# MODELLING OF CYCLIC SHEAR MODULUS AND FACTOR OF SAFETY IN CLAY SOIL

**Mohammed Ganiyu Oluwaseun[1] and Charles Kennedy[2]**

[1]Department of Civil and Environmental Engineering
University of Port Harcourt

Email: ahmedgo2001@gmail.com

[2]Civil Engineering Department, School of Engineering,
Kenule Beeson Saro-Wiwa Polytechnic,
P.M.B. 20, Bori, Rivers State, Nigeria.

Email: ken_charl@yahoo.co.uk

**Cite this article:**

Mohammed Ganiyu Oluwaseun, Charles Kennedy (2024), Modelling of Cyclic Shear Modulus and Factor of Safety in Clay Soil. International Journal of Mechanical and Civil Engineering 7(1), 43-56. DOI: 10.52589/IJMCE-4QKHY8JF

**Manuscript History**

Received: 3 Nov 2023

Accepted: 15 Dec 2023

Published: 29 Jan 2024



**ABSTRACT**: *The liquefaction potential of Niger Delta soil was studied through formulated models based on cyclic shear modulus and factor of safety (FS). Data from the experiment were fitted into models to predict the cyclic shear modulus and Factor of Safety. The test analysis shows effective prediction of cyclic shear modulus for a given number of cycles (1–40) and cyclic shear strain (0.01–5 %). Comparison of results shows no significant differences between the measured and predicted cyclic shear modulus, especially from 0.1% shear strain and above. Similarly, the values of factor of safety predicted by the model were very close to those obtained from the experiment; the predicted FS obtained at depths close to 30 m across the sites were slightly greater than 1.0, as against the observed results. Despite this slight variation, the FS model still shows a high degree of prediction. Therefore, the formulated models can be utilised in the study of liquefaction potential, especially in the Niger Delta region.*

**KEYWORDS:** Liquefaction, Soil, Factor of Safety, Earthquake.

## INTRODUCTION

When saturated granular soils are subjected to cyclic seismic loading, liquefaction occurs, resulting in a temporary loss of shear strength and stiffness. According to Youd et al. (2001), this can cause serious soil damage such as loss of bearing capacity, lateral spreading, boiling of sand and excessive foundation settlement. You can destroy the infrastructure. The most liquefiable materials are non-cohesive sands, but clays can also soften under significant shear stress and acquire high pore pressures during cyclic mobility. Therefore, a realistic assessment of liquefaction requires an accurate assessment of the cyclic shear strength and deformation properties of the clay.

The factor of safety (FS) against liquefaction failure and cyclic shear modulus (G) are the two most important factors affecting strength and cyclic shear response. Under dynamic loading, the shear modulus pairs stress and shear strain and decreases non-linearly with the shear strain amplitude. An equivalent linear analysis of the seismic response requires a robust mathematical model of the shear modulus degradation curve (Kramer, 1996). FS measures the resistance of soil to liquefaction with respect to cyclic shear loads caused by earthquakes. In practice, simplified empirical methods are often used to calculate FS and liquefaction potential. However, due to its drawbacks, more accurate analytical material models need to be developed.

This background provides a context for research attempting to simulate the complex relationships between composition, states, stress conditions, and dynamic loading that affect the cyclic shear modulus and liquefaction resistance of clays. Simplified correlations still serve as a basis, but combining them with new analytical techniques can improve the accuracy of seismic site response and liquefaction risk analysis.

A number of empirical, semi-empirical, and analytical models have been presented to calculate the cyclic shear modulus degradation in clays. Early models (Seed & Idriss, 1970) related the maximum shear modulus Gmax to index properties such as plasticity index (PI) and effective limiting pressure ′c. The normalized G/Gmax modulus and shear strain amplitude were then correlated using a laboratory test database (Vucetic & Dobry, 1991).

Recent studies have indirectly modeled the complex relationships between G regulatory elements using methods such as neuro-fuzzy systems (Darendeli, 2001) and genetic programming (Gamoussi et al., 2020). Some scholars have advocated fundamentally derived analytical frameworks based on soil mechanics principles, despite the potential of such data-driven methods. To model the non-linear hysteretic stress-strain behavior of soil under cyclic loading, Lambrakos (1985) used fracture analysis. However, it is still difficult to validate the theoretical model using experimental data obtained in different environments.

In general, analytical materials models have been developed to support a more robust integrated approach to predict shear modulus degradation in clays under cyclic seismic loading and to complement the benefits of simplified empirical methods.

An important aspect of seismic resistivity assessment is the assessment of soil resistance to liquefaction. The number of hits in a standardized penetration test (SPT) was used in an initial simplified approach by Seed and Idriss (1971) to estimate the cyclic resistance ratio (CRR) using empirical correlations. However, this approach is limited by its reliance on location-specific variables. Consequently, many analytical and semi-empirical modeling methods have


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

been proposed. Using the hollow cylinder test, Juang et al. (2013) developed a physics-based correlation between CRR, relative density, and effective confinement pressure. Numerical simulations, calibrated by laboratory experiments, were also performed to analytically simulate the cyclic stress ratios that induce initial fluidization (Ku et al., 2010). Models for liquefaction resistance and cumulative strain under cyclic loading have been developed using energy-based techniques such as potential theory of shear strain (Drucker, 1975; Liang, 1995).

Although these constitutive modeling techniques help overcome the lack of site-specific empirical elements, some researchers suggest enhancing rather than completely replacing simple methods that remain effective in practice (Youd & Idriss, 2001). This underscores the need to integrate the benefits of optimized correlations and develop analytical models for accurate estimation of liquefaction.

The references given cover various methods, e.g., B. Field tests for site characterization (Jawaid, 2010; Nwankwoala & Oborie, 2014), laboratory element tests to assess cyclic response (Arion & Neagu, 2012; Tsai et al., 2010), physical modeling through rocking table and centrifuge studies (Marques et al., 2013; Marasini & O. Kamura, 2012) and numerical simulations of Youd et al. (2001) and Liang (1995) developed empirical, semi-empirical and analytic materials models for liquefaction estimation as well as for numerical simulation of dynamic soil-foundation interactions (Bertalot et al., 2013; Shahir & Pak, 2010).

Despite the limitations associated with site-specific factors, in practice, simplified empirical correlations based on typical penetration test data are often used to estimate liquefaction potential on a routine basis (Youd et al., 2001).

$$FS = \frac{CRR}{CSR} \tag{1}$$

According to Juang et al. (2013), laboratory element testing provides useful information for building a semi-empirical model that relates cyclic resistance to density, limiting pressure, and other contributing parameters.

Physical modeling reproduces important field-scale mechanisms, such as B. subsidence of bedrock after liquefaction and increase in pore pressure (Dashti et al., 2010). However, there are concerns about the scaling effect. Complementing the assessment of the physical model, numerical simulation can simulate complex soil-foundation interactions under dynamic loads at the prototype field scale (Asgari et al., 2014). However, a proper model of land constitution is needed.

According to Ku et al. (2010), a basic constitutive modeling approach has the potential to overcome the shortcomings of semi-empirical techniques. However, extensive calibration and validation is required prior to reliable use.


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

Integrating the flexibility of the evolving analytical model with the robustness of an optimized empirical approach can result in a more accurate seismic analysis and liquefaction assessment (Youd & Idriss, 2001).

In general, it is recommended to use hybrid analytical-empirical modeling methods to account for the complex interactions between composition, conditions, stress conditions and dynamic loading effects on clays when predicting cyclic shear strength parameters such as shear modulus and liquefaction resistance. Simplified correlations provide a useful starting point, but as laboratory and field data accumulate and numerical simulation tools improve, so do constitutive modeling capabilities.

For the purpose of generating relevant data inputs for design, construction and averting earthquake disasters, it is essential that factors that affect such disasters be studied and understood. Therefore, the liquefaction potential of Niger Delta soil is studied by mathematical models through the cyclic shear modulus and factor of safety.

## METHODOLOGY

The models were formulated with the aid of experiments conducted at various locations across the Niger Delta region. The mathematical models were formulated to predict the cyclic shear modulus ($G_s$) and liquefaction potential of soil, which could be used to study the occurrence of earthquakes in the region.

### Cyclic Shear Modulus

In this study, the output response of cyclic shear modulus ($G_s$) was formulated as a function of shear strain ($\varepsilon_s$) and number of cycles ($N_c$). This model was formulated based on observed field data, which indicated the variation of $G_s$ as shear strain and the number of cycles were changing in values. Thus, the cyclic shear modulus ($G_s$) is represented by the mathematical expression given as follows:

$$G_s = \beta \frac{N_c^{\ x}}{\varepsilon^y} \tag{2}$$

The variable $\beta$ is the constant of the model, while *x* and *y* are power indices relating to the number of cycles and cyclic shear strain. To enable the determination of the constants $\beta$, *x* and *y*, Equation (2) is linearised by taking the natural logarithm of each term. This step leads to Equation (3).

$$\log G_s = \log \beta + x \log N_c - y \log \varepsilon \tag{3}$$


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

Using the method of multiple linear regression, and utilising experimental results, the constant parameters can now be determined. The evaluated parameters are then substituted into the Equation (2), which can be used to predict the cyclic shear modulus ( $G_s$ ) for every shear strain ( $\varepsilon_s$ ) or number of cycles ( $N_c$ ).

## Liquefaction Potential

The possibility of liquefaction occurrence in Niger Delta soil as experimentally investigated is further studied to establish a mathematical relationship that can be utilised to predict the potential of liquefaction in the region. Thus, in this stud, the factor of safety (FS), as one of the key parameters used in predicting the possibility of liquefaction occurrence, was expressed as a function of SPT-N (*N*), percentage of fines (*f*), soil depth (*d*) and effective vertical stress ( $\sigma$ ). This is mathematically expressed as:

$$FS = K_s \frac{N^a f^b d^c}{\sigma^e} \tag{4}$$

The constant coefficients *a*, *b* *c* and *e* are power indices relating to *N*, *f*, *d* and $\sigma$ respectively. Like in cyclic shear modulus, the constants *a*, *b*, *c* and *e* are determined by taking the natural logarithm of each term in Equation (4). This step leads to Equation (5).

$$\log FS = \log K_s + a \log N + b \log f + c \log d - e \log \sigma \tag{5}$$

After the parameter evaluation, they can be substituted into the Equation (4) to predict the FS for every change in SPT-N (*N*), percentage of fines (*f*), soil depth (*d*) or effective vertical stress ( $\sigma$ ).

## RESULTS AND DISCUSSION

Table 1 shows the experimental results for the formulation of the factor of safety cyclic shear modulus model, while Table 2 is the experimental results for the formulation of the factor of safety model. Tables 3 and 4 are the predicted values of cyclic shear modulus and factor of safety, respectively, as compared with the experimental results.

In this work, the cyclic degradation of the soil shear modulus at different shear stress amplitudes is investigated. The shear modulus is a measure of the soil's resistance to shear deformation and its shear stiffness. It can be observed that the shear modulus gradually deteriorates with increasing shear stress under repeated cyclic loading, for example, during earthquakes (Sawicki & Mierczynski, 2009). Previous studies have linked the magnitude of the induced soil shear stress with the degradation of the shear modulus (Arion & Neagu, 2012). The modulus remains essentially constant at a shear strain below 0.001%. The modulus decreases non-linearly to a residual value when the strain increases above the elastic threshold (Ishihara, 1993). This stress-dependent decline can be expressed as follows (Marasini & Okamura, 2014):


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

Gmax is the highest possible shear modulus, r is the reference shear strain at which the modulus drops to 0.5. Gmax is the shear strain and an is the curve fitting parameter. Soil plasticity affects the reference strain r and the degradation rate a.

Cyclic shear modulus data for sandy silty soils at five shear strain levels (0.01% to 5%) are presented in Table 1. After 40 loading cycles, the data show the expected degradation trend, with a decrease in G from 12-14 MPa at 0.01% elongation to 6-7 MPa at 5% elongation. The cumulative damage from repeated loading is evidenced by a progressive decrease in G with more cycles at each loading level.

The plot of shear versus strain modulus is similar to that reported by Khan et al. (2016) and Lu (2017), acceptable for muddy sands. This information allows calibration of cyclic degradation models, which is important for assessing the potential for settlement and liquefaction due to loss of soil stiffness and strength caused by earthquakes (Bertalot et al., 2013). It is recommended to carry out additional laboratory tests with a larger parameter range to fully determine the dynamic performance. The results provide useful input for numerical simulation of the behavior of soil elements under seismic loa

**Table 1: Data for Formulation of Cyclic Shear Modulus Model**

| | **0.01% Strain** | **0.1% Strain** | **1% Strain** | **2.5% Strain** | **5% Strain** |
|---|---|---|---|---|---|
| **No of cycle** | **Shear modulus $G_s$ (Mpa)** | | | | |
| 1 | 12.36 | 11.72 | 9.42 | 8.58 | 7.04 |
| 2 | 13.47 | 13.517 | 9.37 | 8.4 | 6.86 |
| 3 | 13.22 | 13.03 | 9.33 | 8.31 | 6.79 |
| 4 | 14.36 | 12.975 | 9.28 | 8.25 | 6.72 |
| 5 | 14.11 | 12.429 | 9.24 | 8.21 | 6.73 |
| 6 | 13.84 | 12.351 | 9.23 | 8.19 | 6.68 |
| 7 | 13.62 | 12.308 | 9.19 | 8.17 | 6.67 |
| 8 | 13.43 | 12.152 | 9.18 | 8.15 | 6.64 |
| 9 | 13.15 | 12.069 | 9.16 | 8.13 | 6.63 |
| 10 | 12.34 | 12.114 | 9.14 | 8.12 | 6.59 |
| 11 | 13.25 | 12.07 | 9.14 | 8.11 | 6.59 |
| 12 | 13.4 | 12.093 | 9.12 | 8.1 | 6.56 |
| 13 | 13.29 | 12.106 | 9.12 | 8.08 | 6.55 |
| 14 | 13.33 | 12.095 | 9.1 | 8.07 | 6.55 |
| 15 | 13.32 | 12.119 | 9.1 | 8.06 | 6.54 |
| 16 | 13.24 | 12.123 | 9.08 | 8.06 | 6.53 |
| 17 | 13.19 | 12.028 | 9.08 | 8.05 | 6.51 |
| 18 | 13.25 | 12.061 | 9.07 | 8.04 | 6.51 |
| 19 | 13.29 | 12.138 | 9.06 | 8.03 | 6.51 |
| 20 | 13.18 | 12.147 | 9.05 | 8.03 | 6.5 |
| 21 | 13.08 | 12.143 | 9.05 | 8.02 | 6.49 |
| 22 | 13.14 | 12.04 | 9.05 | 8.02 | 6.49 |
| 23 | 13.13 | 12.036 | 9.03 | 8.01 | 6.49 |


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

| | | | | | |
|---|---|---|---|---|---|
| 24 | 13.06 | 11.991 | 9.03 | 8.01 | 6.48 |
| 25 | 13.02 | 12.021 | 9.02 | 8 | 6.47 |
| 26 | 13.01 | 12.035 | 9.01 | 8 | 6.47 |
| 27 | 13.03 | 12.045 | 9.01 | 8 | 6.46 |
| 28 | 12.99 | 12.061 | 9.02 | 7.99 | 6.46 |
| 29 | 12.95 | 11.946 | 9 | 7.99 | 6.45 |
| 30 | 12.88 | 11.967 | 9 | 7.98 | 6.45 |
| 31 | 12.88 | 12.006 | 9 | 7.98 | 6.44 |
| 32 | 12.89 | 12.048 | 8.99 | 7.97 | 6.44 |
| 33 | 12.78 | 11.948 | 8.99 | 7.97 | 6.44 |
| 34 | 12.85 | 12.007 | 8.99 | 7.97 | 6.43 |
| 35 | 12.79 | 12.025 | 8.98 | 7.96 | 6.43 |
| 36 | 12.8 | 12.002 | 8.99 | 7.96 | 6.42 |
| 37 | 12.74 | 11.949 | 8.98 | 7.96 | 6.42 |
| 38 | 12.78 | 11.981 | 8.98 | 7.96 | 6.42 |
| 39 | 12.76 | 11.982 | 8.97 | 7.95 | 6.41 |
| 40 | 12.68 | 12.018 | 8.96 | 7.95 | 6.41 |

The following is a detailed explanation of the Safety Factor (FS) data listed in Table 2:

Based on standard penetration testing (SPT) data, this study investigates how the safety factor against liquefaction varies with depth. The ratio of the soil's ability to resist liquefaction to seismic demands is called the factor of safety. According to Youd et al. (2001), an FS value above 1 indicates sufficient soil strength, while a value below 1 indicates the possibility of liquefaction initiation.

According to previous research (Bray & Dashti, 2010), FS is affected by a number of variables including overburden stress, specific gravity, fine fraction and seismic loading. Empirical correlations were made to estimate FS from the number of SPT blows (N) and effective overvoltage (′v), according to (Olson). According to Olson and Stark (2002), the empirical dependencies are determined to estimate the FS from the number of SPT strokes (N) and the effective overload voltage (′v) as follows:

$$FS = (N/Nliq)^{m} \times K\sigma \times K\alpha$$

Nliq is the number of critical liquefaction shocks, K is the stress dependent correction, K is the age/cement correction, and m is the exponent. This shows the importance of SPT-N in determining liquefaction resistance.

According to the results in Table 2, the FS decreased from 5–8 at shallow depths to 0.5–1 at 30 m depth. This reduction is consistent with the results reported by Pham et al. observed trends in stress depth. 2021. With depth, FS decreases with lower N and higher ′v. However, FS is a variable, with some deep intervals showing local improvement. According to Noor et al. (2019), this may be due to variations in relative density, with denser layers offering better stability.


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

In general, the FS profile calculated from the SPT data provides an early indication of liquefaction potential in various seams. For further investigation using shear wave velocity data and laboratory testing, FS values below 2–3 can be liquefied at depths of more than 10 m (Idriss & Boulanger, 2008). With the help of an integrated database, liquefaction models can be calibrated for conditions in the field. Assessing the stability of a structure under seismic loads requires the use of this knowledge**.**

**Table 2: Data for Formulation of Factor of Safety Model**

| ***d*** **(m)** | ***N*** **(blows)** | ***f*** **(%)** | $\sigma$ **(kPa)** | **Factor of Safety** |
|---|---|---|---|---|
| 0.85 | 4 | 68 | 0.231 | 5.276 |
| 2.3 | 6 | 63 | 0.226 | 4.704 |
| 3.8 | 6 | 67 | 0.307 | 4.778 |
| 5.3 | 9 | 59 | 0.245 | 4.449 |
| 5.65 | 3 | 83 | 0.172 | 7.928 |
| 7.15 | 6 | 70 | 0.314 | 6.705 |
| 7.8 | 3 | 79 | 0.212 | 7.429 |
| 9.3 | 10 | 54 | 0.293 | 3.846 |
| 10.8 | 16 | 36 | 0.311 | 2.651 |
| 12.3 | 13 | 39 | 0.319 | 2.959 |
| 13.8 | 19 | 28 | 0.304 | 2.615 |
| 15.3 | 22 | 21 | 0.315 | 2.263 |
| 16.8 | 25 | 15 | 0.309 | 2.067 |
| 17.75 | 18 | 27 | 0.248 | 2.576 |
| 19.25 | 25 | 14 | 0.321 | 1.953 |
| 20.6 | 20 | 22 | 0.301 | 2.298 |
| 22.1 | 37 | 3 | 0.322 | 0.907 |
| 23.6 | 33 | 6 | 0.321 | 1.014 |
| 25.1 | 30 | 9 | 0.313 | 1.198 |
| 28.95 | 33 | 7 | 0.234 | 1.124 |
| 27.45 | 38 | 2 | 0.312 | 0.832 |
| 28.1 | 39 | 1 | 0.205 | 0.757 |
| 29.6 | 41 | 0.7 | 0.324 | 0.506 |
| 30 | 36 | 4 | 0.275 | 0.913 |


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

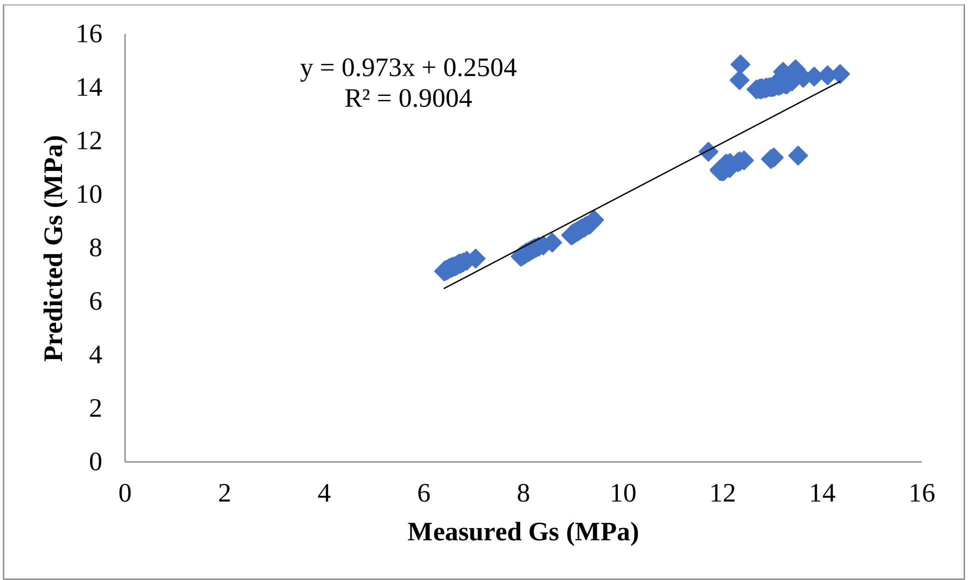


**Figure 1: Measured and predicted cyclic shear modulus**

Figure 1 shows the correlation between the predicted and measured cyclic shear modulus of sandy soil subjected to 400 kPa shear stress. The data obtained from one of the selected sites was used for the analysis, representing the other sites. The $R^2$ was obtained as 0.9004, which implies that 90.04% of the measured cyclic shear modulus of the soil has been explained by the model, indicating that the model can be applied to predict the cyclic shear modulus for a given number of cycles and cyclic shear strain. Similarly, from the regression analysis, the constant coefficients expressed in model Equation (2) were determined as $\beta =$ 9.05224, $x =$ -0.01768 and $y =$ 0.10776. Hence, the predictive model for cyclic shear modulus is expressed as $G_s = 9.05224 \frac{N_C^{-0.01768}}{\varepsilon_s^{0.10776}}$. Thus, the utilization of the model shows appreciable prediction of cyclic shear modulus at the corresponding measured number of cycles for a given cyclic shear strain. Meanwhile, the predictability of the model showed some weakness at very low percentage strain. As a result, comparison of predicted values with experimental data as presented in Table 3, shows that at 0.01% shear strain, the predicted values were higher than those obtained from the experiment at all cycles, but from 0.1 to 5% strain, the differences between the measured and predicted cyclic shear modulus were not much. From the results (Table 3), the cyclic shear modulus predicted by the model between 1 to 40 cycles, increased from 23.71 to 29.19 MPa at 0.01 % shear strain as against 24.40 to 25.76 MPa obtained from the experiment; 13.64 to 16.79 MPa as against 16.46 to 18.2 MPa for experiment at 0.1 % shear strain; 7.85 to 9.66 MPa as against 8.38 to 10.024 MPa for experiment at 1.0 % shear strain; 6.30 to 7.75 MPa as against 5.61 to 7.61 MPa for experiment at 2.5 % shear strain; and 5.33 to 6.56 MPa as against 4.40 to 6.66 MPa for experiment at 5 % shear strain.

Table 1 demonstrates that the cyclic shear modulus decreased with increasing percentage of shear strain, and at 0.01 % strain, the values of cyclic shear modulus were about four (4) times more than those obtained at 5 % strain for every corresponding number of cycles. The effective decrease in cyclic shear modulus at reduced percentage strain has also been reported by previous authors, which was attributed to shear deformation characteristics of soil (Arion &


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

Neagu, 2007, 2012), while increase in strain decreased the factor of safety, and hence, the potential for soil liquefaction (Tsai *et al*., 2010; Sadek *et al*., 2014).

However, the relationship between cyclic shear modulus of soil, number cycle and shear strain, as indicated in this study, has also been outlined in a study by Narepalem and Godavarthi (2019). Other studies have stated the importance of soil evaluation via mathematical models in the design of engineering structure to reduce the impact of liquefaction (Erzin & Tuskan, 2019; Geyin et al., 2020; Pham, 2021; Subedi & Acharya, 2022).

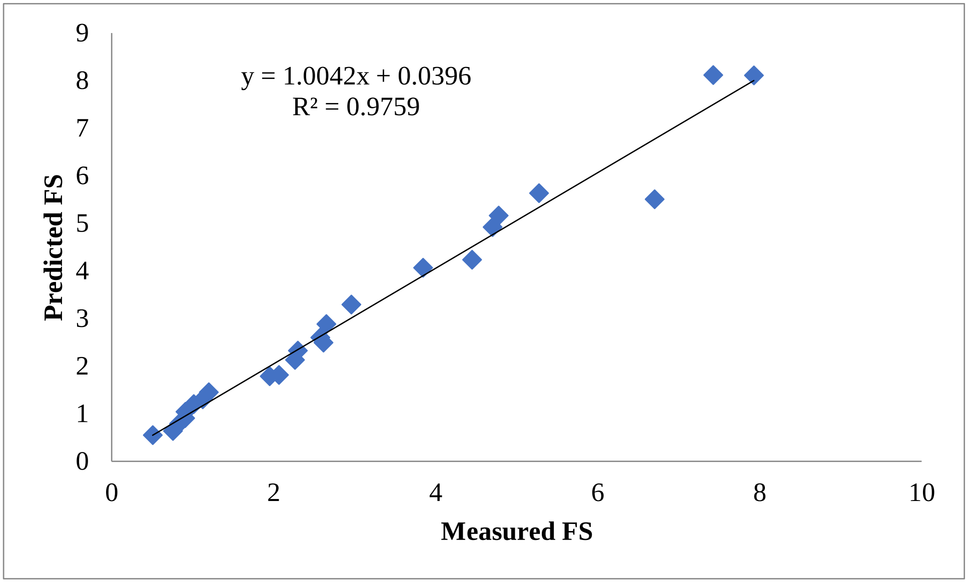


**Figure 2: Measured and predicted factor of safety**

Figure 2 shows the profiles of the measured factor of safety (FS) for Akwa-Ibom, Bayelsa, Delta and Rivers States. Again, experimental data obtained from one of the sites was selected and used to determine the constant coefficients contained in the factor of safety model using the multiple regression analysis. From the data analysis, the power indices relating to the variables in Equation (4) were determined as $a = 0.1357$, $b = 0.3516$, $c = -0.3215$ and $d$ = 0.3899. However, a correction factor of 0.5 was inserted into the model to reduce over estimation of factor of safety. Therefore, the predictive model for factor of safety is expressed as $FS = 0.5\dfrac{N^{0.1357} f^{0.3516} d^{0.3899}}{\sigma^{0.3215}}$ with a standard error of 0.0664.


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

To ascertain the predictability of the model, it was tested with the mean experimental data obtained for each State, as shown in Figures 3. From the profiles, it can be deduced that the factor of safety predicted by the model behaves similar to those of the experiments shown in Figure 3, but the high fluctuations associated with the measured values were reduced for the predicted counterparts across the States. From the predicted FS values (Table 4), it can be said that the occurrence of liquefaction that would cause disaster in the Niger Delta is low, as all the FS predicted were higher than 1. However, from the experiment, the FS at some soil depth shows possibility of liquefaction occurrence, as they are below 1.0. According to studies, a factor of safety less than 1 implied that liquefaction may occur (Karim et al., 2010). Also, the use of models to study liquefaction potential has been reported using the factor of safety as a determining parameter (Jawaid, 2010; Khan et al., 2016). In Karim et al. (2010), the factor of safety was modelled as a function of depth, SPT values, cyclic stress and fine content. Diez et al. (2019) used numerical techniques to estimate the impact of pore pressure on factors of safety against soil liquefaction.

Several authors in recent times have equally used mathematical models in the analysis or prediction of factor of safety against soil liquefaction using various input variables such as earthquake magnitude, peak ground acceleration, standard penetration test, saturated unit weight, fines content, depth of ground water level or soil depth, as functional parameters (Erzin & Tuskan, 2019; Geyin et al., 2020; Pham, 2021; Subasi et al., 2021; Subedi & Acharya, 2022). These studies showed the efficacy of mathematical modeling as a powerful tool for rapid and accurate prediction of factor of safety against liquefaction (Erzin and Tuskan 2019: Pham, 2021; Subasi et al., &; Katona & Karsa, 2022; Subedi & Acharya, 2022). Hence, the model is imperative in the preliminary stage of design for the factor of safety against liquefaction.

## CONCLUSION

Liquefaction is an important variable in the assessment of soil. The analysis of liquefaction or earthquake is undoubtedly expensive, but through development of an appropriate model, it can reduce the rigorous tasks and costs involved in liquefaction analysis. From the experimental results obtained from the sites, models were formulated to predict the cyclic shear modulus and Factor of Safety as a function of the soil characteristic variables. For cyclic shear modulus, the model was formulated as a function of the number of cycles and shear strain, while Factor of Safety was dependent on SPT-N, fines, soil depth and effective vertical stress. From the analysis, it was shown that the models were able to predict the measured cyclic shear modulus and Factor of Safety for the given dependent variables of the respective models. Therefore, based on the level of prediction, the formulated models can be utilised in the study of liquefaction potential, especially in the Niger Delta region.


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

**REFERENCES**

Adamidis, O. and Madabhushi, G.S.P. (2015). Deformation mechanisms under shallow foundations during earthquake-induced liquefaction, Proc. 6th International Conf. on Earthquake Geotechnical Engng, Christchurch, New Zealand, 1–4 November 2015.

Ahmad, S., Khan, M.Z., Anwar, A. and Husain, S.M. (2015). Determination of Liquefaction Potential By Sub-Surface Exploration Using Standard Penetration Test, *International Journal of Innovative Science, Engineering & Technology*, 2(10), 751-760.

Almani, Z. A., Ansari, K. and Memon, A. A. (2012). Mechanism of liquefaction-induced large settlements of buildings, *Mehran University Research Journal of Engineering and Technology*, 31, 635–650.

Amadi, A.N; Eze, C.J; Igwe, C.O; Okunlola, I.A and Okoye, N.O (2012). Architect's and geologist's view on the causes of building failures in Nigeria. *Modern Applied Science*, Vol.6 (6): 31 – 38.

Arion, C. and Neagu, C. (2012). Dynamic laboratory investigation for soil seismic response, 15$^{th}$ *World Conference on Earthquake Engineering*, Lisbon, Portugal, 24-28$^{th}$ September, 2012.

Asgari, A., Golshani, A. and Bagheri, M. (2014). Numerical evaluation of seismic response of shallow foundation on loose silt and silty sand, *Journal of Earth System Science*, 123(2), 365–379.

Bertalot, D., Brennan, A. J. and Villalobos, F. A. (2013). Influence of bearing pressure on liquefaction-induced settlement of shallow foundations, *Geotechniqu*e, 63(5), 391–399.

Dashti, S. and Bray, J.D. (2013). Numerical simulation of building response on liquefiable sand, *Journal of Geotechnical and Geoenvironmental Engineering*, 139(8), 1235–1249.

Dashti, S., Bray, J. D., Pestana, J. M., Riemer, M. and Wilson, D. (2010). Mechanisms of seismically induced settlement of buildings with shallow foundations on liquefiable soil, *Journal of Geotechnical and Geoenvironmental Engineering*, 136(1), 151–164.

Diez, J.L.G., Galindo, J.G. and Pena, A.S. (2019) Adjustment of a numerical model for pore pressure generation during an earthquake, *PLoS ONE*, 14(9), 1-24.

Erzin, Y. and Tuskan, Y. (2019). The use of neural networks for predicting the factor of safety of soil against liquefaction, *Scientia Iranica A*, 26(5), 2615-2623.

Geyin, M., Baird, A.J. and Maurer, B.W. (2020). Field assessment of liquefaction prediction models based on geotechnical versus geospatial data, with lessons for each, *Earthquake Spectra*, 36(3), 1386–1411.

Ishihara, K. (1993). Liquefaction and flow failure during earthquakes, *Geotechnique*, 43 (3), 351-415.

Jawaid, S.M.A. (2010). Comparison of Liquefaction Potential Evaluation Based on Different Field Tests, *Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, San Diego, California, U.S.A., 24–29$^{th}$ May, 2010.

Karim, H.H., Fattah, M.Y. and Hasan, A.M. (2010). Evaluation of Some Geotechnical Properties and Liquefaction Potential from Seismic Parameters, *Iraqi Journal of Civil Engineering*, 6 (3), 30-45.

Katona, T.J. and Karsa, Z. (2022). Probabilistic safety analysis of the liquefaction hazard for a nuclear power plant, *Geosciences*, 12, 192 - 106.


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

Khan, M.A. Khan, M.Z. and Khan, M.B. (2016). Effect of Fines on Liquefaction Resistance in Fine Sand and Silty Sand, *International Journal of Engineering Research and Applications*, 6(1), 102-106.

Lu, C.W. (2017). A Simplified Calculation Method for Liquefaction-Induced Settlement of Shallow Foundation, *Journal of Earthquake Engineering*, 21(8), 1385-1405.

Marasini, N. and Okamura, M. (2014) Numerical simulation of centrifuge tests on seismic behavior of residential building on liquefiable foundation soil, *Computer Methods and Recent Advances in Geomechanics*, 148, 847–852.

Marques, A.S., Coelho, P.A.L.F., Cilingir, U., Haigh, S. K. and Madabhushi, S. P. G. (2013). Centrifuge modelling of liquefaction-induced effects on shallow foundations with different bearing pressures, Proceeding of 2nd Eurofuge Conference on Physical Modelling in Geotechnics, Delft.

Narepalem, S. and Godavarthi, V.R.S.R. (2019). Constitutive behaviour of fine grained soils of Vijayawada region, *KSCE Journal of Civil Engineering*, 23(6), 2463-2470.

Noor, S.T., Islam, S. and Reza, S.M.S (2019). SPT Based Soil Liquefaction Susceptibility Assessment: A Review, *Journal of Environmental Treatment Techniques*, 7(3), 295-299.

Nwankwoala, H.O. and Oborie, E. (2014). Geo-technical Investigation and Characterization of Sub-soils in Yenagoa, Bayelsa State, Central Niger Delta, Nigeria, Civil and Environmental Research, 6(7), 75-83.

Pham, T.A. (2021). Application of feedforward neural network and spt results in the estimation of seismic soil liquefaction triggering, *Computational Intelligence and Neuroscience*, Article ID 1058825, 1-14. Accessed from: https://doi.org/10.1155/2021/1058825 19th May, 2022.

Sadek, S., Najjar, S. Mostapha, Y. and Mostapha, M. (2014). Assessment of the Potential for Earthquake-Induced Liquefaction in Granular Soils, *Spreadsheets in Education*, 7(2), 1-30.

Sancio, R. B., Bray J. D., Durgunoglu, T. and Onalp, A. (2004). Performance of Buildings Over Liquefiable Ground in Adapazari, Turkey, Proc. of 13th World Conference on Earthquake Engineering, No. 935.

Sawicki, A. and Mierczynski, J. (2009). On the behavior of liquefied soil, *Computers and Geotechnics*, 36, 531–536.

Shahir, H. and Pak, A. (2010). Estimating liquefaction-induced settlement of shallow foundations by numerical approach, *Computers and Geotechnics*, 37(3), 267–279.

Subasi, O., Koltuk, S., Akbas, M. and Iyisan, R. (2021). A numerical study on liquefaction induced settlements by using PM4Sand model, *IOP Conf. Series: Materials Science and Engineering*, 1203, 032029. Accessed from https://www.doi:10.1088/1757-899X/1203/3/032029 19th May, 2022.

Subedi, M. and Acharya, I.P. (2022). Liquefaction hazard assessment and ground failure probability analysis in the Kathmandu Valley of Nepal, *Geoenvironmental Disasters*, 9(1), 1-17.

Tsai, P.H., Lee, D.H., Kung, G.T.C. and Hsu, C.H. (2010). Effect of Content and Plasticity of Fines on Liquefaction Behaviour of Soils, *Quarterly Journal of Engineering Geology and Hydrogeology*, 43, 95-106.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B. and Stokoe, K.H. (2001). Liquefaction Resistance of Soils: Summary Report from


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF

the 1996 NCEER and 1998 NCEER/NSF Workshops Evaluation of Liquefaction Resistance of Soils, *Journal of Geotechnical and Geoenvironmental Engineering*, 127(10), 817-833.

Youdeowei, P.O and Nwankwoala, H.O (2013). Suitability of soils as Bearing Media at a Freshwater Swamp Terrain in the Niger Delta, *Journal of Geology and Mining Research*, 5(3), 58 – 64.


Article DOI: 10.52589/IJMCE-4QKHY8JF
DOI URL: https://doi.org/10.52589/IJMCE-4QKHY8JF