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Decipherment of Earth Dam Stability Status from Geoelectric Data Analysis

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Abstract

This article gives an overview on the use of linear and nonlinear arrays vertical electrical soundings in combination of rock mechanic principles towards decipherment of stability status of an existing earth dam. The dam, Tiga dam (commissioned in 1974), is located in the Basement Complex of Northern Nigeria between $N\,11^0\ 28'\ 09.0''$ to $E\ 8^0\ 22'\ 04.7''$ ***from the west and*** $N\,11^0\ 27'\ 37.6''$ ***to*** $E\ 8^0\ 25'\ 55.4''$ ***from the east at an elevation of about 500m above mean sea level. It has an approximate embankment length of 6 km. The stability status was estimated by calculating the range in safety factor. The calculated safety factor has minimum value of 0.80. This proved to be very important as the probability of failure is closely tied to the value of safety factor. The probability of failure is high whenever the factor is less than unity. These values when compared with the minimum value of 1.0 quoted from previous study, published in 2002, demonstrated that the range of factor of safety obtained now is a reasonable risk of failure for short term conditions. However, in a long term, this range of failure is not acceptable considering the level of investment downstream and cost of repair.***

Keywords: solar, Geophysical, Mechanics, Rock, Safety and Stress.

1.0 Introduction

Rock failure is highly linked to the behavior of fractures. Rock at depth is subjected to stresses resulting from the weight of overlying strata and from the locked in stress of tectonic origin. Establishing human artifacts like dams and other engineering structures disturb the stress field.The cumulative effect of different stress fields brings about instability. This createsinstability problems to a site of large dams and the possibility of a dam failure that threaten the safety of people and industrial properties as well as cause substantial environmental effects. Studies of past earth dam failures show three major causes: (1) seepage and internal erosion in the embankment (2) seepage and erosion of the foundation and (3) erosion of the overtopping [1]. In this work results obtained from past geophysical activities[2] combined with past history and principles of rock mechanics were used to study the stability status of Tiga Dam. The dam, Tiga dam (commissioned in 1974), is located in the Basement Complex of Northern Nigeria between $N\,11^0\ 28'\ 09.0''$ to $E\ 8^0\ 22'\ 04.7''$ from the west and $N\,11^0\ 27'\ 37.6''$ to $E\ 8^0\ 25'\ 55.4''$ from the east at an elevation of about 500m above mean sea level. It has an approximate embankment length of 6 km [3]. The geophysical results provided estimates on depth of the fracture underneath the embankment and secondary porosity of the region. The arithmetic mean porosity of the oblique fracture porosity obtained about the location of the fracture was found to be 0.20 (20 %). Moreover, it regarded the fracture as anthropogenic in origin. The geology identifies the rock type and morphology of the fractures. The rock mechanics gave the relations between the fractures and the stress field.The stability status was estimated by calculating the range in safety factor. The calculated safety factor extended to arange below 1.0 as against a value, greater than one, given in a previous (2002) study that gave a value greater [3]. However, this result should be taken with caution as there was no rock in-situ data and in addition there is no generally acceptable (universal) rock model.And that the calculated safety factor is dependent on variables which their values lie within ranges rather fixed.Some of the formulae are empirical.Despite these shortcomings, the study provide guide that was not available earlier. This proved to be very important as the probability of failure is closely tied to the value of safety factor. The probability of failure is high, the closer to zero the factor.

2.0 Theory and Methodology

Risk is defined mathematically as product of vulnerability and hazard (H). Hazard is a function of material and life loss or casualty. The vulnerability (here taken as product of factor of safety (FS) and failure facilitators (F)) is tied to the

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geologic strength index of the host environment and pervading stress field which dictate the probability of rock failure. The factor of safety is defined as the ratio of strength or resisting force (C) of rock element to demand (D) (stress or disturbing force).Thus the Factor of Safety can be expressed as

$$FS = \frac{C}{D} \quad (1)$$

Failure would occur when factor of safety is less than unity, i.e. "when the stress is equal to the strain" [4]. Failure facilitators include all such effects that reduce rock strength and amplify stress and stress concentration. Moreover, the stress field is disrupted when an opening or a load is added. The fundamental steps in understanding and predicting rock failure involves the identification and location of tectonically significant fractures. At shallower depths brittle rock fracture is strongly influenced by increasing strain rate, confining pressure, pore pressure and chemical interactions. The former two have more influence in crystalline (granitic) rock. When a sample is overloaded in tension, pre-existing flaws act as stress concentrators, intensifying the stress field at their tips. According to Deere [5], stress intensity increases as square root of crack length. If a remote stress is increased until stress intensity on the critical flaw tips exceeds the fracture toughness, the flaw will grow in the plane perpendicular to the maximum tensile stress direction. For constant sample boundary conditions, each increment of crack growth will result in further increase in stress intensity, thereby further, accelerating the fracturing process. There are large varied forms of stress fields and other competing unquantifiable factors that influence the fracture process which preclude the development of a universal law that can be used in any practical way to predict fracture strength for arbitrary rock. As a result, variety of empirical formulae and classification schemes have been used as predictive tools in estimating stress bearing capacity of various rock types. There was conflict in authority between the stresses that led to extension of the fracturing and compaction of the bedrock blocks. Hoek andBrown [6] have given a treatise on estimating the values of strength of rock masses and mathematical relationships for calculating values of stress components (vertical and horizontal stresses) for specific sites based on world stress map. They showed that there were definite trends (increase in stress with increasing depth) which emerged from statistical fitting of their empirical failure criteria to published triaxial data for coarse grained polymerallic igneous and metamorphic rocks (amphibolites, gabbro, diorite and granodiorite). According to Hoek-Brown criterion [6], for intact rock, the major principal stress σ_1 is related to the minor principal stress σ_3 (Figure 1) by

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_{ci}\sigma_3 + s\sigma_{ci}^2} \quad (2)$$

Where $\sigma_{ci}$is the uniaxial compressive strength of intact rock piece, and m and s are Hoek-Brown unitless constants for the rock type. The parameter m is related to the degree of particle interlocking present: for intact rock this is high and reduces as the degree of brokenness increases. The parameter s relates to the degree of fracturing present in the intact rock. It is the representation of the cohesion of the rock and its value ranged from zero for highly fractured

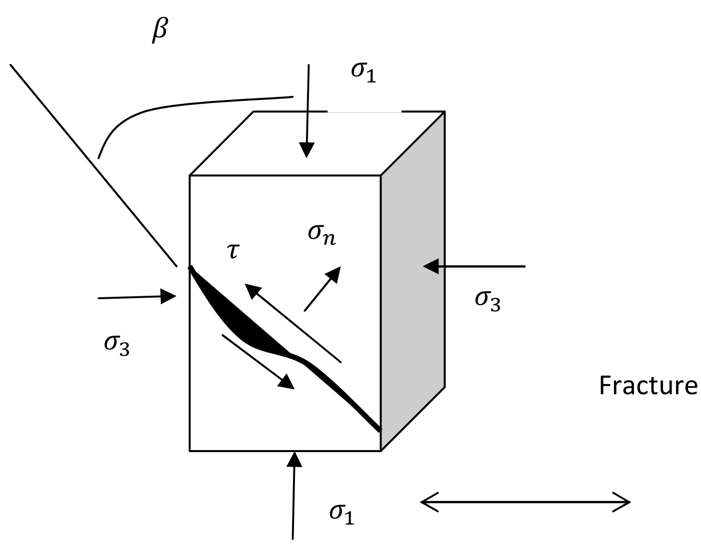

Figure .1: Dispositions of Stresses about Fracture

rock to unity for intact rock. The parameter σ_{ci} is uni-axial compression strength of the intact rock. For jointed rock (specific rock type) mass, the modified form as given in [7] is used. The modified expression is

$$\sigma_1' = \sigma_3' + (m_b\sigma_{ci}\sigma_3' + s\sigma_{ci}^2)^a \tag{3}$$

Where $m_b = mexp\left(\frac{GSI-100}{28-14N}\right)$, $s = exp\left(\frac{GSI-100}{9-3N}\right)$, N =nature of rock type with a value ranging from zero for intact rock to one for highly disturbed rock, and GSI = Geologic Strength Index, has variable values for different rock type. For GSI > 50, the superscript a takes a value of 0.5 while a tends to 0.65 for GSI < 50.

According to Mohr-Navier Coulomb criteria [6], the relationships between the stresses at failure are

$$\tau = \frac{(\sigma_1-\sigma_3)\sin 2\beta}{2} \tag{4a}$$

and

$$\sigma_n = \frac{(\sigma_1+\sigma_3)}{2} - \frac{(\sigma_1-\sigma_3)\cos 3\beta}{2} \tag{4b}$$

Where τ is the shear stress and $\sigma_n$is effective normal stress and β is the angle between the failure plane and the major principal stress direction.

Hoeh and Brown [6] gave expressions for estimating the components of stress intensity due to local rock overburden σ_{loc}as follows;

(a) Vertical Component of stress due to rock masses

$$\sigma_v = \gamma z \tag{5a}$$

(b) Horizontal Component of stress due to rock masses

$$\sigma_h = \frac{v}{1-v}\sigma_v \tag{5b}$$

Where γ is unit weight with a value of 0.027MPa/m, v = is poisson's ratio for the rock type under consideration. The resultant stress due to local overburden (rock) is

$$\sigma_{loc} = \sqrt{\sigma_v^2 + \sigma_h^2} \tag{6}$$

Consequently the total overburden compressive stress σ_{tot} is given by the sum of the world tectonic stress and local stress fields and is given by

$$\sigma_{tot} = \sigma_w + \sigma_{loc} \tag{7}$$

However, according to Biot's law [8], the effective compressive stress in which the effect of pore pressure is incorporated is expressed as

$$\sigma_{eff} = \sigma_{tot} - \alpha\sigma_{hy} \tag{8}$$

Where $\alpha = 1$ for incompressible material and σ_{hy} is pore pressure. Thus from equation (1) D is equivalently σ_{eff}

3.0 Data Analysis

The values of the parameters used were obtained from world stress index map and local environment stress distribution and contributors. To evaluate equation (3) the work of[7] gave the valuesof m =32$\pm$3 where as the GSI for granitic rock is 60. This value of GSI connotes that a = 0.5. For the present study an intermediate values of m = 32 and N = 0.5 were used based on the nature of the oblique fracture as the region was considered to be partially fractured. Putting the values of the above parameters enabled us to obtain the m_b and s expressions enabled as follow

$$m_b = 32exp\left(\frac{60-100}{28-14xo.5}\right) = 4.76 \tag{9a}$$

and

$$s = exp\left(\frac{60-100}{9-3x0.5}\right) = 0.005 \tag{9b}$$

From the work of [6], $\sigma_{ci} = 223.7\ MPa$ for granitic rock, this is the basement rock type in the region of study. So also, gave the range of $\sigma_3$in terms of σ_{ci}as $0 < \sigma_3 < 0.5\sigma_{ci}$. On taking the intermediate value, $\sigma_3 = 55.83MPa$. Putting these values in equation (3), we got the strength of rock at the site,

$$\sigma_1' = 55.83 + \left(4.76x223.7x55.83 + 0.005x223.7^2\right)^{0.5} = 300.16\text{MPa} \tag{10}$$

based on the value resistivity and porosities in the N-E site [2] the fracture location must be along the center of anomalous feature. Thus β is here taken as 50^{o}. Substituting the values of σ_1, σ_3 and β in equations (4a) and (4b) yields

$$\tau = \frac{(300.16-55.83)\sin 100}{2} = 120.31\text{MPa}$$

$$\sigma_n = \frac{(300.16+55.83)}{2} - \frac{(300.16-55.83)\cos 150}{2} = 283.8\text{MPa}$$

.These results gave the value of resultant world stress σ_w for granitic rock as

$$\sigma_w = \sqrt{\tau^2 + \sigma_n^2} = 308.25\text{MPa} \tag{11}$$

To evaluate local stress, for granitic rock, $v = 0.4$ and z is the thickness of the overburden (about 118.0 m for the N-E, when depth of fracturing is regarded as extension of the embankment [2]). Putting these values in equations (5a) and (5b) as appropriate gave

$$\sigma_v = 0.027x118.0 = 3.19MPa$$

and

$$\sigma_h = \frac{0.4x3.19}{1-0.4} = 2.13MPa.$$

The resultant stress due to local overburden (rock) is

$$\sigma_{loc} = \sqrt{\sigma_v^2 + \sigma_h^2} = 3.84\ MPa. \qquad (12)$$

Consequently the total overburden compressive stress σ_{tot} is given by the sum of the world tectonic stress and local stress fields and is given by

$$\sigma_{tot} = \sigma_w + \sigma_{loc} = 308.25 + 3.84 = 312.84MPa \qquad (13)$$

From equation (8), $\alpha = 1$ for incompressible material and σ_{hy} is pore pressure. The pore pressure is synonymous to hydraulic pressure due to overhead water in the reservoir that is exerted in the pore of the rocks. The hydraulic pressure σ_{hy} can be calculated from

$$\sigma_{hy} = \delta gz$$

Where $\delta$is density of water and has the value of 1000kgm^{-3}, g is acceleration due gravity, taken as 9.81ms^{-2}. This yields the value of hydraulic compressive stress as $\sigma_{hy} = 1000x9.81x118$ = 1.16MPa

Thus the effective resultant compressive stress σ_{eff} in the field is given by

$$\sigma_{eff} = 312.84 - 1.16 = 311.68MPa \qquad (14)$$

The ranges for unconfined compressive and tensile strength for granite are respectively given as 233.7MPa [9] to 400MPa (peak value) and 6MPa to 18MPa in accordance with Beryee's law [8]. When sum of the extreme values are considered as representative strength range of granitic basement complex in the area, then the strength of the basement rock will range from 239.7MPa to 418MPa.

In line with the calculated values in the previous paragraphs, C ranged from 239.7 MPa to 418.0 MPa where as D = σ_{eff} =311.68 MPa. Therefore dividing the strength range with the effective resultant stress at the NE site gave the range in value of factor of safety as 0.80 to 1.34.

4.0 Discussion

The range in safety factor calculated here is 0.80 to 1.34, and in rock mechanics failure is assumed to occur whenever safety factor is less than unity. Thus the range of factor of safety obtained now despite its accompanying artifactshas provided information that never existed prior and serves as the only most up-to-date information on status of the structure.The major setback to such approach, as put forward by Lockner [8], is that "even in the same rock type there is variability in rock strength due to many locality specific factors". However, in a long term, this range of failure is not acceptable considering the level of investment downstream and cost of repair. Consequently, effective remedial measures should be instituted by conducting stress sensitivity analysis and other extensive studies of the site. This result should be taken with all seriousness when compared with the safety factors calculated by Haskoning [3]. The values quoted were for upstream under rapid draw down as 1 to 1.2 for downstream slope, steady speepage, static condition.

5.0 Conclusion

The work has demonstrated the use of Geophysical approach complemented by principles of rock mechanics to decipher the stability status for host environment of cultural artifacts. This, study conducted in 2010, eight years after the previous one, has value that showed depreciation in the values of safety factor by 0.2. The perceived minimizers of the safety factor are on the increase. Typically the recent heavy down pour, witnessed during the 2012 rain reason, that led to overflows of rivers and dams.

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