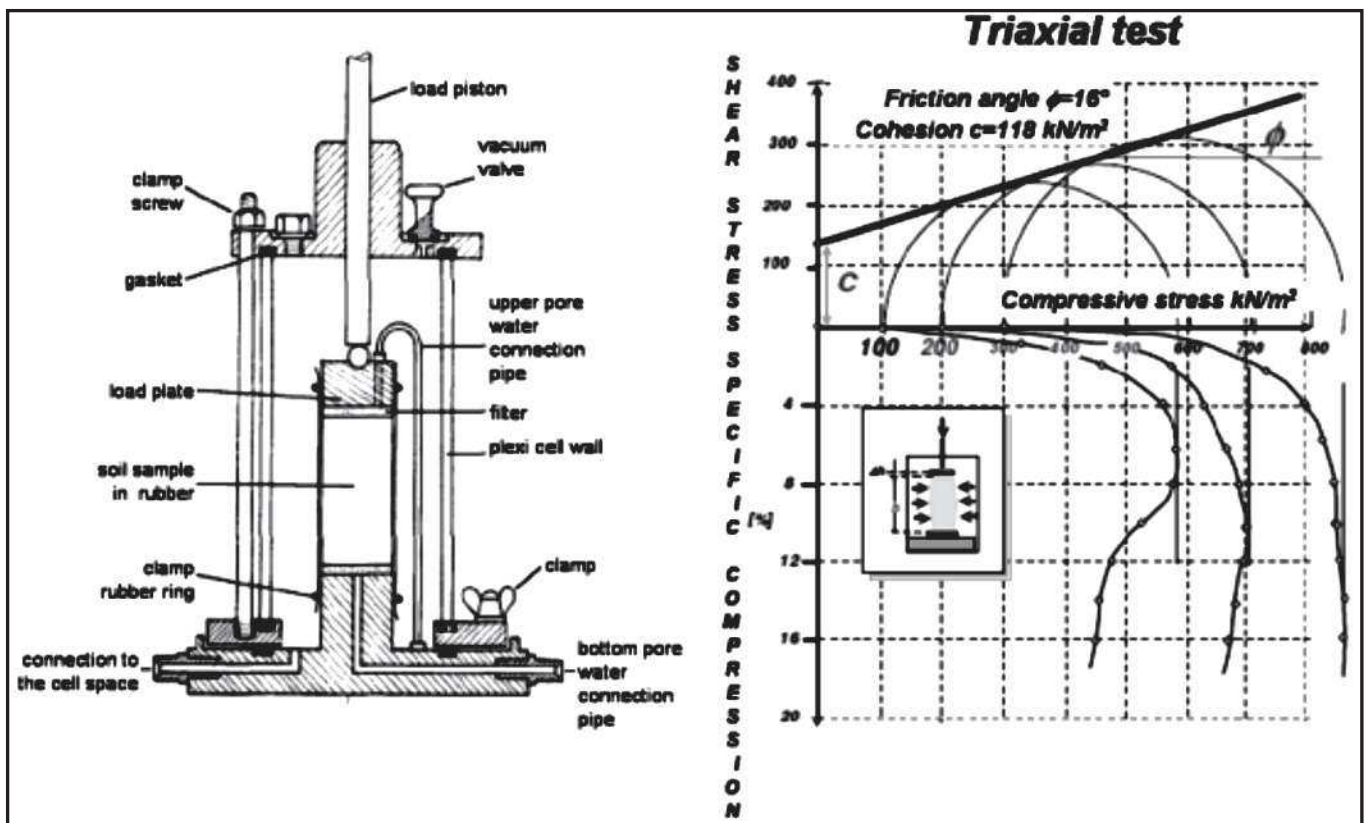


Fig. 12: Triaxial strength test and output

Compressibility enables the determination of Consolidation Settlement – Total and Differential Settlement which has largely been covered under the section on CPT. It is important to say that understanding or being able to determine and apply settlement data to foundation design is a necessary precondition for a successful project, particularly for structures with rigid skid systems with low tolerance to settlement. Manifestation of foundation settlement problems (Fig.13) can be very unsettling.

Compaction: Field Application of Laboratory compaction results

Compaction tests (Fig.14) are used to determine the optimum water content and maximum dry density, which can be achieved for a particular soil using a designated compactive effort. Results are used to determine appropriate methods of field compaction and to provide a standard by which to judge the acceptability of field compaction. Results of compaction test are

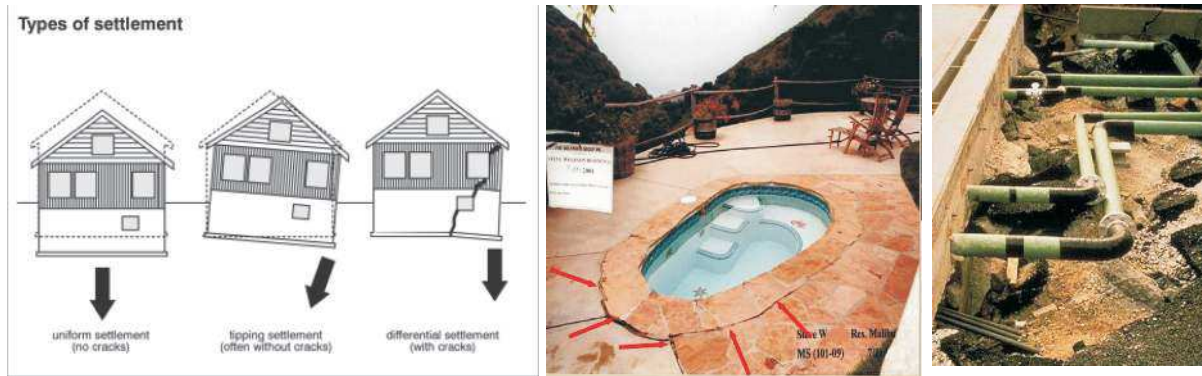


Fig. 13: Manifestation of foundation settlement related problems

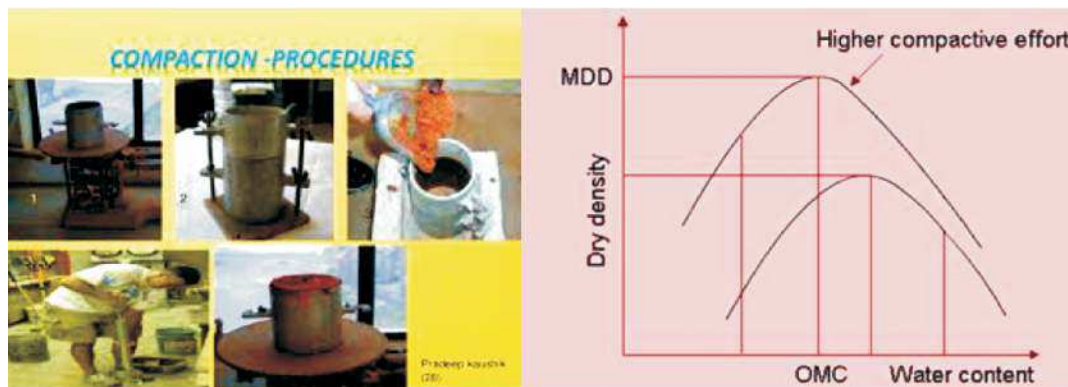


Fig. 14: Compaction test and output

applied to the field processes to control operations in order to achieve the desired compaction levels at the specified optimum moisture content. Usually, compaction trials are undertaken to determine the suitability of different compactors, the number of passes required to achieve the required dry density usually not less than 95% of the laboratory maximum and the thickness of the lifts (ie fill materials).

Permeability of a soil is important for many reasons. Permeability determines Settlement Rate, governs Seepage, Water tightness and Impoundment issues

relating to Dam, water ingress issues in tunnels and foundation stability, material selection for roads, fills and filters. Permeability can be determined in the laboratory as well as in the field. The preferred method of test (Fig.15) depends on the application and use of the information. For major construction where rock mass properties play significant roles, field methods of permeability test are advised. For construction in relatively small sites, the test may be determined by the predominant soil type, falling head for mainly clayey soils and constant head for prominently sandy soils.

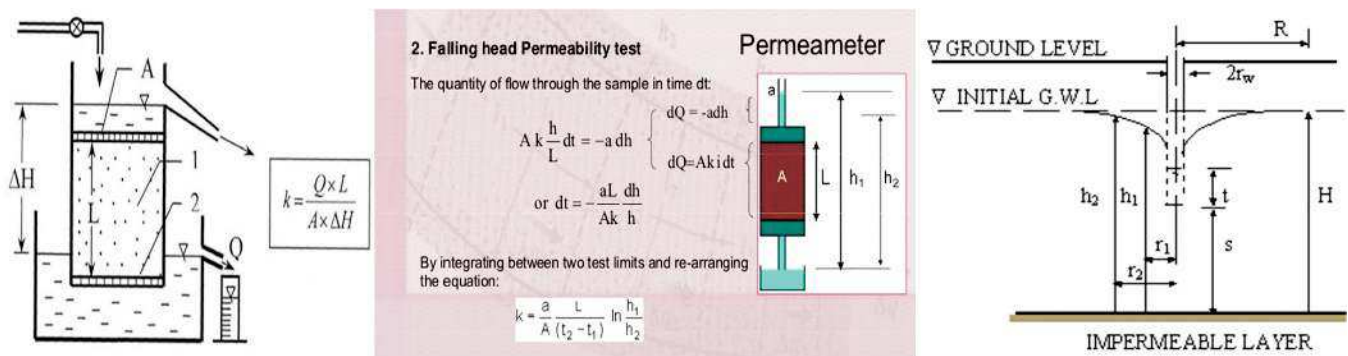


Fig. 15: Laboratory and field measurements of permeability

Conclusions

Although incidences of building collapse are widespread in Nigeria, they can be minimized with adequate soil investigation. When foundation predictions are based on inaccurate or insufficient information, the result is a partial to total difference between expected and encountered conditions with potential consequence for failure. Soil investigation

provides the basis for anticipating in a reliable and specific way the behavior of the ground. The paper has brought clarity to the goal, objectives and procedures for soil Investigation for civil engineering projects. The paper has also highlighted the potentials in the applications of the Cone penetrometer to soil characterization and ultimately provided some insight into the future of Soil Investigation in Nigeria.

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Geotechnical Characterization of Subgrade Soils in Southwestern Part of Nigeria

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Abstract

Causes of road failure are often associated with poor construction materials or inadequate design without cognizance of the underlying soils. Engineering properties of ten bulk soil samples collected from the subgrade of Arigidi/Oke-Agbe highway were investigated to determine their suitability for highway pavement. Results showed that subgrade soils below the failed locations have higher plasticity indices and are classified as A-7-6 clayey soils with high water absorption capability (16.1 – 22.4%) compared to subgrade soils from the stable locations. Low compacted density ($1325 - 1928 \text{ Kg/m}^3$), poor CBR values; 8 – 31% (unsoaked) and 3 – 8% (soaked) indicating 77% reduction in strength on exposure to moisture and the predominance of fines (> 59%) in the soils are responsible for the degree of instability. Also, soft to low stiffness ($49 - 131 \text{ kN/m}^2$) of the subgrade materials underlying the pavement result to the failure characteristics witnessed. The coefficient of permeability ranges from $3.11 \times 10^{-8} \text{ cm/sec}$ to $1.39 \times 10^{-5} \text{ cm/sec}$. The soils have shown their entire unsuitability as fill or subgrade materials in their natural state as indicated from their weak bearing strength and low permeability. This study has shown that the suitability and behaviour of subgrade soil is dependent on its engineering properties.

Keywords: Engineering properties, Failure characteristics, Permeability, Highway pavement, Strength

Introduction

Engineering structures are constructed with the expectation to stand a test of time (Ademila, 2016). The suitability of a soil for a particular use depends on its response to that use. So, utilization of soils needs adequate knowledge of their properties and factors affecting their behaviour and performance. This is encouraged to constitute a precondition for its use in civil engineering construction works. Nigeria is among the countries with the highest records of road accidents with continuous loss of lives and properties due to bad roads. The relationship between highway pavements and their foundation soils cannot be overemphasized in any nation that desires to develop. Performance of highway pavement mostly the flexible one depends on the functions of the component layers especially subgrade.

Failures on Nigeria highways are generally due to poor geotechnical properties of the underlying soils which constitute the entire road pavement. Adeyemi (1995) emphasized the role of parent rock factor on index and moisture-density relationship of subgrade soils developed over certain Precambrian parent rocks of Southwestern Nigeria. A review of the factors influencing the performance of a pavement has been described by Adlinge and Gupta (2010) including the different types of road failure ranging from cracks, potholes to road-cut leading to differential heave of the

pavement material causing frequent bumps on the highways. Lack of adequate knowledge of the behaviour of soils and application of geotechnical parameters of soil have resulted in wrong design and wasteful construction of many of our roads. However, no investigative work on the geotechnical properties of the soils of the area has been reported in the literature to predict the stability of the flexible highway pavement. Accordingly, the area was chosen because of the negligence on the side of the government, designers and constructors of the road which, after some reconstruction and rehabilitation works the failures still persist. This paper discusses the geotechnical properties of ten samples obtained along the failed road and compared with standard values. With the believe that the more the geotechnical data that are available on the subgrade layer the better, as this can lead to more understanding of their engineering performance.

Location, Geomorphology and Geologic Setting of the Study Area

Arigidi/Oke-Agbe highway is an important road that links Abuja (Federal Capital City) and other parts of Southwestern Nigeria. It lies within latitudes $7^\circ 34' \text{ N}$ and $7^\circ 40' \text{ N}$ and longitudes $5^\circ 47' \text{ E}$ and $5^\circ 54' \text{ E}$. It covers an area extent of about 16 Km^2 (Fig. 1). It is situated in the humid tropical region of Nigeria, characterized by alternating wet and dry seasons with a mean annual rainfall of over 1500 mm. The area is

majorly drained by River Auga and other streams which are seasonal. The river dominates the drainage system of the study area and it's mainly dendritic. The area is moderately to highly undulating with an average surface elevation of about 371 m.

Geologically, the area falls within the Precambrian Basement Complex rocks of southwestern Nigeria. It is underlain by migmatite-gneiss-quartzite complex with the granite gneiss and granite being the major rock unit (Fig. 1) with minor amount of grey gneiss, charnockite and quartzite. Granite gneiss is the metamorphosed granites, widely distributed in the study area and it is of two types; the biotite rich gneiss and the banded gneiss. The biotite rich gneiss is fine to medium grained and show strong foliation trending westwards and is usually dark in colour. The banded gneiss show parallel alignment and alteration. It occurs mostly as hills, boulders and flat lying exposures which are dark to light grey in colour and porphyroblastic in texture. There are several quartzite intrusions cutting across the granite.

Grey gneiss in the study area varies from light to dark grey. The gneissic rocks occur as highly weathered and low-lying outcrop. The granite rocks are of the older granite suite. Migmatite gneiss is the predominant rock unit that underlies the study area.

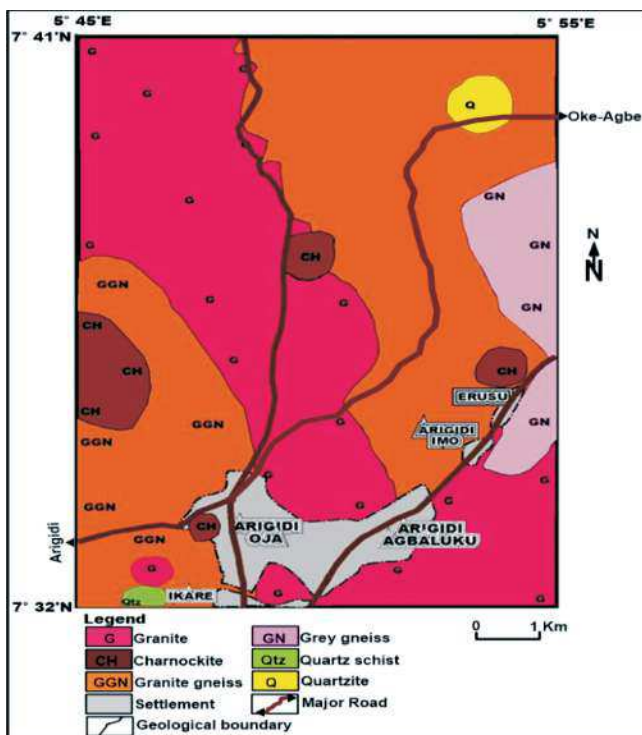


Fig. 1: Geological Map of the part of Southwestern Nigeria showing the study road

Materials and Methods

This research include detailed geological field mapping in order to ascertain the local geology of the area and identification of failed and stable sections of the studied road. On the field, the rock and soil exposures were observed and described. Ten bulk soil samples from the subgrade were collected from ten different locations along the road. Eight samples from eight failed sections and two samples from two stable sections of the road serving as control were obtained at sampling depths of 1 m to obtain true representative samples of the sub-grade which is the placement level of flexible highway pavement. The sample collection was done in a systematic way so as to ensure proper collection of samples and total coverage of the study area. A global positioning system (GPS) was used at each sampling point to measure coordinates of the station and heights above sea level. All the soil samples were carefully labeled in sample bags and then taken to the laboratory in sealed polythene bags to prevent contamination and loss of moisture. The natural moisture content was determined immediately in the laboratory. Soil samples were air dried for two weeks to allow partial removal of natural water before other analysis. After the drying, lumps in the samples were gently grinded with minimal pressure not to reduce the sizes of the individual particles. The following laboratory tests were conducted on the samples: natural moisture content, specific gravity, consistency limits, linear shrinkage, grain size distribution, compaction, California bearing ratio (CBR) and unconfined compressive strength. These laboratory analyses were carried out according to British Standard Methods of test for soils for civil engineering purposes (BS 1377:1990) and ASTM Standard D1557 (2009). Samples for grain size analysis were soaked in a weak calgon solution to facilitate disaggregation during wet sieving. The compaction test was conducted on soil samples compacted in three layers in a CBR mould each 25 mm thick and applying 56 number of blows of 4.5 kg rammer falling freely through a height of 450 mm. The CBR test was carried out with a mould of capacity $945 \times 10^{-6} \text{ m}^3$ at optimum moisture content and 96 hours soaking period which simulates the prolonged inundation and submergence encountered during the peak of rainy season between July and October.

Results and Discussions

Satisfactory pavement performance is largely attributed to a good foundation that provides adequate strength

and low compressibility. This is dependent upon the index and engineering properties of the underlying subgrade layer that serves as the foundation of pavement structure to distribute loads uniformly and effectively.

Index Properties

The results of the index tests carried out on the soil samples are summarized in Table 1

Table 1: Index properties of the soils of the study area

Road condition	Pit No.	Moisture content (%)	SG (Gs)	Consistency limits				Soil type
				Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Linear shrinkage (%)	
Failed	L1	22.4	2.74	61.2	28.6	32.65	19.3	Clayey
Stable	L2	8.0	2.69	31.3	21.3	10.00	5.7	Silty of clayey gravel sand
Failed	L3	16.1	2.75	48.1	22.4	25.70	20.0	Clayey
Failed	L4	18.2	2.68	33.5	21.3	17.20	9.2	Clayey
Failed	L5	17.8	2.65	40.9	22.4	18.50	9.9	Clayey
Failed	L6	19.6	2.64	37.0	19.6	17.40	11.0	Clayey
Failed	L7	16.3	2.72	51.8	23.1	28.75	10.0	Clayey
Stable	L8	8.8	2.64	27.7	19.1	8.65	7.8	Silty of clayey gravel sand
Failed	L9	21.3	2.72	44.2	23.2	20.95	10.0	Clayey
Failed	L10	17.1	2.73	50.8	23.1	27.70	10.7	Clayey

properties of the soils. The natural moisture content of the analyzed soil samples from the failed sections of the road varied from 16.1 – 22.4% (Table 1) while, those from the stable sections varied from 8.0 – 8.8%. This shows that soils from the failed sections have high natural moisture content, because these values are higher than the average range (5 – 15%) specified by FMWH (2000) for engineering construction. This is an indication of high water adsorption capability of the soil material.

Specific Gravity

The specific gravity (SG) of the samples varied from 2.64 – 2.75 (Table 1). All the soil samples met the specific gravity standard (≥ 2.6). The results showed that the soil samples contained sodium and calcium feldspar (2.62–2.76) in nature (Das, 2000). On the basis of this specific gravity, the studied soils can be classified as inorganic soils (Ramamurthy and Sitharam, 2005).

Consistency Limits

The results of the liquid limit, plastic limit, plasticity

Natural Moisture Content

Moisture and movement of water are determining factors in the potential subgrade condition as they influence the design and construction of highway pavement structure. It is used as an indicator for the shear strength of soils, as increase in the moisture content results in a decrease in the shear strength of the material. The high moisture content obtained conform to the generally high porosity and low permeability

index and linear shrinkage tests are summarized in Table 1. The liquid limit, plastic limit and plasticity index of the soils from the failed sections ranged from 37 – 61%, 20 – 29% and 17 – 33% respectively while those from stable sections ranged from 28 – 31%, 19 – 21% and 9 – 10% respectively. According to Whitlow (1995), liquid limit less than 35% indicates low plasticity, between 35% and 50% indicates intermediate plasticity, between 50% and 70% high plasticity and between 70% and 90% very high plasticity. On the basis of this classification, the soils of the stable sections have low plasticity, while soils from the failed sections have moderate to high plasticity characteristics, thereby causing significant deformation under load. The Casagrande chart classification (Fig. 2) also places the soils of the failed sections in the medium and high plasticity/compressibility region. The plots on the plasticity classification chart revealed that the soils mostly fall above the A-line, indicating that they composed of inorganic materials with low-medium-high plasticity.

However, the Federal Ministry of Works and Housing (2000) specified a maximum liquid limit of 40% and a

maximum plasticity index of 20% for highway subgrade material. The results showed that the soils from the failed sections are not good subgrade material for foundation of pavement structures because of their poor geotechnical properties. Plasticity index tends to have a great influence on the activity of the subgrade soils. The plasticity index of the soils of failed sections are higher than 12% and hence, not suitable for use as subgrade materials for roads and bridges as specified by FMWH (2000).

The linear shrinkage of the soils from the failed sections varied from 9.2 – 20.0%. The value is greater than the maximum 8% recommended by Madedor (1983), based on this consideration, the soils would be subjective to swelling and shrinkage during alternate dry and wet seasons of the humid tropical climatic condition of the southwestern Nigeria, hence, the soils are unsuitable as highway subgrade material. The linear shrinkage of the soils from the stable sections varied from 5.7 – 8.3%, which is within the recommended value for subsoil material good for road construction as foundation material. The fine soil mixtures from the failed sections contained clay with higher plasticity index and linear shrinkage values. They are expected to create field compaction problem because of their high values of linear shrinkage. Generally, the lower the linear shrinkage, the lesser the tendency for the soil to shrink when dried (Jegade, 1999).

Grain Size Distribution

The results of the grain size distribution are summarized

in Table 2. The soils from failed sections have percentage passing No. 200 (0.075 mm) ranged from 58.5 – 80.2%, while those of stable sections ranged from 21.3 – 29.2%. It can be observed that the soils from failed sections have high proportion of fines (> 59%) indicating that the soils have the tendency to shrink and swell repeatedly during alternate dry and wet seasons of the humid tropical climatic condition of the southwestern Nigeria, thereby causing distress noticeable on the road. Failures observed in the study area are believed to be caused by pressures developed by the swelling soils. The tested soils of the stable sections (L2 and L8) have percentage passing 0.075 mm of less than 35%, with less amount of clay (14.7%), and fall under group A-2-4 of American Association of State Highways and Transport Officials (AASHTO) classification rated good subgrade highway material (FMWH, 2000). The soils of the failed sections fall under group A-7-6 of AASHTO classification (Table 2) suggesting poor highway subgrade materials being clayey soil type (Table 1). The predominance of fines in the subgrade materials of the failed sections of the road is responsible for the degree of instability witnessed.

Compaction Characteristic

The strength and performance of a pavement is dependent on the load bearing capacity of the subgrade soil. Design of pavement also depends upon the strength of the subgrade soil, which affects the thickness of pavement. Compaction reduces the detrimental effects of water. Lower optimum moisture content is expected to achieve maximum dry density for stability of soil

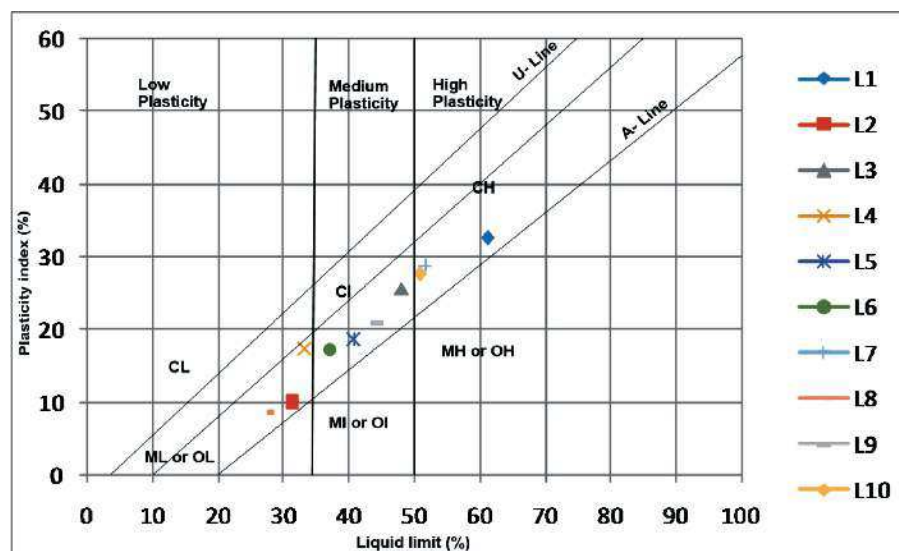


Fig. 2: Casagrande chart classification of the soil samples

Table 2: Grain size distribution parameters

A Road Condition	Pit No.	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	Amount of Fines (%)	AASHTO Classification
Failed	L1	0	19.8	17.0	63.2	80.2	A-7-6
Stable	L2	1.3	69.5	14.5	14.7	29.2	A-2-4
Failed	L3	1.2	40.3	17.4	41.1	58.5	A-7-6
Failed	L4	1.0	33.1	14.9	51.0	65.9	A-7-6
Failed	L5	1.0	36.1	17.3	45.6	62.9	A-7-6
Failed	L6	1.3	34.1	16.3	48.3	64.6	A-6
Failed	L7	1.0	35.1	17.1	46.8	63.9	A-7-6
Stable	L8	5.4	73.3	9.2	12.1	21.3	A-2-4
Failed	L9	1.0	40.2	18.0	40.8	58.8	A-7-6
Failed	L10	1.0	38.6	17.2	43.2	60.4	A-7-6

under field conditions. The maximum dry density and optimum moisture content of the soils at standard proctor compaction energy are shown in Table 3.

Dry density is an important parameter that affects soil reaction to stress. The maximum dry density (MDD) of the soils from failed sections of the highway ranged from 1325 – 1928 Kg/m³ at optimum moisture content (OMC) of 15.3 – 32.7%, while those from stable sections ranged from 1960 – 1995 Kg/m³ (MDD) at 13.4 – 14.4% (OMC) (Table 3). This low range in density with high moisture content of the soils from failed sections indicates low strength with wetness. The interaction of the subgrade with water greatly reduced the strength and therefore continuous failure of the

overlying pavement. However, the best soil for foundation is the soil with high maximum dry density (MDD) at low optimum moisture content (OMC) (Jegede, 1999). Failure of civil engineering structures and road pavements increase when the underlying soils are always soaked with water. The results showed that the compacted densities were very low. So, the subgrade must always be compacted above the MDD and OMC to yield maximum strength, prevent ingress of water and distribute wheel loads uniformly into the pavement structures.

California Bearing Ratio (CBR)

California bearing ratio is a semi-empirical test for

Table 3: Strength properties of the soils of the study area

Road condition	Pit No.	Compaction characteristics		California bearing ratio, CBR (%)		Percentage reduction in strength (%)	Unconfined compressive strength (kN/m ²)
		MDD (Kg/m ³)	OMC (%)	Unsoaked CBR (%)	Soaked CBR (%)		
Failed	L1	1325	32.7	8	3	62.50	48.72
Stable	L2	1960	14.4	32	10	68.75	149.92
Failed	L3	1606	24.6	16	4	75.00	89.14
Failed	L4	1890	16.4	31	8	74.19	123.03
Failed	L5	1854	17.4	28	7	75.00	98.15
Failed	L6	1928	15.3	31	8	74.19	131.76
Failed	L7	1529	26.8	13	3	76.92	71.44
Stable	L8	1995	13.4	37	11	70.27	165.11
Failed	L9	1581	25.3	17	6	64.71	78.07
Failed	L10	1546	26.3	14	6	57.14	75.29

evaluating highway subgrade/subbase soils. The soaked CBR test is also used in simulation of the condition that soils are exposed to in-situ upon ingress of water. It is the resistance a soil possesses when stressed or under load. The values of unsoaked and soaked CBR of the soils from the failed sections ranged from 8–31% and 3–8% respectively. It shows reduction in strength up to 77% with wetness (Table 3). The soaked CBR showed that subgrade soils at failed sections are susceptible to volume change on exposure to excessive moisture. It was observed that unsoaked (32–37) and soaked (10–11) CBR values of samples from stable sections are higher than those from failed sections. The soaked CBR values obtained showed that subgrade soils of the failed sections are liable to critical changes in strength and load bearing capacity as a result of inundation. The CBR values of the study area are generally low indicating low strength of the soil. Soil improvement measures would be necessary for stable structures. The minimum CBR requirements for subgrade, subbase and base courses are 10% (soaked), 30% (soaked) and 80% (unsoaked) respectively (FMWH, 2000).

None of the analyzed samples have the required 30% soaked and 80% minimum unsoaked CBR value recommended for highway sub-base and base course. The soil samples taken from stable sections are within the specification of 10% minimum soaked CBR value for subgrade, which means they are adequate as subgrade material. The tested soils of the failed sections yielded extremely poor CBR values, both in the unsoaked and soaked types. From these results, the soils have shown their entire unsuitability as fill or subgrade materials in their natural state.

Unconfined Compressive Strength (UCS)

Consistency of a soil can be expressed not only in terms of Atterberg limits but also in terms of unconfined compressive strengths of soils which in turn determines the strength of the soil. An understanding of the compressive behaviour is essential for geological and geotechnical engineering purposes and is the core basis for modeling the stress-strain relationships of soils (Liu et al., 2013). Its value is a measure of suitability of such soil as a foundation material. Table 3 shows the summary of unconfined compressive test results. According to Das, (2000), consistency of a clayey soil can be as follows; between 0-25 kN/m² indicates very soft, between 25-50 kN/m² indicates soft, between 50-100 kN/m² indicates medium, between 100-200 kN/m² indicates stiff, 200-400 kN/m² indicates very stiff and

greater than 400 kN/m² indicates hard clay. On this note, consistency of soil samples from failed sections is soft, medium and low stiff (49–131 kN/m²) while that of stable sections remained stiff (Table 3). Soft and low stiffness of the poor subgrade materials underlying the pavement result to the failure characteristics witnessed. This initial consistency can be upgraded/modified by means of soil stabilization.

Permeability Characteristic

According to Siddique and Safiullah (1995), permeability governs important engineering problems like consolidation of clay foundation under applied load and the flow of water through or around engineering structures. The coefficients of permeability of the soils from failed sections ranged from 3.11×10^{-8} cm/sec to 1.39×10^{-5} cm/sec while that of stable sections ranged from 4.00×10^{-5} cm/sec to 2.40×10^{-4} cm/sec (Table 4). Based on the classification by Lambe (1954), the permeability of 10^{-5} cm/sec to less than 10^{-7} cm/sec indicates a very low to practically impermeable soil. However, due to the poor permeability of these soils, water is retained within the soil matrix, resulting in weaker strength and lower bearing capacity. There is a good correlation with the results of the soaked CBR with significant reduction in strength up to 77% in the presence of excessive moisture influx. From this result, it can be deduced that adequate drainage is highly important in the area to prevent ingress of water beneath the pavement, which could lead to the significant reduction in strength of the subgrade soils and thus failure of the pavement. This is observed on the road by wearing away of soil of some sections of the road by erosion caused by non provision of rainwater run-off. Thus, road designers and constructors need to put in place drainage channels on roads to ensure free flow of water. Improvement in pavement design should include detailed description of weather conditions especially temperature and rainfall, as changes in temperature also influence distress in this flexible highway.

Conclusions

Soils encountered in the subgrades of a part of southwestern Nigeria were investigated by their engineering property tests. The soils of the failed sections fall under group A-7-6 of AASHTO classification suggesting poor highway subgrade materials being clayey soil type. The predominance of fines in the soils is responsible for the degree of instability witnessed.

The natural moisture content of the analyzed soil samples from the failed sections indicates a high water adsorption capability of the soil material. The Casagrande chart classification places the soils in the medium and high plasticity/compressibility region. The soils exhibit high shrinkage and low strength properties (MDD, CBR and UCS), which is the main reason the flexible highway pavement constructed on these soils is relatively unstable. From the results, the soils have shown their entire unsuitability as fill or subgrade materials in their natural state. Furthermore, the permeability characteristics generally show poor to practically impervious drainage condition of the soils. The design/reconstruction of the road should be based

on the engineering understanding of the underlying soil with regard to the environmental conditions particularly temperature and rainfall. Thus, good drainage systems should be provided in the area in order to prevent pore water pressures to develop below pavement structures. Removal and replacement of the subgrade and subbase materials of the flexible pavement upto 1 m or soil stabilization will improve the strength for stable structures. Results of this study will be useful in reconstruction works of the failed sections of the road and would guide future pavement design in construction of roads. This is an indication that for effective design of roads, in-depth knowledge of the characteristics and behaviour of the soils is necessary.

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Engineering Geological Characterization of Soils from Banana Island, Lagos, Southwestern Nigeria

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Abstract

Geological and geotechnical conditions of subsurface litho-sequence in Banana Island, Lagos were studied on the bases of field description, in-situ geotechnical test and laboratory analyses aimed at assessing the suitability of the shallow formations as foundation materials. Three rock units were identified as silty sand, silty clay and sand. The silty sand has estimated N-value of 3 – 19, angle of internal friction of 22 – 32°, void ratio of 0.45 – 0.60 while the silty clay has low N-value of 2 – 3, cohesion of 20KN/m² and average void ratio of 1.14. The relatively competent sandy layer at the base has N-value of 17 – 25, angle of internal friction of 33 – 34° and average void ratio of 0.4. The intercalations of very loose to medium dense silty sand sequence from 0.0 – 17.0m render the upper layer technically incompetent for foundation of high rise structure while the presence of highly compressible silty clay at depths approximately 17.0 – 20.0m portends serious danger to engineering applications, particularly for foundation construction. Hence, pile installed within the medium dense sandy layer at depth of 25.0m is considered suitable for foundation under higher loadings in the area.

Keywords: SPT, Banana Island, Foundation, Engineering Geological, Nigeria

Introduction

The growing need for housing and construction has forced authorities and development agencies to exploit lands of poor geotechnical characteristics with little or no consideration for the mechanical stability and bearing capacity of the ground. This practice has resulted in structural defects and/or total collapse of buildings (Amadi *et al.*, 2012). Lands reclaimed from water bodies such as lagoon, sea, swamp, river, flood zones common in Lagos area but are unstable as foundation engineering material (Ademeso *et al.*, 2016). Several workers have reported the presence of clayey and peaty material with undesirable engineering properties in different part of Lagos metropolis (Adebisi and Fatoba 2013; Amadi *et al.*, 2012; Aderoju and Ojo 2015; Oyedele *et al.*, 2011, Oyedele and Olorode 2010, Oyedele *et al.*, 2014). This is however, without consideration to the present study area which has attracted sporadic civil constructions lately. Thus, It is important to give due consideration to the subsoil types and their competence during the conceptualization, planning, execution and maintenance of every construction projects within the area.

The study area is an artificial Island (reclaimed) located in the Lagos Lagoon within Ikoyi district in Eti-Osa Local Government Area of Lagos State (Fig. 1). It occupies a sand-filled area of approximately 1,630,000 square metres divided into 536 plots at 8.6 kilometres East of Tafawa Balewa Square. It falls within

coordinates 6°28'N, 3°27'E and 6.467°N, 3°450'E. It is a zone of coastal creeks and lagoons which is almost encompassed by the Lagos lagoon system. It is located near the eastern margin of the Nigerian sector of the Dahomey Basin, within the Western Nigeria coastal zone which consists largely of coastal creeks and lagoons developed by barrier beaches associated with sand deposition underlain by recent deposits which vary from the littoral and lagoon sediments to the coastal belt and alluvial deposits of the major rivers. Geologically, it falls extensively within the Eastern Dahomey Basin which is a combination of inland/coastal/offshore sedimentary basin in the Gulf of Guinea (Obaje, 2009). Stratigraphically, the basin is divided into Abeokuta Formation, Ilaro Formation, Coastal Plain Sands and Recent Alluvium sediments (Jones and Hockey, 1964). However, for engineering applications, attention is on the Quaternary sequence in the basin which comprises Coastal Plain Sands and recent littoral Alluvium (Durotoye, 1975) and consists of poorly sorted sands, silts and clay deposits with traces of peat in parts. The sands are in parts cross-bedded and show transitional to continental characteristics. The age is from Oligocene to Recent. It directly underlies the study area and is composed of deposits which can be divided into the littoral and lagoonal sediments of the coastal belt and the alluvial sediments of the major rivers. They consist predominantly of unconsolidated sands, clays and mud

with a varying proportion of vegetative matter. The Coastal Plain Sands and recent littoral Alluvium serve

as foundation of engineering structure within the study area.

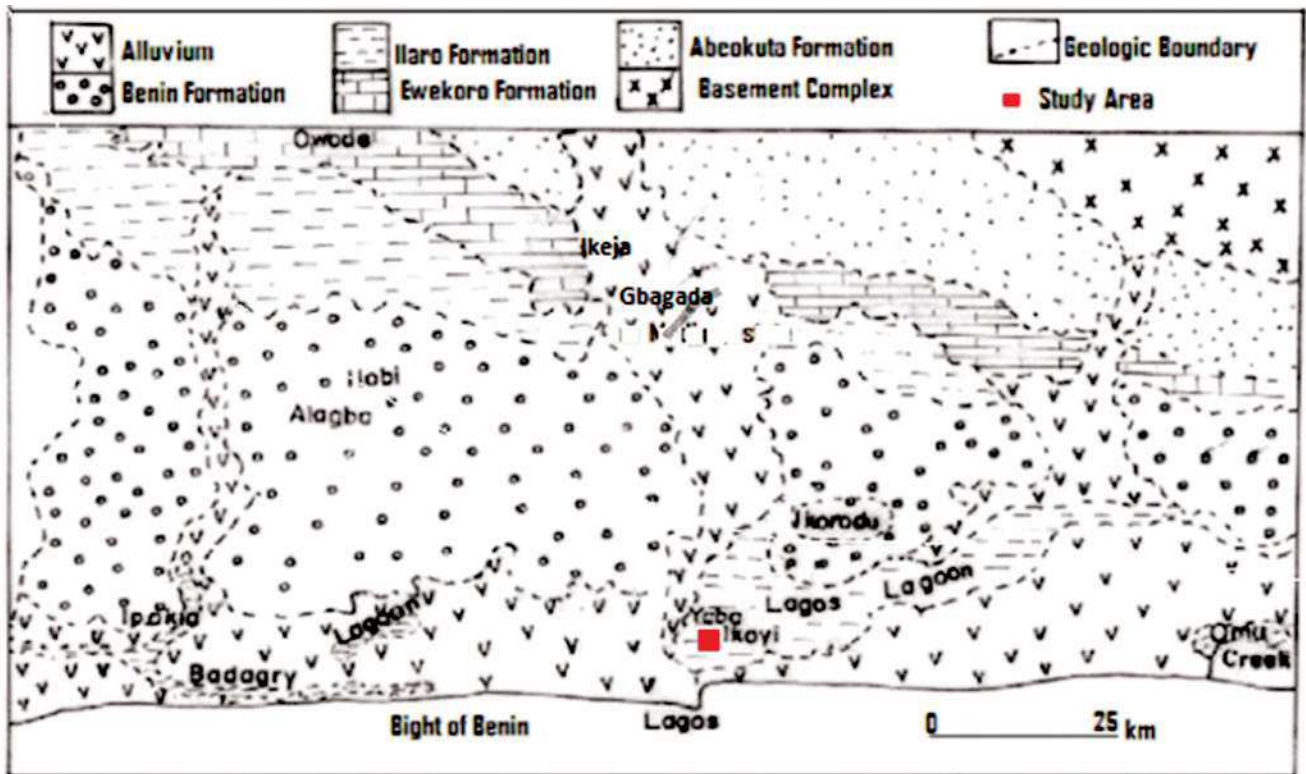


Fig. 1: Geological map of Lagos State showing the study area (modified after Oyedele and Olorode, 2010).

Large and multi-storey construction activities are currently ongoing in different parts of the Island necessitating the need to provide geotechnical database for such developments. The site where this study was carried out within the residential section of the Island is designated for the construction of multi-storey buildings. This study determines the stratigraphy of the superficial deposit underlying the area as well as the relevant engineering characteristics of the deposits to enable appropriate foundation design of structures in the area.

Materials and Methods

Boring and Standard Penetration Testing (SPT)

Two boreholes were drilled to depth 30.0m each with 250mm, 200mm and 150mm diameter steel casings using a Percussion motorized Shell and Auger rig employing light cable percussion boring techniques with a fully equipped motorized Pilcon Wayfarer

drilling rig. Borehole 1 and 2 is at approximately 50.0m from each other. During drilling operations, disturbed soil samples were regularly taken at depth interval of 0.75m and wherever a change of stratum is observed. Samples comprise those from the split spoon of standard penetration test and those of the cutting shoe of 100mm diameter sampler. All samples recovered from the borehole were examined for field characteristics, identified and classified. They were later taken to the laboratory for detailed investigations where a total of 18 samples from each hole were subjected to grain size distribution by wet sieving and Atterberg limit tests accordingly. Effort was made to ensure that all strata encountered were tested appropriately. Standard Penetration Test (SPT) was also carried out at 1.5m intervals.

In carrying out the SPT test, a 50mm diameter split spoon sampler is driven into the soil using a 63.5kg hammer with a 760mm drop, and the penetration resistance is expressed as the number of blows (SPT-N-value) required obtaining a 300mm penetration below

an initial 150mm penetration seating drive. The SPT-N-values were corrected for borehole and dilatancy where necessary and all pertinent borehole data, penetration resistance, and sample data were recorded on the boring log sheet (Figs 2 – 3).

Classification Tests

A series of classification tests were carried out on the samples in strict compliance with relevant geotechnical engineering standards including British standards (BS 1377 (1990); Vickers, 1978; Tomlinson, 1999). Laboratory classification tests were conducted on a number of soil samples to verify and improve on the field identification. These tests include natural moisture content, unit weights, Atterberg limits (liquid and plastic) and grain size distribution. The bearing capacity analysis for shallow foundation was limited to depth 2.0m underlain by loose silty sand while the pile safe working load analysis was limited to the medium dense to dense soil sequence at depths 8.5.0 – 30.0m.

Calculation of Bearing Capacity

The poisson ratio and modulus of elasticity were evaluated from the method of Bowles (1997). The angle of internal friction was estimated from penetration tests based on Sanglerat (1972), Bowles (1997) and Meyerhof (1965) methods.

Plasticity Index (PI) = LL - PL, where LL = liquid limit, PL = plastic limit

$$\text{Liquidity Index (LI)} = \frac{w - \text{PL}}{\text{PI}} \dots\dots\dots(i)$$

Where w = Natural water content, PL = plastic limit, PI = plasticity index.

Teng (1969) equation used to estimate the net allowable bearing capacity under shallow foundation for 40.0mm permissible settlement.

$$\text{Net allowable capacity} \\ (q_{na}) = 55(N-3) \frac{(B+0.3)^2}{(2B)^2} \times k_d \times R_w \dots\dots\dots(ii)$$

Where q_{na} is the net allowable bearing capacity, N is the average SPT-N value of the bearing stratum at base of the foundation, B is the width of the foundation, k_d is the depth correction factor, R_w is the water table correction factor.

Using the equation of ultimate bearing capacity for a driven pile;

$$Q_f = q_f \times A_b + f_s \times A_s \dots\dots\dots(iii)$$

Where

Q_f = ultimate load that can be applied at the top of the pile,

q_f = ultimate bearing capacity of the stratum on which the pile is supported,

f_s = the average shearing resistance of soil per unit area

A_b = area of the pile at the base,

A_s = cylindrical surface area of the pile.

SPT-N method was employed using Meyerhof (1976) equation for driven pile;

$$q_f = 40 \times N \times D (\text{limited by } 400N) \dots\dots\dots(iv)$$

$$f_s = 2N_a \text{ where}$$

N is the SPT-N at the vicinity of the base of the pile,

N_a is the average SPT-N value over the embedded depth of the pile.

Since the above equation is applicable for driven piles, the value of q_f obtained was further multiplied by 0.33 while that of f_s was multiplied by 0.5 to derived corresponding values for bored piles.

Allowable bearing capacity:

$$q_a = q_u / F.S \dots\dots\dots(v)$$

where F.S. is factor of safety = 2.5.

Results

Classification Tests

The results of grain size analysis and Atterberg consistency tests are as shown in tables 1 and 2 respectively while figs. 4 and 5 showed the grain size distribution curves. Interpretation of the result showed that the samples of cohesion less soil at different depths have ranges of: (0.02 – 0.12) d_{10} , (0.10 – 0.20) d_{30} , (0.14 – 0.40) d_{60} , (1.6 – 10.0) C_u and (0.6 – 5.6) C_c . The soils are generally classified as poorly graded sand since their coefficient of uniformity (C_u) < 6 (Wagner, 1957; Casagrande, 1959). Also, Atterberg consistency results (table 2) revealed that the soft silty clay at the fourth layer has (94 – 136%) natural water content, (95 – 99%) liquid limit, (22 – 26%) plastic limit, (72 – 76%) plasticity index, (0.99 – 1.49) liquidity index and (69 – 79%) swelling potential. The high water content (94 – 136%) and liquid limit (95 – 99%) is indicative of highly compressible clay which is inimical to the founding of foundation of civil engineering structures. The plasticity index (72 – 76%) is quite high and above the 55% limits

set for material with very poor and undesirable engineering properties (Holtz and Cubbs, 1956). In addition to these, the liquidity index (0.99 – 1.49) is high since it is >1 and is suggestive of sensitive clay stratum with which may be prone to liquefaction under sudden loadings. The swelling potential of (69 – 79%) is very high and undesirable in construction of foundation of engineering structure since it is above 25% (Seed et al., 1975).

Standard Penetration Test

The result of boring and Standard Penetration Test is as presented in borehole logs in Figs. 2 and 3. Three lithologic units divided into six geotechnical layers based on N-values were delineated from ground surface to about 30.0m depth. These comprises essentially of silty sand, silty clay and sand deposits which are typical of swamp and creek environments. The first layer is made up of loose to medium dense silty sand (0.0 – 2.0m) with an average N-value of 17 (Fig. 2) which is capable of withstanding low to medium foundation pressures. The second layer consists essentially of very loose silty sand (2.0 – 8.50m) with an average N-value of 3 which not only render the layer unsuitable as foundation material but also portend serious danger to the first layer. It is pertinent to mention that the presence of this very loose layer can trigger appreciable immediate settlement under loadings when structures are sited on the first layer. These layers are further underlain by medium dense silty sand (8.50 – 17.0m) with average N-value range of 17 making it suitable for foundation of engineering structure under medium loadings. It is further underlain by high collapsible and compressible soft silty clay (17.0 – 20.0m) with visible traces of incompletely decayed peaty materials. It has average N-value of 2 which is undesirable in foundation under loadings. Beneath the soft silty clay is a layer of loose silty sand (20.0 – 23.0m) with average N-value of 6 rendering it inappropriate for foundation of high rise structure. The sixth layers at the base consist predominantly of medium dense light grey sand with gravelly materials (fig. 3). It is highly resistive with an average N-value of 20 making it the most competent for the foundation of engineering structure under intense loadings.

Depth (m)	Sample NO	Legend	(SPT) Values "N"	Strata Description	Thickness(m)
0.00	1	.X..X..	18	Loose to medium dense dark grey Silty Sand	2.0
2	2	...X...X.			
3	3	...X...X.			
4	4	...X...X.			
3.00	5	...X...X.	3	Very loose dark grey Silty sand	6.50
6	6	...X...X.			
7	7	...X...X.			
8	8	...X...X.			
6.00	9	...X...X.	4		
10	10	...X...X.			
11	11	...X...X.			
12	12	...X...X.			
9.00	13	...X...X.	14	Medium dense dark grey Silty Sand	8.50
14	14	...X...X.			
15	15	...X...X.			
16	16	...X...X.			
12.00	17	X.....	17		
18	18	...X...X.			
19	19	...X...X.			
20	20	X...X...X			
15.00	21	...X...X.	19		
22	22	...X...X.			
23	23	...X...X.			
24	24	...X...X.			
18.00	25	...X...X.	3	Firm dark brownish Silty Clay	3.0
26	26	...X...X.			
27	27	...X...X.			
28	28	...X...X.			
21.00	29	...X...X.	6	Loose dark grey Silty Sand	3.0
30	30	...X...X.			
31	31	...X...X.			
32	32	...X...X.			
24.00	33	...X...X.	18	Medium dense light grey Sand with tiny gravel	7.0
34	34	...X...X.			
35	35	...X...X.			
36	36	...X...X.			
27.00	37	...X...X.	22		
38	38	...X...X.			
39	39	...X...X.			
40	40	...X...X.			
30.00	41	...X...X.	24		
		...X...X.			
		...X...X.			
		...X...X.			
				End of borehole	

Fig. 2: Characteristic of lithologies along the Borehole 1

Discussions

Table 4 represents a compilation of geotechnical engineering parameters estimated from field and laboratory data. The borehole logs (figs. 2 and 3) revealed the occurrence of six geotechnical layers from ground surface to approximately 30.0 m depth. The uppermost layer of silty sand (0.0 – 2.0m) with an angle of internal friction of 32° and N-value of 17 (table 4) is considered a moderate foundation material. The second layer of very loose silty sand with angle of internal friction of 22° and N-value of 2 is considered technical unsuitable for foundation of engineering structures under higher loadings without some form of soil improvements. The layer may also trigger appreciable settlement if the upper layer is heavily loaded. The third layer of medium dense silty sand with angle of internal friction of 31° and N-value of 17 is also considered medium foundation material. The fourth layer of soft silty clay with low cohesion value of 20.0KN/m^2 , N-

Depth (m)	Sample NO	Legend	(SPT) Value "N"	Strata Description	Thickness (m)
0.00—1		..X..X..	17	Loose to medium dense dark grey Silty Sand	3.0
—2		...X..X.			
—3		...X....			
—4		...X....			
3.00—5		X.....	2	Very loose dark grey Silty sand	7.0
—6		X.....			
—7		X.....			
—8		X.....			
6.00—9		X.....X.....	3	Very loose dark grey Silty sand	7.0
—10		X.....X.....			
—11		X.....X.....			
—12		X.....X.....			
9.00—13		X.....X.....	14	Medium dense dark grey Silty Sand	6.50
—14		...X....			
—15		...X....			
—16		...X....			
12.00—17		X.....	16	Medium dense dark grey Silty Sand	6.50
—18					
—19					
—20		...X....			
15.00—21			18	Firm dark brownish Silty Clay	5.50
—22					
—23		X.....			
—24		X.....			
18.00—25		X.....	2	Firm dark brownish Silty Clay	5.50
—26		X.....			
—27		X.....			
—28		X.....			
21.00—29		X.....	5	Loose dark grey Silty Sand	2.50
—30		X.....			
—31		X.....			
—32		X.....			
24.00—33		X.....	17	Medium dense light grey Sand with tiny gravel	6.0
—34		X.....			
—35		X.....			
—36		X.....			
27.00—37		X.....	23	Medium dense light grey Sand with tiny gravel	6.0
—38		X.....			
—39		X.....			
—40		X.....			
30.00—41		X.....	25	Medium dense light grey Sand with tiny gravel	6.0
		X.....			
		X.....			
		X.....			
				End of borehole	

Table 1: Grain Size Distribution Results

Serial No	Sample Designation	Depth (m)	Effective size (d_{10})	Effective size (d_{30})	Effective size (d_{60})	Coefficient of Uniformity (C_u)	Coefficient of Curvature (C_c)	Remark
1	Ban 1	1.50	0.02	0.15	0.20	10.0	5.6	SP
2	Ban 2	6.75	0.09	0.15	0.30	3.3	0.8	SP
3	Ban 3	12.00	0.10	0.20	0.40	4.0	1.0	SP
4	Ban 4	16.50	0.09	0.15	0.20	2.2	1.3	SP
5	Ban 5	20.50	0.06	0.10	0.15	2.3	1.1	SP
6	Ban 6	24.00	0.10	0.15	0.30	3.0	0.7	SP
7	Ban 7	25.00	0.12	0.18	0.30	2.5	0.9	SP
8	Ban 8	29.25	0.10	0.15	0.40	4.0	0.6	SP
9	Ban 9	2.25	0.02	0.15	0.20	10.0	5.6	SP
10	Ban 10	6.75	0.10	0.15	0.25	2.5	0.9	SP
11	Ban 11	13.50	0.12	0.22	0.50	4.1	0.8	SP
12	Ban 12	16.50	0.10	0.15	0.40	4.0	0.6	SP
13	Ban 13	21.00	0.09	0.10	0.14	1.6	0.8	SP
14	Ban 14	24.00	0.10	0.19	0.30	3.0	1.2	SP
15	Ban 15	27.00	0.10	0.16	0.30	3.0	0.9	SP
16	Ban 16	30.00	0.10	0.15	0.30	3.0	0.8	SP

SP = Poorly graded sample

Table 2: Atterberg Limits Results

Sample Designation	Depth (m)	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit, (%)	Plasticity Index, PI(%)	Liquidity Index	Swelling Potential S_p (%)
Ban 17	17.0	136	99	23	76	1.49	84
Ban 18	18.0	94	95	25	70	0.99	69
Ban 19	19.0	102	96	22	74	1.08	79
Ban 20	20.0	112	98	26	72	1.19	74

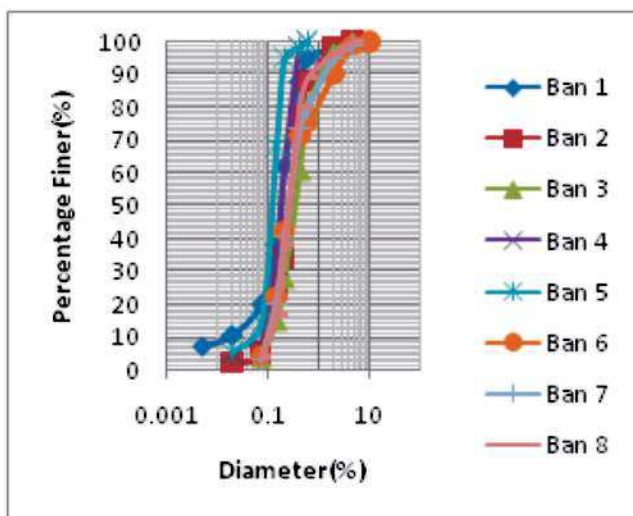


Fig. 4: Grain size distribution curves for borehole 1

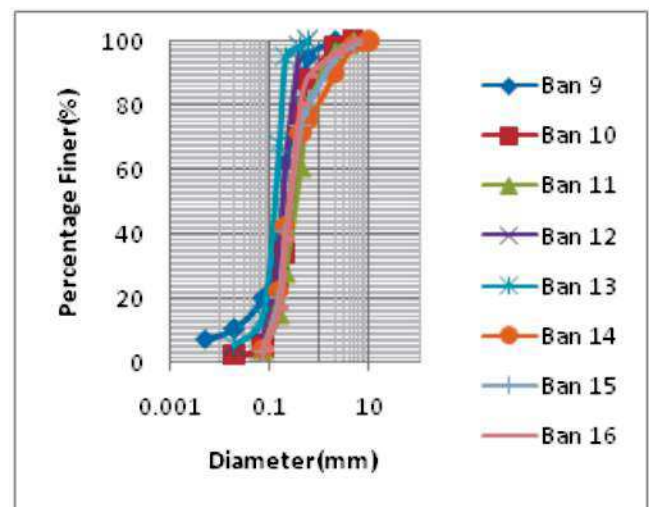


Fig. 5: Grain size distribution curves for borehole 2

Table 3: Summary of Engineering Properties

Geotechnical Engineering Parameters									
Stratum Depth Range (m)	Thickness (m)	Type of Soil	Average SPT Value	Void Ratio	Cohesion (kN/m ²)	Unit Weight(kN/m ³)	Angle of Internal Friction	Estimated modu Elasticity (kN/m ²)	Poisson Ratio
0.00 – 2.00	2.00	Medium dense Silty Sand	18.0	0.50	-	17.50	32°	9,787	0.30
2.00 – 8.50	6.50	Very loose Silty Sand	4.0	0.65		14.50	22°	2,448	0.30
8.50 – 17.00	8.50	Medium dense Silty Sand	17.0	0.45		18.50	31°	21,13 ₃	0.30
17.00 – 20.00	3.00	Soft Silty Clay	2.0	1.14	20	15.50	-	1,961	0.50
20.00 – 23.00	3.00	Loose Silty Sand	6.0	0.60		16.00	29°	5,884	0.30
23.00 – 30.00	7.00	Medium dense Sand	20.0	0.40		19.50	34°	25,51 ₆	0.30

Table 4: Estimated Pile Safe Working Load (Stanlerat, 1972)

Pile Type		Driven Pile		Bored Pile	
Diameter (mm)	Founding Depth (m)	Ultimate Capacity (KN)	Safe Working Load (KN)	Ultimate Capacity (KN)	Safe Working Load (KN)
300	12.0	732	293	487	195
	25.0	1077	431	666	266
400	12.0	1189	476	809	324
	25.0	1658	663	1765	706
500	12.0	1748	699	1207	483
	25.0	2357	943	1532	613
600	12.0	2421	968	1499	600
	25.0	3167	1267	2093	837

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Site Physiography and Geotechnical Assessment of Subgrade Soils Along Ago-Iwoye and Ijesha-Ijebu Road, Southwestern Nigeria

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Abstract

Site physiography and characteristics of laterised subgrade soils between longitudes 3° 44'E and 3° 52'E, and latitudes 6° 54'N and 6° 56'N in Southwestern Nigeria were studied for road design and construction. Geographical locations were used to generate a vector-based digital image of the area. Twenty (20) each of disturbed and undisturbed soils samples were obtained along Ago-Iwoye and Ijesha-Ijebu road taking cognisance of parent rocks. All laboratory geotechnical tests were performed following the British Standard 1337. The three-dimensional vector image revealed a high ground between Ijesha-Ijebu and Irolu, and a dip along a level ridge crest between Ijesha-Ijebu and Omogunwarun Market. Grading and plasticity showed inorganic clay (4-25 %), tropical iron-rich laterite, and lateritic soils with medium silt (8-26 %) and fine sand (6-35 %). Maximum dry density and optimum moisture content ranges were 1667 - 1954 kg/m³ and 9.5 - 19.0% respectively, having about 1.5% of the variation of fines associated with compacted density. Unsoaked California Bearing Ratio (CBR) varies between 11.57 and 57.71%, and soaked CBR ranges from 6.25 to 22.20%. From the test data, it is obvious that the road would require thick pavement to spread the wheel load over the greater low level ridge area.

Keywords: Physiography, Image, Subgrade, Road and Pavement.

Introduction

Experience has shown that terrain configuration and surface water hydrology are fundamental to pavement's martial behaviour. These significantly affect the engineering performance of subgrade soils. Adeyemi (1992) noted that the influences of pedogenic factors of parent rock and topography on the clay mineralogy, geochemistry and highway geotechnical properties of the soils are highly significant. Adeyemi and Wahab (2008) found the California bearing ratio and uniaxial compressive strength to vary significantly over shallow depth, and recommended detailed and thorough sampling of lateritic soils for proper evaluation of highway subgrade soils.

Oyediran and Williams (2010) showed that lateritic soils without typical organic content but high degree of laterisation are suitable for construction purposes. However, such soils were rated poor as subgrade materials or as sub-base under bituminous surfacing. Quadri *et al.*, (2012) compared subgrade and base with a view to ascertaining their susceptibility to volume changes when wet. It was found that the subgrade samples are excellent to good while base sample is fair to poor. Adebisi *et al.*, (2014) combined geotechnical data with drainage system to elucidate the engineering performance of native subgrade soils. The research identified surface water flow is an important component of road construction project.

In order to improve the level of research on the causes of road failure in Southwestern Nigeria, a three-dimensional terrain geomorphology and surface water flow pattern is incorporated into the present study. This is with a view to unravel the detrimental effect of moisture on subgrade soils' properties governing design for pavement materials.

Location and Geology

The study area stretches between longitudes 3° 44'E and 3° 52'E, and latitudes 6° 54'N and 6° 56'N, part of Southwestern Nigeria. The parent rocks for the subgrade soils include the crystalline Basement Complex, Precambrian rocks such as gneisses and quartz-schist (Oyinloye, 2011). Adebisi *et al.*, (2014) identified tropical climate in combination with other pedogenic factors, as prominent in the formation of lateritic soils. These rocks were known to have disintegrated and transformed of into soil which have variable thicknesses, geochemical, mineralogical and geotechnical characteristics (Boul *et al.*, 2003). Twenty (20) disturbed and undisturbed samples were obtained between 0.6m and 0.95m depths, along a road under construction between Ago-Iwoye and Ijesha-Ijebu (Fig. 1).

Method of Investigation

The geographical location and ground elevation

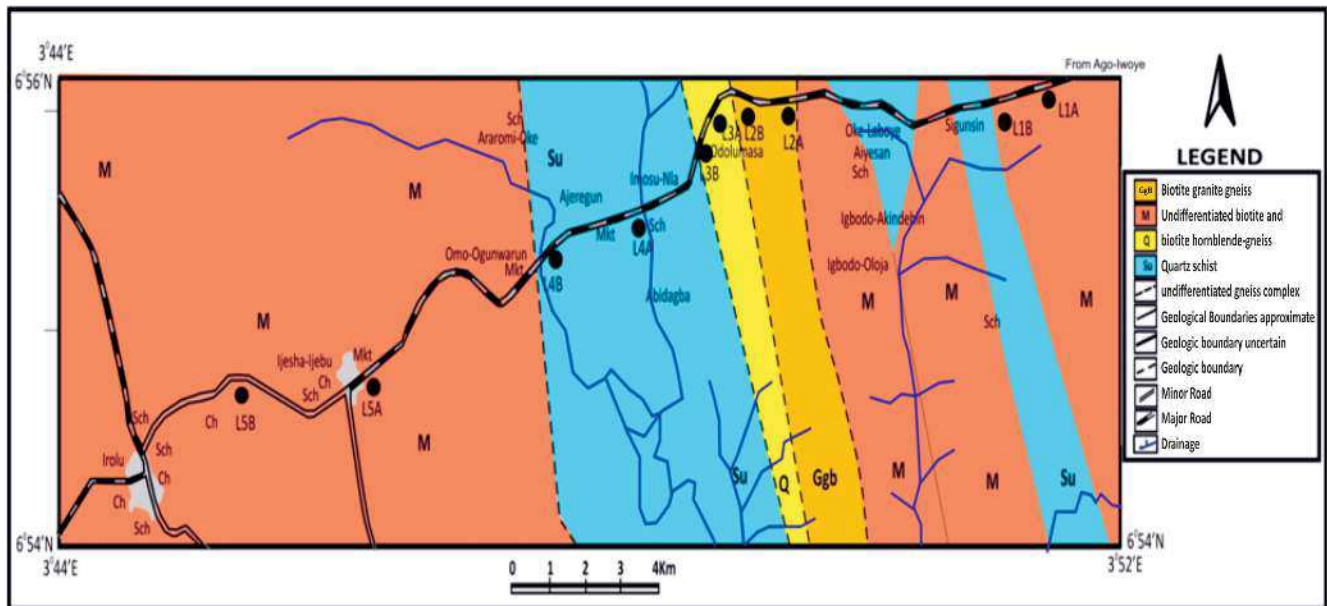


Fig. 1: Map showing the various bedrocks, the road under construction and sample locations.

measurements were simultaneously recorded using the Ground Positioning System (GPS). In this application, contour lines in digital format were generated using 'Suffer 8' software to produce a 3-dimensional vector-based digital image of the area. A drainage pattern layer of the area was created from the existing topographic map, and overlain onto the surface model of the area.

The specific gravity (G_s) of the soils was determined from the ratio of the weight of a given volume of the

soils to the weight of an equal volume of distilled water at 20°C. Coarse soils were shaken in meshes of various sizes, while their fine particles were analysed in a hydrometer through sedimentation to determine complete gradation. Three numbers of blows for closure of clay pat groove were recorded with the corresponding 25th moisture contents, to obtain a flow curve for liquid limit (L_L) determination. The plastic limit (L_P) was determined by moisture content of a thread of a clay pat on a flat plane glass surface. The linear shrinkage (L_S) of

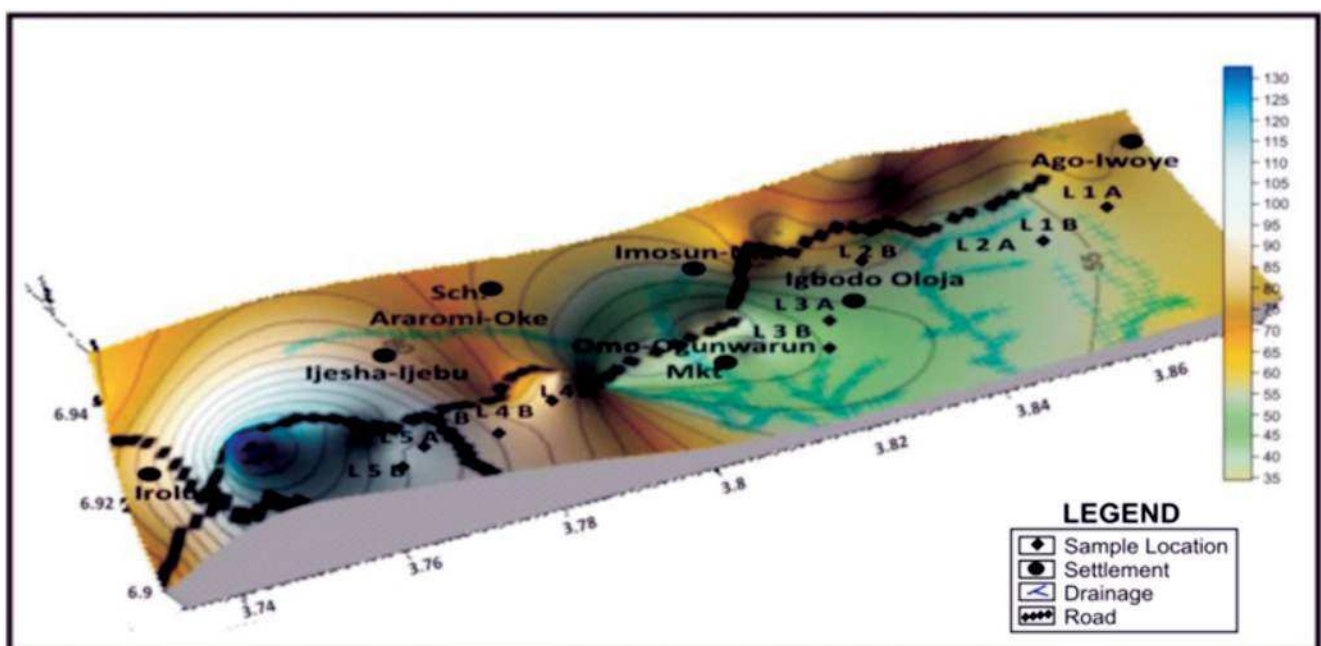


Fig. 2: A three-dimensional vector-based digital image of the study area

the soils was determined from the percentage of the original dimension of the fine soil mass, when the moisture content is reduced from the liquid limit to an oven-dry state in a mould. Compaction test was conducted to determine the soils' optimum moisture content (OMC) and maximum dry density (MDD) at West African level. The OMC and MDD of each sample were used to prepare every specimen for unsoaked California Bearing Ratio (CBR) test, and soaked CBR after 24hrs of soaking. Load was recorded at penetration of 2.5 and 5.0mm. The average CBR for both ends of the moulded specimen was then calculated.

Results and Discussions

Terrain Configuration and Drainage Pattern

A 3-dimensional vector image of the study area (Fig.2), shows a sloping line of high ground between Ijesha-Ijebu and Irolu. A dip along a level ridge crest exists between Ijesha-Ijebu and Omogunwarun Market, with a stretched-out groove in Araromi Oke suspected to be formed by a stream. There is either an almost unnoticeable slope at Imosun Nla, with no area of low ground surrounded by higher ground in all directions. Drainage system in the area is dendritic, which reflects

the nature of topography and associated crystalline bedrocks (Adebisi *et al.*, 2014).

Geotechnical Assessment

The grading curves for the studied soils (Figure 3) are indicative of soils containing a variety of particle sizes. Index and highway geotechnical properties of the soils are summarized in Table 1. Numerically, grain sizes revealed excess and deficiency of medium silt (8-26%) and coarse sand (1-25%) respectively. However, gravel-sized particles were completely missing from few samples. The specific gravity (G_s) ranges from 2.56 to 2.67, and agrees with Swan, (2015) in relation to the size of soil grains. The critical water contents are presented in terms of Atterberg limits with a plasticity chart (Figure4), which shows soils of low to slightly high plastic inorganic kaolinitic silty clay. This is based on plasticity ratio (R) for fair indication of clay mineralogy (Savage, 2007). Based on American Association of State Highway and Transportation Officials (AASHTO) standard, the soils fall into two classification subgroups; plasticity index of A-7-5 subgroup is equal to or less than the liquid limit ($L_L - 30$), and plasticity index of A-7-6 subgroup is greater than the liquid limit ($L_L - 30$).

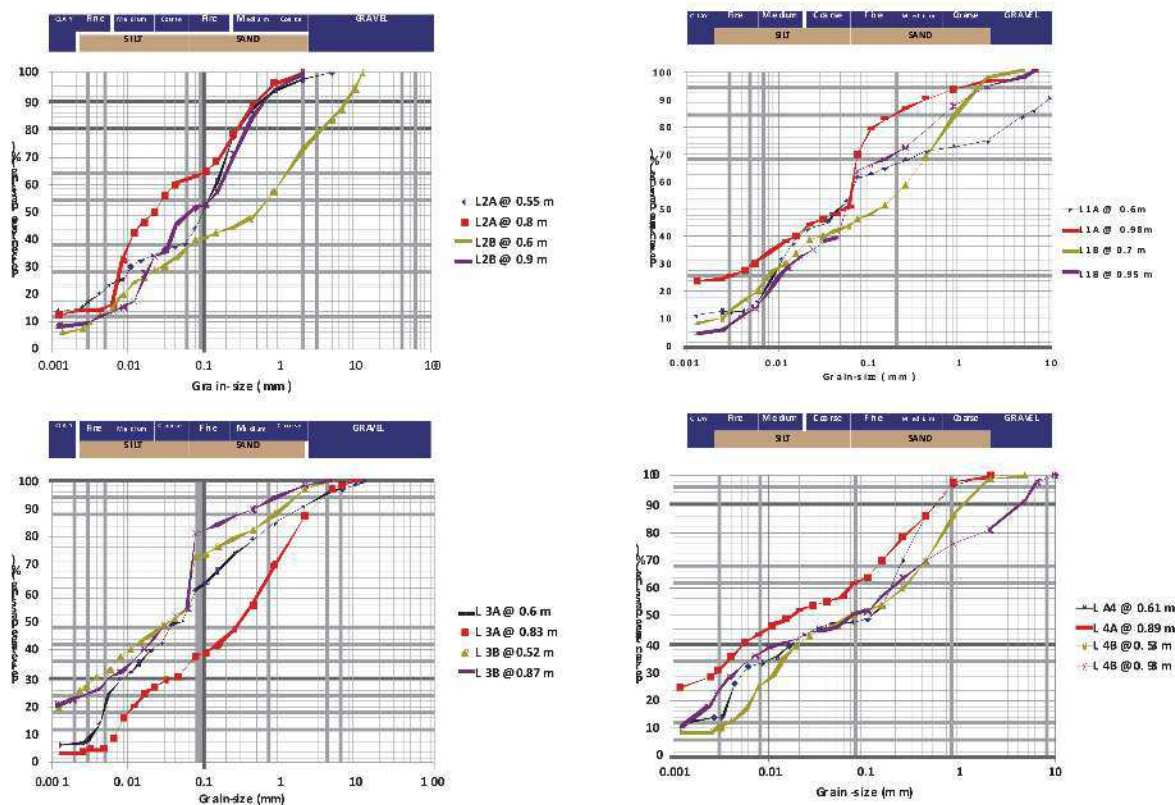


Fig. 3: Grading Curves for Soils obtained from Location 1 to 4.

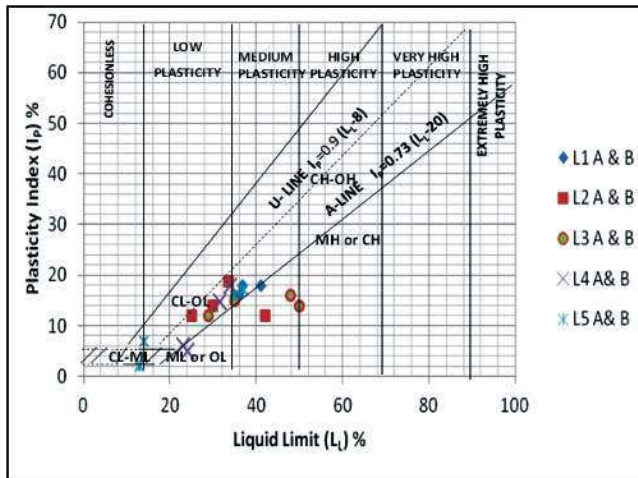


Fig. 4: Casagrande Chart for Soil Classification.

The engineering performance of the subgrade soils is further assessed in terms compaction characteristics and California Bearing Ratio (CBR). The respective maximum dry density (MDD) and optimum moisture content (OMC) ranges are of 1667 - 1954 kg/m³ and 9.5 – 19.0%. Unsoaked CBR ranges from 11.57 to 57.71%, while soaked CBR varies between 6.25 and 22.20%. The maximum dry density increases with decreasing OMCs, both being favourable results (Fig. 5). Considering the CBR values, the FMWH (1997) recommendation of at least 30% and above 80% CBR for unsoaked and soaked samples respectively shows that the soils are weak in terms of strength in most cases.

Conclusions and Recommendation

The drainage pattern developed over the study area is related to the topography, and the nature of the underlying impervious rocks. Surface water in the area would concentrate at the lowest points on slopes, which represent the least desirable locations for the road. The specific gravity, quantity of the size of grains and consistency limits reveal inorganic clayey, tropical iron-rich kaolinitic laterite of sand particles probably with quartz.

Compaction characteristics and California Bearing Ratio confirm that the soils are weak in their ability to withstand vibratory load. On the basis of specified standard, thick pavement is recommended. This will enable spreading of wheel loads over a greater area of

the weak subgrade soils in the stretched-out groove and slopes in the area. This recommendation is a safety measure against soil deformation, which might cause the pavement failure within a short time.

Table 1: Summary of Index and Highway Geotechnical Properties of the Soils.

	RANGE	Minimum	Maximum	Average
GRAIN-SIZE DISTRIBUTION (%)	Clay	4	25	9
	Fine	2	19	12
	Medium	8	26	15
	Coarse	6	18	10
	Fine	6	35	23
	Medium	6	28	8
CRITICAL MOISTURE CONTENTS (%)	Gravel	0	26	5
	Liquid Limit (L _L)	13	48	36
	Plastic Limit (L _P)	15	30	13
	Plasticity Index (I _P)	2	18	23
	Linear Shrinkage (L _s)	4	20	10
	SPECIFIC GRAVITY (G _s)	2.56	2.67	2.62
MOISTURE-DENSITY RELATIONSHIP	PLASTICITY RATIO (R)	1.18	2.13	2.0
	INFERRED CLAY MINERALS	Kaolinite – Illite		
	Optimum Moisture Content (OMC) %	9.5	24.3	21.7
	Maximum Dry Density (MDD) kg/m ³	1459	1954	1837
	Unsoaked CBR	11.67	57.71	46.87
	Soaked CBR	4.59	22.20	12.41

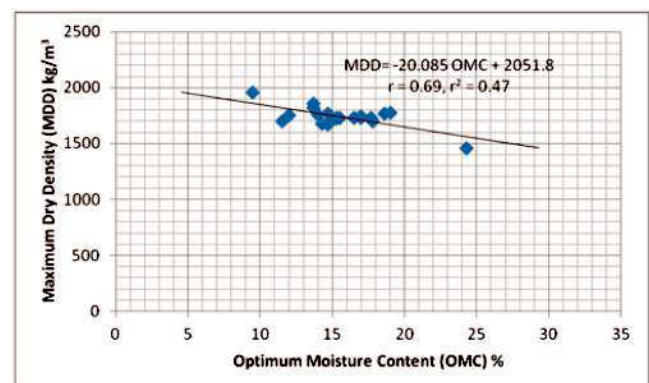


Fig. 5: Linear Regression Plot of Maximum Dry Density and Optimum Moisture Content

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Geophysical Considerations in Subgrade Characterization for Pavement Design – A Case Study of the Warri – Eku Road, Delta State, Nigeria

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Abstract

A combination of electrical resistivity method of geophysical prospecting and conventional geotechnical testing methods was employed in the study of subgrade materials underlying a 30km road route extending from Eku to Warri in Delta State, Nigeria. The purpose of the study was to determine the suitability of the materials as subgrade soils and appraisal of latent causes of pavement failure along the road route. Index property, consistency limits, proctor compaction and California bearing ratio tests were conducted on bulk samples extracted at approximately 1.5 m depth along the road route at intervals of 1 km. Results of the laboratory tests indicate samples meet specified requirements for subgrade materials according to Federal Ministry of Works and Housing. Consequently, it is surmised that latent causes of failure beneath the road route will not be primarily as a result of subgrade materials. Vertical Electrical Soundings using the Schlumberger electrode configuration were conducted at intervals of 1 km along the road route and the result of the interpretation indicates progressive decrease in resistivity values towards the Warri end of the road. This progressive drop is here interpreted as representing effects of groundwater; an assertion that was corroborated by groundwater levels measured within the trial pits. It is therefore believed that groundwater rather than subgrade soils will pose greater challenge to stability of pavements in the study area. Furthermore, the importance of surface geophysical prospecting methods in geotechnical studies is highlighted particularly in reconnaissance investigations.

Keywords: *Subgrade Characterisation, Pavement, Geophysical, Geotechnical, CBR, FMWH, Electrical Resistivity*

Introduction

In recent years, the use of geophysics in geotechnical investigations has been on the increase in Nigeria, particularly, as it relates to its importance for studying causes of road failures. Nevertheless, much of these studies have been concentrated in areas underlain by crystalline rocks where event of fracturing in and fissility of host rocks could pose serious problems to the stability of roads. Little has been done within sedimentary environments, and where they have been studied, the studies are generally confined to regions underlain by clayey deposits and those underlain by variable geology. Furthermore, much of these studies are carried out in the aftermath of road failures to decipher the possible cause of such failure assuming engineering design and construction materials were adequately catered for. In this study, the use of geophysics in geotechnical characterization for road design is embarked upon within a region underlain by coastal sand deposits, to determine potent cause(s) of failure along a road route which had been proposed for dualization.

Site Location and Description

The study route is a 30km road from Warri to Eku in Delta State, Southern Nigeria and lies between geographic coordinates N 050 34.498'; E 0050 48.061'

and N 050 44.175'; E 0050 58.954' (Fig. 1). The road trends approximately NE – SW with average elevation of 6m in the south and 13m in the North. The southern portion of the road falls within a flood plain and as such hosts several water logged portions along the road shoulder.

Geological Setting

Delta State falls within the Niger Delta basin; a major sedimentary basin covering much of the coastal States in Nigeria. Deposits within the basin date from Paleocene to Holocene and deposition has a fining sequence seawards. A similar fining sequence is observed vertically within the basin starting with coarse sandy deposits and ending with marine clays (Odebode, 2013).

Although, the basin consists of five main formations, the study route is underlain by two of the Formations, viz: the Sombreiro Deltaic Plains and the Coastal Plains Sands (Fig. 2). However, owing to similarity in constituents, both Formations have been classified as belonging to a larger group – the Benin Formation (Short and Stauble, 1967). Furthermore, approximately 87% of the road route lies on the Sombreiro Deltaic Plains, therefore, the road route can be considered as being underlain essentially by a single Formation.

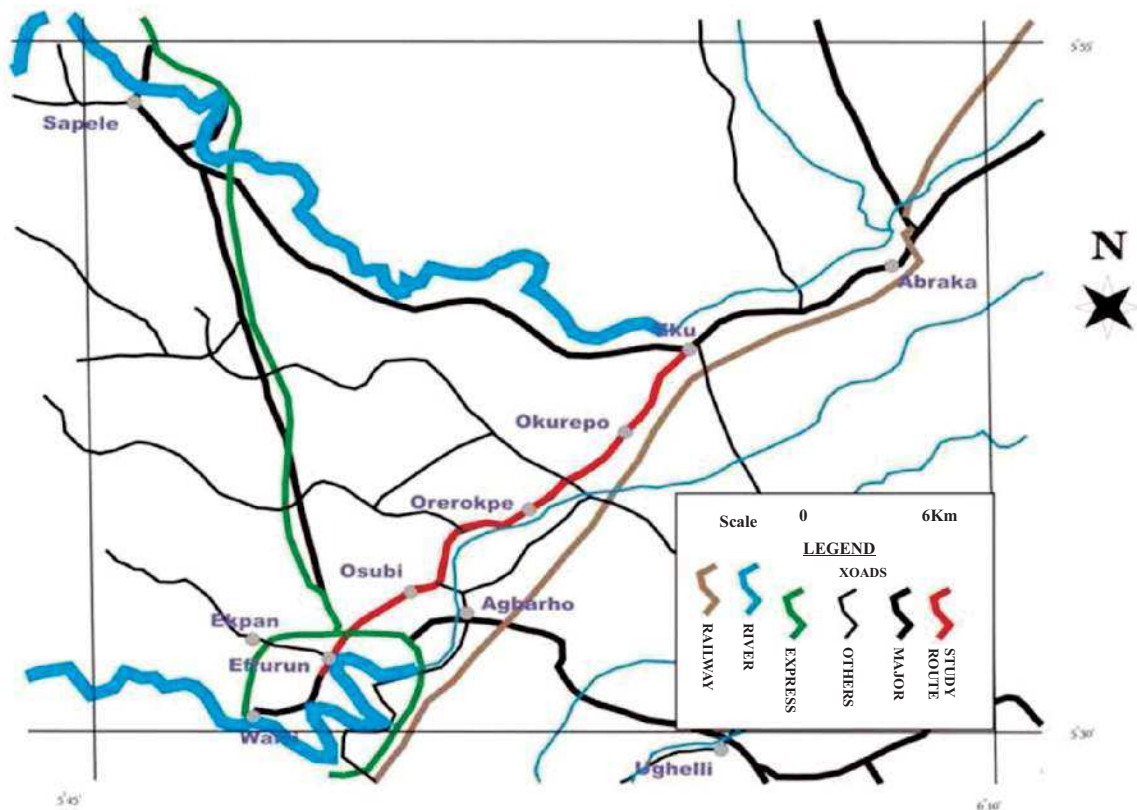


Fig. 1: Drainage along the study road route

Method of Study

Bulk samples from trial pits located at intervals of 1km along the road route were collected for the geotechnical exploration program at a uniform depth of 1.5m. Laboratory tests comprising Grain size analyses, Consistency Limits, Proctor Compaction and California Bearing Ratio (CBR) tests were carried out on the samples. For the geophysical investigation program, the Vertical Electrical Sounding (VES) technique was employed using the Schlumberger Array and sounding stations were made to coincide with trial pit positions; thus, a total of 31 VESs were conducted. The sounding data were obtained using the PASI 16GL Earth Resistivity meter and the sounding conducted while varying current electrode separation between 0.6m and 100m.

The VES data were plotted on bi-Log graphs and the VES curves were interpreted quantitatively and qualitatively by partial curve matching with 2-layer model curves of Orellana and Mooney and a computer aided 1-D forward modeling software; WinResist.

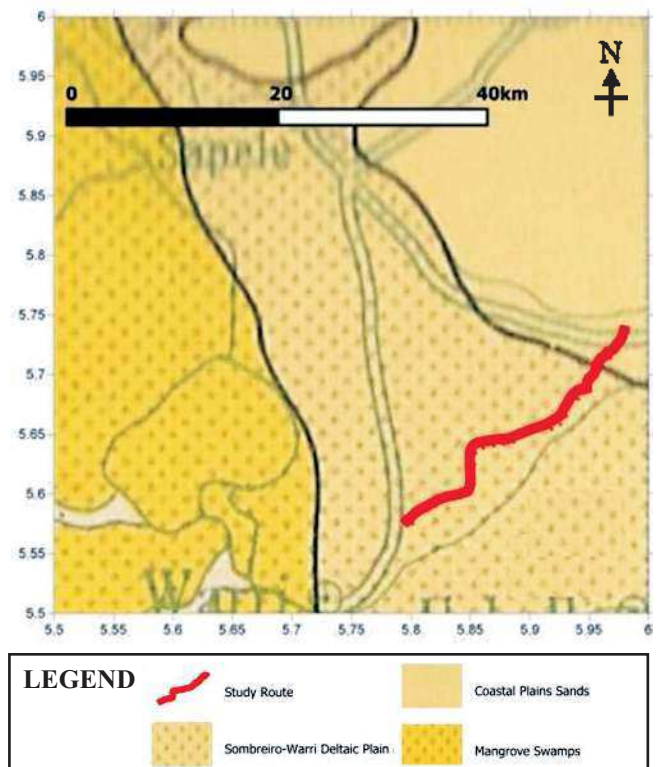


Fig. 2: Geological Map beneath the Study Road Route

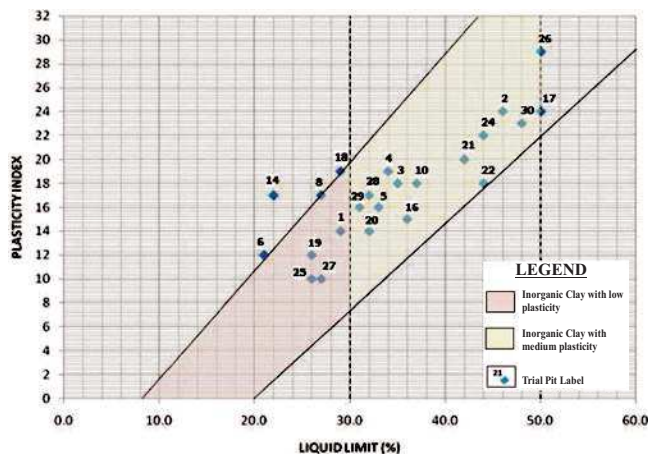


Fig. 3: xxx

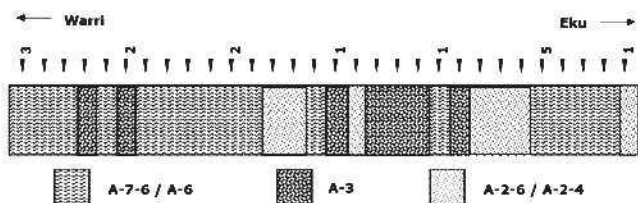


Fig. 4: Soil Type Distribution beneath the Study Road Route

Results and Discussions

Soil Types and Index Properties

50% of samples tested fall classify as fine grained soils having more than 50% of their aggregates constituted of materials with grain size less than 75 μm . 30% of the materials have fines content between 20% and 50% with the remaining 20% of the samples classifying as coarse grained having fines content less than 20%. Atterberg limits tests conducted on the fine grained soils record low plasticity indices generally less than 30% with liquid limits predominantly less than 40% which indicates moderate plasticity for the fine grained soils (Figure 3). Consequently, based on the AASHTO classification scheme, materials underlying approximately 50% of the road are characterized by A-7-6 / A-6 soils which are generally poor as subgrade materials coupled with another 25% of soils characterized by A-2-6 / A-2-4 which are fair to good as subgrade materials. The remaining 25% classify as A-3 which are considered good as subgrade materials (Figure 4). The A-7-6 / A-6 soils are ubiquitous beneath the entire road route with the A-3 soil types concentrated between Chainage 9 and 15.

Soil Activity

Considering the predominance of fine grained soils beneath the road route, activity of the soils, which is

dependent on clay fractions present, could be of significance in their suitability as subgrade materials. Using empirical methods for determination of clay fractions within a given soil after Savage (2007), it was found that although approximately 50% of the soils have more than 50% of their materials finer than 75 μm , clay fractions within the materials rarely exceed 30% (Figure 5). Thus, the soils will better classify as silty clayey sands. Consequently, determination of Skempton's activity for the various soils yields an activity of between 0.5 and 1.3 (Figure 5) which implies the clay fractions in the soil classify as inactive to normal soils. Therefore, failure of subgrade arising from clay activity beneath the road is precluded.

Compaction and Strength Characteristics

Average maximum dry density (MDD) of 1.98 was

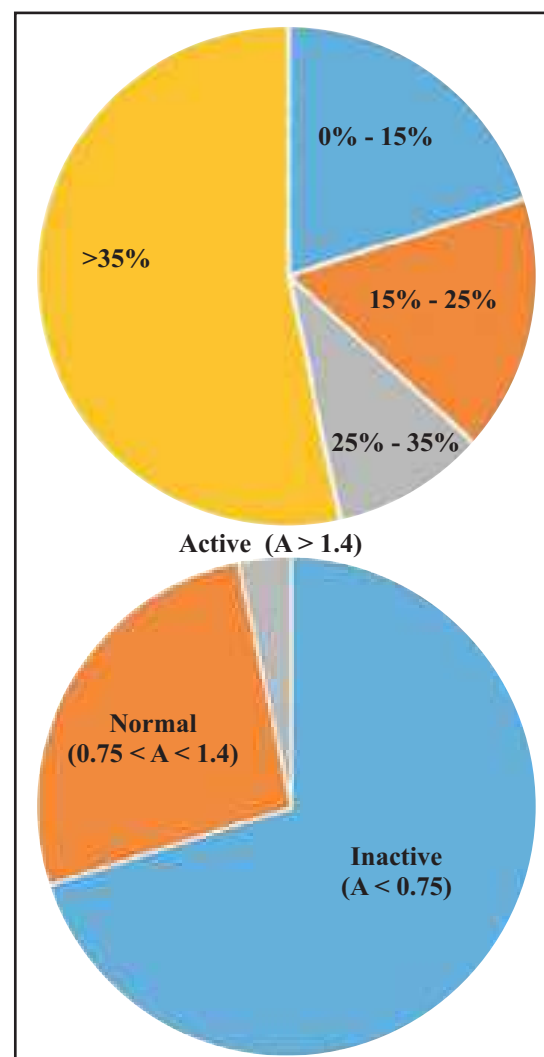


Fig. 5: Percentage Clay Content and Activity beneath the Road Route

recorded for the samples albeit varying OMCs essentially between 10% and 12%. Lower OMCs between 7.1% and 9.8% were recorded for some of the coarse grained samples which classified as A-3 and A-2-6 / A-2-4 soils. Results of CBR tests conducted on the soil samples indicate 50% of the materials have unsoaked CBR values between 30% and 40%, and 80% of the materials have 48 hrs soaked CBR values of 15% and 30% with approximately 3.3% having values less than 15% from the soaked test.

According to clause 6201 of the Federal Ministry of Works and Housing (F.M.W & H) Specification requirement (Olafusi et. al., 2012) the minimum strength of base course material shall not be less than 80% C.B.R. (unsoaked) while for subgrade / fill shall not be less than 10%. Thus, soils underlying the entire road route are considered suitable as subgrade material.

Electrical Sounding

Results of the geophysical investigation indicate similar resistivity structure beneath the sounding stations. However, progressive drop in resistivity values towards the Warri end of the road was observed from a cross section obtained beneath the road route (Figure 6). It is readily observed from the drainage map of the study

area that a stream cuts the road about 2 km from Orerokpe; precisely between VES stations 19 and 18 (Figure 1). The progressive drop in resistivity values coincides with this region and thus, the resistivity drop is here interpreted as indicating groundwater effect; an assertion corroborated by presence of groundwater within trial pits dug on the South-western end of the road route as against the North-eastern end.

Conclusions

Laboratory analyses of soils obtained along the road route indicate that although materials underlying the road route contain significant proportion of fine materials, the F.M.W & H requirement indicate the materials bear significant strength to be used as subgrade materials with maximum dry density ranging between 1.88g/cm³ and 2.2g/cm³. Nevertheless, failure was observed along sections of the road toward the Warri end of the route. Thus, this failure was believed to have been as a result of some other causes other than soils underlying the route. A probable cause of this was shown from the resistivity structure beneath the road route as progressive drop in resistivity structure was observed towards the Warri end. This structure is here interpreted as being indicative of effect of groundwater.

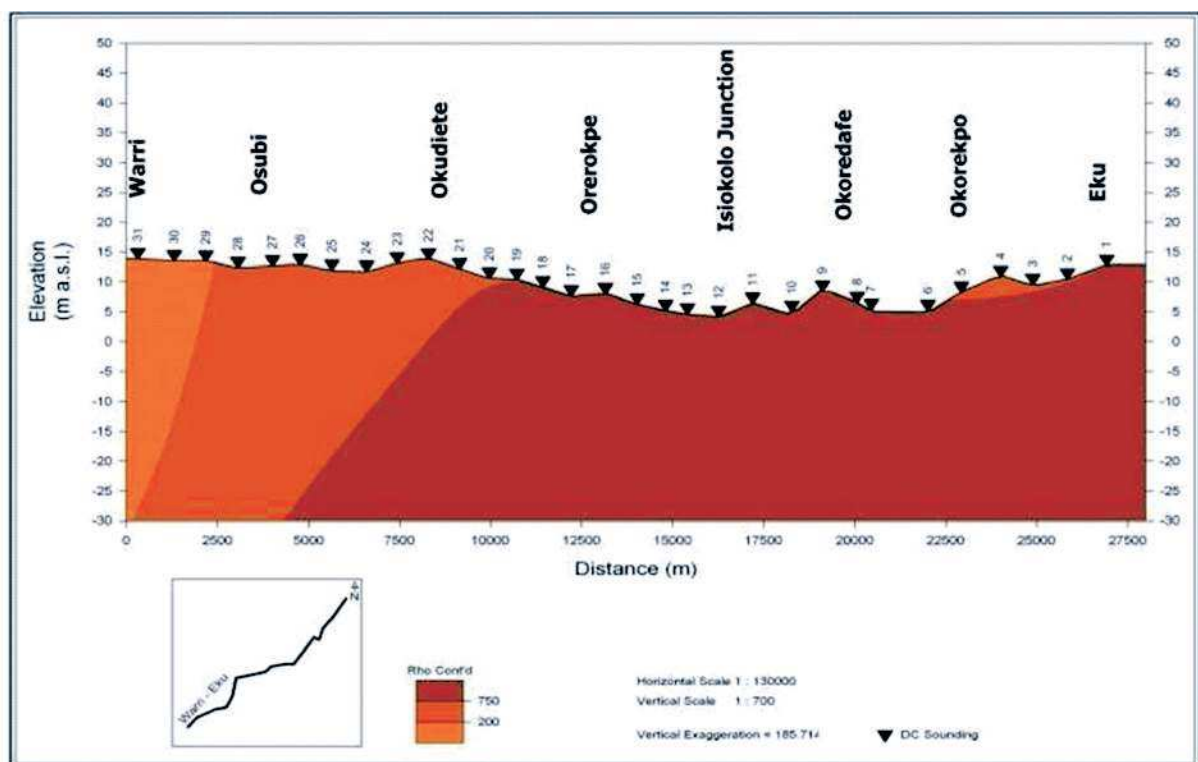


Fig. 6: Resistivity Structure beneath the Study Road Route

Consequently, perhaps in addition to soils being composed of significant clay fractions; although essentially normal to inactive clays, groundwater is a major contributor to failure along the road route and should be appropriately considered in pavement

construction within the study area. Summarily, the use of geophysical tools in geotechnical studies is of immense importance in reconnaissance studies for geotechnical investigations in pavement design.

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Impact of Geology on the Stability of Minna-Lambata Road, NorthCentral Nigeria

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Abstract

The rocks and soils underlying Lambata-Minna road and the neighborhood in central Nigeria were mapped and subjected to various analyses in order to determine their impact on the stability of the road they underlay. Vertical electrical soundings were done along the road to determine the soil profile of the road. The geoelectric studies revealed that the soil profiles of the road are composed of laterite, sand and clayey soils. Road traffic statistics revealed that the traffic density road is within the permissible limit. A total of 5 rock samples were collected and each subjected to thin section, XRD and XRF analyses. Groundwater table of wells along the road was observed to determine the variation in seasonal groundwater variation. 35 water samples were collected from wells along the road and subjected to hydrochemical. A total of 47 soil samples were collected and subjected geotechnical analyses. The field mapping, thin section and XRD revealed the road is underlain by migmatites, gneisses, granites, marble, granodiorite and schist. The groundwater occurring within migmatite terrain has higher ionic concentration and physical properties. The soils are gravelly soils. Low permeability ranges from 4.73×10^{-4} cm/s to 9.78×10^{-3} cm/s were obtained. Optimum moisture content ranges from 18 to 27%. The soaked CBR of soils ranges from 0.9 to 70% while the unsoaked CBR ranges from 5 to 70%. The soils underlying the granites are generally more competent than those underlying the migmatite. The consistent failure of the Lambata-Minna road portion underlain by migmatite gneiss and granite is because the soils occurring within those terrains have poor geotechnical properties to serve as either sub-grade, sub-base or base material in their natural state.

Keywords: *Geology, Stability, Lambata-Minna, Road, hydrochemical*

Introduction

A lot of money and time have been spent by the Nigerian government on the maintenance of Lambata- Minna road. Yet, the road is still unstable. The road condition is worse where it underlay crystalline rocks that are partly weathered to clay minerals. Adequate attention on the understanding of the geology that underlies this road is therefore necessary. This is because the road pavement requires sound and durable support from its subsurface material (underlay geologic material) (Okogbue. 1988). In choosing the route for the highway, one of the important factors considered is the geotechnical properties of the sub-base and subgrade material. When design technique, construction procedure and materials used for road are adequate, and the road still experiences failure, the geological, mineralogical, geotechnical, soil, climate and drainage conditions become critical.

This road has continued to experience failure after construction, and therefore subjected to frequent maintenance. This research was carried out to determine the possible geologic factors responsible for the road failure. Gidigas, (1972) have attributed the majority of road pavement failures in the tropics to mineralogical and geotechnical factors. Ajayi (1982) noted that the

road failure he studied occurred where the pavement was founded on saprolite rather than the strong lateritic horizons. Adeyemi (1992) noted that the degree of stability of the road he studied increased with the amount of kaolinite present in the subgrade soil and with increase in the California Bearing Ratio (CBR) and unconfined compressive strength (UCS) of the sub-grade soils. Adeyemi and Oyeyemi (2000), have noted that the soils below the stable section of the Lagos-Ibadan expressway have higher maximum dry density, unsoaked California Bearing Ratio (CBR), uncured unconfined compressive strength than those below the unstable section. They also noted that the soils below the stable portions have a lower proportion of fines and clay sized fraction and a lower optimum moisture content and linear shrinkage than the soils below the unstable section. Jegede and Oguniyi (2004) attributed the incidence of highway pavement failures for Nigerian highways to improperly compacted edges of the pavements to non- provision of drainage facility along the roads, and low California Bearing values among others.

Some other workers like those of Weinert (1960) Farquhar (1980), Gidigas (1974), Mesida (1987), Ajayi (1987), Ayangade (1992) and Adeyemi

(1992) have shown that the majority of highway failures can be attributed to geological, geotechnical and hydrogeological factors.

Clare and Beaven (1962), and Okogbue and Uma (1988) have observed that the patterns of pavement performance in West Africa are considerably controlled by geology, topography, soil and drainage conditions. Gidigas (1983) and Okogbue and Uma (1988), have noted that depth to water table appears to be the most dominant of the climatic, topographic and drainage factors that affect pavement performance. According to them, when the water table is at the depth of less than 1m, the chances of failure seem to be highly independent of the climate and other environmental conditions.

The Study Area

The study area is located along the Lambata-Minna Road. It lies along longitude (06°31'32.4"E) to (06°22'53.8"E) and latitude (09°35'44.4" N) to (09° 05' 38.3"N). The rock types mapped included fine to medium grained biotite granite, granite gneiss, migmatite, marble and schist. The granite has been affected by the Pan African Orogeny with late tectonic emplacement of granites and granodiorites. The end of the orogeny was marked by faulting and fracturing according to Abaa, (1983; Gandu *et al.*, 1986; Olayinka, 1992). The granites are thus fractured, jointed and deeply weathered in some places. The area has an average annual rainfall of about 1,000mm. Rainy season usually starts during the month of April, and reaches the peak by August and ends in October. Low average temperature of 24°C is recorded between the months of July and September while high temperature is usually recorded during the months of February to April, at an average of 35°C. The harmattan wind is experienced between December and January.

Method of Investigation

Geological field mapping and soil profile study were performed. Fresh rock samples were collected with the aid of geological hammers and carefully labeled for chemical and mineralogic analysis. Trial pits were dug from where soil samples were collected for geotechnical studies. Vertical electrical sounding (VES) was conducted in order to obtain information on the character of the subsurface formations along the road. Groundwater fluctuation that may affect moisture content of subgrade was monitored. Static water levels

were measured from hand-dug wells located very close to the road during the peak of the dry (April) and rainy (September) seasons. The depths of the wells (D), dry (A) and wet seasons (B) water depths were measured from which the groundwater fluctuation and percentage of variation ($E = \frac{B}{A} \times 100$) were obtained.

The thin section preparation for petrographic analysis was carried out on four rock samples using the Hiiquist Models 1005 and 1010, for cutting and polishing, respectively for the mineralogy. Representative bulk samples were crushed milled and homogenized to fine powder of approximately 10 – 15 µm size, pressed into pellets and then analyzed. The Philips semi-automatic X-ray spectrometer (PANalytical Axios WDXRF) machine was used for the X-Ray Fluorescence analysis in order to know the chemical composition of the rocks. X-Ray Diffractogram was carried out on four rock sample using PW goniometer supply diffractometer with CuKα radiation generation at 40KV and 30mA as explained by Velde (1992) and Brindley and Brown (1980) to also know their mineralogic composition.

The wet sieving method was employed during this test as specified in BS 1377 (1990) in order to know the grain sizes of the soils. Atterberg Limits tests conducted are liquid limit (LL) and plastic limit (PL) using the cone penetrometer method as described by Brain (1978). The liquid and plastic limits were each determined with about 300 g of soil samples passing 0.425mm sieve in accordance with BS 1377 (1990) so as to know the plasticity of the soils. The constant head permeameter method was employed for the determination of the permeability in this study. Compaction tests were done using the Standard Proctor mould (4.5 kg rammer) while CBR were performed on samples at their maximum dry densities in accordance with the West African Standard, ASTM D1883 and AASHTO T193 in order to identify the engineering properties of the soils.

Water samples (35) were analyzed for anions and cations as well as their physical parameters in order to relate the ionic concentration to weathering and the stability of the road. Colorimetric, Titrimetry and Flame Photometry (Sherwood Flame Photometry) methods were employed using HACH DR/890 Colorimeter, Sherwood Analog Colorimeter Model 252 as described by Idris-Nda (2010).

Results and Discussion

The rock types mapped included fine to medium grained

biotite granite, granite gneiss, migmatite, marble and schist. The granite has been affected by the Pan African

Orogeny with late tectonic emplacement of granites and granodiorites (figure 1a).

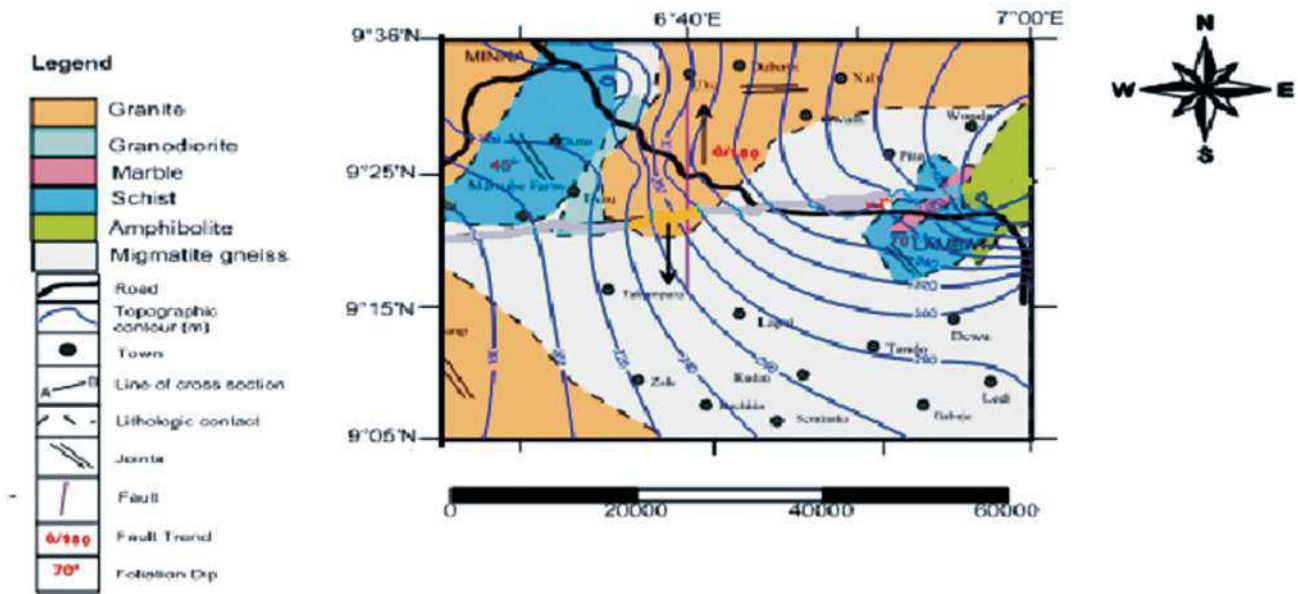


Fig. 1a: Geological map of the study area

The inter-seasonal water table variation varies according to the rock type is presented in figure 1b.

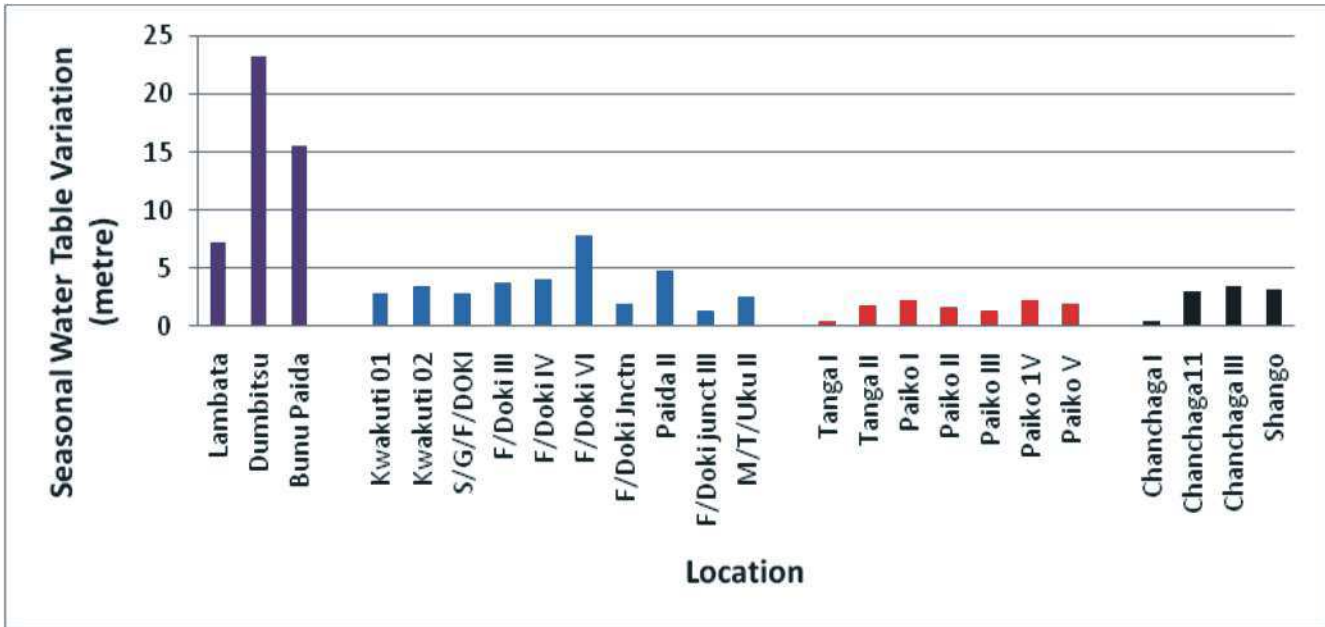


Fig. 1b: Inter-Seasonal water table variation among the observed water wells according to the rock types along Lambata-MinnaRoad (from the left, 1st batch = migmatites; 2nd = Gneiss; 3rd = Granite; 4th = Schist)

Figure 1b shows that the, migmatites have the highest inter-seasonal variation than the granite indicating that permeability also vary with the different rock types

along the road. The fracturing of the granites could be responsible for the lower inter-seasonal water variation observed within the granites.

Four geo-electric layers have been identified within the granitic rocks (Figures 2 b) namely laterite, sand, weathered and fresh basement, while five geo-electric layers namely laterite, sand, clay, weathered and fresh

basement occur within the migmatite (figure 2a and 2c). The high soil/weathered zone spatial depth variation can be attributed to the extensive fracturing and weathering of the underlying rocks.

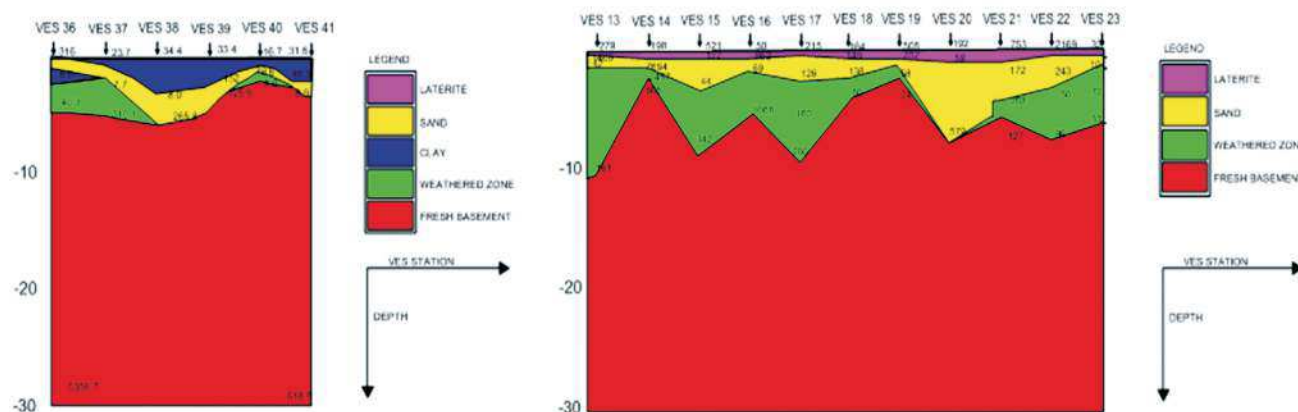


Fig. 2. a & b: Geo-electric section of VES L36 - L41 and L13 - L23 along Lambata-Minna Road

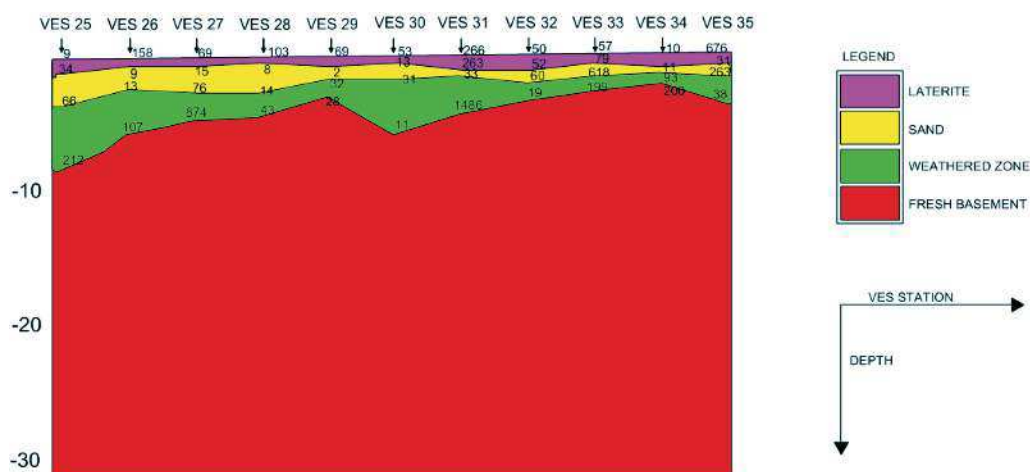


Fig. 2c: Geo-electric sections of VES a L36 - L41 b L13 - L23 and c L25 to L35 along Lambata-Minna Road

Table 1a is the result of the X-Ray Diffraction and photomicrograph of the rocks while the chemical composition is presented on table 1b.

Table 1a shows that the studied rocks consist of quartz, biotite, muscovite, plagioclase, graphite, and opaque minerals. The abundant oxides in the basement rocks along the road are in the order $\text{SiO}_2 > \text{Al}_2\text{O}_3 > \text{TiO}_2 > \text{Fe}_2\text{O}_3 > \text{CaO} > \text{K}_2\text{O} > \text{MgO} > \text{Na}_2\text{O}$ (table 1b). The dominant cations from the analyzed water samples are Ca^{2+} , Mg^{2+} and Na^+ with an average of 19.85, 14.82 and 9.19 mg/l respectively while the dominant anions are Cl^- and SO_4^{2-} with average of 49.81 and 5.2.8 respectively (figure 2d).

The pH of the water samples ranges from 5.4 to 9.64; the electric conductivity ranges from 13 to 1,113 $\mu\text{S}/\text{cm}$; the total dissolved solids (TDS) ranges from 8.71 to 754.7 mg/l and the hardness ranges from 10 to 502 mg/l. It can be seen from Figure 2d that the concentrations of these ions and cations vary with the underlying lithology. Water samples collected from the schist/gneiss (metamorphic) terrain have higher ionic and cation concentration than those collected from the granitic terrain. This is attributed to the higher weatherability of these metamorphic rocks relative to the granitic rocks as well as to their high composition of minerals like biotite, cordierite, hornblende and epidote that are rich in Mg^{2+} , Ca^{2+} and/or Na^+ . This agrees with

Table 1a: X-ray diffraction and photomicrograph results of rocks and soil samples collected along Lambata-Minna Road

SAMPLE	LOCATION		X-RAY DIFFRACTION MINERALS	PHOTOMICROGRAPHY ANALYSES		INFERRED ROCK NAME
	Longitude	Latitude		Minerals (%)	Texture	
A	07°00'08.5"	09°21'28.7"	Quartz, Albite, Cristobalite (quartz), Siderophilite	Biotite (45), Plagioclase feldspar (30), Muscovite (10), Quartz (10), Opaque mineral (5)	Granular	Gneiss
E	07°42'42.5"	09°25'56.8"	Graphite, Annite (Biotite), Nacaphite, Berlimite(phosphate)	Biotite (40), plagioclase feldspar (20), Homblende (15), Quartz (10), Accessory minerals (15), Opaque minerals (10)	Granular	Gneiss
F	06°38'08"	09°25'56.8"	Quartz, Muscovite, Albite, Wulfenite	Quartz (35), Biotite (25), Homblende (25), Opaque minerals (10), Accessory minerals (5)	Equigranular and poikilitic	Granite
H (soil)			Quartz, Baumite			
D	06°34'55.0"	09°31'28.3"	Zn-Al silicate, Nacaphite, Feriwinchite	Biotite (30), muscovite (20), Cordierite (20), Quartz (20), Sericite (10)		Schist
G	06°34'55.0"	09°31'28.3"	Zn-Al silicate, Nacaphite, Feriwinchite	Biotite (50), Muscovite (20), Quartz (15), Cordierite (10), Epidote (10)	Lipidoblastic	Schist

Table 1b: Major oxides of rock along Lambata-Minna road

Sample No.	Major Oxides (pp)									
	SiO ₂	AlO ₃	K ₂ O	Na ₂ O	CaO	MgO	TiO	MnO	Fe ₂ O ₃	Total
F	70.2	2.78	6.73	3.61	2.49	1.2	0.94	0.16	8.17	96.8
A	61.7	13	3.61	0.78	6.03	0.91	1.41	0.10	10.52	98.68
D	51.2	15	1.13	0.34	8.23	1.14	1.03	0.27	19.08	98.62
E	49.6	9.61	2.32	0.68	12.4	1.48	2.5	0.23	19.08	98.91
G	53.8	14.01	5.15	1.03	3	0.88	1.99	0.18	17.97	98.71

works of Gidigas (1974), Transport and Road Laboratory (1974) and Offodile(2002) that ions occurring in groundwater of basement rocks result from the weathering of the basement rocks. It is generally believed that the nature of the parent rock controls the pH in the soil and water (Kovalevsky, 2004). These higher concentrations of these cations (Mg^{2+} , Ca^{2+} and/or Na^+) clearly explain why the samples have higher electrical conductivity and hardness. The neutral to acidic medium nature of most of the studied groundwater supports the weathering in the study area. Higher EC and TDS values as well as higher concentrations of magnesium, calcium, sodium and potassium are recorded in the waters from the migmatitic than from the granitic environment.

The soil profiles examined from the excavated trial pits along the studied highway show variability in soil types both vertically and laterally except for the topsoil, which is generally homogeneous. Visual inspection suggests that the soil is composed of fine to medium grained sand, clayey sand, sandy clays and silty sandy clays while medium to coarse grained materials were also

observed in some profiles. Some of the soil horizons contain fragments suspected to originate from gneiss and schist. In some instances, the rocks have weathered into thick saprolite (tropical clayey soil formed from weathering of igneous/metamorphic rocks). The clayey materials swell on absorbing water during the wet season and shrink during the dry season forming cracks that can extend beyond one meter downwards. This seasonal swelling and shrinkage lead to seasonal differential settlement of the road, which can also result to road instability especially where the road base cuts through the mottled horizon. The heterogeneity of the soil profile is a reflection of their parent rocks. The results of the geotechnical analyses is superimposed on the geology and shown in figure 3.

From Figure 3 it can be seen that the studied road is generally underlain by gravelly (GW and GP) and sandy (SW and SP) soils, it can also be seen that the spatial distribution of the soil type is to an extent controlled by the underlying geology. The portions of the road underlain by granite and gneiss, which are coarse grained rocks, are mostly composed of gravelly soils

while the portions underlain by marble and schist, which are medium grained rocks, are composed for gravelly and sandy soils. These indicate that these soils are actually residuals that originated from the underlying lithology of the studied road. The soils occurring within the migmatite have higher optimum moisture content (OMC) and lower maximum dry density (MDD) than soils derived from the granites but there is no clear difference in the permeability of the soils derived from the different rock along the road. Soils derived from granite have higher California Bearing Ratio (CBR) than of soil collected from the migmatite environment. The results also show that the CBR, OMC and MDD are controlled by the geology. The possible causes of failure of the studied roads are discussed based on the suitability of analyzed soil samples, underlain by different lithologies, for the stated road sections as specified by the Federal Ministry of Works and Housing, 1970. The suitability of the soil samples are summarized in Tables 2a to 2b.

Table 2a: Suitability of soils underlain by granite and granodiorite along Lambata-Minna road (assessed based on Federal Ministry of Works and Housing (FMWH, 1970) specification) for different road sections

Road section	Property	Soil samples underlain by Granite										Granodiorite		Suitability percent (scale of 100.00)
		13	15	16	18	19	21	24	27	30	31	33	36	
For Fill & road Sub-grade	Soil type (USCS)	GW-GC	SM-SC	GP-GM	SM-SC	SM-SC	GP	SP-SM	SW-SM	GW	SC-SM	GW	GC	
	% of fines	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Not suitable	Suitable	Suitable	91.67
	Liquid limit (%)	Suitable	Suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Suitable	Suitable	Suitable	Not suitable	Not suitable	41.67
	Plasticity index (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	0.00
	Soaked CBR (%)	Suitable	Not suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Not suitable	Not suitable	75.00
	OMC (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Suitable	Suitable	Not suitable	Not suitable	Suitable	Not suitable	Not suitable	25.00
For sub-base	MDD (mg m ⁻³)	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	100.00
	Liquid limit (%)	Suitable	Suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Suitable	Suitable	Suitable	Not suitable	Not suitable	41.67
	Plasticity index (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	0.00
	*Soaked CBR (%)	Suitable	Not suitable	Suitable	Not suitable	Not suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Not suitable	Not suitable	58.33
For Base course	% of fines	Suitable	Suitable	Suitable	Suitable	Suitable	Not suitable	Not suitable	Suitable	Not suitable	Not suitable	Suitable	Not suitable	58.33
	Liquid limit (%)	Suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Suitable	Suitable	Suitable	Not suitable	Not suitable	33.33
	Plasticity index (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	0.00
	Unsoaked CBR (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	0.00

USCS=Unified Soil Classification System; MDD=Maximum Dry Density; OMC=Optimum Moisture Content; * = West African Standard; CBR=California Bearing Ratio; % of fines=% less than 75µm

Table 2b: Suitability of soils underlain by *migmatite gneiss* and *schist* along Lambata-Minna road (assessed based on Federal Ministry of Works and Housing (FMWH, 1970) specification) for different road sections

Road Section	Property	Soil samples underlain by Migmatite Gneiss										Schist	Suitability percent (scale of 100.00)
		1b	4b	10b	15b	20b	2	4	7	10	39		
For Fill & road Sub-grade	Soil type (USCS)	GW-GM	SW-SM	GW	SP-SM	SW-SM	GW-GM	GW-GM	GP	GW-GC	SM		
	% of fines	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable		100.00
	Liquid limit (%)	Not suitable	Not suitable	Suitable	Not analyzed	Suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable		10.00
	Plasticity index (%)	Unsuitable	Unsuitable	Unsuitable	Not analyzed	Unsuitable	Unsuitable	Unsuitable	Unsuitable	Unsuitable	Unsuitable		0.00
	Soaked CBR (%)	Not suitable	Not suitable	Not suitable	Suitable	Suitable	Suitable	Not suitable	Suitable	Not suitable	Suitable		50.00
	OMC (%)	Not suitable	Suitable	Suitable	Marginal suitable	Suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable		30.00
For sub-base course	MDD (mg m ⁻³)	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable	Suitable		100.00
	Liquid limit (%)	Not suitable	Not suitable	Not suitable	Not analyzed	Suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable		10.00
	Plasticity index (%)	Not suitable	Not suitable	Not suitable	Not analyzed	Not suitable	Suitable	Suitable	Not suitable	Suitable	Not suitable		30.00
	*Soaked CBR (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable		0.00
For base course	% of fines	Suitable	Suitable	Not suitable	Suitable	Suitable	Suitable	Suitable	Not suitable	Suitable	Not suitable		70.00
	Liquid limit (%)	Not suitable	Not suitable	Not suitable	Not analyzed	Not suitable	Not suitable	Not suitable	Suitable	Not suitable	Not suitable		10.00
	Plasticity index (%)	Not suitable	Not suitable	Not suitable	Not analyzed	Not suitable	Not suitable	Not suitable	Not suitable	Suitable	Not suitable		10.00
	Unsoaked CBR (%)	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable	Not suitable		0.00

USCS=Unified Soil Classification System; MDD=Maximum Dry Density; OMC=Optimum Moisture Content; *=West African Standard;

CBR=California Bearing Ratio; % of fines=% less than 75µm

Table 2a shows that none of the 12 analyzed soil samples occurring within the granite and granodiorite terrain along Lambata-Minna road have plasticity index (PI) suitable for any of the road sections (sub-grade, sub-base or base) while less than half (41.67, 41.67 and 33.33%) of the analyzed samples have liquid limits (LL) adequate to serve as road sub-grade, sub-base or base material. Only 25% of samples have optimum moisture content suitable for sub-grade. 75% of the samples have California bearing ratio (CBR) suitable for sub-grade; 58% have CBR suitable for sub-base while none (0.00%) have CBR suitable for base material. About half (58.33%) of the samples contain the proper amount of fines needed in base materials. It implies that the soils occurring at this portion of the road (granite and granodiorite terrain) cannot satisfactorily serve as sub-grade, sub-base or base course materials in their natural state. This explains the cause of the constant road failure at this portion of the road.

The non-suitability of the soil for any of the road section may be due to the weathering of orthoclase and plagioclase feldspar content of their parent rocks (granite and granodiorite), during the soil formation, to less stable minerals. As it can be seen from Table 2a that the 2 soil sample, occurring in granodiorite terrain, showed non-suitable CBR relative to those occurring in granite terrain, it suggests that residual soils forming from sodic plagioclase-rich rocks (granodiorite) are less competent than those forming from orthoclase/microcline-rich rocks (granite). This is reflected in the poor geotechnical properties of soils that formed from migmatite gneiss (see Table 2b) that is rich in plagioclase. Only 5 samples (50%) in migmatite gneiss terrain along Lambata-Minna road have CBR suitable for road sub-grade while none (0.00%) have CBR suitable for either road sub-base or base. Only 1 sample (10%) each have LL suitable for road sub-grade, sub-base or base. The PI of the soil is also inadequate for any of the road sections. Again, the soil underlain by migmatite gneiss cannot satisfactorily serve as road sub-grade, sub-base or base in its natural state which also explains the cause of the road portion underlain by migmatite gneiss along the studied road Gidigas (1972), Federal Ministry of Works and Housing (1970).

The soil occurring within the migmatite gneiss can serve better as sub-grade on the addition of stabilizers like lime, marble dust or limestone ash (Okagbue and Yakubu, 1997; Okagbue and Onyeobi, 1999) in correct proportions to improve the CBR, LL and PI of the soil while the addition of the stated stabilizers to soil underlain by the granites will enable the soil to serve better as sub-grade and sub-base. Stabilizing the soils within the migmatite gneiss terrain with the above stated

substances to serve as either sub-base or serve as road base may not be economical. This is because the cost of stabilizing such soil will be more than the cost of replacing the soil.

Conclusion

The groundwater collected from wells within the migmatite terrain along Lambata-Minna road has the highest ionic concentration and groundwater fluctuation. Soils occurring within the granitic environment are more competent than soils occurring within the migmatitic rocks implying that soils derived from granitic rocks are more stable than those derived from migmatitic rocks. Portions of the road underlain by granitic rocks are the most stable than portions underlain by migmatite. The cause of the constant failure of Lambata-Minna road is because most soils occurring within the migmatite gneiss and fractured granite terrain of this road is affected by groundwater and has resulted in the poor geotechnical properties. The soil cannot therefore satisfactorily serve as road sub-grade, sub-base and/or base course in their natural state.

Recommendation

The soil can serve as an effective sub-grade when mixed with correct proportion of stabilizers like lime, wood ash or marble dust. On the long run, it will be economical when compared with huge maintenance budget being wasted on it. These results and recommendations are of common applicability in areas with similar lithologic compositions.

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Geophysical, Geotechnical and Water Quality Assessments of Kogi State Designated Dumpsite, Felele, Nigeria

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Abstract

Geophysical, geotechnical, water quality studies within and around Kogi state designated dumpsite at Felele in Lokoja, southwestern Nigeria have been carried out. This is with a view to investigate the feasibility of upgrading the open dumpsite to a modern sanitary landfill. 15 vertical electrical soundings were carried out to geoelectrically characterized the site. 15 soil samples and five leachate water samples were analyzed to determine their geotechnical and physicochemical characteristics respectively. Results show that depth to basement varies between 8.5 and 13.0m. Pseudo-section shows leachate vertical migration to a depth of about 20m and lateral migration to the western region as favoured by the slope direction. The soils are fair to poorly graded gravelly sandy clay, medium to high plasticity having normal to active clay type. The coefficient of permeability ranges between 10^{-7} and 10^{-5} m/s indicating susceptibility to migration of leachate to contaminate the subsurface water body. Slight elevation in the concentration of some of the heavy metals within the site poses danger to quality of neighboring water bodies, thus requiring urgent attention. In conclusion, that the dumpsite does not possess the required requirements for upgrade to a modern sanitary landfill.

Keywords: Heavy Metal, Landfill, Lokoja, Geotechnical, Dumpsite, Leachate

Introduction

Increase in population has resulted in high demand for goods and services thereby leading to increased urbanization and industrialization has the environment to dumping site for waste materials. Historically, dumpsite in many places around the world has remained the most common method of un-organized waste disposal (Abdus-Salam *et al.*, 2011). This is because they are neither lined nor prepare the basement prepared for absorption of toxic substances thereby making the ecosystem (air, water and soil) victim of released leachate. Cost, lack of enforcement of relevant laws on waste disposal, poor regional and urban planning etc have contributed greatly to the presence of dumpsites within towns and cities areas. Thus, this research is aimed at assessing impact of leachate generated from an open dumpsite in Lokoja, Kogi state and possible conversion into a modern sanitary landfill.

The investigated site is located along the Federal highway close to Felele at the entrance of Lokoja in Kogi state, Nigeria (Fig. 1). The site marks the contact between basement complex and sedimentary rocks in the southern Bida basin area. It is located between latitude $7^{\circ}49'$ North of the equator and longitude $6^{\circ}44'$ East of Greenwich meridian at elevation 53meters above sea level. The climate is characterized by wet and dry season and the annual rainfall is between 1026mm and 1524mm with the mean annual temperature of 27°C (Alabi 2009). It has a population of 100,573 males and

94,688 female (National Population Census, 2006). The town typifies an un-planned urbanization and indiscriminate waste depo-center which has resulted to incessant dumping of the generated wastes to river banks, open spaces, road sides and un-engineered site.

Materials and Methods

15 vertical electrical soundings were carried out along three profiles across the site using Schlumberger array. Resistivity values were iterated and curve matched using the IPI2WIN software. Also, 15 soil samples, involving 13 within site and 2 control (Fig. 2) were collected into polythene bags differently through ground pitting at depths range from 0.6 -6.3m and analysed at the Yaba College of Technology, Lagos, Nigeria in accordance with the BSI 1377(1990). Furthermore, eight leachate samples, involving five within site and three control water samples (Table 1) were also collected at depths range from 0-2.28m depth into clean plastic bottles. They were preserved with concentrated HNO_3 which is non-complexing and slightly oxidizing acid. The determination of all physico-chemical parameters was carried out according to APHA(2005).

Results and Discussions

Geophysical survey

Results of the VES data set were interpreted to reveal

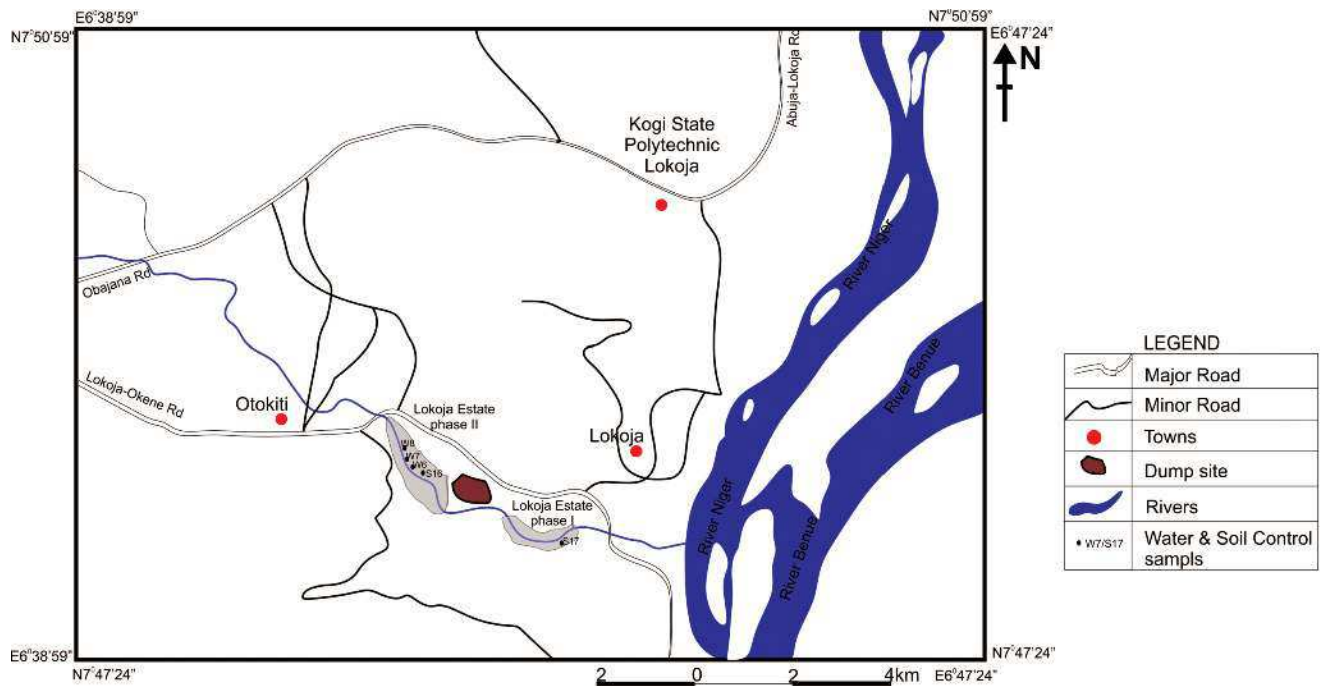


Fig. 1: Site location map of the studied dumpsite

depth to basement ranging between 8.5m and 13.0m with highest depth been observed along profile 1 (Fig. 2 and 3). It also shows three to four geoelectric layers which are inferred to be the top soil layer, lateritic/clay layer, weathered rock layer and the fresh basement. From the three pseudo-sections, areas with low

resistivity were assumed to reflect the presence of leachate which was inferred to occur laterally in western direction of the dumpsite up to depth of 22m depth (Figs 3 and 4). This probably gained its migration pathway through the pore spaces of the interconnected grains.

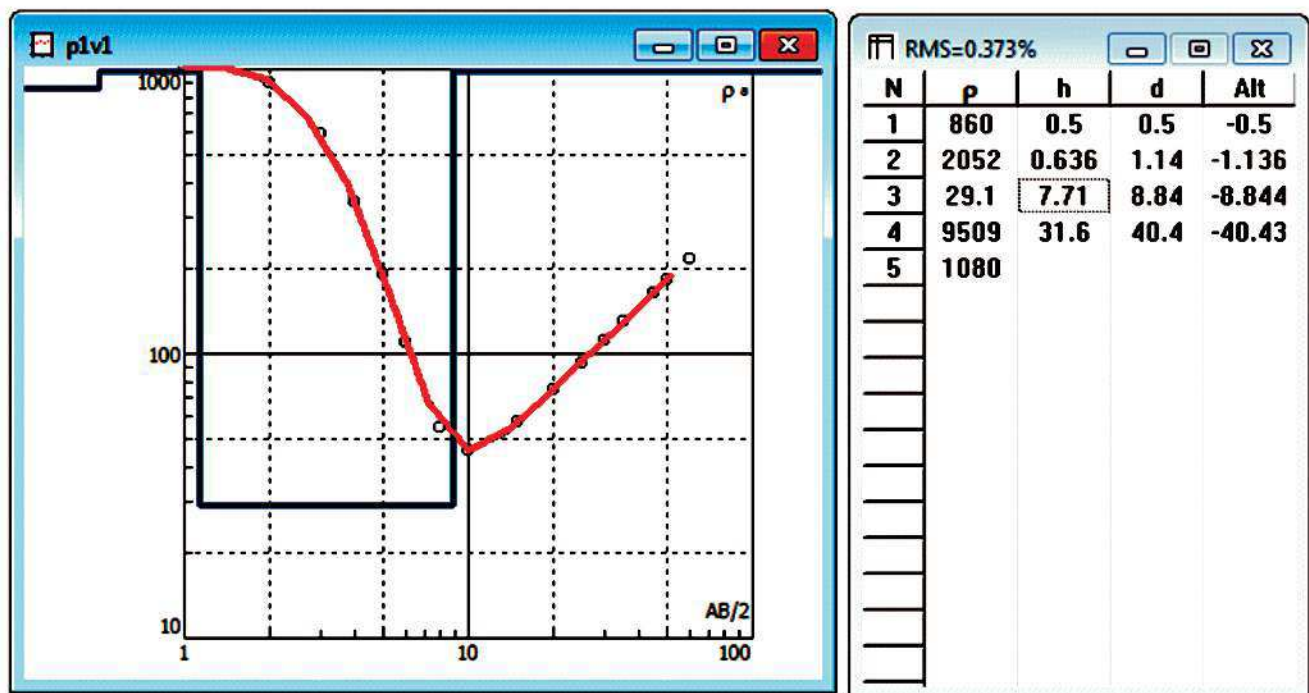


Fig. 2: Typical VES curve over the site

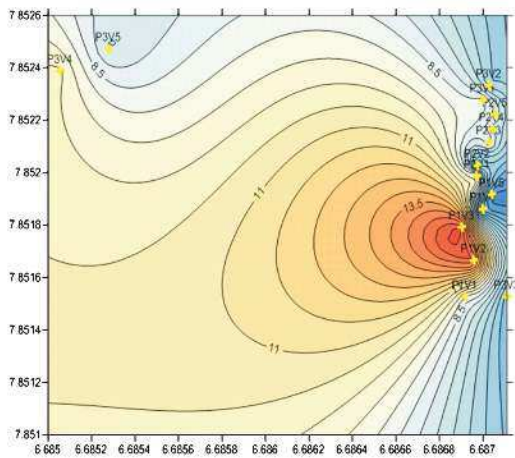


Fig. 3: Depth Basement map over the site.

Geotechnical Characteristics

The results show that gravel has percentage ranging from 10 to 54%, percentage of sands range from 19 to 76%. The percentage of silt ranges from 4.46 to 21% whereas clay ranges from 1.54 to 38% (Fig. 4). These values largely fall below $\geq 50\%$ specified for clay liner materials (Frempong and Yanful, 2004). Thus, they are

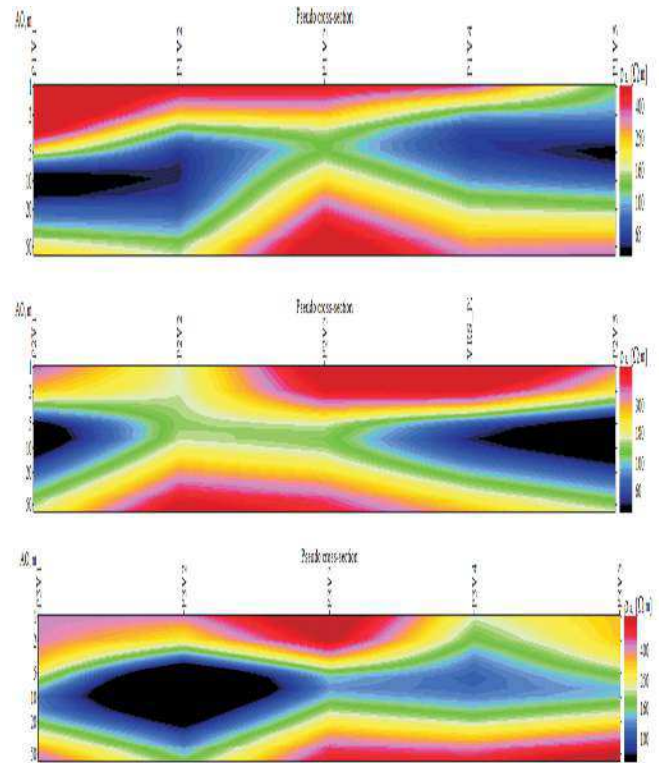


Fig. 4: Pseudo-section for Profilethree

Table 1: Location of Leachate Samples within and around the site

S/N	SAMPLE ID	Location	GPS	SWL (m)	Distance from site (m)
1	S1	Within the site	N07°05'15.5" N E06°04'12.9" E	0.30	-
2	S2	Within the site	N07°05'17.3" N E06°04'13.1" E	0.35	-
3	S3	Within the site	N07°05'16.7" N E06°04'13.1" E	0.00	-
4	S4	Within the site	N07°05'18.6" N E06°04'16.2" E	0.00	-
5	S5	Within the site	N07°05'18.2" N E06°04'13.2" E	0.00	-
6	*S6 (surface water)	River Awori (along Agbaja Village)	N07°05'17.9" N E06°04'11.4" E	0.99	74 (up site)
7	*S7 (surface water)	Peace house bustop Felele phase 2	N07°05'13.5" N E06°04'21.5" E	0.61	150 (down site)
8	*S8 (shallow well water)	H.A Patient medicine store Felele phase 2	N07°05'21.3" N E06°04'21.5" E	2.28	320 (down site)

*S1 - Control Sample

not good enough to curtail leachate migration. However, the specific gravity values range between 2.51 and 2.74 which is favoured by 2.2 recommended by ONORMS 2074 (1990). The PI values range from 10.62 to 41.15 indicating inorganic clay of medium that possibly contain kaolin as dominant mineral type (Fig. 5). The Skempton classification of clay plasticity value and clay

mineral type show that all samples contain illite as dominant clay mineral except pit 2 and 6. Also, 26.6% of the samples are active while others are either normal or inactive (Table 2).

The coefficient of permeability of any soil is sensitive to the MDD value. Soils with MDD of 1.71 g/cm^3 indicate

easy workability on the field (ONORMS 2074). Eight samples were subjected to compaction because of high portion of sand and gravel size in other samples. Only 5 of these samples are parabolic in shape while other are skewed (Fig. 6). The MDD of soil samples are far undesired when compare with the standard recommended values. The coefficient of permeability of the analysed soils ranges from 10^{-6} m/sec to 10^{-5} m/s which has greater permeability and undesirable lining material when compare with standards. It is generally taken that the overriding requirement of a material for use as landfill liner is its capability of achieving permeability (k) of $\leq 1 \times 10^{-9}$ m/s following compaction (Ige, 2007). The obtained results implied soils with increased pores and thus not suitable as potential liners.

Table 2: Summary of LL, PL, PI and Skempton (1953) classification of Clay.

Sample number	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Linear shrinkage limit (%)	Activity of clay	Clay mineral type	Classification of clay
Pt 1	54	21.63	30.37	9.5	1.32	Illite	Active
Pt 2	59	17.85	41.15	9.9	2.74	Montmorillonite	Active
Pt 3	23	12.38	10.62	5.5	0.82	Illite	Normal
Pt 4	31	15.08	15.92	6.8	0.88	Illite	Normal
Pt 5	56	20.85	40.15	9.9	0.74	Illite	Normal
Pt 6	42	24.65	15.92	8.5	0.42	Kaolinite	Inactive
Pt 7	35	16.98	17.02	6.2	0.95	Illite	Normal
Pt 8	41	25.65	30.35	6.0	1.17	Illite	Normal
Pt 9	39	20.06	19.78	8.5	0.64	Illite	Inactive
Pt 10	20	17.32	11.66	6.5	0.82	Illite	Normal
Pt 11	27	14	12.5	5.8	0.78	Illite	Normal
Pt 12	35	18.4	16.1	5.5	1.34	Illite	Active
Pt 13	51	20.65	30.35	6.0	1.17	Illite	Normal
Pt 14	53	21.33	31.67	6.5	1.44	Illite	Active
Pt 15	50	27.18	21.82	7.5	0.64	Illite	Inactive

Water Quality Evaluation

The pH of all the leachate samples was about acidic to neutral, the range being 6.60 to 7.45. The EC in the studied site range between 3760 and 4210 $\mu\text{S cm}^{-1}$ and was found to be an indication of high amount of dissolved materials especially within the site. The range of TDS at all location falls between 271 and 1994 mg l^{-1} which is interpreted to be remarkably high due to

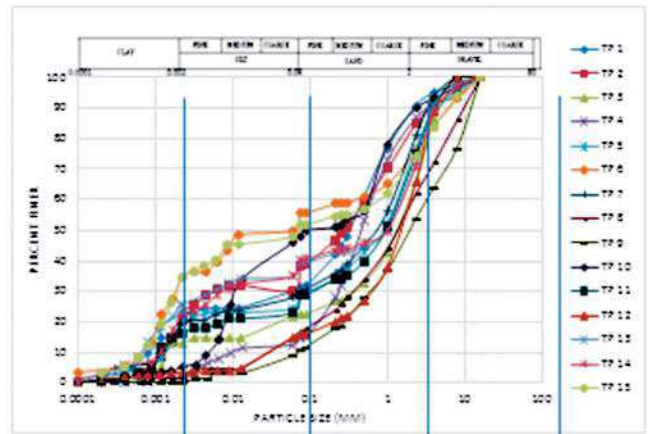


Fig. 4: The Grain Size Distribution curves of soil samples

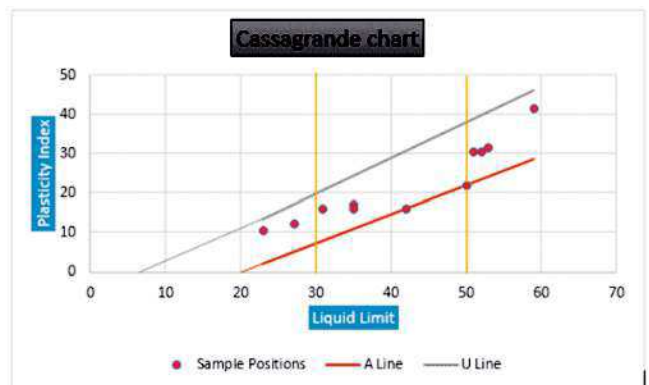


Fig. 5: Positions of the soil samples in the Casagrande's Chart

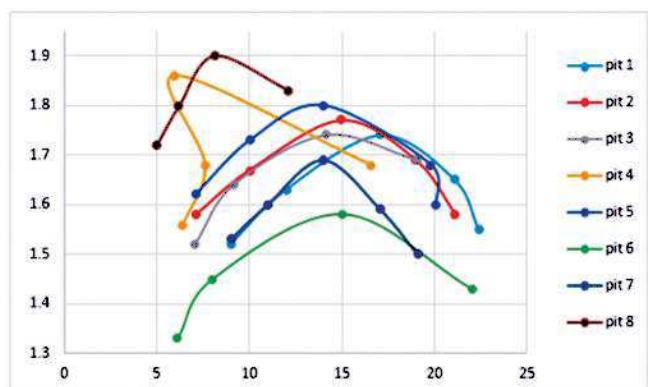


Fig. 6: Compaction curves

possible leaching of salt (Rabinove et al., 1985). COD is a measure of oxygen equivalent to the organic matter content of the water susceptible to oxidation by a strong chemical oxidant and thus is an index of organic pollution. The COD level in the groundwater samples varied from 7 to 25 mg l^{-1} , indicating the presence of organic contaminants in the water.

Ca^{2+} and Mg^{2+} are the important parameters for total hardness. Ca^{2+} concentration in groundwater ranged from 66 to 564 mg l^{-1} . Ca^{2+} often comes from carbonate-based minerals, such as calcite and dolomite. The concentration of Ca^{2+} found to be very high in samples within the site. Elevated concentration of Ca^{2+} causes concretions in the body such as kidney or bladder stones and irritation in urinary passages. The concentration of Mg^{2+} ions varied from non-detectable (nd) to 420 mg l^{-1} . Mg^{2+} salts are cathartic and diuretic and high concentration may cause laxative effect, while deficiency may cause structural and functional changes (Chu *et al.*, 1994). It is essential as an activator of many enzyme systems (WHO, 1997). The concentration of Na^{+} in water samples varied from 10 to 219 mg l^{-1} . The high concentration of Na^{+} may pose a risk to persons suffering from cardiac, renal and circulatory disease (WHO, 1997). The concentration of K^{+} in the water samples varied from 11 to 91 mg l^{-1} and was found to be well within the permissible limit of WHO. The NH_4^{+} concentration in the samples ranged from nd to 4.3 mg l^{-1} and likely indicate its origin from leachate.

The concentration of Cl^{-} in the groundwater samples ranged between 28 mg l^{-1} to 737 mg l^{-1} . High Cl^{-} content of groundwater is likely to originate from pollution sources such as domestic effluents, fertilizers and septic tanks, and from natural sources such as rainfall, the dissolution of fluid inclusions. An excess of Cl^{-} in water is usually taken as an index of pollution and considered as a tracer for groundwater contamination (Loizidou and Kapetanios, 1993).

The concentration of F^{-} in the studied water samples ranged from 0.37 to 1.13 mg l^{-1} and found to be slightly high within the site. F^{-} at low concentration ($<1 \text{ mg l}^{-1}$) in drinking water has been considered beneficial but high concentration may cause dental fluorosis (tooth mottling) and more seriously skeletal fluorosis (Ravindra and Garg, 2005). The concentration of NO_3^{-} in water sample varied from nd to 41 mg l^{-1} . Although only one sample exceeds the permissible limit but it shows a moderately high concentration. Jawad *et al.* (1998) have also reported increase in NO_3^{-} concentration in groundwater due to wastewater dumped at disposal site and likely indicate the impact of leachate. The concentration of NO_2^{-} varied from nd to 0.71 mg l^{-1} . PO_4^{3-}

in the water sample was well below the permissible limit and the concentration varied from nd to 0.06 mg l^{-1} . Concentration of SO_4^{2-} in water sample ranged from 12 to 1096 mg l^{-1} and was significantly high.

Heavy Metals

Metal mobility is controlled by physical sorptive mechanisms which are sometimes limited by landfill liners (Loizidou and Kapetanios, 1993). Cu, Fe and Zn, Pb, Cd, Cr and Ni which are classified as undesirable toxic metals in drinking water are generally slightly higher than permissible concentration within the site except Cr and Ni. This is an indication of the potential for contamination of water bodies and health risk. Cu ranges from nd to 3.90 mg l^{-1} ; Fe ranges from 4 to 27 mg l^{-1} ; Zn ranges from nd to 2.01 mg l^{-1} ; Cd varied from nd to 0.71; Pb varied from nd to 1.6, Cr varied from nd to 0.03 and Ni ranged from nd to 0.69. Though the concentrations of all metals reduce away from the site centre but are expected to increase with time as waste deposition increases in rate and quantity.

Conclusion

The investigated site lies at the Felele in southern contact between the Precambrian Basement Complex and Sedimentary of Nigeria. Geophysical survey revealed depth to basement ranging between 24.9 and 37.4m with about three to four geoelectric layers. The unfractured granite gneiss is unconfirmably overlain by the massive-clay supported sandstone. The overall engineering characteristics of the soil samples are inorganic clay with slightly high plasticity. Generally, these types of soils possess undesirable characteristics to minimize hydraulic conductivity of compacted soils. The basic properties (liquid limit, plastic limit, percentage fine, percentage gravel, activity etc) of the soil samples did not satisfy the requirements as barrier materials in landfills. The soils have average hydraulic conductivity of $1 \times 10^{-6} \text{ m/s}$ when compacted with the modified Proctor compaction efforts which is indicative of difficulty during construction. This is dangerous as it will facilitate free movement of leachate to pollute the subsurface. Results of physico-chemical and heavy metal analyses indicated slightly elevated concentrations above standard permissible limits especially within the site.

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A GIS Based Approach to Mapping and Visualizing Engineering Soil Data

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Abstract

Geographic information science (GIS) technology was applied to map and visualize engineering soil data collected from Ogun State. Physical and geotechnical tests were carried out on 161 soil samples in order to determine their engineering properties. The resulting data, topographic and geologic information were computed in GIS environment to delineate soil attributes/boundaries and thus produce soil zonation map of the area. The results showed range of well graded sandy silty clay with gap gradation curve displaying fines (2 to 81.4% passing 0.075 mm sieves). The soils plasticity varied from low to very high with predominantly sandy silty clay (43%) and silty sand (22%). Other classes include inorganic clays of low plasticity (CL), inorganic silt with micaceous fine sand (MH), inorganic silts and very fine sands (ML), poorly graded sand with silt (SP, SP-SM) of varying percentage. Sandy clay was concentrated around the southwest area and stretched eastward while other soil types were sparsely located. The dominant soil group was 15% (A-6), 14% each (A-2-4, A-2-6, A-7-5), 11% each (A-7-6, A-3); 9% each (A-2-7, A-4); and 3% (A-2-5) types. The results suggest that the soils will exhibit poor engineering qualities due to high moisture loss tendency when exposed to dryness.

Keywords: *Geographic Information System, Zonation Maps, Geo-Engineering Maps, Geology*

Introduction

With advances in digital technology, application of geographic information science (GIS) in science research is increasingly being used for spatial data management and manipulation. Soil geotechnical evaluation often requires large amount of spatial data and GIS is a powerful tool useful for such evaluation because series of analyses can be done on very large data and large areas in a very short time. Important feature to the use of GIS tool is the fact that new information can be generated from diverse datasets through integration provided they share a common spatial referencing system. GIS technology has been widely used in different geo-environmental fields such as natural geological hazards, groundwater vulnerability assessment, site selection for waste disposal, landslide susceptibility, suitability of foundation materials (Dai et al, 2001; Kolat et al, 2006; Orhan et al, 2007; Tosun et al, 2007).

In engineering, geotechnical information is important in foundation works and ensures that the effects of projects on the environment and natural resources are properly evaluated and mitigation measures are applied where necessary (Nwankwoala et al. 2009). The objective of pavement design is to provide a structural and economical combination of materials to carry traffic in a given climate over a given soil conditions for a specified time interval. Subgrade material is one of the crucial

parts in pavement foundation (Oladeji & Raheem, 2002; Okunade, 2007) which represent a key factor that affect pavement structural design. Since all pavements derive their support from underlying Subgrade (Yoder & Witzack 1975), a good knowledge and understanding of soil materials becomes essential in order to avoid premature failures during construction.

There are multiple approaches and methodologies which have been implemented in GIS to produce accurate data such as multi-criteria decision analysis, fuzzy logic and artificial networks. These methods have been used by geoscientists to produce series of maps of different coverage extents and material suitability. Chacon et al. (2006) created engineering geological maps by means of landuse susceptibility. Kolat et al. (2006) also used multi geo-data handling process with multi-criteria decision analysis to determine a liquefaction potential of an area. Seismic zonation maps were also created using GIS (Orhan et al. 2007; Tosun et al., 2007). More recently, application of GIS was used to map soil types based on the relationship that exists between individual soil properties and the radioisotopes of the environment (Taylor et al, 2002; Fakeye et al, 2012; Viscarra et al. 2006).

In view of its importance in geotechnical study, zonation maps of some soil properties have been prepared to visualize the variability of soil characteristics and its suitability for foundation works. These data generated

forms a database to engineers and contractors in the preliminary planning stage of any projects and the gathered facts will serve as a useful guide for further work.

Objectives

The main objective of this work is to develop comprehensive information on engineering properties of subgrade soils in Ogun State, create soil database and zonation maps of different soil type as well as determine the suitability of the soil for construction work.

The Study Area

Ogun State is located in southwestern Nigeria, bordered by Lagos State to the south, Oyo and Osun states to the north, Ondo State to the east and the Republic of Benin to the west. The State occupies a land area of about 16,980.55 km² and situated between Latitude 6°00'N and 8°00'N and Longitude 2°30'E and 5°00'E with the annual rainfall varying between 105cm to 128cm. The average monthly temperature ranged from 23°C in July to 32°C in February. The geologic unit of the area belongs to the Tertiary age which is made up of sedimentary and basement complex rocks, which underlie the remaining surface area of the State. 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 under the Basement complex composed of gneiss, migmatites complex (Figure 1). This in turn is overlain by Ewekoro, Oshosun and Ilaro Formations, which overlies the Coastal Plain Sands (Benin Formation) (Ugbe 2011).

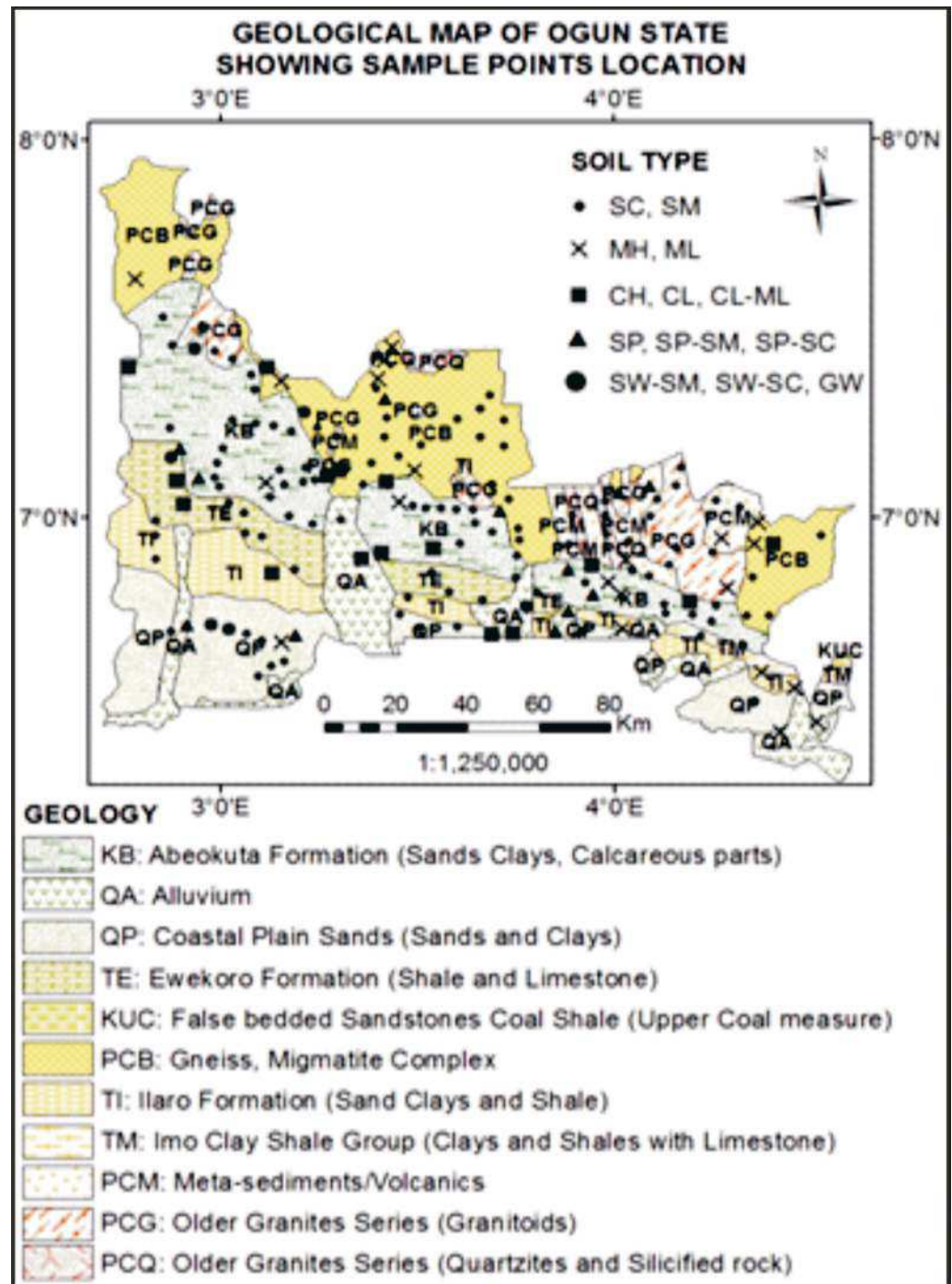


Fig. 1: Geologic map of Ogun State showing the sample points locations

Materials and Methods

Materials

The data sets used include topographic, geologic map and geotechnical test data. 1: 50,000 scale topographic and lithologic maps were georeferenced and digitized to obtain digital elevation model (10x10m cell size) and different lithology of the study area respectively.

Field investigation

Soil samples (161 nos) were collected in the field using a random sampling technique with hand augers at a depth of 1.5 meters subsurface and geographic positioning system (GPS) to determine point location in a grid network. In-situ moisture content was determined using the Thomas Ashworth speedy meter and groundwater conditions from available records. In the field, visual examination and description of soil samples and topographic notes were taken.

Laboratory tests

The 161 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 and hydrometer analyses, Atterberg limit test, water content and density were determined using the above standards. All grain size distribution curves were drawn, percentages of gravel, sand and fines (silt-clay) were determined. The D_{10} , D_{30} , and D_{60} values were read from the curves and the necessary coefficients of uniformity (C_u) and curvature (C_c) were calculated for classifying the coarse-grained soils. Based on the results, the soils were classified according to Unified Soil Classification System (USCS) and American Association of State Highway and Transportation Officials (AASHTO) where soils were grouped according to their suitability as subgrade or embankment materials.

Data Analysis

The geotechnical data were imported into ArcGIS software and series of data analysis were carried out such as spatial interpolation, digitization and modeling. When conducting soil classification, transformation techniques such as principal component analysis (PCA) and band combination (RGB) were used in selecting reliable data. Various data (topographic, geology,

geomorphic map, etc) were aggregated from a variety of sources and analysed using GIS platform to extract useful information.

Spatial interpolation in GIS environment was applied to the soil data using inverse distance weighting method being one of the most frequently used deterministic model in spatial interpolation by geoscientists (Lu and Wong, 2008). The datasets were classified based on fuzzy rules criteria to produce zonation maps and further aggregated in a single view to provide efficient processing, analysis and presentation. Soil database was created during processing and managed. The subgrade soil map was digitized, point and line layers created for location, town and roads. By using vector analysis, a module in Arcmap, various queries were run and zonation maps such as free swell, plastic and liquid limits were produced.

Results and Discussion

Laboratory Test Results

Grain size distribution of soil samples

The parameters of basic physical properties and statistical analysis is shown in Table 1 which indicates that the grain size distribution vary in a wide range of proportion. Table 2 summarizes the frequency distribution and proportion occurrence of percentage fines in soil groups as A-2 (41%) and A-7 (25%) with few occurrences of A-6 (15%), A-3 (11%) and A-4 (8%). Only the A-2 soils contain some amounts of gravels with percent fines between 5-35%. Majority of the soils are fine grained. The grain size distribution graphs in Figure 2 indicate that the soils are silty and clayey sands for A-2 and A-7 soil groups. Generally, the grain size distribution indicates a range of poorly graded clayey silty sands with gap gradation curve displaying fines (2 to 81.4% passing 0.075 mm sieves). Hydrometer analysis conducted showed high amount of silts and clay fractions (fines) and extremely low percentage of gravels (< 10%) which renders the soil unsuitable for construction of granular roadbase and subbase courses. Some of the A-2 materials with less than 20% fines can however be used for subbase, general filling and embankment construction.

Soil Classification

The liquid and plastic limits varied from 20 - 82% and 12 - 48%, respectively. The plasticity index was

Table 1: Statistical summary of test results

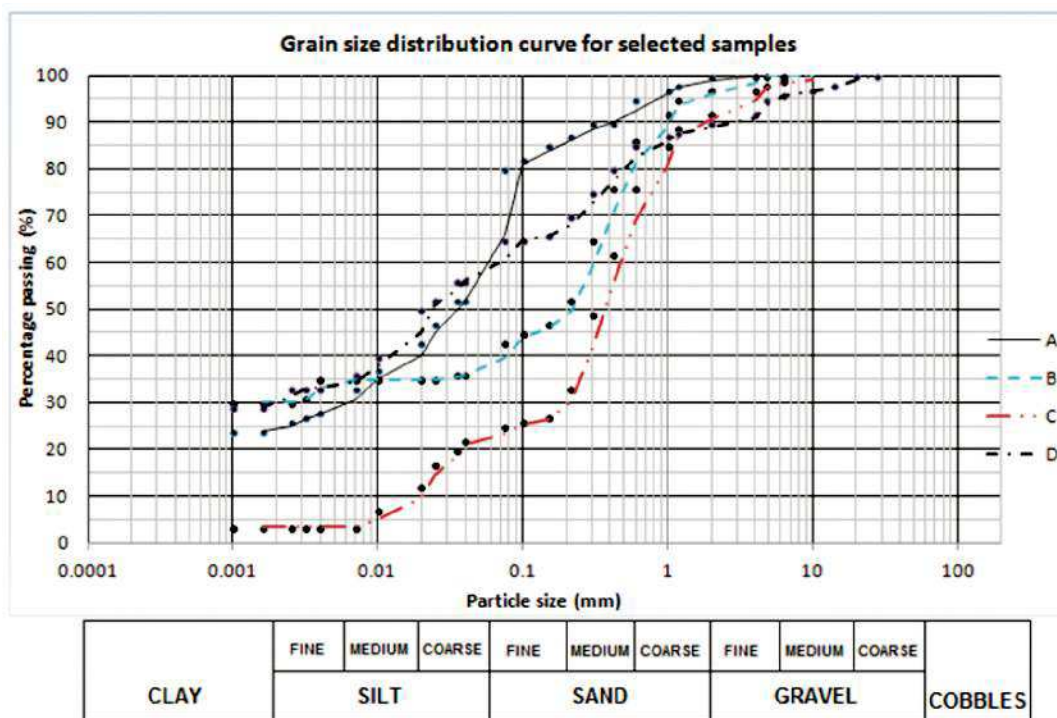
Test		Min	Max	Mean	SD	Range
Percent passing	4.75mm	43.8	100	95.6	8.7	56.2
Sieve	2mm	26	100	90.0	14.0	74
	425 μ m	16.6	96.9	69.3	17.4	80.3
	75 μ m	2.2	81.4	35.6	17.2	79.2
	2 μ m	0.3	48	18.1	11.2	47.7
	1 μ m	0.3	47	16.9	10.5	46.7
Liquid limit (%)		20.2	82	42.6	14.5	61.8
Plastic limit (%)		12	46	25.1	8.1	34
Plasticity Index (%)		3.2	57.7	17.5	8.1	54.5
Free swell (%)		0	50	21.5	8.7	50
CU		1.1	9200	728.8	1139.7	9198.9
CC		0.002	216	16.3	31.3	216
MDD (mg/m ³)		1.1	2.4	1.9	3.8	1.3
OMC (%)		5.4	28.2	14.8	0.2	22.8

CU, coefficient of uniformity; CC, coefficient of curvature; OMC, optimum moisture content; MDD, maximum dry density

Table 2: Frequency distribution of the percentage fines in the soil groups in the study area

ST	NS	RPF	F	PO
A-2	66	0-10	8	5
		11-20	12	7
		21-35	46	29
A-3	18	0-10	5	3
		11-20	6	4
		21-35	7	4
A-4	12	35-45	3	2
		46-55	9	6
A-6	24	35-45	11	6
		46-55	9	6
		56-85	4	3
A-7	41	35-45	11	7
		46-55	17	10
		56-75	13	8

ST, soil types; NS, number of samples; RPF, range of percentage of fines; F, frequency; PO, proportion of occurrence.

**Fig. 2:** Grain size distribution curves of selected soil samples showing gradation of soil samples

between 3 - 58%. According to the classification of Casagrande chart modified by Bell (2007), the soils varied from low to very high plasticity where 58 soil samples were classified as low plasticity, 60 soils as intermediate plasticity, 33 soils as high plasticity and 10 as very high plasticity. The variations in the plasticity indices within each group increased as the soils became progressively more plastic.

The plasticity index as shown in Table 3 classified the soil as having low to medium swelling potential according to Ola (1982) which corroborates the result in Table 2 where the dominant percentage of fines of the A-7 type ranges between 46-55%. This suggests that the soils would exhibit poor engineering qualities because they showed great tendencies to lose moisture when they get exposed to dryness.

The results of liquid limits and plasticity indices of the soil samples plotted on the Casagrande plasticity chart showed their respective plasticity characteristics and classes with respect to the A-line as shown in Figure 3. From the chart, some of the soils plot above the A-line while others plot below the A-line. This indicates that they are essentially both silts and clays which

corroborate the result of the grading test. A-2 soils have liquid limits ranging from 20 to 80% and plasticity index between 3 to 35%. When compared with the Federal Ministry of Works (1997) standard ($W_L \leq 30$, $PI \leq 12$), some of the A-2 soils can be used for general filling embankment, roadbase and subbase courses.

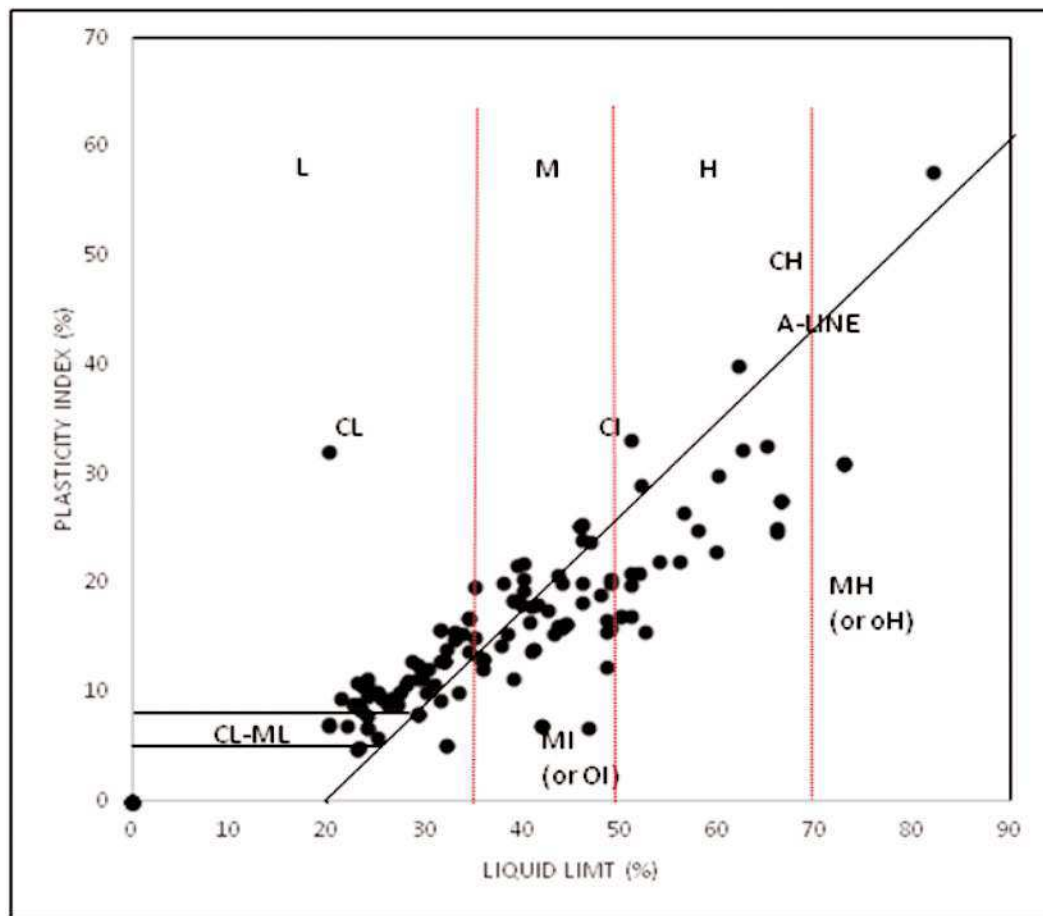


Fig. 3: Plasticity chart of the soils

The results of the free swell test ranged between 0 – 50% with a mean of 20% and standard deviation of 9.07. Using the classification scheme proposed by earlier researchers (Holtz 1959; Prakash & Sridharan, 2005; Ola, 1980) to identify the degree of swelling of the soils used in the study, it can be observed that the degree of swelling in the present study area varies from low to high, depicted as L and H, respectively in that Fig. 3.

Statistically, soil data analysis show that about 43% of the soils classify as clayey sands (SC) as shown in Fig. 4.

About 22% are silty sand (SM). Some 21% classify as inorganic clays of low plasticity (CL), inorganic silt with micaceous fine sand (MH), inorganic silts and very fine sands (ML). About 6% are poorly graded sand with some containing silt (SP, SP-SM). Other soil classes shares the remaining 7% (SW-SC, SW-SM, GW, CH, SP-SC, and CL-ML). Most silty sand soils are located around Idiroko, Ishara & Alakuko while clayey sands are concentrated at Iwoye, Abeokuta, Owode and Ijebu Igbo areas.

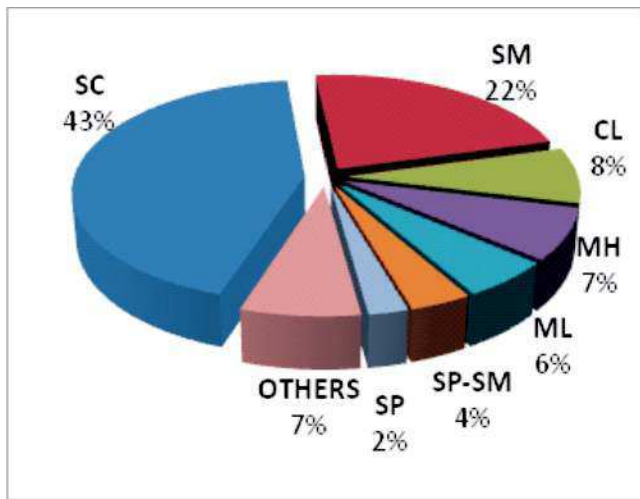


Fig. 4: Statistical overview of USCS soil classification.

From AASHTO classification system, the dominant groups and the proportion of occurrence are 15% (A-6) and 14% each (A-2-4, A-2-6, A-7-5) types with a few percent occurrences of 11% each (A-7-6, A-3); 9% each (A-2-7, A-4); and 3% (A-2-5) types. About 74% of the soils in the State are materials of poor engineering quality and the likelihood that these poor engineering

materials will be used to construct pavement subgrades is very high. The likelihood of experiencing pavement construction problems is also high if remedial measure is not applied during construction. The other 26 percent considered good could however, be used as subbase material.

Compaction characteristics

The maximum dry density (MDD) gives appropriate information on its gradation while optimum moisture content (OMC) gives information on the clay and silt content. As expected, the moisture-density result shows an increase in soils MDD between 1.1 – 2.4 unit which was inversely proportional to OMC between 5.4 – 28.9 as presented in Table 3. The maximum dry density however, decreases with increase in fines thereby making the A-2 material exhibit the highest maximum dry density while A-7 soils have relatively lower densities. They are therefore not suitable for construction of roads. Generally, as expected also, maximum dry densities and optimum moisture contents of soils in the study area are influenced by the grading and plasticity.

Table 3: Value range of each property of the AASHTO soil group in the study area

AASHTO CLASS	NO OF SAMPLES	MDD (mg/m ³)	OMC (%)	LL (%)	PL (%)	PI (%)
A-2-4	23	1.4-2.2	8-22	20-50 (16) 60-80 (7)	13-30 (16) 31-45 (7)	3-10 (6) 11-25 (10) 26-35 (7)
A-2-5	5	1.8-2.2	9-18	42-52 (5)	22-35 (5)	7-10 (1) 20-21 (4)
A-2-6	23	1.1-2.4	8-20	20-50 (16) 51-68 (7)	12-20 (7) 21-39 (16)	6-10 (8) 11-25 (12) 26-32 (3)
A-2-7	15	1.6-2.3	10-25	26-35 (5) 40-65 (10)	17-20 (6) 25-35 (9)	10-20 (11) 21-33 (4)
A-3	18	1.1-2.2	5-23	24-35 (6) 36-58 (12)	13-25 (11) 26-34 (7)	10-15 (8) 16-25 (10)
A-4	12	1.6-2.1	9-24	20-50 (10) 51-77 (2)	13-30 (9) 31-42 (3)	5-14 (9) 21-35 (3)
A-6	24	1.5-2.1	9-28	24-38 (15) 43-46 (4) 66-82 (5)	12-20 (13) 21-30 (7) 40-42 (4)	5-10 (5) 11-25 (14) 32-58 (5)
A-7-5	23	1.3-2.2	10-21	23-32 (4) 43-50 (12) 51-76 (7)	13-25 (6) 26-35 (15) 36-46 (2)	9-20 (12) 21-30 (11)
A-7-6	18	1.2-2.2	9-22	23-35 (8) 40-55 (7) 62-74 (3)	15-25 (10) 26-30 (5) 33-39 (3)	8-14 (9) 16-25 (6) 31-40 (3)

MDD, maximum dry density; OMC, optimum moisture content; LL, liquid limit; PL, plastic limit; PI, plasticity index

Evaluation of some geotechnical data and their implications

As aforementioned, the geotechnical data was evaluated by means of zonation map (Figure 5) in order to visualize their spatial variations and relationship. In this work, the variation of Atterberg limit and free swell properties are evaluated with respect to the two major soil types (clayey and silty sand). The fine grained nature of the soils offer a low permeability potential and the proportion of silty clay fractions may have significant influence on the behaviour of the soils, thus giving rise to high compressibility. Because of the soils compressible nature, the influence of imposed load may

result in consolidation settlement. This imposed load impact may be exacerbated by the thickness and consistency of the compressible layer. This in addition to other intrinsic factors strongly contributes to the failure of civil engineering structures. In view of this, it is therefore recommended to evaluate the strength of the soil for proper design of shallow foundation system that would bear the weight of structures. Also, the result of the AASHTO classification suggests a predominantly fine-grained silt and clay soils. These soils exhibit poor strength, high volumetric instability, and durability therefore there is a great need to stabilize the soil during foundation to improve soil properties and strength.

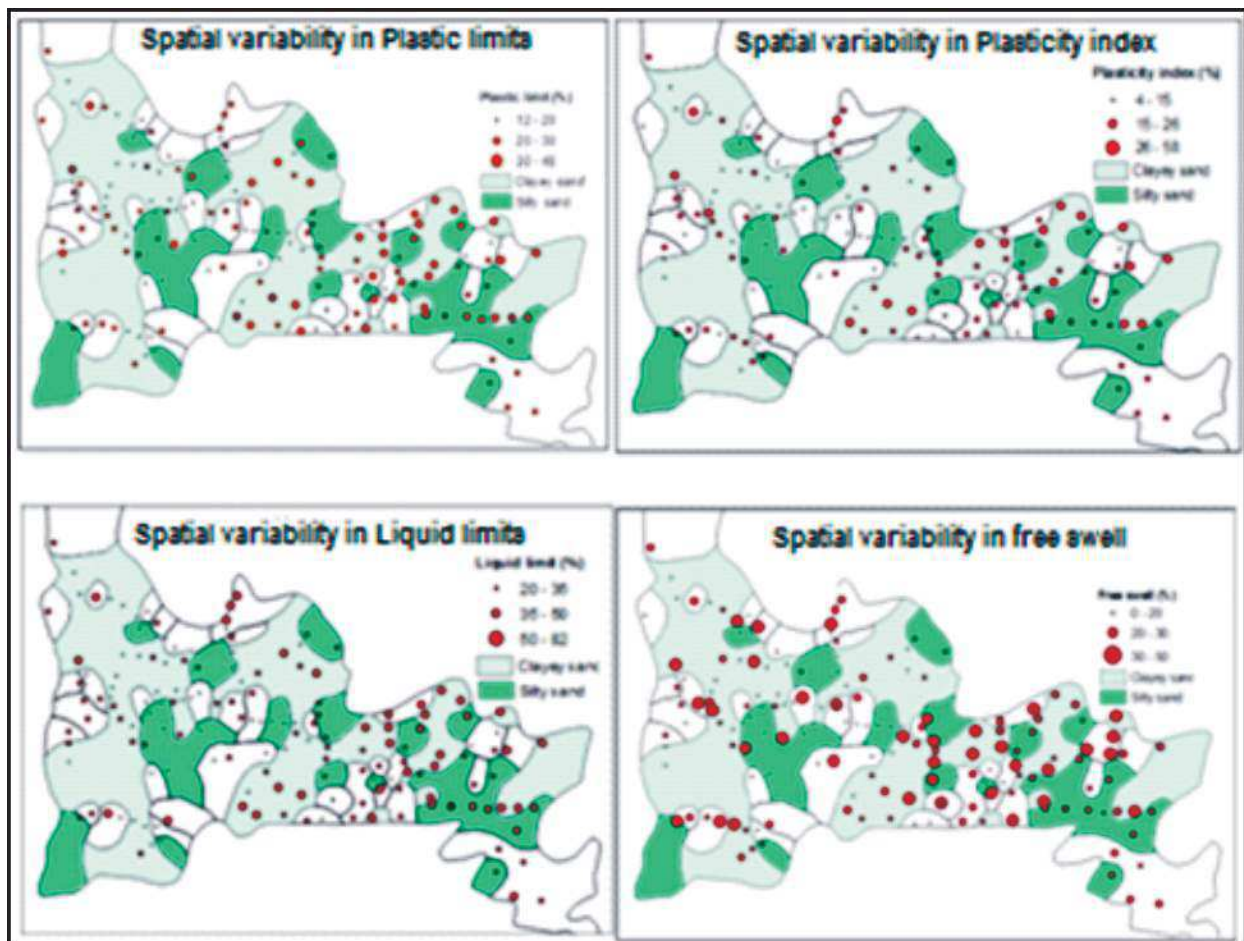


Fig. 5: Spatial variations in Atterberg limits and free swell properties overlain on soil map

Digitized Subgrade Soil Map of Ogun State

Digitized subgrade soil map of Ogun State classified based on Unified Soil Classification System (USCS) is shown in Figure 6. Throughout the study area, different soil types are observed with varying attribute information which are inputted into a soil geodatabase.

This information can be displayed, retrieved, queried, edited, and analyzed for further processing using software modules which can analyze or manipulate the retrieved data in order to generate desired information like "what is the soil property variation at a certain location". This is a standard querying language (SQL) used in relational database to update or retrieve

information. The query can be used to access spatial information from the original data in the database. In figure 7, the variation of plasticity index is spatially visualized in the study area. In GIS, every feature has attribute associated with it. A feature can have one or more attributes which could be point, line or polygon and which could be integrated or overlain to obtain a new feature that can explain a cause. In this study, the

attribute information are stored in geodatabase that can be displayed and analyzed on a map without affecting the underlying feature's geometry. The application of GIS also allows an incorporation of additional database into an existing soil system without changing their spatial referencing system. It is therefore, highly expandable.

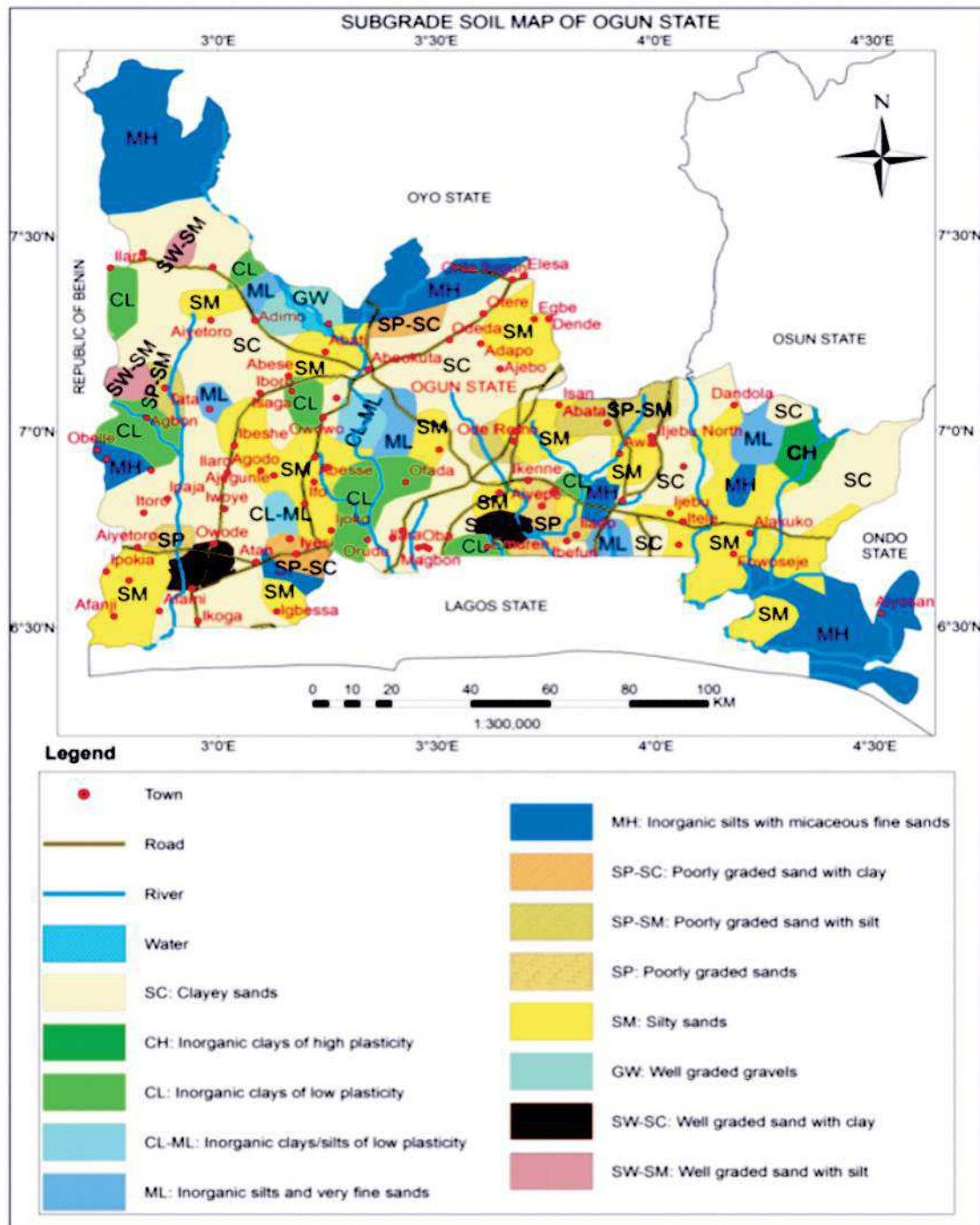


Fig. 6: Digitized Subgrade soil map of the area

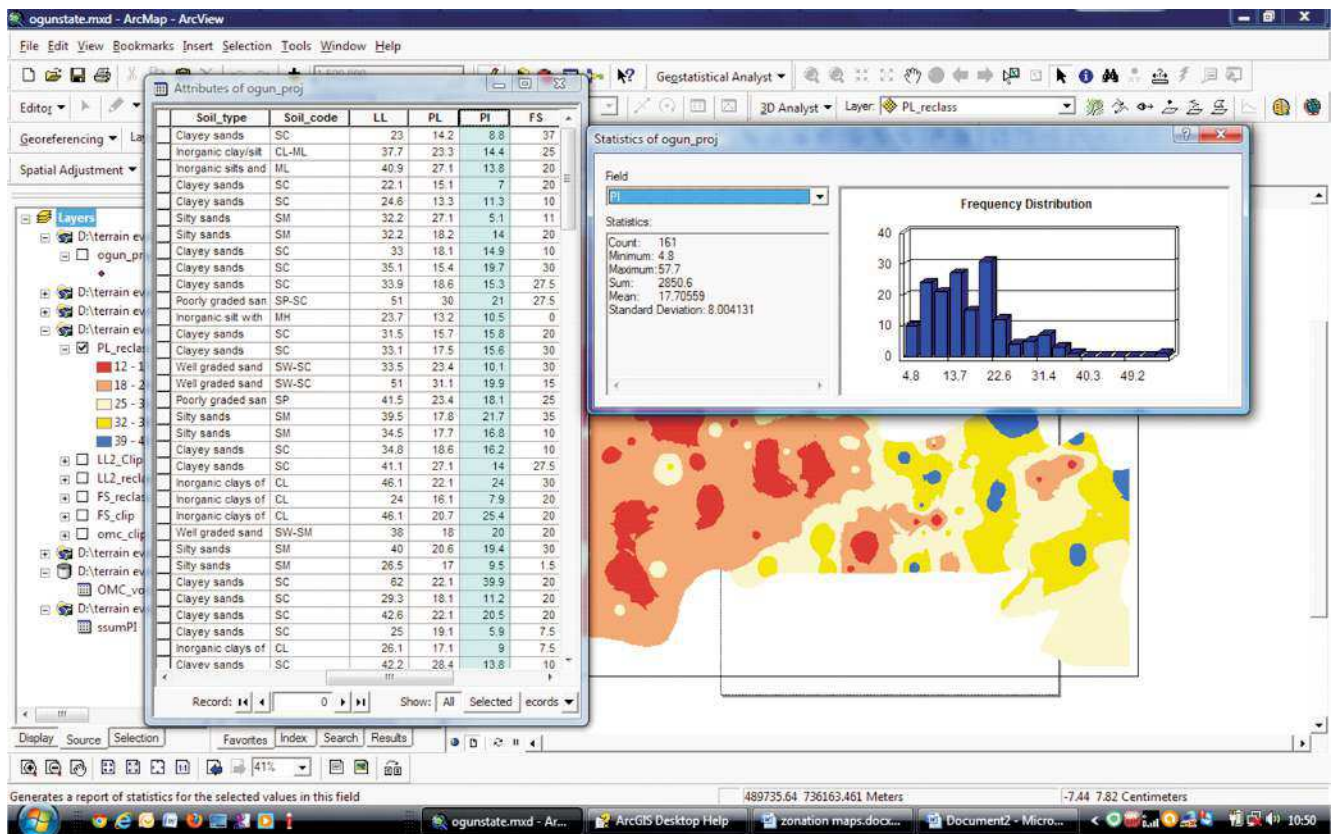


Fig. 7: Resultant map from ArcGIS query showing the plastic limit variation and its histogram

Conclusion

GIS approach proves to be a useful technique that could be used in mapping and enhancing soil information. This paper reveals the basic engineering properties of subgrade soil in Ogun State. The soils were found to be of A-2, A-3, A-4, A-6 and A-7 types with A-2 and A-7 type soils being dominant. The soils have fines content of silty and clayey sands with a mean of about 35% and low gravels percentage which makes the soil unsuitable for road base and subbase courses. Their mixed ratio varies in proportion, showing low to medium swelling potential with plasticity index up to 57%. This indicates that the soils cannot be used to construct durable roads except they are stabilized with appropriate materials that are compatible with the soil properties. Hence, adequate precaution should be taken to accommodate the swelling potential and associated deformation.

By means of spatial interpolation module in the software, a well defined soil distribution map was created which can be used as a valuable tool to the planner, engineer, consultant and contractor. Geotechnical engineers can run more queries of various combinations regarding the various properties of soil which will help in decision making process. This work will be of benefit to geotechnical engineers, consultants, investigators and clients by maintaining updated information regarding soil index properties, reduces time for decision making, generate various maps in vector analysis module which will depict the characteristics of soil, help contractors in knowing about the soil profile before starting their work.

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Geo-Engineering and Environmental Concerns Related to Shales in the Abakaliki Area of Southeastern Nigeria

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Abstract

The impact of the shale formation to the geotechnical and hydrogeo-environmental problems prevalent in the Abakaliki area (southeastern Nigeria) has been evaluated. Analysis on crushed shale samples showed liquid limit (49-54%) and plasticity index (34-38%) that are moderately high, an indication of materials with swellable or compressible clays. Natural moisture content (with mean value of 20%) was also significantly high. Hydrochemical analyses, on the other hand, revealed that the water samples from the shaley formation had comparatively high content of ions and dissolved particles; Ca^{2+} ranged from 8.0 to 134.0 mg/l, Mg^{2+} ranged from 0.2 to 3.3 mg/l, HCO_3^- ranged from 21.8 to 1176.1 mg/l and total dissolved solids ranged 56.9 to 1415.0 mg/l. Dissolution of soluble minerals (predominantly calcite and other rock salts) contained in the shale might likely be responsible for the release of these particles. Geotechnical behaviours of the shale and hydrogeochemical characteristics of water samples from the shale formation, thus, provided insights into the probable causes of seasonal waterlogging, consistent poor groundwater quantity and quality, structural and foundation problems that are prevalent in the Abakaliki area.

Keywords: Abakaliki area; Environmental issues; Geotechnical problems; Shales; Water quality

Introduction

Shale is considered the most predominant sedimentary rock type on the earth's surface. It has been widely noted for its range of composition and properties. In construction projects, environmental and water resources development, shales are known to possess properties that call for concern. The problems possessed by shales in shaley terrains of the world are, in most cases, being influenced by mineralogy, especially the predominant clay mineral type(s) (Okagbue, 1989; Obiora and Umeji, 2004) as well as the climate and physiography of the area under consideration (Ezeribe, 1994).

Abakaliki, the capital city of the current day Ebonyi State, southeastern Nigeria, is an example of a terrain predominantly underlain by shales of the Albian Asu River Shale (Reyment, 1965; Murat, 1972; Nwachukwu, 1972); Ofoegbu and Amajor, 1987; Benkhelil et al., 1989). As a result of their abundance, together with high cost of haulage of the other alternative and often more suitable construction materials, these shales serve both as road pavement subgrades and as aggregates in most construction projects. A survey of the roads and other civil engineering projects in the area had, however, revealed that these projects suffer incessant failures.

Previous studies (e.g., Underwood, 1967; Sowers and

Sowers, 1970) note low permeability, perhaps, low hydraulic conductivity, low specific yield and appreciable hardness (Todd, 1980) as some of the attributes of shale formations (that is, aquicludes). This work evaluated impacts of shale units on the geotechnical, hydrogeological and environmental problems prevailing in the Abakaliki area. The evaluation primarily highlights the shales contribution to road pavement failures, shortage and poor quality of groundwater, as well as environmental issues such as flooding and waterlogging consistently experienced in the area.

Geology

The Asu River Group sediments were deposited during the first marine transgression of the Benue Trough, which is generally believed to have started in the mid-Albian period. Fig. 1 shows the distribution of the Asu River Group and other lithologic units in the lower (or southern) Benue trough, Southeastern Nigeria. The Asu River group sediments are predominantly shales, commonly referred to as the 'Abakaliki shale formation' in and around the Abakaliki metropolis (about 452 sq.km), and localized occurrences of sandstone, siltstone and limestone intercalations (Ofoegbu and Amajor, 1987).

Emplaced in these Asu River group sediments are intermediates to basic intrusive, extrusives and

Pyroclastics (Murat, 1972; Nwachukwu, 1972; Ofoegbu and Amajor, 1987). The group has average thickness of about 2000 m and rests unconformably on the Precambrian Basement (Benkhelil et al., 1989). The Abakaliki shale formation, which has an average thickness of about 500 m, is dominantly shale, dark grey in colour, blocky, and non-micaceous in most locations. It is calcareous (calcite-cemented) and deeply weathered to brownish clay in the greater part of the formation (Okogbue and Aghamelu, 2010a).

Physiography and Climate

The Abakaliki area has a generally undulating relief pattern. No location exceeds 400 m above-sea-level, except the Juju Hill, which is at the centre of the metropolis. Hills and hillocks formed by the pyroclastic bodies represent the major relief structure in the area. No trend has been established by previous research (Ofoegbu and Amajor, 1987) of these conical shaped hills and other residual hills that spread sporadically within the area. The predominant shale has favoured the low erodability of the lithology, resulting in absence or near absence of deep cut valleys and erosion channels. The major river that drains the area is the Ebonyi River and its tributaries; Udene and Iyiokwu Rivers. Both tributaries are perennial and usually overflow their banks at the peak of the rains. Stunted trees and pockets of derelict woodland exist where the lithology has undergone high degree of laterization. In other locations, typical characteristics of the tropical rain forest are represented by occurrence of multitude of evergreen trees, climbing plants, parasitic plants that live on the other plants, and creepers.

Two main seasons exist in the Abakaliki area, the dry season which lasts from November to March and the rainy season which begins in April and ends in October with a short period of reduced rains in August commonly referred to as "August break". Temperature in the dry season ranges from 20 to 38°C, and results in high evapotranspiration, while during the rainy season temperature ranges from 16 to 28°C, with generally lower evapotranspiration. The average monthly rainfall ranges from 31 mm in January to 270 mm in July, with the dry season experiencing much reduced volume of rainfall unlike the rainy season, which has high volume of rainfall. Average annual rainfall varies from 1,500 to 1,650 mm. These climatic conditions are responsible for the development of thick lateritic soils in the area.

Land Use

The major economic activity within the Abakaliki area is subsistence agriculture. Statistics show that more than 60% of the population is engaged in it. One of the main cash crops grown is rice. This has necessitated setting up of rice milling industries in the Abakaliki area. The available land for agriculture is fertile and supports rice and cassava cultivation. The main industries in the area, apart from rice milling industry, are quarrying and rock crushing. Lead-zinc mining occurs around Enyigba and Ameka; in the outskirts of Abakaliki metropolis. The traffic comprising mainly of heavy-duty vehicles, resulting from the transportation of agricultural produce and other economic activities, mount pressure on the existing road and highway networks in the area.

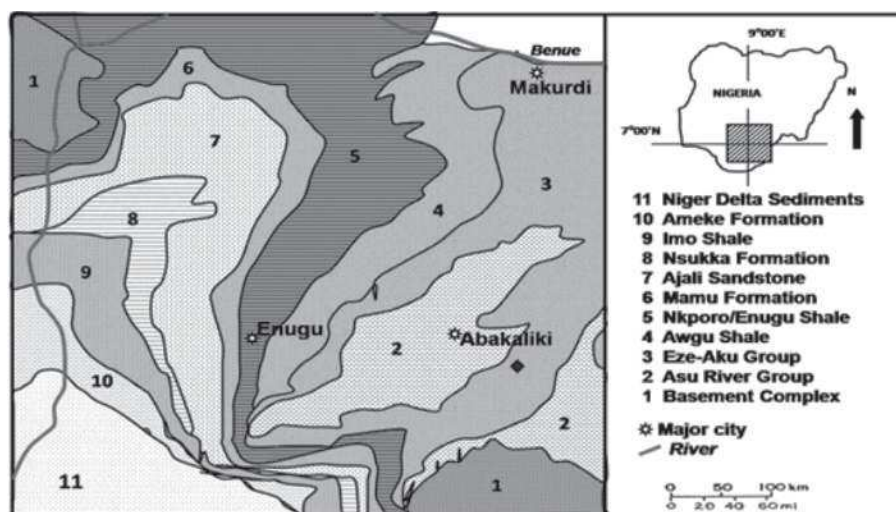


Fig. 1: Generalized Geological map of southeastern showing the distribution of the Asu River Group and other lithologic units (Modified after Okogbue and Aghamelu, 2010b).

Methodology

Sampling

A total of six (6) shale samples were taken from some shale outcrops within the Abakaliki area; two (2) each from Ntezi Abba, Iyiokwu River and Juju Hill. For all the shale samples collected sampling depths ranged from 0.5 to 2.0 m. The shale sampling strictly followed standard procedure for soil sampling specified in British Standard Institution (BSI) 1377 (1990). The *in-situ* samples of the Abakaliki shale produced effervescence on contact with dilute HCl.

The effervescence buttressed calcareous nature of the shale formation. All sampling was done in the dry season, between December and February, 2008. Five (5) water samples (3 from hand dug wells and 2 from boreholes) were collected from Mgbabo and Ogbaraugo areas within the study area. Depth of hand-dug wells sampled ranged 6 to 8 m. Depths of sampled boreholes were not ascertained, however, general borehole depths in the Abakaliki area range between 12 and 55 m. Sterilized 1 L plastic containers were used for conveying the water samples to the laboratory for the analyses of some parameters that could not be carried out in the field. Conservative measures specified by Appelo and Postma (1993) were taken to prevent changes in the chemical composition of the sample before analysis; 100 ml of water sample for Ca^{2+} , Mg^{2+} and Na^+ was acidified to pH >2 using 0.7 ml of 65% HNO_3 , while water sample for NO_3^- was stored cool at 4°C.

Laboratory Analyses

Geotechnical

The tests carried out on the shale samples included Atterberg limits (liquid and plastic limits), specific gravity, natural moisture content, compaction, California Bearing Ratio (CBR) and shear strength. The sample preparations and laboratory testing for these geotechnical parameters followed standard methods of testing soil for civil engineering purposes. The liquid limit and plastic limit tests were carried out on air-dried samples that passed 0.425 mm (BSI No. 36) sieve; both tests then followed standard procedures specified by BSI 1377 (1990). Crushed shale samples that passed through 0.076 mm (BSI No. 200) sieve and oven-dried at 105°C for 24 h were utilized for the specific gravity tests. The tests were also carried out with the aid of a

100.15 ml pycnometer bottle and distilled water with specific gravity of 0.99654 (distilled water temperature was 27°C), in accordance with testing method described by Lambe (1951). Laboratory procedure of this test attempted determination of the specific gravity of the shale excluding air and water contents. Natural moisture contents determination followed simple method outlined by Akyrod (1957). The laboratory compaction tests were limited to particles of shale that passed through 19.05 mm BSI sieve and followed procedure specified by BSI 1377 (1990).

CBR tests were performed on compacted samples in both unsoaked and soaked conditions, following the procedure of Bailey (1976). However, soaking was done overnight (24 h) in a water-filled bathtub. Samples for the shear strength tests were prepared and tested following the BSI 1377 (1990) standard. The test type was the triaxial compression strength test (consolidated undrained). However, the triaxial cell for the tests was the 76 mm diameter type, hence, the tested samples were proportionately scaled down to 152 by 76 mm, height by diameter, respectively. Pressured water was also used for confining the samples inside the cell.

Hydrogeochemical

Water samples were collected from 5 different groundwater sources (hand dug wells and boreholes) and tested for chemical constituents. The tests included pH, TDS, hardness and basic ions; while Ca^{2+} , Mg^{2+} and Na^+ , were the cations tested for, Cl^- , HCO_3^- and NO_3^- were the anions determined in the water samples (all ions measured in mg/l). Apart from pH which was determined in the field using HACH portable pH meter, the rest of the parameters in the water samples were analyses in the laboratory adopting analytical procedures specified by the American Association of Public Health (APHA, 1985) standards. All the water samples were, however, analyzed within 24 h of collection, at the Analytical Laboratory of the Department of Chemistry, Federal University, Ndufu-Alike, Ikwo.

Results and Discussions

Geotechnical Properties

The summary of the geotechnical properties of the Abakaliki shale is presented in Table 1. As shown in the table, the Abakaliki shale recorded values of Liquid Limit (LL) which ranged from 49 to 54, Plastic Limit

(PL) 15 to 18. The PI was considerably high (ranged from 34 to 38). Sowers and Sowers (1970) note that PI >31 should be considered high and indicates high content of expansive clays, most probably illite or illite-montmorillonite mixed layer. Obiora and Umeji (2004) notes predominance of illite and mixed layer clay minerals in the Asu River Group sediments.

On the basis of LL and PI values, the Abakaliki shale could be classified as moderate to high plasticity soil, in accordance with Sowers and Sowers' (1970) rating. Such a soil usually has the ability to retain appreciable amount of total moisture in the diffuse double layer, especially by means of absorption. This fact was buttressed by a significant mean value (20%) of natural moisture content recorded by the shale samples (Table 1). Moderate to high plasticity materials are usually susceptible to high compressibility (Seed and Woodward, 1964; Sowers and Sowers, 1970; Coduto, 1999). An increase in plasticity of material also decreases its permeability and hydraulic conductivity (Sowers and Sowers, 1970).

Table 1: Summary of the geotechnical properties of the tested samples of the Abakaliki Shales

Property	Sample location		
	Ntezi Abba	Iyiokwu River	Juju Hill
LL (%)	54	53	49
PL (%)	18	15	15
PI (%)	36	38	34
SG	2.28	2.25	2.42
NMC (%)	17	23	20
MDD (mg/m³)	1.85	1.89	1.83
OMC (%)	12.1	12.0	12.5
CBR	22	16	22
(Unsoaked)			
(%)			
CBR (Soaked)	11	10	12
(%)			
φ (°)	23	30	31
c (kN/m²)	30	50	55

Results indicated that the Abakaliki shales has mean Gs value of 2.32 (Table 1); a value less than that of a potentially durable construction aggregate, which should have Gs value of 2.625 or above, according to Reidenouer (1970). A comparison of specific gravity values of rock aggregates that serve well in construction project with the Abakaliki shales (Table 2) points out the fact that the materials are unlikely to be durable in most construction projects. Ezeribe (1994) on the basis of slake durability assessment, rates shale samples

collected from two locations on the Asu River Group (Ishiagu and Amenu Okposi areas) as very low durability shales.

Table 2: Average specific gravity of various rock types

Rock type	SG ^a
Igneous	
Andesite	2.22
Basalt	2.77
Gabbro	3.0
Granite	2.67
Rhyolite	2.4
Sedimentary	
Limestone	2.69
Sandstone	2.58
Shale^b	2.3 - 2.4
Metamorphic	
Gneiss	3.12
Marble	2.73
Slate	2.77

^aapparent SG data from Krynine and Judd (1957)

^bbulk SG data from this study

Results indicated that the compacted Abakaliki shale achieved mean Maximum Dry Density (MDD) value of 1.86 Mg/m³ at mean Optimum Moisture Content (OMC) of 12.2 (Table 1). The MDD was considerably high; MDD of other shales from Southeastern Nigeria ranges from 1.50 to 1.68 Mg/m³ (Okogbue and Aghamelu, 2010b). In addition to compactive effort, Okogbue and Aghamelu (2010b) point out that calcite cementation, nature of the compacted sample and moisture condition of the sample may have also contributed to the relatively high MDD recorded by the Abakaliki shale. Results of the laboratory CBR tests showed that the mean CBR values of 20 and 11%, for soaked and unsoaked conditions, respectively, were recorded in the Abakaliki shale. The CBR values showed a reduction in the stability of the shale to about 45%. The reduction in CBR suggested that moisture influx would be detrimental to subgrades of pavements constructed with the Abakaliki shale. Clay mineralogy of the shales, might also be responsible for reduction in CBR, which was a geotechnical signal for cautions when used as pavement material.

Shear strength showed that the angle of internal friction for Abakaliki shales ranged from 23 to 31°C, while

cohesion ranged from 30 to 55kN/m². Coduto (1999) and Punmia et al. (2005) remark that shales that are predominated only by clays and areas well non-cemented, most often, record very low values of f (close to 10°C). Reasonable values of c recorded denoted a cohesive material, similar to consolidated clay and other materials with high clay content. The fact that values of c were significantly greater than 10°C and c reasonably high would suggest that the Abakaliki shale as subgrade and construction material would have considerable strength, and is likely to withstand shear stresses. Moisture influx, however, would deteriorate its constituent minerals, especial clays, resulting in strength reduction and perhaps, bearing capacity loss, during the engineering life of such project. CBR values reduction have already clued-up such suspicion.

Water Quality

The summary of the hydrogeochemical investigations is presented in Table 3. Average pH values for both hand dug wells and boreholes were almost equal. Mean pH for all the tested water samples, however, showed neutrality (6.9 on the pH scale). A comparison of the concentrations of the tested water samples with normal concentrations of elements in unpolluted fresh water (data presented in Table 5) showed the tested water samples had high values of ions, with HCO₃⁻ and NO₃⁻ being exceptionally high (above normal range). Probable source of the ions in the water included dissolution of calcite and other rock salts, as well as mica and organic matters within the shale (Table 5). Occurrences of siltstone, sandstone and limestone beds have also been noted in the Abakaliki shale (Reyment,

1965). NO₃⁻ source might also be anthropogenic that is (due to activities of human beings in the area).

A borehole in the Southwestern part of the area at (Abiaji Ogharaugo) recorded highest enrichment of Ca²⁺, HCO₃⁻ and carbonate hardness. This may be either due to the fact that the borehole tapped deeper shaley bed that have secondary enrichment due to effects of leaching or that static that is (steady) and deeper water table in boreholes aided more dissolution. Most hand dug wells, and even some boreholes, in the Abakaliki area dry up in the middle of dry season leaving the town with acute shortage of portable water. In the Abakaliki area, ground water yield is poor and water mainly exists in fractured and weathered zones within the Asu River Group.

According to Ismael (1990), the fractured and weathered zone shale-aquifer systems have specific yields values as follows; range 1.26×10^{-3} to 1.60×10^{-3} litre per second (l/s) and 1.22×10^{-3} to 1.81×10^{-3} l/s, with average values of 1.48×10^{-3} l/s and 1.59×10^{-3} l/s, respectively. Both shale-aquifer systems have only served for the establishment of hand-pump boreholes and hand-dug wells. All the water sources recorded considerable amounts of Total Dissolved Solids (TDS) and appreciable hardness.

Hardness in water is caused by the presence of any polyvalent metallic cations (Kiely, 1997) but principally Ca²⁺, Mg²⁺ and, less so, Fe²⁺ and Mn²⁺. Kiely (1997) notes that total hardness is usually computed based on the concentration of Ca²⁺ and Mg²⁺ and is usually expressed in mg/l as CaCO₃. Details on the softening processes and

Table 3(a): Hydrogeochemistry of the water samples from the Abakaliki area

Sample location	Source type	Parameter ^a								
		pH	Ca ²⁺	Mg ²⁺	Na ⁺	Cl ⁻	HCO ₃ ⁻	NO ₃ ⁻	Hardness	TDS
Mgbabo Achara	Hand dug well	6.2	8.0	0.2	4.9	7.0	21.8	15.0	17.9	56.9
Mgbabo Achara	Borehole	6.7	88.0	3.0	10.1	4.0	392.0	15.0	322.2	512.1
AzuIsuma, Ogharaugo	Hand dug well	7.4	8.0	0.6	2.0	2.0	37.0	15.0	30.4	64.6
Abiaji, Ogharaugo	Borehole	6.8	134.0	3.3	10.7	71.0	1176.1	20.0	966.6	1415.0
Okposi, Umuogharu	Hand dug well	7.2	26.0	3.3	5.7	3.0	67.52	20.0	55.5	125.5

^aall in mg/l, except pH

^bhardness as CaCO₃ concentration

Table 3(b): An engineering evaluation of the geotechnical properties of the Abakaliki shales

Properties	Average range of values (Underwood, 1967)		Abakaliki shales	Remarks
	Unfavourable	Favourable		
c (kN/m ²)	35 - 700	700 - 10,500	30 - 55	Unfavourable
ϕ (°)	10 - 20	20 - 65	23 - 31	Favourable
NMC (%)	1.13 - 1.76	1.76 - 2.56	1.83 - 1.89	Favourable
Dry density (Mg/m ³)	20 - 35	5 - 15	17 - 23	Unfavourable
Predominant clay minerals	Montmorillonite, illite	Kaolinite, chlorite	Illite ^a	Unfavourable

^adata from Obiora and Umeji (2004)

other physical, chemical and microbiological techniques that can be utilized to achieve acceptable standard for the groundwater in the Abakaliki area are presented in American Water Works Association(AWWA) (1990), Thomas and King (1991) and Kiely(1997).

All the water samples, from both hand dug wells and boreholes, could be classified as 'fresh', due to the fact that their TDS values all fall within the range of 0 – 1000mg/l (Freeze and Cherry, 1979; Fetter, 1990). On the basis of mean TDS values, however, water samples from hand dug wells would classify as moderately hard, while those from boreholes would classify as very hard (Sawyer and McCarty, 1967).

Conclusions

1. Geotechnical assessment showed that the Abakaliki shales are of moderate to high plasticity, most probably, due to its content of illite and mixed layer clay minerals. Attributes of material with high plasticity, like the Abakaliki shale, include swelling on moisture influx, high compressibility, low bearing capacity and low permeability that is (low drainage). These properties, most probably, would negate few favourable properties, like high MDD and high shear strength parameters (Table 6), recorded by the shale and ultimately render it unsuitable for use for most construction purposes, especially where moisture influx cannot be effectively controlled.
2. Hydrogeochemical analyses indicated that the water samples from the area near neutrality, but showed high concentration of cations and anions, resulting in high values of hardness and TDS. Dissolution of the constituent minerals, especially calcite and organic

matter, might likely be the source of these particles in the water. Enrichment of calcium and carbonate ions was observed in one of the tested boreholes within the area and probable explanations to this would be either that the effect of element leaching mechanism was most in the area or that steady water table in the borehole (depths of boreholes are generally higher than that of hand dug wells) necessitated higher degree and persistent dissolution of minerals in the borehole area.

3. Although a general situation of high HCO_3^- , TDS and hardness was observed in the water samples, it requires comparison with established standards, for example World Health Organization (WHO, 2006) guidelines, to ascertain the appropriateness of the water samples for use for various purposes. Treatment of groundwater hardness in the area is possible and could follow standard procedure by American Water Works Association (AWWA) (1990), Thomas and King (1991) and Kiely (1997).
4. On the issue on environmental hazards in the area; high affinity of plasticity material that is (the Abakaliki Shale) for water, its low permeability, topography of the area and high volume of rainwater and runoffs, result to water logging and flooding in the middle of the rainy season, while the same affinity of plasticity material for water, highly reduced volume of rainwater and increased evapotranspiration result to hand dug well and some borehole dry-ups, as well as differential settlement and structural damages of engineering projects founded within the shale formation.

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Determination of Suitable Site for Solid Waste Disposal Using GIS-Based Multi-criteria Analysis

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Abstract

The need to identify a suitable site for disposal of solid waste in urban area is crucial due to the increasing urbanization with multiplying quantity of waste generation within the cities. The current system is manual, costly, labour intensive and time consuming. For this reason a study was conducted to identify a suitable site and verify the suitability of the existing dumpsite in Ado-Odo Ota using GIS-based Multi-Criteria Evaluation (MCE). This technique is faster, easier, more economical and effective than manual method. In this paper, Landsat Enhanced Thematic Mapper plus (ETM+) 2016 and Shuttle Radar Topographic Mission (SRTM) imagery along with soil map and topography map were used. Both primary and secondary data collected were used to create and classify five criteria such as soil type, built up area, water body and river, digital elevation model (slope) and roads in relation to waste disposal site selection. These criteria were examined using spatial analytical tools in ArcGIS 10.4, and each criterion was identified and weighted using Analytical Hierarchy Process by pairwise comparison matrix to create a suitability map for waste disposal with considerations to environmental laws and local conditions. Existing waste disposal site was overlain on suitability map for evaluation. The result showed that an area of 741.15km² of the total area was unsuitable, 79.11km² was poorly suitable, 55.08km² was moderately suitable and 7.38km² was most suitable. The existing waste disposal site was found to fail its suitability test.

Keywords: Waste disposal, Geographic Information System, Multi-criteria analysis, Landsat, Radar imagery

Introduction

Solid waste disposal is an important part of waste management system, which requires much attention to avoid creation of environmental pollution and health problems (Chaulya, 2003). The indiscriminate dumping of wastes into water bodies, open, abandoned lands and any appealing sites without proper treatment have led to serious environmental pollution and health-related problems in many cities (Adeofun et al., 2011). According to the United State Environmental Protection Agency (USEPA, 2005), waste is defined as any garbage, refuse, sludge from a waste water treatment plant, water supply treatment plant or air dried material, including solid, liquid, semi-solid or gaseous material resulting from industrial, commercial mining and agricultural operations and from community activities.

There are various methods of disposing solid waste but among the options (recycling, incineration and composting), landfill is the most preferred because of its easy mode of operation, low cost and less technological involvement (Kontos et al., 2005; Yesilnacar and Cetin 2005). In order to locate a good sanitary landfill, it requires overcoming some environmental and political restriction. The situation of solid waste disposal in Ado-Odo Ota area poses a serious threat to the environment which is not different from other urban areas in Nigerian

with poor waste management system. As a result of urbanization and industrialization, the population of Otta has grossly increased and the waste generated has also increased. Hence, waste generated is indiscriminately dumped at roadsides, disposed directly in the soil or water courses, without any concern to the environment around the metropolis. This has raised concerns regarding public health and safety, property values, community pride, and overall quality of life, pollute the air, soil and sometimes ground and surface water as well as deteriorate the beauty of the area (Botkin and Keller, 1998.)

The first important step in planning solid waste landfill is the site selection for solid waste disposal (Waele et al., 2004). Sitting a landfill is a complicated process because it involves social, environmental and technical parameters. The disposal site must not cause damage to biophysical environment and the ecology of the surrounding, economic factors and geomorphologic features must be considered, also. Several techniques have been found in literatures for site selection (Lucasheh et al., 2001; Sani et al., 2010; Sener et al., 2006; Gemitzi et al., 2006; Mutluturk and Karaguzel, 2007). Daneshvar et al. (2005) ascertained that GIS is an ultimate method for preliminary site selection as it efficiently stores, retrieves, analyses and displays information according to user-defined specification.

The technology is an important tool for solid waste site selection. Many researchers (Akinpelumi, 2011; Onuigbo, 2014; Mohammed et al., 2014; Yahaya et al., 2010; and Suman, 2012) have employed the use of GIS in site selection and solid waste disposal study. The overall GIS-based approach to landfill site selection process involves evaluation of geology, hydrogeology, and morphological properties and assessment of potential landfill sites based on various criteria by weighting linear combination (Allen et al., 2003). This work aims to identify the most suitable site for waste disposal using multi-criteria analyses function in ArcGIS software.

Objectives

The main objective of this work is to identify a suitable site for dumping solid waste in Ado-Odo Ota LGA to overcome the menace of roadside dumping using GIS

technique. Other specific objectives are to identify and assess the suitability of the existing dumpsite in Ota and determine potential location for solid waste disposal using multi criteria analysis.

The Study Area

The study area is the Ado-Odo Ota local government in Ogun State, Nigeria. Geographically, it lies between Latitude $6^{\circ}30' - 6^{\circ}46'N$ and Longitude $2^{\circ}51'$ and $3^{\circ}16'E$. This LGA was created in May 29th 1989 and covers a land area of 1,263 square kilometer. It is the most industrial area with industrial community area of 878 km² and a population of 526,565 at the 2006 census. The geology of the area belongs to the Tertiary age, made up of sedimentary rocks. It comprises of Abeokuta and Ilaro Formations and Coastal Plan sands (Ugbe 2011). Soils in the area are poorly drained, and varies from alluvial soils, clay, sand and silt.

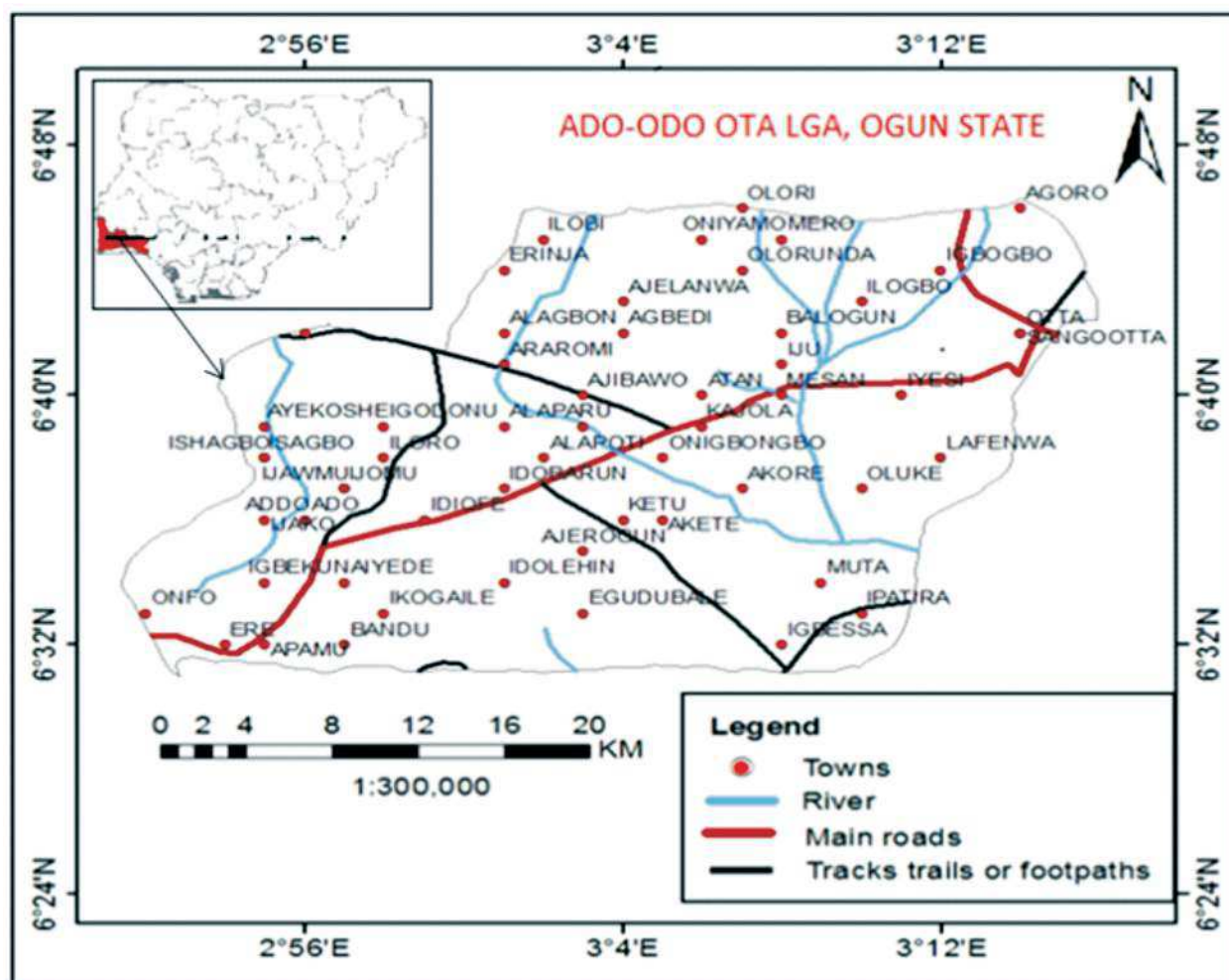


Fig. 1: Map of Nigeria (top left) showing the study area

Materials and Methods

Materials

The dataset used include topographic, geologic map and geotechnical test data. 1: 50,000 scale topographic map was georeferenced and digitized to obtain digital elevation model (10x10m cell size), delineate river and road network, Landsat ETM+, 30m resolution was used to generate landuse map, SRTM imagery covering the study area was also used to generate slope map respectively. Soil map was used to extract the soil types.

Field Investigation

The existing dumpsite in Ota was visited and soil samples were collected using hand auger at a depth of 1.5 meters subsurface and geographic positioning system (GPS) to determine point location. In-situ moisture content was determined using the Thomas Ashworth speedy meter and groundwater conditions from available records. In the field, visual examination and description of soil samples and topographic notes were taken.

Laboratory Tests

The collected soil samples were earlier subjected to laboratory test in accordance with BS 1377 (1990) and ASTM D4318-00 (ASTM, 2000) test methods to characterize the engineering properties such as sieve and hydrometer analyses, Atterberg limits, water content and density. The soils were classified according to Unified Soil Classification System (USCS).

Data Analysis

The Landsat data was imported to ENVI 5.0 to classify the different landuse types. Both unsupervised and supervised classifications were done using maximum likelihood classifier. The classified image exported to ArcGIS for further analysis and slope map was generated from Landsat ETM+ imagery. Five different physical criteria examined were slope, built-up, soil, roads, and waterbody. Each criterion was identified and weighted by analytical hierarchy process (AHP) and mapped using GIS. The methodology was divided into two sub-methods: creation of landuse/landcover maps and site selection criteria using weighted linear combination following the formulated criteria from environmental protection agency (EPA) manual 2006 (Table 1). AHP is a flexible, powerful and easy to use

tool in multi-criteria decision making and it is used in the site selection for suitable landfill. The main feature of this method is based on pair-wise criteria comparison weightage which is given to individual factor before overlaying operation (Sener et al., 2010). Overlay operation such as creation of factor maps, Euclidean distance analysis, reclassification and weighted overlay were performed to generate suitability map.

Table 1: Factor Criteria formulated from EPA landfill manual 2006

Criteria	Least Suitable	Moderately suitable	Highly suitable
Distance to water body & river	160m - 480m	480m - 960m	> 960
Slope	10 - 20°	5° - 10°	0° - 5°
Distance to Road	> 2000m	1000m - 2000m	100m - 1000m
Distance Built up Area	300m - 500m	500 - 800m	> 800m
Soil	Fine sand	Silty sand	Clay

Results and Discussion

Landuse Map

The land cover type is given in Figure 2 which classified the study area into five classes such as light vegetation, thick vegetation, built-up area, bare soil and waterbody (natural and artificial) as shown in the landuse map. Light vegetation covered over 64% of the area while 20% was covered by thick vegetation. Only 15% was used as built-up area, 0.36% as waterbody and 0.63 as bare soil/hills.

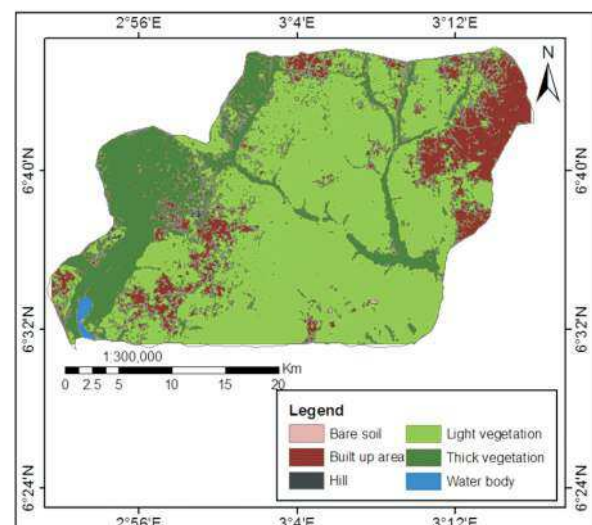


Fig. 2: Landuse map of the study area

Soil Type Map

As shown in Figure 3, the soil types comprised of clay, loam, fine sand, sandy loam, sandy clay loam and loamy fine sand. When the existing dumped site was overlain on the soil map to assess its suitability, the result showed that the site was built on the highly permeable loamy fine sand soil failing to meet the stipulated standard by Environmental Protection Agency (EPA) manual for landfills 2006 (Soil with low permeability e.g. clay rich environment is required).

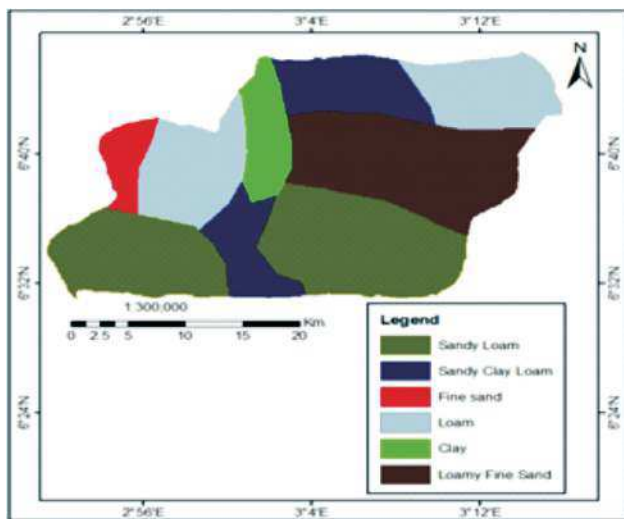


Fig. 3: Soil map of the study area

Slope Map

Slope is another important factor considered for citing a dumpsite as higher slope will increase the runoff and pollutants from the dumpsite, thereby contaminate area farther away and perhaps groundwater (Lin & Kao, 1999). The slope map derived from DEM is shown in Figure 4. From the result, the existing dumpsite is located on a high slope (10–15°) which is a less suitable region based on EPA criteria in Table 1.

Water Body and Drainage Map

A dumpsite is not meant to be located near water resource to protect surface water from contamination by leachate. Safe distance from river is required to prevent waste from eroding into rivers and major streams. Based on EPA criteria in Table 1, the water body and drainage map showed a close distance of about 200m to the landfill (Figure 5) which makes it least suitable. From existing record, the area has high groundwater table (Aizebeokhai, 2011).

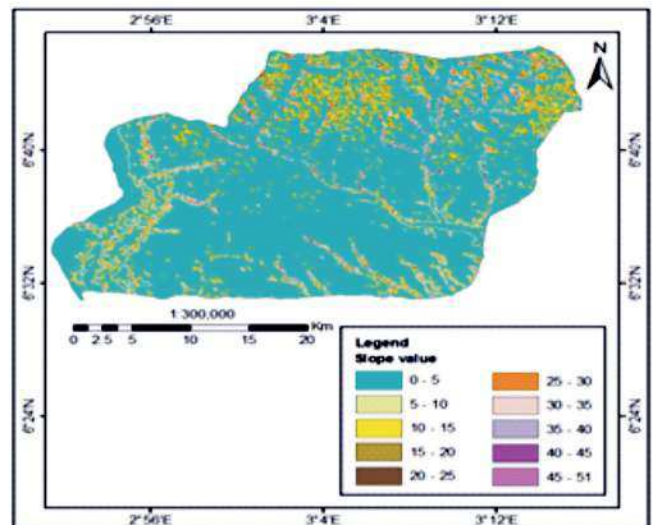


Fig. 4: Slope map of the study area

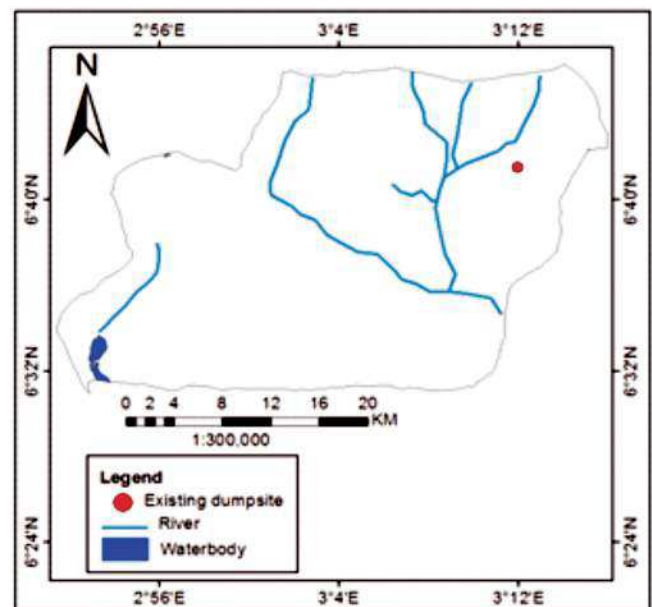


Fig. 5: Water body and drainage map

Built-Up Area and Road Map

The built up area map extracted from the landuse map generated from Landsat ETM+ is seen to be too close to the dumpsite about 300m (Figure 6). According to EPA criteria in Table 1, the allowable stipulated distance to dumpsite has to be greater than 500m and minimum distance to road is 100m. This makes the dumpsite location unsuitable while the road is in good condition and its distance within the stipulated 100 – 1000m by EPA standard (Figure 7).

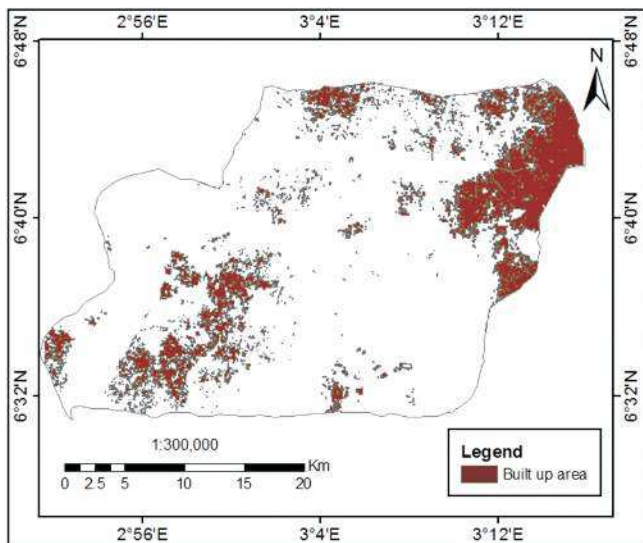


Fig. 6: Built-up area map

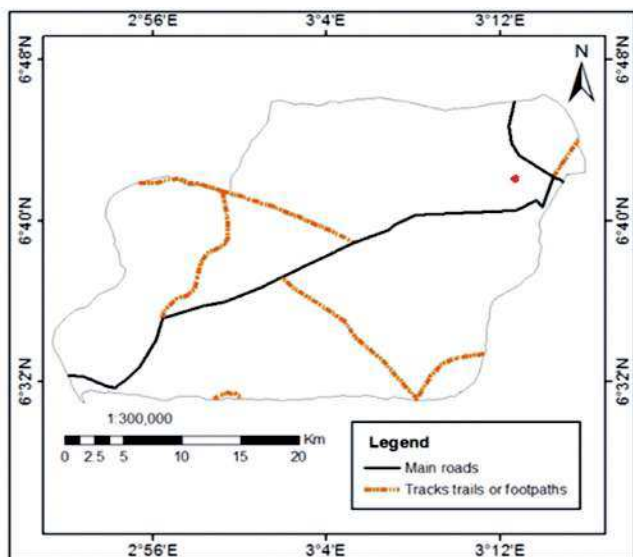


Fig. 7: Road network map

The Buffering and Weighted Overlay Analysis

This is one of the most used approaches for overlay analysis to solve multi-criteria problems such as site selection and suitability models. A set of buffering analysis were performed on the drainage, landcover and road layers at 160m, 30m, and 200m distance adapted from the specification of EPA and overlaid with the candidate site. It was seen that the site overlapped the resources with economic and ecological values. For the weighted overlay analysis, each of the criteria layer was ranked on a scale of 1 to 10 and further reclassified into a scale of 1-3 for easy identification. The result showed

that only a small area of 7.38km² was most suitable where the EPA criteria was fulfilled while a large area of 741.15km² was unsuitable as illustrated in figure 8. This suitability map was used to evaluate the existing dumpsite and was seen to have failed to meet the required standard according to the Environmental Protection Agency 2006.

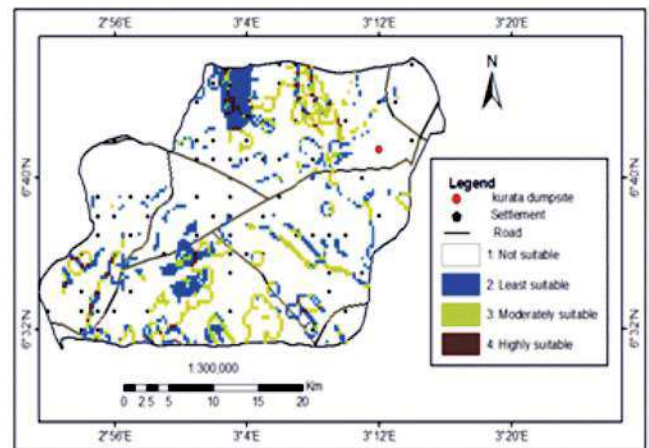


Fig. 8: The Kurata dumpsite overlaid on the suitability map

Conclusions and Recommendations

The present study attempted to locate a suitable dumpsite and also assess the existing one using GIS approach based on multi criteria analysis. According to the AHP technique, the weight of each criteria was calculated. All the five criteria were mapped ranging from distance to drainage, built-up, road network, soil type, and slope using remote sensing and GIS techniques and structured in a geo-database to ensure data consistency during spatial analysis such as slope analysis, Euclidean distance analysis, reclassification, buffering and weighted overlay. Result showed that only the road network criterion met the required EPA standard. Solid waste disposal in Ado-Odo Ota at its present site did not fulfill the required conditions. Furthermore, the high rate of population growth will inevitably make the problem more severe in future if care is not taken to relocate the site to a more suitable area.

Having identified the most suitable area to cite a dumpsite, it is therefore recommended that the Environmental Department of the Local Government Areas and the Town Planning Authority should relocate the Kurata dumpsite to a more suitable place while using this model as a guide before any approval for dumpsite.

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Deterioration of Some Pelitic Rock Aggregates from Asu-River Group as a Function of their Individual Mineralogy and Water Sorption

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Abstract

Ten pelitic rock samples collected from five Albian Asu-River Group outcrops were subjected to x-ray diffraction (X-RD) to determine their dominant and water-absorbent minerals. The samples were also subjected to long soaking and complete drying degradability test to determine their susceptibility to deterioration caused by inter-seasonal wetting and drying and water sorption potentials. The X-RD (water-absorbent minerals) and degradability tests results were subjected to regression analyses to ascertain the water-absorbent mineral species that contribute mostly to deterioration of the rocks. The X-RD results revealed that the rock samples were dominated by micas, amphibole and serpentines while nine of the analyzed samples contain significant (>35%) amount of water-absorbent minerals notably: clay-minerals, zeolite and oxides/hydroxides. The degradability test results revealed that five samples, which showed significant (>1%) deterioration, do not absorb the same amount of water in every wetting process. Combined results of X-RD, degradability and regression analyses revealed that amongst the identified water-absorbent minerals, zeolite and oxide/hydroxide contribute most to deterioration of the rocks. This work has shown that individual water-absorbent mineral species has more deterioration effect on pelitic rocks than the summed water-absorbent minerals and that deterioration of pelitic rocks is followed by increased water sorption.

Keywords: Deterioration, Pelitic Rock, Water-absorbent Minerals, Degradability, Water Sorption

Introduction

Rock deterioration is the physical depreciation of rock when exposed to weathering agents like water, heat and/or organisms. The extent to which different rocks deteriorate is a test of their relative durability. Rock deterioration is controlled both by its (rock) inherent properties like mineralogy and porosity and by environmental factors like rainfall (wetting) and temperature. Works by Fitzner and Kalde (1991) and Weiss (1992) had shown that the hygroscopic properties (water-absorbing potential) of minerals contained in a rock control the rate at which the rock deteriorates. Rocks rich in water-absorbent/soluble minerals like clay-minerals, zeolites, feldspar and calcite are susceptible to deterioration. Such rocks deteriorate due to repeated expansion and contraction caused by cyclic wetting and drying respectively (Hudec, 1980; Christaras, 1991 and Ruedrich et al, 2010).

Works by Hudec (1993), Cobanoglu et al (2003) and Garcia-del-cura et al (2011) reveal that the extent to which rocks absorb water and deteriorate is controlled not only by their mineralogy but also by their effective porosity (connected pores). Rocks rich in water-absorbent minerals and high effective porosity will deteriorate more than those rich in water-absorbent minerals but have low effective porosity. The above

cited authors based their assessment of rock deterioration on the summed water-absorbent minerals and also paid no attention to the amount of water absorbed in both deterioration and non-deterioration periods of the rocks.

The present work assesses the contribution of individual water-absorbent minerals to deterioration of some pelitic rocks and compares the amount of water absorbed during deterioration cycles with the amount absorbed during non-deterioration cycles. This work shall edify geoscientists that not all water-absorbent mineral contained in a rock contribute to deterioration of the rock and that the moisture absorbed by rocks susceptible to deterioration is not the same in every wetting period.

Regional Geology and Climate

The study area is underlain by the Albian Asu-River Group. As the oldest litho-stratigraphic unit, Asu-River Group occurs at the core of the major sedimentary basin (Benue Trough) in Nigeria (Nwajide, 2013). According to Grant (1971) and Burke et al (1971), the Benue Trough is the third failed arm of a triple rift system formed due to the Neocomian/early Gallic Epoch separation of South American plate from African plate. The Basin received its first Asu-River sediments, which

is composed mostly of shales, limestones and sandstones, from bordering Basement complex in Albian Stage (Obiora, 2002; Obiora and Umeji, 2004). These Asu-River sediments were also intruded by pyroclastics at some places.

The part of Asu-River Group studied is within the South-eastern Nigeria. According to Kottek et al (2006) and Peel et al (2007) South-eastern Nigeria is within the tropical savanna climatic region characterized by wet and dry seasons. The wet season occurs from April to October with monthly average precipitation varying from 90mm to 250mm and atmospheric temperature varying from 24°C to 32°C. Dry season occurs from November to March with monthly average precipitation varying from 15mm to 50mm and atmospheric temperature varying from 22°C to 32°C. During the wet seasons, rocks experience saturation due to high precipitation (rain/dew) while during dry seasons they (rocks) are completely dried due to prolonged high temperature and very low precipitation. Generally, the repeated rock saturation common during the wet seasons has its effects on rocks particularly the excavated/quarried ones.

Study Methods and Tests

Field Study

This study was done between latitudes 6°11"N and 6°23"N and between longitudes 8°02"E and 8°11"E (Fig. 1) from where ten pelitic rock samples were collected from five rock quarry outcrops. The pelitic rock outcrops were located at Onyikwa, Umuoghara, Agu-Akpu, Ezzamgbo and Enyigba towns of Ebonyi state. According to Ugwoke (2014) and Okogbue and Ugwoke (2015), rocks occurring at these outcrops have aggregate impact value and Los Angeles abrasion value acceptable as good rock aggregates.

However, it was observed at locations also indicated in Fig. 1 that the pelitic rocks do not serve as good aggregates for unpaved structures like road embankments and pavement base. The rocks start deteriorating after about 4 months of exposure to cyclic wetting and drying. The deterioration manifests in form of cracking and spalling. Thus, the mechanical competency and low durability (deterioration) of the rocks necessitated the present study.

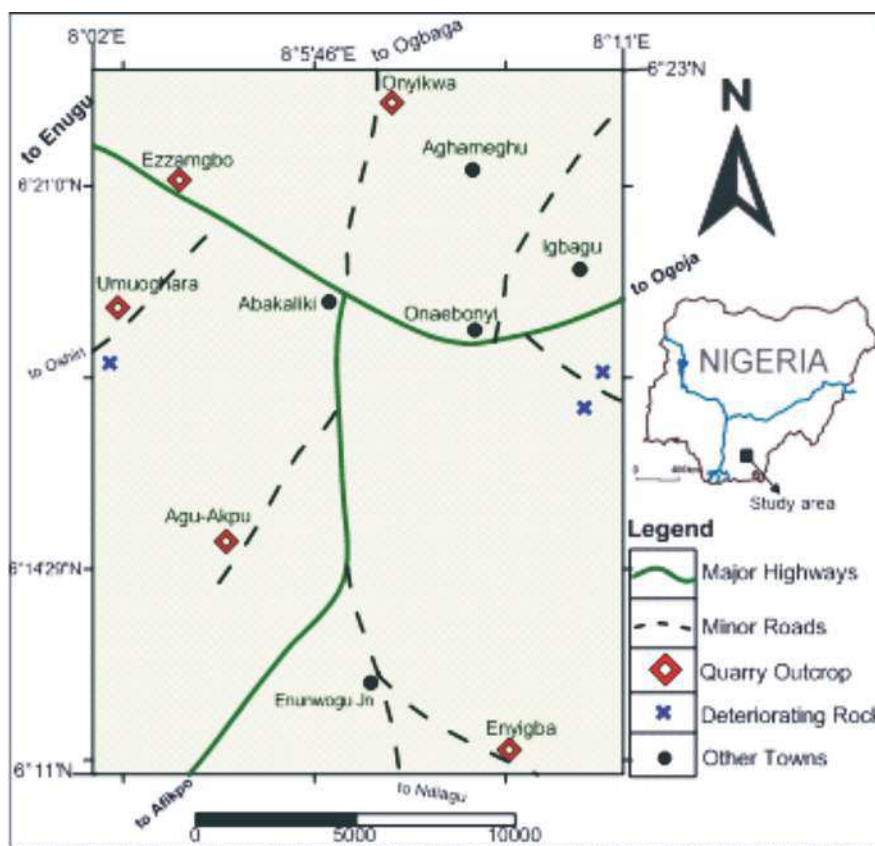


Fig. 1: Map showing quarry units from where the analyzed rock samples were collected and locations of the observed deteriorating rocks

Sampling

Each of the rock samples collected from the five quarry outcrops was divided into two portions. The first portion was subjected to x-ray diffraction (X-RD) test to identify their dominant and water-absorbent minerals. The second portion was subjected to long soaking and complete drying degradability test to simulate inter-seasonal wetting and drying common in south-eastern Nigeria and also to ascertain the variation in amount of water absorbed between deterioration periods and non-deterioration periods. Results of the X-RD and degradability tests were further subjected to regression analysis to determine the water-absorbent minerals that contribute most to deterioration of the rocks.

X-ray Diffraction Test

About 5g of dry rock sample was pulverized and subjected to X-ray diffraction test using Shimadzu X-Ray diffractometer (XRD-6000) for a scan range of 0 and 70 2θ to generate the diffractogram. Peaks of the diffractogram denote minerals present in the analyzed sample. The minerals were identified and labelled by matching the peaks with software mineral cards installed in the diffractometer. The peak matching was done two times - to identify the dominant minerals and water-absorbent minerals contained in the rock. Relative abundances of the identified minerals were calculated from heights and population of the peaks.

Degradability Test

A batch of dry lump rock test samples, weighing between 120g and 180g, were washed with water and finger pressure to remove loose/dust particles adhering to their surfaces, surface-dried with water-absorbent cloth to dry the water adhering to the surface and air-dried for 24 hours to achieve its natural dryness. The weight range 120g to 180g, which is triple the slake durability sample weight range of 40g to 60g (ASTM D4644-87, 1998), was used in order to observe a bigger picture of the samples deterioration as the use of the 40g to 60g weight range in trial tests resulted to near total breakdown of some samples and hence difficulty in assessing their actual degree of deterioration.

The test sample was weighed (W_{dry1st}), placed in a porcelain (non-corrodible) can and filled with potable water of natural climatic temperature (23°C to 26°C). The temperature was determined with a mercury glass-tube thermometer inserted in the water and the set-up

was left for 6 days (144 hours). The water was carefully decanted and the sample washed with fresh water and finger pressure. Particle(s) that got detached during the soaking and/or washing process was/were carefully picked, oven-dried at 105°C for 24 hours to achieve complete drying, cooled to ambient temperature and thereafter weighed (W_p). The intact rock samples were surface-dried with water-absorbent cloth to dry water adhering to their surfaces, weighed (W_{wet}) and oven-dried at 105°C for 24 hours to achieve complete drying. This process was repeated for 6 cycles simulating 6 years of wet and dry seasons. At the end of each cycle, the completely dried intact rock was used as the next subsequent test sample (W_{dry}). The percentage mass of water absorbed during each soaking ($WtAb$) and cumulative percentage of mass lost (Ma_{Lost}) were calculated using Eq. (1) and Eq. (2) respectively.

$$WtAb = \left[\frac{W_{wet} - W_{dry}}{W_{dry1st}} \right] \times 100\% \dots \dots \dots (1)$$

$$Ma_{Lost} = \left[\frac{\sum_{i=1}^n W_p}{W_{dry1st}} \right] \times 100\% \dots \dots \dots (2)$$

Where

W_{wet} ≤ Weight of intact rock after soaking
 W_{dry} ≤ Weight of intact rock after complete drying
 W_{dry1st} ≤ Dry weight of intact rock before first soaking
 W_p ≤ Dry weight of the detached particles

$$\sum_{i=1}^n W_p = \text{sum of } W_p$$

Regression Analysis

The cumulative percentages of mass lost (Ma_{Lost}) and the identified water-absorbent mineral groups were subjected to statistical regression analysis using Statistical Package for Social Sciences (SPSS 16.0) to access the mineral group(s) that contributes most to deterioration of the samples. The analysis was done following stepwise linear regression method; using Ma_{Lost} as the dependent variable and the amounts (in % by mass) of water-absorbent mineral groups as the predictor variables.

Results and Discussions

Examples of the labelled X-RD diffractogram are shown in Fig. 2. The minerals contained in the rocks are shown in Table 1 while the cumulative relative abundance of the water-absorbent mineral groups is shown as bar chart in Fig. 3. Cumulative percentage of mass lost by the samples due to the degradability test is

shown in Fig. 4. The regression analysis between the cumulative percentage mass lost and identified water-absorbent minerals is shown in Table 2. Table 1 and Fig. 3 reveal that among the water-absorbent minerals (clay-minerals, zeolites, oxides/hydroxides and feldspar), identified in the rocks, clay-minerals have the highest proportion.

Then again, among the identified clay-minerals, Illite is the commonest phase. Illite occurs in 9 samples while each of smectite and kaolinite phases occurs in 5

samples. Einsele (2000) and Nichols (2009) had earlier stated that illite is more stable to diagenetic and metamorphic processes than kaolinite and smectite. According to Dunoyer De Segonzac (2011), kaolinite and smectite phases progressively transform to other phases like nacrite, dickite, chlorite and/or illite as diagenesis and metamorphism intensifies. It can also be seen from Table 1 that 5 samples (N1, N2, N3, Z1 and G2) containing pure kaolinite also contain nacrite and dickite.

Table 1: Relative abundance of minerals contained in the analyzed rock samples

Sample	Mineralogy (the mineral groups and their relative abundance are indicated in parenthesis)		
	Water-absorbent minerals		
	Dominant minerals	Clay minerals	Other water-absorbent minerals
N1	Muscovite (mc, 23.37), Riebeckite (am, 18.89)	Kaolinite (k-cl, 14.43), Dickite (cl, 10.14), Illite (i-cl, 9.40), Halloysite (cl, 7.87), Nacrite (k-cl, 6.24), Montmorillonite (s-cl, 0.74)	Sanidine (fd, 8.78)
N2	Grunerite (am, 33.25), Osumillite (mi, 16.45)	Montmorillonite (s-cl, 13.84), Illite (i-cl, 10.06), Kaolinite (k-cl, 9.92), Dickite (cl, 7.97), Nacrite (k-cl, 4.37), Halloysite (cl, 2.73), Vermiculite (cl, 1.36)	
N3	Osumillite (mi, 27.62), Muscovite (mc, 27.62)	Dickite (cl, 14.80), Nacrite (k-cl, 10.63) Montmorillonite (s-cl, 10.21), Kaolinite (k-cl, 7.18), Illite (i-cl, 1.55)	
Z1	Anthophyllite (am, 34.45), Tremolite (am, 28.47)	Illite (i-cl, 7.99), Dickite (cl, 3.08), Kaolinite (k-cl, 1.95), Montmorillonite (s-cl, 0.83), Halloysite (cl, 0.55)	Orthoclase (fd, 12.45) Mordenite (ze, 8.13), Lime (ox, 2.09)
U1	Truscottite (tr, 25.08), Paragonite (mc, 23.28)	Illite (i-cl, 12.68), Montmorillonite (s-cl, 7.31), chlorite-vermiculite-montmorillonite (x-cl, 6.11), Beidellite (s-cl, 2.08)	Faujasite (ze, 23.43)
U2	Lizardite (sp, 49.84), Chrysotile (sp, 35.20)	Illite (i-cl, 14.95)	
G1	Truscottite (tr, 41.18), Talc (Ta, 17.14)	Illite (i-cl, 11.11)	Faujasite (ze, 30.55)
G2	Parahopeite (ox, 32.28), Muscovite (mc, 28.34)	Kaolinite (k-cl, 9.70), Illite (i-cl, 7.79), Nacrite (k-cl, 7.72), Dickite (cl, 2.41)	Mordenite (ze, 10.00), Periclase (ox, 1.73)
E1	Paragonite (mc, 22.72), Pyrophyllite (am, 20.45)	Halloysite (cl, 22.44), Chlorite-vermiculite-montmorillonite (x-cl, 14.20), Montmorillonite (s-cl, 11.92)	Portlandite (oh, 8.24)
E2	Antigorite (sp, 41.84), Tremolite (am, 19.80)	Illite (i-cl, 18.80), corrensites (s-cl, 6.42)	Mordenite (ze, 13.12)

Note: mc≤mica, am≤amphibole, sp≤serpentine, mi≤milarite, ph≤phosphate, tr≤truscottite, ze≤zeolite, se≤sepiolite, ox≤oxide, fd≤feldspar, ta≤talc, oh≤hydroxide, k-cl≤kaolinite clay phase, cl≤other clay-mineral phase, i-cl≤illite clay phase, s-cl≤smectite clay phase, x-cl≤mixed clay

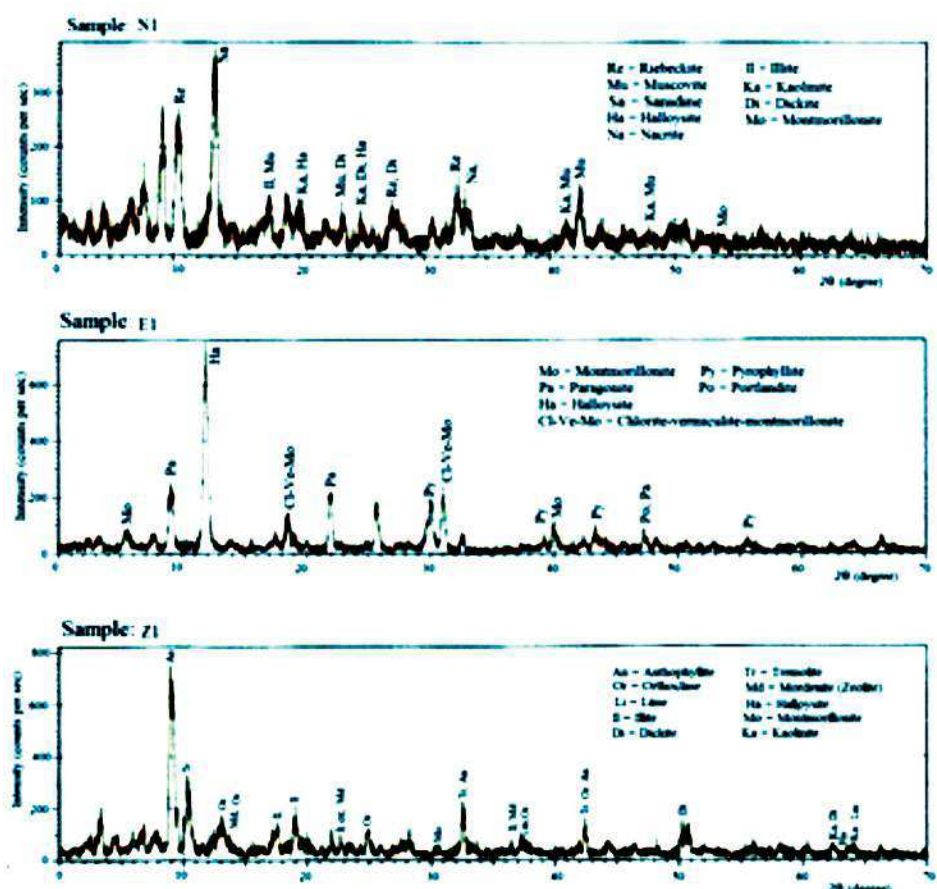


Fig. 2: Examples of the labelled mineral X-RD diffractograms

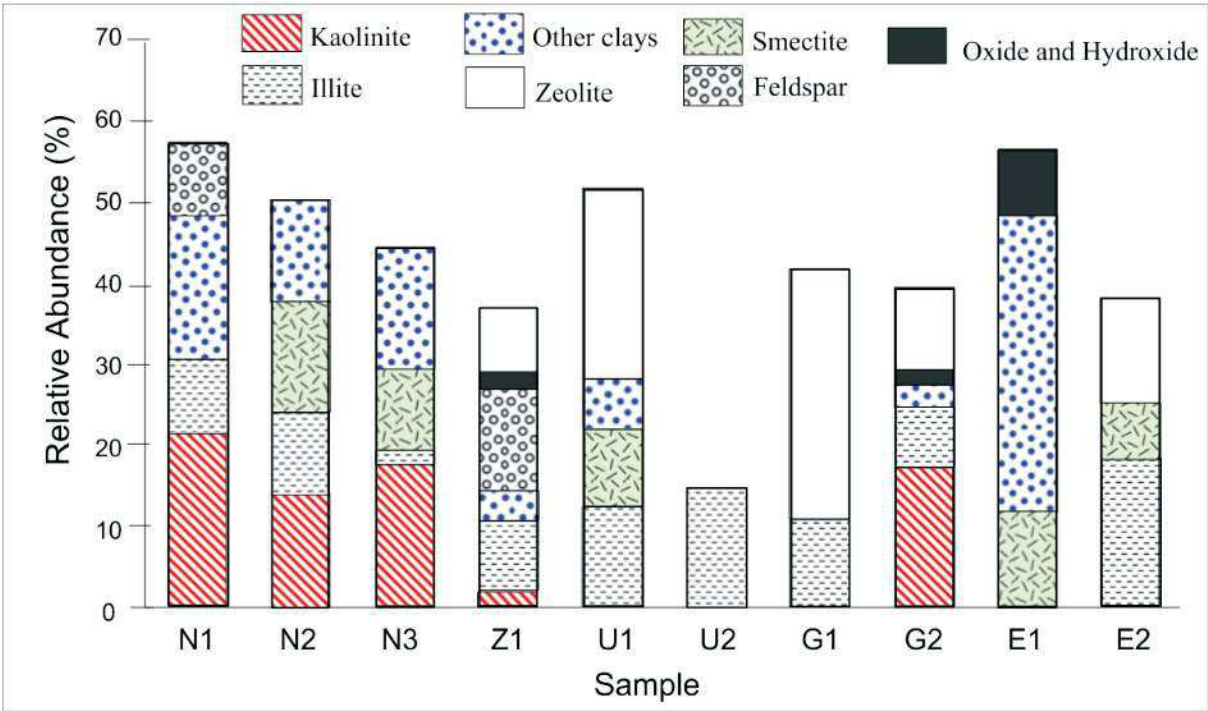


Fig. 3: Cumulative relative abundance of water-absorbent minerals contained in the rock samples

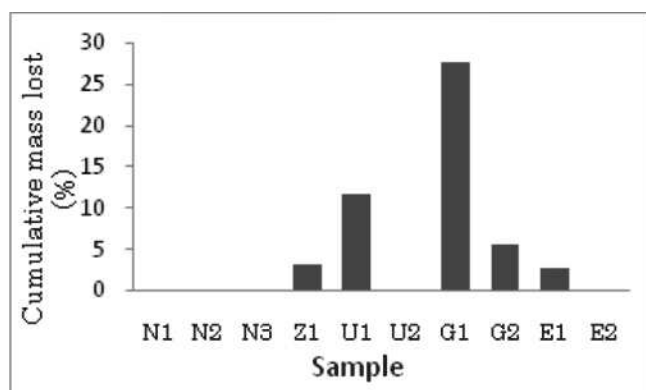


Fig. 4: Cumulative percentage of mass lost in the degradability test

This means, perhaps, that the nacrite and dickite formed from pure kaolinite suggesting that kaolinite has greater chances of transforming to nacrite and dickite than smectite.

Relationship between Mineralogy and Deterioration of the Rocks

Comparing Table 1 and Fig. 3 with Fig. 4, it can be seen that the extent of deterioration is not commensurate with either the clay mineral content or the summed water-absorbent minerals (SumWam). Sample G1 that showed the highest deterioration (27.6%) contains 11.1% of clay minerals and 41.6% SumWam.

On other hand, sample N1, which did not show

Table 2: Relationship of the rocks' deterioration and water-absorbent minerals

Predictor Variables		Correlation and Regression indices		
Variables	Minerals	Rp	sig	Beta
Variables Entered (predictors)	Zeolite	0.882	0.001	0.701
	Illite	0.103	0.388	NA
	Smectite	-0.286	0.212	NA
Variables Removed (Excluded)	Kaolinite	-0.367	0.149	NA
	Other Clay	-0.308	0.194	NA
	Feldspar	-0.190	0.299	NA
	Oxide And Hydroxide	-0.114	0.377	NA
	SumWam	0.053	0.442	NA

Note: Rp≤Pearson Correlation coeff., sig≤Significance, Beta≤Regression coefficient.

SumWam – summed water-absorbent minerals.

NA≤not applicable;

Dependent variable ≤ Percentage of mass lost in the degradability test.

Regression method: Stepwise

(Criteria: Enter if sig. ≤ 0.050, Remove if sig. >0.050)

significant deterioration contains 48.8% clay minerals and 57.6% SumWam. Neither the clay minerals nor SumWam contributed significantly to deterioration of the studied rocks. Table 2 reveals that amongst all the water-absorbent minerals (including SumWam) used as potential predictors, only zeolite emerged as the significant predictor.

It follows that zeolite contributes more to deterioration of the studied pelitic rocks than other water-absorbent minerals. A closer look at Figs. 3 and 4 reveals that amongst the five samples (Z1, G1, G2, U1 and E1) that showed significant (1%) deterioration, E1 that showed the least deterioration does not contain zeolite but contains the highest amount of oxide/hydroxide. Also, none of the five samples that did not show significant deterioration contains either zeolite or oxide/hydroxide. Deterioration of the samples is controlled mostly by zeolite followed by oxide/hydroxide. Therefore, although it is a widely accepted that mineralogy of rock contributes to rock deterioration, it may not be the summed water-absorbent minerals contained in a rock that contribute to its deterioration but the individual water-absorbent mineral species. In the present case, zeolite and hydroxide/oxide minerals caused the deterioration.

Relationship between Deterioration and Water Sorption

The deterioration history curves of samples that showed significant (>1) deterioration in the degradability test are shown in Fig. 5 while their water absorption history curves are shown in Fig. 6.

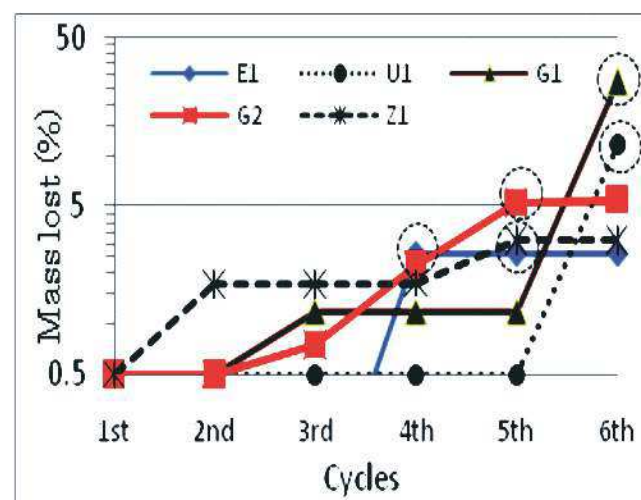


Fig. 5: Deterioration history curves of samples that showed significant deterioration

Noteworthy, 1% has been taken as significant deterioration in order to be as conservative as possible as samples that showed deterioration less than 1% were regarded as been durable or stable.

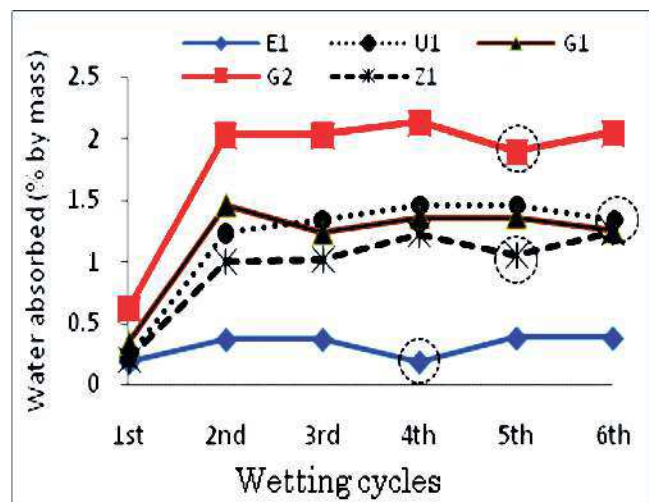


Fig. 6: Water absorption history curves of samples that showed significant deterioration.

Figure 5 shows that none of the samples that showed significant deterioration started deteriorating in the first cycle. Each started in a subsequent cycle. This agrees with work of Ruedrich et al (2010) that rocks deteriorate due to cyclic wetting and drying not by either wetting or drying. Comparing Figs. 5 and 6, it can be seen that the deterioration cycle is accompanied by a graphical decrease in water sorption (circled points of Figs. 5 and 6). This is due to the water sorption and deterioration processes associated with wetting and drying of the rocks. During wetting (soaking) process, the outer portions ordinarily become wet and saturated before the inner portions due to water permeation through possible deterioration pathways (connected pores, micro-

fractures and/or water-absorbent minerals) of the out parts (Koch et al, 2008). Deterioration (detachment) of such outer portion is accompanied by two sequential water sorption processes. Firstly, decrease in amount of water absorbed by the current intact rock relative to the initial intact rock. Secondly, exposure of new outer portion (increased surface area) results to permeation of more water through deeper deterioration pathways and thus an increase in the amount of water absorbed during the next wetting process. This explains the increase in water absorbed in cycle following the deterioration cycle. It implies that, in service, excavated/quarried pelitic rocks absorb more water in wetting cycles following deterioration caused by inter-seasonal wetting and drying.

Conclusions

The following conclusions are drawn from this work:

1. The individual water-absorbent mineral species contributes more to deterioration of pelitic rocks than the summed water-absorbent minerals. In the present study, zeolite and oxide/hydroxide minerals contributed more to deterioration of the rocks than other water-absorbent minerals.
2. The studied pelitic rocks are richer in illite clay mineral phase than kaolinite and smectite phases. The Illite phase contributes least to deterioration of the rocks.
3. Pelitic rocks that are susceptible to deterioration due not absorb the same amount of water in every complete wetting cycle. Deterioration of pelitic rocks caused by inter-seasonal wetting and drying process is followed by increase in water absorption by the rock. There is further evidence that deterioration of rocks is caused by cyclic wetting and drying and not by either wetting or drying.

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Assessment of Plume Concentration and Protective Capacity of a Dumpsite's Underlying Soils: An Electrical Resistivity Approach

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Abstract

Studies which employs electrical resistivity method to determine contamination/pollution from locations of accumulated refuse have been carried out by several researchers. However, a study which maps and identifies profile of soils underlying a dumpsite is yet to be introduced into environmental geophysics literature. In this study the basic electrical resistivity field method was followed to present the geometry associated with the Oke-Tako dumpsite's leachate contamination, and variation in the underlying soil's protective capacity. An 'Omega Resistivity' meter was employed with the Schlumberger Vertical Electrical Sounding (VES) for the survey. Twenty nine (29) VES curves were interpreted to produce iterated curves in order to determine the resistivity, thickness and depth of the subsurface layers. The three (3) major geoelectric layers prominent in the area are, the topsoil (resistivity values range of 17.3 - 294.2Ωm and thickness range of 0.8 - 1.1 m), a sandy clay layer (resistivity values range of 17.2 - 183.7Ωm and thickness range of 1.4 - 20.2 m) and a fractured basement (resistivity values range of 100.8 to 539.1Ωm). The resistivity values help in defining, saturation and contamination zones with depth in the study area. An overlay of iso-resistivity maps of the underlying soils layers reveals a plume flow of northeast-southwest direction with the source of plume concentration in the south. The protective capacity of the underlying soils varies between poor and weak.

Keywords: Dumpsite, Geoelectric, Iso-resistivity maps, Soils and Protective capacity.

Introduction

Dumpsite is a location of accumulated refuse and other discarded materials, which are potential sources of pollution to the soil, surface water and groundwater (UNICEF & WHO, 2008). UNICEF & WHO's report clearly revealed indiscriminate disposal of refuse occurs all over the world as a major problem. Agunwamba (1998) emphasized the need for research on impact of wastes on public health populace. This has been of increasing concern over a decade. According to Ariyo and Enikanoselu (2007), acquisition of two dimensional images aided understanding of geological variability of the subsurface.

Based on electrical resistivity values, Ariyo *et al.*, (2013) delineated four layers (topsoil, sandy layer, limestone and sandy clay) in a soil profile below Sotubo dumpsite along Sagamu-Ikorodu Road. The resulting pseudosection revealed that leachate from the dumpsite has infiltrated the aquiferous sandstone layer. Dare and Fatoba (2014) ascertained leachate plume generation and migration direction at the popular landfill site along Ring road Ibadan. These findings enabled the impact of the leachate from the dumpsite on the surrounding soil, and the groundwater to be clearly defined.

Bayowa *et al.*, (2015) employed surface direct current resistivity technique to investigate the subsoils which underlain an abandoned dumpsite in Ido Osun, Southwestern Nigeria. The research work identified revealed its risk to groundwater contamination. Ganiyu *et al.*, (2016) investigated groundwater contamination potential due to leachate migration from an active dump site in Ibadan. It was revealed that groundwater in the area has been contaminated due to leachate percolation from the active dump site. Ofomola *et al.*, (2016) employed two-dimensional resistivity imaging, and radial resistivity sounding to define dumpsite-induced groundwater contamination in part of Delta State, Nigeria. The groundwater Vistas version 4 was helped to deduce particle/contaminant tracking, flow direction and velocity determination.

At Oke-Tako, Ijebu-Igbo area, huge amount of various types of wastes are dumped on daily basis, and deposited in a chaotic manner. This could lead to release of potentially disastrous materials into the subsurface environment. The present research follows a line of investigation behind the basic electrical resistivity field method, and survey geometry associated with the dumpsite's leachate contamination rating around Oke Takpo site. It is meant to theoretically

model the profile which underlain the basement rocks at different depths of burial, and to determine extent of plume contamination as well as the underlying soil's protective capacity at the location.

Location and Geology

The study area covers a range of latitudes 6.963°N - 6.966°N, and a range of longitudes 3.996°E - 4.005° E in the Southwestern part of Nigeria as shown in Figure 1. Oke Tako is accessible through a series of interconnecting minor roads, footpaths and a few major roads. The area is experiencing tropical savannah climate, which is seasonally characteristic of damp and very humid (Omogbai, 2010). The area is characterised by contrasting topography, which resulted from geomorphic processes responsible for shaping the

terrain. The major geomorphic features in the study area have been covered by houses due to urban development. In the area, rainfall is the only source for surface water bodies and also for groundwater recharge. The area is drained by River Omi.

Crystalline rocks formation underlain all portions of the area. These include gneisses and pegmatite of the Precambrian Basement Complex (Obaje, 2009). According to Oyinloye (2011), the granitic rocks and gneisses show lineation/foliation and the foliations usually trends in a defined direction. Adebisi *et al.*, (2014) reiterated that these rocks have disintegrated and transformed into overburden soils, which have variable thicknesses, geochemical, mineralogical and geotechnical characteristics.

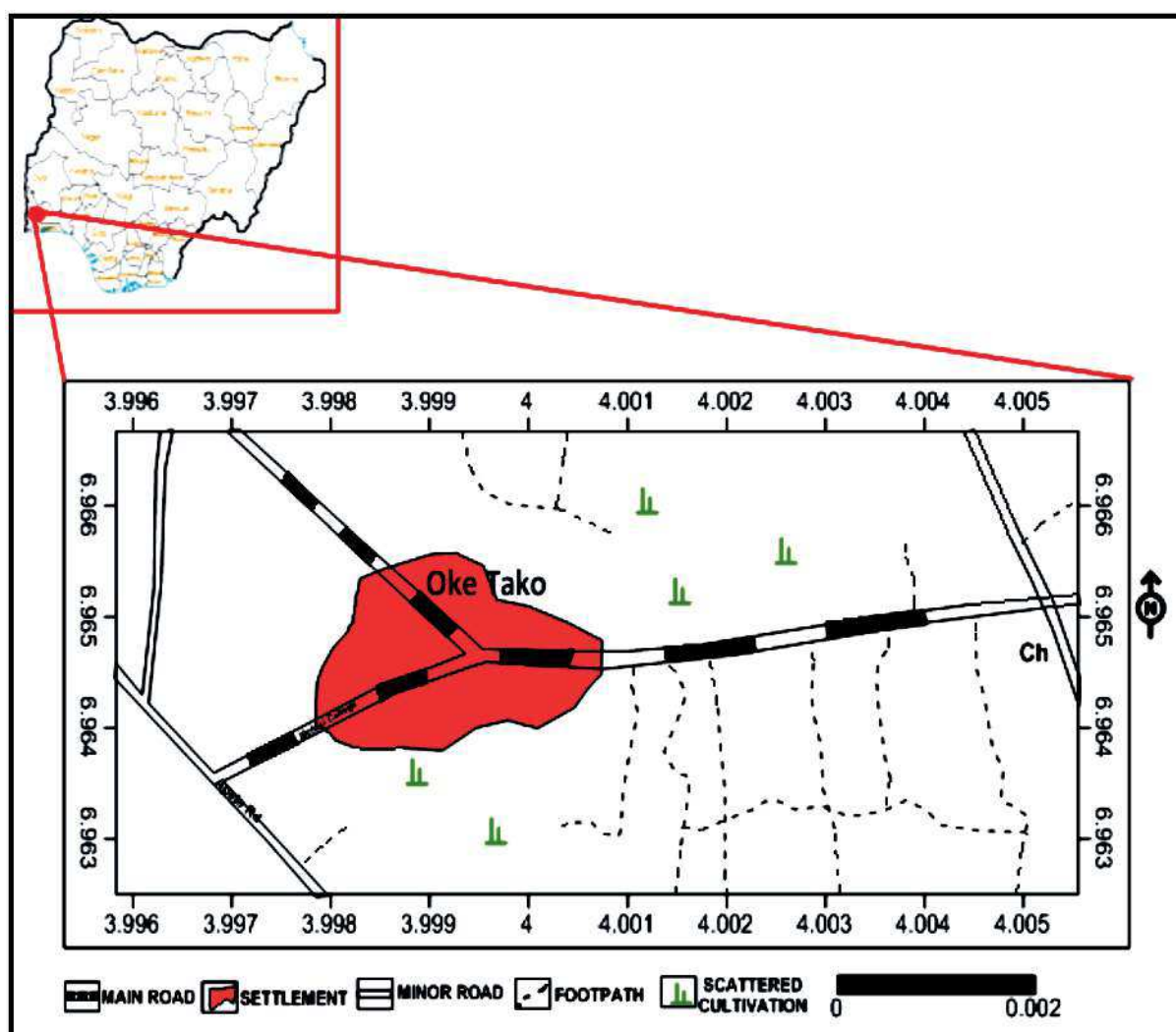


Fig. 1: Map of Oke Tako, showing the studied dumpsite

Materials and Method

The electrical resistivity survey was conducted using an 'Omega Resistivity' meter. The adopted Schlumberger array electrode configuration was adopted for the survey. A survey was carried out in an area outside the dumpsite, to serve as a control such that there will be no influence from the leachate on the resistivity values. A record of the variation of the apparent resistivity of the subsurface with depth for each test line was obtained.

Twenty nine (29) VES curves were interpreted to produce iterated curves for the determination of resistivity, thickness and depth of the subsurface layers. The data were interpreted quantitatively using the partial curve matching method followed by computer iteration using WinResist software package.

Results and Discussion

Electrical Resistivities and Layer Thicknesses

Results were obtained from the analysis of the 29 theoretical VES curves for the overburden thickness and basement and shown in Table 1. The dominant resistivity curve pattern obtained for this study is the H-type. This accounted for about 90% with subordinate A-type curve as some of those shown in Figure 2. Interpretation of few VES curves dataset acquired across the dumpsite is presented in the geoelectric sections shown in Figure 3.

The geoelectric parameters of the area are shown in Table 1. The first layer is the topsoil which has resistivity values range of 17.3 - 294.2 Ω m and the thicknesses range of 0.8 - 1.1 m. The second layer is clayey with resistivity values range of 17.2 – 183.7 Ω m and thicknesses range of 1.4 - 20.2 m. The third geoelectric layer is the fractured basement which has resistivity values ranging from 100.8 to 539.1 Ω m.

The geoelectric sections of the area revealed three (3) basic profile description in the lateral, and sub-surface configuration. Transverse 1 is made up of Five (5) VES points (VES 1, 4, 6, 7 & 8) with electrical resistivity of the top soil ranging from 35.4 to 74.9 Ω m. The second

Layer is clayey having resistivity values of 29.1, 18, 34.9, 28.1 and 23 Ω m, with respective thicknesses of 10.0, 6.3, 4.7, 3.5 and 2.7m. The last layer which is sandy and has resistivity values ranging between 162.2 and 265.9 Ω m. The presence of clay in this transverse is expected to hold the leachate from contaminating any aquifer present.

Transverse 2 is made up of Three (3) VES points (VES 11,16 and 23). The resistivity values of the top soil are 17.3, 26.3, and 24.5 Ω m. The second layer is clayey having resistivity values of 45.1, 37.5 and 98.6 Ω m with thicknesses of 23, 19 and 20.2m respectively. The last layer is sandy has resistivity values of 403, 412.1 and 539.1 Ω m respectively. The presence of Clay at this transverse will hence hold the leachate from percolating down to the groundwater.

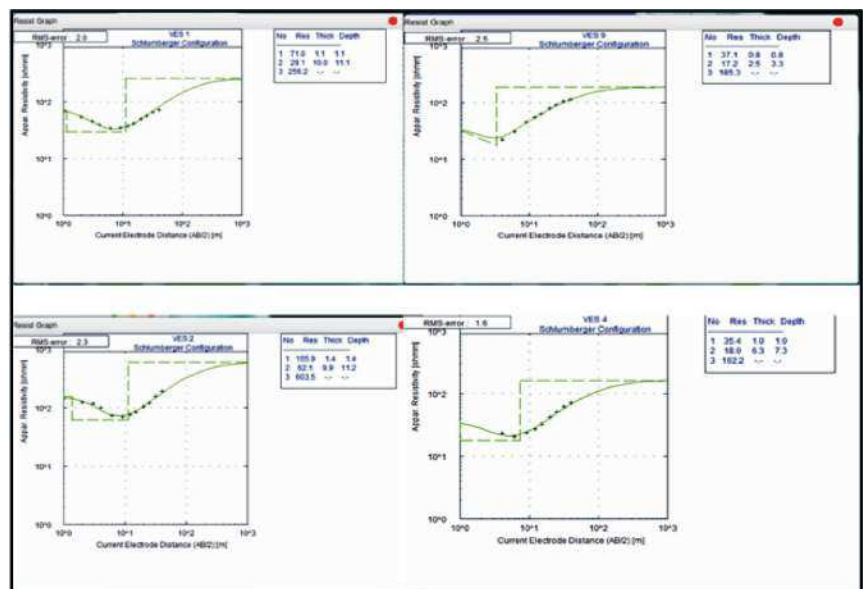


Fig. 2: Selected Vertical Electrical Sounding (VES) curves

Table 1: Analysis of selected theoretical VES curves for the overburden and basement

VES	Layers	Resistivity (Ω m)	Thickness (m)	Depth (m)	Lithology
17	1	154.3	1.2	1.2	Top soil
	2	104.4	31.3	32.5	Sandy clay
	3	1773.5			Fresh basement
18	1	112.4	1	1	Top soil
	2	160.4	35.2	36.3	Sandy clay
	3	1521.5			Fresh basement
19	1	252.7	1.1	1.1	Top soil
	2	166.6	21.6	22.8	Sandy clay
	3	702.6			Fractured basement
20	1	320.8	1.2	1.2	Top soil
	2	177.5	17.7	18.9	Sandy clay
	3	1119.8			Weathered basement

Transverse 3 made up of Five (5) VES point (VES 12, 13, 24, 27 and 29). The resistivity values of the top soil are 114.3, 198.9, 47.1, 103.7 and 67.2 Ω m. The second clayey layer has resistivity values of 55.5, 90.1, 19.5, 55.7 and 26.7 Ω m with thicknesses of 8.3, 11.6, 15.2, 9.4 and 12.6m respectively. The last layer which is sandy with resistivity values of 129.6, 166.4, 134.2, 125.5 and 237.3 Ω m.

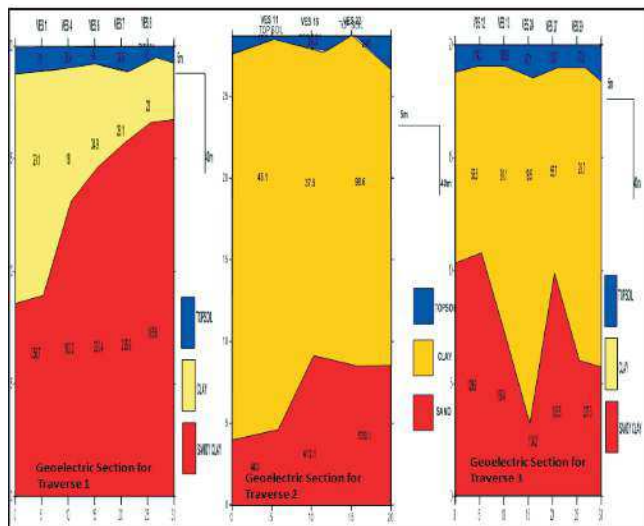


Fig. 3: Geoelectrical sections for soil layers of the overburden with depth.

Contamination Rating and Protective Capacity

The thickness of the underlying soil is an important factor to be considered at a dumpsite location. True resistivity and layer thickness helped to determine the protective potentials of the underlying soils in the study area. Figure 4 is an overlay of iso-resistivity maps of soils layers, which underlain the dumpsite at 1.0, 5.0 and 10.0m. The direction of plume flow is generally, northeast-southwest with the source of plume concentration in the south. The contamination rating of the underlying soils is based on typical resistivity values of such materials. Following Eduvie (1998), Dan-Hassan and Olurunfemi, (1999) and Aboh and Osazuwa (2002), areas with resistivity values ranging between 1 and 30 Ω m are characterized as saturated dumpsite region. Those with resistivity values ranging from 30 to 90 Ω m are characterized by the unsaturated dumpsite region (Tables 2, 3 and 4). The resistivity distribution with depth is indicative of contamination zones within the study area. Areas with high resistivity values show that the contamination of leachate in this region is insignificant. However, areas with low resistivity values are indicative of leachate contamination. The leachate infiltrated the sandstone layer throughout the area.

Table 2: Selected contamination rating of the study area at depth of 1.0m

Select Geoelectrical Location	Electrical Resistivity (ρ_s) Ω m	Inferences		
VES 1	67.340163	leachate region	Uncontaminated	Unsaturated
VES 2	143.33423	leachate free	Uncontaminated	
VES 3	49.76541	leachate region	Contaminated	Unsaturated
VES 4	34.6534	leachate region	Contaminated	Unsaturated
VES 5	131.19814	leachate free	Uncontaminated	
VES 11	35.876	leachate region	Contaminated	Unsaturated
VES 26	290.71	leachate free	Uncontaminated	
VES 27	94.179	leachate region	Contaminated	Unsaturated
VES 29	67.009	leachate region	Contaminated	Unsaturated

Table 3: Selected contamination rating of the study area at depth of 5.0m

Selected Geoelectrical Location	Electrical Resistivity (ρ_s) Ω m	Inferences		
VES 1	37.552346	leachate region	Contaminated	Unsaturated
VES 4	28.573152	leachate region	Contaminated	Saturated
VES 12	63.505333	leachate region	Contaminated	Unsaturated
VES 13	101.862	leachate free	Uncontaminated	
VES 24	27.715333	leachate region	Contaminated	Saturated
VES 25	213.76333	leachate free	Uncontaminated	
VES 27	70.006667	leachate region	Contaminated	Unsaturated
VES 28	135.68433	leachate free	Uncontaminated	

Table 4: Selected contamination rating of the study area at depth of 10.0m

Selected Geoelectrical Location	Electrical Resistivity (ρ_s) Ω m	Inferences		
VES 1	35.769588	leachate region	Contaminated	Unsaturated
VES 2	72.224481	leachate region	Contaminated	Unsaturated
VES 14	110.82	leachate free	Uncontaminated	
VES 15	161.63	leachate free	Uncontaminated	
VES 16	41.657667	leachate region	Contaminated	Unsaturated
VES 17	114.01333	leachate free	Uncontaminated	
VES 27	57.924333	leachate region	Contaminated	Unsaturated
VES 28	131.94433	leachate free	Uncontaminated	
VES 29	31.903667	leachate region	Contaminated	Unsaturated

The longitudinal unit conductance values shown in Table 4 express the overburden protective capacity in the area. The overburden medium acts as a natural filter to percolating leachate with the help of water during aquifer recharge. Its ability to retard and filter percolating leachate is shown in a characteristic longitudinal unit conductance map (Figure 5), prepared from for all the VES locations was used for the overburden protective capacity rating of the study area.

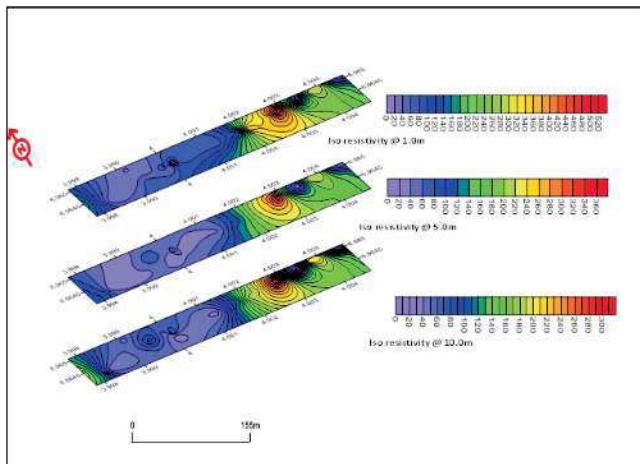


Fig. 4: Overlay of isoresistivity maps of soils layers underlying the dumpsite at 1.0, 5.0 and 10.0m

Table 5: Selected longitudinal unit conductance for protective capacity of underlying soils

VES	Longitudinal Conductance(s) (mhos)	Inferred Protective Capacity
1	0.36	Moderate
2	0.17	Weak
3	0.14	Weak
14	0.05	Poor
15	0.02	Poor
16	0.59	Moderate
20	0.10	Weak
24	0.81	Good
29	0.49	Moderate

The area between latitudes 6.963°N - 6.966°N, and longitudes 3.996°E - 3.9985° E and latitudes 6.965°N - 6.966°N, and a range of longitudes 4.004°E - 4.005° E falls within the poorly overburden protective capacity. The area between latitudes 6.9643°N - 6.966°N, and longitudes 3.9984°E - 4.005° E falls within the weakly overburden protective capacity, while the area between latitudes 6.965°N - 6.966°N, and longitudes 3.9982°E - 4.0036° E and latitudes 6.9643°N - 6.9665°N, and a

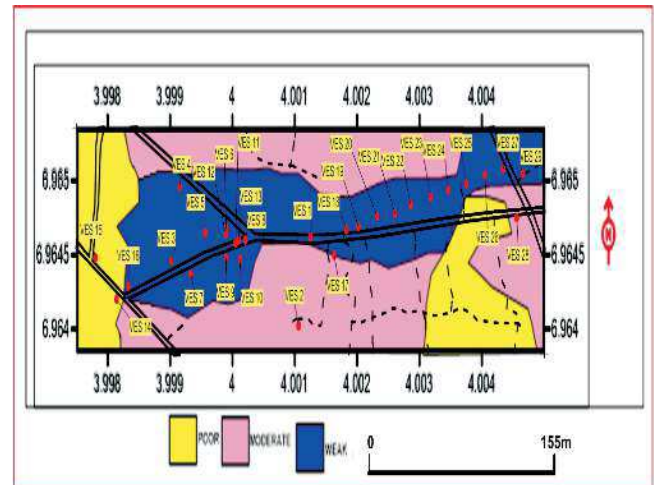


Fig. 5: Longitudinal unit conductance map for protective capacity of the underlying soils.

range of longitudes 3.9984°E - 4.003° E falls within the moderately overburden protective capacity.

Summary and Conclusions

Soils which underlain the Oke Tako dumpsite showed three (3) major geoelectric layers. The topsoil has thicknesses ranging between 0.8 and 1.1 m, above a clayey layer with thicknesses varying from 1.4 to 20.2 m and sandy clay over the fractured basement, which have electrical resistivity values ranging from 100.8 to 539.1Ωm. The presence of clay in the layers is bound to prevent the leachate from percolating to the groundwater system. The general electrical resistivity distribution with depth in the area shows that leachate contamination in the region is insignificant. However, areas with low resistivity values are indicative of leachate contamination. The leachate infiltrated the sandstone layer throughout the area. The presence of sand in the underlying soils informed the low protective potential but the clay to an extent enhanced their protective capacity. In general, the soils which underlain the dumpsite have weak to poor range protective capacity for leachate percolation to the subsurface.

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Evaluation of Sub Base / Subgrade Soils Along Ikorodu-Shagamu Road, Shagamu, Southwestern, Nigeria

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Abstract

Geotechnical instigation was carried out on Sagamu-Ikorodu highway with a view to establishing the geotechnical factors responsible for the incessant failure. Dynamic cone penetration (DCP) and 2D electrical resistivity survey were undertaken. Trial and borrow pits were employed for collection of Bulk soil samples along the road alignment. Soil Classification (particle grains size distribution, Atterberg limit and specific gravity tests), compaction test and California Bearing Ratio [CBR] test (both soaked and unsoaked) were carried out on the soil samples. 2D electrical imaging shows resistivity values ranging from 4 - 40 $\Omega \cdot m$ within the soil subgrade, a layer which was found to consist of clay/sandy clay soil. This relatively thick clay/sandy clay material is capable of causing road failure due to its low level of competency. The sieve analysis revealed that the subgrade soils have fines of 17% - 22% and percentage sand of 34% - 65%. All samples collected from the failed section have moisture content ranging from 35% - 55.1%, specific gravity from 2.3 - 2.7, liquid limit from 32.9% - 58.2%, plastic limit from 10.4% - 17.6%, plasticity index from 22.8% - 40.5% and fall within fair to poor highway material. Amount of fines is much higher than the recommended 15% maximum value, they can thus be said to have poor geotechnical properties and all locations fall under group A-7-5 and A-7-6 of AASHTO classification. This grading represents fair to poor sub grade materials under the same specification. The compaction characteristics of the soil maximum dry density range from 1670 kgm^{-3} to 1800 kgm^{-3} and optimum moisture content ranging from 17.7% to 21.1%. The unsoaked and soaked CBR varied from 8.23 - 14.67 and 1.98 - 6.89 respectively, for samples compacted at the modified AASHTO level. DCP test gave insitu estimate of the bearing capacity between 12.73 KN/m^2 - 167.1 KN/m^2 of highway subgrade materials and fall within the A-C curves of number of vehicles per day exceeding 3 tons laden weight thickness of pavement layers curves for respective traffic volume. The soils in the unstable section of the Ikorodu-Sagamu highway from the road alignments have a low level of laterization, poorly graded and rich in fines, and the soil loses strength significantly when wet, hence poor as subgrade material.

Keywords: Sagamu-Ikorodu, Zonation Maps, Dynamic Cone Penetration, CBR, 2D, Electrical Resistivity Survey

Introduction

Road has plays an important role in the trade and transportation system throughout the world, and total collapse of the road several times often rapid increase the reconstruction in the pavement infrastructure development in Nigeria. Structure of a specific pavement represents three separate layers of surface, base, sub-base and sub grade whereas, the characteristics of the soil bed over which the entire pavement system rests on represents pavements geotechnical properties (McGhee, 2010). The stability and durability of pavement depends on the traffic load or intensity and the strength of pavement layers (Kiehl and Briegleb, 2011). If the pavement is not perfectly designed considering present and future traffic conditions, pavement failure becomes inevitable.

According to Aigbedion (2007), Road Failure could be defined as a discontinuity in a road pavement resulting in cracks, potholes, bulges and depressions. A road pavement is supposed to be a continuous stretch of asphalt lay for a smooth ride or drive. Visible cracks, potholes, bulges and depressions may punctuate such

smooth ride. The punctuation in smooth ride is generally regarded as road failure. Failed roads are characterized by potholes, polishing / pavement surface wash, block and longitudinal cracks, drainage collapse, depressions / sinking of roadway, over flooding of the carriageway, gullies and trenches, rutting and raveling (FMW&H 1992), all of which are evident along the Ikorodu-Sagamu expressway under study confirming its failure.

Inadequacies in design and poor workmanship, improper mix proportion, poor quality control during construction, poor drainage, inadequate thickness of the pavement section to support the loads that travel across the asphalt and weak or unstable sub grade components could also lead to the failure of a pavement. Poor soils can seriously impede construction of adequate sub grades, as well as affect the long-term performance of a pavement during its service life The performance of any structure is strongly dependent on the foundations. Road pavements are no exceptions and their foundations need to be fundamentally modelled using sound geotechnical engineering principles. Thus, it is very important to have sufficient information about the

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.

Asphalt deteriorates when weather and traffic wear "fines" aggregates away. When the "fines" are worn away, there is nothing to bind the larger aggregates together and the surface begins to unravel. This weakens the pavement section and eventually allows water to penetrate the sub grade, which leads to pavement failure. About 90% of the time when we have pavement failure, it is a problem with the sub grade. Inadequate support in the base and soil directly beneath the pavement can cause alligator cracking and other pavement failures. In the light of the recurring pavement failure along the Sagamu-Ikorodu road, some questions constantly come to mind: what exactly is the cause of this problem? Again, since not all sections of the road failed, or at least failed equally, do soil characteristics play any role in the durability of the roads? Considering

the cost of constructing and maintaining this road, the answers to these questions have become a necessity particularly now that the impacts are multiplying. It is to this effect that a need to investigate on the role of the geotechnical characteristics and geophysical parameters of the soil on which the road is built in the failure of the pavement.

Objectives

The main objective of this work is to determine the place of soil geotechnical characteristics and geophysical parameters in the failure of the Ikorodu –Shagamu highway pavement.

The Study Area

The study area is located along Ikorodu-Sagamu road is a major road linking part of Lagos State and Ogun State located within the Federal Republic of Nigeria. It lies within latitude $7^{\circ}41'31.2''\text{N}$, longitude $5^{\circ}56'61.0''\text{E}$ and latitude $7^{\circ}54'08.0''$, longitude $5^{\circ}69'25.5''$ (Fig. 1). It is characterized by an altitude of 90 m above sea level.



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

It falls within the Eastern Dahomey Basin (Fig. 2). The basin is a peri-cratonic basin developed during the initiation of rifting associated with the opening of the Gulf of Guinea in early Cretaceous to Late Jurassic (Kingston et al., 1983; Whiteman, 1982; Klemme, 1975; Burke et al., 1971). The crustal separation and thinning of the basin was accompanied by an extended period of thermal-induced basin subsidence through the mid Cretaceous to Tertiary times as the South American and the African plates entered a drift phase to accommodate the emerging Atlantic Ocean (Mpanda, 1997; Storey, 1995). The research area is underlain by the littoral and lagoon deposits of recent sediments. The coastal belts vary in width from about 8km near the

republic of Benin boarder to about 24 m toward the eastern end of Lagos lagoon (Jones and Hockey, 1964). Hydro-geologically, Kampsax and Kruger (1987) subdivided the aquifer in the coastal terrain of Lagos and Ogun into four with the first aquifer representing the recent sediments and the second and third aquifers being within the coastal plain sands. The water table aquifer occurs in recent sediments along the coast and within alluvial plains of the river valleys. However the water table is shallow with results of chemical analysis of the water from recent sediments showing higher TDS and conductivity than water from the coastal plain sands aquifer (Oyedele, 2006).

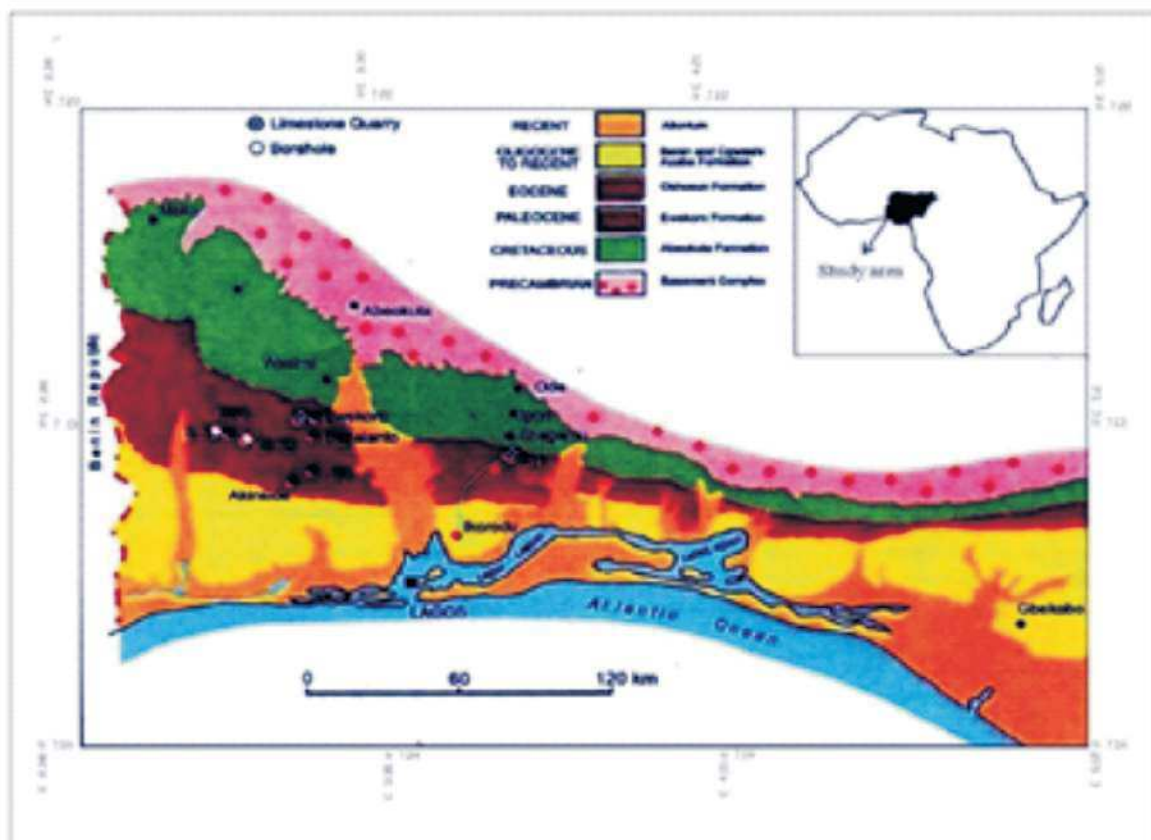


Fig. 2: Generalized geological map of Eastern Dahomey Basin (Modified after Billman, 1976)

Materials and Methods

Materials

The data sets used include topographic map, geologic map, 2D-electrical resistivity data, and geotechnical test data.

Field Investigation

The 2D electrical resistivity survey was carried out with the aid of ABEM Terrameter. The process was carried out progressively until the distance of 2km (AB) was covered along the Sagamu-Ikorodu road. The DCP test was conducted at twelve locations arbitrarily named

PT1 - PT12 test points. The penetration values of 10 blows interval were measured to evaluate the DCP value at different longitudinal distances of Sagamu-Ikorodu highway. The Dynamic Cone Penetrometer (DCP) is an instrument designed for the rapid *in-situ* measurement of the structural properties of the existing road pavements constructed with unbound materials (Russell *et al.*, 2004). It was also used for determining the *in-situ* CBR value of compacted soil sub-grade beneath the existing road pavement. Large test pits (1.5m X 1.5m X 1.5m) was excavated at the pavement –shoulder interface to a depth of 2m minimum eight pits dug for each homogenous road segment or change of soil as per TOR staggered left/right and one from borrow pit to carrying out laboratory analysis.

Table 1: 2D, DCP test point and Trial Pit along the Road Alignment

2D No	DCP Test Point No	Trial Pit No.
2D 1	DCP 1, 2, 3 and 5	(A1, A2 and A3)
2D 2	DCP 4, 6 and 7	(B4, B5 and B8)
2D 3	DCP 8, 9 and 10	(D7 and D9)
2D 4	DCP 11 and 12	(E10)
		(C6) Borrow 1 Pit



Fig. 3: Map of the Study Area showing accessibility and position of Tests points along the alignment.

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 and hydrometer analyses, Atterberg limit test, water content and density were determined using the above

standards. Visual examination, soil classification tests were carried out on the samples. Compaction test and California Bearing Ratio (CBR) test carried out.

Data Analysis

Electrical resistivity imaging was carried out along the Ikorodu-Sagamu road (Figure 3) using the ABEM terrameter in order to delineate the resistivity of the subsurface materials by injecting electric current through current electrodes (AB) and picking the potential difference from the potential electrodes (MN). The Instrument is designed to measure the resistance; initial parameters used for the survey include a current of 5 mA and four cycles for averaging the resistance value. The Wenner array was chosen because, of all common arrays, it has the strongest signal strength which is a significant factor in the study site with high background noise. Wenner array is good in resolving vertical changes (i.e. horizontal structures) in subsurface resistivity below the center of the array Oyedele *et al.*, 2012.

Results and Discussion

Laboratory Test Results

Grain size Distribution of Soil Samples

The studied soils from WAPCO gate (A1-A3) one are poorly graded being deficient in intermediate size particles, sand between 39-43% and silts 56-60%, they are classified as based on Universal Soil Classification System (USCS) as (CL) and as clay and silts soils, as well as poorly graded soils. This implies that subgrade materials were poorly graded with high percentage of clayey soil composition which affects the rate of compaction of the road; however, fines is higher than the required 15% maximum value on this basis, they can be said to be poor sub-grade materials because the lower the quantity of fines in a soil, the better its engineering characteristics. The soils from (A1-A3) locations fall under **group A7 of AASHTO classification** and they are rated as fair to poor subgrade materials under the same specification. (See table 2).

The result shows that moisture content ranges from 17.9-21.1%, Plastic Limit (PL) between 10.1-10.8% and Liquid Limit (LL) 33.4-35.3%. The results obtained shows that it is within acceptable limit. However, results of plasticity index (IP) which is plotted below on plasticity chart in Fig.4 above shows that test point 1, 2,

Table 2: Particle Size Distribution and Atterberg Limits of the Studied Soils at Wapco gate

Soil Sample Property	A 1	A 2	A 3
% gravel (>4.25mm)	1	2	1
% sand (<4.25mm to 200mm)	39	39	43
% fines (<0.075mm)	60	59	56
Specific Gravity	2.3	2.7	2.2
Cu	0	0	0
Cc	0	0	0
Liquid limit (%)	33.6	35.3	33.4
Plastic limit, %	10.8	10.1	10.4
Plasticity Index, %	22.8	25.2	22.8
Linear Shrinkage	7	8	7
Colour	Reddish Brown	Reddish brown	Reddish Brown
AASHTO Classification	A-7-5	A-7-6	A-7-5
USCS Classification	CL	CL	CL

and 3 contain soils of intermediate clay content (**CI**) the proportion of the inorganic clays is low to medium plasticity. The presence of clay soil in foundation materials is capable of causing swelling and brokerage of road surfaces when in contact with moisture during wet season. Term **clayey** is applied has **fine fractions have $PI > 11$** . The grain size distribution curves for the studied soil from bad spot along the road alignment and borrow pit studied soils (B4, B5, D7 & C6) are presented in Table 3.

B4, B5 and D7 sand ranges between 57- 60% indicating

Table 3: Particle Size Distribution and Atterberg Limits of the Studied Soils bad spots along the road alignment and Borrow pit point

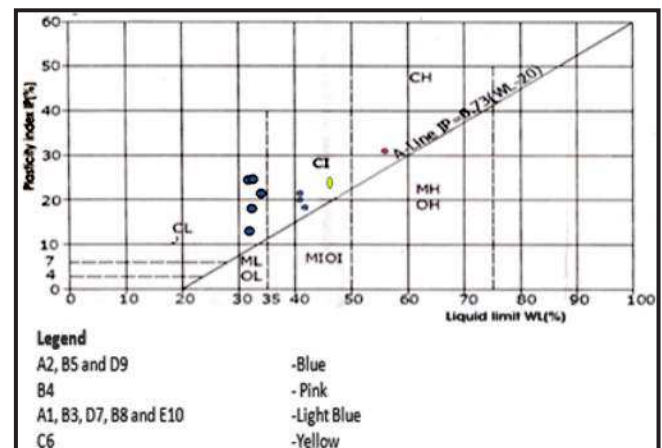
Soil Sample Property	B4	B5	C6	D7
% gravel (>4.25mm)	2	3	2	8
% sand (<4.25mm to 200mm)	53	60	57	47
% fines (<0.075mm)	45	48	41	45
Specific Gravity	2.6	2.7	2.6	2.4
Cu	0	0	0	0
Cc	0	0	0	0
Liquid limit (%)	58.2	46	45.2	34.8
Plastic limit, %	27.7	20.9	21.8	13.5
Plasticity Index, %	30.5	25.1	23.4	21.3
Linear Shrinkage	15.6	13.5	11.4	10.2
Colour	Reddish Brown	Reddish brown	Reddish Brown	Reddish Brown
AASHTO Classification	A-7-6	A-6	A-7-6	A-7-6
USCS Classification	CL	CL	CL	CL

gravely Clay, sandy Clay as well as poorly graded soils and C6 sand content is 47%, this implies that subgrade materials were poorly graded with low percentage of sandy composition which affects the rate of compaction of the road; however, the proportion of fines in both locations is about 48% on the average. Moisture content ranges from 17.9 - 21.1%, Plastic Limit (PL) between 13.5-27.7% and Liquid Limit (LL) 34.8- 58.2%. Table 4 shows the soil classification on B4, D5 and E10. They are poorly graded and deficient in intermediate size particles, sand between 34 and 65%. They are classified as sandy clay, sandy clay as well as poorly graded soils.

Table 4: Particle Size Distribution and Atterberg Limits of the Studied Soils bad spots along the road alignment

Soil Sample Property	B8	D9	E10
% gravel (>4.25mm)	3	4	2
% sand (<4.25mm to 200mm)	62	35	64
% fines (<0.075mm)	35	61	34
Specific Gravity	2.3	2.7	2.4
Cu	0	0	0
Cc	0	0	0
Liquid limit (%)	32.9	34.2	43.4
Plastic limit, %	14.8	13.8	14.6
Plasticity Index, %	18.1	20.4	28.8
Linear Shrinkage	7.3	8	13
Colour	Reddish Brown	Reddish brown	Reddish Brown
AASHTO Classification	A-7-5	A-7-6	A-6
USCS Classification	ML	CL	CL

Subgrade materials were poorly graded with high percentage of silt soil composition which affects the rate of compaction of the road; however, the proportion of fines in both locations is about 60% on the average. B8 is inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts and they are very poor graded, less than 10 based on (USCS) ML. The quantity of fines in these two locations is higher than the required 15% maximum 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.

**Fig. 4:** Plasticity Chart for the classification of clayey and silty soil

Compaction Test

Table 5 shows the bad spots along the road alignment samples (D9 and E10) laterite. It has the highest maximum dry density (MDD) of 1.88g/cm³ (Table5). Sample B5 soil has a MDD value of 1.58 g/cm³ while the Samples at A1, A2, A3 and Sample B5 laterite has the

lowest MDD values of 1.675g/cm^3 - 1.72g/cm^3 and 1.67g/cm^3 respectively. Compaction places soils in a denser state and decreases further settlement, increases shear strength and decreases permeability. It is possible to control the dry density and moisture content so that the soils produced exhibit most of the properties desired, such as consolidation, permeability, etc. (Gidigas, 1976). The relatively poor to fair values of the compaction properties possessed by these soils make them fair engineering construction materials. In general, they may be suitable for use as fill materials but not suitable for sub-grade materials in road construction. In the construction of barriers, compaction is done to achieve a soil layer of improved engineering properties. While that of location two ranges between 19.07% and 23.41%, the mean being 20.30%.

At the same level of compaction, the maximum dry density (MDD) for location one averages 1675.00 kg/m^3 while that of location two averages 1880.00 kg/m^3 . Both locations have good compaction parameters. The CBR test often gives a good estimate of the bearing capacity of highway subgrade materials. The average value for the unsoaked CBR are 13.50 and 3.56 on location one (A1, A2, A3 B4 and B5) and two (D7, B8, D9, E10 and C6) respectively.

Table 5: Maximum dry density and Optimum moisture content

Soil sample	MDD (KN/m ³)	OMC(%)
A1	1700	18.8%
A2	1720	18.0%
A3	1675	19.0%
B4	1680	17.9%
B5	1670	18.8%
C6	1760	18.7%
D7	1830	19.0%
B8	1740	20.0%
D9	1850	20.2%
E10	1880	21.1%

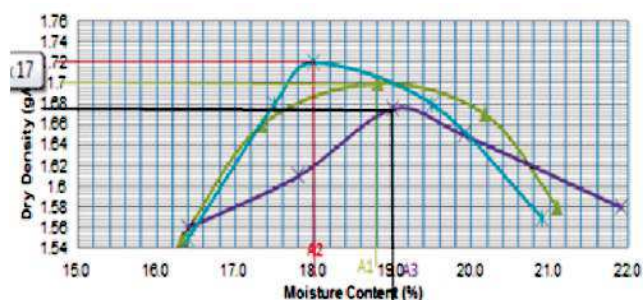


Fig 5: Dry Density and Moisture Content (for Sample A1, A2 and A3)

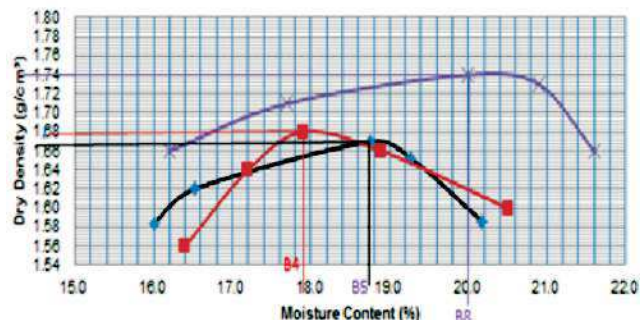


Fig 6: Dry Density and Moisture Content (for Sample B4, B5 and B8)

Soaked CBR

Location one (A1, A2, A3, B4 and B5) has a higher soaked CBR value compared with location sample for D7, B8, D9, E10 and C6. However none of these soils from both locations meet the earlier specified value none could be adjudged fit as sub-base or base-course materials. CBR of the soils reduce greatly as a result of soaking, and the amount of water absorbed is more than 20%. This further stresses the fact that these soils have high water retention capacity and that adequate rainage is required in order to prevent ingress of water below the pavement, which could result in a significant loss of strength of the subgrade soils and hence failure of the overlying pavement. Values of CBR between 5 and 7 are obtained from laboratory testing. The values of the CBR indicate the presence of clayey materials within the soil samples extracted from these test pits. This soil can be used only as filler material for embankment materials and not suitable for road construction. Location D7, B8, D9, E10 and C6 has high CBR value but unfortunately the P.I. value is relatively high (23). This material can be used as base or subbase material for roads construction if the PI is reduced to less than 9 or 15 for respectively.

Fig. 7 shows 2D resistivity structure correlated with the DCP 2 and 3 along traverse 1. The first geo-electric

Table 4: Maximum dry density and Optimum moisture content have average values of 9.07 and 21.33 for locations one and two respectively. The strength reduction is 48.7% and 29.3% for location one and two respectively.

Sample Number		Moisture dry density relationship					
		Moisture dry density relationship Modified AASHTO		California bearing ratio Modified AASHTO			
				MDD (kg/m ²)	OMC (%)	Unsoaked CBR	% of water absorbed
Location 1 (Wapco Gate and Dump site along the road alignment)	A1	1700.00	18.80	13.56 19.02	99.0	3.56 8.03	26.0
	A2	1720.00	18.00	12.78 18.34	93.3	1.98 6.89	14.5
	A3	1675.00	19.00	14.67 20.56	107.1	2.45 9.56	17.9
	B4	1680.00	17.90	11.57 20.15	84.5	4.56 9.56	33.3
Location2 (Bad Spot along the road alignment)	B5	1670.00	18.80	13.89 21.23	101.4	6.67 13.5	48.7
	B8	1740.00	20.00	8.23 15.68	60.1	5.66 7.73	41.3
	D7	1830.00	19.50	8.57 15.98	62.6	5.23 7.98	38.2
	D9	1850.00	20.20	8.98 16.78	65.5	5.54 8.02	40.4
	E10	1880.00	21.10	9.07 18.56	66.2	5.78 8.46	42.2
Location Borrow Pit	C6	1760.00	18.70	9.34 16.45	68.2	6.89 8.89	50.3

layers represent the topsoil with resistivity value that ranges from 3 - 36? m and depth of about 10m beneath the surface. This layer is composed of clay/sandy clay and extends from the surface to the depth of 10m beneath the surface has shown in the DCP graph The clay is considered primarily firm to stiff inconsistency. It exhibits low to moderate shear strength and consolidation characteristics and therefore deemed appropriate as a medium within which conventional foundations may be placed to safely support low to moderate superstructure loads. Figure 4 shows 2D resistivity structure correlated with the DCP 4, 5, 6 and 7 along traverse 2. The first geo-electric layers represent the topsoil with resistivity value that ranges from 3 - 46 ? m and depth of about 10 m beneath the surface. This layer is composed of clay/sandy clay and extends from the surface to the depth of 10m beneath the surface as shown in the DCP graph. The clay is considered primarily soft to firm becoming stiff in consistency. It exhibits low to moderate shear strength and consolidation characteristics and therefore deemed appropriate as a medium within which conventional axle load may be placed to safely support low to moderate superstructure loads.

Figure 8 shows 2D resistivity structure correlated with the DCP 8, 9, 10 and 11 along traverse 3. The second

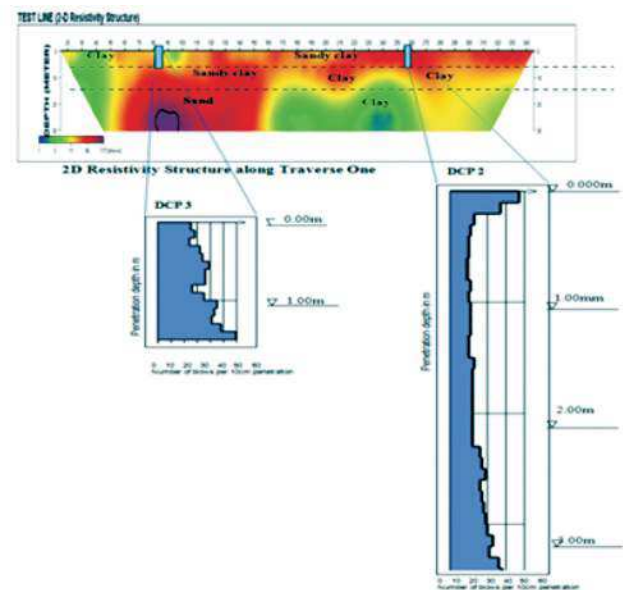


Fig. 7: Correlation of Result along Traverse One with DCP 2 and 3

geo-electric layers was delineated from depth of about 10 m composed of clay/sandy clay/sand and extend to 30m beneath the surface with resistivity value that ranges from 4 - 143? m. The lateritic clay, 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. Two zones of the clay were encountered. The first occurs within the first 2.25m from the ground surface.

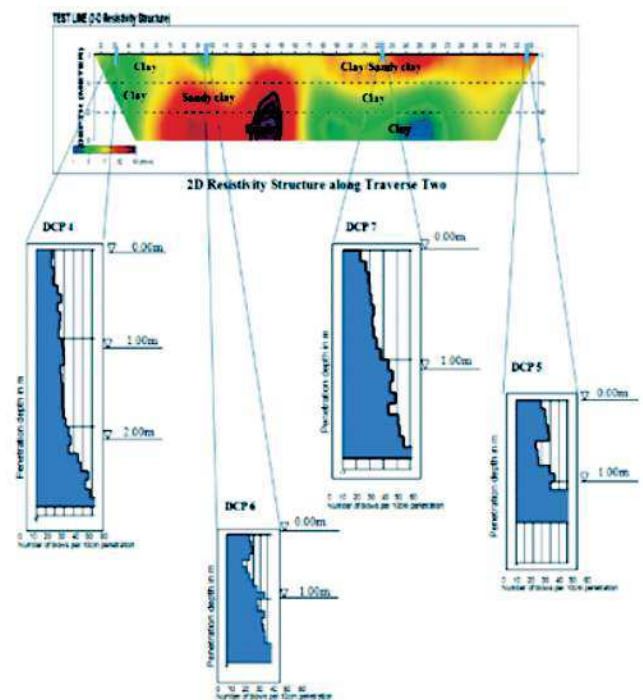


Fig. 8: Correlation of Result along Traverse Two with DCP 4, 5, 6 and 7

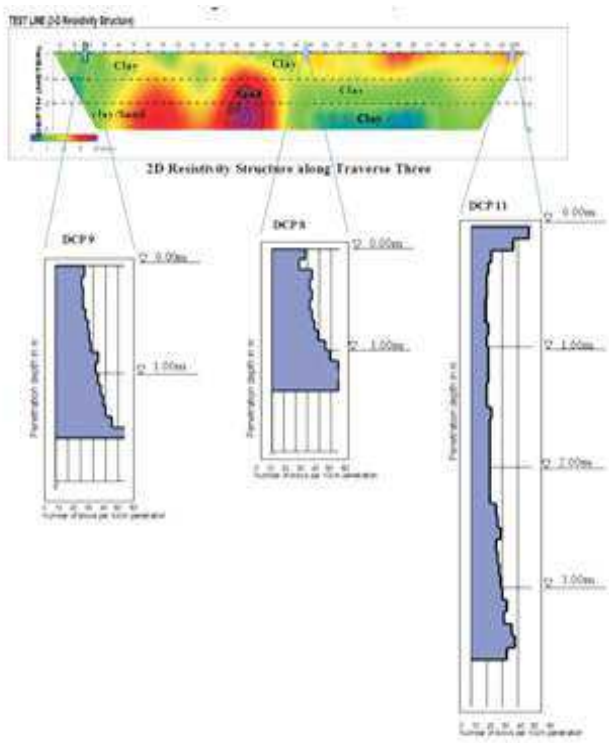


Fig. 9: Correlation of Result along Traverse Three with DCP 8, 9 and 11

The material encountered within this depth is soft to firm in consistency. This is shown by end resistance, N values as low as 11. The strength and consolidation properties of the clay subsoil from the ground surface to about - 2.25m are relatively poor. This is thought to be generally caused by the exposure of the original subsoil material to the ground surface conditions. This exposure has resulted in some weakening and softening due to the effects of weathering agents. Beyond - 2.25m, the strength properties of the lateritic clay improve rather. This indicates the limit of the zone adversely affected by weathering effects. However, at lateral distance of 120 - 170m, a sandy material was delineated at depth of about 13 - 30m having resistivity value that ranges from 70 - 143? m.

Figure 9 shows 2D resistivity structure correlated with the DCP 1, 12, and 10 along traverse 4. The first geo – electric layer represent the topsoil with resistivity value that ranges from 2 - 50? m-m and depth of about 10m beneath the surface. This layer is composed of clay/sandy clay and extend from the surface to the depth of 10m beneath the surface. The relatively thick clay/sandy clay material will cause road failure because it is not a good competent material for road construction. The soil stratification encountered from the ground surface to the end of the investigation appears simple

and uniform, being made up mainly of the lateritic clay and silty sands commonly found occurring in the Quaternary Coastal Plain Sands, a geological Formation of extensive occurrence in southwestern Nigeria. However, in areas a thin mantle of agricultural top soil was encountered with rootlets. The first subsoil zone occurs within the first 1.0m to 1.5m from the ground surface. Here, end resistance, N, values range between 2 and 15. This indicates a soft to firm consistency with relatively low shear strength and the potential for considerable volume compressibility on load application.

Figure 10 zones may only be used in supporting relatively low superstructure loads via the use conventional flexible pavement. In comparison with areas of similar occurrence, the lateritic clay within this upper zone appears to exhibit lower strength characteristics. This is thought to be due to exposure, over time, of the area to the weathering process, leading to the softening and weakening of the lateritic clay. Beyond - 1.5m the second and lower zone of the lateritic clay was encountered. The improved N values indicate within this zone is symptomatic of a reduction of the adverse effects of weathering, burnt ashes in DCP 1, 10 and 11, a concomitant improvement of the engineering properties of the subsoil.

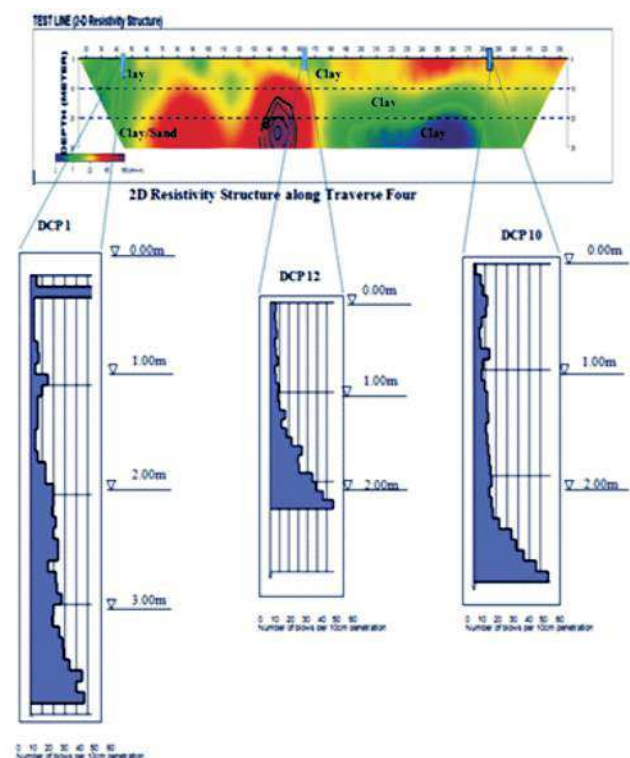


Fig.10: Correlation of Result along Traverse Four with DCP 1, 12 and 10

Figure 11, 12 and 13 are graphical presentation of the comparison between DCP - CBR with unsoaked and soaked CBR laboratory values. The insitu CBR values exceed those gotten from laboratory determination. Meanwhile, the unsoaked CBR values are higher than the soaked CBR. The very low soaked CBR values simulates the condition along the road alignment that lacks drainage system, thus leaving the soils wet and undrained.

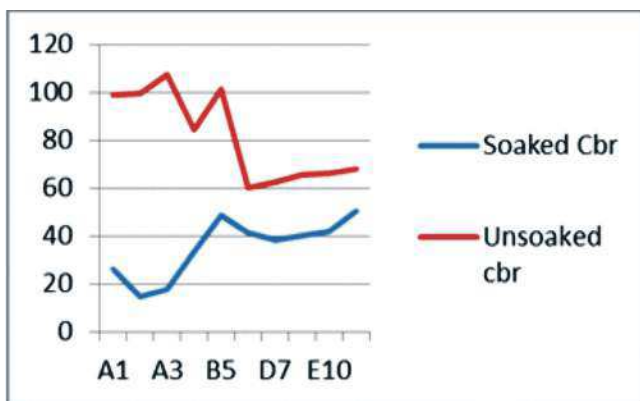


Fig. 11: Graphical Representation of Soaked CBR against Unsoaked CBR

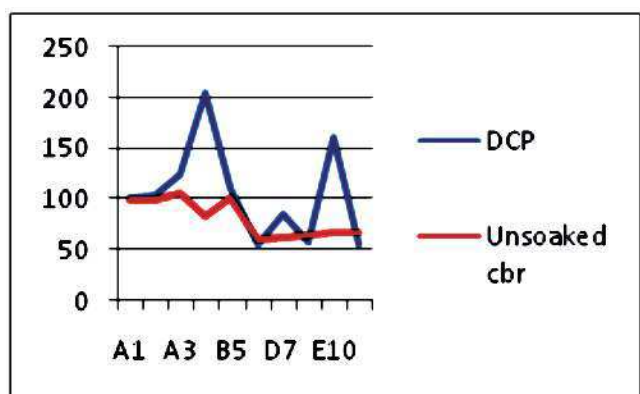


Fig. 12: Graphical Representation of Unsoaked CBR against DCP CBR

Conclusion

The current deteriorating condition of Shagamu - Ikorodu road is majorly due to inappropriate design of

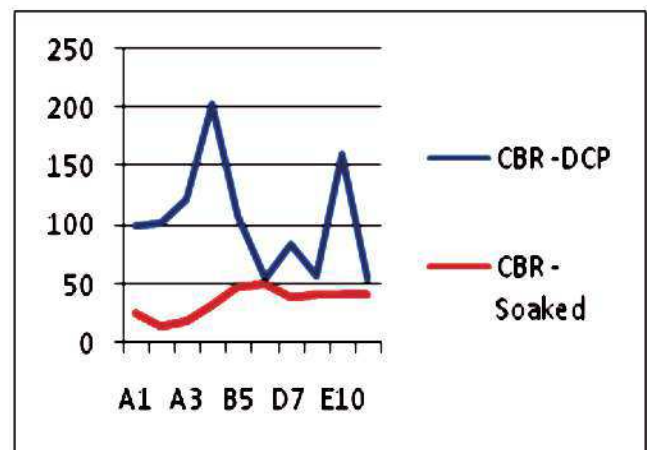


Fig. 13: Graphical Representation of Soaked CBR against CBR DCP

the pavement and poor geotechnical properties of sub-base and sub-grade soil evidenced by high percentage of fines. Other factors like improper drainage and improper maintenance treatments also contributed to the failure. Appropriate thickness of different layers was not given adequate consideration considering the importance of the studied road. This relative thickness of clay/sandy clay material is capable of causing road failure due to low level of competency.

The observed inconsistency between road side soil and sub-grade soil might get an impact on sub-surface drainage system and might contribute to road failure. The subgrade soils from borrow pits falls within the A-6 and A-7-5 AASHTO classification which are referred to as poor subgrade soils. In addition, there is a significant reduction in strength after soaking, therefore the soil is susceptible to significant loss of strength during raining season, as there no drainage systems along the road route. It is recommended that materials to be used in the construction of the road pavement must conform to specifications. Also, the drainage system should be constructed to sustain pavement life. Finally, regulatory agencies should enforce that only appropriate or design axle loads are allowed on specific pavement road.

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