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Geotechnical Investigations for Design of Foundations for Erosion and Flood Control Structures at Unwana Beach, Afikpo, Ebonyi State, South-Eastern Nigeria

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GEOTECHNICAL INVESTIGATIONS FOR DESIGN OF FOUNDATIONS FOR EROSION AND FLOOD CONTROL STRUCTURES AT UNWANA BEACH, AFIKPO, EBONYI STATE, SOUTH-EASTERN NIGERIA.

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ABSTRACT

Erosion occasioned by annual floods along the Cross River at Unwana Beach, Afikpo, situated on latitude 8^o 00' 00'' North of the Equator and longitude 5^o 40' 15'' East of the Greenwich Meridian, has threatened the location of a Water Pumping and Treatment Station nearby. Geotechnical investigations carried out using the Shell-and-Auger rig along the shoreline of the Unwana Beach indicated that the subsurface consists of between 3.00 to 8.00 meters of Yellowish brown silty clay layer (CL) underlain by about 3.00 meters of Dark, clayey sands (SC). These are further underlain by between 0.00 – 4.00 meters of Reddish gray, mottled silty clay (CL) and between 1.00 – 1.50 meters of Black stiff silty clays (MH). Underlying all these is a Black fissile Shale layer that extends beyond the limiting 20.00-meter depth of boring prescribed by the clients.

Standard Penetration Tests (SPT) indicate that the upper-most Yellowish brown silty Clay layer has N-values of between 15 and 45, while the underlying Dark clayey sand layer has average N-values of 20. The Reddish brown, mottled silty clay layer (CL) beneath has average N-values of 22, while the Black fissile shale layer has N-values more than 50 (that is, refusal).

The computed allowable bearing capacities (based on N-values and Terzaghi's classical soil mechanics approach) for the subsurface materials at the project site indicate that the upper Yellowish brown silty clays (CL) have $q_{(allowable)}$ of 236.06 and 139.51 kPa respectively; Dark clayey sands (SC) have 188.84 and 195.67 kPa respectively and the Reddish gray mottled silty clays (CL) have 173.11 and 201.60 kPa respectively.

Bathymetric surveys carried out perpendicular to the shoreline at five sections indicated that the maximum depth to the river bed at the proposed site for a Landing Jetty, at the date of investigations (11-22-2002), was 6.20 meters.

Steel sheet piles were recommended and used as foundation systems for the shore protection works with the length of sheet piles equal to $H + D_f + h$**,** where H = depth to the bottom of the river bed at low-low water, D_f = depth of embedment of pile into the bearing medium and h = height of sheet pile above the river bank cliff (free-board) at the time of investigations (18th – 28th November, 2002). Wales of steel-type were used as reinforcement for the emplaced sheet piles, with their vertical separations approximately equal to 1.50 meters. Steel tie-backs were used to restrain the emplaced sheet piles from undergoing flexural and / or buckling failures, with tie-back vertical separations equal to 1.50 meters and tie-back horizontal separations approximately equal to 2.00 meters. Additionally, anti-corrosion protection for the tie-backs was asphaltic materials and concrete encasements.

Key Words: Erosion, Flood-control, SPT-values, allowable-bearing-capacities, bathymetry, sheet-piles.

INTRODUCTION

During the recessional stages of the annual floods that occur along the Cross River in southeastern Nigeria, much of the river bank materials are eroded away leading to shoreline encroachment of previously established structures along the banks of the river. One of such threatened areas was the Unwana Recreational Beach located at Afikpo in the Ubeyi local Government Area of Ebonyi State of Nigeria. Flood and erosion control structures proposed to be constructed at the beach necessitated geotechnical investigations to be carried out to provide ***some geologic as well as geotechnical engineering parameters*** which would form the basis for ***sound engineering design of foundation systems*** for the proposed flood and erosion structures.

Scope of Paper.

The general scope of the paper provides the geologic sequence at the project site to depths commensurate with deep foundation systems using the Shell-and-Auger rig. It also presents through laboratory and field testing, the physical properties, strength characteristics and bearing capabilities of the sub-soils that influenced both the choice of foundation systems and design considerations. Thirdly, the paper provides recommended construction techniques that were needed according to the dictates of the subsurface conditions at the project site.

Description of project site. Geographically, the project area is situated approximately on ***latitude 8^0 00' 00'' North*** of the Equator and ***longitude 5^0 40' 15'' East*** of the Greenwich Meridian. The Project is situated along a stretch of the shoreline of the project site.

Topography and vegetation.
The topography of the project area is flat-lying at the beach with a very steep incline behind the river bank. The vegetation around the project area consists mostly of tall trees with lush undergrowth**.**

HYDROGRAPHIC CONDITIONS AT THE PROJECT SITE

The project site is situated along the concave segment of the Cross River at the Unwana Beach, in Ubeyi Local Government Area of Ebonyi State, Nigeria. The river flow is faster at the concave segment which is also prone to shoreline erosion as a result of flow impact on this shoreline.

Bathymetric Surveys

In order to determine the depth of the river at the shoreline, the flow velocity as well as the river bed configuration along the proposed segments of the shoreline for both the *wharf wall and the jetty*, a *bathymetric survey* was carried out perpendicular to the shoreline at ***(5) points***. The bathymetric surveys were carried out with the aid of a ***hand-dug canoe*** that conveyed both survey crew and equipment. The Datum was taken as the 1973 Maximum Flood Level (MFL) during the survey measurements at the project site. In addition, the MFL mark for 1974 was also taken for comparison purposes. Lastly the 2002 MFL was also taken. All measurements of elevations were then taken from the ***1973 datum*** to aid in the final design of the wharf as well as the Landing Jetty.

Bathymetric profiles perpendicular to shoreline at time of study. The following perpendicular profiles were measured along the shoreline:-

(i) Jetty point
(ii) 25.0 metres downstream of the jetty point
(iii) 50.0 metres downstream of the jetty point
(iv) 75.0 metres downstream of the jetty point
(v) 100.0 metres downstream of the Jetty anchor point along the shore protection structures.

The Jetty Point.. The first (1^{st}) bathymetric profile was taken at the site of the proposed Jetty at the project site. From the bathymetric profile, it was observed that as at the date of the survey (22/11/02), the vertical distance from the crest of the bank to the river bed at this point was approximately ***6.20 meters with*** the deepest point as ***4.20 meters*** at a distance of ***40.00 meters*** from the shoreline. The angle of curvature of the river bed from the shoreline was approximately ***20^o*** from the horizontal.

50.0 metres downstream of the jetty point. The third (3^{rd}) bathymetric profile was taken at a point 50.00 metres downstream of the ***Jetty location.*** The crest profile is not too different from that at the jetty site with the vertical distance from the bank crest to the river bed here as approximately ***5.00 metres***. The deepest point at this crossing was ***3.90 metres*** at a distance of between ***60.00 and 90 metres*** from the shoreline. The angle of curvature of the river bed from the shoreline was approximately ***10^o*** from the horizontal.

100.0 meters downstream of the jetty point. The fifth (5^{th}) Bathymetric profile was taken at a point 100.00 meters downstream of the Jetty location. At this river crossing the bathymetric profile on the date of the survey (22/11/02) shows that the vertical distance from the crest of the bank to the river bed was approximately *5.20 meters* with the deepest point as ***3.40 meters*** at a distance of ***10.00 meters*** from the shoreline.

The angle of curvature from the shoreline was approximately ***5°*** from the horizontal towards to the coast.

SUBSURFACE CONDITIONS

Subsurface conditions at the project site were studied by boring holes with the aid of *Shell-and-Auger Rig*. Soil samples were retrieved at specific depth intervals of ***1.00 meter*** for purposes of visual examination, laboratory testing and classifications. Also, Soundings in the form of *Standard Penetration Testing (SPT)* were carried out during the boring process.
The bearing capabilities of the various soil horizons at the project site were assessed using the *Standard Penetration Test data.* which gave valuable information about the subsurface characteristics at the project area.

Local Geology

The local geology of the project area is basically that of the Nkporo Shale and Eze Aku Shale Groups of Upper Cretaceous (Turonian to Senonian) age**.** The general area consists of sedimentary rocks ranging from shale, mudstones to siltstones.

Water Tables. The Water Table at the site was observed to be located at a depth of *1.00m* below the beach level at the shoreline at the time of the investigation but this is highly variable depending on the season under consideration. High water tables are obtained during the flood level stage.

Subsurface Explorations. The subsurface exploration program at the project site comprised Shell-and-Auger borings, soil samplings and Standard Penetration Tests (SPT) which were also carried out during the Shell-and-Auger boring exercise.

Borings. A total of three (3) Shell-and-Auger borings each to a depth of ***20.00 meters***, except the boring at the location of the jetty that went down to 30.00 meters**,** were made along the shore of the proposed Water Front protection site. A Fence Diagram of the borings carried out at the site is shown in *Fig. 1*..

Samplings. In general, disturbed samples were obtained during the drilling program in all Shell-and-Auger holes. Within the zone of cohesive materials such as clays or sandy clays, undisturbed soil samples were obtained with the aid of U-tubes. Sampling intervals during the drilling were *1.00 meter* apart up to the end of the boring. All depths are in relation to ground level at the time of investigations.

Standard Penetration Tests (SPT). Standard Penetration Tests were carried out at the sampling depths where cohesion-less *(c)* materials or *c - ϕ soils* were encountered, during the boring exercise. These values show, to a large extent, the ability of the various layers of the soils at the project site to carry foundation loads imposed on them.

Subsurface profiles and descriptions at the various sites. Details of the various lithologies encountered during the subsurface exploration program are presented in a Fence Diagram *(Fig.2).* Generally the soils / rock down to 30.00 meters depth at the location of the Jetty, can be categorized on the basis of consistency, gradation and strength into *five (5) types* namely:

(i) Yellowish brown Clay Layer *(CL)*
(ii) Dark, soft Clayey Sand *(SC)*
(iii) Reddish Gray mottled Silty Clay *(CL)*
(iv) Black stiff Silty Clay *(MH)*
(v) Black fissile Shale *(Shale)*

Yellowish to reddish-gray mottled Clay Layer (CL). These materials make up the ***uppermost sections*** of the subsurface at the project site and are found to be about **8.00m** *thick* at the site of the proposed Jetty. It is basically made up of *yellowish to reddish-gray, mottled Clay* with moisture contents varying from a low ***12.5 to 16.4%.*** Under the ***Unified Soil Classification System (USCS)***, the materials in this layer can be classified as *yellowish to reddish-gray, mottled Clay* ***of low consistencies (CL).*** They have ***Plasticity Indices*** (PI) in the range of ***8.5 – 14.2%.***

Unconsolidated – Undrained ***(U-U)*** triaxial test results on samples from this layer indicate that the undrained friction angles ($\varnothing_u$) vary from 4^0 ***to*** $6.^0$ with corresponding cohesion ***(Cu)*** values ***between 45.00 and 56.00 kPa.***
Consolidation Tests indicate that the *Coefficient of Consolidation* C_v under a load of 50.00 kPa was between *0.95 and 1.24* m^2 *per year*, while the *Coefficient of Volume compressibility* M_v was between *1.05 and 1.45* m^2 *per KN*. Under a load of 400kPa, the values of C_v varied between *1.28 and 1.75* m^2 *per year*, while the M_v varied between *1.75 and 1.88* m^2 */ KN*, respectively.
The gradation patterns of the materials are such that their *Coefficients of Uniformity* (C_u) range from *2.5 to 2.80.*

Dark, soft Clayey Sand (SC). This layer, which extends from a depth of about ***8.0 meters to 9.00 meters*** at the site, has an approximate thickness of about ***1.00 meter***. These materials can be classified as ***SC (Clayey Sands, Sand-Clay mixtures)*** under the ***Unified Soils Classification (USC) system***.

They have moisture contents varying from ***10.2 to 15.8%.*** Unconsolidated – Undrained ***(U-U)*** triaxial test results on samples from this layer indicate that the undrained friction angles *($\varnothing_u$)* vary from ***18^0 to 22.0^0*** with corresponding cohesion ***(Cu)*** values ***between 20.75 and 27.50 KPa.*** Consolidation Tests carried out on materials from this layer indicate that the *Coefficient of Consolidation C_v* under a load of 50.00 kPa was between *0.75 and 1.75 m^2 per year*, while the *Coefficient of Volume compressibility M_v* was between *0.80 and 0.95 m^2 per KN*. Under a load of 400kPa, the values of C_v varied between *1.05 and 1.25 m^2 per year*, while the M_v varied between *1.10 and 1.20*, respectively.
The gradation patterns of the materials are such that their Coefficient of Uniformity *(C_u)* range from *1.5 to 1.75*.

Stiff, reddish gray mottled Silty Clay (CL). This layer, extends from a depth of about *9.0 meters to 12.60 meters* at this site, has an approximate thickness of about *3.60 meter*. These materials can be classified as CL (Silty Clays) under the Unified Soils Classification (USC) system. The materials have moisture contents varying from *20.2 to 40.2%. Unconsolidated – Undrained (U-U) tri-axial test* results on samples from this layer indicate that the undrained friction angles *($\varnothing_u$)* vary from *4^0 to 6.0^0* with corresponding cohesion *(C_u)* values of *75.50 KPa.*
Consolidation Tests carried out on materials from this layer indicate that the *Coefficient of Consolidation C_v* under a load of 50.00 kPa was between *2.20 and 2.25 m^2 per year*, while the *Coefficient of Volume compressibility M_v* was between *1.55 and 1.75 m^2 per KN*. Under a load of 400kPa, the values of C_v varied between *1.60 and 1.70 m^2 per year*, while the M_v varied between *1.25 and 1.75*, respectively.
The gradation patterns of the materials are such that their *Coefficients of Uniformity (C_u)* range from *2.2 to 2.5.*

Black Stiff Silty Clays (MH). This layer extends from a depth of about *12.0 meters to 13.00 meters* at this site { Boring # 1}, and has an approximate thickness of about *1.00 meter.* These materials can be classified as *MH* (Inorganic Silts and elastic silts and silty clays) under the *Unified Soils Classification (USC) system.* The materials have moisture contents varying from *22.4 to 22.8%. Unconsolidated – Undrained (U-U) triaxial test* results on samples from this layer indicate that the undrained friction angles *($\varnothing_u$)* averaged about *8^0* with corresponding cohesion *(C_u)* values of *76.80 kPa.*. *Consolidation Tests* carried out on materials from this layer indicate that the *Coefficient of Consolidation C_v* under a load of 50.00 kPa was an average of *1.20 m^2 per year*, while the *Coefficient of Volume compressibility M_v* was average of *1.70 m^2 per KN*. Under a load of 400kPa, the values of C_v averaged *1.25 m^2 per year*, while the M_v averaged *1.70*, respectively.
The gradation patterns of the materials are such that their *Coefficient of Uniformity (C_u)* range from *2.5 to 3.5.*

Black fissile Shale (Shale). This layer, which extends from a depth of about *15.0 meters to depths beyond the limits of explorations* at this site, has an approximate thickness of over *5.00 meters*. These materials can be classified as *Shale* since these are virtually rocks and not necessarily soils to be classified under the *Unified Soils Classification (USC) system.* The materials have moisture contents varying from *14.4 to 14.8%.* SPT N- values obtained on these materials indicated values are in the range of over *70 blows per 30 cm (refusals)*.

Soil Bearing Capacity. In addition to the field *Standard Penetration Tests (SPT)*, laboratory *one-dimensional oedometer consolidation tests* were also carried out on selected undisturbed samples obtained in U-4 tubes during the subsurface investigations. The bearing pressures of the various soil layers were computed using the classical *Terzaghi Theoretical equation* given by Terzaghi and Peck [1967] as:

$$q_u = q_c / F.S = 1/F.S \{\{ (1\text{-}0.2\, B/L)\, \gamma\, B/L.N_\gamma \} + \{ (1 + 0.20\, B/L)\, c\, N_c\} + \{ (\gamma D_f\, N_q)\}\} \qquad (1)$$

where:

B = width of Foundation;
L = Length of Foundation
γ = unit weight of soil at foundation level
N_γ, N_c, N_q = Terzaghi factors.

The bearing capacity values for the various soil layers at the project site, computed on the basis of F.S = 3.0; are given in Table 1 below. These values are comparable to those of Teme [1992 and 2000].

Table 1. Bearing capacity values for soil layers

Soil Type	Bearing capacity values qu (kPa}
Yellowish brown Clay Layer (CL)	*139.51*
Dark soft Clayey sands Layer (SC)	*195.67*
Reddish gray mottled Silty Clay (CL)	*201.60*
Black stiff Silty Clays (MH)	*224.40*

Table 2: Consolidation and drainage characteristics of materials at the Unwana Beach project site.

	(m)	M_v (m^2/MN) 50.kPa	400 kPa	(C_v) (m^2/yr) 50.kPa	400 kPa	(K) (cm/sec)	(corrected)	
Yellowish brown Clay (CL)	0.00 to 7.00	1.86 to 2.12	0.94 to 1.05	0.40 to 0.65	0.36 to 0.80	4 x 10-6	15 – 45	Compressible very low permeability, poor drainage
Dark soft clayey Sand (SC)	7.00 to 9.00	0.86	0.76	0.66	0.48	4 x 10 –4	20 – 28	Low compre-ssibility, good permeability, fairly drained
Reddish-gray mottled silty clay (SC)	9.00 to 11.00	1.98	1.08	0.92	0.54	1 x 10 –4	22	High compressi-bility; low perm-ability, poor drainage
Black-stiff silty clay (MH)	11.00 to 13.0	1.60	1.76	1.20	1.25	2.25 x 10 –4	--	Fairly high compressibility moderate perm-eability, fairly low drainage
Black fissile shale layer (Shale)	13.00 > 20.00	N/A	N/A	N/A	N/A	N/A	> 50	Very low com-pressibilty and highly impermeable

Consolidation. Two parameters namely (a) the *Coefficient of Volume Compressibility (M_v)* and (b) *Coefficient of Consolidation (C_v)* were determined during the laboratory consolidation testing. Oedometer tests carried out on these site soil samples indicate that the *Coefficient of Consolidation (C_v)* varied from 0.40 to 0.65 m^2/yr and 0.36 to 0.80 m^2/yr under confining pressures of 50 and 400 kPa, respectively, for the upper *yellowish brown Clay (CL)*; 0.66 and 0.48 m^2/yr under confining pressures of 50 and 400 kPa, respectively, for the *Dark soft Clayey Sands (SC)*; 0.92 and 0.54 m^2/yr under confining pressures of 50 and 400 kPa, respectively, for the *Reddish gray mottled Silty Clay (CL)* and 1.20 and 1.25 m^2.yr under confining pressures of 50 and 400 kPa, respectively, for the *Black Stiff Silty Clays (MH)*. On the other hand, the *Coefficient of Volume Compressibility (M_v)* obtained for the same set of soil samples under similar confining pressures varied from 1.86 – 2.12 m^2/MN for the upper *yellow brown Clays (CL)*; from 0.76 – 8.6 m^2/MN for the *Dark soft Clayey Sands (SC)*; from 1.08 – 1.98 m^2/MN for the *reddish gray mottled Silty Clays (CL)* and from 1.60 – 1.70 m^2/Mn for the *Black stiff Clays (MH).*
Values of *Coefficient of Consolidation (C_v)* and *Coefficients of Volume Compressibility (M_v)* obtained from the laboratory tests are contained in *Table 2.*

CPT values. The penetration resistance obtained from the Dutch Cone Penetration Test as converted from the SPT curves (according to the method of Peck, Hanson and Thornburn, 1974) are contained in the form of *resistance versus depth values* and shown in Table 2.
These values assist to indicate to a large extent the incremental and continuous values of resistance of soil layers with depth at

the project site and also indicate possible horizons where foundation loads can be borne.

Drainage Conditions

Generally, the *Static Water Table* at the project site was encountered at a depth of between *– 2.00 and – 10 meters* below the ground surface at the time of the drilling process. The Water table is observed to be very highly variable due to the annual floods that inundate the entire area from July to October every year.

The overlying *Yellowish brown Clay Layer(CL)* materials are compressible with values of *Coefficient of Permeability (K)* (as determined according to Hazen [1893]) of $4x10^{-6}$*cm/sec* or lower and therefore has poor drainage characteristics. The underlying *Dark Soft Clayey sand(SC)* materials are just slightly compressible with values of *Coefficient of Permeability (K)* of $4x10^{4}$*cm/sec* or higher and therefore have fairly good drainage characteristics.

The *Reddish Gray Mottled Silty Clay(CL)* materials have *permeability (K) of* $1x10^{-4}$*cm/sec* and therefore have fairly poor drainage characteristics. The *Black Stiff Silty Clay (MH)* materials beneath are fairly highly compressible with values of *Coefficient of Permeability (K)* of 2.25×10^{-4}*cm/sec* and therefore have fairly low drainage characteristics. However, the *Black fissile Shale (Shale)* are virtually incompressible and impervious.

FOUNDATION TYPE OPTIONS ADOPTED AT PROJECT SITE.

Based on the sub-surface lithology at the site, the annual variable *Water Table* consequent upon the cyclic floods and taking into consideration the type of project proposed for the site (Flood and Erosion Control Structures), *Deep Foundation Types in the form of Steel Sheet Piles* with *Wales* and *Tie-rods* to provide tie-back support for the sheet piles were recommended for the project site.

Design Considerations for Steel Sheet Piles.

The design of the sheet piles took into consideration the *Length of sheet piles (L); depth of sheet pile embedment* (D_f) and *the stresses on the embedded piles [Meyerhof, 1963; Hansen, 1968; Hansbo, 1994; Bowles, 1977; Tomlinson, 1980]*

Length of sheet piles *(L)*. In order to find the appropriate length of sheet piles, it was necessary to know the maximum height of past floods at the project site which was found to be equal to *20.00 meters.* Therefore the *Length of the Sheet Pile* was taken to be approximately equal to:

$$H + D_f + h = 20.00 \text{ meters}. \qquad (2)$$

where: H = height from top of cliff to the top of bearing medium ~ *13.0 metres* {based on subsurface profile at site}; D_f = depth of penetration of the sheet pile into the bearing layer and h

Paper No. 5.44

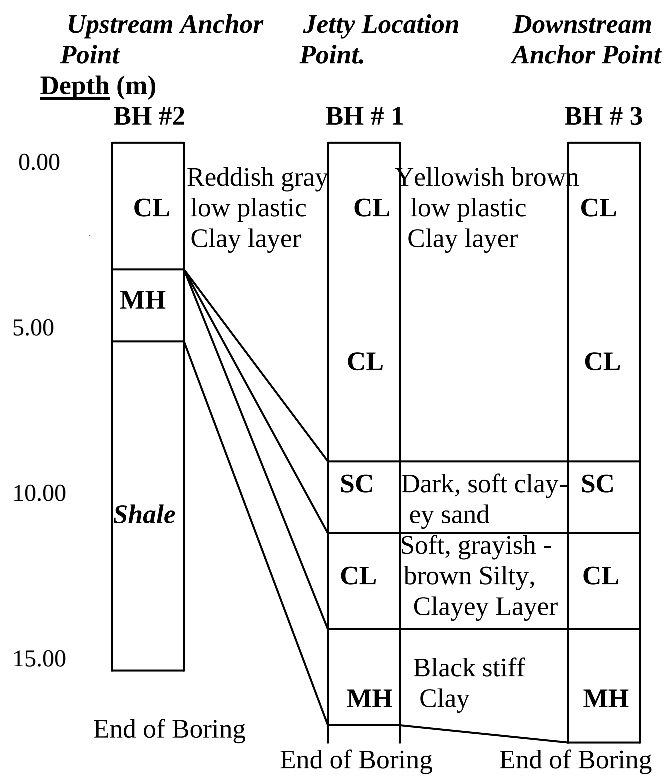

Fig. 1. Schematic representation of a Fence Diagram to show the subsurface disposition of soil profile along the shoreline at Unwana Site

Depth of embedment of sheet piles (D_f). This was estimated by using values of *Standard Penetration Test (N) –values* (> 50 in this case) obtained from field investigations as contained in the Table 3 below.

Free-board above the crest of the cliff at the river edge.*(h).*. The maximum flood height during the *2002 flood* period was *10.50 meters* above the bottom of river bed. From the boring records, it is *13.00 meters* to the top of the *Black fissile shale* at the jetty site, therefore, from *equation (2)* above, we have:

$H + D_f + h = 20.00$ *meters* $= 13.0 + 4.65 + h$; and h = *2.35 meters.*
The schematic of the river bank with respect to sheet piling at the project site is shown in *Fig. 2.*

Table 3: Approximate values of depth of pile penetration, (D_f)

Standard Penetration Resistance, SPT *N-values*	Relative Density, D_r	Depth, D_f
0 – 4	Very loose	2.0H
5 – 10	Loose	1.5H
11 – 30	Medium dense	1.25H
31 – 50	Dense	1.0H
> 50	Very dense	0.75H

(after Cernica 1995)

Hence, $D_f = 0.75H = (0.75)\ (6.20\ )\ m.$
$\therefore\ D_f = 4.65$ *meters.*

Stresses on the embedded piles. For the given site where there are mixed (c-ϕ) soils, the probable *Pressure Distribution Patterns* behind the installed Sheet Pile are as shown in the *Fig. 3* below. These figures show the *Pressure Distribution, Active pressure on sheet pile wall* and the *forces on wall above point of zero shear*, respectively.

Foundation construction considerations. In this project site, both barge and land-based equipment were mobilized to site for purposes of the sheet pile emplacement along the shores of the project site. Also free-draining granular materials were recommended and used for back-filling after the emplacement of the sheet piles.

Emplacement of wales: For structural rigidity and avoidance of flexural failure of sheet piles, ***'Wales'*** were necessary for stability of the sheet pile system at the project site. The Wales were designed as simply supported beams with spans equal to the distances between tie-rods and attached to the backfill face of the piling in order to have a flush-front face. Where necessary, splices in channel sections of Wales were staggered to avoid weak points.

Vertical intervals of wales emplacement. Vertical intervals of Wales were maintained at *1.50 meters* (such as shown schematically in *Fig. 4a* below).

Emplacement of tie-backs. Tie-rods made of threaded steel bars were used to provide the anchor for the embedded steel sheet piles. Concrete encasements were provide as corrosion protection to the tie-rods used.

Vertical intervals of tie-back emplacements. The vertical intervals recommended for the tie-backs were $x = 1.50$ *meters(such as shown schematically in Fig. 4a)*

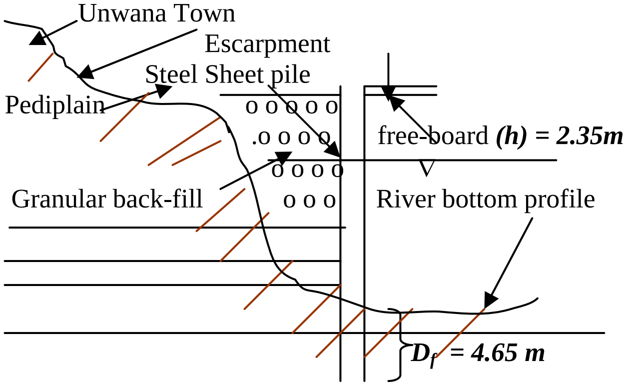

Fig. 2: Schematic of the river bank with respect to sheet piling at project site.

Horizontal intervals of tieback emplacements. The horizontal intervals used for the sheet piling were $x = 2.00$ *meters.* The schematic cross-sectional outline for the sheet piling at the project site is as shown in *Fig. 4a*.

Emplacements of anchors. It was recommended that the anchor system to be used at the project site be *Dead- man-type*. This was constructed by pouring a concrete beam in place at site, such as schematically shown in *Fig. 4b* below.

SUMMARY AND CONCLUDING REMARKS

On the basis of field investigations and laboratory testing carried out on soil samples obtained from the project site, it is observed that basically *five (5) identifiable soil horizons* are present.
The maximum depth to the river bed at the proposed site of the Landing Jetty at the time of field investigations was *6.20 meters*. Steel Sheet Piles were recommended and used
as foundation systems for the shore protection works at the Unwana Beach erosion and Flood Control Project site, Afikpo in Ubeya Local Government Area of Ebonyi State in southeastern Nigeria.

The length of the sheet piles used was equal to $\boldsymbol{H + D_f + h}$, where $\boldsymbol{H}$ = depth to the bottom of the river channel at low-low water, $\boldsymbol{D_f}$ = depth of embedment of pile into the bearing medium and $\boldsymbol{h}$ = height of sheet pile above the river bank cliff at time of investigations (18th – 28th November, 2002).

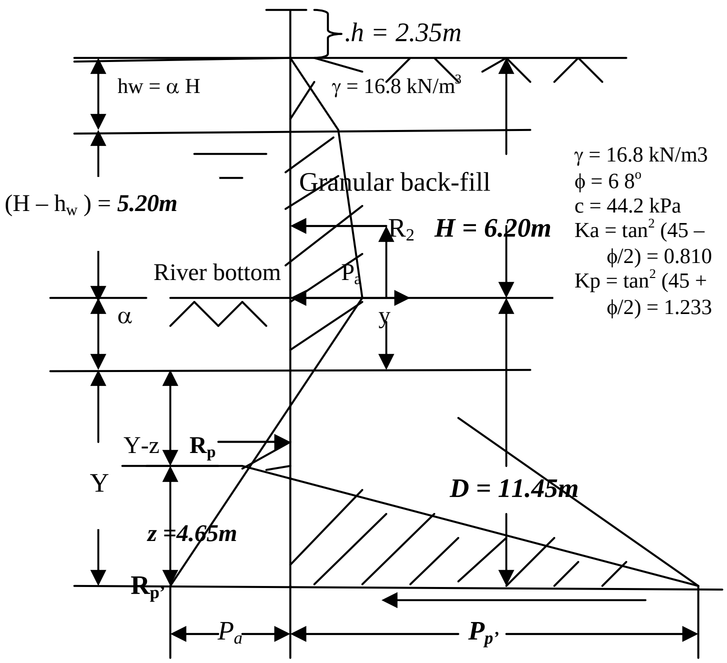

Fig. 3: Likely Pressures on Sheet-Piling at the project site

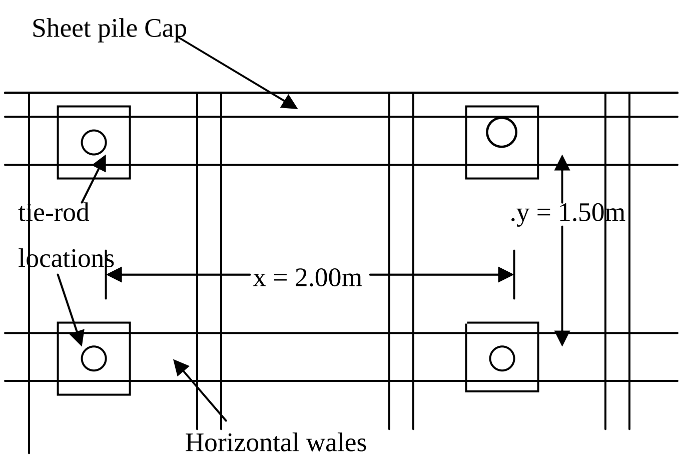

Fig. 4a: Schematic diagram showing the vertical distance between two Wales at the backfill face of the sheet piling.

Steel Wales were used for reinforcing the emplaced sheet piles at the site. The vertical separation of the wales was approximately 1.50m apart.

Tie-backs were used to restrain the sheet piles from undergoing flexural and / or buckling failures. The vertical separation of tie-backs was *approximately 1.50 meters apart*, while the horizontal separation was *approximately 2.00 meters apart.* Anti-corrosion protections for the tie-rods were either *asphaltic materials or concrete encasements*.

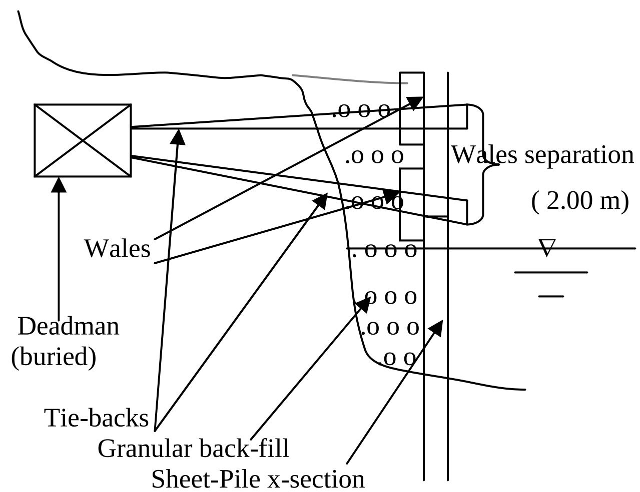

Fig. 4b: Schematic cross-sectional outline showing the vertical distances between tie-backs and Deadman-type anchors behind the sheet pile wall.

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