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Slope Stability Analysis of the Afaha Ekpenedi Integrated Farm Site, Esit Eket, Eastern Niger Delta, Nigeria

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Abstract
Natural and or engineered soils slopes failure can be very catastrophic with attendant sliding and or slumping downslope and infrastructural developments on slopes are at risk of instability and failure. Slope instability may be initiated by combination of forces gravitation, seepage water, erosion by run off, sudden lowering of water table adjacent to the slope and earthquakes. The proposed development of an integrated farm to include fisheries and aquaculture, poultry and piggery with associated feed and product processing factories, office complexes and power supply units situated at the slope toe region with slope angle of about 32^0; and visible and large tension cracks having measured and calculated depths ranging from 0.4m – 0.6m and 4.63 - 5.13m respectively in a perennial coastal microclimatic rainfall zone. Geotechnical investigation and slope stability analysis was considered for a sustainable project development. 7 borings were carried out to a depth of 4m using hand auger and samples collected and tested. The soil profile depicts a two critical layers' problem, one with a 0 – 2.0m layer of CI-CH clayey-silt with porosity 0.44 – 0.59, void ratio 1.04 – 1.3, hydraulic conductivity 2.04 x 10^{-3} – 2.95 x 10^{-3}, undrained cohesion 34 – 40kN/m^2, friction angle 6^0 – 8^0 and a second layer at 2 – 4.0m of CI – CH clayey-sand of 0.45 – 0.51 porosity, 0.98 - 1.22 void ratio, hydraulic conductivity 1.37 x 10^{-3} – 1.17 x 10^{-2}cm/sec, undrained cohesion 34 – 40KN/m^2 and friction angle 6^0 - 9^0. Three slope design sections were evaluated for instability using the analytical method of Fellenius. Results yielded mobilized cohesion and friction angles of 11.61KN/m^2 and 7.9^0, 12.40KN/m^2 and 5.9^0, and 12.12KN/m^2 and 5.9^0 respectively. Design sector 2 had a failed slope with factor of safety 0.91, while the other 2 slope design sectors were under limit equilibrium conditions. Slope retaining walls were recommended with rectangular foundation bearing capacities of 123.7KN/m^2, 133.9KN/m^2 and 116.2KN/m^2 based on the soil's shear strength respectively for each of the slope design sectors.
Keywords: Mobilized Cohesion, Friction Angle, Factor of Safety, Limit Equilibrium
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1. Introduction
1.1. Site Conditions
A total area of 3.114 hectares of land that slopes from the south to the north with visible tension cracks and erosional gullies crisscrossing the entire site, an average slope angle of 32^0 and a stream nearly at the toe in a perennial coastal microclimatic tropical equatorial rainforest (rainfall >3500mm in the wet season) was proposed for development of an integrated farm. The facilities include poultry (layers and broiler) houses, fishery and aquaculture units, products processing factories, feed mill, biogas digester, sales pen, farm offices and houses, elevated water tanks, internal access roads and a power house. These civil engineering activities will involve excavations which can be at the toe of the slope sections. In view of the project objective and site's topography, site geotechnical assessment and slope stability analysis was considered. Das and Sobhan observed that any ground that stands at an angle to the horizontal is a slope and whether natural or engineered, slopes have a tendency to move with such movements varying in origin and magnitude (Das and Sobhan, 2012). Slope failure and sliding can also range from near surface disturbances of weathered zones to deep seated displacement of rock masses (Blyth and De Freitas, 1982). Natural or engineered slopes' failure can be very catastrophic with attendant sliding and/or slumping and these failures are classified into fall, topple, slide, spread and flow (Cruden and Varnes, 1996). Slope instability may be initiated by combination of the forces of gravitational, seepage water, erosion by run off, sudden lowering of water adjacent to the slope and earthquake. Look noted that slope failure and the attendant sliding occurs when the frictional resistance of the earth's materials (soils and rocks) is exceeded by the disturbing forces or moments (shear stress) (Look, 2007), which attain magnitudes exceeding the shear strength of the materials (Abija *et al*, 2019). The failure and sliding occurs when the stress exceeds the mobilized shear strength reducing the factor of safety to less than unity. In practice, a soil slope fails when all the shearing strength along a critical surface is overcome (Murthy, 2014; Punmia *et al*, 1974; Murthy, 2007) and failure follows the most critical circle which must be determined in the stability analysis. Circular failure occurs dominantly in slopes of soils, mine dumps, weak rocks and highly jointed rock masses; and failure may be rotational, translational or compound. Though it is important to determine the possible position of the slip surface, the centre of the most critical circle can only be found by trial and error. Various slip circles may be

analyzed and the one yielding the minimum factor of safety can eventually be obtained. The major geological factors responsible slope failure includes soil or rock, geological structure, groundwater effect on the strength of the geological materials and the forces operating on the materials and the in situ stress magnitude and orientations, seismic disturbances from earthquakes, weathering, and erosion (Blyth and De Freitas, 1982; Look, 2007). All but earthquake forces pose a high risk of slope instability and or failure at the project site. Human occupation or the design of any engineering project on a natural slope or slopes of embankments of dams, roads or levees require stability analysis. Traditionally, two methods, namely the analytical or closed form and the numerical approaches which are a simplification of actual geological, mechanical and hydrologic conditions at the site (Das, 1941) are adopted. Pre-failure analysis assesses safety conditions and its planned performance while post failure of back analysis determines failure and the processes that caused it. Effective stability analysis of slopes requires site characterization of all geological factors and information regarding in situ engineering properties of the site, groundwater and seepage conditions and associated pore water pressures, lithology, stratigraphy, and geologic details disclosed by borings; their geologic interpretations, stresses, shear strength, geological structure, including bedding, folding, and faulting, alteration of materials by faulting, joints and joint systems, weathering, cementation, slickensides, field evidence relating to slides, earthquake activity, movement along existing faults, and tension jointing and general kinematic mechanisms. The engineering properties of the geomaterials necessary for the stability analysis include possible variation in natural deposits or borrow materials, natural water contents of the materials, climatic conditions, possible variations in rate and methods of fill placement, and variations in placement water contents and compacted densities that must be expected with normal control of fill construction in engineered slopes. Design of slope protection and stabilization has to include two components, vegetational-biological and mechanical-structural. The structural-mechanical component of slope protection can consist of conventional retaining walls, either the gravity or cantilever type, or a reinforced earth structure. This study was aimed at characterizing the subsurface soil profile, their engineering properties and stability analysis of the slopes for the design of structural mechanical stabilization using retaining walls.

1.2. Stability Analysis of Soil Slopes

Generally, there exist a state of geomechanical equilibrium in the ground forming the slope which can be distorted when the strength of the materials occurring along a failure surface at a depth (z) below the ground is exceeded under the action of in situ stresses and pore water pressure. Stability analysis of soils slopes involves two methods, the mass procedure and the method of slices. In the later, the mass of soil above the surface of sliding is taken as a unit and is useful when the soil is assumed to be homogeneous. In the later, the soil above the surface of sliding is divided into a number of vertical parallel slices and stability of each slice is calculated separately. Method of slices fully accounts for the inhomogeneity of the soil layers, pore water pressure and variation of the normal stress along the potential failure surface. The limit equilibrium and numerical modeling methods are most widely applied. Limit equilibrium is an analytical method that determines the force or moment equilibrium conditions against rotational shear failure and is based on stress - strain relationships. Das noted that stability analysis of the slope involves the determination of factor of safety which generally can be defined as the ratio of the shear strength to the shear stress on the potential failure surface (Das and Sobhan, 2012). It measures the overall amount by which the strength along the potential failure surface would have to fall short of the values defined by the cohesion and friction angle in order for the slope to fail. Factor of safety is the ratio of the resisting to the driving forces or moments on the failure surface or plane. Two types of factors of safety are used namely factor of safety with respect to shearing strength (friction) and factor of safety with respect to cohesion (height). A slope fails when the factor of safety falls below the conditions of limiting equilibrium or state of impending (factor of safety = 1.0) thus mobilizing the materials downward and outward along the failure plane and or surface which in soils slopes, is a circular arc that initiates sliding along the surface of contact between the slice and its base (Murthy, 2007). A slope is stable when the factor of safety is > 1.0 and unstable when the factor of safety is less than unity. The standard engineering requirement for soil slope stability is a factor of safety of 1.25 - 2.0 under static load and >1.1 under seismic load (Das, 1941). For slopes in cohesive soils, if the slope angle exceeds the angle of internal friction of the materials on the failure surface, the slope can only be stable up to a critical depth z after which a critical condition is reached and the slope attains a state of incipient failure because the stress equates the strength. For any given value of the slope angle and angle of internal friction, the critical depth is proportional to the cohesive strength of the materials. Information required for the assessment of the stability of a slope against circular failure include the location, orientation, and shape of a potential or existing failure surface, distribution of the materials within and beneath the slope, types of material and their representative shear strength parameters, drainage conditions, distribution of the piezometric levels along the potential failure surface and slope geometry.

Figure 1. Akwa Ibom State showing Esit Eket area.

2. Study Area

The study area (Figure 1) is a coastal microclimatic region embedded in an overall tropical climate and is marked by two seasons. Geohydrologically, the area is controlled by freshwater rivers which form receptacles for sediments from the hinterlands. The water table is very close to the ground surface and varies from 0 to 4meters (Oghenero *et al*, 2014) and the hydraulic conductivities of the aquiferous sand units vary from 3.82 x 10^{-3} to 9.0 x 10^{-2}cm/sec which indicates a potentially productive aquifer (Akpokodje, 1989). The geomorphology is dry flatland and a coastal plain with scattered seasonal swamps and five geomorphic zones have been recognized by (Fubara, et al, 1988) in the basin to include dry flat land and plains, the Sombrero Warri Deltaic plains with abundant fresh water back swamps, fresh water swamps, meander belts and alluvial swamps, salt water or mangrove swamps, and active/abandoned coastal ridges. The Niger Delta has been subdivided into three hydrometeorolical zones as coastal, transition mangrove and freshwater zones (Fubara, et al, 1988). The coastal zone is made of sand bars and ridges under ebbing and flooding tides of the coastal saline waters. The soils are dominantly cohesionless sands and gravels with high plasticity clays in places. The transition (mangrove) zone is very significant in terms of geotechnical engineering due to the preponderance of very plastic and cohesive clays and muds. These soils have a friction angle varying from $1^0 – 8^0$ and sometimes zero under saturated conditions (Oghenero *et al*, 2014; Teme, 2017) and depth of occurrence ranges from 10 – 15m and in exceptional places can extend up to 35 – 42m (Teme, 2017). Below these clayey/muddy layers, underlie well graded sands at varying levels of consolidation. The freshwater zone consists of the upper Niger Delta dominantly composed of cohesionless silty and lateritic soils characterized by acidic clayey soils (Gidigasu, 1976).

The project site lies within the coastal plain sands of the Niger Delta basin also called the Benin formation. These coastal sands whose lateral equivalent is the Ogwashi-Asaba formation conformably overlie Agbada formation. These sands were deposited during the Pleistocene – Oligocene period of the Tertiary era. The site's subsoil conditions reveal general average top soil strata ranging from 0 - 300mm depth underlain by predominantly fine to medium density clayey silts/sands deposited from coastal influence. This deposit continues as homogenous mixture of loose fine to medium silt with occasional clay content to a depth of 2m.

Table 1. Borehole coordinates and measured ground elevation at the site.

S/no	Northings	Eastings	Ground Elevation
BH1	N04°39 37.'899'	E008°06 37.113'	23m
BH2	N04°39 37.215'	E008°06 38.486'	26m
BH3	N04°39 38.060'	E008°06 38.724'	24
BH4	N04°39 37.040'	E008°06 38.560'	25m
BH5	N04°39 37.578'	E008°06 36.986'	26m
BH6	N04°39 35.547'	E008°06 35.763'	26m
BH7	N04°29 35.5'	E008°06 36.1'	29m

3. Methods of Investigations

3.1. Field Exploration

The study method involved field geotechnical investigation, laboratory analysis of soils samples, and slope stability analysis using the analytical limit equilibrium method. Field exploratory investigation employed visual site reconnaissance inspection, measurement of tension cracks, and sub-surface exploratory boring of 7 number geotechnical boreholes by conventional auger boring method to a maximum depth of 4.0m to derive disturbed and undisturbed samples according to the specifications of (BS 1377, 1990). Disturbed samples were obtained at depths where changes in lithology were encountered during the boring programme using the Shell-and-Auger equipment. Undisturbed samples were obtained by advancing a core cutter and the bottom of the hole and all samples were protected using by sealing of the core cutter with candle wax and disturbed samples in cellophane bags to prevent loss of moisture and consistency. Laboratory investigation included determination of soil index properties and shear strength tests. Classification tests using the disturbed samples included Moisture Content, Atterberg Limits and Sieve Analysis while for the design, in situ standard penetration test were conducted at depths as specified. These were field tests and laboratory analysis on sampled cores were conducted by mobilizing a crew with the following tools, hand auger, core cutter, accessory tools and sample tubes and parkers (BS 1377, 1990).

3.2. Laboratory Analysis and Material Characterization

All the geotechnical engineering tests necessary for slope stability analysis and bearing capacity of the in situ soil under effective stress condition were conducted in accordance with (BS 1377, 1990) standards for testing of soils for civil engineering works. These were identification and classification tests on disturbed samples recovered from the boreholes. Shearing strength tests were also carried out to. The following laboratory tests were carried out on the soil samples recovered from the borings Natural moisture content, specific gravity, particle size distribution Analysis, consistency limits, bulk density and unit weights determinations and shear strength tests.

3.3. Method of Slope Stability Analysis

Using measurement obtained from the site for critical slope design sections, engineering properties of the soils determined in geotechnical laboratories and slope from GPS elevations of the site for the slope stability analysis, the Fellenius, (1927) method was adopted considering that the groundwater level rises and the slope toe and base becomes submerged during the rainy season subjecting the subgrades to full saturation ($\phi = 0$) condition under its worst case scenario and precipitation being the major causative factor of slides. The undrained shear strength was also assumed to be constant with depth at each of the slope design sections in accordance with the Fellenius, (1927) method. Trial circular failure surfaces with centre O and radii, R were assumed and the factor of safety determined by repetitions and the value (lowest factor of safety) corresponding to the critical circular arc of logarithmic spiral selected. Only the critical factors of safety in the analysis have been reported. Equations (1) to (8) were employed in the analysis to the determine the saturated unit weight, mobilized cohesion, mobilized and critical friction angle and other parameters defined. Results obtained were evaluated in to determine factor of safety and overdesign with measured and calculated depths of the tension crack ranging from 0.4 - 0.6m and 4.63 - 5.13m respectively.

$$\gamma_{sat} = \frac{Gs+e}{1+e} \quad (1)$$

$$C_m' = \gamma_{sat} H \cos^2\beta\ (\tan\beta - \tan\phi_m') \quad (2)$$

$$\phi_m' = \arctan\ (\tan\phi_u)/Fs \quad (3)$$

$$\phi_c = 0.5(\beta + \phi_m') \quad (4)$$

$$Z_0 = 2C'/\gamma \quad (5)$$

$$H_c = C/\gamma[1/(\tan\beta - \tan\phi)\cos^2 i \quad (6)$$

$$Ns = C'/\gamma_{sat}H_c = [1 - \cos(\beta + \phi_m')]/[4 \sin\beta \cos\phi_m'] \quad (7)$$

$$F_oS = C_u/C_m \quad (8)$$

where,
C_m' = mobilized cohesion equivalent to the ratio of the actuating moment to the resisting moment, H = slope height, β = slope angle, ϕ_m' = mobilized angle of friction, ϕ_c = critical angle of the failure surface, N_s = stability number (dimensionless), H_c = critical depth, C' = effective cohesion, γ_{sat} = saturated unit weight, F_oS = Factor of safety

3.4. Investigation of the Bearing Strength of the Foundation Subgrade

The ultimate bearing capacity of the subgrades was determined at 1m and 2m depths using the formula for rectangular foundation footings in equations (9) and (10). For the stability of retaining walls, acceptable factor of safety against bearing capacity failure is 3. This was applied to obtained the allowable bearing capacity at the specified depth.

$$Q_{ult} = cN_c[1+0.3B/L] + \gamma D_f N_q + 0.5[\gamma B N_\gamma(1- 0.2B/L)] \quad (9)$$

$$Q_{all} = Q_{ult}/3 \quad (10)$$

4. Results and Discussion

4.1. Soil Stratigraphic Profile and Engineering Characteristics of the Subgrades

The typical soil profile (figure 2) characterizing the site's subsurface stratigraphy for the top 4.0m depicts a soft black humus layer extending from 0.00 – 0.30m followed by a firm light brown clayey silt/sand from 0.35 – 2.0m. The third layer is a firm reddish clayey sand that underlie the site from 2.0 - 5.0m. For purpose of slope stability analysis, the thin top and second soil layers were considered as one thus two composite layers for the analytical problem under limit equilibrium condition were assumed.

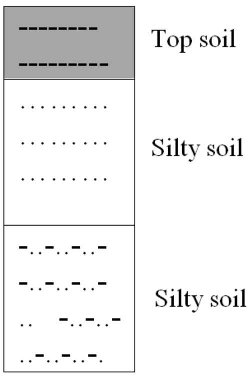

Figure 2. Typical profile of the soil at the project site.

The subgrades are poorly to well graded classifying as CI – CH silty sands under the unified soils classification system with gravels ranging from 0.8% to 30.0% with an average of 6.13%, the soils becoming gravellier with depth. The percentage composition of sands ranges from 65.4% to 89.7% with an average of 80.55%; the fines (silt/clay) fractions varies from 4.2% to 20.5 with an average of 13.36%. The liquid limit of the soils varies 43% to 58% with a mean of 48.57%, plastic limit from 11.5% to 35.9% averaging 24.25% and plasticity index from 14.1% to 33.5% with an average of 24.31%. Their coefficient of uniformity varies from 1.83 to 8.32 with an average of 5.67, coefficient of concavity varies from 0.7 to 2.63 with an average of 1.648 (Table 1). The moisture content of the subsoils varies from 18.42% – 26.5% averaging 21.1%, with standard deviation of 3.04 and coefficient of variation of 0.144. The specific gravity ranges from 2.61 - 2.68 with and a mean of 2.64, standard deviation of 0.026 and coefficient of variation of 0.01. The wet density varying from 1.353 -1.678g/cm^3 with an average of 1.484(g/cm^3), standard deviation of 0.121 and coefficient of variation of 0.08. Correspondingly, the wet unit weight varies from 13.27kN/m^3 to 16.46 kN/m^3 with an average of 14.56 kN/m^3, a standard deviation of 1.187 and a coefficient of variation of 0.82. The porosity ranged from 0.44 to 0.56 with a mean of 0.49, standard deviation of 0.05 and coefficient of variation of 0.1. The void ratio of the subgrades varied from 0.98 to 1.27 averaging 1.14 with a standard deviation of 0.12 and coefficient of variation of 0.11. The soils specific surface area ranged from of 620.75cm^2/cm^3 to 2456.0cm^2/cm^3 with an average of 1714.34cm^2/cm^3, a standard deviation of 665.84 and coefficient of variation of 0.39. Their coefficient of permeability ranged from 1.25 x 10^{-3}cm/sec to 1.17 x 10^{-2}cm/sec with a mean of 3.69 x 10^{-3}cm/sec, standard deviation of 0.04 and coefficient of variation of 1.078 (Table 2).

The shear strength properties of the soils indicate a range of 6^0 to 9^0 and 34.0 kN/m^2 to 40.0kN/m^2 angle of internal friction and cohesion respectively (Table 3 and Figure 3) and under the natural moisture conditions, the effective stress varies from 2537.3 - 3365.4kN/m^2 at a depth of 1.0m to 2537.31kN/m^2 - 7092.2kN/m^2 at a depth of 2.0m (Table 1) which is highly susceptible to increase when the soils are submerged and fully saturated.

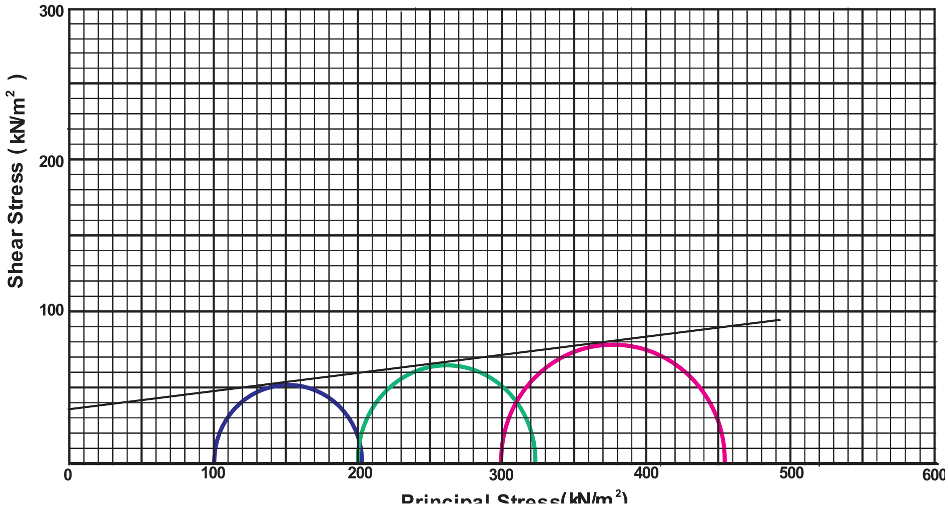

Figure 3. Typical Mohr circle (c-ϕ) of the undrained triaxial shear strength tests on the soils samples (a) 2.0m depth (b) 4.0m depth.

4.2. Slope Stability Analysis

Considering the two-layer slope stability problem and three slope design sections with a slope angle of 32^0, and cohesive strength of 36kN/m^2, 40kN/m^2 and 34kN/m^2 with slope height of 2.12, 2.11 and 2.2m respectively with zero friction under full saturation, a saturated unit weight of 17.23kN/m^3, 17.60kN/m^3, 17.20kN/m^3 on the design sections would mobilize a cohesion of 11.61kN/m^2, 12.40kN/m^2, 12.12kN/m^2, at slip surface radii (R) of 21m, 7.3m and 7.4m respectively. The slope sections depicted factors of safety of 1.0, 0.9 and 1,0 with over-design factors of 0.91, 0.80 and 0.41 in design sections 1, 2 and 3 respectively showing sections 1and 3 are under limit equilibrium condition while design section 2 has failed already (Table 4).

Silty sands overlying the slope sections with tension cracks portend a high susceptibility to liquefaction of the slope materials under full saturation and factors of safety depicting failed or limiting equilibrium conditions at the site, development will be unsustainable. The soils and or sediments below the water table will temporarily lose strength and behave as a viscous liquid rather than soil and leading to slope instability, lateral spreading of ground, settlement, increased lateral loads on retaining walls and piles, and loss of foundation support. Therefore, slope stabilization using reinforced concrete retaining walls was recommended.

4.3. Bearing Capacity of the Foundation Subgrades

To support the proposed cantilevered retaining walls for slope stabilization, the bearing strength of the subgrades was assessed. The ultimate bearing capacities of the foundations subgrades have determined 371.2, 380.1, 396.1 and applying a safety factor of the and allowable bearing capacity of 123.7 kN/m^2, 126.7 kN/m^2 and 132.0 kN/m^2 for proportioning of the cantilevered rectangular footings to support the earth pressure from the fills and natural ground. With a base thickness of 0.5m, breadth base of 2.25m, wall stem thickness of 0.5m, height of 1.6m, and breadth toe of 0.5m have been recommended for design and design must assess overturning, sliding and bearing capacity failures of the footings (Table 5).

Table 2. Soil grading and consistency properties

Sample Id	D_{10}	D_{30}	D_{60}	C_u	C_c	% Gravel	% Sand	% Silt/ Clay	LL (%)	PL (%)	I_p (%)	σ_{eff} (kN/m^2)	USCS class
BH1,1m	0.032	0.122	0.250	7.81	1.86	1.6	80.9	17.5	44.6	18.6	26.0	3365.4	CI
BH1,2m	0.131	0.317	1.09	8.32	0.70	30.4	65.4	4.2	45.0	11.5	33.5	2537.31	CI
BH2,1m	0.042	0.134	0.275	6.55	1.55	1.5	84.3	14.5	58.0	35.9	22.1	4072.4	CH
BH2,2m	-	-	-	-	-	1.7	77.8	20.5	54.8	30.6	24.2	4123.71	CH
BH4,1m	0.03	0.149	0.263	1.83	2.63	0.8	85.2	14.0	43.0	28.9	14.1	7092.2	CI
BH4,2m	0.07	0.168	0.269	3.84	1.50	0.8	89.7	9.5	46.0	20.0	26.0	3269.2	CI
Minimum	0.03	0.122	0.25	1.83	0.7	0.8	65.4	4.2	43.0	11.5	14.1	2537.31	
Maximum	0.131	0.317	1.09	8.32	2.63	30.4	89.7	20.5	58.0	35.9	33.5	7092.2	
Mean	0.061	0.178	0.429	5.67	1.648	6.13	80.55	13.36	48.57	24.25	24.31	4076.70	
Std. dev	0.042	0.079	0.37	2.76	0.696	11.89	8.445	5.807	6.23	9.057	6.31	1588.91	
Coefficient of variation	0.693	0.4471	0.86	0.49	0.42	1.94	0.11	0.43	0.13	0.373	0.259	0.38975	
95% confidence interval	0.052	0.0987	0.46	3.43	0.86	12.48	8.86	6.095	6.54	9.51	6.626	1667.73	
99% confidence interval	0.087	0.1638	0.76	5.68	1.43	19.57	13.90	9.56	10.25	14.90	10.39	2615.44	
Skewness	1.561	1.983	2.23	-0.67	0.124	2.443	-1.28	-0.602	0.975	-0.16	-0.35	1.712	
Kurtosis	2.102	4.078	4.99	-1.55	1.25	5.977	2.06	0.007	-1.20	-1.23	1.701	3.523	

Table 3. Geotechnical properties of the subgrades

Sample Id	Wn (%)	G_s	$\rho_{b(wet)}$ (g/cm^3)	$\rho_{b(dry)}$ (g/cm^3)	γ_{dry} (KN/m^3)	γ_{wet} (KN/m^3)	Porosity	Void ratio	Specific surface (cm^2/cm^3)	K cm/sec
BH1`DS1,1m	18.65	2.65	1.353	1.151	11.29	13.27	0.57	1.3	2258.09	2.95×10^{-3}
BH1DS1,4m	18.42	2.68	1.453	1.209	11.86	14.25	0.45	1.22	620.75	1.17×10^{-2}
BH2,DS2,1m	22.32	2.62	1.573	1.287	12.62	15.43	0.49	1.04	1822.21	2.04×10^{-3}
BH2DS2,4m	26.50	2.62	1.678	1.325	12.99	16.46	0.51	0.98	2456.00	1.37×10^{-3}
BH4,DS3,1m	21.20	2.61	1.391	1.149	11.27	13.64	0.44	1.27	1807.98	1.25×10^{-3}
BH4,DS3,4m	19.55	2.65	1.457	1.210	11.87	14.29	0.46	1.19	1321.06	2.87×10^{-3}
Minimum	18.42	2.61	1.353	1.149	11.27	13.27	0.44	0.98	620.75	1.25×10^{-3}
Maximum	26.5	2.68	1.678	1.325	12.99	16.46	0.57	1.27	2456.0	1.17×10^{-2}
Mean	21.1	2.64	1.484	1.22	11.98	14.56	0.49	1.14	1714.34	3.69×10^{-3}
Std. dev	3.04	0.026	0.121	0.071	0.698	1.187	0.05	0.12	665.84	0.004
Coefficient of variation	0.144	0.01	0.08	0.058	0.058	0.82	0.10	0.11	0.39	1.078
95% confidence interval	3.19	0.028	0.127	0.075	0.733	1.246	0.05	0.15	698.87	0.004
99% confidence interval	5.0	0.043	0.1989	0.118	1.149	1.95	0.079	0.25	1096.01	0.0066
Skewness	1.287	0.66	0.827	0.52	0.51	0.827	1.112	-0.49	-0.811	2.273
Kurtosis	1.464	-0.594	-0.195	-1.286	-1.292	- 0.2	0.815	-2.23	0.319	5.332

Table 4. Slope Stability Analysis Data

Slope design section	C_u (kN/m^2)	ϕ^0	Slope Height (m)	γ_{dry} (kN/m^3)	γ_{sat} (kN/m^3)	C'$_m$ (kN/m^2)	R (m)	β^0	Θ_c^0	Tension crack depth (Z_0) in (m)		F_s	Over-design factor	Slope stability status	Remark
										Measured	Calculated				
1	36	0	2.12	14.4	17.23	11.61	21	32	20	0.6	5.0	1.0	0.91	Limiting equilibrium	Stabilization Required
2	40	0	2.11	15.6	17.60	12.40	7.30	32	19	0.4	5.13	0.9	0.80	Failed	Stabilization Required
3	34	0	2.2	14.7	17.20	12.12	7.4	32	1	0.5	4.63	1.0	0.40	Limiting equilibrium	Stabilization Required

Table 5. Soil Strength and Bearing Capacity Data

Sample Id	Undrained Cohesion (C_u) (kN/m^2)	Undrained friction angle ($\phi_u{}^0$)	Dry Unit weight (γ_{dry}) (kN/m^3)	Ultimate bearing capacity (Q_{ult})	Factor of Safety (F_s)	Allowable bearing capacity (Q_{all})	Recommended Foundation Type
Design Section 1 (BH1,1m)	35	8	14.7	371.2	3	123.7	Rectangular
Design Section 1 (Bh1,2m)	34	9	15.6	424.30	3	141.43	Rectangular
Design Section 2 (BH2,1m)	36	8	14.4	380.1	3	126.7	Rectangular
Design Section 2 (BH2,2m)	40	6	14.9	401.8	3	133.9	Rectangular
Design Section 3 (BH4,1m)	40	7	14.9	396.1	3	132.0	Rectangular
Design Section 3 (BH4,2m)	34	6	14.7	348.44	3	116.2	Rectangular
Minimum	34	6	14.4	348.44		116.2	
Maximum	40	9	15.6	424.3		141.43	
Mean	36.5	7.33	14.86	386.99		128.99	
Std. dev	2.81	1.21	0.403	26.37		8.77	
Coefficient of variation	0.077	0.165	0.027	0.068		0.068	
95% confidence interval	2.95	1.27	0.42	27.68		9.21	
99% confidence interval	4.63	1.99	0.66	43.41		14.43	
Skewness	0.689	0.075	1.297	-0.09		-0.08	
Kurtosis	-1.95	-1.55	2.696	-0.08		-0.08	

W_n = moisture content, G_s =, Specific gravity, Wet Density. $\rho_{b(wet)}$ (g/cm^3), Dry Density $\rho_{b(dry)}$ (g/cm^3), Dry Unit Weight, γ_{dry} (kN/m^3), Wet unit weight γ_{wet} (kN/m^3), Porosity, Void Ratio, Specific Surface (cm^2/cm^3), Coefficient of permeability, K (cm/sec)

5. Conclusion

The investigation and pre-analysis which guides in choosing slope stabilization for which a cantilevered reinforced retaining wall has been proposed reveals the geo-environmental risk of any human occupation on the land. Probable occurrence of failure and potential sliding shall be prevented by the incorporation of the results into design and construction of the retaining walls and implementation of the foundation recommendations into the proportioning of the footings.

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