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GEOTECHNICAL SITE INVESTIGATION OF SIGIRI, LOWER NZOIA FLOODPLAIN, WESTERN KENYA

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ABSTRACT

Geotechnical investigations were carried out to establish the bearing capacity of the geological materials that occur at the site. The investigations involved an analysis of the properties of the materials in the area using soil mass characterization, river depth analysis, geotechnical testing and slope stability analysis. The investigations identified several key stability issues such as the depth to the firm foundation, the bearing capacity of the foundation rock, liability to liquefaction and the stability of the slopes.

Keywords: floodplain, construction site, bearing capacity, foundation rock, geotechnical investigations

1. INTRODUCTION

Sigiri in Western Province is one of the areas most frequently devastated by floods in Kenya. It occurs at the lower course floodplain of River Nzoia, where the floodwaters spread over a length of more than 10 km forming swamps that enjoin those of River Yala to the south. There is an increasing need to build a bridge across River Nzoia at Sigiri to promote better communication and trade within the region, hence, the carrying out of the geotechnical studies. These studies, used for characterising the properties of a construction site, were important in establishing the foundation requirements for the future building of the bridge and improving the road network.

The geotechnical results revealed a weathered foundation on the future northern abutment and a firm foundation on the intermediate pier. The firm strata were found to be capable of accommodating stresses of up to 240 kN/m^2 at depths between 9.75m to 13.75m or between 15m and 19m. The foundation of the southern pier and abutment was found to be composed mainly of Holocene sediments. Strength test results indicated a safe bearing capacity of 100 kN/m^2 at a depth of 14m and this indicates the possibility of differential settlement should the foundation be set at a depth less than 18m without ground treatment. Some materials were established to be having N-values of 20 or less indicating susceptibility to liquefaction.

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Liability to rapid consolidation and settlement was noted in most soil materials. Most slopes in the area are unstable.

River Nzoia is one of the longest rivers that flow through western Kenya into Lake Victoria. At Sigiri, the width of the river at the floodplain is approximately 40m. The Sigiri Bridge will be designed as a span bridge with a length of approximately 100m. The northern site of the bridge will lie within an area underlain by sediments of the Kavirondo Super group and intruded by volcaniclasites. The southern site of the bridge will lie within an area underlain by Holocene sediments. The geology of the site includes reddish-brown clays, highly weathered trachytic phonolite, and fresh- gray phonolites, medium to coarse-grained sand and boulders. The sedimentary bedrock units exposed on the hills adjacent to the site dip approximately 70° towards the river.

Unfortunately many of the geotechnical properties in this floodplain are still poorly understood. No geotechnical advances have been made in the past. Perhaps the least understood geotechnical property of this part of the floodplain is the settlement/ consolidation due to building of structures, particularly how this property can be related to the likely surface area to be affected. Estimates of such parameters would be of great scientific interest in establishing the ability of the ground to support the structures. In this study, we present results of geotechnical site investigation at Sigiri for construction of a bridge and adjoining roads.

2. MATERIALS AND METHODS

The sampling sites comprised of the area at which a bridge is proposed at Sigiri, covering the lengths to which the road will be improved since the bridge will be constructed on a floodplain where the water in the river bursts its banks. The bridge will be raised to provide sufficient slope for gravity drainage and also to avoid flooding of the bridge. Preliminary investigations involved measurement of the depth of the river as the site. The northern side of the river was found to be shallower that the southern side. Stability of artificial slopes created during construction of the dykes was analysed. Plate testing was done up to two kilometres away from the southern bank Investigations at the bridge site involved sinking of three boreholes by rotary drilling with core recovery. The boreholes were drilled to facilitate the understanding of soil stratification and to identify the depth to firm strata. The borehole positions were identified and clearly marked on the ground and they were sunk near the piers and abutment for the proposed bridge. Drilling was carried out according to British standards, BS 5930: Code of Practice for Site Investigation [2]. Two boreholes were drilled to depths of 21.50m and 23.50m for the northern site and one borehole was drilled to a depth of 27m on the southern site since no intact rock was encountered.

Material recovered from boreholes were properly identified and stored in wooden core boxes. A detailed log of material recovered, along with in situ tests, was carried out after which all samples were photographed in colour. Any physical properties for instance strength of the core after recovery was noted and natural features such as joints were distinguished from those produced either by drilling or by changes that have occurred later e.g. by stress relief or desiccation.

Laboratory testing was done on selected undisturbed and disturbed samples to establish the bearing capacity of the firm strata and other overlying soil layers. Suitability of the ground for supporting the road was investigated by particle size analysis, establishment of Atterberg limits, plasticity index, linear shrinkage and free swell volume. Soils were tested for strength by shear-box and triaxial methods. Oedometer method was used to determine the consolidation /settlement characteristics of soil materials The firm strata was tested for bearing capacity by Point Load and Unconfined Compressive Strength tests [6,9,11]. In the final stages, all sections were compiled to guide in geotechnical evaluation of the site investigated as regard foundation excavation and ground treatment.

3. RESULTS AND DISCUSSION

Important geologic features identified during investigation include shear zones, high permeability fracture zones and weak clay beds of high plasticity. Localized open fissures were encountered on the upper portion of the northern pier. Highly weathered zones were encountered throughout the entire length of both abutments. High permeability fracture zones were identified in the saturated, highly weathered trachytic phonolite in the in the valley foundation of both abutments. These fracture zones represent areas of settlement for the foundation. The fracture zone could transmit pore pressure to the underlying soils. Clays were identified underlying the proposed southern abutment.

The strength of the soils was established from the values of C and ϕ obtained from direct shear and triaxial tests. These results were complimented with those of standard penetration test and are important in the design of earthworks. The C and ϕ values were used in the analysis of the long term stability of the soils as regards their ability to support foundation structures. Ultimate and safe bearing capacities were determined, with the following assumptions

Type of footing Pile

Size of footing 1.5 m by 1.5 m

For borehole 1, the design parameters are

Unit weight $\gamma = 14.59$ kN/ m^2

Depth of sample 2 m

Apparent cohesion, C = 58 kN/ m^2

Friction angle $\phi = 17^{\circ}$

Thus, Nc = 15

Nq = 5

Nγ = 2.5

As based on Terzaghi's bearing capacity coefficients.

The ultimate bearing capacity is given by

$$q_{nf}=(1.3*58*15)+(0.4*14.59*1.5*2.5)+(14.59*2*5)$$
$$=1298.8 \text{ kN/ m}^2$$

In order to safeguard against shear failure irrespective of any settlement a factor of safety of 3 was adopted

$q_{safe} = 1298/3$
$= 433\ kN/m^2$

Values of safe bearing capacity of the various samples tested are summarised in the (Table 1).

Table 1. Values of safe bearing capacity of the various samples tested

B/H No.	Depth (M)	Bearing capacity value (kN/m^2)	
		Triaxial	SPT
1	2	433	151
2	14	130	184
3	8	321	918
3	14	221	162
3	18	105	-

In the oedometer test, the coefficients of consolidation and compressibility were established. These coefficients were used to classify soils into various groups according to the degree of compressibility /consolidation and plasticity. The coefficient of consolidation was used in estimating the rate at which consolidation of the soils would proceed when subjected to external loads and the probable duration of associated settlement.

The degree of compressibility of each soil sample was compared with the British Standards volume compressibility table that classifies into various groups outlined by Lambe and Whiman [1,2,3]. These tables consist of standard ranges of values of coefficient of consolidation and compression index for soils and describe soils according to plasticity and plasticity index range. The soils were classified into on of the following three group, low consolidation, $C_v < 10$ mm/year rapid consolidation, $C_v = 10$ to 100 mm/year, very rapid consolidation $C_v > 100$ mm/year. The values of the rate of settlement in millimetres per year for the samples tested from the three boreholes (Table 2)

Table 2. Values of the rate of settlement in millimetres per year

B/H No.	Depth (m)	Rate of settlement (mm)	Probable rate of consolidation
1	2	179	Very rapid
2	14	39	Rapid
3	8	165	Very rapid
3	14	67	rapid
3	18	65	rapid

The expected maximum settlements are calculated by assuming a simplified triangular stress distribution beneath the foundation [10]

Thus settlement $S_{oed} = M_v*0.55q_n*1.5B$

Where,

M_v = average coefficient of volume compressibility = 3.25* 10E-4m^{2}(BH3)

q_n = net foundation pressure

B = width of foundation (pile) = 1.5m

Therefore,

S_{oed} =3.25* 10E-4*0.55*433*1.5*1.5*100

=179 mm

From the core logs, it was established that most of the soils under the proposed bridge site are cohesionless, leading to a poor total core recovery. In some cases the percentage of core material recovered was as low as 2% and the soils were poorly graded. The fact that caving of borehole materials from the upper layers occasionally occurred made the interpretation of the strata very complex. At this point it should not be assumed that the loose materials observed in the core boxes strictly came from the depths indicated.

Using the test results in the table below, the strength of the rocks and the bearing capacity of the foundation rock was calculated as shown below using Goodman's formula [8]

$$q_f = q_u (N\phi + 1)$$

where

q_f = Ultimate bearing capacity in kN/m^2

q_u = Unconfined compressive stress value in kN/m^2

N_ϕ = constant given by $\tan^2 (45 + \phi/2)$

where ϕ is the angle of breaking for a particular rock type.

Results of UCS test of the sample recovered from borehole 1 at a depth of 9.7m which gives the lowest stress of 9,700 kN/m^2 were used. An assumption of internal angle of failure of rock was made as equal to 30˚ based on the mode of failure of the rock.

Therefore,

$$q_u = 9{,}700 \text{ kN/m}^2$$

$$N_\phi = 3$$

$$\phi = 30^\circ$$

Hence,

Ultimate bearing capacity, q_f = 9,700(3+1)

= 38,800 kN/ m^2

Up to this depth the fracture index could be regarded as infinite and hence a Factor of

Safety (FoS) of 40 was applied to give q_f = 970 kN/ m^2. This factor takes care of the weariness of the rocks beyond the considered depth. To obtain the Safe Bearing Capacity, a factor of three was adopted which gives

$$q_{safe} = 320 \text{ kN/ } m^2$$

The conservative factor of 40 used due to fractures takes care of any settlements that may arise (both immediate and long term).

For the Point Load test, the lowest rock strength index of 300 obtained from borehole one at a depth of 10m was used to calculate the compressive stress using the formula

Thus,

Compressive stress, $q_u = 24^* I_{50}$

$$q_u = 24^*300 = 7200 \text{ kN/ } m^2$$

In order to obtain the Ultimate Bearing Capacity, a Factor of Safety of 10 was used for weathering and fractures,

Hence, q_f= 720 kN/ m^2

A factor of three was used to obtain the Safe Bearing Capacity [4].

$$q_{safe} = 240 \text{ kN/ } m^2$$

4. CONCLUSIONS AND RECOMMENDATIONS

The river channel was found to be deepest on the southern part. Since river valleys and other depressions of the earth do not generate fortuitously but especially because of presence of weak rocks it is postulated that the river valley follows a direction of a fault at Sigiri Bridge site. Because of the step-like shape of the valley, it is indicated that the principle fault was accompanied by several subsidiary faults

Ground water level was carefully investigated with special attention paid to wetter periods. Loss of cohesion in soils for instance in the clays to the east of Nakhasiongo Hill results from infiltration of rainwater into surface layers. The normal stress, σ is small near the surface and thus only a section of the strength curve in the Mohr diagram is valid, where the curve bends down towards the coordinate origin. This section of Mohr curve displays a very small magnitude of C but ϕ =40˚~45˚. Considering the statical loading by penetrating water, the safe angle of slope should be less than 20˚. Steeper slopes deform, suffer from cracks which are penetrated by water and the surface layer collapses.

Natural and artificial slopes in Port Victoria are very stable because the soils and rocks in the area are very dry. Cut slopes in the clays to the east of Nakhasiongo hill are very unstable due to the fact that they were cut at very high angles during the construction of the adjacent dyke. A very unsuitable situation arose during the construction of the dykes in the area. The subgrade of the route was situated lower than the surrounding ground surface, so that the necessary gradient of drainage was unavailable. A roof -like reverse gradient was created to obtain sufficient drainage of the fill slopes. However in some areas, this drainage is so insufficient such that the dykes have been cut through by the violent floodwaters.

The other cause of slope failure in the fissured clay to the East Nakhasiongo Hill is that the fissures open because of stress relief or desiccation, so that water can penetrate the soil. Groundwater present in the slope decreases the effective stress between the clay particles while the tangential forces do not diminish. The effective stress decreases because of the presence of water molecules between the soil particles, the tangential forces do not diminish because of existence of a free surface of the slope towards which the face materials can collapse.

The nature and position of bedrock was established on the northern side of the proposed bridge. The character of the bedrock varied from borehole to borehole and it was therefore not possible to draw a cross-section of the bridge site. If the northern pier is situated out of reach of faults, difficulties connected with the deepening and extension of foundation will be evaded. Also, other apparently secondary problems may arise from poor geological conditions

No rock material was encountered on the site for the southern pier and abutment even after the borehole was drilled beyond the postulated 20m depth. The soils encountered in this borehole were either loose or cohesionless or of high plasticity and this indicated the possibility of differential settlement should the foundation of the bridge be set on them. The river valley was also found to be deepened at this site, and is about 20m deep.

The rock materials encountered in the boreholes at the site for the northern abutment showed shearing and fracturing. In some cases, an intact rock was encountered and just below it, a weak soil showing that the rock sample could be representing a boulder transported from the hills. The rock quality designation for the northern pier satisfies the ISRM standards [5,6,7] for supporting a foundation though the possibility of another sheared zone existing beyond the drilled depth cannot be ignored.

To minimise uneven settlement on both abutments, it is recommended that compaction be done from below the base of the southern abutment leaving 300mm foundation level which should be filled with a proper mix of concrete keyed to the abutment to form an apron at least 1m off the wall surface. For the northern pier, if the foundation is set above the highly weathered depth below sound rock, concrete of suitable mix should be injected into the rock fissures around the foundation level [3].

The design bearing pressure should be limited to a maximum value slightly below the lowest safe bearing capacity (240 kN/m^2) obtained by the point load test. A safe bearing capacity of 200kN/m^2 can be used for design purposes. The cost effects considered will determine whether piling has to be done on the northern side on excavation

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