Determination of Shear Strength and Rock Mass Parameters of Two Nigerian Limestone Deposits

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Abstract
The research work determined the shear strength parameters (angle of friction and cohesion), and the rock mass parameters (Tensile Strength, Global Strength and Deformation Modulus) of Obajana and Ewekoro limestone deposits of Nigeria. The Hoek – Brown constants were also evaluated and the Mohr – Coulomb Envelope plotted for the deposits. Obajana Deposit is unique while that of Ewekoro is of three different categories based on hardness, density and mere visual inspection. Schmidt hammer hardness and Density tests were performed. The results obtained were used to evaluate the Uniaxial Compressive Strength (UCS) and consequently, the Point load index (Iz) of the rock studied. The shear strength parameters for the kinematic analyses were obtained using Roclab 1.0 from Rocscience with the evaluated intact uniaxial compressive strength, estimated geological strength index, material constants, disturbance factor of the rock and the intact modulus as input parameters. This gives friction angles of 28.980, 24.720, 31.860 and 28.980; and cohesion of 3.414MPa, 1.928MPa, 5.566MPa and 2.902MPa for Obajana, Ewekoro Type I, Ewekoro Type II and Ewekoro Type III deposits respectively. The intact modulus of 12000MPa and highest level of disturbance by blasting (1) were used. These two chosen values are to ensure maximum factor of safety if the result of the shear strength and rock mass parameters are used in mining and civil design works.

Keywords: friction angle, cohesion, density, in-situ, fracture, strength,

INTRODUCTION

Rock Strength Criteria and Properties
A peak strength criterion is a relation between stress components which will permit the peak strengths developed under various stress combinations to be predicted. Similarly, a residual strength criterion may be used to predict residual strengths under varying stress conditions. In the same way, a yield criterion is a relation between stress components which is satisfied at the onset of permanent deformation (Brady and Brown, 2006).

Equation (1) is the general form of the peak strength criterion.

\[ \tau = f(\sigma_1, \sigma_2) \]  

This is sometimes written in terms of the shear, \( \tau \) and normal stresses, \( \sigma_1 \), on a particular plane in the specimen as given in Equation 2.

\[ \tau = f(\sigma_1) \]  

Because the intermediate principal stress, \( \sigma_2 \), has less influence on peak strength than the minor principal stress, \( \sigma_3 \), all of the criteria used in practice are reduced to Equation (3).

\[ \tau = f(\sigma_1) \]  

The Plane Griffith Criterion
The essence of the Griffith criterion is that for a material to break in tension owing to the presence of an existing microcrack, sufficient energy must be released to provide the necessary new surface energy as the crack propagates. The rate of strain energy release must be equal to or greater than the required surface energy increase (Hudson and Harrison, 1997). The Griffith criterion enables a relation to be derived between the uniaxial tensile strength and the triaxial compressive strength as shown in Equation 4.

\[ (\sigma_1 - \sigma_2) \cdot \left[ \frac{1}{\gamma} (\sigma_1 + \sigma_2 + \sigma_3) \right] \leq 2 \gamma S \]  

which for uniaxial compression with \( \sigma_3 = 0 \) gives

\[ \sigma_1 = \frac{2 \gamma S}{\gamma} \]  

This relation has been modified by various researchers for a variety of factors, particularly friction across the crack surfaces.

The Hoek – Brown Strength Criterion
Hoek and Brown (1980) represented the shear strength of rock as a curved Mohr envelope. They introduced their failure criterion to provide input data for the analyses required for the design of underground excavations in hard rock. The relationships they established have been re-examined.
and new elements introduced from time to time to account for the wide range of practical problems to which the criterion was being applied (Hoek et al., 2002). Typical of these enhancements were the introduction of the idea of “undisturbed” and “disturbed” rock masses (Hoek and Brown, 1988), and of a modified criterion to force the rock mass tensile strength to zero for very poor quality rock masses (Hoek et al., 1992). The Hoek – Brown criterion is defined by Equation (5).

\[ \sigma_1 = \sigma_2 + \sigma_3 + \left( \frac{C}{\sigma_2} \right)^2 \]  

Where \( \sigma_1 \) and \( \sigma_3 \) are respectively the major and minor effective principal stresses at failure, \( \sigma_1^c \) is the uniaxial compressive strength of the intact rock material and \( m \) and \( s \) are material constants; \( s = 1 \) for intact rock.

**The Generalized Hoek – Brown Strength Criterion**

In 1994, Hoek also introduced the concept of the Generalized Hoek–Brown criterion in which the shape of the principal stress plot or the Mohr envelope could be adjusted by means of a variable coefficient \( a \) in place of the 0.5 power term in Equation (5) as reflected in Equation (6).

\[ \sigma_1 = \sigma_2 + \sigma_3 + \left( \frac{C}{\sigma_2} \right)^{2a} \]  

Where \( m_1 \) is a reduced value of the material constant \( m \) for intact rock as shown in Equation (7).

\[ m_1 = \frac{(3s - 200)}{(s - 20)} \]  

There is an inbuilt chart in RocLab for estimating the values of \( m_1 \) for a wide variety of rock types, and \( s \) and \( a \) are constants for the rock mass given by Equations (8) and (9) respectively.

\[ s = \frac{\sigma_1^c}{C} \]  
\[ a = \frac{1}{2} \left( \frac{2}{m} - m_1 \right) \]  

\( D \) is a factor that depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses; guidelines for the selection of appropriate values for \( D \) are inbuilt in RocLab software.

The uniaxial compressive strength of the rock mass is obtained by setting \( \sigma_3 = 0 \) in Equation (6), giving rise to Equation (10).

\[ \sigma_1 = \sigma_2 \]  

The tensile strength is obtained in Equation (11) by setting \( \sigma_2 = 0 \); \( \sigma_3 = \sigma_3^c \) in Equation (6).

\[ \sigma_1 = \frac{\sigma_3}{m_1} \]  

This represents a condition of biaxial tension. Hoek (1983) showed that, for brittle materials, the uniaxial tensile strength is equal to the biaxial tensile strength. Normal and shear stresses are related to principal stresses by the Equations (12) and (13) published by Balmer (1952).

\[ \sigma_1 \pm \sigma_2 \pm \sigma_3 = \frac{1}{2} \left( \sigma_1 - \sigma_2 \right) \]  
\[ \sigma_1 \pm \sigma_2 \pm \sigma_3 = \frac{1}{2} \left( \sigma_1 + \sigma_2 \right) \]  

The Hoek–Brown failure criterion also allows the rock mass modulus of deformation to be calculated (in GPa) using Equation (15).

\[ E = \frac{\sigma_1 - \sigma_2}{\sigma_1 - \sigma_2} \]  

The original Equation proposed by Hoek and Brown (1997) was modified by including the factor \( D \), to allow for the effects of blast damage and stress relaxation.

The principal use of the rock mass modulus of deformation is in numerical analysis to calculate strain in rock slopes.

**The Mohr – Coulomb Criterion**

The analysis of failures in rocks involves examination of the shear strength of the rock mass on the sliding surface expressed by the Mohr–Coulomb failure criterion. Therefore, it is necessary to determine friction angles and cohesive strengths that are equivalent between the Hoek–Brown and Mohr–Coulomb criteria. These strengths are required for each rock mass and stress range along the sliding surface. This is done by fitting an average linear relationship to the curve generated by solving Equation (6) for a range of minor principal stress values defined by \( \sigma_3 < \sigma_2 < \sigma_3. \) The fitting process involves balancing the areas above and below the Mohr–Coulomb plot. The outcomes are Equations (16) and (17) for the angle of friction \( \phi' \) and cohesive strength \( c' \).

\[ \phi = \arctan \left( \frac{1 + \sigma_3}{1 - \sigma_2} \right) \]  
\[ c = \frac{1}{2} \left( \frac{\sigma_1 + \sigma_2}{\sigma_1 - \sigma_2} \right) \]  

\[ \sigma_3 = \sigma_3 \text{max} \]  

The value of \( \sigma_3 \text{max} \), the upper limit of confining stress over which the relationship between the Hoek–Brown and the Mohr–Coulomb criteria is considered has to be determined for each individual case.
The Mohr–Coulomb shear strength $\tau$, for a given normal stress $\sigma$, is found by substitution of these values of $c'$ and $\phi'$ into the following Equation (18).

\[ \tau = c' + \sigma \tan \phi' \]  \hspace{1cm} (18)

Equivalent plot, in terms of major and minor principal stresses, is defined by in Equation (19).

\[ \tau = c' + \sigma_1 \tan \phi' = c' + \sigma_3 \tan \phi' \]  \hspace{1cm} (19)

However, for slope stability, failure is initiated along a sliding surface within the slope where the rock is subject to a biaxial stress field and it is useful to consider the overall behavior of a rock mass rather than detailed failure propagation process. This leads to the concept of a global “rock mass strength” and Hoek and Brown (1997) proposed that this could be estimated from the Mohr–Coulomb relationship in Equation (20).

\[ \tau = \frac{2c' \sin \phi'}{1 - 2\sin \phi'} \]  \hspace{1cm} (20)

with $c'$ and $\phi'$ determined for the stress range $c' < \frac{2\sigma}{\phi'}$ giving Equation (21) for the rock mass strength $\sigma_{\text{eq}}$.

\[ \sigma_{\text{eq}} = \frac{c' \sin \phi'}{1 - 2\sin \phi'} \]  \hspace{1cm} (21)

Determining the appropriate value of $\sigma_{\text{eq}}$, for use in Equations (16) and (17) depends on the specific application. For the case of slopes, it is necessary that the calculated factor of safety and the shape and location of the failure surface be equivalent. This is given by $\sigma_{\text{eq}}$, for both Mohr – Coulomb and Generalized Hoek – Brown criteria. Equation (22) is the relationship between $\sigma_{\text{eq}}$, the rock mass strength $\sigma'_{\text{eq}}$, and the stress level on the sliding surface.

\[ \sigma_{\text{eq}} = \sigma'_{\text{eq}} \]  \hspace{1cm} (22)

The stress level on the sliding surface is related to the slope height $H$ and the unit weight of the rock $\gamma_r$ is given by Equation (23).

\[ \sigma_{\text{eq}} = \frac{\gamma_r H^2}{2H} \]  \hspace{1cm} (23)

1.6 Uniaxial Compressive Strength

The Uniaxial Compressive Strength (UCS) of rock may be regarded as the highest stress that a rock specimen can carry when a unidirectional stress is applied, normally in an axial direction, to the ends of a cylindrical specimen (Olabiyi and Ojo, 2004). International Society of Rock Mechanics Commission, ISRM (1981) suggested Equation (24) for determining the uniaxial compressive strength.

\[ \sigma_{\text{UCS}} = \frac{F}{A_{\text{spec}}}, \text{MPa} \]  \hspace{1cm} (24)

Where $\sigma_{\text{UCS}}$ is the compressive strength in MPa, and $F$ is the load at failure in kN. According to Bell (1992), classification of intact rocks based on uniaxial compressive strength as suggested by ISRM is shown in Table 1.

<table>
<thead>
<tr>
<th>Strength Classification</th>
<th>Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>Under 6</td>
</tr>
<tr>
<td>Moderate</td>
<td>6 – 20</td>
</tr>
<tr>
<td>High</td>
<td>20 – 60</td>
</tr>
<tr>
<td>Very High</td>
<td>60 – 200</td>
</tr>
<tr>
<td>Extremely High</td>
<td>Over 200</td>
</tr>
</tbody>
</table>

Source: Bell, 1992

Point Load Strength Index

The point strength test in its role as an index test is one member of the indirect tensile test family that is carried out to assess the strength characteristics of any rock types. The point load test can be used to estimate the compressive strength for rock slope design. Because the point load test provides an index value for the strength, usual practice is to calibrate the results with a limited number of uniaxial compressive tests on prepared core samples.

The test procedure involves placing the sample between the platens and applying a load with the hydraulic jack to break the sample in tension. If $P$ is the point load breaking strength, then the point load index, $I_p$, is given by Equation (25).

\[ I_p = \frac{P}{A_{\text{spec}}}, \text{MPa} \]  \hspace{1cm} (25)

Where $D_{\text{eq}}$ is the equivalent core diameter, defined as:

\[ D_{\text{eq}} = D^2 \]  \hspace{1cm} (Diametral tests where $D$ is the core diameter),

Or

\[ D_{\text{eq}} = \frac{4A_{\text{spec}}}{\pi} \]  \hspace{1cm} (Axial, block or lump tests),

Where $W$ is the specimen width and $D$ is the distance between the platens. The term (WD) is the minimum cross-sectional area of a lump sample for the plane through the platen contact points (Wyllie and Mah, 2005).

Broch and Franklin (1972) gave a classification of intact rock material based on point load strength index as shown in Table 2. The index, $I_p$, varies with $D_{\text{eq}}$ and so size correction must be applied in order to obtain a unique point load strength index for a particular rock sample for use for strength classification. Wherever possible, it is preferable to carry out diametral tests on 50–55 mm diameter specimens.

<table>
<thead>
<tr>
<th>Strength Classification</th>
<th>Point Load Strength Index (MPa)</th>
<th>Equivalent Uniaxial Compressive Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely High Strength</td>
<td>Over 10</td>
<td>Over 160</td>
</tr>
<tr>
<td>Very High Strength</td>
<td>3 – 10</td>
<td>50 – 160</td>
</tr>
<tr>
<td>High Strength</td>
<td>1 – 3</td>
<td>15 – 60</td>
</tr>
<tr>
<td>Medium Strength</td>
<td>0.3 – 1</td>
<td>5 – 16</td>
</tr>
<tr>
<td>Low Strength</td>
<td>0.1 – 0.3</td>
<td>1.6 – 5</td>
</tr>
<tr>
<td>Very Low Strength</td>
<td>0.03 – 0.1</td>
<td>0.5 – 1.6</td>
</tr>
<tr>
<td>Extremely Low Strength</td>
<td>Less than 0.03</td>
<td>Less than 0.5</td>
</tr>
</tbody>
</table>

Source: Broch and Franklin, 1972
The size-corrected Point Load Strength Index, \( I_{PL}\), is defined as the value of \( I_{L} \) that would have been measured in a diametral test with \( D = 50 \text{ mm} \). The results of several series of tests carried out by a number of investigators show that the value of \( I_{L} \) determined in a test of equivalent diameter, \( D_{eq} \), may be converted to an \( I_{PL} \) value by the relationship in Equation (26).

\[
I_{PL} = I_{L} \left( \frac{D_{eq}}{D} \right)^{0.6} 
\]

Beginning with Broch and Franklin (1972), a number of investigators have developed correlations of the Point Load Index, \( I_{L} \), with the uniaxial compressive strength, \( f_{c} \). The most commonly used correlation is Equation (27).

\[
f_{c} \approx (22 - 23) I_{L}^{0.6} \quad \text{where} \quad I_{L} = \frac{P}{D^{2}}
\]

Also, Bieniawski (1975) suggested Equation (28) as an approximate relationship between \( f_{c} \), \( I_{L} \) and the core diameter, \( D \), measured in millimeters.

\[
f_{c} = (4 + 0.07D) I_{L}^{0.6} \quad \text{where} \quad D \leq 50 \text{ mm}
\]

### METHODOLOGY

#### Density

**Density Test**

The objective of the test is to measure the dry density of rock samples of irregular form from Obajana and Ewekoro deposits. The Saturation and Buoyancy technique for irregular rock sample was adopted and the procedures follow the standard suggested by ISRM (1981) and conform to ASTM (1994).

The saturated volume of the sample was calculated using Equation (29).

\[
V_{s} = \frac{V_{f} - V_{i}}{\rho_{w}} 
\]

where \( V_{f}(\text{ml}) \) is the final water level and \( V_{i}(\text{ml}) \) is the initial water level in the cylinder after the immersion of the irregular rock sample.

The dry density of the rock samples was calculated using Equation (30).

\[
\rho_{d} = \frac{M}{V_{s} - V_{i}} 
\]

where \( M \) (g) is the oven dried mass at a temperature of 105°C.

### Hardness Test

The test involved the use of Schmidt impact hammer of type N for the hardness determination of lump rock samples. The rebound value of the Schmidt Hammer is used as an index value for the intact strength of rock material, but it is also used to give an indication of the compressive strength of rock material (ISRM, 1981). The result of the hardness test is used to evaluate the Unconfined Compressive Strength (UCS) and consequently, the Point Load Index Values. The standard method for the Schmidt hammer test as described by ISRM (1981) and ASTM (1994) was followed. The measured test values for the samples were ordered in descending order. The lower 50% of the values were discarded and the average obtained of the upper 50% values to obtain the Schmidt Rebound Hardness (ISRM, 1981).

The average values obtained from the Type – N machine was converted to Type – L readings by using the relationship established by Aydin and Basu (2005) as depicted in Equation (31).

\[
R_{L} = 1.095 R_{N} - 0.207 
\]

where \( R_{N} \) = Rebound Hardness value from Type N Hammer, and \( R_{L} \) = Rebound Hardness value from Type L Hammer.

### Unconfined Compressive Strength (UCS)

The Uniaxial Compressive Strength of the rock samples were estimated from the values of the equivalent Type L Schmidt Hammer Hardness and the density of the rock.

The UCS values were estimated by an Equation (32) developed by Xu and Mahtab (1990).

\[
UCS = 0.215 \times 10^{6} R_{L} \times \rho^{0.5} 
\]

where \( UCS \) = Uniaxial Compressive Strength (MPa),

\( R_{L} \) = Rebound Hardness value of Type L Hammer, and

\( \rho \) = Density of rock (g/cm³).

The UCS was used for the strength classification and characterization of the intact rock for the generalized Hoek – Brown criterion and equivalent Mohr – Coulomb criterion for obtaining the friction angle and the cohesion.

### The Point Load Index (\( I_{L} \))

The Point Load Index (\( I_{L} \)) Values were estimated from the Uniaxial Compressive Strength (UCS) values using Equation (33) established purely for carbonate rocks by Osman (2010).

\[
I_{L} = 0.257 UCS - 0.3267 
\]

The \( I_{L} \) (like the UCS) was also used for the strength classification and characterization of the intact rocks. It was further used with the Fracture Spacing Index (I) and the Geological Strength Index (GSI) for the assessment of the most economic excavation method with the aid of appropriate charts.

### Shear Strength Parameters

The shear strength of the jointed rock mass is based on the Hoek – Brown strength criterion which defines the strength as a curved envelope (Wyllie and Mah, 2005). The friction angle and the cohesion for this criterion are evaluated using the program RocLab 1 (RocScience, 2002) which implements Equations (16) and (17) respectively.

### RESULTS AND DISCUSSION

#### Density Result

The density is shown in Table 3. All the rock samples tested possess an average density of between 2.4 – 2.7g/cm³. Ewekoro has three different types of deposits. The three rock types have densities of 2.40,
2.70 and 2.50g/cm$^3$. Obajana deposit is unique with a density of 2.6g/cm$^3$.

Table 3 is a summary of the density result. The Table shows that type II deposit of Ewekoro has the highest density values while type I of the same Ewekoro has the least.

Table 3: Summary of density Results

<table>
<thead>
<tr>
<th>Test No</th>
<th>Obajana</th>
<th>Ewekoro Type I</th>
<th>Ewekoro Type II</th>
<th>Ewekoro Type III</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.65</td>
<td>2.40</td>
<td>2.69</td>
<td>2.51</td>
</tr>
<tr>
<td>2</td>
<td>2.66</td>
<td>2.41</td>
<td>2.68</td>
<td>2.49</td>
</tr>
<tr>
<td>3</td>
<td>2.63</td>
<td>2.40</td>
<td>2.76</td>
<td>2.54</td>
</tr>
<tr>
<td>4</td>
<td>2.50</td>
<td>2.37</td>
<td>2.72</td>
<td>2.49</td>
</tr>
<tr>
<td>5</td>
<td>2.56</td>
<td>2.43</td>
<td>2.67</td>
<td>2.47</td>
</tr>
<tr>
<td>AVG</td>
<td>2.60 (g/cm$^3$)</td>
<td>2.40(g/cm$^3$)</td>
<td>2.70 (g/cm$^3$)</td>
<td>2.50 (g/cm$^3$)</td>
</tr>
</tbody>
</table>

Schmidt Hammer Hardness Values

Table 4 shows the arrangement of the rebound hardness in descending values. The lower 50% of the values were discarded and the average obtained of the upper 50% values for each of the rock samples as Suggested by ISRM (1981). The average of the upper half is taken to represent the average rebound values of the hardness test.

Type N Schmidt Hammer test is mostly use for concretes while ISRM (1981) recommends the use of type L for rocks. Therefore the average values of type N obtained were converted to type L reading using Equation (31). The results are shown in Table 5.

Table 4: Descending Values of Schmidt Rebound Hardness

<table>
<thead>
<tr>
<th>S/N</th>
<th>Obajana</th>
<th>Ewekoro I</th>
<th>Ewekoro II</th>
<th>Ewekoro III</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>46</td>
<td>39</td>
<td>50</td>
<td>42</td>
</tr>
<tr>
<td>2</td>
<td>45</td>
<td>37</td>
<td>49</td>
<td>42</td>
</tr>
<tr>
<td>3</td>
<td>43</td>
<td>37</td>
<td>48</td>
<td>41</td>
</tr>
<tr>
<td>4</td>
<td>42</td>
<td>37</td>
<td>48</td>
<td>41</td>
</tr>
<tr>
<td>5</td>
<td>41</td>
<td>36</td>
<td>47</td>
<td>40</td>
</tr>
<tr>
<td>6</td>
<td>41</td>
<td>36</td>
<td>46</td>
<td>39</td>
</tr>
<tr>
<td>7</td>
<td>40</td>
<td>36</td>
<td>46</td>
<td>39</td>
</tr>
<tr>
<td>8</td>
<td>39</td>
<td>35</td>
<td>45</td>
<td>38</td>
</tr>
<tr>
<td>9</td>
<td>39</td>
<td>35</td>
<td>45</td>
<td>37</td>
</tr>
<tr>
<td>10</td>
<td>39</td>
<td>34</td>
<td>44</td>
<td>35</td>
</tr>
</tbody>
</table>

The result shows that Ewekoro Type II deposit has the highest value of the rebound hardness and closely followed by Obajana.

UCS and the Point Load Index Values

The Uniaxial Compressive Strength (UCS) of the rock samples was evaluated from the Schmidt Hammer hardness values using Equation (32). The Point load Strength was in turn estimated from the UCS values by using Equation (33). Table 6 shows the Uniaxial Compressive and Point Load Strengths results and their classification according to Table 2 as given by Broch and Franklin (1972).

Table 6: UCS, Point Load Values and their Rock Class

<table>
<thead>
<tr>
<th>Samples</th>
<th>UCS (Mpa)</th>
<th>Is (Mpa)</th>
<th>Rock Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Obajana</td>
<td>69.04</td>
<td>2.92</td>
<td>High to Very High Strength</td>
</tr>
<tr>
<td>Ewekoro I</td>
<td>47.91</td>
<td>1.92</td>
<td>High Strength</td>
</tr>
<tr>
<td>Ewekoro II</td>
<td>96.00</td>
<td>4.18</td>
<td>Very High Strength</td>
</tr>
<tr>
<td>Ewekoro III</td>
<td>58.70</td>
<td>2.43</td>
<td>Very High Strength</td>
</tr>
</tbody>
</table>

All the rock types tested are of "Very High Strength class" to "High Strength Class". Ewekoro type II deposit has the highest value which is in agreement with its high density and hardness values.

Analysis of Rock Mass Strength

The rock mass strength parameters were estimated from RocLab 1.0 software. Table 7 shows the input parameters while Table 8 shows the results obtained for each of the strength parameters. Figures 1 – 4 are Hoek – Brown and the Mohr – Coulom curves of the four tested samples as obtained from the software.

Table 7: Input Data for the determination of Rock Mass Parameters

<table>
<thead>
<tr>
<th>Sample</th>
<th>Sigci (Mpa)</th>
<th>GSI</th>
<th>m2</th>
<th>D</th>
<th>E4 (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Obajana</td>
<td>69.04</td>
<td>70</td>
<td>12</td>
<td>1</td>
<td>12000</td>
</tr>
<tr>
<td>Ewekoro I</td>
<td>47.91</td>
<td>65</td>
<td>10</td>
<td>1</td>
<td>12000</td>
</tr>
<tr>
<td>Ewekoro II</td>
<td>96.00</td>
<td>75</td>
<td>12</td>
<td>1</td>
<td>12000</td>
</tr>
<tr>
<td>Ewekoro III</td>
<td>58.70</td>
<td>70</td>
<td>12</td>
<td>1</td>
<td>12000</td>
</tr>
</tbody>
</table>
Table 8: Results of the Estimated Rock Mass Parameters

<table>
<thead>
<tr>
<th>Sample</th>
<th>Hooke – Brown Classification</th>
<th>Mohr-Coulomb Fit</th>
<th>Rock Mass Parameters (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ω_H</td>
<td>S</td>
<td>a</td>
</tr>
<tr>
<td>Obajana</td>
<td>1.408</td>
<td>0.0067</td>
<td>0.501</td>
</tr>
<tr>
<td>Ewekoro I</td>
<td>0.821</td>
<td>0.0029</td>
<td>0.502</td>
</tr>
<tr>
<td>Ewekoro II</td>
<td>2.012</td>
<td>0.0155</td>
<td>0.501</td>
</tr>
<tr>
<td>Ewekoro III</td>
<td>1.408</td>
<td>0.0067</td>
<td>0.501</td>
</tr>
</tbody>
</table>

S sigc = Intact Uniaxial Compressive strength (MPa)
GSI = Geological Strength Index
ω₀, ω₁, ω₂, s and a = Material Constants
ω₃ = Cohesion (MPa)
σₑ = Friction Angle (Deg.)
σₑ = Tensile Strength (MPa)
σₑ = Uniaxial Compressive strength (MPa)
Erm = Global Strength (MPa)

CONCLUSION
From the detail analyses conducted, the following conclusions are reached:

1. Ewekoro Type II deposit has the highest Uniaxial Compressive Strength (UCS) and Point Load Index I, values of 96.00MPa and 4.18MPa respectively. The least is Ewekoro Type I having UCS and I of 47.91MPa and 1.92MPa respectively.

2. Ewekoro Type II also has the highest cohesion and friction angle of 5.566MPa and 32° respectively while Ewekoro Type I also has the least of 1.92MPa and 24.72° respectively.

RECOMMENDATION
The results obtained for the Hoek – Brown constants, Mohr – Coulomb’s Fit and the Rock Mass parameters can be used for various Mining and Civil Engineering design purposes.

REFERENCES


APPENDIX

Figure 1: Hoek – Brown Classification and Mohr – Coulomb Envelope for Obajana Deposit

Figure 2: Hoek – Brown Classification and Mohr – Coulomb Envelope for Ewekoro Type I Deposit
Figure 3: Hoek – Brown Classification and Mohr – Coulomb Envelope for Ewekoro Type II Deposit

Figure 4: Hoek – Brown Classification and Mohr – Coulomb Envelope for Ewekoro Type III Deposit