DESIGN PRACTICE OF BORED PILES IN NUBIAN FORMATION CASE STUDY: FOUNDATION AND BRIDGES IN KHARTOUM

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مُسْـتَخْـلَص

يستخدم مصطلح تكوين الحجر الرملي النوبي في السودان لمجموعة من سحنات الصخور الرسوبية الافقية التطبق والتي تتكون من الكونجلوميرات (الرصيص)، الجريت (الحصباء)، الحجر الرملي، الحجر الطيني الرملي والحجر الطيني. تتوضع هذه التشكلات لا توافقياً مع صخور الأساس وصخور الحجر الرملي من الحقبة القديمة. يغطي تكوين الحجر الرملي النوبي مناطق واسعة من شمال، وسط غرب وشرق السودان والذي عادة ما تتكشف سحناته علي السطح أو تكون مغطاة بطبقة رسوبية من تكوينات الحقبة الحديثة أو الرباعية. لقد مثلت هذه التشكلات بولاية الخرطوم قاعدة إنشائية للعديد من البنيات الثقيلة كالجسور النهرية والبنايات الشاهقة. تعتبر المواد اللاحمة للحبيبات ودرجة تجانس وتجوية الحجر الرملي إضافة إلى درجة قوام الصخور الطينية وخواص التكوينات الأخرى التي تعلوها من العوامل الأساسية التي تتحكم في مستوي الأداء الهندسي لهذه الصخور.

تلخص هذه الورقة الخصائص الجيوتقنية لمواقع خمسة كباري على روافد النيل في الخرطوم والطرق التي استخدمت لتقدير الأعمال التصميمية للإسفينيات الداعمة لأعمال هذه الكباري علما بأنها تخترق الحجر. تمت مقارنة الأعمال التصميمية بنتائج اختبارات التحميل للإسفينيات في هذه المواقع وأوضحت المقارنة أن تقديرات الأعمال التصميمية في الحجر الرملي تعتبر غير واقعية ومحافظة مما يتطلب طرق مختلفة لتقدير الأعمال التصميمية أو الطرق المتبعة لتصميم الإسفينيات.

Abstract

The term Nubian Sandstone Formation or Nubian Formation (NF) is applied in the Sudan to those bedded and usually flat-lying conglomerates, grits, sandstones, sandy mudstones and mudstones that rest unconformably on the Basement Complex and the Paleozoic sandstones. These formations are either exposed or covered by the recent quaternary formations. They cover large areas in Northern, Central Western and Eastern Sudan. Several important heavy structures such as bridges across the rivers and high rise buildings in Khartoum are supported on these formations. The factors controlling their performance as foundation materials are the type and amount of cementing material for the sandstones; the heterogeneity, degree of weathering and the type and consistency of the mudstones and the characteristics of the formations covering them. Different approaches were used for the design of piles resting or penetrating NF.

This paper summarizes the geotechnical characteristics of five bridge sites in Khartoum and the approaches used by the designers for estimating the bearing capacity of the piles supporting these bridges. The piles were all socketed into the NF. The designs were compared with the results from pile load tests carried out in the bridge sites. The analysis has shown that the approaches used for estimating the pile capacities in the NF are very conservative and un-realistic. Alternative design approaches or improvements of the currently used designs are needed.

Keywords: *bored pile, Nubian formation, bridges, Khartoum state*

1 INTRODUCTION

The term Nubian Sandstone Formation or Nubian Formation (NF) stands for cretaceous continental sediments composed of mudstones, sandstone, conglomerate and some laterites [1]. These formations cover about 45% of the surface area of Sudan and extend from latitude 22^o North southwards to latitude 10^o south, i.e. the southern borders of Sudan Figure 1. The northwestern desert is wholly covered by these formations. The NF is also found in Gedarif-Shawak region in eastern Sudan Figure 1. The deposition of sediments forming NF was by a system of braided channels in purely continental environment except for Gedarif-Shawak where the deposition was reported to be fluvial in littoral environment [2]. These formations are generally covered with recent quaternary deposits of variable depth.

Numerous large projects which constitute bridges, high rise buildings, industrial developments and dams have been built on NF. In Greater Khartoum almost all the bridges across the Niles, connecting the city, are resting on NF deposits. The dam Complex of Upper Atbara (DCUA) project in Shawak area in eastern Sudan is built on NF. The relatively intact sandstone has been also used as building material since the Colonial times (the 18th century).

The NF in Khartoum city is encountered at depths 7 m to 25 m, whereas the NF is almost exposed on the ground in parts of Omdurman city. A contour map for the depths of the NF in Khartoum is given in Figure 2 [3]. The bridges and high rise buildings in greater Khartoum are founded on bored pile foundation of various lengths. The piles have been designed to rest on or be socketed in the NF.

This paper reviews the geotechnical conditions of five bridge sites in Greater Khartoum Figure 3, discusses the methods used for estimation of the carrying capacity of the piles used to support the bridges and evaluates these capacities in comparison with the pile load tests.

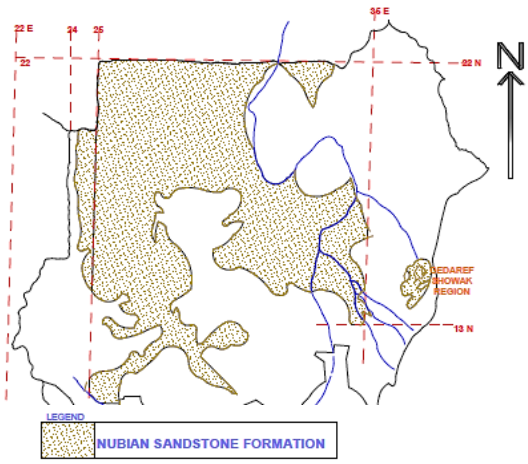

Figure 1: The distribution of NF in Sudan

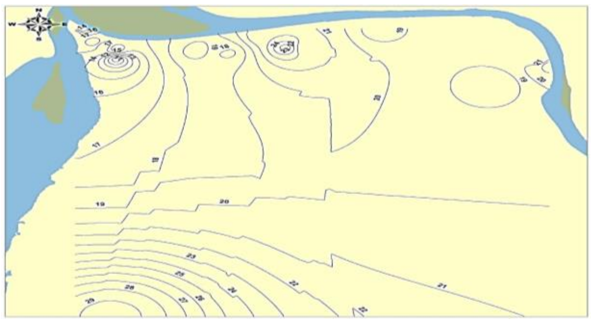

Figure 2: Contour map of NF depths in Khartoum

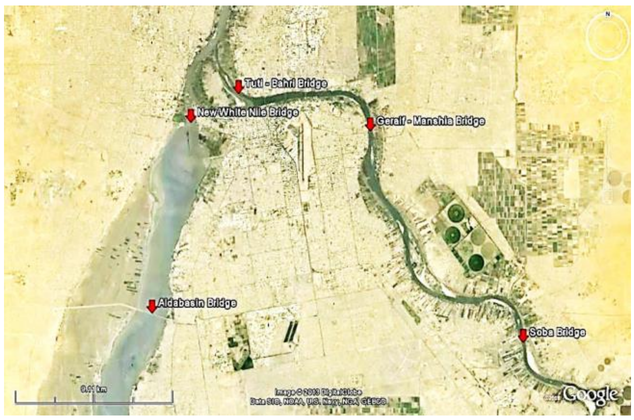

Figure 3: Locations of the studied bridge sites

2 GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS OF THE NUBIAN SANDSTONE FORMATION

Yousif [1] carried out an intensive study on the geotechnical properties of the NF, mainly the sandstones and mudstones of Khartoum and Shawak areas. The main geological and geotechnical outcomes of significance to their performance as foundation support formations is summarized here-under:

i) The NSF is jointed, cracked and suffer facies changes and strength characteristics of weak rocks. The mudstones and sandstones occur in beds which are generally horizontal with slight dipping towards the N and NW directions. The NF rests uncomfortably on the basement complex with thickness up to 150 m. Intrusion of volcanic basaltic rocks “dikes” have been observed in Shawak area.

ii) The sandstone is principally composed of angular to sub-angular quartz grains of variable sizes floating in a matrix of fine material. It is characterized by color variations and the cementing material is basically kaolin. The strength of the sandstone is controlled by the fines content. Friction angle varies with grain size distribution. Strength is adversely affected by wetting. Shear planes are not affected by the presence of laminations.

iii) The argillaceous sandstone is weak. Gradation is skewed towards the coarse side, grain to grain contact is rare and the grains are surrounded by fine cementing matrix.

iv) The mudstone is blocky, massive, fissile or laminated and composed of clay and silt size particles showing remarkable color variations. The clay is basically kaolin with very small amounts of smectite and calcite. it is weak to very weak, highly over-consolidated with low potential for swelling. It shows brittle to ductile failure and the strength is adversely affected by water or wetting. Linear shrinkage and plasticity index are small.

v) There are no signs of holes, cavities or solution channel in the NSF.

3 THE PROFILE DESCRIPTION FOR STUDIED BRIDGE SITES

Data have been collected for five bridge sites in Greater Khartoum on the Blue and White Niles Figure 3. Two of the studied bridges have been constructed during the last three decades and the other three are under construction nowadays.

The bridges on the Blue Nile are Geraif-Manshia bridge, Tuti-Bahri bridge and Soba bridge whereas the White Nile and Aldabasin bridges are on the White Nile. The bridges were all founded on groups of bored concrete piles socketed in the NF. The bridges were all built by Foreign Contractors. The geotechnical investigations were carried out at the bridge sites by local geotechnical firms. Recommendations for pile capacities were presented in the geotechnical reports. The structural designs were all carried out by the civil contractors through design and build contract type.

The investigations comprised drilling several boreholes in the centerline of each bridge using auger and rotary drilling; continuous flight auger in the upper alluvium and wash boring or rock coring in the NF. The strength of the NF was evaluated either by the Standard Penetration Test (SPT) N-values or the unconfined compression test on core specimens and rarely with pressure meter. SPT-N values were reported as N>50 or N>101. The latter was reported when the number of blows needed to penetrate the first 150 mm is greater than 100.

The geotechnical conditions of the bridge sites will be described here-under with emphasis given to the characteristics of the NF whether it is sandstone, mudstone or conglomerate.

3.1 Geraif- Manshia Bridge

A profile description of the geotechnical conditions along the axis of Manshia bridge is given in Figure 4. Six boreholes were drilled in the centerline of the bridge, two on-shore and four off-shore. Thick layer of medium dense to very dense silty sand (SM) or poorly graded sand (SP) was encountered on the left and right shores down to 21.0 m depth. The alluvium deposit on-shore is underlain with moderately weathered conglomerate (about 2.0-3.0 m thickness) overlying highly weathered mudstone on the left bank and sandstone on the right bank.

The formations below the river bed showed alluvium deposits of cohesionless nature (SM/SP) underlain with Nubian Formation. The NF was in the form of interchanging layers of highly weathered mudstone and/or sandstone. The mudstone or siltstone, found in pockets, is basically non-plastic to low-plastic whereas the fine material is predominantly silt. The sandstone is dominant below the river bed.

3.2 Soba Bridge

This bridge is located on the Blue Nile few kilometer up-stream of Manshia bridge site. The bridge is under construction and less than 20% of the substructure has been completed.

The layout of the encountered formations is similar to Manshia bridge showing an upper layer of quaternary alluvial deposits underlain with the NF. The Nubian Formation at this site is found at depths greater than 20m on the banks and 12m under the river bed level see Figure 5. The formation is basically highly weathered sandstone (SC/SM and SP) with thin pockets of mudstone detected in Boreholes 1, 5 and 8. Conglomerate was encountered on the top of the NF in borehole 8 for a single core run (2.0 m). The encountered formations were penetrated using normal drilling in most cases.

3.3 The New White Nile Bridge

Observation of the ground profile section, depicted in Figure 6, suggests three distinct subsurface zones. The first consists of highly plastic silt or silty clay covering about 5.0 m to 7.0 m below ground surface underlain with a transitional zone of stiff sandy clay or dense clayey sand (2.0 -4.0 m thick). The third zone is the Nubian Formation which consists of inter-bedded layers of highly weathered mudstone and sandstone. The mudstone is found in pockets.

3.4 Al Dabasin Bridge

Aldabasin Bridge is located on the White Nile upstream of the New White Nile Bridge linking Khartoum with Omdurman. It is a steel plate girder bridge with composite concrete deck and has a total length of 1670 m .The project is under construction and the substructure has been completed.

A typical geological section for Aldabasin bridge is given in Figure 7. The encountered formations comprise two distinct zones, an upper alluvial deposit and lower Nubian Formation. The upper zone extends to more than 20 m depth and is composed of loose to medium dense sand SP/SM/SC. The NF is composed of interchanging strata of highly weathered bedded sandstone, sandstone intercalated with mudstone and mudstone. Some core runs showed interchanging thin layers of sandstone and mudstone. Core drilling was used to penetrate the NF. The recovery was poor and RQD was very low to low. However, the quality of the NF looks better than the quality at the other bridge sites.

3.5 Tuti-Bahri Bridge

The bridge is under construction nowadays and the pile foundations on Bahri side have already been constructed. The geotechnical investigation comprised drilling four boreholes in this site, in the abutment and pylon locations, since the river course is relatively narrow. The boreholes were carried down to 50 m depth Figure 8. The Nubian Formation was encountered below the quaternary alluvial deposits. The NF was encountered at depths 18-20 m on-shore and 10.0-12.0 m off-shore. It is formed of alternating layers of highly weathered mudstones and sandstones with poor RQD and core recovery.

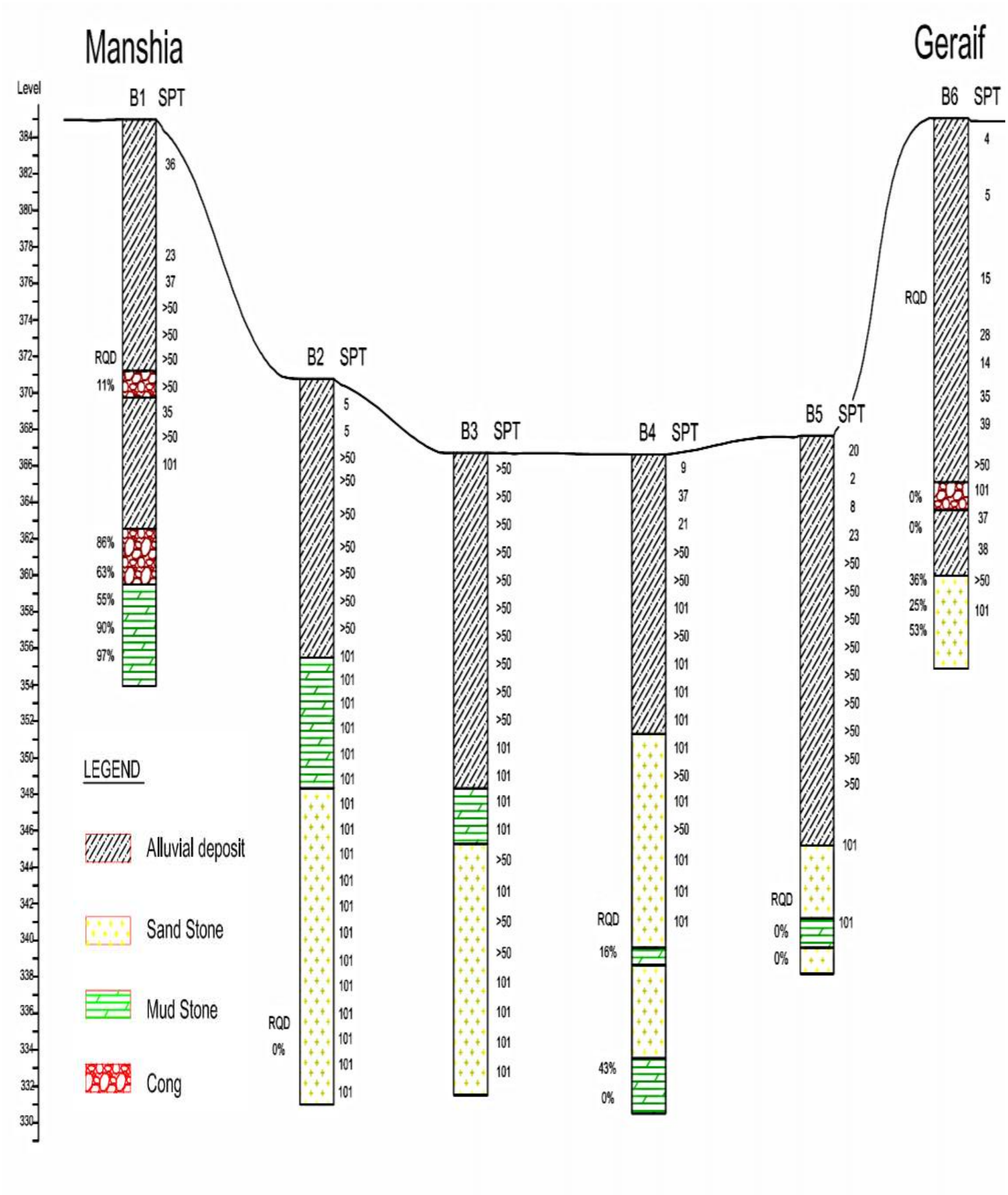

Figure 4: Subsurface profile of Geraif- Manshia Bridge site

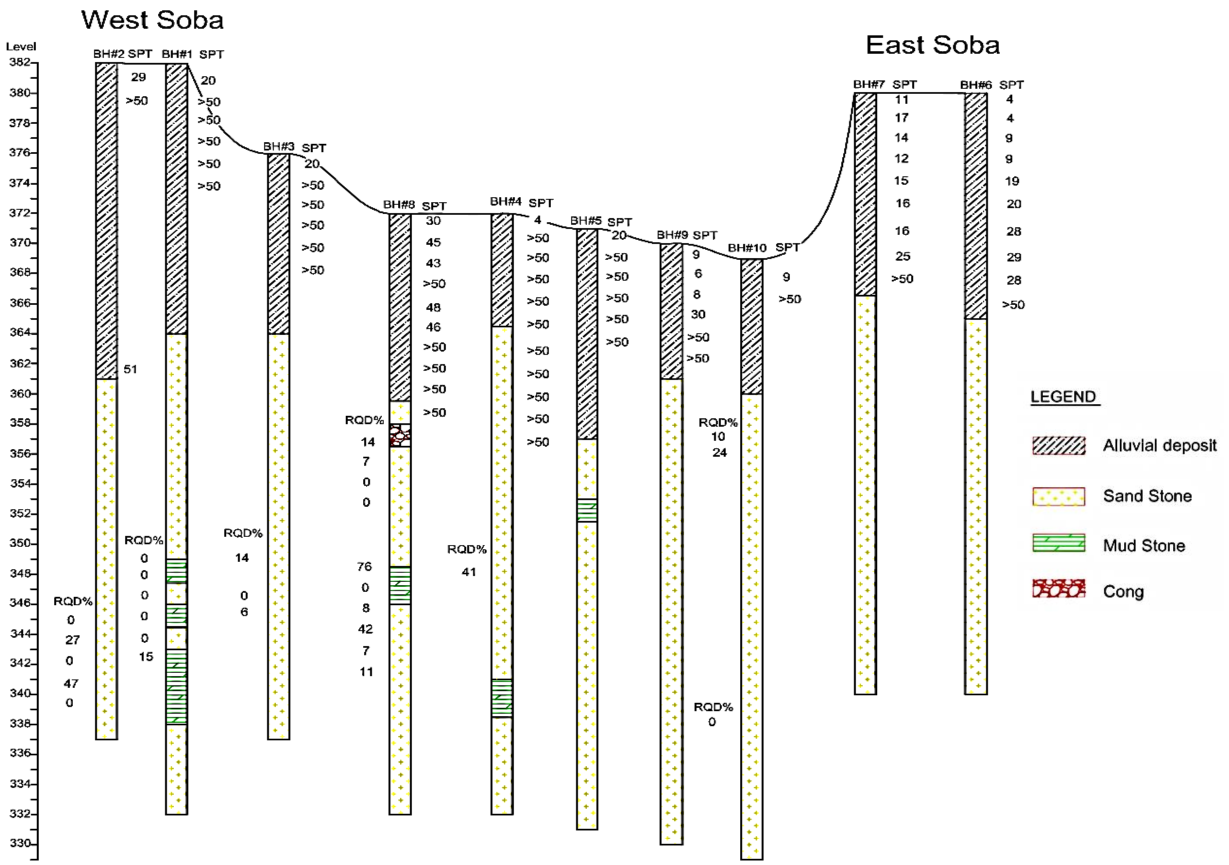

Figure 5: Subsurface profile of soba bridge site

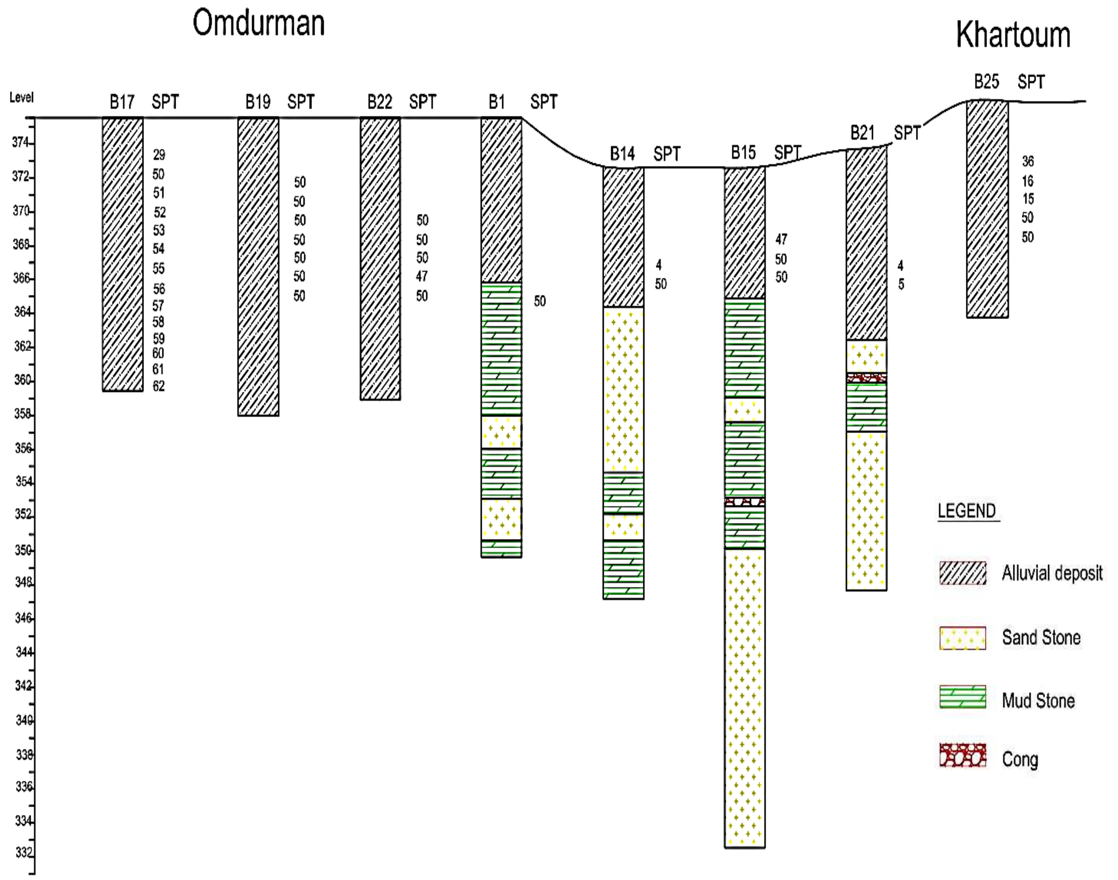

Figure 6: Subsurface profile of New White Nile bridge site

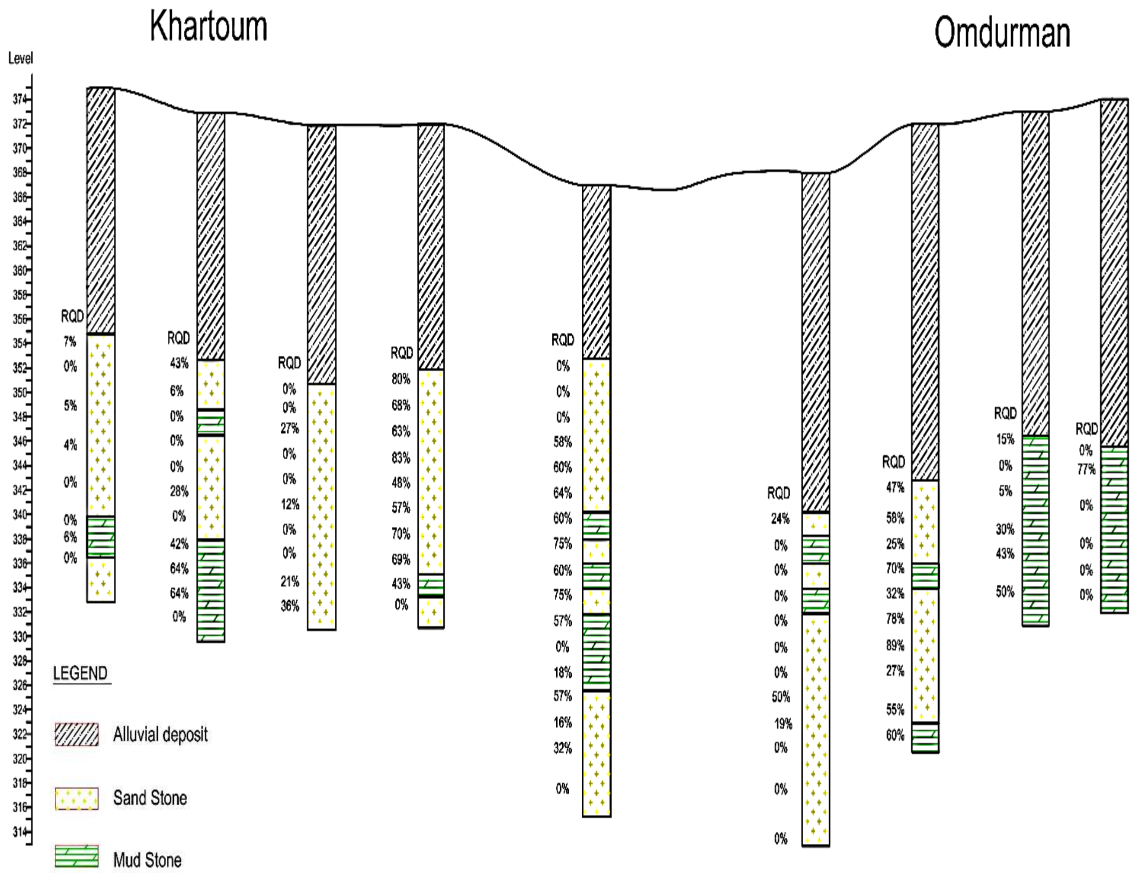

Figure 7: Subsurface profile of Al dabasin bridge site

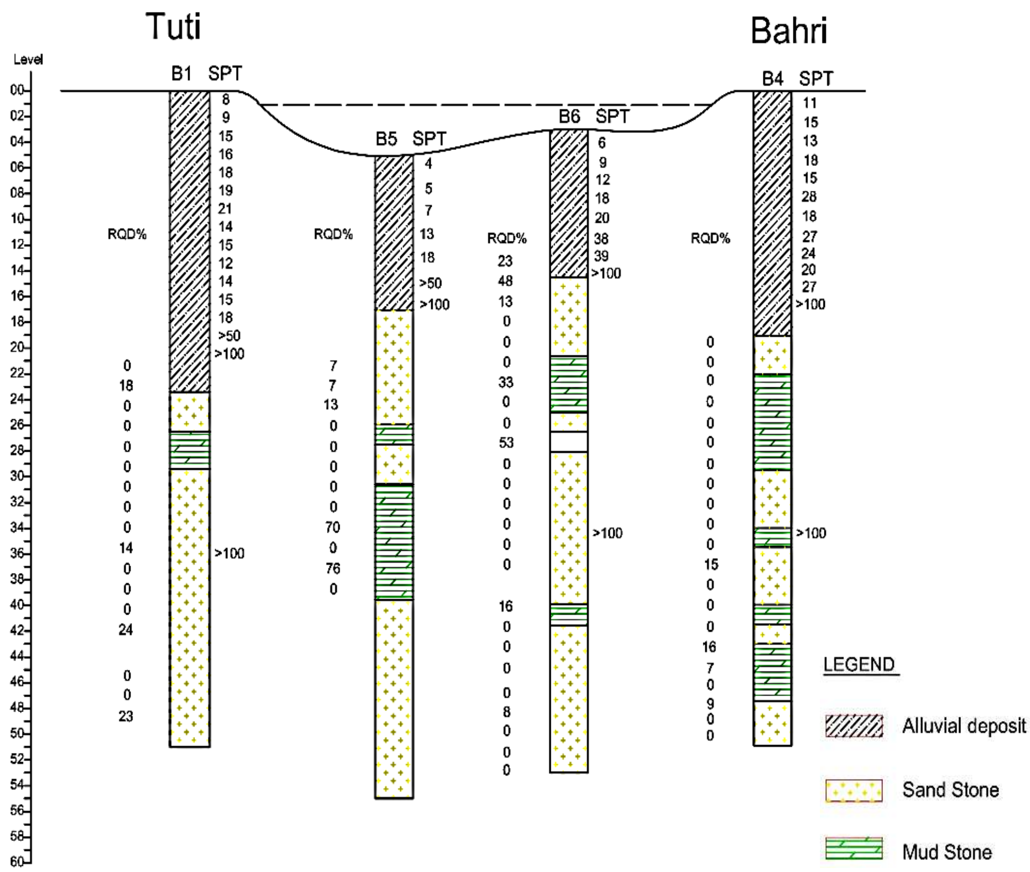

Figure 8: Subsurface profile of Tuti-Bahri bridge site

4 THE GEOTECHNICAL CHARACTERISTICS OF THE NUBIAN FORMATION AT THE BRIDGE SITES

The geotechnical investigations carried out for the bridge sites have shown that the NF is located below the upper quaternary alluvial deposits at variable depths. The following are the basic engineering characteristics of the Nubian Formation on which the foundations of the bridges are resting:

- The formations are basically sandstones with inter-bedded layers or pockets of mudstones and rare occurrence of conglomerate. The formation is highly weathered and could be penetrated with normal auger/rotary drilling, when weakly cemented. Core samples were also obtainable wherever the deposits are intact and hard.
- The formation is not uniform in terms of rock-type and quality. Good examples are the stratification in the New White Nile bridge (Boreholes 1 &15, Figure 6) and Geraif-Manshia bridge sites (Boreholes 4 & 6, Figure 4), Aldabasin bridge (all borehole) and Tuti-Bahri bridge (Boreholes 4 and 6, Figure 8).
- The sandstone has poor core recovery and rock quality, RQD. The sand grains are medium to fine. The cementing material is mostly kaolin but some thin layers of hard crust where the cementing material was reported as iron oxide and silica were reported.
- The mudstone is predominantly non-plastic ML/CL to low plastic CL and is rarely highly plastic CH (exception is Tuti-Bahri). The very low RQD indicates poor core recovery and rock quality. The fines content of the mudstone is mostly very high.
- The strength of the weathered Nubian Formation was measured either by the SPT test (N value), the unconfined compressive strength of the retrieved samples or the pressuremeter- test (E modulus). The SPT test was stopped when the penetrated value was higher than 50 or penetration was not possible. As for the unconfined compressive strength test, the rock quality was low and consequently the specimens taken for UCS testing were generally of length/diameter ratio less than 2; therefore shape correction was applied. The number of pressuremeter tests was rather limited and the test was performed with a test probe of NX size at specified locations.
- The tested sandstone samples measured very low strength in general, exception are the specimens from Aldabasin bridge site. However, high strength values were measured when the cemented material was reported as ferrous oxide and silica (qu =17070 Kpa has been measured in BH AX 60 depth 24.0 m-26.0 m of Aldabasin bridge). The high strength is often coupled with high density.
- The qu values for the sandstones of the five bridge sites range from 263 Kpa to as high as 17000 Kpa. Two levels of strength were observed, the high strength data which corresponds to the specimens cemented by ferrous oxide or mixed with gravel and the low strength values which correspond to specimens cemented with kaolin. A frequency distribution of qu data is given in Figure 9. If data corresponding to these specimens were grouped separately, then the kaolin cemented samples with minimum strength of 263 KN/m^2 and maximum strength 7132 KN/m^2, would have an average strength of 2717 KN/m^2, standard deviation 1732 KN/m^2 and coefficient of variation 64%. It is worth mentioning that, in general, the sandstone specimens from Aldabasin measured high

strength compared to the specimens from the other bridges.

- Few mudstone samples were retrieved and tested for their unconfined strength. The minimum measured qu value was 272 KN/m^2 and the maximum value was 12471 KN/m^2. The average value is 2574 KN/m^2 and the standard deviation is 2122 KN/m^2 for only 10 specimens.
- Extensive data for UCS and water absorption were reported for Aldabasin bridge site. Absorption is generally between 15-20% for most of the tested specimens. Few specimens dissolved during testing.
- Few pressuremeter tests were carried out in-situ at different depth in the sandstones and the mudstones of Soba and Tuti-Bahri using Mennard type NX size pressuremeter. Boring was done by rotary drilling. The elastic modulus (E in Kpa) was reported. The minimum reported E-value for sandstone was 1048 Kpa whereas the maximum value was 13758 Kpa with a median value of 2113 Kpa. Only two tests were carried out on mudstone specimens from Tuti-Bahri site. The reported E-modulus for the mudstone were 204 Kpa and 228 Kpa.

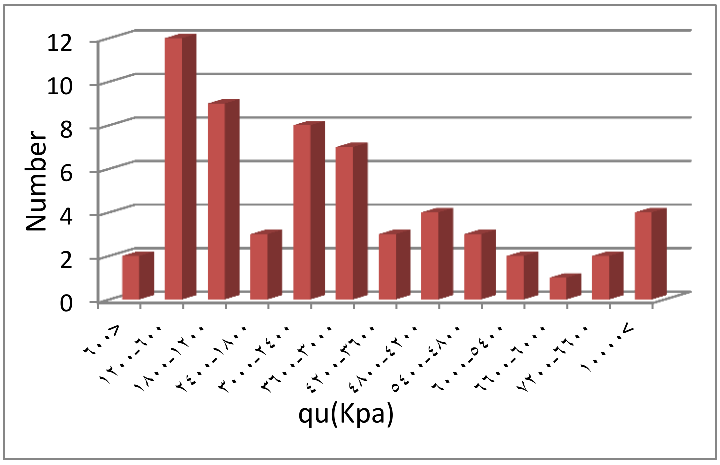

Figure 9: Frequency distribution for qu values of the sandstone tested specimens

5 EVALUATION OF THE DESIGN OF THE PILES

Review of the geotechnical data of the bridge sites has shown that the recommendation of the geotechnical firms, which executed the geotechnical investigations, was to use large diameter bored piles to support the structural loads. Recommendations have been given, in the reports, for the suggested pile sizes (diameters and lengths). The sizes used or adopted do not necessarily comply with the recommended sizes. Pile load tests were carried on the adopted sizes to confirm their carrying capacity. This section reviews the methods by which the geotechnical firms computed or estimated the load carrying capacities for the recommended piles, the obtained values, pile sizes adopted or used for construction in comparison with the results of the pile load tests data.

5.1 Geraif-Manshia Bridge Site

The recommendation in the geotechnical report was to use bored concrete pile groups to support the structural loads. Two pile diameters were selected 1.20 m and 1.50 m “based on the Client’s desire”. The recommendation in the report was to extend the piles into the very dense sand (N > 50) or the NF and the recommended depths were 15.0 m, 21.0m and 27.0 m. The short piles (15.0 m) were recommended in the river course whereas the longer ones (21.0 and 27 m) were recommended on the banks. Idealized or model soil profile was not given. The methods used for estimation of pile capacities were Vesic [4] and Touma and Reese [5]. A factor of safety of 3.0 was used for shaft friction and base resistance; the calculated allowable bearing capacity of the piles is given in Table 1.

Table 1: Calculated pile capacities for Manshia Bridge (from geotechnical report)

Case	Pile Length m	Pile Diameter m	Qu (Vesic) KN	Qa (Vesic) KN	Qu (Toma and Reese) KN	Qa (Toma and Reese)KN	F.S	Pile Location at site	Formation Type
1	15	1.2	21752	7251	22043	7348	3	River Course	V. dense sand
2	15	1.5	66988	1130	27553	9184	3	River Course	V. dense sand
3	21	1.2	30453	10151	22043	7348	3	River Bank	V. dense sand
4	21	1.5	47582	15860	27553	9184	3	River Bank	V. dense sand
5	27	1.2	39154	13051	22158	7386	3	Right Bank	V. dense sand

It is noted from the Table that Vesic method gave higher allowable capacities compared to Touma and Reese and the difference could be more than 100%. It is noted from the design assumptions that consideration was not given to the presence of pockets of mudstone and/or conglomerate in this site.

One pile was load tested close to the location of Borehole 1 on the left bank of the Blue Nile. The pile was 1.2m diameter and 21.5 m long. Referring to Figure 4 (Borehole 1), the test pile penetrated the very dense sand (N>50, depth 13.0m to depth 21.5m) and was resting directly on the NF (sandstone). The pile was load tested to 8000 KN and the test was stopped due to the appearance of hair cracks in the anchor piles. The total measured settlement was 4.3 mm. It appears from the results of the load test that the frictional resistance had not been mobilized on the application of 8000 KN. The very dense sand is offering high frictional resistance along 8.5 m of penetration in it and is acting as an intact socket layer rather than a purely cohesionless material. The unit ultimate frictional resistance of the very dense sand exceeds 100 KN/m^2.

5.2 Soba Bridge

The geotechnical report recommended the use of bored concrete piles to support the structural loads. Idealized ground profile was not given in the report, however, the Nubian Formation was assigned a density of 19 KN/m^3 and a friction angle of 45^o. Recommended pile sizes were 0.6, 0.8, 1.0 and 1.2 m diameters and 30 m length. A software "All Pile" was used by the geotechnical consultant to compute the allowable pile capacities given a factor of safety of 2.5 for shaft friction and 3.0 for base resistance. The recommended allowable bearing sizes and their capacities in KN are thus given in Table 2.

Table 2: Allowable pile capacity of bored piles at Soba site (from geotechnical report)

Pile diameter. (m)	Pile Length (m)	Qa (KN)	Comments
0.6	30	3000	
0.8	30	4900	
1.0	30	7100	
1.2	30	9500	

The structural designer adopted 1.8 m and 1.2 m diameter piles (in groups) to support the structural loads. The small diameter piles will be used on-shore at the abutments whereas the large diameter piles will be used to support the central piers. The offshore pile cap is supported on three 1.8m diameter bored concrete piles whereas the on-shore pile cap is supported on six 1.20 m diameter bored concrete piles. Idealized soil profiles used for capacity calculations on-shore were given in the structural design report. The NF is modeled as very dense silty sand with effective un-factored friction angle ϕ' equals 42^o and factored friction angle of 35.8^o. subgrade modulus K value of 40000 KN/m^2 was used. Capacity was determined for the smaller diameter (1.2m) using Tomlinson [6] and assuming pile penetration length of 27 meters on-shore and 31.0 m off-shore. The computed allowable capacities are given in Table 3 for the 1.2m diameter piles. Factor of safety of 2.5 was applied for friction and 3.0 for end bearing. The Nq value for the NF "very dense sand" was taken as 50 (ϕ' equals 35^o). The piles embedment depth was checked using the computer program GROUP for lateral stability.

The pile test was carried out on 1.2m diameter pile located between the West Abutment and Pier (P1) of the bridge. The test pile was designed for a test load of 12,000kN (twice the maximum working load). The test pile was installed to a depth of 28m with 23 meter penetration in the NF. The calculated ultimate capacities for the test pile using the same parameters stated above were:

shaft friction, Qsu = 2,854kN

end bearing, Qbu = 14,081kN

However the total measured settlement on the application of 1200 ton was only 3.3 mm and the plastic settlement only 0.9 mm. The pile load test revealed the following:

- The maximum shaft resistance is far from being fully mobilized. Full mobilization of the shaft friction will necessitate settlements greater than 3.3 mm, i.e. about 1% of pile diameter (> 12 mm).
- No movement was generated at the base of the pile and therefore no end bearing was mobilized.
- It appears that the Nubian formation in which the pile was socketed provided much greater shaft friction than the conventional design parameters suggest
- If we consider that the maximum applied load (12000) KN had been resisted by the shaft friction through 23 m of penetration in the rock, then the average applied unit shaft friction would exceed 160 KN/m^2. The ultimate unit frictional resistance will certainly be greater than 200 KN/m^2; if compared with the computed values (< 40 KN/m^2) , it will be clear that the NF is acting more as a socket than as a normal cohesionless material.
- The assumption that the NF will act like a very dense cohesionless soil is not sound.

5.3 New White Nile Bridge

The data analysis in the geotechnical report was based on the statement that "Client "Contractor" wanted to use bored concrete piles 1.2m diameter and each pile should safely carry 660 tons"; the geotechnical firm was asked to suggest the suitable pile depth that satisfies the given requirements.

Idealized soil profile was given in the report. The highly weathered Nubian Formation extends from 7.0 m depth down to more than 50.0 m. It consists of alternating layers of sandstone and mudstone. The NF was modeled as very dense sand with effective friction angle of 41^o and effective unit weight of 7.5 KN/m^3. The Nq value for base resistance was taken as 70. The design or safe pile length was found to be 24 meters provided that all piles should rest on the weathered sandstone. The ultimate base resistance was computed to be 14250 KN whereas the ultimate shaft resistance was 2160 KN. A factor of safety of 2.5 was assumed.

Table 3: Ultimate pile capacities as determined by the designer (from design report)

Pile dia. (m)	Pile Length (m)	Ultimate Unit Friction (KN/m^2)	Ultimate end Bearing (KN/m^2)	Ultimate Total Friction (KN)	Ultimate Total End Bearing (KN)	Qu (KN)	Qa (KN)	Comments
1.2	27	33.92	14055	3628	15896	19528	7113	On-shore
1.2	31	35.8	12332	2769	13936	16705	6030	Off-shore

Pile load test data was not found for this site. The simplified assumptions used in this analysis are the same as those used for the design of piles in Soba and Manshia. The main shortcoming of the analyses is neglecting the probable degrading effects of the mudstone layer on the pile capacity computations.

5.4 ALdabasin Bridge

The substructure for this bridge was executed in two stages due to changes in design and Contractor. The first stage, completed in 2006, comprised of construction of pile groups at the location of the two abutments and 24 piers on both sides of the river. The piles supporting the abutments are 0.9 m diameter and 24-25m long whereas those supporting the piers were 1.5 and 1.35m diameter with total effective length 23.5 to 35m below existing grade.

The methodology specified in LRFD (Load Resistance Factor Design) AASHTO Code 1 was followed for estimating the bored pile axial resistance using the method of O'Neill and Reese [6]. According to LRFD geotechnical design philosophy the nominal pile resistance (ultimate pile capacity) which includes the friction and bearing resistances should be multiplied by geotechnical resistance reduction factors, rather than dividing by one global safety factor, to obtain the factored pile resistance (allowable capacity).These geotechnical factors account for many uncertainties related to design, construction and side condition details. The resistance factor differs depending on the embedment soil, i.e., whether it is clay, sand, gravel or intermediate geomaterial (IGM).

Two different approaches were followed in modeling the NF for the computation of side and tip resistances; these are:

- The NF is assumed to behave as very dense sand. The side and tip resistances were calculated according to AASHTO 2007 [7]
- The NF is regarded as IGM. The procedure used for capacity calculations were based on FHWA [8] and O'Neill and Reese [9].

A plot showing a comparison of the two factored resistances computed according to the above design methods is presented for bored piles installed at the 24 Pier locations Figure 10.

Two pile load tests were performed at the location of the eastern and western bridge abutments on piles 0.8m diameter and 21.0 and 25.0m long, respectively. Both test piles were subjected to sequence of loading to a maximum of 6570 KN which is assumed to be the maximum service load.

Interpretation of the load tests was made based on the so-called L1-L2 graphical method proposed by Hirany and Kulhamy [11]. In this method the values of L1 and L2 are the loads corresponding to pile tip settlements equal to 0.4 and 4% of the pile tip diameter. The maximum loads applied in the two tests produced total settlements equal to 1.25% and 2.05% of pile diameter. Accordingly the elastic limit load (L1) values were estimated to be 4200KN and 2200KN for the east and west abutment locations, respectively. The unit friction values for the NF exceed 300 KN/m^2

The test pile on the western side penetrated about 8.0 m in the hard clay. The pile was load tested in increments to 6570 KN. The total measured settlement was 16.41mm and the plastic settlement was 9.6 mm. The total settlement exceeded 2% of the pile diameter therefore the friction could be considered as fully mobilized.

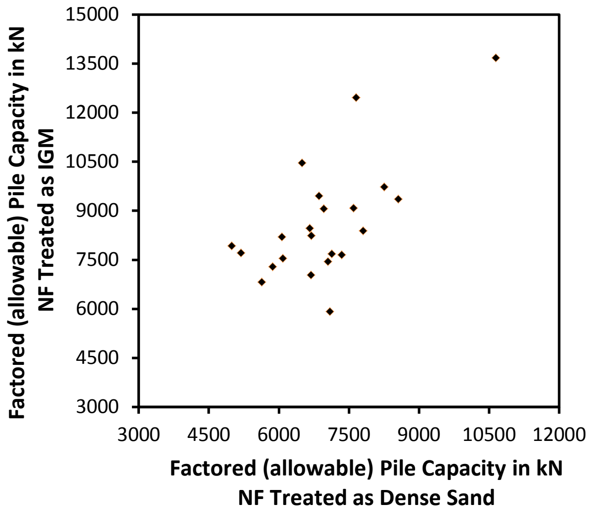

Figure 10: Factored Pile Capacities using LFRD design methods

5.5 Tuti-Bahri Bridge

The geotechnical consultant recommended using 0.8 m to 1.2 m diameter piles socketed in the NF to 32.0 m depth from the original ground or river bed level. The recommended sizes and the corresponding allowable capacities are given in Table (4). The capacities were determined using "ALLPILE" software. Idealized soil profile and soil parameters were not given.

The structural designer adopted bored piles with the following configurations:

- 1.8 m diameter and 40 m long for the two pylons
- 1.5m diameter and 26 m long for the two piers
- 1.2 m diameter and 22.0 m long for the abutments

The AASHTO LRFD [7] Code was followed for the computations of the nominal side and tip resistances of individual piles and the factored resistance were obtained using the appropriate resistance factors. The NF was assumed to behave as IGM and the total factored axial pile resistances were estimated to be as follows: for pylons support 19400 KN; piers support 10500-12300 KN and for abutments support 6050 to 7400 KN. The lower values of the capacities at the abutments and piers location correspond to Tuti side of the bridge.

One pile load test was carried out. The test pile was installed on the eastern river bank close to Borehole #4. The profile near the test pile constitutes an upper nile silt layer (0.0 to 4.0m) overlying medium dense silty sand layer extending down to depth 18.0 m. The silty sand is underlain with 3.0 m thick highly weathered sandstone layer overlying a thick layer of highly weathered mudstone (to depth 28.5m) followed by interchanging layers of weathered sandstone and mudstone down to 50.0 m depth. The test pile has a diameter of 1.0 m and is extended down to 40.0 m depth. The pile was tested to carry 1000 ton maintained maximum load. The total measured settlement under this load was 5.34 mm (~0.5% of pile diameter). It is evident from the load test that the total friction was not mobilized along the pile when 1000 ton load was applied to the pile head. The unit friction for 22.0m penetration in the NF exceeds 300 KN/m^2.

Table 4: Allowable capacities of suggested pile sizes for Tuti-Bahri (from the geotechnical report)

Pile diameter (m)	Pile Length (m)	Qa (KN)	Comments
0.8	32	3100	
0.9	32	4300	
1.0	32	5600	
1.1	32	7300	
1.2	32	9400	

6 CONCLUDING REMARKS

The following remarks and observations may be drawn as general conclusions derived from the analysis and discussion on the characteristics of the NF and the design criteria and methodologies adopted for the bored piles supporting the substructures of the five bridge projects:

- The foundations of all the bridges across the Niles in Khartoum are resting on or socketed in the NF underlying the upper alluvial formation.
- The NF is basically sandstones with pockets of mudstones and rare occurrence of conglomerates. The formation is highly weathered within the depth range of the foundations and is rated as very poor rock based on the core recovery and RQD. The rock may be described as weak rock based on SPT-N values and unconfined compressive strength according to CIRIA [10] report classification (80<N60,200; 600< qu<1250 KN/m^2).
- The gradation of the sandstone is medium to fine sand often cemented with silt and kaolin and seldom with iron oxide. The SPT-N value of the sandstone is much greater than 100 and the average measured unconfined strength of the retrieved core specimens is 2700 KN/m^2. Accordingly and based on the Construction Industry Research and Information Association, CIRIA, [] the sandstone is considered as weak rock (600<qu<12500 KN/m^2). The iron cemented sandstone is weak to moderate.
- The mudstone, within the investigated depths, is blocky and highly weathered. The core recovery was very poor and it is composed of high percentage of silt and clay particle size. The consistency is generally non-plastic to low plastic with rare occurrence of highly plastic mudstone (Tuti-Bahri site). The mudstone is weaker than the sandstone (average qu is 1431 KN/m^2). It is considered according to CIRIA [10] as very weak rock.
- Very few pressuremeter test results were reported for the NF at the studied sites. The results showed that the elastic modulus of the sandstone was showing considerable variations (1048 to 13758 KN/m^2). The sandstone is more stiff compared to the mudstone which measured very low elastic modulus. The test results are known to be very sensitive to the quality of drilling [10].
- Bored concrete piles were proposed and used to support the structural loads for all the bridges across the Niles in Khartoum. Review of the geotechnical reports has shown that two approaches were used for estimation of the bearing capacity of individual piles. For the first approach the NF was modeled as very dense sand whereas the second one used AASHTO (2007) LRFD code to compute the pile capacities. The NF for the latter was assumed to behave as IGM.
- For the first design approach, the ultimate load capacity of the piles was computed using effective angle of friction of 37^o to 43^o for the NF or the very dense sand. The recommended Nq values ranged from 50 to 70 for end bearing computations and the unit friction along the pile shaft was found to be less than 40 KN/m^2.
- Figure 10 compares the factored resistances of single piles determined assuming the NF to behave as IGM versus behaving as very dense sand for Aldabasin site. The results show that the latter assumption gives higher capacities compared to the former. The factored pile capacities when the supporting layer is assumed as IGM are about 1.25 times those when the supporting formation is considered as very dense sand.
- The pile load test data was analyzed for four bridge sites. The load tests reflect the fact that there is considerable under-estimation of the ultimate pile capacities when the two theoretical approaches are used. For most of the pile load tests, the

frictional resistance had not been fully mobilized. The estimated unit friction for the NF would exceed 200 KN/m^2. The estimated unit frictional resistances from the pile load tests clearly indicate that the NF under all in-place conditions is acting more as a socket than as a normal cohesionless material.

- This evaluation has shown that alternative design approaches or improvement in the used ones are needed for the designers to come out with a technically viable and sound design for piles socketed in the NF. The call for design approach should take into consideration the variations in rock types, properties and weathering conditions to arrive at more representative evaluation of the unit side friction and unit tip pile resistance values.

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