Evaluation of Liquefaction Potential of Some Sites in Uyo Metropolis, Akwa Ibom State, Southeastern Nigeria

Abidemi O Ilori (✉ bidemiini@gmail.com)

University Of Uyo <https://orcid.org/0000-0002-5432-9085>

Iniobong Uloh Unufi

University of Uyo

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Evaluation of liquefaction potential of some sites in Uyo metropolis, Akwa Ibom State, Southeastern Nigeria

By

Abidemi O Ilori
Department of Civil Engineering,
Faculty of Engineering. University of Uyo.
Uyo, Akwa Ibom State. Nigeria.
bidemiini@gmail.com, abidemiilori@uniuyo.edu.ng

and

Iniobong Uloh Unufi,
Department of Civil Engineering,
Faculty of Engineering. University of Uyo.
Uyo, Akwa Ibom State. Nigeria.
iniobongunufi@yahoo.com

Abstract

Liquefaction potential analysis were carried out for some locations within Uyo metropolis, Akwa- Ibom state, Southeastern Nigeria using mostly cone penetration test data. The dominant soil is the Coastal Plain Sands. Unified Soil Classification System places soil within 20 m depth in the clayey sand (SC), silty sand(SM), poorly graded sand(SP) and dual combination of the three, namely SC-SM, SM-SP. Factor of safety(FS) was calculated for potential earth tremors with 7.5 and 4.5 moment magnitude and a peak ground acceleration (PGA) value of 0.16g. Other than the first layer in some the sites in which values of FS is greater than 1.0, the factors of safety for the 7.5 magnitude for all depth up to 20.0 m are less than one. For the 4.5 moment magnitude, the FS is less than 1.0 in some layers and greater than 1.0 in others. Liquefaction potential index (LPI) values for the sites range from 0 to 26.65, this places the level of liquefaction severity for all the sites in the ‘very low’ to ‘very high’ category based on level of severity classification. Potential settlement of the liquefiable layers estimated at the sites ranges from 8.38 cm to 58.48 cm. Settlement values were used to correlate liquefaction zones. Liquefaction potential index has a coefficient of correlation of 0.54 with settlement values.

Keywords. Uyo, liquefaction, factor of safety, potential settlement, correlation

1.0 Introduction

Although Nigeria is not located within the major seismic zones of the world, over the years, several earth tremors have been experienced in some parts of the country. These tremors with local magnitudes (between 4 and 5) were experienced from the early 1930s, and this led to in depth studies into the seismology of the country (Tsalha et al. 2015). Some studies have reported that earth tremors in Nigeria are triggered by intraplate tremors unlike what occurs in other regions of Africa (Eze et al. 2012). These tremors were attributed to regional stresses created as a result of the country's position between the West Africa craton and the Congo craton, and zones of weakness resulting from magmatic intrusions and other tectonic activities in the sedimentary areas (Eze et al. 2012; Afegbua et al. 2011). Most of the earlier earth quakes in Nigeria were not well documented because of the absence of seismological equipment then. Table 1 presents a list of historical earthquakes and tremors felt in Nigeria. Some missing information of some of the events in the table indicates that such were not documented. One of the most recent earth tremor event in Nigeria is the one of 2009 which was felt in Abeokuta, Ago-Iwoye, Ajambata, Ajegunle, Imeko, Ijebu-Ode, Ilaro and Ibadan towns, all in south western Nigeria. The tremors which had a local magnitude of 4.4 were reported to be triggered by a ruptured fault within the upper crust (Akpan et al. 2014). In 2016, series of earth tremors and quakes were felt in Saki, Oyo state also in south western Nigeria; such tremors and earthquakes also occur n communities in Bayelsa, and Rivers State in Southern Nigeria; in Jabal area of Kaduna state in North western Nigeria, (Vanguard.com 2017). The most recent occurred in September 2018 in the country's capital city, Abuja (Premium Times.com 2018). The causes of these events are presently uncertain and studies on the events are still ongoing, a clear indication that Nigeria may not be free of earth tremors in the near future. These events have led to damage of infrastructures such as buildings, even though not extensive. An earth tremor was experienced between Rivers and Bayelsa states in July and August 2016, which led to damage of road pavement as reported by Daily Post news (2016). The straight line geographical distance between this area and Uyo, is approximately about 180 km. Unfortunately this incidence was not scientifically documented as there are no earth tremors monitoring station in this area. One of the major phenomenon that accompany an earthquake or tremors is liquefaction of soil. The above tremor incidences have led to the present study which aims to investigate the effects of

such tremors on possible liquefaction of soil around Uyo metropolis, in Akwa –Ibom State, South eastern Nigeria.

From Table 1, earthquakes with magnitudes in the range of 3.7 on local magnitude scale (ML) to 6.5 on the Richter scale (Mr.) has been reported in the country. Oluwafemi et al (2018), using previous earthquakes values (Mr) make prediction of possible earthquakes in Nigeria of magnitude in the range of 3.0 to 7.1. In evaluating the 11th September, 2009 earthquake in south western Nigeria, Akpan et al, (2014) determined the local magnitude (M_L) to be 4.5, and 4.2 moment magnitude (M_w). Adepelumi et al (2011), based on the September 11, 2009 earthquake of magnitude 4.8 (M_w), estimated likely peak ground acceleration (PGA) values to be in the range from 0.16g to 0.69g. This study evaluates liquefaction potential of the soils at some locations in Uyo, based on earthquake magnitude (M_w) of 4.5 and a peak ground acceleration of 0.16g.

The study uses Cone Penetration Tests (CPT), Standard Penetration Test (SPT), and lithologic borehole with Soil Behavior Type (SBT charts- Robertson and Cabal, 2005) to characterize, and profile soil at seven sites in Uyo metropolis, southeastern Nigeria. The liquefaction potentials of the soil were then evaluated by estimating the factor of safety for soil profiles using different criteria for the CPT and SPT data as detailed by relevant previous workers (Boulanger and Idriss, 2014, Youd et al, 2004) on liquefaction. Potential settlements associated with the liquefiable layers were calculated, and implications of the results for the study area were highlighted

2.0 Study objectives

1. Geotechnical characterization of the selected sites using Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT) data
2. Using SPT data but mostly CPT, determine soil resistance developed by the soil in a Magnitude 7.5 event; the cyclic resistance ratio(CRR)
3. Determine shear resistance, due to magnitude 7.5 and 4.5 event. The cyclic shear ratio (CSR).

4. Estimate the factor of safety for each of the event for the different soil layers characterized in '1' above.
5. Evaluate liquefaction potential index for each of the site
6. Evaluate the potential settlement of areas where factor of safety of the magnitude 4.5 event is 1.3 or less

3.0 Description of study area and geology

Uyo is the capital of Akwa Ibom State in Nigeria, a State located in the coastal southern part of the country within the oil rich Niger Delta. Generally, it has relatively flat terrain with no undulating hills or valleys, except at the North eastern part of the city which has deep gully erosion sites. Uyo, lies between latitudes $4^{\circ} 58'$ and $5^{\circ} 6'$ North, and longitudes $7^{\circ} 48'$ and $8^{\circ} 02'$ East; and is located within the West African tropical rainforest belt. Its location in the Niger delta region necessarily leads to extensive urbanization that is characterized by infrastructural developments.

According to the Nigerian Geological Survey Agency (2006) base map of Akwa- Ibom State, the geology of Uyo is dominated by the Tertiary – Recent (Quaternary) sediments Coastal Plain Sands (Short and Stauble, 1967). The Coastal Plain Sands commonly called 'Benin Formation' is one of the six major geomorphic units that makes up Niger delta. The grains are sub angular to well rounded, and are believed to have been deposited in a continental fluvial to deltaic environment.

4.0 Ground water level and ground condition

Sand or sandy soil will not liquefy in unsaturated condition. Laboratory test results (Sherif et al., 1977) show liquefaction resistance for soils increases with decreasing degree of saturation. The ground water table underlay the study area at an average depth of 20 m (Evans et al, 2009). It is common knowledge in earthquake events that ground water is often forced to the ground surface which eventually causes the sandy layers it is forced through to liquefy. The potential for liquefaction of such layers of sandy soils in the study area forms the main theme for this study. Also in their work Evans et al, (2009) had indicated five layers of superficial sediments in delineating aquifer in the study area. These layers are dominantly sand and are

prone to continuous wetness especially during raining season. The layers on top of the layer containing aquifer form part the recharge channel for the aquifer which is within the fourth layer ; thus the likely hood of liquefaction of these layers in the presence of earth tremors with significant moment magnitude.

5.0 CPT versus SPT tool of liquefaction analysis

Among other features such as Quality control, repeatability of test, and detection of variability of soil deposits, the cone penetration test (CPT) has greater repeatability and reliability, and provides a continuous profile compared to SPT. It is the main tool used in this study, and also the fact that within the study area, SPT data are scare.

6.0 Materials and Method

6.1 Data acquisition.

Cone penetration test data from seven locations were utilized in the study. Their approximate geographical coordinates are listed in Table 2 and are indicated in Figure 2. One of the seven locations has Standard penetration test (SPT) data which is the only SPT data utilized in the study, while one other site has lithology borehole drilled up to 20.0 m in addition to CPT data at the site. The lithologic borehole is used as a check on the soil types indicated by the CPT data. Disturbed soil samples up to 3.0 m depth were taken at other location for soil type identification and classification purposes.

Five of the CPT data were acquired with 2.5 ton Guada Dutch cone penetrometer, while two were acquired with a 10 ton motorized rig CPT. The cone penetration tests acquires cone resistance (q_c), sleeve friction (f_s). Three of the sites were located within University of Uyo permanent site campus, while four sites were located in various parts of Uyo.

6.2 Data analysis

6.2.1 Cone Penetration Test data

The analysis of CPT data was carried out as follows.

- i. The CPT data was used to compute unit weights of the soil up to the depth of investigation using the equation by Robertson and Cabal (2005),

$$\frac{\gamma}{\gamma_w} = 0.27 [\text{Log} R_f] + 0.36 \left[\log \left(\frac{q_c}{P_a} \right) \right] + 1.236 \quad (1)$$

where R_f = friction ratio $= \left(\frac{f_s}{q_c} \right) 100 \%$

γ_w = unit weight of water in same units as γ

P_a = atmospheric pressure in same units as q_t (q_t is modified as q_c since Guada Dutch cone used does not acquire pore pressure)

- ii. Using normalized Soil Behavior Type (SBT chart, Robertson, 2010), the soil is classified into profiles with depth using the following equations

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5} \quad (2)$$

where:

I_c = soil behaviour index

Q_t = normalized cone penetration resistance (dimensionless)

$$= \left(\frac{q_c - \sigma_{vo}}{P_a} \right) \left(\frac{P_a}{\sigma'_{vo}} \right)^n \quad (3)$$

$$n = 0.381(I_c) + 0.05 \left(\frac{\sigma'_{vo}}{P_a} \right) - 0.15 \leq 1.0 \quad (4)$$

F_r = normalized friction ratio, in %

$$= \frac{f_s}{(q_c - \sigma_{vo})} X(100\%) \quad (5)$$

The two steps above were used to established soil profile at each of the CPT test location

iii. For each identified soil layer, an average cone, and friction resistance is determined by summing up the different resistances that make up the layer and finding their average.

The average cone resistance is then normalized for overburden pressure using the following equations;

$$q_{c1N} = C_Q \left(\frac{q_c}{P_a} \right) \quad (6)$$

$$C_Q = \left(\frac{P_a}{\sigma'_{vo}} \right)^n \quad (7)$$

where, q_{c1N} = equivalent clean sand normalized cone resistance for over-burden (Robertson and Wride 1998);

C_Q = normalizing factor for cone resistance;

q_c = cone tip resistance;

P_a = atmospheric pressure;

σ'_{vo} = vertical effective stress;

n = stress exponent defined in equation (4) above

iv. The cyclic resistance ratio at 7.5 quake magnitude, $CRR_{7.5}$ is then calculated as given by Boulanger and Idriss, (2014) which is dependent on the normalized cone resistance, $(q_{c1N})_{CS}$

$$CRR_{M=7.5, \sigma'_v=1 atm} = \exp \left[\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000} \right)^2 - \left(\frac{q_{c1Ncs}}{140} \right)^3 + \left(\frac{q_{c1Ncs}}{137} \right)^4 - 2.8 \right] \quad (8)$$

$CRR_{7.5}$ = cyclic resistance ratio for an equivalent magnitude 7.5 event

And $(q_{c1N})_{cs}$ = equivalent clean sand normalized cone penetration resistance;

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N} \quad (9)$$

q_{c1N} is as defined in equation (6) above, and

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6} \right) \exp \left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2} \right)^2 \right) \quad (10)$$

Where FC = Fine content (%) and given by the expression

$$FC = 80(I_c + C_{Fc}) - 137 \quad (11)$$

C_{Fc} is a constant that varies with soil layers. The percentage of fines obtained from shallow investigation was used to model this value on each site

Vii) Cyclic stress ratio, CSR for a 7.5 magnitude event is calculated using

$$'CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma_v'} r_d \frac{1}{MSF} \frac{1}{K_\sigma} \quad (12)$$

Where ,

a_{max} = peak horizontal ground acceleration;

g = acceleration of gravity;

σ_v = total vertical overburden stress and

σ_v' = effective vertical overburden stress, respectively, at a given depth below the ground surface;

r_d = depth-dependent shear stress reduction factor

MSF is the magnitude scaling factor, which is equal to 1 for the 7.5 event

K_σ = the overburden correction factor computed using equation given by Boulanger and Idriss, (2014)

$$K_\sigma = 1 - C_\sigma \ln \left(\frac{\sigma'_v}{P_d} \right) \leq 1.1 \quad (13)$$

Where C_σ is computed as

$$C_\sigma = \frac{1}{37.3 - 8.27 (q_{c1Ncs})^{0.264}} \leq 0.3 \quad (14)$$

And depth-dependent shear stress reduction factor computed as

$$r_d = \exp[\alpha(z) + \beta(z)M_w] \quad (15)$$

Where

$$\alpha(z) = -1.012 - 1.126 \sin \left(\frac{z}{11.73} + 5.133 \right) \quad (16)$$

and

$$\beta(z) = 0.106 + 0.118 \sin \left(\frac{z}{11.28} + 5.142 \right) \quad (17)$$

z = depth, the terms in bracket are in radians.

And the MSF is computed for each of the soil layer as

$$MSF_{max} = 1.09 + \left(\frac{q_{c1Ncs}}{180} \right)^3 \leq 2.2 \quad (18)$$

For Moment magnitude other than the 7.5.

For Moment magnitude 7.5, MSF_{max} is equal to 1. Moment magnitude are both 7.5 and 4.5 for this study

Equation 18 is proposed by Youd et al. (2001) to replace

$$MSF = 6.9 \exp \left(\frac{-M_w}{4} \right) - 0.058 \leq 1.8 \quad (19)$$

Where M_w is Moment magnitude

Viii) Factor of safety is then computed for the 7.5 and 4.5 Moment magnitude as

Factor of safety,

$$FS_{7.5} = \frac{CRR_{7.5}}{CSR_{7.5}} \quad (20)$$

and

$$FS_{4.5} = \frac{CRR_{4.5}}{CSR_{4.5}} \quad (21)$$

respectively

6.2.2 Standard Penetration Test data

SPT analysis proceeded as follows

- i) The soil profile as indicated by the SPT records is first set up for the site up to 20.0 m with respective layer's SPT blows
- ii) The field value of SPT blows is then corrected into standard value, $(N_1)_{60cs}$ as follows

$$(N_1)_{60cs} = N_m C_N C_E C_B C_R C_S \quad (22)$$

Where N_m = Field SPT 'N' value

Where

C_N = is a factor to normalize N_m to a common reference effective overburden stress;

C_E = correction for hammer energy ratio (ER);

C_B = correction factor for borehole diameter;

C_R = correction factor for rod length; and

C_S = correction for samplers with or without liners

C_S , C_B , and C_E are assumed to be 1.0, 1.0, and 0.6, respectively. Rod length correction with

respect the depth (CR) at the borehole location is corrected using the table presented by

Murthy (2007)

C_N is calculated using the equation

$$C_N = \left(\frac{P_a}{\sigma'_V} \right)^\alpha \leq 1.7 \quad (23)$$

Where, P_a = atmospheric pressure, 100kPa,

σ'_V = Effective overburden pressure, and

$$\alpha = 0.784 - 0.0768\sqrt{(N_1)_{60}} \quad (24)$$

iii) Calculation of cyclic resistance ratio

Cyclic resistance ratio (CRR) is calculated with the equation by Idriss and Boulanger (2006)

$$CRR = \exp \left\{ \frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126} \right)^2 - \left(\frac{(N_1)_{60cs}}{23.6} \right)^3 + \left(\frac{(N_1)_{60cs}}{25.4} \right)^4 - 2.8 \right\} \quad (25)$$

Where

$(N_1)_{60cs}$ = SPT N values corrected into standard form plus correction for percentage of

Fines,

$= (N_1)_{60} + \Delta(N_1)_{60}$, and

$$\Delta(N_1)_{60} = \exp \left(1.63 + \frac{9.7}{FC+0.1} - \left(\frac{15.7}{FC+0.1} \right)^2 \right) \quad (26)$$

iv) A correction for Overburden, K_o using equation (14).

Where

$$C_o = \frac{1}{18.9 - 2.5507\sqrt{(N_1)_{CS60}}} \leq 0.3 \quad (27)$$

- v) The cyclic stress ratio (CSR) is calculated using the same equation (24) above
- vi) Factors of safety computed as in equations (20) and (21) above.

6.3 Computation of liquefaction potential index

Iwasaki et al. (1978, 1982) proposed liquefaction potential index (LPI) as a single value expression to evaluate regional liquefaction potential for soil profile up to 20 m depth, they proposed the following;

$$LPI = \int_0^{20} F(z).w(z)dz \quad (28)$$

Where z = is the midpoint of the soil layer

dz = is the differential increment of depth

$F(z)$ = severity factor, which is calculated using the following expressions

$$F(z) = 1 - FS \text{ for } FS < 1.0$$

$$F(z) = 0, \text{ for } FS \geq 1.0.$$

$w(z)$ = weighting factor and is computed as

$$w(z) = 10 - 0.5z \text{ for } z < 20 \text{ m}$$

Instead of a single value of factor of safety assigned to depth less than 20 m, Luna and Frost (1998), proposes presents the following procedure which evaluates LPI of each soil layering and then sum them up. Their procedure is as follows

$$LPI = \sum_i^n w_i F_i H_i \quad (29)$$

Where

$$F_i = 1 - FS_i, FS_i < 1.0$$

$$F_i = 0 \text{ for } FS_i \geq 1.0$$

H_i = is the thickness of the discretized soil layers;

n = is number soil of layers;

F_i = is liquefaction severity for i -th layer;

FS_i = is the factor of safety for i -th layer;

w_i = is the weighting factor

$$=10-0.5z_i$$

z_i = is the depth of the i -th layer (m).

The Luna and Frost procedure is the one adopted for computation of LPI in this study.

6.4 Computations of settlements

Possible settlements in the case of seismicity of Magnitude 4.5 for each location were estimated using the relationships developed by Tatsuoka et al. (1990). The equations uses cone resistance values corrected for fines as developed by Youd et al. (2001), to estimate volumetric strain in percent. These relationships are presented in Appendix. The settlement of each soil layer is estimated based on

$$S = \sum_{i=1}^j \varepsilon_{vi} \Delta Z_i \quad (31)$$

Where

S = the calculated liquefaction-induced ground settlement at the CPT location

ε_{vi} = the post liquefaction volumetric strain for the soil sublayer i

ΔZ_i = the thickness of the sublayer i

j = the number of soil sublayers.

Due to the large volume of computations involved, excel worksheet was used in carrying out all the computations in this study.

7.0 Results and discussion

7.1 Soil indices and classification

The results of laboratory analyses used to identify and classify the soil types at three of the seven penetration sounding locations are presented in Table 3. The table presents one of these

results for shallow depths starting from the ground surface up to 4.0 m and the inferred soil type from soil behaviour type analysis (SBT) up to refusal depth. The table also presents the soil log from SPT and lithologic boreholes at two locations up to 20 m. Both borings provides soil profile at these locations up to 20 m.

Table 4 presents a typical worksheet analysis of CPT data for the classroom site location. The table presents soil index values I_c , unit weights, division of the soil into layers based on both soil index (I_c), and unit weights. The two parameters were reconciled to be able to group and profile the soils into layers. From all the CPT data, I_c values ranges from 1.22 to 4.26 indicating soil types ranging from gravelly sand to dense sand to organic soils – clay.

Laboratory analyses of both samples obtained from shallow depths, SPT, and lithologic borings indicate sandy soils, which are silty, or clayey, and sometimes pure sand. The soils are of low plasticity and not very large liquid limit indexes. Plasticity index ranged from 9% to about 18%, while liquid limits values are not more than 48%. The soils are classed into SC, SM, SC-SM, and SP soils under Unified Soil Classification System (USCS). From the SPT borehole presented there is occurrence of clay soil which is classed as Inorganic clay (CL), at around 18.0 m to 20.0 m depth.

7.2 Unit weights, soil behavior Index and site characterization

The soil classification results from samples recovered from shallow depth, 0 - 4.0m , were used in conjunction with the SBT index values and chart, unit weights, and the SPT boring logs to characterize soil profile in each site. Comparing the values of the unit weights computed with using equation (1), and the units weights obtained from the only SPT borings utilized in the study, indicates slight variations between the two. The average of the unit weight up to 20 m from the SPT boring log is 18.23 kN/m^3 while that of those computed by equation (1) is 18.19 kN/m^3 , a difference of 0.04 kN/m^3 ; equation (1) therefore can be said to reliable estimate of unit weights within the Coastal Plain Sands lithology and depth of investigation.

By reconciling the soil behavior index values(I_c), and the computed unit weights, the soil profile is developed for each site. Soil profiles at each site indicate highly stratified pattern. . For example the classroom site has seventeen soil strata within 11.75 m range, and the Bank Avenue site has thirty two strata within a 20 m depth range. Table 4 presents a typical profile The soil

profile for the study area is made up of sand mixtures which consists of clayey sand and silty sands, interspersed frequently with clays and silt mixtures, and sometimes pure sands.

7.3 Potential for liquefaction due to soil type

Figure 1 is a chart developed by Tsuchida (1970), which is a sieve analysis plot that shows envelopes for both potential liquefiable soils and soils that are liquefiable. A plot of sieve analysis results of soil recovered from SPT boring, borehole three(BH3) and from lithologic boring puts all the soil within the envelope for liquefiable soils. This indicates that soils within the study area will undergo liquefaction.

7.4 Liquefaction analysis

The liquefaction analysis results can be group into two, those on University of Uyo permanent site campus, and those outside the campus. Tables 5, and 6, presents typical liquefaction analysis results for two sites; namely; the hostel block site, and the classroom block site, all at the University of Uyo permanent site campus, while Table 7 presents SPT analysis for the specialist hospital site. Table 8 presents a summary of the factors of safety with depth for all the sites studied.

For the 7.5 magnitude event, the factor of safety (FS) are less than 1.0 for all the soil layers in all the site investigated, except the first layer. On all the sites the factor of safety is more than 1.0 except the auditorium, Abak road and Bank avenue sites, which have factor of safety for the same layer less than 1.0. The first layer has a thickness of 0.50 m to 0.75 m in most of the site.

Table 8 presents factor of safety values for the 4.5 event. From this table, for layer 1; the 1000 seater auditorium and the Abak road site have FS less than 1.0 while the remaining five sites have FS greater than 1.0 for this layer and the layer thickness varies from 0.5 m to 0.75 m.

For other layers, the hostel block site at University of Uyo campus has two depth ranges in which FS are least at 1.09 and 1.19, though slightly more than 1.0. The first between 0.50 m and 3.25 m depth with a thickness of 2.75 m, and the second at 10.0 m to 12.0 m depth with a thickness of 2.0 m. These two layers contribute more than ninety percent of the settlement at this

site. The first zone with FS of 1.09 extends to the classroom block site, occurring at 1.50 m to 4.5 m, a thickness of 2.5m although interspersed with a layer having FS greater than 1.0 m. The classroom block, and the hostel block site are within the same location and are about 170 m apart, although the classroom block is at a slightly lower elevation than the hostel block; hence the inter fingering of the liquefiable layers. The auditorium site is on the other hand has liquefiable soils all through up the 15.75 m depth with FS for all the soil layers less than 1.0. It is also at some distance and at a far lower elevation than the other two blocks.

Outside University of Uyo, the Dominic Etuk site has soil with FS less than 1.0 occurring at the depth range of 0.5 m to 4.0 m. a thickness of 3.5 m. This is within the depth range of the two sites in University of Uyo that is characterized with FS less than 1.0, except the auditorium site.

The Specialist hospital site and the Bank Avenue site even though widely spaced apart based on their geographical coordinates, have similar profile with respect to liquefiable soil layers. The specialist hospital site has a layer with FS less than 1.0 in the 0.75 -1.00 m depth range, bounded by the non-liquefiable layers at the top and bottom up to 1.25 m depth, thereafter liquefiable layer is up to 13.0 m. The 13.0 m to 20 m depth is interspersed with two layers of 3.0 m and 2.0 m thick non liquefiable soil strata. For the Bank Avenue site, continuous soil layers with FS less than 1.0 is from 0.70 m depth to 12.60 m depth, thereafter the soil up to 20 m depth is interspersed with a liquefiable layer of 1.1 m thickness in the 14.10 m and 15.20 m depth range. The Abak road site with FS less than 1.0 up to refusal depth of 14.75m is continuously liquefiable up to that depth.

The SPT analysis presented in Table 7 indicate FS is less than 1.0 in layer 2 of the soil strata, and FS greater 1.0 in all other layers in discordant to the result from CPT analysis for the same site. Hoque et al (2017), in a presentation involving comparative analysis of the use of SPT and CPT to evaluate the liquefaction potential of soil at the bank of Jamuna River, Bangladesh, Indicate similar result. Four SPT and CPT data were utilized in their study. For the depths investigated which is between 3.0 m to 20.0 m, the results of the four analyses consistently gave higher FS values by SPT than that given by the CPT. The results of the only SPT analysis obtained in the present study when compared with the CPT analysis result are in consonance

with reported trend. SPT analysis gives less conservative values or overestimate FS values than that of CPT. CPT values are therefore to be relied upon for evaluation purposes.

7.5 Correlation of liquefiable layers

It is possible to correlate the thickness of continuously liquefiable layer among three of the sites. The Bank Avenue and the Ibom specialist hospital sites have continuous liquefiable layer of 12.4 m and 12.25 m respectively. The Bank Avenue site at an average elevation of 66.4 m above sea level is at about 3.0 m below the specialist hospital level which is at an elevation of 69.4 m. Referencing the same level at the specialist hospital site, the remaining thickness of the continuous liquefiable layer at the hospital site will be 10.25 m. This layer is believed to extend in stratum and thickness to the Bank Avenue site with an approximate thickness of 9.90 m, and is also the one that outcrops at the Abak road site where the thickness is 14.75 m.

The Soil Behaviour Type index I_c parameter, which identifies the type of soil present is also used to correlate the soil layers at these sites. The Soil Behaviour Type index values I_c , for the top part of the liquefiable soil at Abak road, the specialist hospital and Bank avenue sites are from computations 2.37, 2.79 and 2.37. Based on Robertson (2010) chart, the soil represented by these values are ‘silty sand to sandy silt’ and silt mixtures (between the lower part of zone 5 and upper part of zone 4 of the chart), these are the soil that forms the top part of the liquefiable layers on the three sites thereby correlating their lithology at the sites. Figure 3 present a correlation of liquefiable layers from the three sites.

7.6 Liquefaction potential index (LPI) values

Liquefaction potential index was estimated based on Luna and Frost (1998). The values for each site are presented in Table 8. The values are from ‘0’ to 26.65. Based on table of classification of level of liquefactions severity by Dixit et al (2012) presented in Table 9, these values severity level ranged from ‘very low’ to ‘very high’ based on Iwasaki et al. (1982), Luna and Frost (1998), and MERM (2003) classification schemes. Figure 2 presents a map of Uyo metropolis showing possible liquefaction zones based on computed LPI values.

7.7 Settlement values.

Probable settlements due to possible liquefaction computed as indicated in section '6.4' above are computed for each soil layer at each site. A value for total settlement at each site is calculated by summing up the settlements for each layer. These values are presented also in Table 8. The probable settlement values can be grouped into three. The classroom and the hostel block at 12.71 cm and 8.37 cm constitute one set; the 100 seater auditorium, the Abak road, the specialist hospital, and the Bank Avenue sites constitute a group with settlement values at 43.41, 58.4 cm, 57.64 cm, and 55.34 cm. This group consists of the sites whose liquefiable layers are correlated in section 7.5 above. These settlement values for them serves further substantiate the correlation. The Dominic Etuk site stand-alone but constitutes a group with settlement value at 28.24 cm.

The LPI values were correlated with the settlement values and a correlation coefficient 'r' of 0.54 was obtained. Generally the settlement values are proportional to the thickness of liquefiable layers in each site, with a correlation coefficient 'r' of 0.92. Liquefaction potential index (LPI) was also correlated with thickness of liquefiable layer; a correlation coefficient value 'r' of 0.80 was obtained. Expectedly thicknesses of liquefiable layers have direct bearing on LPI values and also the amount of expected settlement.

Zhang et al. (2002), uses a similar method to estimate settlement due to the 1989 Loma Prieta earthquake in District and Treasure Island all in San Francisco area of California, U.S.A. He presented the results of the method and the actual measurement of settlement that took place. This method of settlement estimation gives values slightly on the conservative side in some situation and are not in some others. In some cases, the prediction almost and sometimes matches actual settlement values. Based on the above, the estimated total settlements in this study can be said to be conservative. In situations where a seismic event does not trigger liquefaction in all the liquefiable layers, settlement will be sum of the layers that are affected. The fact that some layers of soil do not liquefy in an earthquake event, even though they are liquefiable was reported by Bertalot et al, (2013). In their study, Zhang et al. (2002) provide a graphic signature log of normalized cone values corrected for fines, $Q_{c(1N)cs}$ with depth and factor of safety values with depth; these two graphic log signatures are similar. The same log signature types from this study are presented for the hostel site. These are presented in Figure 4. The signatures are similar;

though Zhang et al. uses Robertson and Wride (1998), approach in calculating $q_{(c1N)cs}$, whereas this study uses the method by Boulanger and Idriss, (2014).

8.0 Implication of findings to the study area

There is at present no provision for seismic consideration in building design codes for the study area. In the light of the findings of this study, provision for such consideration should now be given in the design and construction of buildings within this area. Most of the buildings in the study area are bungalows structures, single or two storey structures, and multistorey structures whose foundations are mostly placed at between 1.20 m to 1.80 m depth representing the second layer in most of the sites except for the multistory which are mostly on deep foundation. All such structures will undergo settlement associated with the soil layer on which they are founded upon if such liquefy. In a study of building that were affected by 1999 Turkey earthquake. Called “Adapazari failures”, Gazetas et al (2004) carried out measurement and noted that significant tilting and toppling were observed only in relatively slender buildings (with aspect ratio: $H / B > 2$), provided they were laterally free from other buildings on one of their sides. For the prevailing soil conditions and type of seismic shaking; most buildings with $H / B > 1.8$ overturned, whereas building with $H / B < 0.8$ essentially only settled vertically, with no visible tilting. Soil profiles based on three SPT and three CPT tests, performed in front of each building of interest, reveal the presence of a number of alternating sandy-silt and silty-sand layers, from the surface down to a depth of at least 15 m. Estimate of peak ground acceleration was 0.2g to 0.3g This soil profile in the cited case is similar to the one under study, based on the above, and as a first step in limiting potential damaged to buildings within the study area, the ratio of building height to width should be limited to the ratio $H/B < 0.8$ since from the case cited it ensures such buildings will not undergo tilting and ensure uniform settlement. Although the type of shallow foundation (independent footing or mat) the buildings have was not indicated in the example, it appears it is not too significant to the result.

9. Conclusion

The soil profile at seven sites within Uyo metropolis were evaluated for liquefaction potential based on assumed 7.5 and 4.5 moment magnitude earth tremors with a possible peak ground acceleration (PGA) of 0.16g.

The soils at the sites based on classification of liquefiable soil types classify as ‘most liquefiable soil’ types

Except at four sites where factor of safety is greater 1 for the first layer, FS for first layer and other layers at depth for all the sites are less than 1.0 for the 7.5 moment magnitude earth tremors.

For the 4.5 moment magnitude, FS takes on values greater than 1.0 and less than 1.0 at various depths on the sites giving rise to different thicknesses of liquefiable layer for the different sites. Liquefaction potential index values places all the sites in ‘very low’ to ‘very high’ classification. The associated potential settlement estimate due to liquefiable layers at site is also high at a least value of 8.37 cm and largest value of 58.4 cm. LPI values have some correlation with settlement values.

The study shows that some location within Uyo city may be more affected by liquefaction than others as indicated by LPI values that range from 0 to 29.74

This study represents a baseline study for liquefaction potential evaluation for the study area.

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Declaration

The authors’ wishes to declare that the interpretation of the CPT data provided by the two bodies acknowledged above and as used in this study are the authors interpretation and in no way have influence on the way or decision such data have been used by such bodies.

Competing Interest - The authors declare no competing Interest.

References

Adepelumi AA, Yakubu TA, Alao, OA, Adebayo AY (2011). Site Dependence Earthquake Spectra Attenuation Modeling: Nigerian Case Study. *International Journal of Geosciences*, 2011, 2, 549-561. <http://www.SciRP.org/journal/ijg>. doi:10.4236/ijg.2011.24058

Allen JRL (1965) Late quaternary Niger delta and adjacent areas: sedimentary environmental and lithofacies. *AAPG Bull* 49(5):547–600

Akpan OU, Yakubu TA (2010). A review of earthquake occurrences and observations in Nigeria. *Earthquake Science* (3):289 – 294.

Akpan OU, Isogun MA, Yakubu TA, Adepelumi AA, Okereke CS, Oniku AS. and Oden MI, (2014) An Evaluation of the 11th September, 2009 Earthquake and Its Implication for Understanding the Seismotectonics of South Western Nigeria. *Open Journal of Geology*, 4, 542-550. <http://dx.doi.org/10.4236/ojg.2014.410040>.

Afegbua KU, Yakubu TA, Akpan OU, Duncan D, and Usifoh ES, (2011). Towards an integrated seismic hazard monitoring in Nigeria using geographical and geodetic techniques, *International Journal of the Physical Sciences* 6 (28): 6385 – 6393.

Bertalot, D, Brennan AJ, and Villalobos FA, (2013). Influence of bearing pressure on liquefaction-induced settlement of shallow foundations. *Géotechnique*, 63(5), 391–399. <https://doi.org/10.1680/geot.11.P.040>

Boulanger RWI, and Idriss M, (2014). CPT and SPT based liquefaction triggering procedures. Center for Geotechnical Modeling. Report no. UCD/CGM-14/01
Daily Post, September 25 dailypost.ng/2016/09/25/earth-tremor-rivers-bayelsa-communities-cry-out/

Dixit J, Dewaikar DM, and Jangid RS, (2012). Assessment of liquefaction potential index for Mumbai city. *Natural Hazards and Earth System Sciences*. doi:10.5194/nhess-12-2759

Evans UF, George NJ, Akpan AEI, Obot B, and Ikot AN, (2009). A Study of Superficial Sediments and Aquifers in Parts of Uyo Local Government Area, Akwa Ibom State, Southern Nigeria, Using Electrical Sounding Method

Eze CL, Sunday VN, Ugwu SA, Uko ED, and Ngah SA, (2012). Mechanical model for Nigerian interpolate earth tremors. *Advances in Science and Technology* 6(2): 80 – 84.

Gazetas G, Apostou M, and Anasta- Sopoular J, (2004), Seismic Bearing Capacity Failure and Overturning of Terveler Building in Adapazari 1999, *Proc. Fifth Inter. Conf on Case histories in Geotechnical Engineering*. New York CD ROM –SOAP11(1-51), 2004.

Hoque MM, Ahmed M, Siddique AA, (2017). Evaluation of Liquefaction Potential from SPT and CPT: a Comparative Analysis. *Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering*, Seoul 2017.

<https://www.issmge.org/uploads/publications/1/45/06-technical-committee-02-tc102-13.pdf>
(accessed 27/5/2020)

Idriss IM, and Boulanger RW, (2006). Semi-empirical procedures for evaluating liquefaction potential during earthquakes, *Soil Dynam. Earthq. Eng.*, 26, 115–130,

Iwasaki T, Tokida K, Tatsuko F, and Yasuda S, (1978). A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan, *Proceedings of 2nd International Conference on Microzonation*, San Francisco, 885–896,.

Iwasaki T, Tokida K, Tatsuoka F, Watanabe S, Yasuda S, and Sato H, (1982). Microzonation for soil liquefaction potential using simplified methods, *Proceedings of 2nd International Conference on Microzonation*, Seattle, 1319–1330,

Luna R. and Frost JD, (1998). Spatial liquefaction analysis system, *J. Comput. Civil Eng.*, 12, 48–56.

Microzonation for Earthquake Risk Mitigation (MERM), (2003). *Microzonation Manual*, World Institute for Disaster Risk Management,

Murthy VNS, (2007). *Soil Mechanics and Foundation Engineering*. CBS Publishers. New Delhi. p 593

Nigerian Geological Survey Agency, (2006). *Geological and Mineral Map of Akwa-Ibom State*, Nigeria

Oluwafemi, J., Ofuyatan, O., Oyebisi, S., Alayande, T., Abolarin, J.,(2018). Probabilistic Seismic Hazard Analysis of Nigeria: The Extent of Future Devastating Earthquake. IOP Conf. Series: Materials Science and Engineering 413 (2018) 012036
doi:10.1088/1757-899X/413/1/012036

PremiumTimes.com2018.<https://www.premiumtimesng.com/top-news/282825-Abuja-tremor-not-earthquake-nigerian-govt.html>,date. Retrieved: 15 Sept 2018.

Robertson PK, and Cabal KL, (2010). Estimating soil unit weight from CPT. 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, May 2010

Robertson PK, and Wride (Fear) CE, (1998). Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, 35(3): 442–459.

Robertson PK, (2010). Soil behaviour type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT'10,Huntington Beach, CA, USA. www.cpt10.com

Sherif MA, Ishibashi I, and Tsuchiya C, (1977). "Saturation Effects on Initial Soil Liquefaction," Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT8, .

Short KC, Stauble AJ (1967) Outline of geology of Niger Delta. AAPG Bull 51:767

Tatsuoka F, Zhou S, Sato T, and Shibuya S, (1990). Method of evaluating liquefaction potential and its application. In Report on seismic hazards on the ground in urban areas, Ministry of Education of Japan, Tokyo. (in Japanese.)

Tsuchida H, (1970)."Prediction and Countermeasure Against the Liquefaction in Sand Deposits," Abstract of the Seminar in the Port and Harbor Research Institute (in Japanese),

Tsalha MS,. Lar UA,.Yakubu TA, Kadiri UA, and Duncan D, (2015). The review of the historical and recent seismic activity in Nigeria. IOSR Journal of Applied Geology and Geophysics 3 (1): 8-56.

Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, Christian JT, Dobry R, Finn WDL, Harder LF, Hynes ME, Ishihara K, Koester JP, Liao SSC, Marcuson WF, Martin GR, Mitchell JK, Moriwaki Y, Power MS, Robertson PK, Seed RB, and Stokoe KH, (2001).Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSFworkshops on evaluation of liquefaction resistance of soils, J. Geotechnical and Geoenvironmental Eng.,ASCE 127(10), 817–33.

Vanguard.com. Earthquake in Nigeria: measures to avert devastating impacts – experts 2017, Retrieved 2017 – 10 29.

Zhang G, Robertson PK, and Brachman RWI, (200). Estimating liquefaction-induced ground settlements from CPT for level ground. Canadian Geotechnical Journal. 39: 1168–1180. National Research Council, Canada..DOI: 10.1139/T02-047

Appendix

Tatsuoka et al. (1990) relationships for computation of settlements due to liquefaction

1. If $FS \leq 0.5$, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \leq (q_{c1N})_{cs} \leq 200$
2. If $FS = 0.6$, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \leq (q_{c1N})_{cs} \leq 147$
3. If $FS = 0.6$ $\varepsilon_v = 2411(q_{c1N})_{cs}^{-1.45}$ for $147 \leq (q_{c1N})_{cs} \leq 200$
4. If $FS = 0.7$, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \leq (q_{c1N})_{cs} \leq 110$
5. If $FS = 0.7$ $\varepsilon_v = 1701(q_{c1N})_{cs}^{-1.42}$ for $110 \leq (q_{c1N})_{cs} \leq 200$
6. If $FS = 0.8$ $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \leq (q_{c1N})_{cs} \leq 80$
7. If $FS = 0.8$ $\varepsilon_v = 1690(q_{c1N})_{cs}^{-1.46}$ for $80 \leq (q_{c1N})_{cs} \leq 200$
8. If $FS = 0.9$, $\varepsilon_v = 102(q_{c1N})_{cs}^{-0.82}$ for $33 \leq (q_{c1N})_{cs} \leq 60$
9. If $FS = 0.9$, $\varepsilon_v = 1430(q_{c1N})_{cs}^{-1.48}$ for $60 \leq (q_{c1N})_{cs} \leq 200$
10. If $FS = 1.0$, $\varepsilon_v = 64(q_{c1N})_{cs}^{-0.93}$ for $33 \leq (q_{c1N})_{cs} \leq 200$
11. If $FS = 1.1$, $\varepsilon_v = 11(q_{c1N})_{cs}^{-0.65}$ for $33 \leq (q_{c1N})_{cs} \leq 200$
12. If $FS = 1.2$, $\varepsilon_v = 9.7(q_{c1N})_{cs}^{-0.69}$ for $33 \leq (q_{c1N})_{cs} \leq 200$
13. If $FS = 1.3$, $\varepsilon_v = 7.6(q_{c1N})_{cs}^{-0.71}$ for $33 \leq (q_{c1N})_{cs} \leq 200$
14. If $FS = 2.0$, $\varepsilon_v = 0$ for $33 \leq (q_{c1N})_{cs} \leq 200$

Table 1. List of Historical Earth Tremors in Nigeria (Adapted from Akpan and Yakubu 2010 and Eze et al 2012)

S/N	Year-Month-Day	Origin Time	Felt Areas	Intensity/ Magnitude	Probable Epicenter	Coordinates	
1	1933	-	Warri	-	-	05° 45' 23"E	05° 31' 42" N
2	1939-06-22	19:19:26	Lagos, Ibadan, Ile-Ife	6.5 (Ml)	Akwapin fault in Ghana	03° 23' 00"E	06° 30' 11" N
3	1948-07-28	-	Ibadan	-	Close to Ibadan	-	-
4	1961-07-2	15:42	Ohafia	-	Close Ohafia area	07° 47' 21"E	05° 37' 15" N
5	1963-12-21	18:30	Ijebu-Ode	V	Close to Ijebu-Ode	-	-
6	1981-04 -23	12:00	Kundunu	III	At Kundunu village	-	-
7	1982-10-16	-	Jalingo, Gembu	III	Close to Cameroun Volcanic Line	-	-
8	1984-07-28	12:10	Ijebu-Ode, Ibadan, Shagamu, Abeokuta	VI	Close to Ijebu-Ode	-	-
9	1984-07-12		Ijebu Remo	IV	Close to Ijebu - Ode	03°22' 00"E	07° 11' 45" N
10	1984-08-02	10:20	Ijebu-Ode, Ibadan, Shagamu, Abeokuta	V	Close to Ijebu-Ode	-	-
11	1984-12-08	-	Yola	III	Close to Cameroun Volcanic Line	-	-
12	1985-06-18	21:00	Kombani Yaya	IV	Kombani Yaya	-	-
13	1986- 07-15	10 :45	Obi	III	Close to Obi town	08 °46'E	08° 22'N
14	1987-01-27	-	Gembu	V	Close to Cameroun Volcanic Line	11° 15'E	06° 42'N
15	1987 – 03-19	-	Akko	IV	Close to Akko	10° 57'E	10° 17'N
16	1987-05-24	-	Kurba	III	Close to Kurba village	10° 12'E	11° 29'N
17	1988-05-14	12:17	Lagos	V	Close to Lagos	-	-
18	1990-06-27	-	Ibadan	3.7(ML)	Close to Ijebu-Ode	03° 58'E	07 °22'N
19	1990-04-5	-	Jerre	V	Close to Jerre Village	-	-
20	1994-11-07	05:07:51	Ojebu-Ode	4.2(ML)	Dan Gulbi	-	-
21	1997	-	Okitipupa	IV	Close to Okitipupa Ridge	-	-
22	2000-08-15		Jushi-Kwari*	III	Close to Jushi Kwari village	07° 42'E	14° 03'N
23	2000-03 -13	-	Benin	IV	Benin City (55Km from Benin)	-	-
24	2000-03-07	15:53:54	Ibadan, Akure, Abeokuta, Ijebu-Ode, Oyo	4.7(ML)	Close to Okitipupa	-	-
25	2000-05-07	11:00	Akure	IV	Close to Okitipupa Ridge	-	-
26	2001-05-19	-	Lagos	IV	Close to Lagos city	-	-
27	2002-08-08	-	Lagos	IV	Lagos city	-	-
28	2005-03	-	Yola	III	Close to Cameroun Volcanic Line	-	-
29	2006-03-25	11:20	Lupma	III	Close to Ifewara- Zungeru Fault	-	-
30	2009-09-11	-	Abomey-Calavi	II	Close to Benin	-	-
31	2011-11-05	-	Abeokuta	4.4	Close to Abeokuta	-	-

Table 2. Geographical coordinates of test locations used in the study

Site locations	Approximate geographical coordinates	Mean elevation above sea level (m)	Testing tool
Hostel block site,	5° 2' 29.19" N		
University of Uyo	7° 58' 24.27" E		Cone Penetration Test
Classroom block site,	7° 58' 32.64" E		Cone Penetration Test
University of Uyo.	7° 58' 41.19" E		Cone Penetration Test
1000 seater auditorium site,	7° 56' 25.11" E		Cone Penetration Test, and
University of Uyo	7° 53' 2.4" E	69.4	Lithologic borehole
Dominic Etuk site,	5 ° 1' 46.31" N		Cone Penetration Test, and
Specialist hospital site	7° 54' 41.49" E	65.0	Standard Penetration Test.
Abak road site	4° 59' 54.25" N	66.4	Cone Penetration Test
Bank Avenue site	7° 55' 36.05" E		Cone Penetration Test

Table 3 Soil profile, soil types from SPT log, lithologic borehole and at shallow depth from three of the sites investigated.

[illegible]

Table 4 Soil profile, soil layering, and soil types for the classroom block site as indicated by CPT soil index values(I_c), SBT chart, and unit weights

Depth (m)	Cone resistance, qc (kN/m2)	Friction resistance fs (kN/m2)	Friction ratio (%)	Normalized cone resistance Qtn	Normalized friction ratio	Overburden pressure	Unit weight (kN/m3)	Average Unit weight for each soil strata (kN/m3)	SBT Index, I_c	Soil type from SBT chart
0	0	0.00	#DIV/0!	0.00	#DIV/0!	0.00			#NUM!	
0.25	2942.1	30.00	1.02	29.42	1.02	4.33	17.33		2.35	Sand mixtures – silty sand to sandy silt
0.5	2451.8	30.00	1.22	24.52	1.23	8.63	17.26	17.26	2.46	Sand mixtures – silty sand to sandy silt
0.75	1961.4	30.00	1.53	19.61	1.54	12.88	17.18		2.59	Sand mixtures – silty sand to sandy silt
1	1471.1	30.00	2.04	14.71	2.06	17.07	17.07		2.77	Silt mixtures – clayey silt to silty clay
1.25	1471.1	20.00	1.36	14.71	1.38	20.75	16.60	16.94	2.67	Silt mixtures – clayey silt to silty clay
1.5	1765.3	30.00	1.70	17.65	1.72	25.71	17.14		2.66	Silt mixtures – clayey silt to silty clay
1.75	2451.8	30.00	1.22	24.52	1.24	30.21	17.26		2.46	Sand mixtures – silty sand to sandy silt
2	2451.8	30.00	1.22	24.52	1.24	34.53	17.26		2.46	Sand mixtures – silty sand to sandy silt
3.5	2255.6	30.00	1.33	22.56	1.37	60.31	17.23		2.51	Sand mixtures – silty sand to sandy silt
3.75	2451.8	30.00	1.22	24.52	1.26	64.74	17.26		2.46	Sand mixtures – silty sand to sandy silt
4	2746	50.00	1.82	27.46	1.87	71.58	17.90	17.90	2.52	Sand mixtures – silty sand to sandy silt
4.25	2451.6	70.00	2.86	24.52	2.95	77.52	18.24	18.24	2.68	Silt mixtures – clayey silt to silty clay
4.5	2451.6	10.00	0.41	24.52	0.42	72.00	16.00	16.00	2.25	Sand mixtures – silty sand to sandy silt
4.75	2451.8	90.00	3.67	24.52	3.81	88.01	18.53		2.75	Silt mixtures – clayey silt to silty clay
5	2451.8	100.00	4.08	24.52	4.24	93.25	18.65		2.78	Silt mixtures – clayey silt to silty clay
5.25	2451.6	100.00	4.08	24.52	4.25	97.91	18.65	18.77	2.78	Silt mixtures – clayey silt to silty clay
5.5	2451.6	130.00	5.30	24.52	5.54	104.23	18.95		2.86	Silt mixtures – clayey silt to silty clay
5.75	2647.9	120.00	4.53	26.48	4.73	108.61	18.89		2.79	Silt mixtures – clayey silt to silty clay
6	2451.8	130.00	5.30	24.52	5.56	113.71	18.95		2.86	Silt mixtures – clayey silt to silty clay
6.25	3728.7	80.00	2.15	37.29	2.21	115.96	18.55	18.55	2.46	Sand mixtures – silty sand to sandy silt
6.5	2942.1	130.00	4.42	29.42	4.61	123.64	19.02		2.75	Silt mixtures – clayey silt to silty clay
6.75	2942.1	130.00	4.42	29.42	4.62	128.39	19.02		2.75	Silt mixtures – clayey silt to silty clay
7	2942.1	150.00	5.10	29.42	5.34	134.30	19.19	19.08	2.79	Silt mixtures – clayey silt to silty clay
7.25	2942.1	170.00	5.78	29.42	6.07	140.14	19.33		2.83	Silt mixtures – clayey silt to silty clay
7.5	3246.3	110.00	3.39	32.46	3.54	141.50	18.87		2.64	Silt mixtures – clayey silt to silty clay
7.75	3922.8	100.00	2.55	39.23	2.65	145.93	18.83	18.83	2.49	Sand mixtures – silty sand to sandy silt
8	3922.8	170.00	4.33	39.23	4.51	155.52	19.44	19.44	2.65	Silt mixtures – clayey silt to silty clay
8.25	3922.8	130.00	3.31	39.23	3.45	157.83	19.13		2.57	Sand mixtures – silty sand to sandy silt
8.5	3432.5	100.00	2.91	34.33	3.06	159.62	18.78		2.58	Sand mixtures – silty sand to sandy silt
8.75	4413.2	130.00	2.95	44.13	3.06	167.79	19.18	19.01	2.50	Sand mixtures – silty sand to sandy silt
9	3628.6	130.00	3.58	36.29	3.76	171.91	19.10		2.62	Silt mixtures – clayey silt to silty clay
9.25	4413.2	100.00	2.27	44.13	2.36	174.59	18.87		2.42	Sand mixtures – silty sand to sandy silt
9.5	4413.2	190.00	4.31	44.13	4.50	186.32	19.61	19.61	2.62	Silt mixtures – clayey silt to silty clay
9.75	3432.5	70.00	2.04	34.33	2.15	179.09	18.37		2.48	Sand mixtures – silty sand to sandy silt
10	4413.2	130.00	2.95	44.13	3.08	191.77	19.18		2.50	Sand mixtures – silty sand to sandy silt
10.25	4413.2	130.00	2.95	44.13	3.08	196.56	19.18	19.11	2.50	Sand mixtures – silty sand to sandy silt
10.5	4903.5	200.00	4.08	49.04	4.26	206.98	19.71		2.57	Sand mixtures – silty sand to sandy silt
10.75	3922.8	130.00	3.31	39.23	3.50	205.66	19.13		2.58	Sand mixtures – silty sand to sandy silt
11	4413.2	270.00	6.12	44.13	6.44	220.19	20.02		2.73	Silt mixtures – clayey silt to silty clay
11.25	3922.8	150.00	3.82	39.23	4.05	217.08	19.30	19.66	2.62	Silt mixtures – clayey silt to silty clay
11.5	10787.7	270.00	2.50	107.88	2.56	234.14	20.36	20.36	2.17	Sand mixtures – silty sand to sandy silt
11.75	19614	200.00	1.02	196.14	1.03	237.87	20.24	20.24	1.71	Sands – clean sand to silty sand

Table 5 Liquefaction analysis result for the hostel block site for 7.5 and 4.5 moment magnitude for 0.16g ground acceleration																	
Depth (m)	Depth range (m)	Thickness layer (m)	Soil bulk weight (kN/m ³)	Effective pressure, σ'_v (kN/m ²)	pressure, σ_v (kN/m ²)	Soil behavior (SBT) index , Ic	Average Skin f _s (kN/m ²)	Average cone e q _c , (kN/m ²)	Normalize resistance, q _{c1N}	Normalized cone corrected for fines, q _{c1n(cs)}	Overburde correction factor,,Ko	Cyclic resistance 7.5 event, CRR _{7.5}	Cyclic stress ratio for event, CSR _{7.5}	Factor of safety, FS =CRR/CS R	Stress reduction Mw = 4.5	Cyclic stress ratio event, CSR _{4.5}	Factor of safety for 4.5 event, FS for CRR/C SR
0.5	0.0 -0.5	0.5	15.73	5.50	7.87	2.27	75.00	2059.47	127.49	167.94	1.10	0.48	0.14	3.52	1.00	0.03	13.78
3.25	0.5 -3.25	2.75	14.9	35.75	48.84	2.22	40.00	2407.19	40.11	75.71	1.09	0.11	0.13	0.88	0.93	0.10	1.08
5.75	3.25 - 5.75	2.5	16.46	63.25	89.99	2.73	161.67	2770.48	30.53	69.88	1.06	0.11	0.13	0.81	0.85	0.08	1.37
6.25	5.75 - 6.25	0.5	17.06	68.75	98.52	2.73	220.00	3446.49	34.94	65.22	1.06	0.10	0.13	0.78	0.84	0.07	1.54
8	6.25 - 8.00	1.75	16.46	88.00	127.33	2.28	135.71	6080.36	50.56	83.09	1.01	0.12	0.14	0.87	0.78	0.09	1.32
8.5	8.0 - 8.5	0.5	17.62	93.50	136.14	2.10	250.00	11278.05	90.94	150.72	1.01	0.30	0.14	2.16	0.76	0.07	4.16
9.5	8.5- 9.5	1	16.76	104.50	152.90	2.37	150.00	6006.80	42.68	91.56	1.00	0.13	0.14	0.93	0.73	0.09	1.45
10	9.5 - 10.0	0.5	17.32	110.00	161.56	2.00	185.00	12258.75	89.01	128.98	0.99	0.20	0.14	1.42	0.71	0.08	2.55
12	10.0 - 12.0	2	16.54	132.00	194.64	2.43	125.00	5577.76	31.80	63.07	0.97	0.10	0.14	0.75	0.65	0.09	1.19
14.75	12.00 -14.75	2.75	17.51	162.25	242.79	2.28	233.64	10521.14	51.75	86.05	0.95	0.12	0.14	0.90	0.57	0.07	1.69

Table 7 Liquefaction analysis from Standard penetration test result for the specialist hospital site for peak acceleration of 0.16g

Soil layer No.	Depth (m)	Depth range (m)	Layer thickness (m)	SPT 'N' Value (Uncorrected)	Effective overburden pressure, σ_v' (kN/m ²)	Vertical pressure, σ_v (kN/m ²)	SPT N values Corrected for fines (N_1) _{60cs}	Cyclic resistance ratio, CRR	Overburden correction factor K_o	Constant in K_o (C_o)	Stress reduction coefficient r_d	Cyclic stress ratio, CSR for 7.5	Factor of safety, FS = CRR/CSR for 7.5	Cyclic stress ratio, CSR for 4.5	Factor of safety, FS FOR 4.5 = CRR/CSR
1	1	0.40 -1.0	0.6	3,4,5 @ 1.5 m	6.6	10.8	12.84	0.14	1.52	0.10	1.00	0.13	1.04	0.10	1.32
2	3	1.0-3.0	2	2,3,3 @ 3.0 m	28.6	46.8	8.90	0.11	1.35	0.09	0.98	0.16	0.71	0.12	0.90
3	4.5	3.00 – 4.5	1.5	3,3,4 @ 4.5 m	45.1	73.8	10.21	0.12	1.39	0.09	0.97	0.16	0.73	0.12	1.01
4	6	4.5- 9.0	4.5	3,6,7, @ 9.0 m	94.6	154.8	10.76	0.12	1.29	0.09	0.95	0.18	0.69	0.12	1.04
5	9	9.0 - 10.5	1.5	4,5,5 @ 10.5 m	111.1	181.8	8.90	0.11	1.37	0.09	0.91	0.19	0.59	0.11	1.01
6	12	10.5 – 12.0	1.5	4,7,9, @ 12.0 m	138.6	226.8	11.78	0.13	1.36	0.10	0.89	0.20	0.66	0.10	1.31
7	13	12.3 -13.0	1	4,7,9, @ 13.05m	149.6	244.8	11.47	0.13	1.45	0.10	0.85	0.21	0.62	0.09	1.44
8	16	13.70 -16.0	2.3	2,4,7, @ 15.0m	174.9	315.1	6.53	0.10	1.27	0.08	0.81	0.22	0.42	0.09	1.14
9	16.9	16.0 - 16.9	0.9	3,7,8, @ 16.5 m	200.2	331.3	9.73	0.12	1.43	0.09	0.79	0.23	0.50	0.08	1.47
10	19.3	18.3 -19.3	1	3,3,6 @ 18.00 m	208.2	349.3	8.62	0.11	1.40	0.09	0.76	0.25	0.44	0.07	1.49
11	21	19.3 -21.0	1.7	3, 6 6, @ 19.50 m	221.8	379.9	6.24	0.09	1.33	0.08	0.74	0.26	0.36	0.07	1.31
12	22.5	21.0 - 22.5	2.5	3, 8, 13 @ 21.0 m	194.7	427.4	6.87	0.10	1.30	0.08	0.72	0.34	0.28	0.09	1.12

Table 8. Factor of safety, probable settlement values, for soil layers at the different sites investigated

1000 seater auditorium site				Classroom block site				Hostel site				Dominic Etuk			
Depth range (m)	Thickness of soil layer (m)	FS for 4.5 = CRR/C SR	Settlement (cm)	Depth range (m)	Thickness of soil layer (m)	FS for 4.5 = CRR/CSR	Settlement (cm)	Depth range (m)	Thickness of soil layer (m)	FS for 4.5 = CRR/CSR	Settlement (cm)	Depth range (m)	Thickness of soil layer (m)	FS for 4.5 = CRR/CSR	Settlement (cm)
0.0 -2.5	2.50	0.82	8.11	0 - 0.75	0.75	1.29	0.21	0.0 -0.5	0.50	13.78	0.00	0.0 -0.5	0.50	1.22	0.23
2.5 - 3.0	0.50	0.85	1.50	0. 75 -1,5	0.75	1.26	0.00	0.5 -3.25	2.75	1.08	1.60	0.5 - 2.75	2.25	0.55	7.74
3.0 -8.75	5.75	0.73	25.89	1.75 -3.75	2.00	0.81	7.34	3.25 - 5.75	2.50	1.37	0.00	2.75 -4.00	1.25	0.55	4.10
8.75 - 9.50	0.75	1.03	0.77	3.75 - 4	0.25	0.89	0.80	5.75 - 6.25	0.50	1.54	0.00	4.00 - 8.50	4.50	0.60	14.77
9.50 -14.25	4.75	1.03	6.64	4.0 - 4.25	0.25	1.34	0.00	6.25 - 8.00	1.75	1.32	0.58	8.50 - 8.75	0.25	0.79	0.54
14.25 -15.75	1.50	1.29	0.50	4.25 -4.5	0.25	0.76	1.00	8.0 - 8.5	0.50	4.16	0.00	8.75 - 11.00	2.25	0.72	0.86
	15.75	Total	43.41	4.5 - 6.0	1.50	1.22	0.18	8.5 -9.5	1.00	1.45	0.00		11.00	Total	28.24
				6.0 - 6.25	0.25	0.94	0.36	9.5 - 10.0	0.5	2.55	0.00				
				6.25 - 7.5	1.25	1.20	0.14	10.0 - 12.0	2	1.19	6.20				
				7.5 -7.75	0.25	1.17	0.12	12.00 - 14.75	2.75	1.69	0.00				
				7.75 - 8.0	0.25	1.30	0.00		14.75	Total	8.38				
				8.0 - 9.25	1.25	1.16	0.13								
				9.25 -9.5	0.25	1.33	0.00								
				9.5 -10.75	1.25	0.95	2.44								
				10.75 - 11.25	0.50	1.41	0.00								
				11.25 - 11.5	0.25	6.00	0.00								
				11.5 -11.75	0.25	275.09	0.00								
					11.50	Total	12.71								
Thickness of liquefiable layer (m)			8.75		4.0				0.00					10.5	
Liquefaction Potential Index (LPI)			13.23		3.80				0.00					26.65	

Table 8 (Contd) . Factor of safety, probable settlement values, LPI, for soil layers at the different sites investigated

Abak road				Specialist Hospital				Bank Avenue			
Depth range (m)	Thickness of soil layer (m)	FS For 4.5 =CRR/CSR	Settlement (cm)	Depth range (m)	Thickness of soil layer (m)	FS For 4.5 =CRR/CSR	Settlement (cm)	Depth range (m)	Thickness of soil layer (m)	FS For 4.5 =CRR/CSR	Settlement (cm)
0.00 - 0.75	0.75	0.94	0.13	0.0 - 0.75	0.75	1.52	0	0.0 -0.5	0.5	1.46	0.00
0.75 - 1.25	0.5	0.82	1.98	0.75 - 1.0	0.25	0.92	0.69	0.5- 0.7	0.2	1.08	0.12
1.25 - 2.5	1.25	0.74	4.67	1.0 - 1.25	0.25	1.09	0.30	0.70 -1.50	0.8	0.93	0.36
2.05 - 4.50	2	0.69	8.52	1.25 -2.25	1	0.83	3.30	1.5 -2.10	0.6	0.80	2.44
4.50 - 10.5	6	0.72	31.56	2.25 - 2.5	0.25	0.77	0.90	2.10 -2.20	0.1	0.79	0.43
10.50 - 11.00	0.5	0.85	1.97	2.5 -2.75	0.25	1.02	0.31	2.20 -2.30	0.1	0.70	0.45
11.00 - 11.75	0.75	0.90	2.80	2.75 -7.25	4.5	0.84	15.27	2.30 -3.20	0.9	0.92	3.22
11.75 - 12.50	0.75	0.89	3.20	7.25 - 10.25	3	0.69	19.00	3.20 - 4.30	1.1	0.69	7.94
12.50 - 14.75	2.25	1.02	3.66	10.25 - 10.75	0.5	0.83	2.10	4.30 - 4.60	0.3	0.67	1.19
	14.75	<i>Total</i>	58.48	10.75 -12.0	1.25	0.87	5.34	4.60 - 5.50	0.9	0.68	3.86
				12.0 - 13.0	1	0.88	6.19	5.50 -6.50	1	0.74	4.03
				13.0 - 16.0	3	1.08	3.01	6.50 -6.70	0.2	0.64	0.86
				16.0 - 17.0	1	1.46	0.00	6.70 - 8.20	1.5	0.79	6.02
				17.0 - 17.75	0.75	1.23	0.42	8.20 - 8.60	0.4	0.84	1.52
				17.75 - 19.75	2	1.98	0.68	8.60 - 8.70	0.1	0.82	0.39
				19.75 -20.0	0.25	1.27	0.14	8.70 -9.0	0.3	0.85	1.13
					20.00	<i>Total</i>	57.65	9.00 - 9.60	0.6	0.84	2.40
								9.60 - 10.2	0.6	0.77	2.43
								10.20 - 10.70	0.5	0.89	1.97
								10.70 -11.00	0.3	0.85	1.27
								11.0 - 12.30	1.3	0.91	5.62
								12.30-12.60	0.3	0.92	1.31
								12.60 - 12.80	0.2	0.96	0.35
								12.80 -13.10	0.3	0.88	1.23
								13.10 -13.70	0.6	1.01	1.03
								13.70 -14.10	0.4	1.17	0.22
								14.10 -14.30	0.20	0.82	0.68
								14.30 -14.50	0.2	0.99	0.20
								14.50 -15.20	0.7	0.90	2.03
								15.20 - 15.80	0.6	1.28	0.20
								15.80 - 16.90	1.1	1.28	0.45
								16.90 - 20.00	3.1	1.41	0.00
									20.00	<i>Total</i>	55.34
Thickness of liquefiable layer (m)			12.50				12.20				13.30
Liquefaction Potential Index (LPI)			17.86				12.50				15.51

Table 9. Level of liquefaction severity

LPI	Iwasaki et al (1998)	Luna and Frost(1998)	MERM (2003)
LPI = 0	Very low	Little to none	None
0 < LPI < 5	Low	Minor	Low
5 < LPI < 15	High	Moderate	Medium
15 < LPI	Very high	Major	High

Figures

Figure 1

Grain size analysis for soil sample from SPT borings and Lithologic borehole

Figure 2

Map of Uyo metropolis showing test locations and zones with their Liquefaction Potential Index (LPI) values

Figure 3

Correlation of liquefiable soil layers from three sites.

Figure 4

Log pattern of the normalized cone resistance and factor of safety values with depth for the hostel block site for 4.5 magnitude event.