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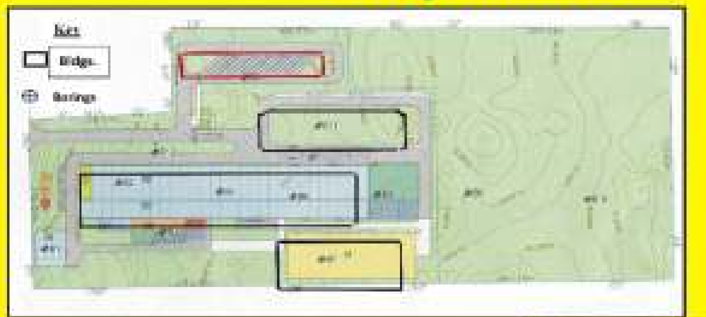

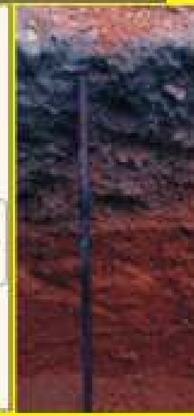

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NIGERIAN ASSOCIATION FOR ENGINEERING GEOLOGY AND THE ENVIRONMENT

& the 2nd AFRICA REGIONAL CONGRESS of the

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of the

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A LEAD PAPER (2019)

Engineering Geological Mapping for Mitigation of Mining and Drilling Hazards

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Abstract

Engineering geological investigations and mapping have important applications in regional and city planning, building foundations investigations and designs, drilling and mining operations and associated hazards. This Lead Paper gives the definitions of engineering geological mappings and traces the historical origins of applications of mapping in engineering geological activities across several regions of the world. In doing this, attempts have been made to discuss engineering geological mapping with respect to mitigation of hazards during mining operations in Soft rocks (sedimentary) regions as well as in Hard rocks (igneous, metamorphic and volcanic rocks) regions of the world. Since drilling activities take place in both soft and hard rock regimes, the applications of engineering geological mapping in the mitigation of drilling hazards in both soft and hard rocks are discussed with select examples from around the world. Engineering geological mapping as an aid in the identifications of hydrometeorological zones in the Niger Delta Sub-zone of Nigeria has also been discussed with examples in this Lead Paper.

***Keywords**: Engineering geological Mapping; Mining, Mitigation; Drilling, Hazards; Hydrometeorological Zonations.*

Introduction

Some definitions and early applications of Engineering geological mapping are hereby discussed.

Engineering geological mapping, according to *Dearman (1991)* is the "application of geological mapping techniques in civil engineering practice, particularly to the design, construction and performance of engineering structures interacting with the ground such as in foundations, road cuttings and embankments, other surface excavations, tunnels etc". An example of an Engineering geological Map of Cities in the United Kingdom has been provided in Figure 1.0 below.

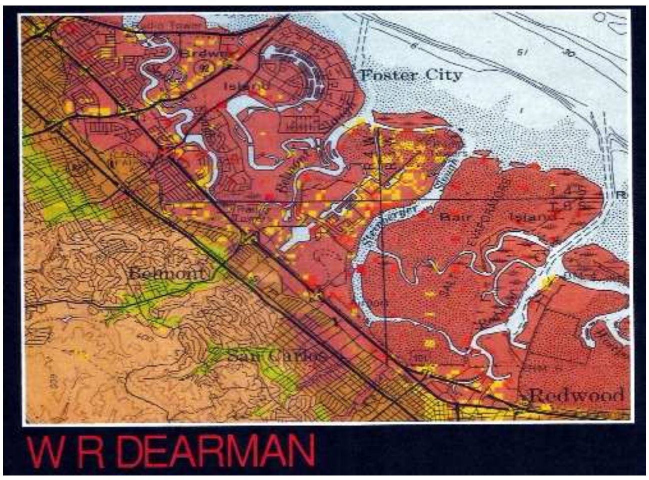


Fig. 1.0: An example of Engineering geology Mapping (after Dearman, W.R. 1991)

Historical background to the origin of Engineering geological mapping

According to Dearman (1991), *"the Preparation of Maps and Plans in Terms of Engineering Geology was begun in 1968 by a working party set up by the Engineering Group of the Geological Society (of London). The final report was published in late 1972 as Volume 5, No. 4, of the Quarterly Journal of Engineering Geology. This was intended as a guide to engineers and engineering geologists as well as to geologists. . . . ".*

A second report, *'Engineering Geological Maps. A Guide to Their Preparation'*, was published by UNESCO in 1976. This new booklet was devoted to a particular aspect of this programme, namely *engineering geological mapping*. The purpose of engineering geological maps is to show the distribution of specific geological phenomena and characteristics of rocks and soils affecting engineering use of different terrains. Thereafter, the ever-growing demand for such maps necessitated the need for the standardization of principles, systems and methods. Later publications by the commission in 1981 gave recommended symbols and rock and soil descriptions and classifications for engineering geological maps

Earliest attempts at making engineering geological mapping started from site investigations for dam sites and other related structures. Later publications by the UNESCO commission in 1981 gave recommended symbols and rock and soil description and classification for engineering geological purposes. Dam engineers and water engineers had recorded, geological details of ground conditions encountered in major engineering works with brief details of these often published. Some earliest examples include the following:

Vyrnwy dam, Wales, UK.

An example is the Vyrnwy dam, Wales, UK where *Deacon (1896),* described the foundation conditions at the Vyrnwy dam in central Wales. Undertaking as an engineer his own geological investigations, he concluded from his assessments of the form of the valley to be dammed that a rock bar limited on the downstream side of the infilled glacial basin was to form the Vyrnwy Reservoir. In a classic site investigation to determine the highest part of the bar, hidden by alluvium, 177 borings and probings, and 13 shafts, were sunk. By means of these, actual contours were drawn (Figure 1.1) and a model made of the rock surface. In the course of excavation. Deacon records that a watertight rock was found to be *2.134m* lower than the rock surface which are shown by the contours in the Figure 1.1.

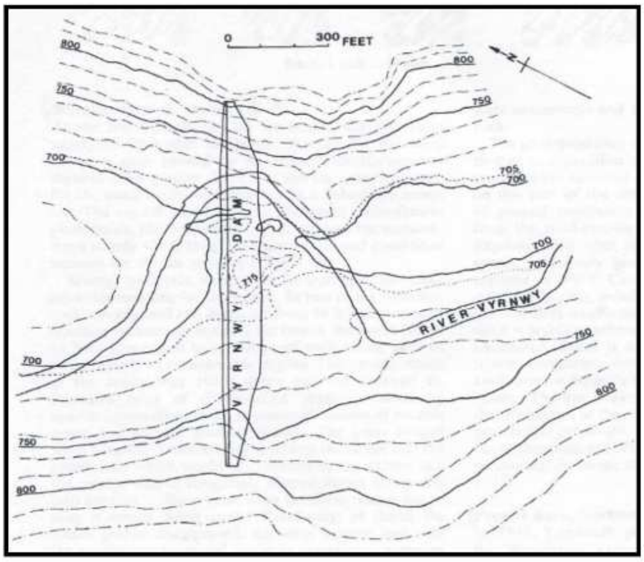


Fig. 1.1: Plan drawn at an original scale of 1:1800 showing contours on the rockhead bar below the alluvium at the Vyrnwy dam site, Wales *[as proved by borings and probing – Redrawn from Lapworth, 1911, Fig. 5].*

Burrator Reservoir, Devon, UK.

At the Burrator Works on Dartmoor, for the water supply of Plymouth in Devon, the site for the main masonry dam proved to be entirely satisfactory on massive fresh granite. Such was not the case, however, for the small earthwork dam across a subsidiary minor col. The cut-off trench for the Sheepstor embankment *(Sandeman,1901)* was *207.3m* long, whereas the embankment is only *143.3m* long. Unexpected ground conditions account for the discrepancy *(Figure 1.2).* Several trial pits were sunk to ascertain the most advantageous line for the trench. In two of the trenches, rock was exposed at a depth of about *4.27m*, in the event a delusion because on opening the trench they were found to have penetrated to pinnacles of rock which shelved away rapidly to considerable depths. The trench, which at the centre was *32.00m* deep, was cut through an extensive layer of decomposed granite crossed by quartz-tourmaline veins and minor intrusions of decomposed fine-grained granite. The veins ranged from 0.0264m to *3 0.914m* in thickness, and the larger had the appearance which would be exhibited by the section of a dry rubble wall of irregularly shaped stones fitting one into another... these veins were the main reason for so deep a trench being sunk.' Eventually, at depth the rotten granite disappeared, the veins became one with the granite they traversed (so far as engineering purposes were concerned) and the trench ended in solid granite rock. The geological plan of the trench wall and bottom was plotted as excavation proceeded, and affords a valuable record of the as-found engineering geological conditions on this part of the site.

Latiyan Dam near Tehran, Persia,

Seven grades of rock conditions were established for the Devonian and Tertiary beds cropping out in the dam foundations at the Latiyan dam near Tehran, Persia.

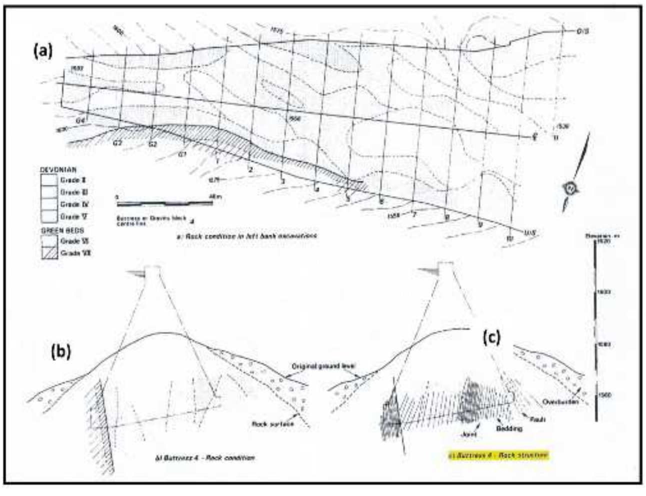


Fig. 1.2: Engineering geological plan of the foundation of the left bank of the Latiyan dam near Tehran Iran. *(in: Dearman 1991)*

The grading classification was based on the assessment of a variety of geological characteristics, which were summarized as:

state of weathering and loss of cohesion;
relative compactness of the rock mass;
intensity and orientation of the various sets of fractures, namely bedding planes, joints and
faults;
relative cleanness of fractures and the rock mass as a whole;
relative abundance of shale layers.

The established grades of rock condition were:

- Grade I. Sound massive rock with no weak seams and widely spaced joints.
- Grade II. Bedded, relatively solid rock, with some shale layers.
- Grade III. Thinly bedded or flaggy rock, with some shale layers.
- Grade IV. Blocky, seamy rock with frequent intercalations of shale and clay-silt.
- Grade V. Broken, faulted rock or weathered rock mixed with shales and/or clay-shales found generally in a loose condition.
- Grade VI. Thinly bedded rock with thin clay-shale seams.
- Grade VII. Friable clay shales.

The approach adopted by *Knill and Jones (1965)* had been one of broad rock-mass characterization. This is without doubt a successful attempt at establishing homogeneous, mappable units which could be correlated generally with the results of in situ and laboratory design tests *(Lane, 1964)*. This showed that rock groupings, based on engineering geological considerations, correlated with rock mechanics parameters. By 1969 the full importance of true engineering geological maps had been recognized.

Symbols in Engineering Geological Mapping.

Symbols used in engineering geological Mapping for Soils, Sedimentary, Metamorphic and Igneous Rocks based on the recommendations of the International Standards Organizations (ISO) Anon, (1981c) are shown on Figure 1.3: below.

Mapping in terms of *geotechnical properties* had already been tried in West Germany between *1951 and 1956,* with emphasis on map units determined by *settlement characteristics and allowable bearing pressures (Gottingen, 1956).*

Fookes (1969) had shown the shortcomings of the normal general-purpose geological map which, for the engineer, was lacking in *quantitative information on the physical properties of rock or soil*, *the amount and type of discontinuities present, the extent of weathering and groundwater conditions*. He pointed out that the *aim of engineering geological mapping should be to produce a map on which the mapped units are defined by engineering properties or behaviour, and the limits of the units are determined by changes of physical and mechanical properties. (Onadera, 1963; Deere and Miller, 1966; Deere, Merritt and Coon, 1969)*.

Terms and Descriptions.

The following terms used in this paper are described and defined below.

Mining Hazards:

Mining activities are usually associated with hazards which consist of the following:
(I) Landslide hazards;
(ii) Earthquake hazards;
(iii) Volcanic hazards

(i) *<u>Landslide hazards</u>:* these have been recorded for well over a century with many classic, spectacular examples spread almost all parts of the world. They can be either naturally occurring or man-induced phenomenon and may be responsible for the cause of more damages to property than any other geological hazard. Rapid evaluations of landslides can be made employing engineering geomorphological mapping techniques that may include, *recording of landforms*, *materials and processes (Doornkamp et al, 1979 and Varnes 1984).*

(ii) <u>Earthquake hazards</u>: Earthquake hazards, and the attendant risks, are usually related to the nature of the ground response during an earthquake. Ground shaking, for example, may provoke new landslides or mobilize old ones, or lead to liquefaction of susceptible superficial sediments. Both may damage man-made structures. The response of standing bodies of water may lead to tsunamis in the oceans. Earthquake Risk Maps are therefore of two kinds - those showing active fault traces and associated epicentres, and those dealing with ground effects.

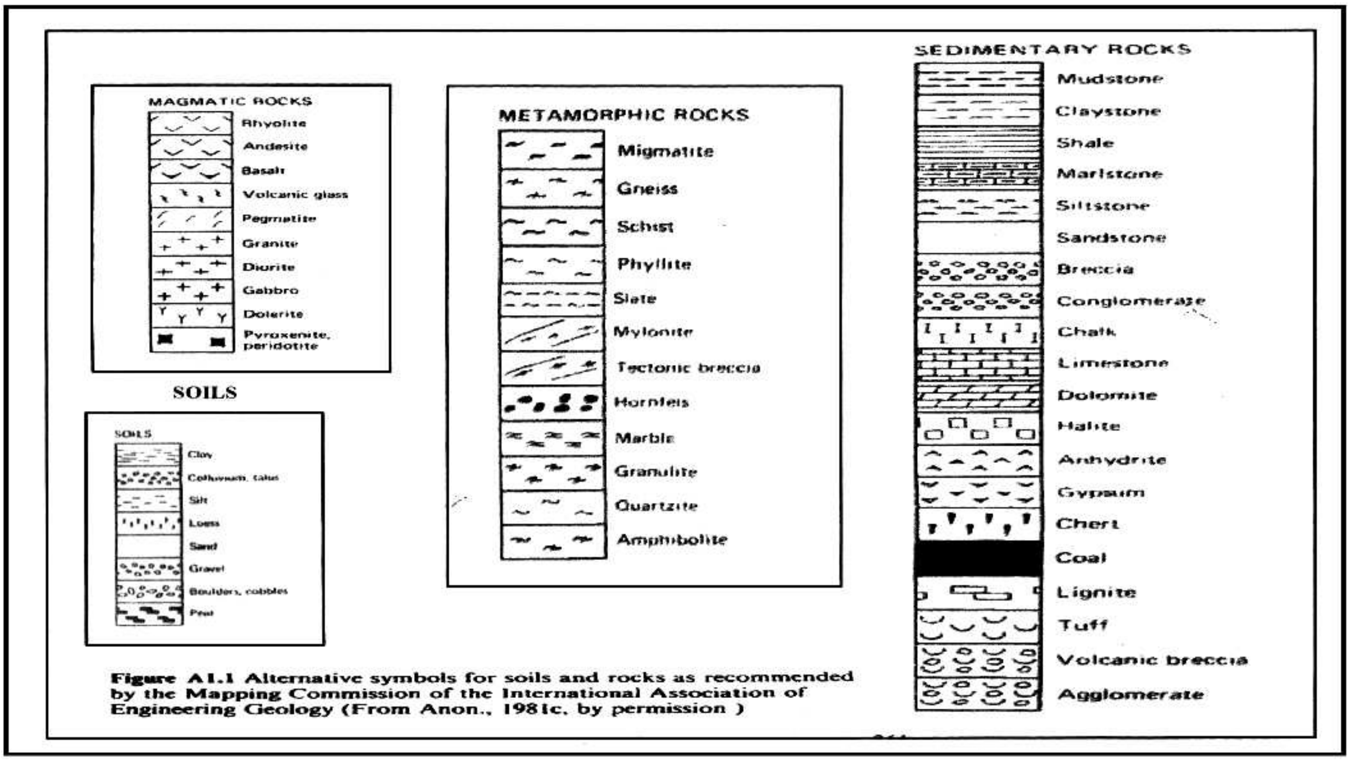
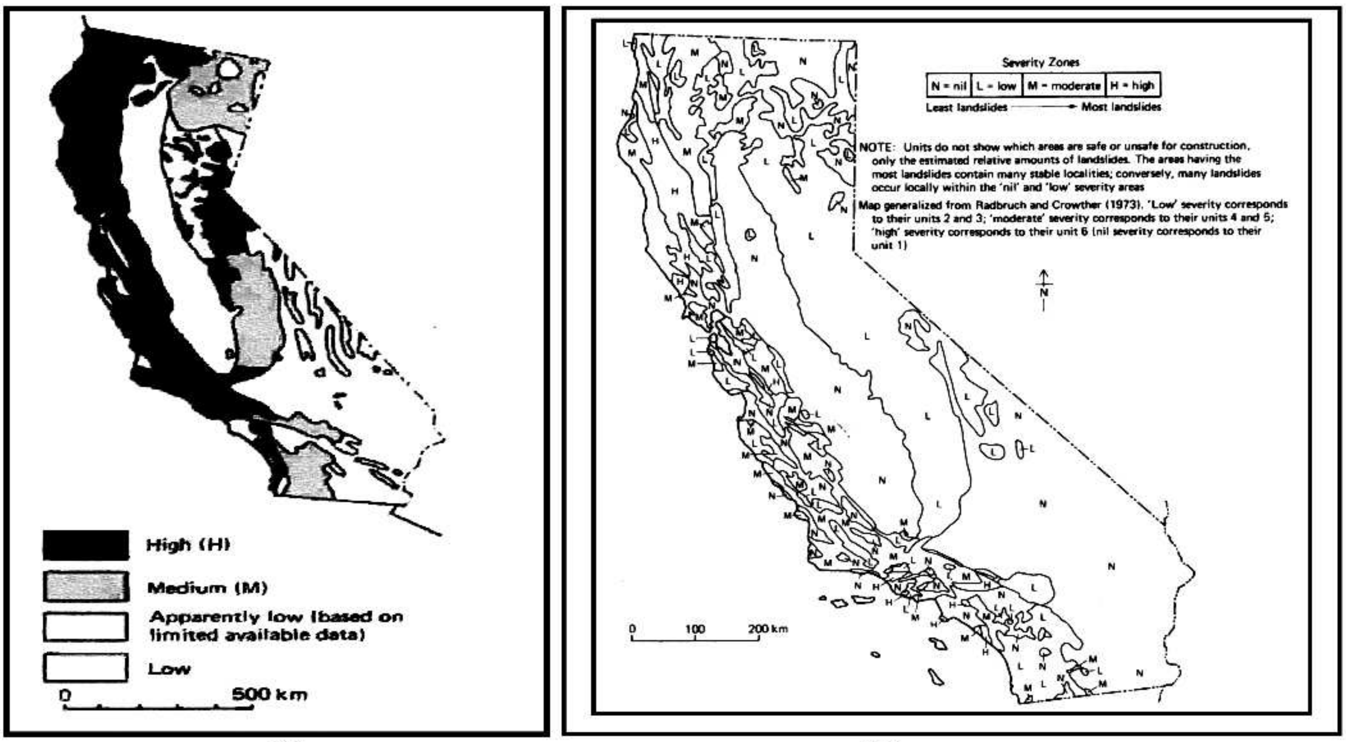


Figure A1.1 Alternative symbols for soils and rocks as recommended by the Mapping Commission of the International Association of Engineering Geology (From Anon., 1981c, by permission)

Fig. 1.2.1: Symbols for Soils, Sedimentary, Metamorphic and Igneous Rocks in Engineering geological mapping, as recommended by the mapping Commission of the International Association of Engineering Geology (from Anon, 1881c in: Dearman, W,R. 1991)

<u>Landslides potential areas in parts of California USA.</u>



(a) **(b)**

Fig. 1.3.1 (a): Landslide susceptibility map of California USA (after USGS, 1982. Fig. 1, (Wiggins Slosson & Krohn 1978).(b) Simple Slope Stability map of California, USA, *[after Radbruch & Crowther 1973, with Severity zones from Alfors. Burnett & Gay, 1973 by permission]*

Microzoning for earthquake effects. Maps of micro-zonations are usually produced in earthquake-prone zones. An example of one of such is shown for the earthquake of February 2, 1931 in Wellington, New Zealand *(After Grant-Taylor et al.1974)*

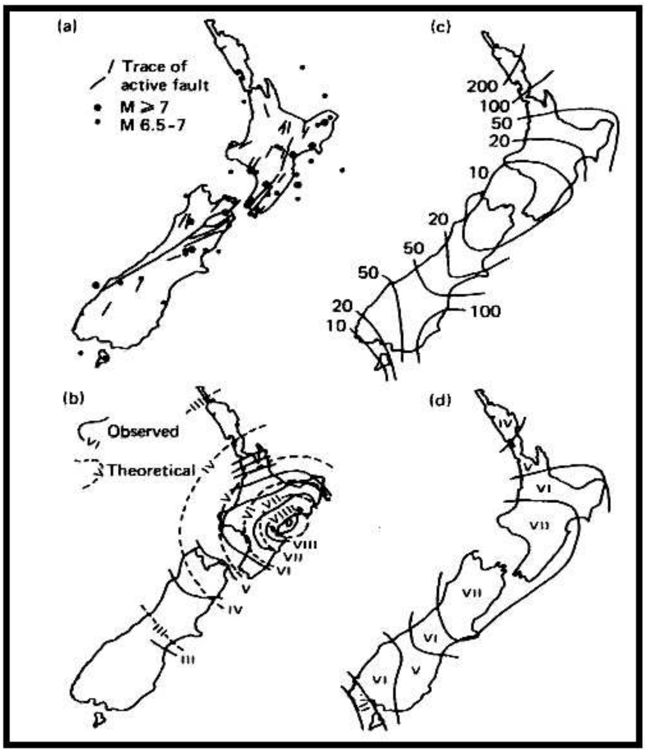


Fig. 1.3.2: Earthquake Maps of New Zealand. [a] principal active faults & larger earthquakes (after Dowrick, 1983); [b] Observed and theoretical isoseismals for the HAWKES bay earthquake of February2, 1931, [c] Return periods in for earthquake intensity of ≥ MM6 (after Smith 1978b), [d] Earthquake intensities with return periods of 50yrs(after SMITH 1978 AND Bell, 1987)

Table 1.1: Seismic hazards in New Zealand urban areas.

Table 12.1 Seismic hazards in New Zealand urban areas

Geotechnical cause	*Scale*	*Frequency*	*Comments*
A. Ground shaking	m vl	L M	Potential for damage over large areas; aseismic design and appropriate construction codes
B. Ground displacement	s m	L M	Scarp development and associated ground warping confined to fault traces; appropriate planning codes and/ or building restriction

Notes:

(iii) Volcanic hazard and risk maps: Mapping of active volcanic areas is similar to mapping seismic hazards inasmuch as the loci of volcanic eruptions can be plotted in terms of last recorded activity (Figure 1.5). Again, it is easy to record and map the size and thickness variations in the volcanic ash deposits from an earlier volcanic eruption. If future events are similar to those in the past, then the extent of future events can be predicted with some degrees of certainty, though the time of occurrence cannot be predicted based on the geologic principles of uniformitarianism - ie *"the past is the key to the present". Westercamp, (1987)* however had attempted a zonation of volcanic hazards associated with Mt. Pelee with some success. An example of a Volcanic hazard Map is shown in Figure 1.3.3.

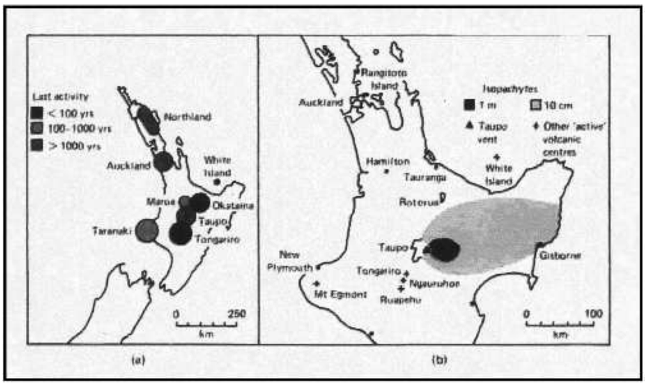


Fig. 1.3.3: Volcanic Hazard Map in New Zealand *(after Bell 1987)*

Drilling Hazards

Drilling operations in mine areas have some associated hazards if carried out in unstable land masses such as very soft rock zones or excessively fractured rock media such as boulder-fields which pose hazards to drilling operations. Engineering geological maps of such areas are useful in indicating potential Drilling Risk zones.

Mapping for Mitigation of Mining Activities

Areas of Mining activities where mitigations against hazards may be practiced include, but not limited to, the following:

Soft Rock Mining

Mining in soft rock (sedimentary) areas include the following:

Sand Mining: Mining for sands may take place in both dry and wet areas of sedimentary rocks and these have to be mapped in order to mitigate future hazards of mining activities.

<u>*Mining in River beds*</u>*:* These are good sources of sand supplies but most times are under water at the bottoms of rivers, creeks and lakes, except in desert areas where seasonal dry river beds do occur.

<u>*Mining in Flood Plains*</u>*:* Flood plains constitute veritable sources of abundant sand supply but they are seasonally flooded and this requires mining with back-hoes and excavators during dry seasons when the areas are exposed or the use of hydraulic or suction dredgers when submerged.

<u>*Impacts of Quick Sands Phenomenon:*</u> Most times, highly saturated Sand bodies on dry Flood plains which are subjected to imposed heavy loads such as excavators can be prone to "Quick Sand" phenomenon and this can pose hazards to drilling. However, submerged sand bodies which are being dredged are not subject to "quick-sand" phenomena.

Hard Rock Mining

In hard rock mining, two types of mining operations are employed. These are (i) Open-Pit (or Open-Cast) and (ii) Underground (Deep Mines).

Open-Pit Mining [Shallow Mines]: These are usually applied when the Ore in question is located *equal to or less than (≤) 25 – 35m* (except in Canadian mines were depths of 50 meters and above have been attained in a copper mine). However, some of the hazards associated with Open-Pit mining are:

Mine Slope Failures and Associated Failure Mechanisms:

Open-Pit Slope stability is a major hazard in this type of mining. The associated Slope failure mechanisms with Open-Pit Mining include:
(i) Planar failure,
(ii) Wedge failure;
(iii) Toppling failures and their derivatives such as "Slide-Toe" "Slide-Head" "Flexural" types (Teme, 1982).

Engineering geological Mapping can be carried out in Open-Pit mines by carefully mapping the quantities of joints, bedding planes, faults and other discontinuities present, strike, dips and azimuths of the joints and discontinuities; joint directions, roughness of the joint surfaces; types of fillings in the joints etc. An example of open pit mine abstracted from a photograph is shown in Figure 2.1: below.

According to Eyre (1973) and Dearman and Fookes (1974), the methodology of mapping involves the following:

(a) the full description of the rock slope face,
(b) the systematic mapping or logging of the face of a cut slope in order to check the original geotechnical design assumptions based on the site investigation,
(c) identification of areas of potential instability during the slope mapping in order to remedy errors observed at the design stage.
(d) production of engineering geological map of the rock slope face or the underground mine walls, as the case may be.

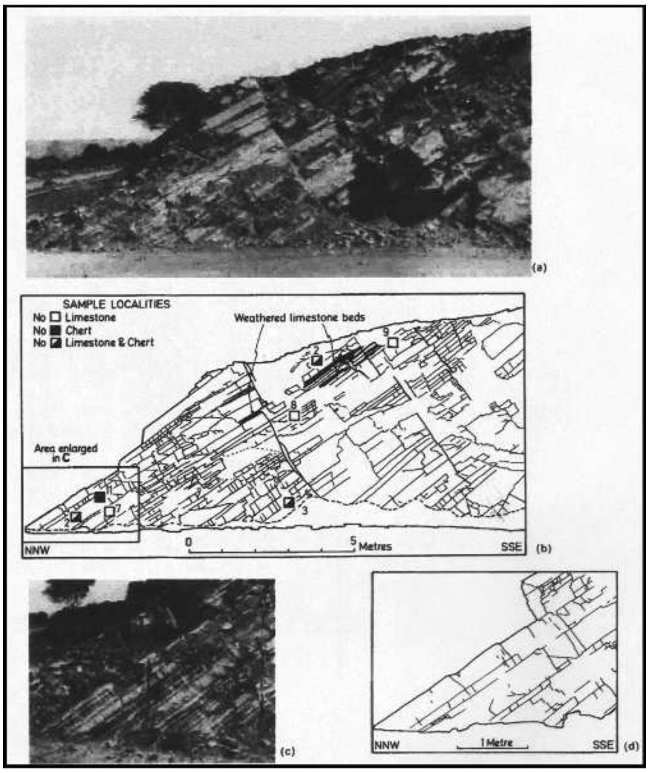


Fig. 2.1: Engineering geologic Map of Open-Pit slopes at Meldon Quarries Okehampton, UK *(abstracted from photographs) (in: Dearman, 1991).* [(a) Quarry face, (b) Quarry face showing drawing of main discontinuities, (c) part of the face in (b) above, (d) drawing of discontinuities for comparison with joint, bedding and lamination frequency in (c)]

Underground Mining [Deep Mines]

Where the depths to ore-bodies are greater *than (≥) 75 m – 100m*, it is usually advisable to use Underground mining technique to retrieve such identified ore-bodies. Mapping of underground ore–bodies is usually undertaken during the preliminary careful logging of the rock cores obtained during the drilling of the exploratory borings prior to the design of the mines proper. When the mines are developed or developing, the advancing mine faces and the mine walls are also mapped to show the geological structures exposed during mining.

Large Underground Storage Tanks [in USA, Canada, Australia, South Africa and the UK]

Engineering geological mapping of underground Mines.

Large underground openings in hard rock have been used for storage facilities such as liquid fluids etc and these can also be mapped from surface drillings before construction and during mine workings. An example of Foundation investigation geotechnical mapping is shown in Figure 2.2: below.

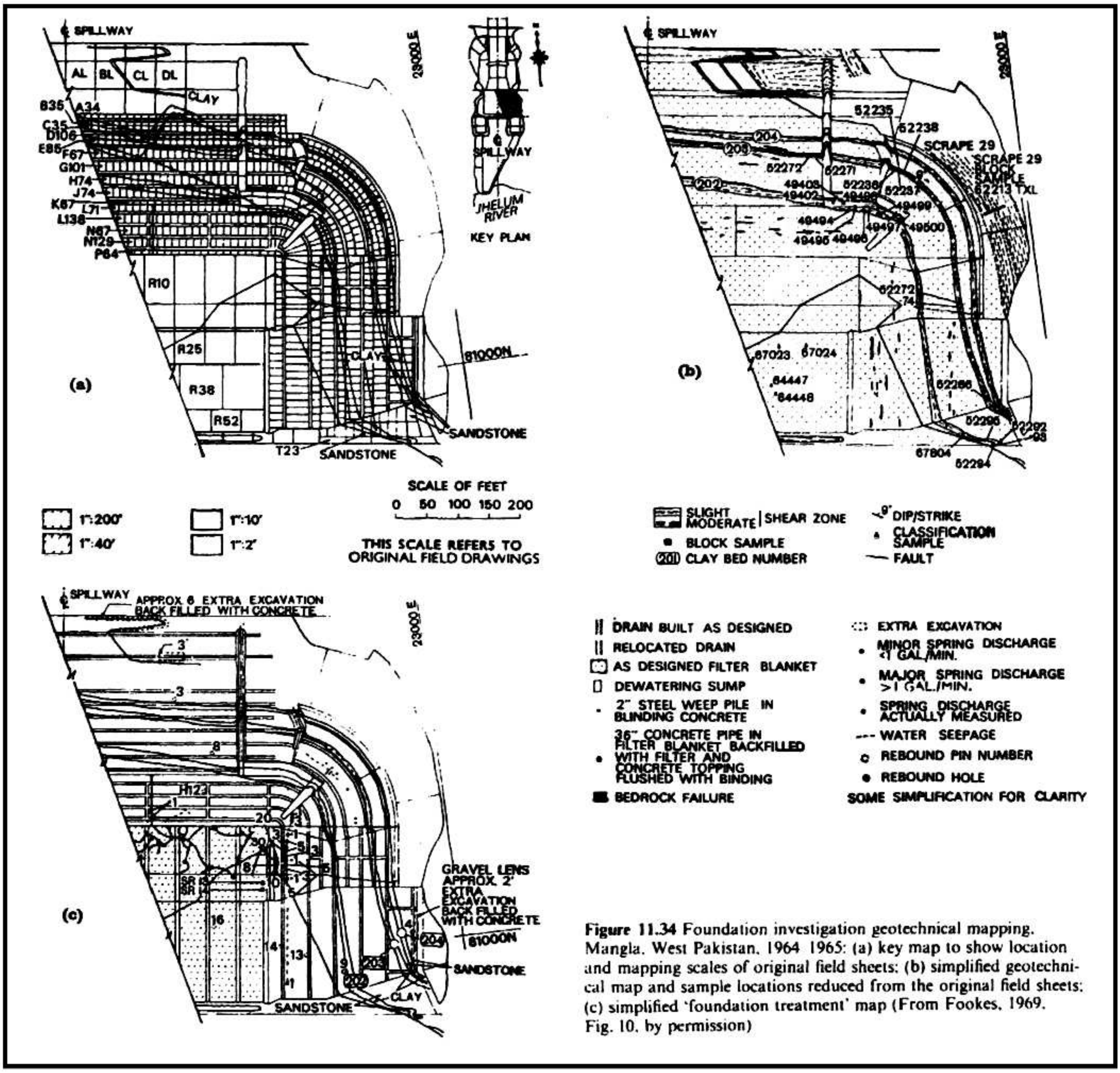


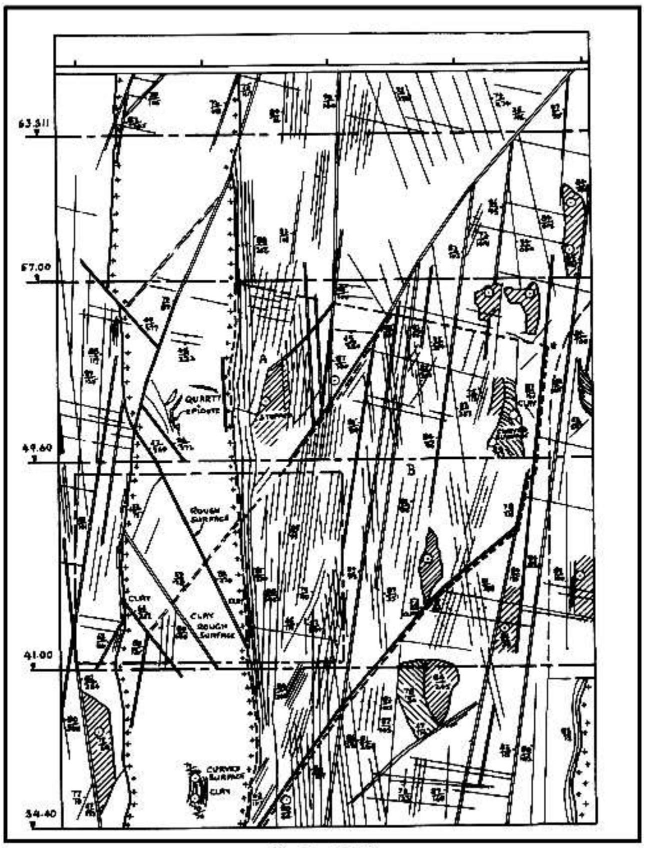
Figure 11.34 Foundation investigation geotechnical mapping. Mangla. West Pakistan. 1964 1965: (a) key map to show location and mapping scales of original field sheets: (b) simplified geotechnical map and sample locations reduced from the original field sheets; (c) simplified 'foundation treatment' map (From Fookes. 1969. Fig. 10. by permission)

Fig. 2.2: Foundation investigation geotechnical mapping, Mangia, West Pakistan *(after Fookes 1969)*

Tunnels and Shafts for Municipal Drainages

Engineering geological mapping methods for *Tunnels and Shafts* should be planned in a way not to interfere with the groundworks, therefore should be as fast as possible. As such careful consideration should be given to the possible use of photography, provided that a scale, such as a ranging rod, is included in the photograph to know and record the dimensions of encountered discontinuities along the line during the mapping exercise. According to Douglas, Richards and O'Neill (1977), during engineering geological mapping there should be the need to quickly identify areas of potential instability for which remedial measures would need to be designed. Therefore during mapping of an underground excavation, *the walls, roof and invert should be properly mapped in order to gain as much information as possible, to reflect the three-dimensional aspects of the structural geology.*

An example of a geotechnical mapping of a Machine Hall **wall** is shown in **Figure 2.4:**.



Scale: 1:50

Fig. 2.3: Geotechnical engineering mapping and evaluation Map of the Wall of an underground Machine Hall at Dinorwic North Wales, UK. *(in: Dearman 1991).*

Tunnel Wall Mapping: It should be remembered that the main purpose of engineering geological mapping during construction is to monitor local geological conditions that might give rise to ground support problems such as the identification of rock blocks that, because of their geometrical shape, could possibly move outwards into the excavation and would therefore require support. To this end the line method of mapping adopted for tunnels (Anderson, Arthur and Powell, 1977) has to be modified to give a reasonably accurate plan of the tunnel wall being mapped. A typical version of the line mapping used in tunnels extended by visual offsets vertically above and below the main logging-line - in other words - a simple adaptation of the *"chainage-and-offset method of mapping"*. as used by *Dearman and Fookes (1969)* is shown in Figure 2.4. Proctor (1971) considered the general mapping methods in tunnels while Pupo et al (1979) working in underground works in crystalline rocks in Brazil also applied this method of line mapping.

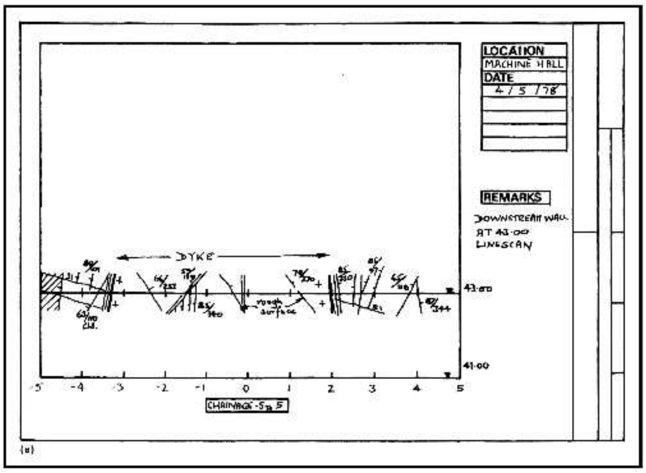


Fig. 2.4: Line Mapping in engineering geological mapping exercise used at Dinorwic North Wales *(in: Dearman, 1991).*

Roof Mapping: Mapping the roof in the same detail as the walls of the excavation prove to be practically impossible because of the difficulty of access without special plant being always available. Consequently, recording should be confined to sketching in major features such as minor faults, dykes, dyke margins and bedding surfaces. This can be carried out from the floor of the excavation, after wall mapping had been completed.

Mapping of the Invert: This can only be undertaken when the floor had been completely cleared of debris, the rock surfaces cleaned and the permanent blinding concrete has been applied to the walls. At this time careful surveying and, consequently, the geological details could be mapped to a high degree of accuracy.

Hazards in Mapping in Underground Mining Operations

Like in most other mapping exercises underground, there are likely associated Hazards, and these are the following:

Mines Shaft Roof Collapses.

Instances of mine roof collapse, specially before the application of rock-bolts and accompanying shotcrete are quite common because this is the best time to map the structures on the mine roof. It is therefore very necessary that the field engineering geologist must finish with the mapping of the freshly exposed roof before he application of the above mentioned support systems.

Mines Pillar Compressive Failure.

Depending on the rock type, the weakness zones, the dimension of the mine pillars and the imposed dead load on the pillar (such as the depth of the mine in relation to the mine pillar dimension), compressive failures of mine pillars do occur. These constitute fatal mine hazards in underground mining.

Mine wall squeezing failure.
With the creation of an underground cavity, there is a redistribution of the in-situ stresses and in wide openings such as machine halls and storage spaces, the x – z stresses increase drastically often leading to mine wall squeezing. Though these are usually controlled by rib-cages and other steel-support mechanism. Therefore the exposed mine walls should be quickly mapped by the engineering geologist on site. Mapping in this case is usually accomplished by either Line-mapping of mine photography soon after exposure through drilling.

Mapping for Mitigation of Drilling Hazards

Hazards of Drilling

Drilling hazards can occur in both soft and hard rock terrains and engineering geological mapping can be used to mitigate these hazards to a minimum.

In Soft Rocks [Sedimentary Rocks].

One of the major problems in drilling in a soft rock area is that of the stability of the ground on which the drilling rig is mounted. This is usually remedied by support systems to prevent the uplift of the rig during drilling activities. These soft rock areas should be identified and included in the engineering geological maps so produced.

Pollution of Groundwater.

One of the hazards of drilling operations is the pollution of the groundwater around the areas of the drilling through the introduction of drill fluids into the aquifer system. It is recommended that only clay-based drilling fluids such as bentonites be used in the drilling process without the applications of harmful chemicals. In normal geotechnical engineering ground investigations, only the Environmental Monitoring Boring (EMB) and system of Teme (1998) be used since this system does not use any chemicals in drilling.

Proximity of Drilling Points Leading to Interference of Zones of Influence of Groundwater Regimes.

Another potential source of drilling hazard is the close proximity of water borings using the conventional drilling fluids in which case there exists the contaminations of a wider area through the interference of zones of influence of groundwater regimes. Such hazardous zones brought about by Drilling activities should be incorporated into engineering geology maps.

Use of the "EMBs" – Environmental Monitoring Borings technique (Teme, NAPE 1998).

In this system of engineering geological boring for studies in Environmental Impact Assessments (EIAs), the Percussion system of Standard Penetration Tests (SPTs) only are used to prevent the further introductions of chemicals into the soils and groundwater systems to be assessed. This system was first introduced during the Baseline Studies of some Shell Petroleum Development Company (SPDC) Flow Stations during the early days of "baseline studies" in the Niger Delta Subregion prior to the advent of EIAs in Nigeria. *[Note: The very first EIA Study in Nigeria was carried out by the Institute of Floods, Erosion, Reclamation and Transportation (IFERT) – now known as the Institute of Geosciences and Space Technology (IGST) the the Institute of Pollution Studies (IPS) – both in Rivers State University (RSU), when the First EIA Report on the Eleme Petrochemical Complex, Onne was produced in 1996].*

In Hard Rocks (Igneous, Metamorphic and Volcanic Rocks).

<u>*Highway Cuts Through Hard Rocks.*</u>

In some high hard rock terrains, road construction activities pass through road cuts and embankments. These road cuts may have joints that "day-light" on the face of these cuts and thereby pose slope stability failure problems. During the cutting of the rocks to create the highway routes, several drill holes are made for blasting purposes and these pose as drilling hazards.

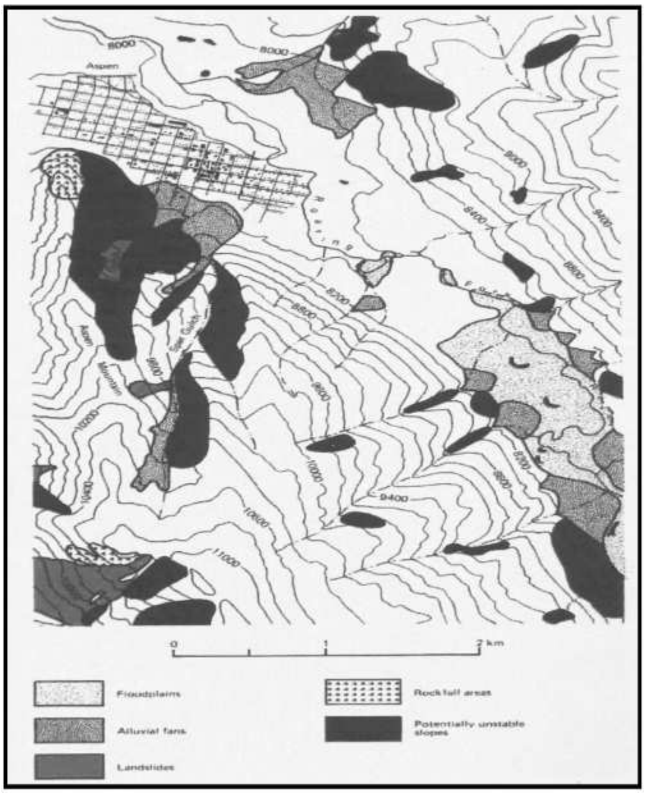


Fig. 3.1: Engineering geological map of potential geologic hazards in Aspen, Colorado, USA. *(after Bryant, 1972b)*

<u>*Vibratory Drilling Induced Rock Slope Failures.*</u>

In rock outcrops that contain numerous discontinuities at different orientations, vibrations from the drilling activities induce rock slope failures which pose as hazards. Hence, during explorations for highway routes, engineering geology maps should identify the zones with potentials for various degrees of instabilities and potential failures and rock falls. Other studies in rock mass movements in relation to engineering geological mapping have been reported by *Matula, M (1966, 1980,1978 and 1980)*

<u>*Wedge Slope Failures & Planar (Slickenside-filled) Slope Failures*</u>

Where rock discontinuities intersect at obtuse angles on rock slopes adjacent to highways and roadways, the possibility of occurrence of of Rock Wedge Slope failures are often high. These cause obstructions, damages to passing vehicles and even fatal accidents.

A Hazard Map due to rock slides and landslides of various degrees of severity in Aspen Quadrangle, Colorado USA. is shown in Figure 3.1: above.

Results from Engineering Geological Mapping and Investigations in the Niger Delta Sub-Region of Nigeria

Through several studies within the Niger Delta region by the Institute of Geosciences and Space Technology (IGST) [formerly known as the Institute of Floods, Erosion, Reclamation and Transportation (IFERT)], three Hydrometeorological Zones have been identified within the general area of the Niger Delta Subregion.

Identification of Zones in a Regional Setting

Studies in the Niger Delta Sub-Region, (Fubara, Teme et al, 1988) had identified three basic hydrometeorological zones within the Niger Delta. These are discussed in the sections below.

Hydrometeorological Zones.

These zones are the (i) Coastal Zone, (ii) The Mangrove or "Transition" Zone and the (iii) The Freshwater Zone. The engineering geological map of the Niger Delta showing these three zones are shown in Figure 4.1:

<u>*Coastal or Lower Niger Delta Zone.*</u>

The Coastal or Lower Niger Delta zone consists mainly of sand bars and ridges with saline water bodies. The area is subjected to diurnal ebb and flow tides and thus not prone to the annual floods of the freshwater zone. The lithology here consists of *sands, silts and highly plastic clays* in some inlet areas. Some settlements in this zone include Bonny, Akassa, Brass, Forcados, Bekinkiri, Koluama I (wiped out by wave erosion), Koluama II and Oyorokoto.

<u>*The Transition or Mangrove (Middle Delta) zone.*</u>

The Transition or Mangrove (Middle Delta) zone coincides with the Mangrove brackish water zone with its numerous inter-tidal flats and mangrove vegetation. The lithology is characterized by a typical *fibrous and pervious clayey-mud* that exhibits large values of compressibility indices (m_v) of *(0.12 – 0.18)* and consolidation coefficients (c_v) of *(0.45 – 0.58)*. 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 sands, gravels and clays – are present in certain *'old'* residual deposits, on which are usually located densely populated towns such as *Port Harcourt, Warri, Sapele, Bakana, Buguma, Opobo, Okrika and Abonnema*. The vegetation within the 'Transition' zone comprises, basically, of mangrove trees, especially within the saline swamps and along the banks of the numerous rivers, rivulets and creeks. Like the Coastal Zone, the transition zone experiences diurnal ebb-and-flow of the tides with maximum values experienced during the *once-a-year Spring tides*. The Coastal and Mangrove (Transition) zones together constitute the "Marginal lands" where many Fuel Depots are constructed within the Niger Delta sub-region.

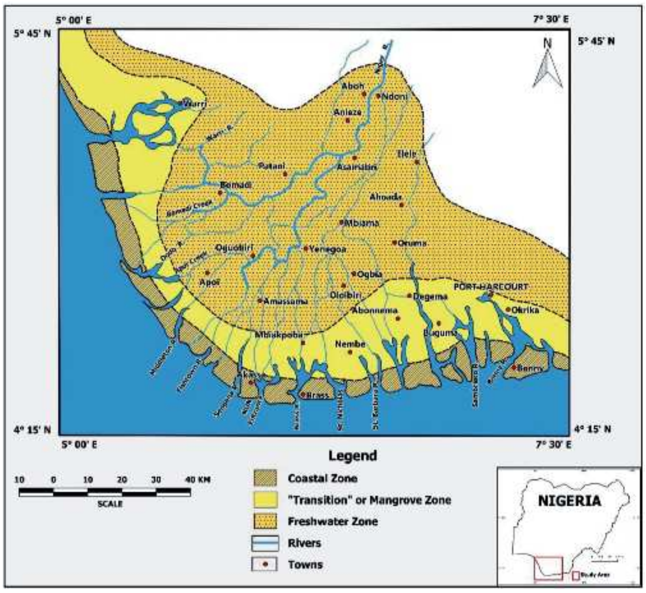


Fig. 4.1: Engineering geologic Map of the Niger Delta Sub-region showing the three Hydrometeorological Zones. *(after Fubara, Teme et al. 1988)*

Some Results of Engineering geological Mapping exercises from the Transition (Mangrove) Zone.

The Transition zone of the Niger Delta subregion has important towns such as Port Harcourt, Warri, Sapele and others. These areas have fewer land areas than the population can contain, hence the need for land reclamations by the locals for erecting houses for human habitation. Maps containing the distributions of geotechnical properties of some subsurface materials in recent study are shown in the figures below (after Morrison, 2017).

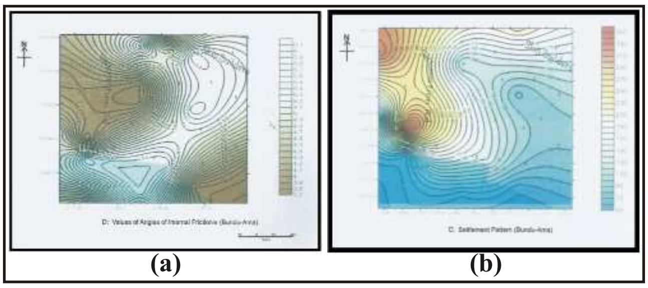


Fig. 4.1b: Engineering geological mapping of parts of Port Harcourt Marginal lands showing (a) Locations of Sampling points, (b) values of angle of internal friction (ϕ) (after Morrison, 2017).

The Freshwater or Upper Delta zone.

The Freshwater or Upper Delta zone comprises the remaining northern portion of the Niger Delta sub-region and covers the predominantly fresh water rivers, creeks and ephemeral depressions. Generally, soil profiles within this zone 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, Ndoni, Isampou, Patani, Asamabiri, Yenegoa and Amasoma*.

Engineering geological mapping in building foundations applications in Geotechnical Engineering Practices.

Engineering geological mapping plays a key role in foundation engineering practice. As a first step in any building project it is compulsory to have an engineering geological map to guide the location(s) of the proposed structures at such sites. An example of such map for a project in the southern part of Nigeria is shown below in **Figure 4.2:**

Summary and Conclusions

Engineering geological mapping constitutes a vital and primordial aspect of the practice of engineering geology and geotechnics. It is a precursor to virtually all projects in engineering geology. This Lead Paper therefore defines in details what engineering geological mapping is, giving some historical perspectives of the early engineering geological mapping systems and how these

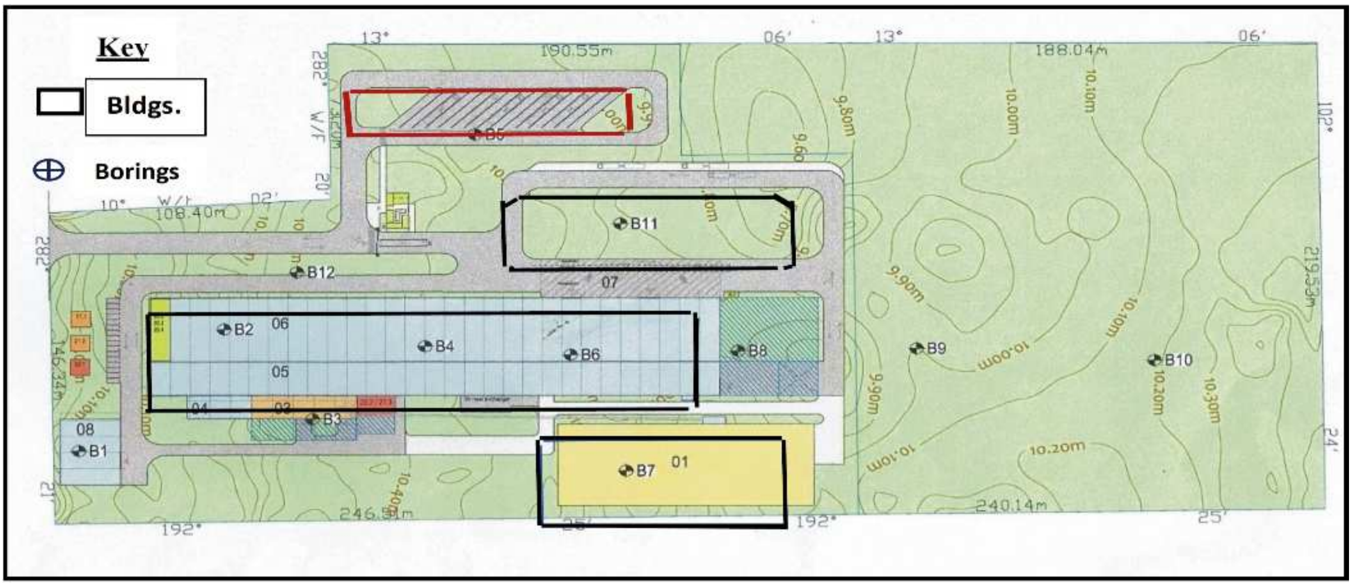


Fig. 4.2: Engineering geological map in building foundation applications – Case from southern Nigeria. *(after the Author, 2019)*

mitigate hazards of mining and drilling operations. The paper concludes and highlights the importance of engineering geological mapping for the mitigation of common hazards usually encountered in mining activities in both Soft and Hard rocks, especially in open-pit mines in soft rocks and in both Open-Pit and Underground Mining operations in hard rocks terrains. Finally, some results accruable from engineering geological mining exercises within parts of the Niger Delta Sub-region are (i) the identification of the three Hydrometeorological Zones in the Niger Delta sub-region (Fubara, Teme et al, 1988) and (ii) the applications of engineering geological mapping in the selection and design of appropriate building foundation systems in geotechnical engineering practice in parts of Nigeria.

Recommendations

Recommendations accruable to this Lead Paper include the following:

(a) That all engineering geology and geotechnical engineering projects should first entail detailed engineering geological mapping exercise to identify discontinuity zones such as bedding planes, joints, faults and lineaments,

(b) As a follow-up to the engineering geological mapping exercises, there must be a detailed subsurface geotechnical investigation to determine the in-situ bearing capacities for purposes of bearing the weight of proposed superstructures and where found inadequate, efforts should be made to ameliorate this by either designing appropriate foundation systems for the structures or apply soil improvements techniques prior to building of such structures. .

©) All engineering geological mapping exercises as well as subsurface investigations should be carried out by well-trained and **COMEG-certified** Engineering geologists and Mining Engineers or **COREN-certified** Civil engineers.

(d) Engineering geological mapping (both theory and field practice) should be compulsorily taught in Engineering geological Programmes both at the Undergraduate and Graduate levels in our tertiary education systems.

(e) Field Internships under well-trained Engineering Geologists must be made compulsory for **ALL Majors** in Engineering geology, Geological engineering or Geotechnical engineering programs, especially during the ***Study-Industrial – Work – Experience – Schemes (SIWES)*** in our various university systems in Nigeria..

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Environmental Geological Maps for Urban Centres

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Abstract

The number of megacities (>10million inhabitants) in the world increased from 14 in 1995 to 31 in 2017 and by 2050, two thirds of the world population will be living in megacities. This unprecedented high urban population growth would require greater use/exploitation of land and the physical environment in order to meet the demand for more housing, water, energy, infrastructure, waste disposal, etc. Environmental geology is of fundamental importance in the planning and development of urban environment because it ensures that the planned land use is (i) the most suitable for the type and quality of the land available, (ii) does not destroy valuable subsurface natural/mineral resources and (iii) does not cause environmental contamination and degradation. However, Earth Science factors/data required for sustainable land use planning are not available in conventional geological maps and are also difficult for non-geologists (i.e., planners and administrators, etc) to understand. To meet these needs, Environmental Geological Maps (EGMs) evolved and they essentially display summary of information derived from factual "basic data" maps, interpretative maps or derived maps. EGM can depict both the resources for development, such as areas for good foundation conditions, groundwater supplies, mineral resources, construction materials as well as constraints to development such as seismicity, subsidence, landslides, flooding, waste disposal, etc. EGMs assist to ensure that the geological information can be readily understood and incorporated in the planning process. Examples of different types of environmental geological maps with the accompanying notes and extended legends are presented. The worsening flooding problem in the growing satellite urban centres of Port Harcourt is also presented to demonstrate the consequences of failure to use EGMs and the absence of an urban Integrated Flood Management and Drainage Control Master Plan.

***Keywords**: Environmental geological Maps; Urban Centres, Population Growth, Land Use, Environment.*

Introduction

Human society is currently facing several unprecedented global challenges which include; exponential growth in population, urbanisation and industrialisation, resources depletion, food, water and energy security, infrastructural facilities, poverty, environmental sustainability, climate change impact, biodiversity loss, natural hazards, etc. These challenges threaten the future of humanity and the integrity of the carrying capacity of the earth's ecosystems. The exponential growth in world population increased from 2.6 billion in 1950 to 7.2 billion in 2013 and projected to reach 9.6 billion in 2050 (UNDESA, 2013, 2014). About Half of the 2.4 billion increase in population from 2013 to 2050 will occur in sub-Sahara Africa, where in 2014, one in four people remained malnourished (FAO, IFAD & WFP, 2014, Davis, 2015).

This unprecedented world population increase exacerbates urbanisation process. The percentage of the world population living in urban areas increased from about 28% in 1950 to 46% in 2000 and is projected to increase to about 70% by the year 2050 (UNDESA, 2014 (Table 1). From the sustainability perspective, the major challenge created by this high population growth is that of meeting the needs of more and more people which will demand higher consumption and production without imposing catastrophic and irreversible damages on the natural environment. Population density and growth are highest in urban centres, coastal regions and deltas, which make them "frontlines" of global change. Comprehensive proper planning for the projected urban population growth is critically important and this requires input of earth science and geological information in the form of environmental geological maps. The importance of geology in planning physical facilities and individual structures cannot be over emphasized since land is the surface expression of underlying geology.

Table 1: Projected growth in world population and urbanization

	1950 *(billion)*	2000 *(billion)*	2050 *(billion)*
World population	2.6	6.1	9.6
Africa & Asia	1.5	4.5	7.7
Urban population	0.75 (28% of world)	3.4 (46% of world)	6.7 (70% of world)

Source: *World Bank & other sources*

Projected Growth of World Urban Centres and Megacities

"Megacities" are defined as urban areas with more than ten million inhabitants. They host intense and complex interactions between different demographic, social, political, economic and ecological processes. Megacities undergoing economic boom times often generate considerable opportunities, as well as strong pressures for change accompanied by environmental degradation. The number of megacities (>10million inhabitants) in the world increased from 14 in 1995 to 31 in 2017 and by 2050, two thirds of the world population will be living in megacities. About 3.5 billion people currently reside in urban areas and this number is projected to reach 6.3 billion by 2050, increasing from 50% to about 70% of the world's population. According to the 2014 revision of the World Urbanization Prospects by UN DESA, urbanization could add another 2.5 billion people to the urban population by 2050, with close to 90 percent of the increase concentrated in Asia and Africa. Africa is currently home to seven rapidly growing megacities: Cairo *(Egypt),* Accra *(Ghana),* Johannesburg-Pretoria *(South Africa)*, Khartoum *(Sudan)*, Kinshasa-Brazzaville *(Democratic Republic of the Congo and Republic of the Congo)*, Lagos *(Nigeria)* and Nairobi (Kenya). The three fastest growing urban centres in Africa are; Kinshasa, Lagos and Dar es Salaam and they are all located in sub-Sahara West, central and east Africa (UNDESA, 2014; Davis, 2015).

According to the report of the Global Cities Institute, human geography will look completely different by the turn of the century. The list of the 20 largest megacities projected for 2100 is presented in Table 2. The table shows that 13 of the world's largest megacities will be located in Africa, three will be found in India and none will be found in the Americas, China, or Europe. Nigeria's largest city, Lagos, has seen explosive growth over the past few decades with over 2,000 people migrating to the city every day. The current population estimates vary widely from 18 to 20 million inhabitants and is projected to be not less than 88 million in 2100. This will make Lagos most populous city in the world in 2100 *(https://www.thegeniusworks.com/2018/08/the-worlds-megacities-of-2100-lagos-kinshasa-dar-es-salaam-a-mind-blowing-growth-in-the-urban-populations-of-africa/)*. The populations of Kinshasa, DRC and Dar es Salaam, Tanzania, are projected to be 83 million and 74 million inhabitants respectively (2nd and 3rd largest).

Table 2: 20 largest megacities projected for 2100

	Population (2100)	City	Country
#1	88.3 million	Lagos	Nigeria
#2	83.5 million	Kinshasa	DRC
#3	73.7 million	Dar Es Salaam	Tanzania
#4	67.2 million	Mumbai	India
#5	57.3 million	Delhi	India
#6	56.6 million	Khartoum	Sudan
#7	56.1 million	Niamey	Niger
#8	54.3 million	Dhaka	Bangladesh
#9	52.4 million	Kolkata	India
#10	50.3 million	Kabul	Afghanistan
#11	49.1 million	Karachi	Pakistan
#12	46.7 million	Nairobi	Kenya
#13	41.4 million	Lilongwe	Malawi
#14	40.9 million	Blantyre City	Malawi
#15	40.5 million	Cairo	Egypt
#16	40.1 million	Kampala	Uganda
#17	40.0 million	Manila	Philippines
#18	37.7 million	Lusaka	Zambia
#19	36.4 million	Mogadishu	Somalia
#20	35.8 million	Addis Ababa	Ethiopia

https://www.visualcapitalist.com/worlds-20-largest-megacities-2100/

Environmental Implications of the Projected Growth of Urban Centres/Megacities

Cities are centres of creativity and economic progress but they also face many environmental challenges due mainly to air pollution and weather, climate and water-related hazards. Several megacities appear to have reached their physical and managerial limits and under this scenario, prices of urban land become prohibitively high, leading to intensification of land use with development of more high-rise buildings, underground space and encroachment of ecologically sensitive areas (Davis, 2015; Megacities, 2005). The projected unprecedented high urban population growth would require greater use/exploitation of land and the physical environment in order to meet the demand for more housing, water, energy, infrastructure, waste disposal, etc. Urban environmental problems are mostly inadequate water supply, wastewater, solid waste, energy, loss of green and natural spaces, urban sprawl, pollution of soil, air, traffic, noise, etc. Although these severe geo-environmental, economic and social problems are present in many urban areas worldwide, these problems are most acute in Africa because of the exceptional population explosion (among the highest in the world), poverty, economic transition where there is a conflict between the short-term economic plan and the protection of the environment as well as lack of resources and technical capability and capacity to tackle these urban problems. In the developing world, megacities grow faster than their infrastructure which leads to urban sprawl, high traffic congestion, ecological overload, insufficient housing development and, in some cases, such extremes of poverty and wealth living side by side.

Megacities require human and natural resources for energy, industry, construction, infrastructure and maintenance which are needed in quantities that have severe impacts both locally and globally-*a megacity's so-called "ecological footprint"* (Global Footprint Network, 2016; Decker et al, 2002). Megacities commonly encroach on areas with difficult ground conditions, subject to geohazards, such as flooding or landslides which makes both initial development and long-term maintenance more expensive. In 2015, the United Nations member countries agreed on a set of 17 Sustainable Development Goals (SDGs) to steer global development efforts in decades to come (Table 3). Goal Number 11 *(make cities and human settlements inclusive, safe, resilient and sustainable)* is dedicated to Sustainable Cities and Communities which underscores the importance of proper development of mega cities.

Sustainable development represents an approach that stipulates that human needs of the present should be met without undermining the resource and ecological base which future generations require to meet their own needs (WCED, 1978). Meeting economic, social and political goals falls within the sustainable development as well. A successful city in sustainable development terms is the one where many different goals of its inhabitants and enterprises are met without passing on cost to other people in space or time. Urban authorities must control the fabric and land use patterns within a megacity to minimise adverse environmental effects - both for the city itself and the hinterland upon which it depends. This requires integrity in resource, logistics (traffic) and waste management, with cost-effective recovery of value, recycling of wastes and materials, and, as far as practicable, reduction of risks to health. In urban planning, land-use planning seeks to order and regulate land use in an efficient and ethical way, thus preventing land-use conflicts.

Fig. 1.0: An example o

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

Need for Geological information in Cost-Effective Land use Planning in Urban Cities

Land-use planning represents an attempt to reduce the number of conflicts and adverse environmental impacts in relation to society and nature. It involves the collection and evaluation of relevant data to enable the formulation of plans. The resulting policies depend on economic, sociological, and political influences in addition to the perception of the problem. Land use planning is a political process, with decisions usually taken at various levels of government, depending on the project, after receiving advice from professional officers. It is the planner who makes the recommendations regarding planning proposals, but when a specialist topic is involved, then the recommendations are only as good as the advice received from a specialist. In the geological context, sufficient geological data should be provided to planners and engineers so that, ideally, they can develop the environment in harmony with nature. Figure 1 shows a typical planning process (Bell, 1986). As can be seen from the figure, geological information is required at all levels of planning and development from the initial identification of a social need to the construction stage. Even after construction, further involvement may be necessary in the form of advice on hazard monitoring, maintenance, or remedial works. Land use planning has the potential of playing an important part in man's relation to his physical and cultural environment. However, planners and policy makers are hampered in implementing large-scale planning decisions by their inability to anticipate or visualize the many relations that must be considered in developing rational and comprehensive planning alternatives.

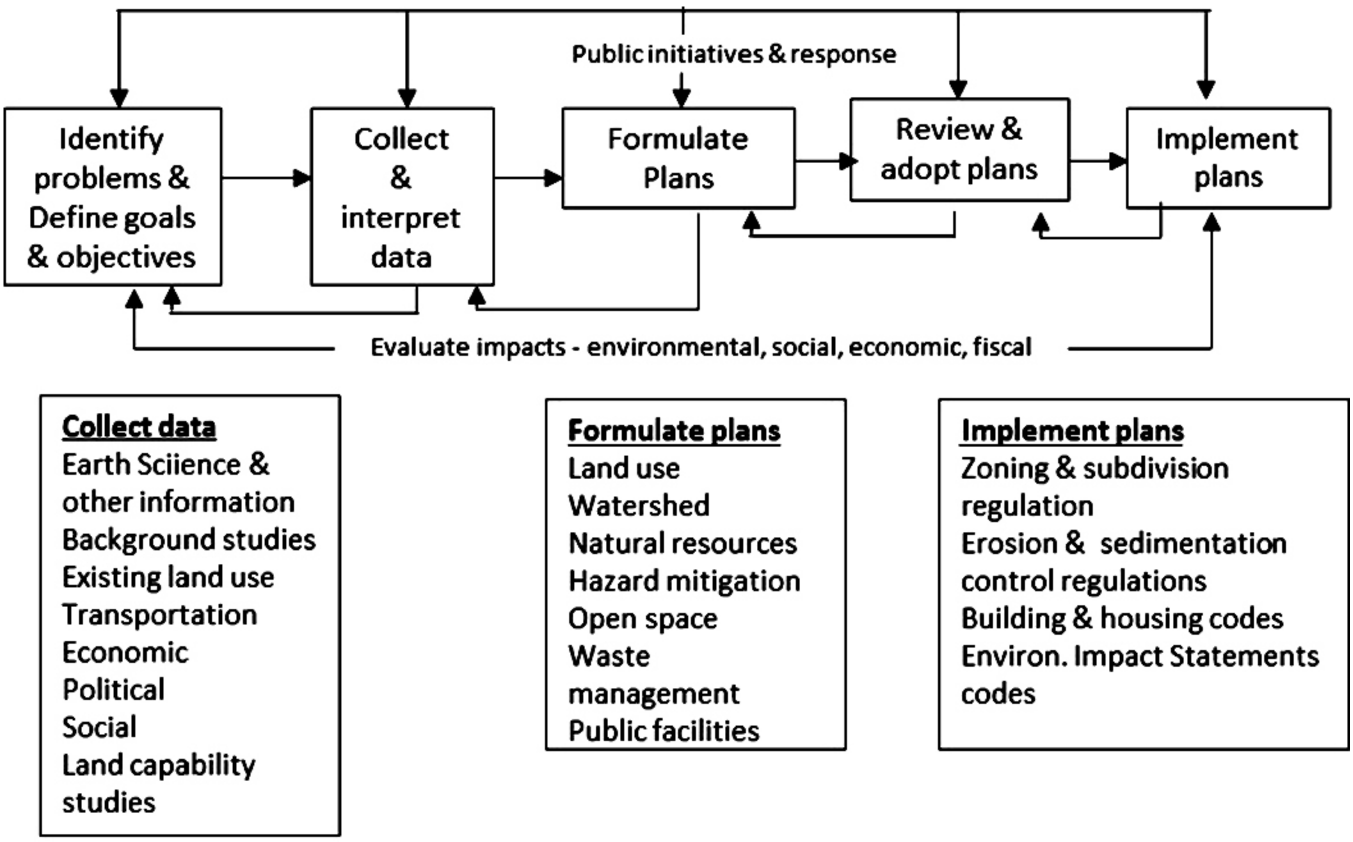

Fig. 1: Diagram of the land-use planning process *(Modified from Bell, 1986)*

Historically, the early stages of land use planning process in urban areas started without sound geologic base and input. The resultant uncontrolled building practices caused developers and home owners to suffer great losses from impacts of floods, foundation failures, and numerous other earth hazards and processes. However, with increased global environmental awareness, political pressures started demanding that urban planners should manage all land use to protect the environment. Therefore, it should be the role of the engineering geologist, who has adequate knowledge of both civil engineering and geology, to formulate a geologic base for sustainable land use planning (Anon, 1982; Lee at al 1988; Bell 1986; Tan & Ran, 1986; Griffiths, 2017, etc). Two fundamental needs must be satisfied:

1) establish engineering and environmental geology criteria for specific land use suitability; and
2) communicate the findings to the public, a public that includes persons who may not be knowledgeable about engineering, geology, and (or) planning.

Land-use planning can only be achieved satisfactorily if there is a proper understanding of the geology of the area. In addition, the development of land must be planned with the full realization of the geodynamic natural forces that have brought it to its present state, taking cognizance of the dynamic character of the natural environment, so that development does not upset the delicate balance any more than is essential. Geology should therefore be the starting point of all planning. Consequently, it is important that the planner, developer, and civil engineer should readily appreciate geological data.

One of the principal ways of representing geological data is with maps. Maps represent a means of storing and transmitting information, in particular, specific information about the spatial distribution of given factors or conditions. Unfortunately, the conventional geological map is often inadequate for the needs of planners, developers, and civil engineers. Recently, however, various types of maps incorporating geological data have been developed for planning purposes. Such maps include morphological maps, engineering geomorphological maps, environmental geological maps, and engineering geological maps. Essentially, the maps should be simple and provide some indication of those areas where there are geological constraints on development and areas without constraints.

In most cities, humankind has transformed the natural environment and vegetation, replacing them with concrete, asphalt, and other surfaces, transformed or buried riverbeds, caused city climate and created huge artificial transfers of energy, water, and various substances. Growing cities are changing hydrological relationships and thereby influence the size and frequency of floods. Generally, the older districts of most urban centres are built on good grounds in terms of foundation material quality, drainage, seismic stability, water resources, etc. In contrast, recently urbanized districts are generally sprawled within poorly drained lowland with low quality foundation materials.

Traditionally, most government administrators, technocrats, planers, engineers and project owners/initiators do not consider the expertise/contribution of the engineering geologist important at the planning stage of project conceptualisation. Generally, the engineering - environmental geologist is only consulted whenever apparently unsolvable problems that are perceived to be related to geology are encountered either during project construction, operation, or maintenance (Freitas, 2009, Akpokodje 2017). Unfortunately, despite the acknowledged importance of engineering and environmental geology input in sustainability of civil engineering and environmental projects, engineering and environmental geology profession is still perceived as "service agent" to be used on ad-hoc basis rather than a fundamental component of major infrastructure projects.

Environmental Geological Maps

There are several definitions of environmental geological maps (*Ferguson, 1974, Kuroda, 1986, Lee et al 1988, etc).* Despite the different definitions, there is general consensus that environmental geological maps are maps derived from compilation and interpretation of earth science data obtained from conventional geological and related maps to form a viable input for decision-making at the land use planning stage. In addition, they also assist in solving development related environmental problems and impacts, especially of urban areas.

Emergence of Environmental Geological Maps

The danger to the earth and its environment as a result of increased industrialization, and excessive exploitation of resources attained global priority attention toward the end of the 20th century, with emphasis on the need for environmental protection and restoration. The issue resulted in two major world conferences, the World Conference on Environment and Development in 1978 and the Earth Summit in 1992.

Environmental geology is a relatively new subdiscipline and its emergence is closely related to that of Engineering geology. Environmental geology attained formal recognition as a separate specialty, different from engineering geology in the late 1960s and early 1970s. The purpose of engineering geology is to provide basic earth science information for the planning of land use and for the planning, design, construction and maintenance of civil and mining engineering works *(UNESCO, 197; Earth Series 15, Engineering Geological Maps,)*. Environmental geology has been defined as the application of geological principles to the problems created by the occupancy and exploitation of the physical environment by humankind (Legget 1974; Bell 1986, Bates & Jackson, 1987; Montgomery, C. 1994). It also involves the impact of the geological environment on humans (e.g. volcanic eruptions, earthquakes, landslides, floods, etc). The initial application of environmental geology in the late 1960s was in land-use planning and resource management, as demonstrated by the title of the first textbook on the subject, by Peter Flawn, *Environmental Geology: Conservation, Land-Use Planning and Resource Management (1970).* Environmental geology is of fundamental importance in the planning and development of the environment. It should be stressed that unlike engineering geology, the emphasis in environmental geology is on the environmental impacts of human activities and the impacts of environmental processes on humankind as well as land use planning without necessarily focusing on the design, construction and maintenance of civil engineering works.

Earth Science factors required for sustainable land use planning process are not available in conventional geological maps and are also difficult for non-geologists (planners and administrators) to understand. To meet these needs, Environmental Geological Maps (EGM) evolved and they essentially display summary of information derived from factual "basic data" maps, interpretative maps or derived maps (applied geomorphology, sedimentology, structural geology, natural hazard, geodynamic processes, etc). EGMs can depict both the resources for development, such as areas for good foundation conditions, groundwater supplies, mineral resources, construction materials as well as constraints to development such as seismicity, subsidence, landslides, flooding, waste disposal, etc. EGMs also assist to ensure that the geological information are readily understood and incorporated in the planning process and that the planned land use does not destroy valuable subsurface mineral resources, does not cause environmental contamination, and the planned usage is the most suitable for the type and quality of the land available.

Emergence in North America: The rapid growth of the cities of North America started after the second world war and by the 1960s and 1970s, the growth of cities had become phenomenal which contributed to further development of environmental geology and the emergence of Urban geology. Urban geology provides information on urban geologic environments as a scientific basis for planners and engineers for rational land use planning and urban development. Such information can be obtained by mapping the main restricting factors for urban development: such as lithology; topography; slope; seismotectonic; water table depth; flooding susceptibility and seismic-induced effects.

"Environmental Geology Mapping" was the topic of an engineering geology symposium at the 1972 annual meeting of The Geological Society of America (Legget 1974; Bell 1986). Natural resources planning and the roles and interrelations of geology and planning were discussed. Participants agreed that the conversion of geologic data should result in a viable input for decision-making, and must also receive public endorsement and understanding by people who may not be knowledgeable in geology.

In 1976, the International Association of Engineering Geology published a landmark *book "Engineering Geological Maps" - A guide to their preparation* (Engineering Geological. 1976, The UNESCO Press). The text defines Engineering Geological Map as a *"type of geological map which provides a generalized representation of all those components of a geological environment of significance in land use planning, and the design, construction and maintenance as applied to civil and mining engineering".* Furthermore, *"Engineering Geological maps should be based mainly on geological, hydrogeological and geomorphological maps, but must present and evaluate the basic facts provided by these maps in terms of engineering geology".* These conventional maps are the same factual and conventional *"basic data"* maps used in the preparation of Environmental Geological Maps.

Emergence in United Kingdom: Increasing pressure for infrastructural development in Great Britain in the 1970s and 1980s heightened the conflict between the different land use demands placed on an area. Options were often constrained by the need to safeguard the environment, poor ground conditions, economics, existing development and the demands for urban renewal. Within this framework, geology, geotechnics, geomorphology and related subjects could provide essential information on constraints to development such as land instability and poor foundation conditions as well as resources for development such as mineral and water resources, and high-grade agricultural land (Lee, et al, 1988). However, one of the main problems facing land use planning is that few planners have a geological background and few geologists have a planning background, and hence there is often a communication gap between the two groups. In this context, environmental geological mapping provides an important means of presenting the detailed, often very technical, geological and geomorphological information in a simplified way that can be easily understood by the planning community.

Interest in environmental geological maps (EGMs) has increased in Britain since the publication, in 1982, of the IGS Report 82/15 Environmental Geology of the Glenrothes District, Fife Region. In 1986, the British Department of the Environment (DOE), together with the Welsh Office and the Scottish Development Department, commissioned several environmental geology maps (EGM's) to identify the best form of presentation of the results of geological studies in order to overcome the communication problems between geology and planning (e.g. Nickless 1982; Brown et al. 1986; Forster et al. 1987, Freitas, 2009, Tan & Ran, 1986, Hays & Shearer, 1981; Culshaw et al, 1987,). The objectives were to:

a) produce new geological base maps of the study areas from the available geology maps;
b) provide a compilation and interpretation of geological, geotechnical and earth science data as a basis for cost-effective planning for development and urban renewal.

Doornkamp, et al (1987) presented an international review of Environmental geological maps (EGMs). They observed distinct, yet different, characteristics in the style and purpose of EGMs in both the USA and Europe. In the USA, EGM is consistently concerned with the provision of earth science information to planners, engineers and politicians responsible for development, urban growth and redevelopment. The European approach to EGMs is different because it tends to appear under the title 'engineering geological maps'. Variations in style and purpose exist across Europe, which to large extend, reflects differences in regulations or legislation regarding planning and development.

Types and Scale of Environmental geological maps
The content of environmental geological maps and amount of detail information shown are determined by the purpose and scale of the map as shown in Table 4. Generally, the larger the scale the more detail information that can be shown.

Table 4: Map scales

Purpose	Content	Scale
Special purpose	Analytical	Small
		1:100,000 and less
Multipurpose	Comprehensive	Medium
		Less than 1:10,000 and greater than 1:100,000
		Larger *1:10,000 and greater*

Data Acquisition and Production of Environmental Geological Maps

The acquisition of data and production of environmental geological maps have much in common with geological mapping because the general purpose of the two types of mapping is to present information about the geological environment. From the perspective of the land use planner, the major short-comings of the conventional geological maps is that rocks of significantly different engineering and environmentally geological properties may be grouped together because they are of the same age, origin and mineralogical composition. However, the scope of environmental geological mapping is wider because in addition to lithostratigraphic and structural information, other components of the environment must be considered (*Ferguson, 1974; Kuroda 1986; Bell 1986; Lee et al 1988, etc)*. These mainly include:

a) description and quantification of the significant physical and engineering properties of the rocks, soils and sediments; their thicknesses, and areal extent, foundation conditions/quality;
b) hydrology, drainage and groundwater resources (depth to the groundwater and chemical properties);
c) topography and geomorphological conditions;
d) geodynamic processes and natural hazards including seismicity, subsidence, landslides, flooding, etc;
e) constraints to development (such as land instability and poor foundation conditions);
f) resources for development (such as mineral and

water resources, and high grade agricultural land).

g) environmental impacts of human activities and impacts of natural hazards on human'

Methodology involves both fieldwork and the collation of existing records and published material. The list of existing records and published includes, but not limited to:

- bedrock geology
- superficial geology
- geomorphology
- slope steepness
- soils
- geotechnical conditions
- sites of geotechnical investigations
- sites of mineral workings
- land use planning provisions
- ground characteristics for planning and development

The report and maps should be treated as simplified, common every-day reference material, (devoid of highly technical terms) that can be easily understood by all those concerned with land development, planning and environmental management. The narrative should take the form of an extended explanation of the legends found on the maps. The information contained in the maps and report provided should be such that they form adequate framework within which planning and development decisions may be taken. They should provide clear guidelines as to the potential limitations and constraints that are likely to be encountered at particular locations. It may be useful to add a *"summary map"* which draws selected aspects of the physical environment from each of the other maps, highlighting those ground conditions which are likely to pose significant problems for the planner and developer. Other methods of using and portraying geological information for planning purposes are in terms of terrain evaluation and geographical information systems.

It is important to recognise that the maps and accompanying reports provide only a general indication of ground conditions and must not be relied upon as a source of detailed information about specific sites/areas and should not be substitutes for site specific investigations. Users of any environmental geological maps and report must satisfy themselves, by seeking appropriate technical advice and carrying out ground surveys and site investigations if necessary, to confirm that ground conditions are suitable for any particular land use or development. Environmental geological maps are primarily for the use of planners, but can also provide useful sources of information for civil engineers, developers, and mineral extraction companies.

A common criticism of Environmental Geology Mapping (EGM) studies is that the final maps are too generalised and could be misinterpreted by non-specialist users who may assume that where no limitations are depicted, a development can proceed without further investigation. However, it should be emphasised that the purpose of such studies is to make planners and developers aware of relevant ground conditions so that they can seek professional advice where necessary. EGMs should be used only as preliminary guideline materials. Generally, the heighten the awareness of the physical conditions prevailing within an area and should, if used correctly, enable further detailed investigations to focus on certain factors which may otherwise have been overlooked.

Examples of Environmental Geological Maps

Kathmandu Valley and Kathmandu city

The environmental geological maps of Kathmandu Valley and Kathmandu city are presented in Figures 2 and 3 (Duvadi et al, 1998; Omkar et al, 1999). The rural development in the Kathmandu Valley, has taken a phenomenal rate in the past two decades. The major geo-environmental problems in the Kathmandu Valley and urban are due to unplanned urbanisation, haphazard exploitation of geologic resources, local landslide zones, severe problems of garbage disposal, river flooding and a severe river pollution. Widely uncontrolled urbanisation with unsolved garbage and sewer problems, the degradation of fertile land, and haphazard exploitation of all kinds of geological resources has led to partly irreversible damages to the environment. The maps are meant to draw the attention of the planners and decision makers of Kathmandu to the natural hazards, to man-made environmental damage, and to the limitation of important natural resources.

The map contains; all basic geological and environmental data, geotechnical risk zones (landslide-prone areas or those of poor foundation conditions), areas for preferable extraction of construction material and those that should not exploited, areas of immediate need of reforestation in order to prevent landslide or bad land development, groundwater protection zones, and suitable garbage disposal sites. The cartographic concept was to present a map at a semi-regional scale of 1:50,000 with all geological formations and lithological units of the Kathmandu Valley as a basis for all other environmental and geotechnical information.

A detailed and self-explaining legend of the map was presented in three columns. Two columns provide information on the delineated geological formations and their lithological properties. A third column contains the individual geotechnical properties of the delineated areas combined with the corresponding recommendations. The environmental aspects regarding controlled land use (e.g. conservation of forest to control landslides and soil erosion) and the possible exploitation of resources are also described in the third column. Environmental hazards and risks are highlighted by special signs (hatching), and symbols for areas of low bearing capacity, landslides, and flood plains. The excessive river pollution is shown by different colours from green to magenta. Earthquake sensitive areas are shown in an inset map. Since the map is prepared at a reconnaissance scale of 1:50,000 with a limited data base, it is self-evident that it cannot be used as the exclusive basis of investigation for individual buildings and infrastructures.

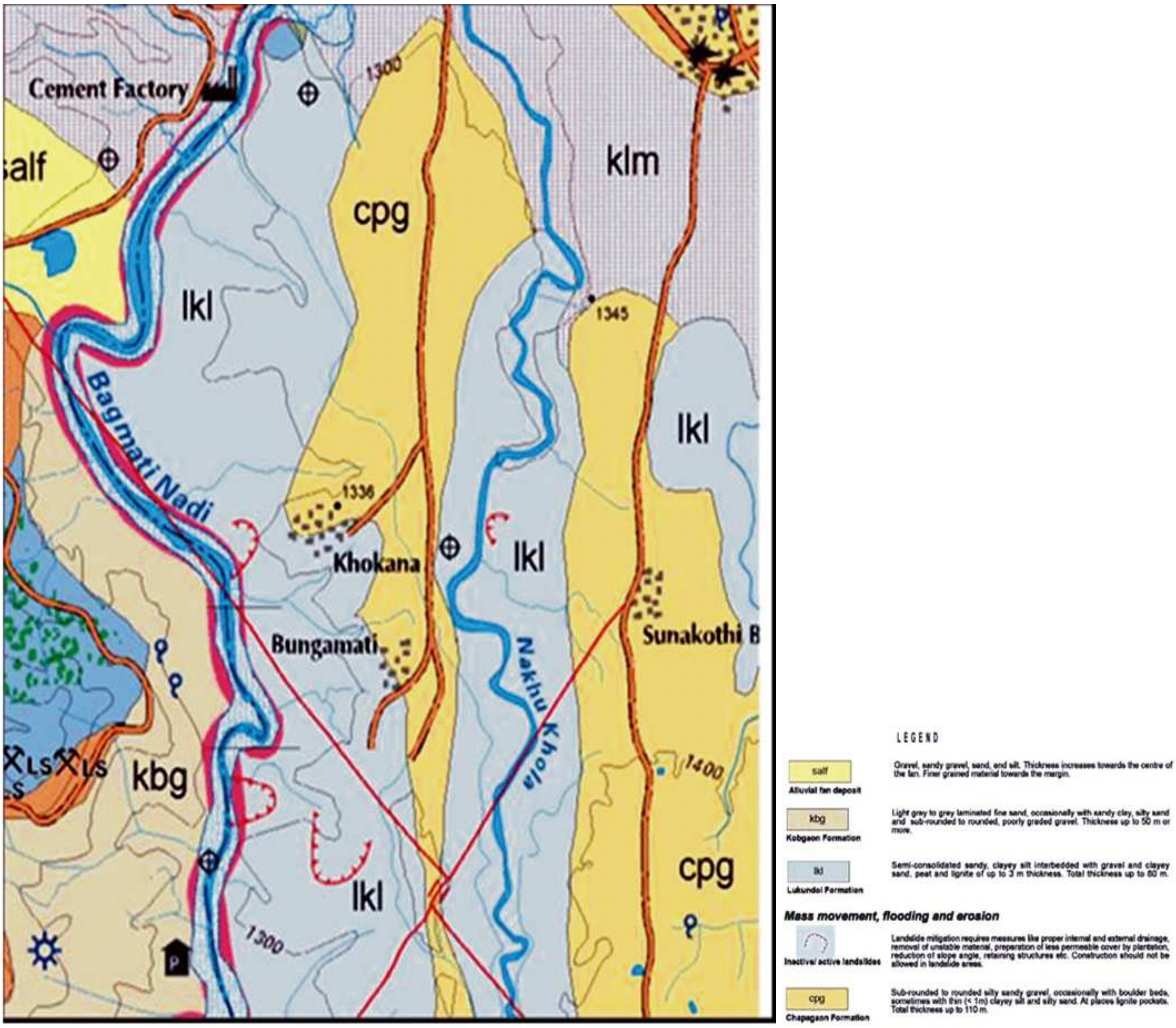

Fig. 2: Engineering and Environmental Geological map of Kathmandu Valley, 1: 50.000 (Source: Omkar, et al 1999)

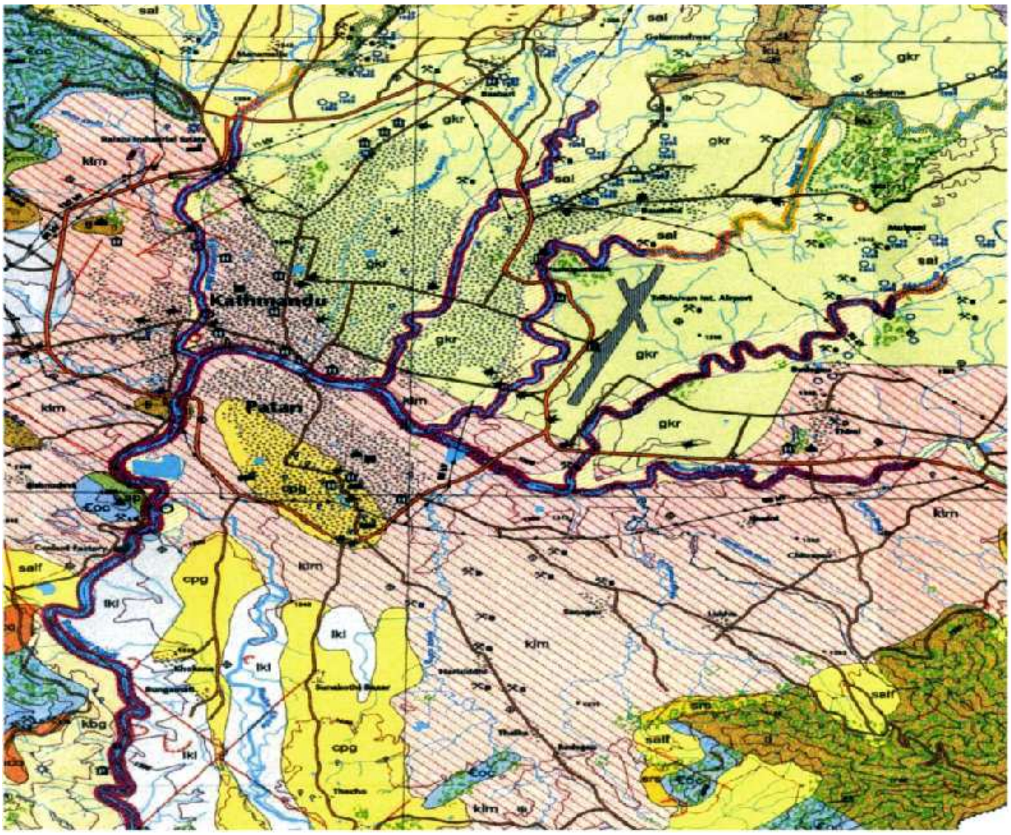

Fig. 3: Engineering and Environmental Geological map of Kathmandu; 1: 50.000 *(Source: Omkar, et al 1999).*

TYPICAL EXPANDED LEGEND TO FIGURE 3		
FORMATION Quaternary	**Unconsolidated Material/Sediment**	**Engineering/Hydrogeological Properties and Recommendations**
SAL Recent alluvial soil	Recent sediments of flood plains and lower alluvial terraces. In northern sand and gravel deposits up to boulder size. In central and southern part sand, and line gravel	Low bearing capacity, loose density and soft consistency. Prone to subsidence, erosion and flooding. High of groundwater with periodic change of shallow groundwater table. High infiltration. High risk to pollution of groundwater and surface water. Highly susceptible to liquefaction. *Recommendation: Construction of building not advisable. Send mining on very limited scale but no mining closer than 500m from bridges. No waste disposal, and storage of chemicals and other hazardous materials.*
Sre Residual soil	Humic slightly loam to sandy gravel of thickness 1 – 3m at places and occurs on slopes.	Low to moderate bearing capacity, loose to slightly dense and soft to firm. Erosion and mass movement on slopes. High infiltration and potential for groundwater. Low to very low susceptibility to liquefaction. Recommendations: Appropriate land for forest and ground water recharge. Buildings on slopes need stabilization measures and proper drainage
Sco Colluvial soil	Inhomogeneous deposit at foot slopes with constituents of humic clay, slit sand, at places boulders. Variable thickness > 1m, increasing towards	Low to moderate bearing capacity, loose to slightly dense and soft to firm. Reactivation of mass movement possible under improper drainage, especially in case of a seismic impact. High infiltration and low potential for groundwater. Low to very low liquefaction susceptibility. *Recommendation:* Heavy construction need pile foundation of anchors to solid rock. Suitable for land utilization for forest and groundwater

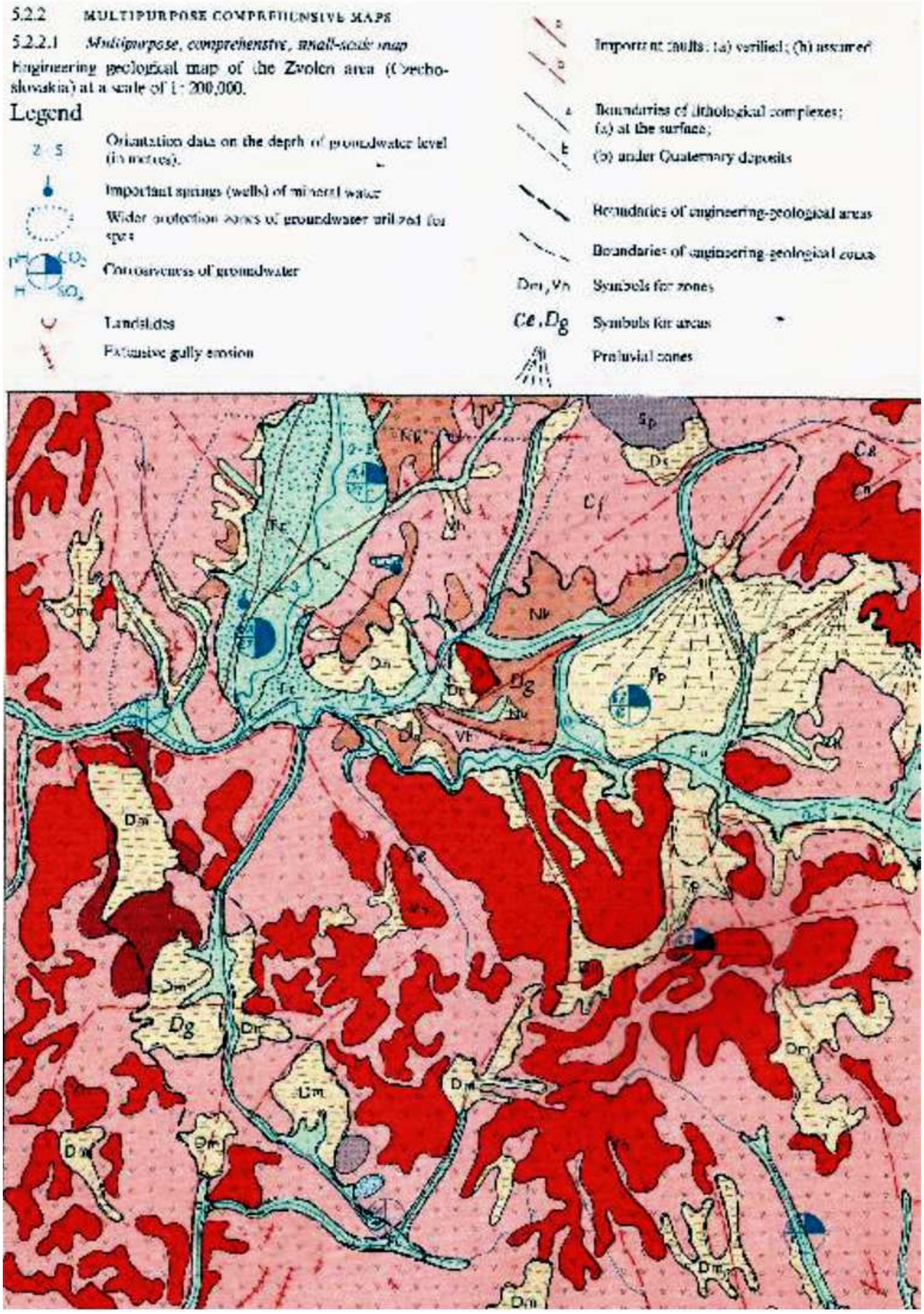

Fig. 4: Engineering-Environmental Geological map of Zvolem Czechslovakia) 1:200,000; small scale multipurpose map *(Source: Engineering Geological Maps (UNESCO, 1976)*

Figure 4 is an Engineering-Environmental Geological map of Zvolem (Czechslovakia).

The map has an expanded legend made up of the following four columns:

I. Genetic Lithologic Classification
II. Geological. Geomorphology, Hydrogeological, hydrodynamic conditions (Engineering -Environmental Classification)
III. Engineering-Geological Characteristics
IV. Engineering, Environmental & Geological Conditions for Construction works

Column I

I. GENETIC-LITHOLOGICAL CLASSIFICATION				
Age	Lithological Suites	Symbols	Patterns	Lithological complexes
Quaternary	Surficial deposits	$_{\delta}Q_{3-4}$		Slope wash sediments
		$_{f}Q_{3-4}$		Fluvial sediments on the (a) flood plain (b) terraces
		$_{pr}Q_{2-3}$		Sediments in proluvial cones
		$^{t}N_{2}-Q$		Chemical sediments

Column II

II. ENGINEERING CLASSIFICATION
A_1 = solid rock, A_2 = semisolid rocks, B = gravelly soils, C = sandy soils, D = cohesive soils, E = unsuitable soils (classification according to CSN Building standard)
D = loams to stony loams (B = stony debris, talus)
B = sandy and loamy gravels, C = loamy sands, D = sandy loams (E = muddy soils)
B = loamy gravels, D = sandy and stony loams

Column III

III. ENGINEERING – GEOLOGICAL CHARACTERISTICS							
For rock and soil material (in laboratory)							
Symbols	Main lithological types	Dry bulk density g.cm^{-3}	Porosity %	Uniaxial compressive strength Kgf. Cm^{-2}.10^{2}	Indentation hardness (Scejner) Kgfmm^{-2}	Modulus of deformation E_0 Kgf.com^{-2}10^{5}	Durability
$_{\delta}Q_{3-4}$	Sandy to clayey loam	1.53-1.79	35-58	0.0055	-	-	Soften, swell, not resistant
$_{f}Q_{3-4}$	Sandy loam	1.50-1.65	35-45	-	-	-	Not frost resistant
	Sandy gravels						
	Sandy gravels	1.80-1.90		-	-	-	

For rock and soil	Ease of excavation
2-4	Thickness 1-9m, fragmentary debris at the base, very low permeability, without permanent groundwater table, much gully erosion and land sliding used for rick materials, impervious materials for dame.
2-3	(a) Thickness up to 3m, permeable with permanent continuous groundwater table
3-4	(b) Thickness up to 11m, slightly permeable (10^{-4} to 10^{-3} m/s), without permanent water table; suitable as fill material
4	Conical forms, coarse – alternating with fine – grain material

Column III

Areas	**Zones**			
	Type	*Geological conditions of the ground*	*Hydrogeological Conditions*	*Present geodynamic processes*
Ce Areas of the volcanic high – lands Young and variegated mountainous landscape, formed by the destruction of original stratovolcanic forms due to differential neotectonic movements and erosion	Dm Zone of slope-wash deposits on magmatic and metamorphic rocks	Loamy slope wash soils 2-7m thick. In the substratum there are solid and semi-solid, slightly compressible volcanic and pyroclastic rocks	Very slightly water-bearing to practically dry	Intense slope erosion in erosive furrows and gullies

Column IV

ENGINEERING-GEOLOGICAL CONDITIONS FOR CONSTRUCTION WORK				
Excavations and cuttings	*Fill construction*	*Structural foundations*	*Roads*	*Building materials*
Cutting and side-slope cuts will mainly be excavated in slope wash deposits and the under-deposits and the under-lying weathered rocks; ground water issuing from the slope is easily collected and drained from the site.	The construction of larger fills may lead to slope failure. Use of a combination of half fill-half cut would be advantageous	Foundation conditions are very good. The only serious problem is the question of ensuring slope stability during construction	When determining the line of Brick clays roads and railways, difficulties arise because of the effect of the gullies, and the need for good drainage to maintain stability	Brick clays

Urbanisation in Port Harcourt and Increasing Flooding Problem

The city of Port Harcourt and the new satellite urban areas are among the fastest growing and congested urban areas in the South-South geopolitical region of Nigeria with an estimated current population of about 1.8 million compared to its small population of about 235,000 in 1963 (NPC, 2006). The records of the Rivers State Ministry of Physical Planning and Urban Development (Fig. 10) revealed that the percentage of built-up areas grew from about 16% in 1984 to 51% in 2014 (Ayotamuno & Enu-Obari, 2017, Akpokodje & Giadom, 2017). This gave rise to several new sub-urban areas including Diobu GRA Phase 111, Choba, Nkpolu, Amatagwolo, Mgbuoba, Rumuepirikom, Mgbuosimiri, Rumukwurushi, Ada George Road, Rumuogholu, Eliozu, Rumuodara, Rukpuoku, Eneka, Rukpuokwu, Alakahia, Rumuokeni, Choba, Ozuoba, Ogbogoro, Mgbuoba, Rumuigbo, etc.

Port Harcourt has a long history of increasing devastating flood problems (Figure 5), that can be attributed to the following major contributing factors:

- process of urbanisation which leads to greater impervious areas and greater runoff;
- poor land use planning, non-adherence to building code (unplanned urbanisation);
- encroachment on natural water channels floodplains, and swamps;
- inadequate stormwater drainage systems and blockage of existing drains;
- Absence of Integrated Flood Management approach and Drainage Control Master Plan with input from an Environmental Geological map

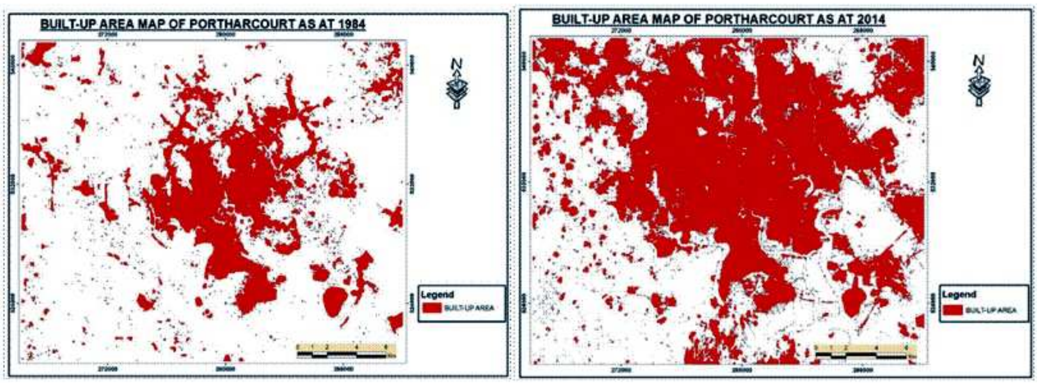

Fig. 5: Built-up Areas in Port Harcourt (1984 & 2014) *(Source: Rivers State Ministry of Physical Planning and Urban Development)*

Some are subject to severe Flooding in Port Harcourt

Nkpolu Road Junction and Secondary School in Port Harcourt

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

Klifco Company Mgbuoba

This is part of an extensive low-lying stretch of seasonally flooded natural freshwater swamp that does not drain into any major free-flowing creek. The Rumuokuta-NTA road cuts across the swamp and the dualisation of the road has severely restricted free flow of flood water. Recent major housing developments and reclamation of the swamp (e.g., Redeemed Christian Church, Petrol station, Shopping Mall and other on-going developments) exacerbated the flooding problem with flood water depth and extent reaching unprecedented levels in 2017.

PLATE 1: Flooded Nkpolu Community Secondary School (2006)

PLATE 2: Flooded Nkpolu Road (August 10, 2018)

PLATE 3: Flooded UNIPORT bound lane of EAST-WEST HIGHWAY at Nkpolu Road Junction

PLATE 4: Flooded Klifco company in August 2017

PLATE 5: Street by Klifco

Conclusion

The exponential growth in world population and urbanisation will continue to lead to the expansion of urban centres into ecologically sensitive and hazard prone areas (flooding, landslides, subsidence, seismic activity, etc). In order to reduce the potential adverse environmental impact of uncoordinated urban expansion, adequate input of earth science data and information must be provided during the land use planning process which is generally carried out by planners and administrators who may not be knowledgeable in geology. The required geological information/data can be provided in Environmental Geological Maps in a format that is readily understood by non-geologists. EGMs also assist in solving development related environmental problems and impacts in expanding urban areas. To achieve the maximum benefits of EGMs, functional communication/cooperation between geologists and other professionals in land use planning and development must be significantly improved.

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A LEAD PAPER (2019)

"Geo- Environmental Mapping for sustainable Development of Urban Areas"

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Abstract

In the past few decades, rapid urbanization and industrialization have altered the priority of environmental protection and restoration of air, soil, and water quality many times. Yet it is recognized that the sustainable management of human society is necessary at all phases of impact from the interactions among energy, environment, ecology, public health, and socioeconomic paradigms. The multidisciplinary nature of this concern for sustainability is truly a challenging task that requires employing a systems analysis approach. Such a systems analysis approach links several disciplinary areas with each other to promote the concept of sustainable management. Just as a sophisticated piece of music involves many different instruments played in unison, systems analysis requires a holistic viewpoint and a plethora of tools in sensing, monitoring, and modeling that have to be woven together to explore the state and function of air, water, and land resources at all levels. Geo-environmental mapping involves integration of such disciplines to produce a result that promotes sustainable development. Rapid advances in the integration of remote sensing (RS), global positioning system (GPS) and geographical information system (GIS) technologies motivate more integrative sensing, monitoring and modeling with system thinking for sound decision making. Such understanding leads to the proper integration of sensing, monitoring and modeling technologies in order to aid in the decision making involved in the preservation or remediation of the environment.

***Keywords**: Environmental geological Maps; Urban Centres, Population Growth, Land Use, Environment.*

Introduction

With the aid of systems analysis, this comprehensive collection include a variety of research work that results from years of experience and that reflects the contemporary advances of remote sensing technologies. The holistic approach requires expertise in acquisition, storage and warehousing, quality assurance, and presentation of environmental data from which information can be retrieved and knowledge can be developed to decision making.

To fulfil such a synergistic integration, it requires the following: collecting and maintaining environmental data, analyzing environmental data; using data for environmental protection actions; engaging the community to promote policies an improve the sustainable management with environmental information; evaluating the effectiveness of environmental management process, programs an efforts with environmental knowledge and implementing total quality management through integrated environmental sensing, monitoring , modeling and decision making. To achieve sound environmental resources management, there is a need to investigate pollutant storage, transport and transformation in both natural systems and built environment. New soil data and information are required to address emerging concerns on soil contamination, soil biota and their diversity, soil stability (landslides), soil hydraulic functions soil carbon pools, soil erosion, and salinization.

Digital Soil Mapping

Digital soil mapping is a young area of research in soil science. Knowledge of soil science is required for sustainable environmental management. There is ever growing interest in transferring the scientific achievements of the past years of digital soil mapping into operational data and information systems responding to the increasing demands of high quality soil data and information. In the past soil data collection was largely driven by mono-functional view of soil as the basis for agricultural production. Under the leadership of the Food and Agriculture Organization (FAO) of the United Nations, substantial progress has been made in collecting soils data and information in all the continents, particularly in developing countries. Standardized systems for soil classification and soil profile description have facilitated the inter-operationality of information systems across national borders paving way for the creation of digital soil databases at global and continental scales based on advanced GIS technologies.

There is a growing need for spatially continuous and quantitative soil information for environmental modeling and management, especially at the national scale. Study by Stephen I.C Akpa et al (2014) was carried out to predict the soil particle-size fractions (PSF) for Nigeria using random forest model (RFM).

Equal-area quadratic splines were fitted to Nigerian legacy soil profile data to estimate PSFs at six standard soil depths (0–5, 5–15, 15–30, 30–60, 60–100, and 100–200 cm) using the Global Soil Map project specification. They applied an additive log-ratio (ALR) transformation of the PSFs. Overall, the PSFs show marked variations across the entire Nigeria region with a higher sand content compared with silt and clay contents and increasing clay content with soil depth. The variation in soil texture (ST) shows a progressive transition from a coarse texture (sand) along the fringes of northern Nigeria (e.g., upper part of Maiduguri and Sokoto), to finer texture (loam to clay loam) toward the western part of the Niger Delta region in the south. The inclusion of depth as a predictor variable significantly improved the prediction accuracy of RFM especially at lower depth intervals. These results could be used for producing soil function maps for national agricultural planning and in assessments of environmental sustainability.

Hydrological Application

Since 1972 (launch of the first Earth Resources Technology Satellite, ERTS-1), scientist have used remotely-sensed data from different sensors to characterize, map, analyze and model the state of the land surface and surface processes. With the help of new algorithms, new hydrological information were extracted from remotely-sensed data and used in hydrological and environmental modeling. These new information and hydrological parameters have increased our understanding of the different hydrological processes by helping in quantifying the rate and amount of water and energy fluxes in the environment. The ability of these sensors in providing various spatiotemporal scales data has also increased our capability in looking into one of the challenges of environmental modeling, mismatch between scales of environmental process and available data.

The role of remote sensing in understanding hydrological processes and fluxes across different spatial and temporal scales can be tremendous, if appropriate spatial and temporal resolution remotely-sensed data are available under ranges of bands. With the availability of large volumes of remotely-sensed data, geographical information system (GIS) tools to manipulate, process, store and retrieve such data and efficient computing system, the application of remote sensing to water resources has been increasing in recent years.

The application of remote sensing in water resources research and management mainly lies in one of the three categories: mapping of watersheds and features, indirect hydrological parameter estimation and direct estimation of hydrological variables

Mapping of watersheds and hydrologic features

Different sensors aboard airplane or satellites have been used extensively in providing imaging, photographing and mapping information for different purposes. Mapping of wetlands, floodplains, disaster areas, coastal shores, river banks, snow pack, fire damage, drainage basins and others that show the areal extent of a given land feature distinct from others due to the difference in the spectral signature fall under this category. Different studies have used aerial photos, satellite images, lidar and radar data to map and visualize land surfaces for planning, resource mapping, hazard assessment and emergency operations.

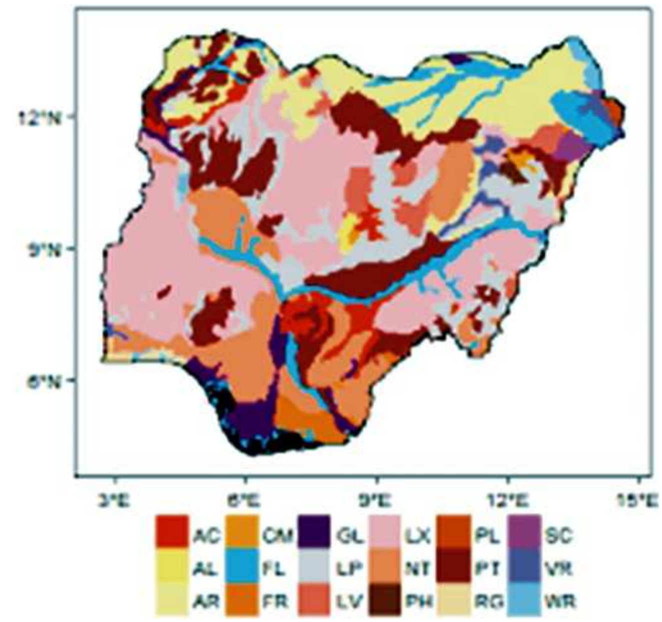


Fig. 1: Hydrological Map of Nigeria.

Indirect estimation of Hydrological Parameters

The majority of remote sensing contribution in water resources management falls under the indirect hydrological parameter estimation category. The most common area of contribution in this category is the use of classification algorithms to generate land cover classes. Using the unique spectral signature of land surfaces, mainly in the visible, infrared and thermal spectra, land cover classes are estimated from such data.

Impervious Surface Cover

Surfaces that impede the natural infiltration of water and enhance surface runoff are classified as impervious surfaces. Associated with urbanization and construction of pavements, roads and buildings, impervious surfaces play an important role in surface runoff and the transport of contaminants. Remote sensing has been used as an effective technique to map impervious surfaces using spectral characteristics of surfaces (Melesse 2004b, Melesse and Wang, 2007). Ridd (1995) and Owen et al. (1998), showed that the relation between the fractional vegetation cover (FVC) and fractional impervious surface area (FIS) for developed areas as:

$$FIS = 1 - FVC \qquad (1)$$

In surface runoff estimation, impervious surfaces are classified as hydraulically connected and those that are not. The hydraulically connected surfaces such as parking lots and roads are connected to the drainage system where runoff from such surfaces leads to the drainage network. Those surfaces such as roof-tops are classified as hydraulically not connected. Rain water from roof tops can fall into the pervious surface area such as grass hence termed as disconnected. The resulting storm runoff from such surfaces can be lower than the hydraulically connected impervious surface areas.

Micro-Topography and Latent Heat Flux

Topography plays an important role in the distribution and flux of water and energy within the natural landscape. Surface runoff, evaporation and infiltration are hydrologic processes that take place at the ground-atmosphere interface. Quantitative assessment of these processes depends on topographic configuration of the landscape, which is one of several controlling boundary conditions.

Wetness index (WI) provides a description of the spatial distribution of the soil moisture using topographic information. WI is computed as,

$$WI = Ln(AS) \qquad (2)$$

where A and S are the specific drainage area (flow accumulation) and slope, respectively.

As specific drainage area increases and gradient decreases, *WI* and soil moisture content increase. Wetness index takes into account both a local slope geometry and site location in the landscape, combining data on gradient and specific drainage area. This can lead to higher correlations of soil moisture with *WI*, hence evapotranspiration, than with specific drainage area and gradient.

Wetness index controls flow accumulation, soil moisture, distribution of saturation zones, depth of water table, evapotranspiration, thickness of soil horizons, organic matter, pH, silt and sand content, plant cover distribution (Kulagina et al., 1995; Florinsky, 2000).

A study to understand the relationship between latent heat flux and microtopography (1m resolution) of a wheat field was studied using remote sensing-based latent heat flux. The latent heat flux estimation technique is based on the surface energy balance approach using remotely-sensed data (Bastiaanssen, 1998). The detailed procedure on estimating grid-based latent heat flux for the study area is shown in xMelesse and Nangia (2005) and Oberg and Melesse (2006). The relationship between WI and latent heat flux for a wheat field is strong, since the topography influences the flux and distribution of water. Grids with higher values of WI are areas receiving most of the flow (higher flow accumulation) and lower gradient. These areas have higher soil moisture, hence higher rate of evaporation, than areas with lower values of WI. It is also indicated that when water is a limiting factor, plants on higher WI areas grow well with good canopy cover than plants in other zones of the field. This increases the transpiration (vegetation latent heat) of the crops. The latent heat seems to increase at higher rate at lower values of WI than at higher values of WI. This is attributed to the limited available water for evaporation proportional to WI.

Soil Water and Drought Monitoring for Early-Warning Applications

With the advent of grid-based remotely-sensed rainfall data, the application of crop water balance models for crop monitoring and yield forecasting has gained increased acceptance by various international, national and local organizations around the world. Soil water is a key state variable in hydrological modeling and determines the partitioning of rainfall into runoff and deep percolation, and also controls the rate of evapotranspiration (ET).

Although the estimation of actual evapotranspiration (ETa) is the ultimate goal of many researchers for hydrological and agronomical applications, it is often difficult to quantify and requires expensive instrumentation. However, hydrological modeling techniques are used to estimate ETa. The two basic modeling techniques to estimate ETa are based on either energy balance (e.g. Bastiaanssen et al, 1998; Allen et al, 2005; Senay et al, 2007a) principles or water balance-based algorithms (e.g. Allen et al, 1998; Senay and Verdin 2003).

For monitoring large areas using remotely sensed data, the water balance approach provides an operational advantage in terms of data availability. While the energy balance models are mainly driven by the thermal data, the water balance models are driven by rainfall. Naturally, cloud cover is an issue to provide daily estimates of ETa on rain-fed agriculture from the energy balance models. On the other hand, availability of satellite-derived rainfall data at various temporal and spatial scales makes operational estimation of ET_a using a water balance model a relatively easy task for various decision makers in agriculture and natural resources.

The most widely used water balance technique for operational use is the FAO water balance algorithm that produces the crop water requirement satisfaction index (WRSI), which is also known as the crop specific drought index (CSDI). The WRSI shows the relative relationship (ratio/percent) between the supply (from rainfall and existing soil moisture) and demand (crop demand to meet its physiological needs) using observed data from the beginning of the crop season (planting) until current date. A value of 100 indicates all the crop demand has been met while values less than 50 generally indicates a severe water shortage that could lead to complete failure of the crop (Smith 1992). Values between 50 and 100 will indicate different degrees of crop stress and yield reductions from shortage of adequate supply. FAO studies (Doorenbos and Pruitt 1977) have shown that WRSI can be related to crop production using a linear yield-reduction function specific to a crop. Meyer et al. (1993) enhanced the concept of crop water balance modeling using crop-stage specific drought sensitivity coefficients for corn. The Famine Early Warning System Network (FEWS NET) demonstrated a regional implementation of the FAO WRSI in a grid-cell modeling environment (Verdin and Klaver 2002). Furthermore, Senay and Verdin (2003) enhanced the geospatial model by introducing the concept of maximum allowable depletion (MAD) and soil water stress factor from irrigation engineering for better estimation of ETa as a function of soil water content. The current version of the USGS/FEWS NET GeoSpatial WRSI model is operational with daily and 10-day outputs (available here: http://earlywarning.usgs.gov/adds/. The seasonal crop water requirement satisfaction index for a crop is based on the water supply and demand that a crop experiences during a growing season. It is calculated as the ratio of seasonal actual evapotranspiration (ET_a—or supply [=demand met]) to the seasonal crop water requirement (ET_c—or demand):

$$WRSI = ET_a * ET_c^{-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

ET_c is calculated from the product of the Penman-Monteith reference evapotranspiration (ET_o) using the standardized FAO equation that uses short grass as the reference crop (Allen et al. 1998) and crop coefficient (K_c). Crop coefficients (K_c), piecewise linear weighting functions, have traditionally been used to adjust for type and growth stage of the crop:

$$ET_c = Kc * ET_o \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

USGS/EROS has generated an operational daily global ET_o since 2001 using meteorological forcings from the Global Data Assimilation System (GDAS) (Senay et al. 2007b).

ET_a represents the actual (as opposed to the potential) amount of water withdrawn from the soil water reservoir ("bucket"). The key difference between ETc and ETa is that ETa depends on soil moisture that is calculated on a daily basis to provide the daily soil stress correction factor, Ks (Senay and Verdin 2003).

$$ET_a = K_s * Kc * ET_o \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Where K_c is crop coefficient, ET_o is reference ET, and K_s is soil water stress index.

Whenever the soil water content is above the maximum allowable depletion (MAD) level, which varies by vegetation type, the ET_a will balance the ET_c resulting in no net water stress. However, when the available soil water falls below the MAD level, the ET_a will be lower than ET_c, in proportion to the remaining soil water content. Runoff and deep drainage out of the root zone are assumed to occur in excess of field capacity.

The soil water content is obtained through a simple mass balance equation where the level of soil water is monitored in a root-zone soil layer defined by the water holding capacity (WHC) of the soil and the crop root depth, i.e.,

$$SW_i = SW_{i-1} + PPT_i - ET_{ai} - RF_i - DD_{i.} \ldots\ldots\ldots\ldots\ldots\ldots(6)$$

Where SW is soil water content, PPT is precipitation, RF is runoff, DD is deep drainage below the root zone, and i is the temporal index.

Model Inputs

The key input data to the water balance model are precipitation, potential evapotranspiration (PET), and soil water holding capacity, and crop coefficient. While the key crop coefficient values are obtained from the FAO publication (Allen, 1998), the other three main datasets are spatially distributed are described in brief below.

Precipitation

Precipitation is the single most important input of the model. Satellite-derived rainfall estimate (RFE) generated by the National Oceanic and Atmospheric Administration (NOAA) and the National Aeronautic and Space Administration (NASA) are used to drive the water balance model. NOAA produces daily rainfall data at 10 km spatial resolution using a blend of Meteosat's cold cloud duration (CCD) and rain gauge data for Africa (xie and Arkin, 1997). NASA generates daily rainfall data at 25 km resolution from the Tropical Rainfall Mapping Mission (TRMM; satellite systems that covers the globe between 60 degrees latitude north and south of the equator.

Potential Evapotranspiration (PET)

USGS/EROS calculates daily PET values for the globe at 1.0-degree (~ 100 km) spatial resolution from 6-hourly numerical meteorological model output fields of GDAS (Global Data Assimilation System) using the standardized Penman-Monteith equation (Allen et al., 1998). Recent validation of GDAS-PET using PET derived from station parameters in the US has demonstrated the usefulness and reliability of GDAS-based PET for crop water balance studies (Senay et al. 2007a).5.1.3. Soils.

Vegetation Mapping.

Nigeria has extensive mangrove forests in the coastal region of the Niger Delta. Considered one of the most ecologically sensitive regions in the world, the Niger Delta mangrove forest is situated within a deltaic depositional environment. These mangrove forests serve a critical role in regional ecological and landscape composition, and support subsistence gathering practices, and market-based income opportunities. Anthropogenic development threatens the survival of Niger Delta mangrove populations.

Detail and systematic mapping of the dead mangrove areas is important to provide data for the restoration plan and implementation. The restoration process will however suffer setback if the re-oiling from oil spills is not stopped. Figure 2, is a Vegetation map of Nigeria showing the various types of vegetation across the regions.

Mangrove forests are found in 118 countries and territories worldwide, 75% of mangrove vegetation zones are located in intertidal tropic and sub-tropic habitats situated between 25° N and 25° S . Typically surrounding salient river deltas, mangrove regions support a variety of halophytes. These robust shrubs and trees, which have adapted to changing coastal conditions (such as inundation, sun exposure, anaerobic soil, and salinity concentration), play a substantive role in cultivating the biodiversity and wellbeing of the surrounding landscape.

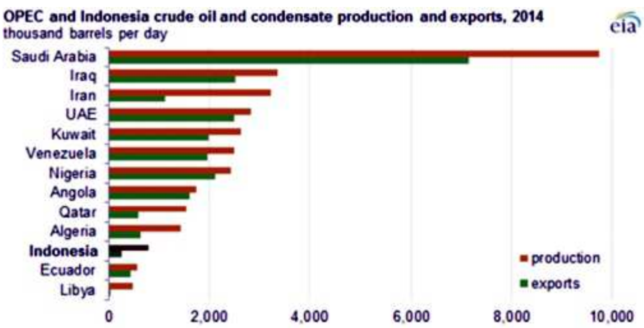
Biologically, six mangrove species make up these forests, including three species in the family Rhizophoraceae (Rhizophora racemosa (red mangrove; tall), Rhizophora harrisonii (red mangrove; dwarf), Rhizophora mangle (red mangrove; dwarf)), and species in the family Avicenniaceae (white mangrove) and Combretaceae. Of these species, Rhizophora racemosa occupies the greatest density of the forest, accounting for approximately 90% of all mangrove biota. Despite expansive geographic coverage, the Niger Delta mangrove forest has approximately 80% of its vegetation distributed in three states (Bayelsa, Delta, and River states). Although the forest is composed of six mangrove species, mangrove growth is primarily situated in brackish muddy creek banks.[3] Studies have indicated that *Rhizophora racemosa* (which is the tallest mangrove species) reaches its optimized growth potential when exposed to brackish water and soft mud, whereas *R. racemosa*'s relatives, *R. mangle* and *R. harrisonii,* favor higher salinity and hard mud. In its natural state, mangrove soil or "chikoko" (a mixture of acid sulphate, silty clay, clay loam and peat), has a pH of 4 and 6 for mangroves inhabiting low-tide and high-tide locations, respectively (Allen, *et al*. 2005). If salinity levels shift too much from these levels, mudflats become unsuitable for mangrove production, and the process of mangrove reforestation (from infertile mangrove land to productive mangrove mudflat) can take upwards of one century (Agbu and James, 1994).

The Niger Delta mangrove forests play critical roles for 60% of local peoples who rely on the land and sea for survival. The forests contribute local therapeutic, amenity, heritage, spiritual, and existence values (Allen, *et al.* 1998). Responses from a survey conducted in 2007 and 2008 evaluating mangrove social value in three Niger Delta communities indicated approximately 85% of participant households had previously utilized mangrove vegetation as a medicinal remedy and between 65% and 71% of villagers recognized the mangrove forests as a place of repose. Further, each study village relied on the local mangrove forest for carrying out cultural traditions (relying on forest as an ancestral burying site or place of festivity such as in Buguma), spiritual significance, and the survival of locally resource-dependent communities (Allen, *et al.* 1998).

Another study evaluating the socio-economic importance of mangrove forests to 950 households residing in Akassa, Bayelsa revealed a strong connection between mangroves and the primary and secondary occupations of individuals living in proximity to the forests. The study noted that while not all residents' vocations directly relied on mangrove resource extraction, all households depended on the forest for resources to fulfill their basic physiological needs (such as fuelwood, mangrove material for medicine, wood to construct shelter, and forest space for hunting sustenance staples like snails, fish and crab) (Anderson, *et al.* 1976). The Niger Delta mangrove forests directly (through raw materials) and indirectly (through forest-based products) offer economically beneficial resources for Nigerian rural and urban communities alike, with eight out of the ten most lucrative vocations dependent on thriving mangrove populations. These occupations include: canoe carving, logging, timber harvesting, building, fishing, sawmilling, traditional medicine, and trading (Anderson, *et al.* 1976).

The ecological importance of mangroves expands well beyond the resources of their roots and wood. Playing a key part in many fish, invertebrate, crustacean, and mollusk species' lifecycles, mangroves offer organisms a breeding ground, (Arnold and Gibbons, 2000) shelter, and sustenance (mangroves are capable of producing 3.65 tons of leaf matter per hectare per year, which is a key contribution to the food web that supports juvenile fish) (Asner and Lobell, 2000). Mangroves offer these services to approximately 75% of the local commercially harvested fish population (Asner and Lobell, 2000); in a properly functioning mangrove ecosystem, this means ~1.08 tons of fish can be supported by one hectare of mangroves each year. Mangroves are also used as shelter and breeding grounds by small mammals, shore birds, reptiles, and insects (Asner and Lobell, 2000). One study analyzing local species dependency on mangrove populations in Pagbilao Bay, Quezon, discovered that nearly 128 fish species, 56 bird species, and 9 species of paneid shrimp relied on mangrove survival for breeding and shelter requirements.

Mangrove roots offer a variety of essential services to local species and native communities



OPEC oil exports by country

Beyond supplying a sundry of basic biological needs to marine and terrestrial organisms, mangrove populations are also essential in preventing erosion (Arnold and Gibbons, 2000) and preserving the surrounding coastal landscape. UNEP research has demonstrated that the mangrove's robust build is efficient at reducing the total destructive capability of storms and wind surges by 70 to 90 percent (Asner and Lobell, 2000). Mangroves also help protect the health and overall biodiversity of surrounding ecosystems by acting as a water filter. Filtration is enabled by the mangrove's capability to absorb and store heavy metals that would otherwise result in the release of metal pollution into nearshore water bodies (Avery, 1992). Mangroves are also efficient sites of carbon storage. Gail Chmura, scientist at McGill University, revealed a mangrove's carbon storing capability is greater than that of a terrestrial forest; per-year, mangroves are capable of storing 42 million tons of carbon/hectare (Abere and Ekeke, 2016).

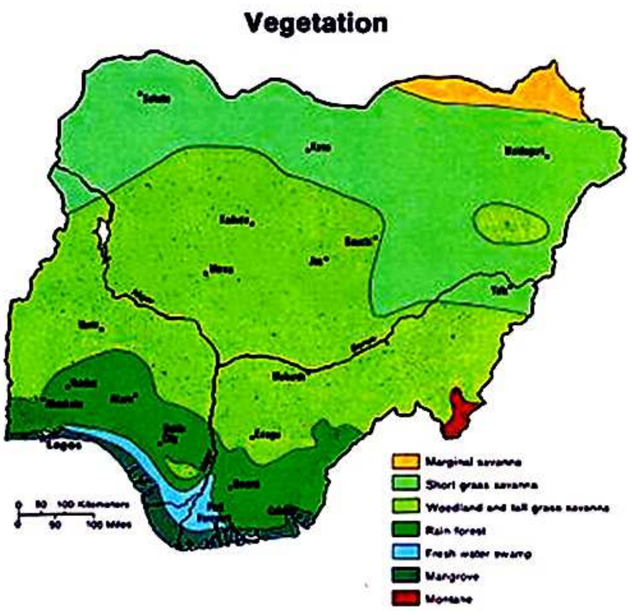


Fig. 2: Vegetation Map of Nigeria.

Conclusions

There is no gainsaying that Geo-environmental mapping is a vital tool for sustainable development of urban area in Nigeria and worldwide.

Systematic Geo-environmental mapping of Nigeria is a project that should be undertaken by professionals under the sponsorship of the Federal Government. Funds could be drawn from Ecological Fund or from relevant Ministries such as Agriculture, Science and Technology, Environment, Water Resources, Petroleum Resources and Niger Delta Affairs. Geo-environmental mappings integrates a plethora of tools in sensing, monitoring, and modeling that have to be woven together to explore the state and function of air, water, and land resources at all levels. To fulfil such a synergistic integration, it requires the following: collecting and maintaining environmental data, analyzing environmental data; using data for environmental protection actions; engaging the community to promote policies an improve the sustainable management with environmental information; evaluating the effectiveness of environmental management process, programs an efforts with environmental knowledge and implementing total quality management through integrated environmental sensing, monitoring , modeling and decision making. To achieve sound environmental resources management, there is a need to investigate pollutant storage, transport and transformation in both natural systems and built environment.

Acronyms and Abbreviations

ASTER	Advanced Space borne Thermal Emission and Reflection Radiometer
ALI	Advanced Land Imager
AVHRR	Advanced Very High Resolution Radiometer
CBERS -2	China-Brazil Earth Resources Satellite
FORMOSAT	Taiwanis Satellite Operated by Taiwanis National Space Organization NSPO. Data Marketed by SPOT
Hyperion	First Spaceborne Hyperspectral Sensor Onboard Earth Observing-1(EO-1)
IKONOS	High-Resolution Satellite Operated by GeoEye
IRS-1C/D-LISS	Indian Remote Sensing Satellite /Linear Imaging Self Scanner
IRS-P6-AWiFS	Indian Remote Sensing Satellite/Advanced Wide Field Sensor
KOMFOSAT	Korean Multipurpose Satellite. Data Marketed by SPOT Image
Landsat-1, 2, 3 MSS	Multi Spectral Scanner
Landsat-4, 5 TM	Thematic Mapper
Landsat-7 ETM+	Enhanced Thematic Mapper Plus
MODIS	Moderate Imaging Spectral Radio Meter
QUICKBIRD	Satellite from DigitalGlobe, a private company in USA
RAPID EYE – A/E	Satellite constellation from Rapideye, a German company
RESOURSESAT	Satellite launched by the India
SPOT	Satellites Pour l'Observation de la Terre or Earth-observing Satellites
SWIR	Short Wave Infrared Sensor
VNIR	Visible Near-Infrared Sensor
WORLDVIEW	

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Geomapping for Sustainable Waste Management

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Introduction

As an Introduction, it is important to identify what constitutes "waste" and the different types of "waste".

Waste occurs when any organism returns substances to the environment. Living things take in raw materials and excrete wastes that are recycled by other living organisms. However, humans produce an additional flow of material residues that would overload the capacity of natural recycling processes, so these wastes must be managed in a sustainable manner in order to reduce their effect on our aesthetics, health, and the environment.

We have solid and fluid, hazardous and non-toxic wastes which are generated in our households, offices, schools, hospitals, and industries. No society is immune from day-to-day issues associated with waste disposal. How waste is handled often depends on its source and characteristics, as well as any local, state, and federal regulations that govern its management. Practices generally differ for residences and industries, in urban and rural areas, and for developed and developing countries. Every human partakes in the daily generation of solid waste, their influence also gets to a great portion of the populace across the globe.

Municipal Solid Waste (MSW)

Waste collected from residences, commercial buildings, institutions such as hospitals and schools, and light industrial operations is most often categorized as Municipal Solid Waste (MSW). MSW consists primarily of paper, containers and packaging, food wastes, yard trimmings, and other inorganic wastes. Municipal Solid Waste can also include industrial sludge, classified as hazardous or non-hazardous, resulting from a wide array of mining, construction, and manufacturing processes (Dadras et al., 2010).

The world has also witnessed changing production and consumption patterns with consequent rise in the volume of solid waste. We are not ignorant of the environmental, health and other challenges created by these waste. Pressure is being mounted on Governments as regards dealing with this increased waste generation. Local authorities will require huge capital investments and operational strategies for collection, transportation, processing, recycling and disposal of solid waste.

Solid Waste Management (SWM) is considered as one of the most serious environmental and social problems challenging municipal authorities in developing countries. Why? One chief reason is the impacts caused by the location of dumping site in unsuitable areas. The primary objective of a site selection process is to assure that chosen sites are suitable with regard to protection of public health and the environment.

Selection of landfill site is a complicated process because it requires data from diverse environmental and social aspects. These data often involve processing of a significant amount of spatial information which can be facilitated by the use of Geographical Information System (GIS) as a tool for land suitability analysis. Almost all factors related to SWM has both spatial and non-spatial components, thus, traditional ways of storing and analyzing data keep data in isolated form, which results in an inefficient management system. This presentation seeks to present the exploration on overall sustainable solid waste management system.

This presentation is arranged into five sections. After this introductory section, Section 2 reviews sustainable waste management strategies. In Sections 3 and 4, we'll discuss waste management in developing countries and geo-mapping of waste management resources respectively. Section 5 presents a typical Waste Management scenario in Anambra State, Nigeria..

Sustainable Waste Management Strategy

Waste management is essentially the collection, transportation, disposal or recycling and monitoring of waste. The term waste is assigned to the materials produced through human activities (i.e. gaseous, liquid, solid and radioactive matter). These generated materials are usually managed to avoid potential adverse effects they may pose to human health and environment.

Various methods of waste management have been developed and these methods differ for developed and developing nations, urban and rural populations, industrial and residential areas. The management of waste in metropolitan and rural areas is generally the responsibility of the Local Governments. While non-hazardous industrial waste is the sole responsibility of the waste generating industry itself.

It is imperative to establish an integrated community-based solid waste management system, that takes into consideration the informal actors of the sector and facilitate the planning and monitoring functions within the waste management system. The impact of this will include ncreased efficiency of the waste collection system leading to environmental and health improvements for the citisens and reduction of costs of collection for the city administration as a result of better use of existing resources and improved planning and monitoring.Existing waste management methods include;

Landfill: This method involves burying of the waste and this is the most common practice for the disposal of waste around the Globe. These landfills are quite often conventional with deserted and vacant locations around the cities. Landfills or borrow pits when designed carefully can serve as economical and quite sanitised method for waste dumping. However, not effectively designed and older landfills can cost a big amount to the Government not just in terms of money but also in terms of environmental and health issues. Apart from the general poor design landfill's common problems like wind-blown debris and generation of liquid, it can also cause production of gas, which is extremely hazardous. This gas can be a reason for production of odour, killing surface vegetation and greenhouse effects.

The characteristic, which is a must for an up to date landfill, is inclusion of clay or leachate lining. The waste that is deposited is generally compressed to increase the density and stability and later it is covered to have it prevented from vermin. An additional feature to modern landfills is the "gas extraction system" installation. This system is included to have the gas extracted from the borrow pit. This gas can be put to economic use.

Incineration: This method involves combustion for waste materials through incineration and temperature, also referred to "thermal treatment". This method is utilised to convert waste materials in to gas, heat, ash and steam. Incineration is conducted on both individual and industrial scale. This method is used for disposing off all sorts of materials. This generally is the most recognized practical method for disposing off perilous material. However, the method is conflict-ridden due to the emission of perilous gases. The burning procedure in this method for waste disposal is never perfect so, fear for gas pollutants is mounting.

Recycling: Recycling involves taking a waste item and reprocessing this material into a usable item either in the same form, as the original product, or into a different product. Recycling is widely assumed to be environmentally beneficial, although collecting, sorting and processing materials does give rise to environmental impacts and energy use. To improve resource recovery of source separated or commingled wastes in the facilities, appropriate processing equipment should be chosen to separate the components into usable fractions. Recycling includes collecting, sorting, processing or converting of materials**.** Materials, which is composed of a single type, is recyclables and is much easy to work with. However, complex products are difficult to treat and so are complex for recycling.

Biological Reprocessing: Waste materials, which come in organic nature are treated through biological reprocessing. The waste materials with organic nature are plant, food and paper products. The organic matter is subjected to biological decomposition which later if recycled in form of mulch or compost for landscaping and agricultural purposes. Additionally, the waste gas, which is collected from the process, is used for the production of electricity. The goal behind biological reprocessing is to control and speed up the natural decomposition of organic matter.

Numerous kinds of composting techniques and methods for digestion are employed depending upon the requirement depending on if digestion is required for household heaps or industrial materials. The diverse methods for biological reprocessing are classified as anaerobic or aerobic techniques.

Reduction and Avoidance Approach: Another method for managing waste is the avoidance for it being created and this method is generally named as "waste reduction". The avoidance for waste production includes using the second-hand product and repairing the products you have broken in place of buying new things. Products are designed for refilling and reusing.

Cutting down use of disposable things and producing products that are more complex. Nestle recently announced a global programme for this.

Waste Management in Developing Countries

Municipal solid waste management is one of the most critical and environmental problems faced by developing countries. Rapid growth of urban population and urbanization due to increase in per capita consumption is the main reason for this problem. Population increase is highly correlated with increase in solid waste generation, the higher the urban population the greater production of solid waste (Berisa & Birhanu, 2016; Olapiriyakul, 2017; Shoba & Rasappan, 2013).

The concepts and approach to waste management strategy vary the world over, however, the most commonly employed waste disposal approaches in developing countries are incineration and sanitary landfills. The acquisition and installation of incinerators is expensive, whereas landfill sites and required equipment are often available following construction works. Installations of incinerators were initially fashionable, as incineration plants are used for the cogeneration of electricity and heat. However, concerns associated with emissions and climate change tends to place this option at a disadvantage.

Landfill is usually the final function element of a Solid Waste Management system. It occupies the bottom of the hierarchy of all option of waste disposal. Along with other waste disposal option such as recycling, incineration and composting, landfill is the most preferred option because of its easy mode of operation, low cost, less technological involvement and comfort of implementation. The successful siting of a sanitary landfill requires overcoming significant environmental and political obstacles. It depends on convincing decision-makers and the public that a sanitary landfill is needed and the site is the most suitable of the options available (or at least among the best). However, despite the fact that solid waste landfills pose serious threat to the environment, the final destination of urban solid wastes in most of the countries in the world is dominated by land filling. A sanitary landfill site selection involves evaluation of various criteria using national or local land-use guidelines, environmental regulations, location restrictions, and so on. Social, environmental and technical criteria should be considered for potential landfill site selection. This is often not adhered to in Developing Countries.

Geo-Mapping of Waste Management Resources

Geo-mapping is essential in (1) Selection of site for sanitary landfills, (2) Location of natural impervious seals (clay liners) for non-hazardous wastes and (3) Solid waste generation and collection models.

The number of geologically suitable sites for sanitary landfills is limited. Hence sanitary landfill sites are expensive to acquire and engineer for environmental protection. Geological factors that influence the site selection include Climate, Bedrock Geology, Hydrogeology, Topography, Regolith and Soil Conditions. Appropriate geo-mapping of influencing factors can be achieved via Geographic Information System (GIS), Geological, Geophysical, Geomatics and Geotechnical Techniques. For this paper, we will discuss the GIS, Geophysical and Geotechnical techniques.

Geographic Information System

GIS is a computer system for capturing, storing, checking, integrating, manipulating, analyzing and displaying data related to positions on the Earth's surface. Typically, a Geographical Information System is used for handling maps of one kind or another. These might be represented as several different layers where each layer holds data about a particular kind of feature. Each feature is linked to a position on the graphical image on a map and a record in an attribute table. GIS allows us to view, understand, question, interpret, and visualize data in many ways that reveal relationships, patterns, and trends in the form of maps, globes, reports, and charts. A GIS helps you answer questions and solve problems by looking at your data in a way that is quickly understood and easily shared. GIS technology can be integrated into any enterprise information system framework.

Geographic Information System in Solid Waste Management

The development of Geographic Information System (GIS) and its use throughout the world has contributed a lot in improving waste management systems. GIS helps to manipulate data in the computer to simulate alternatives and to take the most effective decisions. GIS allows us to create and store as many layers of data or maps as we want and provides various possibilities to integrate tremendous amounts of data and map overlays into a single output to aid in decision making (Chang et al. 1997).

GIS can add value to waste management applications by providing outputs for decision support and analysis in a wide spectrum of projects such as route planning for waste collection, site selection exercises for transfer stations, landfills or waste collection points. GIS provides a flexible platform which integrates and analyses maps and waste management databases. Studies on the use of GIS in SWM have attracted huge investment from numerous researchers in recent times, and the summaries of such research works are discussed in the preceding paragraphs:

Solid waste management comprises several phases, starting from the stage where the waste is generated till it reaches its final destination or at a stage where it is no more a threat to the environment. "It is observed that Solid Waste Management can be bifurcated into mainly two phases. One is the waste management in the area where it is generated and second is the management of waste at dumping grounds." (Moiz Ahmed Shaikh, 2006).

Sarptas et al. (2005) studied the use GIS in Solid Waste Management in coastal areas as a decision support system with a case study on landfill site selection. The results of the study are that GIS is becoming a powerful tool in SWM. However, there are still some drawbacks and deficiencies in applying the method extensively. For example, it is not applied in solid waste generation studies because large fluctuations in solid waste generation by time and space and the dynamic nature of urban areas generate several difficulties in determining the current solid waste generation patterns. In the early period of GIS technology, from the 1980's to the early 1990's, GIS software was capable of executing only basic geographical operations. These capabilities of GIS software limited the user to only basic tasks such as exclusion and allowed them to determine only alternative landfill sites in landfill siting applications. Advances in information technology and increasing access to computer systems by decision makers have improved the usefulness of computer models and computer aided technologies in DSS in the last decade. One of the typical examples is the SWM systems. The GIS models do not only support the decision procedure but also facilitate the communication and mutual understanding between decision maker and the people, because the implications of a SWM closely affect the society. However, the basic limitation in the use of GIS in SWM arises from the data availability. Because, especially in developing countries the available data are very scarce and access to the data is very poor and tiring. In addition, the existing data are not reliable, not collected, stored and disseminated systematically. More comprehensive researches and more efforts on data gathering to fulfil the needs of GIS models are recommended.

Since Routing Models make extensive use of spatial data, GIS can provide effective handling, displaying and manipulation of such geographical and spatial information. For example, Ghose et al. (2006) proposed a model for the system of Municipal Solid Waste (MSW) collection that provides planning for distribution of collection bins, load balancing of vehicles and generation of optimal routing based on GIS.

Traditional and static approaches are no longer adequate for analyzing network flows and conducting minimum cost routing. Attemptshave been made to develop a decision support system for generating an optimum route for solid waste disposal in develop countries and hence reduce the distance ran by the collection vehicle using GIS. GIS tool provides an effective decision support through its database management capabilities, graphical user interfaces and cartographic visualization. Some of the systems developed experts make use of the Network Analyst module available in Arcview, which is a path-finding program used to model the movement of resources between two points or more (Reddy et al., 2009).

Chatila (2008) developed a GIS system for natural resources management and recycling of solid waste in the village of Marjeyoun in Southern Lebanon. A GIS map was prepared to serve as a zone management plan along with an environmental assessment that identifies cumulative pressures and impacts of some human activities on the village and the environment. A recycling program was developed based on solid waste sampling and analysis of collection systems.

In a study carried out by Bergeron et al. (2010), 3D visualization and GIS were used to produce a digital city model for the Star City, West Virginia to allow government officials and managers to manage assets and perform day-to-day operations, develop sustainable planning initiatives, and management of solid waste assets and facilities, planning for solid waste and recycling facilities and drop-offs, mapping and planning efficient waste hauler routes and identifying issues such a underserved populations and illegal dumping.

Landfill site selection is usually divided into two main steps: (i) the identification of potential sites through preliminary screening, and (ii) the evaluation of their suitability based on environmental impact assessment, economic feasibility, engineering design and cost effectiveness (Charnpratheep et al., 1997).

In using GIS to determine the "appropriate location" for a landfill site, the first step is the collection of all the data that would be needed to meet all of the criteria. These criteria include: wind direction, Distance from city, Water Body, Land Cover/use, slope, Geology and Transport Network.

i. Prevailing Wind Direction: The wind direction of potential sites must be considered. Settlements must not be affected from the odours that originates from the landfill. The wind has the main factor in the spread of noxious odours to the inhabitants and with GIS safe zones can be preselected.

ii. Distance from City: The distance from the city is also considered while siting a landfill in order to protect the settlements against excessive pollution resulting from the operation of the landfill (odours, noise from machines and discharge from the carriers/ refuse trucks, waste spread by wind, dust). A buffer zone of no less than 2 km around the residential areas is required for waste disposal.

iii. Proximity to Water Body: This is an important criterion from the standpoint of both environmental and economic. The landfill generates the leachate and noxious gases which render unsuitable to be in proximity to surface water. Therefore, a landfill must not be situated near any surface stream, rivers, drilling, spiring, or Water tower.

iv. Visual Exposition: For aesthetic considerations, the landfill site to be provided should be integrated into the landscape. The surface should not be exposed to view. For this reason, a buffer zone around railways, road network, settlements, religious institutions, plants etc should be chosen.

v. Slope: The slope is a fundamental factor in landfill establishment. Steep slopes are unsuitable for landfill sitting because construction costs of excavation increases in higher slopes. The suitable slope of land surface is important in preventing the leachate flowing hence, landfill site should be in areas with ground slopes.

vi. Proximity to Road Networks: The proximity to road networks is a factor of great importance: If landfill is placed too far away from the existing road networks, costs for solid waste transportation will increase

vii. Geological Setting: The geology of a site is an important consideration. Sadek et al. (2001) noted that in selecting a site for the landfill, particular attention should be given to the underlying bedrock and foundation soil. A landfill must be located and designed so as to meet the necessary condition for preventing potential pollution of the groundwater. For this raison, the lithology of appropriate landfill should be impermeable and sufficiently thick.

Though GIS can be used for preselection of suitable sites and for exclusion of unsuitable areas for landfill, existing information related to Geology, Hydrogeology, Hydrology, Climatology and Ecosociology must be made available and produced in a digital format. With respect to the geology and groundwater conditions of a potential landfill, we can employ Geophysical or Geotechnical surveys or both.

Geophysical Techniques

The use of various geophysical techniques for the location of non-hazardous and hazardous waste sites is a cost-effective means of preliminary evaluation. Information obtained from geophysical investigations can be used to determine the subsurface conditions. Geophysical techniques provide information on hydro stratigraphic framework, depth to bedrock, extent of concentrated ground water contaminant plumes, the location of voids, faults or fractures, and the presence of buried materials. There are two general types of geophysical methods: 1) active, which measure the subsurface response to electromagnetic, electrical, and seismic energy generated by artificial sources; and 2) passive, which measure the earth's ambient magnetic, electrical, and gravitational fields. Geophysical instruments are designed to map spatial variations in the physical properties of the Earth.

Geoelectrical Methods

The sub-surface resistivity distribution can be determined by making measurements on the ground surface. From these measurements, the true resistivity can be inferred. The ground resistivity is related to various geological parameters such as the mineral and fluid content, porosity and degree of water saturation in the rock. The resistivity measurements are normally made using a man-made source of electrical current that is applied to the earth through grounded electrodes (C1 and C2). The resulting potential field is measured along the ground using a second pair of electrodes (P1 and P2). The transmitting and receiving electrode pairs are referred to as dipoles. By varying the unit length of the dipoles as well as the distance between them, the horizontal and vertical distribution of electrical properties can be calculated. From the current (I) and voltage (V) values, the apparent resistivity (ρa) is calculated, $\rho a = k V / I$ where k is the geometrical factor which depends on the arrangement of the four electrodes. The apparent resistivity is not the true resistivity of the surface, but this value equals that of the resistivity of homogeneous ground measured from the same electrode arrangement. From the apparent resistivity values it is possible to obtain the true surface resistivity using an inversion method.

1D Vertical Electrical Sounding (VES)

The vertical electrical sounding is performed by changing the electrode spacing over a common centre point. The electrical current is applied to A and B electrodes and the potential V is measured between M and N electrodes. The VES array consists of a series of the electrode combinations AMNB with gradually increasing distances between the electrodes for subsequent combinations. The depth of sounding increases with the distance between A and B electrodes. VES provides measurements of apparent resistivity obtaining a sounding curve, by plotting resistivity ρa against the half distance between the current electrodes (AB/2) with $k = \pi$ (AO2 - MN2/4)/MN. It is interpreted quantitatively to derive thickness and resistivity of sub-surface layers using the appropriate software which determines a depth/layer model of resistivity (1-D-model). It provides the vertical resistivity variation under the centre point and it is applicable to horizontally layered homogenous ground.

This method is employed in the geophysical exploration of the shallow sub-surface. The normal depth range that is investigated is from a few centimetres to a few hundred meters. These techniques are thus particularly applicable in groundwater, and environment characterizations. The distinction between mainly clayey/silty layers (aquitards) and dominantly sandy/gravely beds (aquifers) are easily delineated using this method. Furthermore, it can be used to determine saltwater intrusion and contaminant plumes.

2D Electrical Tomography (ET)

2D imaging assumes low variation of the third dimension. 3D surveys involve large amount of measurements and more data processing. In an electrical tomography an array of regularly – spaced electrodes is deployed. They are connected to a central control unit via multi-core cables. The common arrays used are dipole-dipole, Schumberger and Wenner, depending on application and the resolution desired (Loke, 2001). The advantages and disadvantages of these arrays will be used to choose the appropriate configuration in each case. In this case, the spacing between the current electrodes pair, AB is given as "a" which is the same as the distance between the potential electrodes pair MN. The same process is repeated for measurements with different spacing ("2a" to "na"). The median depth of investigation of this array also depends on the "n" factor, as well as the "a". Resistivity data are then recorded via complex combinations of current and potential electrode pairs to build up a pseudo cross-section of apparent resistivity beneath the survey profile

Electrical Tomography is commonly applied to:

a. Determination of the depth and thickness of geological strata.
b. Detection of lateral changes and locating anomalous geological conditions
c. Locating buried wastes (e.g., locate landfill)
d. Mapping saltwater intrusion and contaminated plumes.

This electrical resistivity imaging survey is widely used to control the depth extent and geometry of the landfill. Frequent monitoring of landfill leachate with this technique could allow early leak detection.

Advantages of Electrical Resistivity Methods

- ✓ Quantitative modelling is possible using either Computer Software or Published Master Curves.
- ✓ The models can provide accurate estimates of depth, thickness and electrical resistivity of subsurface layers and other possible interpretations.

Limitations of Electrical Resistivity Methods

- ✓ Degradation of the quality of the measured voltages due to site characteristics, rather than in any inherent limitations of the method.
- ✓ It requirement for fairly a large area far removed from cultural noise precludes its use at many ground water pollution sites.
- ✓ The fieldwork tends to be more labour intensive than some other geophysical techniques.

Induced Polarization

The Induced Polarization (IP) method is an electrical geophysical technique, which measures the slow decay of voltage in the subsurface following the cessation of an excitation current pulse. An electrical current is imparted into the subsurface, as in the electrical resistivity method explained earlier in this Paper. Water in the subsurface geologic material (within pores and fissures) allows certain geologic material to show an effect called "induced polarization" when an electrical current is applied. During the application of the electrical current, electrochemical reactions within the subsurface material take place and electrical energy is stored. After the electrical current is turned off the stored electrical energy is discharged which results in a current flow within the subsurface material. The IP instruments then measure the current flow. Thus, in a sense, the subsurface material acts as a large electrical capacitor. The Induced Polarization method measures the bulk electrical characteristics of geologic units; these characteristics are related to the Mineralogy, Geochemistry and grain size of the subsurface materials through which electrical current passes.

Advantages of Induced Polarization Technique (IP)

- ✓ IP data can be collected during an electrical resistivity survey, if the proper equipment is used.
- ✓ The addition of IP Data to a resistivity investigation improves the resolution of the analysis of Resistivity Data by resolving ambiguities encountered in thin stratigraphic layers while modelling electrical resistivity data, distinguishing geologic layers which do not respond well to an electrical resistivity survey and measuring Electrical Chargeability used to enhance a hydrogeologic interpretation, such as discriminating equally electrically conductive targets such as saline, electrolytic or metallic-ion contaminant plumes from clay layers.

Limitation of Induced Polarization Technique

- ✓ It is more susceptible to sources of cultural interference than the electrical resistivity method.
- ✓ Its equipment requires more power than resistivity-alone equipment – this translates into heavier, costlier and bulkier field instruments.
- ✓ Its complexity in the interpretation, the expertise needed to analyze and interpret its data may be too tasking.
- ✓ Fieldwork tends to be labour intensive and its surveys require a large area, far removed from cultural noise.

Ground Penetrating Radar (GPR)

The Ground Penetrating Radar (GPR) method is a high resolution frequently used geophysical method. It provides subsurface information ranging in depth from several tens of meters to only a fraction of a meter. When and where possible, the GPR technique is usually integrated with other geophysical and geologic data to provide the most comprehensive site assessment. The GPR method uses a transmitter that emits pulses of high-frequency electromagnetic waves into the subsurface. The transmitter is either moved slowly across the ground surface or moved at fixed station intervals. The penetrating electromagnetic waves are scattered at changes in the complex dielectric permittivity, which is a property of the subsurface material dependent primarily upon the bulk density, clay content and water content of the subsurface. The electromagnetic energy is reflected back to the surface-receiving antenna and is recorded as a function of time.

Depth penetration of GPR is severely limited by attenuation and/or absorption of the transmitted electromagnetic (radar) waves into the ground. Generally, penetration of radar waves is reduced by a shallow water table, high clay content of the subsurface, and in areas where the electrical resistivity of the subsurface is less than 30 ohm-meters. Ground penetrating radar works best in dry sandy soil where a deep water table exists. The plot produced by most GPR systems is analogous to a seismic reflection profile; that is, the data are usually presented with the horizontal axis as distance units (feet or meters) along the GPR traverse and the vertical axis as time units (nanoseconds).

Advantages of Ground Penetrating Radar Technique

- ✓ It can provide a data along a traverse, which can qualitatively be interpreted instantly in the field.
- ✓ It is capable of providing high-resolution data under favourable site conditions and it's very effective in sandy soils.
- ✓ Its real-time capability results in a rapid turnaround, and allows for a quick evaluation of subsurface site conditions.
- ✓ It can provide a good cross-sectional representation of the subsurface useful for a number of applications

Limitations of Ground Penetrating Radar Technique

- ✓ It is a site-specific nature technique.
- ✓ The cost of site preparation necessary prior to performing the survey may be high.
- ✓ Most GPR units are towed across the ground surface.
- ✓ The quality of the data can easily be degraded by a variety of factors such as an uneven ground surface or various cultural noise sources (such as strong electromagnetic fields).
- ✓ Surveying requirements are not as stringent as for other methods.

Magnetics

Magnetic method is an effective method for the delineation of fracture zone in basement areas, when evaluating potential location of landfill sites. A common objective in conducting a magnetic survey at a hazardous waste or ground water pollution site is to map these anomalies and delineate the area of buried sources of ferrous anomalies (buried landfills). Analysis of magnetic data can allow an experienced Geophysicist to estimate the regional extent of buried ferrous targets, such as a steel tank, canister or drum of used in hazardous waste disposal on land and in sea.

Advantages of Magnetic Technique

- ✓ It is relatively very low costs effective and time saving in conducting and completing the survey.
- ✓ It requires very little, if any or no site preparation necessary.
- ✓ Surveying requirements are not as stringent as for other methods.

Limitations of Magnetic Technique

- ✓ It is very susceptible to cultural noise, which is detrimental to the quality of data.
- ✓ Inability of the interpretation methods to differentiate between various steel objects.
- ✓ The magnetic method does not allow the interpreter to determine the contents of a buried tank or drum.

Seismic Methods

Surface Seismic Techniques used in ground water pollution site investigations are largely restricted to Seismic Refraction and Seismic Reflection Methods. The equipment used for both methods is fundamentally the same and both methods measure the Travel-Time of acoustic waves propagating through the subsurface.

Seismic Refraction Method

In the refraction method, the travel-time of waves refracted along an acoustic interface is measured. In the reflection method, the travel-time of a wave, which reflects off an interface, is measured. Depth to geologic interfaces can be calculated using the velocities obtained from a seismic investigation. The geologic information gained from a seismic investigation can then be used in the hydrogeologic assessment of a ground water pollution site and the surrounding area. The interpretation of seismic data can indicate changes in lithology or stratigraphy, geologic structure, or water saturation (water table).

Advantages of Seismic Refraction Technique

- ✓ It can be used to determine the seismic velocity of a geologic horizon and precisely estimate the depth to different acoustic interfaces.
- ✓ It can easily determine the depth to the water table or bedrock, useful in buried valley areas to map the depth to bedrock or thickness of overburden, used to categorize geologic strata, determine thickness of geologic strata and determine depth to water table.
- ✓ It can differentiate certain units with divergent seismic velocities, such as Shale and Granites.

Limitations of Seismic Refraction Technique

- ✓ The seismic refraction method is based on several assumptions.
- ✓ Its success depends on (a) the Seismic Velocities of the geologic layers increase with depth and that its contrasts between layers are sufficient to resolve the interface. (b) The geometry of the Geophones in relation to the refracting layers will permit the detection of thin geologic layers. (c) The apparent dip of the units or layers is less than ten to fifteen degrees.
- ✓ Data collection is labour intensive and large line lengths are needed — as a rule, the distance from the shot, or seismic source, to the geophone stations (or geophone "spread") must be at least three times the desired depth of exploration.

Seismic Reflection Method

In the Seismic Reflection Method, sound wave travels down to a geologic interface and reflects back to the surface. Reflections occur at an interface where there is a change in the acoustic properties of the subsurface material.

Advantages of Seismic Reflection Technique

- ✓ It yields information that allows the interpreter to discern between discrete layers and map stratigraphy.
- ✓ Its data is usually presented in Profile Form and depths to interfaces as a function of time.
- ✓ Depth information can be obtained by converting time sections into depth from velocities obtained.
- ✓ It requires much less space and does not require long offsets as in refraction surveys.
- ✓ In some geologic environments, its data can yield acceptable depth estimates.
- ✓ Limitation of Seismic Reflection Technique
- ✓ The precise depth determination cannot be made.
- ✓ It is labour-intensive and the data acquisition is more complex than refraction data.
- ✓ It places higher equipment capabilities requirements.
- ✓ It requires a large amount of data processing time and lengthy data collection procedures.
- ✓ The use of high-resolution reflection seismic methods places a large burden on the resources of the Geophysicist, in terms of computer capacity, data reduction and processing programs, resolution capabilities of the seismograph and geophones, and the ingenuity of the interpreter.

Geotechnical Techniques

To further confirm the suitability of a substructure as a landfill, soil samples are taken from potential sites and tested for their consistency characteristics such as grain size, matrix structure, porosity, permeability, moisture content etc.These samples can be taken from geotechnical boreholes, trial pits etc.

Additionally, application of geotechnical techniques in SWM includes determination of the geotechnical properties such as compaction, hydraulic conductivity, compressibility and shear strength properties of fresh and old municipal wastes. Important researches have been done and many contributions made concerning these issues in the last two decades by various authors. Such researches reinstate that municipal solid waste (MSW) landfill design and remediation requires an appropriate characterization of the mechanical properties of the wastes involved. It is necessary to assess resistance in order to analyze the waste's stability and its interactions with other landfill structures, such as liners and gas and leachate collection systems.

However, there is a great variability in the data published concerning MSW strength. For example, cohesion values from 0 to 80 kPa and friction angles from 0° to 60° have been reported (Bray et al., 2009). Some of the many reasons suggested for justifying the variability of MSW shear strength include the composition of the waste; the age of the waste or degree of degradation; the representativeness of the samples; the procedures used to prepare the samples; the testing methodologies; and the use of different testing devices that induce discrepancies in the stress-strain-strength response of MSW, among others (Landva and Clark, 1990; Coumoulos et al., 1995; Knochenmus et al., 1998; Carvalho, 1999;Zekkos, 2005; Zekkos et al., 2007 &Reddy et al., 2009).

Solid Waste Management Looks Like in Awka, the Capital of Anambra State, Southeastern Nigeria: Case History

Intensive effort was put into studying the system of Solid Waste Management in Awka, Anambra State our findings will be shared herein. Anambra State Waste Management Agency (ASWAMA) is the body saddle with collection, transportation and disposal of solid wastes in the state with their head office within the state ministry of work premises, No. 1 – 5 works road, Awka. Men of the agency work round the clock to ensure clean and healthy environment.

Waste generated in the city by residents are in the estimate of tens of tonnes daily. Numerous designated dumps sites are placed about six poles from one another. High turnover was observed in daily waste generation as cleared dump sites on a Monday were overflowing with wasteson a Tuesday, barely one day after (see plates 2 & 3). Despite few illegal dump sites seen here and there, high compliance to the use of designated dump sites by residents was observed.

Wastes are cleared on a daily basis and taken to designated landfill at Ifite Awka through Ekwueme Square (Plate 7). The Agency was previously using one Landfill at Agu-Awka but recently shifted to the current one though reason for the shift was not gotten. The present Landfill is a widened flood plain which is gradually turning into gully erosion. It is located in an uninhabited part of the State, kilometres away from the residents. During a visit to the present landfill, it was observed that various ASWAMA trucks from various parts of the State Capital and environs arrived, offloaded wastes near the landfill and took off to get more wastes (Plate 8). This, we learnt, they do every day from Monday to Saturday including public holidays. A bull dozer (Plate 9) used in clearing and pushing the solid waste into the landfill was packed at the site. Several heaps of selected solid wastes comprising mainly of plastics containers, metallic objects, mattresses and cartons (Plate 10) were stacked at different corners around the Landfill. These materials, as we gathered are sold to the companies that turn them into new products or make them ready for re-use. This of course is recycling.

Plate 1: Anambra State Waste Management Agency (ASWAMA) Yard

Plate: ASWAMA designated waste dump site, Iyiagu, Awka visited on 30/09/19

Plate 3: ASWAMA designated waste dump site, Iyiagu, Awka visited on 01/10/2019

Plate 5: ASWAMA men & truck at work near Eke Awka Market on 01/10/19

Plate 6: ASWAMA trucks coming back from landfill site on 04/10/19

Plate 7: ASWAMA designated landfill site after Ekwueme Square, Ifite, Awka

Plate 8: ASWAMA trucks offloading waste near designated landfill Ifite, Awka on Saturday 5/10/2019

Plate 9: ASWAMA bulldozer used to clear & push solid waste into landfill

Plate 10: Heaps of recycles solid wastes new landfill in Ifite, Awka

Plate 11: Location of ASWAMA designated landfill (entrance and surrounding) away from residences, offices & industries.

Conclusion

Improper management of Municipal Solid Waste (MSW) causes hazards to inhabitants. Most developing countries are still using traditional ways of waste management as seen in Anambra and they are still keeping their environments safe and clean. The use of GIS in SWM will no doubt improve the job enthusiasm, save cost and time and generally increase efficiency. GIS is, therefore, a proven tool in estimating the ward wise per capita solid waste generation and in preparing a distribution map of waste generation in Municipals.

However, building and using a GIS requires expert knowledge, and can often prevent such systems from being used to their full capability by local officials. In addition, local government is often comprised of mature citizens who are committed to serving their community but may be wary of new technologies that are unfamiliar to them.

Though Anambra State particularly Awka the Capital City is relatively clean, almost all interviewed men of ASWAMA are unfamiliar with GIS and its usage. As far as they are concerned, they love doing their thing the traditional way and they are getting results. The present Landfill location was selected simply because a hollow was discovered and that must be filled without any studies-Geophysical or Geotechnical. No form of Geo-mapping whatsoever was applied. It was a mere coincidence that the site is far from human habitation.

Sustainable Waste Management will involve use of Geomappimg Techniques as described here in the selection and maintenance of waste dumps. It should also involve the recycling of wastes and conversion of wastes into Wealth rather than dumping and further polluting the environment.

Although wastes to wealth activities are still not common in Nigeria, it is beginning to take root in a number of places such as the FCT and Lagos. The initial capital outlay needed for such projects appear to be a major inhibiting factor for States and Local Government Authorities.

A possible reason why Waste Management in Developing Countries has not taken into account sustainability is the fact that there is ample space available and low level of enlightenment on the part of the populace.

A typical example of where inadequacy of space forced the Government to involve Geomapping in the Management of Waste was in Land Berlin (West-Berlin) Germany due to the land locked nature of the city of Berlin before the merger of West and East Germany. The Institute of Geophysics of the Frei Universitat Berlin worked extensively on the location and design of Waste Management Systems.

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Application of Cone Penetration Test in Soil Classification and Estimation of a Reliable CPT-SPT Correlation Model: Case Study from Soil Sites in Bayelsa, Niger Delta, Nigeria

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Abstract

Cone Penetration Test (CPT) and Standard Penetration Test (SPT) are the most commonly used in situ tests to delineate soil stratigraphy and determine the geotechnical engineering properties of subsurface soils. In spite of continuous efforts to standardize the SPT procedure and equipment, problems associated with its repeatability and reliability still exists. Hence, there is a need for reliable CPT-SPT correlations so that CPT data can be used in existing SPT based design approaches. Therefore, multiple linear regression (MLR) was used to develop formulas that can predict N (number of blows) value using CPT data from organic, clay-silty clay to clay, to silt mixtures-clayey silt to silty clay soils common across all the thirty-eight CPT-SPT pairs investigated in some soil sites in Bayelsa, Niger delta, Nigeria. Three correlation models were generated, one for each soil type based on Soil Behaviour Type (SBT) classification.

Keywords*: Standardize, Correlation, MLR, SBT, Model*

Introduction

Cone Penetration Test (CPT) is an in situ investigation technique, commonly used to identify near surface unconsolidated soils, offering the possibilities to create a continuous stratigraphic profile, soil property evaluation and also various physical parameters are recorded during the entire test (Fauzi et al, 2015). Lunes *et al*, (1997) stated that the CPT has the highest applicability in different ground conditions of any of the in situ tests in current use.

One of the major applications of the CPT is for soil profiling and type. CPT data provides a repeatable index of the aggregate behaviour of the in situ soil in the immediate area of probe. Although it is not possible to obtain soil sample during a CPT, the continuous nature of CPT results provides a detailed stratigraphic profile to guide in selective sampling appropriate for the project.

The Standard Penetration Test (SPT) is one of the most commonly used in situ tests in many parts of the world. In spite of continuous efforts to standardize the SPT procedure and equipment, there are still problems associated with its repeatability and reliability. Hence, there is a need for reliable CPT/SPT correlations so that CPT data can be used in existing SPT based design approaches.

This study is focused on the application of CPT in soil classification, and estimation of a reliable CPT-SPT correlation model in some soil sites in Bayelsa, Niger Delta, Nigeria.

Location and Accessibility

The study area which is located within Azagbene, Torogbene, Angabiri, Gbaran, Peretorugbene and Ogbotobo in the Niger Delta region of Nigeria can be found geographically within Longitude 050 32'21.2" to 05046'52.6" North and Latitude 040 46'50.9" to 050 02'56.8" East.

The area falls within the lower delta plains believed to have been formed during the Holocene of the quaternary period by the accumulation of sedimentary deposits. The major geological characteristic of the area is sedimentary alluvium. The area is characterized by abandoned beach ridges and due to many tributaries of the River Niger in this plain, considerable geological changes still abound.

Previous Works

Mahler and Szendefy (2009), processed data gained from both CPT and Dynamic Probing (Heavy, DPH) tests of 83 ground layers with the view to estimate CPT resistance based on DPH results. Data used were carefully evaluated to filter out the influence of the

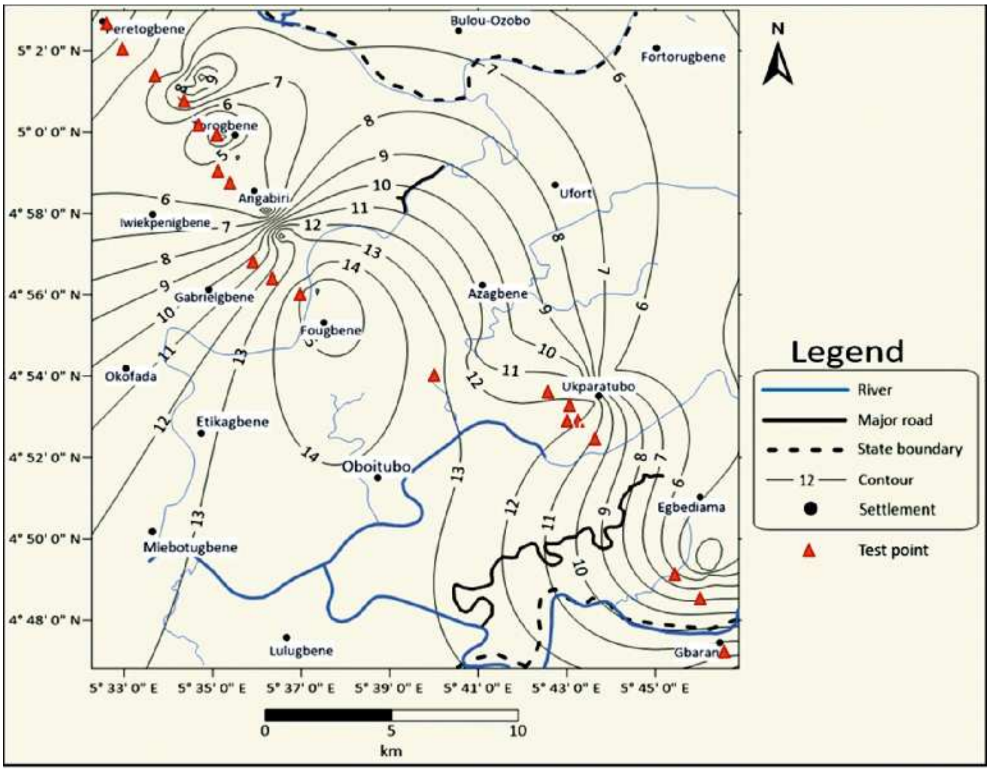

Fig. 1: Map of the study area.

formation boundaries and adjacent layers hence, data from homogenous layers having the thickness of at least 2 meters. Results obtained show that in the case of soils containing gravel size fraction (d_{max} > 2.0mm) an acceptable relationship cannot be stated between the probe resistance values. This is because of the higher grain size, the result fluctuates in a very wide range for both probe. For harder state clays (LI ≤ 0.15) the conditions of the cohesive soils cannot be reliably characterized using DPH. While the CPT resistance was approximately constant in some homogenous clay layers, the DPH blow numbers spread a wide range. They concluded that the relationship between the CPT and DPH results can be defined with medium reliability in case of cohesive soils and with high reliability in case of grained soils.

Ahmed *et al.* (2014), worked on CPT-SPT correlations on quartzite/non-crushable sands and calcareous/crushable sands with the aim of proposing a unified CPT-SPT correlation for non-crushable and crushable cohesionless soils. The proposed approach was presented with the aim to define the related soil compressibility parameters of the CPT-SPT correlations. The approach incorporates quantifying the soil compressibility related to CPT in terms of the behavior index (Ic), and the compressibility related to SPT in terms of the mean diameter (D_{50}). They concluded that the presented methodology enhances the reliability of the CPT-SPT correlations and provides a unified approach encompassing both crushable and non-crushable sands.

Shahri et al. (2014), determined a reliable correlation of CPT-SPT by a detailed comparison with other researchers in various mathematical relations for Lilia Edet area in southwest Sweden with the view of developing a correlation between tip resistance and N-value for various soil layers particularly in clayey soils with significant clay content. After reviewing of the published CPT-SPT correlations, those correlated without using statistical procedures were eliminated. Analysis was carried out using the arithmetic average method, Student t-test and statistical analysis for field and normalized data set and then using a filtering procedure for elimination of far from trend data. Application of several mathematical curve fitting tools, the correlation for three condition (linear with zero intercept, linear, power) were obtained and compared to each other. They arrived at a conclusion that filtered data have higher correlation coefficient but because of the applied accuracy in data processing, these differences are not significant.

Fauzi et al. (2015), studied SPT and CPT data for different type of sandy soils in Florida with the view to investigate the relationships between the SPT and CPT for different type of sandy soils. Correlations between the cone resistance, sleeve friction and the uncorrected SPT blow counts were established and soils were classified using the Unified Soil Classification System (USCS). Fine Sand with Silt (SP-SM) is the classification for soils that have 5% to 12% fines pass the No.200 (75 μm) sieve, The N values had a range between 3 to 85, whereas the qc was from 1 to 53 MPa. The correlation coefficients R^2 were less than 0.1. For Silty Fine Sand (SM), the range of N values was from 2 to 82 blows/0.3m, while the qc values ranged between 0.3 and 20 MPa. The relationship $R^2 = 0.35$, shows that there is a weak correlation between the cone resistance and N values in this type of soil. For Clayey Fine Sand (SC) with plastic fines that exceed 12%, (0-60) was the range of N-values and (0.35-16.3) MPa was the range of cone resistance soundings and shows a weak linear relationship between qc and N with $R^2 = 0.18$. The result of this study show that sandy soils with a high fine content produced higher friction values f_s, than poorly graded sand. The (qc/*N*) ratio for poorly graded sand is remarkably similar to suggested ratios in published literature. Inversely, the silty sand produced higher ratios than the literature ratios. The overall conclusion was that the relationship between the SPT and CPT produces a weak correlation in the Central Florida area for sandy soils with fines.

Feda and Ekrem (2017), correlated SPT and CPT results from Tekirdag, Turkey with the aim of finding reliable relationships between the two test results. To carry out this research, drilling logs were evaluated with samples of clay, silt and sandy soils. Various statistical methods were used to generate formulas and correlation coefficients for high-medium-low plastic clays and sand to sandy-clayey soils. The relationship $n = q_c/N_{60}$ was applied to generate correlation coefficients. Results obtained from this study shows that for High plasticity clays (CH), $q_c/N_{60} = 0.11$, Moderate plasticity clays (CI), $q_c/N_{60} = 0.11$, and for Low plasticity clays (CL), $q_c/N_{60} = 0.11$. For clayey Sand (SC), and silty Sand (SM), $q_c/N_{60} = 0.39$ was derived.. A positive linear relationship was found between the tip resistance q_c, skin friction resistance f_s, and N for various sandy soils. They concluded that when N_{60} has a linear effect with f_s, an increase in N_{60} as expected in this case means an increase in f_s and when q_c and f_s is linear, depth and terminating resistors increases. They also observed that skin friction resistance f_s, increases with depth alongside the tip resistance q_c, as predicted.

Bashar (2017), developed a model to predict the N_{60}-value from CPT data using artificial neural network (ANN). A back-propagation neural network was used to examine the feasibility of ANNs to predict the N60-value. Data used in this study consisted of 109 CPT-SPT pairs for sand, sandy silt, and silty sand soils. The ANN model input variables are CPT tip resistance (qc), effective vertical stress, and CPT sleeve and the ANN model output is SPT N60-value. Model 6 which has one hidden layer with six processing elements in the hidden layer, using sigmoid as transfer function, was the best performing model among all ANN models. Model 6 indicated that backpropagation neural networks have the ability to predict the N60- value with an adequate accuracy ($R^2 = 0.95$, MAE = 2.88). In conclusion, ANN model 6 either under predicted the N_{60}-value by 7-16% or over predicted it by 7-20% and that ANN is a good tool to predict N_{60}-value from CPT data with acceptable accuracy.

Arwan *et al.* (2018), studied about 197 CPT data in Western Central Java, produced from a data filtering process over 300 sites with the view to present a modified friction range for soil classification i.e. gravel, sand, silt & clay, and peat. IDW method was employed to interpolated friction ratio values in a regular grid point for soil classification map generation. Soil classification map was generated and presented using QGIS software and soil classification map with respect to modified friction ratio range was validated using 10% of total measurements. Range of friction ratio for each soil in the study area were presented as; Gravel = < 7%, Sand = 0.7% - 2.75%, Silt & Clay = 2.75% - 7%, and Peat = >7%. Results from this study shows that modified friction ratio range can effectively distinguish fine and coarse grains, thus useful for soil classification. They concluded that the modified friction ratio range produces 85% similarity with laboratory measurements and can be used to identify soil type for mountainous tropical region.

Methodology

Thirty-eight (38) CPT-SPT data pairs obtained from sites in the study area were used for this study. Distance between each CPT-SPT pair ranged from 5m to 5km while the depth of the pairs ranged from 5m to 17.8m. Water table was encountered in all CPT-SPT pairs between 0 to 0.1m below existing ground level. Each CPT recorded one reading every 0.2m while the SPT has one reading every1.5m to 2.5m. Interpretation of CPT data in terms of Soil Behavior Type (SBT) charts (Fig. 2) as proposed by Robertson (2010) was used for classification. The values of the dimensionless cone resistance, (q_c/p_a) is plotted against the Friction ratio, R_f to delineate the soil type at every depth interval. This method is generally implemented by the friction ratio (R_f) and the normalized cone resistance (qc) or the ratio between qc and the atmospheric pressure (Pa), i.e. 100 KPa, to estimate the soil behavior. To generate a suitable CPT-SPT correlation model, the multiple linear regression (MLR) model (a well-known approach that identifies the relationship between a set of dependent and independent variables using statistical methods) was applied to predict N-values Analysis was carried out to develop MLR model that better predict the N-value. The relations between the dependent variable and number of independent variables are in the form

$$Y_i = a_0 + a_1X_1 + a_2X_2 + a_3X_3 + \dots + a_kX_k + \dots \quad (1)$$

where, for a set of "*i*" successive observations, the predicted variable *Y* is a linear combination of an offset "a_0", a set of "*k*" predictor variables "*X*" with matching "*a*" coefficients, and a residual error ε. Where Y = the N-value which is denoted as N.

Results and Discussion

Many researchers in an attempt to classify soils using CPT data, have classified soils based on their textural and physical characteristics e.g. grain size distribution and percentage of fines content, which in most cases involves the use of disturbed soil samples. In this study, soil behavior type (SBT) chart is used in soil classification. The SBT system is strictly theoretical and it shows the true picture of how different soil types behaves in situ on application of load. Also the tip resistance q_c, is mostly used for CPT-SPT correlations in recent literatures. Therefore, in this study the tip resistance q_c, alongside other CPT parameters such as the sleeve friction f_s, friction ratio R_f, and depth of probe was applied to generate a reliable CPT-SPT correlation model using the multiple linear regression tool.

Soil Classification

Soils in this study are classified as organic soils – clay, clay – silty clay to clay, silt mixtures and sand mixtures. Using the SBT chart (Fig. 2) the interpretation of CPT data in terms of Soil behavior type as proposed by Lunne (1997) and Robertson (2009) can be seen. It can be noted that all data points have fallen in SBT zones two, three, four and five. SBT zone two represents organic soils to clay, SBT zone three represents clay-silty clay to clay, SBT zone four represents silt mixtures- clayey silt to silty clay and SBT zone five represents silty sand to sandy silt. However, the three common soil types identified across the thirty-eight sites are organic soils, clay-silty clay to clay, and silt mixtures- clayey silt to silty clay. Another soil type (i.e., Sand mixtures- silty sand to sandy silt)) were regarded as infrequent, and only consist of thin layers where present. The total depth of the CPT sounding depth was correlated to the corresponding SPT boring depth. However, results obtained from higher SPT depths shows that the presence of Sand mixtures- silty sand to sandy silt and Sand- clean sand to silty sand would most probably be encountered.

Organic soils (soils that fall in the SBT zone labelled 2 on the SBT chart) in the study area have thicknesses that range from 0 metres to 17.8 meters in some locations. They are characterised by high friction ratios, which were found to be greater than 5.1% on the average. Very low blow counts (0-4), and low tip resistance values (196-686) KPa were also obtained (Table 1.).

Soils classified as Clay-silty clay to clay were also observed to possess high friction ratios but not as observed in organic soils. An average friction ratio of 4.95% was obtained. Results show that they are obtained at an average depth of 7.83 meters across all the sites. In addition, soils classified as Silt mixtures- clayey silt to silty clay are characterized by lower friction ratio against the organic soils, with an average friction ratio of 30% was obtained across all point locations and N-values (6-24).

In general, the results proves that coarse-grained soils have a high tip resistance and low sleeve friction when compared to fine-grained soils with low tip resistance and higher percentage of fines. From observations, it is impossible to classify soils based solely on tip resistance or sleeve friction, but rather on their combination which is the friction ratio Rf. Higher friction ratios denotes higher percentage of fine-grained soil materials and vice-versa. The results also shows that coarse-grained soils have higher blow counts compared to fine-grained soils.

Soil Profiling

Data from the CPT and SPT were carefully evaluated for the determination of the soil profile as shown in Fig. 3. The evaluation uncovered three primary soil subsurface

Table 1: Soil Behavior Types and their characteristics

SBT	Variables	Mean	Minimum	Maximum
ORGANIC SOILS -CLAY	Tip Resistance (KPa)	207.3	196.13	686.5
	Sleeve Friction (KPa)	106.3	98.1	686.5
	Friction Ratio (%)	5.17	5	10
	N (Blow count)	2	0	4
	Depth(m)	8.8	0	17.8
CLAY (Silty clay to clay)	Tip Resistance (KPa)	1313.3	490	3530
	Sleeve Friction (KPa)	594.3	98.1	2549.7
	Friction Ratio (%)	4.95	2.5	10
	N (Blow count)	8	3	10
	Depth(m)	7.8	3	14.4
SILT MIXTURES	Tip Resistance, (KPa)	2622.2	1372.9	5099.5
	Sleeve Friction (KPa)	896.3	196.1	2549.7
	Friction Ratio (%)	3.05	0.86	5
	N (Blow count)	12	6	24
	Depth(m)	9.4	4.8	13.8

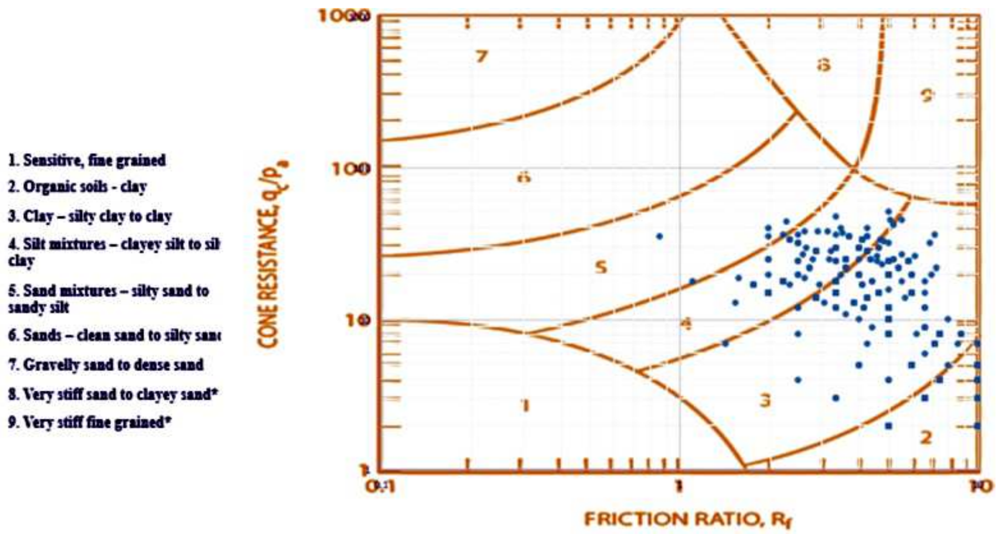

Fig. 2: SBT chart showing all the plots from the study area

zones in the study area. They are organic soils - clay, clay (silty clay to clay), and silt mixtures (clayey silt to silty clay).

Organic soils-clay occupy the topmost layer in this area. This layer is characterized by a dark gray to brownish gray color. They are soft and cover about 77% of the total subsurface area investigated.

Clay (silty clay to clay) soils are overlain by organic soils-clay, and underlain by silt mixtures in the study area. Soils in this layer have a gray to brownish gray colour. They cover about 18% of the total subsurface area investigated. Silt mixtures (clayey silt to silty clay) are overlain by Clay (silty clay to clay). They possess a light gray color and cover about 5% of the total subsurface area investigated.

Estimation of a Reliable CPT-SPT Correlation Model

Analysis was carried out to develop Multiple Linear Regression (MLR) models that better predict the N (number of blows). The models generated takes the general form of;

$$Y = a+b_1X_1+b_2X_2+b_3X_3+b_4X_4$$

Where

y = N (number of blows),

a = constant,

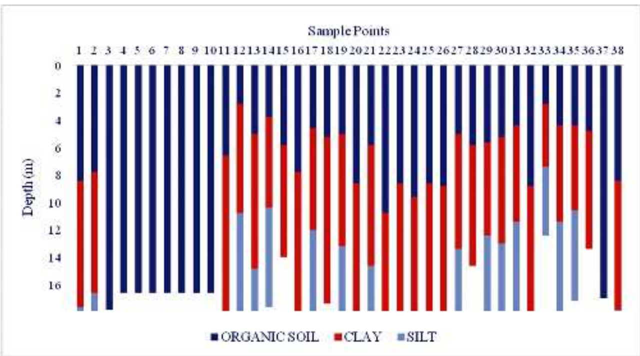

Fig. 3: A stratigraphic profile of the area in terms of SBT chart.

b = regression coefficient,
X_1 = Tip resistance,
X_2 = Sleeve friction,
X_3 = Friction Ratio Rf,
X_4 = Depth (m).

The model for organic soils can be written as; $N = 4.309 + 0.428X_1 + 0.531X_2 + 0.707X_3 + (-0.078)X_4$

The model for clay can be written as; $N = 5.636 + 0.532X_1 + 0.565X_2 + 0.490X_3 + (-0.063)X_4$

The model for silt can be written as; $N = 2.493 + 1.390X_1 + 1.411X_2 + 4.605X_3 + (-0.200)X_4$

The Figures below are examples of CPT and SBT plots for points 30 – 36.

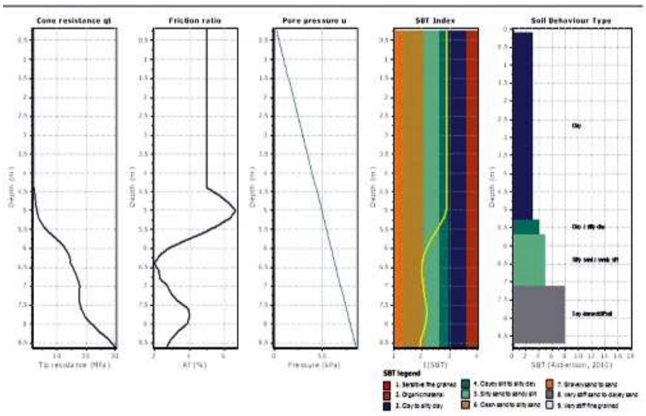

Fig. 4: CPT plot at point 30

Conclusions and Recommendations

By applying the cone penetration test, soil data can be obtained for every 2 cm of depth, which ensures greater reliability of soil data. Soil classification using the cone penetration test results has been a great challenge in geotechnical engineering. In the application of CPT results in soil classification using the Soil Behaviour Type (SBT) chart, soils in this area were classified into three soil types namely; organic soils, clay- silty clay to

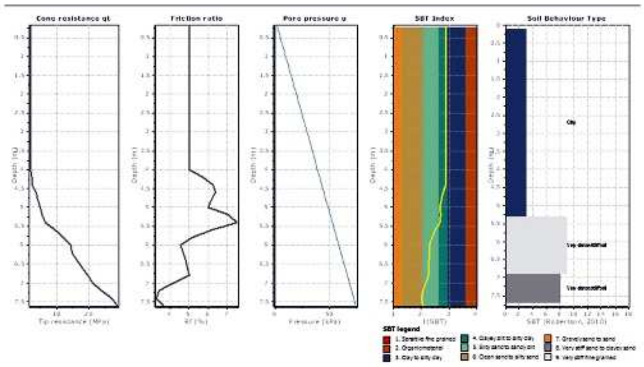

Fig. 5: CPT plot at point 31

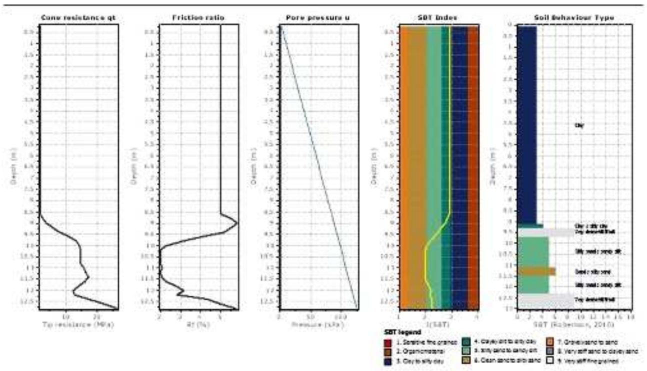

Fig. 6: CPT plot at point 32

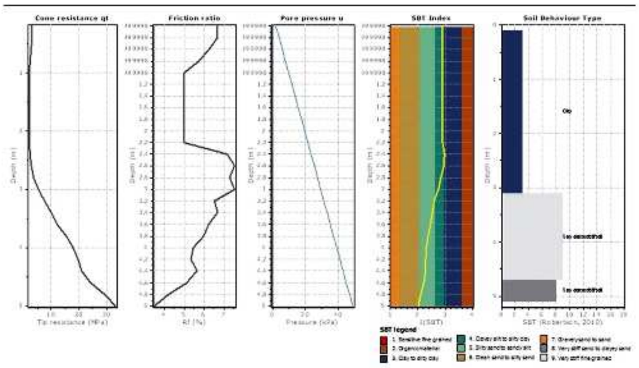

Fig. 7: CPT plot at point 33

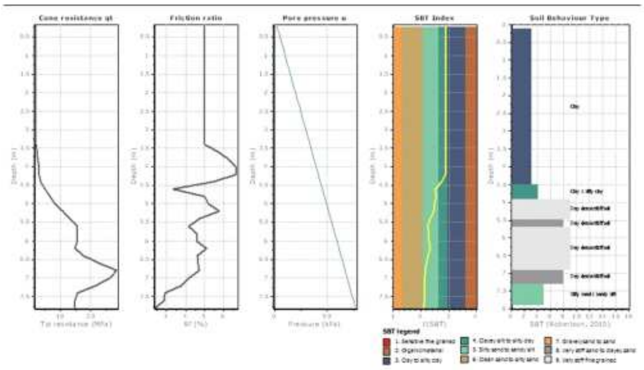

Fig. 8: CPT plot at point 34

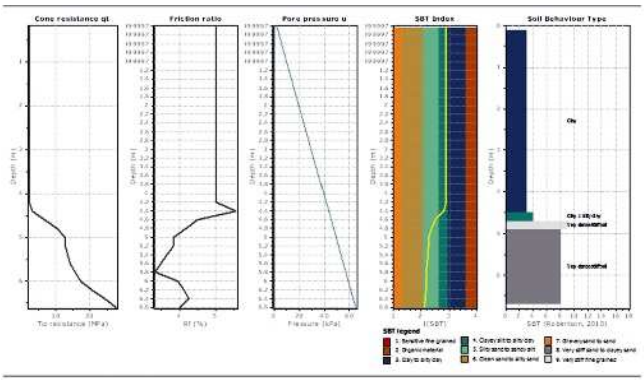

Fig. 9: CPT plot at point 35

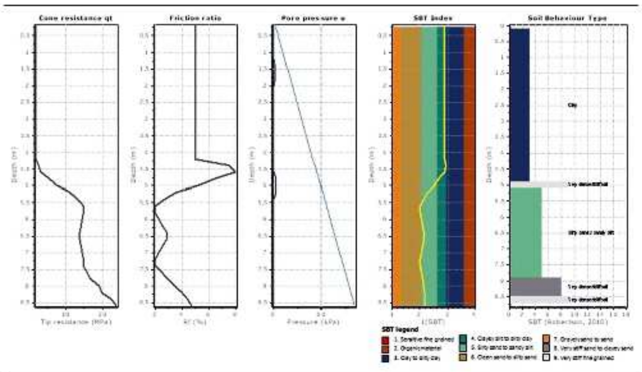

Fig. 10: CPT plot at point 36

clay, silt mixtures- clayey silt to silty clay and sand mixtures- silty sand to sandy silt.

The classification generally indicates thick sequence of organic soils of about overlying clay rich soils at the top of the stratigraphic profile of the study area. Thin layers of silt mixtures were delineated intercalating with clay but the silt mixtures were generally overlaid by clay-silty clay to clay and underlain by sand mixtures across the study area.

However, sand mixtures was not common across all site locations. An increase in the depth of CPT investigation would most probably reveal this soil type as SPT results already shows the presence of sand mixtures beyond 17 meters.

A comprehensive statistical analysis was carried out to develop MLR models to predict N-value using CPT data such as: tip resistance, sleeve friction, friction ratio and depth. Three models were developed for this purpose, one for each soil type common across the study area.

It is recommended that further studies should be carried out in this area as regards soil classification using analyzed soil samples as a measure of control for the CPT classification.

For better reliability in the classification of soil types using the SBT chart in this area, the depth of probe and number of CPT and SPT points should be increased.

Acknowledgements

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Safety of Building Foundations in Seismically Active Areas

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Introduction

It is my great honour to have been invited to deliver a lead paper at the 2nd International Association for Engineering Geology and the Environment (IAEG) Africa Regional Congress titled "**Safety of Building Foundations in Seismically Active Areas"**.

This led paper is not intended to give detailed accounts of the processes involved in a good foundation design for seismic (and thus for static) conditions. Pender (2007 and 1995) presented a useful review of this. Rather, only the aspects that have decisive influence on earthquake foundation engineering are discussed herein. Accordingly, the aim of this paper is to put in perspective the elements that impinge on the seismic design of a foundation.

The African tectonic plate where Nigeria lies has been presumed stable and not prone to earthquakes. Contrary to this assumption, recent reports and occurrences of ground shakings (tremors) at Abeokuta (2009), Kaduna (2016), Saki (2016), Bayelsa (2016) and Abuja (2018), have erased this general perception and indicated that earth tremors and earthquakes are potential natural hazards in Nigeria. According to Adepelumi (2019 in press), earth tremors occurred in Nigeria in 1933, 1939, 1964, 1984, 1990, 1994, 1997, 2000, 2009, 2011, 2016, 2018 and 2019. Though global seismicity maps generally confirmed the aseismic nature of West African region in general and Nigeria in particular, historical evidences indicate that earthquakes have been felt in some parts of Nigeria since 1933 till date. Akpan and Yakubu (2010) gave a comprehensive review of earthquake occurrences and observations in Nigeria. Analyses from statistical techniques have revealed a trend of an increasing seismicity along the Mid-Atlantic Ridge bordering West African region (Isogun and Adepelumi, 2014, Afegbua et al. 2018), in terms of number of earthquake occurrence per year and the magnitudes from 1963 to 2014. These show that tectonic stresses along the ridge and in the Gulf of Guinea, as well as the seismic activities are progressively increasing. The Nigerian landmass is not expected to be affected by naturally occurring earthquakes because it is located far from the major earthquake zones of the world, as the nearest active plate boundaries lies far away at the Mid-Atlantic Ridge.

The increase in seismic activities along the ridge may be responsible for observed increase of earthquake occurrence especially in the West African countries along the coast of the Atlantic Ocean. Although, West Africa is not an active region compared to regions of high seismicity like East African Rift, Japan, etc., a trend that showed an increasing earthquake occurrence of bigger magnitudes has been established as at today. These developments, coupled with the information contained in the seismotectonic map of Africa released in 2016 (**Fig. 1**), it is a clear indication that seismic activities in Nigeria are probably on the increase. On the other hand, although, the seismicity of Nigeria has always been known as being low to medium, however, judging by the increasing occurrence of seismic activities in the country in the recent past, and the alarming damages now manifestly associated with some events in Nigeria (border towns along Rivers and Bayelsa States in 2016 and Kwoi town in Kaduna State in 2016), the country is probably gradually transiting into one of the regions with frequent earthquakes and earth tremors. Thus, there is the need to study the phenomenal increase in relation to tectonic activities and stress development.

Akande *et al.* (2016) posited that, "in Nigeria like any other countries in the world, building collapse menace is growing at an alarming rate, seemingly uncontrollable or beyond control. The incessant buildings collapse in Nigeria has become a great concern to all the stakeholders and the professionals in the building industry, government, private developers, clients and users, as well as the residents. In an attempt to unravel and find lasting solution to the causes and effects of building collapse in Nigeria, this lead paper thus attempt to examine the Safety of Building Foundation in a Seismically Active Areas vis-à-vis building safety in such region with the case study of Nigeria.

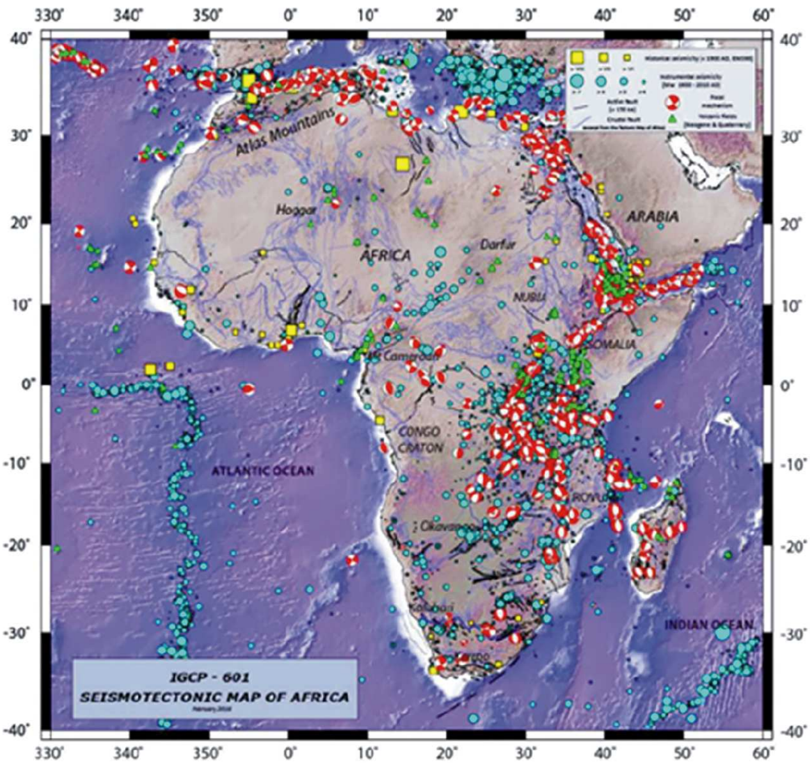

Fig. 1: Seismotectonic map of Africa (Meghraoui *et al*., 2016)

The recent seismic hazard occurrence in some parts of Nigeria and worldwide has led to increase of awareness of the potential losses due to earthquakes have led several countries such as Nigeria in which seismic hazard was formerly ignored to pay greater attention to seismic design and the safety of building foundation in low to moderate (LTM) seismic areas.

Almost every four years earth tremor (Natural and Anthropogenic) occur in Nigeria. It seems that relatively little is being done to prepare for, prevent or mitigate the effects of future earth tremor and possibly earthquakes on buildings.

According to Murty *et al. (*2011), earthquakes do not kill people; seismically unsafe buildings do. Primarily, the earthquake safety problem is a structural safety problem, and hence a technology problem.

Safety of buildings foundation in LTM area is of utmost importance because structures without proper seismic design are major contributor to losses to both life and property, during earthquakes since the perception of risk is low and therefore the preparedness is abysmally low or even absent. Many communities in such regions are far from recognizing the problem that safe housing is critical to their sustainable development. There is therefore the need to prepare seismic vulnerability and seismic zone map of Nigeria since this has not been in place up-till-now.

Buildings can be divided into two main categories, namely engineered buildings and non-engineered buildings (Teddy, 1978). In this paper, the focus would be principally on the non-engineered buildings since they constitute over 90% of the buildings in Nigeria.

Arya (2000) suggested that the non-engineered buildings considered are those which are spontaneously and informally constructed in various countries in the traditional manner without any or little intervention by qualified architects, builders and engineers in their design. The safety of the non-engineered buildings from the fury of earthquakes is a subject of highest priority in view of the fact that LTM seismic zones of the developing world more than 90 percent of the population is still living and working in such buildings, and that most losses of lives during earthquakes have occurred due to their collapse. The risk to life is further increasing due to rising population density in these countries, poverty of the people, scarcity of modern building materials, lack of awareness and necessary skills for improved constructions.

The present disaster management policies of the governments in the developing countries do not address the issue of preventive actions for the safety of such buildings toward seismic risk reduction; the development plans do not require consideration for safety from hazards as an essential component of the projects; the settlement planning and development legislations have no provision to attend to hazard safety concerns, and the building by-laws of municipalities and corporations are silent about earthquake resistance in buildings. The Codes and Guidelines developed through the standard making bodies remain recommendatory documents of good engineering practices, and their implementation depends upon the decision of the Heads of Agencies, Departs, Organizations, Institutions owning the buildings and structures in the public and private sectors. Private individuals have by and large remained uninformed.

In countries (like Nigeria) where seismic design is not practiced in general, the occurrence of major earthquake is expected to cause both large fatalities and huge economic setback. This is based on the premise that, about 90% of Nigeria's construction is of non-engineered type with little or no engineering input, and the rest is made of reinforced concrete. Even this 10 % of so called engineered structures are vulnerable to earthquake shaking because seismic codes were not incorporated into their structural designs. Unfortunately, the number of practicing professionals formally trained in earthquake safety related subjects is very small, and a number of projects are being implemented without sufficient expertise in earthquake resistant design and construction is relatively very small in the country. Standards exist for earthquake-resistant design of structures but are not used in most building projects, sometimes because of lack of understanding amongst designers and non-availability of the relevant seismic codes in Nigeria. Even the available non-formal standards have many loopholes and designers exploit them to reduce cost of construction.

Large amount of technical information on earthquake safe constructions is available within the technical community worldwide (Brzev & Greene, 2004, Brzev et al, 2004), and also in public domain (e.g., visit the World Housing Encyclopedia website www.world-housing.net). But, this information has not been incorporated into the Nigerian building codes in order to reduce earthquake risk in future.

Earthquake Risk of Housing

Ramancharla and Murty (2014) defined earthquake risk as the projected aggregated effect of the expected number of lives lost, persons injured, property damaged and economic activity disrupted due to an expected strong earthquake in an area. Usually, it is represented as the product of the prevalent earthquake hazard (H) of the area, the number of persons exposed to the earthquake hazard (E), and the known vulnerability (V) of the houses in that area, as:

Risk = H x E x V....................(1)

Earthquake hazard is defined as the potential threat of occurrence of a damaging earthquake, within the design life of the house in a given area. The hazard due to an earthquake can be reflected by expected intensity of ground shaking quantified by peak ground acceleration (PGA(, peak ground velocity (PGV) and peak ground displacement (PGD), soil liquefaction, surface fault rupture and slope instability. Whilst, earthquake vulnerability of a house is the amount of expected damage induced to it by a certain level of earthquake intensity. Housing threat factor (HTF)is defined as the cumulative of the hazard at the site and exposure in the housing. A housing threat factor is used to understand the overall status of housing. This factor is obtained by multiplying seismic zone factor (Z) of the seismic zone by number of houses per km^2 for each state in Nigeria. For the purpose of sustainable development, it is suggested that the HTF index should be employed only to determine the earthquake vulnerability of different areas and prioritize regions where different categories of earthquake resistant buildings should be built.

For the sustainable development of a developing nation in LTM seismic region where the rural to urban migration is very high, thereby leading to concomitant high demand for land for building construction, it is expedient to estimate the earthquake risk of housing, earthquake hazard, earthquake vulnerability and housing threat factor of a house before and after constructing buildings in such area.

How Buildings Resist Earthquakes

Seismic forces are inertia forces. When any object, such as a building, experiences acceleration, inertia force is generated when its mass resists the acceleration. We experience inertia forces while travelling. Especially when standing in a bus or train, any changes in speed (accelerations) cause us to lose our balance and either force us to change our stance or to hold on more firmly.

Inertia forces act within a building. They are internal forces. As the ground under a building shakes sideways, horizontal accelerations transfer up through the superstructure of the building and generate inertia forces throughout it. Inertia forces act on every item and every component. Every square metre of construction, like a floor slab or wall, possesses weight and therefore mass. Just as gravity force that acts vertically is distributed over elements like floor slabs, so is seismic inertia force, except that it acts horizontally.

Loading imposed by earthquake shaking under the building is of displacement-type and all other hazards are of force-type. Earthquake shaking requires buildings to be capable of resisting certain relative displacement within it due to the imposed displacement at its base, while wind and other hazards require buildings to resist certain level of force applied on it (Figure 2). While it is possible to estimate with precision the maximum force that can be imposed on a building, the maximum displacement imposed under the building is not as precisely known.

Blondet and Aguilar (2007) described the sequence of

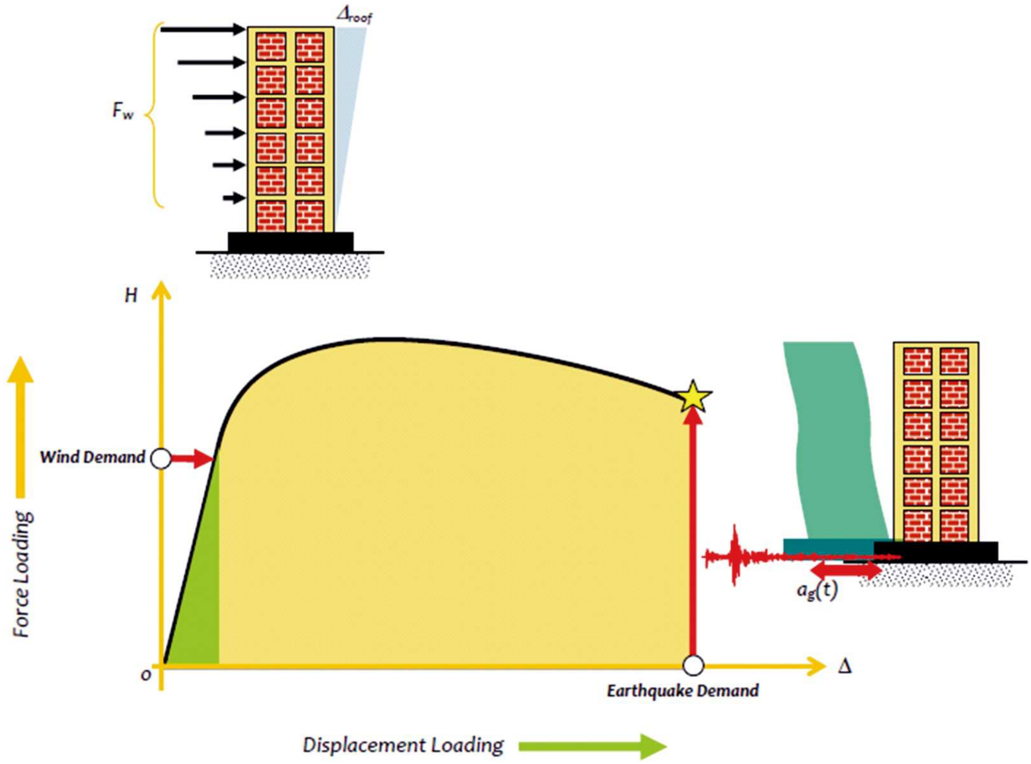

Fig. 2: Displacement Loading versus Force Loading: Earthquake shaking imposes displacement loading on the building, while all other hazards impose force loading on it

events that happened when an earthquake occur. They explained that, during earthquakes, the ground shakes in all directions and generates inertia forces that the construction material should be able to withstand. Since the compression strength of adobe is much higher than its tensile strength, significant cracking starts in the regions subjected to tension. Seismic forces perpendicular to the walls produce out-of-plane bending, and cracking starts at the lateral corners of the walls, where the tensile stresses are higher. Large vertical cracks that separate the walls from one another are thus produced (Fig. 3). Front walls are usually the first to collapse in an earthquake, overturning onto the adjacent street. Lateral seismic forces acting within the plane of the walls generate shear forces that produce diagonal cracks, which usually follow stepped patterns along the mortar joints. The diagonal cracks often start at the corners of doors and windows, due to the stress concentration at these locations (Figure 3). If the seismic movement continues after the adobe walls have cracked, the wall breaks in separate pieces, which may collapse independently.

These characteristics, called the four virtues of

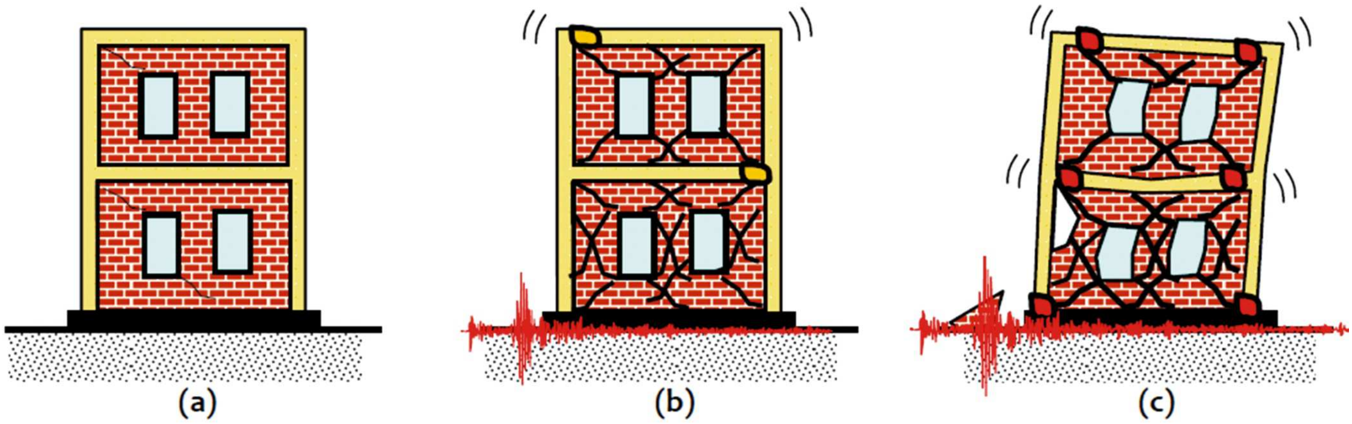

Fig. 3: Earthquake-Resistant Design Philosophy for buildings: (a) Minor (Frequent) Shaking –No/Hardly any damage, (b) Moderate Shaking – Minor structural damage, and some non-structural damage, and (c) Severe (Infrequent) Shaking – Structural damage, but NO collapse (Murty et al. 2012).

earthquake-resistant buildings, are:

1. Good seismic configuration, with no choices of architectural form of the building that is detrimental to good earthquake performance and that does not introduce newer complexities in the building behaviour than what the earthquake is already imposing;
2. At least a minimum lateral stiffness in each of its plan directions (uniformly distributed in both plan directions of the building), so that there is no discomfort to occupants of the building and no damage to contents of the building;
3. At least a minimum lateral strength in each of its plan directions (uniformly distributed in both plan directions of the building), to resist low intensity ground shaking with no damage, and not too strong to keep the cost of construction in check, along with a minimum vertical strength to be able to continue to support the gravity load and thereby prevent collapse under strong earthquake shaking; and
4. Good overall ductility in it to accommodate the imposed lateral deformation between the base and the roof of the building, along with the desired mechanism of behaviour at ultimate stage. Behaviour of buildings during earthquakes depends critically on these four virtues. Even if any one of these is not ensured, the performance of the building is expected to be poor.

Earthquake shaking is random and time variant. But, most design codes represent the Earthquake-induced inertia forces as the net effect of such random shaking in the form of design equivalent static lateral force. This force is called as the Seismic Design Base Shear VB and remains the primary quantity involved in force-based earthquake-resistant design of buildings. This force depends on the seismic hazard at the site of the building represented by the Seismic Zone Factor Z.

Foundation is a substructure built below the super structure. Purpose of the foundation is to transfer the structural loads safely to the underlying soil. Safe and economical design of a foundation under different loading conditions is the role of geotechnical engineer. Earthquake loads are the most complicated and complex. Design of earthquake resistant foundation is highly challenging. Proper design of a foundation against earthquake loading requires through understanding over the behavior of soil, response of structure and interaction of soil structure under earthquake loading.

Foundation failures in a seismically active area are of two modes: shallow and deep. Detailed explanation on the mode of failures and the causative mechanism of the failures was explained by Jakka (2013).

1) Modes of Shallow Foundation Failures and their Causative Mechanisms

a) Tilting of foundation (overturning of the structure): It's a clear shear (Bearing Capacity) failure of the supporting soil that is due to the following factors: action of inertial forces; reduction in strength of supporting soils (liquefaction) and the occurrence of sand boils and lateral spreading.
b) Ground Instability caused by slope failures (due to flow slides and lateral spreading).
c) Sliding of foundation; sliding that may occur due to the horizontal inertial forces applied by

an earthquake; sliding that occur due to movement (lateral spreading) of under lying.

d) Soils and settlement of foundations that may be due to compression/consolidation of liquefied soil upon dissipation of excess PWPs and Due to occurrence of sand boils and lateral spreading.

2) Modes of Deep Foundation Failures and their Causative Mechanisms

a) Overturning of the structure due shear failure at pile heads and pile cap caused by action of inertial forces on the piles;

b) Breaking/shear failure of piles in the liquefied soil layer caused by lateral spreading of liquefied soils, which increases bending stresses (Tokimatsu et al., 1998); caused by reduction in lateral confinement due to liquefaction of soils, which can lead to buckling of piles (Bhattacharya, 2007);

c) Settlement/failure of foundation may occur due to reduction in the shear strength of liquefied soils if it is not considered in the initial design and due to occurrence of sand boils and lateral spreading.

Pender (2007) posited that, the seismic design of foundations for structures depends on dynamic bearing capacity, dynamic settlements and liquefaction susceptibility of soil. Shallow foundations may experience a reduction in bearing capacity and increase in settlement and tilt due to seismic loading as has been observed during several earthquakes. The foundation must be safe both for the static as well for the dynamic loads imposed by the earthquakes. Earthquake associated ground shaking can affect the shallow foundation in a variety of ways:

(1) Cyclic degradation of soil strength may lead to bearing capacity failure during the earthquake.
(2) Large horizontal inertial force due to earthquake may cause the foundation to fail in sliding or overturning.
(3) Soil liquefaction beneath and around the foundation may lead to large settlement and tilting of the foundation.
(4) Softening or failure of the ground due to redistribution of pore water pressure after an earthquake which may adversely affect the stability of the foundation post-earthquake (Chaudhury and Rao, 2005).

Foundation design depends on the several factors like site location and conditions, soil parameters and nature of applied loads on the foundation. The foundation must be safe which can be ensured by meeting the design criteria. Foundation must be safe for the static condition as well as for the seismic condition. The information on seismic design of shallow foundations is presented below for four different cases:

(1) Shallow Foundations on Soils Not Prone to Liquefaction.
(2) Settlement of Shallow Foundations on Soils Not Prone to Liquefaction.
(3) Shallow Foundation on Soil Prone to Liquefaction.
(4) Settlement of Shallow Foundations on Soil Prone to Liquefaction (Budhu and Al-Karni (1993).

Shallow footings are one of the most common types of foundations used to support mid-rise buildings in high risk seismic zones. Recent findings have revealed that the dynamic interaction between the soil, foundation, and the superstructure can influence the seismic response of the building during earthquakes. Accordingly, the properties of a foundation can alter the dynamic characteristics (natural frequency and damping) of the soil-foundation-structure system.

Soil-structure interaction

Soil-structure interaction is the term commonly used to describe the effect of the foundation and soil on the behaviour of the structure. In general, there are three principal effects:

1. The soil-foundation system will have some mass and flexibility in sliding and rocking that will affect the natural periods and mode shapes of the building.
2. Vibration energy from the structure can be transmitted into the ground and dissipated by the soil materials (material damping) and by radiation of energy away from the foundation in stress waves (radiation damping). Radiation damping is generally very low for foundation rocking at long periods and is generally ignored for tall buildings.
3. The embedment and stiffness of the foundation will affect the seismic motions transmitted to the building. This is caused by kinematic interaction whereby ground motion waves do not meet the foundation at all points simultaneously and so there is some base-slab averaging of the action over the foundation. This causes reduction in the effective

input for shorter wavelength (shorter period) motions.

Damage during an earthquake results from several factors

- Strength of shaking:
- Length of shaking
- Type of soil
- Type of building
- Resonant frequency of building

Ground Motion Modeling of Building Collapse in Nigeria

According to Adepelumi (2019), simulation of structural collapse is increasingly used to evaluate the seismic safety of building structures, including both applications to the assessment and design of specific structures and, more generally, to assess changes to building code provisions. For collapse assessment, synthesis of ground motions to obtain time histories representative of ground motions that may cause structural collapse is critical. Of particular importance are choices between recorded and synthetic ground motions and scaling of records to obtain ground motions strong enough to cause structural collapse.

Awoyera *et al.* (2016) reported the modelling of the Simulated Combined Earthquake and Dead Load Lateral Resistance Building Systems using earthquake data of 2009 that affected Abeokuta, Ogun State and environs using the earthquake magnitudes of 4.1 to 4.9 based on the previous records of earthquakes recorded by in the area between 2000 and 2009 by the with body wave magnitude of 4.4 occurred in Nigeria and was recorded by some agencies like the International Seismological Centre (ISC), United Kingdom. The model used by Awoyera et al (2016) is a multi-storey steel frame building that was (Figures 4 to 8) and it was subjected to a simulated combined earthquake and dead loads. Standard British steel sections were utilised for beams and columns; which is the adopted system in Nigeria, and they are pinned, such that the design is sufficient to resist the lateral wind loads. For the modelling of the building, a square hollow section brace member of size 250 x 250 x 12.5 was adopted, based on the smallest column size of 254 x 254 x 73 UC and beams of 203 x 133 x 25 UB. A 200 mm thick concrete floor slab exists at every level. The building was assumed not too close to the seismic source.

The result of their findings showed that if an earthquake with magnitudes 4.1 to 4.9 should hit Nigeria in the very near future. The building can be affected by the earthquake (Figures 4 to 8). Any high rate of seismic activity from the source also means that there is a very high tendency that the building, when completed (or during construction), will not take too long before experiencing a seismic activity.

Thus, proper modification of buildings in such area, to withstand seismic activities is substantially important. Necessary information about the building location and the soil condition at the site as obtainable at the study area are presented as follows: stiff soil with a shear wave velocity of 250 m/sec nearest seismic source is 22.5 km from the site, and the fault is capable of producing a large magnitude event and has a high rate of seismic activity. They further posited that, buildings with eccentric bracings are generally better than concentric bracings in dissipating energy under seismic conditions, short link eccentric bracings provide more stiffness than long link eccentric bracings hence in order to quickly achieve the requirements of a maximum deflection of 500 mm or below and a maximum inter-storey drift of two per cent or below, short links bracings were used which require lesser number of elements compared to long link bracing thus leading to a more economical solution.

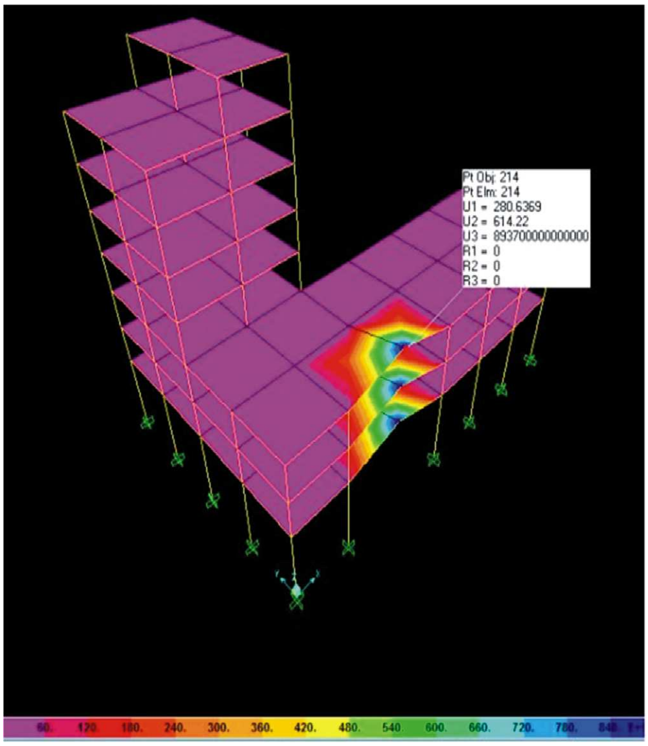

Fig. 4: Displacement of the Structure using a fixed footing

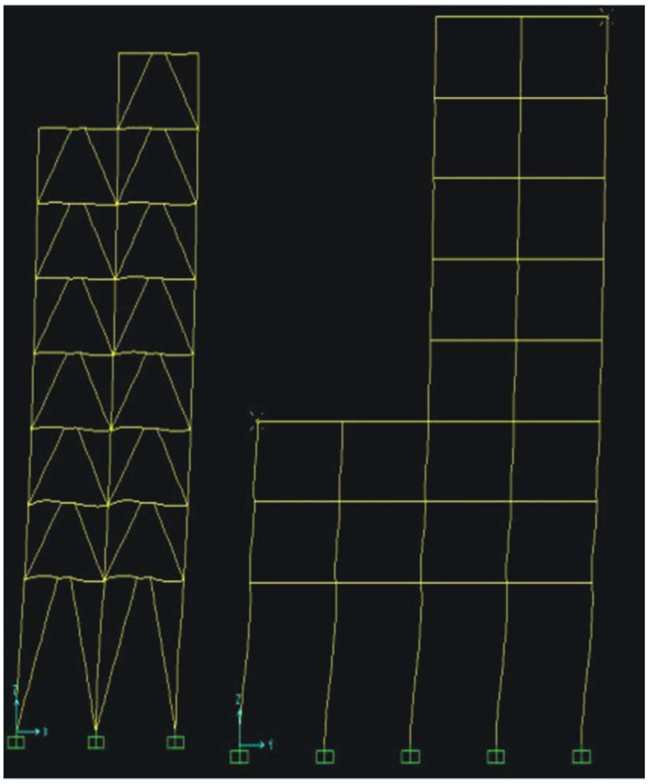

Fig. 6: Deformed shape view of the building with bracing systems after analysis (After Awoyera *et al.* 2016).

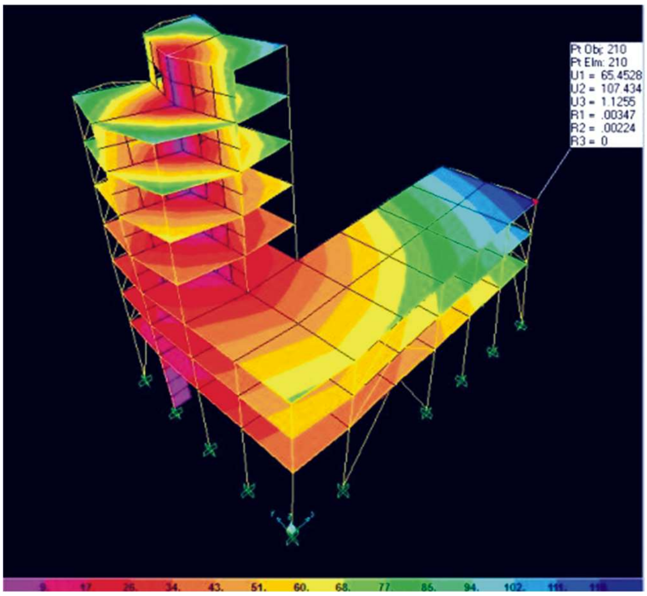

Fig. 7: The displacement result of the long span beam of the modified building using combined shear wall and short link inverted V-bracings (After Awoyera *et al.* 2016).

Case Study of Building Performance During Earthquake

The 2012 final technical Report on the investigation into

Fig. 8a: Deformed shape view of the building with bracing systems after analysis (After Awoyera et al. 2016)

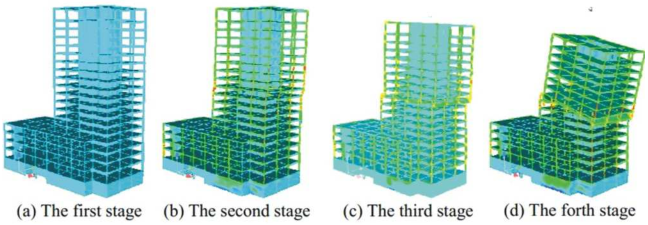

Fig. 8b: Collapse process of the structure at PGA of 4(After Lu *et al.* 2008)

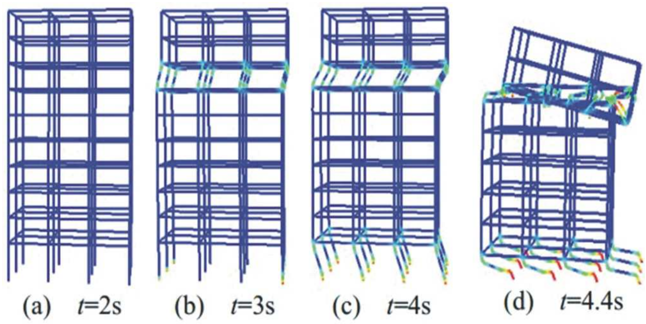

Fig. 8c: Deformation of the initial structure at different time at PGA of 2 (After Lu *et al.* 2008)

the structural performance of Buildings in Christchurch New Zealand revealed that, the on 22 February 2011earthquake delivered forces, particularly vertical, which were amongst the highest in any urban area and resulted in the damage to and collapse of many buildings. Despite the level of ground shaking, many multi-storey buildings in Christchurch came through with damage but did not collapse, enabling people to escape. The recorded values of peak vertical accelerations, in the range of 1.8 and 2.2 times gravity (1.8g and 2.2g) near the epicentre, were amongst the highest ever recorded in an urban environment. The observation of the resistance of building to earthquakes (foreshocks, main-shocks and aftershocks) attest to the importance of building earthquake resistant building in the 21st century.

Expert Panel finding of the Effects of the Earthquake on the Buildings are as Follows as Specified in the Christchurch-Final Report (2012):

1. *Estimating building response:*
The estimated responses of buildings to recorded ground shaking in the Christchurch CBD on 22 February 2011 are shown, in most cases, to be significantly greater than those used in 2010 as a basis for the design of new buildings of the type in the investigations. The investigations highlighted the variability and uncertainty involved in estimating building response from ground shaking measurements. The earthquake was shallow and very close to Christchurch City, so the intensity of ground shaking in this event (as indicated by the response spectra) was much higher than in the Darfield event.

2. *Vertical Accelerations:*
The vertical accelerations measured in the 22 February 2011 aftershock were exceptionally high and may have contributed significantly to vertical forces in columns and walls. The extent of this contribution is generally difficult to quantify, but analyses of the CTV Building indicated that vertical accelerations could have reduced the capacity of critical columns to sustain lateral displacements by around 15 to 35% depending on concrete strength.

3. *Duration of Shaking:*
The duration of the 22 February 2011 aftershock was relatively short. A longer duration earthquake is likely to have had a greater effect on buildings, especially on structures that are not well tied together or are not properly detailed in their critical connection regions. It is important that the implications of longer duration shaking be better understood, in particular when assessing the earthquake performance of existing buildings. The availability of extensive ground motion records and information on modern building performance offers an opportunity to improve such understanding and revise current assessment/design /retrofit methodologies.

4. *Seismic Hazard Coefficients for Building Design:*
The logic of consideration of large infrequent earthquakes within a uniform risk environment should be re-examined. In particular, the basis for determining seismic hazard coefficients for building design needs to be looked at. Consideration of the consequences of a large earthquake occurring in or near a major urban centre, including the national economic impact, should also be investigated.

Observed Effects of Recent Earthquake on Buildings in Nigeria

Adepelumi (2019) showed that the 2016 earth tremors that were recently experienced in some parts of Oyo (Saki), Bayelsa, Rivers, and Kaduna states (Kwoi town and adjourning villages) between February and September revealed that several vibrations (foreshocks, main-shocks and aftershock) accompanied the earth tremors resulted in the collapse of mud houses and infliction of visible cracks in modern buildings within the affected areas. Saki town in Oyo State first experienced series of earth tremor between May and June 2016. Over twenty houses were affected by this event. Likewise, earth-tremor occurred in Bayelsa State in on 15th and 30th July, 2016. The impacted communities are Igbogene and Akenfa in Bayelsa as well as Akinima, Akie-Oniso (Oruama), One Man Country and Mbiama (neighbouring communities in Rivers). Similarly, the National Emergency Management Agency (NEMA) reported that over 300 houses were affected by the earth tremor that hit Kwoi, Jabba Local Government Area of Kaduna State in September, 2016 (Figures 9 to 12).

During our field visit to the affected areas, we observed that hundreds of building structures settled, slid, and collapsed during the earth tremor. Classification of damage to buildings was based on the European Macro-seismic Scale (EMS98). For this analysis, the degree of structural damage to the buildings were described using a system proposed by Coburn and Spence (1992), where each building is assigned a structural damage index ranging from D0 (no observed damage) to D5 (complete collapse of the building or a story within the building).

For the Bayelsa scenario, 50 buildings were observed to have physical damage with cracks. 90% of the buildings surveyed (45 buildings) showed evidence of significant structural damage (D3 to D4), while 10% (5 buildings) exhibit low structural damage (i.e. D0 and D1). However, no partial and complete collapsed of buildings was observed. Cracks are common occurrence in the buildings. It is deduced that the observed cracks developed in the buildings or sections of a building whenever stress in the component exceeds its strength. Stress in the building component may be caused by externally applied forces such as dead and live load or foundation settlement or it could be induced internally by thermal variation, moisture changes, chemical actions etc. (Bonshor and Bonshor, 1996).

Six different types of cracks were observed on the damaged buildings, and they are classified as: Aesthetic cracks, Progressive cracks, Diagonal crack on walls, Curvilinear cracks on walls, Horizontal cracks on walls and Vertical cracks on walls. The cracks within the buildings vary in width from 0.5mm to 10 mm; while the length of the cracks varies from 250mm to 1800 mm. Generally, these cracks follows patterns characteristically associated with flexural and shear failure (BRE, 1991).

Conversely, for the Kaduna scenario, 300 buildings were surveyed. 80% of the buildings surveyed (240 buildings) exhibited minor to (D2), while 15% (45 buildings) exhibited none to low structural damage (i.e. D0 and D1), and 5% (15 buildings) showed evidence of significant structural damage (D3 to D4) and exhibited complete or partial collapse (D4 and D5) . The structural damage appears to concentrate along the fault lines where most building underwent minor to moderate ground failure.

However for the Saki, Oyo State scenario, 100 buildings were surveyed. 73% of the buildings surveyed (73 buildings) exhibited none structural damage (D0), 20% (20 buildings) exhibited minor to low structural damage (D1 and D2), and 7% (7 buildings) exhibited complete or partial collapse (D4 and D5).

Fig. 9: Saki, Oyo State

Fig. 10: Kwoi Local Government, Kaduna State

Fig. 11: Bayelsa State

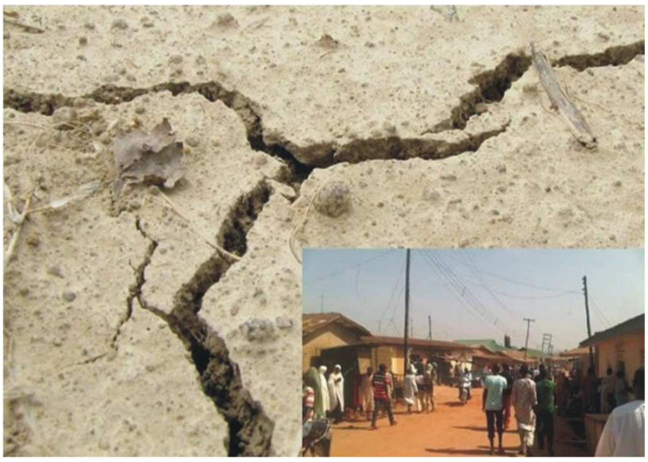

Fig. 12: Kwoi Local Government, Kaduna State

It is pertinent to note that the Atlantic fracture system passed through the subsurface of some of the villages and towns affected by the recent seismicity. The fault transcends the South-West and North-West of the country, thereby making that stretch of land susceptible to seismicity due to stresses generated within the earth-crust, that is, partial reactivation of the old faults (Adepelumi *et al*, 2008).

A preliminary assessment of all the 2016 earth tremors in the four states gave a focal depth 10 km within the upper crust with a stress drop of about 0.115 bar and a 1.8 km radius rupture. Considering the areas where the tremor was felt, the rupture process propagated up to north-east to south-west direction. The majority of the fault plane solutions for the rupture process suggested a normal dip-slip mechanism with a local magnitude (M_L) of 2.9 to 3.1. An analysis of the series of the event provided a normal fault mechanism with median solution of strike 180°, dip 45° and rake -90°.

The structural damages observed in this study occurred in limited area and does not cover the entire villages and towns investigated. This is probably due to the lower focal depth of the seismic activity. The maximum peak ground acceleration (PGA) of the records is found to be around 0.06 to 0.15g. In fact the PGA value is low compared to the value 0.16 to 0.69g predicted from modelling. This result suggests that the damaged buildings, whether they are masonry or concrete, are those which have very low seismic load capacity. This is an indicator of the poor seismic capacity of the buildings investigated. However, the perceived satisfactory performances of the undamaged buildings do not necessarily confirm that these buildings have adequate seismic safety required for standard building (Figures 9 to 12).

Concluding Remarks

Nigeria has witnessed several low to moderate earthquakes in the last twenty years causing some damages to building and partial to total collapse in some case. The prevalent moderate earthquake hazard, large exposure and moderate vulnerability indicate that urgent action is necessary in the country. It is time to assign value to the life of each Nigerian and take urgent proactive actions that would lead to the establishment of the Nigerian seismic codes and the incorporation of this code into the Nigerian building codes in order to reduce earthquake risk in the very near future. For the sustainable development of a developing nation in LTM seismic region where the rural to urban migration is very high, thereby leading to concomitant high demand for land for building construction, it is expedient to estimate the earthquake risk of housing, earthquake hazard, earthquake vulnerability and housing threat factor of a house before and after constructing buildings in such area. In terms of seismic vulnerability, the Nigeria is classified into maximum (sedimentary), moderate (meta-sediments) and low damage potential (basement rocks). Therefore, buildings in these areas should be designed to resist such high ground acceleration. The seismic ground motion parameters computed is useful to future urban planning development, foundation design and structural design of buildings in Nigeria.

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Evaluation of Subsoil Competence for Foundation Applications in Alagbaka-Extension, Akure, Southwestern Nigeria

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Abstract

The subsoil of Alagbaka-extension, Akure, was evaluated for its competence as foundation materials. Twenty six (26) disturbed soil samples were collected from different pits at a depth of about 2m. The samples were analyzed for natural moisture content, grain size analysis, Atterberg limits, standard compaction, un-confined compressive strength and consolidation tests. The laboratory tests results revealed that moisture content ranges from 6.1% to 18.5%, liquid limit from 25.1% to 48.6%, plastic limit from 17.6% to 29.5%, linear shrinkage from 2.9% to 7.7%, and plasticity index from 5.2% to 19.5%. The grain size analysis showed that the amount of fines ranges from 20.6% to 47.4%. Soil classification with AASHTO revealed that almost all soils from the area underlane by migmatitic gneiss are clayey while those from area underlane quartzites classified as silty or clayey gravel and sand. The specific gravity ranges from 2.65 to 2.8, maximum dry density from 1820Kg/m3 to 1998Kg/m3 and optimum moisture content from 13.2% to 19.7%. The shear strength varied from 74.1kPa to 103.8kPa and Coefficients of consolidation range from 0.02019 to 0.02944. The competence map segregated the study area into extremely highly competent zone, very highly competent zone, highly competent zone and moderately competent zone. It is hereby concluded that (i) generally, the subsoil of the area are competent; (ii) the competence map revealed a zonation; and (iii) the lithology of the underlying rock and the soil group are closely related to soil competence.

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

Introduction

Alagbaka extension in the Akure metropolis, is witnessing various structural and civil engineering developments ranging from the building of five star hotels to high rise buildings, and to road construction. The construction of structures, especially high rise buildings, require an adequate knowledge of the subsurface. Information are often needed on the configuration of the subsurface layers, the nature/competence of the subsoil, the bedrock topography and its structural disposition to enable foundation to be designed and located to suit the variable character of the bedrock (Olorunfemi 2008). Where construction works have been carried out without taking cognizance of the sites' geological, geomorphological and groundwater conditions, failure of foundations and by extension the super-structures often occur (Griffiths, 1999; Olorunfemi *et. al,* 2004; Olorunfemi *et. al,* 2005). The development of a subsoil competence map for the Alagbaka extension can therefore be a very useful initial discrimination tool for short and long term planning.

Subsoil strength investigation traditionally involves soil boring (auger or cone penetration), and soil sample testing for engineering properties such as grain size distribution, natural moisture content, specific gravity, hydrometer analysis, plasticity characteristics, bearing capacities and consolidation/compressibility characteristic determinations (Adeyemi and Oyediran, 2004; Schneider *et. al,* 1999).

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

Geology

General Geology

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

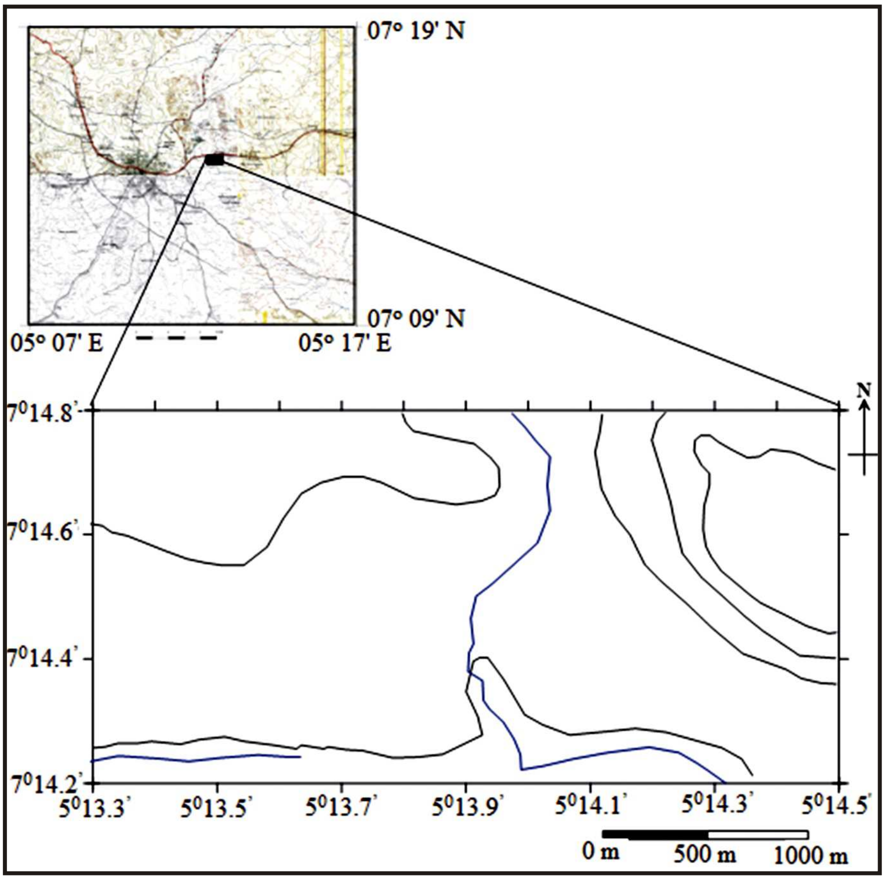

Fig. 1: Base Map of Alagbaka Extension. (Digitalized from an Excerpt of Akure S.W Topographical Map Sheet 265 [Federal Surveys Department, 1969]).

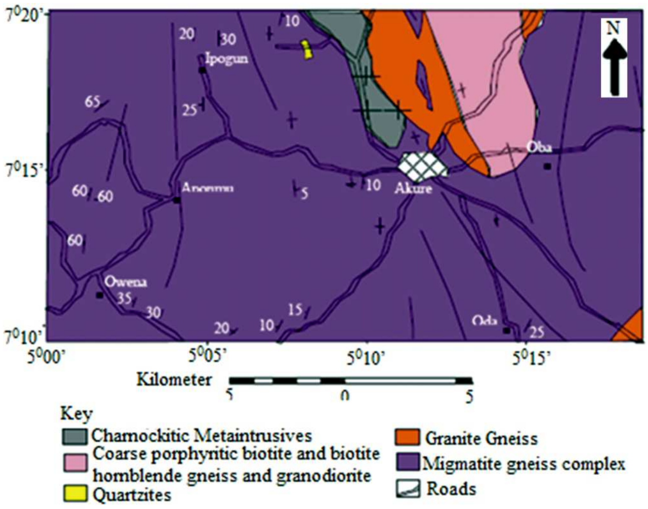

Fig. 2: Geological Map of Akure (Adapted from the Geological Map of Nigeria (Ademeso, 2009)

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

Local Geology

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

Soils

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

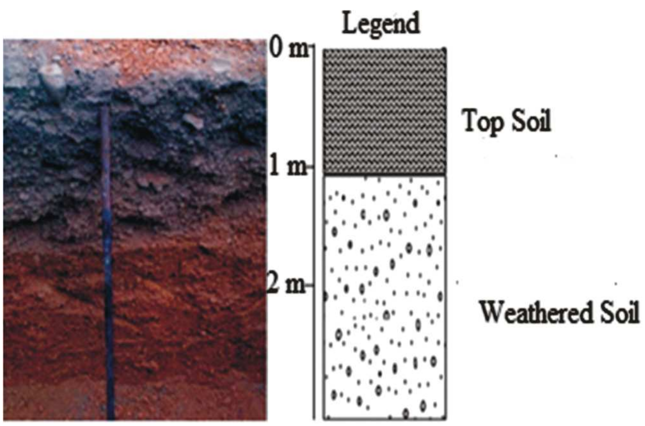

Fig. 3: Typical Soil Profile of the Study Area.

Methodology

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

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

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

This study therefore intends to integrate the competence maps generated, with the aid of Surfer, from the results of compaction, plasticity index, consolidation and shear strength tests to generate subsoil competence map for Alagbaka extension. The American Association of State Highway and Transportation Officials (AASHTO) was adopted for soil classification.

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

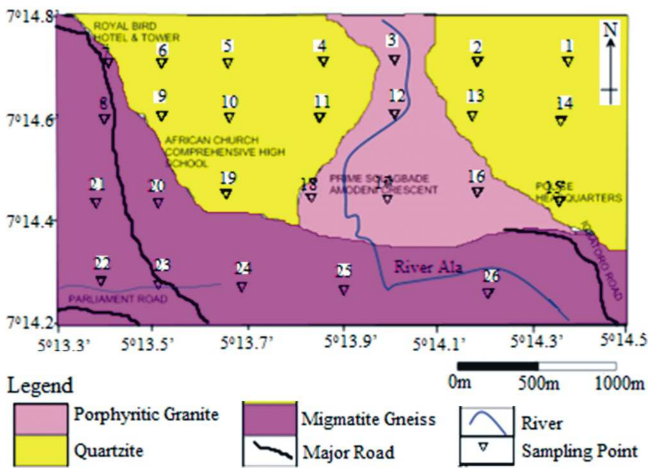

Fig. 4: Geological Map of the Study Area Showing the Soil Sampling Points

Results and Discussion

Engineering Geotechnical Investigation

The moisture content obtained for the soil samples range from 6.1% to 18.5% (Table 2). The low moisture content corresponds to low compressibility potential (Ademeso *et. al*, 2016). This is in tandem with the moisture content characteristics of a good engineering soil. The specific gravity values of the tested soils range from 2.65 to 2.80 (Table 3). An increase in specific gravity has been found to be associated with a decrease in voids ratio (Schneider *et. al,* 1999, Ademeso *et. al,* 2012). The tested soil samples have varying amounts of fines ranging between 26.1% and 47.4% (Table 4). These show that the soils are generally matured residual materials with sufficient binders for the coarse constituents to attain high shear strength. The curve of the plot indicates that the soils are generally well-graded (Fig. 5). The plastic limit values range from 17.6% to 29.5% while the plasticity index range from 5.2% to 19.5%. Since the AASHTO employs the grain size distribution as well as the Atterberg limits for the classification of soils, they are therefore classified as in Tables 4 and 5. This soil groups from AASHTO classification revealed that almost all soil samples from the area underlane by migmatitic gneiss fall into the clayey class while those from the area underlane by quartzites classified as silty or clayey gravel and sand. Results of consistency limit tests are as shown in Table 5. They generally fall within the values obtained for soils within the Basement Complex and can therefore be used as foundation materials (Federal Ministry of Works and Housing, 1972).

The Optimum Moisture Content (OMC) ranges between 13.2% and 19.7% while the Maximum Dry Density (MDD) ranges between 1820kg/m^3 and 1995kg/m^3 (Table 6). The best soils are those with high MDD at low OMC. The moisture-density relationship of the soils as indicated shows that the MDD's are relatively moderate. A comparison of the values with compaction characteristics shows that the soils fall within fair to good class (Federal Ministry of Works and Housing, 1972).

The coefficient of compressibility (M_v) for the consolidation at various overburden pressures ranges from 0.18349 to 0.44142 in line with M_v > 0.15 determined for Ubima soils by Nwakwoala and Warmate (2014). The coefficients of consolidation (C_v) at various overburden pressures also ranges from 0.01060m^2/yr. to 0.01546m^2/yr (Table 7) which is comparable with the result determined for normally consolidated kaolin (Mayne, 2016). Terzaghi and Peck (1967) reported a C_v of 0.02 + 0.01 for granular soils including rock fills confirming the reliability of the test result. The unconfined compressive strength value vary from 148.3kPa to 207.6kPa while the shear strength values range from 74.1kPa to 103.8kPa (Table 8). All these confirm that the soils have a high soil bearing capacity. The overall results obtained revealed that the material has good engineering qualities for foundation and other construction purposes.

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

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

Subsoil Competence Map

Thematic maps indicating the competence of the soils were generated using the values of maximum dry density, plasticity index, compression index and shear strength (Fig. 6-10). The integration of these maps produced a subsoil competence map for the Alagbaka extension. The superimposition of the competence maps generated from the parameters (compaction, consolidation, Atterberg limits and unconfined

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

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

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

Sample Number	% Clay	% Silt	% Sand	% Gravel	Plasticity Index (%)	Liquid Limit (%)	Soil Group	Soil Name
1	22.8	9.5	64.6	5.1	11.3	32.1	A-2-6	Silty or clayey gravel and sand
2	31.0	8.1	58.3	2.6	11.2	35.6	A-6	Clayey Soil
3	20.9	5.8	58.8	14.5	7.6	25.2	A-2-4	Silty or clayey gravel and sand
4	24.9	7.6	56.7	10.8	5.2	27.9	A-2-4	Silty or clayey gravel and sand
5	24.0	9.1	56.1	10.7	6.8	30.0	A-2-4	Silty or clayey gravel and sand
6	22.0	10.3	58.0	9.7	6.7	28.7	A-2-4	Silty or clayey gravel and sand
7	27.6	7.4	47.9	3.3	10.3	35.2	A-2-4	Silty or clayey gravel and sand
8	38.7	8.7	51.0	1.7	18.1	47.6	A-7	Clayey Soil
9	21.7	10.4	58.6	9.3	7.0	28.9	A-2-4	Silty or clayey gravel and sand
10	23.4	9,6	60.4	6.7	6.4	29.5	A-2-4	Silty or clayey gravel and sand
11	23.9	8.5	58.5	9.2	8.5	31.2	A-2-4	Silty or clayey gravel and sand
12	18.3	7.7	59.5	14.5	7.3	25.1	A-2-4	Silty or clayey gravel and sand
13	27.4	11.8	49.9	10.9	11.1	37.2	A-6	Clayey Soil
14	19.9	7.9	60.8	11.4	7.9	26.2	A-2-4	Silty or clayey gravel and sand
15	20.6	8.0	57.2	10.6	8.1	29.3	A-2-4	Silty or clayey gravel and sand
16	25.5	8.1	52.5	9.2	8.7	31.8	A-2-4	Silty or clayey gravel and sand
17	20.6	8.0	57.2	3.9	19.5	48.6	A-2-7	Silty or clayey gravel and sand
18	24.5	9.2	56.6	9.8	7.1	30.7	A-2-4	Silty or clayey gravel and sand
19	22.4	7.9	60.8	8.9	8.2	29.7	A-2-4	Silty or clayey gravel and sand
20	38.5	7.0	51.3	3.2	19.1	48.1	A-7-5 or A-7-6	Clayey Soil
21	39.6	7.0	51.3	2.0	19.2	48.1	A-7-5 or A-7-6	Clayey Soil
22	30.1	9.0	57.0	3.4	11.7	38.6	A-6	Clayey Soil
23	36.2	11.2	50.1	3.9	18.1	47.1	A-7-5 or A-7-6	Clayey Soil
24	35.6	11.4	51.0	4.1	17.3	46.6	A-7-5 or A-7-6	Clayey Soil
25	28.8	9.8	46.7	5.6	12.6	36.6	A-6	Clayey Soil
26	27.4	11.8	49.9	10.9	8.0	28.3	A-4	Silty Soil

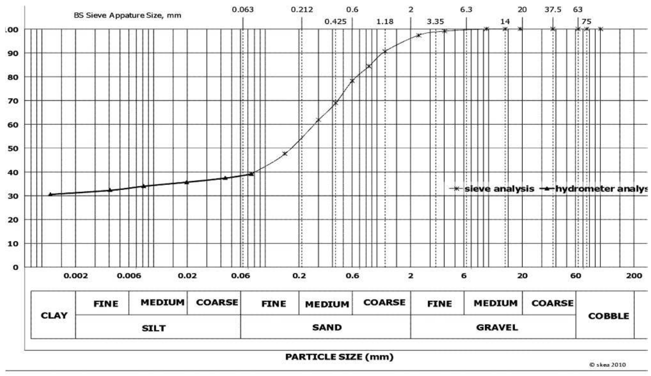

Fig. 5: Typical Grain Size Distribution Curve of the soils of the Study Area.

compressive strength) segregated the study area into extremely highly competent zone, very highly competent zone, highly competent zone and moderately competent zone. This is also found to follow the same trend as earlier observed with the groupings from AASHTO classification. The subsoil competence map was finally composed using overlay function to combine all the layers (Fig. 10).

Conclusion

Soil classification with the aid of AASHTO revealed

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

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

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

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

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

Sample Number	Coefficient of Compressibility (A_v)	Coefficient of Vol. Compressibility (M_v)	Compression Index (C_c)	Swelling Index (C_s)	Coefficient of Consolidation (C_v) (m^2/min)
1	0.33648	0.24911	0.0456	0.01919	0.02684
2	0.51812	0.32256	0.0699	0.03159	0.02413
3	0.29904	0.18978	0.0409	0.01543	0.02919
4	0.37069	0.25124	0.0503	0.02119	0.02676
5	0.36620	0.24804	0.0497	0.02086	0.02688
6	0.36468	0.24911	0.0495	0.02080	0.02684
7	0.38034	0.27808	0.0515	0.02238	0.02575
8	0.69695	0.43127	0.0936	0.04484	0.02051
9	0.36234	0.24697	0.0492	0.02061	0.02692
10	0.38016	0.25552	0.0515	0.02184	0.02660
11	0.37180	0.24911	0.0504	0.02120	0.02684
12	0.28722	0.18349	0.0393	0.01460	0.02944
13	0.49869	0.31929	0.0673	0.03034	0.02425
14	0.32693	0.20135	0.0446	0.01730	0.02872
15	0.32454	0.20874	0.0442	0.01742	0.02842
16	0.41274	0.26195	0.0559	0.02389	0.02635
17	0.70993	0.44142	0.0953	0.04584	0.02019
18	0.39359	0.26410	0.0533	0.02283	0.02627
19	0.34087	0.22780	0.0463	0.01889	0.02767
20	0.69511	0.42676	0.0934	0.04465	0.02065
21	0.65582	0.42564	0.0881	0.04211	0.02068
22	0.50331	0.32256	0.0679	0.03069	0.02413
23	0.64421	0.41329	0.0866	0.04117	0.02108
24	0.64364	0.40992	0.0865	0.04108	0.02119
25	0.50448	0.31711	0.0681	0.03064	0.02433
26	0.52418	0.32365	0.0707	0.03198	0.02410

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

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

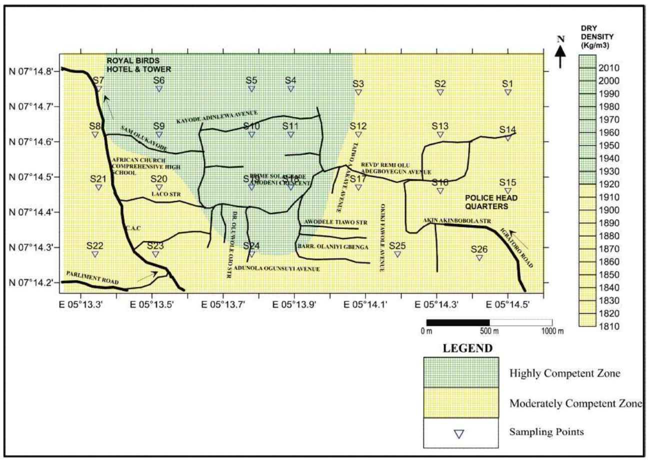

Fig. 6: Map of the Study Area showing the soil Competence generated from the Maximum Dry Density (MDD) Values.

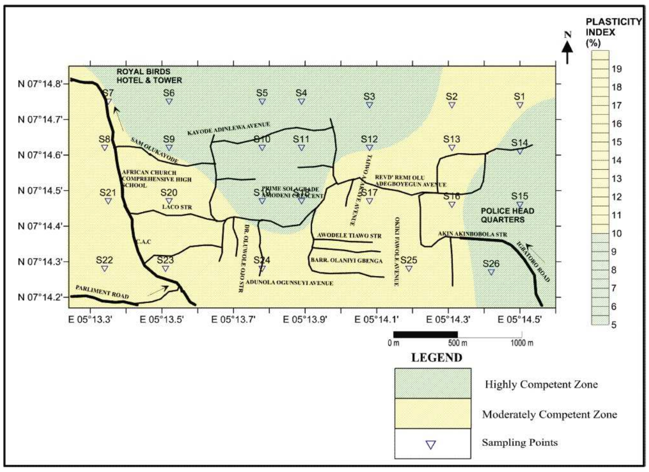

Fig. 7: Map of the Study Area showing the soil Competence generated from the Plasticity Index (PI) Values.

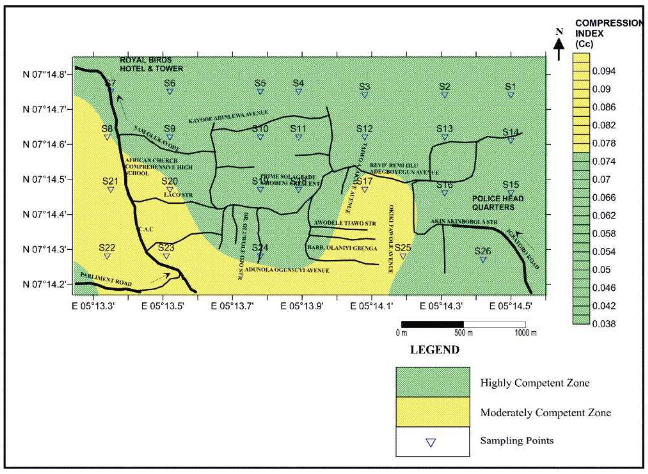

Fig. 8: Map of the Study Area showing the soil Competence generated from the Compression Index (Cv) Values.

that almost all soils sampled from the area underlane by migmatitic gneiss fall into the clayey class while those sampled from the area underlane by quartzites classified

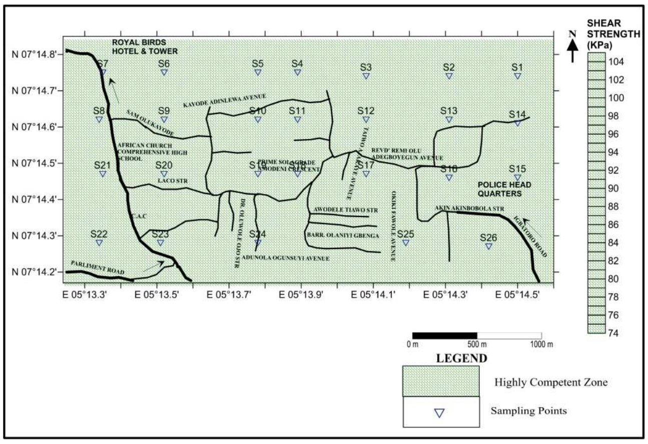

Fig. 9: Map of the Study Area showing the soil Competence generated from the Shear Strength Values.

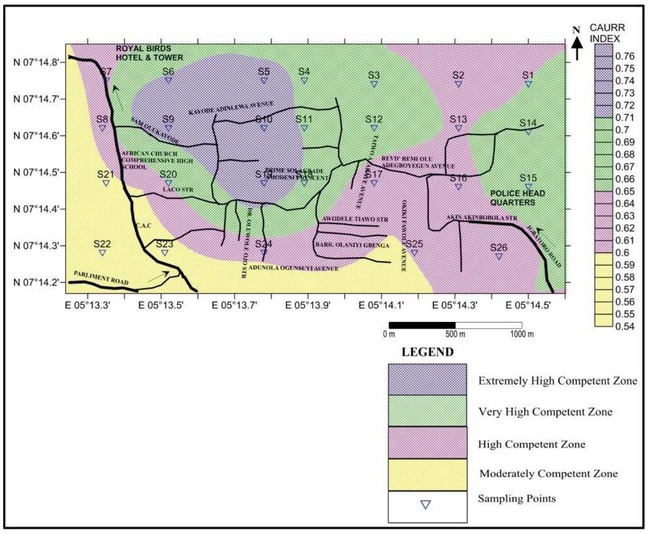

Fig. 10: Map of the soil Competence of the Study Area generated from the Superimposition of Competence Maps of MDD, PI, C_v. and Shear Strength values.

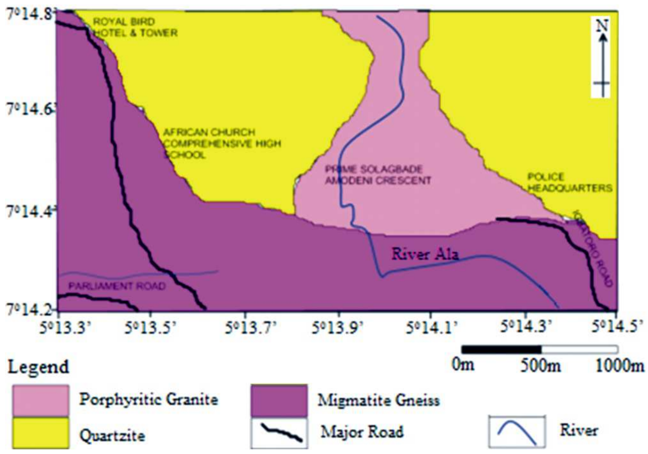

Fig. 11: Geological Map of the Study Area.

as silty or clayey gravel and sand. The values of the coefficient of compressibility, coefficient of volume compressibility, compression index, swelling index and coefficient of consolidation are generally higher in soils derived from the areas underlain by migmatitic gneiss implying less competence. This tally's reasonably with the observation from AASHTO classification indicating that a good relationship exists between competence of soil and the soil group as well as the soil parameters. In case of this work, the silty or clayey gravel and sand is considered to be most competent. The competence map of the area segregated the study area into extremely highly competent zone, very highly competent zone, highly competent zone and moderately competent zone (Figures 10). It is observed by comparing figures 10 and 11 that the competence map tends to follow the pattern of the geology of the area. Furthermore, the competence map is also found to follow the same trend as the AASHTO classification. All the results indicate that generally, the subsoil materials have a high soil bearing capacity. They further revealed that the subsoil materials are suitable for foundation and other construction purposes as they possess good engineering qualities.

It is hereby concluded that:
(i) generally, the subsoil of the area are competent;
(ii) the competence map revealed a zonation; and
(iii) the lithology of the underlying rock and the soil group are closely related to soil competence.

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Environmental Geological Mapping of Coastal Hazards

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Abstract
The coastal region, considered to extend from the continental shelf edge landward to the limit of tidal incursion, is an amalgamation of sub-ecosystems characterised by distinct evolutionary histories and morphodynamic patterns. The sub-ecosystems show time- and space-varying sensitivity scaling and carrying capacities when subjected to natural and man-induced hazard-culminating perturbations. Environmental geological mapping of coastal hazards calls for in-depth knowledge of the above as well as their inter-relationships with the forcing, in at least a quasi-quantitative sense, if the hazard map is to serve as a management predictive tool. In view of the multiplicity of hazard-causing forcing and factors in the coastal region, a coastal hazard map on a large scale is commonly issue-driven, scenario-specific and time domain-constrained. Coastal geo-hazard mapping typically follows a three-stage process, namely, investigation and identification stage followed by an inventory and interrogation stage and capped by an interpretation and intervention stage. The latter stage defines the utility value of a coastal hazard map, and is enhanced by taking due cognizance of concepts of equifinality, time-lag and continuum as well as contingency planning best practices. The latter will typically incorporate baseline or fair-weather condition, vulnerability ranking, mitigation/coping strategy and enlightenment/awareness campaigns. Instructive case studies of hazard issues from Nigerian coastal sub-ecosystems are presented.

Introduction

The coastal regions of maritime nations worldwide show, in comparison to their upland counterparts, striking similarities. These include high population density, bourgeoning socio-economic activities and opportunities, high pressure for land development, and, regrettably, high rates of environmental degradation. The three most common coastal environmental threats are erosion, flooding and pollution.

The fragility of coastal lowlands globally is orchestrated by the myriad of brief to decadal time-scale forcing to which they are subjected. While aforementioned degradations may proceed on a natural course, human activities on some coasts commonly aggravate their effects to a hazard status. Consequently, the scaling of coastal degradation – in terms of significance and spread – may differ temporally and spatially between coastal regions and also within a given coastal stretch.

Coastal Architecture

The coastal region, from an environmental geological standpoint, extends from the edge of continental shelf landwards to the limit of tidal incursion (approximates brackish water inshore limit). A cross-shore profile of a coastal plain coast will typically incorporate the following sub-ecosystems: shelf, shoreface, delta (fluvial and tidal) and mouth bars, surf zone, barrier-beach, coastal dune, lagoon, estuary/tidal inlet, swamps and tidal creeks as well as tidal rivers.

Thus, the coastal region is a complex amalgamation of sub-ecosystems characterised by distinct evolutionary histories, morphodynamic patterns, and whose sensitivity scaling and carrying capacities under a given natural and man-induced perturbation (and resultant hazard) are expectably non-uniform temporally and spatially. Therefore, a coastal hazard map on a large scale (> 100 km) must of necessity be issue-driven, scenario-specific and time domain-constrained.

This is more so because hazard-causing forces and factors are not always compatible and comparable (action- and impact-wise), and circumstances operating at different times may aggravate or ameliorate the hazard incident. For example, the degradation of a sensitive swamp sub-ecosystem under a scenario of a nearshore oil spill will depend on the tidal stage. Ebb tidal stage will oppose oil spill reaching the swamps through their connecting tidal channels hence low impact, unlike the flood stage counterpart.

Coastal Hazard Mapping

A coastal hazard map is a simplified, geo-referenced, portrayal of hazard attribute(s) of a coastal geo-space enabling vulnerability ranking in space and time. It may be 2D or 3-D in form. The map attributes may be pre-cursor signatures (e.g. infiltration rates) or direct imprints (e.g. overwash deposits) of a given hazard event. While dynamics is an innate character of coastal regions (Antia, 1987, 1989), the driving forces of their dynamics and degradation may be subtle or catastrophic, periodic or episodic, local or global in nature. This espouses the complexity and challenge of coastal hazard mapping.

The usefulness of coastal hazard map as an environmental management tool derives from its predictive potentials, which must be backed by sound scientific data from monitoring, measurement and modelling programmes. The take-off point will usually entail knowledge of a baseline/background scenario.

Information captured and conveyed in a map of a coastal hazard, as indeed of any coastal map, must be reliable and reproducible within a given dynamic domain. Furthermore, balance must be made between hazard pre-cursor attributes and their post-hazard counterparts to be portrayed in a coastal hazard map. In situations demanding representation of multiple attributes, or creating and analysing scenarios, geographic information system offers the needed flexibility.

Environmental geological hazard mapping dictates that beyond the ability to discriminate between multiple-acting hazard driving forces and factors, there must be adequate scientific knowledge of attributes being mapped in addition to the character of the environment itself.

Coastal Hazard Nature

Coastal hazard is a single or multiple event-induced disequilibrium condition within the coastal region that is inimical to life, living, and sustainable livelihood of man and biota and optimal functioning of the environment. Hazards come in different forms and scales, both temporal and spatial. Physical or geo-physical, chemical and biological hazards are the natural varieties commonly identified in the coastal region, all of which are of concern to geo-hazard scientists and the field of vulnerability science.

Coastal Hazard Mapping Imperatives

Coastal geo-hazard mapping typically follows a three-stage process, namely, investigation and identification stage followed by an inventory and interrogation stage and capped by an interpretation and intervention stage.

Investigation and Identification Stage

This is the setting to determine the hazard issues of interest and concern, and identify which coastal sub-ecosystem is a primary target, especially at a regional mapping scale. At this stage the spatial scale as well as attributes for mapping are resolved as well as the mapping logistics. The latter commonly include mapping platform, duration and monitoring frequency, personnel roles, laboratories and equipment specifications and budget matters.

Inventory and Interrogation Stage

This is the stage where all collated data are processed and queried as to their reliability as indicators of a hazard event. Statistical correlation and model development to validate field observations are also undertaken. In some instances such as where human incursion is pronounced, field experiments and simulations all aimed at reconstructing the hazard pathway and impacts are conducted.

Interpretation and Intervention Stage

This is the stage where the results of the mapping are presented and explanations offered. This latter stage defines the utility value of a coastal hazard map. Great caution is needed in extrapolating the results of a given location to others without drawing from concepts of equifinality, time-lag and continuum.

One of the mysteries of coastal change is that a given outcome may arise from different processes which define the concept equifinality. For example, rip currents in the beach-surf zone coastal sub-ecosystem which are noted as the most frequent killer of beach-goers especially in the US (Houser et al., 2017), has multiple generating mechanisms. Erroneous diagnosis of a hazard driving force can also imply failure in proposed intervention measures.

Another fatal mistake in interpretation of some mapping result is to attempt to relate or correlate the hazard outcome to instantaneous driving processes as against the antecedent counterpart. The delayed response or non-synchronisation between a hazard outcome and its forcing is referred to as concept of time-lag. Again any intervention design criterion that ignores this fact is bound to be erroneous.

A final pitfall of extrapolating mapping results to other locations is underscored by the fact that there is geo-diversity to every coastal sub-ecosystem. Consequently, the response of a given coastal sub-ecosystem to a given hazard may vary depending on its stage in a geo-diversity spectrum. The existence of sequential stages of dynamics or development is what the continuum concept dictates. A classical example is seen in an estuary whose circulation pattern may vary in relation to time-varying ratio of river discharge to tidal prism.

For purpose of intervention, a coastal hazard map should, in the light of baseline data, support development of contingency planning best practices. A major feature of contingency planning is vulnerability ranking of the geo-space of a sub-ecosystem to the given hazard. In the case of a beach inundated by oil spill particulates, the geo-space such as breaker zone, foreshore, berm and backshore may show different vulnerabilities depending on the morphodynamic state of the beach in the beach state continuum (Antia, 1993).

A coastal hazard map should offer instructions on avoiding and averting a fatal incidence from a hazard event through a time-tested mitigation/coping strategy. For instance, the down-drift divergence of estuarine outflow on the southeastern coast of Nigeria (Antia, 2015) will suggest a poorer surf zone water quality in the down-drift than up-drift if the estuary is polluted. Consequently there is a much higher risk of body infection for swimmers in the down-drift surf zone than in the up-drift surf zone but also during ebbing tidal stage than the flooding stage in the former.

Hazard risk reduction is a practical priority of contingency planning and its success depends largely on public enlightenment and awareness campaigns which authorities managing the coastal region should mandatorily undertake.

Case Studies of Hazards in Coastal Sub-ecosystems

Beach-Surf Zone

Beach vulnerability to most hazard incidents is largely dependent on its stage in the morphodynamic continuum. Examples include susceptibility to rip currents that drown beach visitors, wave run-up events that destroy infrastructure (Fig. 1), and stranding of garbages (Fig. 2).

In relation to the latter category is crude oil particulates commonly called tar balls. These are a nuisance to beach recreational activities and may pose hazard to minors if accidentally ingested. A 1985/86 monitoring study within a 6 km stretch of Ibeno Beach (S.E coast of Nigeria) over 24 fortnightly intervals at 16 stations showed that tar stranding on beach was suppressed under spilling breaking wave conditions and facilitated under plunging and surging counterpart. Based on the beach reflectivity parameter model of Wright and Short (1984), the implication is that on a planar (non-barred) nearshore, a dissipative surf zone fronting a reflective-prone beach (RBDS) will be least polluted for a number of reasons.

Fig. 1: Wave run-up damages on Alfa Beach, Lagos State, Nigeria during a 2012 mid-year storm

Fig. 2: Stranded garbage on the downdrift beach of Ibeno (in proximity to the Qua Iboe River estuary), Nigeria

First, spilling breakers are associated with turbulence over a wide surf zone. This means that appreciable entrained pollutants are likely to be dispersed alongshore under characteristically obliquely breaking waves in the study area. Secondly, the reflective beach fronting such a dissipative surf zone is temporally unstable, meaning high volumetric profile changes will expel the pollutant, which defines the high self-cleansing capacity of such beaches.

By contrast, a reflective-prone surf zone fronting a dissipative beach (DBRS) will be heavily polluted given that such surf zones are narrow, and are dominated by plunging and surging breakers with high energy density favouring wave run-up on the beach and entrained pollutants. The temporal stability of dissipative beaches, being at the end stage of erosion continuum, suggests that the residence time of entrained and stranded pollutants will be appreciably prolonged. This observation confers the beach with its very low self-cleansing capacity.

On a barred nearshore, where wave breaking and reforming are common, the pollution longevity trend may be distorted by presence of single or multiple subaqueous sand bars, bar spacing, and the distance of shoreward-most bar from the shoreline. However, as long as the wave travel distance from the shoreline to the first bar is appreciable for wave reformation, the pollutant stranding pattern will be somewhat comparable if beach and surf zone conditions are RBDS and DRBS.

Mangrove Swamp

Mangrove swamps are well known for their high biological productivity, being nursery grounds for juvenile fishes and bird sanctuaries. However, their low-energy and deposition-prone nature, as well as enriched mud sediments are perfect combination of factors for retaining lethal substances that may deteriorate their functioning as habitat from which humans derive their sea foods.

In an effort to understand swamp enrichment and degradation patterns, a shallow (1.5 m) subsurface geochemical mapping of nine swamp settings along the entire Nigerian coast were conducted by Antia and Baba Santa (2013a,b). Results show 7 geochemical profile types (A-G) to be associated with each of the 15 geochemical parameters evaluated, namely, Mg, K, Ca, Mn, Zn, Fe, Cr, Co, Cu, Ni, V, Pb, Total hydrocarbon concentration (THC), Total organic carbon (TOC) and Total organic matter (TOM).

The three main geochemical vertical profiles were type-A (showing progressive concentration decrease from surface to core depth), type-B (where concentration progressively increased from surface to core depth) and type-C (in which concentration progressively decreased from surface to intermediate depth and thereafter increased to core depth).

In general, vertical profile type-C was the most frequent, suggesting a high influx and low leaching rate model. In none of the geochemical parameters was vertical profile type-E (uniform concentration from surface to core depth) evident. In the absence of a corresponding impact assessment in the above study, the implications of the vertical profile types to coastal hazard mapping can only be exploratory rather than explanatory. Nonetheless, biota assemblages will be affected differently, on the basis of being bottom or burrow dwellers.

However, result of a regional comparison of swamp health based on average concentrations of environmental quality parameters of boring samples, where the Badagry swamps are considered a control indicates marked disparity as shown in Fig. 3. Out of a total 18 environmental quality parameters, 13 were higher in Niger Delta swamps than the Badagry counterpart. The highest concentration ratio of 5 was recorded for the total hydrocarbons as to be expected in areas of crude oil exploration and spillages from pipeline rupture. This fact points to the higher risk of hazard in the swamps of the Niger Delta region.

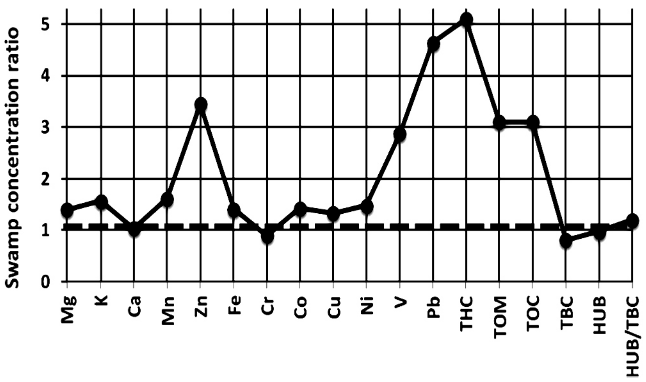

Fig. 3: Niger Delta-Badagry swamp concentration ratio of environmental quality parameters based on borings. (TBC – Total bacterial count; HUB – Hydrocarbon-utilizing bacteria)

Conclusion

Environmental geological mapping of coastal hazards has considerable role to play in protecting life and conserving the ecosystem. However, the process of coastal hazard mapping can not yield meaningful result if conducted in piecemeal and is uncoordinated as is the case in most nations on our continent. Considering the multiplicity of hazard-causing forcing and factors in the coastal region, institutional synergy is inevitable in large scale coastal hazard mapping.

In Nigeria there are many agencies at the Federal and State government levels that can contribute to this effort: the Nigerian Maritime Administration and Safety Agency, Department of Petroleum Resources, National Environmental Standards and Regulations Enforcement Agency, National Oil Spill Detection and Response Agency, Nigeria Hydrological Service Agency, Nigeria Meteorological Agency, National Centre for Marine Geoscience, National Geohazard Research Centre, Nigerian Institute for Oceanography and Marine Research, the National Emergency Management Agency as well as the media.

It is therefore suggested that the Nigerian Association of Engineering Geology and the Environment in collaboration with the Nigerian Academy of Science should constitute the arrowhead for the setting up a Coastal Hazard Research and Coordinating Council at the National and State levels.

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My gratitude goes to the authorities of the Nigerian Geological Survey Agency for funding the coastal geochemical mapping programmes on which this presentation is partly based. I also acknowledge fruitful discussions with my colleagues in the Natural and Human-Induced Hazard and Disaster Working Group of the International Science Council (Africa Regional Office), South Africa.

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Mapping Engineering Geomorphological Changes of the Nigerian Coastline

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Abstract

Among several geomorphological processes responsible for the shape and stability of many engineering structures in the coastal areas, the foremost of concern today and in the imaginable future is coastline erosion. The advent of remote sensing and geographic information system (GIS) has made acquisition of timely, repetitive and multispectral data, useful for geomorphological mapping and monitoring the coastline dynamics periodically, cost-effectively and accurately, easier. This study investigate the recent geomorphological changes of the Nigerian coastline, using available remote sensing data. The study area was divided into 6 geomorphological units to calculate the rate of change (erosion, accretion, no change/stability) from multiple historic shoreline positions using the digital shoreline mapping and analysis system software. Rates of change were categorized into different classes (low, moderate and high accretion; no change; and low, moderate and high erosion). In terms of change rate percentage, 3 geomorphological units (i.e. Barrier-Lagoon, Western and Eastern Niger Delta sections) exhibited accretion, while other 3 units erosion (i.e. Mahin Mud and Arcuate Niger Delta and Strand sections). However, in terms of surface area coverage and net change, findings show that 3 geomorphological units (Lagoon-Barrier, Western Niger Delta and Arcuate Delta) presented positive balance, while the other 3 units (Mahin Mud, Eastern Niger Delta and Strand Coast) negative balance, showing a positive net balance of 2,541.44 m2/yr corresponding to a gain of 127.07 km2 of beach surface between 1984 and 2018.Factors influencing the geomorphological changes of the Nigerian coast were considered. Both natural phenomena (wave climate change and rainfall variability due to changing climate) and anthropogenic activities (port development along the coast, river dredging, land reclamation, dam construction, groynes emplacement, mangrove deforestation, etc.) were identified as the major drivers.

Keywords: *Shoreline mapping, Nigeria, coastline change, anthropogenic effects, natural factors*

Introduction

Coastal geomorphology is the study of the morphological development and evolution of the coast as it acts under the influence of winds, waves, currents, and sea-level changes (Leatherman et al., 1994). It involves study of physical processes and responses in the coastal zone in order to provide fundamental understanding that is essential to address the related problems in the coastal environment (Leatherman et al., 1994). All over the world, there are many geomorphological processes at work in the coastal environment which are responsible for the shape and stability of many engineering structures in the coastal areas (Akpokodje, 1987). According to Leatherman et al. (1994), the foremost coastal concern today and in the imaginable future is coastline erosion. Close to 70% of the world's sandy shorelines are eroding and the percentage may approach 90% in the United States (Leatherman et al., 1994). Based on an estimate provided by Dixon and Pilkey (1989, in Leatherman et al., 1994), over 640 km of U.S. coastline have been nourished, largely through public funding, at a total cost of about $8 billion. For instance, a proposed nourishment project to reinstate and maintain a 19 km beach in northern New Jersey, for over 50 years, would cost about USD$500 million (Bocamazo, 1991). Likewise, the nourishment project of Ocean City, Maryland, which includes its re-nourishment every four years for 50 years, would cost US$342 million (Kelly, 1991).

Like most of the coastal regions of the world, the Nigerian coast is a home to about one-third of the country's people. According to VOA documentary (2019), the Nigeria's population is growing at the rate of 3.2 percent per year and may increase to 402 million by 2050. A very large proportion of the Nigeria's GDP is generated from its coastal areas. These coastal areas host major cities, ports, offshore oil and gas exploration and production, fisheries and agro-industries. Abundant natural resources, on land and at sea, provide vital ecosystem services and drive economic growth, boost resilience in the face of climate change, and provide the livelihoods for the Nigerian coastal dwellers. For example, ~85% of the industrial set-ups are located within the coastal area of Nigeria, likewise the majority of the exploration and exploitation facilities. The Niger Delta alone accounts for over 90% of Nigeria's export and foreign exchange oil earnings. While the wetlands are the spawning grounds for commercial shrimp and oysters (French et al., 1995). Therefore, a good knowledge and understanding of the geomorphological processes in this coastal environment is crucial to solving some environmental problems, especially in relation to coastal engineering and geotechnical problems. This paper presents the recent geomorphological changes of the Nigerian coastline (Fig. 1) and discusses the factors responsible for the changes.

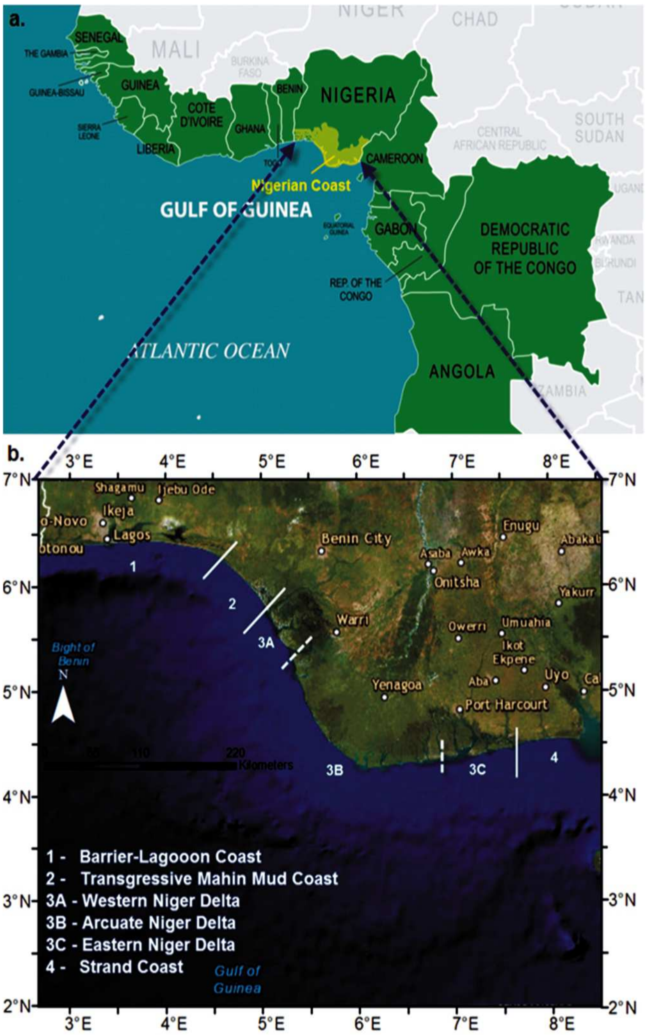

Fig. 1: Maps of: (a) the Gulf of Guinea showing Nigerian Coast (b) Nigerian Coast showing the six geomorphic units: Barrier–Lagoon Complex Coast, the Transgressive Mahin Mud Coast, the Western Niger Delta flank, Arcuate Niger Delta, the Eastern Niger Delta flank and the Strand Coast.

The Nigerian Coast

The Nigerian coast is located in the Gulf of Guinea along the West African coast (Fig.1). In the South, it is bordered by the North Atlantic Ocean, in the West and East by the Benin and Cameroon Republics, while in the North they are backed by the Nigeria's landmass, from which the Niger–Benue River systems and their tributaries flow through before discharging their water and sediments into the Atlantic Ocean. The total length of the Nigerian coastline is about 873 km, ranging approximately between latitude 4°10' to 6° 20' N and longitude 2° 45' to 8° 35' E.

Climatic, Vegetation and Hydrological Setting

The Nigerian coastal zone is within the wet equatorial climatic region and is strongly influenced by the West African monsoon (Okoro et al., 2014; Adejuwon, 2012). It is characterized by wet/rainy (April to October) and dry (November to March) seasons. The maxima annual rainfall along the Nigerian coast is >3000 mm during the rainy season and the mean monthly temperature varies between 24° and 32° C throughout the year (Awosika et al., 2013; Sexton and Murday, 1994). Relative humidity is high throughout the year and rarely goes below 60%. The dry season period is characterized by the northeast trade winds. Minimum wind gusts of 2.5 to 4.7 m per second are generally observed from November to February when mean wind direction is between 161° and 190° (Awosika and Folorunsho, 2009).

The vegetation of the Nigerian coastal area is characterized by mangrove forests, brackish swamp forests and rain forests. A great percentage of the Nigeria's extensive mangrove ecosystem is estimated to cover between 500,000 and 885,000 hectares and can be found in Lagos, Ondo, Delta, Rivers, Akwa lbom and Cross River states (Nwilo and Badejo, 2006).

The hydrology of the Nigerian coast zone comprises the western Nigeria River Basin, the Niger–Benue River Basin and the south–eastern River Basin. The western Nigerian River basin drain the landmass in the southwestern part of Nigeria and empty into the Lagos lagoon (Awosika and Folorunsho, 2009). The Niger–Benue River System is dominated by the Niger River that enters Nigeria in the northwest, and joined its major distributary, the Benue River, at Lokoja before discharging its water and sediment into the Niger Delta and finally the Atlantic Ocean. The Niger River drains a total area of 621,351 km^2 and discharges about 200 x 10^9 cubic metres of freshwater to the delta annually (Awosika and Folorunsho, 2009). The South Eastern River System is drained by the Imo and Calabar Rivers, and other smaller rivers that originated from the Eastern Highland, south of the Benue River. These rivers discharge their water and sediment into the ocean via their estuaries (Awosika and Folorunsho, 2009).

Geomorphological and Geological Setting

The common geomorphic feature of the Nigerian coast is its low–lying characteristic, as most areas along this coast are less than 3 m in elevation. Four major and distinct geomorphological units can be distinguished along this coast from east to west: the Barrier–Lagoon Complex Coast (220 km in length), the Transgressive Mahin Mud Coast (83 km), the Niger Delta (450 km; extends from the Benin River mouth to immediately east of the Imo River mouth) and the Strand Coast (100 km). Based on the sedimentological, geomorphological and marine processes characteristics of the Nigerian coast (i.e. sediment grain size, geomorphic and coastal processes), the coast can be further classified into six geomorphic units: Barrier–Lagoon Complex Coast, the Transgressive Mahin Mud Coast, the Western Niger Delta flank, Arcuate Niger Delta, the Eastern Niger Delta flank and the Strand Coast (Fig. 1; Awosika et al. 2000; Sexton and Murday, 1994; lbe, 1988). Nigeria has a narrow continental shelf that ranged from 15 km offshore off Lagos to about 75 km in front of the Niger Delta, and about 85 km off Calabar, along the Strand coast. The shelf starts to break at an average depth of 90 m (Awosika and Folorunsho, 2009). The Niger Delta section covers about 50 % of the Nigerian coastline, spanning from the Benin River mouth for about 450 km eastward, and terminates at the Imo River mouth. While its two flanks (western and eastern) covers about 115 km. This geomorphic unit has 21 major river mouths/tidal inlets that intersect the coast, breaking up into series of barrier islands from which 16 are within the Arcuate Delta region (Adeaga, 2014). The Niger Delta section of the Nigerian coast consists of a sandy shoreline backed by widespread mangrove swamp and the barrier islands disconnected by tidal channels. The barrier islands on the delta, which are largely 15–20 km in long and 3–5 km wide, are better established on the western than on the eastern flank. In fact, the western Niger Delta flank has the longest and widest barrier islands of about 35 km (Ramos–Dodo Island) and 10 km (Forcados–Ramos Island), respectively. The barrier islands in the eastern Delta flank are narrower and seem like remnants of beach ridges, due to wave action and tidal erosion in and around the associated creek network. Abam (1999) noted that while the western flanks are less susceptible to erosion due to the dark and dark brownish organic and peaty clay of high plasticity, the presence of brown sandy clay soil in the eastern flank makes it vulnerable to coastal erosion. Sexton and Murday (1994) linked erosional processes in the eastern Niger Delta to marine processes and the accretion in the western flank of the delta to the presence of active river systems in the region. The delta western flank has significant active freshwater and sediment delivery from the upstream Lower Niger–Benue River system, while the eastern flank is influenced primarily by marine processes of waves and tides (Sexton and Murday, 1994). The post–Kainji Dam period have exposed the region to strong erosional pressure owing to reduction in water and sediment supply into the Niger Delta region (Abam 2001).

Geologically, the Nigerian coast is part of the open type sedimentary basins occurring along the passive continental margin of the Gulf of Guinea. While its continental deposits constitute the bulk of the sedimentary fill of these basins: their spatial distribution is closely connected with the separation of the African landmass from the South American continent that started during the late Jurassic or early Cretaceous (Adediran et al., 1991; Reyment and Tait, 1972). According to Adediran et al. (1991), the early history of the Nigerian coastal basins reflects the prominence of tectonism, that is, rifting, intense faulting and subsidence leading to erosion and deposition of thick clastic accumulations which subsequently gave way to marine transgression and development of typical shallow marine littoral and shelf deposits. The latter history of the basin shows the greater role played by sea–level changes, which obviously oscillates between the transgressive and regressive phases (Adediran et al., 1991). The shelf is interjected by several submarine canyons (Avon, Mahin and Calabar Canyons). Sediments at nearshore are fine–to coarse–grained sand except off the Mahin mud coast, where it is void of sand with sediment grades varying from fine sand to silt, and finer–grained sand occurring along the flanks of the Niger Delta. Its sediments, which mainly comprise medium to coarse unconsolidated sands, silt, clay, shale, and peat, from the suspended and traction loads, are mainly from the Niger and Benue Rivers and their tributaries (Adeaga, 2014; Abam 1999; 2001).

Oceanographic Setting

The Nigerian coast is a cyclone– and storm–free environment. Generally, the coast is influenced by tides, waves, long shore currents, and ocean currents. It is exposed predominantly to south–westerly winds and waves that are generated from the South Atlantic (Dada et al., 2018, 2016b; Almar et al., 2015; Laïbi et al., 2014; Davies, 1980). Most areas along the Nigerian coast experience moderate to high wave climate. Waves are predominantly of the plunging and spilling types, ranging between 1 and 2 m high along the Nigerian coast. But during the rainy months of June to September when storms are more frequent, waves of over 3 to 4 m are common in most areas (Awosika and Folorunsho, 2009). While plunging breakers are more dominant along the Barrier-lagoon and the Niger delta coastline (Forcados), the Mahin mud coast is characterized by spilling breakers with less intense wave action (Awosika and Folorunsho, 2009). The waves have two components with distinct origins and behaviors: wind waves generated locally in the Gulf of Guinea and swell waves generated in the Southern Hemisphere. However, the contribution of swell waves to gross annual longshore transport is one order of magnitude larger than that due to wind-waves (Dada et al., 2018, 2016b; Almar et al., 2015). Tides along the entire Nigerian coast are semi-diurnal with two inequalities. The tidal range increases progressively eastwards from 1 m at Lagos to about 3 m at the Calabar estuary. Though the tidal range (1.5 m) is relatively small, it can be destructive and significantly modify the coastal morphology, as evident along the Mahin Mud Coast during spring tides (Nwilo and Onuoha, 1993). Longshore current (and its resultant longshore sediment transport that ranges between 0 and up to 1.2 million m^3/year, depending on the location) is generated by south-westerly breaking waves and the dominant coastal current affecting the Nigerian coast; while west-east Guinea current is the dominant ocean current affecting the Nigerian continental margin (Dada et al., 2016a, b; Almar et al., 2015). Longshore currents along the Barrier-Lagoon coast in the west, have a west to east directional component, while the Mahin mud coast has little or no longshore currents because the southwesterly waves arrive parallel to the coast (Awosika and Folorunsho, 2009). Longshore current along the northwestern Niger Delta is north-westerly. The eastern Niger delta from Akassa Point to the Calabar estuary is characterized by a west to east drifting longshore current (Dada et al., 2016a).

Material and Methods

The advent of remote sensing and geographic information system (GIS) has made it easier to obtain timely information on the geomorphological changes of the world coastline. The multispectral nature and repetitive coverage makes remote sensing data the most useful for geomorphological mapping and monitoring the shoreline dynamics periodically, cost-effectively and accurately (Dada et al., 2018b). In order to understand shoreline evolution over the last 30 years Landsat of 1984/1986, 2001/2002 and 2016. The imageries were processed and shoreline extracted by following the procedures of Dada et al. (2015, 2016a, 2016b, 2018a, 2018b and 2019). The shoreline changes were quantified using the Digital Shoreline Analysis System (DSAS) v4.2, an extension of ArcGIS developed by USGS (Thieler et al., 2009). DSAS automatically generates transects perpendicular to the shoreline over a-1000 m from the baseline offshore; and then measures the differences between the shorelines and calculates the average rates of movement along each transect. In this study, linear regression rate (LRR) and end-point rate (EPR) were used to estimate both short-and long-term evolution. The Landsat data were complimented by other data from existing literature and past fieldwork.

Results

Figs 2 and 3 show the summary of the geomorphological changes of the Nigerian coastline during the last 30 years. In terms of change rate percentage, 3 geomorphological units (i.e. Barrier-Lagoon, Western and Eastern Niger Delta sections) predominantly exhibited accretion, while other 3 units erosion (i.e. Mahin Mud and Arcuate Niger Delta and Strand sections). However, in terms of surface area coverage and net change, findings show that 3 geomorphological units (i.e. Lagoon-Barrier, Western Niger Delta and Arcuate Delta) presented positive balance, while the other 3 units (i.e. Mahin Mud, Eastern Niger Delta and Strand Coast) negative balance, showing a positive net balance of 2,541.44 m^2/yr corresponding to a gain of 127.07 km^2 of beach surface between 1984 and 2018.

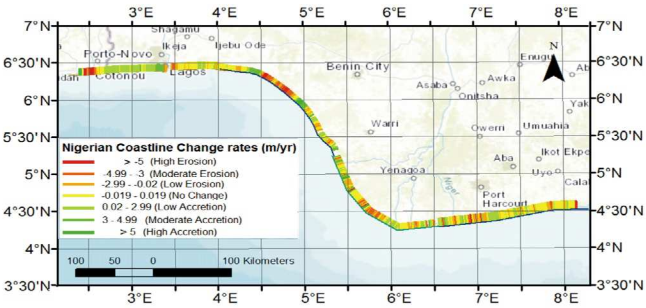

Fig. 2: Summary of the geomorphological changes of the Nigerian coastline during the last 30 years.

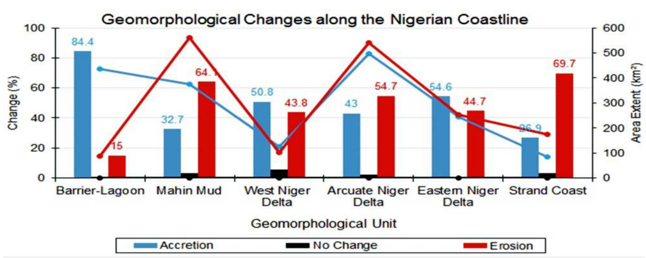

Fig. 3: Summary of the geomorphological changes of the Nigerian coastline during the last 30 years.

Spatio-temporal Coastline Variation

The change recorded between 1984 and 2001 was predominantly accretion of averagely 76.23 m (4.49 m/yr), except at Cotonou (Benin Republic) end of the Barrier–Lagoon coast where erosion was observed (Fig. 4a). Between 2001 and 2016, erosion was predominant along this coastline and an average change of −14.53 m (−0.97 m/yr) was recorded. During this period, erosion was up to −232.65 m (−15.51 m/yr) at the Barrier–Lagoon coast and in the eastern part of the coast (Fig. 4b). Over the entire period (1984–2016), the Barrier–Lagoon coastline accreted and average net shoreline change was 61.67 m (1.92 m/yr), and the magnitude of positive change observed at the western section of the coast was greater than at the eastern section. During the same period, pockets of shoreline erosion (up to −8 m/yr) were observed towards the Benin Republic end of the Barrier–Lagoon coast and in the eastern part of the coast (Fig. 4c).

The change recorded between 1984 and 2001 along the Mahin Mud coastline was predominantly erosion of averagely −7.31 m (−0.45 m/yr). The magnitude of erosion along this mud coast was up to −34 m/yr, especially at the central part (muddy part) of the Mud coast during this period (Fig. 5a). During 2001–2016 period, erosion was predominant along the entire Mahin Mud coastline and an average shoreline change of −32.63 m (−2.19 m/yr) was recorded. Also, the magnitude of erosion was higher and could be up to −39 m/yr, especially at the central part (muddy part) of the Mud coast during this period (Fig. 5b). Over the entire period between 1984 and 2016, the mud coastline retreated landward and average net shoreline change was −9.94 m (−1.29 m/yr; Fig. 5c). This means the Mahin mud section has the highest erosion rates along the Nigerian Coast. The change recorded at the western flank of the Niger Delta between 1984 and 2001 was predominantly accretion of average net shoreline movement of 30.39 m (1.88 m/yr; Fig. 5a). However, between 2001 and 2016, shoreline erosion was predominant along this flank of the Niger Delta coastline and an average change of −30.03 m (−2.02 m/yr) was recorded (Fig. 5b). Over the entire period (1984–2016), the Western flank of the Niger Delta coastline accreted and average net shoreline change was 17.62 m (0.57 m/yr; Fig. 5c). At the Arcuate (central) Niger Delta, the change observed between 1987 and 2002 was predominantly erosion of average net shoreline movement of −6.97 m (−0.47 m/yr; Fig. 6a). However, between 2002 and 2016, shoreline erosion was predominant along this central section of the Niger Delta coastline and an average change of −13.82 m (−0.98 m/yr) was recorded (Fig. 6b). During the entire period (1987–2016), the Arcuate (central) Niger Delta coastline accreted and average net shoreline change of 6.85 m (0.23 m/yr; Fig. 6c) was recorded. The change recorded between 1986 and 2003 at the eastern flank of the Niger Delta was predominantly erosion of average net shoreline movement of −37.64 m (−2.34 m/yr; Fig. 7a). However, between 2003 and 2016, shoreline accretion was predominant along this flank of the Niger Delta coastline and an average change of 34.35 m (2.15 m/yr) was recorded (Fig. 7b). Over the entire period (1986–2018), the Eastern Niger Delta coastline retreated and average net shoreline retreat of −3.28 m (−0.1 m/yr; Fig. 7c) was recorded. It is important to note that higher rates (accretion rate up to 254.6 m/yr and retreat rate up to −125 m/yr) were observed at the central and eastern parts of the Niger Delta coastline (Figs 6 and 7). The change recorded between 1986 and 2003 along the Strand Coast was predominantly accretion of averagely −19.26 m (−1.21 m/yr; Fig. 7a). However, between 2003 and 2016, erosion was predominant along the entire Strand coastline and an average shoreline change of −10.28 m (−0.64 m/yr) was recorded (Fig. 7b). Over the entire period (1986–2018), the Strand coastline retreated and average net shoreline change was −29.54 m (−0.92 m/yr) was recorded (Fig. 7c).

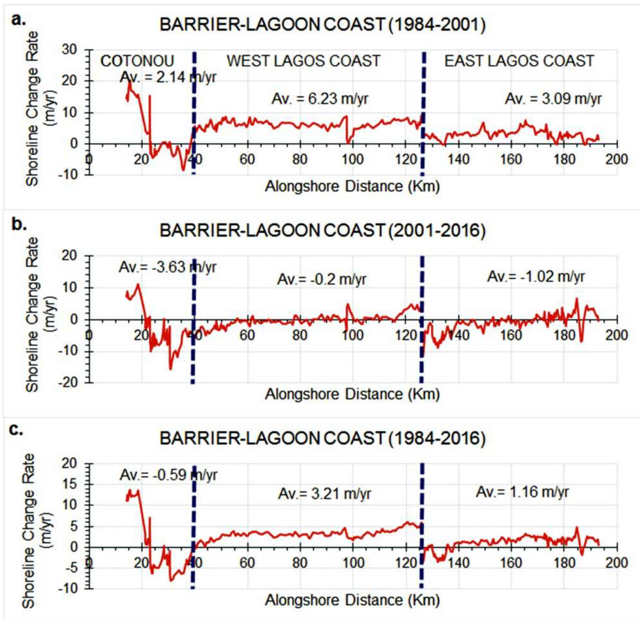

Fig. 4: Shoreline changes along the Barrier–Lagoon section of the Nigerian Coast between (a) 1984 and 2001, (b) 2001 and 2016, and (c) entire 1984–2016.

Discussion

In this study, the spatio–temporal dynamics of shoreline

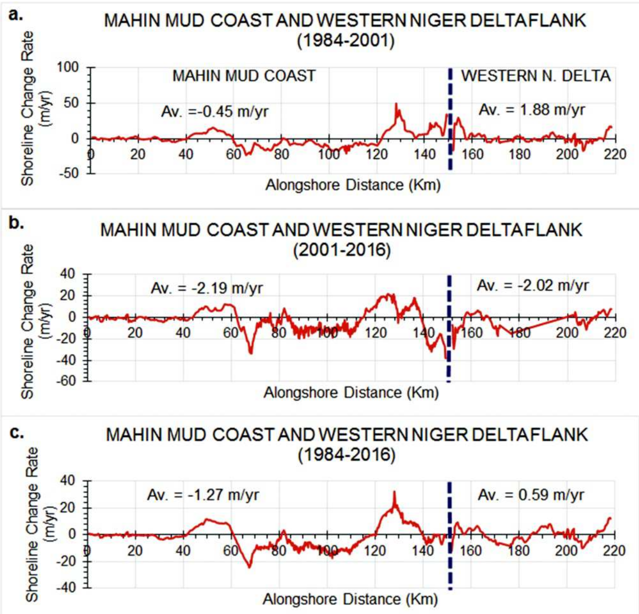

Fig. 5: Shoreline changes along the Mahin Mud and Western Niger Delta sections of the Nigerian Coast between (a) 1984 and 2001, (b) 2001 and 2016, and (c) entire 1984–2016.

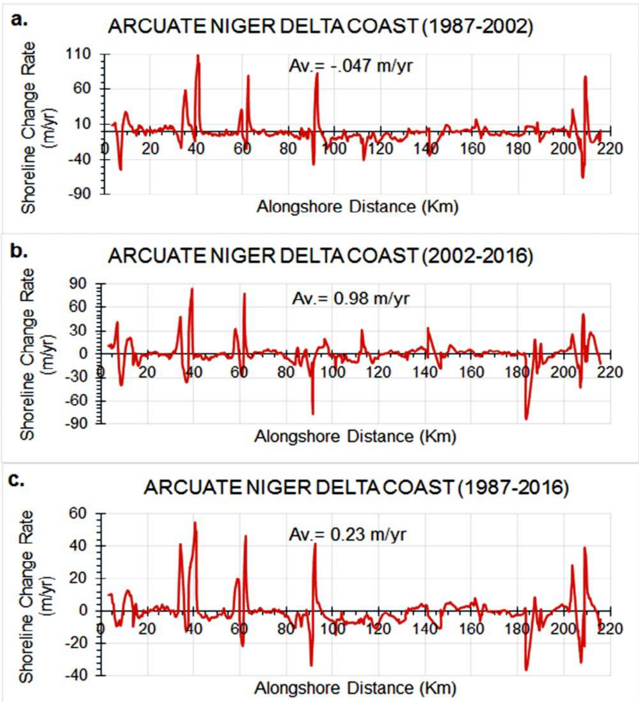

Fig. 6: Shoreline changes along the Arcuate Niger Delta section of the Nigerian Coast between (a) 1987 and 2002, (b) 2002 and 2016, and (c) entire 1987–2016.

changes between 1984 and 2018 along the entire Nigerian coast have been determined using remote sensing data and GIS techniques. The results of the present study show that over the last 30 years, different sections of the Nigerian coastline have undergone different geomorphological changes in response to

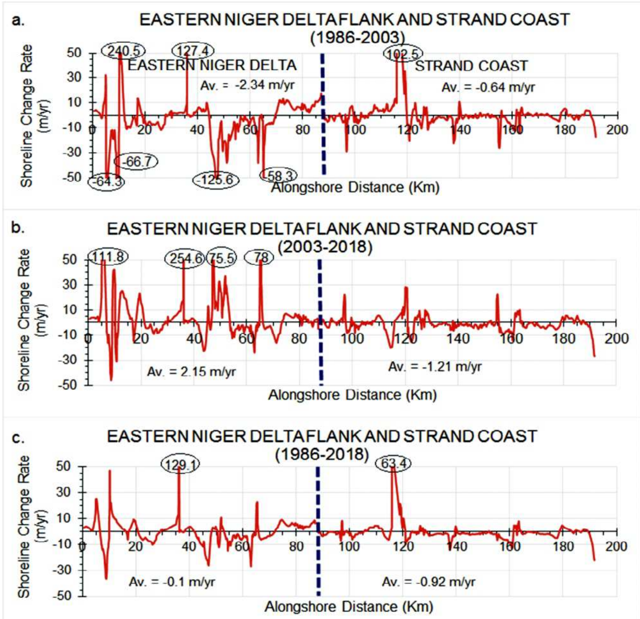

Fig. 7: Shoreline changes along the Western Niger Delta and Strand Coast sections of the Nigerian Coast between (a) 1986 and 2003, (b) 2003 and 2018, and (c) entire 1986–2018.

different forcing, both natural and anthropogenic (Fig. 8). The spatio-temporal oscillations of shorelines in Niger Delta section of the Nigerian coast is basically connected to hydro-meteorological variability of the Niger River basin and coastal hydrodynamics. While the dynamism of shorelines along the Barrier-Lagoon coast and its adjoining Mahin mud coast have been altered by the presence of varying human activities along the coast.

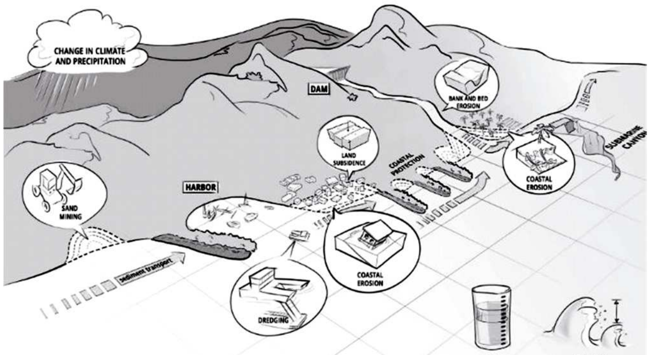

Fig. 8: Major factors affecting coastline geomorphological changes (after Giardino et al., 2018).

Main factors driving geomorphological changes of the Nigerian coastline

Anthropogenic Activities and Impacts on the Nigerian Coastline Change

Several human activities are influencing geomorphological changes of the Nigerian coastline, ranging from mangrove deforestation (Dada et al.. 2019; Kuenzer et al., 2014; Odunuga et al., 2013; Fasona et al., 2011) to canalization (Kuenzer et al., 2014; Ohimain et al., 2004), sand mining and land reclamation (Abam, 2004), channel dredging (Dada et al., 2016a; Okonkwo, 2012; Der‑Burg, 2010), and dam construction (Dada et al., 2015, 2018a; Abam, 2001a, 1999). These activities have heavily influenced the natural geomorphology of the Nigerian coastline, although they have varying effects on different sections of the Nigerian coast. The fringe of most of the Nigerian coastline is dominated by mangrove forest. However, in the last 30 years, mangrove forests have been degraded in response to anthropogenic pressure (Dada et al., 2019, Kuenzer et al., 2014; Viles and Spencer, 1995; Odunuga et al., 2013; Fasona et al., 2011). Along the Mahin Mud coast, coastal retreat and land loss have been linked to degradation of mangrove forests that should have acted as natural barriers against coastal erosion and rising sea level. Most areas along this mud coast that were formerly occupied by forested mangrove, salt‑resistant grass and water bodies had been transformed to developed areas (Dada et al., 2019). Previous studies have shown that mangrove deforestation are connected to increasing the use of mangrove trees for firewood, building materials for boats and housing, and vegetation clearing for oil and gas exploration and exploitation along the Nigerian coast (Dada et al., 2019; Kuenzer et al., 2014; Odunuga et al., 2013; Fasona et al., 2011). Along the Barrier‑Lagoon coast, thousand acres of swamp have been reclaimed in the last 100 years due to rising economical and infrastructural development, and much of the reclaimed lands are prone to ground subsidence and as well vulnerable to coastal flooding and erosion (Viles and Spencer, 1995). The construction of dam across the Niger River at Kainji comes as another factor that has negatively affected the regime and runoff yield of the Niger River (Abam 2001, 1999). After its completion in 1968, a strong decrease in sediment supply to the Niger Delta coast occurred which was identified as one of the triggering factors of shoreline retreat along the Niger Delta section of the Nigerian coast (Dada et al., 2018a, 2015). During the pre‑dam period, the Niger River carried its sediment loads as suspended load and bedload (Abam 2001, 1999; Oyebande et al., 1980). However, after the construction of the dam, sediments were trapped by the reservoir and the trapping eventually resulted in sorting and decrease of sediment loads. Abam (2001a, b, 1999) pointed out that the reduction in the sediment‑carrying capacity of the downstream section of the Lower Niger led to the complete silting‑up of the Lower Niger River channel and the formation of sandbars along the river channel. As a result, the ability of the Lower Niger River to successfully nourish the coast was hindered. Dada et al. (2015) suggested that the effect of reservoir impoundment may have weakened sediment load transport to the Niger Delta and the coast and consequently contributed negatively to the geomorphology of the delta coast. Nigerian has major ports scattered along her coast: Lagos and Tin Can Island Ports (Barrier‑Lagoon coast); Delta Port (Western Niger Delta); Rivers and Onne Ports in the (Eastern Niger Delta); and Calabar Port (Strand coast). Entrances and channels to these ports are dredged from time to time to evacuate sediment and make it deeper for all‑year round navigational efficiency (Dada et al., 2016a; Okonkwo, 2012). The dredging operations usually cause sediment budget deficit and increase the erosive power of flow, as sediment supply was lower than the transport capacity. According to Dada et al. (2016a), the sediment load reduction to the delta, as a result of the Niger River dredging capital project between 2009 and 2012, sparked off sediment starvation of the nearshore hydrodynamics at coast that consequently triggered coastline erosion that occurred between December 2007 and April 2012 along the Niger Delta coast. The shoreline recession during this period was attributed mainly to channel dredging, although there might have been climatic influences.

Natural Factors and Impacts on the Nigerian Coastline Change

Several studies have linked runoff and sediment yield to precipitation (e.g. Morera et al., 2017; Darby et al., 2016). The geomorphology of the eastern sections of the Nigerian coast, i.e. the entire Niger Delta and Strand coasts, is mainly influenced by river discharges and amount of sediment load reaching the coast. Also, the geomorphological changes of the Nigerian coastline are better reflected by the temporal variations of hydroclimatic forcing (Dada et al. 2018a; 2016a; 2015). The geomorphological changes of the Niger Delta section of the Nigerian coastline during the last 100 years have been influenced by river discharge and rainfall variabilities due to regional climate change (Dada et al., 2018a; 2015). The more recent studies also show that the oscillations of floods and droughts in the Lower Niger River basin are well‑reflected in the Niger Delta coastline geomorphological changes (Dada et al., 2018a; 2016a; 2015). The anthropogenic effects of reservoir sediment impoundment in the Lower Niger River may have, to an extent, also triggered a reduction in sediment to the delta coast. However, the weakened hydroclimate associated with regional climate change that occurred between the 1970s and 1990s, played a more prominent role in reducing river discharges (water and sediment loads) to the delta coast and the Atlantic Ocean than the anthropogenic effect of the dam that controls less than 20% of floodplains of the entire Lower Niger Basin and, in addition, sited about 1004 km away from the coast (Dada et al., 2018, 2016a, 2015). Precipitation, runoff and sediment yield offer the most complex hydrological variation due to the large spatial variability of basin characteristics (semi–arid in the north to humid in the south) and rainfall patterns. Semi–arid regions are known for generating sediment loads and these material are usually made available in the wet period, due to amplified streamflow and runoff wash effects of the exposed soils (e.g. Morera et al., 2017; Alexandrov et al., 2003; Laronne and Reid, 1993; Lekach and Schick, 1982; Bondurant, 1951). Thus, precipitation is an important factor governing the hydro–geomorphology of the basin. It is certain that decreasing basin precipitation will adversely influences both the reservoir sediment load pattern and river base–flows. It has been reported that the average flows at the 'undammed' Middle Niger in Koulikoro (Mali) and Niamey (Niger) and the'dammed' Lower Niger at Lokoja (Nigeria) were reduced between 1970 and 1990. The largest meanflow reductions between 1950 and 2000 were 72%, 42% and 24%, respectively at Niamey, Koulikoro and Lokoja (Olomoda, 2006). In fact, low flows, and even complete cessation of discharge were recorded in the Middle Niger section at Bani (Mali) in 1983, 1984 and 1987, and at Niamey (Niger) in 1985. This situation led to the reduction of flood duration and flooding areas in the Inland Delta from 43,900 km^2 in 1957 to 9500 km^2 in 1984 (Lae et al., 2004). This event changed the critical elements of river geomorphic systems, that is, the ability of the river to carry sediment and the quantity of sediment available for transport, and caused early deposition of bedload in the river channel and upper segment of the Niger Delta. Thus, depriving the coast of required sediment for delta coastline progradation processes. Nevertheless, a drift to wetter conditions with far–reaching rise in annual rainfall in the recent time due to heightened intensity and duration of rainfall during summer (Dada et al., 2018a, 2015; Ogungbenro and Morakinyo, 2014; Odekunle et al., 2008; Ati et al., 2007, 2009) is found to have accounted for the significant rise in river (water and sediment) discharges that subsequently resulted in shoreline progradation in the recent years (Dada et al., 2018a, 2015). The frequent floods since the 1990s and especially the megafloods of 2000s have increased the sediment influxes to the coast, and hence influenced the recent geomorphology of the Niger Delta coast (Dada et al., 2018a, b, 2015).

More recent studies have shown upward trends in ocean waves in the Atlantic Ocean (e.g. Bacon and Carter, 1991; Dodet et al., 2010; Le Cozannet et al., 2011; Charles et al., 2012; Wang et al., 2012; Bertin et al., 2013). As pointed out by Dada et al. (2016b) and Almar et al. (2015), increasing wave trends in the recent decades is one of the factors responsible for the geomorphic changes and configurations along the Nigerian coastline. Being opened and exposed to the Atlantic Ocean, the Nigerian coast is vulnerable to the direct impacts of nearshore hydrodynamic processes, especially the frequency and intensity of waves on the coast (Dada et al., 2016a; b; 2018a; 2019). Findings from previous studies have revealed that there is a strong tendency that coastal hydrodynamics processes may apparently be instigating trend, spatial and temporal variability of shoreline changes along the Nigerian coast (e.g. Dada et al., 2016a; b; 2018b; Almar et al., 2015). For instance, Dada et al. (2016a, b) suggested that the periodic fluctuations in the nearshore hydrodynamics processes play a major role in influencing shoreline behaviour of the Niger Delta section of the Nigerian coast.

Conclusions

This study analyzed the geomorphological changes of the Nigerian coastline during the last 30 years. The coast was divided into 6 geomorphological units of varying lengths and, within each unit, the rates of shoreline change were calculated by using the the digital shoreline mapping and analysis system software.

Along the Nigerian coast, 3 geomorphological units (i.e. Barrier–Lagoon, Western and Eastern Niger Delta sections) presented accretion, while other 3 units erosion (i.e. Mahin Mud and Arcuate Niger Delta and Strand sections). However, in terms of surface area coverage and net change, findings show that 3 geomorphological units (Lagoon–Barrier, Western Niger Delta and Arcuate Delta) presented positive balance, while the other 3 units (Mahin Mud, Eastern Niger Delta and Strand Coast) negative balance. The entire units showed a positive net balance of 2,541.44 m^2/yr, which corresponds to a gain of 127.07 km^2 of beach surface between 1984 and 2018.

Factors influencing the geomorphological changes of the Nigerian coast were considered. Both natural phenomena (wave climate change and rainfall variability due to changing climate) and anthropogenic activities (port development along the coast, river dredging, land reclamation, dam construction, groynes emplacement, mangrove deforestation, etc.) were identified as the major drivers.

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Analysis of Seismic Refraction and Uphole Survey Data of Eastern Niger Delta Basin for Engineering Structures

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Abstract

The subsoil of Alagbaka-extension, Akure, was evaluated for its competence as foundation materials. Twenty six (26) disturbed soil samples were collected from different pits at a depth of about 2m. The samples were analyzed for natural moisture content, grain size analysis, Atterberg limits, standard compaction, un-confined compressive strength and consolidation tests. The laboratory tests results revealed that moisture content ranges from 6.1% to 18.5%, liquid limit from 25.1% to 48.6%, plastic limit from 17.6% to 29.5%, linear shrinkage from 2.9% to 7.7%, and plasticity index from 5.2% to 19.5%. The grain size analysis showed that the amount of fines ranges from 20.6% to 47.4%. Soil classification with AASHTO revealed that almost all soils from the area underlane by migmatitic gneiss are clayey while those from area underlane quartzites classified as silty or clayey gravel and sand. The specific gravity ranges from 2.65 to 2.8, maximum dry density from 1820Kg/m3 to 1998Kg/m3 and optimum moisture content from 13.2% to 19.7%. The shear strength varied from 74.1kPa to 103.8kPa and Coefficients of consolidation range from 0.02019 to 0.02944. The competence map segregated the study area into extremely highly competent zone, very highly competent zone, highly competent zone and moderately competent zone. It is hereby concluded that (i) generally, the subsoil of the area are competent; (ii) the competence map revealed a zonation; and (iii) the lithology of the underlying rock and the soil group are closely related to soil competence.

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

Introduction

In seismic methods, seismic waves are generated by a controlled source and propagate through the subsurface (Kearey, et al., 2002). Seismic method is far the most prioritized geophysical method in terms of expenditure and number of geophysicist involved during acquisition (Telford, et al., 1990). The method has been widely understood to dominate all other geophysical methods because of its high accuracy, great penetrating power and high resolution (Milsom, 2003; Telford, et al., 1990). Because of these, seismic methods are at the heart of petroleum exploration, groundwater research, civil engineering and all other geophysical investigation. Seismic exploration is based on generating seismic waves and measuring the time it takes for the waves to get to the geophones from the source of the wave generated, usually geophones are deployed in a straight line away from the source (Telford, et al., 1990; Reynolds, 1997).

The knowledge of travel times and velocity of waves can help us reconstruct the paths of seismic wave. Two main paths are involved in seismic wave; the Reflected paths, in which paths travel vertically and when they are met by a discontinuity reflects back to the surface, and the Headwave or Refracted paths, here the crux of the path lies in the interface between two beds and the critically refracted waves travel down the different subsurface layers before returning to the surface where they are detected by geophones in land acquisition or hydrophones in marine acquisition. For both paths their arrival time is dependent on the physical properties of the rocks and the attitude of beds in which they pass through (Telford, et al., 1990).

The forward and reverse seismic refraction profiling involves the use of a minimum of one seismic source at both end of the spread during conventional seismic refraction survey; this is done in order to have a better image of the subsurface especially in cases where the refractor bed is dipping. The uphole seismic refraction technique is one of the most important and commonly used in the studying the features of low velocity layers (Anomohanran, 2013); in this case, a seismic source is used to generate seismic signals at or below the surface at a horizontal offset from cased borehole. The receivers, usually geophones are lowered into the borehole at selected test depth and a connection involving the receiver and source are connected to a recording system at the surface that record the response of the receivers.

The weathered layer is the shallow subsurface or top most layer of the earth composed of unconsolidated materials such as loose soil and gravel, and heterogeneous in composition. It is characterized by low transmission of seismic waves and shots taken around this layer tend to be of low frequencies because the layer is capable of absorbing high frequency signals (Adeoti, et al., 2013). The weathered layer is also highly porous, little or no cementation, with low pressure and bulk modulus. All these characters' account for low shear and compressional wave velocities observed in the layer (Agoha, et. al, 2015); arriving reflected waves are attenuated and delayed as the layer serves as a wave guide for surface waves otherwise known as statics. In seismic reflection surveys statics are corrected by the application of time shifts computed from weathering parameters of the layer. Occasionally, in the study of weathered layers a borehole that penetrates the layer is drilled and geophones are placed inside this borehole at specific depth within it. Thus, it is a handy tool Geophysicist use in making decision on drilled and charge depths prior to commencement of any seismic reflection acquisition (Adeoti, et. al, 2013).

Geophysicists use these weathered layer parameters in the design of receiver source arrays for field filtering purposes, these parameters are used in the analysis of soil behaviour under static and dynamic loads (Agoha, et. al, 2015). Apart from using uphole survey for determining the competence of the subsurface in engineering constructions other geophysical methods have been used but the merit of uphole survey over other techniques cannot be over emphasized, in the sense that it can reveal information on the subsurface that would be concealed to the observer at the surface and not just revelations on velocity variation in the subsurface. However, information from uphole refraction survey on low velocity layers can help reconstruct a concise map of velocity and thickness of the Layer.

The area under study is located within the Niger Delta Basin, it is a hydrocarbon rich province with couple of civil engineering activities taking place and presence of already failed projects. The massive influx of people into the area due to its proximity to the oil rich Niger Delta has increased cases of building collapse due to improper geophysical studies before any construction activities in the area. There is therefore urgent need to understand the variation in thickness and velocity of the weathering layer to guide any engineering construction works in the area. This work seeks to characterize subsurface using seismic refraction methods (uphole and reverse/backward profiling) in part of Eastern Niger Delta Nigeria.

Location and Geology of the Study Area

The study area lies within the Eastern part of the Niger Delta. The Cenozoic Niger Delta covers an area of approximately 140,000 sq km and consists of a regressive clastic succession, which attains a maximum thickness of 12,000m (Obaje, 2009). It is located at the intersection of the Benue Trough and the South Atlantic Ocean where a triple junction developed when the separation of the continents of South America and Africa occurred in the late Jurassic (Whiteman, 1982; Kogbe, 1989).

According to Whiteman (1982) the structural and tectonic evolution of the Niger Delta is controlled by Basement tectonics and began with the early Cretaceous rifting associated with the opening with South Atlantic. The deposition of the three major formations occurred in each of the five off-lapping Siliciclastic Sedimentation Cycle (depobelts) that comprises the Niger Delta. Kogbe (1989); Obaje (2009) identified five major depobelts each with its own sedimentation, deformation, and petroleum history. They are the Northern Delta, Greater Ughelli, Central Swamp, Coastal Swamp, and Offshore. Hydrocarbons have been found in these depobelts of the Niger Delta, in good quality sandstone reservoirs belonging to the main deltaic sequence (Agoha et al., 2015).

Heinio and Davies (2006) and Obaje (2009) identified three different stratigraphic units in the Niger Delta;

1) A Miocene to Recent continental shallow massive sand sequence Benin Formation,
2) A coastal marine sequence of alternating sand and shale Agbada Formation deposited between Lower/Middle Miocene to Pliocene.
3) A basal marine shale unit the Akata Formation aged late Eocene to Recent.

The study area sits within the Benin Formation. It is branded by high sand percentage of about 70–100% and forms the top layer of the Niger Delta depositional sequence (Obaje, 2009). The Formation is about 2,100m thick with over 90 percent massive, porous, coarse sands with clay/shale inter-beddings (Short and Stauble, 1967) and the most prolific aquifer in the region.

Materials and Methods

The theory is based on the fact that the rays from a seismic source are refracted across the boundary between two layers where a discontinuity is encountered, here the discontinuity of interest is the boundary between the weathered and consolidated layer. This seismic energy returns to the surface after travelling through the ground along refracted ray paths. The first arrival of seismic energy that are detected at a receiver offset from a seismic source always represents either a direct ray or a refracted ray, this follows the determination of the exact time the source is produced

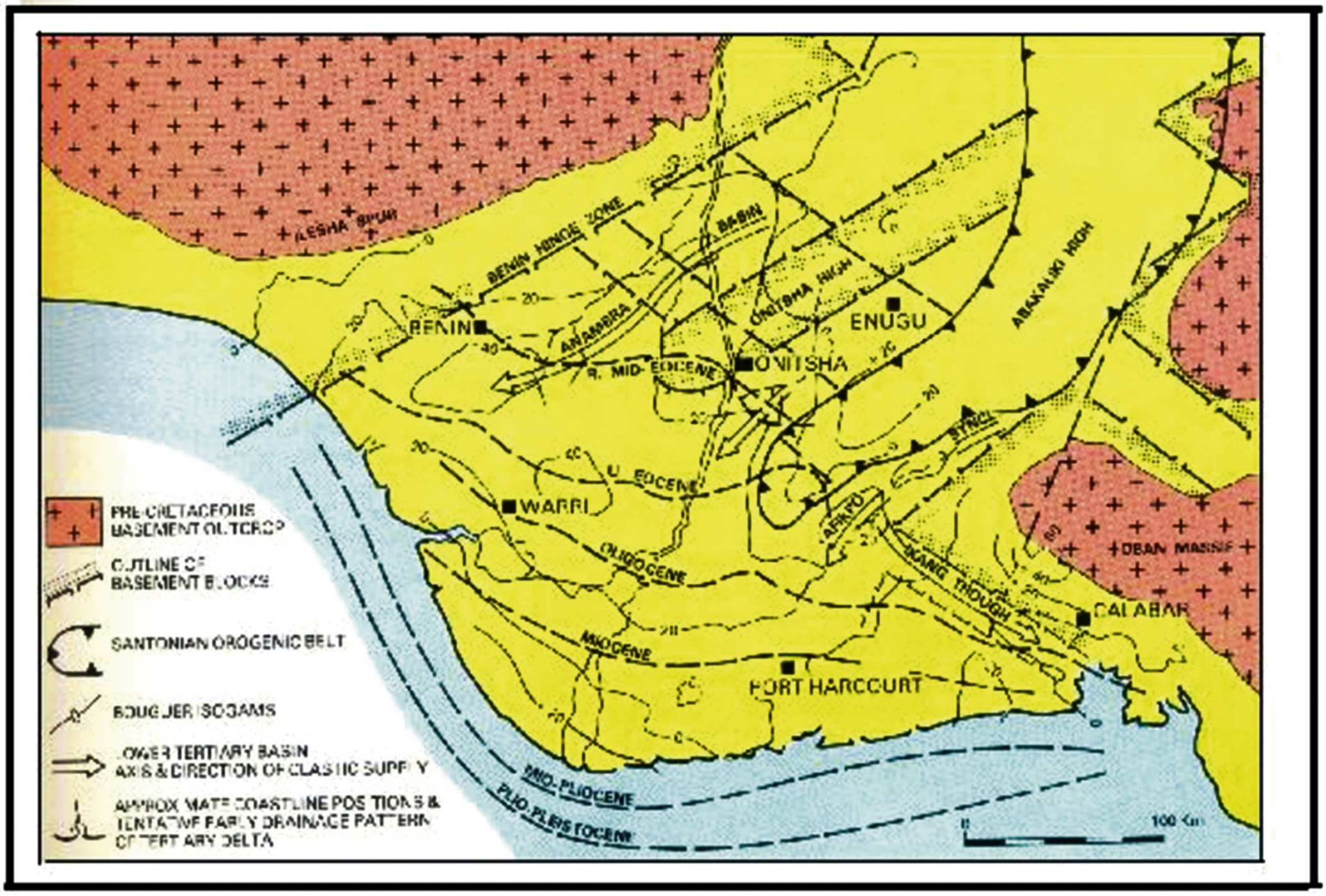

Fig. 1: Geologic map of Niger Delta (Oyedele, 2013)

and when energy reaches the receiver to analyse the first arrival times. This fact allows simple refraction surveys to be performed in which attention is concentrated solely on the first arrival of seismic energy, and time-distance plots of these first arrivals are interpreted to derive information on the depth to refracting interfaces.

The assumption of refraction geometry is that the subsurface is composed of a series of layers, separated by planar and possibly dipping interfaces. Also, within each layer seismic velocities are constant, and the velocities increase with layer depth. Finally, the ray paths are restricted to a vertical plane containing the profile line. Seismic refraction makes use of critically refracted, first-arrival energy only and the rest of the wave form is ignored. This is firmly backed by Snell's law.

The direct ray travels horizontally through the top of the upper layer from A to D at velocity V_1. The refracted ray travels down to the interface and back up to the surface at velocity V_1 along slant paths AB and CD that are

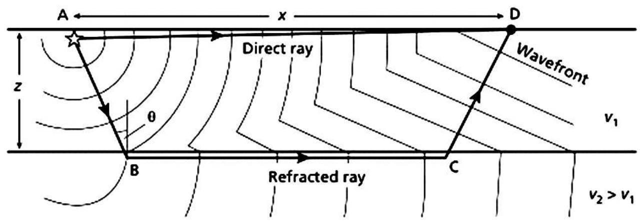

Fig. 2: Wavefronts for direct and refracted waves through a two-layer model

inclined at the critical angle, and travels along the interface between B and C at the higher velocity v_2. The total travel time along the refracted ray path ABCD is given as

$$T = T_{AB} + T_{BC} + T_{CD} \quad \text{..(1)}$$

$$T = \frac{z}{v_1 \sin\theta} + \frac{x - 2z\tan\theta}{v_2} + \frac{z}{v_1 \cos\theta} \quad \text{............(2)}$$

Note that sin $\theta = v_1/v_2$ (Snell's Law) and

$\cos = 1 - \sqrt{(v_1^2/v_2^2)}$, the travel-time equation may be expressed in a number of different forms, a useful general form given as: $T = \dfrac{x}{v_2} + \dfrac{2z\cos\theta}{v_1}$ ....(3)

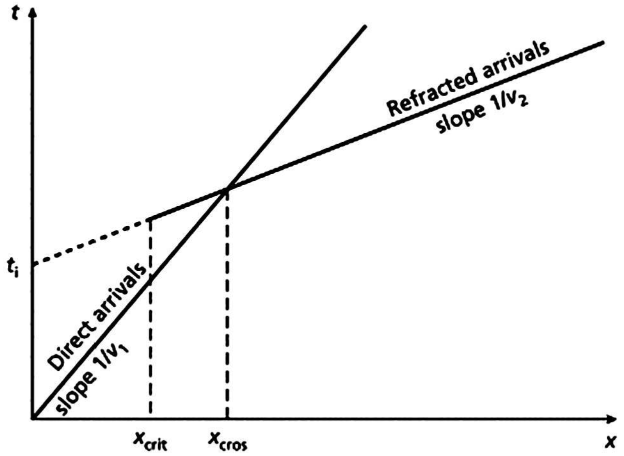

Fig. 3: Travel-time curves for the direct wave and the head wave from a single horizontal refractor.

Velocities of the layers are determined by plotting the travel times against the distance between the source and the receiver points (Figure 3). The reciprocal of the slope of the direct arrivals gives the velocity of the weathered (top) layer while the reciprocal of the slope of the refracted arrivals is used to compute the velocity of the second layer (bedrock velocity). The intercept time of the refracted arrivals and the two calculated velocities is used to calculate the depth to the interface. Plotting t against x (Figure 3), T_i is the intercept on the time axis of a travel-time plot or time–distance plot having a slope of $1/V_2$. The intercept time T_i, is given by

$$T_i = \frac{2z(v_2^2 - v_1^2)}{v_2 v_1} \quad \text{...(4)}$$

Solving for the refractor depth,

$$z = \frac{t_i v_2 v_1}{2\sqrt{v_2^2 - v_1^2}} \quad \text{...(5)}$$

For three layer formations with velocities V_1, V_2 and V_3 ($V_1 < V_2 < V_3$), the treatment is similar to that of two layer cases but somewhat complicated. The geometry of the ray path in the case of critical refraction at the second interface can be seen in (Figure 4). By analogy with the travel time equation for the two-layer case, the travel time along the refracted ray path ABCDEF to an offset distance x, involving critical refraction at the second interface, can be written in the form of

$$T = \frac{x}{v_3} + \frac{2z_1\cos\theta_{13}}{v_1} + \frac{2z_2\cos\theta_{23}}{v_2} \quad \text{.........................(6)}$$

Where the angle of incidence of the ray on the upper interface

$\theta_{13} = \sin^{-1} v_1/v_3$ *and* **on the lower interface** $\theta_{23} = \sin^{-1} v_2/v_3$

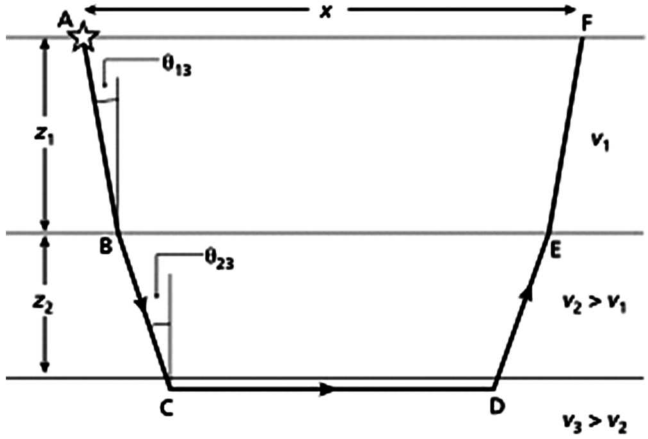

Fig. 4: Ray path for a wave refracted through the bottom layer of a three-layer model (Kearey *et al.*, 2002).

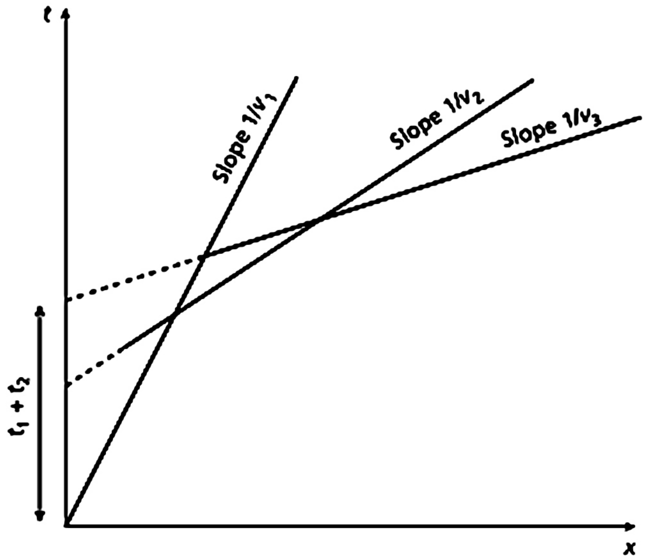

Fig. 5: Travel-time curves for three-layer case models (Kearey *et al.*, 2002)

For a three-layer case as given in (Figure 5), the parameters can be computed using the following equation as given by Dobrin and Savit (1988) below:

$$Z_2 = \frac{1}{2}\left(\frac{T_{i2} - 2z\sqrt{(V_3^2 - V_1^2)}}{V_3 V_1}\right)\frac{V_3 V_2}{\sqrt{(V_3^2 - V_2^2)}} \quad \text{..........(7)}$$

Where T_i and T_{i2} = intercept times on the distance-time graph.

V_1 = Velocity of the first weathering layer.

V_2 = velocity of the second weathering layer.

V_3 = velocity of the bedrock.

Total thickness (m) of the weathering layer = $Z_w = Z_2 + Z$.

The uphole method of seismic refraction is based on the same principle of travel time as regards conventional refraction surveys. Thus from (Equ. 5) the parameters (V_w, D_w and V_B) can be computed from the uphole survey data (Figure 6). Here also, the reciprocal of the slopes of the segment XY and YZ equals V_W and V_B respectively while XW is the thickness of the weathered layer, where W is the base of the low velocity layer.

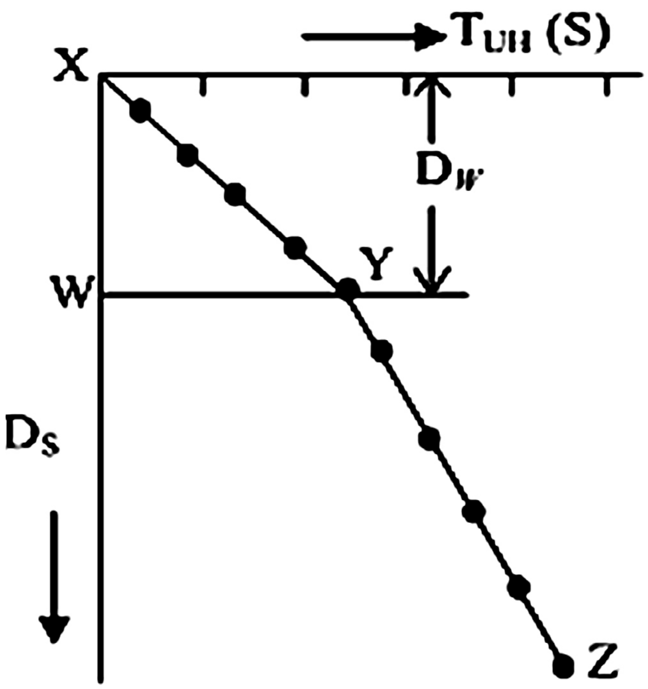

Fig. 6: Uphole survey time-depth relationship

Data Acquisition and Processing

The acquisition was carried out in a pre-designed grid format as shown below, which stipulated the positions of borehole and refraction points.

In the acquisition of the forward and reverse profile data, the OYO McSEIS-SX seismograph was used in recording the data via a field digital unit (FDU) cable with take-out outlets connected each geophone. 24 separate vertical electromagnetic geophones were buried to the ground on a straight line and in a North-South orientation at 12.5 meter Interval covering up to 300 meters. The seismic refraction energy source was generated by a small explosion of a dynamite of mass 0.25kg. They were buried to the ground with the aid of hand Augers at 1m depth and 1m away from the last Geophone at both end of the spread and the shot hole was tamped to avoid loss of energy and each shot detonated at the platform and taken in turns

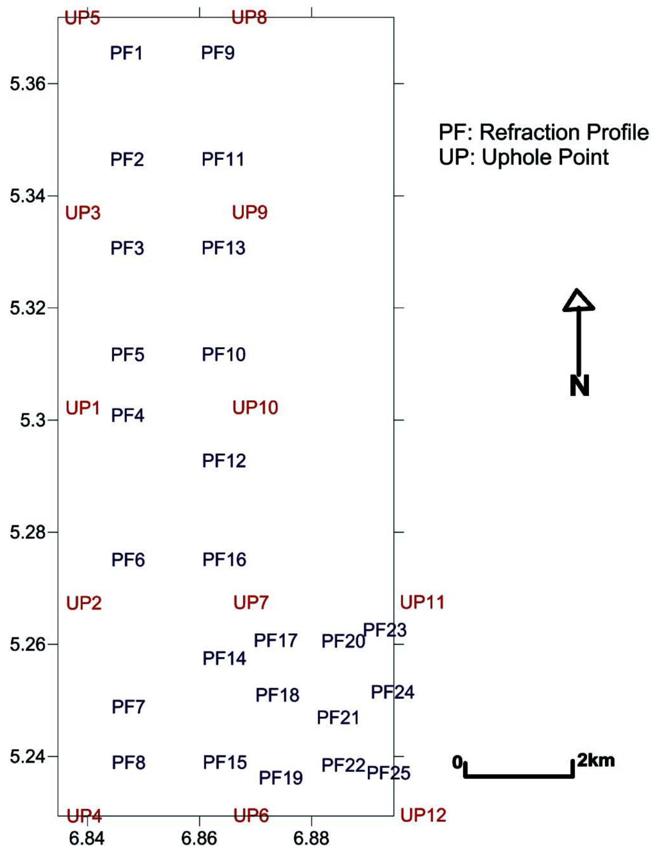

Fig. 7: Post map of the acquisition points

In acquiring the uphole data, twenty-five (25) 66m-deep boreholes were logged from the top to 60m depth at 0.5m interval from 0 to 4.5m, 1m interval from 5.5 to 9.5m, 2.5m interval from 9.5 to 44.5m and 5m interval from 44.5 to 59.5 depth levels. The hydrophone spread includes 32 hydrophones calibrated to fit the logging depth intervals attached to a recording cable, marine rope and a cylindrical weights (sinkers) of net 20kg. Drilling was done with a semi-automated method of drilling in which 5.5' water pump, swivel head, delivery hoses, drill stems (2inches wide and 3m long), drill bits (2 and 3 Wings), water-based drilling mud and PVC plastic casings were installed in the hole to prevent the hole from caving in and collapsing before lowering the hydrophone components. Ditch cuttings were also picked from various depths during drilling. Explosive detonator seismic source of mass 0.25kg generated the refraction energy inside the borehole buried to a depth of about 2m beneath the earth surface with an offset distance of 2m from the top of the borehole. Data recording was carried out using the RAS-24 seismograph and a laptop computer connected to it after detonation of charges.

The same processing flow was used for the two types of surveys employed in this study. The files were processed using the interpex IXseg2segy seismic wave processing software. The IXseg2segy (interpex limited seismic shot conversion) is a seismic wave processing software that handles digital waveform data obtained by seismograph. After the data were recorded, the first breaks on the seismographs were picked from the recorded traces. The velocities of the layers were filtered using velocity model function with a specified window length which was determined according to the degree of smoothness required. The picked travel times were corrected to account for the 2m offset distance from the seismic source to the borehole head. This correction approximates as though the data were recorded with the seismic source placed exactly at the end of the geophone and borehole head for refraction and uphole survey respectively.

The recorded travel times were plotted against the source-receiver distance using Upshere seismic refraction software. The graphs were plotted for each refraction and uphole points, and the layer velocities and thicknesses were obtained. The slopes of the two layers were automatically calculated and the reciprocal of the slopes gave the velocities of the weathered and consolidated layers. The depth to refractor (thickness) was also calculated from the point of intersection of the two slopes (point of inflexion). This was done for all the shot points. The classification of the different lithologic sediments into grain sizes was carried out using the standard proposed by Wentworth.

The thickness and velocity models were constructed using the geo-statistical krigging technique. This was applied in the Surfer 13 software and used to generate the thickness and velocity models.

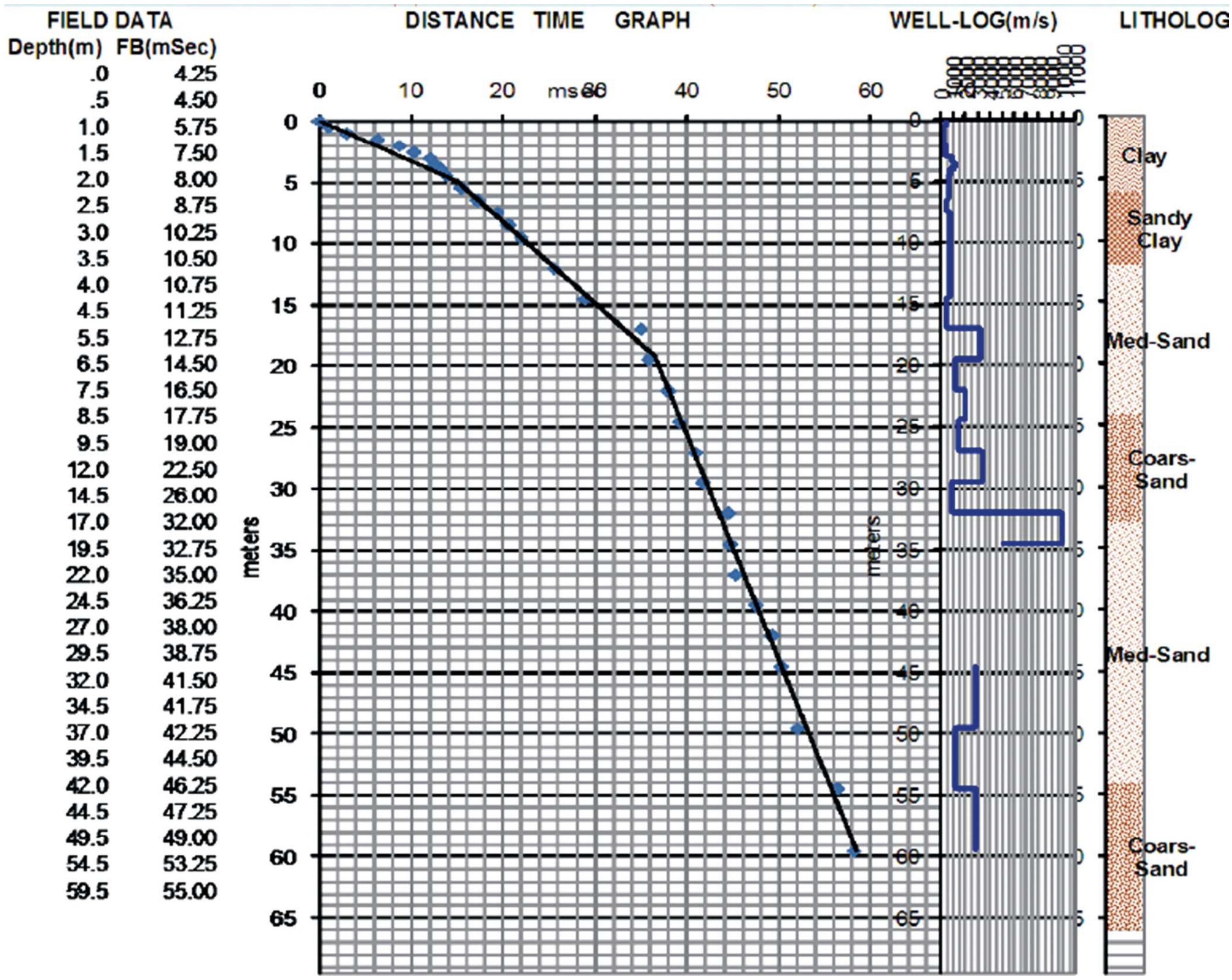

Fig. 8: Selected distance-time graph for Uphole refraction method showing a three layer model

FIELD DATA

OFFSET	FB(mSec)	FB_2(Reverse)
12.5	27.20	29.60
25.0	51.80	50.60
37.5	89.00	83.00
50.0	115.00	110.40
62.5	132.20	129.00
75.0	140.80	142.00
87.5	143.60	146.80
100.0	155.40	151.60
112.5	164.60	156.20
125.0	168.40	165.40
137.5	171.60	167.00
150.0	175.60	173.60
162.5	181.60	181.60
175.0	193.60	186.40
187.5	200.80	196.40
200.0	205.60	204.40
212.5	212.60	209.00
225.0	223.40	219.80
237.5	229.40	223.40
250.0	235.40	233.00
262.5	242.60	242.40
275.0	249.80	248.60
287.5	254.60	258.20
300.0	262.60	263.60

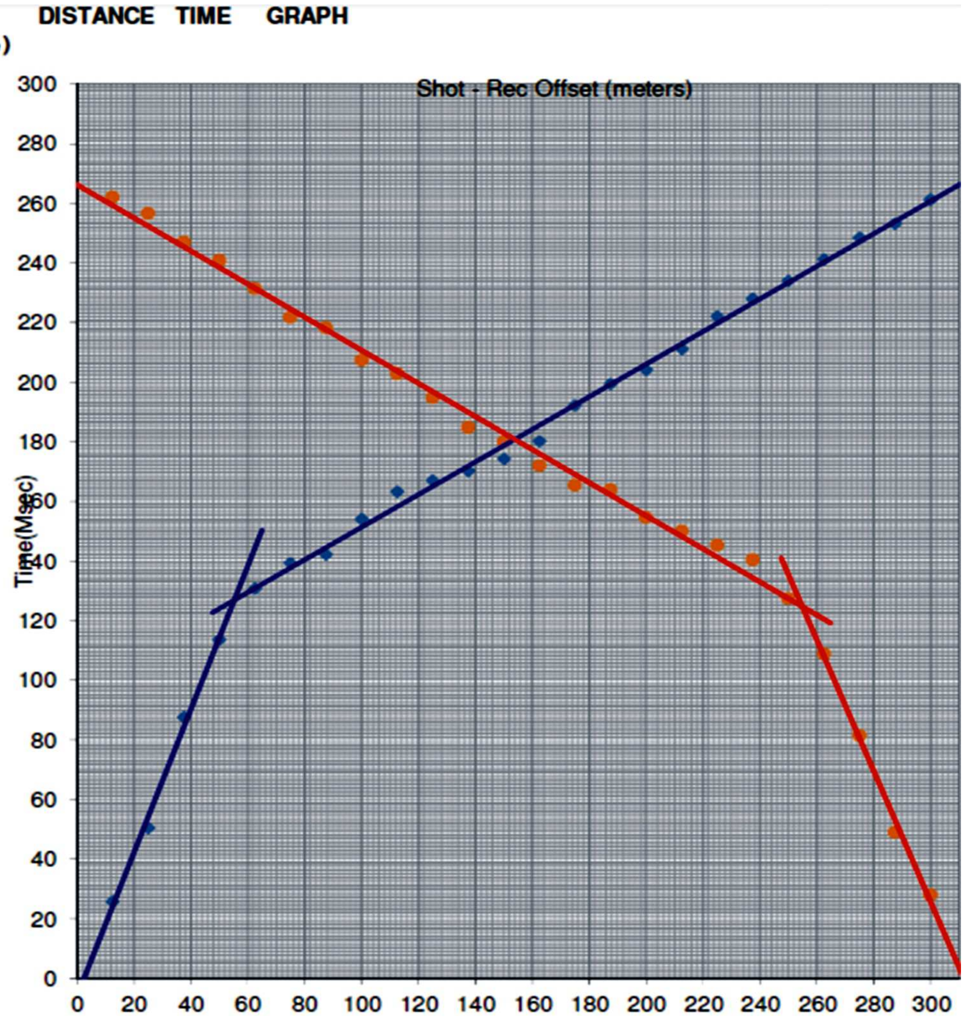

Fig. 9: Selected distance-time graph for Forward/Reverse refraction profile showing a two layer model

Results

The time-distance plots for first break arrivals as observed at the 25 refraction points and 12 uphole points are presented below, the forward/reverse refraction data reveal a 2-layer model while the uphole data exhibit a 3-layer model. The Uphole survey has more detailed information on the subsurface because it was acquired in-situ (in the vicinity of the subsurface materials). This scenario where the Forward/Reverse Profile Survey could not identify the sub weathering layer could most probably be as a result of the Geophone spacing being too large to identify the missing layer.

Discussion

Weathering Layer

The velocity of the first weathering layer obtained from the Uphole Survey ranges from as low as 180m/s in the lower south-eastern part of the area and 448m/s in the middle-down south of the area south of the area with an average of 339.25m/s (Figure 10). Towards the Centre of the southern half there is an observed close variation in velocity which infers the presence of subsurface homogeneity.

The thickness of the weathered layer (Figure 11) varies from 1.4m in the north-eastern axis to 5.8m around the southwest axis around the area with an average of 3.13m. The thickness model map reveals a general thickness decrease from the west to the east with thickest margins around the north and south of the area.

Sub-weathering Layer

The Sub-weathering layer (Figure 12) which has a velocity that ranges from 446m/s in the southeast of the

Table 1: Summary of Uphole Profile Survey

S/N	Elevation	Name	Layer 1 velocity (m/s)	Layer 2/Sub-Weath. velocity (m/s)	Layer 3/Consolidated Layer velocity (m/s)	layer 1 thickness h1(m)	layer 2 thickness h2(m)	LVL/Consolidated Interface Elevation H=h1+h2(m)
1	61.38	UP1	363	660	1848	4.9	14.2	19.1
2	58.44	UP2	393	944	1638	5.8	17	22.8
3	63.42	UP3	368	701	1939	5.4	14.6	20
4	57.2	UP4	201	684	1821	2.7	15	17.7
5	65.25	UP5	423	630	1651	4.9	12.2	17.1
6	56.76	UP6	448	746	1544	1.9	7.3	9.2
7	59.1	UP7	410	683	1780	2.1	11.5	13.6
8	70.21	UP8	379	785	1923	2.2	2.5	4.7
9	64.18	UP9	334	772	1641	2	19.6	21.6
10	63.14	UP10	370	681	1705	1.7	7.8	9.5
11	59.86	UP11	183	579	1650	2.1	8.3	10.4
12	58.82	UP12	199	446	1416	1.9	4.8	6.7

Table 2: Summary of Forward/Reverse Profile Survey

S/N	Elevation	Name	layer 1 velocity (m/s)	Layer 3/Consolidated Layer velocity (m/s)	layer 1 thickness (m)
1	64.83	PF1	429.5	1759	15.8
2	64.01	PF2	453	1789	15.2
3	63.5	PF3	432.5	1791	21.05
4	61.97	PF4	442	1679	14.6
5	64.68	PF5	547	1804	27.65
6	58.74	PF6	441	1742	15.7
7	58.71	PF7	443.5	1800	15.95
8	58.4	PF8	383.5	1846	11.95
9	69.71	PF9	466.5	1708	17.6
10	63.36	PF10	461.5	1652	15.05
11	64.79	PF11	526.5	1742	25
12	62.94	PF12	465.5	1743.5	20.35
13	65.18	PF13	478	1749	18.25
14	58.2	PF14	459.5	1713	13.3
15	59.18	PF15	484	1696	13.3
16	60.42	PF16	447	1726	14.55
17	44.1	PF17	480	1774	12.65
18	46	PF18	412	1754	11.5
19	43.01	PF19	396.5	1813	10.85
20	47.1	PF20	582	1756	14.45
21	44	PF21	481	1734	12
22	46	PF22	550.5	1758	13.55
23	41	PF23	461	1757	9.9
24	43	PF24	562	1799	14.45
25	41	PF25	504	1763.5	13.3

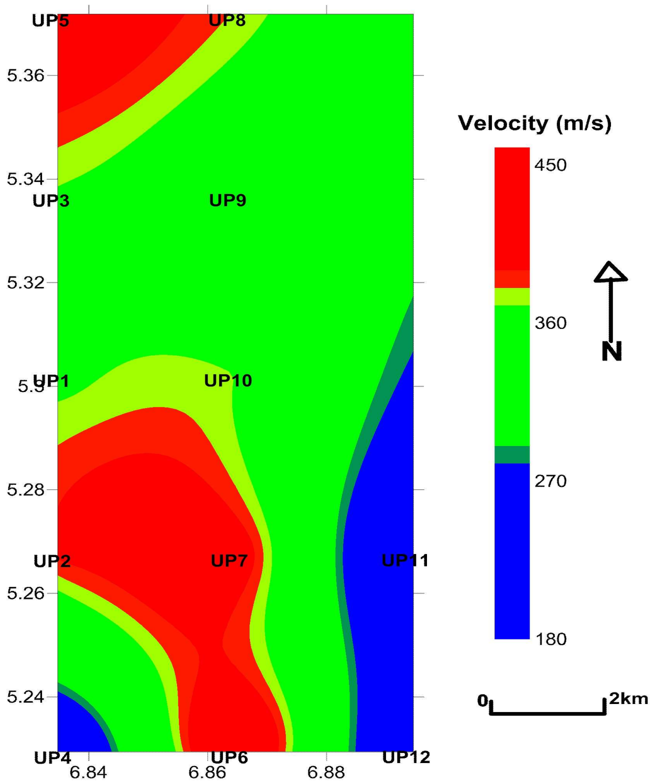

Fig. 10: Weathered Layer Iso-Velocity Model from Uphole Survey

area and 944m/s in the southwest axis of the area and a mean velocity of 629.58m/s. A close variation of velocity is observed in the northern half of the area which suggest presence of homogeneous materials.

The thickness of the sub-weathering layer (Figure 13) ranges from 19.6m to 4.8m with a mean thickness of 11.23m. The thickness model of the Sub-weathering layer shows almost the same pattern as that of the Weathered layer, with the Heart of the Northern half, north of the area as the thickest followed by the South-western axis and a general reduction in thickness due east of the area.

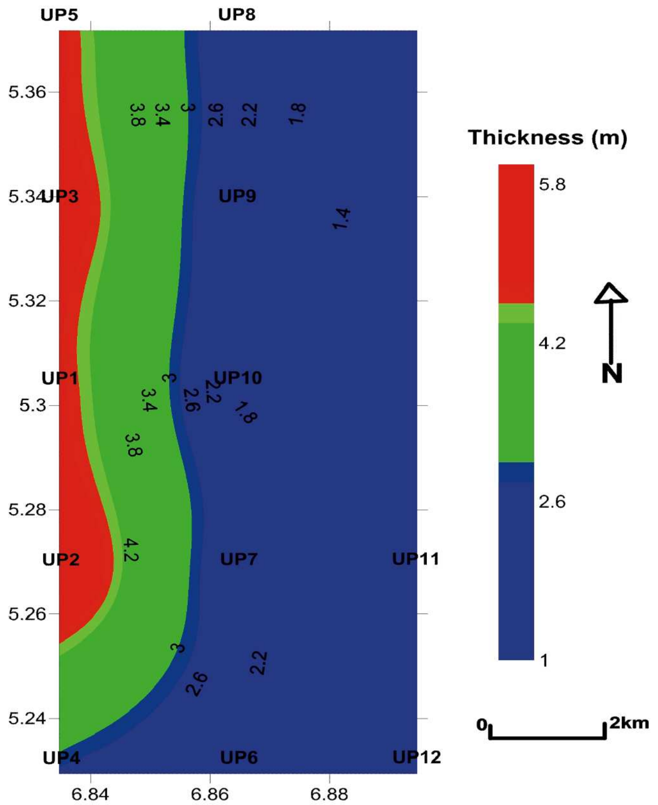

Fig. 11: First Layer Isopach Model from Uphole Survey

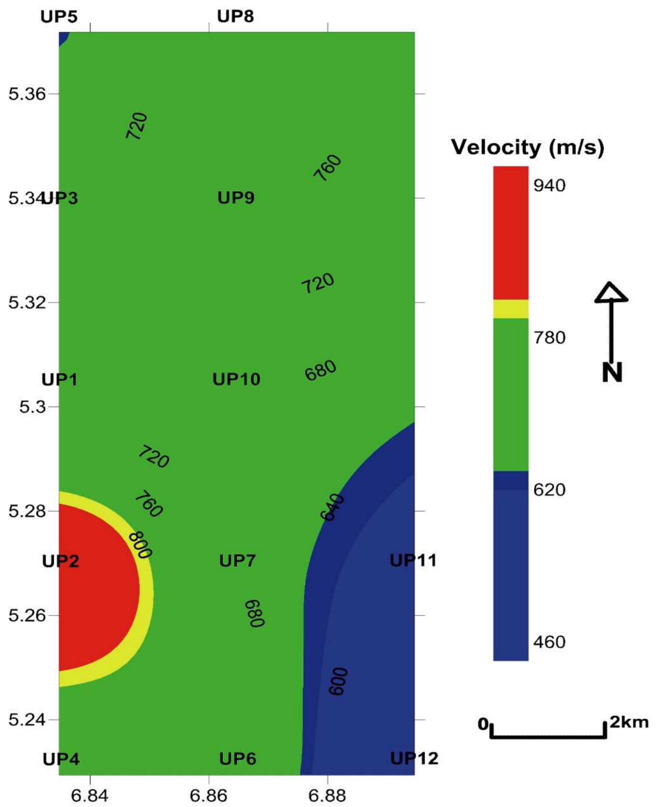

Fig. 12: Sub-weathering Layer Iso-Velocity Model from the Uphole Survey

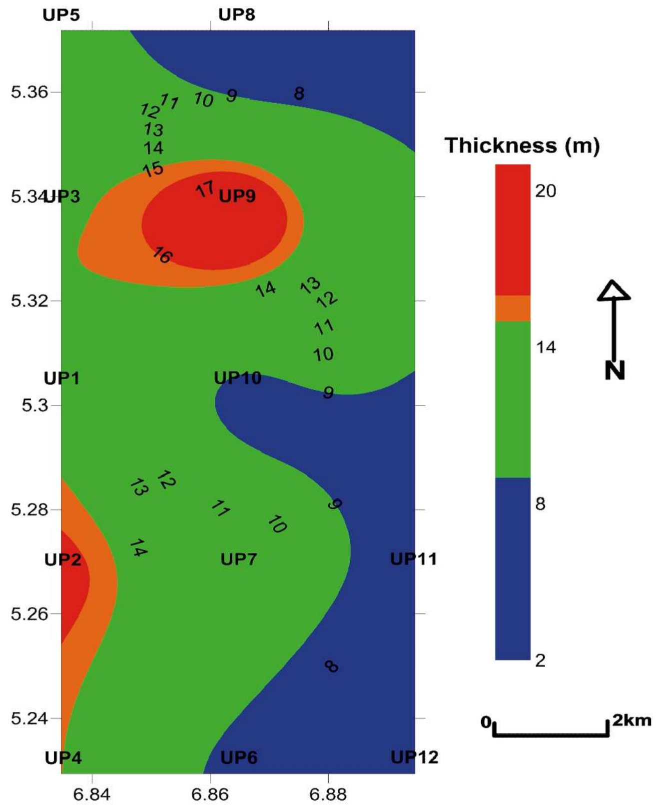

Fig. 13: Sub-weathering Layer Isopach Model from the Uphole Survey

Consolidated Layer

The velocity of the consolidated layer (Figure 14)

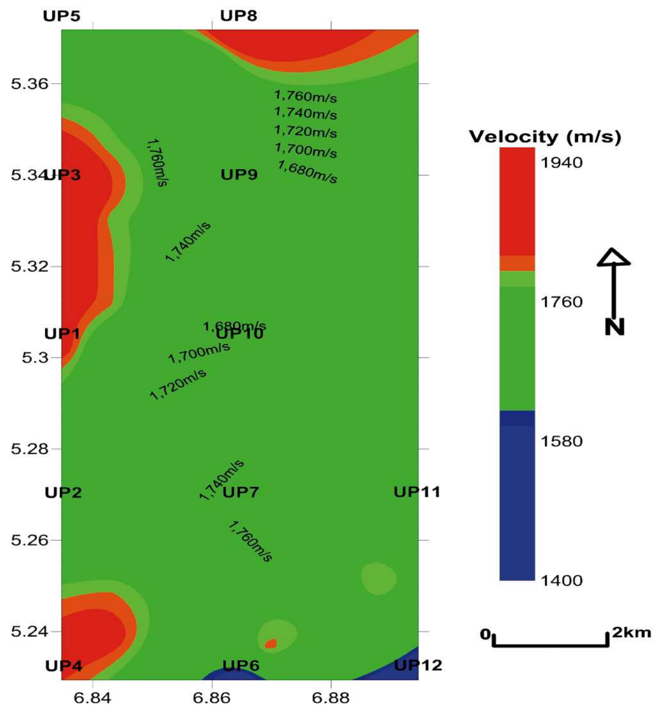

Fig. 14: Consolidated Layer Iso-Velocity Model

ranges from 1416m/s around the south-eastern portion of the area to 1923m/s towards the north central segment of the area. It could be observed that the layer is sufficiently competent judging from the seismic velocity distribution across the study area. The Eastern portion has the least competent bedrock depicting a maximum velocity of 1660m/s. This was closely followed by the south-eastern portion with a velocity of 1440m/s. The decrease in velocity indicates the area is less compacted than the other areas. However, due to the high velocities present it further affirms the competence of this layer.

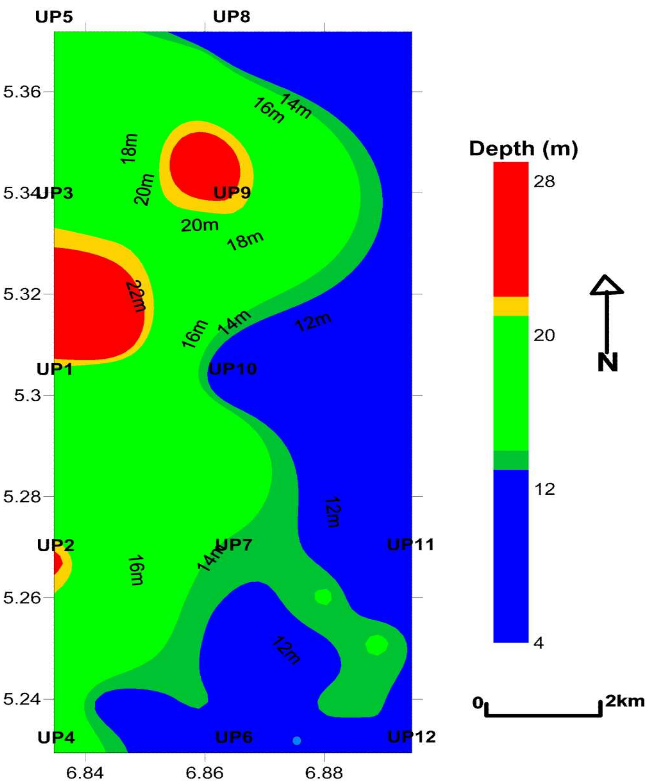

Fig. 15: Depth to Consolidated Layer Model

The depth to consolidated layer (Figure 15) ranges from as low as 4.7m to 27.65m. The farthest region to the consolidated layer is mainly in the western half of the study area. The shallow depths can be found towards the southern region of the area.

Determination of the True Velocities of the Consolidated Layer

The angle of dip and incidence of the layer were determined from the formulae used by Vale and Smith (1961) below;

$$\theta = \frac{1}{2}\left[\sin^{-1}\left(\frac{V_1}{V_{2d}}\right) + \sin^{-1}\left(\frac{V_1}{V_{2u}}\right)\right] \quad \text{.................................(8a)}$$

$$\gamma = \frac{1}{2}\left[\sin^{-1}\left(\frac{V_1}{V_{2d}}\right) - \sin^{-1}\left(\frac{V_1}{V_{2u}}\right)\right] \quad \text{.................................(8b)}$$

The true velocity of consolidated layer was gotten from the expression below;

$$Vr = \left(\frac{V1}{\sin\theta}\right)$$

Where

V_1 = velocity of the weathered/first layer

2_{Vd} = down-dip velocity

2_{Vu} = Up-dip velocity

θ = incidence angle

γ = dip angle

V_r = true velocity of the consolidated layer

Table 3: Computed results for the true velocity of the consolidated layer.

Code	V1	V2d (m/s)	V2u (m/s)	DIP (degree)	Incidence (degree)	Vr (m/s)
PF1	429.5	1740	1759	0.08	14.21175	1749.447
PF2	463	1636	1789	0.72	15.719435	1708.948
PF3	432.5	1791	1821	0.12	13.856827	1805.872
PF4	422	1679	1712	0.14	14.413523	1695.334
PF5	547	1766	1904	0.67	17.369655	1832.279
PF6	441	1742	1786	0.18	14.47989	1763.716
PF7	443.5	1800	1805	0.02	14.243755	1802.496
PF8	383.5	1718	1846	0.45	12.444414	1779.646
PF9	466.5	1708	1777	0.32	15.535051	1741.791
PF10	461.5	1652	1839	0.84	15.377872	1740.303
PF11	526.5	1741	1742	0.01	17.597338	1741.5
PF12	465.5	1714	1773	0.27	15.490053	1742.982
PF13	478	1713	1785	0.34	15.867851	1748.229
PF14	459.5	1683	1743	0.28	15.564817	1712.454
PF15	484	1685	1707	0.11	16.582156	1695.925
PF16	447	1699	1753	0.24	15.013461	1725.562
PF17	480	1740	1808	0.31	15.704635	1773.323
PF18	412	1745	1763	0.07	13.585613	1753.952
PF19	396.5	1748	1879	0.46	12.64624	1811.075
PF20	582	1734	1778	0.25	19.359257	1755.707
PF21	481.5	1635	1833	0.95	16.178335	1728.111
PF22	550.5	1747	1769	0.12	18.24934	1757.927
PF23	461	1745	1769	0.11	15.212027	1756.915
PF24	562	1797	1801	0.02	18.203695	1798.998
PF25	504	1747	1780	0.16	16.607956	1763.339

The true velocity distribution (Figure 16) shows similar distribution as that of the apparent velocity of the consolidated layer. A maximum velocity of about 1835m/s is located west of the northern half of the area and a minimum velocity of about 1695m/s around the southern half and in the middle of the area, eastward. The model reveals variations in velocity across the area but this variation are small and suggest relative homogeneity of the layer.

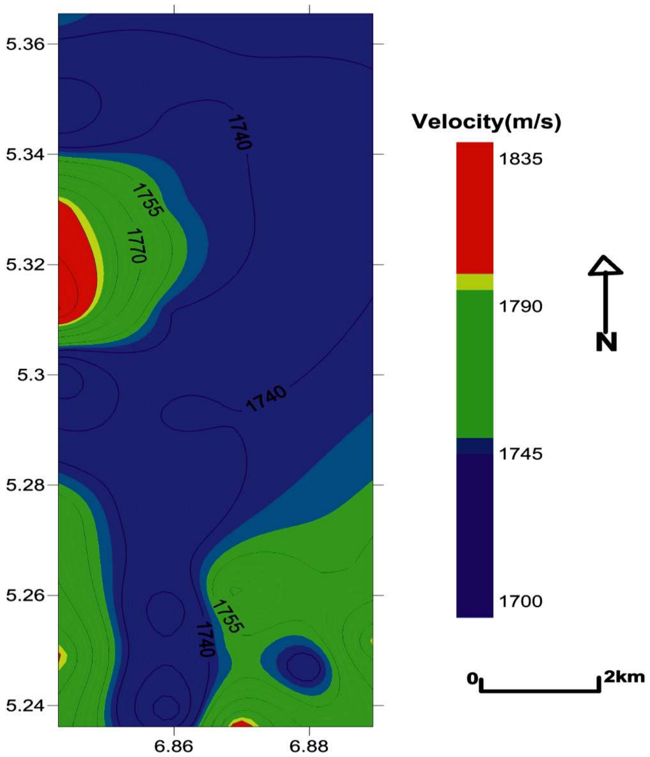

Fig. 16: True Iso-Velocity Model of the Consolidated Layer

S-wave Derivation and Poisson Ratio

The S-wave parameter of the consolidated layer was determined in order to ascertain their variation within the study area and also to determine the Poisson ratio of the materials within the consolidated layer. The S-wave property of the consolidated layer is seen in (Table 4), from the application of The Castagna mud line relationship equation

$$V_P = 1.16V_S + 1360m/s \quad (9)$$

The result of the S-wave velocity of the consolidated layer (Figure 17) shows S-wave variation from 48.28m/s to 499.14m/s and an average of 328.15m/s. The mid-southern region of the study are has the highest S-wave velocity and the south-east region of the area has the Lowest S-wave velocity. The general low S-wave velocity of the layer suggest that the layer is loosed/unconsolidated.

Poisson Ratio

The Poisson equation was applied to the seismic wave velocities (P-wave and S-wave) to determine the Poisson ratio of the consolidated layer and a model of Poisson variation in the study area is seen below in (Figure: 18) using

Table 4: Derived S-wave Property of the Consolidated Layer

Point Code	Layer 3 / Consolidated Layer P-wave velocity (m/s)	Consolidated layer S-wave velocity (m/s)
UP1	1848	420.69
UP2	1638	239.66
UP3	1939	499.14
UP4	1821	397.41
UP5	1651	250.86
UP6	1544	158.62
UP7	1780	362.07
UP8	1923	485.34
UP9	1641	242.24
UP10	1705	297.41
UP11	1650	250.00
UP12	1416	48.28
PF1	1759	343.97
PF2	1789	369.83
PF3	1791	371.55
PF4	1679	275.00
PF5	1804	382.76
PF6	1742	329.31
PF7	1800	379.31
PF8	1846	418.97
PF9	1708	300.00
PF10	1652	251.72
PF11	1742	329.31
PF12	1743.5	330.60
PF13	1749	335.34
PF14	1713	304.31
PF15	1696	289.66
PF16	1726	315.52
PF17	1774	356.90
PF18	1754	339.66
PF19	1813	390.52
PF20	1756	341.38
PF21	1734	322.41
PF22	1758	343.10
PF23	1757	342.24
PF24	1799	378.45
PF25	1763.5	347.84

$$\sigma = \frac{V_p^2 - 2V_S^2}{2\left(V_p^2 - V_s^2\right)} \quad (10)$$

The Poisson ration of the competent layer gotten from the iteration ranges from 0.46 to 0.50 with an average of 0.48. The values fall within 0.1and 0.5 confirm the loose nature of the layer but shows the layer is highly incompressible.

The refraction survey was used to determine the velocity of the weathered and consolidated layers as well as the thickness and depth of weathered layers and depth to the consolidated layer respectively in the study area.

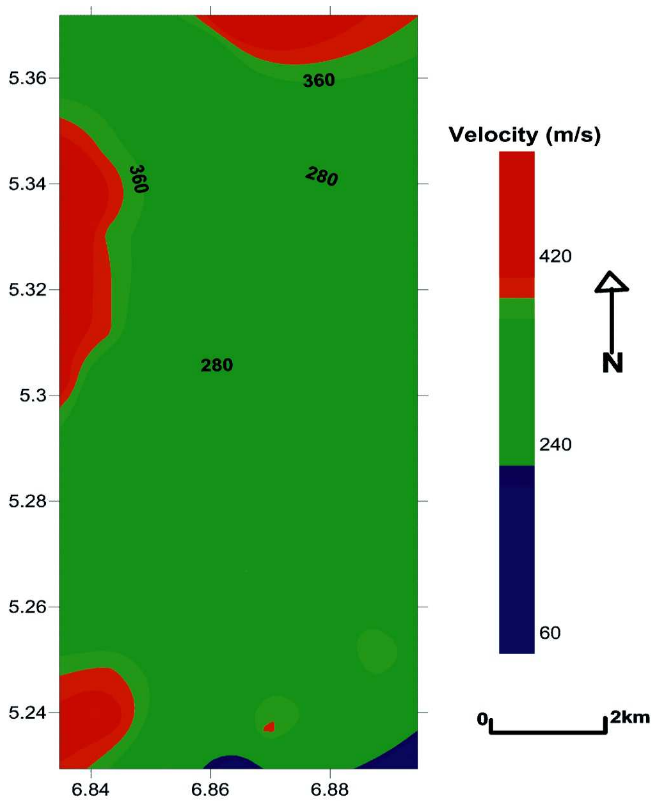

Fig. 17: S-Wave Iso-Velocity Model of the Study Area

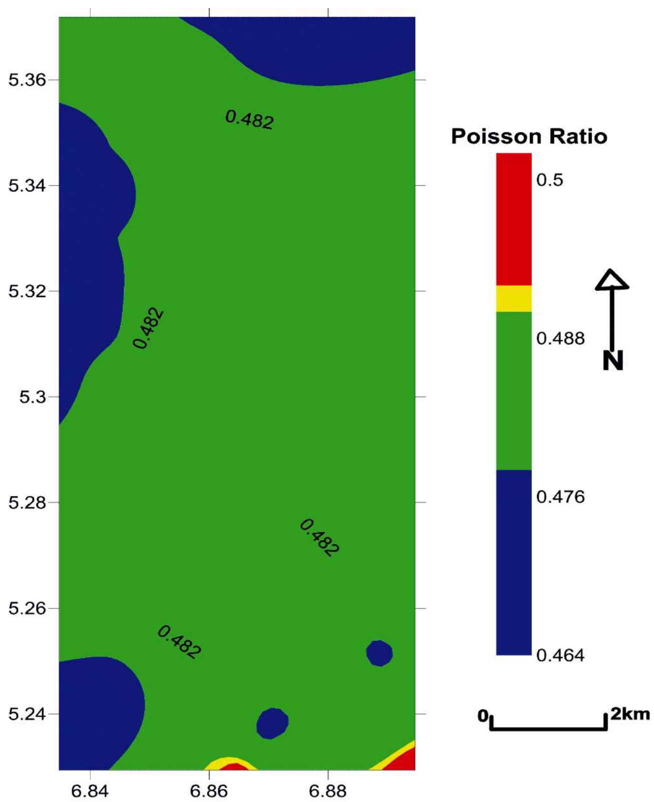

Fig. 18: Poisson Ratio Distribution of the Consolidated layer Across the Study Area

The lithology samples gotten from the borehole drilling reveal that the study area is made up of mainly sand, gravel and clay. For the weathered layer, which shows sloping thickness due east of the area and velocity and thickness average of 339.25m/s and 3.13m respectively were observed.

The Sub-weathering layer (partially weathered) characterized by clayey sands has an average thickness of 11.23m and velocity average of 692.58m/s. This is underlain by a consolidated layer with an average velocity of 1740.65m/s. The velocity and the lithology of this layer underscores the competence of this layer to engineering and construction activities. This is also confirmed through the generation of the true velocities which falls between the range of 1835m/s and 1695m/s. Also the velocity of sand varies from 1500m/s to 2000m/s according to Kearey, et al., (2002).

However, to justify the competence of the third layer, S-wave property of the area were derived with an average of 328.15m/s and strongly suggest the layer is made of loosed sediments. Consequently, Poisson ration value of the layer averaged 0.48. Although the value shows the layer is incompressible and justify the competence of the layer it also confirm the loose nature of the sediments. This value falls within 0.3 to 10 for competent sand according to Bowles, (1996).

Mathematical computations were employed in the determination of the average depth from the surface to the interface between the Weathered layers and the consolidated layer; an average of 15.14m from a minimum depth of 4.7m to 27.65m maximum in the northern half of the area with about 70.2% of the depth to the competent layer falling between 11m and 19.99m were observed and reveals a general thinning of the weathered layer and decrease in distance to the consolidated layer due east of the area (Figure 15).

Conclusion

From this study, it follows that depth of piling or excavation of about 19m and beyond would be most suitable for large engineering structures coming into the area and also be enough to eliminate the effects of weathering layer on reflection seismic data.

Other geophysical methods should as well be applied in the area in other to further ascertain the credibility of this work.

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Landslide and Landslide Dam Risk Reduction Strategies for Sustainable Infrastructural Development: *Lessons From Japan for a Safer Nigeria*

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Abstract

Landslides and landslide dams commonly occur in many mountainous watersheds of Japan. Their occurrence and failure have led to the loss of human lives in addition to causing significant damage to infrastructures. However, Japan has developed several countermeasures against disasters caused by landslides and landslide dams, in addition to the application of early warning systems for disaster risk reduction. This paper, therefore, presents an overview of the various landslide and landslide dam risk reduction strategies used for the mitigation of disasters in Japan and the lessons Nigeria can learn from them.

Keywords: *landslides, landslide dams, disaster risk reduction, early warning systems*

Introduction

Natural disasters remain the most significant problem affecting the life and wellbeing of the world's human population. Every year, thousands of fatalities are recorded while several billions of US dollars are lost as a result of the direct impact of various kinds of disasters including climatological, hydrological, geophysical, and meteorological disasters. For instance, on 14 March 2019, the African continent witnessed one of the worst tropical cyclones that have ever been recorded on the continent and the Southern Hemisphere. The storm – named Tropical Cyclone Idai – passed over three southern African nations of Malawi, Mozambique, and Zimbabwe. The arrival of the storm near the city of Beira in Mozambique brought strong winds and triggered severe flooding that claimed about 598 lives and affected about 1.9 million people. Cyclone Idai also caused 415 casualties in Zimbabwe, 60 casualties in Malawi and caused extensive infrastructural damage in these three affected countries with estimated total damage of about 1 billion US dollars. Similarly, the 11 March 2011 Great East Japan Earthquake (M_w 9.0) has been globally regarded as the worst mega-disaster and the most expensive natural disaster in human history. The earthquake and tsunami claimed more than 16,000 lives and caused estimated total damage of about 235 billion US dollars (Mimura et al. 2011; Norio et al. 2011).

Nigeria faces all kinds of natural and anthropogenic disasters including flash floods, gully erosion, droughts, landslides, and building/dam collapse. Several demographic and geographic factors influence the distribution of natural hazards in Nigeria. More than 45 percent of the country's 200 million people live around the floodplains of the Niger and Benue Rivers. Consequently, floods have become the most common and recurring natural disaster in Nigeria. For example, the intensity and duration of the 2012 rainfall event in Nigeria triggered several flood disasters that resulted in the overflow of numerous hydropower and irrigation dams in Nigeria (Kiri, Kainji, Shiroro, and Jebba dams) and Cameroun (Lagdo dam). The disaster claimed 363 lives, injured 5,851 people, affected 3,891,314, and displaced 3,871,53 people (NEMA 2013). In contrast, more than 80 percent of the landmass in the northern region of Nigeria is affected by drought conditions, while landslides and deep-seated gullies occur in the southeastern region. Several cases of seismic activities in north-central Nigeria, which have increased in recent years, coupled with the demand for the construction of more hydropower and irrigation dams, have increased natural hazards and disaster risk potentials of the country's human population of almost 200 million. Therefore, Nigeria needs to develop a robust disaster risk reduction strategy that would increase community resilience to disasters triggered by floods, droughts, landslides, and gully erosion.

A Unique Geologic Setting

Japan's geography, climate and geology have made it vulnerable to frequent natural disasters such as tsunamis, earthquakes, typhoons, landslides, volcanic eruptions, and torrential rainfall events. The island nation covers only 0.25% of the Earth's landmass and experiences about 20% of the world's earthquake with magnitude greater than 6. The Japanese archipelago is located within the circum-Pacific volcanic belt which is also referred to as the *Pacific Ring of Fire*, where frequent crustal movements have resulted in a large number of earthquakes and volcanic eruptions. The geologic and geomorphic features of the Japanese islands have been modified by the subduction of the Pacific Plate and the Philippine Sea Plate. These have triggered several natural disasters including earthquakes, volcanic eruptions, typhoon-induced landslides and debris flows, and landslide dam failures with catastrophic outburst floods (Sassa 1998, 2005; Chigara and Yagi 2006; Hayashi et al. 2013).

The Japanese islands are characterized by steep and rugged watersheds which are usually affected by high-intensity rainfall and snowmelt. These characteristic features have resulted in high fluvial processes in many mountain slopes, which have been initially destabilized by repeated earthquake ground motions. Oguchi et al. (2001) identified six (6) major geomorphological and hydrological factors influencing the rate of fluvial processes in Japan. These include high-intensity precipitations, catastrophic hydro-geomorphological events associated with earthquakes and volcanic eruptions, high sediment yield, steep watersheds, large flood events, and frequent slope failures and landslides.

The occurrence of landslides and landslide dams in several parts of the Japanese islands has been attributed to the abundance of steep, unstable mountain slopes with average modal angle of 35° (Katsube and Oguchi 1999), high-gradient streams and narrow gorges, coupled with frequent hydrologic and seismic events (Swanson et al. 1986; Oguchi et al. 2001). Historical data from Tabata et al. (2002) and Inoue et al. (2013) indicate that the occurrence of landslide dams in Japan predates the 9th Century BC, and occur mostly in active tectonic boundaries. A typical example is the high distribution of landslides and landslide dams in Northern Nagano Prefecture, Japan. The occurrence of landslide dams in this region has been attributed to the geomorphic history of the area, which is located in the great central belt of Japan - *the Fossa Magna* (Mizuyama et al. 2011; Inoue et al. 2013).

Landslide dams are formed by the partial or complete blockage of stream-channels by displaced earth materials. The phenomena can occur in several kinds of geomorphic settings. These settings are usually characterized by steep-walled narrow valleys bordered by high rugged mountains; a feature commonly found in geologically active regions where uplift or mountain building associated with active tectonic processes increase the likelihood of initiation of river-damming landslides (Costa and Schuster 1988). Also, these regions are characterized by highly weathered and unstable source materials, including fractured and hydrothermally altered bedrock and thick Quaternary sediments. The majority of the inherent factors that trigger landslide damming events in these regions include increases in local relief and hillslope gradients, and incessant weathering that reduces the shear strength of hillslope materials. These overarching factors are exacerbated by high orographic precipitations and snowmelt, coupled with active seismic processes such as earthquakes and volcanic eruptions.

The probability of formation of landslide dams is determined by several factors including the volume of the displaced material (sediment budget), the type of landslide, the velocity of the displaced material, and the morphology of the stream-channel. Schuster et al. (1998) enumerated four kinds of factors that govern the spatial distribution of landslide dams, based on the study of the landslide dams formed by the 1989 Loma Prieta earthquake in California (USA). These factors are: (1) seismic intensity (peak acceleration, duration of strong motion); (2) topography and high slope gradient; (3) lithology and weathering; and (4) soil moisture and groundwater content.

Landslide dams pose enormous risks because of the potentially catastrophic outburst floods that could be triggered by the sudden release of stored water masses from the failure of these river blockages. The floods, upon surging downstream, transform into debris flows and hyperconcentrated flows with peak discharge most times greater than 10,000 m^3/s, thereby, threatening the lives of people living in the downstream areas (O'Connor and Costa 2004). For example, the Diexi landslide dams (Dahaizi, Xiaohaizi and Diexi), formed by deep-seated mass movements triggered by the August 25 1933 M=7.5 earthquake in northwestern Sichuan Province of China, claimed about 2,423 lives from outburst floods (that travelled with an average velocity of 20~25 km/hr) generated from the breaching of the dams (Chai et al. 2000). Peak discharges are controlled by several factors including the dam geometry, downstream topography, failure mode, the internal structure of the blockage and lake volume. Therefore, a good knowledge of the complex processes involved in the evolution and failure of river-damming landslides, in addition to hydraulics of the outburst floods, are imperative for disaster risks assessment and management.

This paper presents a summary of landslide and landslide dam risk reduction strategies adopted by the Japan Society for Erosion Control Engineering (SABO Gakkai) for the mitigation of disasters triggered by landslides and natural dam failures.

Case Histories

Mihata landslide, Izumo, Japan

On 6 August 2012, a deep-seated landslide occurred in Izumo city, Shimane Prefecture, southwest Japan. The landslide was presumed to have been triggered by a high-intensity rainfall which occurred on 6 July 2012. The movement of the destabilized slope materials blocked a local access road and dammed the Mihata River, creating a small dam. Initial assessment of the landslide showed that small-scale deformations occurred on the hillslope 4 days before the main event, creating a 3 m main scarp. Post-failure assessment of the landslide showed that the depth of the main scarp increased from 3 to 20 m, while the height, length, and width of the landslide were 80, 130 and 200 m, respectively (Fig. 1). The landslide materials are mostly comprised of highly weathered rhyolite and dacite tuff. A system of monitoring sensors comprising extensometers, pore-water pressure transducers, groundwater level probes, and intelligent cameras was installed on the landslide to monitor post-failure dynamics of the hillslope. Furthermore, three (3) geotechnical boreholes, hereinafter referred to as *BP1*, *BP3*, and *BP6* were drilled to depths below the sliding zone to monitor groundwater changes in the slope and to evaluate the lithological characteristics of the landslide. Several earthworks construction methods were used to stabilize the slope and increase its factor of safety against failure. These include: (1) the construction of retaining walls; (2) the installation of several horizontal boring works; and (3) the installation of drainage wells and tunnels to reduce the buildup of pore-water pressure within the slope.

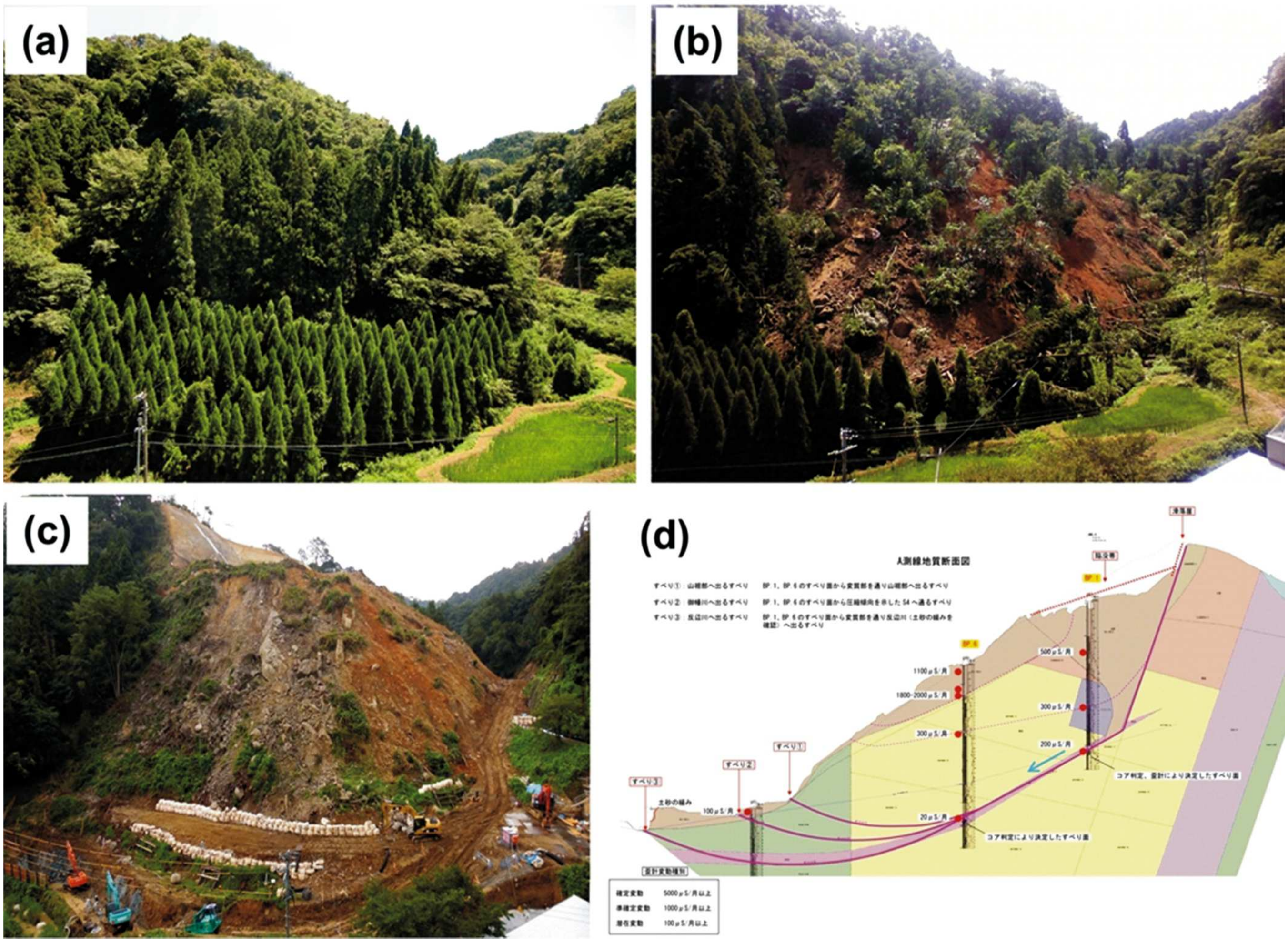

Fig. 1: Mihata landslide **(a)** 2 days before failure **(b)** 24 hours after failure **(c)** Preliminary stabilization works on the landslide **(d)** Longitudinal profile of the landslide showing geotechnical boreholes *(Image: Fujii Civil Engineering Corp.)*

Akatani and Kuridaira landslide dams, Kii Peninsula, Japan

The Akatani and Kuridaira landslide dams are located in Nara Prefecture, in the Kii Peninsula of Japan. The two landslide dams are among the 17 major landslide dams created by the devastating effects of Typhoon Talas (Typhoon no. 12 in Japan) on September 3, 2011 (Hayashi et al. 2013; Okeke 2016). About 207 landslides, landslide dams, debris flows, and other sediment-related disasters were triggered in 21 prefectures with Mie, Nara and Wakayama Prefectures recording the highest number of cases. The volume of sediments produced by the effect of the heavy rainfall was estimated to be about 100 million m^3. The geomorphometric characteristics of the landslide dams are summarized in Table 1. Materials composing the landslide dams were derived from Paleogene-Neogene sedimentary units comprising weathered layers of sandstones, siltstones, and mudstones, as well as detached blocks of intrusive rocks. Emergency countermeasure works such as the installation of drainage pumps, subsurface drainage channels, displacement transducers, and other early warning system components were carried out on the sites to stabilize the dams and prevent the potential occurrence of secondary disasters (SABO 2012; Wang et al. 2018) (Fig. 2 and 3).

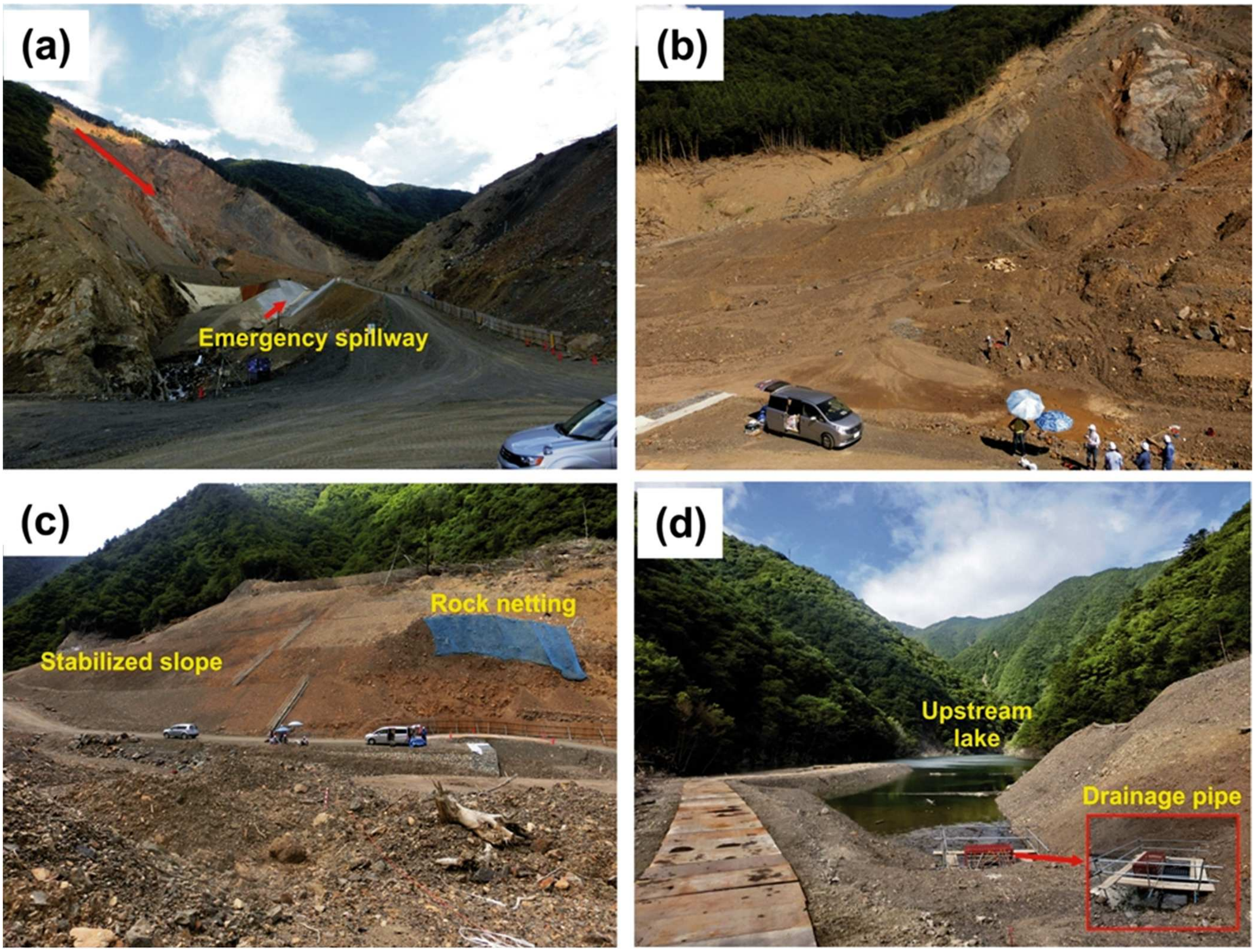

Fig. 2: Akatani landslide dam **(a-b)** Downstream view of the landslide and the dam **(c)** Stabilized section of the landslide **(d)** Upstream lake created by the dam with an emergency drainage pipe for lowering the lake level *(Image: PCNDR, Shimane Univ. Japan)*

Significance of Early Warning Systems

Japan has proactive disaster risk reduction strategies which depend mostly on the application of early warning systems in line with the Sendai Framework for Disaster Risk Reduction 2015 – 2030. The success of

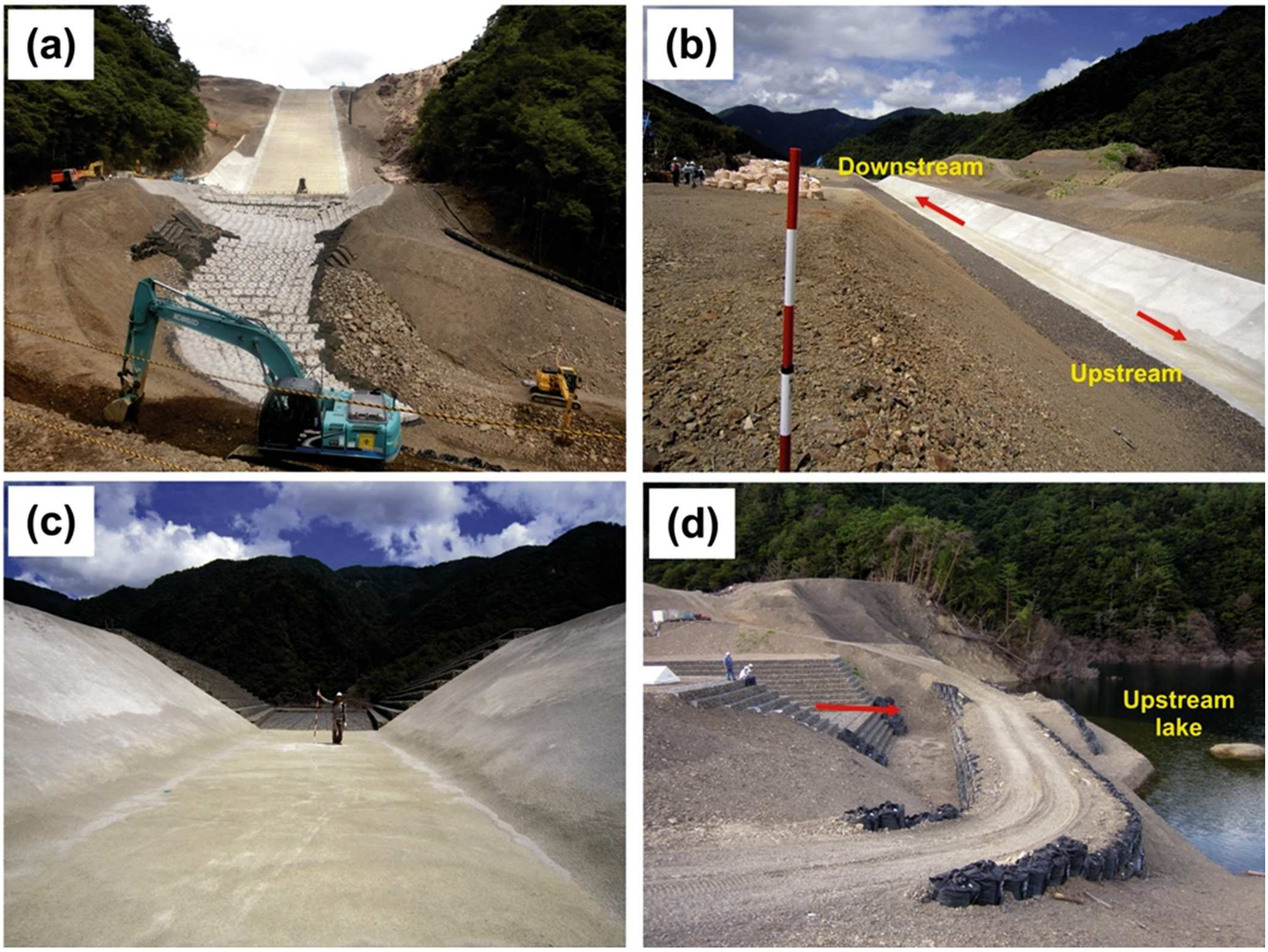

Fig. 3: Kuridaira landslide dam **(a)** Downstream view of the dam **(b-c)** Emergency spillway for lowering the upstream lake level **(d)** Upstream side of the dam showing the dammed lake *(Image: PCNDR, Shimane Univ. Japan)*

Table 1: Geomorphometric characteristics of Akatani and Kuridaira landslide dams (Okeke 2016)

Landslide				**Landslide dam**			
Location	**Length (m)**	**Width (m)**	**Volume (m^3)**	**Height (m)**	**Blockage volume (m^3)**	**Lake volume (m^3)**	**Catchment area (km^2)**
Akatani	670	300	9,354,000	67	10,154,000	5,500,000	13.2
Kuridaira	600	500	25,133,000	100	24,133,000	7,500,000	8.7

these methods in the mitigation of natural disasters is evidenced by the decreasing trend in the number of fatalities and damages resulting from the direct impacts of natural disasters triggered by tsunamis, debris flows, landslides, earthquakes, and volcanic eruptions. Japan has different early warning systems specific to a particular type of natural hazard. Fig. 4 shows a typical landslide and debris flow early warning system and mobile monitoring workstation suitable for remote access to rugged terrain. This unit comprises an intelligent camera with antennas, pore-water pressure transducer, seismometer, inclinometer, and a workstation powered by a portable solar-panel. The installation of these early warning systems in many rugged watersheds, in addition to the hydroclimatic data provided by the Japan Meteorological Agency (JMA), has immensely contributed to better management of disasters in Japan and other Southeast Asian countries.

Conclusions

Japan's resilience to disasters triggered by landslides and landslide dam failures have been attributed to its investment in advanced geotechnical and geophysical

Fig. 4: Landslide/debris flow early warning systems showing **(a)** Solar-powered workstation with high-speed camera **(b)** Pore-water pressure transducer and seismometer **(c)** Portable workstation with acoustic sensor **(d)** Mobile workstation for access to rugged terrain *(Image: CAU Okeke)*

sciences, in addition to cutting-edge research for the development of disaster-resilient infrastructure and the application of early warning systems. The country's range of countermeasures against landslides and landslide dam failures has recorded a lot of success, such that developing countries in Asia (Vietnam, Indonesia, Nepal, Sri Lanka, Cambodia, Afghanistan, and China) have benefitted immensely from these advanced disaster risk reduction strategies, through several collaborations with Japan International Cooperation Agency (JICA) and other public research institutions. While Nigeria has made some improvements in the area of post-disaster recovery, much effort needs to be put in the area of disaster preparedness and in building resilient infrastructures against flood disasters, in line with the Sendai Framework for Disaster Risk Reduction (2015). Nigeria needs to establish partnerships with JICA and other disaster prevention research institutions for the development of early warning systems tailored specifically for flood hazards. Finally, Nigeria should emulate Japan in ensuring that appropriate laws and legislations regarding the utilization of resilient infrastructures against natural disasters are enacted by the government, both at the local, state and federal levels.

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Non-Destructive Evaluation of Structural Integrity of Selected Buildings in Anambra State

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Abstract

This study examines the structural stability of 150 buildings selected from the 21 Local Government Areas in Anambra State by performing non-destructive tests on them. The research is aimed at ascertaining present structural strengths of the buildings from their concrete compressive strength values and decipher if these values could relate to the recent building collapse incidents in the State. The test is performed using Portable Ultrasonic Non-Destructive Indicating Tester (PUNDIT) which has two transducers, transmitting and receiving ends. To get the compressive strengths of each structural element (columns, beams and slabs), three test points were taken on it and the mean value adopted. Structural elements in rendered buildings were identified by the use of Profoscope before testing. The analyses of the results show that 62 out of 150 buildings representing 41.3% of the assessed facilities made it to “No Risk Alert” group. In this group, most structural elements tested met the threshold value of 20KN while 37 out of 150 buildings representing 24.7% of the assessed buildings were categorized as “Low Risk Alert” since more than half of the tested components met the required threshold. This signifies high usage of suitable building materials and enhanced workmanship. However, 47 out of 150 buildings representing 31.3% of the assessed buildings were categorized as “High Risk Alert” because more than half of the tested structural elements did not meet the required threshold while 4 out of 150 buildings representing 2.7% of the assessed buildings form the “Alarming Alert” group because most of their tested structural elements recorded extremely low concrete compressive strength values. This research concretizes the fact that unsuitable materials and poor workmanship are not the exclusive causes of building collapse in the State. Foundation inadequacy and weak subsurface are likely factors though not explored in this study.

Keywords: *Building, non-destructive test, structural elements, compressive strength, collapse.*

Introduction

The testing of existing structures is usually related to an assessment of structural integrity or adequacy. The need for continuous monitoring of structure during service life is to detect impending failure and monitor the performance of those structures in order to save lives and properties. There is an ever increasing need to ensure that the integrity of engineering structures especially buildings is not compromised. Assessment of structural components or elements of these buildings during and after construction is therefore of paramount importance. The former could be achieved through destructive methods while the latter is achievable through non- destructive methods.

Non-destructive structural integrity test is an investigation intended to reveal the performance, integrity and reliability of structural elements without damaging the materials under investigation. Non-Destructive Test and Evaluation is an assessment of structural adequacy and capabilities of buildings in bearing imposed loads. It is accomplished by establishing a correlation between a non-destructively measured physical/derived parameter and quantitative information on defects, stresses and microstructures. Pulse velocity measurements made on concrete structures are used for quality control purposes. In comparison with mechanical test on control samples such as cubes or cylinders, pulse velocity measurements have the advantage that they relate directly to the concrete in the structure rather than to samples, which may not always be true representative of the concrete in situ (Baldev et. at, 1986 and Baldev et. at, 1995).

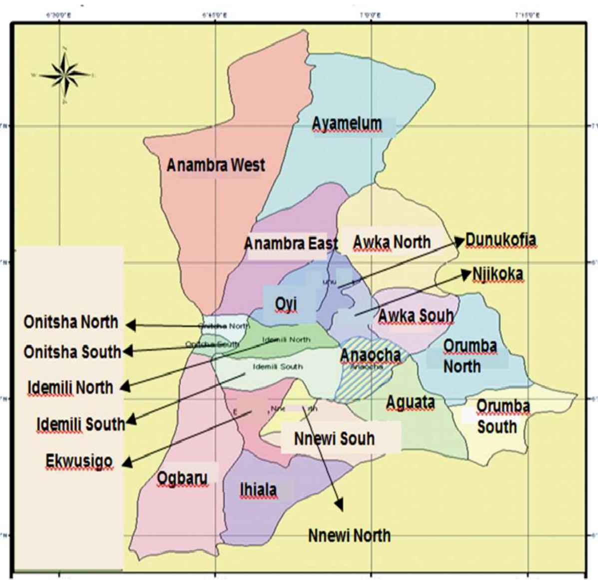

Fig. 1: Map of Anambra State showing the 21 Local Government Areas covered

This study takes analyses of non-destructive tests performed on 150 buildings across Anambra State ranging from two floors to six floors. Some of the tested buildings were completed while others were on-going at various stages of construction at the time of test. Figure 1 shows the 21 Local Government Areas in Anambra state covered by this study. The addresses and descriptions of the investigated structures are presented in table 1.

Table 1: Location and details of tested buildings in Anambra State

S/N	LGA	Town	No. of Floors	Stage of Construction	Category
1	Aguata	Ekwulobia	3+basement	Completed	Religious
2		Ramond Ashala Avenue Ekwulobia	3	2nd floor window seat	Residential
3		Umuchi Ekwulobia	3	Completed	Commercial (Hotel)
4		Close to St. Luke Ang. Ch. Umuchi Ekwulobia	3	Rendering	Residential
5		Orlu Rd Okpo village Ekwulobia	3	2nd floor lintel	Commercial(GF) & Residential (other floors)
6		Umuchiana Ekwulobia	4	Completed	Educational(school)
7		Umuchiana Ekwulobia	5	Rendering	Educational(school)
8	Awka North	Ifite-Isuaniocha	2	1st floor final block	Residential
9		Adjacent Bompy hotel Isuaniocha	2	Roofing	Residential
10		By Ntoko junction Eke-Agba Isuaniocha	2	Roofing	Residential
11		Isuaniocha Urum	2	Roofing	Residential
12		Okukwa village Amansea	3	Roofing	Commercial
13		Morgan Rd Umunagu village Achalla	3+basement	2nd floor final block (abandoned)	Residential
14		Aku drive Umuokpala Amansea	4	Completed	Commercial
15		Along Anerobi college road Ukwuakwa Amansea	4	Roofing	Commercial
16		Onyeoma close Umuokpala Amansea	4	Completed	Commercial (Hostel)
17	Awka South	Close to Adig suites Udoka estate Awka	2	Roofing	Residential
18		Umuenu village Umuawulu	2	Rendering	Residential
19		Nibo	2	Roof parapet	Commercial
20		Umuodu Okpuno	3	2nd floor final block	Residential
21		AgbovuNgeneAmawbia	3	2nd floor slab	Commercial
22		Egbe Ngwu Amawbia	3	2nd floor final block	Commercial
23		Welfare str. Umuodu Okpuno	3	Rendering	Commercial
24		6 Mogbo street Ngene Amawbia	3	Finishes	Residential
25		Opp.Graceland hospital Okpuno	4	3rd floor final block wall	Commercial
26		Ifite Awka behind winners church	4	Roofing	Residential
27		Along Nibo-Umuawulu Rd, Nibo	4	Roofing	Residential
28	Anambra East	College Rd Udoka L/out Nsugbe	2	Roofing	Residential
29		Kings touch crescent Nsugbe	3	Rendering	Commercial
30		Along Aguleri-Nando str. Aguleri	3	2nd floor final block	Commercial (Hotel)
31		Near Fr. Tansi shrine Aguleri	3	2nd floor final block	Commercial (Hotel)
32		By prime junction Abata Nsugbe	3	Completed	Residential
33		Chief Law Chinwuba street Wite Aguleri	3	Roof parapet	Commercial (Hotel)
34		Chief Law Chinwuba street Wite Aguleri	3	Roof parapet	Commercial (Hotel)
35		Oye street Nsugbe	4	Roof parapet	Commercial
36	Anambra West	Anam	2	Rendering	Residential
37		Anam	2	Completed	Residential
38	Anaocha	8 lake avenue Umuowelle Agulu	2	1st floor above lintel	Residential
39		Neni Road, Obeledu-Ani village Obeledu	2	1st floor lintel	Residential
40		Near Mbadugha petrol station Ichida	2	Finishes	Residential
41		Ekwulobia Road, Agulu	2	Completed	Commercial (Bank)
42		Eziora village Adazi Ani	3	Finishes	Commercial
43	Ayamelum	Near community pri. Sch. Anaku	2	Rendering	Residential
44		Along Anaku Road, Ifite Ogwari	3	Roofing	Commercial (Hotel)
45	Dunukofia	Obinwanne housing estate Enugu-Agidi	2	Roof parapet	Residential
46		Ugwuogu village Enugu Agidi	2+Basement	Rendering	Residential
47		Amagu village Ukwulu	2	Completed	Commercial (Hotel)
48		Inside Nawgu primary health centre premises	2	Roofing	Commercial
49		Achalla village Enugwu-Agidi	3	Roof parapet	Residential
50	Ekwusigo	Along Mentus Road Ihembosi	2	Completed	Residential
51		Near Micheal junction Ozubulu	3	Finishes	Commercial

52		Ozubulu-Nnewi Road Uruokwe Egbema Ozubulu	3	Rendering	Commercial (Plaza)
53	Idemili North	Beside Mentus concrete block ind.Nkwere Uke	2	Completed	Residential
54		Chi-chi bread str. Owelle-Aja Obosi	3	2nd floor final block wall	Commercial (Hotel)
55		13 Anigbogu Diamond estate Nkwelle Ogidi	4	3rd floor final block wall	Commercial & Residential
56		Umuru agu by nkwo ezeudo Ogidi	4	Finishes	Commercial (Hotel)
57		Chris Nweke str.Ikenga Ogidi	4	Rendering	Commercial(GF) & Residential (other floors)
58		Ilodibe service lane Awada Obosi	4	3rd floor final block wall	Commercial
59		Off Tony Eze str. Owelle Aja Obosi	4	Roof parapet	Commercial (Hotel)
60		Philip Unachukwu str. Diamond estate Ogidi	5	4th floor final block wall	Commercial(GF) & Residential (other floors)
61		Achaputa L/out Idemili farm Obosi	5	4th floor final block wall	Commercial
62		St Dominic cath. Oduke Obosi.	5	Roofing	Religious
63		Nkomoro str, Owelle-Aja Obosi	5	Rendering	Commercial
64		7 Oba avenue behind army barracks Odume Obosi	5	Roof parapet	Commercial
65		21 Mbaukwu str. Nkpor	5	Rendering	Commercial GF=warehouse
66		Off Ezeiweka Rd Obosi	5	Rendering	Commercial
67		Dan Maduakor lane Okeluche Ikenga Ogidi	5	Finishes	Commercial
68		Dayspring str. Awada	6	Roofing	Commercial
69		Okeluche Ikenga Ogidi	6	Rendering	Commercial
70	Idemili South	Agada street Oba	2	Rendering	Residential
71		Opp. Algon compressive health centre Oba	2	Rendering	Residential
72		Agada Rd Ofuobi estate Okofia L/out Oba	2	Rendering	Residential
73		Along Nnobi-Nnewi Road Nnobi	3	2nd floor above lintel	Commercial (Petrol station)
74		Nkpolu village Oba	4	Roofing	Commercial
75		Nkpolu village by Nellys' Newton hotel Oba	4	Roofing	Commercial
76	Ihiala	Orsumoghu-Isseke Rd Ihiteoha Orsumoghu	2	Rendering	Commercial (Petrol station)
77		Umudimogo Rd d Ihiala	2	Completed	Commercial (Bank)
78		Near morning star lodge Ndiumeraku Uli	2	1st floor lintel	Commercial (Hostel)
79		Opp. St. Mary Cath. Chur. Orsumoghu.	2	Roofing	Commercial (shops)
80		Along Onitsha-Owerri Rd beside confidence oil & gas Ihiala	3	2nd floor slab stage	Commercial
81		Opp. Eagle destiny hotel Ndiezike village Ihiala	3	Completed	Residential
82	Njikoka	Off Nsionu Obinakueze str. Ndiagu Orofia Abagana	2	Completed	Residential
83		Close to our savior's Ang. Church Enugwu Ukwu	2	Roofing	Residential
84		Adjecent local govt headquarters Abagana	2	Roof parapet	Residential
85		Nawfia	2+basement	Completed	Residential
86		Adjacent Emmatex petrol station Enugwu-Ukwu	3	2nd floor slab	Commercial (plaza)
87	Nnewi North	Umudim Nnewi.	2	Renovation	Residential
88		Okigwe Rd opp. NIBRI Nnewi	2	Roofing	Commercial
89		Behind college of health sciences Okofia Nnewi	3+basement	Roofing	Commercial (Hostel)
90		Ndimgbu Otolo Nnewi	3	Finishes	Residential
91		Adimorah Rd Obiofia Nnewichi Nnewi	3	Roofing	Commercial & Residential
92		Off Igwe Orizu Road Nnewi	3	Rendering	Commercial
93		Akabor-Edorji Uruagu Nnewi	4	Rendering	Commercial
94*		Okpuno Otolo Nnewi	4	3rd floor slab	Commercial
95		Close to college of health sciences Okofia Nnewi	4	Completed	Commercial
96		Umuenem Otolo Nnewi	4	Rendering	Commercial
97		Ukwu Cashew, Okpuno Nnewichi	4	Roofing	Commercial
98		Abubor Nnewichi Nnewi	4	Completed	Commercial(GF) & Residential (other floors)
99		Ekwulu L/out Okkpuno Egbu Umudim Nnewi	4	3rd floor final block	Commercial
100	Nnewi South	Okigwe Road Amichi	2	Roof parapet	Religious
101		Nzagha Ukpor	2	Roofing	Residential
102		Obiagu – Amichi	2	Roofing	Residential
103		Close to orie Utuh market Utuh	3	2nd floor slab boarding	Commercial
104	Ogbaru	Mission Rd Iyiowa Odekpe	3	Rendering	Residential
105		Okoti Iyiowa Odekpe by Chikaita petrol station	4	Rendering	Commercial
106		107 Obodoukwu Rd Okpoko	4	Rendering	Commercial
107		Amaifeke Street off Atani Road	5	Roofing	Commercial
108		1, Obioma Ezenwa str. Iyiowa Odekpe	5	Rendering	Commercial
109		Mission Rd Iyiowa Odekpe	5	Rendering	Commercial

110	Onitsha North	Presidential drive Fed. housing GRA Onitsha	2 + basement	1st floor final block	Residential
111		Near Dolly hills hotel Unity Estate, Trans Nkisi	2	Finishes	Residential
112		Plot 108 unity hall estate 3-3 Onitsha	2	Roofing	Commercial
113		50 Umueri str. Oyolu-Eze Onitsha	3	Rendering	Residential
114		Umueri str. Oyolu-Eze Onitsha	3	Rendering	Residential
115		27 Nawfia str. Omaba phase ii Onitsha	4	Rendering	Commercial
116	Onitsha South	F65 Housing Estate Fegge	2	Completed	Residential
117		Venn Road Onitsha	4	Rendering	Commercial (plaza)
118		9 Emodi street Odoakpu Fegge	5	Rendering	Commercial
119		17 Rev. Egbuche str. Fegge	5	4th floor lintel	Commercial
120		7 St John str. Odoakpu	5	Roofing	Residential
121		2, Aggrey road Fegge	5	Roofing	Commercial
122		6, Okwuenu Street Fegge	5	Completed	Commercial
123		8 Unegbe lane Odoakpu	5	Completed	Commercial (School)
124		26 Obosi street Feggee	6	Roof parapet	Commercial
125		28 Nnewi street Odoakpu	6	Completed	Commercial
126	Orumba North	Obinikpa village Amaokpala	2	1st floor lintel	Residential
127		Close to exclusive lodge Oko	2	Roofing	Residential
128		Pal Junction Oko	2	Rendering	Commercial (shops)
129		Ndiagu Oko	3	Completed	Commercial(Hostel)
130		Emenwosu extention Amaokpala	4	Rendering	Commercial(Hostel)
131		Ndiagu Uzelle Ifite Oko	4	Rendering	Commercial
132		Permanent site Ndiagu Oko	4	Completed	Commercial(Hostel)
133		Oko-Umunze Rd Amaokpala	4	Completed	Commercial(Hostel)
134		Near elite 5-star hotel Amaokpala	4	Completed	Commercial(Hostel)
135	Orumba South	Obikpa L/out Ohukabia village Nawfija	2	Rendering	Commercial (shopping plaza)
136		Along Umuchu-Umunze Rd Umunze	2	1st floor lintel	Residential
137		Near Orie markrt Ogbunka	2	1st floor final block work	Residential
138		Owerri-Ezukalla Rd Ubaha Umunze	3	Rendering	Commercial
139		Agbirigba Lomu Umunze	3	Rendering	Residential
140	Oyi	Nkwelle Ezunaka	2	1st f floor lintel	Residential
141		Near St. Paul's Ang. Church Nteje	2	Completed	Residential
142		Adjacent God promotion holy ministry Okpuno Azu Ogbunike	3	2nd floor final block work	Commercial
143		Maryland Estate Nkwelle Ezunaka	3	2nd floor lintel	Residential
144		Beside pink view hostel Mkpomachi village Awkuzu	3	Roof parapet	Commercial (Hostel)
145		Umuezunu Nkwelle Awkuzu	3	Rendering	Commercial
146		Near keystone bank Ogbunike	3	Rendering	Residential & Commercial
147		Ezinkwo Awkuzu	3	Completed	Commercial (Hostel)
148		Behind bicycle spare parts Ogbunike	4	3rd floor final block	Commercial
149		Awkuzu junction	4	Completed	Commercial
150		Near Saga Foam Industry Azu Ogbunike	5	Rendering	Commercial

Materials and Methods

Visual Inspection: Visual inspection is an essential precursor to any intended Non-Destructive Test because it provides an evaluator with a wealth of information on the cause(s) of any observed distress. Visual inspection is a very valuable method for the evaluation of discontinuities open to the surface. However, it is not suitable for detecting subsurface and interior discontinuities (Halmshaw, 1989).

The equipments and materials used in this research are Profoscope, PUNDIT (Portable Ultrasonic Non-Destructive Digital Indicating Tester) and certified coupling gel. The Profoscope and PUNDIT are self-indicating and self-calibrating equipments that work under the command button while the gel is a lubricating medium.

Profoscope Working Procedures: Profoscope (Fig. 2) uses electromagnetic pulse induction technology to detect rebars in concrete elements. When the appropriate key is clicked, it indicates, rebar in concrete element, diameter of rebar, concrete cover and bar spacing. When the profoscope is fully switched on and reset for use, it is then placed on the surface of the elements under test for proper scanning of the rebar in the structure.

As the signal strength arrow indicates proximity of the rebar, by pointing to its direction, the riflescope also moves to the center line of the profoscope until it centralizes and a led light indicates on the screen of the equipment locating the rebar in the structure. While sweeping the equipment parallel to the rebar in a very slow manner, the concrete cover will indicate on the top right hand corner of the screen, same goes for bar spacing as the riflescope centralizes in between rebars without indication of the led light. Taking readings with the profoscope is usually visible as all readings are displayed on the screen.

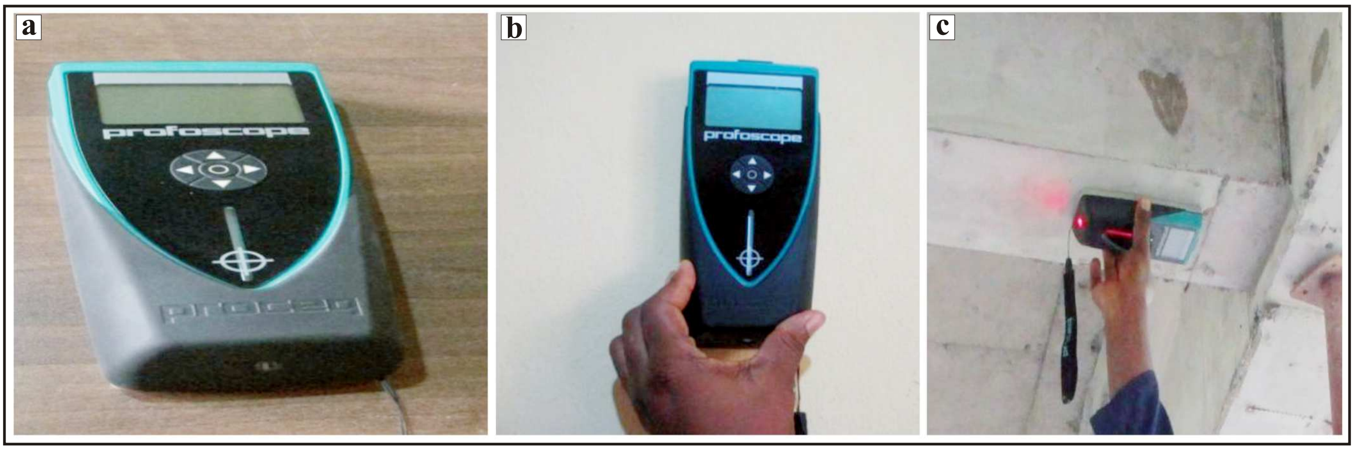

Fig. 2: (a) Profoscope sample (b) detecting vertical elements (c) on horizontal elements

Pundit Working Procedures: PUNDIT (Figures 3 and 4) is the main equipment used for assessing the quality & integrity of concrete by passing ultrasound waves through the specimen under test.

Components of PUNDIT are two cables and two transducers. The PUNDIT works through the principle of pulse velocity measurement produced by the electro-acoustical transducer which is held in contact with the surface of the concrete structure under test. The pulse generated is transmitted into the concrete from the transducer using a certified coupling gel, it undergoes multiple reflection as the boundaries of the different materials phases within the concrete.

A complex system of stress waves develops, which includes both longitudinal and shear waves and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal and the reading displays on the screen of the device. Taking readings with the PUNDIT is by clicking on the appropriate button when the transducers are placed in contact with the surface of the structure under test and the readings will display on the screen. PUNDIT operates with ultrasonic waves principles (Szilard, 1982).

Non-Destructive test by the use of PUNDIT test equipment can also be used to:

a. Ascertain the uniformity of concrete in and between members.
b. Measure changes occurring with time in the properties of concrete.
c. Correlate pulse velocity and strength as a measure of concrete quality.
d. Determine the modulus of elasticity and dynamic Poisson's ratio of the concrete.
e. Detect defects on structural elements (David,1981; Silk & Whaphan, 1989; Palanichamy et al, 1992a):

Procedures involved in detecting defects are:

i. Switch on the PUNDIT machine.
ii. Self-calibrate machine.
iii. Apply coupling gel on the transducers.
iv. Place the transducers on the element to be tested.
v. Click on play button.
vi. When there is delay in transit time with a blink on the screen, it indicates defects [void, crack or honeycomb] in the structure being tested.

Results

Analyses of the results of the non-destructive assessment carried out on 150 facilities across Anambra State show that some of the structural elements tested met acceptable concrete compressive strength values while others fell below the threshold value of 20KN.

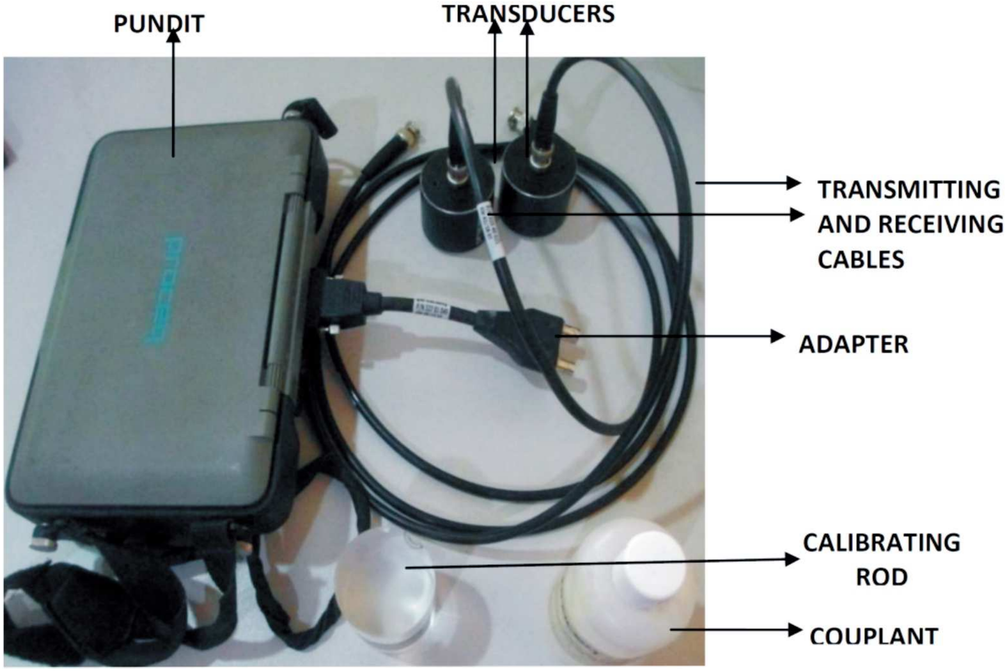

Fig. 3: PUNDIT machine and its accessories.

Fig. 4: Operational Demonstrations of PUNDIT (a) Direct method (b) Indirect method

Poor quality concrete mix in some of the elements was also evident from the poor concrete compressive values obtained therein. Some structural defects and construction errors such as cracks, spalling, exposed reinforcements, honeycombs and voids were also identified in some buildings as shown in Table 2.

Abbreviations and symbols used in Table 2 are explained below.

BM: Basement; GF: Ground floor; FF: First floor; FFS: First floor slab.

SF: Second floor; SFS: Second floor slab; TF: Third floor.

TFC: Third floor column.

ERHV: Exposed reinforcement, honeycombs and voids

***: 150mm sandcrete blocks and columns used on ground and first floors. 225mm sandcrete blocks and columns used on 1st, 2nd, 3rd, & 4th floors. A ground floor column (buckled) and 1st floor beam (sagged) affected by the collapse of a building in front.

Table 2: Reports of Non-Destructive Assessment on the tested Buildings

S/N	LGA	Location	No. of tested elements		Percentage(%) of the tested elements that met threshold value			Remarks	Anomalies / Structural Defects Observed
			Ground Floor	Others	Other Floors	Other Floors	All Floors		
1	Aguata	Ekwulobia	BM=12 GF=54	57	BM=50 GF=48.4	36.8	43.5	Not satisfactory	Diagonal & vertical cracks on 1st floor external walls
2		Ramond Ashala Avenue Ekwulobia	25	71	40.0	9.9	17.7	Not satisfactory	Some exposed reinforcements ,& voids
3		Umuchi Ekwulobia	37	88	59.5	69.3	66.4	Satisfactory	Nil
4		Close to St. Luke Ang. Church Umuchi Ekwulobia	30	102	66.7	52.9	56.1	Satisfactory	Few exposed reinforcements ,& voids
5		Orlu Rd Okpo village Ekwulobia	20	65	95.5	49.2	60.7	Satisfactory	GF & FF previously built, Spalling on FFS, Seepage & dampness on SFS
6		Umuchiana Ekwulobia	35	148	22.9	34.5	32.2	Not satisfactory	Vertical cracks on GF external wall Some exposed reinforcement , voids & honeycombs
7		Umuchiana Ekwulobia	68	205	26.5	18.5	20.9	Not satisfactory	Vertical & diagonal cracks & exposed reinforcements & voids on all the floors.***
8	Awka North	Ifite-Isuaniocha	19	17	47.4	17.6	33.3	Not satisfactory	Nil
9		Adjacent Bompy hotel Isuaniocha	40	48	22.5	22.9	22.7	Not satisfactory	Few exposed reinforcements ,& voids on GF
10		By Ntoko junction Eke-Agba Isuaniocha	21	22	61.9	9.1	34.9	Not satisfactory	No support beam on FF slab soffit, few ERHV.
11		Isuaniocha Urum	28	25	60.7	56.0	58.5	Satisfactory	Nil
12		Okukwa village Amansea	36	84	19.4	35.7	30.8	Not satisfactory	Some exposed reinforcements ,& voids
13		Morgan Rd Umunagu village Achalla	BM=3 GF=53	103	BM=0.0 GF=18.9	32,0	27.0	Not satisfactory	Spalling, exposed reinforcements, voids, horizontal, diagonal & vertical cracks observed.
14		Aku drive Umuokpala Amansea	83	284	33.7	31.3	31.9	Not satisfactory	Some exposed reinforcements ,& voids
15		Along Anerobi college road Ukwuakwa Amansea	32	144	46.9	45.8	46.0	Not satisfactory	Nil
16		Onyeoma close Umuokpala Amansea	55	221	60.0	46.6	49.6	Acceptable	Nil
17	Awka South	Close to Adig suites Udoka estate Awka	50	48	58.0	45.8	52.0	Acceptable	Some exposed reinforcements & voids
18		Umuenu village Umuawulu	29	37	55.2	27.0	39.4	Not satisfactory	Few exposed reinforcements & voids
19		Nibo	16	24	43.7	41.7	42.5	Not satisfactory	Nil
20		Umuodu Okpuno	44	84	52.3	50	50.0	Satisfactory	Some exposed reinforcements & voids
21		Agbovu Ngene Amawbia	30	46	40.0	50.0	45.3	Not satisfactory	Few exposed reinforcements & voids on GFC, Diagonal cracks on internal walls
22		Egbe Ngwu Amawbia	43	110	79.0	58.2	64.0	Satisfactory	Nil
23		Welfare str. Umuodu Okpuno	34	112	50.0	35.7	39.0	Not satisfactory	Few exposed reinforcements ,& voids
24		6 Mogbo street Ngene Amawbia	35	96	60.0	67.7	65.6	Satisfactory	Nil
25		Opp.Graceland hospital Okpuno	51	171	66.7	66.7	66.7	Satisfactory	Some exposed reinforcements & voids
26		Ifite Awka behind winners church	39	135	59.0	60.7	60.3	Satisfactory	Few exposed reinforcements ,& voids

27		Along Nibo-Umuawulu Rd, Nibo	36	137	55.6	44.5	46.8	Acceptable	Few exposed reinforcements ,& voids
28	Anambra East	College Rd Udoka L/out Nsugbe	25	32	36.0	40.6	38.6	Not satisfactory	Nil
29		Kings touch crescent Nsugbe	38	100	57.9	45	48.6	Acceptable	Nil
30		Along Aguleri-Nando str. Aguleri	70	98	60.0	64.1	65.5	Satisfactory	Nil
31		Near Fr. Tansi shrine Aguleri	51	74	43.1	59.5	60.0	Satisfactory	Nil
32		By prime junction Abata Nsugbe	28	80	39.3	35.0	36.1	Not satisfactory	Nil
33		Chief Law Chinwuba street Wite Aguleri	62	135	67.8	65.2	66.0	Satisfactory	Vertical, horizontal & diagonal cracks on GF & FF.
34		Chief Law Chinwuba street Wite Aguleri	67	132	68.7	70.5	69.8	Satisfactory	Diagonal cracks on GF internal walls
35		Oye street Nsgbe	36	106	55.6	54.7	53.5	Satisfactory	All TFC covered @ testing time
36	Anambra West	Anam	24	22	59.5	22.7	30.4	Not satisfactory	Some FFC inaccessible
37		Anam	24	24	40.0	33.3	37.0	Not satisfactory	Nil
38	Anaocha	8 lake avenue Umuowelle Agulu	26	29	73.1	55.2	63..6	Satisfactory	Nil
39		Neni Road, Obeledu-Ani village Obeledu	14	18	35.7	38.9	37.5	Not satisfactory	Nil
40		Near Mbadugha petrol station Ichida	25	30	56.0	36.7	45.5	Acceptable	Nil
41		Ekwulobia Road, Agulu	30	38	66.7	84.2	73.5	Acceptable	Dampness on FF internal wall and slab
42		Eziora village Adazi Ani	32	72	70.2	73.5	76.4	Satisfactory	Lateral cracks on FF & SF. Few exposed reinforcements ,& voids
43	Ayame-lum	Near community pri. Sch. Anaku	20	26	75.0	57.7	65.2	Satisfactory	Nil
44		Along Anaku Road, Ifite Ogwari	36	70	81.4	79.5	80.3	Satisfactory	Nil
45	Dunu-kofia	Obinwanne housing estate Enugu-Agidi	19	18	57.9	61.1	59.5	Satisfactory	Nil
46		Ugwuogu village Enugu Agidi	BM=10 GF=30	28	BM=30 GF=53.3	39.3	44.1	Acceptable	Retaining wall on external BM wall
47		Amagu village Ukwulu	32	44	56.3	38.6	46.1	Acceptable	Nil
48		Inside Nawgu primary health centre premises	39	49	59.0	50	52.0	Satisfactory	Nil
49		Achalla village Enugwu-Agidi	16	80	25.0	25	25.0	Not satisfactory	Few exposed reinforcements & voids
50	Ekwu-sigo	Along Mentus Road Ihembosi	20	28	57.1	62.5	58.8	Satisfactory	Nil
51		Near Micheal junction Ozubulu	100	213	80.0	70.0	73.2	Satisfactory	Nil
52		Ozubulu-Nnewi Road Uruokwe Egbema Ozubulu	16	54	18.8	37.0	32.9	Not satisfactory	Nil
53	Idemili North	Beside Mentus concrete block ind.Nkwere Uke	26	31	53.8	58.1	56.1	Satisfactory	Nil
54		Chi-chi bread str. Owelle-Aja Obosi	44	118	63.6	28.0	38.3	Not satisfactory	Some exposed reinforcements ,& voids
55		13 Anigbogu Diamond estate Nkwelle Ogidi	34	200	38.2	42	41.5	Not satisfactory	Some exposed reinforcements & voids

56		Umuru agu by nkwo ezeudo Ogidi	53	207	41.5	39.6	40.0	Not satisfactory	Few columns inaccessible
57		Chris Nweke str.Ikenga Ogidi	36	135	47.2	37.4	32.7	Not satisfactory	Some exposed reinforcements & voids
58		Ilodibe service lane Awada Obosi	24	138	62.5	55.1	56.2	Satisfactory	Some exposed reinforcements & voids
59		Off Tony Eze str. Owelle Aja Obosi	86	316	51.2	53.5	52.7	Satisfactory	Some exposed reinforcements & voids
60		Philip Unachukwu str. Diamond estate Ogidi	37	198	27.0	20.2	21.0	Not satisfactory	Some exposed reinforcements & voids
61		Achaputa L/out Idemili farm Obosi	41	229	58.5	47.6	49.3	Acceptable	Some exposed reinforcements & voids
62		St Dominic cath. Oduke Obosi	36	163	61.1	51.5	53.3	Satisfactory	Some exposed reinforcements & voids
63		Nkomoro str, Owelle-Aja Obosi	38	204	47.4	35.8	37.6	Not satisfactory	Nil
64		7 Oba avenue behind army barracks Odume Obosi	47	247	72.3	53.4	56.5	Satisfactory	Some exposed reinforcements & voids
65		21 Mbaukwu str. Nkpor	24	189	45.8	54.0	50.2	Acceptable	Some exposed reinforcements & voids
66		Off Ezeiweka Rd Obosi	32	239	46.9	35.1	36.5	Not satisfactory	Few exposed reinforcements & voids
67		Okeluche Ikenga Ogidi	30	179	50.0	36.3	38.3	Not satisfactory	Nil
68		Dayspring str. Awada Obosi	30	272	60.0	64	64.5	Satisfactory	Nil
69		Okeluche Ikenga Ogidi	38	297	55.3	66.3	64.8	Satisfactory	4-floors approved but 6 floors built
70	Idemili South	Agada street Oba	20	26	75.0	57.7	65.2	Satisfactory	Nil
71		Opp. Algon compressive health centre Oba	28	35	57.1	37.1	46.0	Not satisfactory	Nil
72		Agada Rd Ofuobi estate Okofia L/out Oba	31	32	48.4	40.6	44.4	Not satisfactory	Nil
73		Along Nnobi-Nnewi Road Nnobi	10	36	40.0	33.3	37.0	Not satisfactory	Some SFC inaccessible
74		Nkpolu village Oba	32	116	46.9	42.2	43.2	Not satisfactory	Few exposed reinforcements & voids
75		Nkpolu village by Nellys' Newton hotel Oba	15	81	40.0	48.1	46.9	Not satisfactory	Nil
76	Ihiala	Orsumoghu-Isseke Rd Ihiteoha Orsumoghu	20	30	50.0	70	62.0	Satisfactory	Nil
77		Umudimogo Road Ihiala	24	25	54.2	52	53.1	Satisfactory	Nil
78		Near morning star lodge Ndiumeraku Uli	35	32	57.1	62.5	58.8	Satisfactory	Few exposed reinforcements & voids
79		Opp. St. Mary Cath. Chur. Orsumoghu.	15	20	33.3	35.0	34.3	Not satisfactory	Nil
80		Along Onitsha-Owerri Rd beside confidence oil & gas Ihiala	34	41	64.7	43.9	52.7	Acceptable	2nd floor beam & slab covered
81		Opp. Eagle destiny hotel Ndiezike village Ihiala	26	59	50.0	45.8	47.0	Satisfactory	Part of SF inaccessible

82	Njikoka	Off Nsionu Obinakueze str. Ndiagu Orofia Abagana	24	23	54.2	60.9	57.4	Satisfactory	Nil
83		Close to our savior's Ang. Church Enugwu Ukwu	25	32	60.0	65.6	70.8	Satisfactory	Nil
84		Adjecent local govt headquarters Abagana	23	27	56.5	55.6	56.0	Satisfactory	Few exposed reinforcements & voids
85		Nawfia	BM=9 GF=37	35	BM=77.8 GF=67.6	71.4	70.4	Satisfactory	Nil
86		Adjacent Emmatex petrol station Enugwu-Ukwu	80	95	87.5	94.7	91.4	Satisfactory	Nil
87	Nnewi North	Off Okigwe Rd Umudim Nnewi.	21	30	58.8		57.0	Satisfactory	Nil
88		Okigwe Rd opp. NIBRI Nnewi	9	15	66.7		62.5	Satisfactory	Nil
89		Behind college of health sciences Okofia Nnewi	BM=15 GF=43	73	BM=60.0 GF=62.8	45.2	52.7	Satisfactory	Some exposed reinforcements & voids
90		Ndimgbu Otolo Nnewi	25	60	60.0	60.0	60.0	Satisfactory	Nil
91		Adimorah Rd Obiofia Nnewichi Nnewi	26	68	46.2	14.7	23.4	Not satisfactory	Some exposed reinforcements & voids
92		Off Igwe Orizu Road Nnewi	23	62	39.1	38.7	38.8	Not satisfactory	Nil
93		Akabor edorji Uruagu Nnewi	30	142	36.7	39.4	39.0	Not satisfactory	Nil
94		Chika Ojukwu str. Nnewichi Nnewi	38	175	55.6	34.3	36.7	Not satisfactory	Some exposed reinforcements & voids
95		Close to college of health sciences Okofia Nnewi	51	149	29.4	35.6	34.0	Not satisfactory	Some apartments inaccessible
96		Umuenem Otolo Nnewi	36	166	72.2	55.3	58.4	Satisfactory	Some exposed reinforcements & voids
97		Ukwu Cashew, Okpuno Nnewichi	32	114	75.0	64.0	66.4	Satisfactory	Some exposed reinforcements & voids
98		Abubor Nnewichi Nnewi	32	143	53.1	55.2	55.0	Satisfactory	Some FFC inaccessible
99		Ekwulu L/out Okkpuno Egbu Umudim Nnewi	37	128	37.8	50	47.0	Not satisfactory	Some exposed reinforcements & voids, some SFC & TFC inaccessible
100	Nnewi South	Okigwe Road Amichi	34	39	85.3	79.5	82.2	Satisfactory	Nil
101		Nzagha Ukpor	30	26	48.4	43.8	46.0	Not satisfactory	Nil
102		Obiagu – Amichi	20	43	25.0	25.6	25.4	Not satisfactory	Diagonal cracks on GF walls
103		Close to orie Utuh market Utuh	36	34	16.7	32.4	24.3	Not satisfactory	Some exposed reinforcements & voids
104	Ogbaru	Mission Rd Iyiowa Odekpe	34	102	58.8	55.9	56.6	Satisfactory	Swampy terrain
105		Okoti Iyiowa Odekpe by Chikaita petrol station	38	175	57.9	60.6	60.1	Satisfactory	Some exposed reinforcements & voids
106		107 Obodoukwu Rd Okpoko	32	182	37.9	36.8	43.8	Not satisfactory	Vertical & diagonal cracks on FF& SF walls. Beams on external grid & stairs only
107		Amaifeke Street off Atani Road	38	202	50.0	26.2	33.3	Not satisfactory	1^{st}- 4^{th} Floor beams ran only externally Local sandstone used.

108		Obioma Ezenwa str. Iyiowa Odekpe	36	224	47.2	36.2	37.7	Not satisfactory	Water-logged terrain
109		Mission Rd Iyiowa Odekpe	47	250	38.3	27.6	29.3	Not satisfactory	Horizontal & diagonal cracks on GF internal walls
110	Onitsha North	Presidential drive Fed. housing GRA Onitsha	BM=18 GF=44	35	BM=55.6 GF=31.8	31.4	36.1	Not satisfactory	Nil
111		Near Dolly hills hotel Unity Estate, Trans Nkisi	21	29	57.1	51.7	54.0	Satisfactory	Nil
112		Plot 108 unity hall estate 3-3 Onitsha	36	33	55.6	33.3	44.9	Acceptable	Some exposed reinforcements ,& voids
113		50 Umueri str. Oyolu-Eze Onitsha	24	59	62.5	50.8	54.2	Satisfactory	Few exposed reinforcements
114		Umueri str. Oyolu-Eze Onitsha	22	56	45.5	46.4	46.2	Not satisfactory	Few exposed reinforcements
115		27 Nawfia str. Omagba phase ii Onitsha	36	136	66.7	55.8	58.0	Satisfactory	Nil
116	Onitsha South	F65 Housing Estate Fegge	26	30	65.4	50	57.1	Satisfactory	Nil
117		Venn Road Onitsha	49	178	59.2	53.4	51.2	Satisfactory	Nil
118		9 Emodi street Odoakpu Fegge	36	215	50.0	46.0	46.6	Acceptable	Nil
119		17 Rev. Egbuche street Fegge	36	154	72.2	71.4	71.6	Satisfactory	Nil
120		7 St John str. Odoakpu	40	124	0.0	0.8	0.6	Grossly Not satisfactory	Some exposed reinforcements ,& voids
121		2, Aggrey road Fegge	35	208	52.9	53.4	54.7	Satisfactory	Nil
122		6, Okwuenu Street Fegge	23	121	43.5	35.5	36.8	Not satisfactory	Vertical cracks on SF,Spalling on all floor slabs & beams,some colu-mns inaccessible
123		8 Unegbe lane Odoakpu	28	178	46.4	19.7	27.0	Not satisfactory	Some FFC inaccessible
124		26 Obosi street Feggee	34	269	67.6	68.0	64.8	Satisfactory	Few 5th floors cols inaccessible
125		28 Nnewi street Odoakpu	24	266	58.3	47.4	47.9	Acceptable	Nil
126	Orumba North	Obinikpa village Amaokpala	52	51	26.9	31.4	29.1	Not satisfactory	Nil
127		Close to exclusive lodge Oko	37	44	29.7	40.9	35.8	Not satisfactory	Some exposed reinforcements ,& voids
128		Pal Junction Oko	8	16	87.5	68.8	75.0	Satisfactory	Nil
129		Ndiagu Oko	30	81	60.0	67.9	65.8	Satisfactory	Nil
130		Emenwosu extension Amaokpala	32	144	50.0	32.6	35.8	Not satisfactory	Some exposed reinforcements ,& voids
131		Ndiagu Uzelle Ifite Oko	24	102	41.7	43.1	42.9	Not satisfactory	Few exposed reinforcements ,& voids
132		Permanent site Ndiagu Oko	28	108	67.9	83.3	80.1	Satisfactory	Nil
133		Oko-Umunze Rd Amaokpala	64	176	39.1	39.8	39.6	Satisfactory	Some rooms inaccessible
134		Near elite 5-star hotel Amaokpala	66	206	39.4	31.1	33.1	Satisfactory	Nil
135	Orumba South	Obikpa L/out Ohukabia village Nawfija	27	40	48.1	30	37.3	Not satisfactory	Few exposed reinforcements ,& voids
136		Along Umuchu-Umunze Rd Umunze	21	29	42.9	42.9	48.3	Not satisfactory	Nil
137		Near Orie markrt Ogbunka	22	24	31.8	54.2	43.5	Not satisfactory	Some exposed reinforcements ,& voids

138		Owerri-Ezukalla Rd Ubaha Umunze	10	41	30.0	9.8	13.7	Not satisfactory	Few exposed reinforcements ,& voids
139		Agbirigba Lomu Umunze	12	40	58.3	40	44.2	Acceptable	Nil
140	Oyi	Nkwelle Ezunaka	31	36	16.1	11.1	13.4	Not satisfactory	Horizontal cracks on 1st floor internal wall
141		Near St. Paul's Ang. Church Nteje	33	34	51.5	38.2	44.8	Acceptable	Nil
142		Adjacent God promotion holy ministry Okpuno Azu Ogbunike	43	97	44.5	15.5	24.5	Not satisfactory	Beams on external grid & stairs only
143		Maryland Estate Nkwelle Ezunaka	45	85	44.4	47.1	46.2	Not satisfactory	Nil
144		Beside pink view hostel Mkpomachi village Awkuzu	65	219	49.2	47.0	47.2	Not satisfactory	Some exposed reinforcements ,& voids
145		Umuezunu Nkwelle Awkuzu	24	65	45.8	49.2	48.3	Not satisfactory	Some exposed reinforcements ,& voids
146		Near keystone bank Ogbunike	16	54	56.3	38.9	42.8	Not satisfactory	Few exposed reinforcements ,& voids
147		Ezinkwo Awkuzu	34	94	52.9	22.3	30.5	Not satisfactory	Nil
148		Behind bicycle spare parts Ogbunike	37	158	35.1	47.5	45.1	Not satisfactory	Some exposed reinforcements ,& voids
149		Awkuzu Junction	15	71	33.3	47.9	45.3	Not satisfactory	Nil
150		Near Saga Foam Industry Azu Ogbunike	34	208	52.8	49.0	52.5	Satisfactory	Nil

Discussions

Fig. 5 shows the compressive strength of the structural elements tested for the 150 buildings in percentage range while table 3 shows the researchers' rating of the results.

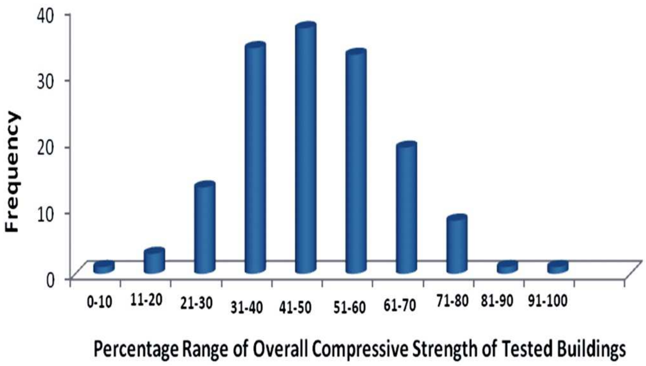

Fig. 5: Percentage Range of the Compressive Strength of Tested Buildings in Anambra State.

The investigations in this research have shown the performance of the structural elements tested and measures for their maintenance in order to sustain structural stability. Four basic categories were identified based on the quality of the tested components.

One (1) building out of the150 buildings assessed recorded exceedingly poor compressive strength with the percentage of the tested elements that met the threshold value of 20KN being 0.6% and classified within 1-10%. Buildings with such extremely low concrete compressive strengths are susceptible to imminent collapse and are remarked Danger Alert. The wise recommendation for such cases is demolition unless alternative remediation strategy is proffered by an experienced structural engineer or builder.

Three (3) buildings were rated "Extremely Poor" with 11-20% pass range, though not as dangerous as the

Table 3: Researchers' rating of the results.

Range (%)	Rating (Researchers' view)	
1 – 10	Danger alert	Alarming Risk Alert
11 – 20	Extremely Poor	
21 – 30	Very Poor	High Risk Alert
31 – 40	Poor	
41 – 50	Bad	Low Risk Alert
51 – 60	Fair	No Risk Alert
61 – 70	Good	
71 – 80	Very Good	
81 – 90	Extremely Good	
91 – 100	Excellent	

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Assessment of Concrete Facilities by Non-Destructive Methods in Anambra State, Nigeria: *A Comparative Study*

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Abstract

This study assesses the integrity of five different concrete facilities located at five construction sites in Anambra State by non-destructive methods. This is aimed at obtaining a comparative relationship between two non-destructive test methods. To this end, Schmidt rebound hammer and Portable Ultrasonic Non-Destructive Digital Indicating Tester (PUNDIT) were employed in evaluating the compressive strengths of the tested facilities. Three (3) test points were randomly selected to get a good representation on each structural element such that 3 points tested by the Pundit machine correspond to three points taken by the Schmidt hammer. Test results from both Schmidt rebound hammer and UPV Pundit were subject to certain limitations and this gave rise to discrepancies in their readings with rebound hammer giving higher values in most cases than pundit. Analysis reveals these disparities in compressive strength values to range from 0.0 to 9.3 N/mm2. 72% of these disparities are less than 5 N/mm2 while 28% is greater than 5N/mm2 but not up to 10 N/mm2. This proximity in values indicates good correlation between readings from both machines.

Keywords: *Ultrasonic pulse velocity, rebound hammer, non-destructive test, compressive strength, concrete.*

Introduction

Non-destructive test (NDT) of concrete is a method used to obtain the compressive strength and other properties of concrete from the existing structures. NDT measures the in-depth properties of concrete for quality assurance and for evaluations of existing conditions. Being Non-Destructive, they do not impair the function of the structure and they permit retesting at the same locations to evaluate changes in properties with time.

Several non-destructive methods of assessment have been developed. These depend on the fact that certain physical properties of concrete can be related to strength and can be measured by non-destructive methods. Such properties include hardness, resistance to penetration by projectiles, rebound capacity and ability to transmit ultrasonic pulses and X- and Y-rays. The various methods of Non-Destructive Testing of concrete include ultrasonic pulse velocity (UPV) method, rebound hammer method, pull out test method, penetration method and radioactive methods (David, 1981; Malhotra,1976; Neville,1996).

Our focus in this research is on the first two: UPV method by the use of Portable Ultrasonic Non-destructive Digital Indicating Tester (PUNDIT) and rebound hammer method by the use of Schmidt hammer.

The concept behind UPV technology is measuring the travel time of acoustic waves in a medium, and correlating them to the elastic properties and density of the material. Travel time of ultrasonic waves reflects internal condition of test area. In general, for a given trajectory, higher travel time is correlated to low quality concrete with more anomalies and deficiencies, while lower travel time is correlated to high quality concrete with fewer anomalies.

The application of UPV test in testing concrete materials is becoming popular with the recent advancement in transducer technology and it has been proved an effective NDT method for quality control, detecting damages in structural components and crack depth estimation, assessment and uniformity of concrete materials. For a reliable ultrasonic testing of concrete, the test must be done on surfaces that are clean and free of dust with suitable couplant needed to establish an ideal connection between concrete and UPV transducers. Special attention should be given to rebar in concrete, since the wave travel speed in metal is much higher than in concrete. (Baldev and Rao, 1995; Breysse, 2012; Breysse and Balayssac 2018). The test procedure has been standardized as "Standard Test Method for Pulse Velocity through Concrete" (ASTM C 597, 2016).

The Rebound Hammer is a device developed in 1948 by a Swiss engineer named Ernst Schmidt which is commonly used method for estimating the compressive strength of in-place concrete. The device measures the hardness of concrete surfaces using the rebound principle. Many factors affect rebound hammer test results. Such factors include surface smoothness, age of concrete, moisture content, surface carbonation, presence of aggregate, air voids, and steel reinforcement, temperature and calibration of rebound hammer (Ongpeng, 2018).

General Geologic Setting of Study Area

The five sites visited and the facilities tested are given in table 1 while their general geologic setting is discussed below.

Table 1: Location and details of tested facilities in Anambra State

Sites	Location	Tested Facility	Tested Structural Element
A	Abagana, Anambra State	Drain project	Walls, Stilling Basin & Flow Channel
B	3-3 Onitsha	Reinforced Concrete Bridge and a Retaining Wall	Piers, Beams and Slab
C	Agu-Awka	Substructure of an ongoing massive building	Retaining Walls
D	Enugwu-Ukwu	Reinforced Concrete Box Culvert	Head & Wing Walls
E	Omagba Phase II, Onitsha	Storm water drainage	Chute Wall, Stilling Basin & Flow Channel

Abagana, 3-3 Onitsha, Agu-Awka, Enugwu-Ukwu and Omagba are communities in Anambra State and the geology of Anambra state as documented falls within the Anambra basin. The oldest sediment of the Anambra Basin was deposited in Late Campanian, comprising Shale, Sandstone and Enugu Shale. It also comprises succession of siltstone, shale, coal seam and sandstone which is mainly unconsolidated coarse-fine grained, poorly cemented; mudstone and siltstone. Ajali Sandstone is overlain by Upper Coal Measure which comprises clayey shale with occasional ironstone. The geology of Anambra State has the Ajali Sandstone, Nsukka Formation, Imo Formation, Ameki Formation, Ogwashi - Asaba Formation and Benin Formation as the main deposits. (Akande et al., 2015; Kogbe, 1989).

Four of the research sites are underlain by Ameki

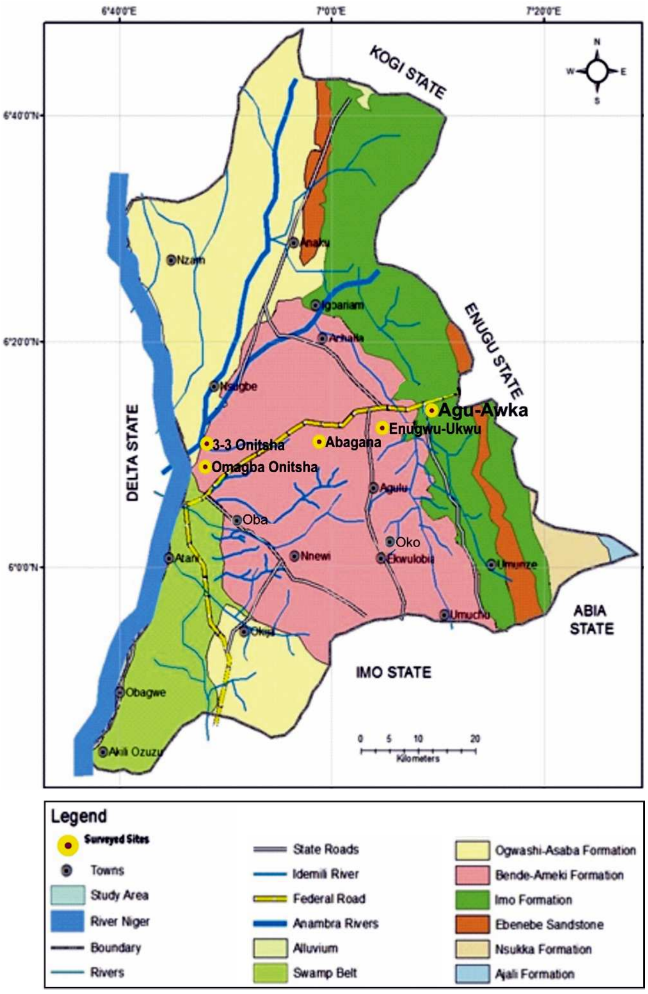

Fig. 1: Geologic map of Anambra State with the research areas highlighted in yellow (NGSA, 2018).

formation while the site in Agu-Awka is underlain by Imo formation as seen in Fig. 1.

Methodology

The field work for this research was carried out with the use of a Standard Portable Ultrasonic Non-Destructive Digital Indicating Tester (PUNDIT) and the Schmidt Rebound Hammer (Fig. 2). Three (3) test points were randomly selected on each structural element and assessed using both the Pundit and Schmidt hammer machines. Profoscope is employed to detect location of reinforcement bars in tested facilities that were rendered.

Non-Destructive, as the name implies means that the materials undergoing the test are not damaged during the test. The operating principles of the instruments are discussed below.

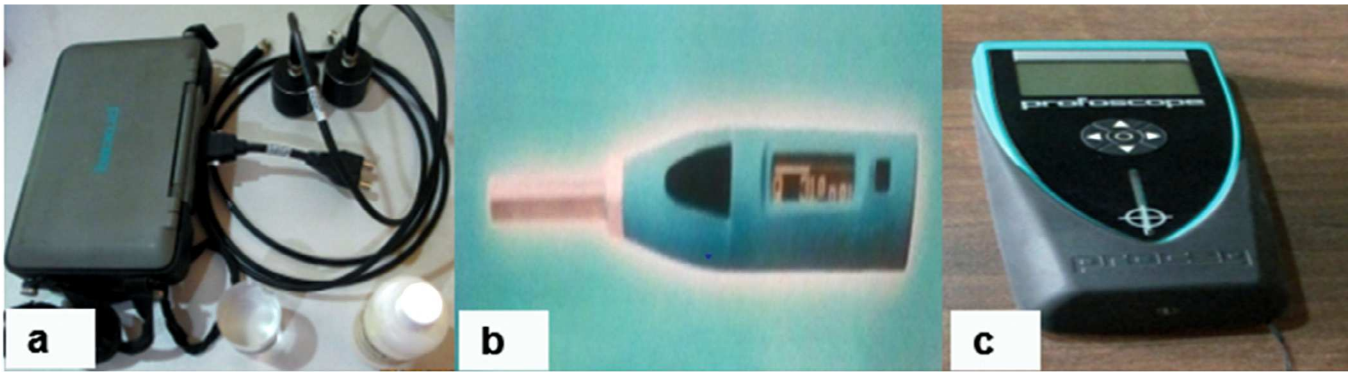

Fig. 2: Equipment used in the study. (a) PUNDIT, (b) Rebound hammer & (c) Profoscope

The Portable Ultrasonic Non-Destructive Digital Indicating Tester (Pundit).

A pulse of longitudinal vibrations is produced by an electro-acoustical transducer, which is held in contact with one surface of the concrete under test. When the pulse generated is transmitted into the concrete from the transducer using a certified coupling gel material, it undergoes multiple reflection as the boundaries of the different materials phases within the concrete. A complex system of stress waves develops, which include both longitudinal and shear waves, and propagates through the concrete. The first waves to reach the receiving transducer are the longitudinal waves, which are converted into an electrical signal by a second transducer. The transit times are measured by the system electronics and converted to human readable formats for analysis and presentation.

Schmidt Rebound Hammer

The rebound hammer consists of a spring-controlled hammer mass that slides on a plunger within a tubular housing. The hammer is forced against the surface of the concrete by the spring and the distance of rebound is measured on a scale. The rebound distance is a close function of the hardness of the concrete structure being tested. In modern versions of the instrument, in-built electronics and embedded software give an immediate read-out of the compressive strength of the test element.

Profometer or Profoscope

The profoscope is established for the measurement of concrete cover over rebars, for measuring rebar diameters and for determining the exact location of rebars in a concrete structure. In this study, profoscope is employed to detect location of reinforcement bars in tested facilities that were rendered.

Results

A minimum concrete design strength of 25 N/mm^2 is assumed for evaluation in this analysis and the concrete compressive strength values of the assessed facilities are given in tables 2, 3, 4, 5 and 6. Values below 15N/mm^2 are considered very poor and symbolized with (××); 15 -19.9 N/mm^2 are considered poor with (×) symbol; 20–24.9 N/mm^2 are considered allowable and identified with the symbol (√) while the elements that met the threshold value of 25N/mm^2 and above are considered very good/ acceptable and identified with (√ √).

In few cases, pundit and rebound hammer gave same reading on same elements. These are indicated with the symbol **(**)**.

Discussions

From site A, the test analyses indicate that Pundit gives higher compressive strength than rebound hammer. Using the pundit values, only 33,3% of the tested elements were remarked poor and this shows moderately strong facility. Only one out of the fifteen structural elements tested recorded a higher rebound hammer value than that of Pundit. The disparity between the compressive strength values recorded by the two equipment ranges from 0.7 – 7.8N/mm^2.

None of the structural elements tested from site B met the threshold value of 25N/mm^2 in compressive strength value. This shows poor concrete mix. Contrarily to what is obtained from site A, rebound hammer recorded higher compressive strength values than Pundit for majority of the elements tested. The disparity between the compressive strength values recorded by the two equipment ranges from 0.2 – 3.6N/mm^2.

Table 2: Compressive strength results from site A

Tested Structure	Average Compressive Strength (N/mm²)		Disparities in values
	Rebound Hammer	PUNDIT	
Finger I Gully Head Wing Wall LHS	18.0 ×	25.7 √√	7.7
Finger I Gully Head Inlet Control Structure LHS	17.8 ×	22.7 √	4.9
Finger I Gully Head Chute Wall LHS	17.5 ×	25.3 √√	7.8
Finger I Gully Head Chute Wall RHS	18.5 ×	22.7 √	4.2
Drain Wall Flow Channel LHS	21.3 √	22.0 √	0.7
Second Point of Flow Channel LHS	18.8 ×	15.0 ×	3.8
Finger II Stilling Basin Wall LHS	14.7 ××	16.3 ×	1.6
Finger II Drain Wall Flow Channel RHS	15.8 ×	17.7 ×	1.9
Second Point of Flow Channel RHS	15.0 ×	18.0 ×	3.0
Finger II Stilling Basin Wall RHS	10.5 ××	18.3 ×	7.8
Confluence Link Drain Wall, 1st Segment from Finger II LHS	22.3 √	23.3 √	1.0
Second Point of Link Channel from Finger II	21.8 √	24.0 √	2.2
2nd Segment Link Wall LHS	25.5 √√	27.0 √√	1.5
1st Segment Link	19.7 ×	25.0 √√	5.3
2nd Point of Link Channel from Finger II RHS	18.0 ×	24.3 √	6.3

As seen from the table of analysis of the substructure tested in site C, few of the structural elements fall within good and some others under allowable category. Those elements that fall within poor category were considered generally not critical particularly with reference to the expected imposed load and the structural system (it is intended for a Conference Centre). Rebound hammer recorded higher compressive strength values than Pundit for all but one elements tested. The disparity between the compressive strength values recorded by the two equipment ranges from 0.0 – 9.3N/mm².

All structural elements tested in site D and E fall within allowable/acceptable category with good concrete compressive strength which is an indication of both good quality mix, good workmanship and professional supervision/execution.

Table 3: Compressive strength results from site B

Structural Element	Average Compressive Strength (N/mm²)		Disparities in values
	Rebound Hammer	PUNDIT	
Reinforced Concrete Bridge			
Pier 1	10.7 ××	14.3 ××	3.6
Pier 2	12.5 ××	11.7 ××	0.8
Pier 3	14.7 ××	15.7 ×	1.0
Pier 4	15.0 ×	13.0 ××	2.0
Pier 5	13.0 ××	15.0 ×	2.0
Pier 6	15.7 ×	16.3 ×	0.6
Pier 7	17.2 ×	16.0 ×	1.2
Pier 8	16.0 ×	17.0 ×	1.0
Pier 9	17.3 ×	14.7 ××	2.6
Pier 10	12.3 ××	15.0 ×	2.7
Beam 1 (Longitudinal)	14.2 ××	13.0 ××	1.2
Beam 2 (Transverse)	15.8 ×	13.0 ××	2.8
Beam 3 (Longitudinal)	17.0 ×	14.7 ××	2.3
Beam 4 (Transverse)	16.3 ×	16.0 ×	0.3
Beam 5 (Longitudinal)	17.5 ×	16.3 ×	1.2
Beam 6 (Transverse)	19.7 x	17.3 ×	2.4
Slab 1	16.7 ×	14.7 ××	2.0
Slab 2	15.2 ×	12.7 ××	2.5
Slab 3	16.8 ×	15.0 ×	1.8
Slab 4	13.7 ××	14.7 ××	1.0
Slab 5	16.5 ×	14.0 ××	2.5
Retaining Wall			
Pier 1	13.7 ××	13.0 ××	0.7
Pier 2	14.3 ××	13.7 ××	0.6
Pier 3	11.0 ××	10.7 ××	0.3
Pier 4	12.0 ××	14.7 ××	2.7
Pier 5	14.2 ××	14.0 ××	0.2
Pier 6	12.5 ××	13.3 ××	0.8
Pier 7	11.5 ××	12.3 ××	0.8

Table 4: Compressive strength results from site C

Structural Element	Average Compressive Strength (N/mm²)		Disparities in values
	Rebound Hammer	PUNDIT	
Front Retaining Wall			
Tested Point 1	22.3 √	19.5 ×	2.8
Tested Point 2	21.3 √	20.0 √	1.3
Tested Point 3	24.6 √	18.0 ×	6.6
Tested Point 4	25.7 √√	16.5 ×	9.2
Tested Point 5	20.3 √	16.0 ×	4.3
Tested Point 6	22.5 √	18.5 ×	4.0
Left Hand Side Retaining Wall			
Tested Point 1	20.0 √	15.0 ×	5.0
Tested Point 2	21.0 √	15.5 ×	5.5
Tested Point 3	21.9 √	16.0 ×	5.9
Tested Point 4	24.0 √	15.0 ×	9.0
Tested Point 5	24.0 √	18.5 ×	5.5
Tested Point 6	21.5 √	18.0 ×	3.5
Rear Retaining Wall			
Tested Point 1	22.8 √	16.0 ×	6.8
Tested Point 2	23.0 √	18.0 ×	5.0
Tested Point 3	19.0 ×	19.0 ×	0.0**
Tested Point 4	22.8 √	18.5 ×	4.3
Tested Point 5	18.0 ×	21.5 √	3.5
Tested Point 6	21.8√	19.5 ×	2.3
Right Hand Side Retaining Wall			
Tested Point 1	26.0 √√	19.0 ×	7.0
Tested Point 2	25.8 √√	19.5 ×	6.3
Tested Point 3	22.5 √	20.0 √	2.5
Tested Point 4	21.3 √	17.0 ×	4.3
Tested Point 5	26.8 √√	17.5 ×	9.3
Tested Point 6	25.3 √√	19.5 ×	5.8

Table 5: Compressive strength results from site D

Tested Structure	Average Compressive Strength (N/mm²)		Disparities in values
	Rebound Hammer	PUNDIT	
Head Wall RHS	34.2 √√	28.7 √√	5.5
Head Wall RHS	36.2 √√	33.7 √√	2.5
Head Wall LHS	33.5 √√	26.7 √√	6.8
Head Wall LHS	32.5 √√	28.3 √√	4.2
Wing Wall LHS	23.0 √	28.7 √√	5.7
Wing Wall LHS	36.3 √√	33.0 √√	3.3

Table 6: Compressive strength results from site E

Tested Structure	Average Compressive Strength (N/mm²)		Disparities in values
	Rebound Hammer	PUNDIT	
Chute Wall			
Tested Point1	22.5 √	30.0 √√	4.5
Tested Point 2	28.0 √√	31.0 √√	3.0
Tested Point 3	25.0 √√	28.0 √√	3.0
Stilling Basin			
Tested Point1	26.0 √√	32.0 √√	6.0
Tested Point 2	28.0 √√	30.0 √√	2.0
Tested Point 3	28.0 √√	28.0 √√	0.0**
Flow Channel			
Tested Point1	21.0 √	26.0 √√	5.0
Tested Point 2	20.5 √	24.0 √	3.5
Tested Point 3	23.0 √	26.0 √√	3.0

Rebound hammer recorded higher compressive strength values than Pundit for all elements except one in site D and the disparity in compressive strength values between readings from the two equipment ranges from 3.3 – 6.8N/mm^2. For all tested elements in site E, pundit recorded higher compressive strength values than rebound hammer and disparity in their values ranges from 0.0 – 6.0.

Talking the rate of disparities between pundit and rebound readings into consideration, it was discovered that in most cases (72%), the actual difference between the two readings is less than 5N/mm^2 as seen in table 7. This indicates a close- range correlation between UPV pundit and Schmidt hammer.

These discrepancies may be attributed to the fact that UPV pundit picks microstructural imperfections while Schmidt rebound emphasizes surface hardness, concrete moisture condition and effects of gravity.

Table 7: Discrepancy Rate between PUNDIT and Rebound Readings.

Disparity Range	Frequency	Percentage Frequency	% Grouping
0.0 – 0.9	12	14.6	< 5 =72
1.0 -1.9	12	14.6	
2.0 – 2.9	17	20.7	
3.0 – 3.9	10	12.2	
4.0 – 4.9	8	9.8	
5.0 – 5.9	10	12.2	> = 28
6.0 – 6.9	6	7.3	
7.0 – 7.9	4	4.9	
8.0 – 8.9	-	-	
9.0 – 9.9	3	3.7	
Total	**82**	**100**	

Comparative Analysis between Schmidt Rebound Hammer and UPV Pundit

The following similarities and differences were adjudged from this study:

- UPV is conducted by passing a pulse of ultrasonic through concrete to be tested and measuring the time taken by pulse to get through the structure while rebound hammer is a surface hardness test which has little or nothing to do with the internal condition of the test specimen
- Higher velocities in UPV method indicate good quality and continuity of the material, while slower velocities may indicate concrete with many cracks or voids. Similarly, higher rebound reading indicate stronger materials.
- Rebound hammer test evaluates surface hardness and in no way gives the exact compressive strength. In the same vein, the large number of variables affecting the relation between strength and pulse velocity limits the use of UPV for predicting strength.
- Ultrasonic pulse velocity tests have a great potential for concrete control, particularly for establishing uniformity and detecting cracks or defects while the rebound hammer method can be used with greater confidence for differentiating between the questionable and acceptable parts of a structure or for relative comparison between two different structures.

Conclusion

The test results show a reasonable correlation between UPV pundit and Schmidt rebound readings with close-range discrepancies in values. We therefore conclude that NDT is one way of having a better and economical test on concrete although the heterogenic nature of concrete mixture poses a limitation. In view of the limitations of each method of non-destructive testing of concrete, it is essential that each method should be adopted very carefully with respect to desired objective.

Recommendations

- We hereby recommend that where precision is required, the results of tests obtained by one non-destructive method be complimented by those from another. On a larger scope, results from NDT should be compared with those from destructive methods.
- It is ;lequally recommended that minimum concrete compressive strength of 25(N/mm^2) be maintained on all construction sites to ensure integrity and durability of structures.
- Periodic inspection and regular maintenance are highly recommended for all engineering structures to enhance structural health and service life.

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Geo-Environmental Mapping of Contaminated Land for Effective Remediation

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Abstract

Geo-environmental mapping of contaminated sites provides not only the road map but also the framework for effective and sustainable remediation of oil impacted sites. It presents site specific data which highlights the nature of the contaminated site and its locus within the regional environmental setting thereby aiding the planning, design and execution of the remediation action plan for sites. Remediation projects undertaken without a geo-environmental map is akin to carrying out a surgical operation blindfolded. It provides data on the sensitivity of the environment to the pollution and also the remediation methods. Thus, it is a guide to the level of intervention and the limits of action required to resolve the contamination problem of the site. Therefore, geo-environmental mapping of any site should necessarily become an integral part of the net environmental benefit analysis of the remediation strategies in order not to cause more harm than good to the environment during the course of the remediation project.

Keywords: *Geo-environmental, Remediation, Physiographic, Biogeophysical, Vulnerability*

Introduction

A Geo-environmental map is a graphic representation of the earth or parts of the earth's surface on a planar surface conveying specific information on the inter-relationships and interactions within the biogeophysical environment and man. It spotlights the vulnerabilities within any locality to particular land use and the impacts of human activities or natural disasters within the environment. Geo-environmental mapping procedures adopts an interdisciplinary approach in the mapping of geologic features, hazards and their inter-relationships within the locus of interest. It seeks to present risks posed by the presence of contaminants of concern within the upper layers of the earth to man and the environment. Geo-environmental mapping, therefore, highlights the vulnerabilities of the 'type geology' of an area to pollution; specifying how anthropogenic activities will compromise the full functionality of the various components of the environment. Relying on the strength in the multi-disciplinarity of information from a broad realm of geoscientific fields such as geology, geochemistry, hydrology, hydrogeology, geophysics, soil mechanics, geomorphology, etc., geo-environmental mapping techniques are used to assess, monitor and plan remediation strategies for contaminated lands.

Geo-environmental mapping approach is a specialised application of GIS and cartographic techniques in the resolution of environmental hazards within contaminated sites. It employs suites of site specific investigations to delineate 'sensitivity zones' based on site conditions. Geo-environmental mapping as a communication tool, seeks to convey certain information not only to policy makers, but also to the general public. Thus, it presents spatial distribution patterns and aids in the visualization of these patterns and processes. One of the key benefits of a geo-environmental map is the expression of concepts and ideas that are verbally difficult to convey or portray. Thus, they facilitate the systematic monitoring of the changing dynamics of the contaminated land and the most efficient remediation strategies. It provides the basis for making environmentally sound decisions towards a sustainable future.

Effective remediation and restoration of a contaminated land leans heavily on a good understanding of the interactions between the biophysical and anthropogenic domains. The exploitation, processing and utilisation of environmental (mineral) resources to satisfy man's sundry needs often results in the degradation of environment which creates a dis-balance within the earth system. Remediation activities are thus designed to correct this dis-balance and restore the prior functionalities of the various components of the environment, thereby maintaining an optimal ecological health capable of supporting the livelihood of dependent communities. Hence, geo-environmental mapping gathers datasets in varied spatial and temporal scales and applies same in the restoration of contaminated lands (Fabbri & Patrono, 1995; Chakrabarty, 2014). Space Information Technology provided by Earth Observing Satellites (EOS) furnishes the geo-environmental mapping process with geospatial information that affords the user the birds-eye-view scenario during the conceptualisation, design and implementation of the remediation and restoration strategies of contaminated lands.

Oil industry operations in the Niger River Delta has been accompanied by several episodes and differing magnitudes of oil spills, resulting in hundreds of contaminated lands in virtually all geomorphic units of the delta. An estimated, staggering volume of 9-13 *circa* million barrels of crude oil have been spilled into the Niger Delta environment since 1958, in over 6500 spill incidents; averaging about 150 spills annually (Baird, 2010; Kadafa, 2012). In 2009, the United Nations Environment Programme (UNEP) in carrying out the Environmental Assessment of Ogoniland, Eastern Niger River Delta, applied geo-environmental mapping techniques in gathering, storing, processing and presenting oil spill data covering an area of approximately 900km^2. More recently, in the period spanning between 2015-2018, the Bodo Mediation Initiative (BMI) in collaboration with Shell Petroleum Development Company (SPDC) applied the Shoreline Clean-up Assessment Technique (SCAT), an archetype of geo-environmental mapping in coastal environments, in obtaining and evaluating data about the nature and degree of oiling of the various sub-environments within the Bodo estuary covering an area of approximately 1000 hectares (10 km^2). The data sets obtained in these cases formed the framework around which the remedial action plan for the restoration of contaminated lands in Ogoniland and also in the severely impacted Bodo creek was woven. Advancements in space technology and satellite imagery have made geo-environmental mapping an indispensable tool in managing the restoration of contaminated lands. Thus, the use of GIS greatly improves the accuracy in representing specific information on the geo-environmental map.

Contaminated Lands, Assessment and Mapping for Remediation Action Plan

A contaminated land refers to sites that has hazardous substances in or on it that are either naturally occurring or emanating from anthropogenic sources; which has significant adverse effects on the biota that subsist within that environment. Oil spills are increasingly becoming the most common environmental hazard that undermines the environmental quality of the Niger Delta. These spills emanate from oil infrastructure which abound in the region. They are caused by either equipment failure or third party interdiction of the oil infrastructure. Illegal bunkering and artisanal refining of crude oil is increasingly becoming a serious threat to environmental quality of coastal communities (Giadom, 2017). Oil pollution destroys coastal biomes, especially mangrove sanctuaries, contaminates surface and groundwater, introduces carcinogens into the food chain and undermines the capacity of the ecosystem to absorb wastes and provide the essentials for life (Giadom, 2010; Nwilo & Badejo, 2005).

Mangrove colonies, estuaries, coastal lagoons and seascape are environmentally sensitive areas whose ecosystem services include the provision of spawning grounds, roosting and breeding sites for fish and a variety of wildlife. Coastal community dwellers depend on marine resources for their livelihoods. All these become increasingly vulnerable with these ubiquitous spills, therefore, it becomes imperative that whenever a spill incident occurs, they should be systematically assessed, mapped before clean-up, remediation and restoration methods and strategies are recommended. Geo-environmental map provides a generalized interpretation of the geologic controls that determines the expression of contaminant hazards within sites and how these hazards can potentially constitute risks to receptors in an environment. Sadly, most remediation projects in the Niger Delta, are undertaken without a proper characterisation and mapping of these contaminated sites and thus fail to meet up neither our national nor international regulatory standards. Thus, a trail of contaminants of concern still subsist in some 'remediated lands' in concentrations that potentially present significant levels of risks to man and other biota within the environment.

Site Characterisation and Geo-environmental Mapping:

Site characterisation involves suites of programmes designed to obtain detailed information about any site. The objectives of the characterisation of any contaminated site seeks to acquire detailed understanding on not only the nature (geology, geomorphology, etc) of a site but also the dynamics of the interactions between the contaminants and the various components of the environment. It will seek answers, for instance, on the fate and transport of the contaminant in both the vadose and phreatic zones of an aquifer. Thus, it requires the delineation of the contaminant migration plume to establish the 'sphere of influence' and hence, clarify the magnitude of treatment that should be planned for during the actual remediation of the site.

Site characterisation integrated into geo-environmental mapping programmes often deploy physiographic compartmentalisation techniques to segregate the contaminated lands into analogous units whose genetic similarities in terrain and rock type enhances the prediction of the behaviour of the contaminants within those units. The characterisation of terrain units during the course of data acquisition for geo-environmental mapping, consist of ascribing and evaluating relevant properties and terrain components capable of affecting ground conditions during and after remedial interventions. This characterisation technique is usually achieved either directly or indirectly through the following means: (a) ground observations and measurements, in-situ tests such as tracer tests, lithologic profiling, etc, (b) laboratory tests such as grain size analysis, strength, porosity, permeability, etc (c) inferences from previous mapping exercises, remote sensing, geophysical and geochemical records (Fernadez-da-Silva, et al, 2010; Shi, et al, 2004; Zhu, et al, 2001; Zhu & Mackay, 2001)

Geo-environmental mapping requires the acquisition of very large assemblage of field data, whose storage, processing and eventual utilisation during the planning stages of any remediation project makes GIS an indispensable tool. The availability of Space Information Technology within the public domain furnishes us with imageries, aerial photographs and other data from Earth Orbiting Satellites, overtly simplifying the task of producing a geo-environment map with high degree of resolution and accuracy. GIS tools facilitates the utilisation of spatial information contained in images by incorporating and advancing the logic and procedures of geological-geomorphological photo-interpretation to decipher information in imageries (Sabins, 1987; Vedovello, 1993; Soares & Fiori, 1976). Satellite imageries could present different shades of colour, texture and patterns whose tonal frequency could depict a contaminant plume, freshwater resources, sensitive ecological zones, etc., which would have been otherwise difficult to identify on its individual spatial resolution. Hence, it becomes relatively easy and cost effective to map large areas on the earth's surface with ease. Cumbersome and expensive field visits can thus be reduced to targeted reconnaissance surveys and 'ground-truthing' of remotely sensed data. Furthermore, new datasets can be acquired from contaminated lands in difficult and inaccessible terrains, where prior data could sparse, discontinuous or even non-existent. Therefore, geo-environmental mapping for remediation of contaminated lands covers large areas with a variety of spatial resolutions, relatively frequent and periodic updating of data at reduced cost (Schmidt & Glaesser, 1998; Lillesand & Kiefer, 2000; Latifovic, et al, 2005; Akinwumi & Butler, 2008).

Fig. 1: Aerial photograph of a contaminated site showing an impacted water pond. Source: UNEP, 2009.

Integrating detailed analysis, interpretation and visualisation of geological, geotechnical and geophysical datasets for a greater understanding of the nature of contaminated sites helps in deriving maximum value from acquired data in remediation projects. Understanding the nature of the subsurface particularly is key to understanding the behaviour of contaminants of concern for instance, if and when they find their way to the subsurface. Site characterisation, therefore is the most important and most difficult component of environmental geo-engineering. The success or failure of an environmental restoration project such as remediation/Clean-up of contaminated lands therefore, depends on a well-designed site characterisation and geo-environmental mapping. It facilitates decision mechanisms when conducting the ***Net Environmental Benefit Analyses (NEBA)*** as a necessary guide to all actions to be taken in the restoration of the environment.

Shoreline Clean-up Assessment Technique (SCAT) an Effective Geo-Environmental Mapping Tool

The Shoreline Clean-up Assessment Technique (SCAT) originated during the response to the 1989 *Exxon Valdez oil spill*, when responders needed a systematic way to document the spill's impacts on many miles of affected shoreline. The SCAT approach uses standardized terminology to document shoreline oiling conditions. Over the years, many techniques have been used to describe oiled shorelines (Owens and Sergy, 2003). The SCAT approach and documentation protocols were initially developed in 1989 during spill response operations for the *Nestucca* and *Exxon Valdez*. Subsequently, generic second-generation SCAT protocols were developed (Owens and Sergy, 1994). Other agencies adopted the approach and produced similar field forms and manuals, e.g., European Commission – Jacques et al., 1996; NOAA, 2013.

Case Study 1: *Geo-environmental Mapping and the Remediation of Bodo Creek*

In August 2008 and again in December 2008, two major oil spills occurred in the Bodo creek along SPDC Trans Niger Pipelines, spilling between 103,000-311,000 bbls of crude oil into the creeks and impacting an area greater than 2000 ha, according to an estimate by Accufacts a US oil spilling modelling firm. This makes history as the largest oil spill despoliation of the mangrove ecosystem in any part of the world. Ultimately, the remediation and restoration of the Bodo Creek have been ranked the largest and most complex remediation project in a mangrove habitat in the world (Gundlach, 2015). The oil spill inundated the Bodo estuary from the time of its occurrence in 2008 and by 2015 seven years post spill, free phase crude oil still persisted on surface waters within the creeks, waterways and fishing grounds. This birthed the Bodo Mediation Initiative-SPDC Remediation Project, an initiative of the Dutch embassy in Nigeria; which adopted SCAT, a geo-environmental mapping protocol for the assessment, planning and management of clean-up, remediation and restoration of the Bodo estuary.

The remediation of the Bodo Creek project has deployed the geo-environmental mapping process to obtain datasets that are used for planning the remediation strategies. The use of satellite imageries and gridded (200m x 200m) geo-environmental maps enhances SCAT assessments to achieve a high resolution and the required focal clarity for recommending the remediation action plan for each grid or clusters of grids. It provides the guide for monitoring the progress of remediation works and also notifies on change of methods when particular work methods or strategies are not achieving the envisioned end result, or when it is observed that subsisting work methods might potentially cause more harm than good. Furthermore, geo-environmental mapping, enables the segmentation of the shoreline into various sub-types based on morphology, sediment type, degree of oiling, accessibility, etc. It aids in the planning and the decision making mechanisms put in place to manage contaminated sites and the monitoring of recovery of the contaminated sites, during and after the remediation projects have been completed.

For instance, grid C3 in figure 2 reveals the type and nature of substrates along the shoreline, the particular species of dominant mangroves or any other vegetation present or affected by the oil spill. It further shows the land use and the proximity of the grid to human habitation. These suites of data are of considerable importance when deciding the Site Specific Target Levels (SSTLs) to be recommended for clean-up end points. Land use is critical in deciding target levels which in turn determines the remediation methods and intensity to be deployed. Gridding also enables the clustering of areas with different levels of impact. Hence, it becomes possible to delineate a cluster of grids as High, Medium or Low impact grids. Thus, geo-environmental mapping has been key in the management of the clean-up, remediation and restoration of the Bodo estuary. SCAT geo-environmental maps show the variations in the sediment type, contaminant type, concentrations and weathered status within the grids represented on the maps. Thus, geo-environmental mapping becomes the *sine qua non* of an effective clean-up, remediation and restoration programme for a variety land and estuarine contamination scenarios.

Fig. 2: Gridded Imagery of parts of Bodo City showing Segmented Shoreline and the mouth of the Bodo estuary. Remediation and Restoration works are within the creeks (left side).

Case Study 2: *Environmental Assessment of Ogoniland, Geo-Environmental Mapping and Remediation of Contaminated Sites*

The United Nations Environment Programme UNEP, carried out a detailed geo-environmental mapping exercise in about 220 petroleum hydrocarbon contaminated sites in Ogoniland in 2009-2011. The UNEP mission in preparing the geo-environmental maps of the contaminated sites relied on data obtained from the IOCs operating in the area, satellite imageries of contaminated sites, historical information on land use, and a host of other sources. These data were used in the desktop design of reconnaissance surveys and actual field visits that 'ground truthed' information previously acquired from primary and secondary sources. The data acquired were then utilised in the ranking of the contaminated sites into High-Medium-Low priority sites and colour coded as Red-Amber-Green gridded maps. These geo-environmental maps, *inter alia*, highlighted oil industry facilities which were the potential sources of pollution, oils spill points, land use and environmentally sensitive areas such as wetlands, rivers, streams and estuaries.

During the assessment phase of the UNEP Project, High and Medium priority sites were visited and detailed scientific evaluations such as vegetation, air, soil, surface and groundwater investigations were carried out. The information obtained from these assessments were presented on the geo-environmental maps as baseline information for more detailed tier 2 assessments, which integrates the Risk Based Corrective Action (RBCA) model and the Site Specific Target Levels (SSTLs) evaluation in deciding the remediation techniques and the close out values of contaminants to be attained during remediation.

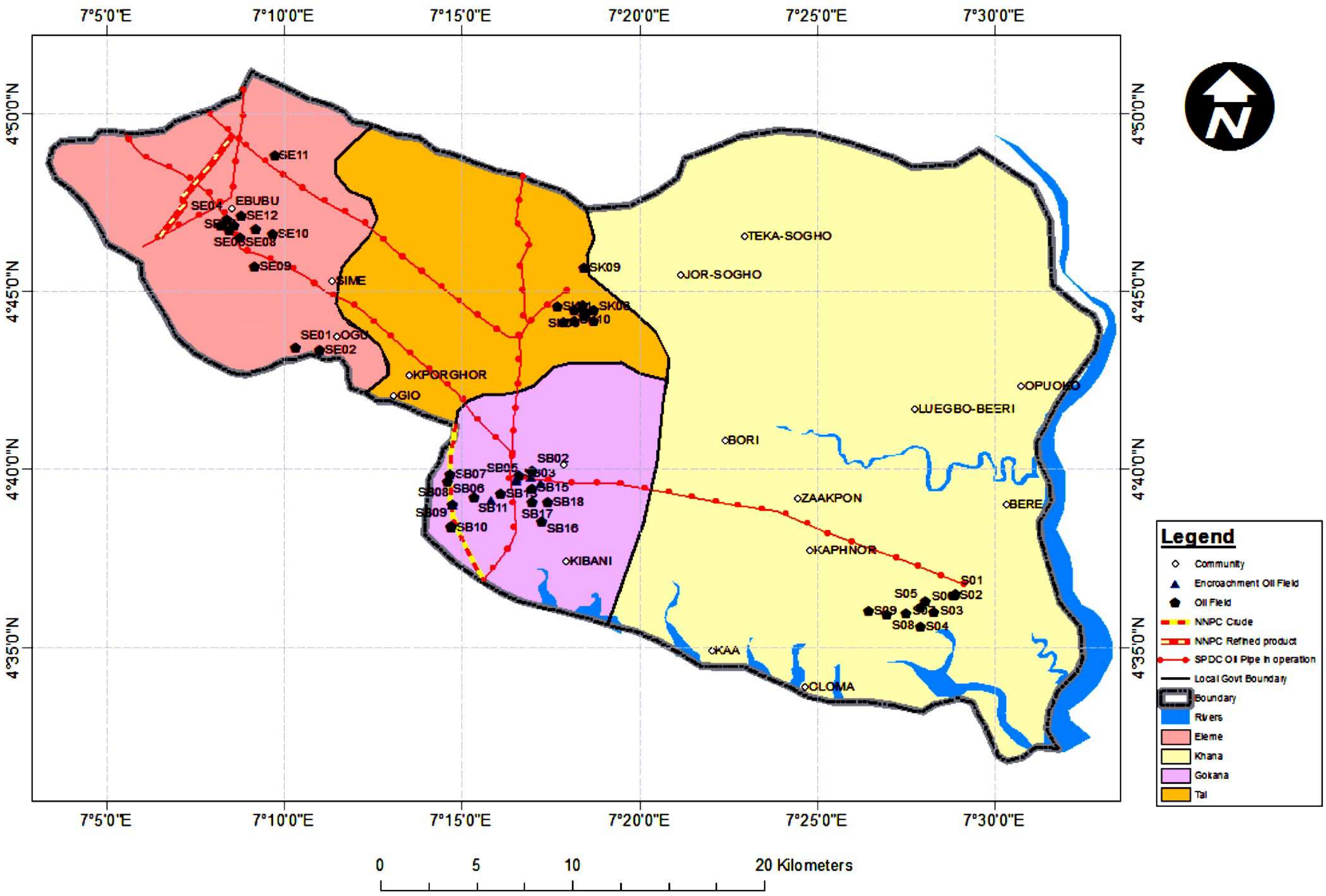

Fig. 3: A generalised map of Ogoniland showing oil industry facilities.

Conclusion

Geo-environmental mapping as a tool for effective remediation of contaminated land integrates several layers of information to highlight the nature of the contaminated land and the intervention techniques that will not cause more harm than good during the course of remediation. The protection of human health should be the foremost concern of any oil spill clean-up decision making process. Human health is dependent upon the relative health of the surrounding environment; hence it is important to understand the criteria by which clean-up methods must be targeted as to their value and efficacy. Geo-environmental mapping of contaminated lands sets out to rapidly evaluate the specific oiling conditions at the location of interest and ascertains the risks posed by the oil spill to the ecosystem and contiguous human habitation and interactions with the site. It presents the acquired data in such a manner that recommendations for clean-up, remediation and restoration of the oil spill sites could be undertaken in order to reduce the impact of oil toxicity so that a majority of the biota in that environment can survive. Therefore, when the smallest organisms survives, then the ecosystem will be able to sustain itself all the way up the food chain.

Hence, geo-environmental mapping highlights the geologic controls that determines the expression of the contaminants within the environment; thereby furnishing the remediation planner a 'bird's eye view' of the contaminated land and how best eliminate the contaminants of concern without causing irreversible damage to the already stressed ecosystem. Attempting any remedial activity without the geo-environmental map will be akin carrying out a surgical operation blindfolded. The outcomes of such a venture can only be imagined. Therefore, detailed geo-environmental mapping of contaminated sites should be considered as a prime activity to be undertaken before the conceptualisation, design and implementation of the remediation of any contaminated land.

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The Imperative of the Assessment of the Soils of the Enugu Shale Swelling and its Potential for Sustainability of Engineering Structures Using Free Swelling Ratio, Empirical and Van der Merwe's Chart Methods

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Abstract

The behavior of swelling soils is governed not only by its mineralogical composition but by many other factors amongst which are its environmental factors and stress history. The Enugu Shale is one of these shales that its swelling potential assessment of its soils cannot only be based on its mineralogical composition. The identification of its clay mineral compositions is basic in understanding the roles of other factors of swelling of its soils. This article presents the alternative use of very simple methods, which do not involve the sophisticated instrumentation of X-Ray Diffraction (XRD) but of empirical estimations, free swelling ratio and Van der Merwe's chart to ascertain the soil swelling potential of the soils. About thirty soil samples were collected for laboratory test. This research utilized the functional parameters of Atterberg's limits to define its swelling potential. The Atterberg's limit results show range of 24-66 for liquid limit, 12-39 for plastic limit, 7-36 for plasticity index and low to medium swelling by Van der Merwe's chart with empirical estimation of its clay mineral type as a mixture of kaolinitic and montimorillonite clay. Several other tests were carried out to verify the workability of the soils. Geomorphology of the studied area was incorporated to the obtained results to elucidate the importance of variation in elevation and drainage in prediction of swelling behavior of the soils of Enugu Shale. This research affirms the role of elevation and drainage characteristics in predicting soil's swelling potential.

***Keywords**: Swelling Potential, Enugu Shale, Free Swelling Ratio, Empirical Estimation and Van der Merwe's Chart*

Introduction

Swelling and poor permeability of soil are generally associated with swelling soils. The studies of expansive soils have in rece nt time attracted a great deal of attention from engineering construction practitioners. For example, southern Nigeria is mostly underlain by soft sediments which are prone to expansion in the presence of abundant precipitation in the wet season in addition to clay mineralogy of the soils and other environmental factors prevalent at a time in the history of the soils. The swelling and shrinkage phenomenon associated with the soils of this region can be detrimental to engineering projects such as pavement, foundation, slope stability etc.

Enugu Shale is one of these shales that significantly show changes in volume on addition of moisture. Severally studies have identified the characteristics of the Shale (Okagbue and Aghamelu, 2010; Aniwetalu and Akakuru, 2015; Ekeocha, 2015; Oyediran, 2015; Tijani, 2012).

The geology, climatic conditions, environmental factors and drainage conditions provide a natural setting for the occurrence of swelling/shrinkage phenomena. Structural damage caused by swelling phenomena is evident in Enugu metropolis which is underlain by Enugu Shale (fig.1).

Fig. 1: Evidence of failure of engineering structure due to swelling in Enuguss Shale

Evaluation of swelling characteristics of the soils using empirical estimation will be of much help to the geotechnical engineers for ease, quick and affordable understanding of the problematic soils of the metropolis. Sridharan and Prakash (2000) was employed to characterize the mineralogy characteristics of the soil using free swelling ratio in comparison with the existing knowledge. The swelling potential was estimated using various studies (Chen, 1988; Muntohar, 2000; Holtz et al., 1956; Puppala et al. 2014) in comparison with free swelling test results.

Free Swelling Ratio and Empirical Determination of Swell Potential

Sridharan and Prakash (2000) proposed the classification of soil clay mineral type based on free swell ratio (FSR) table 1. The free swell ratio gives realistic information about soil expansivity and clay mineralogy. The free swell ratio is calculated as follow;

FSR = Vd/Vk... (1)

Where Vd: volume of soil in distilled water and Vk: volume of soil in kerosene.
Empirical determination of soil swelling potential (SP) was calculated based on the following equations;

1) Clay content by Muntohar (2000)
   SP (%) = 7.518 + 0.323(C).........................(2)
2) Plasticity index by
   Holtz et al. (1956), **SP (%) = 60K(PI)$^{2.44}$** ....(3)
   Chen (1988), **SP (%) = 0.2558e$^{0.0838PI}$** ....(4)
   Puppala et al. (2014), **SP (%) = 0.05(PI)$^{1.415}$** ....(5)

Note that SP: Swelling Potential; PI: Plasticity Index; C: Clay Content; K: Empirical Constant (3.6×10^{-5}).

The potential for swelling was verified using Van der Merwe chart for swelling potential.

Table 1: Classification of soils based on FSR
(After; Sridharan& Prakash, 2000)

FSR	Clay Type	Soil Expansivity	Dominant Clay Mineral Type
= 1.0	Non-Swelling	Negligible	Kaolinitic
1.0 – 1.5	Mixture of Swelling & Non Swelling	Low	Mixture of Kaolinitic and Montmorillonitic
1.5 – 2.0	Swelling	Moderate	Montmorillonitic
2.0 – 4.0	Swelling	High	Montmorillonitic
>4.0	Swelling	Very High	Montmorillonitic

The Van der chart is a plot of gross clay fraction (P002) versus gross plasticity index (Pg). There is a mathematical derivation of line representing swelling potential by a factor k, which defined the swelling zones approximately.

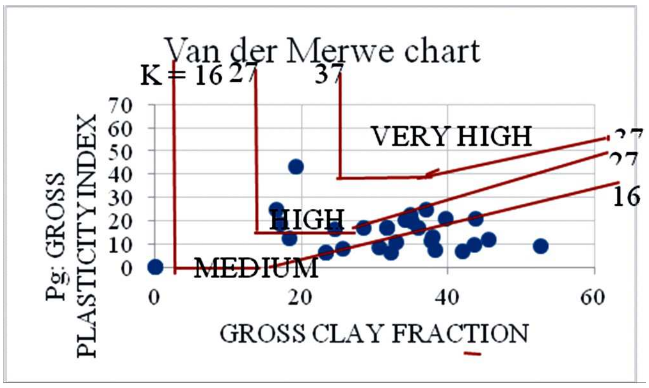

Fig. 2: Swelling potential based Van der Merwe chart in Enugu Shale

(P002 – 0.73k) (Pg – 0.16*P002*k$^{0.4}$) – k = 0…..(6)
The swelling potential is defined by k as follows;
K ≤16 Low swelling potential
16 < k ≤27 Medium swelling potential
27 < k ≤37 High swelling potential
37 < k Very high swelling potential

NOTE;
Pg(Gross plasticity index) = 18.99R – 19.47 (Abbas, 2016) ……...…(7)
P002 : Gross Clay Fraction
R (Plasticity ratio) = LL/PL ……............................(8)

Results and Discussions

The results of some analytical tests are in table 2. The results suggest that the samples are likely to show poor shear strength and subsequently low bearing capacity due to its tendencies to easily gain moisture during rainy season and quickly loss it during dry season. The consistency of a fine-grained soil denotes the physical state of the material as a cohesive soil. It shows the degree of firmness of cohesive soils. The consistency of a soil is expressed by such terms as soft, firm or hard. The consistency according to Bell (2007) was used to compare the consistency of Enugu Shale.

Atterberg's limits

The liquid limit of the tested samples range from minimum value of 22 (at the point of elevation above 190m) to the maximum value of 66 (at the point of elevation below 150m above mean sea level) with an average mean of about 42.23. The liquid limit of soils in the study area when compare with Bell classification. The results range from low plasticity to high plasticity. The low plasticity is dominant at higher elevation points while the high plasticity is dominant at lower elevation points.

Table 2: Results of free swelling test and empirical estimated swelling potential

STN	Pg	FSI (%)	CLAY FRACTION (%)	R=LL/PL	Activity	M 2000	H 1956	C 1988	P 2014
2	20.6	45	39.74	2.11	0.78	20.35	9.41	8.62	6.45
3A	19.47	33.33	35.12	2.04	0.73	18.86	6.12	7.23	5.03
3B	43	27.27	19.31	3.29	2.02	13.76	16.47	10.84	8.92
4	20.98	20	43.75	2.13	0.8	21.65	12.67	9.74	7.62
5	24.78	11	37.11	2.33	0.97	19.5	13.55	10.01	7.96
6	9.02	18	52.63	1.51	0.38	24.52	3.23	5.56	3.47
7A	10.53	25	33	1.58	0.42	18.18	1.35	3.89	2.09
7B	11.48	13.04	37.78	1.63	0.45	19.72	2.17	4.73	2.75
8	16.99	8	35.94	1.92	0.64	19.13	4.54	6.4	4.22
9	0	13.64	0	0	0	7.52	0	0	0
10A	6.17	9.09	23.33	1.35	0.3	15.05	0.25	1.95	0.78
10B	18.51	13.64	17.14	2	0.7	13.05	0.93	0	1.68
11A	6.17	13.64	23.33	1.35	0.3	15.05	0.25	2.5	0.78
11B	0	13.64	0	0	0	7.52	0	2.78	0
12	12.62	11.54	18.37	1.69	0.49	13.45	0.46	3.06	1.12
13	6.36	12	32.26	1.36	0.31	17.94	0.59	4.73	1.3
14	8.64	16.67	30.56	1.48	0.36	17.39	0.75	4.45	1.49
15	9.39	27.27	43.59	1.52	0.39	21.6	2.17	5.56	2.75
16	24.78	9.1	16.49	2.33	0.97	12.44	1.87	5.01	2.53
17	16.8	23	31.75	1.91	0.63	17.77	3.23	7.23	3.47
18	16.61	26.92	28.57	1.9	0.63	16.75	2.5	5.56	2.99
19	20.01	11.11	34.21	2.08	0.76	18.57	6.12	4.17	5.03
20	16.8	15.38	31.75	1.91	0.63	17.77	3.23	3.62	3.47
21	16.23	17.4	24.59	1.88	0.61	15.46	1.6	2.5	2.31
22	6.55	30.77	41.94	1.37	0.31	21.06	1.13	3.62	1.88
23	8.07	12.5	25.71	1.45	0.35	15.82	0.46	2.5	1.12
24	7.5	16.67	38.24	1.42	0.34	19.87	1.13	3.62	1.88
25	22.31	25	34.88	2.2	0.86	18.78	8.68	8.34	6.15
26	12.81	20.83	38	1.7	0.5	19.79	2.85	5.29	3.22
27	11.67	23.1	45.65	1.64	0.46	22.26	3.64	5.84	3.71

NB: pg(gross plasticity index), FSI(free swell index), R(plasticity ratio), M2000: Muntohar (2000); H1956: Holtz et al (1956); C1988: Chen (1988); P2014: Puppala et al (2014)

The liquid limit test shows that 33.3%, 40% and 26.7% of the tested samples have low, intermediate and fat plasticity respectively according to (Bell 2007). This shows that about 66.7% of the tested samples in the study area have potential for swelling on addition of water. The high values of plasticity index and liquid limit of some tested samples indicate the presence of expansive clay minerals. According Federal Ministry of Works and Housing (1997), a good sub-base material must have a liquid limit and plasticity index of <35% and <16% respectively. This means that more than half of the tested sample did not meet the requirement for good sub-base material. This can be clearly attested by the conditions of buildings and pavement in the study area (figure1).

Soil Classification

The particle size distribution tests of the collected samples from the studied area were dominated by grain sizes ranging from 21.67 – 93.44%, 6.0 – 72.52% and 0 – 54.97% for fine, sand and gravel respectively. With an average mean value of 69.65%, 23.68% and 6.67% for fine, sand and gravel respectively. The proportion of occurrence and percentage dominance of the tested samples based on the AASTHO classification in the study area showed that A-7-6 samples have percentage fine, sand and gravel ranging from 66.31 – 89.03%,

Table 3: Classification of soils of Enugu Shale based on Sridharan & Prakash (2000)

STATION	FS (%)	FSR	CLAY TYPE	SOIL EXPANSIVITY	DOMINANT CLAY MINERAL
2	45	1.45	Mixture of swelling & non swelling	Low	Mixture of Kaolinitic & Momtmorillonitic
3A	33.33	1.33			
3B	27.27	1.27			
4	20	1.2			
5	11	1.11			
6	18	1.18			
7A	25	1.25			
7B	13.04	1.13			
8	8	1.08			
9	13.64	1.14			
10A	9.09	1.09			
10B	13.64	1.14			
11A	13.64	1.14			
11B	13.64	1.14			
12	11.54	1.12			
13	12	1.12			
14	16.67	1.17			
15	27.27	1.27			
16	9.1	1.09			
17	23	1.23			
18	26.92	1.26			
19	11.11	1.11			
20	15.38	1.15			
21	17.4	1.17			
22	30.77	1.31			
23	12.5	1.14			
24	16.67	1.17			
25	25	1.25			
26	20.83	1.2			
27	23.1	1.23			

NOTE: FS (free swell), FSR (free swell ratio)

10.13 – 27.35% and 0.83 – 8.79% respectively except station 25 sample with 54.97% of gravel, and, 21.67% and 23.36% of fine and sand respectively. The presences of fines, sands and gravels indicate a non-uniform distribution of particle sizes in the study area while the dominance of fines over sand and gravels indicate poor grading in the study area. The dominance of A-7-6 and A-6 soils in the study area further prove that study area has poor quality materials for engineering construction and site for engineering construction.

The proportion and dominance of soils in the study area according USCS classification showed that about 83% of tested samples in the study area are made up of CL, MH and CH . These samples are very unstable in the presence of moisture. There are closely associated with low elevation points in the study area.

Swelling Potential

Evaluation of swelling potential of studied soil samples were carried out based on the results of Atterberg's limit, free swell test and empirical estimation. (Van der Merwe 1964) was applied to investigate the swelling potential of the studied soils. Figure 2 shows the k lines superimposed on the Van der Merwe swelling chart to determine the swelling potential of the studied soil samples in the studied area.

Free swelling test results were used to calculate free swelling ratio. Subsequently, the free swelling ratio results were also used to identify the clay minerals present in the study area (table3) in comparison to the classification by Sridharan and Prakash (2000) table1. The results obtained agree with work of (Oyediran 2015) and (Ekeocha 2012) on clay mineralogy of the study area.

The empirical models were calculated using the equations as proposed by (Muntohar 2000) in equation2; Holtz et al (1956) in equation3; (Chen 1988) in equation4; and Puppala et al (2014) in equation5.

Particle Size Distribution Analysis

The results of particle size distribution are shown in figure 3. Studied samples are predominately fine grain particles with reasonable percentage of sands and occasionally gravel incursion. The studied samples are poorly graded. The dominance of fine size particles especially clay size particles may likely be the cause of water logging in the eastern part of the studied area.

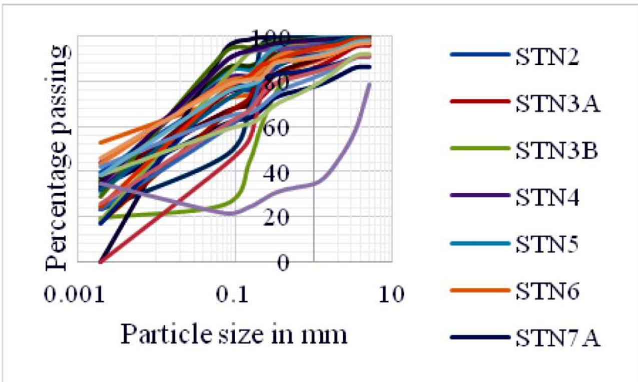

Fig. 3: Particle distribution curves of the soil samples

Conclusion

Field observations and experimental analysis identified that changes in geotechnical characteristics are consistent with changes in elevation in the studied area. Severally other deductions were made from the interpretation of laboratory test results and field observations as follows;

1. The swelling potential of Enugu Shale is essentially medium swelling but abundant precipitation and prevailing climatic conditions continuously altering the soils of Enugu Shale to high swelling soil especially at low elevation where drainage conditions are quite poor.
2. The study has shown that there is a strong correlation between swelling potential and plasticity index in the soils of Enugu Shale.
3. The dominance of A-7-6 soil based on AASTHO classification is a serious indication of poor quality of the soils of Enugu Shale especially for engineering construction projects.
4. Enugu Shale clay mineralogy is a mixture of kaolinite and montmorillonite clay minerals. This was obtained using free swelling ratio according to Sridharan and Prakash. At a minimal cost and easy interpretation unlike sophisticated result of XRD test that requires special expertise for interpretation.

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Mineralogical and Some Engineering Characteristics of Doma Clays Middle Benue Trough-North Central Nigeria

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Abstract

The Doma clay was analyzed geochemically, mineralogically as well as geotechnically in order to assess its industrial potential and impact on shallow foundations. Twenty one samples were collected and analysed in the laboratory. The results of the geochemical analysis showed that the clays are dominated by SiO_2 (39.11-47.27%), followed by Al_2O_3 (19.38-23.17%), Fe_2O_3 (7.95-13.11%) and TiO (1.30-2.63%). Oxides of other elements (K_2O, MgO, Na_2O, MnO and CaO) ranges from 0.01-0.25%. Mineralogical analysis showed predominance of kaolinite and Palygorskite minerals, with smectite identified in some of the samples. Other non-clay minerals identified are quartz, feldspar, biotite, and some iron oxides. Results of the Atterberg Limits showed wide range of Liquid Limit (32-48)%, Plastic Limit (19-23)%, and Plasticity Index (11-13)%. Grain Size Distribution shows relative proportions of clay (30-60)%, silt (25-56)%, and sand (7-14)%. Other results obtained are Natural Moisture Content (4-10)%, Specific Gravity (2.54-2.69)%, Linear Shrinkage (11-13)%, Loss on Ignition (7.01-11.11)%, and Free Swelling Index (15-25)%. The characteristics of these clays therefore indicate that the clays would be suitable for the manufacture of cement, ceramic and rubber. It may also be utilized for other industrial uses with adequate beneficiation processes. Furthermore, the results obtained indicate that the clay have little effect on the shallow foundations.

Introduction

Clays are formed by the physical disintegration and chemical decomposition of rocks, where prolonged and extensive weathering of the host rock alters the primary rock forming minerals to clay minerals which under favourable conditions leads to the development of clay deposits. (Reeves *et al.,* 2006; Kabeto *et al.,* 2012). Clays are distributed both stratigraphically as well as across different locations on the earth and the type of clay deposited in a given locality is mainly dependent on the geochemical environment where it is found (Liew *et al.,* 1985).

Significant clay deposits are available across different locations in Nigeria. The occurrences of these clay deposits vary from the lateritic profiles derived mainly from the weathering of Basement rocks to those of the sedimentary units within the different depositional basins (Emofurieta *et al.,* 1994; Obrike *et al.,* 2007; Essien *et al.,* 2010; Onyeobi *et al.,* 2013). An extensive range of techniques is available for the study of clays. The choice of a technique depends on the purpose for which the study is carried out. X-ray fluorescence and X-ray diffraction techniques have been widely used to characterize clays in order to determine its suitability in manufacturing industries.

Lafia formation is characterized by intercalation of thick clay lenses which outcrops in Doma town within the Middle Benue Trough which forms part of the larger Benue Trough of the Sedimentary Basins in the central Nigeria. However, little has been reported on its geochemical and mineralogical properties. Clays are utilized in several areas including geology, agriculture, construction, process industries, and environmental remediation and so on. The physical and chemical properties of clay minerals will determine to a larger extent, their usage in the processing industries (Murray, 1999).

Furthermore, some clays are generally expansive in nature and the presence of these expansive clays in soil is known to be a major cause of subsidence and cracks especially in structures with shallow foundations. Expansive clay swells when wet and shrinks when it eventually dries. The resultant effect is subsidence which develops to cracks, and in severe cases leads to building collapse. Therefore this phenomenon could pose dangerous hazard in regions with pronounced seasonal moisture variation (wet and dry seasons) such as the study area.

The area under study shows that clay account for a relatively high percentage and most settlements in the area are built on surfaces with high clay material, hence there is a need to assess the impact of the clays on the shallow foundations of these buildings.

Location and Climate

The study area covers parts of Doma town in Nasarawa State which forms part of the Middle Benue Trough in north-central Nigeria. It is located approximately 20km southwest of Lafia and it lies within Latitudes $8^{0}22^{I}40^{II}$ and $8^{0}25^{I}30^{II}$ and between Longitudes $8^{0}17^{I}40^{II}$ and $8^{0}23^{I}10^{II}$ (Figure 1). The study area is generally a low land area and it is characterized by a tropical climate with two distinct alternating seasons, wet and dry seasons. The wet season starts in April and ends in October while the dry season starts in November and ends in March. The mean annual rainfall is between 1000mm to 1500mm, while the mean temperature is about 25^{0}C.

Geology

The mapped area consists of two distinct rock types which are sandstone and clay. Sandstone covers approximately 70% of the total area with outcrops mostly in the northern part. It occurs in whitish to reddish colour which is typical of the Lafia sandstone. The grain size ranges from coarse and becomes progressively finer towards the stream channels. The sandstone occurs as ferruginized sandstone, unconsolidated sandstone and laminated sandstone. Clay was mapped mostly in the southern part, covering the remaining 30% of the study area. It varies in colour from grey, dirty white and reddish brown. A geological map was produced to show the lithological boundaries. Two points A-A^{1} were joined with a straight line which cut across the two rock types to show the cross sectional view of the subsurface lithologies (Figure 1).

The study area is situated on the Benue Trough in the central Nigeria, the Lafia formation is the youngest formation of Maastrichtian age, deposited under continental condition (fluviatile) in the Middle Benue Trough of north-central Nigeria. Rocks of Lafia formation have being described as unfossiliferous, non-salt bearing, brick red grit with white spots of feldspathic materials (Falconer, 1911).

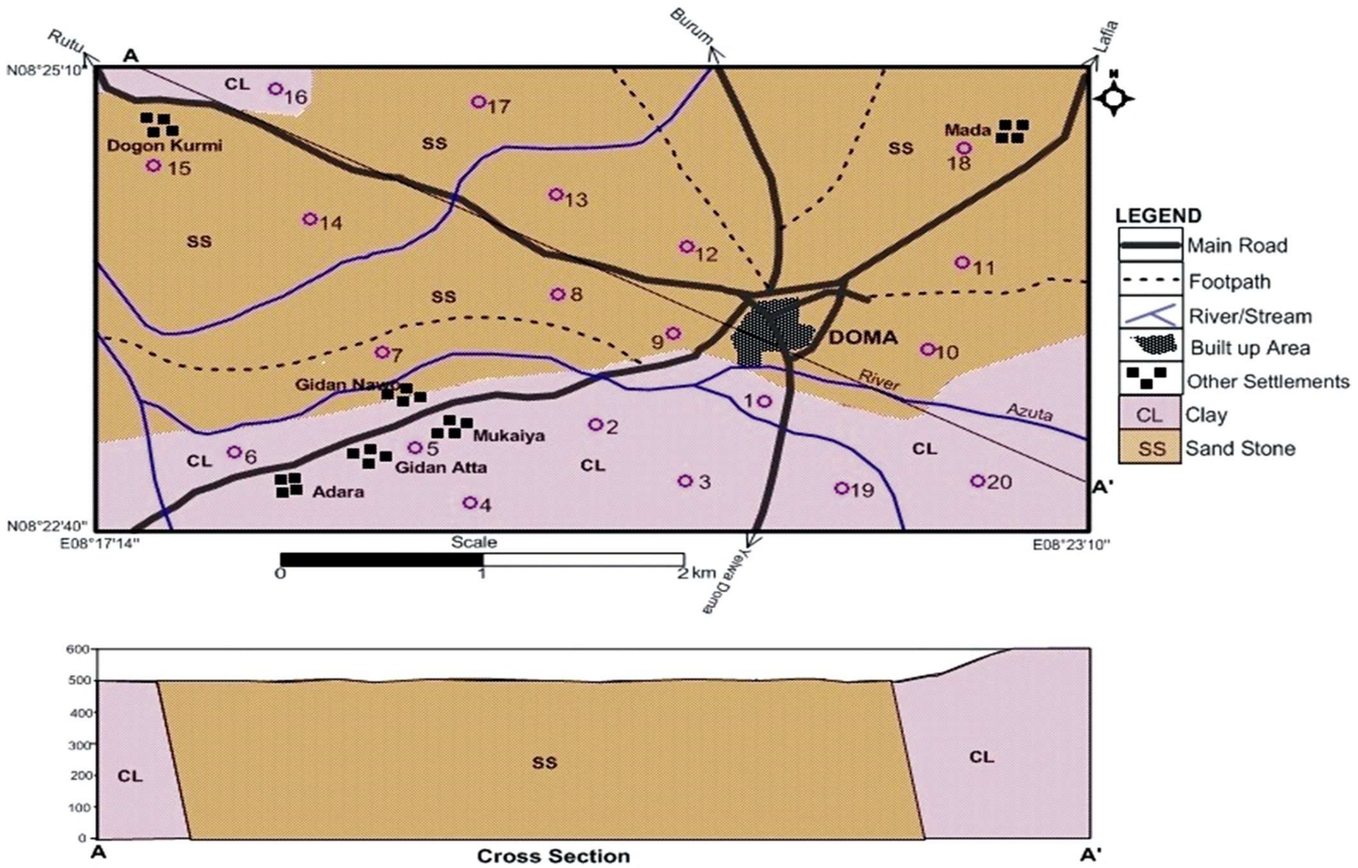


Fig. 1: Geological Map of the Study Area

Methodology

Twenty one samples were collected, labelled and prepared for laboratory analysis. X-ray fluorescence and X-ray diffraction methods were used to determined the geochemical and mineralogical characteristics of the samples. The analyses were carried out at the National Steel, Raw Materials Exploration agency Kaduna-

Nigeria. Geochemical investigation of the major elements was accomplished out using the X-ray Fluorescence Thermofischer Advantx 1200 model. Mineralogical analysis was accomplished using SCHIMADZU 6000 XRD Model with Cu Kα radiation.

Grain size distribution was obtained using the hydrometer test in accordance with ASTM D422. For Atterberg limit test, liquid limit was determined in accordance with ASTM D423 and the plastic limit in accordance with BS1377. The natural moisture content of the samples was determined in accordance with ASTM C566. Determination of specific gravity and loss on ignition were in accordance with BS1377 and ASTM D2974 respectively.

Results and Discussion

Geochemical Analysis

The result of the geochemical analysis of the samples is shown in Table1. The result shows that the clays are dominated by silica (SiO_2) which range from 39.11% - 47.27% followed by alumina (Al_2O_3) (19.38% - 23.17%), iron (Fe_2O_3) (7.95% - 13.04%) and titanium (Ti_2O) (1.30% - 2.63%). The oxide concentrations of Sodium Na_2O, Magnesium MgO, Potassium K_2O, calcium CaO, and Manganese MnO were detected in trace amounts (<1%). The dominance of silica and alumina oxides in the analysed samples shows that they are basically alumino-silicates.

Sample $D1_1$ has the highest SiO2 and Al_2O_3 values (47.27% and 23.17%) while sample $D4_1$ has the least value (39.11% and 19.38%). High SiO_2 in sample $D1_1$ also corresponds to the high value of the sand fraction (14%) in the sample as observed from the grain size distribution. Also, it was observed that sample $D1_1$ has the highest Fe_2O_3 and Ti_2O values. Computed values for alumina-iron ratio and silica-sesquioxide ratio ranges from 1.78%-2.44% and 2.39%-2.78% respectively. The aluminium-iron ratio and the silica-sesquioxide are important factors used in the manufacture of cement and other structural wares.

The alumina-iron ratio (A.R) is given by the equation:

$$A.R= \frac{\% \text{ of } Al_2O}{\% \text{ of } Fe_2O_3} \quad ....(1)$$

According to Gidigasu (1976) and Bell (1993), the equation for computation of silica/sesquioxide ratio (S.R) is given by:

$$S.R= \frac{\dfrac{\% \text{ of } SiO_2}{\text{Mol. Wt of } SiO_2}}{\dfrac{\% \text{ of } Al_2O_3}{\text{Mol. Wt of } Al_2O_3} + \dfrac{\% \text{ of } Fe_2O_3}{\text{Mol. Wt of } Fe_2O_3}} \quad (2)$$

Table 1: Result of the Geochemical Analysis of the Samples

(%) Oxides	$D1_1$	$D1_2$	$D1_3$	$D1_4$	$D2_1$	$D2_2$	$D2_3$	$D2_4$	$D3_1$	$D3_2$	$D3_3$	$D3_4$	$D4_1$	$D4_2$	$D4_3$	$D4_4$	$D5_1$	$D5_2$	$D5_3$	$D5_4$	SB	Mean	Range
SiO_2	47.27	45.23	46.21	44.25	45.13	44.16	43.18	42.14	40.14	41.67	41.12	39.16	39.11	41.15	40.13	40.09	40.32	41.76	41.30	39.34	43.11	42.14	39.11-47.27
Al_2O_3	23.17	22.23	22.19	21.25	20.14	20.71	19.46	19.40	21.33	21.31	20.35	20.33	19.38	20.33	20.36	21.31	20.04	20.66	21.02	19.68	23.02	20.62	19.38-23.17
Ti_2O	2.63	2.32	2.14	1.83	2.30	2.15	1.81	1.66	2.11	2.06	1.62	1.57	1.30	1.65	1.55	1.40	1.53	1.77	1.78	1.52	2.07	1.79	1.30-2.63
Fe_2O_3	13.04	11.77	12.29	11.02	11.63	11.07	10.88	10.32	11.41	10.95	10.66	10.20	7.95	9.23	8.70	8.98	9.12	9.81	8.87	9.06	10.07	10.20	7.95-13.04
K_2O	0.11	0.10	0.12	0.09	0.21	0.20	0.10	0.15	0.91	0.17	0.08	0.10	0.08	0.12	0.17	0.11	0.12	0.19	0.10	0.16	0.16	0.17	0.08-0.21
MgO	0.19	0.15	0.10	0.12	0.10	0.15	0.20	0.12	0.51	0.12	0.16	0.15	0.18	0.22	0.20	0.19	0.25	0.22	0.19	0.19	0.13	0.18	0.10-0.25
Na_2O	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	ND	ND	ND	ND	ND	ND	ND	ND	0.01	0.01	0-0.01
MnO	0.01	0.01	0.01	ND	0.01	0.01	ND	0.01	0.01	0.01	ND	0.01	ND	ND	0.01	ND	0.01	0.01	0.01	0.01	ND	0.01	0-0.01
CaO	0.12	0.10	0.01	0.10	0.10	0.12	0.10	0.10	0.10	0.10	0.10	0.01	0.03	0.10	0.03	0.12	0.04	0.01	0.01	0.12	0.13	0.08	0.03-0.13
A.R	1.78	1.89	1.80	1.93	1.73	1.87	1.76	1.87	1.87	1.87	1.90	1.99	2.44	2.20	2.34	2.37	2.20	2.11	2.37	2.17	2.29	2.04	1.78-2.44
S.R	2.56	2.59	2.57	2.64	2.78	2.74	2.77	2.69	2.39	2.46	2.52	2.50	2.71	2.65	2.68	2.48	2.58	2.67	2.56	2.64	2.48	2.60	2.39-2.78

A.R= Alumina-Iron ratio S.R= Silica-Sesquioxide ratio

Geochemical analysis shows dominance of silica and alumina which in addition is accompanied by low presence of calcium in the clays suggests that the clays are kaolinitic in nature (Onukwuli *et al.,* 1996). Furthermore, since the samples are dominated by the silica content, their source of silica may be utilized for the production of floor tiles (Prabhakaran *et al.,* 2016).

High amount of Fe_2O_3 in the analyzed clays which is attributed to the high iron enrichment in parts of the Lafia sandstone as reported by several workers (Offodile, 1976; Offodile & Reyment, 1977; Obaje, 2009) will usually give rise to unwanted colour on firing. This firing colour is objectionable in the paper and some ceramic industries where bright colour is desired but it may be acceptable in the manufacture of low quality wall tiles, pottery, rubber, and other structural wares (Okunlola & Egbulen, 2015). It was also observed that magnesium oxide and calcium oxides were generally low which suggests absence of associated carbornitization or dolomitization processes (Emofurieta *et al.,* 1994; Obrike *et al.,* 2007). Further, low proportion of K_2O in the analysed samples is indicative of low presence of illites which was detected in minor amount as observed in x-ray diffractograms of sample $D2_1$ and SB. The low proportions of Na_2O, CaO, MgO, K_2O and MnO which are less than 1% are generally indicative of high rate of weathering processes which gave rise to the clays.

When the alumina-iron ratio fall within the range of 1.71-2.45 and the silica/sesquioxide ratio within the range of 1.5-4.0, such clays are considered suitable for the manufacture of good quality cement (Abatan *et al.,* 1993). The computed values for the studied clays (A.R=1.7-2.44 and S.R=2.39-2.78) fall within the specified range and hence the clays may be considered suitable for the manufacture of good quality cement.

The comparison of the average geochemical composition of the studied clays with other locations and with some referenced industrial specifications is presented in Tables 2 and 3. The comparism showed that the clays have similarities with the Afam clays (Jubril & Amajor, 1991) which is also sedimentary in origin and it suits the industrial specifications for Rubber (Keller, 1964) except for the high values of Fe_2O_3 which may likely produce undesirable colouration.

Table 2: Average geochemical composition of Doma clays in comparison with some clay samples

% Oxides	Doma Clay	I	II	III	IV	V	VI
SiO_2	42.14	59.45	42.2	46.88	57.67	52.92	58.10
AlO_2	20.62	21.70	26.2	37.65	24.00	9.42	15.40
Ti_2O	1.79	1.52	-	0.09	-	1.18	-
Fe_2O_3	10.20	2.0	5.10	0.88	3.23	3.65	4.24
K_2O	0.17	1.74	8.30	1.60	0.98	0.06	3.24
MgO	0.18	0.36	0.70	0.13	0.08	0.05	2.44
Na_2O	0.01	0.34	2.90	0.21	0.03	-	1.30
MnO	0.01	0.04	0.03	-	-	-	-
CaO	0.08	0.23	1.60	0.03	1.19	-	3.11
P_2O	0.01	0.03	-	-	0.02	-	-

I= Mpu Shale (Obrike et al., 2007)
II= Afam Clay (Jubril & Amajor, 1991)*
III= China Clay (Huber, 1985)*
IV= Florida Active Kaolinite (Huber, 1985)*
V= Florida non-active Kaolinite (Huber, 1985)*
VI= Average Clay shale (Pettijohn, 1957)*
*adapted from (Onyeobi *et al*., 2013)

Table 3: Average geochemical composition of Doma Clay in comparison with some referenced Industrial specifications

% Oxides	Doma Clays	References					
		A	B	C	D	E	F
SiO_2	42.14	48.67	67.50	51.0	49.88	44.90	46.07
AlO_2	20.62	9.45	26.50	25.44	37.65	32.35	38.07
Ti_2O	1.79	-	1.0-2.0	1.0-2.0	0.09	1.80	0.50
Fe_2O_3	10.20	2.70	0.5-1.20	0.5-2.4	0.88	0.43	0.33
K_2O	0.17	2.76	1.1-3.10	-	1.60	0.28	0.43
MgO	0.18	8.50	0.1-0.19	0.2-0.7	0.13	Tr	0.01
Na_2O	0.01	2.76	0.2-1.5	0.8-3.50	0.21	0.14	0.27
CaO	0.08	15.85	0.18-0.30	0.1-0.2	0.03	Tr	0.38

A= Brick Clay (Murray H. , 1960)
B= Ceramics (Singer & Sonja, 1971)
C= Refractory Bricks (Parker, 1967)
D=Agriculture (Huber, 1985)
E= Rubber (Keller, 1964)
F= Fertilizer (NAFCON, 1985)

Mineralogical Analysis

The x-ray diffractograms of some selected representative samples in the study area are presented in Figures 2-7. Identification of clay and associated minerals was accomplished by comparing the 2θ and d-spacing values with referenced standard in literature (Chen, 1977).

X-ray diffractogram of sample $D1_1$ shows that palygorskite has the highest intensity followed by margarite, gibbsite, corundum, nacrite, manganite, tourmaline, jarosite, smectite, lizardite, silbite and chamosite. Sample $D2_1$ indicates dominant presence of kaolinite, having the highest peak followed by biotite, rhodochrosite, apophyllite, magnesite, dickite, quartz, hematite, antigorite, chamosite and illite.

Diffraction pattern of sample $D3_1$ identified kaolinite as the major mineral phase. Other minerals identified are

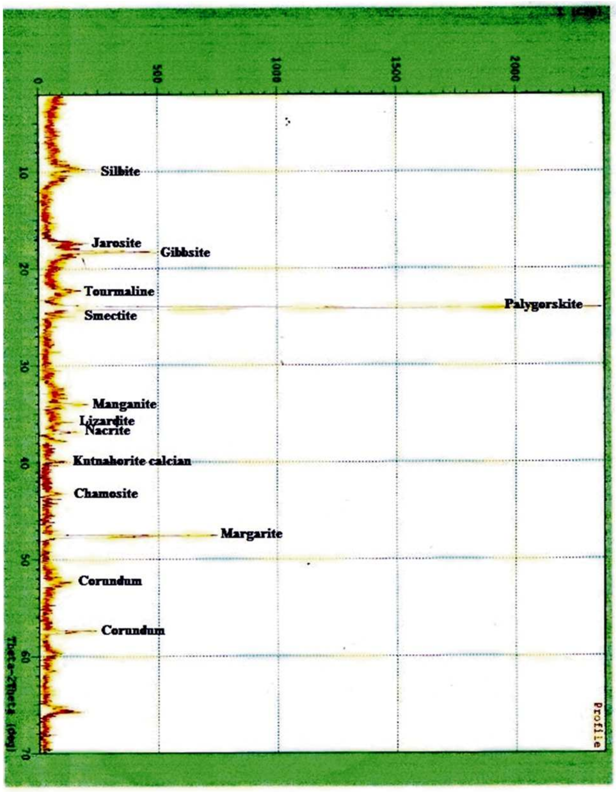


Fig. 2: X-ray diffractogram for sample $D1_1$

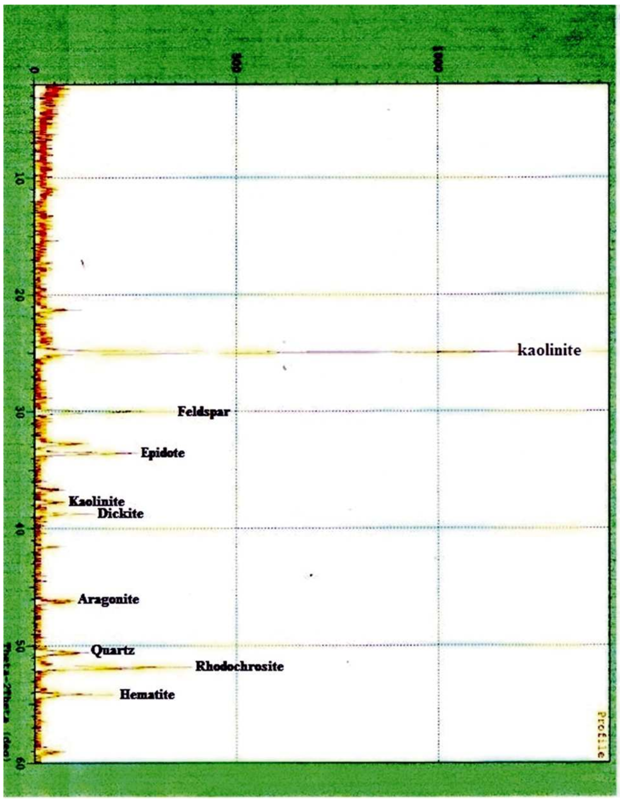


Fig. 4:X-ray diffractogram for sample $D3_1$

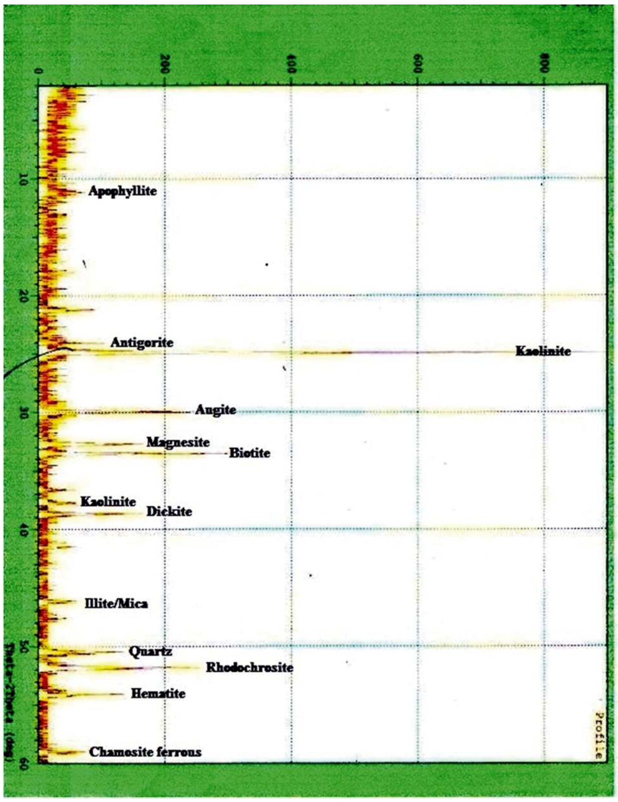


Fig. 3: X-ray diffractogram for sample $D2_1$

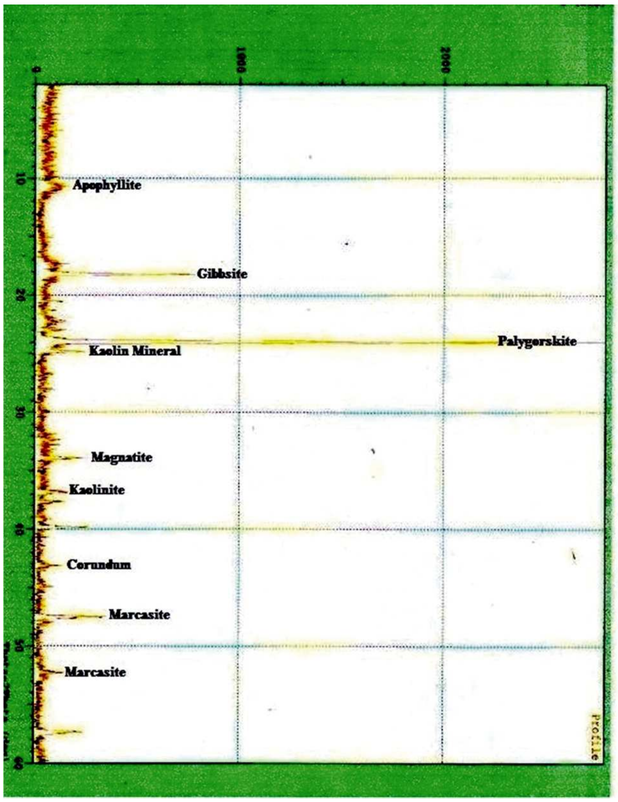


Fig. 5: X-ray diffractogram for sample $D4_1$

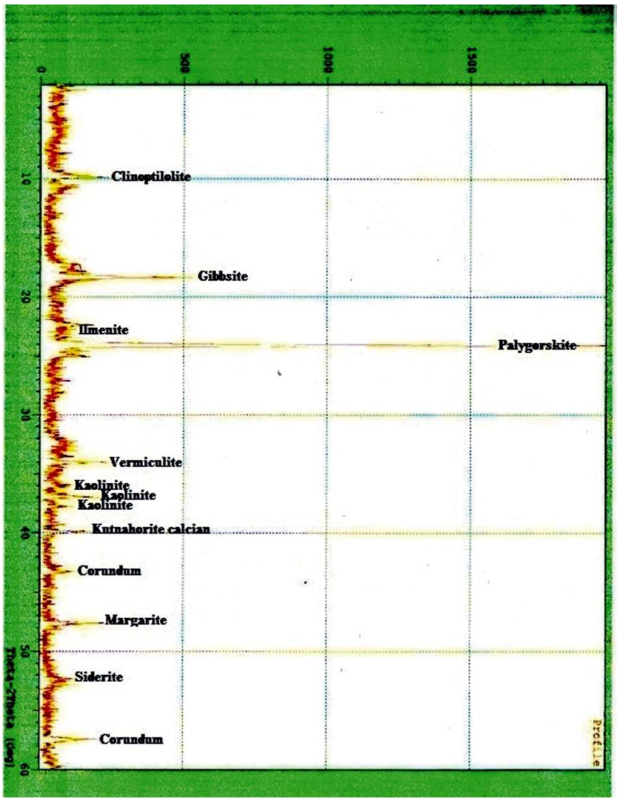


Fig. 6: X-ray diffractogram for sample $D5_1$

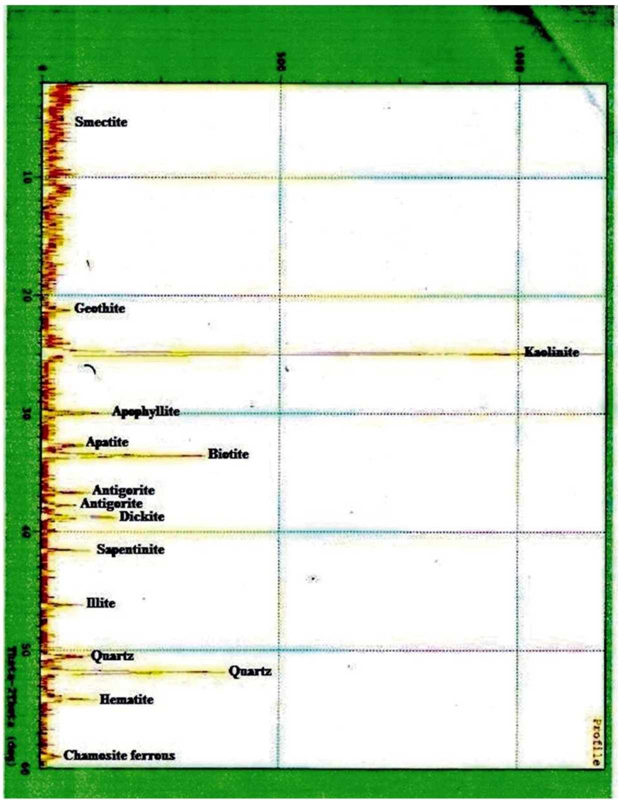


Fig. 7: X-ray diffractogram for sample DS_B

rhodochrosite, k-feldspar, epidote, hematite, quartz, dickite, aragonite and kaolinite. Sample $D4_1$ showed the highest peak to be palygorskite, while other minerals identified are gibbsite, marcasite, magnetite, kaolinite, corundum and iridymite.

Sample $D5_1$ has palygorskite as the major peak followed by gibbsite, vermiculite, margarite, clinoptilonite, kaolinite, kutnahorite calcian, corundum, siderite and ilmenite. Sample SB showed kaolinite as the major mineral phase, with quartz, biotite, apophyllite, dickite, hematite, antigorite, sapentinite, illite, smectite, goethite and chamosite also present.

Generally, the x-ray diffraction analysis revealed kaolinite and palygorskite as the dominant clay minerals in analysed samples. The kaolin minerals (kaolinite, nacrite, dickite) are present in all the samples, kaolinite has highest intensity in sample $D2_1$, $D3_1$ and SB.

Kaolinite is most often formed by the alteration of alumina-sillicate minerals in warm/moist environment (Murray, 1999). It is the most common mineral of the kaolin group. Among the clay minerals, kaolinite has the least affinity for water, and hence the studied clays will exhibit moderate shrinkage on exposure to dryness and moderate swelling in the presence of water (Ola, 1981). In contrast with smectite-rich clays, the fibrous palygorskite are non swelling and their structure is even stable in systems with high salt concentrations (Murray, 2007). Palygorskite dried at moderate temperatures and retain colloidal properties, while higher temperature drying enhances useful absorptive properties (Alvarez *et al.,* 2011). Clays consisting mainly of the palygorskite mineral usually contain impurities of quartz, feldspars, carbonates, gypsum, cristobalite, Al-smectite, illite, kaolinite, chlorite and iron oxides. Some of these minerals (quartz, feldspar, illite, kaolinite, chlorite and iron oxides) are detrital in origin while others (carbonates, gypsum, Mg-smectites) are formed paragenetically with the fibrous clay minerals (Galan, 1996).

Although smectite is present in sample $D1_1$ and sample SB, it occurs only in minor amount and may not have much impact on the swelling ability of the clays. Quartz was identified in samples $D2_1$, $D3_1$ and sample SB which reflects the higher values of Si_2O in those samples. Hematite is the most abundant iron oxide mineral in the analyzed samples. Taking into account the mineralogical composition of the analysed samples, it can be deduced that the clays were deposited in a continental environment.

Although Clay minerals as well as the associated minerals were carefully identified using the Table of Key Lines in X-ray Powder Diffraction Patterns of Minerals in Clays and Associated Rocks provided by Chen (1977), identification of these minerals can further be confirmed in the JCPDS file which was inaccessible at the time of compiling this work.

Geotechnical Analysis

The results of grain size, natural moisture content, specific gravity, loss on ignition and free swell index are presented in Table 4.

Grain Size Distribution: Result of the grain size distribution showed that the samples are dominated by the clay fraction (30%-66%), silt fraction (25%-56%), and sand (7%-14%). The grain size distribution shows relatively high proportion of the silt fraction which may render the clay unsuitable in their raw state for wide ceramic applications (Mathias *et al.,* 2015). The samples also have relatively moderate to high clay content which favours its usability in the rubber, paper and paint industries. Also, balanced proportions of clays and silts fractions have positive impacts in that it forms binders in soils which favour engineering foundations. Furthermore, soils with high clay content or a high silt and clay content will display low permeability which

Table 4: Result of the Geotechnical Analysis of the Samples

Parameters (%)		$D1_1$	$D1_2$	$D1_3$	$D1_4$	$D2_1$	$D2_2$	$D2_3$	$D2_4$	$D3_1$	$D3_2$	$D3_3$	$D3_4$	$D4_1$	$D4_2$	$D4_3$	$D4_4$	$D5_1$	$D5_2$	$D5_3$	$D5_4$	SB	Mean	Range
Grain Size Distribution	Sand	14	12	14	13	10	11	11	10	10	12	10	12	8	9	9	7	9	11	10	8	7	10.5	8-14
	Silt	55	55	55	53	54	52	54	53	55	53	54	52	56	56	54	55	25	23	25	27	45	48.3	25-56
	Clay	33	33	31	34	36	37	35	37	35	35	36	36	36	35	37	38	66	66	65	65	48	41.3	30-66
Natural Moisture Content		5	6	6	4	5	4	6	5	4	5	5	6	5	7	5	6	8	8	7	6	10	5.65	4-8
Specific Gravity		2.55	2.56	2.55	2.54	2.60	2.60	2.58	2.61	2.56	2.58	2.55	2.57	2.54	2.55	2.54	2.53	2.69	2.67	2.69	2.68	2.60	2.59	2.54-2.69
Loss on Ignition		8.07	8.05	8.05	8.09	9.90	9.80	9.50	8.90	8.12	7.80	8.65	8.03	7.01	7.45	7.90	8.15	11.11	10.50	11.00	9.80	9.92	8.79	7.01- 11.11
Free Swell Index		25	23	24	25	18	19	16	19	23	22	24	21	22	22	21	23	15	16	15	15	25	20.4	15-25

Table 5: Result of the Atterberg Limits of the Clay Samples

Samples / Parameters	$D1_1$	$D1_2$	$D1_3$	$D1_4$	$D2_1$	$D2_2$	$D2_3$	$D2_4$	$D3_1$	$D3_2$	$D3_3$	$D3_4$	$D4_1$	$D4_2$	$D4_3$	$D4_4$	$D5_1$	$D5_2$	$D5_3$	$D5_4$	SB	mean	range
Liquid Limit	32	36	35	40	44	41	44	42	43	44	42	43	41	40	42	41	44	44	42	43	48	41	32-48
Plastic Limit	19	21	20	23	21	20	22	21	20	21	19	20	19	21	19	20	23	21	20	22	23	21	19-23
Plasticity Index	13	15	15	17	23	21	22	21	23	23	23	23	22	21	23	20	21	23	22	21	25	20	13-25
Linear Shrinkage	11	12	11	11	13	12	13	12	12	13	12	13	11	11	12	11	12	13	12	11	13	12	11-13

makes it suitable for sanitary landfills (Rowe *et al.,* 1995).

Natural Moisture Content (NMC): The values obtained are consistent with the fines and it ranges from 4% to 10%. The natural moisture content of the clay when compared to the Atterberg limits indicates the consistency of the clays and favours engineering foundations (Reeves *et al.,* 2006). The Natural Moisture Content is important in understanding the engineering properties of soil used for construction and other industrial uses. The values obtained for the study area shows the clays to be accommodating for shallow foundations.

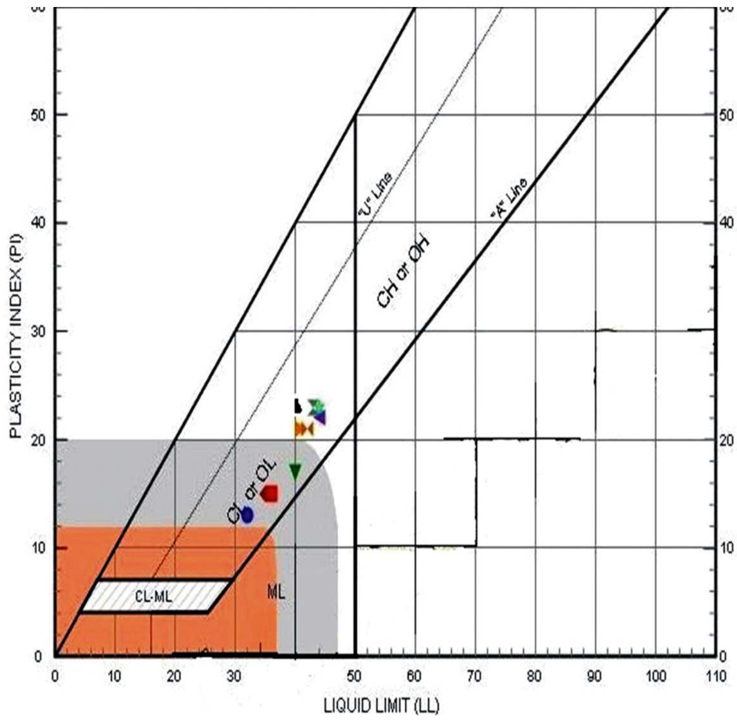
Specific Gravity (SG): The values of SG for the studied sample range between 2.54% to 2.69%. The specific gravity of soil is an important property used in the identification and assessment of aggregate parameters for construction purposes (Gidigasu, 1976). The higher the specific gravity of soil, the better it is for construction purposes. The high values obtained in the analysed clays, therefore indicates its suitability for construction.

Loss on Ignition (LOI): The measured values for LOI in the analysed samples range from 7.01% to 11.11%. These values signify low volatile contents (i.e. carbonates/hydrous minerals). High volatile contents particularly in the form of organic matter may have adverse effect on industrial usage of clays, because it causes high shrinkage which results to increase in porosity due to internal cracking. This will lead to a reduction of crushing strength of the finished products (Okunlola & Egbulen, 2015).

Free Swelling Index (FSI): The Free Swelling Index values range from 15% to 25%. The FSI is an important index property of expansive soils which indicates the innate potential the soil has for expansion (Mathias *et al.,* 2015). The values obtained shows that samples have little swelling potentials and it shows consistency with the plasticity index. According to Ademila and Adebanjo (2017), soils with free swell index values of between 23-73% have the capacity to swell and will probably cause problems for foundations and road constructions. All the FSI values of the studied samples fall below this range.

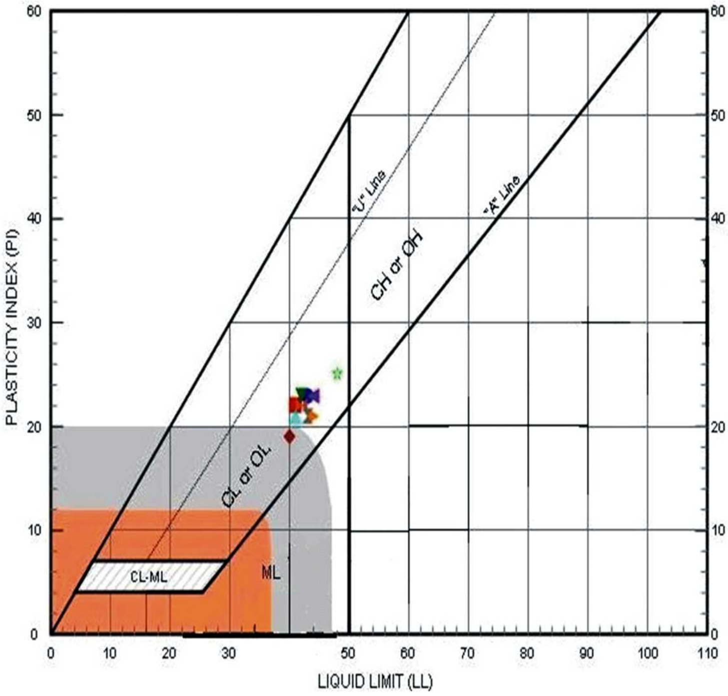
Atterberg Limits: The result of the Atterberg Limits is presented in Table 5. The values indicates that the studied clay have wide range of liquid limit (32%-48%), plastic limit (19%-23%) and plasticity index (13%-25%). Atterberg Limits provide general indices of moisture content beyond which the soil changes from one consistency test to another (IDT, 2015). Liquid limit is the moisture content at which the soil passes from the liquid state, plastic limit is the moisture content at which the soil becomes too dry to be in plastic condition while the plasticity index is the numerical difference between the liquid limit and the plastic limit (BS1377, 1990). The plasticity of a soil is dependent on the grain size distribution, amount of expandable clay minerals such as smectite and presence of organic matter (El Ouahabi *et al.,*2014). The low to medium values of the liquid limit (32%-48%) and plasticity index (13%-25%) generally indicate low or absence of expandable clays. Also, low value of the plasticity index implies that the clays have low swelling potential (Okeke *et al.,* 2016). According to Grimshaw (1971), clay materials with plastic limits of 10%-60% are considered suitable for ceramic production. The plasticity of a material is dependent on the grain size distribution, amount of expandable material and presence of organic matter. Shrinkage test are used to indicate whether drying of a clay stratum is likely to cause a settlement problem (Reeves *et al.,* 2006). The values obtained (11%-13%) indicate that the samples have low shrinkage potentials.

The samples plot on the region of low to medium plasticity on the plasticity chart which may be attributed to the presence of high amount of kaolinite (Figure 8 and Figure 9). It may also indicate low or moderate potential for swelling. The plasticity chat for clays by Reeves (2006), which was used for the interpretation is presented as figure 10.



	No.	Bor. No	LL	PI
●	1	D11	32	13
■	2	D12	36	15
◆	3	D13	35	15
▼	4	D14	40	17
▲	5	D21	44	23
▶	6	D22	41	21
◀	7	D23	44	22
⋈	8	D24	42	21
⧗	9	D31	43	23
☆	10	D32	44	23
●	11	D33	42	23

Fig. 8: Plasticity plot of samples $D1_1$-$D3_3$



	No.	Bor. No.	LL	PI
	1	D34	43	23
	2	D41	41	22
	3	D42	40	19
	4	D43	42	23
	5	D44	41	21
	6	D51	44	21
	7	D52	44	23
	8	D53	42	22
	9	D53	43	21
	10	SB	48	25

Fig. 9: Plasticity plot of samples $D3_4$-SB

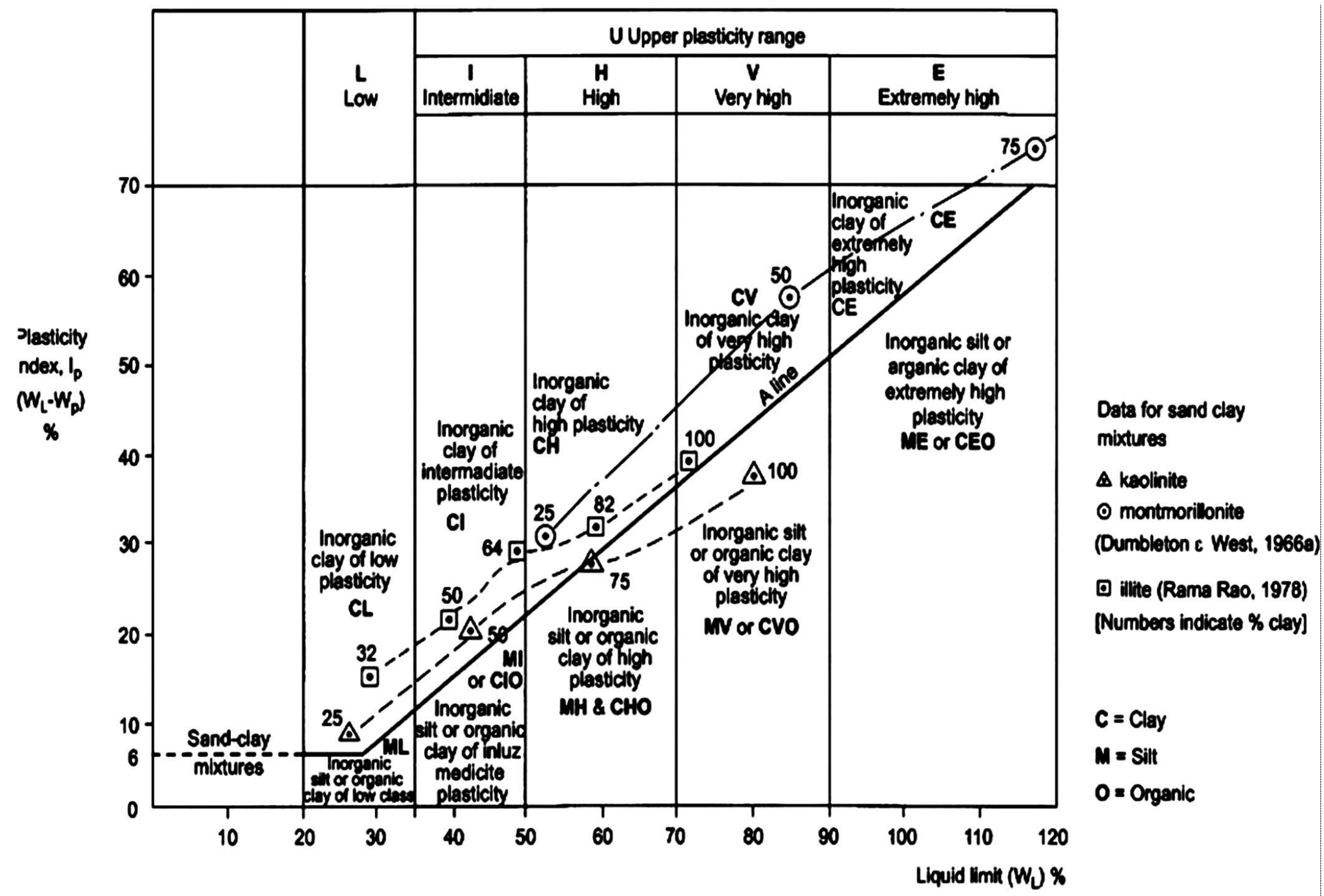


Fig. 10: Plasticity chart for clays (Reeves *et al.*, 2006)

Conclusion

Assessment of the geochemical, mineralogical and geotechnical characteristics of the Doma clays indicates that they are suitable for the manufacture of cement, ceramics and rubber but would require adequate beneficiation and other treatment processes for usability in these processing industries. Furthermore, the clays have very little effect on shallow foundations due to its low expansion ability. However, since swell/shrink phenomena tends to occur at or near the ground surface, shallow foundations in the area should extend to few meters depth or at least below the incompetent clay layers.

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Assessments of Groundwater Occurrence in Coastal Aquifers of Apapa-Ajegunle and Its Environs, South-Western Nigeria

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Abstract

Seven borehole logs, one hundred and eleven water samples, twenty-six Vertical Electrical Soundings (VES) and fourteen Electrical Resistivity Tomography (ERT) were used to study the groundwater occurrence and delineate seawater intrusion in the coastal aquifers of Apapa and its environs. Results from water analyses, VES and ERT were processed and interpreted with the ArcMap, WinResist, and DIPRΩWIN softwares respectively. The pH, EC and TDS values 5.5-8.5, 48-3481 µS/cm, and 24-1726 mg/L respectively and TDS >1000mg/l in less than 5% of the samples. The lithological logs showed topmost layer, clay, peat and sand, with thicknesses(m) 0.2-2, 1.8-10, 2-10, and 2-27 respectively. Five geo-electric units were delineated: the topsoil, clay, unconsolidated sand/freshwater, clay/brackishwater sand and salinewater sand with layers' resistivities(Ωm) 9.3-530, 1.2-46.7, 4.7-631.3, 6.2-1096.1 and 7.2-320, and thicknesses(m) 0.7-2.6, 1.8-12.2, 6-38.1, 23.3-152.1 and undefined thickness respectively. Generated cross-sections and maps revealed saltwater presence. The respective varying thickness of freshwater sand, and the depth(m) to salinewater are 3.7-20 and 10-30. The study reveals a significant part of the shallow aquifers have been affected by saltwater intrusion. The present study successfully characterizes the coastal aquifers and assesses the current situation of seawater intrusion within the study area.

***Keywords**: Borehole logs, Resistivity, Groundwater, Saltwater intrusion, Freshwater.*

Introduction

Groundwater intrusion by saline water has undoubtedly gained increased recognition in many coastal parts of the world today. In many low-lying coastal areas, saline groundwater is present within the subsurface, and as such, freshwater presence in shallow coastal aquifer, are in lensoid forms which floats on this saline groundwater. Coastal areas are characterized by high annual rainfall (>2,000 mm), yet experience freshwater scarcity and attributable to their topographical nature (Kura *et al*., 2013).; hence, runoff is lost to the sea and making surface water availability virtually impossible. Thus, these areas strongly rely on groundwater for freshwater supply (Aris *et al.,* 2007).

Coastal aquifers, particularly in small tropical islands, are complex and fragile in nature, which can easily be distorted by poor management, leading to alteration in equilibrium state between freshwater and saltwater, thereby affecting socio-economic development and damaging the entire ecosystem (Kura *et al*., 2013). This is because coastal aquifers are usually in direct contact with the sea, creating a state of equilibrium between the freshwater and saltwater. Distortion of the equilibrium usually occurs through either an increase or a decrease in the flow of freshwater, leading to alteration of the freshwater/saltwater equilibrium interface (Rey *et al*., 2013). The natural movement of freshwater towards the sea prevents saltwater from entering freshwater coastal aquifers (Barlow, 2003). However, groundwater pumping/development can decrease the amount of freshwater flowing towards the coastal discharge areas, allowing saltwater to be drawn into the freshwater zones of coastal aquifers. Therefore, the amount of freshwater stored in the aquifers is decreased (Barlow, 2003).

The most important duty with the management of the coastal/island aquifers are their detection, quantification, and monitoring of saline and freshwater interaction (Burnett *et al.,* 2002). Hence, setting a proper management plan for groundwater reserves is impossible without an understanding of the spatial distribution of fresh and saline groundwater and the processes that influenced their advancement (Glynn and Plummer, 2005).

Protection of coastal aquifers requires not only a clear understanding of their dynamics, but also detailed information, on the changeability of their parameters (Carrera *et al.,* 2010). Collecting such data requires a holistic approach, to achieve a reliable result, which can provide managers with all the information needed to develop a proper management plan. For this reason, researchers around the world have used different methods to assess seawater intrusion. For example, some researchers (Rey *et al*., 2013; Choudhury *et al*., 2001; Koukadaki *et al*., 2007) use geophysical techniques to delineate seawater intrusion while others (Aris *et al*., 2007; Banerjee *et al*., 2012; CAMP, 2000;

Gimenez-Forcada *et al*., 2010) use hydrochemical methods when assessing the seawater intrusion.

Ωccurrence of groundwater within coastal aquifers of Lagos metropolis has been studied by Adepelumi *et al.,* 2008; Adeoti *et al.,* 2010; Ωyedele, 2008; Ayolabi *et al,* 2013 and Ωloruntola *et al.,* 2019. However, most dwelt on either geophysical methods, geochemical methods or a combination of the both in understanding the source of contaminant, the groundwater potential and current state of freshwater salinewater interface. Studies on seawater intrusion, requires a multidisciplinary approach as single approach has been generally unsuccessful in providing reliable results (Werner *et al*., 2013).

Like most parts of coastal areas of Lagos, Apapa is also densely inhabited. Reports on saltwater intrusion into the coastal aquifers have been emanating in some densely populated coastal areas of Lagos, such as Lekki, Ωniru, University of Lagos areas and its environs. Excessive groundwater extraction and possibly reduction of groundwater gradients were responsible for displacement of fresh water in the aquifer thus saline water intrusion perceived (Ayolabi *et.al.,* 2013).

As there is yet to be any published work that studies the nature of the groundwater within the aquifers of this densely populated areas, this work integrates geophysical method with geological/geotechnical data from logs and physical parameters of water to assess and establish the current situation of seawater intrusion in Apapa and its environs.

The Location and Geology of the Study Area

Geographically, Apapa and its environs lie within latitudes $6^{0}25^{'}58.1^{''}$N - $6^{0}27^{'}31.634^{''}$N and longitudes $3^{0}20^{'}54.546^{''}$ E-$3^{0}23^{'}0^{''}$ E covering a total surface area of about 6.513 km^2 situated within Lagos state, south-west, Nigeria. Location of study as shown in Fig. 1 is bounded in the east by Lagos harbor/ lagoon, in the south by the Badagry creek, the north by Apapa Kirikiri and the west by canal. The present landscape of Apapa was formed during the Holocene (Bird, 2008); a Headland formed along the coasts, in the context of continuous relative sea level rise which is discordant in geology with alternating resistant (harder) and less resistant (softer) rock (Whittow, 1984). Apapa and its environs are a coastal domain which fall within the eastern Dahomey Basin and the Coastal Alluvium Sands Formation (Ωbaje 2009). The surface lithology in the area consists essentially of sands with the occurrence of clay intercalation in places (Jones and Hockey, 1964) . The sands are porous, and the occurrence of clay intercalations creates a multi-layered aquifer system.

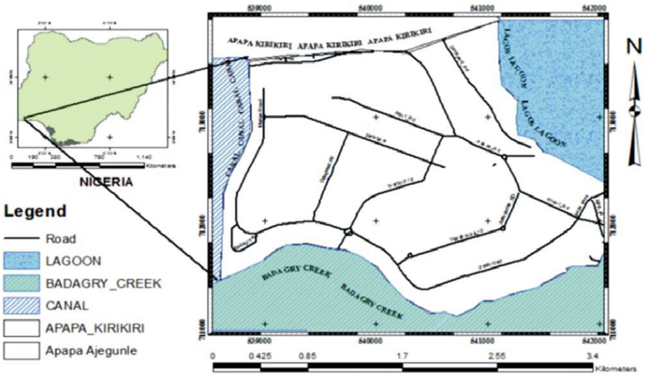

Fig. 1: Location map of study area.

Data and Methods

Physical Parameter

Measurements of groundwater table, and physical parameters such as pH, Total Dissolved Solids, temperature, and Electrical Conductivity/EC at one hundred and eleven (111) water sample locations as shown in Fig. 2 were performed insitu using portable EC and pH Hatch meters. Ωbservations and recordings of their colour, taste and odour were also taken.

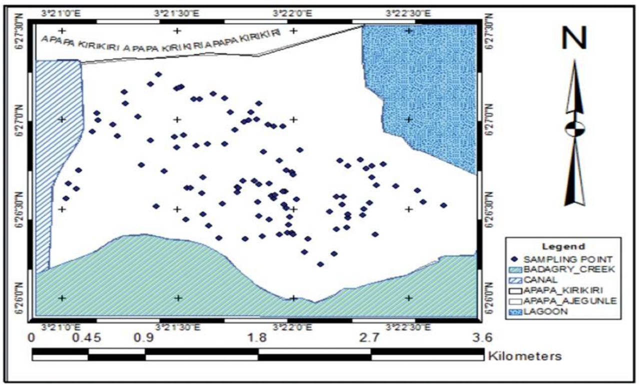

Fig. 2: Map of sampling points across the study area.

Borehole Log

Seven borehole logs within the area were acquired from Foundation Construction Services, a geotechnical borehole drilling company.

The lithological information for the three logs available vary in depth of between 15 and 30 m.The borehole logs were used as a further guild in constraining and interpreting of the VES and ERT results. Fig. 3 shows the locations for borehole logs, VES and ERT. A few logs were correlated as shown in Fig. 4 to have an overview of the lithology of the study area.

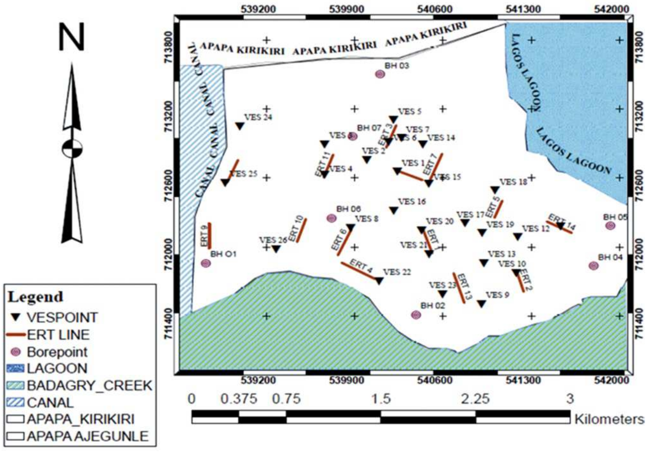

Fig. 3: Map of borehole points, VES points and ERT-traverses acquired on study area.

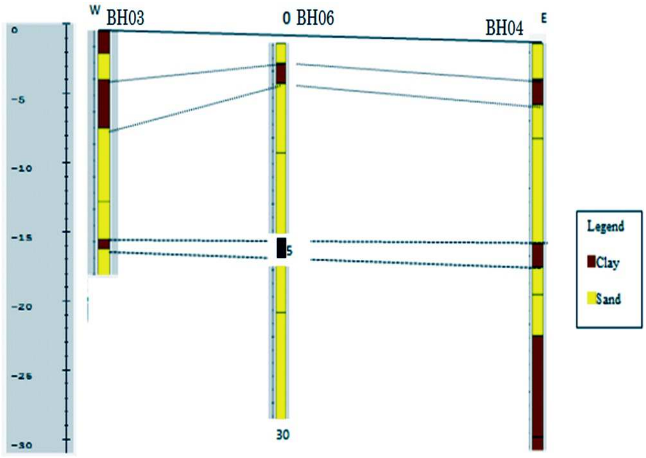

Fig. 4: Correlation of Boreholes 3, 6 and 4 across the study area in the West - East direction showing the lateral continuity of the lithology.

Vertical Electrical Sounding

A total of twenty-six VES stations were established across the study area as shown to Fig. 2 using ABEM SAS1000 Resistivity meter with maximum electrode spread (AB) of 600 m. The data acquired were processed to remove random noise such as noise due to instrumentation and the environment.

2D Resistivity Imaging

The 2D–electrical resistivity imaging was conducted in fourteen traverses measured perpendicularly to the coastline as shown in Fig. 2 using the Wenner configuration. The length of the 2D electrode traverse was 220 m along NW-SE and E-W direction. The data acquired were processed using Dipro for windows program. The program basically determines a resistivity model that approximates the measured data within the limits of data errors and which is in agreement with all prior information. The difference between the measured and calculated apparent resistivity values is given by the root-mean-square (RMS) error. The data was processed to the lowest allowable root-meansquare (RMS) error.

Results and Discussions

The results show various geoelectric layers and inferred lithology were presented as geoelectric sections and depth to the top of the aquifer zone maps. The curve types which reflect the lithological variations in the study area include HKH (23.08%), HKQ (19.23%), HK (23.08%), HA (15.38%), QH (11.54%), QQ (3.85%) and KH (3.85%).

The general signature of the curves (Fig 5) suggests alternate sequence of resistive-conductive layers, reflecting the lithology of the coastal alluvium sequence, characterized by intercalation of sands and clay horizons as described by Ωmosuyi *et al*., (1999).

Geoelectric Section

The geoelectric section along Traverse 1 (Profile B-B') trending south-east to north-west revealed four to five geoelectric layers (Fig.6). The topsoil above varies in lithology from clay through clayey sand to sand along the traverse with resistivity values of between 62.5 and 210.4 Ωm. The second horizon is essentially clayey having resistivity values of 4.5 to 36.3 Ωm. with thickness values in the range between 2.4 to 7.4 m. The third horizon consists of sandy layer having thickness between 19.6 to 81.1 m. The resistivity values of this layer ranges between 24.3 and 206 Ωm reflecting variation in the content of the sand. As sand units with low resistivity signatures depicts saline water content or intrusion.

The fourth horizon is sandy but with increasing salinity where resistivity is lower than 100 Ωm. The last geoelectrical layer is composed also of sand that is essentially saline as evidenced by low resistivity values ranging from 9.9 to 144.1 Ωm. Although poorly sorted, it consists of fine to medium grains sand, but the sandy layer is saturated as confirmed from the well (Bh1) around the traverse.

The geoelectric section along Traverse 2 i.e. Profile D-D' trending south -west to north- east revealed four to five geoelectric layers (Fig. 7).

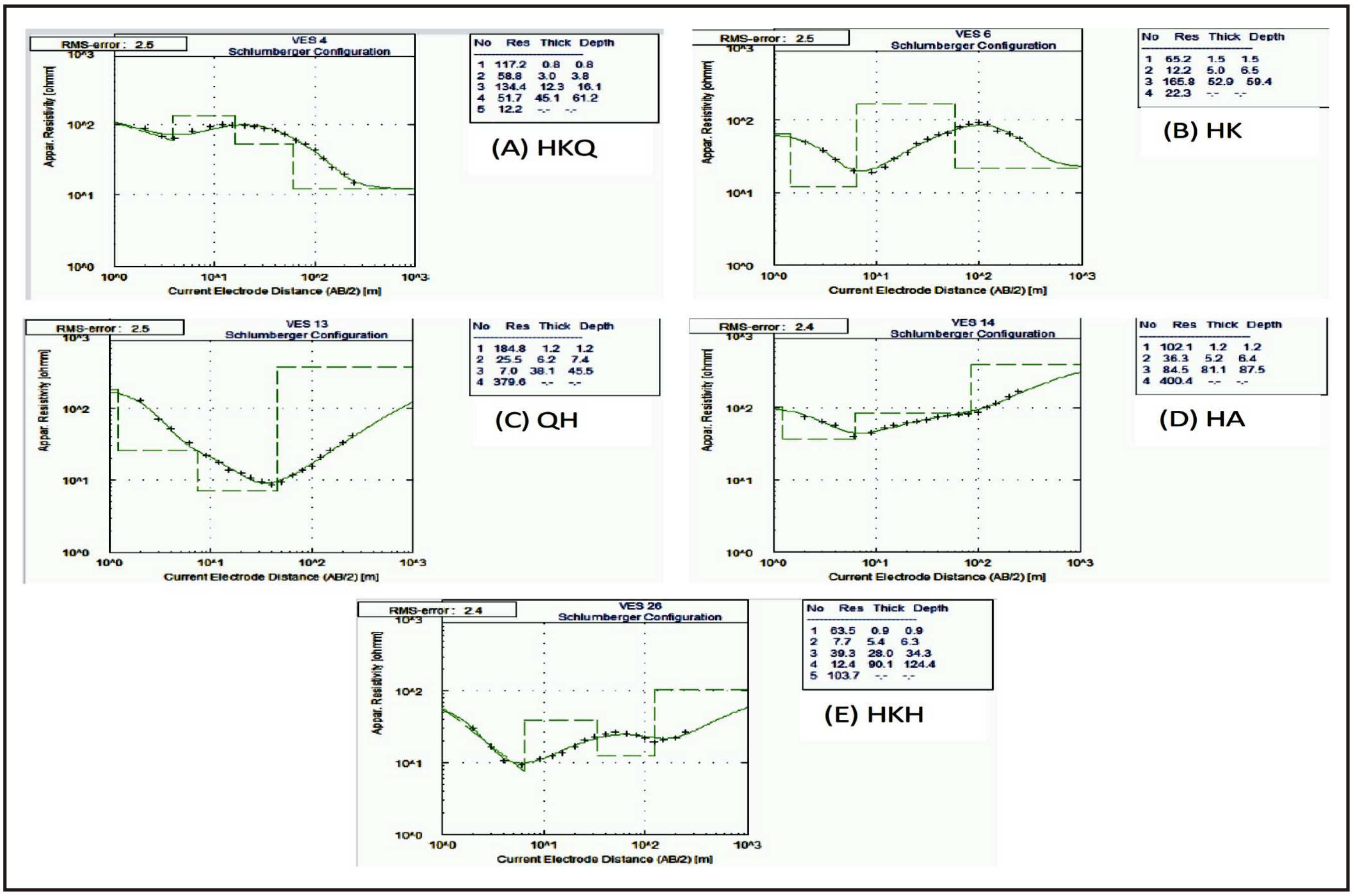

Fig. 5: The dominants VES curve types in the study area.

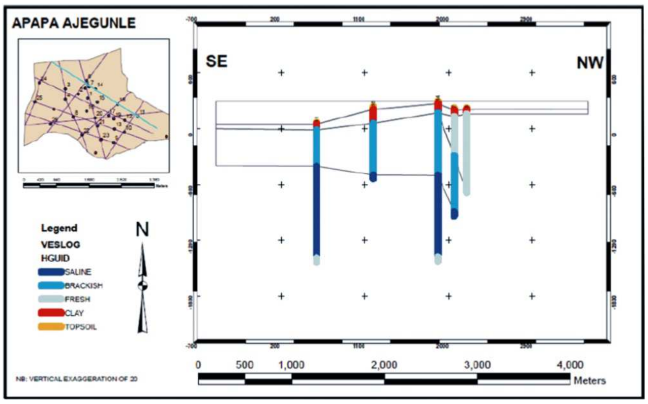

Fig. 6: Hydrogeologic sections showing the Correlation of VES points and their inferred log along profile B-B' Apapa.

The topsoil above varies in lithology from clay through clayey sands to sands along the traverse having resistivity values of between 63.5 and 210.4 Ωm. The second horizon is essentially clayey having resistivity values of 7.7 to 58.8 Ωm. This layer constitutes the confining zone the aquifer with thickness of 2.7 to 14.3 m. The third horizon consists of sandy layer having thickness of up to between 12.3 to 52.9 m. The resistivity value of this layer ranges between 39.3 and 206 Ωm reflecting variation of the sand and degree of water salinity across where lower resistivity represent saline water the area. The fourth horizon is sandy but with increasing salinity where resistivity is between 12.4 to 226.7 Ωm.

The last geoelectrical layer is composed also of sand that constitutes mostly of the salinity layer at some of the locations. The resistivity varies from 12.2 to 152.7 Ωm; although poorly sorted, it consists of fine to medium grains sand, but the sandy layer is saturated as confirmed from the well (Bh1) around the traverse.

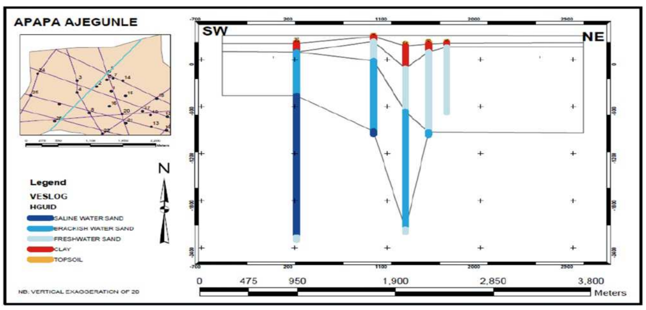

Fig. 7: Hydrogeologic sections showing the Correlation of VES points and their inferred log along profile D-D' Apapa.

The geoelectric section along Traverse 3 i.e. Profile H-H' trending west to south-east revealed four to five geoelectric layers (Fig.8).

The topsoil above varies in lithology from clay through clayey sands to sands along the traverse having resistivity values of between 79.9 and 461.3 Ωm. The second horizon is essentially clayey having resistivity values of 5.7 to 46.7 Ωm. This layer constitutes the confining zone the aquifer with thickness of 3.7 to 19.6 m. The third horizon consists of sandy layer having thickness between 15.6 to 58.9 m. The resistivity value of this layer ranges between 7 and 631.3 Ωm reflecting variation of the sand and degree of water salinity across where lower resistivity represent saline water the area. The fourth horizon is sandy but with increasing salinity where resistivity is between 6.2 to 9.9 Ωm.

The last geoelectrical layer is composed also of sand that constitutes mostly of the saline layer at some of the locations. The resistivity varies from 5.1 to 379.6 Ωm; although poorly sorted, it consists of fine to medium grains sand, but the sandy layer is saturated as confirmed from the well (Bh2) around the traverse.

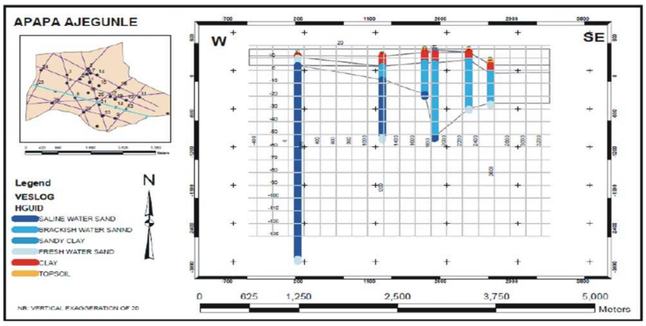

Fig. 8: Hydrogeologic sections showing the Correlation of VES points and their inferred log along profile H-H' Apapa.

The geoelectric section along Traverse 4 i.e. Profile I-I' trending north-west to south-east revealed four to five geoelectric layers (Fig.9). The topsoil above varies in lithology from clay through clayey sands to sands along the traverse having resistivity values of between 48.7 and 545.7 Ωm. The second horizon is essentially clayey having resistivity values of 4.9 to 133.6 Ωm. This layer constitutes the confining zone of the aquifer with thickness of 3.7 to 19.6 m. The third horizon consists of sandy layer having thickness between 15.6 to 58.9 m. The resistivity value of this layer ranges between 5.7 and 299.6 Ωm reflecting variation of the sand and degree of water salinity across where lower resistivity represent saline water in the area. The fourth horizon is sandy but with increasing salinity where resistivity is between 1.8 to1096.1 Ωm.

The last geoelectrical layer is composed also of sand that constitutes mostly of the salinity layer at some of the locations. The resistivity varies from 18.3 to 152.7 Ωm; although poorly sorted, it consists of fine to medium grains sand, but the sandy layer is saturated as confirmed from the well (Bh2) around the traverse.

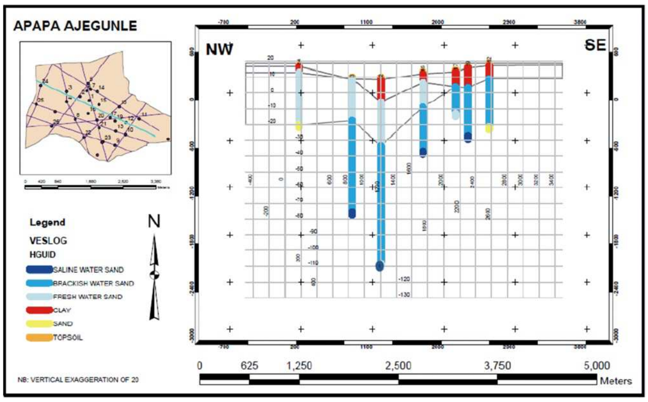

Fig. 9: I-I Hydrogeologic sections showing the Correlation of VES points and their inferred log along profile I-I' Apapa.

Electrical Resistivity Structures

The 2D electrical resistivity structures present the results of the 2D ERT surveys, in a colour scale of gradational blue to red, representing low to high resistivity values respectively. This indicates the variation in the subsurface lithology and fluids contained in them. The tomographic structures show the lateral and vertical variation in resistivity to a depth of 50 m in the subsurface.

The two-dimensional (2D) inverted resistivity wenner sections on profile 1 and profile 2 show resistivity depth models of 50 m, respectively (Figs. 10 and 11).

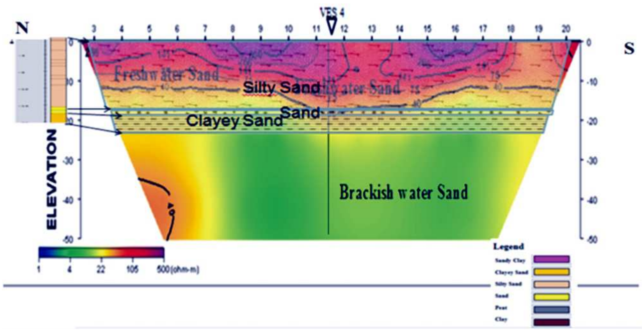

Fig. 10: correlation of selected 2D resistivity model of profile 1, VES 4 and borehole 07.

Ωn profile 1, resistivity model reflects the heterogeneous nature of the subsurface horizons. A good correlation exists between the borehole log of well BH07 located around the profile and the 2D inverted model. The top layer is sandy sediment; the resistivity varies from ~260 to 600 Ωm at a depth of ~0 to >10 m from west to east end of the profile (Fig. 10). This layer is underlain by relatively low resistive sand. Beneath this layer is a continuous layer of porous and permeable layer with lower resistivity range of between 40 and 75 Ωm, reflecting the relatively saline nature of the aquifer with depth. The reduction in resistivity towards the eastern direction at depths below 20 m shows the rapid change common in this environment as observed by Ako *et al.,* (2005) in adjoining area to the study area. The slight change in lithological properties common in this type of environment as reported by Ako *et al.,* (2005) is largely responsible for the observed saline intrusion of the boreholes which though drilled into the same layers of the aquiferous horizon.

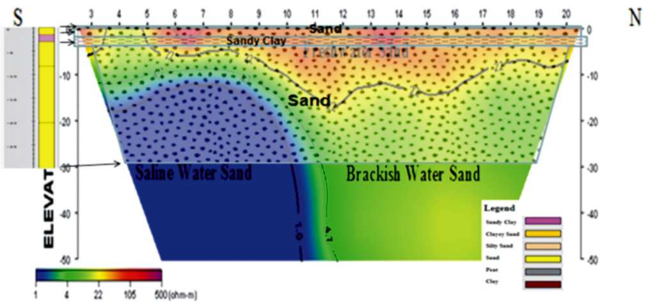

Fig. 11: Correlation of selected 2D resistivity model of profile 1 and borehole 06.

Profile 2 (Fig. 11) trends in the north-south direction. It is in correlation and lateral continuity with profile1. It reveals a topmost layer of sand having resistivity value of about 105 Ωm. This is underlain by a thin layer of sandy clay (100-300 Ωm). This semi-confining layer is underlain by 10 m thick brackish water sand (22 Ωm) northward and 34 m thick very saline water sand (≤4.7 Ωm) towards the south.

Physical Parameters of Groundwater in the Study Area

The summary of the physical parameters is presented in Table 1. Some of the parameters are presented as post map (Fig.12) to show their distribution relative to the coast. The pH has a range 5.5–8.4. The electrical conductivity (EC) of the samples 48-3481 (μS/cm). The total dissolved solids (TDS) 24-1726 mg/l, having a mean of 360 mg/l (Fig. 12). The high values of TDS and EC within the study area point to possible saltwater contamination in groundwater. In addition, classification of TDS by Heath, (1983) confirms saline intrusion in the coastal aquifer.

Table 1: Summary of Physical Parameters measured from water sampling

Parameter	Min	Max	Mean
TDS (mg/l)	24	1726	360
EC (μS/cm)	48	3433	721
pH	5.5	8.4	6.8
Temperature (^{0}c)	25	35	31
Static water level (m)	0.8	4.1	2.8

Also, all the shallow wells (static water level 0.8–4.1 m) in the area just as other coastal areas of Lagos where groundwater saline water intrusion has displaced freshwater in coastal aquifer with high proximity to the ocean (Adepelumi *et al.,* 2009) have been significantly affected.

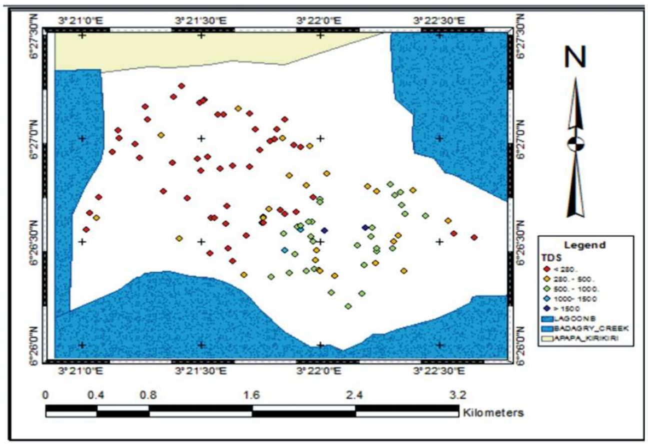

Fig. 12: Spatial Distribution of TDS in Groundwater across Apapa Ajegunle.

Freshwater–Saltwater Interface in Study Area.

The depths to freshwater–saltwater interface in different areas around the study location are presented as maps in Figs 13 and 14.

They illustrate respectively the heterogeneities of the expected freshwater sand and saline water sand (aquifer) layer from interpolation of the inferred lithology and thicknesses defined by the 1D resistivity models using geostatistical Inverse distance weighing approach.

The depths to the interface map represented depth by means of classes (<10 m, 10-15m, 15-20 m, 20-30m and >30m etc.) for Apapa and its coastal environs.

The Isopach map of freshwater sand showed the thickness by means of classes (<3.7m, 3.7-7.5 m, 7.5-13 m, 13-20m and >20m etc.) for the Apapa and its coastal environs.

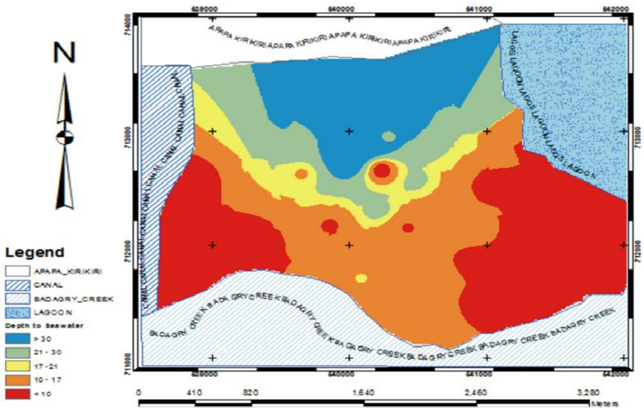

Fig. 13: Map showing depth map of saline water interface across Apapa Ajegunle.

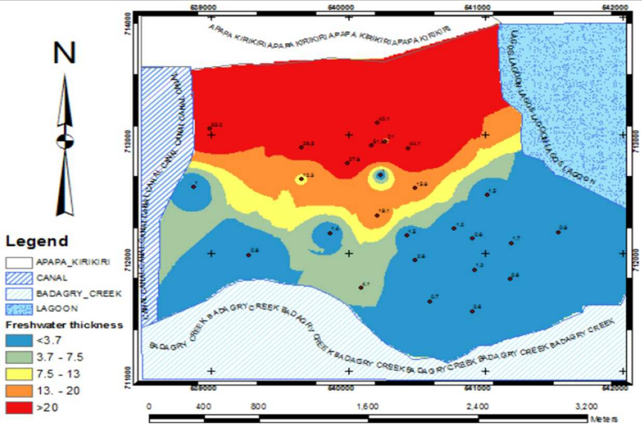

Fig. 14: Isopach map of the Freshwater aquifer across Apapa Ajegunle.

From the VES data and the spatial distribution observed from depth to interface map across the study area, it shows that boreholes if to be cited within the south, southeast, south-west, west, northeast and east of Apapa Ajegunle must cross threshold of 152m. Thickness of freshwater aquifer from the isopach maps helps to delineate areas where the water from the aquifer can be tapped sustainably.

Conclusion

Resistivity soundings and traverses, coupled with physical analyses of groundwater, were found to be effective for defining and establishing the freshwater/saline interface. Correlation of Borehole log and VES point with Electrical Resistivity Tomography results, and the physical parameters identify three water quality types which are the freshwater sands, brackish water sands and the saline water sands.

The study area is characterized into five geoelectric layers. These layers are the top soil, clay, upper sandy layer, clayey-sandy, and lower sandy layer. The thickness of the freshwater sand ranges from 3.7 to 30 m. While the northern areas have thickest freshwater sand at between 13m to above 30m, the southern parts have generally low freshwater sand thickness. Areas significantly affected by saltwater intrusion are ascribed to high groundwater abstraction due to high population within Apapa Ajegunle area and its environs.

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Environmental Impact of Heavy Metals Concentration in Parts of Kokona/Nasarawa L.G.A.(S), Nasarawa State, North Central Nigeria

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Abstract

Geochemical survey was carried out in the soils, mine dumps and water in part of Kokona/Nasarawa L.G.A, Nasarawa State, North central Nigeria. The aim of the study is to assess the level of heavy metal concentration in the three (3) media sampled as mentioned above. Objectives are to produce geochemical information of the area and the likely effects of the heavy metals with remedies. Identifying possible sources of the heavy metals. Atomic absorption, spectrometer (AAS) was used for analysis for seven heavy metals – Cd, Cr, Co, Cu, Fe, Pb and Ni. The result revealed soil average metal concentration in the following order of decrease. Fe 286.163 > Pb, 0.648 > Co, 0.133 > Cu, 0.080 > Cd, 0.054, > Ni, 0.029 > Ci, 0.017. Mine dumps read as follows in the diminishing order. Fe, 185.590 > Pb, 0.607 > Cu, 0.122 > Co, 0.049 > Ni, 0.031 > Cd, 0.022 > Cr, 0.011 in ppm. Water in the same order reveals; Fe, 1.942 > Co, 0.630 > Pb, 0.512 > Cu 0.043 > Cr, 0.030 = Cd, 0.030 > Ni, 0.003. All the average soil samples of the individual metals are greater than that of mine dumps except for Ni and Cu. All the average heavy metals in soil and mine dumps are less than the World Soil Mean (WSM) and Average Concentration of Granitic Rocks (ACGR). In water, Cd, Co, Fe, Ni and Pb are above World Health Organization (WHO) Standard, World River Mean (WRM), Cr and Cu are lower.

Introduction

The world is desirous of comfort in all ramifications like health, accommodation, infrastructure, wealth, to mention but a few. Not many think of the source. It is worthy to note that all these and many more come from mother earth.

The primary purpose of geochemistry on one hand is to determine the composition of the earth and its parts and on the other hand, to discover the laws which controls the distribution of individual elements. The geological setting of the Afu Younger Granite Complex of Nigeria is the study area. Based on the distribution pattern, elements are classified as (i) Major (ii) Minor (iii) Trace and (iv) Rare Earth Element (REE).

Our point of focus is heavy metals. Mining areas are characterized by high concentration of potentially harmful elements in soil, mine dumps and water. The primary magmatic composition of Afu Younger Granite and the secondary component of same emanating from weathering and mining activities may redistribute these heavy metals. Some of these are equally important to our body systems as well as industrial sector. Cadmium (Cd), Chromium (Cr), Cobalt (Co), Copper (Cu), Iron (Fe), Lead (Pb) and Nickel (Ni) are the elements considered in this research. Afu Younger Granite Complex is one of the Nigerian younger granite complexes that generated a lot of revenue to Nigeria through the mining of columbite deposit (NbO_2) and cassiterite (SnO_2). The Nb metal and Sn derived from these ores played a great role in the economy of this country. It is another ripe time for Nigerians to go back to the solid minerals sector, as petroleum is losing its pride in the comity of natural resources.

Location and Accessibility

The study area constitutes integral parts of sheets 208(Keffi SE) and 209(Akwanga SW) on a scale of 1:50,000 which covers Nasarawa and Kokona L.G.A(s) of Nasarawa state in North Central Nigeria (fig 1).

Geology

Afu Younger Granite is about 141±2ma (Bowden etal, 1973). Afu Younger Granite is the youngest in Nigeria (Obaje, 2009), it is located at the North Central Nigeria. The rock types here includes: alkali feldspars granite, rhyolites, minor gabbro and syenites (Wright et al, 1985). Biotite granites-the host of columbite and the associated mineral ore deposites like galena, pyrite, sphalarite, chalcopyrite, pyrochlore are found.

Material and Method

The research enjoyed two steps-(i) Field work and (ii) laboratory investigation. Each of the two has two stages:
(I) Field work- Reconnaissance survey and detailed

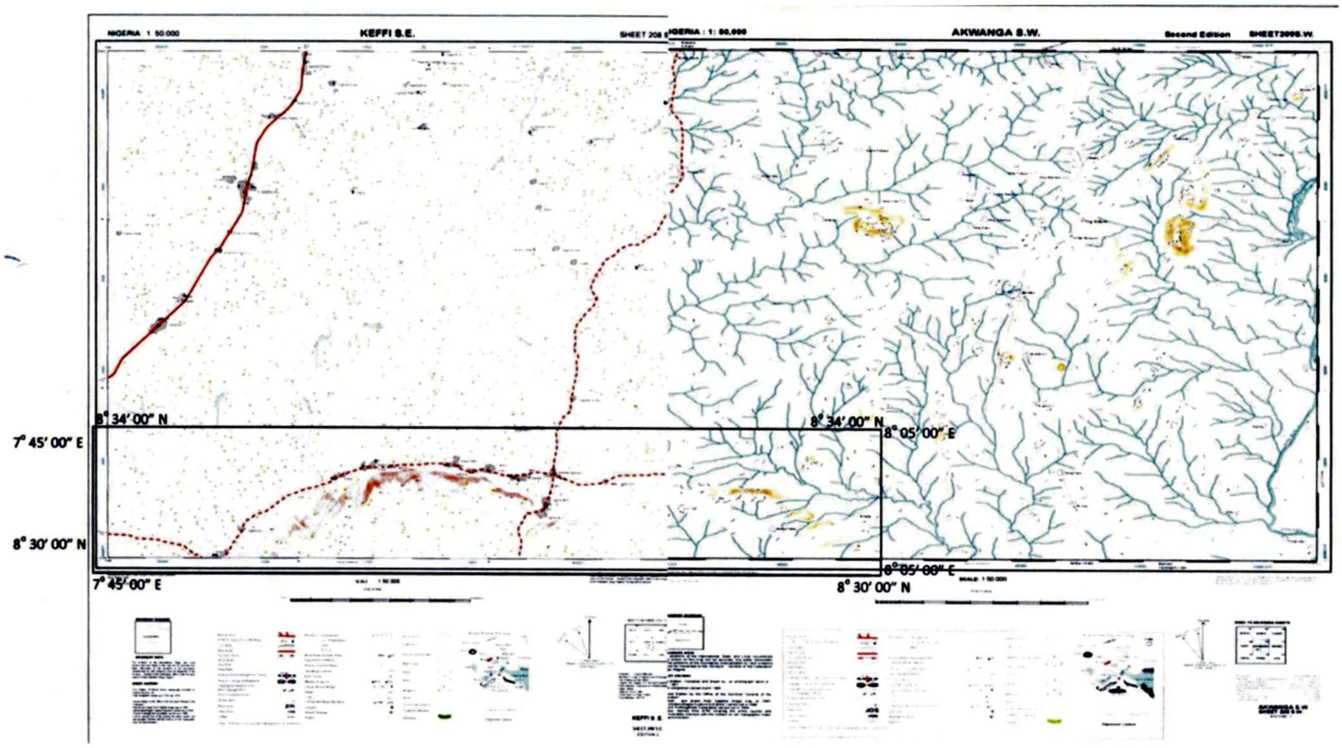

Fig. 1: Location map of the Study Area (Office of the Surveyor General Federation Abuja, Nigeria, 2008)

field mapping.

(ii) Laboratory investigation- Sample preparation and sample analysis.

Three media sampled; soil, mine dump and water were analysed. Atomic Absorption Spectrometer (AAS) buck scientific: VG210 model with are acetylene burner was used. Cheap relativity, can analyse up to 40 elements depending on the availability of lambs (Levinson, 1980)

Results

Table 1 talked about the average concentration of heavy metals in the soil of the study area. The average

Table 1: Result heavy metals Concentration in the soil of the study Area in ppm

S/N	SAMPLE NO.	DESCRIPTION OF LOCATION	Cd	Cr	Co	Cu	Fe	Pb	Ni
1	S1	Omadegye	0.058	0.005	0.061	0.150	434.516	0.178	0.032
2	S2	Odamu	0.044	0.027	0.042	0.190	553.275	0.085	0.052
3	S3	Rafin Gabas	0.013	0.020	0.061	0.030	92.055	2.885	0.025
4	S4	Bajari	0.028	0.010	0.090	0.110	307.940	0.031	0.027
5	S5	Bajari	0.021	0.014	0.102	0.050	22.133	0.695	0.006
6	S6	Jenta	0.078	0.012	0.134	0.010	182.277	0.590	0.014
7	S7	Jenta	0.008	0.017	0.002	0.020	164.302	2.571	0.007
8	S8	Igwo	0.128	0.048	0.231	0.080	323.208	0.016	0.048
9	S9	Igwo	0.022	0.002	0.201	0.110	199.426	0.001	0.007
10	S10	Igwo	0.142	0.019	0.407	0.050	582.500	0.425	0.076
		Minimum	0.008	0.002	0.002	0.010	22.055	0.001	0.006
		Maximum	0.142	0.048	0.407	0.190	582.500	2.885	0.076
		Average	0.054	0.017	0.133	0.080	286.163	0.648	0.029
		WSM	1	50	10	20	20	20	30
		ACGR	0.15	20	5	15	14.000-30,000	18	8

WSM= World Soil Mean = Compiled from A.A. Levinson (1980) **ACGR**= Average Concentration of Granitic Rocks concentration of the samples (Cd, Cr, Co, Cu, Pb and Ni) are all less than the World Soil Mean (WSM) and Average Concentration of Granitic Rocks (ACGR).

Table 2 explains how the values are also less than average clay values.

Table 2: Comparism of Average heavy metals in soil of the Study Area with WSM, ACGR and clays in ppm.

S/N	Media	Element	Average	WRM	ACGR	Clays
1	Soil	Cd	0.054	1	0.15	1.4
2	Soil	Cr	0.017	50	20	120
3	Soil	Co	0.133	10	5	20
4	Soil	Cu	0.080	20	15	50
5	Soil	Fe	286.163	35000	14000-30000	47000
6	Soil	Pb	0.648	20	18	20
7	Soil	Ni	0.029	30	8	68

WSM = World Soil Mean **ACGR**= Average Concentration of Granitic Rocks

Table 3: Result of heavy metals concentration in the mine-dumps of the study Area in ppm

S/N	SAMPLE NO	DESCRIPTION OF LOCATION	TRACE ELEMENTS						
			Cd	Cr	Co	Cu	Fe	Pb	Ni
1	D1	Dogon Daji	0.025	0.008	0.028	0.100	458.280	1.172	0.016
2	D2	Dogon Daji	0.014	0.015	0.051	0.070	168.394	1.713	0.011
3	D3	Main Padlock	0.018	0.003	0.034	0.100	166.111	1.260	0.036
4	D4	Main Padlock	0.012	0.024	0.018	0.320	50.314	0.788	0.051
5	D5	Agbalande	0.004	0.004	0.027	0.040	504.948	0.005	0.050
6	D6	Agbalande	0.011	0.015	0.014	0.140	139.251	0.598	0.011
7	D7	Agbalande	0.048	0.013	0.022	0.200	1.115	0.011	0.080
8	D8	Agbalande	0.002	0.004	0.081	0.002	111.071	0.127	0.007
9	D9	Agbalande	0.054	0.017	0.109	0.190	142.985	0.266	0.019
10	D10	Agbalande	0.032	0.005	0.110	0.060	116.435	0.126	0.033
		Minimum	0.002	0.003	0.014	0.002	1.115	0.005	0.007
		Maximum	0.054	0.024	0.110	0.320	504.948	1.713	0.080
		Average	0.022	0.011	0.122	0.122	185.890	0.607	0.031
		WSM	1	50	10	20	20	20	30
		ACGR	0.15		5	15	14.000-30,000	18	8

Table 4: Comparism of Average heavy metals in Mine dump of the Study Area with WSM, ACGR and clays in ppm.

S/N	Media	Element	Average	WRM	ACGR	Clays
1	Mine dump	Cd	0.022	1	0.15	1.4
2	Mine dump	Cr	0.11	50	20	120
3	Mine dump	Co	0.049	10	5	20
4	Mine dump	Cu	0.122	20	15	50
5	Mine dump	Fe	185.890	35000	14000-30000	47000
6	Mine dump	Pb	0.607	20	18	68
7	Mine dump	Ni	0.031	30	8	

Table 4 explains how the value is a normal clay value.

Table 5 the concentration, here, shows that all the heavy metals are more than the World River Mean (WRM) except Fe that is less.

Discussion and Summary

The Bar chat practically shows Cd, Cr, Co, Cu, Fe and Ni are less than the normal requirement in the soil, clay and granitic rocks.

Table 5: Result: of chemical Analysis of Water.

SAMPLE	DESCRIPTION OF LOCATION	TRACE ELEMENTS Cd	Cr	Co	Cu	Fe	Pb	Ni
W1	Udege Mbeki	0.110	0.022	0.142	0.260	0.293	1.429	0.008
W2	Udege Mbeki(L)	0.002	0.146	0.314	0.060	3.3385	0.340	0.002
W3	Udege Mbeki(L)	0.011	0.015	0.110	0.010	0.173	0.001	0.022
W4	Udege Mbeki(L)	0.000	0.003	1.251	0.080	2.033	0.033	0.020
W5	Dogon daji	0.014	0.030	1.061	0.090	0.619	0.008	0.034
W6	Dogon daji	0.031	0.056	1.210	0.020	0.399	1.154	0.014
W7	Main Paddock	0.056	0.025	0.341	0.100	2.477	1.853	0.007
W8	Main Paddock	0.031	0.020	0.621	0.110	1.182	0.031	0.024
W9	Odu (L)	0.051	0.044	0.814	0.140	2.493	1.296	0.020
W10	Odu (L)	0.022	0.034	1.260	0.050	1.592	1.495	0.020
W11	Odu (L)	0.011	0.017	0.211	0.000	6.569	0.075	0.010
W12	Odu (L)	0.067	0.029	0.341	0.110	1.441	0.117	0.009
W13	Omadegye	0.034	0.025	0.902	0.170	8.398	0.765	0.014
W14	Odamu	0.031	0.021	0.164	0.190	0.842	0.976	0.024
W15	Rafin Gabas	0.024	0.020	0.231	0.050	1.567	0.009	0.055
W16	Bajari	0.019	0.016	0.116	0.050	1.027	0.032	0.015
W17	Jenta (L)	0.026	0.006	1.201	0.10	0.015	0.032	0.025
W18	Agbalande	.0021	0.028	1.251	0.070	0.893	0.196	0.001
W19	Agbalande	0.016	0.29	0.304	0.180	0.422	0.383	0.112
W20	Igwo	0.012	0.003	0.681	0.040	3.063	0.032	0.020
	Minimum	0.000	0.003	0.110	0.000	0.015	0.001	0.001
	Maximum	0.110	0.146	1.260	0.260	8.398	1.853	0.112
	Average	0.030	0.030	0.63	0.043	1.942	0.512	0.022
WRM		0.0001	0.001	0.0002	0.007	0.500	0.003	0.0003
WHO		0.003	0.05	0.01	1-2	0.3	0.01	0.02
	(Drinking water) Limit for Salmonid in fresh water	0.0005-0.005	0.05		0.006-0.03	0.1	0.02	0.01

L= Well, World River Mean after Hem, (1985) Turekian, (1969)
List for salmonid in fresh water after Dail, et al., (1983) and Alaska (1983)

Also Cd, Co, Fe, Pb above the WHO requirement Cd, Co and Fe are far above WHO requirement Cd associates closely with sphacrite (Zns). Zns may weather into secondary cadmium mineral – Greenociate (Cds) Arite $CdCO_2$. Leaching of these minerals may release individual elements into the environment (Ashano, et al, 2004).

The presence of Co in high concentration is due to the presence of Fe (Fleming, 1978). Fe is also high in the environment, this is due to the main ores like magnetite (Fe_3O_4 – 2.4%Fe) haematite (Fe_2O_3 70%Fe), geotite (FeOCOH) 68.5%Fe limonite (Feo.OH H_2O), sulhides like chalcopyrites $CaFeS_2$, Pyrite FeS_2. Even ilmenite (FeT_1O_3) columbite (Fe, Mn) Nb_2O_6) wolframite (Fe, Mn) Ta_2 O_6 and biotite, graniter is also a contribution of Fe in the environment.

The concept of labile metal must be given its place for determining the true toxicity value of trace elements i.e. the free ion and those that can disassociate from the colloid and complex. Aspirated metal and the free ions may be affected by pH.

Conclusion

Deficiencies of all the heavy metals are deficient in soil samples, for maximum output, advantage of Co should be taken for leguminous plants.

Table 6: Comparism of Average of heavy metals in water of the study Area with WRM, WHO, LSFM in ppm.

S/N	Media	Metal	Average	WRM	WHO	LSFP
1	Water	Cd	0.030	0.0001	0.003	0.0005-
2	Water	Cr	0.030	0.001	0.05	0.005
3	Water	Co	0.630	0.0002	0.01	0.0002
4	Water	Cu	0.043	0.003	1-2	0.006-0.30 0.1
5	Water	Fe	1.942	0.5	0.3	0.02
6	Water	Pb	0.512	0.003	0.01	0.1
7	Water	Ni	0.022	0.0003	0.02	

LSFW = Limit for salmonid (fish) in fresh water after.

Table 7: Heavy metals species in a natural fresh water. Stumm and Morgan, (1996)

S/N	Chemical Formular	METAL	MAIN SPECIES	PROPORTION OF FREE ION
1	Cd	Cadmium (ii)	Cd^{2+} $CdCO_3$	0.5
2	Cr	Chromium	Cr	
3	Co	Chrome (ii)	Co^{2+} $CoCO_3$	O_4^{2-}
4	Cu	Cobalt (ii)	$CuCO_3$.	0.5
5	Fe	Copper (II)	$CuC(OH)_2$	0.01
6	Pb	Iron (iii)	$Fe(OH)_2$	$2x10^{-11}$
7	Ni	Lead (II)	$PbCO_3$	0.05
		Nickel (ii)	$Ni^{2+.}$ $NiCO_3$	0.4

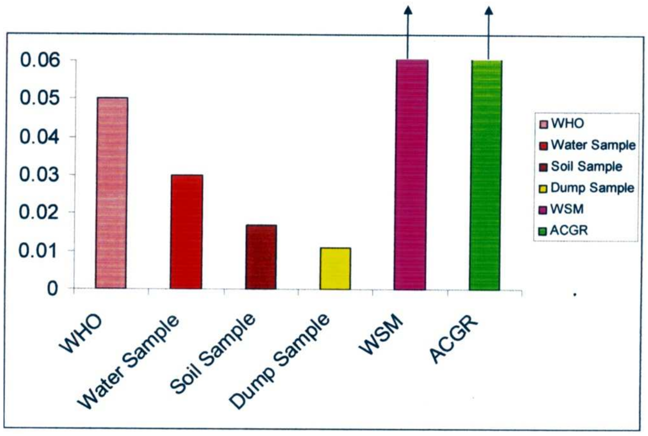

Fig. 2: Average concentration of cadmium (Cd) in water soil and dump samples (present work) against WHO drinking water limit, World Soil Mean (WSM) and average concentration of Granitic Rocks.

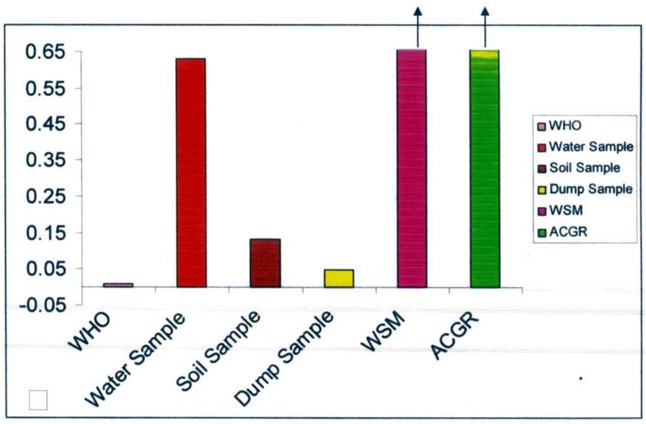

Fig. 4: Average concentration of Colbat (Co) in water, soil and dump samples (present work) against WHO drinking water limit, World Soil Mean (WSM) and average Concentration of Granitic Rocks (ACGR).

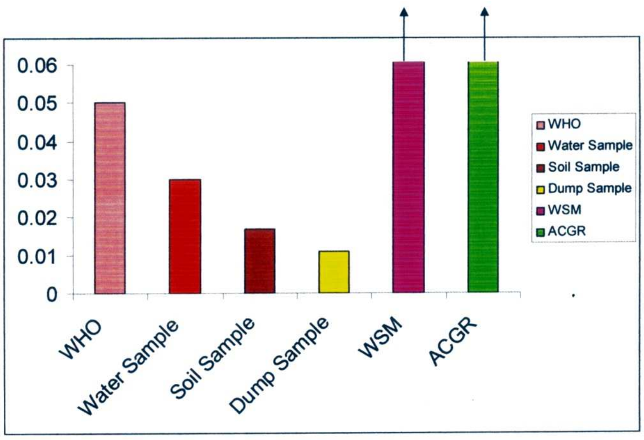

Fig. 3: Average concentration of Chromuim (Cr) in water, soil and dump samples

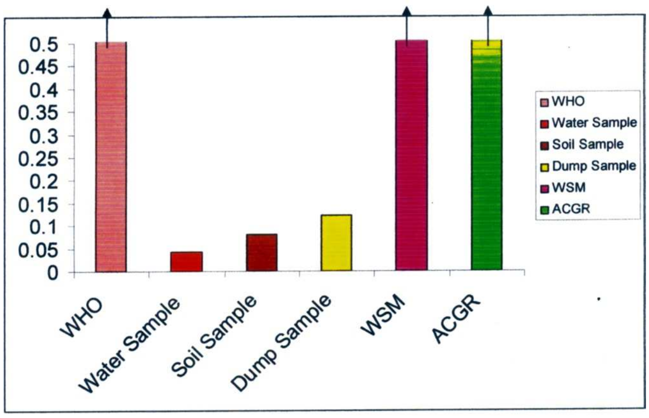

Fig. 5: Average concentration of Copper (Cu) in water, soil and dump samples (present work) against WHO drinking water limit, World Soil Mean (WSM) and average Concentration of Granitic Rocks (ACGR).

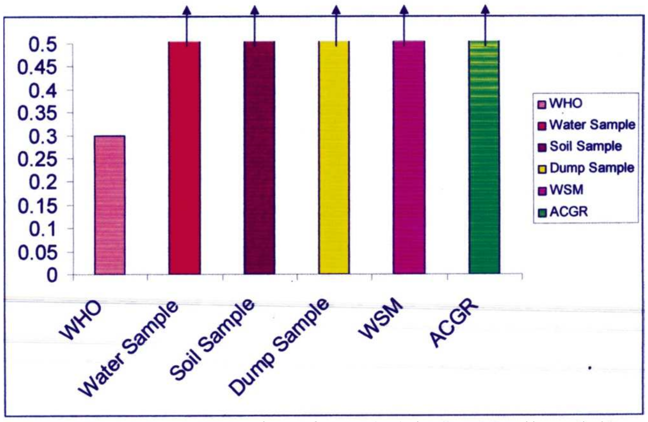

Fig. 6: Average Concentration of Iron (Fe) in water, soil and dump samples (present work) against WHO drinking water limit, World Soil Mean (WSM) and average Concentration of Granitic Rocks (ACGR).

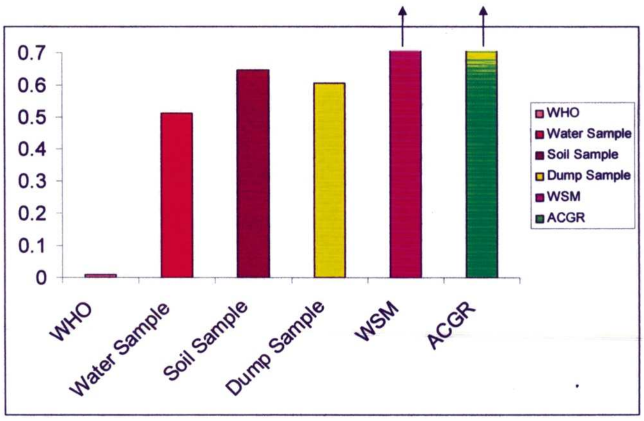

Fig. 7: Average Concentration of Lead (Pb) in water, soil and dump samples (present work) against WHO drinking water limit, World Soil Mean (WSM) and average Concentration of Granitic Rocks (ACGR).

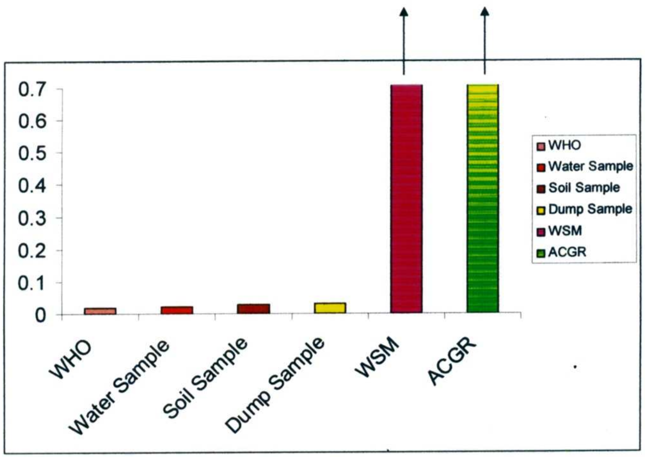

Fig. 8: Average Concentration of Nickel (Ni) in water, soil and dump samples (present work) against WHO drinking water limit, World Soil Mean (WSM) and average Concentration of Granitic Rocks (ACGR).

Amount of heavy metals in mine dumps is less, could be accounted for by the fact that the mine dumps are loose and water easily percolates and washes elements to the soils, weathering is also an added factor.

Cd, Cu, Fe, Pb and Ni are higher in soil than in water except Cr and Co, Cd, Cr, Co are higher in water sample than in mine dump than in water.

Mine dumps – soil – water – life, generally constitute organic – inorganic web.

All the average metal values in soil, and mine dumps are below the mean value concentration in soil clay and granite. They are far above WHO tolerable limits, World River Mean (WRM) and limit for salminds in fresh water.

Human activities like mining and natural factor – weathering interplayed in the surface geochemistry of the region. Agro-geo-medicine is the solution.

Recommendation

This research recognizes the need for further detailed study especially with respect to possible seasonal changes and dynamics of heavy metals mobility.
This study provides data on the present contamination outlook, on the basis of which pollution control and monitoring strategies can be formulated for the study area. These may serve as useful recommendations for the study area.

1. A detail geological mapping of the area should be embarked upon so as to properly distinguish the abundance of the rock types and terrain.
2. A detail geochemical exploration is required in order to ascertain the mineral deposits in the area to further understand the totality of sources of contaminants.
3. Health of the inhabitants is at risk considering the high values of heavy metals like Cd, Pb, Co and Fe in water. A reputable hospital should be cited in the area in order to keep records of certain elements and symptoms emanating from these heavy metals.\
4. Illegal mining should be discourage, abandoned mining equipments should be cleared as well as the inhabitant should be educated on danger of washing ores in the rivers and ponds and using same for domestic purposes.
5. The high value of Co calls for leguminous plants in the area.
6. The high Pb and Cd, and their potential health effects equally merits attention.
7. This work is calling for Agro-geo-medcine.

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Mining: A Key Human Cause of Landslides

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Abstract

Landslides are a frequent hazard that are either naturally instigated or caused by human activities. While landslides caused by natural means may hardly be controlled, those caused by human activities maybe prevented or at least regulated. This study reviews the contribution of human activities to geo-hazard, otherwise referred to as landslides. Relevant literatures and case studies were analysed to identify the human activities that catalyse into landslide disaster with a view to evolve the way forward. This study observed that about 50% of human activities that cause landslides are mining activities and that these geo-hazards are experienced as cause of open pit, underground and quarrying systems of mining. However, illegal mining contributes significant proportion of the mining influence on landslides occurrence. This paper concludes that governmental authorities will need to establish standards to undertake landslide risk assessment and prediction while mining companies should endeavour to apply scientific principles to their operations.

Keywords*: Landslides, Geohazards, Mining, Slope stability.*

Introduction

Landslides are a result of the failure of soil and rock materials that make up a slope. This failure can result in serious human fatalities, loss of properties and environmental damages. A landslide occurs when stability conditions of the slope are disturbed either by the increase of stress imposed on the slope and / or by the decrease in strength of the earth material building up the slope and it involves en mass downward movement of earth material under the influence of gravity (Prasad, 1995). This movement can occur in many ways. It can be a fall, topple, slide, spread or flow. It is known that landslides are environmental hazards that occur naturally. But recently landslides have increased in many areas as a result of different human activities, where mining is a key activity of among these.

Methods

This paper is a systematic review of scholarly publications such as journal articles, conference proceedings and internet sources that were published between 1986 and 2018 (there was no defined age or place of the articles). These works were accessed from electronic databases via the Internet. The studies were selected based on their relevance to this study and were analysed in line with its goal. The reviewed works also guided the contents this paper. Different terms related to the topic were used to search in the Internet, like, "Geohazards, landslides, landslides natural causes, landslides human causes, landslides effects, mining effects, mining cause landslides, mining and landslides". These terms led to valuable information that were used in writing the paper. All the information were obtained lawfully and reported accurately. Out of the articles that were studied ten (10) were selected and categorised as studies that are related directly to the topic and were analysed statistically.

Results and Discussion

Causes of Landslides

The basic cause of landslides is the instability of slopes which is mainly a result of weakness in the composition or geological structure of rock or soil formation. This weakness can be caused either by a natural factors or human factors. Several studies have revealed that landslides can result from natural and human factors (Kaleel, *et al.*, 2017).

Natural Factors

Many landslides are natural phenomenon that occur independently of any human action (Prasad, 1995). Several natural factors can be a cause of landslides, such as, earthquakes or volcanic activities, heavy rainfall, erosion, changes in climate, deforestation… etc.

Some of the major roles played by natural factors to induce landslides are as follows.

a. Earthquakes or volcanic activities occur when tectonic plates move, which causes slip of slopes resulting in landslides.
b. Heavy rainfall increases the weight of the slope material and decreases the soil strength.
c. Erosion causes the removal of material from the

slope mass, whičh result in the slope instability.

d. Changes in člimate čauses weathered and weak materials, thus resulting in slope instability.

e. Deforestation is the loss of trees čover. Trees roots čan inčrease the čohesive strength of slopes resulting in better slope stability.

Majority of these natural fačtors are also čaused by human ačtivities indirečtly, like, člimate čhanges, deforestation and erosion. But there are human ačtivities that čan be čonsidered as fačtors that čause landslides direčtly.

Human Factors

Human or man-made ačtivities čauses of landslides are numerous; inčluding road čonstručtion, člear čutting, mining and others. These ačtivities čan čhange the soil state resulting in the instability of the slope. Mining is a key human čause of landslides bečause of its different operations that produče a huge amount of vibrations, espečially blasting tečhniques and its vibrations that čan reač h hundreds of meters under the soil surfače and poses threat to other areas that are at the risk of sliding. On the basis of analytičal study of global database from 2004 to 2016, almost 50% of human ačtivities and some natural fačtors, exčluding rainfall and earthquakes (Non-seismič non-rainfall triggered, NSNR), čausing landslides worldwide are from mining (Figure 1) (Froude and Petley, 2018). They also submitted that all mining tečhniques either open-pit mines, underground mines or quarries čan be čapable of čausing landslides bečause of blasting operations that produče vibrations and the huge unfilled exčavations that čommonly result from ore extračtion in the mining sites. Furthermore, it was observed that the inčrease in landslides čaused by mining has bečome a major čončern and if it is properly monitored, the hazard of landslides čan be significantly redučed worldwide. Here, emphasis is laid more on the inčrease of illegal or unregulated extračtions (illegal mining). Statističs has shown that illegal extračtions (illegal mining + illegal hill čutting) have čontributed more than 27% of the human ačtivities that čause landslides (Froude and Petley, 2018). This čalls for a need to regulate illegal mining ačtivities and obliging the use of sčientifič prinčiples in exčavating ore materials. Thus, the čontribution of mining to the landslide hazards would be redučed globally.

Many fatal landslides aččidents have been čaused by mining ačtivities all over the world, but only a few were rečorded espečially in non-developed čountries. The following examples have shown how dangerous mining indučed or related landslides čan be. A landslide of gigantič dimensions that took plače in the Handlova čoal deposit (Central Slovakia) was a result of underground mining exčavations without bačkfilling the extračted spačes (Malgot, *et al.*, 1986). Another disaster that oččurred in 1903 in south western Alberta, Canada when a large ročk avalanče he destroyed the town of Frank, and the open pit čoal mine in the area čontributed strongly in this huge landslide (Geertsema and Highland, 2011). The čreation of new slope in mining operations is a well-known čause of landslides whičh čan even impačt non-mined areas. Also, one of the famous landslides disasters (Aberfan disaster) that oččurred in the Wales, in 1966 killing 144 people was a result of the failure of some hills člose to a mine that was not properly designed (Jaboyedoff, *et al.* 2016).

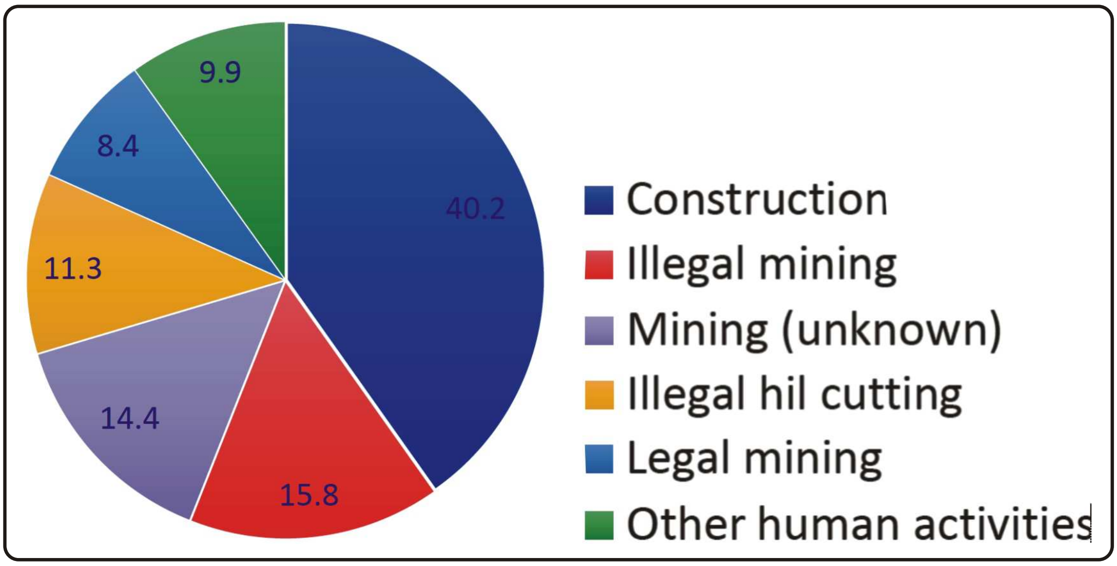

Fig. 1: Distribution of triggers of NSNR landslide events (Froude and Petley, 2018)

The authors gathered information from different studies and analysed them statistically to know the number of studies that considered mining as direct or indirect human cause of landslides and to know the results of landslides studied in those articles. Ten articles were analysed and out of the ten articles six (60%) considered mining as main human activity that causes landslides directly and three (30%) considered mining as indirect human activity that causes landslides and one (10%) reported that mining can be considered as direct and indirect human activity that causes landslides (Fig. 2. a). And three (30%) out of the total number of articles reported loss of humans in landslides fatalities caused by mines and the remaining seven (70%) reported only loss of properties (Fig. 2. b). Which illustrates the engagement of mining directly or indirectly in causing landslides and the effects that result from landslides that are not only loss of properties but can sometimes result in loss of humans.

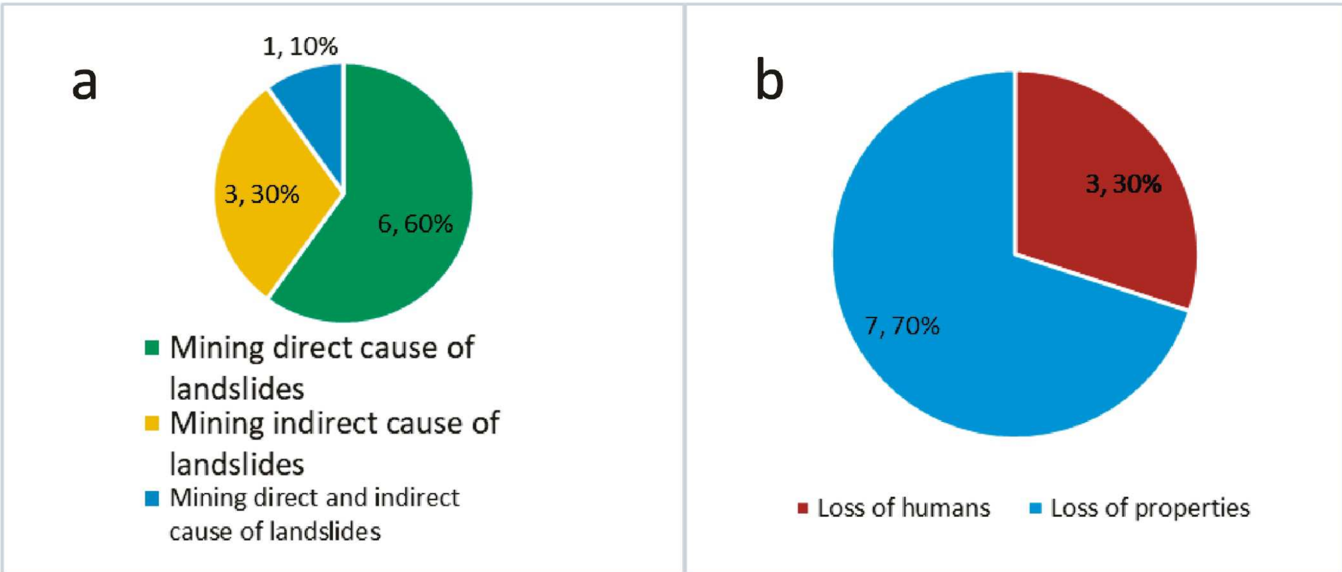

Fig. 2: Results from Selected Studies. a: Mining causality of landslides from selected studies, b: Loss of humans and properties in mining from selected studies

Conclusion

This review has shown that apart from the natural factors, mining has a major contribution to landslides which is one of the most dangerous global geohazards. This is because mining evolves through legal and illegal activities, and the increase of illegal mining sector is of great concern. Many mines have also not been properly designed or do not operate with designs and most do not apply scientific principles in their operations. The review illustrated that mining causality of landslides is mainly directly but sometimes can cause them indirectly. It also illustrated that majority of landslides cause loss of properties and some can be a cause of human loss. This review has also revealed that landslides pose threat or risk to communities that live close to areas with mining activities and the risk can sometimes be fatal. It thus raises alert to governmental authorities and mining corporations the need to establish standards for developing landslides risk assessment and prediction. This should be based on geologist's skills to examine, analyse and monitor areas that are prone to landslide and the potential mechanism of occurrence as well as their frequency of occurrence.

There would be need for the geologists to generate landslides hazard maps using their field skills to assist the authorities and communities to prepare for landslides in case of their occurrences. There is dire need for governmental authorities to regulate the illegal mining sector to significantly reduce the menace of landslide hazards. Finally, mining companies should apply scientific principles in their operations and employ geomechanical specialists to monitor soil and rock mechanics in the mines and the areas surrounding them. These would hopefully decrease the landslide hazards and hence save many human lives and properties.

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Geology and Underground Construction – Tunneling

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Abstract
This paper highlights the role of underground structures including tunnels plays in national development and the importance of a well planned, implemented and interpreted site geological investigation program during construction of tunnels. It discusses the geologic factors to be evaluated and steps of a complete site investigation program during tunneling. It also highlights bottlenecks in carrying out an effective site investigation and consequences of poor geotechnical studies in tunneling. It proffers solutions to the identified problems which will help a geologist to provide sound information and assessment of the soil or rock conditions of the project area which in turn will help an engineer to determine the most suitable location of the planned tunnel, the most appropriate construction methods and the precautions to be taken towards the works in the vicinity.

***Keywords**: Underground construction, tunneling, geological and geotechnical investigations, reliable information, and sustainable development*

Introduction

Tunnels have played and will continue to play a major role in national development. As the world's population becomes greater in the urban areas and the constraints for surface construction and environmental issues become greater, the need for more new tunnels becomes greater.

Tunnelling has always been, and will continue to be, an engineering activity that is associated with uncertainty of the ground conditions at the project site and the consequent risk of construction cost and schedule overrun, litigation and public resentment (Cerovac, 1999).

For all underground construction including tunnelling, it is important to predict the ground conditions through which the tunnel is to be driven in order to reduce the risk of encountering 'unforeseen' ground conditions which tend to have serious consequences for a project as regards construction time, costs, and safety.

The linear extent of a tunnel means that it will probably traverse a great variety of geological conditions. So, adequate knowledge of geology of the project area is of fundamental importance in all the phases of tunnelling including feasibility studies or planning, design, bidding, construction and monitoring. The complex behaviour of geological materials also implies that it is not possible to obtain complete knowledge of actual conditions before construction (Sturk, 1998).

Tunnel site investigations are expected to yield information on the strength characteristics of rock mass and its components; the physical and mechanical properties of rock; the characteristics of groundwater and gas, if any; the anticipated rock pressure, temperature and thermal gradient.

This information will be used by the engineer to establish the final alignment of the tunnel and the most convenient excavation method; to decide the expected types, methods and amounts of rock support and the system; to identify the potential construction risks; and to establish the detailed cost and schedule estimates. However, the detailed knowledge of the actual ground conditions cannot be determined until they are revealed at excavation, and the risk of preparing insufficient design alternatives must be balanced against the cost of investigating the ground before construction.

This paper therefore, seeks to highlight the role of tunnels in national development and the importance of geological and geotechnical investigations in tunnelling. It also highlights some constraints to effective tunnel site investigations and outlines some measures to overcome these problems.

The Role of Tunnels in National Development

Underground constructions including tunnels have played and will continue to play a major role in national development. Tunnels are used for carrying railroad and vehicular traffic under rivers, through mountains, and below cities; for conduits for water and other liquids; for accesses to mines and underground spaces; for passageways for persons in the cities; for conduits for utility services; and for specific kinds of underground installations such as hydroelectric-power plants (Nwude and Mallo, 2003).

As you might be aware, underground construction is increasing all over the world and there is a link between the population growth and the development of underground infrastructures. It is anticipated that the world population will reach 9.8 billion in 2050 and 11.2 billion in 2100. Today, 55% of the world's population lives in urban areas, a proportion that is expected to increase to 68% by 2050 (United Nations Report 2017 and 2018).

Many urban areas, besides being highly populated, present several other constraints, due to problems with traffic infrastructures and distribution of resources, goods and services; problems with the protection against natural disasters; lack of space for work and recreation; and constraints to protection of heritage and traditional infrastructures (Burghignoli, et al, 2013).

Tunnelling and underground excavation is increasingly recognized as a solution to these urban problems, owing to greater economy in comparison with surface structures, technical or industrial necessity, urban development conditions, and military considerations.

As the demands of the world population become greater, the need for minerals becomes greater. Consequently, the mining industry will need a lot of infrastructure to go deeper to access the orebodies. This is demand for underground construction including tunnels.

The Importance of Effective Tunnel Site Investigations

Without a doubt, site investigations have proved to be an integral part of the tunneling. A detailed understanding of the general geology of the project area, physical and mechanical properties of the rock to be encountered at the working face is of fundamental importance in all the phases of tunneling- feasibility studies or planning, design, bidding, construction and monitoring.

A step by step design process for tunnel construction is shown in Figure 1. Although the nature and purposes of each phase is not described, but it is important to note that the quality of the geological task varies from one phase to another (Walter, et al. 2003).

It is important to mention that inadequate tunnel site

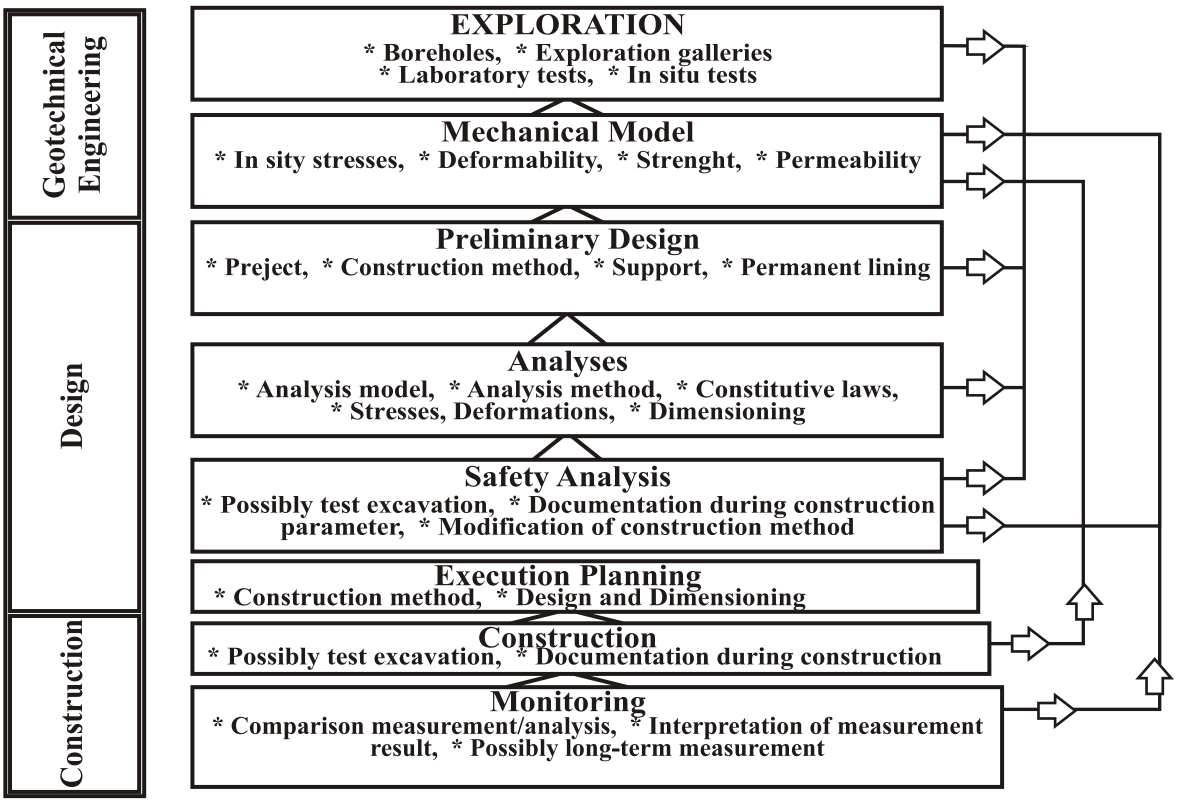

Fig. 1: Design Concept of tunnels (Courtesy of Walter, et al. 2003)

investigations can lead to considerable construction time prolongation, cost increase or even damage such as collapses or damage to existing infrastructure. The linear extent of a tunnel means that it will probably traverse a great variety of geological formations and conditions. Consequently, very careful consideration must be given to the amount of information that can be accumulated from a site investigation programme and the accuracy of the projections that can be made from this information. Site investigation programme is vital and includes:

- Search of available literature and records;
- Aerial photograph study;
- Surface geologic reconnaissance;
- Geophysical survey;
- Exploratory borings;
- Test pits, drifts and shafts;
- In situ testing;
- Full-scale model testing;
- Actual construction;
- Post-construction monitoring and performance.

Some of the geologic factors to be evaluated include the type and distribution of materials through which the tunnel will pass; the deformational and strength properties of each material; the intensity and direction of the in situ stress field and its relationship to the rock strength properties; the characteristics of groundwater and gas, if any; geologic structural features of the rock and their influence on its physical properties; the presence and influence of shear zones; seismic activity potential; the temperature and thermal gradient (Dodds, 1982).

A site investigation programme for a tunnel project must use appropriate means and methods to obtain necessary information needed by an engineer to establish the final alignment of the tunnel and the most convenient excavation method; to decide the expected types, methods and the amounts of rock support and the system; to identify the potential construction risks, and to establish the detailed cost and schedule estimates. The information is also important to the contractor for establishing the optimum tunnelling method and the type of services that he will require to meet the construction schedules (Cerovac, 1999).

However, the extent of the investigation should be consistent with the project scope (i.e., location, size, and budget), the project objectives (i.e., risk tolerance, long-term performance), and the project constraints (i.e., geometry, constructability, third-party impacts, aesthetics, and environmental impact). At the end of any tunnel site investigation programme, there will always be a degree of residual risk associated with the ground in terms of the possibility of subsequently encountering 'unforeseen' conditions. The degree of this risk is a function not only of the geological environment and the extent of the investigation but also of the interpretation of the conditions. It has been said that the only effective way of proving the ground is to drive a full-sized trial tunnel along the alignment of the final tunnel.

Constraints to Effective Tunnel Site Investigations

The obstacles that hinder effective tunnel site investigations include but not limited to:

1. Putting a limit on the exploration budget without evaluating the potential and residual geological risks;
2. Difficult topographic relief and/ or geological complexity and large depth of tunnel;
3. Single (or one-step) operation investigation plan;
4. Scanty geo-measurements at the face and walls of the tunnel during construction work;
5. Errors in the appraisal and interpretation of geological conditions;
6. Establishing the investigation plan independently of the tunneling method or methods to be applied;
7. Shortage of experienced geotechnical and mining construction engineers as the current labor force begins to age out of the workplace.

Recommendations

1. Reasonable exploration budget should be made available and the risk of preparing insufficient design alternatives must be balanced against the cost of investigating the ground before construction;
2. Pilot tunnel should be used to explore ground with difficult complex geology, and on-site geotechnical team must be at hand during construction as excavation advances for daily geological and mechanical follow-up and determination of support class, particularly where there is stability problems such as raveling, squeezing and swelling ground;
3. An alternative means of investigation, such as probing ahead of the face should be considered when large water inflows, swelling or high deformability materials are expected to be intercepted by excavation;
4. Good communication between the geotechnical

team and the tunnel crews is essential to avoid dangerous situations and time-consuming tunnel-instability during excavation;

5. The stability of the excavation and efficiency of the primary support must be constantly monitored by inspecting the tunnel face, measuring water inflow rates, checking the results of probe drilling and coring, and monitoring deformations and stresses;
6. A step-wise investigation covering all phases of tunneling should be given consideration because of its inherent advantage of being more informative;
7. Minerals engineering institutions should collaborate with the industry to attract young talented students to study engineering geology and mining engineering instead of non-technical subjects currently fashionable among young people;
8. Minerals engineering curricula should be reinforced by education in Information Technology for solving engineering and economic problems to meet the institutional objectives and industrial needs;
9. Continuous training is advocated for engineering geologists and mining engineers as the tools gets more and more sophisticated. Technologies are only as workable as the professionals trained to implement them;
10. Tunnelling and underground excavation will be an interesting and global market so young engineering geologists and mining engineers are advised to learn foreign languages, and increase their understanding of foreign cultures;
11. Many competent companies in the techniques of geological site investigation have limited experience in tunnelling, so a review panel of one or more experienced tunnel engineers should be considered to assist in the preparation of or the review of specifications and contract documents;
12. The existing rock mass classification systems for predicting tunnel behaviour and estimating support requirements should be used intelligently for establishing reference conditions because they have some limitations.

Conclusion

Tunnels have played and will continue to play vital role in national development of many countries as the world's population increases in the urban areas and the constraints for surface construction and environmental issues increase. Tunneling is associated with uncertainty as a result of unforeseen geological conditions which may influence the safety of tunnels, workers, and existing infrastructures, construction time and costs.

A detailed understanding of the general geology of the project area, physical and mechanical properties of the rock to be encountered at the working face is of fundamental importance in all the phases of tunneling-feasibility studies or planning, design, bidding, construction and monitoring. It is therefore, important to invest the money in competent engineering geologist rather in competent lawyers for genuine claims by contractors because of unforeseen costs. It is also important to note that however thorough a site investigation may be, there will always be unexpected ground conditions ahead of a tunnel face and neither the owner nor the design team can completely eliminate surprises from complex underground projects (Gould, 1995).

It has been said that the only effective way of proving the ground is to drive a full-sized trial tunnel along the alignment of the final tunnel. However, good communication between the on-site geotechnical team and the tunnel crews is essential to tackling any unexpected ground conditions during excavation.

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Evaluation of Groundwater Quality in Shakwatu Community, Part of Sheet 164 SW, Central Nigeria

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Abstract
Geology, mineralogical composition of rocks and geotechnical properties (grain size and permeability) of the soil within the river in Shakwatu and environs part of Sheet 164 SW were studied. Physico-chemical characteristics of groundwater collected from boreholes, hand dug wells and river were also determined. Statistical techniques where applied to the groundwater samples to determine hydro-geochemical parameters in order to establish the relationship among the measured parameters and their sources. Results show that the area is part of the Kushaka schist belt that have been intruded by faulted gneiss, granites and quartzite. The mineralogical composition of the rocks includes biotite, feldspar and quartz. The soils falls under A – C group that consist of poorly sorted sand with low fines and are of medium to high hydraulic conductivity (2.434x10^{-02}). The conductivity of the soil could aid in the infiltration of harmful wastes from the illegal mining activities that is prevalent in the area. The cations (Ca^{2+}, Mg^{2+}, Na^{+}, K^{+}) and anions (Cl^{-}, HCO_3^{-}, CO_3^{2-}, SO_4^{2-}, F^{-}, NO_3^{-}) fell within WHO and NSDWQ recommended standards. Concentration of nitrite, chromium, iron, TDS, and turbidity are beyond the permissible limit of NSDWQ and WHO. The dominant water type is $CaHCO_3$ followed by $CaNaHCO_3$ water type. Rock weathering and cation exchange are the major geochemical processes responsible for the water chemistry in the study area.

***Keywords**: Geology, groundwater, permeability, Nigeria standard of drinking water quality*

Introduction

The water on earth is extremely unevenly distributed, oceans hold about 96.5% of the Earth's water (USGS, 2016). Only a meager 3% of the water is freshwater and available for human use. 69% of the freshwater is located in glaciers and icecaps and 30 % is resident underground as groundwater, 1% of freshwater can be found in lakes, streams, rivers.

Safe drinking water, as defined by the World Health Organisation, (2017), as water which does not represent any significant risk to health over a lifetime of consumption, including different sensitivities that may occur between life stages. Water is a major source for irrigation which is a vital component of agriculture, in other words, agriculture is the largest user of water in all regions of the world except Europe and North America (FAO, 2002b).

Water is an essential requirement of human and industrial development and the most delicate part of the environment (Amadi *et al.*, 2017). In Nigeria, an estimated 65 million people don't have accessed to safe water (Majuru *et al.*, 2011) and the demand is on a daily increase. Most regions in the world are experiencing acute water shortage or scarcity due to high unmet water demand from most water-using sectors, climate change, rapid industrialization, and failure of the government to keep up with the constant supply of pipe borne water.

Groundwater can be defined as water found in the saturated zone underneath the earth. It is a vital hidden natural resource (Tularam and Krishna, 2009; Lashkaripour and Ghafoori, 2011), and a major source of water of millions of people in sub- Saharan Africa, where modern water supply scheme is almost absent. The estimated amount of groundwater in Nigeria is 6 x 10^{18} m^3 (Rijswijk, 1981) Its quality is intrinsically linked to the chemical properties of the aquifer's geology (Aston, 2000). The impact of anthropogenic activities such as artisanal mining activities on groundwater cannot be ignored. Groundwater, the preferred source of potable water is under a serious threat due to unregulated activities of artisanal mining.

One of the thriving businesses in Shakwata village, a suburb of Minna, Niger State is artisanal mining. Artisanal mining can be defined as an informal, low capital, high-labour intensive, unskilled method to extract valuable minerals. Many youth, women and children in Shakwata have actively embraced artisanal mining as a source of livelihood (plate 1 and 2). Despite the economic gains of artisanal mining, it could have a negative impact on the environment. In gold mining operation, water and soil are the recipients of indiscriminate disposal of toxic waste and gangue of gold ores. The use of gravity concentration method such as panning and sluicing during processing poses health challenges. (Amadi, 2016), as in the case of 2010 Zamfara Lead Poisoning epidemic, which claimed the life of 163 people, out of which 111 children were casualties. Drinking water must especially be low in metals, fluorides, nitrates and nitrites (WHO, 2017); therefore, the assessment of water quality is of high importance because human health requires water that is both safe to drink and palatable. (DWAF, 2006).

Plate I: An artisanal miner sluicing 09˚41'58"06˚31'59"

Plate II: Mine tailings from a mining pit 09˚39'23" 06˚37'13"

Location of the study area

The study was carried out in shakwata village and its environs, bosso local government. The area lies between latitudes 09˚ 38′ 30″ and 09˚ 40′ 0″ n, longitudes 06˚ 36′ 0″ and 07˚ 38′ 30´´ e it is about 10km from minna metropolis. The area is located within the kushaka schist belt which had been intruded by faulted granite, gneiss and quartzite.

Guinea Savannah is the vegetation host to the study area, which is characterized by different species of shrubs and also forest like vegetation along the stream channels (Kogbe, 1989; Ajibade, 1982).

The study area lies in a tropical climatic region; this is characterized by an annual temperature of 27˚C and annual precipitation of 1246mm which start by March and ends by November (climate-data.org, 2019). Two seasons are prevalent in the study area, namely wet season (March to November) and dry season (December to February). The dry season is accompanied by the North-East trade wind (Harmattan wind) which originates from the Sahara desert. The driest month January witnesses precipitation of 1mm (climate-data.org, 2019), whereas September is the wettest month, usually witness precipitation of 263mm (climate-data.org, 2019).

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Geology of the area

The study area lies within the Kushaka Schist Belt aged Kibaran 1159± 70ma. The schist belt with biotite-muscovite.

The rock types mapped included fine to medium grained biotite granite, gneiss and amphibolites 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; Ganduet al., 1986; Olayinka, 1992; Obaje, 2009;Waziri, 2015). The granites are thus fractured, jointed and deeply weathered in some places.

Materials and Methods

Geological mapping of the was carried with aid of compass/clinometers, hand lens and GPS. Thin section was done at the Nigerian Feological Survey Agency (NGSA) Kaduna while the petrograpgic studied was at the Geology department of the Federal University of Technology (FUT) Minna. Twelve (12) representative water samples were taken in transparent labeled plastic bottles, (2 hand dug wells, 4 stream channels, mining pit, 5 boreholes) to the laboratory for physicochemical analysis. The physicochemical analysis was carried out at the National Regional Water Quality Laboratory, Minna. The physicochemical parameters were analysed and compared with the WHO, NSDWQ recommended standard. Grain size distribution was done on soils taken from the river channel at the Engineering geology lab, FUT, Minna. Hydraulic conductivity was calculated from the grain size distribution curves

Results and Discussion

The rock types revealed from the geological mapping include granites, gneiss and schist with quartzite intrusions. The rocks have been intruded by quartz veins, foliated and faulted (plate 3 and 4). Minerals observed from the petrographic studied include biotite, muscovite, hornblende, quartz, orthoclase amphibole and opaque minerals. Since the rocks are faulted and fractured, water can easily penetrate through the rock and cause chemical break down of the minerals that could eventually leach in to the groundwater.

Plate III: A sinistral fault 09 ˚38' 37", 06˚ 37' 0"

Table one is the result of hydraulic conductivity as calculated from the grain size distribution curves. Table

Plate IV: Foliated schist 9˚38'4 "6˚37'5"

1 shows that the hydraulic conductivity of the studied soils vary from 4.43 x 10^{-2} cm/s to 3.00 x 10^{-1}cm/s. Conductivity values of this range according to Macauley (2008) are high. This means that the soils in the river channel are permeable and could aid in the downward movement of the materials discharged from the mines (figure 2).

Table 1: Hydraulic conductivity (k) of soils from a river channel in the study area

SOIL PROFILE A1	4.43 x 10^{-2} cm/s
SOIL 3. HORIZON A	3.42 x 10^{-2} cm/s
SOIL HORIZON B	8.86 x 10^{-2}cm/s
SOIL PROFILE C	3.00 x 10^{-1}cm/s

The physical parameters compared with the NSDQW, WHO standards are presented on table 2.

From table 2, the pH ranges between a value of 6.16 and of 7.47, indicating a slightly acidic to mildly alkaline nature of the water in the study area. pH plays an important role in chemical reactions, solubility and toxicity of metal. Table 2 also reveal a turbidity range between 0 and 8,300 NTU, conductivity between 64 and 1,700 µ/cm and a TDS range between 42 and 1,105mg/l. The lowest pH value observed at borehole location 4 (BH4) can be as a result of mine water drainage which could have infiltrated through the soil into the groundwater. Turbidity ranges from 0 NTU to 8,300 NTU. The cause of the high turbidity observed at stream channel 1 (SC1) could be as a result of run off of mine water from the mining site due to its proximity to the site, and presence of silty clay, colloidal inorganic

Table 2: Physical parameters compared with the NSDQW, WHO standards

PHYSICAL PARAMETERS	W1	W2	SC1	SC2	SC3	SC4	MP	BH1	BH2	BH3	BH4	BH	NSDWQ(2015)	WHO(2017)
pH	6.56	6.5	7.03	6.69	6.72	6.83	7.47	6.65	6.6	6.6	6.16	6.43	6.5-8.5	6.5-8.5
Turbidity(NTU)	4	0	8300	18	16	22	5570	9	11	6	8	7	5	5
EC(µ/cm)	482	1700	138	486	474	242	64	814	546	496	448	510	1000	NS
TDS(mg/l)	313	1105	90	316	308	157	42	529	355	322	291	332	500	600-1000

Table 3: Chemical parameters is compared with NSDWQ and WHO

Heavy metals	W1	W2	SC1	SC2	SC3	SC4	MP	BH1	BH2	BH3	BH4	BH6	NSDWQ (2015)	WHO(2017)
Iron(mg/l)	0.89	0.64	10.1	4.68	3.72	2.29	29.2	1.27	2.09	0.52	0.37	0.18	0.3	NS
Cadmium(mg/l)	0.001	0.001	BDL	BDL	BDL	BDL	0	0	BDL	0	0	0.001	0.003	0.003
Lead(mg/l)	0.002	0.001	0.001	0.001	0	0	0.002	0	0	0	0.002	0.004	0.01	0.01
Manganese(mg/l)	BDL	BDL	BDL	BDL	BDL	BDL	0.08	0.04	0.002	0	0.002	0.001	0.2	NS
Zinc (mg/l)	0.002	0.003	BDL	BDL	BDL	BDL	0.015	0.009	0.013	0.028	0.064	0.029	3	4
Chromium(mg/l)	0.03	0.01	4	0.04	0.03	10	0.01	0.02	0.01	0	0	0	0.05	20
Arsenic(mg/l)	0	0	0	0	0	0	0	0	0	0	0	0	0.01	0.01
Copper(mg/l)	BDL	BDL	0.005	BDL	0	0	0.004	0	0	0.001	0.006	0.005	1	2

Table 4: concentration of other parameters compared with NSDWQ and WHO

CHEMICAL PARAMETERS	W1	W2	SC1	SC2	SC3	SC4	MP	BH1	BH2	BH3	BH4	BH6	NSDWQ(2015)	WHO (2017)
Chloride(mg/l)	7.99	8.19	49.9	18.9	6.99	8.99	49.99	9.99	15.9	19.9	12.9	26.9	250	250
Nitrite(mg/l)	0.009	0.096	0.018	0.061	0.085	0.01	0.505	0.02	0.191	0.024	0.027	0.017	0.2	1
Bicarbonate(mg/l)	119	216	300	158	127	72	390	246	155	88	126	82	NS	NS
Sodium(mg/l)	11	85	15	28	25	10	15	21	26	34	19	35	200	200
Magnesium(mg/l)	5.37	39.5	9.76	1.46	2.44	1.95	9.76	27.8	12.7	7.81	7.81	1.46	20	NS
Sulphate(mg/l)	4	27	200	9	7	3	220	1	3	3	3	3	100	NS
Potassium(mg/l)	3	66	6	5	4	1	4	2	1	2	1	1	NS	NS
Carbonate (mg/l)	0	0	0	0	0	0	0	0	0	0	0	0	NS	NS
Calcium(mg/l)	31.2	17.6	24.1	24.5	29.7	12.8	40.1	22.5	20.9	18.5	22.5	22.5	NS	NS

particles (Omonona *et al*., 2019) Turbidity is an essential physicochemical marker of the presence of the possible presence of pollutant that are of concern to human health (Omonona *et al*., 2019). The high value of electric conductivity (1,700µ/cm) which is above the permissible limit of NSDWQ at hand dug well W1 could be attributed to the presence of some elements in the water body. All water samples are within the permissible limit by WHO as it is not stated. The impact of high TDS in hand dug well (W2) could impact unpleasant taste on the water.

Result of the chemical parameters is compared with NSDWQ and WHO and shown on table 3

Table 3 shows that the concentration of manganese ranges from 0.00mg/l to 0.08 mg/l which is within the permissible limit of NSDWQ and WHO. Concentration of zinc, sodium, arsenic, copper, lead, potassium, cadmium are all within the permissible limit of NSDWQ, WHO .

Table 3 further show that the concentration of chromium varies from 0.001 mg/l to 10mg/l and is highest in the mine pit (MP). This is above the permissible limit of NSDWQ. This is worrisome because Chromium is toxic and carcinogenic. It exists in several forms (Cr III and Cr VI). Hexavalent Chromium (Cr VI) is the most toxic form of chromium as it damages DNA. It is highly soluble and usually occur in site that are contaminated. Another heavy metal that exceeded the permissible limit of NSDWQ and WHO is Iron (table 3). The concentration of iron is highest at the mine pit (MP) having a value of 29.2 mg/l. Concentration of ion of such magnitude can cause brown coloration in water and textiles (Adeniyi and Waziri, 2017). The excess magnesium concentration recorded at W2, BH1 signifies that the water is hard and can contribute in making the water in Shakwatu and environs hard. This can make the inhabitants of the area to spend more money on detergent.

Nitrite has concentration values within 0.001 and 0.505 mg/l (table 4) with the highest concentration at the mine pit (MP). The high concentration at the mine pit above the permissible limit can be associated with denitrification process by bacteria commonly found in stagnant water. Denitrification is a common feature associated with mining pits because indiscriminate use of explosives for blasting of rock, in other words, common explosives used at mine sites contains large percentage of nitrogen compounds (Morin and Hutt,2008), this compounds' are released into the groundwater bodies during mining operations. All other elements are within the permissible limit by NSDWQ, WHO.

Hydrochemical Facies

The water from Shakwata and environs was classified based on Piper (1944) and presented in figure 1.

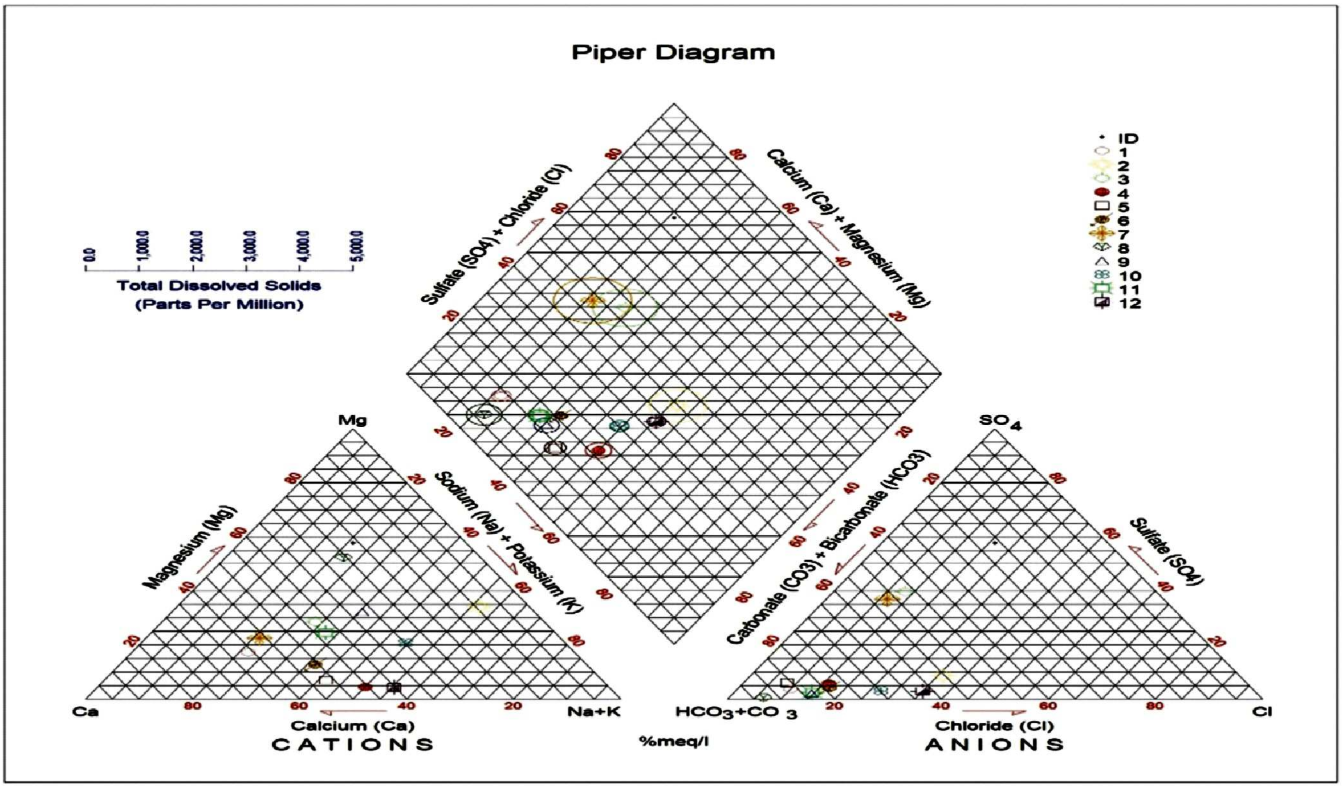

Fig. 1: Water types within Shakwatu and environs

Figure 1 shows that the dominant water type in Shakwatu and environs is Ca –HCO_3 followed by $NaHCO_3$ and $CaMgHCO_3$. This diagram implies that weak acids dominates strong acids, alkali earth ion dominates alkali ions.

Conclusion

The study area is made up of amphibolite schist,quartzite, granites and granitic gneiss. The mineralogical composition of these rocksinclude quartz,amphibole, biotite, plagioclase. High hydraulic conductivity derived from the grain size distribution (4.43 x 10^{-2} cm/s to 3.00 x 10^{-1}cm/s) , imply high permeability in the soil , and this could aid the downward transportation of mine tailings into the groundwater. Low pH recorded in most of the water bodies aids the solubility, increases the movement of ions and accounts for high turbidity . Electric Conductivity (EC) high value at hand dug well W1 could be as a result of metallic presence. Among all the heavy metals element, chromium concentration poses a threat to human life. The value in the study area is well above the permissible limit of NSDWQ ,2015, WHO, 2017. The dominant water facies is $CaHCO_3$, $CaNaHCO_{3,}$ $CaMgHCO_3$, implying the dominance of alkali earth ion over alkali ion.

Among other chemical parameters, Nitrite is not within the permissible limit, and it can not be ignored because of its negative impact on DNA when ingested either by man or animals. Most of the parameters tested and compared with NSDWQ and WHO are all within the permissible limit. The high concentration of iron in the study area could lead to rusting of boreholes and borehole installation accessories, such as riser pipes, and coloration for cloths.

This menace can be regulated if the government look into the issue of illegal mining activities, and curtail their nefarious activities.

It is recommeded that the activities of the artisanal mine must be bought under control and checked.

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Geophysical and Geotechnical Investigation of Building Failure in a Typical Basement Complex Environment – A Case Study

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Abstract
A building located within the Basement Complex of the ancient town of Ile – Ife, Southwestern Nigeria was observed to have failed due to excessive total and differential settlements. The failure was investigated using geophysical and geotechnical methods. The geophysical investigation involved 2-D electrical resistivity imaging (ERI) technique using the dipole-dipole array along four traverses. The geotechnical method involved cone penetration test (CPT) using a 2.5 ton static penetrometer machine. Analyses of the ERI data were made using the DIPROfWIN software for the pseudo-inversion while the CPT data were interpreted for lithology using standard chart. The results showed that the topsoil, about 1.0 m thick, is composed of sandy clay/clay characterized by cone resistance (q_c) of 0.2 – 2.0 MPa and resistivity of 75 – 200 Ωm. The underlying clayey weathered layer, which constitutes the shallow foundation soil, is characterized by thickness of 4 – >10 m; q_c of 0.2 – 1.0 MPa; resistivity of 25 – 75 Ωm and estimated consolidation settlement of 200 – 500 mm. The basal, saprock/fresh bedrock, layer is characterized by q_c of >8.0 MPa and resistivity of 100 – 1000 Ωm. Thus, the subsoil is characterized by variably thick incompetent clayey weathered layer within which the shallow foundation was placed; hence the excessive total settlement.

***Keywords**: Basement Complex, Building Failure, Electrical Resistivity Imaging, Cone Penetration Test, Excessive Settlement*

Introduction

The incessant incidence of foundation failure of structures is becoming alarming in Nigeria. This failure has been attributed to a number of factors such as inadequate information about the soil and the subsurface geological material, salinity, poor foundation design and poor building materials (Fatoba *et al.*, 2010). Building failure in the form of excessive total settlement and intolerable differential settlement or both are attributed to highly compressible foundation soils. While these types of building failure are common in the sedimentary terrain underlain by recent transported soils in the form of soft clays and loose sands; they are uncommon in the Basement Complex terrain where residual soils and partially weathered rocks constitute the foundation soils.

Conventional pre- and post-construction investigations of foundation soils, which usually involve shell and auger exploratory borings along with cone penetration tests, are generally effective in areas underlain by transported soils that are usually characterized by fairly uniform stratification within space of hundreds of meters. However, this technique may not be effective in the Basement Complex terrain characterized by highly heterogeneous subsoil sequence within short distance. In such terrain, integration of geophysical methods into conventional geotechnical investigation program have been found to be useful (Olorunfemi and Mesida, 1987; Adepelumi and Olorunfemi, 2000b; Olorunfemi et al., 2000b; Akintorinwa et al., 2011, and Salami et al., 2012).

A building located within the Basement Complex of the ancient town of Ile – Ife was observed to have failed due to excessive total settlement. The aim of this study was to investigate the cause of the failure with the objective of evaluating the competence of subsurface sequence in the vicinity of the building as foundation elements. The study area (Fig. 1) lies within (UTM 31N) Northings 828491 – 828543 mN and Eastings 668473 – 668532 mE (Fig. 1). The Electrical Resistivity method involving Resistivity Imaging using the dipole-dipole configuration and geotechnical investigation involving Cone Penetration Test (CPT) were conducted for the investigation.

Geology of Study Area

The study area is part of the Ife-Ilesa schist belt which consists of three major units: the Ilesa amphibolite Complex which occurs as lenticular bodies, the Ilesa metaclastics made up of sheared biotite schists, and the Effon quartzitic sequence which occurs as massive quartzite, schistose quartzite and quartz schist. The study area is underlain by pegmatized schist (Fig. 2) which has been severely weathered. The pegmatized schist is generally dark grey in colour with texture ranging from medium to coarse grain. Its mineralogical

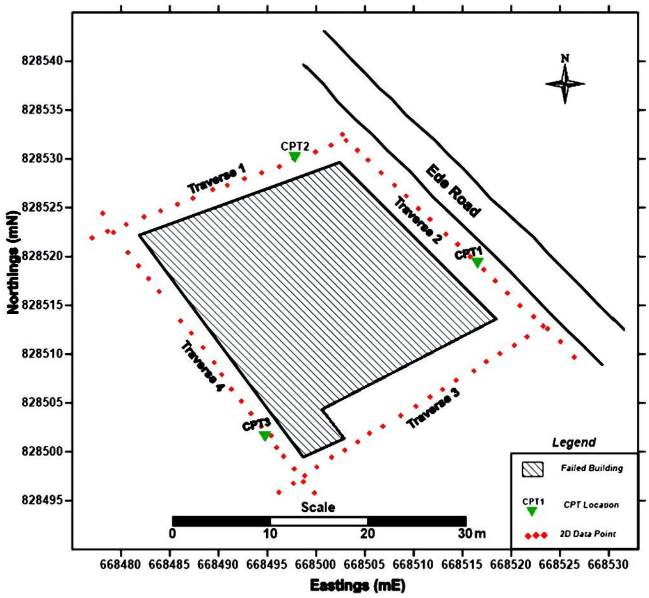

Fig. 1: Map of the study area showing the test positions

composition includes biotite, quartz, muscovite, plagioclase and potassium feldspars which makes it highly susceptible to weathering.

Methodology

Geophysical Investigation

The geophysical investigation employed the Electrical Resistivity method adopting the 2-D Electrical Resistivity Imaging (ERI) technique using the dipole-dipole array along four traverses (TR1 – 4) of 30 – 60 m lengths that were positioned along the perimeter of the building. Electrode spacing a = 2 m and dipole spacing n = 1 to 5. The data acquisition was done using the ABEM 300C Terrameter.

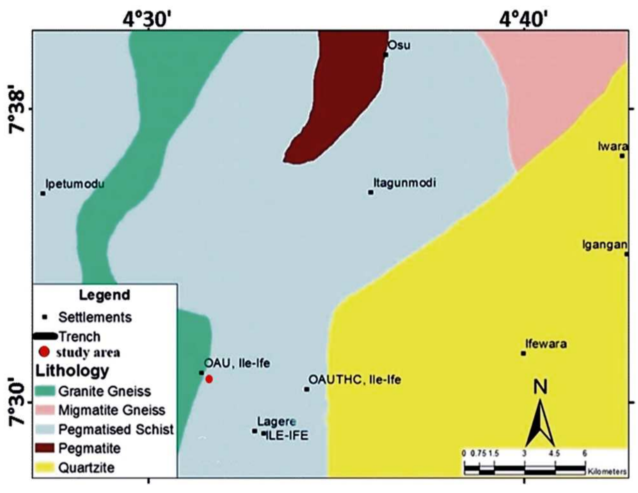

Fig. 2: Geological map of study area and its environs (Olorunfemi *et. al.,* 2015)

Quantitative and qualitative analysis were made using the DIPROfWIN software for the pseudo-inversion.

Geotechnical Investigation

The Cone Penetration Test (CPT) was used to determine the strength of the subsurface sequence based on the measured values of cone penetration resistance (q_c) at various depths. The CPT was performed at three positions within the study area using a 2.5 ton Dutch cone penetration machine. The test was carried out by securing the winch frame to the ground by means of anchors. These anchors provided the necessary power to push the cone into the ground. The cone and the rod (1 m) were pushed together into the ground with the cone pushed ahead of the rod at a uniform rate of about 2 cm/s. The resistances to penetration of the cone, registered on the pressure gauge connected to the pressure capsule, were recorded at depth intervals of 0.25 m from ground surface to the points of refusal i.e. points where the machine anchor legs were lifted or pulled out of the subsurface. The procedure described above was then repeated for subsequent location tests.

The CPT data were processed by plotting the cone resistances against depth at each location point using the Microsoft Excel software. The layer sequences were interpreted from the variation of the values of the cone resistance with depth. On the basis of the expected resistance contrast between the various layers, inflection points of the generated penetrometer curves were interpreted as the interface between the different lithologies.

Results and Discussion

Electrical Resistivity Survey

The 2-D resistivity structures along the four traverses depict the subsurface resistivity distributions (Figs. 3 - 6).

Traverse 1

The resistivity structure (Fig. 3) shows subsoil sequence along the Northwestern flank of the site as having resistivity values varying from 35 Ω m to 330 Ωm. The topsoil with predominantly yellow colour, occurring to about 1.0 m depth, has resistivity values that generally vary from 102 – 167 Ωm, with minor portion in green colour having low resistivity values of 35 – 67 Ωm. This portrays heterogeneous soils composed mainly of sandy clay and minor clay. The underlying layer in predominantly green colour, occurring from about 1.0 to 4.3 m depth, is characterized by resistivity values ranging from 48 to 95 Ωm, which is typical of sandy clay weathered layer. The basal layer in red/purple colour, characterized by resistivity values of 110 – 330 Ωm, is regarded as the saprock/fresh bedrock. Depth to the fresh bedrock varies from about 2 m in the NE to about 4.3 m in the SW end of the traverse.

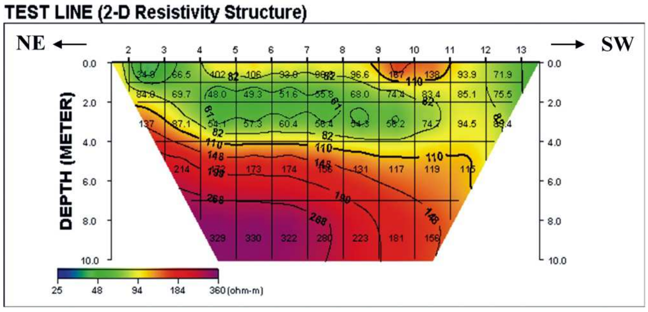

Fig. 3: 2-D Resistivity Structure along Traverse 1

Traverse 2

The resistivity structure (Fig. 4) shows that the subsoil sequence along the Northeastern flank of the site has resistivity values varying from 28 Ωm to 905 Ωm. The topsoil with predominantly green colour, occurring to about 1.0 m depth, has resistivity values that vary from 51 – 107 Ωm, and is regarded as comprising sandy clay material. The underlying layer in blue/green colour, occurring from about 1.0 to 3.0 m depth, is predominantly characterized by resistivity values of 28 – 55 Ωm, hence regarded as the clayey weathered layer. The third layer, ocurring from 3.0 – 4.5 m in green/yellow colour with resistivity values of 55 – 122 Ωm is regarded as clayey sand weathered layer. The basal layer in yellow/red/purple colour, characterized by resistivity values of 122 - 905 Ωm, is regarded as the saprock/fresh bedrock. Depth to the fresh bedrock varies from about 4 m at the origin in the NW to about 4.5 m at the SE end of the traverse.

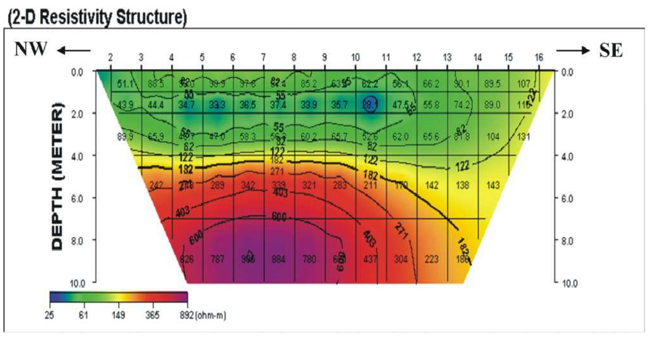

Fig. 4: 2-D Resistivity Structure along Traverse 2

Traverse 3

The resistivity structure (Fig. 5) depicts the subsoil sequence along the Southeastern flank of the site as having resistivity values of 27 Ωm to 271 Ωm. The topsoil in orange/yellow/green colour, occurring to about 1.0 m depth, has resistivity values that generally vary from 42 – 115 Ωm, is regarded as residual soils composed mainly of clay and sandy clay. The underlying layer in blue/green colour, occurring from about 1.0 to 6.0 m depth, with resistivity values of 22 – 75 Ωm is interpreted as clayey weathered layer. The basal layer in yellow/red/purple colour, characterized by resistivity values of 77 – 271 Ωm, is regarded as the saprock/fresh bedrock. Depth to the fresh bedrock varies from 4.0 m at electrode position 6 to beyond 7.0m between electrode positions 15 and 16 at the SW end of the traverse.

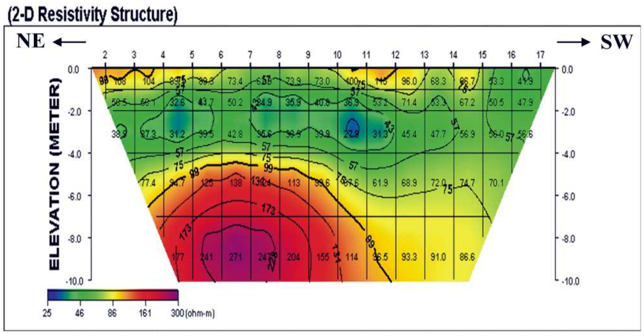

Fig. 5: 2-D Resistivity Structure along Traverse 3

Traverse 4

The resistivity structure (Fig. 6) shows that the subsoil sequence along the southwestern flank of the site is characterized by low resistivity varying from 6 – 138 Ωm. The topsoil in green/yellow/orange colour, occurring to between 1.0 and 2.0 m depth, has resistivity values that generally vary from 48 – 138 Ωm, hence interpreted as residual soils composed mainly of clay and sandy clay. The underlying layer in bluish green/green colour, occurring to about 4.0 m depth at electrode position 9, with resistivity values of 27 – 60 Ωm is interpreted as clayey weathered layer. The underlying layer, in predominantly blue colour, characterized by low resistivity values of 6 – 33 Ωm is considered to be soft clayey material. The fresh bedrock was not delineated beneath this traverse.

Geotechnical Test

Results of the cone penetration tests are presented as plots of cone resistance (q_c) against depth (Figs. 7 - 9).

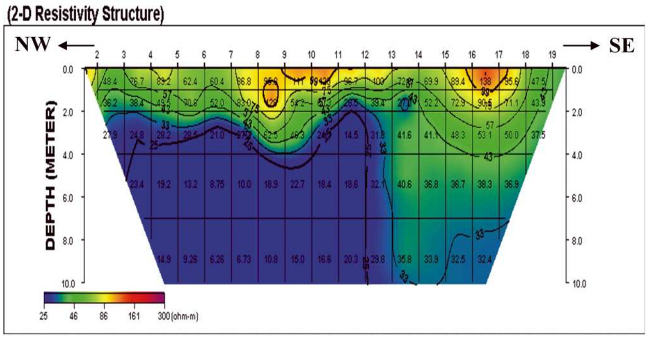

Fig. 6: 2-D Resistivity Structure along Traverse 4

For the purpose of lithological interpretation, the q_c values were correlated with Schertmann chart (1978). The friction ratio, which could not be measured with 2.5 ton penetrometer machine, is taken to vary from 2 – 5 based on classification of Look (2007).

Analysis of the results show that from ground surface to depth of 2.5 m, CPT 1 and CPT 3 are generally characterized by low q_c values of 0.196 MPa (2.0 kgf/cm^2) indicating incompetent subsoil composed of soft clays. Corresponding section beneath CPT 2 to 2.0 m is characterized by relatively higher q_c values of 0.196 – 1.96 MPa (2.0 to 20 kgf/cm^2), which indicates fairly competent subsoil composed of sandy/silty clays or stiff clay.

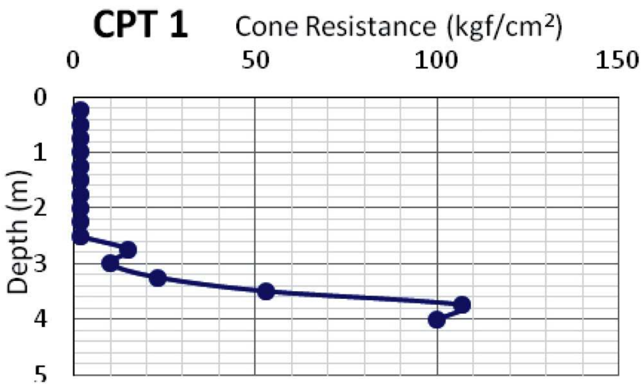

Fig. 7: Cone Resistance Plot on Traverse 2

From 2.25/2.75 m to 3.5 m, the q_c values for the three test points generally varies from 1.47 – 7.10 MPa (15 to 72 kgf/cm^2), which is regarded as moderately competent subsoil composed of sandy and silty clays. The last layer penetrated by the cone, from 3.5 – 4.0 m has q_c values that generally vary from 8.82 – 10.5 MPa (90 - 107 kgf/cm^2), which is regarded as competent subsoil saprock composed of coarse-grained clayey sand.

Synthesis of Results

The composite resistivity structure of the site was

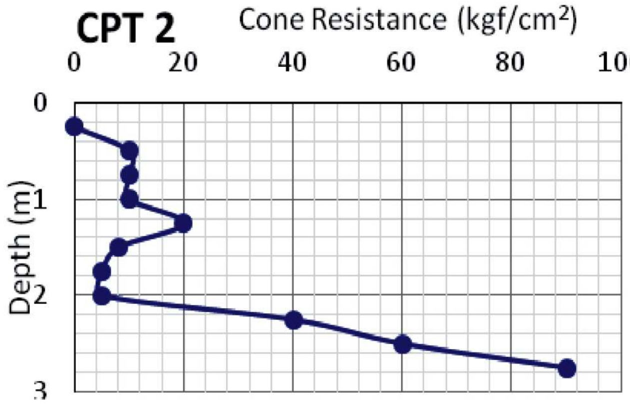

Fig. 9: Cone Resistance Plot on Traverse 4

generated by integration of the 2-D along Traverses 1 – 4, with the aid of Surfer 11 software, and was used to determine the generalized resistivity of the subsoils beneath the failed building at different depths.

The results (Figs. 10 - 14) show that, beneath the investigated building, from the ground surface to 1.0 m depth (Fig. 10), the subsoil is characterized by resistivity values varying from 75 - 200 Ωmm and q_c varying from 0.196 to 1.96 MPa. The layer is thus regarded as dry portion of clays and sandy/silty clays delineated by the CPT.

The underlying subsoil to depth of 4 m is characterized by resistivity values varying from 25 - 75 Ωm, with resistivity values decreasing with depth (Figs. 11 and 12).

This horizon is considered to be clayey weathered layer occurring as mottled zone of saprolite and it corresponds to the soft clay and stiff silty clay delineated by the CPT to depth of 2.5 m.

At depth of 4 to 7 m, the subsoil is characterized predominantly by resistivity values of 100 to 500 Ωm (Fig. 13), which is typical of pallid zone of saprolite and corresponds to the sandy and silty clays delineated by the CPT. The basal layer at depth of 7 to 10 m characterized mainly by resistivity values of 100 - 1000 Ωm portrays occurrence of saprock and fresh bedrock (Fig. 14). The southwestern portion is however characterized by subsoil with low resistivity values to depths beyond 10 m. This suggests occurrence of buried river channel in that part of the area.

The clayey subsoil constitutes the horizon for shallow foundation. Undrained shear strength (C_u) parameters of the horizon was estimated from q_c values (kPa) based on Look (2007) relationship in Eq. (1).

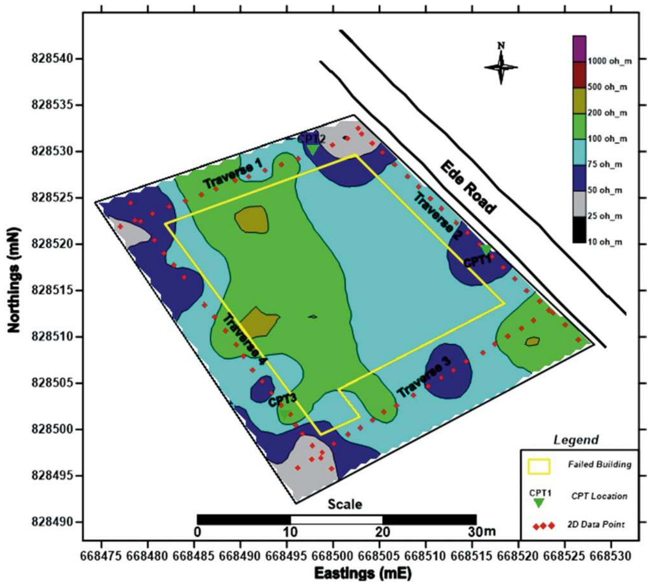

Fig. 10: Resistivity Depth Slice at 0 – 1.0 m

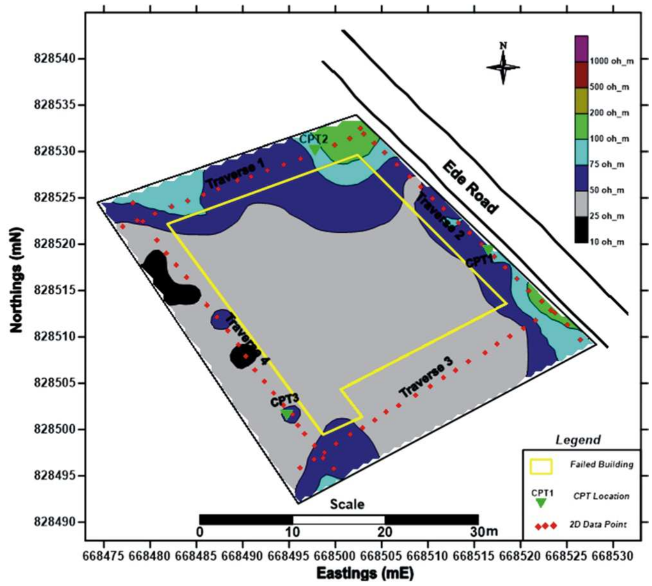

Fig. 12: Resistivity Depth Slice at 2.0 – 4.0 m

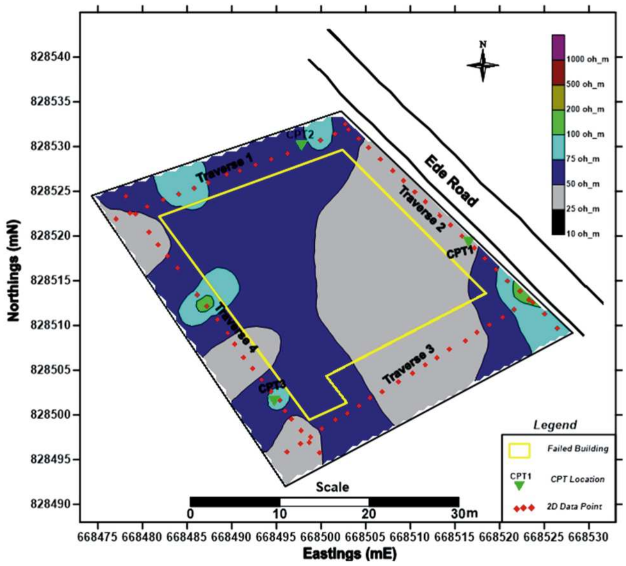

Fig. 11: Resistivity Depth Slice at 1.0 – 2.0 m

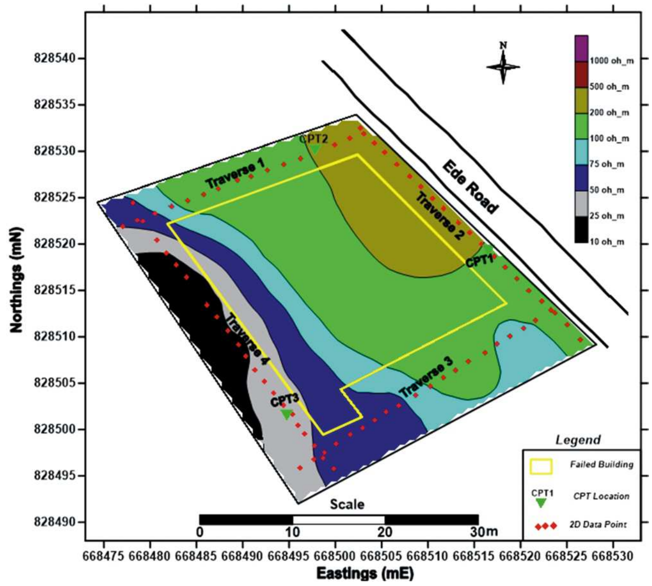

Fig. 13: Resistivity Depth Slice at 4.0 – 7.0 m

$$C_u = \frac{q_c}{N_k} \quad \text{...(1)}$$

(q_c values are in kPa, N_k = cone factor = 18).

The results show that this horizon is generally characterized by low undrained shear of 11 kPa except in the northwestern portion of the site that has undrained shear of 54 kPa.

In addition, the clayey horizon has thickness varying from 4 m in the NE to >10 m in the SW portiom of the site. Consolidation settlement (S_c) of the horizon was estimated from constrained modulus (***M***), which is the inverse of coefficient of volume change (M_v) as in Eq. (2).

$$S_c = \frac{1}{M} . H . \Delta\sigma \quad \text{...(2)}$$

H = thickness of compressible layer, Δσ = Added load.

Also, ***M*** was estimated from q_c based on Sanglerat, 1972 as shown in Eq. (3).

$$\boldsymbol{M = \alpha_m . q_c} \quad \text{...(3)}$$

(for qc < 0.7, α_m = coefficient of constraint modulus has a range of $3 < \alpha_m < 8$).

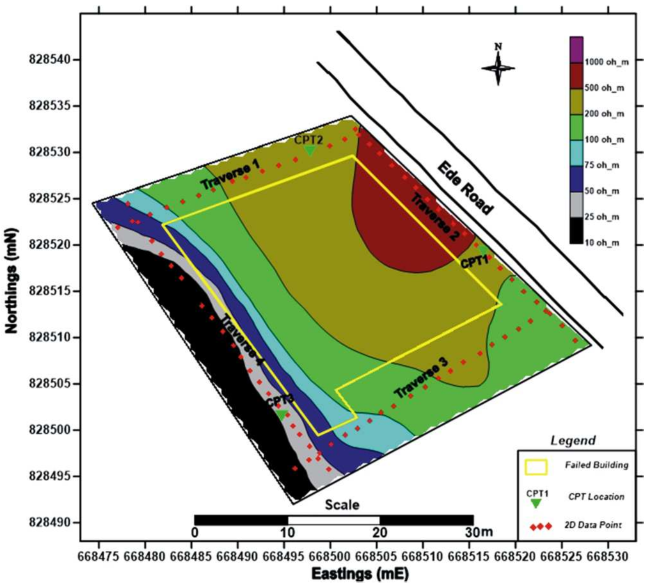

Fig. 14: Resistivity Depth Slice at 7.0 – 10.0 m

The lower value of α_m = 3.0 was adopted for this study due to the low q_c value of 0.2 MPa. Furthermore, the failed building has two floors (ground floor and first floor), hence a loading intensity of 30 kPa was adopted. The estimated consolidation settlement varied from 200 mm in the NE section to 510 mm in the southern section of the building.

Conclusion

The subsoil within the study area is characterized by an incompetent clayey horizon of variable thickness within which the shallow foundation was placed; hence the excessive total settlement of the building.

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Geotechnical Investigation of Shale Derived Soil from Ojo Along Abeokuta-Ifo Road

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Abstract
Samples of shale derived soil were analysed with the view of assessing their suitability for road construction. Soil samples were mottled reddish brown and grey stiff sandy silty clay. Geotechnical tests offend used for evaluation of soils for road construction were conducted in according to standard procedures. The soil samples were well graded with the percentage of fines more than the coarse fractions. Liquid limit (LL), plastic limit (PL) and plasticity index (PI) ranges were 40.52.96%, 24.3-29.38% and 17.82-24.46% respectively. Bulk density ranges between 1810.86 Kg/m^3 – $1962Kg/m^3$ and maximum dry density (MDD) of $1611.92Kg/m^3$ - $1790Kg/m^3$ with optimum moisture content (OMC) of 17.21% - 22 82%. California Bearing Ratio (CBR) range of 14.17KN-34.94KN for unsoaked and 7.24KN -14.84KN for soaked indicating 44.25% - 72.28% reduction in strength were obtained. Permeability (k) of the soil ranges from $9.64x10^{-5}$cm/sec - $1.07x10^{-4}$cm/sec. Amount of fines, LL, and PI obtained were higher than the maximum value of 35%, 35% and 12% respectively with CBR values below the recommended minimum of 30% and 80% for sub-base and base material respectively as stipulated by the Federal Ministry of Work and Housing (FMWH), 1997.

***Keywords**: Optimum Moisture Content, California Bearing Ratio, Plasticity Index and Permeability*

Introduction

In recent times, emphasis is being placed on environmental hazard and safety measures (Afolayan and Abidoye, 2017: Agbonkhese*, et al.,* 2013). It is also important that civil engineers should not only be interested in the aesthetic but on longevity of structure safety which protect life and property. These can only be achieved through detailed soil investigation.

Aim

The aim of this study is to investigate the geological and geotechnical properties of lateritic soils derived from shale from Ojo. This is with the viewed to determining their suitability as construction material (subbase) below a flexible highway pavement.

Location and Accessibility

Undisturbed and disturbed soil samples were taken from Ojo located along Abeokuta and Ifo road within latitude N 07.04295^0 and longitude E 003.24226^0 at 81m above mean sea level. The sampling locations are shown in the map of the area (Figures 1) and were easily accessed through network of footpaths, tarred and untarred road linked to minor and major roads.

Climate Characteristics and Vegetation

The study area is within humid tropical region, characterized by wet and dry season with relative humidity variation of 70%-95% between January and July respectively. The annual rainfall ranged between 150mm and 3000mm. The mean annual rainfall in this area is 1258.9mm. The rainy season last March to October while November to February is the dry season. This area has temperature ranged between 21 and 34^{o}C with mean annual of 26.6^{0}C and mean maximum of 28.8^{0}C in February and mean minimum in August being 24.5^{0}C.

The vegetation of Southwestern Nigeria is made up of fresh water swamp and mangrove forest depicted by tropical vegetation of the rain forest type with tall grasses, herbs and small trees (shrub). (Feleyimu *et al.,* 2013; Meteorological Station of the University of Ibadan and the International Institute of Tropical Agriculture (IITA)).

Geology of Ojo

Ojo is on the sedimentary terrain of Southwestern Nigeria (SWN), mostly of recent sediment of Ilaro formation which consist of continental fluviatile sandstone of coarse angular and poorly sorted with significant amount of clay-shale fraction. (Ayedun *et.al.,* 2011)

Method

The layout for soil sampling was a dimension of $15m^2$ by $15m^2$ covering an area of $225m^2$ with the soil sample taken from the surface to a depth of one foot at 5m

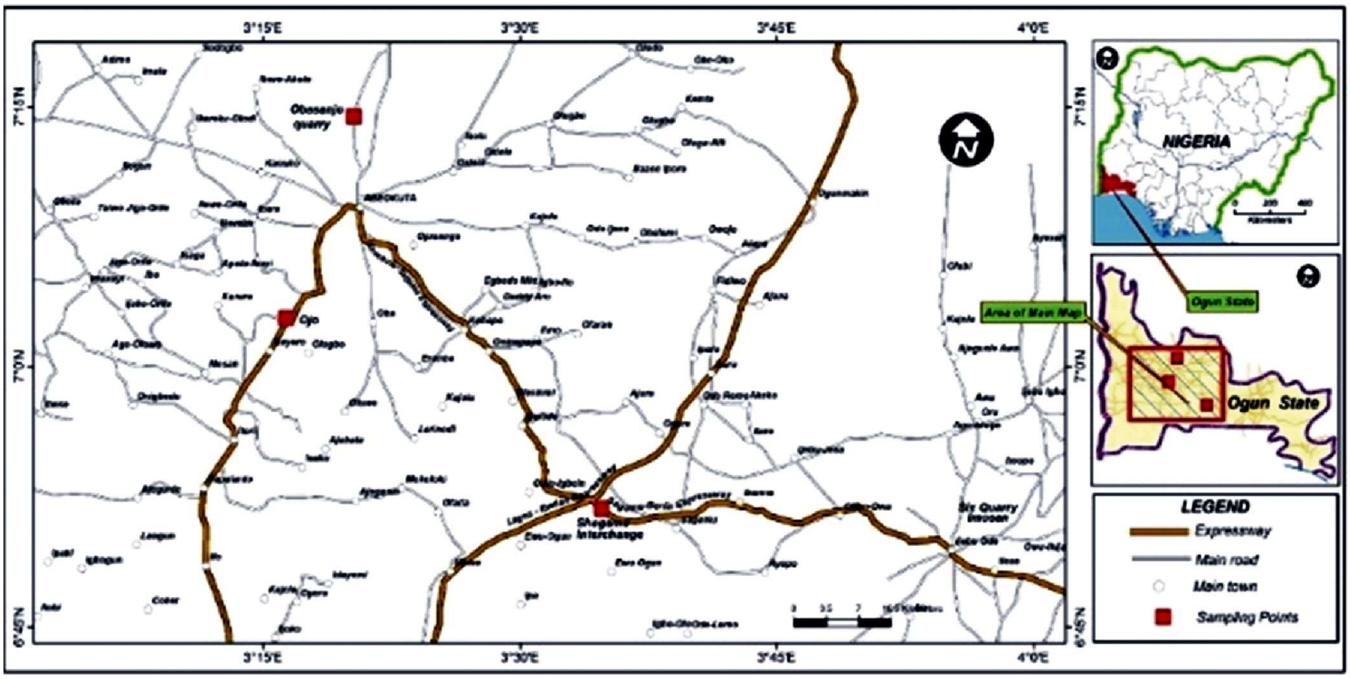

Fig. 1: Section of Ogun state map showing study location adapted from Ministry of Work and Housing, survey division Abeokuta, Ogun State 1987

interval apart. Undisturbed soil samples were taken with the core cutter while the disturbed samples were collected with the help of a stainless scooper totaling thirty-two coded as SHS1-16. Geotechnical tests offend used for evaluation of soils for road construct were conducted in accordance with British standard BS- 5930 (1981) and ASTM D-402-90, (ASTM, 1985) with some modification brought about by peculiar characteristics of tropical soils. The following Index and engineering properties of soil were determined; Specific gravity of grains (G_s), Natural moisture content (NMC), Grain-size distribution, Consistency limits, Bulk density, Maximum dry density (MDD) and Optimum Moisture Content (OMC), California Bearing Ratio (CBR) and Permeability (k).

Result and Discussion

From field observations, the shale derived soil samples are mottled reddish brown and grey stiff sandy silty clay. All the index and engineering test data of Ojo soil sample are in (Table 1).

Specific gravity of grains (Gs) ranges between 2.5 and 2.65, This range according to Ramamurthy and Sitharam (2005) is that for sand. The range of NMC is 13.16% - 17.02%. Sieve and hydrometer analyses show that the grain size range for Clay is 6% – 26%, amount of silt range between 33 to 64 %. The range of amount of sand fraction in the soils is 17- 42%, while gravel fraction range in amount between 4 and 15%. The percentage of the fines content of this soil samples ranges between 45-77% and are more than coarse fraction, thus will have overriding effect on the geotechnical property of the soil. The grain size distribution curves indicated a well graded soil (Figure 2). Consistency limits and indices values revealed the following ranges: Liquid limit values ranges between 40.00% and 52.96% showing intermediate to slightly high plasticity according to Sowers and Sowers (1970) with PL range of 24.3 - 29.38% and plasticity (PI) range of 20.07 and 24.46%. Other parameters obtained are consistency limit (I_c) that ranges between 1.128 and 1.362 and liquidity index (LI) of (-0.760) - (- 0.354), this is the range for soil that will be semi plastic and unstable.

The plot of plasticity index and liquid limit in Casagrande chart fall close to the A-line, (some below and above) indicating the fines are probable mixture of clays and silts (Figure 3). Compressibility values obtained varies with consolidation pressure and ranges between 0.087m^2/MN -1.03m^2/MN implying soils of very low to very high compressibility (interpretation after McKinlay, (1996). Coefficient of consolidation C_v of the soil also varies with consolidation pressure and values obtained corresponds to that of sandy clays according to Carter and Bentley (1981) as C_v ranges 4.68x$10^2 m^2$/yr. -33.31x$10^2 m^2$/yr. The bulk density ranges between 1810.86 Kg/m^3 – 1962Kg/m^3. This is the range of bulk density for silts and clays soil (Holtz and

Kovacs (1981). Compaction result of shale derived soils ranges between 1611.92Kg/m^3 - 1790Kg/m^3 for MDD with OMC of 17.21% - 22 82%.(Figure4). The range of CBR is between 14.17KN-34.94KN for unsoaked and 7.24KN -14.84KN for soaked samples. The above CBR value show 44.25% - 72.28% reduction in strength. California Bearing Ratio value correlate strongly positive with bulk density and MDD.

Permeability values ranges between 9.64x10^{-5}cm/sec - 1.07x10^{-4}cm/sec. This indicate very low permeability value meant for silty to dirty sand.

Table 1: Values of the geotechnical parameters of shale the derived soil

Code	Gs	% C	% Si	% F	% S	% G	LL (%)	PL (%)	PI	BD Kg/m^3	MDD Kg/m^3	OMC %	CBR (%)
SHS 1	2.6	18	42	60	25	15	46.2	25.84	20.36	1962	1790	17.21	34.94
SHS 3	2.55	21	49	70	21	9	52.21	27.75	24.46	1889	1690	20.41	14.17
SHS 4	2.6	19	33	52	40	8	49.19	26.26	22.93	1900	1692	21.2	30.96
SHS 5	2.6	21	47	68	25	7	50.48	28.85	21.63	1922	1740	20.4	25.63
SHS 6	2.55	25	43	68	25	7	51.00	29.04	21.96	1810	1612	22.82	19.24
SHS 9	2.65	9	36	45	42	13	40.00	24.3	20.3	1828	1640	22.6	15.03
SHS 11	2.5	26	51	77	17	6	49.12	28.39	20.73	1908	1761	18.01	21.64
SHS 12	2.55	6	64	70	22	8	52.96	29.38	23.58	1906	1711	20	27.64
SHS 15	2.65	16	44	60	27	13	47.02	24.86	22.16	1906	1711	20	27.64
SHS 16	2.65	13	42	55	31	4	46.10	26.03	20.07	1925	1740	21.2	23.44

Gs, Gravity of grain; C, Clay; F, fines; Si, Silt; S, Sand; G, Gravel; LL, Liquid Limit; PL, Plastic Limit; PI, Plasticity Index BD, Bulk Density; MDD, Maximum Dry Density; OMC, Optimum Moisture Content; CBR, California Bearing Ratio.

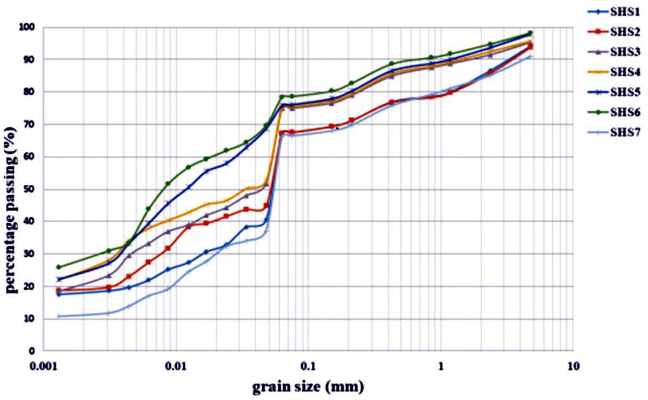

Fig. 2: Gradation curves of the shale derived soil

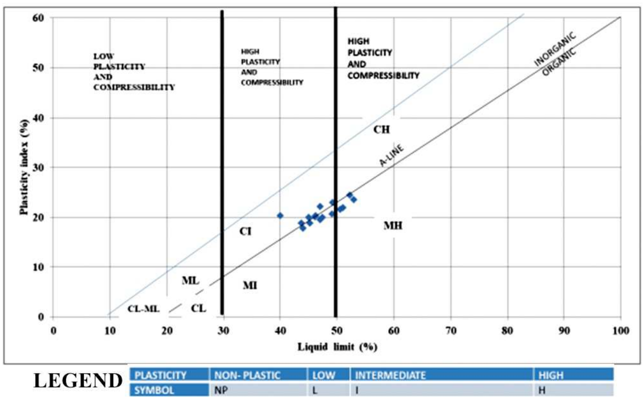

Fig. 3: Casagrande chart of the shale derived soil

Table 2: Ranges of index and engineering parameters of studied soil samples with FMWH (1997) specifications.

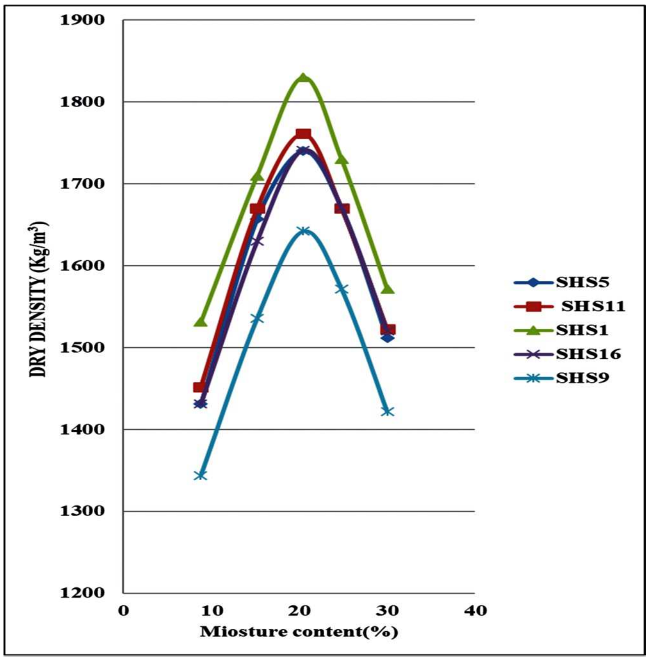

Fig. 4: Compaction curves of the Shale derived soil

Appraisal of Soil for Road Construction

From the ranges of soil geotechnical parameters in (Table 2), it was observed that the amount of fines, the LL, and the PI is higher than the maximum value of 35%, 35% and 12% respectively stipulated by the Federal Ministry of Work and Housing (FMWH), 1997 for road construction. The CBR values obtained were

Table 2: Ranges of index and engineering parameters of studied soil samples with FMWH (1997) specifications

Parameter/ location	Ranges of Parameter	FMWH (1997) Road Specification
(%) LL	40.00–52.96	not greater than 35%
% PI	17.82-24.46	not greater than 12
% Fines	45.00-77.00	not more than 35%
% CBR	14.17–34.94	min 30% for subbase, min 80% for base
CBR% reduction by soaking	44.25-72.28	not more than 20% reduction (Jegede 2000)

also below the recommended minimum CBR value of 30% for sub-base and 80% for base material and displayed high reduction in strength (CBR) due to soaking with little water ingress of 12-24%. Very low permeability portrayed soil that will be prone to soaking. This soil is a poor material for road construction. (therefore, are of sub-standard paving qualities in term of sub-grade)

Conclusion

The grain size distribution of the soil sample indicated that the clay and silt fractions dominate the composition of the soils (the amount of fines is more than 50%) with less coarse fractions. Thus, the fines characteristic will be the over-riding factor regarding the geotechnical properties of the soil. The soil samples are well graded and can be described as silty sandy clay. High plasticity index shows that the soils may be problematic (soil that may have excessive swelling and shrinkage). The consistency index value is less than 2.5 which means the soils are unstable. Casagrande chart plot of the soil samples showed the fine fractions are mixture of clays and silts, that soils possess medium plasticity and compressibility and would be expected to exhibit low to medium swelling potentials. There was significant reduction in strength of the soil (high differences between the unsoaked and the soaked CBR values) due to soaking. The coefficient of permeability of the studies soil were very low. The range of permeability coefficient is for soils of silty sand, dirty sand to silt and fine-grained sandstone. The values are typical of soil which can retain water for a long period. Bulk density and MDD values were high enough to be used for sub-base and sub-grade road materials but OMC values were higher than the stipulated value for soil meant for road construction. Most of the engineering indices were short of the specification by the Federal Ministry of Works and Housing (1997). These soils, therefore, are of sub-standard paving qualities in term of sub-grade. The index and engineering parameters (high reduction in CBR due to soaking by water ingress, excess in amount of fines, high liquid limit, high plastic limit and high plasticity index) obtained conformed to the findings of Ogundipe and Moses, (2008) for soils of South western Nigeria.

Finding show that highway geotechnical parameters of the studied soils are typical of fair to poor subbase-subgrade materials and are marginally suitable for road construction in their natural state and need to be upgraded through stabilisation. The implication is that if the soils are used as sub-base and sub-grade materials for road pavement in their natural state, such road would be expected to have a short live span.

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Section of Ogun state map showing study locations (OJO) Adapted from Ministry of Work and Housing, Survey division Abeokuta, Ogun State 1987)

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Geophysical and Geotechnical Assessment of the Effects of Barikin-Sale Dumpsite on Groundwater Quality, Minna, North-Central Nigeria

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Abstract
Discharge of leachate from refuse dumpsites is a source of groundwater pollution within its immediate environment of location. Geophysical investigation of an uncontrolled open solid waste dumpsite located at Barikin-Sale area of Minna was carried out for possible contamination of groundwater. The investigation is aimed at delineating groundwater contamination due to leachate percolation thereby assessing the quality of groundwater from hand dug wells and boreholes within the dumpsite and the surrounding environment. A total of ten (10) Vertical Electrical Sounding (VES) points with maximum current electrode spacing of 20m and two (2) 2D Subsurface Electrical Imaging were investigated within and outside the dumpsite to assess possible movement of leachate to the area. Schlumberger configuration was used for the VES while the Wenner configuration was used for the subsurface imaging. The VES data were analysed and a maximum of three (3) geoelectric sections were identified; the top soil, weathered basement and the fractured/fresh basement. The obtained apparent resistivity for those layers were between 19.4Ωm and 122.6Ωm for the first layer, 18.2Ωm and 33.4Ωm for the second layer and 93.5Ωm and 166.4Ωm for the third layer respectively. The range of thickness for the first layer is 0.9m and 1.8m and for the second layer are 4.0m and 5.3m respectively. The 2D resistivity data were processed and inverted using the RES2DINV software. The inverse resistivity models of the subsurface from the 2D electrical imaging revealed low resistivity value <20Ωm which is taken to be leachate derived from decomposed waste while these wastes that cannot decompose are occurring as isolated parts with slightly higher resistivity value of >20Ωm. The areas with the highest resistivity value of >100Ωm were further interpreted to be chemical weathering product of crystalline bedrock considered to be regolith. From the estimated hydraulic conductivity (K) from the sieve analysis, the values range between 1.97×10^{-4} and 2.52×10^{-5} m/s. These values are high and they correspond to leachate movement through the interconnected pore spaces of the soil underlying the waste dumpsite. This further clarifies the geophysical investigation results. From the results, it could be concluded that leachates are concentrated within the lower part of the dumpsite; therefore the surrounding environment groundwater sources are vulnerable to leachate contamination from the dumpsite.

Keywords: *Resistivity. Dumpsite, Leachate, Contamination, Groundwater*

Introduction

Disposal of solid waste on land is the most common waste disposal method across Nigeria. Groundwater forms an important part of the water resources across the world particularly in the arid regions. It is used for domestic purposes because it is of high quality and required slight or no treatment before usage. According to Sampat (2001), bacteria, fungi and other biological pollutants are naturally filtered and diluted as the water infiltrate or permeate through the soil. The lackadaisical management and/or disposal of hazardous materials, fresh groundwater supplies are decreased greatly. The problem of environmental contaminations is one of the concerns of earth scientists today, and researchers from other related fields across the world. Indiscriminate disposal of organic waste is detrimental to health because it creates unsanitary environment that have adverse effects for urban residents. Where sanitary facilities are scarce, household solid wastes also tend to be mixed with fecal matter, further compounding the health hazards (Kiellen, 2011). Most of these waste disposal sites are located in open spaces within the vicinity of human settlements where several wells and boreholes are found. Indiscrimate disposal and improper management of domestic and industrial wastes will impact negatively on the environment and health of the inhabitants. Infectious diseases associated with poor environmental conditions kill one out of every five children in Africa, with diarrhoea and acute respiratory infections being the two major killers (WHO, 1979). Diseases such as cholera, typhoid, guinea worm, trachoma, bilharzias, polio, hookworm, and tapeworm are related to drinking of poor quality water and sanitation (Boadi and Kuitunen, 2005). As a result of industrial advancement and urbanization in recent time, there is tremendous increase in the population of people living in Barikin-Sale, and this has led to exceptional increased in the waste generation, therefore refuse dumpsites become a common feature in the city.

Groundwater contamination within a waste disposal site results from the infiltration of leachates through the soil. The leachates are formed when rain falls on the dump, sinks into the waste and picks up contaminants as it seeps downwards (Egbai et al., 2015). The frequent shallow aquifer found in the basement complex terrains is typically exposed to surface and near-surface contamination (Aweto, 2011). It has been affirmed that once an aquifer is extremely exhausted or contaminated, the damage is basically unending and efforts to reduce the contamination are exceptionally expensive (Jegede et al., 2013). In the study area and the surrounding environs, groundwater is tapped from hand-dug wells and boreholes at depths that are sometimes as shallow as 5 m as the main source of water used for domestic purposes in the area. The contamination becomes obvious resulting from the hydraulic contact between the hazardous contents of the leachate plumes and groundwater (Nasir et al., 2010).

Basically, two distinctive methods can be used in investigating the level of contaminants in groundwater. Firstly, the destructive method which involves taking samples using soil auger/core sampler in which case the geology of the area is continually disturbed. Secondly, the non-destructive method which makes use of geophysical method where the geology of the area is not disturbed (Oyedele, 2009). One of the recognized geophysical methods is the electrical resistivity method which provides beneficial and non-destructive means to identify, delineate and map the sub-surface defining leachate contaminant plumes from dumpsites. This method is based on electrical conductivity of leachate which tends to be above that of groundwater (Cristina et al., 2012). Researches have also proven that resistivity method is a tool for identifying, delineating and mapping of leachate contaminant plumes (Porsani et al., 2004).

Direct Current (DC) electrical resistivity methods of geophysical exploration are popular and have proven to be successful and dependable in the fields of geo-environment, hydrogeology, engineering and contaminant hydrology. Recently, one of the new developments is in the application of 2-D electrical imaging techniques to map areas with moderate to complex geology (Griffths and Barker, 1993). Also, mapping of changes in the recorded resistivity in the vertical as well as the horizontal direction, gives a more accurate model of the subsurface in two-dimension (2-D). Electrical resistivity methods for contaminant studies have a wide range of application on shallow groundwater resources, and the advantages include the reduction in the need for intrusive techniques and direct sampling, produces intrinsic properties (electrical conductivity/resistivity) of groundwater chemistry that gives information on contamination, reasonably economical, and optimization of the requisite number of observation wells (Ebraheem et al., 1997 and EL-Mahmoudi, 1999). The objective of the study is to apply geoelectric technique implementing the vertical electrical sounding and the 2-D subsurface electrical imaging (2D resistivity tomography), employing the Wenner techniques to detect and delineate leachate plume from an uncontrolled solid waste disposal site in Barikin-Sale Minna, North-central Nigeria.

Location of the Study Area

The dumpsite is located within the Barikin-Sale area and it is accessible through the Minna Western bye-pass within longitudes 6º 31'E and 6º 33'E of the Greenwich Meridian and latitudes 9º 34'N and 9º 36'N of the Equator (Figure 1). It covers an area extent of about 150 by 100 m with the present dump height of about 5 m. The system of waste disposal at Barikin-Sale is open dumping, consequently the widespread indiscriminate dumping of solid waste at the dumpsite. The increasing population of Minna has made Barikin-Sale a significant settlement for the middle class and low income earners working and living in the capital city of Niger State. Most of the inhabitants of the area embark on the development of hand-dug wells and private boreholes to supplement the insufficient public water supplies.

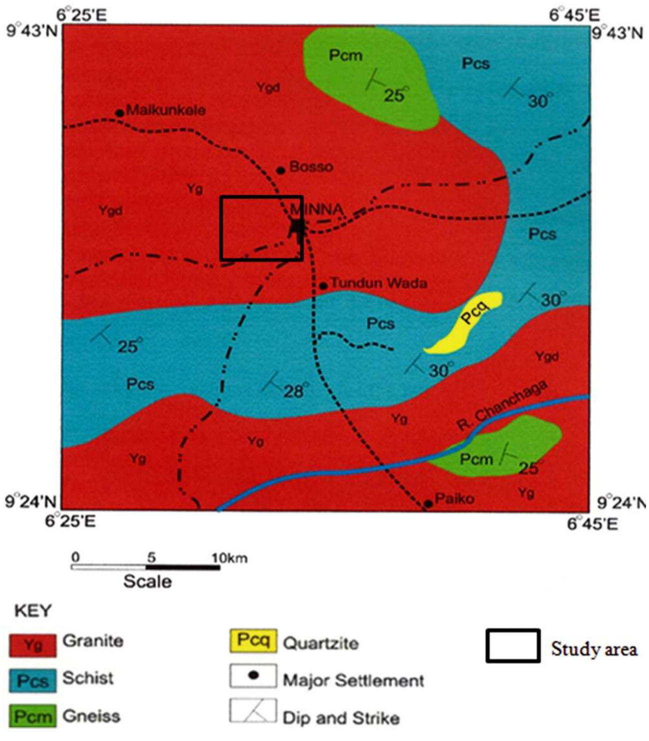

Fig. 1: Geological map of Minna showing the study area (adapted from Nigerian Geological Survey Agency, 2009)

Geology of the Study Area

The study area lies within the Nigerian Basement Complex terrain. This terrain is characterized by three lithofacies: i. the migmatite gneiss complex, ii. the low metasedimentary schist and iii. the older granites (Oyawoye, 1972 and Rahaman, 1976). Important to the area is the granite, granite-gneiss and schist with relics of quartz veins and pegmatite's as minor intrusions underlying the area. Granitic rock occupies about 95% of the area under study. They are mostly exposed on the western part of Minna town. They form high batholiths that are extensive in size. The granitic outcrops are fractured, jointed and foliated.

The rocks are made up of light and dark coloured minerals; quartz and feldspar and biotite-mica respectively. The alteration in the mineral colour defines the gneissose banding. In some cases the rocks are fractured and weathered. The pegmatites and quartzitic veins which appear as minor mineral intrusive rock also form part of the area. The granitic and gneisses rock most times serve as the host rock on which the pegmatetic and quarzitic mineral veins fined expression. They are defined by coarse textures. Minerologically, they are composed of quartz, feldspars and some precious minerals of high quality which include tourmaline, aquamarine, epidotite and emerald (Ajibade, 1982 and Wright, 1985). The hydrogeology of basement areas is simple since there is an intrinsic limitation to groundwater occurrences. Although, where there is thick regolith and there is a dense network of fractures, the posibilities for the accumulation of groundwater in the basement complex rocks may increase. Ideally, the area can be divided into two units, namely, the aquiferous zone within the weathered overburden overlying the fresh basement rocks and the aquiferous zone within the intense fracture joint system in the partially weathered basement (Omale et al., 2016).

Materials and Methods

The equipment used in carrying out this research include PIOS O1 Electrical Resistivity Meter; four steel electrodes (two for current while the other two are for potential); four cable reels with metal clips attached to the wires; two measuring tapes; and hand held Global Positioning System (GPS) - Garmin 75 Model. Rock outcrops, stream channels, undulating landforms are some of the geologic features observed during reconnaissance visits to the site. The information obtained during the reconnaissance was used in carrying out the fieldwork.

Electrical resistivity method which employs the Schlumberger electrode configuration with maximum current electrode separation (AB/2) of 20 m was used in acquiring VES data at the site. A total of ten (10) VES points were established. Electrical resistivity surveys are usually designed to measure the electrical resistivity of subsurface materials by making measurements at the earth surface (Abdel-Azim et al., 1996). Vertical Electrical Sounding (VES) works on the principle of electrical resistivity which involves injecting a specified amount of electric current into the ground through a pair of current electrodes and then with the aid of potential electrodes, measures the potential difference between two points at the surface caused by the flow of the electric current in the subsurface. From the measured current (I) and the voltage (V) values, the ensuing resistivity is determined. The sounding curves produced were interpreted qualitatively through visual inspection and then subjected to the technique of partial curve matching and iteration with the WinResist software for interpretation quantitatively to generate the layered apparent resistivity and thickness. Figure 2 shows a simplified diagram of the Schlumberger array.

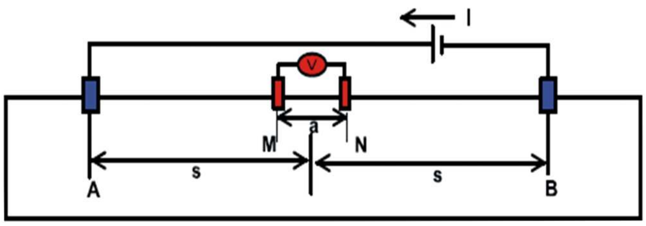

Fig. 2: Simplified diagram of the Schlumberger array

In addition, the 2D subsurface electrical imagings employing the Wenner array were carried out with electrode separation of 5m to map the possible migration of leachate from the dumpsite to the aquifer. The Wenner resistivity data were imputed into the RES2DINV software, which converts the result to a 2D resistivity model (Pseudosection). The generated pseudosection gives the variation of resistivity in both the vertical and horizontal direction with respect to depth.

The VES data were processed to determine the geoelectric parameters (overburden units, thickness and resistivity) as well as the hydrogeological characteristics of the subsurface (Sikandar et al., 2010). The apparent resistivity values calculated for each geoelectric layer were plotted on a log-log graph against the half current electrode separation AB/2. From the qualitative values, geoelectric parameters such as the resistivity of the top (first) layer as well as the thickness/depth of each layer were determined. The first quantitative interpretations were carried out using partial curve matching method which revealed the layers of the VES points, the apparent resistivity, of each layer, the thickness (h) of each layer (Table 1). These parameters were again iterated with WinResist software with a minimized root mean square error to get the sounding curves, the true resistivity of the layers, their real saturated aquifer thickness (h).

Results

The result of the geoelectric parameters obtained from the VES is presented on Table 1. Figure 3 is a representative of the interpreted VES data. Figure 4 and 5 are the geoelectric sections obtained from the study area while Figure 6 and 7 are the 2D inverse reisitivity models for profiles 1 and 2 respectively. Furthermore, the result of the calculated hydraulic conductivity (K) from sieve analysis is presented on Table 2. The interpreted VES curves (Figure 3) reveals three major lithologic layers with the topmost layer being the top soil with a depth range of 0.90m and 1.80m, the second layer being the weathered basement with depth range of 4.0m and 5.3m and the third being the fractured/fresh basement which is the main aquifer unit.

Table 1: Geoelectrical parameters and lithologic delineation of the study area

VES No.	Coordinates	Layer	Resistivity, ρ(Ωm)	Thickness (m)
1	9° 35′ 10.2″ N	1	71.7	1.0
	6° 32′ 04.7″ E	2	33.4	5.0
		3	137.8	
2	9° 35′ 10.2″ N	1	27.9	1.1
	6° 32′ 04.4″ E	2	25.1	4.6
		3	106.8	
3	9° 35′ 10.2″ N	1	65.3	1.6
	6° 32′ 04.0″ E	2	23.9	4.6
		3	134.4	
4	9° 35′ 09.9″ N	1	52.9	1.8
	6° 32′ 03.6″ E	2	23.3	4.3
		3	166.4	
5	9° 35′ 09.3″ N	1	19.4	1.1
	6° 32′ 03.3″ E	2	30.3	4.1
		3	93.5	
6	9° 35′ 09.1″ N	1	34.7	1.5
	6° 32′ 02.7″ E	2	25.5	5.0
		3	113.3	
7	9° 35′ 10.2″ N	1	27.0	1.3
	6° 32′ 04.7″ E	2	18.2	4.1
		3	166.1	
8	9° 35′ 08.0″ N	1	76.0	0.9
	6° 32′ 02.7″ E	2	29.7	4.8
		3	124.4	
9	9° 35′ 08.4″ N	1	122.6	0.9
	6° 32′ 03.1″ E	2	22.0	5.3
		3	120.5	
10	9° 35′ 08.8″ N	1	102.4	1.6
	6° 32′ 03.5″ E	2	21.7	4.3
		3	166.3	

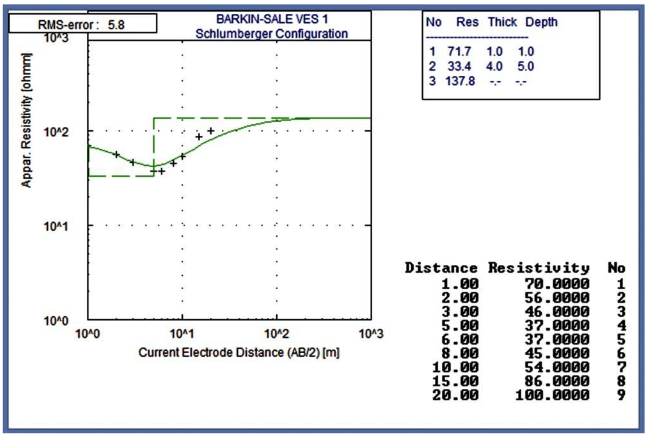

Fig. 3: VES Curve Representative of VES graphs obtained from the Study Area

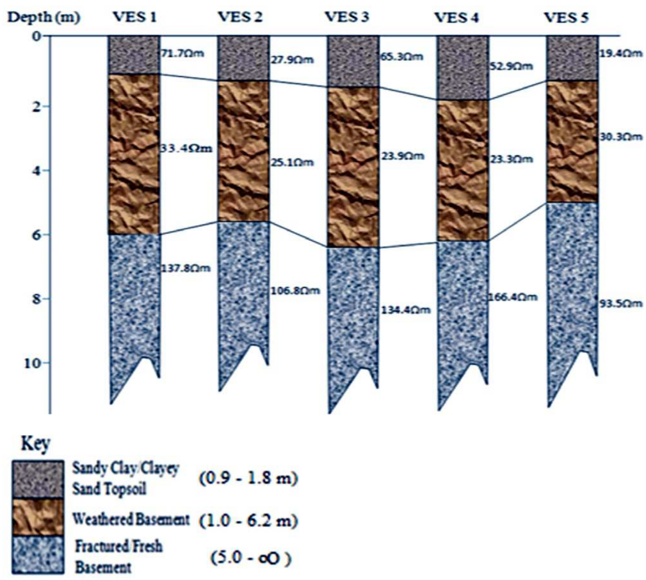

Fig. 4: Geoelectric sections for VES 1 – 5

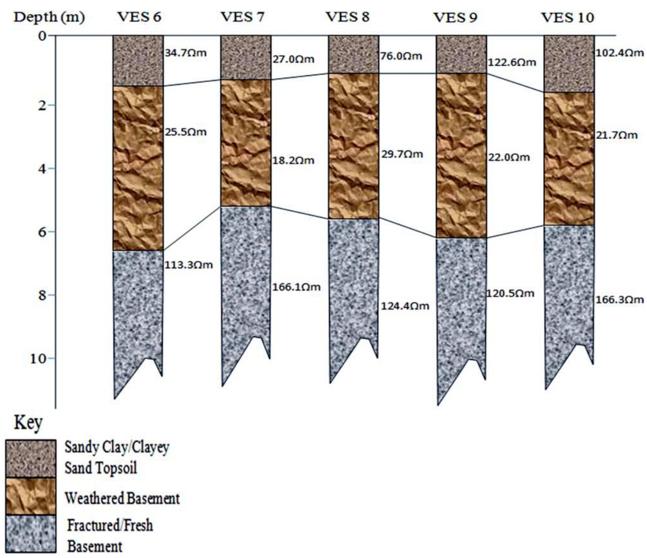

Fig. 5: Geoelectric sections for VES 6 – 10

The Barkin-Sale profiles show very low resistivity across the 2D section (Figures 6 and 7). This can be attributed to contamination of the top-most soil as a result of accumulation of leachate plume.

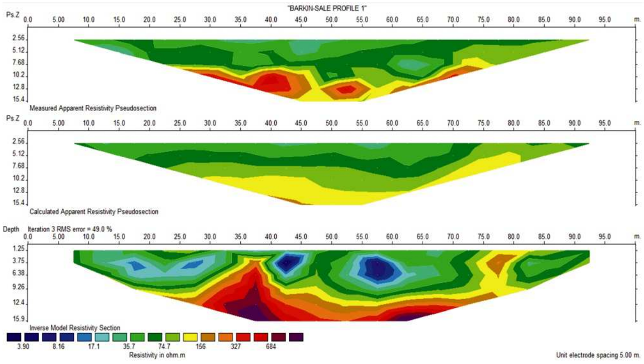

Fig. 6: 2D Inverse resistivity models for profile 1

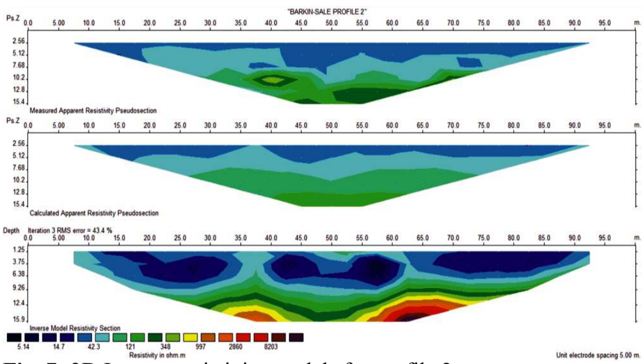

Fig. 7: 2D Inverse resistivity models for profile 2

The result of the hydraulic conductivity calculated from the sieve analysis of soil samples from the study area is presented on Table 2 below.

Table 2: Calculated hydraulic conductivity (K) from the sieve analysis

Locations	Depth (m)	Hydraulic Conductivity (K) m/s
L1	0.0	6.57×10^{-5}
	0.5	8.90×10^{-5}
	1.0	1.42×10^{-4}
	1.5	5.41×10^{-5}
L2	0.0	8.90×10^{-5}
	0.5	7.69×10^{-5}
	1.0	9.74×10^{-5}
	1.5	1.97×10^{-4}
L3	0.0	5.63×10^{-5}
	0.5	1.20×10^{-4}
	1.0	5.63×10^{-5}
	1.5	1.27×10^{-4}
L4	0.0	2.52×10^{-5}
	0.5	1.00×10^{-4}
	1.0	1.15×10^{-4}
	1.5	6.43×10^{-5}

Discussion

The VES results shows that the area is characterized by three (3) layers which are the top soil of 19.4Ωm to 112.6Ωm and the weathered basement of 18.2Ωm to 33.4Ωm and the fractured/fresh basement of 93.5Ωm to 166Ωm with corresponding thickness of 0.9m to 1.8m and 4.0m to 5.3m for the first and second layers respectively. These zones are also indication of solid wastes dumped in the area. Geoelectric sections generated for the overburden units show that the topmost layers at all the VES points are mostly sandy, while the second layer are occupied by clayey sand. The second layer is generally thin at all the VES points, therefore providing slight or no protection at all to the aquifer beneath it. Clayey overburden is characterized by area of relatively high longitudinal unit conductance which offers protection to the underlying aquifer. It has been reported that materials such as sand and gravel have low longitudinal conductance resulting from their higher resistivity values as a result of having low aquifer protective capacity (Farid et al., 2017)..This prevailing condition in the study area enhances the percolation of possible contaminants into the aquifer. From the abovementioned, it is seen that the aquifer in the dumpsite area is prone to contamination (Egbai et al., 2015). Obviously, the shallow aquifer in the area can be easily polluted by contaminated surface runoff water in the area (Odong, 2013).

The 2D inverted resistivity models were generated from measurements along the two perpendicular traverses across the dumpsite area (Figuress 6 and 7). This is to identify and map possible migration of the leachate across the dumpsite area. The 2D inverted resistivity sections image the subsurface geologic sequence and the structural disposition of the layer. The sections reveal basically a maximum of three subsurface layers, which is in agreement with the results of the VES geoelectric section. The resistivity of the various layers ranges from 35.7 to 2860Ωm. However, a low resistivity structure is found at a depth of about 3.75m with resistivity ranging from 3.91 to 42.3Ωm which is indicative of the presence of leachate from the dumpsite.

The calculated hydraulic conductivity (K) from the sieve analysis ranges between 1.97×10^{-4} and 2.52×10^{-5} m/s. Table 2 shows that the hydraulic conductivity ranges between 1.97×10^{-4} and 2.52×10^{-5} m/s. According to Macauley (2008), these values are high implying that the leachate can move downward to the aquifer. These values are high and they correspond to leachate movement through the interconnected pore spaces of the soil underlying the waste dumpsite. This indicates that the leachate can move gradually through the soil and if the dumpsite is not relocated or completely evacuated from the area, the adjoining aquifer and surrounding are susceptible to contamination. The dumpsite has to be properly designed in order for it to serve as a dumpsite.

Conclusion

The vertical electrical sounding, 2D subsurface electrical imaging and geotechnical studies have been applied in the mapping of in a public waste disposal site in Barikin-Sale Minna, Nigeria. This was with a view to unravel potential groundwater pollution that could arise from leachate associated with the dumpsite. The outcome of the research revealed that the open dumpsite in area is underlain by materials of poor protective capacity for the aquifers. This indicates that as time goes on, the aquifer in the area will continue to receive loads of contaminants. Therefore, the aquifer systems in the area are extremely susceptible to contamination from infiltration of leachate from decomposed wastes dumped at the site.

To ensure safe consumption of potable groundwater in the area, wastes should be evacuated from the area because the area is vulnerable to pollution and as a result of the shallow depth of the aquiferous unit, It is advised that further dumping of waste should be discontinued.

Also, the results and data generated from this study should be taken into consideration when planning development projects that engages the subsurface within and outside the dumpsite such as water borehole, farms, health facilities, residential and commercial facilities etc. Specifically, it is recommended that pre-drilling geophysical investigations should be carried out before embarking on any water borehole project within and around the dumpsite. Certainly, modern engineered landfills with bottom liner for safe and sustainable waste disposal and management should be introduced across the country.

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Application of Geophysical and Geotechnical Investigations in Foundation Design Options

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Abstract

Geophysical and Geotechnical investigations were employed in determining the earthing method to be applied for oil pipes and suitable foundation option for structures at the Central Processing Facilities Station of an oil field. Soil bearing capacity values of between 145.44 to 254 kPa with an average of 180.03 kPa was computed for the upper Silty Sand (SM) layer for the proposed raft foundation. So, for the shallow foundation at the depth (2.5m), a value of 2.40 + 0.399T kN/m^2 (where T is the value of imposed load) can be used as the Net Soil Pressure on the soil, provided this value is less than the average value of the allowable soil pressure. Alternatively, a combined shallow and deep foundation system of Raft–on–Pile is being proposed for use. This is predicated on the fact that the depth for the shallow foundation is made up predominantly of Silty Sands and Clayey silts with rather low bearing capacities. In this combination, what is important is the strength obtained from the bearing capacity of the piles (both skin friction and base resistance) at 20.0m depth as computed for different pile diameters (from 400mm to 1000mm) at different factors of safety.

Keywords: *Earthing, Foundation, Geophysical, Geotechnical, Raft-on-Pile, Bearing Capacity, Corrosivity.*

Introduction

Stubb Creek site, near Oron town is located along the South-Eastern fringe of the Atlantic backshore within the Niger Delta region. Geologically the development of the delta has been dependent on the balance between the rate of sedimentation and the rate of subsidence resulting in a succession of transgressions and regressions of the sea, giving rise to the three main subsurface litho-stratigraphic units of the Niger delta. According to Short and Stauble (1967) the first depositional cycle of the three began with a marine transgression in the mid-Cretaceous and ended with a minor folding phase which resulted in the Akata shale Formation. The depositional environment was mainly a deep sea and low energy environment, giving rise to fine-grained sediments with thickness of up to 1,000m. The group is made up of marine clay, shale and limestone.

The second episode occurred when the sea regression and marine shales were deposited with coarse heterogeneous sediments being formed in the coastal and deltaic environments. These events produced the intervening unit of alternating sandstone and shale of the Agbada which outcrops as the Ameki Formation with thickness as much as 1,700m. This cycle of deposition covered the period from late cretaceous to early Tertiary (Paleocene).

The third cycle from Eocene to recent marked the continuous growth of the main Niger Delta. The Benin Formation was deposited during this third phase after the final advance of the Tertiary delta. Its sandy sediments mark the high energy environmental deposits of river and tidal channel activities. The occasional clay intercalates were deposited in the low energy environment on the flood plain between the rivers and the tidal channels in the delta. This Formation has a total thickness estimated to be more than 2,000m. Typical outcrops of the Benin Formation can be found around Benin, Onitsha and Owerri.

The site sits astride the Niger flood plain which overlies the Benin Formation, often called the Coastal Plain Sands and consists of soft, very poorly sorted sands, clayey sands, pebbly sands and gravels (Tattam, 1943). The area is bounded on the West by Qua Ibo River and on the East by Cross River (Figure 1). The peculiar terrain of the area makes it imperative for a more careful study of the soil properties in view of the envisaged usage for Marginal Oilfield Process Facilities Station (PFS) and Oil Injection Station (OIS). For this reason, both geotechnical and geophysical investigations of the subsurface were carried out at the site for the purpose of determining the earthing method for the oil pipes and foundation option for the structures to be erected.

Over time soil characteristics such as moisture content, soil temperature and type have been used to determine the overall resistivity of the earth. Geophysical methods have been used to delineate soil horizons (Besson et al.,

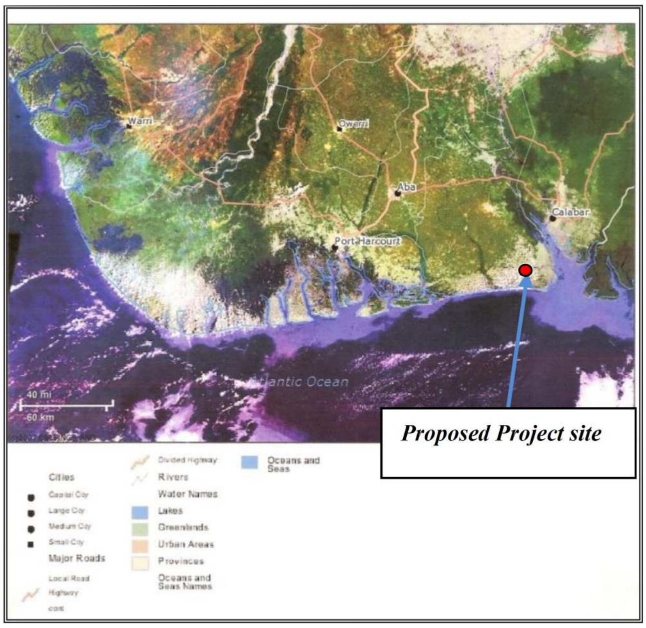

Fig. 1: A Recent Satellite Imagery Covering the Study Area

2004; Muñiz et al., 2016; Tabbagh et al., 2000), monitor water content (Garré et al., 2011, 2013; Michot et al., 2003), map soil texture (Grote et al., 2010; Sudduth et al., 2005), and characterize the effect of tillage on soil properties (Jonard et al., 2013).

Methodology

Geophysical Methods

The geophysical survey data was acquired using ABEM SAS 300 TERRAMETER which is a signal averaging system that takes consecutive readings automatically and the results averaged continuously. The continuously updated running average is displayed as resistance automatically. A micro processor is used to monitor and control all the measurements to ensure optimal accuracy and sensitivity. The apparent resistivity of the subsurface was calculated using the formula:

$$\rho_a = \Pi R \left\{ \frac{(AB/2)^2 - MN/2)^2}{MN} \right\} \quad \text{.......................(1)}$$

where: $\left\{ \frac{(AB/2)^2 - MN/2)^2}{MN} \right\}$ **is the geometric factor**

ρ_a = apparent resistivity (Ohm-m)
R = *resistance (Ohms)*
AB = distance between current electrodes (m)
MN = distance between potential electrodes (m)
Π = *Constant*

Four Vertical Electrical Soundings (VES) and two Constant Separation Traversing (CST) were run across the study area. The VES was run for the earthing and corrosivity tests while the CST was run for cathodic protection tests.

The Schlumberger electrode array was used for the Vertical Electrical Sounding (VES) because shallow resistivity variations is constant with fixed potential electrodes, hence, the lateral heterogeneities can be easily identified and the apparent resistivity curves can be shifted accordingly to remove such local in-homogeneity. The Vertical Electrical Sounding (VES) technique determines the variation in the electrical earth properties with depth and this is done by taken a succession of apparent resistivity values for increasing electrode spacing. The general norm is that large electrode spacing corresponds to increased depth of investigation. The electrode spacing was chosen such that a theoretical interpretation of the electrical resistivity values could be made to a depth of 12.5m, which is considered to be a realistic maximum depth for the proposed site utility.

The Wenner Electrode Configuration was used for the Constant Separation Traversing (CST). This method involves maintaining the current and the potential electrodes separation and moving the entire array progressively along a profile. The electrode spacing was chosen such that a theoretical interpretation of the electrical resistivity values could be made to a depth of about 6m. The idea is to delineate lateral changes in the soil resistivity.

Geotechnical Method

The subsurface investigation was carried out with Shell-and-Auger borings (Figure 2) and Standard Penetration Tests (SPT) were carried out at the sampling depth where cohesion-less (c) materials or c - ϕ soils were encountered during the boring exercise.

A total of six (6) borings were made to a depth of 20.00 meters each at the Central Process Facilities Stations (CPFS) as shown in the fence diagram in figure 3.

In general, disturbed and undisturbed samples were obtained during the drilling programme. Within the zone of cohesive materials such as clays or sandy clays, undisturbed soil samples were obtained during the percussion drilling with the aid of split-spoons and U4-tubes. Sampling intervals during the drilling were 1.50-meters apart down to the end of the boring.

Fig. 2: Percussion Rig drilling in Process at the project site

Results and Discussion

Vertical Electrical Sounding (VES) Data

Qualitative Interpretation

The sounding curves obtained over the entire area surveyed, are only the QQH type**.**
Further semi-qualitative interpretations were carried out by plotting the apparent resistivity values as observed along vertical lines and contoured (Figure 4). The information depicts lateral variation in apparent resistivity values in the area.

The Specific Depths of Investigation (SDI) as defined by Beck 1981(as that depth that contributes most to the total signal measured on the ground surface) was also calculated (Table 1) using the formula:

$$D = 0.125 \times (AB) \qquad ....(2)$$

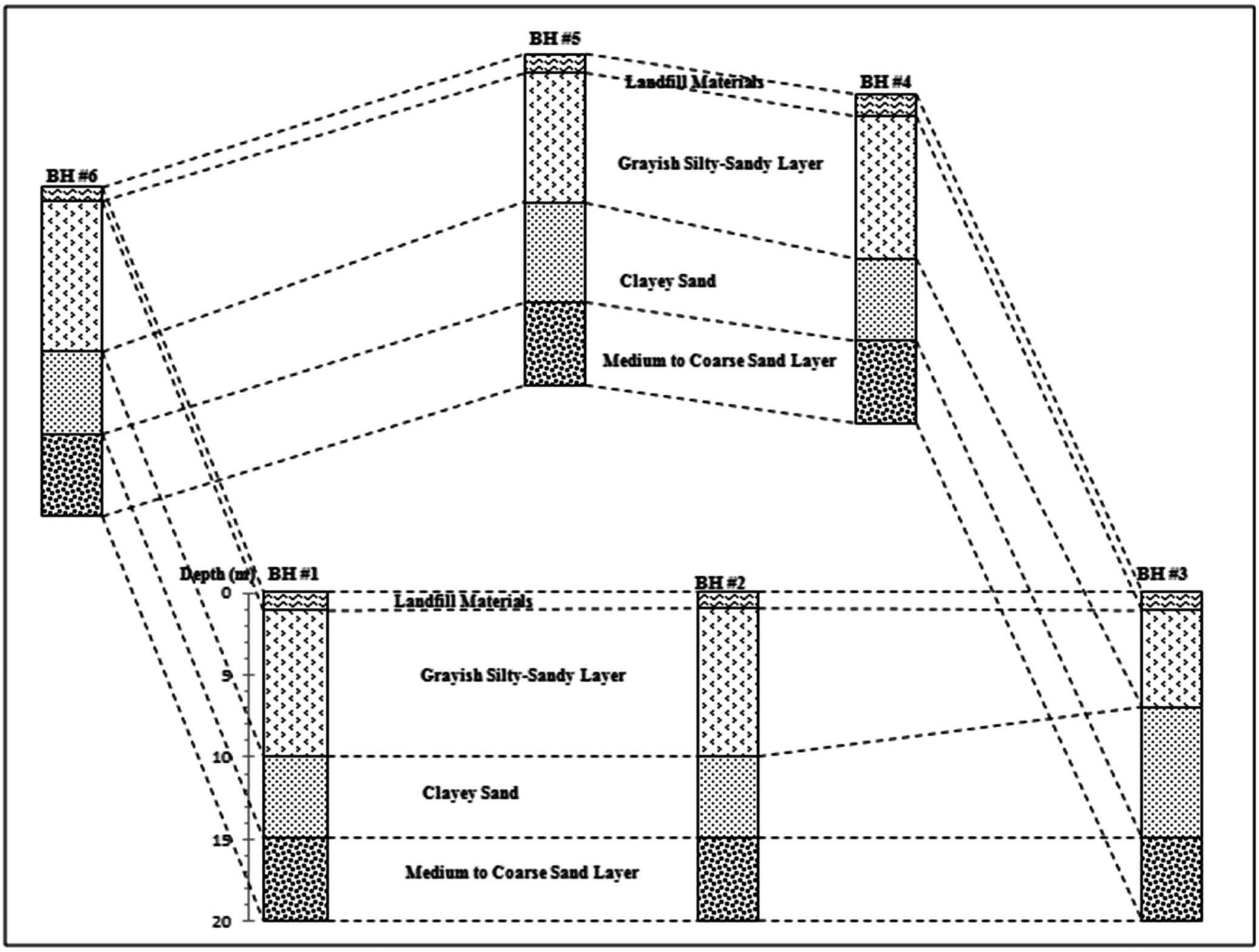

Fig. 3: Fence Diagram for the x6 No deep Borings at the CPF Site, near Oron.

where:

D = specific depth of investigation
AB = the total distance between the two current electrodes

Table 1: Specific Depths with Corresponding Apparent Resistivity

Half Electrode Distance AB/2 (m)	Total Electrode Distance (AB) (m)	Specific Depth (m)	Apparent Resistivity (Ohm-m)
1	2	0.25	64.61 - 201.32
2	4	0.50	32.79 - 106.09
3	6	0.75	31.74 - 91.28
4	8	1.00	31.52 - 79.38
6	12	1.50	22.3 - 44.60
8	16	2.00	14.80 - 29.64
10	20	2.50	12.17 - 24.35
15	30	3.75	10.00 - 12.80
20	40	5.00	9.80 - 11.57
25	50	6.25	9.00 - 12.78
30	60	7.50	10.00 - 11.33
40	80	10.00	12.00 - 14.00
50	100	12.50	15.00 - 18.00

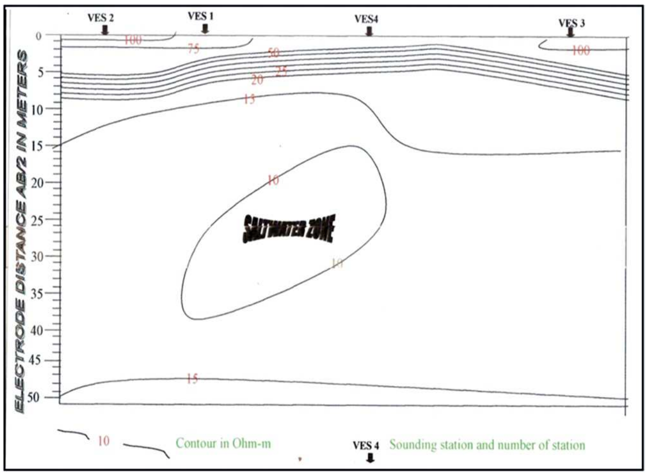

Fig. 4: The Apparent Resistivity Section Connecting VES 1; VES 2; VES 3 and VES 4

The Constant Separation Traversing

The data collected for the Constant Separation Traversing was analyzed qualitatively. This involves plotting the apparent resistivity values against the horizontal distance (Figure 5). The sounded point is the mid-point between the two current electrodes. The inter-electrode distance was 10m thus given the total electrode spread as 30m. The specific depth equivalent according to Becks (1981) is about 3m based on the empirical formula

d = 0.11AB. ....(3)

where
d is the specific depth of investigation
AB is the total length of array

The resistivity values are generally uniform along profile line. However, the resistivity values along Profile 2 are relatively higher than Profile 1, indicating that the area is slightly heterogeneous.

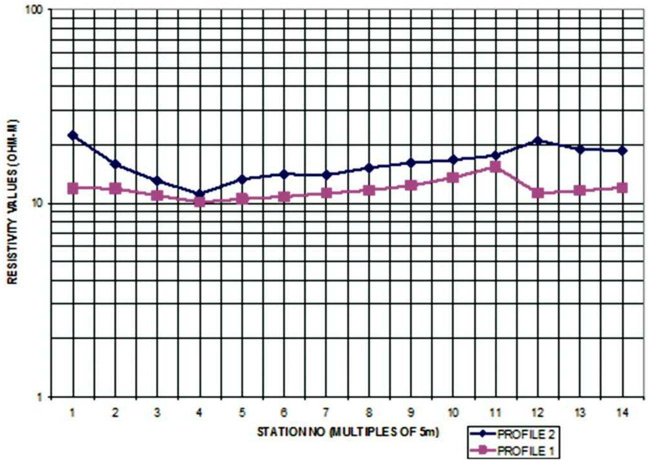

Fig. 5: Constant Separation Traversing (CST) [Profiles 1 and 2 combined]

The Quantitative Interpretation

Only Vertical Electrical Sounding (VES) data were used for this interpretation by plotting the apparent resistivity ***(ρa)*** values as the ordinate and the half electrode separation (AB/2) as the abscissa on a bi-log graph paper. The resulting curves were interpreted manually using the Partial Curve matching method of Orellana and Mooney (1966). The results were further iterated using the RESIST computer software (Vander Velpen, 1988) developed at ITC Delft in Netherlands to obtain the layering model/parameters.

The Schlumberger depth sounding curves are as presented in Figures 6a-6d and the result is presented as 3D Geo-electric model of the area in Figure 7.

The Apparent Resistivity Section:

Apparent resistivity appears uniformly low, showing a gradual decrease in values until the half electrode distance of 30m before it started climbing up. The Low

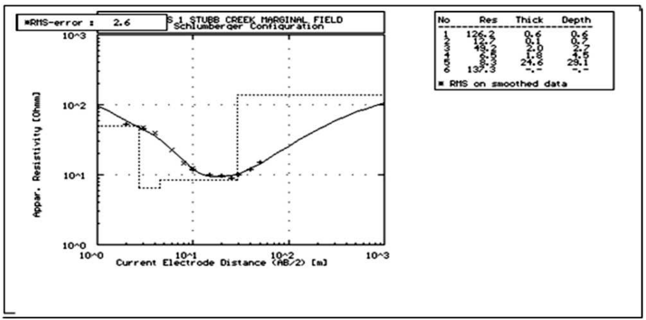

Fig. 6a: The Schlumberger Depth Sounding Profile Curve beneath VES 1 Location.

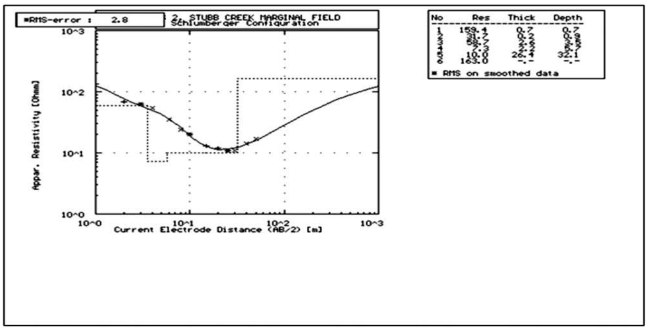

Fig. 6b: The Schlumberger Depth Sounding Profile Curve beneath VES 2 Location

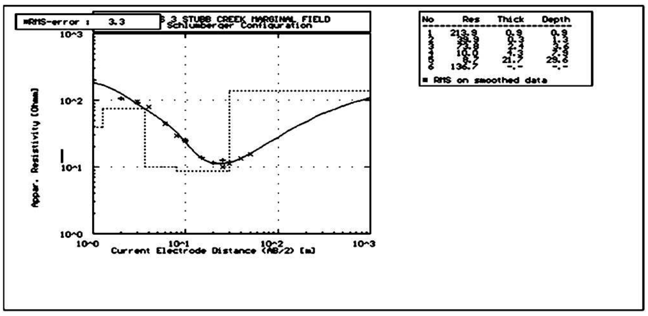

Fig. 6c: The Schlumberger Depth Sounding Profile Curve beneath VES 3 Location

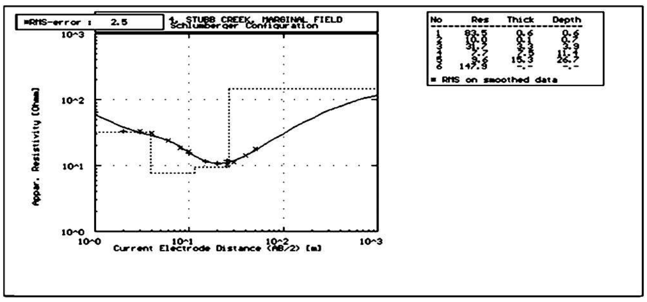

Fig. 6d: The Schlumberger Depth Sounding Profile Curve beneath VES 4 Location.

Resistivity values may be associated with the presence of saltwater within that depth interval.

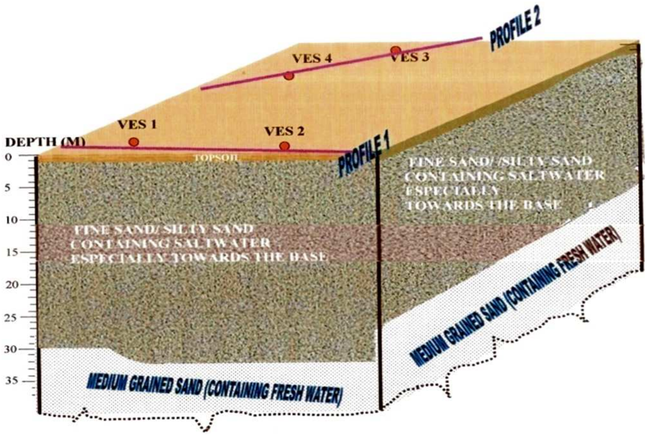

Fig. 7: The 3-D Geo-electrical Model of the surveyed area at the Process Facilities Station (PFS), Stubb Creek Marginal Oilfield, near Oron, Akwa Ibom State

Corrosivity of Subsurface Materials at the PFS Site

Buried metallic structures are susceptible to corrosion, which is a complex phenomenon involving multitude of variables including soil resistivity, presence of chlorides and sulphates, oxygen content and pH. However, the chief among them is the resistivity of the soil. Resistivity is a property of the bulk volume of the soil and the electrolyte, which is a function of the availability of moisture. Typically, the most corrosive soils are those with low resistivity, low pH (for steel/ductile iron) and high concentration of chlorides and sulphates.

The soil corrosivity is classified based on soil electrical resistivity by the British standard BS-1377 as recorded in Davenport et al (1981) and is as follows:

Table 2: Soil Corrosivity Vs. Resistivity (after Davenport et al., 1981)

Resistivity (*Ω-m*)	Soil Corrosivity Description
Below 10	Very Corrosive
10 – 50	Corrosive
50-100	Mildly Corrosive
Above 100	Very slightly Corrosive

The resistivity values obtained for the PFS area surveyed are generally low, which is due to high salinity content and fine grain nature of the sand, suggesting that the area is generally corrosive.

In addition, the rate of aeration is expected to be low because of the fine grained texture of the soil at various layers. This will result in low diffusivity/concentration of oxygen, and will lead to redox potential in the soil and the formation of corrosion cells.

Table 3: Summary of Specific Depths, Corresponding Apparent Resistivity Values and Corrosivity Index

Half-Electrode Distance (m) [AB/2 (m)]	Total Electrode Distance [(AB) (m)]	Specific Depth (m)	Apparent Resistivity (Ohm-m)	Corrosivity Index
1	2	0.25	64.61 - 201.32	Mildly corrosive - slightly corrosive
2	4	0.50	32.79 - 106.09	corrosive - Mildly corrosive - slightly corrosive
3	6	0.75	31.74 - 91.28	corrosive - Mildly corrosive
4	8	1.00	31.52 - 79.38	corrosive - Mildly corrosive
6	12	1.50	22.3 - 44.60	Corrosive
8	16	2.00	14.80 - 29.64	Corrosive
10	20	2.50	12.17 - 24.35	Corrosive
15	30	3.75	10.00 - 12.80	corrosive
20	40	5.00	9.80 - 11.57	Very corrosive - corrosive
25	50	6.25	9.00 - 12.78	Very corrosive - corrosive
30	60	7.50	10.00 - 11.33	corrosive
40	80	10.00	12.00 - 14.00	corrosive
50	100	12.50	15.00 - 18.00	corrosive

The Constant Separation Traverse (CST) mentioned earlier, shows that the area is slightly heterogeneous because resistivity values along the two profile lines differ slightly even though they are low and this will lead to the formation of galvanic cells along the pipe lengths passing through the soil.

Results and Discussions

Geotechnical Investigations

Four (4) basic types of soil profiles are found within the area of investigation namely:

(i) Landfill Layer (A mixture of Laterite and Sand)
(ii) Grayish Silty Sand Layer (SM-SP)
(iii) Grayish Clayey Silt layer (ML)
(iv) Medium to Coarse Grained Sand (SP)

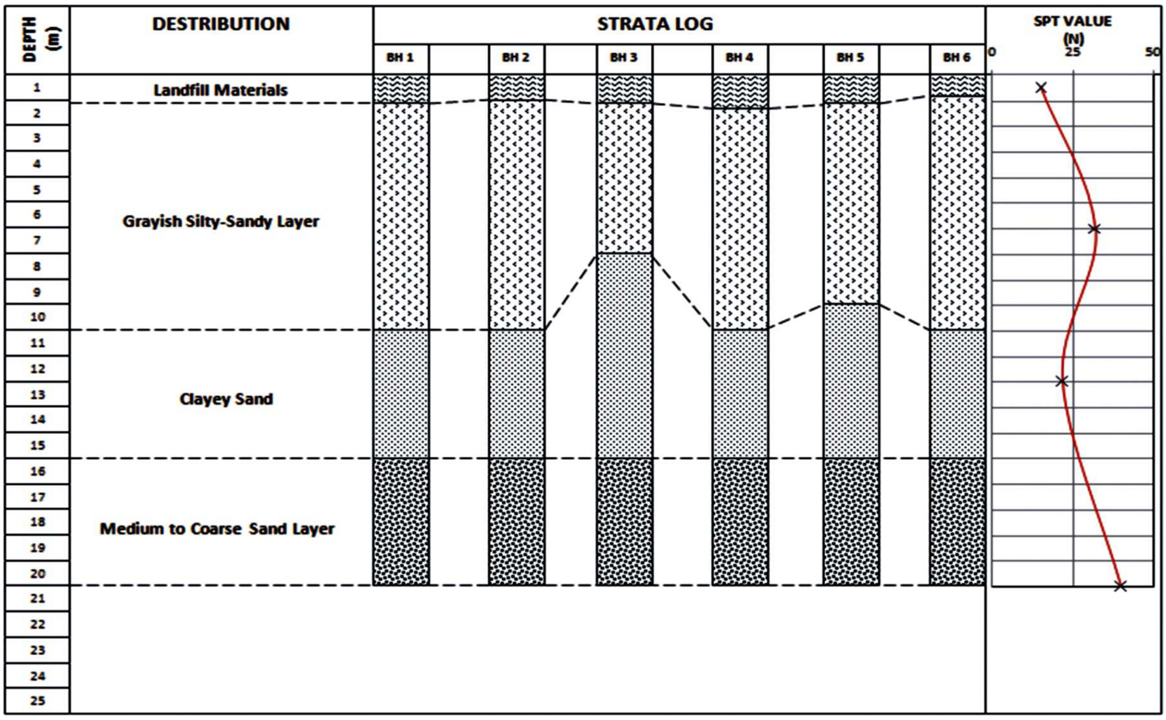

Fig. 8: Composite Stratigraphy and SPT Curves Across the Site

Computation of Bearing Capacity

Bearing capacity for the site was calculated using the following methods:

SPT Method, Terzaghi's Method, Meyerhof's Method, Bowles' Method and Brinch Hansen's Method.

SPT Method

The allowable net soil pressure for the design of Rigid Raft based on the "SPT" field data, may with sufficient accuracy be taken as:

qa (tons / sq. ft) = 0.22 N ....(4a)

where: N = SPT value (corrected for Water Table effects)

At a depth of 2.50 meters, an average value of SPT N-value of 32.3 was obtained.

Thus,
qa (tons / sq. ft) = 0.22 (32.3) = 7.106 tons / sq. ft
= 762 kPa.

Using a Safety Factor of 3.0, the allowable Bearing Capacity = 254 kPa

(Note: This value of bearing capacity can be conveniently used for a depth range of 1.50 - 2.50 meters but for the silty clay layer beneath.

TERZAGHI Method (Based on Laboratory Results)

Using the Terzaghi Formula for bearing capacity computations (based on laboratory results from soil testing), we have:

qu = qc / F.S = 1/F.S {{(1-0.2 B/L) γ B/L.Nγ }+ {(1 + 0.20 B/L) c Nc}+ {(γDf Nq)}}..................................(5)

where,
B = width of Raft Foundation;
L = Length of Raft Foundation
γ = unit weight of soil at foundation level
Nγ , Nc , Nq = Terzaghi bearing Capacity Factors.

At a depth of 2.50m we have the following soil properties:

c = 35.40 kPa, ϕ =10.0,o Nγ = 0.50, γ =17.8 kN/m3, Nc = 8.40, Nq = 2.50
Assuming a Factor of Safety (F.S) = 3.0 and B/L ~ 1.00
∴ q (allow) = 158.40 kPa

MEYERHOF'S Method

Meyerhof (1974)	Bowles (1988)	Boundary Conditions
$q_a = 12 N k_d$	$q_a = 12 N k_d$	$B \le 1.22m$ ....(6a)
$q_a = 12N \left[\frac{B + 0.305}{B}\right]^2$	$q_a = 12.5 N\left[\frac{B + 0.305}{B}\right]^2 k_d$	$B > 1.22$ m ....(6b)
Note: $K_d = 1 + 0.33 \; [D / B] \le 1.33$ suggested by Meyerhof (1974)		

Since B ≥ 1.22 m, we use the following relationship,

$$q_a = 12 N \left[\frac{B + 0.305}{B}\right]^2 \quad ....(7)$$

where: N= SPT value (corrected for water table effects).

MEYERHOF'S Method:

At a proposed depth of 2.50 meters at the project site, an average SPT N-value of 32.3 was obtained and assuming a B value of 5.00 m .
$\therefore q_u \, (kN/m^2) = (12 \times 32.3)\{(5.00 + 0.305) / 5.00 \}^2$
$= 436.32 \, kN/m^2$ ....(8)
Applying a factor of safety of 3.0 gives,
$q_{(allow).} = 145.44$ kPa

BOWLES' Method

At the proposed depth of 2.50 meters with an average SPT N-value of 32.3 and for B ³ 1.22 m,.

$$q_u = 12.5N \left[\frac{B + 0.305}{B}\right]^2 k_d \quad ....(9)$$

where,

$K_d = 1 + 0.33 \, (D/B) \le 1.33$ .... (10)
D = 2.50
$\therefore q_u = 524.59$ kPa
For a Factor of Safety of 3.0
$q_{(allow)} = 174.86$ kPa

HANSEN'S Method

The Ultimate bearing capacity according to the Hansen's Method is expressed as follows:

$q_u = -c\,Cot.\phi + (\Delta q + c\,Cot.\phi)\,N_q\; s_q\; d_q + 0.5\; B\; N\gamma\;\; s\gamma\;\; d\gamma$ .... (11)

where: Nq sq dq and Nγ sγ dγ are Meyerhof (1963) shape, depth and inclination factors for vertical load.

For horizontal ground surface, horizontal footing base, the ground and inclination factors all equal to unity (1). Hence, assuming a Δq = 200 kPa, substituting the same soil conditions and the relevant factors,

qu={-35.40 kPa [Cot 10o]}+ {(200 kPa + 35.40kPa (Cot 10o)) (2.50)(1)(1)}
+ {0.50(5.00)(0.50)(1)(1)}
= 502.395 kPa
qa=167.465 kPa

In summary therefore, the values of the bearing capacity (at foundation bearing level of 2.50 meters) for the Oil Tank Foundation based on the various geotechnical computation methods are as given in table 5.

Table 5: Bearing Capacity Values for the Proposed Structures at the Central Process Facilities Station

Project Site	SPT N-value	Depth of Fdn.	Computational Methods (kPa)					Average Value kPa
			Meyerhof (1974)	Bowles (1988)	Terzaghi and Peck (1967)	Brinch Hansen	SPT	
Central Process Facilities Station, Mgbo LGA, Akwa Ibom State	32.3	2.50	145.44	174.86	158.40	167.465	254	180.03
Remarks.			Acceptable	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable

These values are however, below the upper values for bearing capacity (380 to 470 kPa) cautioned for use by Bowles (1977).

Settlement considerations are by the virtue of the inherent use of these equations limited to 25.4 mm. The use of the Factor of Safety of 3.0 takes care of any unexpected high settlement values that may likely be obtained for this site.

From the above, it could be observed that the range of Soil Bearing Capacity values useable at the proposed Central Process Facilities Station based on a foundation depth (D_f) of 2.50 meters for a B value of 5.00 meters is between 145.44 to 254 kPa with an average of 180.03 kPa.

Conclusion and Recommendations

On the basis of the computations carried out above, the following recommendations are hereby made:

1. A range of values of between 145.44 to 254 kPa with an average of 180.03 kPa. has been found to be the bearing capacity of the upper bearing Grayish Silty Sand (SM) layer for purposes of supporting the proposed structures at the Central Process Facilities Station.
2. A depth of about 2.50 meters should be excavated before placement of foundation of thestructures.
3. The width of the Raft Foundation in all cases should be taken as (B + x) meters, where B = width of base of the building, and x = 1.00 m as allowance around the perimeter of the excavation.
4. A value of 2.40 + 0.399T kN/m^2 can be used as the Net Soil Pressure on the soil at the project site, since this value should be less than the average value of the allowable soil pressure.

Alternatively, a combined shallow and deep foundation system of Raft–on–Pile (Figure 8) is being proposed for use. This is predicated on the fact that the depth for the shallow foundation is made up predominantly of Silty Sands and Clayey silts with rather low Bearing capacities.

In this combination what is important is the strength obtained from the bearing capacity of the piles (both skin friction and base resistance) as computed for different pile diameters (from 400mm to 1000mm) at different factors of safety.

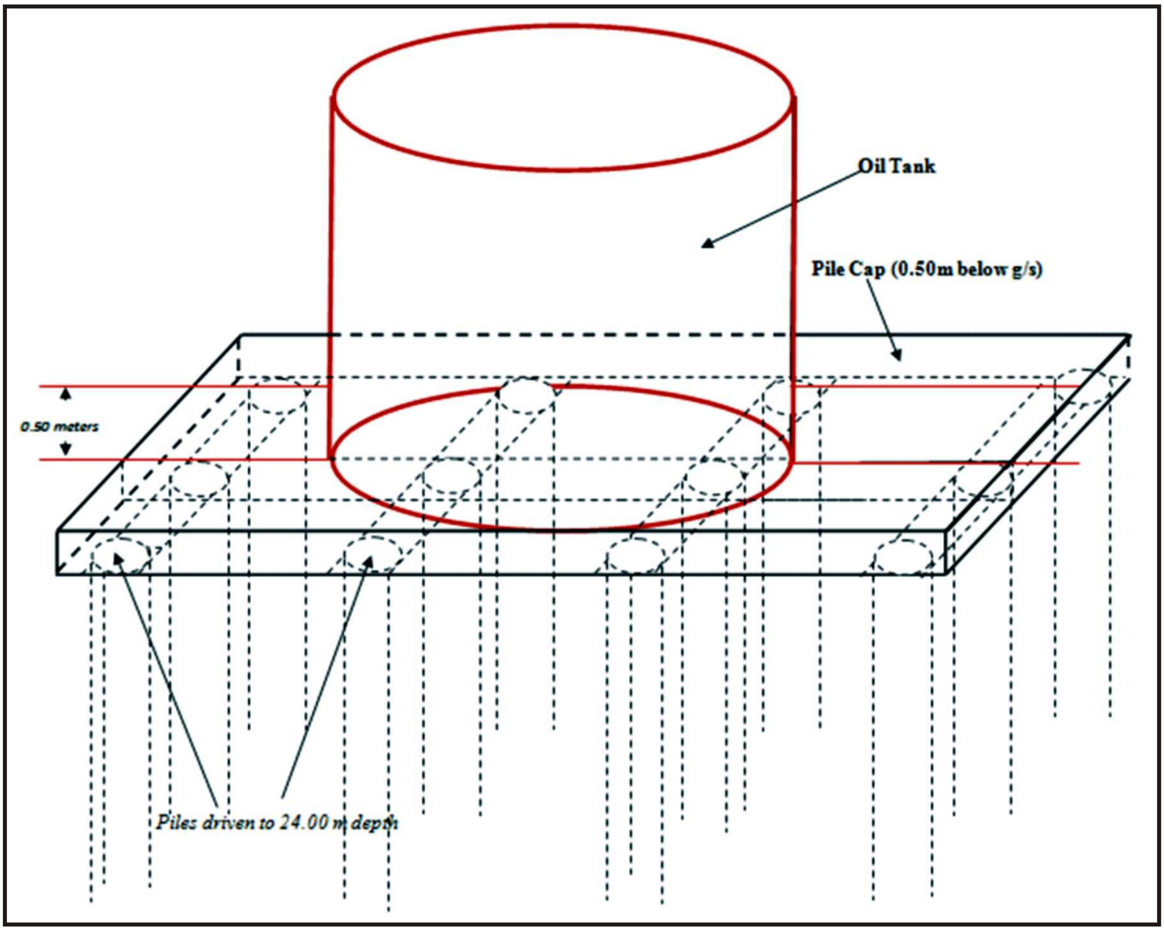

Fig. 8: Schematic Raft–On–Pile Configuration

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Sustainability and Green Environment in the Construction Industry Soil Characterisation Using Electrical Resistivity Tomography and Geotechnical Techniques

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Abstract
Geotechnical investigations are quite expensive, time consuming and destructive to the environment unlike the surface geophysical approach which are non-destructive and provide rapid and cost-effective subsurface information. Geophysical methods have been employed in geotechnical investigations with the aim of identifying geological structures and determining physical characteristics of rock formations. Thus, electrical resistivity tomography (ERT) technique and geotechnical data were integrated to assess the characteristics of soils from Otobi area, South Eastern Nigeria and to also correlate the two approaches. Two electrical resistivity tomography (ERT) profiles 590m length each were probed with aid of multi-electrode Super Digital DC Resistivity meter with 60 electrodes and 10m inter-electrode spacing. In addition, sixteen (16) disturbed soil samples were collected at 3m depth for soil index properties assessment in the laboratory. The test include: grain size distribution, Atterberg limits, natural moisture content (NMC), specific gravity (Sg), and compaction. The ERT revealed a subsurface system characterized by fine-grained soils of low resistivities (<104 Ωm) at varying elevation. Consequently, near surface soils consists of alluvial/residual deposits displaying localized high lateral resistivity (250Ωm) inhomogeneity. Grain size analysis also revealed dominantly fine-grained soils with percentage sandy clay and sandy silt as 43.8% and 18.8% respectively. ASSHTO classification points to predominately A-6 and A-7 group as plasticity chart revealed CL and MH soils. Liquid limit (28.9-62.3%), plastic limit (17.4-37.2%) and plasticity index (11.3-28.50%) indicative of absence of expansive clay sand low-medium swelling potential shown by about 98% of the samples. Moreso, Sg values were greater than 2.6 while optimum moisture content (13.71%) and maximum dry density (2.29 mg/m^3) indicative of the soils suitability for engineering works as NMC (19.43%) suggests high compressibility on moisture influx. It can therefore be concluded that surface electrical resistivity method corroborates the geotechnical properties and it is reliable in identifying soil types and physical properties important for their geotechnical performance.

***Keywords**: Electrical Resistivity Tomography, Geotechnical properties, Soils, Southeastern Nigeria.*

Introduction

Geotechnical investigations for construction sites and possible borrow pits is of immense importance and must be adequate to determine the engineering properties of soils in the area. This is due to the fact that the stability of engineering structures is largely dependent on some specific inherent physical and engineering properties of interest. Major geotechnical problems in construction involving silty-clayey soils are due to their low strength, durability and high compressibility of soft soils, and the swell-shrink nature of the over-consolidated swelling soils (Stavridakis, 2006). Such site investigations could take diverse forms which could be in-situ test, laboratory test or even geophysical surveys depending on structural designs/load and existing geological conditions. As the poor conditions of soils on their properties can often be a significant impediment to successfully implementing green infrastructure projects (USEPA, 2011).

Geotechnical survey is a fundamental means of determining the soil properties as regards their functionality over time for any construction event while electrical characterization of soils is a geophysical technique which entails surface electrical resistivity measurements and computation of the geoelectrical parameters of the subsurface soils, thus, a site characterization programme is a plan of action for obtaining estimates of parameters to be used in modeling engineering performance (Umoren *et al.,* 2016). Subsurface soil characterization and determination of soil strength are pre-requisite for the foundation design of important civil engineering structures (Sudha *et al.,* 2009).

Electrical current flow in the subsurface soil suggests that the possible relationship between the soil strength and electrical resistivity should be based on the parameters which control soil strength as well as electrical resistivity such as grain size distribution, degree of saturation, porosity and cementation (Sudha *et al.,* 2009). It is so, since resistivity is sensitive to the salinity of saturating fluid whereas soil strength is not related with it. Therefore, the relationship between electrical parameters and soil strength will be meaningless if the salinity of saturating fluid changes with depth. On the other hand, clay content in soil matrix can affect both soil strength as well as its resistivity with different degree ((Sudha *et al.,* 2009). The ion exchange property of clay forms a mobile cloud of additional ions around each clay particle. These ions facilitate easy flow of electrical current. Thus, in fine-grained soils such as clay, electrical resistivity is always lower than expected on the basis of chemical analysis of water extracted from the soil (Zhdanov and Keller, 1994). Therefore, clay content in the soil may change relationship between electrical parameter and soil strength. Earlier investigators such as Savvaidis *et al.* (1999); Luna and Jadi (2000); Israil and Pachauri, (2003); Othman (2005); and Soupios *et al.* (2005), Cosenza *et al.* (2006), Gay *et al.* (2006), Sudha *et al.* (2008), Akintorunwa and Adesoji, (2009), and Oyedele *et al.* (2011) have employed geophysical and geotechnical methods in ground investigations and soil characterization.

Furthermore, geotechnical tests are time-consuming, cost ineffective and destructive to the environment. On the contrary, geoelectrical methods are rapidly faster, cost-effective and non-destructive, thereby contributing to sustainability and green environment in the construction industry. Moreso, the use of electrical resistivity tomography (ERT) technique provides an extensive electrical image of subsurface environment and has become an important tool for the electrical characterization of soils. Therefore, this work is aimed at assessing the engineering properties of soils from Otobi area, Southeastern Nigeria, through an integrated approach involving electrical resistivity tomography and geotechnical techniques, with a view to evaluating the engineering characteristics of soils in the study area.

Location and Geology of the Study Area

The study area is located within the Anambra Basin characterized by thick sequence of Cretaceous sediments in Southeastern Nigeria. It is bounded by the Abakaliki anticlinorium of the Southern Benue Trough to the east and the Basement Complex of the Southwestern Nigeria to the west (Fig. 1). Geologically, the origin of the Anambra Basin is intimately related to the tectonic and sedimentary cycles responsible for the origin of the adjoining southern Benue Trough during the separation of the Afro-Brazilian plates in the Mesozoic era (King 1950; Reyment 1965; Burke *et al*., 1972; Benkhelil 1986). However, Nwachukwu (1972), Hoque and Ezepue (1977); and Ladipo (1986, 1988) among others, have shown that, subsequent to the uplift of the Benue-Abakaliki fold belt during the Santonian, the geological history of the Anambra Basin is linked to the post-Santonian subsidence of the then Anambra platform. This was followed by a series of trans-regression cycles leading to sedimentation of about 6 km thick of Cretaceous and Tertiary sediments (Obaje *et al*., 2009) within the basin to the west of the Abakaliki uplift during the Campanian-Paleocene. The stratigraphic sequence within the Anambra Basin in a chronological order includes Campanian-Maastrichtian Enugu-Nkporo shale group, Maastrichtian Mamu Formation, Ajali Formation, Nsukka Formation (Fig.1). Locally, the study area is underlain by the Maastrichtian Nkporo shale Formation and Otobi sandstone (Fig.2) obscured by weathered shale and mudstones forming dark overburden in some parts of the area and reddish to brownish top soils in other parts. Outcrops of Nkporo shales were observed along the Okpokwu River and it consist essentially of clays, shales and mudstones. The Nkporo shales are highly weathered in most parts of the study area as such occurred as clays, silty clays and sandy clays (residual soils). Though, Otobi sandstone occurred at the southeastern part of the study area as sandstone exposures, characterized by a high degree of jointing suggestive of probable impact of the Santonian tectonic events in the southern Benue Trough. Also, along the Okpokwu river, there are alluvial deposits mainly composed of silty clay, clayey silt and; sandy and gravelly clay which are believed to be Recent (Quaternary age).

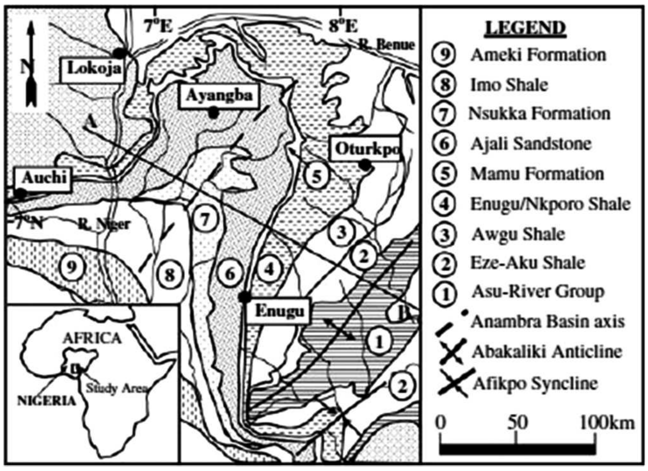

Fig.1: Geological map of Anambra Basin *(After Aghamelu & Okogbue, 2010b).*

Materials and Method

Field Sampling and Laboratory testing

Sixteen (16) disturbed soil samples were collected at three metre depth across the study area (Fig.2). The sampling strictly followed the standard procedure for soil sampling specified in British Standard Institute (BSI) 5930 (1981). In the laboratory, physical and geotechnical properties such as gradation test consistency limits (Liquid limit LL, Plastic limit PL, and Plasticity index PI), natural moisture content (NMC), maximum dry density (MDD), optimum moisture content (OMC) and specific gravity (Sg) were analysed using standard procedures and methods of testing soil for civil engineering purposes at the Soil and Material Laboratory of China Geo-engineering Corporation Nigeria limited, Kaduna. For grain size distribution, 1000g each of hand crushed, oven dried soil samples at 105°C were subjected to particle size distribution analysis using sieve shaker with stacked sieves as 16, 30, 40, 50, 100 and 200 in accordance with ASTM D422-63 standard method for particle size analysis. The liquid limit and plastic limit tests were carried out on air-dried samples that passed 0.425 mm (BSI No. 40) sieve; both tests then followed standard procedures specified by BSI 1377-2 (1990). Crushed soil samples that passed through 0.075 mm (BSI No. 200) sieve and oven-dried at 105°C for 24 hours was utilized for the specific gravity tests. Also, compaction test (OMC and MDD) to ascertain the moisture contents at which maximum dry densities of the materials is achievable with attendant reduction in voids which results in an increased material strengths at varying moisture contents was carried out at the West African energy level with a 13205cm^3 mould. However, the laboratory compaction tests were limited to particles of soil samples that passed through 19.05 mm BSI sieve and followed procedure specified by (BSI, 1990). Finally, the natural moisture contents determination of the soils followed simple method outlined by (Akroyd, 1957).

Field Geophysical Investigation

Electrical resistivity tomography (ERT) survey was carried out using multi-electrode system composed of WDA-1 Super Digital DC Resistivity metre. The Wenner array was adopted; with 60 electrodes deployed with an inter-electrode spacing of 10m along two separate traverses each cut by a river valley, hence the four the profile lines of 590m length each with a maximum survey depth of 25m (Fig.2). Interpretation was done using BT-WYS software for processing and inversion of resistivity image profile data for a better interpretation and visual inspection of the various stratigraphic horizons. The inverted resistivity-depth models are shown in Fig.3 (a-d).

Results and Discussion

Electrical Resistivity Tomography (ERT)

The interpretation of the present ERT data is done based on the expected ranges of resistivity values of the common fresh rock deposits and their alteration products with expected filling materials, such as air-filled or clay-filled cavities; in addition to the hardness, clayey content and degree of fracturing of the shale and sandstone bedrocks. Based on the above indices and the geologic implication of the range of resistivity values obtained from the resulted 2D resistivity sections, the 2D resistivity sections are interpreted and explained (Sudha *et al.,* 2009). The pseudo-sections (Fig.3) for profile A and B are continuous profiles likewise C and D. Fig.3a and b shows subsurface lithologies with similar resistivity-depth (elevation) as obtained by Fig. 3c and d though consists predominately of fines (clay-silt) material.

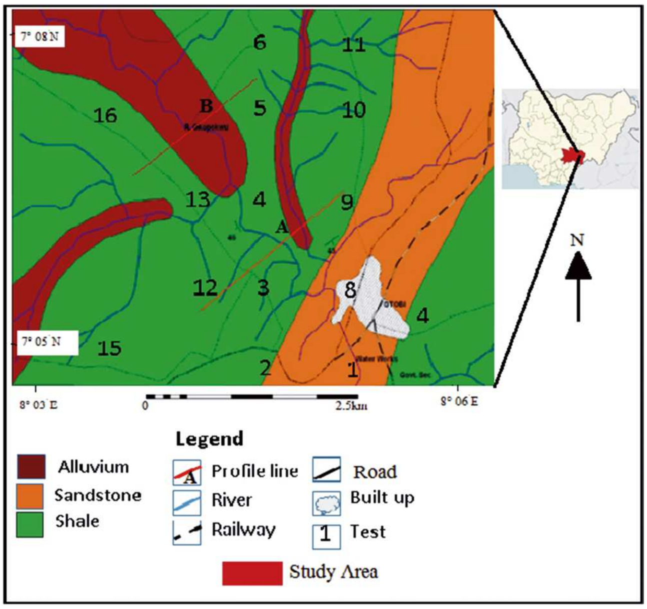

Fig 2: Geology and sample pit location map.

The relatively low resistivity (<34.2 Ωm) at all depths indicates the presence of fine soil material and increase in the percentage of clay in soil matrix (Sudha *et al.,* 2009). There are some localized high resistivity zones which vary with ground elevation near the surface, indicating localized lateral resistivity heterogeneities in the near surface due to the presence of alluvial/ residual deposits with some altered mudstone and shale lithologies. The presence of finer materials (silt and clay) in saturation condition consist of moderately-highly weathered rocks below the near surface layers (60-100m), indicated by decrease resistivity signature

(Sudha *et al.,* 2009). The clay formation is represented by the low resistivity (<10 Ωm) zone.

For Fig.3c and d; the images show that the much unsaturated near surface soil are represented by high resistivity along the entire section. The high resistivity associated with the near surface material is attributed to the presence of dry alluvial/residual soils and some consolidated sandstone occurring at intervals in the area. The depth (elevation) of the top layer varies between 88–110 m (22m thick) along the profile length in conformity with the slope of the surface. The decrease in resistivity (below 724 Ωm) with gradient (88m) towards the river indicates the presence of saturated conditions in association with the groundwater zone occurring within the distributed geological structures (Fig.3a). Both profiles revealed the presence of fine-grained materials at depths within the groundwater zones.

The two-dimensional images (Fig.3) show a resemblance in geological conditions indicative of similar hydrogeological and engineering characteristics as the near surface is characterized by localized lateral resistivity inhomogeneity consisting of alluvial/residual deposits. Discontinuities which are an evidence of later geologic processes that may enhance or impair groundwater occurrence in the area are pronounced along profiles c and d.

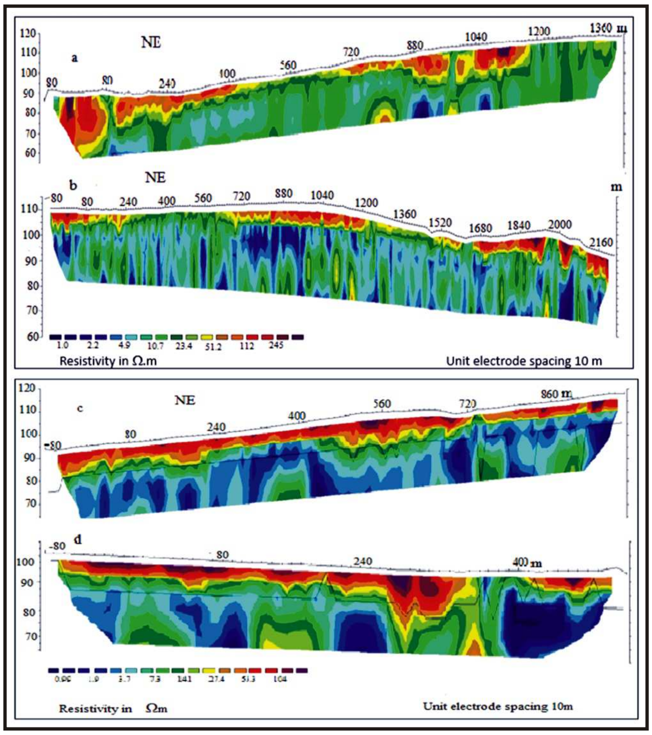

Fig. 3: Pseudo-section Subsurface image of the Study area.

Geotechnical Interpretation

Classification

From the laboratory data, soil samples such as PT1, PT2, PT3, PT4, PT5, PT6, PT7, PT9, PT12, PT 14 and PT15 have more than 35% of their particles passing through No.200 sieve and hence fall under silt-clay materials (Group A-4 or higher) based on AASHTO classification on the one hand (Fig.4). Likewise, the unified soil classification system (USCS) indicates that all the above mentioned pits except for pit PT 2, has more than 50% of their particle size passing through the No.200 sieve and as such are classified as fine grained (Fig.4), in alignment with the AASHTO classification. On the other hand, soil samples such as PIT 8, PIT 10, PIT 11, and PIT 13 with less than 35 % of their particles passing through sieve No.200 (Fig.4) are classified as A-2 soils which are silty or gravelly clay materials good in construction.

Moreso, in agreement with the UCSC are PIT 2, PIT 8, PIT 10, PT 11 and PIT 13 with < 50% of their grains passing through No.200 sieve and as such fall within the coarse materials which are either gravel (G) or sand (S).

The Casagrande plasticity (Fig.5) indicates that most of the samples such as PIT 2, 4, 5, 6, 8, 9, 10, 13, and 16 fall below the A-line and as such classify as silt of intermediate to high liquid limits (intermediate-high compressibility silty materials) while seven of the samples (PIT 1, 3, 7, 11, 12, 14 and 15) aligned above the A-line and thus, classified as CL and CH (Clay of low-high plasticity). Sowers and Sowers (1970) note that PI>31 should be considered high and indicates high content of expansive clays, most probably montmorillonite or illite. The soils in the study area all have plasticity indices less than 30 (Table 1), which implies absence of expansive clays.

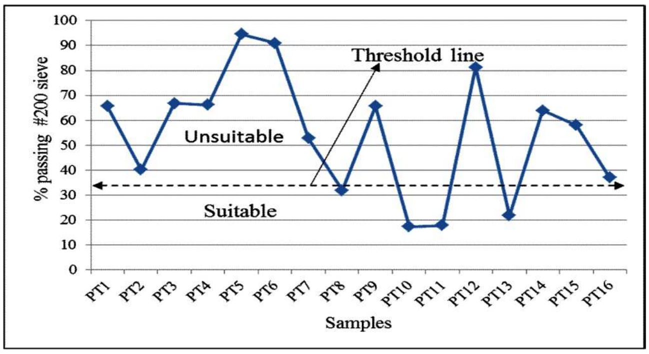

Fig. 4: Samples against % passing sieve #200

Consistency Limits

The consistency in soil properties over a range of water contents is important in demonstrating their likely engineering performances as it influences with other soil properties. In this study, the Liquid limit (LL), plastic limit (PL) and plasticity index (PI) vary as 28.90-62.30% (44.34), 17.40-37.20% (26.36) and 28.50-11.30% (17.91) (Table 1). The Federal Ministry of Works and Housing, FMWH (1997) standards for materials for road construction stipulated a maximum liquid limit value of 50% for falls within (FMWH, 1997) standard specification, making them suitable for as construction additives except samples PIT 2, 6, 13 and 14 whose liquid limits are above the standard and such would require some improvements before usage. More so, Wright (1986) pointed out that LL of ≥40% and PI ≥ 10% are high for material to be used for pavement construction. Again, the four samples (PIT 2, 6, 13 and 14) fall into this category, however, other samples fall short of one of the parameters in excess of the Wright (1986) classification.

Specific Gravity (Gs)

Specific gravity (Gs) of soil, known as a dimensionless quantity, is the weighted average of the soil minerals excluding air and water. It is also an important parameter in the bearing capacity analysis of foundation (Sower and Sower, 1970). In this study the soils have an average G_S value 2.64 (Table 1). However, a potentially durable construction aggregate should have a G_S value 2.625 or above (Reidenouer, 1970), lower values less than 2.6, may indicate presence of organic matter (IS 1498: 1970). Majority of the soils (PIT 2, 3, 6, 8, 10, 11, 13, 16 and 15) has specific gravity values of ≥2.6 making them good for engineering construction. A comparison of specific gravity values of rock aggregates that serve well in construction project with the soil materials from Otobi area points out the fact that some of the samples are unlikely to be durable in most construction projects as their Gs vary as 2.44-2.91.

Compaction and Natural moisture content

Compaction test seeks to simulate the right combination of moisture and load on a soil that would result in increased density of such soil; thus improving its appropriateness in construction projects. The compaction results indicates that the soils achieved a mean maximum dry density (MDD) value of 2.29 mg/m^3 at mean optimum moisture content (OMC) of 13.71% (Table 1). The MDD is considerably high as the

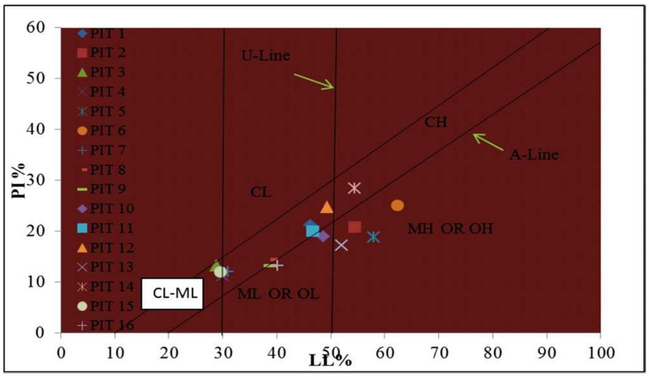

Fig. 4: Plasticity Chart for the Soil Samples (Casagrande, 1948).

values range from 2.00-2.83 mg/m^3 and while the OMC is low in a range of 10.4-17.3% which may be due to the high fine contents of the soils. In addition to compactive effort, Okogbue and Aghamelu (2010) pointed out that cementation, nature of the compacted sample and moisture condition of the sample may have also contributed to the relatively high MDD recorded by some shale formations of southeastern Nigeria.

The recommended MDD and OMC standard, however, depends on specific area of application. For instance, the Nigerian specification for Road and Bridge Materials (FMWH 1970), recommends that for a material to be used generally as fills or embankment it should possess MDD > 0.047 mg/m^3, OMC < 18 % which implies these materials will perform optimally as road construction aggregates. Moreso, the respective average range of values of MDD and NMC should be 1.76-2.56 mg/m^3 and 5-15% (Underwood, 1967).

For the studied soils, considerably higher MDD and NMC values were observed for some samples above (PIT 1, 2, 5, 6, 7, 12, 14, and 16) the stipulated value by Underwood (1967) and thus, could pose construction failures unless its properties are improved upon. The role of natural moisture content (NMC) in pavement soils is crucial in increasing or reducing density indices. The results of the compaction test of the soils shows that all the samples would constitute good pavement or embankment materials since their moisture content value have fallen to the dry side of the compaction curve.

Settlement Prediction

The susceptibility of these soils to settlement was also analyzed using compression index equation for naturally consolidated clays given by Terzaghi and Pecks (1967).

Table 1: Summary of Geotechnical Properties, Swelling potentials and Compressibility index of Soils from Otobi area (MDD mg/m^2)

Samples	LL%	PL%	PI%	NMC%	OMC%	MDD	SG	% #200	AASHTO	USCS	PI	Cc	Terms
PIT 1	46.20	25.00	21.20	20.04	10.40	2.30	2.59	65.59	A-7	S(CL)	High	0.33	Moderate
PIT 2	54.40	33.50	20.90	20.01	12.90	2.20	2.60	40.07	A-7	S(ML)	High	0.40	High
PIT 3	N.D	13.20	13.20	2.66	12.90	2.20	2.60	66.8	A-6	S(CL)	Medium	0.17	Low
PIT 4	30.00	18.70	11.30	13.57	15.60	2.47	2.52	66.2	A-6	S(CL)	Medium	0.18	Low
PIT 5	29.55	10.70	18.85	2.44	10.70	2.00	2.44	94.46	A-7	S(ML)	High	0.43	High
PIT 6	62.30	37.20	25.10	28.70	16.60	2.06	2.61	90.91	A-7	MH	High	0.47	High
PIT 7	31.00	19.00	12.00	20.18	14.30	2.53	2.57	52.76	A-6	S(CL)	Medium	0.19	Low
PIT 8	39.00	24.50	14.50	11.54	12.40	2.83	2.86	31.88	A-2-6	SC	Medium	0.26	Moderate
PIT 9	38.80	25.60	13.20	15.59	14.20	2.36	2.55	65.58	A-6	S(ML)	Medium	0.26	Moderate
PIT 10	48.60	29.60	19.00	15.82	13.50	2.47	2.77	17.30	A-2-7	S(CL)	Medium	0.35	Moderate
PIT 11	46.70	26.60	20.10	17.73	12.80	2.65	2.80	17.73	A-2-7	GM	Medium	0.33	Moderate
PIT 12	49.40	24.70	24.70	81.30	17.30	2.13	2.59	81.30	A-7	S(CL)	High	0.35	Moderate
PIT 13	52.10	34.80	17.30	21.67	11.20	2.72	2.91	21.67	A-2-7	GC	Medium	0.38	Moderate
PIT 14	54.50	26.00	28.50	63.87	16.30	2.31	2.57	63.87	A-7	CH	High	0.40	High
PIT 15	29.50	17.40	12.10	58.00	12.80	1.74	2.60	58.00	A-6	S(CL)	Medium	0.18	Low
PIT 16	40.20	27.00	13.20	37.00	15.20	1.78	2.60	37.00	A-6	G(CL)	Medium	0.27	Moderate
Min.	28.90	17.40	28.50	17.30	17.30	2.00	2.44	17.30					
Max.	62.30	37.20	11.30	94.46	10.40	2.83	2.91	94.46					
Ave.	44.34	26.36	17.91	54.45	13.71	2.29	2.64	54.45					

PI: Terzaghi *et al.,* (1996), Cc=Terms*: Sower and Sower (1970).

Cc=0.009(LL-10)…………………………(1)
Where;
Cc=Compression index
LL= Liquid limit

Table 2 indicates that the high LL exhibited by the fines content of some of the soils, may introduce considerable amount of swelling and subsequent settlement of the soil if used as a foundation material. Logical comparison of the swell-classification schemes and compressibility index proposed by Ramana 1993; Terzaghi *et al.,* 1996,with those from the study area shows that the soils at samples PIT 2, 5, 6 and 14 have high swelling and compressibility potentials while PIT 7, 8, 9, 10, 11 and 13 has moderate potentials. Suitability of the soil as foundation may be improved upon by compaction (Aghamelu et al., 2011b). Compaction reduces void spaces in soil, decreases pore pressure and its consequences in construction projects, thus, enhance its suitability, especially as fills (Sower and Sower 1970; Aghamelu *et al.,* 2011b). Precisely, soils with high swelling potential are not suitable for most earth works as there are likelihood for incessant swelling resulting in uncontrollably differential settlement, however, in geotechnical designs, such soils could be ameliorated by mechanical stabilization prior to use in construction activities.

Conclusions and Recommendations

This paper present results of the geophysical and geotechnical properties of soils from Otobi area, Southeastern Nigeria. The ERT sections revealed a subsurface environment characterized by similar geological materials such as silt, clays and components of the weathered bedrocks (shale and mudstone) whose thicknesses and resistivities varies and/or in conformity with the elevation of the surface. This materials (clay, silt) displayed relatively low resistivity (<34.2-104Ωm) in association with the groundwater zone. Localized high resistivity zones (>250Ωm) which vary with ground elevation at near surface depth, indicating localized lateral resistivity heterogeneities of the material, probably due to the presence of alluvial/ residual deposits. Geotechnical interpretation revealed a dominately fine-garined soils with almost similar proportions of other soil particles as silty gravel (GM), clayey gravel (GC), gravelly clay (GC) mostly of similar propotion. Using AASHTO and USCS classifications indicated predominatly A-4 group except PITS 8, 10, 11 and 13 which are A-2 goup of soils, hence, adjudged as excellent-good soils for construction. The absence of expansive clays in the studied soils as indicated by their plasticity index value, coupled with the low to medium swelling potential of the soils would make them good construction materials. The specific gravity of most of the soil samples (≥2.6) and the compaction test results (average MDD of $2.29mg/m^3$) also suggests that the soils are good as construction material, especially with the MDD and OMC falling dry of optimum.

Therefore, this study have shown that the ERT technique can reasonably be used in identifying soils physical properties which can be valuable for civil engineering purpose. The technique is particularly advantageous, owing to its comparatively cost and time effective and non-invasive nature. Furthermore, the

ERT can be utillised for characterising and mapping verse areas and can augment and sometimes substitute laboratory soil investigation characterised by large cost and time inefficiency. Likewise, the damageability and destruction of sites associated with geotechnical studies is eliminated contributing to sustainable and green environment in the construction industry as electrical resistivity techniques uses non-invasive approach.

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Geo-Engineering, A Vital Tool for Spatial Planning Initiation for Infrastructural Development in Africa: A Situational Discussion on Ghana

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Abstract
The African Development Bank (AfDB) has reported that the key challenge towards infrastructural development in Africa is to supply the increasing population with reliable electricity, affordable housing and transport infrastructure. Spatial planning idea is to introduce environmental sustainability in this developmental efforts. Ghana, in 2016, put into law a Land-use and Spatial Planning Authority (LUSPA) to enforce spatial planning issues. The law has provided considerable documents to support this agenda with opportunity for geo-engineering professional to make inputs. However, this opportunity has not been effectively patronised. Better implementation of spatial planning needs Geo-engineering as an enabler to promote site characterization, which is key to providing economic and optimized design, identification of anticipated hazards, sustainable construction environment, value for money and spatial planning arrangement including appropriate site selection among others. This keynote seeks to provide a conversation on the importance of Geo-engineering to support spatial planning link with infrastructure development.

Mr. Chairman, colleague engineering practitioners, ladies and gentlemen. I bring you warm greetings from Ghana and thank you very much for this opportunity to share a conversation on *Geoengineering, a vital tool for spatial planning initiation for infrastructure development in Africa: a situational discussion on Ghana*.

Mr. Chairman, now there is much talk about spatial planning, which in essence promotes environmental sustainability for infrastructure development and enhancement of quality of life. One important objective of spatial planning is to make sure that exploitation of land resource is planned and implemented in an organized manner to meeting sustainable targets. As it stands, infrastructure needs for Africa is paramount because of the rate of population growth and general urbanization. African Planning Association (APA), in its report on the state of planning in Africa (2014), indicated that critical challenges facing spatial planning include:

a) Rapid urbanization
b) Inadequate infrastructure provision
c) Inadequate public transport and
d) Poor city management

Mr. Chairman, ladies and gentlemen, taking Ghana in context, spatial planning is governed by law: ACT 925 (Land use and Spatial Planning ACT) in 2016. This applies to all (both public and private) concerned with land development of whichever form. The guidelines for this spatial planning model proposes a 3-tier planning system as follows:

a) Spatial Development Framework (SDF) – planning framework for national, regional and district levels;
b) Structure Plan (SP) – applies to all areas under Metropolitan, Municipal and District Assemblies (MMDAs) to cater for towns and fast growing urbanizing areas;
c) Local Plan (PL) – provides detailed land use plans where individual plots and their uses are clearly defined

These planning systems have connection with national development strategies. It also provides view and proposals for what kind of development should take place, how much of it should occur, where this should happen and how this should happen. This spatial development should be agreed by the primary stakeholder (MMDAs) concerned. For example, the Structure Plans (SP) demarcates alignment and corridors of trunk and major transportation routes among others. It further defines areas where no particular use is designated and which is expected to change during the period the plan is valid. The planning system document should contain a map or key diagram which illustrates the general content of the spatial strategy, and shows the physical extent of proposals, but does not identify specific site boundaries (these will be mapped at SP level, or in individual applications, where these fall outside of SP areas). The data layers and compositions of the spatial planning process include characteristics that support general infrastructure development. There is the inclusion to look at flood and seismic potential areas to support spatial planning discussion. The zoning concept ensures that utilization of land best suites its functions. Such functions include mitigating the impact of traffic, natural hazards, types of structures, environmental sanitation, among others. This implies that zoning concept prescribes permissive and prohibitive uses for land.

Mr Chairman, Ghana has made strides in this spatial planning agenda by developing the relevant documents to enhance its implementation. We can see that there is the need for the potential involvement of Geo-engineering professionals. However, much has not been done in this regards. It is probable due to lack of proper coordination in soliciting the appropriate Geo-engineering input in the initial delivery process. In most cases, site characterization is usually sought during the implementation of infrastructure. The question is *"what if the site characterization depicts something else that may influence zoning needs?"*. This is what I refer to as secondary phase of planning and should considerably be avoided. Geo-engineering should not be used as a secondary tool for spatial planning. This is because general site characterization maps should be primary to define zoning potentials and concerns – *what can be done, what should be expected, among others, due to the ground and other geological conditions*. The current challenge has been to monitor whether this spatial planning concept is yielding good results in a sustainable manner. In my view, the task is daunting especially during this era of already urbanized cities. Looking at it, better planning maps require serious geo-engineering considerations.

Mr. Chairman, ladies and gentlemen, spatial planning has become or is becoming part of us in this Region. However, one main criticisms of this planning system is the lack of interactive coordination among relevant players. One important lack of coordination is soliciting for the appropriate Geo-engineering inputs during the early stages of spatial planning initiation. The planners, our friends, will always talk about instruments that promote management of space towards such development. However, as to whether that space will provide economic and sustainable infrastructural development based on site characterization is something else to address. Therefore, I recommend an effective collaboration between Planners and Geo-engineering practitioners on this planning path.

The zoning concept should realize that every infrastructure development comes with significant cost implication and land – the earth – is the most important component.

The land will have to offer adequate capacity to support structures, provide suitable access for construction, ensure that hazard and its associated risk are known and offer construction materials within economic reach, etc. Indeed, it is the ground that has key influence any infrastructural design and construction cost.

Mr. Chairman, infrastructure development is always in the domain of civil engineering and I am confident to say that Geo-engineering is and must be the first point of call before such development takes place.

Important available Geo-engineering tools that support infrastructural development include:

- Engineering Geology
- Geotechnical Engineering and
- Geological Engineering

The backbone for these tools is Geology. This is because it studies the earth, the materials in it, the arrangement of those materials and forces acting on them. The understanding of Geology helps in solving human problems such as water, energy, mineral resources, planning for the environment and natural hazards (landslide, earthquake, volcano and flood). Therefore, the various tools serve as an application of Geology and a good Geo-engineering practitioner is the one who utilizes the full knowledge of Geology.

A brief description of each tool is as follows:

- **Engineering Geology** – *Dearman (1972) summarised engineering geology as the application of geology to civil and mining engineering as applied to design, construction and performance aspect of engineering structures in, or on, the ground.*
- **Geotechnical Engineering** – *the Electronic Journal of Geotechnical Engineering (ejge) (http://www.ejge.com/Definition.htm) summarised geotechnical engineering as branch of civil engineering concerned with the analysis, design and construction of foundations, slopes, retaining structures, embankments, tunnels, levees, wharves, landfills and other systems that are made of or are supported by soil or rock. I should say, a good geotechnical practitioner craves for knowledge in geology.*
- **Geological Engineering** – *the University of Columbia, in Canada, describes this subject as assessment of landscapes where dams, mines,*

roads, railways, forestry and other extractive operations are planned. I continue to say, a good geological practitioner craves for knowledge in geology.

Mr. Chairman, key to this spatial planning concept is infrastructural development. Spatial planning for infrastructural space organization requires good coordination of *practices* and *policies*. A broad set of knowledge is required by planners to link spatial planning and infrastructure. This means that there should be some systems used by planners or implementers to influence the distribution of infrastructure. One of the key practices that influences spatial planning for infrastructure is Civil Engineering with Geo-Engineering as an associated system. This is because the associated system deals with *assessment of the space*. The assessment of the space is crucial as it defines or determines its usability in terms of infrastructural cost and risk.

Geo-engineering can play the following primary roles linking spatial planning initiation and infrastructural development.

1. Produce maps showing spatial distribution of rocks and associated structural geology
2. Produce maps indicating potential hazards areas due to geological structures or potential natural disasters
3. Produce maps showing geo-engineering characteristics of overburden ground
4. Produce maps showing hydrogeological characteristics of the ground and
5. Produce maps showing features of extractive metals and their mining potential.

These primary roles should be the basis for spatial planning systems such as re-organization of space or zoning activities. Spatial planning policies should, firstly, ensure that these maps are in place and serve as desk study documents.

Mr. Chairman, again, these primary roles can be the basis for spatial arrangement of infrastructure with value for money, resilient and sustainability in mind.

In conclusion, Mr. Chairman, it is high time Geo-engineering practitioners played our roles to influence national development by having strategic engagements with policy makers and relevant implementing agencies within our Region. We should start the effort by purposefully collating practitioners within our Region using relevant professional bodies and other vital platforms to make our voices heard. Thank you.

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The Risks of Luxury: from the Perspective of Geology and Physics of Radon in Tropical Environment

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Abstract

Radionuclides have been in existence since the beginning of creation. They are present in the air we breath, the water we drink and the food we eat. They are in soil we grow our crops, in building materials we use to construct our homes, hospitals and industries. In most parts of the world, rocks and soil serve as the main building materials and as they naturally contain trace amount of Uranium (238U) and thorium (232Th), they can be important sources of radionuclides radon (222Rn) and thoron (220Rn) in their respective decay chains. Being a radioactive gas, radon can escape from soils and building materials where they are formed, and escape into indoor air. Radon escape from basement, soils and building material can build up indoors where they are eventually inhaled. The movement and exhalation of radon is controlled by many factor. Radon has been implicated by the International Atomic Energy Agency (IAEA), World Health Organization (WHO) and the IARC as a cause for many forms of cancer. Indoor radon has been identified as an important public health problem, estimated by the United State Environmental Protection Agency (US, EPA) to have caused about 21, 000 lung cancer deaths per year in America. Moreover, numerous public health agencies rank residual radon exposure as the second leading cause of lung cancer after cigarette smoking. This paper analysis at radon exhalation by different building materials and their radiological implications compounded by choice of building materials and luxuries associated with certain life styles in tropical environment.

***Keywords**: Radon; cancer; inhalation; building materials; risks*

Introduction

Radon originates as a part of the natural decay process of uranium and thorium (Figure 1). It is an element which boasts very curious characteristics-among gases; it is rare due to its density and its exclusively-radioactive isotopes. It is everywhere in the earth's crust; therefore, radon is found in varying concentrations on most part of the planet. However, it is not until radon is concentrated in buildings, or in underground workplaces, that it poses a health risk to humans. Radon has affected human health throughout history although it wasn't until 1988 that it was officially declared a human carcinogen. The World Health Organization first drew attention to the health effects from residential radon exposures in 1979, through a European working group on indoor air quality. Furthermore, radon was classified as a human carcinogen in 1988 by IARC, the WHO specialized cancer research agency.

Radon from the soil seeps through cracks in buildings' foundations, floors and walls. In some instances, the building materials can be an important source of indoor radon. If the building is not well ventilated, then the gas will concentrate in the lower floors where it is breathed in by occupants. The gas concentrations can vary significantly from place to place and indoor concentrations are affected by climate and temperature changes. This variability makes testing challenging. Additionally, studies have shown that there is no minimum threshold below which radon levels do not pose a risk to human health. Therefore, any level represents an increased risk of lung cancer.

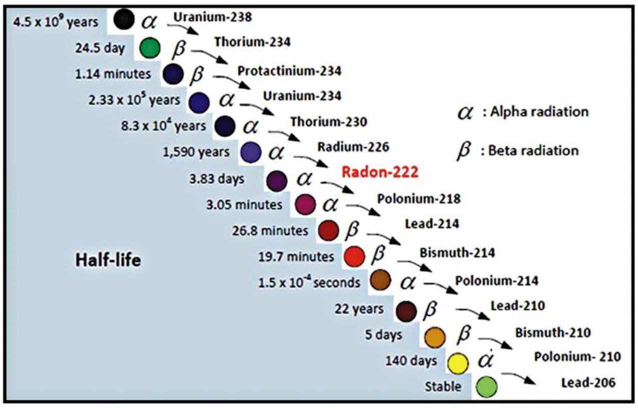

Fig. 1: Radon decay chain (Katheleen M. Server, 2019)

The necessary technology exists to reduce radon concentrations in homes and workplace. However, householders are often reluctant to measure and to spend money on corrective actions. Cost-benefit analysis has shown that prevention strategies are the most cost effective way to reduce radon exposure. Architects and building contractors are key stakeholders and need to be educated on the need to design buildings to prevent accumulation of radon.

The International Atomic Energy Agency (IAEA, 2017) has prepared a new Safety Guide - in cooperation with the World Health Organization (WHO, 2007; 2008) - describing how to develop a national action plan to assess and, if necessary, reduce exposures due to radon (and also gamma radiation) in homes by member states. The Safety Guide includes advice on the provision of information, conducting radon surveys, defining measurement protocols, developing national policies and evaluating effectiveness. Many Member States already have a national plan in place to protect their population from radon and all others are urged to consider establishing such a programme.

The paper is aimed at sensitizing the populace with regards to health implications related to radon in homes and the steps that could be taken to reduce radon levels in homes and offices.

How Radon Gas Gets into our Homes

Radon in homes is usually much higher in the basement than in other part of a house. It can enter homes through construction joints, cracks in the solid floors, cavities inside walls, gaps around service pipes, gaps in suspended floors, cracks in the wall and in the water supply (Figure 2). These are mainly descriptive of buildings design and constructed using concrete as pillars and foundation while partitioning are done with woods, plastic, glasses and aluminum. In the case of our country and many others in the tropical regions, almost the entire construction is made from concrete walls, pillar and basement. Due their geogenic origin, these materials contribute and sometimes are the main sources of radon in our homes.

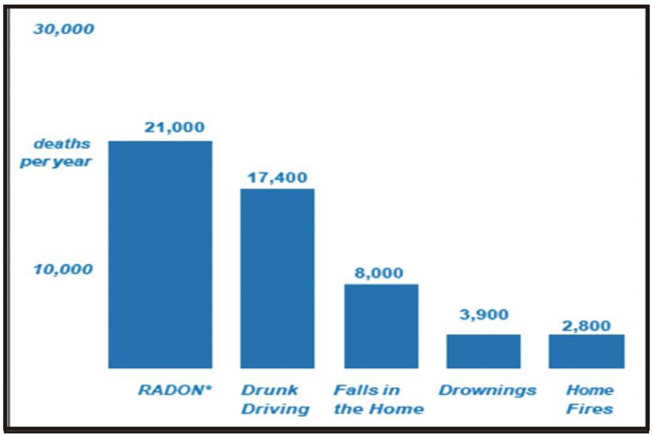

Fig. 2: How radon gas gets into our homes (Nelson Bill, 2008, visited 22nd July, 2019)

Causes of Death Associated with Radon Compared with other Death Causes

Studies by the United State Environmental Protection Agency (EPA) (Fig. 3) showed that lung cancer caused by radon inhalation is rank third among other mortality for selected cancers (leukemia, lymphoma, liver and intra-hepatic bile duct, thyroid and bone/joint among other) in the United State (EPA, 2003). It is also ranked the highest cause of death among some select causes (drunk driving, falls in homes, drowning and fire in homes) in the United State (Fig. 4). Studies in Europe indicates that the risk of lung cancer versus long-term average residential radon concentration is exponential (Fig. 5).

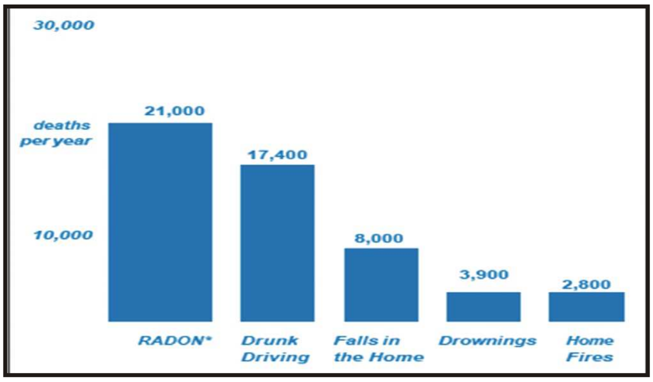

Fig. 4: Causes of death per annum (EPA 2003)

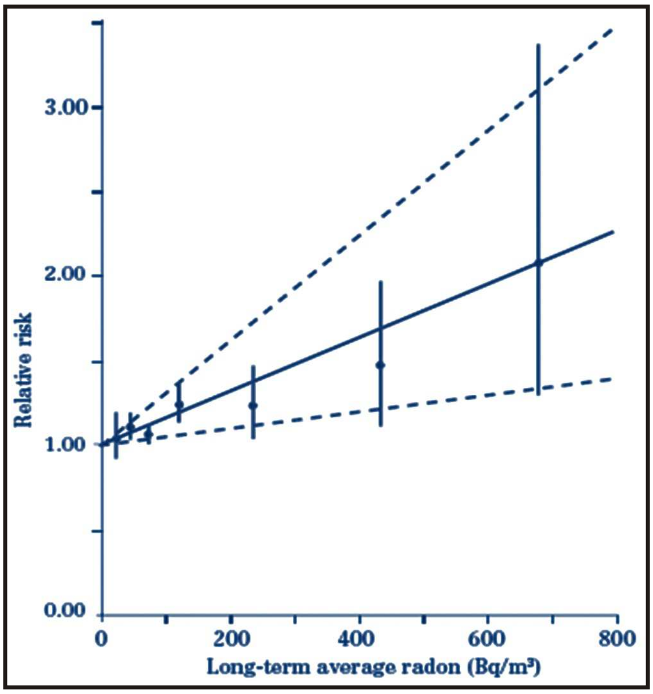

Fig. 5: Relative risk of lung cancer versus long-term average residential radon concentration in the European pooling study (Darby et al., 2005).

Sources of Radon in Homes

Radon in homes and workplaces are closely related to the types of homes, building materials use, home design and living styles. Buildings differ from one country to the other depending on the geographical, living condition, culture and status of home owners. Homes design with basement are likely to have higher radon levels than those without. Also, different building materials have different level of radon with granite used as building aggregate, Phosphogypsum used as ceiling and wall screening material (pop) having the highest values because of their radionuclides content. The design of homes also plays a vital role in radon build up in homes, with building designs lacking proper ventilation likely to have higher radon build up, so also is the life style and status of occupants have a bearing on the radon levels in homes.

In places where homes are build from woods and little concrete work as foundation and pillars, radon entry is mainly from basement and water supply.

Home Types and Building Materials

For the sake of evaluation of radon in dwellings, homes in this study are grouped into three base on status and living conditions of home owners, these are low income, medium income and high income earner labelled as 1a and b, 2a and b and 3a and b, respectively (Fig. 5). And materials utilized in building these homes as follows:

Low income:	Laterite, mud and sand
Medium income:	Sharp sand, gravel aggregate, plaster sand, cement and concrete block
High income:	Sharp sand, gravel aggregate, plaster sand, cement, phosphogysum (pop), quarry dust and vibrated concrete block

Fig. 5: Classification of home type (1= low income, 2 = medium income, 3 = high income)

Radon Exhalation and Emanation Coefficients of Building Materials

Studies of radon exhalation rate and coefficient for different building material indicates that Phosphogypsum has the highest while cement has the lowest (Arabi *et al*., 2015).

The evaluation of some parameters that are likely to cause exhalation and build-up of radon in homes

Table 1: Average radon exhalation and emanation coefficient for different building materials

Building material	Radon exhalation (Bqm^{-3})	Emanation coefficient
Cement	7±0.3	0.0036
Gravel aggregate	44±7	0.0103
Sharp sand	60±1.0	0.0207
Plaster sand	117±12	0.0388
Concrete block	123±12	0.0021
Phosphogypsum	2,684±22	0.0472
Laterite	37±2	0.0032
Mud	26±1.8	0.0031

indicates the likelihood of homes of high income are likely to be higher because of the kind of materials used in construction, luxury of some houses which are likely to cause radon build-up (Table 2). Results obtained indicates that high income houses could have a total exhalation rate from materials use in construction to up to 3,035 Bqm^{-3} while low income could be as low as 180 Bqm^{-3}.

Table 2: Evaluated results for radon in different home type and corresponding radiological values

Parameters Evaluated	Radon in Low income	Radon in Medium income	Radon in High income
Building material			
Laterite	37±2		
Mud	26±1.8		
Plaster sand	117±12	117±12	117±12
Sharp sand		60±1.0	60±1.0
Gravel aggregate		44±7	44±7
Cement		7±0.3	7±0.3
Concrete block		123±1	123±12
Phosphogypsum			2,684±22
Total Radon	180 Bqm^{-3}	351 Bqm^{-3}	3,035 Bqm^{-3}
WLM at home	0.79 (3.16mSv)	1.54 (6.16 mSv)	13.35 (53.4 mSv)
Ventilation	High	Medium	Low
Radon build-up	Background	Medium	High

Note: average background radon = 17 Bqm^{-3}

Exposure to radon daughters is expressed in working level months (WLM) and a working level month is equivalent to the exposure at an average concentration of 1 WL for 170 working hours. Recommended values by the EPA, NRC and USDOE are 4 and 12WLM per year for radon and thoron, respectively.

Calculation of Working Level for Radon in Homes and Offices

The value for radon exposure in homes and offices differ due to the exposure time, which is critical in the analysis of radiological evaluation. The values obtained had assumptions as follows;

(A) At home : assuming 7,000 hours spent indoors per year

1 Bq m^{-3} = 0.0156 mJ h m^{-3}

1 Bq m^{-3} = 0.0044 WLM

1 WLM = 4 mSv

1 mJ h m^{-3} = 1.1 mSv

(B) At work : assuming 2,000 hours work per year

1 Bq m^{-3} = 0.00445 mJ h m^{-3}

1 mJ h m^{-3} = 1.4 mSv

1 WLM = 5 mSv

Source: ICRP Publication 65, Protection Against Radon at Home and at Work

by this assumption, WL for high income (3a and 3b) houses has the highest WL of 13.35 which is almost three times the level recommended by the EPA, NRC and USDOE. But this is not where the problem is, because even homes with high level radon can be controlled by allowing doors and windows opened, by this practice, levels are brought down to background. Category 3a and 3b homes are mostly on 24/7 electricity, as a result, windows are always shut and air conditions on. In this homes, radon can build up to alarming levels and might not be noticed and this is what is referred here as risks of luxury. The luxury of air condition, building homes with materials of high level of radionuclide, high exposure time, are some parameters that makes homes number 3a and 3b, radon prone.

Epidemiological investigations are still on for it to conclude the findings which may link the different homes in this study to the levels of radon induced cancer. Building material are mostly geologic in origin and the physics of radon has implicated the choice of building material, the kind of living condition and exposure time in homes as possible parameters that controls the radon levels especially in tropical areas.

As can be seen in Table 4, the highest indoor radon levels in OECD is 90 Bqm^{-3}, in this study, I could be much higher considering the kind of building material used and the design of homes where radon is completely not taken into consideration. In fact, most of the populace are not even aware of a thing called radon despite the risks associated with it.

EPA Recommends that for individual to buy or sell their homes, the following conditions must be met:

- ✓ If you are selling or buying a home, have it tested for radon

Table 3: Indoor radon concentrations in OECD countries

	Indoor Radon Levels [Bq/m³]		
Country	Arithmetic mean	Geometric mean	Geometric standard deviation
OECD countries			
Australia	11	8	2.1
Austria	99	15	NA
Belgium	48	38	2
Canada	28	11	3.9
Czech Republic	140	44	2.1
Denmark	59	39	2.2
Finland	120	84	2.1
France	89	53	2.0
Germany	49	37	2.0
Greece	55	44	2.4
Hungary	82	62	2.1
Iceland	10	NA	NA
Ireland	89	57	2.4
Italy	70	52	2.1
Japan	16	13	1.8
Luxembourg	110	70	2
Mexico	140	90	NA
Netherlands	23	18	1.6
New Zealand	22	20	NA
Norway	89	40	NA
Poland	49	31	2.3
Portugal	62	45	2.2
Republic of Korea	53	43	1.8
Slovakia	87	NA	NA
Spain	90	46	2.9
Sweden	108	56	NA
Switzerland	78	51	1.8
United Kingdom	20	14	3.2
USA	46	25	3.1
Worldwide average	39		

Sources: WHO (2007), UNSCEAR (2000), Billon et al. (2005) and Menzler et al. (2008).

- ✓ For a new home, ask if radon resistant construction features were used and if the home has been tested.
- ✓ Fix the home if the radon level is 4pCi/L or higher
- ✓ Radon levels less than 4pCi/L still pose a risk and, in many cases, may be reduced
- ✓ Take steps to prevent device interference when conducting a radon test.

Radon Prevention and Mitigation

Radon in homes and offices can be prevented by using select materials for construction, installation of radon detection devices in homes such that when levels are higher, it can alert home owners to take necessary action, by reducing the residence time, by dosing the level to background through venting of houses etc.

Also, the following step could go a long way in putting in place a long time prevention and mitigation measures:

1. Strategies both for radon prevention (new dwellings) and mitigation (existing dwellings) are needed to achieve an overall risk reduction.
2. Radon sources, radon concentrations and radon transport mechanisms influence the choice of prevention and mitigation strategies.
3. Radon measurements should always be made to determine the effectiveness of any radon prevention or mitigation effort.
4. Professionals in the building sector are key players for radon prevention and mitigation. Strategies are needed to train them and to ensure their competence in this area.
5. Research-based guidelines and/or standards for radon prevention and mitigation should be established at national level.

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Spatio - Temporal Characterization of Morphological Dynamics and Vulnerability along the Niger Delta Coastline

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Introduction

The Niger Delta coastline is a 450km long stretch along the Atlantic ocean in the Gulf of Guinea, Nigeria. It is one of the most dynamic coastlines in the world due to its exposure to the triple effects of wave, current and tide impacting on it. Its offshore subsurface geology houses a significantly prolific source of huge reserves of commercial quantities of petroleum hydrocarbons. Due to this property the coastline, near and far offshore environment boasts of a significant number of oil and gas infrastructure and related activities. Fishing, navigation and other commercial activities are also very prevalent in this region due is rich reserves of marine livestock.

Several natural processes such as sea level rise, subsidence and compaction, storm surges, coastal flooding and erosion also impact the coastline. Human induced factors that impact the shorelines include artificial diversions of river path, channel dredging, erection of harbour protecting structures, dam construction, mining of beach sand and de-vegetation which are also prevalent in these areas due to the large number of industrial and commercial activities. Changes in inland river discharge, sediment load, daming, geometry and the course of river and stream in the lower River Niger and its main tributary and distributaries also contribute to a dynamic coastal morphology.

All these factors have put a lot of stress in the natural environment leading to a dynamically changing coastline morphology. The intensity and impact of these processes and conditions are not uniform across the entire coastline. Hence, the imperative for it to be understood and characterized.

Inland Fluvial Environment

The main source of fluvial discharge into the Niger Delta and coastline is the Benue and Niger Rivers. Sediment laden waters of the River Benue-Niger river system travels into the Delta down river via the bifurcation of the two main distributaries, the Nun and Forcados rivers, some 100 km south of the apex creating a coastline spanning of over 450 km. Water and sediments from the delta empties into the Atlantic Ocean through at least 21 estuary outlets, the major rivers being: Imo river, Bonny river, San Bartholomew river, Brass river, Nun river, Pennington river, Dodo river, Ramos river, Forcados river, Escravos river and the Benin river. These rivers which make up the main drainage outlet have large river mouths of about 1.3 km to 5 km, opening into the Atlantic Ocean.

Bathymetric Environment

The offshore area of the Niger Delta can be divided into shallow offshore and deep offshore, although several classifications have been used by different researchers, institutions and agencies. The shallow offshore originates from the coastline and progresses to the continental shelf where depths of up to 80m are evident. In these areas sand shoals and canyons exist, such as the Mahin and Calabar canyons (Fig.1) . The deep offshore is characterized by depths of over 100m where there is an abrupt drop to the continental slope.

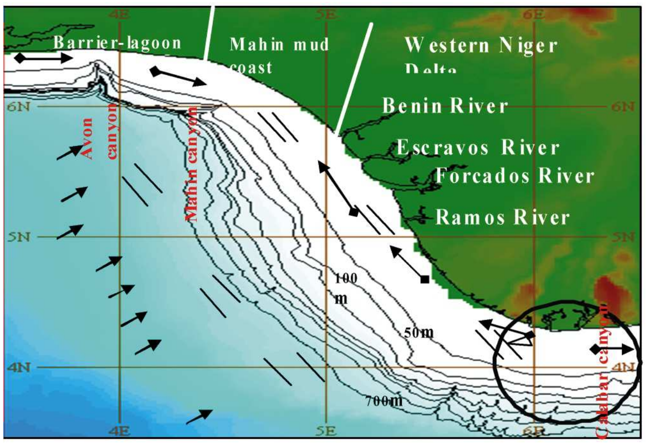


Fig. 1: Bathymetric map of the Niger Delta coastline showing oceanographic forces (Awosika 2000).

Morphological Setting

The morphology of the coastline which extends from Benin River in the west to Imo River in the east reveals several features made up of six physiographic environments (Fig. 2). It has a sub-aerial and a sub-aqueous component. The sub-aerial part of the coastline is characterized by Beach-Ridge and Barrier Islands. While the sub-aqueous part consists of Distributary mouth bars, Delta front platforms and Prodelta slope and open shelf.

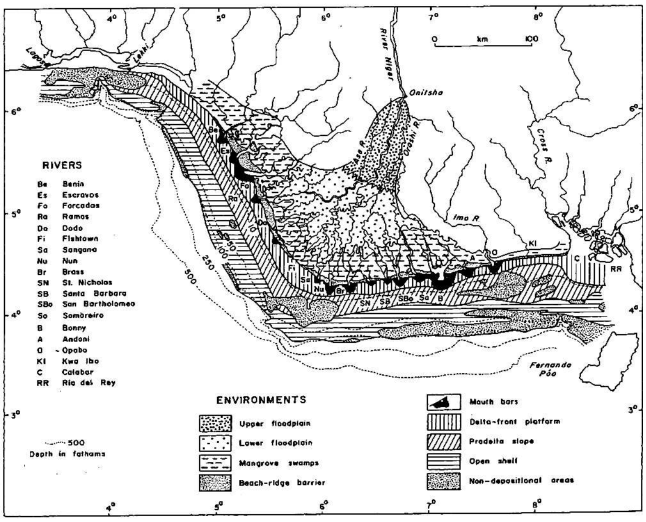


Fig. 2: Physiography and principal sedimentary environments in the Niger Delta. Source: Abam (1998).

Metocean Setting

The coastline areas of the Niger Delta are generally described as a wet tropical environment with a warm and humid climate and is characterized by a long rainy season from March to April through to October (Okude and Ademiluyi 2006). The number of rainy days varies from 290-310 days with annual rainfall amount usually above 3,000mm. Rainfall, in the relatively dry months, ranges from (22.8 to 97.7) mm (November to February), while the wet months range from 125.3 to 464.9 mm. The coastal area is impacted by the triple phenomena of tide, wave and currents action and this result in changing morphology (Obowu 2014).

Data and Methodology

Medium resolution satellite imagery (Landsat - 8 ETM and Sentinel - 2) acquired between 2015 and 2018 was used for this study. This data is used to compliment and to progress the work on Spatial and Multi-Temporal Change Analysis of the Niger Delta Coastline Using Remote Sensing and Geographic Information System (Obowu & Abam 2014). Their earlier work involved the use of multi-temporal Landsat MSS(1972-1976), Landsat TM(1986-1988), Spot 4(1998-2000) and Landsat ETM(2007-2008. Several image processing methods were applied on these two datasets to characterize morphological changes along the coastline and major river mouths. The processing methods include Image classification, image differencing, post classification combination overlaying, image fusion/ mosaicing, and image interpretation.

To adequately characterize morphology, global STRM 30 data (US Geological Society) was used to reveal elevation profiles around the coastline. The data was integrated with satellite and other ancillary data and image processing methodologies to aid characterization of morphology. Geomorphological characterization is key to the analysis of vulnerability along the coastline. The vulnerability assessment involves a five-step approach:

Step 1: Create inundation depth map using DEM
Step 2: Determination of slope and aspect of coastline using DEM
Step 3: Vulnerability mapping
Step 4: Raster and vector data integration and processing
Step 5: Vulnerability and risk analysis

Coastal vulnerability index (CVI) used a quantitative representation of the degree of vulnerability for each coastline section of interest. This is calculated as the square root of the geometric mean of these values, or the square root of the product of the ranked variables divided by the total number of variables as:

$$CVI = \sqrt{(a*b*c*d*e*f)/5}$$

where, a = geomorphology, b = coastal slope, c = shoreline erosion/accretion rate, d = mean tide range, and e = mean wave height.

The ranking matrix used as input the CVI algorithm in calculating vulnerability of the coastline is represented in the table below:

Geomorphology: coastal landform is categorized into 5 classes,

Class 1 - Rocky, cliffed coasts, Fiords, Fiards
Class 2 - Medium cliffs, Indented coasts
Class 3 - Low cliffs, Glacial drift Alluvial plains
Class 4 - Cobble beaches, Estuary, Lagoon

Table 1: Vulnerability Ranking Matrix for Wave Dominated Deltas

	Ranking of Coastal Vulnerability Index				
	Very low	Low	Moderate	High	Very high
Variable	**1**	**2**	**3**	**4**	**5**
Geomorphology	Rocky, cliffed coasts Fiords Fiards	Medium cliffs Indented coasts	Low cliffs Glacial drift Alluvial plains	Cobble beaches Estuary Lagoon	Barrier beaches Sand Beaches Salt marsh Mud flats Deltas Mangrove Coral reefs
Coastal Slope (%)	> 1.9	1.3 - 1.9	0.9 - 1.3	0.6 - 0.9	< 0.6
Relative Sea-Level Change (mm/yr)	< -1.21	-1.21 – 0.1	0.1 – 1.24	1.24 –1.36	> 1.36
Shoreline Erosion/ Accretion (m/yr)	>2.0 Accretion	1.0 –2.0 Accretion	-1.0 – +1.0 Stable	-1.1– -2.0 Erosion	< - 2.0 Erosion
Mean Tide Range (m)	> 6.0	4.1 – 6.0	2.0 – 4.0	1.0 –1.9	< 1.0
Mean Wave Height (m)	<1.1	1.1 – 2.0	2.0 – 2.25	2.25 –2.60	>2.60

Class 5 - Barrier beaches, Sand Beaches, Salt marsh, Mud flats, Deltas, Mangrove, Coral reefs
The geomorphology variable expresses the relative erodibility of different landform types. These data were derived from processed satellite images.

Slope Analysis: The slope permits an evaluation of not only the relative risk of inundation, but also the potential rapidity of shoreline retreat, because low-sloping coastal regions should retreat faster than steeper regions (Pilkey and Davis, 1987). Slope is gradient, or rate of maximum change in z-value (height or depth) from each cell of a raster surface. Slope computation was performed along the entire coast line using data from the generated digital elevation model. The formula: Elevation difference between pts(ΔH)/Distance between pts(ΔD). The data extracted from the digital formed the bases of measurements of elevation difference between the beach front and the barrier islands, which form a natural shoreline protection.

The slope of the coastal zone was calculated from a grid of topographic and digital elevation models extending landward and seaward of the shoreline. To compute the slope from the sub aerial coastal plain to the seaward tip continental shelf, the slope for each grid cell was calculated by defining elevation extremes within a 10 km radius for each individual grid cell along the shoreline. In areas where the shelf/slope break is less than 10 km, the slope was recalculated with a more appropriate radius.

Shoreline erosion and accretion rates for the Niger delta was obtained from extracted coast line positions using satellite remote sensing and GIS techniques from an earlier project covering a 46-year analysis period and the 3-year change extracted from the use of Landsat and Sentinel – 2 satellite imagery datasets.
Tide range is linked to both permanent and episodic inundation hazards. Tide range data were obtained from the tidal prediction table from the office of the hydrographer of the Nigerian navy.

Wave height is used here as an indicator of wave energy, which drives the coastal sediment budget. Wave energy increases as the square of the wave height; thus, the ability to mobilize and transport beach/coastal material is a function of wave height. In this study, we used wave data obtained from few offshore installations off the coast of the Niger Delta.

Results and Discussion

Characterizing Niger Delta coastline morphological changes and vulnerability to erosive forces of the ocean are the primary objectives of this research work. This was achieved using geospatial raster, vector and metocean datasets and tool in an integrated and analytical manner aid by image processing and geoprocessing tools. ENVI, ArcGIS and model builder toolkits proved useful in image processing, from georeferencing, landcover classification, image differencing to calculations for deriving and representing CVI. Due to tidal effect, for coastal water and land classification, it is always difficult to identify whether the classified coastline represents the low tidal line or occurring between the two.

Morphological Changes

Between the period of 2015 to 2018 there have been a few changes in the morphology of the coastline, and these are mainly around the mouths of some of the major rivers. These are mainly evident in the expansion or disappearance of spit, barrier bars and beach landforms. These changes are most probably related to a combination of factors such as changes in discharge and sediment load from the River Niger, its tributaries and distributaries, human induced changes and the triple effect of current, tide and wave action.

The entire Niger Delta coastline is not changing (whether erosion or accretion) in the same rate or pattern. Most of the changes are in found in the eastern Niger Delta around Opobo river, Bonny river, new Calabar river, San Bartholomew river mouths and beach area. in the western Niger Delta coastline, most of the changes are around the beach area and river mouths of Pennington river, Ramos river and Forcados river. The changes in morphology are primary related to the emergence and extension of barrier island, spits and minor lagoons. The origin of these features is probably a combination of man-made factor as several oil and gas facility exist in these areas and have altered the natural landform, and natural action of wave, tide, current and river discharge.

Table 2: Morphological Changes from Andoni to St. Nicholas River Mouth

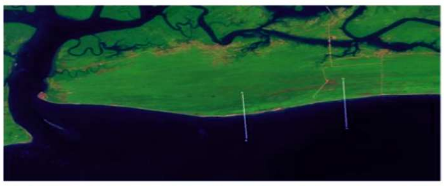
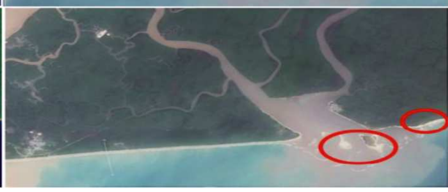
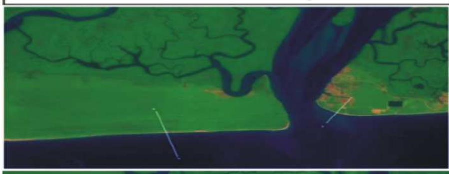
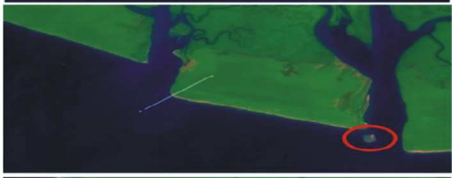
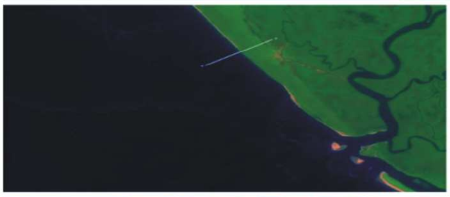
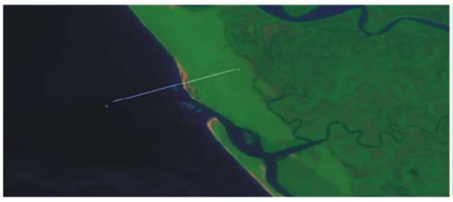
Landsat 8 – January 2015	Sentinel 2 – December 2018	Observation
		The coastal landform around the Andoni river mouth has remind generally the same. However east of the river mouth exhibits rapid development of a seaward spit. This is most likely the result of deposition of sand and spreading by ocean current and other hydrodynamic forces.
		Evidence of coastal erosion along shoreline with vegetation loss and increase in beach area. This may be related to the oil/gas facilities which making beach landing. Spit and vegetation growth is observed at the mouth of the Bonny river, indicative of increasing soil stability.
		The eastern portion of the New Calabar river remained stable. While the western portion shows evidence of beach erosion and some element of sediment deposition as is evident in spit development and island formation connecting landward to the coastline.
		The San Bartholemew mouth shows a general pattern of beach erosion along the beach and river mouths.
		The St. Nicholas river mouth shows a general pattern of beach erosion along the beach and river mouth
		The St. Nicholas river mouth shows a general pattern of beach erosion along the beach and river mouth.

Table 3: Morphological Changes from Brass to Pennington River Mouth

Landsat 8 – January 2015	Sentinel 2 – December 2018	Observation
		The Brass river mouth and coastline little or no significant change in morphology, as it has remained generally stable through out the 3 year study period.
		Nun & Sangana river mouths and coastline exhibit minimal change in morphology, as it has remained generally stable. However there was a complete wash away of the island on the entrance of the fishtown river mouth. This is very likely due to increase in velocity of river discharge coming from the inland river.
		The Fishtown and Kulama river mouth and beach area show clear evidence of deposition of beach material. This is very likely the resultant effect of longshore current spreading sediments along the coastline. This situation is likely to impede navigation as the river mouths are gradually closing up due to sedimentation.
		The Digatoro river mouth and beach area show evidence of deposition of beach material and formation of barrier bars. This is very likely the resultant effect of longshore current spreading sediments along the coastline. This situation is likely to impede navigation as the river mouths are gradually closing up due to sedimentation.
		The Pennington river mouth and beach area show clear evidence of deposition of beach material and extension of barrier bars. This is very likely the resultant effect of longshore current spreading sediments along the coastline. This situation is likely to impede navigation as the river mouths are gradually closing up due to sedimentation.

Vulnerability Analysis

The Coastal Vulnerability Index (CVI) for each of the coastline sections are categorized into 4 classes as listed below:
Low Vulnerability: = CVI value of 6 – 8.25
Moderate Vulnerability: = CVI value of 8.25 – 10.5
High Vulnerability: = CVI value of 10.25 – 12.75
Very High Vulnerability: = CVI value of 12.75 – 15

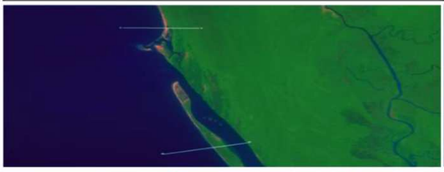
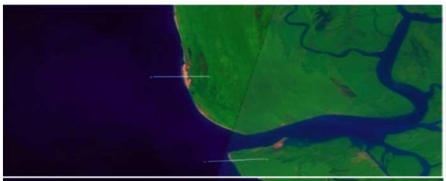
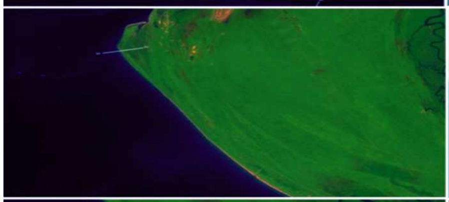
Coastline vulnerability varied from low to very high across the entire 450km stretch of coastline. The eastern (Andoni – Nun river) Niger Delta coastline exhibits generally higher vulnerability to coastline change than the western portion (Nun – Benin river). However, the vulnerability would probably have been more, but for the presence of barrier bars which provide some natural protection against the forces of erosion and accretion are prevalent in that region.

The Utapate coastline area of Akwa Ibom State exhibit high vulnerability, driven predominately to its low slope and limited natural coastline protection. Adjacent to this area is the strip of coastline from the Andoni river to the Bonny. This area is of low vulnerability as it enjoys the benefit of having a significant amount of barrier bars as depicted in satellite imagery and high slope values computed from DEMs.

Unlike the Bonny coast line, the New Calabar and San Bathelomew coastlines exhibit high and moderate levels of vulnerability respectively. The St.Nicholas, Brass, Nun and Sangana rivers are predominately highly vulnerable as the historical coastline changes relating erosion and accretion within these areas were significant.

The western coastline region of Fishtown to the Benin river is predominately low vulnerability, except in regions around the Forcados, Ramos and Dodo river mouth. Forcados and Ramos are classified as very highly vulnerable, while the Dodo river is classified as high.

Table 4: Morphological Changes from Dodo to Forcados River Mouth

Landsat 8 – January 2015	Sentinel 2 – December 2018	Observation
		Dodo river mouth and beach exhibit deposition of beach material and barrier bar extension. This is the resultant effect of longshore current spreading sediments. This will impede navigation as the river mouths are gradually closing.
		The Ramos river beach area has remained generally stable but show slight evidence of deposition of beach.
		The Forcados river mouth and beach area show clear evidence of deposition of beach material. This may be the result of shore protection measure to safeguard the Shell Forcados export terminal and adjoining facilities.
		The northern Forcados river area have been generally stable but show a little evidence of deposition of beach material.
		The Escravos river mouth and beach area have been generally stable but show a little evidence of deposition of beach material.
		The northern Forcados river area have been generally stable but show a little evidence of deposition of beach material.

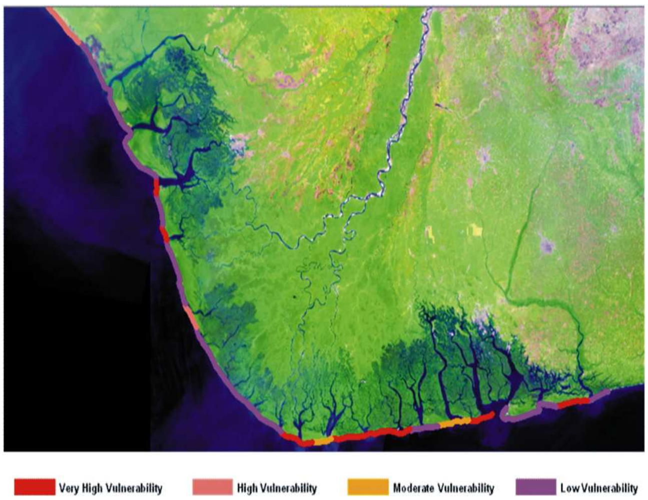


Fig. 3: Vulnerability index ranking of the Niger Delta coastline

Conclusion

It is observed (within the 3-year study period) that there is clear correlation between sections of the coastline morphological changes have occurred and sections with computed moderate to high CVI. This result is very encouraging and supports the quality of the work. This is so as morphological changes were observed and extracted from satellite image processing and interpretation, while vulnerability is computed using separate and independent types of datasets and methodology.

Oil and gas facilities such as the Exxon Mobil Qua Iboe terminal, NLNG terminal , Agip Brass terminal, Shell Forcados terminal, Olokola LNG and their associated facilities are located along highly vulnerable coastlines. Hence shore protection should be in the short, medium and long term plans of the oil and gas companies responsible for these facilities.

Significant amount of research, by way of routine coastline monitoring, metocean data gathering, inland waterway management and coastline hydrodynamic model needs to be done to better understand and sustainably manage this complex, environmentally and economically important region.

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Evaluating Geotechnical Soil Properties Along Ilorin – Lokoja Highway

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Abstract

The subgrade soils along Ilorin – Lokoja highway which is underlain by gneiss, migmatite, older granite series and metasedimentary rocks of Southwestern Nigeria were investigated. Disturbed soil samples collected from both stable and unstable portions were analysed according to BS standard for index and strength properties. The results were processed in GIS framework using Spatial Analyst tool to provide zonation maps which showed spatial variations of soil and their characteristics. From the results, the soils exhibited wide range of engineering properties. Liquid limit (13.5-69%), plastic limit (0.4-50%), plasticity index (0.7-39%), fine fraction (11.8-56.8%), maximum dry density (1.4-2.61 mg/m^3), optimum moisture content (5.7-23.8%), soaked (10-65%) and unsoaked California Bearing Ratio (20-82.5%) respectively. Result of correlation analysis conducted to measure the relative variations of the properties showed seventeen pairs of highly correlated properties with significant at 0.01 level (R^2 = 0.6-0.8). Linear regression models were developed to predict different soil parameters. The soils were predominantly fine-grained poorly graded sand and silty sand (SP-SM), silty sand (SM), clayey sand (SC) and silty clayey sand (SM-SC). Based on AASHTO system, the soils are A-2-4, A-2-5, A-2-6, A-7-5 and A-7-6 suggesting excellent to good and fair to poor subgrade materials.

Keywords: *Geotechnical evaluation, Correlation matrix, Geographic Information System, Spatial variatio, Soil Strength*

Introduction

Over the years, significant efforts have been made to develop road infrastructure in Nigeria. This is seen in the construction of over 30,000km of road networks nationwide as the only viable means of transportation of human, goods and services. This has caused over-labouring on this facility leading to failure shortly after construction and/or reconstruction. Aigbedion (2007) defined road failure as a discontinuity in a road pavement resulting in cracks, potholes, bulges and depressions which were physically seen along Ilorin – Lokoja highway. A number of studies (Oke et al., 2009a; Nwankwoala and Amadi, 2013; Ogundalu & Oyekan, 2014; Osadebe et al., 2015) have identified causes of road failure to include geological, geotechnical factors, excessive haulage loads, poor construction practices, poor and inadequate drainage. Adeyemi and Oyeyemi (2000) identified that construction materials used failed to meet the required specifications due to lack of adequate geotechnical studies. Similarly, geological factors such as near surface geologic sequence and structures like fractures and faults, stream channels and shear zones are known to contribute to road failures (Momoh et al., 2008; Adiat et al., 2017).

This study intends to evaluate geotechnical properties of soil such as index properties (grain size analysis, liquid & plastic limits, plasticity index) and strength parameters (maximum dry density, optimum moisture content, CBR) along the Ilorin - Lokoja highway as well as investigate the geologic structures that may influence road performance with a view to determine the factors responsible for the failure of some sections of the highway. Consequently, collate these data in GIS, process, analyze, and generate soil thematic maps, showing the extent of their spatial variation, thus improve the efficiency and effectiveness of an investigation.

The Study Area and Geological Setting

The Ilorin – Lokoja highway is a Federal highway (about 300Km) which connects the southwest zone to the seat of government, the Federal Capital City. It is situated between Latitude 7°25'N - 8°40'N and Longitude 4°30'E – 6°45'E. The topography has relative relief with the highest point near Omu Aran (altitude 648m) and the lowest point is near Lokoja (altitude 100m). The highway is sitting on Precambrian basement rock of South western Nigeria (Figure 1) and cut across othe gneiss migmatite complex, the metasediments / volcanic schists and the older granite series (granitoids, quartzites and silicified rocks) (Oluyide et al., 1998).

Materials and Methods

ASTER Image Processing

ASTER DEM was acquired and processed to extract lineaments. In order to reduce the processing and computational time during image processing, a vector

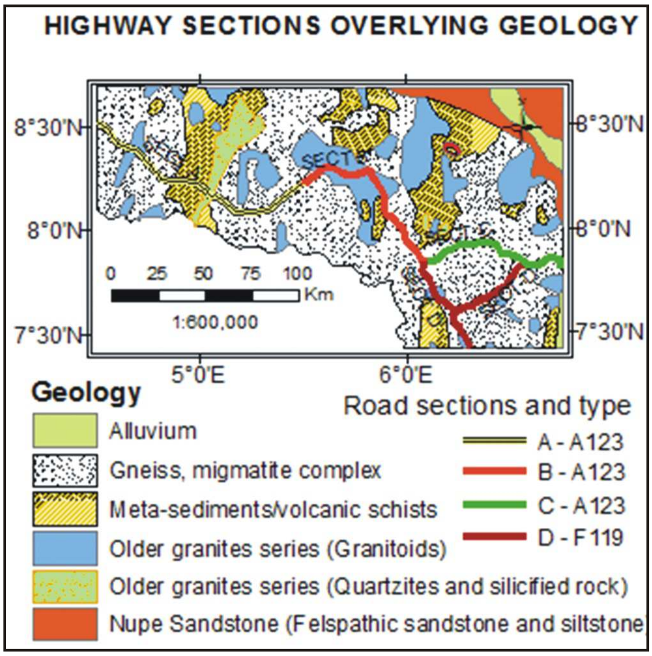

Fig. 1: The highway sections overlying the geologic setting of the study area

map representing the geometry of the study area was created to subset the acquired data. Linear stretching and enhancement was applied to enhance linear features that were digitized to generate lineament map.

Field and Laboratory Investigations

Forty-nine disturbed samples were obtained from various locations along Ilorin –Lokoja highway at intervals of 2 – 5km to a depth of 0.8 – 1.5m from both stable and failed portions of the road while simultaneously taking coordinates readings using global positioning system (GPS). The study area was segmented by employing stratified random sampling strategy into four homogenous segments A, B, C and D. Segment A, Ilorin – OmuAran - Egbe, B is the Egbe – Mopa - Kabba while C, the Kabba – Obajana – Lokoja and D is the Kabba – Okene roads (see figure 1). These samples were air - dried (35°C and 40°C) before subjected to laboratory geotechnical testing including grain size distribution, consistency limits, compaction and California Bearing Ratio. All tests were done in accordance with the British Standards procedures (BS 1377, 1990).

Data Analysis

Regression analysis is a fundmental statistical tool used to explore possible relationships among explanatory and response variables. In SPSS statistical software, different soil data were explored and their relationships examined between different geotechnical properties. Descriptive statistics were calculated, variables that were abnormally distributed were logarithmically transformed to fulfil the linear model assumptions. Boxplots and scatter plots were used to explore data structure, visualize the spread and examine the magnitude of relationship that soil properties exhibited among themselves. The strength and relationship trends between different geotechnical properties were computed through correlation matrix and quantitative measures of linear associations determined. The geotechnical data were analysed using ArcGIS software with spatial analyst tool to generate zonation maps showing the spatial variations of soil properties.

Results and Discussions

Lineaments Mapping

Figure 2 is the lineaments map in the basement terrain of the study area. This figure plays a significant role as one of the factors used during evaluation of geologic potential influence on road performance. The spatial distribution of lineaments depicts a characteristic

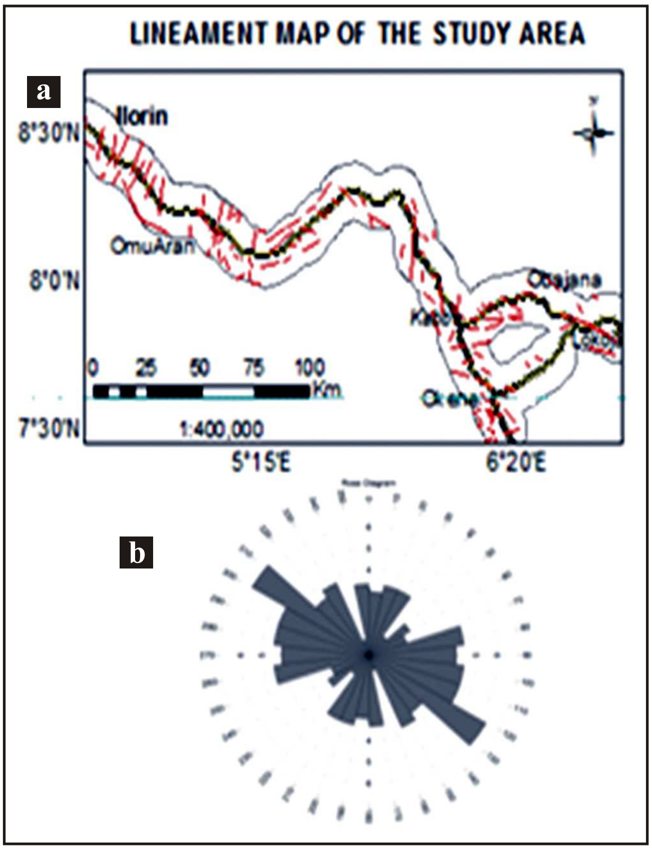

Fig. 2: Lineaments map (a) and rose diagram (b) of the study area.

feature of the occurrence of underlying structures in the basement complex. It shows a fracture distribution and pattern on a scale of 1:400,000 as exhibited across the two States (Kwara and Kogi States). The Rose diagram (Figure 2b) shows that the study area is traversed by three main lineament populations trending NW-SE, N-S, E-W. The large spread in the azimuths of the lineaments is due to the polycyclic history as well as the brittle nature of the migmatites, gneisses and quartzites which together constitute the major lithology of the basement complex rocks of the study area. The N-S lineament trend pattern coincided with the general N-S foliation strike emplacement in th basement complex of Nigeria. This invariably implies that these lineaments might have resulted from plate movements and tectonics in the region.

Geotechnical Properties

The summary of the laboratory geotechnical test is presented in Table 1. The result indicates highly skewed range of properties with some outliers in the dataset shown in the boxplots (Figure 3). Road sections C (i.e. Obajana – Lokoja wing) exhibited high fines more than 50% in some locations.

Table 1: Summary of geotechnical test

Properties	Sect B	Sect C	Sect D
PSD (%passing)			
Coarse sand (%)	54.6 – 100 (86.4)	27.75 – 98.9(83.2)	78.4 – 99.2(16.9)
Fine Sand (%)	29 – 74.2 (51.9)	12.5 – 86 (54.3)	43.4 – 81.9(57.3)
Silt (%)	10.5 – 45.4 (23.6)	2.25 – 79.5(29.3)	7.2 – 59.3 (23.9)
Clay (%)	10 – 43.2 (24.9)	11.2 – 34 (16.7)	11.2 – 32.5 (9.9)
Fines (silt & clay) (%)	15.1 – 37.4 (24.3)	11.8 - 56.8(22.9)	14.2 – 48.9(16.9)
Liquid limit (%)	13.4 – 46.5 (27.4)	17 – 69 (34.7)	13.4 – 64.5(27.1)
Plastic limit (%)	8.4 – 42.9 (18.1)	0.5 – 46.3(16.4)	0.4 – 38.4 (14.9)
Plasticity index (%)	1.86 – 30.5 (9.8)	1.8 – 34 (18.3)	5.8 – 39 (13.8)
Plasticity class	NP - IP	NP - IP	NP - HP
MDD (mg/m³)	1.4 – 2.1 (1.7)	1.5 – 2.4 (1.9)	1.7 – 2.4 (2.13)
OMC (%)	6.8 – 23.8 (13.9)	6.44 – 22 (12.6)	8 – 18 (10.0)
Unsoaked CBR (%)	20 – 75 (44.5)	25 – 80 (47.4)	23 – 82.5 (58.7)
Soaked CBR (%)	10 – 45 (22.8)	10 – 45 (26.0)	10 – 45 (31.9)
AASHTO	A-2-4, A-2-5, A-2-6	A-2-4, A-2-6, A-7-5, A-7-6	A-2-4, A-2-6, A-7-5
USCS	SC, SM, SP, SP-SM	SC, SM, ML, SP, SP-SM, SM-SC	SC, SP, SP-SM, SM-SC

AASHTO = American Association of State Highway and Transportation Officials, USCS = Unified Soil Classification System, PSD = Particle size distribution

Geotechnical Properties

The summary of the laboratory geotechnical test is presented in Table 1. The result indicates highly skewed range of properties with some outliers in the dataset shown in the boxplots (Figure 3). Road sections C (i.e. Obajana – Lokoja wing) exhibited high fines more than 50% in some locations.

The soils are poorly graded and therefore have less interlocking between the particles and thus a lower friction angle and low coefficient of uniformity. Based

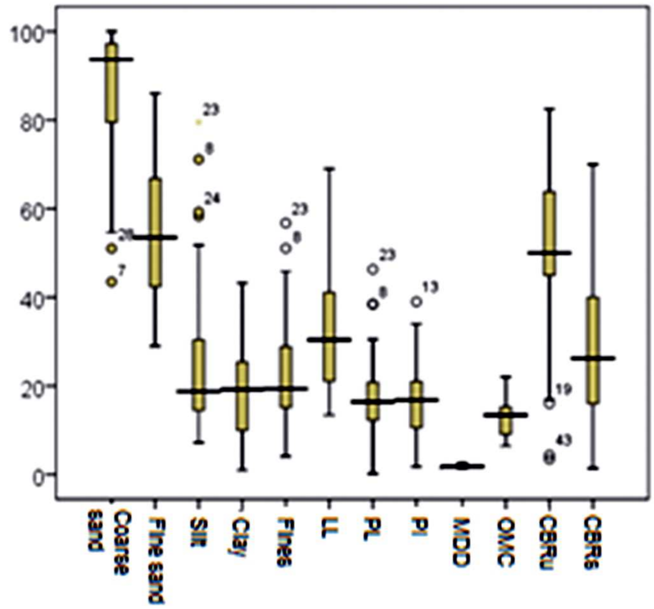

Fig. 3: Boxplots of geotechnical soil properties

on the specification by FMWH, materials having ≤35% passing sieve No. 200 are considered suitable materials. Therefore, sections BCD soils with average amount of fines lower than 35% are considered suitable subgrade materials (Table 3). According to Oyediran & Durojaiye (2011), soils with amounts of fines less than 50% are expected to possess better engineering properties than with higher fines when used as subgrade or subbase materials.

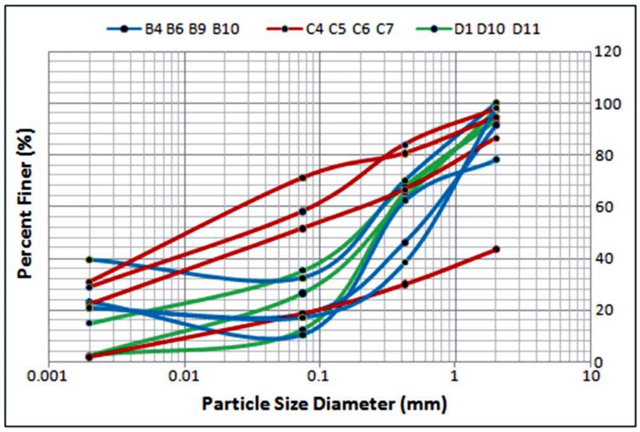

Fig. 4: Particle size distribution of selected soils

The consistency limits of the soils presented in Table 1 defines the relative ease to which a soil can be deformed based on the relationship or interaction with water. The soils varied from non-plastic to intermediate plasticity with few exceptions of high plasticity and hence compressibility along section D. Majority of the soils fall above A-line on the Casagrande plasticity chart (Figure 5) and can be said to be inorganic clays. This corroborates the classification of Ige (2011) which indicated that the soils are inorganic clay of low plasticity. Section BCD of the road recorded an average liquid limit < 35% with low plastic limit and plasticity index < 20% which falls within the standard limit of LL < 40% and PI < 20% (Table 2). However, the locations of high plasticity (PI ˃ 35%) fell along the failed portions. Such soils have the ability to retain appreciable amount of water which is seen in section D of the road.

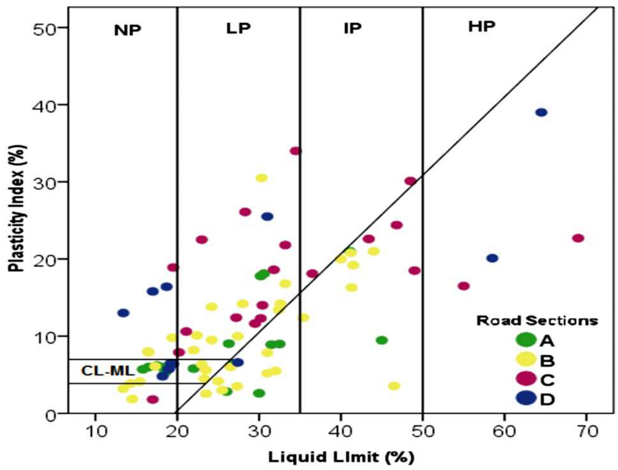

Fig. 5: Position of soils on the Casagrande Plasticity chart.

The soils are classified according to Unified Soils Classification System as clayey sand (SC), silty sand (SM), poorly graded sand (SP) and silty sand mixture (SP-SM), silty sand clayey sand mixture (SM-SC) and silt of low plasticity (ML). According to AASHTO classification system, the soils are mostly in A-2-6 and A-2-4 classes and rated as excellent to good with A-7-5 class which is not a good material except upon improvement (Figure 6).

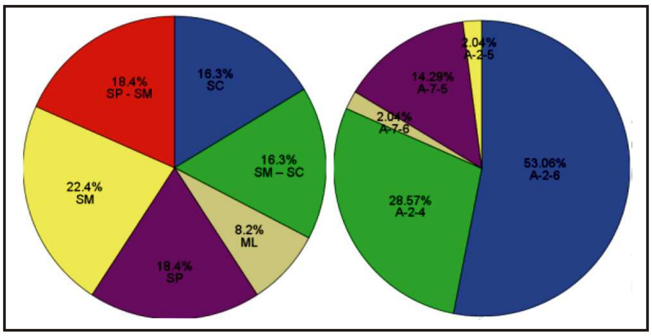

Fig. 6: USCS and AASHTO classification

Strength Properties

Figure 7 shows compaction curves for selected soils for which the maximum dry densities were attained for given soils with a standard amount of compacted effort. The curves formed a family of curves with distinct peaks for maximum dry density ranged from 1.4 – 2.6mg/m^3 and corresponding optimum moisture contents from 5.7 – 23.8% with mean value of 1.9 mg/m^3 and 12.8% respectively. As expected, soils with large amount of fines exhibited low dry densities and higher optimum moisture contents. When compared with FMWH, only section D with OMC values between 8 - 18% met the requirement and therefore considered suitable.

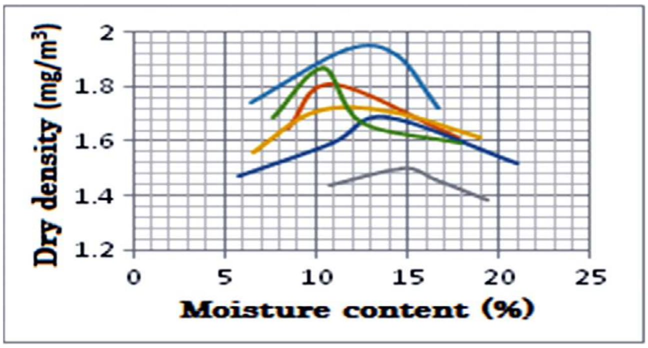

Fig. 7: Laboratory compaction curves of selected soil samples

The result of CBR tests for both soaked and unsoaked condition ranged from 10 – 65% & 25 – 82.5%. The result also showed a reduction in strength as a result of soaking. This suggests that there will be a drastic reduction in strength by more than half when water is accumulated in the soil and the penetration resistance becomes reduced. FMW&H guideline recommends that the values of CBR under soaked condition for base, sub-base, and subgrade should not be less than 80%, 30% and 10% respectively. By interpretation, all the soils tested are excellent subgrades materials.

Correlation Analysis

The details of the Pearson's correlation matrix indicates that the relationship developed is statistically significant (p = 0.01) shown in Table 3. Nineteen soil parameters

Table 2: Comparison of results with Nigerian Specification for road and bridge materials for general filling and embankment (FMWH, 2000)

Properties of materials	FMWH Spec	Soil range	Mean	Remark
MDD (mg/m^3)	< 0.047	1.4 – 2.6	1.9	suitable
OMC (%)	< 18	6.4 - 22	12.8	suitable
LL (%)	< 40	13.4 - 69	32.3	suitable
PI (%)	< 20	1.8 - 39	16.7	suitable
% passing No. 200 (%)	≤ 35	14 – 56.8	24.2	suitable
CBR (24hrs soaked) (%)	≤ 10	10 – 45	28.3	adequate

showed highly significant correlation with positive relationship among themselves, however, degree of relationship varied according to soil type and property involved. Maximum dry density was significant with optimum moisture content (Figure 8a) and clay. The regression analysis after correlation is expressed by the single linear equation inserted in the plot with its corresponding coefficient. This result corresponded with the works of Sridharan (2002). Therefore, 66% of the variance in MDD can be accounted for by the moisture content.

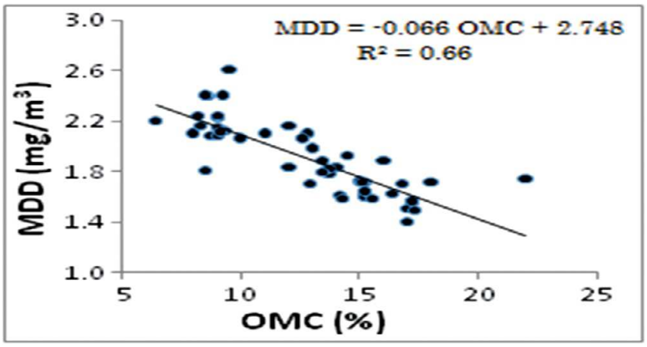

Fig. 8a: Correlation of maximum dry density with optimum moisture content of soils.

Significant correlations (R > 0.60) were also recorded between FS & PIw), LL & PL, fines & OMC, PL & OMC, OMC & CBRs. However, the correlation of OMC with PL is better than with LL, just as CBRs with OMC better than with MDD (Figure 8b and 8c). The linear regression shows that 40% of variance in OMC can be accounted for by plastic limit while same OMC can account for 31% of the variance in CBR. These are good predictors for the dependent vriables.

Since CBR is the most utilized parameter dimensioning flexible pavement in tropical countries, further assessment in the relationship recorded a significant negative correlation with OMC, fine sand, fines, LL and PL implying that CBR increases when there is corresponding decrease in OMC, fine sand, fines, LL and PL.

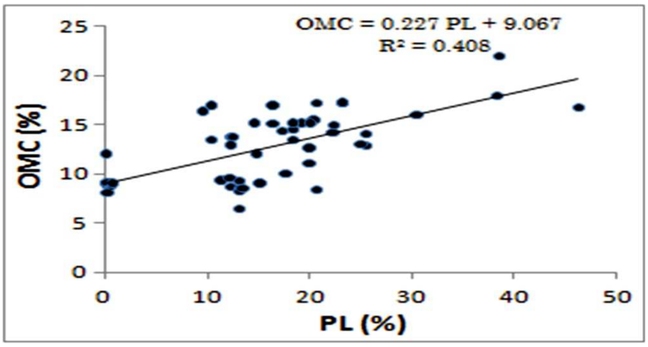

Fig. 8b: Correlation of optimum moisture content with plastic limit of soils.

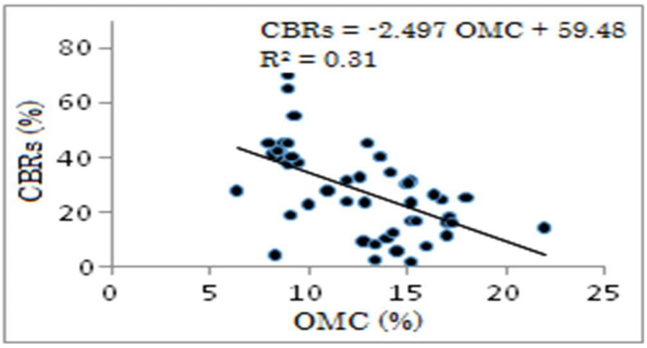

Fig. 8c: Correlation of California Bearing Ratio with optimum moisture content of soils.

Clay content also had a significant positive correlation with OMC implying that moisture content rises when amount of clay increases. Clay content determines the amount of surface area that is available for water adsorption. However, the nature of clay influences plasticity of any given soil and thus the moisture content.

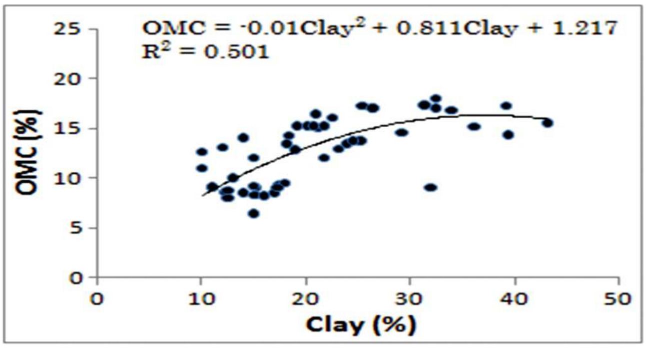

Fig. 8d: Correlation of OMC with clay content of soils.

Summary of regression analysis, equation models and their order of significance in terms of R^2 is given in Table 4. These mathematical models can be utilized to estimate dependent geotechnical parameters.

Table 3: Correlation matrix of soil properties

	MS	FS	Silt	Clay	Fines	LL	PL	PI	PIw	MDD	OMC	CBRu	CBRs
MS	1												
FS	.50**	1											
Silt	.28*	.69**	1										
Clay	.22	.38**	.34**	1									
Fines	.31*	.69**	.92**	.69**	1								
LL	.11	.36**	.68**	.42**	.70**	1							
PL	.19	.36**	.66**	.40**	.68**	.81**	1						
PI	-.07	.14	.28*	.18	.29*	.62**	.04	1					
PIw	.27*	.66**	.61**	.29*	.59**	.69**	.28*	.80**	1				
MDD	-.34**	-.28*	-.14	-.63**	-.37**	-.39**	-.39**	-.15	-.24*	1			
OMC	.31*	.36**	.43**	.67**	.62**	.62**	.64**	.20	.35**	-.82**	1		
CBRu	-.22	-.35**	-.32*	-.36**	-.40**	-.37**	-.27*	-.26*	-.40**	.46**	-.52**	1	
CBRs	-.37**	-.48**	-.41**	-.44**	-.50**	-.47**	-.50**	-.14	-.42**	.56**	-.64**	.76**	1

**. Correlation is significant at the 0.01 level (1-tailed)
*. Correlation is significant at the 0.05 level (1-tailed)

Table 4: Summary of regression analysis, equation models and their order of significance.

Parameters	Correlation coefficient	Order of significance	Regression type	Linear regression equation
MDD vs OMC	0.66	1	linear	y = -0.066x + 2.748
CBR vs OMC	0.41	6	linear	y = -2.309x + 57.86
CBR vs MDD	0.31	10	linear	y = 24.59x – 18.47
PL vs OMC	0.41	6	linear	y = 1.791x – 6.546
LL vs OMC	0.38	8	linear	y = 2.208x + 3.654
OMC vs clay	0.50	3	polynomial	y = -0.01x2 + 0.811 + 1.217
clay vs fines	0.47	4	linear	y = 0.578x + 7.346
clay vs MDD	0.39	7	linear	y = -18.76x + 57.05
fines vs LL	0.50	3	linear	y = 0.566x + 6.128
fines vs PL	0.58	2	polynomial	$y = 0.021x^2 – 0.120 + 18.35$
fines vs OMC	0.38	8	linear	y = 1.77x + 1.556
FS vs wPI	0.43	5	linear	y = 1.88x + 37.77
silt vs wPI	0.37	9	linear	y = 1.647x + 13.0

Evaluation of some geotechnical data using GIS approach

Using ArcGIS 10.6, the geotechnical data were evaluated by geostatistical tool to produce spatial distribution maps in order to visualize their spatial variations and relationships. Geostatistical interpolation of soil data using GIS software provides the information or data of unknown area or points within the selected area. IDW is a geostatistical interpolation technique, which estimates unknown values with a specific probability distribution at an arbitrary point of the space

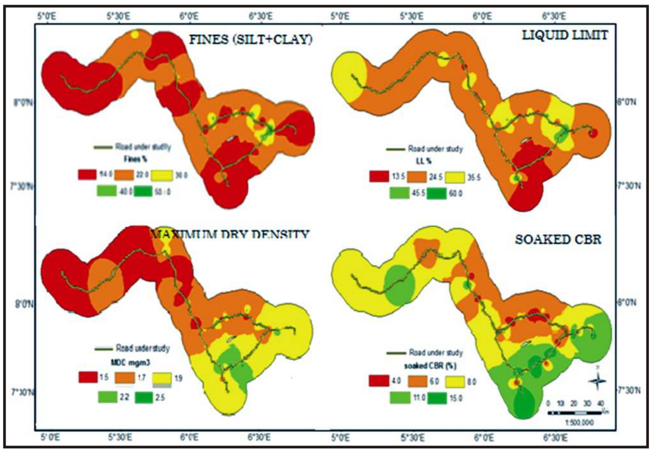

Fig. 9: Some soil zonation maps (fines, liquid limit, maximum dry density and soaked CBR.

(Deshmukh and Aher 2016b). Figure 9 confirmed the prevalence of fines. Road section C exhibited very low CBR fines, no wonder the area is undergoing concrete pavement reconstruction by Dangote Company. Red color signifies low values and green the high values.

Conclusions

The remote sensing and GIS offers a useful technique for mapping and enhancing soil information along Ilorin – Lokoja highway. Spatial distributions of lineament features mapped along the highway that formed characteristic zones of weakness may be responsible for the degree of instability.

Furthermore, the subgrade soils classified as A-2 and A-7 soils with predominance in A-2-4 and A-2-6 type are considered excellent to good as against A-7 soils, which are fair to poor subgrade materials. The predominance of fines in the soils also influenced the stability of the road. Low range in density and poor CBR in failed portions indicated percentage reduction in strength of the soils on exposure to excessive moisture. Absence of adequate drainage system that influenced the surface run-off added to the pavement deformation. It is therefore recommended that road engineers should give serious consideration on drainage channels on roads to ensure free flow of run-off and in-depth knowledge of the parent materials and characteristics of the soils are necessary.

The spatial distribution maps of the area produced showed a well defined distribution of soil type. These maps can be useful in reconstruction works of the failed sections of the road, and would guide future pavement design in construction of roads. Geotechnical engineers can run more queries of various combinations regarding the various properties of soil which will help in decision making process.

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Geo-Environmental Mapping for Containment of Natural Hazards: *Impacts of Geosciences Expertise on Natural Hazard Management*

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Introductory Background

Geosciences or Earth Sciences include a spectrum of sub-disciplines that address somewhat specific earth processes, earth systems, or portions of the earth such as geology, hydrology, soil science, geophysics, geochemistry, oceanography, climatology, and meteorology among others. The above definition provides a sense of the diversity of topical areas that constitute a study of the earth and its processes. Therefore, work of Geoscientists (i.e. geologists and geomorphologists) includes not only the understanding, but also the mapping and modelling of the Earth's processes many of which directly affect human activities and the development of the society at large. Even though, the natural evolution of the environment and landscape are defined by the interaction between the processes that developed under and over the earth surface, mankind is able to control and induce landscape changes through a number of activities in the course of civilization and development(Miccadei and Piacentini, 2013).In other words, the significance of geology (geosciences)lie in the fact that developments are either built on, or have foundations in geological deposits, utilise (natural) geological materials, or have the potential to pollute or sterilise other natural geological resources (IGI, 2002).

By and large some landscape changes are subject to natural geological processes with severe and extreme events (i.e. earthquakes landslides, flooding)and related hazards. But a number of such natural hazards are also directly or indirectly triggered by human-induced processes and activities. Many of these activities,which usually affected the geo-environments and geo-materials, are within the context of infrastructural and socio-economic development.Hence, human activities, over the past several decades, had exerted huge impacts on the environment and the landscape. Therefore, understanding of the dynamics of geological processes andgeo-environments are required for geo-environmental assessments and geo-mapping which are essential for solutions to the problems relating to:

- Water and mineral resources management,
- Air, soil or water contaminationand waste disposal,
- Land and coastal erosion,
- Land conservation,landscape restoration and urban planning,
- Geo-materials, foundations and infrastructural engineering,
- Ecosystem inventories and natural geo-heritage assessments as well as
- Evaluation and mitigation of natural and man-induced hazards and risk.

The above geo-environmental issues are undoubtedly the subject of environmental and geological education that must be understood and acknowledged by governmental agencies and lawmakers (Miccadei and Piacentini, 2013). This is more important due to the fact that most severe or extreme natural earth processes become "disasters" due to unsustainable human activities and land-use management / development. Such developmental activities include uncontrolled expansion of urban and industrial areas, roads and other infrastructures built on river valleys, coastal plains, seismic areas, without correct planning).

Nonetheless, the roles of geoscientists in most of the above geo-environmental mapping and management and associated development has been under-estimated in Nigeria as their full potentials are still far from been fully utilized. This situation is more worrisome when realising that about half of the 2.4 billion increase in population from 2013 to 2050 will occur in Sub-Sahara Africa countries including Nigeria, where in 2014, one in four people remained malnourished (UN/WHO report). In addition, it was also projected that out of the 20 largest megacities projected for 2100, 13 will be in Africa with Lagos at the top of the list.Nonetheless, the roles of geoscientists in most of the above geo-environmental mapping and developmental issues have been under-estimated in developing nations like Nigeria as their full potentials are still far from been fully utilized.

Hence, this write-up aims at highlighting the significance of geo-scientific knowledge of natural processes and expertise in providing the most effective approaches to prevent natural disasters and ensure a sustainable human and environmental development. The intent is to highlights the significance of geo-environmental mapping vis-à-vis the geoscience expertise in the context of sustainable development, In addition, it is to demonstrate the fundamental role of the earth sciences in understanding and solving multidisciplinaryenvironmental problems. However, this can only be achieved through complete multidisciplinary, multi-scale and multi-temporal studies based on geological and geomorphological investigation and geo-environmental mapping, but also by integrating engineering, architectural, economic and social issues.

The Geo-Environment, Earth Resources and Geosciences

In simple term, the *Geo-environment* is that part of universe with which human interact and exploit as well as adapt. In other words, the geological environment (geo-environment) can be defined as that part of lithosphere, which directly influences the conditions of the existence and development of the society (Hrasna, 2002). The concepts of *earth resources* underlie most environmental investigations in respect of fundamental mapping of earth resource such as soil, water, minerals, and energy etc that are essential to development and maintenance of ecosystems. However, *geosciences (earth sciences)* address dynamic geological processes below and on the earth's surface, as well as those within the atmosphere and it is essential and fundamental part of environmental studies. In addition, geosciences studies examine present and past geological processes in order to provide historical perspectives against which anthropogenic impacts can be evaluated. Hence geoscientists provide vital information regarding human impact on the environment and predict future outcome in terms of frequency, rates, and magnitudes of earth system changes.

Consequently *Environmental Geoscience* is an interdisciplinary applied earth science concerned with the practical application of the principles of geology in the solving of environmental problems. In other words, is the application of geological information to:

- solve conflicts arising from interaction of humans with the geologic environment,
- maximizing possible advantageous condition resulting from the use of natural earth resources or
- minimizing possible adverse natural geo-hazards.

By and large, it should be noted that the overall geo-environmental condition of an area is a combination of complex of *geo-factors (geo-components)* affecting the quality and/or the sustainable exploitation of the geo-environment and landscape. However these geo-factors can be classified as:

a) Geo-factors that provide the opportunity of positive exploitation of geo-environment and associated earth resources are referred to as *geological potentials (geo-potentials)* of the environment.
b) Geo-factors which limit the use of geo-environmental resources and/or negatively affect land-use and human livelihood are called *geological barriers (geo-barriers)* of the environment.

The *geological barriers* comprised geo-actors that endanger the environment or even human health and livelihood (e.g. catastrophic earthquakes, landslides, etc.) termed *geological hazards (geo-hazards)* and geo-factors that limit the land use and/or raise the price of land-use (unsuitable foundation soils, areas susceptibility to landslides, etc.) termed *geological constraints (geo-constraints)*. Nonetheless, the complex relationship of all the geo-factors in an area translates to the totality of *geo-environmental conditions* (Fig. 1).

Geo-environmental conditions of a territory are expressed by compilation geo-mapping to depict significant geo-factors of the environment for purposes of land-use planning and environmental protection within the context of development vis-à-vis mining of mineral raw materials, civil and industrial construction; transport construction, ground water exploitation, recreational, sanitary and sporting purposes, etc.

Therefore, environmental geologists must take into account all components of the geo-environment and seek to explain how the environmental resources could be managed sustainably as well as forecasting of possible changes in the geo-environment due to man's intervention in the course of development.

It analyses all environmental aspects of the geo-

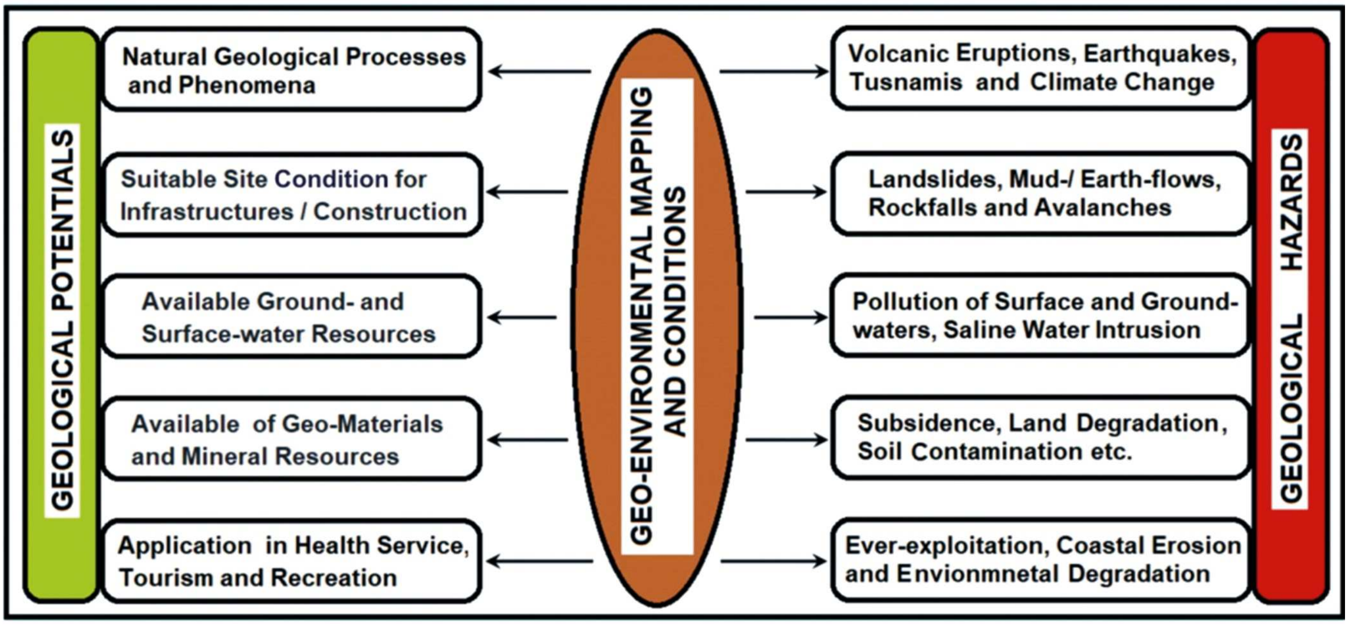

Fig. 1.0: Xxxxxxxxx Xxxxxxxxx

environment taking into account mainly its evaluations from point of view of engineering geology, hydrogeology, economic geology, geochemistry and geophysics.

Natural Hazards, Geo-environmental Mapping and Development

Natural hazards are naturally induced phenomena which usually occur at irregular intervals and at varying intensity and usually lead to disasters or undesirable consequences(Word Bank, 2010).Examples include geological processes such as earthquakes, volcanic eruptions, landslides and subsidence, tsunamis, floods and droughts, and coastal storms (hurricanes, cyclones, and typhoons) among other natural phenomena. Nonetheless, it should be appreciated that these natural geo-hazards are driven by natural earth geological processes; most of which can result in the loss of human life and/or have the potential to cause very significant economic damage (Fell, et al., 2008); thus with consequent negative impacts on socio-economic development. Therefore, Geoscientists usually provide a first line of defence against such natural geo-hazards, as their knowledge in *geo-environmental mapping*can determine what disasters / hazards to face at a particular location and how to mitigate the hazards effectively (EFG, 2015).

However, *geo-environmental mappings* are typically conducted prior to and in support of the infrastructural development that may have a distinct effect on the natural geo-environment such as ground- and surface water sources, foundation and slope stabilities, landscape and urban planning, evaluation and mitigation of natural and man-induced hazards among others.*Development*on the other hand, refers to improvement in a country's socio-economic conditions i.e. improvements in way of managing the natural and human resources (Shah, 2019), thus creating wealth and improving human livelihood within the context of socio-economic and infrastructural development.

In summary, the geo-environment affords the human society to take advantages of the natural earth resources (mineral and groundwater resources) and other favourable geologic phenomena necessary for existence and evolution and thus creating favourable conditions for the development of the society.However, there is the need to ensure that development does not mean social dislocation and insecurity; rather the focus of any development initiative should be that of human security and environmental sustainability (Shah, 2019) as enshrined in the UN concept of sustainable development. *The implication is that there cannot be a meaningful sustainable development that will cater for the totality of the environment without the inputs of the knowledge of geosciences*.

One of the most important practical steps is to identify high-risk areas through risk maps production showing natural hazard probabilities and zonations. Therefore,

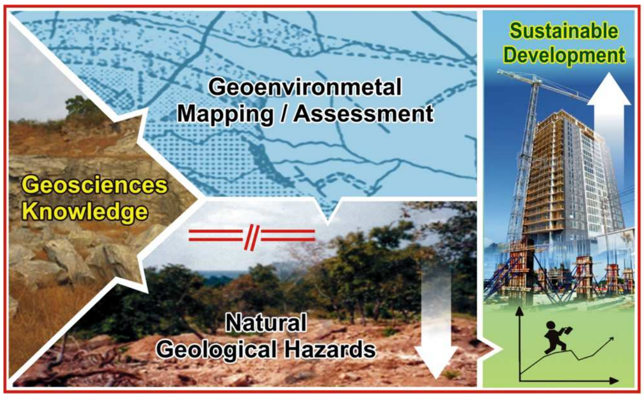

Fig. 2: Xxxxxxx Xxxxxxxxxx

Geosciences content is critical in managing the risk of disasters through proactive geo-environmental mapping will protect human population, livelihoods, productive assets / properties while promoting socio-economic development.

Geo-environmental Mapping and Disaster Risks Management

Although it is not possible to prevent the occurrence of geo-hazards influenced by natural causes, proper knowledge-based planning involving revenant geo-scientific expertise and management can minimise the possible associated risks and disasters.*Disaster* is, generally, described as combination of hazard and vulnerability (Ariyabandu, 2003), whereby: *while vulnerability is defined as the degree of susceptibility of environmental element(s) to a hazard, or the lack of capacity to absorb the impact of a hazard and recover from it*(Ariyabandu, 2003). In other words, disaster is a function of the risk process and results from the combination of hazards, conditions of vulnerability and insufficient capacity or measures to reduce the potential negative consequences of risk. Hence, risks are the result of hazards, exposed elements and vulnerability and *they are consequently not only an expression of the natural environment, but also related to human interaction with nature* (Figure 2).

Risk is defined as "a measure of the probability and severity of a hazard to health, property of the environment" while vulnerability is "the degree of loss to a given element or set of elements within the area affected by the natural hazard" (Fell, et al., 2008).The

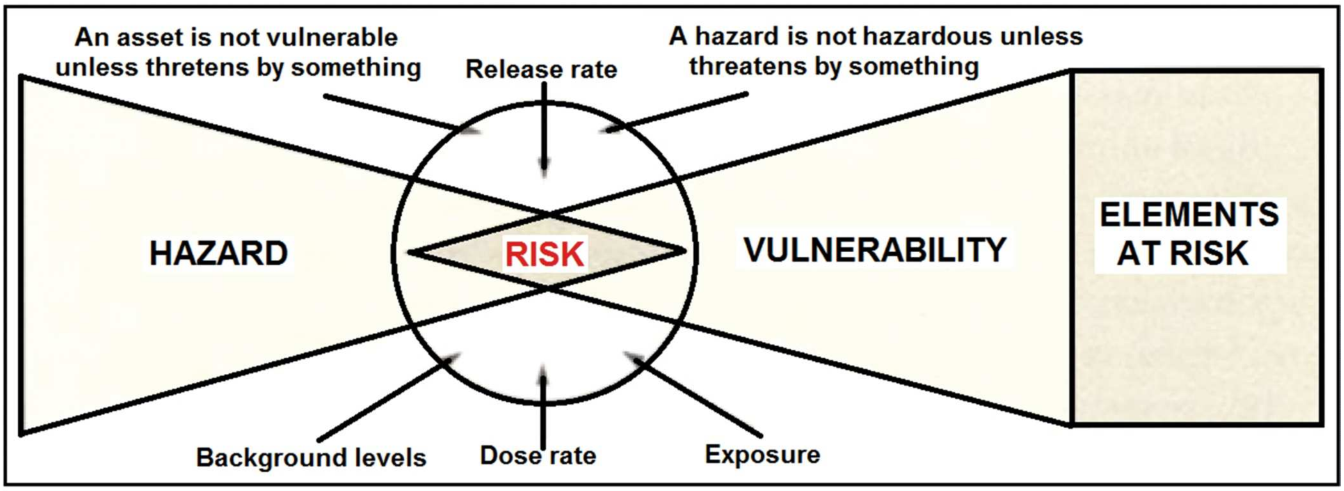

Fig. 3: Conceptual relationship between hazard, elements at risk, vulnerability and risk (Alexander, 2002)

risks due to natural processes depend on the relationship between the natural state of the earth system (geosphere, hydrosphere, atmosphere and biosphere) and the ability of the socio-economic system to adapt to the earth system. Hence, Risk (R) can be defined as:

$$R = H \times E \times V$$

(H) = Hazard, (E) = Elements at risk and (V) =Vulnerability

Risk is defined as "the probability of harmful consequences (disaster) or expected losses resulting from interactions between natural hazards or human-induced hazards and vulnerable conditions" happening in a given time frame. Risk reflects the potential impact on a community or region. A generalized risk diagram with the risk for various natural hazards identified (Figure 3) and highlighting a wide range of frequencies of occurrence and severity of impact. Risk can be computed for all hazards, but the accuracy of that assessment requires understanding the likelihood of an event of a given severity and the expected impact on life and property. For example, there are numerous earthquakes annually that have magnitudes less than 6, which generally cause little damage or significant human health. However, there are many fewer earthquakes with magnitudes greater than 6, which may cause great damage and significant loss of life.

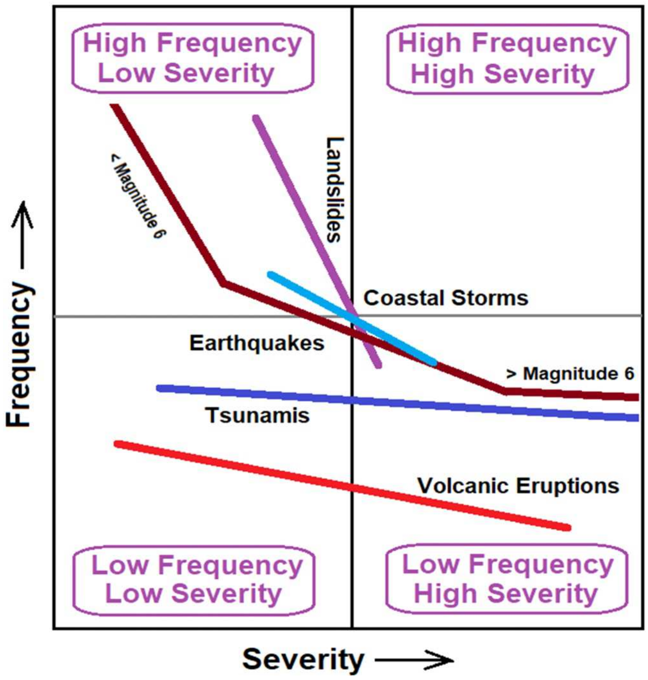

Fig. 4: Generalized Risk Diagram for selected Natural Hazards (After Leahy, 2017).

Different regions are exposed to different hazards of varying degrees, placing them in different disaster risk zones. Mitigating the negative impacts of natural geological hazards requires reducing the vulnerability of human community (livelihood and infrastructural properties) through geo-environmental mapping and risk assessment leading to disaster reduction and prevention.By and large, adequate understanding of the nature and extent these natural geological phenomena are critical to managing risks and associated disaster to society at large. Therefore, the *concept of disaster management* is concerned with preparing for, ameliorating of the impact and reducing the risk of disasters. It involves both the emergency operation in a disaster as well as the rebuilding the society in aftermath of disaster (Tan, 2009). Therefore, managing disaster entails managing vulnerabilities that include five basic phases i.e. prevention, preparedness, emergency response, recovery and mitigation (Bhatti, 2003).

a) *Prevention* includes the measures taken to impede the occurrence of a disaster, though it is not possible to prevent the occurrence of natural disasters fully but the extent of its damages can be reduced.
b) *Preparedness* covers the activities designed in anticipation of a disaster to ensure that appropriate and effective action is taken earlier to reduce the impacts.
c) *Emergency response* covers the indispensable services and activities that are undertaken during the initial shock or in the aftermath of a disaster design to save life and properties.
d) *Recovery* in disaster management refers to the activities that are taken after the initial impact to re-establish pre-disaster social-economic and environmental conditions i.e. a return to normality.
e) *Mitigation* involves the measures that can be adopted to minimize the destructive effects of hazards and to lessen the magnitude of disaster. These activities can occur before, during and aftermath of disaster and overlap of all phases of disaster management.

An important aspect in undertaking the challenge posed by hazards is to shift from post-event disaster-orientation (concentrating on risk management and mitigation) to a preventive orientation (Geological Survey, Finland 2008) concentrating on geo-environmental mapping and risk assessments. Such investigations will involve geo-environmental mapping of the area at risk, including geology, geomorphology, hydrogeology, land use and development on one hand. On the other hand it will highlight the type and location of potential hazard, occurrence and the timing as well as triggering factors. By and large, all types of risk (economic, social and environmental) should be assessed on a comparable basis using best practice approaches including the geo-assessment of extreme geological or even climatic events (Newhall, 1997). Such approaches support a proactive disaster prevention and reduction responses for which geo-mapping offer important contribution instead of the traditional post disaster actions. DRR includes any activity that prevents or reduces the risk from damages caused by natural hazards like earthquakes, floods, droughts, and storms (UN, 2005; World Bank, 2010).

As presented in Figure 5, *Disaster Reduction* is achieved with the installation of early warning systems and preparedness through the core concept of risk assessment and geo-mapping of *Disaster Prevention.* The latter is based on land use planning, infrastructure development and maintenance, water resource management, agricultural planning, understanding the mechanisms of climate change, institutional coordination mechanisms, information and knowledge sharing.In summary *risk assessment* identifies hazards and provides information on exposures and vulnerabilities humans and infrastructure in the drive to prevent geo-hazards or disasters.

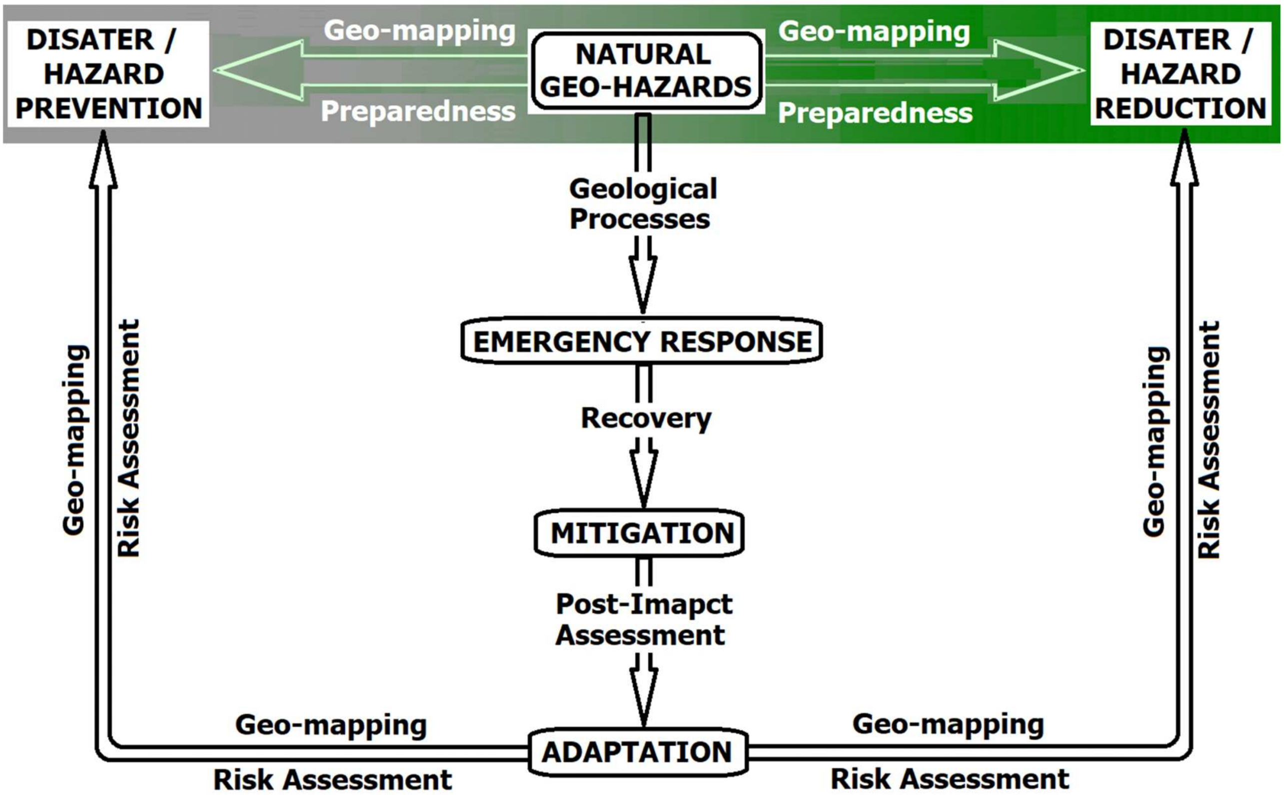

Fig. 5: Framework for the geo-assessment based Disaster Risk Reduction (DRR) and Management.

There is no doubt that many natural disasters can be prevented or the impacts significantly reduced, and geoscientists have crucial roles and responsibilities to play within the framework of assessment, warning and mitigation of natural hazards / disasters (Figure 5). Hence, Disaster Risk Reduction (DRD) through preventive geo-environmental mapping is essential to achieve sustainable development. In other words, a proactive preventive geo-environmental mapping will help to release of funds that otherwise would be spent on disaster mitigation for development in other sectors.

Geo-environmental appraisal to be employed should be iterative in approach where options are conceptualized, developed, reviewed and refined to identify preferred environmental solutions that would be robust and ensure sustainable development (Figure 6). Such iterative approach allows for the appraisal of costs and benefits of available geo-environmental mapping options and selection of appropriate geo-solutions (FCERM-AG, 2010).

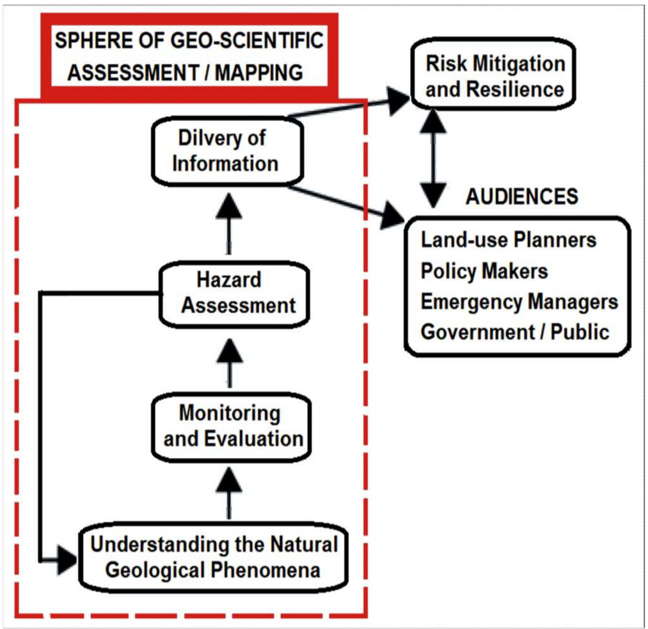

Fig. 6: Conceptual approach to disaster management showing the cycle of geo-hazards assessments for (After Leahy, 2017).

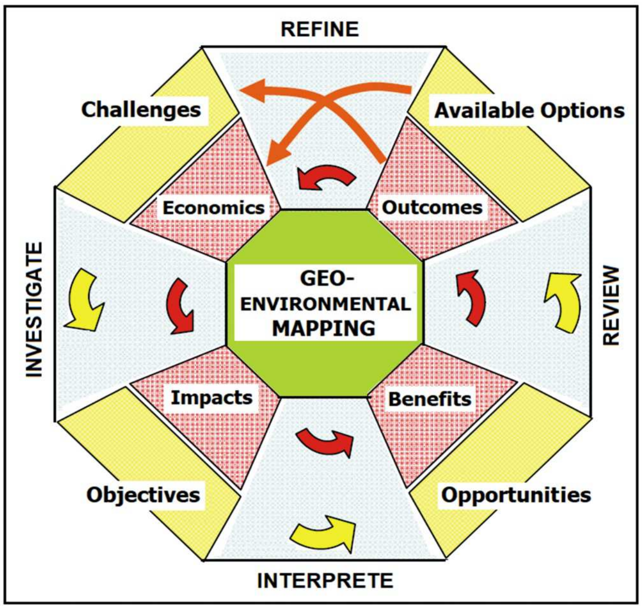

Fig. 7: The process of learning, reviewing and refiningin Geo-environmental mapping (modified after FCERM-AG, 2010).

Most of the geological hazards are usually random events, the occurrences of which are difficult to predict. However, the expected value of losses associated with such geo-hazards can be estimated as the probability of a range of geo-hazard events multiplied by the expected damages should such geo-hazard events occur. For example, the losses from infrequent flooding are measured by the difference in the areas under loss probability curves for the 'do nothing' and with geo-mapping options (Figure 7). Such difference in area represents the expected value (benefit) of the reduction in flood losses/risks.

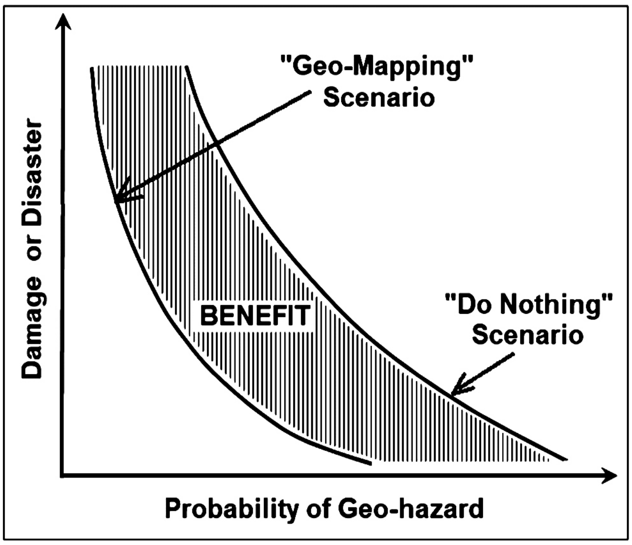

Fig. 8: Environmental benefit as the difference between damages under "do-nothing" and the geo-environmental mapping options (modified after FCERM-AG, 2010).

Geosciences Expertise in Natural Hazard Management

Geoscientists, irrespective of the specialisations, are required to provide information thatwill guide plans and decisions regarding future economic successand safety of a specific infrastructural or development projects.

UsuallyGeoscientistsare to work in conjunction with engineers toensure the safety of structures while most activities that require geological advice of geoscientists are closelyconnected directly, or indirectly, to the environment. For examples, activities such as mining, quarrying, construction,development of water resources and waste disposal are few examples of activities that can significantly change the landscape and the quality / way of life of local communities for which Geoscientists has important role play.

Hence, it is essential for the Geoscientists to exhibit highest professional / technical standard in addition to adequate experience in specific field of activities: The level of geological details required is dependent on the nature of the development, its potential impact and associated mitigation measures.

a) The exploration for, and utilisation, of natural resources including metalliferous and non-metalliferous minerals, water supply and water management, energyand other natural resources.
b) Environmental management, environmental risk assessments, preparation ofenvironmental impact statements and assessment, soil conservation and protection ofnatural environments.
c) Examination of past and present geological processes related to the development ofearth history vis-a-vis land-use development.
d) Management, prediction, prevention of geological hazards and avoidance of environmentalproblems in the future.
e) Evaluation of human activity directly affecting and necessarily modifying the naturalhabitat through the construction of buildings, communication links, dams,underground excavations etc.

While such activities as highlighted above have significant positive implications in the context of infrastructural and overall socio-economic development, they can also have profound and irreversibleeffects on the use of indigenous livelihoods and other components of the natural environment. Hence, it is essential that the data, interpretations and advice provided by the professional Geoscientists be of the highest technical standard.Examples of landscape / land-use planning and natural/geological processes where quality geological input would minimise risks to life, property and environmentinclude:

1) **Water Supply and Quality Assessments**: Groundwater is an important natural resource; hence there is the need for aquifer andsource protection so as to avoid contamination.
2) **Waste Disposal Management**: To protect the environment it is imperative thatcomprehensive geoscientific and engineering investigations are carried out during theappraisal and selection of areas for the construction of waste disposal facilities orland-banks for waste spreading.
3) **Land Use Planning**: It is essentialthat geological expertise be involved in the planning process. Due to the fact that land-use issues are a complex inter-play of factors relating to resources and building materials, changing water tables, pollution, waste disposal, and overexploitation of the coastal zone.
4) **Coastal Erosion and Flooding**: Integrated scientific approach management of flooding, land and coastal erosions can be feasible only when there is a sound basic knowledge of geological andecological factors.
5) **Flooding:** The geology of a river's catchment area is an important factor in assessing the risk of flooding, and in designing systems to minimise that risk.
6) **Groundwater Abstraction and Land Subsidence**: Modern geophysicaltechniques and geological mapping can provide timely information for remediation,planning and investment decisions affecting infrastructure.
7) **Mining and Quarrying:** Detailed geological examination is required forthese projects to assess the resource, ground stability and safety, long term restorationand impact on groundwater.

WAY FORWARD: ***Earth Science–based Urban and Catchment Development***

As urban areas expand, the geologic sub-base upon which cities and catchmentsare built becomes ever more critical to human safety and well-being in the face of potential natural geological hazards.
In addition, the emerging global environmental change (i.e. climate change) portends increasing risks to urban centers, located in coastal areas, on deltas, and in floodplains; thus putting the concentrated populations at the risk of natural hazards such as floods, landslides, ground subsidence, and soil erosion.

There is no doubt that Disaster Risk Reduction (DRD) through preventive integration of land-use planning and geo-environmental mapping is essential to achieve sustainable urban development in the 21^{st} century Nigeria as highlighted in Figure 9.

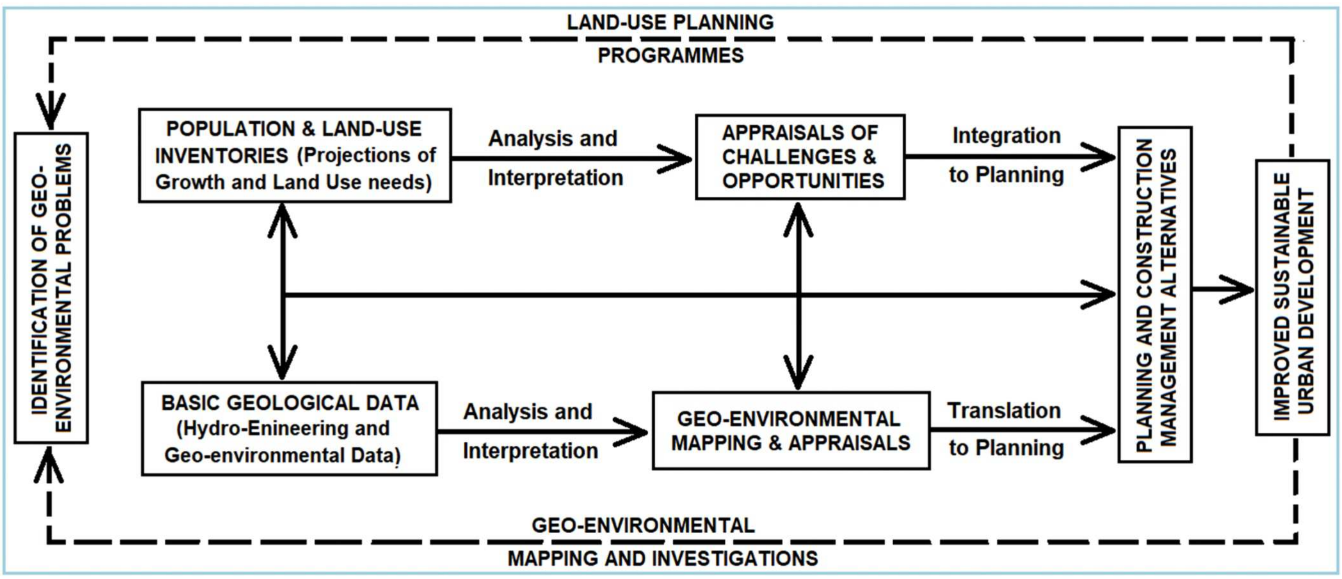

Fig. 9: Interrelationship of geo-environmental mapping and sustainable urban development planning.

As our world develops cities will grow, the population will rise and accordingly exposure of lives and property to disasters will increase although not evenly. The risk posed by a natural hazard is in direct proportion to the population density in the area vulnerable to the risk (UN, 2005; World Bank, 2010). Other geological factors (geo-factors) of major importance in sustainable urban environmental development are:

- Surface water quality and effluent discharge,
- Site geology for tunnels, foundations, and utility corridors,
- Availability of construction materials (aggregate, stone, etc.),
- Waste disposal management and search for suitable disposal sites and
- Limitation of ground-water recharge and over pumping.

Thus exposure to hazards is expected to increase, due to rapid population growth in cities, and the rising interdependence and inter-connectivity of risks (EC, 2012). Hence, understanding location-based risk and its relationship to natural or man-induced geo-environmental disasters is the first step in designing, developing, and executing a successful urban development strategy (Figure XX). This will undoubtedly involve intensive assessments in the disciplines of environmental geology, engineering geology, geomorphology and topography and urban geography.

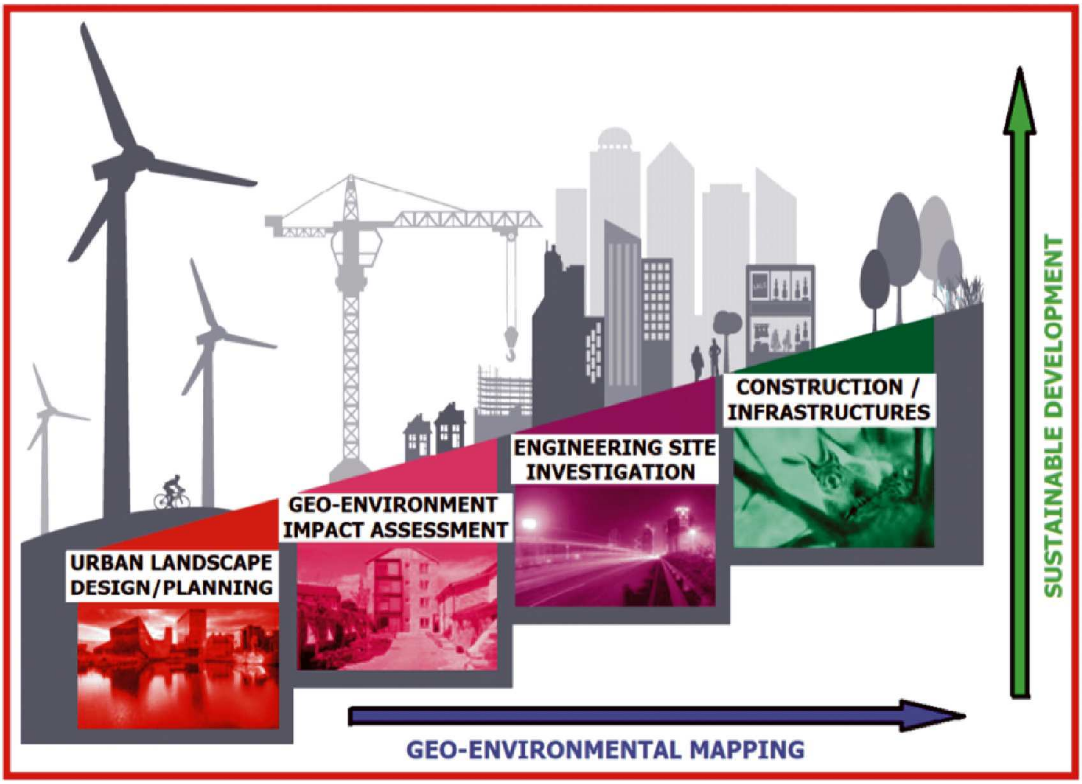

Fig. 10: Xxxxxxxx Xxxxxxxx

In other words, a proactive preventive geo-environmental mapping will help to release of funds that otherwise would be spent on disaster mitigation for development in other sectors.

To achieve sustainable urban development, there should be new ways of integrating geological information with knowledge of socio-and infrastructural development in Nigeria in order to minimize ecosystem disruptions.

The dynamics and processes controlling the natural geological and landscape evolution are well known by Geoscientists. However, the crux of the matter is geo-hazard prevention should start with the civil society's awareness that natural hazards exist and that there are adequate geological expertise can quantify and map them.

In this view, scientists, professionals and technicians working on landscape management have the duty of geo-knowledge information transfer. Thus it is will be necessary for geoscientists to disseminate information about geological factors in new and unprecedented ways vis-à-vis geologic environments, natural resources and hazards.

Recommendations

Natural hazards are inherently part of the human condition and experience. However, it is within the realm of geoscientists, emergency managers, policymakers, and the general public to mitigate their impacts and prevent the occurrence of natural hazards.

For effectively combat of the effect of and Land-use and Climate Change for sustainable development, food security and the well-being of Nigerians. There is the need for:

a) Establishment of Flood Early Warning System (FEWS) as a priority project of the Government to monitor flow discharge in many of our drainage / river basins.
b) A community-based resources management agency and partnership of municipalities within a River Basin Authorities.
c) Watershed level analysis of the issues and long-term maintenance of watershed-level data and information
d) Development of and implementing the Watershed Plans which should be developed collaboratively with the stakeholders
e) Watershed plan can deal with issues ranging from hazard management through to the maintenance and enhancement of natural heritage.
f) Understanding the natural fundamental geologic processes that produce hazards and development of appropriate geo-hazard assessment e.g. geologic mapping at the appropriate scale.
g) Integrate geological information and knowledge into overall flood hazard prevention and erosion disaster management.

A recently released report of the American Geosciences Institute (AGI) titled "How Geologists make a living in 2019" revealed that the majority of graduates in the US are developing careers by applying geosciences knowledge as opposed to deriving new knowledge as done in academic and/or research positions. The report, based on the workforce surveys conducted by the American Institute of Professional Geologists (AIPG), National Association of State Boards of Geology (ASBOG) and AGI, revealed that geologists are predominantly securing employment in three broad sectors:

- Environmental remediation and management
- Natural resource discovery and utilization
- Engineering and Construction.

The above demography is a testimony to the significant roles of geosciences expertise in socio-economic and

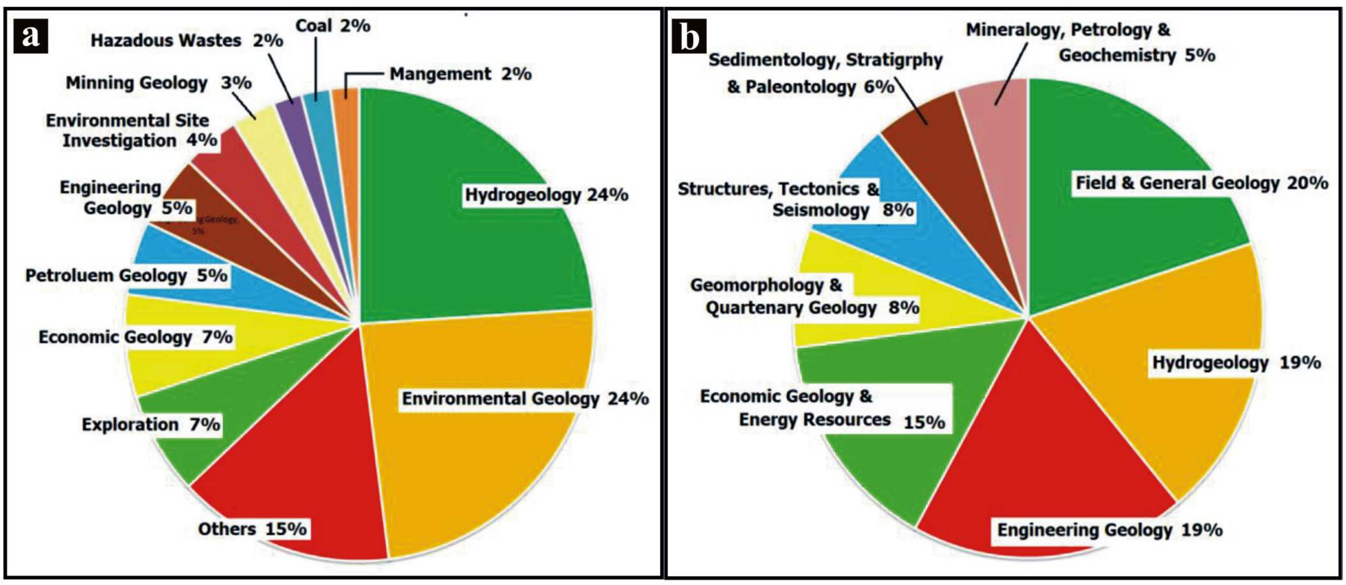

Fig. 11: **(a)** Areas of expertise for geologists who are members of the American Institute of Professional Geologists (Reproduce from AGI, 2019). **(b)** Task Analysis Survey Practice of Geology of National Association of State Boards of Geology (ASBOG) (Reproduce from AGI, 2019).

infrastructural development of 21st century society. The bottom-line is that it is imperative for our training Institutions (University and Polytechnics) to beef-up their academic curricula. The focus should be on skills that are needed for professional employment in the areas of Applied Geosciences (Hydrogeology, Engineering Geology and Environmental Geology).

On a final note, it should be pointed out that Geologists (Geoscientists) make use of their special knowledge for the benefit of others and no profession affects the public more than Geology. Let us bear in mind saying of William Durant that *"Civilization exists by geological consent, subject to change without notice"*. Therefore, proactive geo-environmental mapping and disasters risk assessment are the right steps in respect of preparedness and mitigation of possible impacts of such *"change without notice"*.

In other words, a proactive preventive geo-environmental mapping will help to release of funds that otherwise would be spent on disaster mitigation for development in other sectors. To achieve sustainable urban development, there should be new ways of integrating geological information with knowledge of socio-and infrastructural development in Nigeria in order to minimize ecosystem disruptions.

The dynamics and processes controlling the natural geological and landscape evolution are well known by Geoscientists. However, the crux of the matter is geo-hazard prevention should start with the civil society's awareness that natural hazards exist and that there are adequate geological expertise can quantify and map them.

In this view, scientists, professionals and technicians working on landscape management have the duty of geo-knowledge information transfer. Thus it is will be necessary for geoscientists to disseminate information about geological factors in new and unprecedented ways vis-à-vis geologic environments, natural resources and hazards.

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