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# Bearing capacity and settlement assessment of the soil of Kotto(North-East of Douala) using static penetration and laboratory testing

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***Abstract:****Thisstudy reports on the assessment of the bearing capacity and soil settlement in Kotto, located, in the north-east of Douala(Cameroon)where modern buildings are rising daily. Themethodology entailsinthe analysis of the results from soil identification, direct shear, oedometric and static penetrometertests.Three objectives were envisaged, firstly, the knowledge of the geotechnical nature of the soils of this area,secondlythe knowledge of the mechanical propertiesof these soils, andfinally, a comparative study of the results of laboratory and in-situ tests. The testing procedures adopted were in accordance with French normsNF P 94-113 for the static penetrometer test, NF P 94-071-1 for the Casagrande shear test box, and the XP P 94- 090-1 for the oedometer test. The Terzaghi principle was utilized in calculating the bearing capacity. The results are quite expressive in terms of the difference between the results of laboratory and in-situ mechanical tests: the initial effective vertical stress between the depths 2 and 2.30 m is 23 kPa, while the average bearing capacity at 2 m depth is 322.5 kPa. In general, the bearing capacity of this area is low and requires soil improvement for structures with a load greater than 245 kPa. The settlement increases with depth, at the depth of 2 m, it can atain a value of 4.80 cm.*

***Key words:*** *bearing capacity, settlement, mechanical tests, modern buildings, Kotto.*

## Introduction

Cameroonis located between West and Central Africa and currently experiencing huge construction activities. The quality of construction of civil engineering works is imperative. Therefore, geotechnical studies are required to furnish the necessary soil parameters required in the design and construction of stable sustainable structures. Preliminary evaluation of mechanical characteristics and soil compressibility is not common practice by many project managers. There has been a number of recent building structural collapse such as the collapse of a six storey house under construction in Douala in September 2015, the collapse of five storeybuilding in Douala in the month of June 2016 and the collapse of a five storeybuilding in Dschang in September 2017,is a wake up call for tough scientific and technical approach in the design and construction of buildings.Thus, it is urgent to master soil mechanics in the sub-region. Hence this study on "bearing capacity and settlement assessment of the soil of Kotto (Douala North-east) based on the static penetration test and on laboratory tests". It mainly aims to determine the lift and compressibility of different soil layers. It is obvious that this goal can only be achieved through the identification and classification of soil, the determination of peak ground resistance, the evaluation of settlement through a practical case.

The present work is intended to be a critical analysis of all the activities generally performed in the analysis of the foundation soil of buildings in the city of Douala, specifically in the suburb of Kotto. This will be done through surveys and tests carried out in the laboratory as well as in situ. To achieve this, the work will be organized around three points. First of all the generalities on the study area, then the brief presentation of the methodology used, then a presentation of the different results obtained followed by their interpretation and discussion.

## I. Presentation of the study site

The study site was in Kotto which is the administrative district of the Douala V. It is located at an altitude of 46.0 meters, the GPS coordinates are 4 ° 5'5.64 " of north latitude and 9 ° 45'20.05 " of east longitude. Douala is the economic capital and is one of the coastal cities of Cameroon. It extends over a plain and is subdivided into six districts. It is a port city located on the edge of the Atlantic Ocean, at the bottom of the Gulf of Guinea, at the estuary of the Wouri River.The relief consists of a set of valleys mostly flat-bottomed, wet or dry.The climate is equatorial of the

Cameroonian type and coastal sub-type with two seasons which are: the rainy season which extends from March to November and the dry season which extends from December toFebruary. It is characterized by a constant temperature, around 27.5 ° C and very abundant rainfall. The air is almost constantly saturated with moisture.

The study area is located in the coastal plain of Cameroon stretching from the eastern slopes of Mount Cameroon to the estuary of the Sanaga river in the Douala sedimentary basin. This triangular basin has an area of about 7000 square kilometers (Ngueutchoua, 1996). The Douala Basin has suffered multiple transgressions during its geological history. The work of Njike (1984), Regnoult(1986), Ngueutchoua (1996), Mbesse et al(2012) highlights a certain number of formations. The lithology of the Douala basin from top to bottom shows several formations grouped according to their age of deposition: Quaternary, Tertiary and secondary sediments, with thestudy arealocated in the Tertiary formation. The Tertiary sediments are represented by the Nkapa, Souellaba and Matanda formations. They are characterized by marls, sandstones, silts, black or brown clays, clayey sands and fine sands.

## II.Material and methods

### *II.1. Tests*

Common test have been used (Manefouet, 2012;Enyegué, 2016). The water content was determined in accordance with the NF P 94-050 standards (AFNOR, 1995). The Atterberg limits tests were carried out in accordance with the NF P 94-051 (AFNOR, 1993).

The geotechnical parameters are intended to identify the soil and characterize its state by means of the clay consistency. The three distinguishing characteristics are the liquid limit LL, plastic limit PL and the plasticity index PI which gives information on the extent on the field soil plasticity.

The shear test was carried using the shear box test according to the NF P 94-071-1 (AFNOR, 1994). This test was carried out under drained conditions on all the samples obtained. The shear strength parameters were obtained from the slope of the line of best pit giving the angle of friction φ' (Effective angle of internal friction) and the ordinate at the origin of line giving the cohesion c'. This line represents the shear stress τ as a function of the normal stress σ' and friction angle φ'.

The Odometric test was carried out according to the standard XP P 94-090-1 (AFNOR, 1997), this test makes it possible to establish, for a given sample, two types of curves: the compressibility curve(which indicates the total settlement as a function of the logarithm of the applied stress) and the consolidation curves (which give the settlement of the sample as a function of time under application of a constant stress).The oedometric or compressibility curve $e = f[f(Log\sigma_v)]$giving the variations of the void ratio (e) of the soil as a function of the vertical effective stress applied to the sample will make it possible to determine the initialvoid ratio of the soil in-situ ($e_0$), the compression index Cc , the recompression index Cs and the maximum past effective stress (preconsolidation) pressure$\sigma_p'$.

The static penetration test was determined inAccordance with NF P 94-113standard (AFNOR, 1996). It aims to determine the resistance to penetration of a standardized cone and possibly the lateral friction mobilized on the sleeve. The parameters deduced from the measurements of the static penetration test are:

- the static tip resistance $q_c$ which is equal to the ratio between the tipstress Qc and the cross section Ac of the base of the cone, namely:
  $q_c = \frac{Q_c}{A_C}$ (1)
- the unit lateral friction $f_s$ which is equal to the ratio between the lateral friction force on the sleeve Qs and the lateral surface area of the sleeve As, and is given by:
  $f_s = \frac{Q_s}{A_s}$(2)
- the friction ratio $R_f$ in %, which is equal to the ratio between the unit lateral friction fs and the static peak resistance qc, that is:

$R_f = \frac{f_s}{q_c}$(3)

### *II.2. Calculation of stress and settlement*

The calculation of the shear strength of the soil layers will be performed from static penetration test. The ultimate stress at the cone tip is given as(fascicule 62-V, 1993):

$$q_u = k_c q_{ce} + \gamma D \quad (4)$$

Where

$$k_c = 0.14\left[1 + 0.35 \cdot \left(0.6 + 0.4\frac{B}{L}\right)\frac{D_e}{B}\right] (5)$$

$$q_{ce} = \frac{1}{3a+b}\int_{D+3a}^{D-b} q_{cc}(z)dz \qquad (6)$$

Where: B: width of foundation, L: length, D: height of foundation, De: height of the foundation element contained in bearer layer, $k_c$: lift factor, $q_{ce}$: even tip equivalent strength, $q_{cc}$: corrected equivalent strength,a = B/2 if B > 1 m or a = 0,5 m if B < 1 m; b = min {a ; h}

The permissible stress is given by the expression : $\sigma_{perm} = \frac{q_u}{F}$ with F = 3.

The settlement (S) under the footing is given by the oedometric method. In this case, the effective stress is lower than the pre-consolidation pressure, so the overload first causes the recompression of the soil to a voids ratio corresponding to the pre-consolidation pressure, and then it reaches a unequaled level, causing a strong compression of the soil and the development of a deeper settlement, hence the use of both index (compression index and recompression index). The settlement of overconsolidated soil is given by (Robitaille and Tremblays,1997; Tchouani, 1999 ; Callaud M., 2004):

$$S = H_0 \left\{ \frac{C_s}{1+e_0} log \left[ \frac{\sigma'_p}{\sigma'_{v0}} \right] + \frac{C_c}{1+e_0} log \left[ \frac{\sigma'_{v0}+\Delta\sigma'_v}{\sigma'_p} \right] \right\} \qquad (7)$$

$H_0$is initial thickness of the considered layer. The surcharge of the soil above the depth of settlement ($z_i$ is the thickness of the layers above the stress reference point) is given by:

$$\sigma'_{v0} = \sum \gamma_i . z_i \qquad (8)$$

The stress reference point is usually the center of the layer of which settlement is calculated.

$$\Delta'_{\sigma_v} = \Delta_{\sigma'_v}(z) = I_q (9)$$

Stress increase brought by the footing to the z-side identified from the base of the footing.I is the coefficient of influence according to the dimensions of the footing and the depth z and read on charts (Curve for determining the increase of vertical stresses under the corner of a uniformly loaded rectangular surface).q represent the uniform pressure or stress applied by the footing. The soil initial void ratio in place is noted $e_o$. Cc and Cs are respectively the compression index and the recompression index. The preconsolidation pressure is noted$\sigma'_p$ and the coefficients of consolidation $C_v$.

The calculation is made at the corner of the footing and the origin of the depth is taken at the level of the footing.

## III. RESULTS AND DISCUSSION

### *III.1. Field results*

#### *III.1.1. Manual auger sampling results*

Manual auger drilling has allowed us to determine the soil profile in place at a depth of 3 m, and this is presented in figure 1.This section of the soil is identical for the different trial pits that have been made. It shows the homogeneity of the formation that is in place.

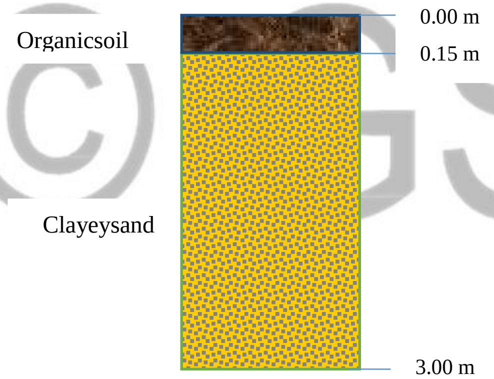



**Figure n°1:** Lithological soil section with manual auger

The results of the static penetrometer testsare given in table n°3. These were processed to obtain equivalent peak resistance, ultimate stress, and the allowable soil stress. The processing was obtained using the following assumptions:

- Square footings of size B = 2 m;
- Penetrometric lift factor Kc taken for soil type A sand from Fascicle 62-V.

The permissible stress was calculated at the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS). To allow for the design using the worst conditions,the SLS values was considered with a factor of safety of 3. The curves resulting from the exploitation of the field results are presented in Figure n°2.

Table n°1 summarizes the peak resistance ranges qc of each trial pit per ground layer traversed. According to this table, it is found that the different layers are of uniform thickness. This shows that it is homogeneous soil. An abnormal peak of resistance at a depth of 6.80 m is observed for CPT1 (Cone Penetration Test 1), a sharp increase in peak resistance from 17.80 m for CPT 2, 3, and 4 until blocking. On the other hand for the CPT 1 there is a decay showing probability of a rocky block which does not extend in all the soil.

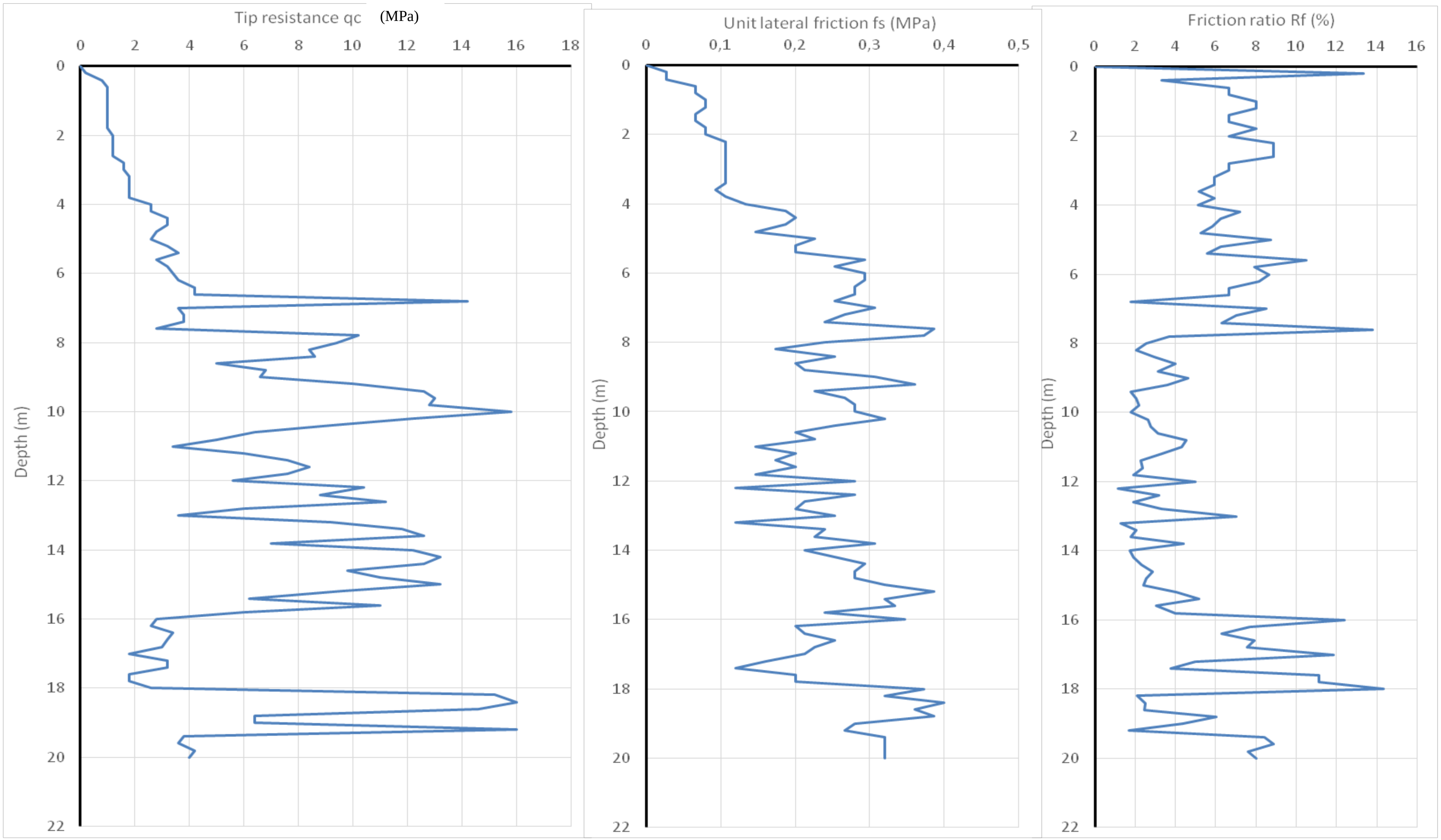



**Figure n°2:** curves of the CPT static penetration test 1

**Table n°1:**Range ofPeak Resistance by boring and perlayer

| N° of boring | Depth (m) | $q_c$ (MPa) | Layers | N° of boring | Prof (m) | $q_c$ (MPa) | Layers |
|---|---|---|---|---|---|---|---|
| CPT 1 | 0.00 – 3.80 | 0.2 – 1.8 | 1st layer | CPT 3 | 0.00 – 4.00 | 0.4 – 2.0 | 1st layer |
| | 3.80 – 7.60 | 2.6 – 14.2 | 2nd layer | | 4.00 – 7.60 | 2.2 – 8.6 | 2nd layer |
| | 7.60 – 9.00 | 5.0 – 10.2 | 3rd layer | | 7.60 – 9.80 | 6.0 – 11.4 | 3rd layer |
| | 9.00 – 10.40 | 9.2 – 15.8 | 4th layer | | 9.80 – 13.00 | 4.0 – 15.0 | 4th layer |
| | 10.40 – 1.,80 | 3.4 – 13.2 | 5th layer | | 13.00 – 15.60 | 9.0 – 15.2 | 5th layer |
| | 15.80 – 18.00 | 1.8 – 3.4 | 6th layer | | 15.60 – 17.80 | 2.6 – 6.0 | 6th layer |
| | 18.00 – 20.00 | 3.6 - 16 | 7th layer | | 17.80 – 19.40 | 9.8 – 26.0 | 7th layer |
| | >20 | Stopping | Stopping | | >19.40 | >26.0 | Refusal |
| CPT 2 | 0.00 – 4.80 | 0.6 – 1.8 | 1st layer | CPT4 | 0.00 – 4.60 | 0.6 – 2.0 | 1st layer |
| | 4.80 – 7.60 | 2.4 – 4.2 | 2nd layer | | 4.60 – 7.60 | 2.2 – 5.8 | 2nd layer |
| | 7.60 – 9.80 | 4.2 – 12.2 | 3rd layer | | 7.60 – 9.80 | 5.6 – 11.4 | 3rd layer |
| | 9.80 – 12.40 | 4.6 – 14.0 | 4th layer | | 9.80 – 12.40 | 4.4 – 14.6 | 4th layer |
| | 12.40 – 15.60 | 8.4 – 17.0 | 5th layer | | 12.40 – 15.60 | 7.2 – 16.6 | 5th layer |
| | 15.60 – 17.80 | 2.8 – 8.2 | 6th layer | | 15.60 – 17.80 | 3.0 – 6.6 | 6th layer |
| | 17.80 – 19.20 | 17.0 – 28.0 | 7th layer | | 17.80 19.60 | 13.4 - 28.0 | 7th layer |
| | >19.20 | >28.0 | Refusal | | >19.60 | >28.0 | Refusal |

The profiles of the average allowable stress obtained as a function of depth are recorded in Table n°2.From this table, the permissible stresses at SLS vary from one survey to another depending on the depth. The sizing of the foundations will be done with the most unfavorable values of the stresses at different depths. The layer to be proposed as the one to be the recommended base of the foundations must have a permissible stress at the serviceable limit state greater than or equal to the total ultimate load.

### *III.2. Results of laboratory tests*

*III.2.1. Results of soil identification tests*

The results of soil identification (water content, specific gravity, particle size analysis and Atterberg limits) are summarized in Table n°2. The unit weight of soil (γ) is $19.27\ kN/m^3$.

The HRB classification is the one used.

**Table n°2:** Summary of soil identification results

| Depth of sampling (m) | Particle size distribution (Sieve openning - mm) | | | | | | Atterberg Limits (%) | | | $\gamma_s$ (T/m$^3$) | ω (%) | IG | HRB Classification | Nature of material |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2.0 | 1.0 | 0.5 | 0.315 | 0.16 | 0.08 | LL | LP | IP | | | | | |
| | Percent passing by weight | | | | | | | | | | | | | |
| 200 – 2.30 | 100 | 90.4 | 76.4 | 57.4 | 43.7 | 36.6 | 54.0 | 29.4 | 24.6 | 2.64 | 19.2 | 4 | A-7-6 (4) | Plastic clayey sand |

The plasticity index indicates a plastic soil. Manual auger drilling has demonstrated the nature of the material as clayey sand, which is confirmed by the HRB classification and is class A-7-6 (4).

*III.2.2. Results of mechanical tests on intact samples*

The results of the shear box test and oedometric tests are shown in Table n°4.

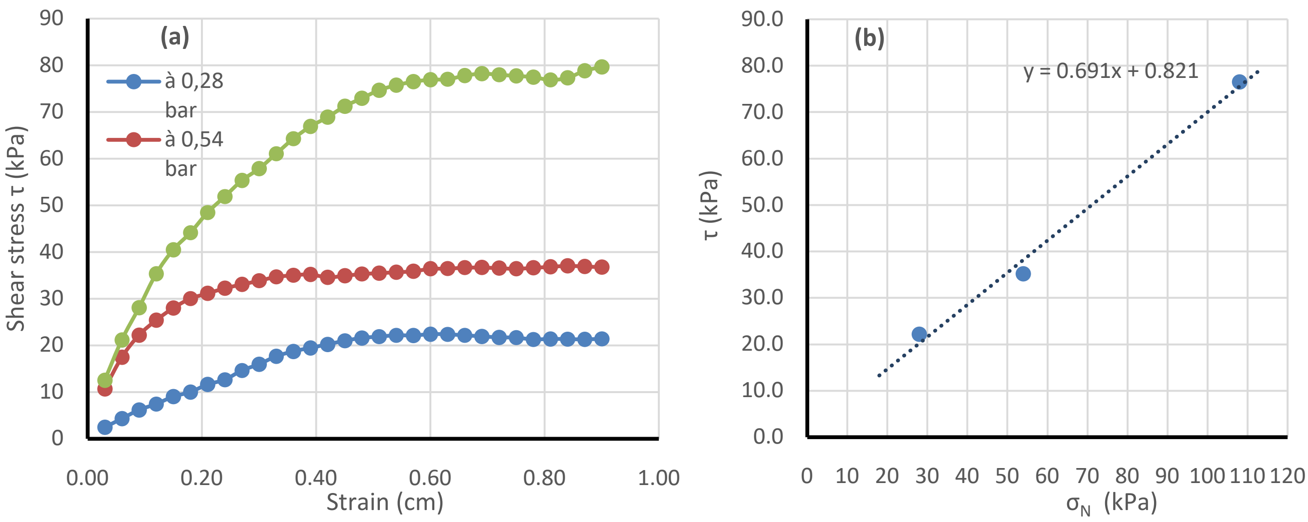
**Table n°4:** Summary of results of mechanical tests on intact sample

| | | |
|---|---|---|
| Sample | Depth (m) | 2.00 – 2.30 |
| | Nature | Yellowish clayey sand |
| Water content (□□□ %) | | 20.8 |
| Unit weight of the solid phase (γs - kN/m$^3$) | | 26.43 |
| Total unit weight ($\gamma_h$ - kN/m$^3$) | | 19.27 |
| Œdometric test | Preconsolidation pressure ($\sigma'_p$ – kPa) | 34 |
| | Effective overburden pressure ($\sigma'_{vo}$ – kPa) | 23 |
| | Initial void ratio $e_o$ | 0.788 |
| | Compression index Cc | 0.06 |
| | Recompression index Cr | 0.01 |
| | Oedometric modulus (kPa) | 23.94 |
| | Permeability coefficient (k - cm/s) | $3.461 \times 10^{-3}$ |
| Straight shear test | Angle of internal friction (φ –degree) | 34.17 |
| | Cohesion ( c –kPa) | 0.821 |

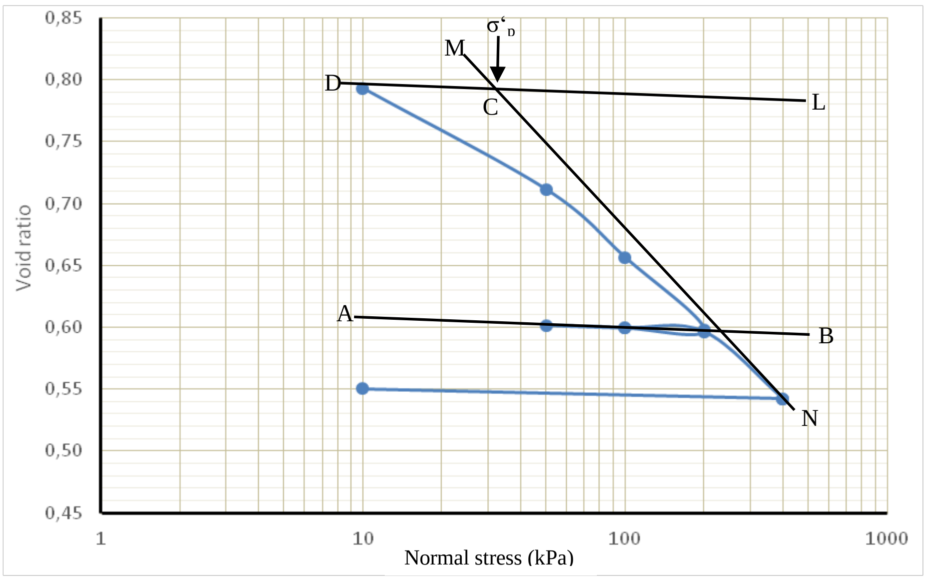
**Table n°3:**Summary of the results of static penetrometer surveys

| Depth (m) | CPT 1 | | | | | CPT 2 | | | | | CPT 3 | | | | | CPT 4 | | | | | Mean $\sigma_{perm}$ (SLS) (MPa) | Mean $\sigma_{perm}$(ULS) (MPa) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $q_{ce}$ (MPa) | Kc | $q_u$ (MPa) | $\sigma_{perm}$ (SLS) (MPa) | $\sigma_{perm}$(ULS) (MPa) | $q_{ce}$ (MPa) | Kc | $q_u$ (MPa) | $\sigma_{perm}$ (SLS) (MPa) | $\sigma_{perm}$ (ULS) (MPa) | $q_{ce}$ (MPa) | Kc | $q_u$ (MPa) | $\sigma_{perm}$ (SLS) (MPa) | $\sigma_{perm}$ (ULS) (MPa) | $q_{ce}$ (MPa) | Kc | $q_u$ (MPa) | $\sigma_{perm}$ (SLS) (MPa) | $\sigma_{perm}$ (ULS) (MPa) | | |
| 1 | 1.34 | 0.15 | 0.21 | 0.069 | 0.104 | 1.16 | 0.16 | 0.18 | 0.060 | 0.090 | 1.50 | 0.16 | 0.24 | 0.080 | 0.120 | 1.33 | 0.16 | 0.21 | 0.069 | 0.104 | 0.070 | 0.105 |
| 2 | 2.01 | 0.16 | 0.33 | 0.109 | 0.163 | 1.57 | 0.17 | 027 | 0.088 | 0.133 | 2.04 | 0.19 | 0.38 | 0.126 | 0189 | 1.82 | 0.17 | 0.31 | 0.102 | 0.153 | 0.106 | 0.160 |
| 3 | 2.62 | 0.17 | 0.45 | 0.148 | 0.223 | 2.05 | 0.18 | 0.36 | 0.121 | 0.181 | 2.58 | 0.21 | 0.54 | 0.180 | 0.271 | 2.32 | 0.18 | 0.41 | 0.137 | 0.206 | 0.147 | 0.220 |
| 4 | 3.85 | 0.17 | 0.67 | 0.222 | 0.333 | 2.52 | 0.18 | 0.46 | 0.154 | 0.231 | 3.41 | 0.23 | 0.80 | 0.266 | 0.400 | 2.96 | 0.18 | 0.54 | 0.180 | 0.270 | 0.205 | 0.308 |
| 5 | 5.01 | 0.18 | 0.90 | 0.299 | 0.449 | 3.47 | 0.18 | 0.64 | 0.213 | 0.320 | 4.46 | 0.26 | 1.15 | 0.385 | 0.577 | 3.94 | 0.19 | 0.73 | 0.243 | 0.365 | 0.285 | 0.428 |
| 6 | 6.29 | 0.18 | 1.16 | 0.386 | 0.578 | 5.29 | 0.18 | 0.96 | 0.320 | 0.480 | 5.84 | 0.28 | 1.65 | 0.552 | 0.827 | 5.62 | 0.18 | 1.04 | 0.345 | 0.518 | 0.401 | 0.601 |
| 7 | 8.78 | 0.50 | 4.39 | 1.463 | 2.195 | 6.76 | 0.50 | 3.38 | 1.127 | 1.690 | 7.82 | 0.50 | 3.91 | 1.303 | 1.955 | 7.34 | 0.50 | 3.67 | 1.223 | 1.835 | 1.279 | 1.919 |
| 8 | 9.78 | 0.50 | 4.89 | 1.630 | 2.445 | 8.60 | 0.50 | 4.30 | 1.433 | 2.149 | 9.23 | 0.50 | 4.62 | 1.538 | 2.308 | 8.95 | 0.50 | 4.48 | 1.492 | 2.238 | 1.523 | 2.285 |
| 9 | 10.05 | 0.50 | 5.03 | 1.675 | 2.513 | 9.70 | 0.50 | 4.85 | 1.617 | 2.425 | 9.91 | 0.50 | 4.96 | 1.652 | 2.478 | 9.82 | 0.50 | 4.91 | 1.637 | 2.455 | 1.645 | 2.468 |
| 10 | 10.55 | 0.50 | 5.28 | 1.758 | 2.638 | 11.01 | 0.50 | 5.51 | 1.835 | 2.753 | 10.80 | 0.50 | 5.40 | 1.800 | 2.700 | 10.93 | 0.50 | 5.47 | 1.822 | 2.733 | 1.804 | 2.706 |
| 11 | 11.42 | 0.50 | 5.71 | 1.903 | 2.855 | 12.43 | 0.50 | 6.22 | 2.072 | 3.108 | 11.94 | 0.50 | 5.97 | 1.990 | 2.985 | 12.21 | 0.50 | 6.11 | 2.035 | 3.053 | 2.000 | 3.000 |
| 12 | 11.20 | 0.50 | 5.60 | 1.867 | 2.800 | 14.23 | 0.50 | 7.12 | 2.372 | 3.558 | 12.73 | 0.50 | 6.37 | 2.122 | 3.183 | 13.50 | 0.50 | 6.75 | 2.250 | 3.375 | 2.153 | 3.229 |
| 13 | 11.17 | 0.50 | 5.59 | 1.862 | 2.793 | 14.10 | 0.50 | 7.05 | 2.350 | 3.525 | 12.65 | 0.50 | 6.33 | 2.108 | 3.163 | 13.40 | 0.50 | 6.70 | 2.233 | 3.350 | 2.138 | 3.208 |
| 14 | 10.11 | 0.50 | 5.06 | 1.685 | 2.528 | 13.19 | 0.50 | 6.59 | 2.198 | 3.296 | 11.67 | 0.50 | 5.84 | 1.945 | 2.918 | 12.46 | 0.50 | 6.23 | 2.077 | 3.115 | 1.976 | 2.964 |
| 15 | 8.74 | 0.50 | 4.37 | 1.457 | 2.185 | 12.37 | 0.50 | 6.18 | 2.061 | 3.091 | 10.59 | 0.50 | 5.30 | 1.765 | 2.648 | 11.50 | 0.50 | 5.75 | 1.917 | 2.875 | 1.800 | 2.700 |
| 16 | 9.03 | 0.50 | 4.52 | 1.505 | 2.258 | 15.09 | 0.50 | 7.54 | 2.514 | 3.771 | 12.17 | 0.50 | 6.09 | 2.028 | 3.043 | 13.64 | 0.50 | 6.82 | 2.273 | 3.410 | 2.080 | 3.120 |
| 17 | 7.62 | 0.50 | 3.81 | 1.270 | 1.905 | 18.37 | 0.50 | 9.18 | 3.061 | 4.591 | 14.80 | 0.50 | 7.40 | 2.467 | 3.700 | 16.84 | 0.50 | 8.42 | 2.807 | 4.210 | 2.401 | 3.602 |
| 18 | 5.84 | 0.50 | 2.92 | 0.973 | 1.460 | 15.99 | 0.50 | 7.99 | 2.664 | 3.996 | 12.72 | 0.50 | 6.36 | 2.120 | 3.180 | 14.61 | 0.50 | 7.31 | 2.435 | 3.653 | 2.048 | 3.072 |
| 19 | 5.14 | 0.50 | 2.57 | 0.857 | 1.285 | 15.05 | 0.50 | 7.53 | 2.508 | 3.763 | 11.90 | 0.50 | 5.95 | 1.983 | 2.975 | 13.73 | 0.50 | 6.87 | 2.288 | 3.433 | 1.909 | 2.864 |
| 20 | 4.51 | 0.50 | 2.26 | 0.752 | 1.128 | 13.30 | 0.50 | 6.65 | 2,217 | 3.325 | 10.70 | 0.50 | 5.35 | 1.783 | 2.675 | 12.25 | 0.50 | 6.13 | 2.042 | 3.063 | 1.698 | 2.548 |

The results obtained during these tests allow to calculate the settlement. The curve related to the shear test is given in Figure n°3 and that of the compressibility curve in Figure n°4.




**Figuren°3:** Shear test results:(a) Shear stress curve, (b) Shear stress line




**Figuren°4: Curve of compressibility**

***III.3. Foundation sizing and settlement computation***

*III.3.1. Dimensioning of foundations*

➢ *Primary data*

As in the work of Kognonsa (2004), it will be considered in this work the following parameters for the design and calculation of foundations:

- shallow foundation with isolated footing;
- post of width b = 20 cm and length l = 30 cm;
- service load at SLS Pser = 610.60 kN;
- the characteristic strength of concrete at 28 days is 25.00 MPa.

Foundation sizing will be done according to the following principles:

- Standard for geotechnical calculations: DTU 13.12
- Standard for reinforced concrete calculations: BAEL 91 mod. 99

➢ *Penetrometer case*

The calculation of the shallow foundations was made between 1 and 6 m deep ($\sigma_{perm}$acceptable from 5m). The allowable SLS stress vary with depth (Table n°5) and the lowest values of the results were retained; therefore, the foundation calculation will be done with these results. The results of the foundation calculation at SLS are shown in Table n°5.

**Table n°5:** Results of the calculation of the dimensions of the footings

| Depth z (m) | $\sigma_{perm}$(SLS) (MPa) | Pser (kN) | b (m) | l (m) | footings | | | | | Pser/S (MPa) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | B (m) | L (m) | S (m²) | d (m) | h (m) | |
| 1 | 0.067 | 610.6 | 0.20 | 0.30 | 2.50 | 3.75 | 9.38 | 0.86 | 0.95 | 0.065 |
| 2 | 0.101 | 610.6 | 0.20 | 0.30 | 2.00 | 3.05 | 6.10 | 0.69 | 0.75 | 0.100 |
| 3 | 0.140 | 610.6 | 0.20 | 0.30 | 1.70 | 2.60 | 4.42 | 0.58 | 0.65 | 0.138 |
| 4 | 0.179 | 610.6 | 0.20 | 0.30 | 1.55 | 2.30 | 3.57 | 0.50 | 0.55 | 0.171 |
| 5 | 0.245 | 610.6 | 0.20 | 0.30 | 1.30 | 1.95 | 2.54 | 0.41 | 0.50 | 0.241 |
| 6 | 0.359 | 610.6 | 0.20 | 0.30 | 1.10 | 1.60 | 1.76 | 0.33 | 0.40 | 0.347 |

In view of the low values of the permissible stress (Table n°5), the dimensions of the footings are very large, which is not advantageous. At 5 m depth, $\sigma_{perm}$ = 0.245 MPa and the dimensions of the footings are acceptable and the safety is verified thanks to relation $\frac{P_{ser}}{S} \leq \sigma_{perm}$ ; nevertheless shallow foundations cannot be designedat such depth, as in the work of Hassan (2010). So these soils do not easily permit the use ofshallow foundations. At least to make an improvement with viabilitymaterial as the case of the pozzolan present in the neighbourhood.

*III.3.2. Calculation of settlement*

The results of the settlement calculation are summarized in Table 6.

**Table n° 6:** Settlement computation results

| | | | | |
|---|---|---|---|---|
| **Width B (m)** | 1.30 | | | |
| **Length L (m)** | 1.95 | | | |
| **Depth Z (m)** | 1.00 | 2.00 | 3.00 | 4.00 |
| **B/Z** | 1.30 | 0.65 | 0.43 | 0.33 |
| **L/Z** | 1.95 | 0.98 | 0.65 | 0.49 |
| **Influence coefficient** | 0.173 | 0.135 | 0.082 | 0.032 |
| **Pressure q (kPa)** | 240.39 | | | |
| **$\sigma'v_0$ (kPa)** | 19.27 | 38.54 | 57.81 | 77.08 |
| **$\Delta\sigma'v$ (kPa)** | 41.59 | 32.45 | 19.71 | 7.69 |
| **Settlement S (cm)** | 2.27 | 4.80 | 7.41 | 10.43 |

The calculation of settlement bellow the base of foundation gives values which increase with depth rightup to 2 m under the footing, and these values are acceptable, whereas from 3 meters they are not. The settlements are acceptable at 2 m because the maximum value for settlement under isolated footing is 6.50 cm.

*III.4.1. Technical discussion*

Soil surveys were carried out by the mechanical point penetrometer at a depth of 20 m. The lithology of the site presents seven layers of soil. The thicknesses of each of the layers as well as their peak resistance ranges are given in table n°7.

**Table n°7:** Summary of Soil Bearings per Layer

| Layers | Depth (m) | qc (Mpa) | $\sigma_{perm}$atSLS (Mpa) |
|---|---|---|---|
| 1st layer | 0.00 – 4.00 | 0.4 – 2.0 | 0.060 – 0.266 |
| 2nd layer | 4.00 – 7.60 | 2.2 – 8.6 | 0.213 – 1.463 |
| 3rd layer | 7.60 – 9.80 | 6.0 – 11.4 | 1.432 – 1.835 |
| 4th layer | 9.80 – 13.00 | 4.0 – 15.0 | 1.903 – 2.350 |
| 5th layer | 13.00 – 15.60 | 9.0 – 15.2 | 1.685 – 2.514 |
| 6th layer | 15.60 – 17.80 | 2.6 – 6.6 | 1.505 – 3.061 |
| 7th layer | 17.80 – 19.60 | 3.6 – 28.0 | 0.751 – 2.664 |

According to Table n°7, the first layer of soil has low values of permissible stress at SLS, despite the fact that some trial pits have some acceptable values (0.266 MPa) at 4 meters depth. It appears that this layer is not favorable for the sizing of the footings. This leads to a depth of foundation calculation of up to 6 meters in the soil (acceptable $\sigma_{perm}$) with optimum dimensions being (1.30 m x 1.95 m) for the footings (Table n°5). This was done for a minimum admissible stress $\sigma_{perm}$ = 0.245 MPa and safety is respected ($\frac{P_{ser}}{S} \leq \sigma_{perm}$ ).

The optimization of these dimensions led to the calculation of settlements (Table n°6). At2 metersdeep, it is estimated at 4.80 cm. According to the recommendations of the Fourth Congress International of Soil Mechanics, in London in 1956, this settlement (global settlement), is acceptable for any type of work.

Permissible stresses increase overall with depth (figure n°5). Between 1 and 6 m depth, the increase is low (from 0 to 0.4 MPa for SLS and from 0 to 0.6 for ULS); at 6 m, an abrupt increase occurs up to 7 m (0.4 to 1.35 MPa for SLS and 0.6 to 1.9 MPa for ULS); between 7 and 12 m, the admissible stresses increase moderately (from 1.35 to 2.1 MPa for SLS and 1.9 to 3.25 MPa for ULS); between 12 and 15 m, the stresses decrease rather (from 2.1 to 1.6 MPa for the SLS and from 3.25 to 2.6 MPa for the ULS); between 15 and 17 m, the stresses increase sufficiently (from 1.6 to 2.25 MPa for SLS and from 2.6 to 3.6 MPa for ULS); at 17 m the stresses decrease to 20 m (from 2.25 to 1.7 MPa for the SLS and from 3.6 to 2.5 MPa for ULS)

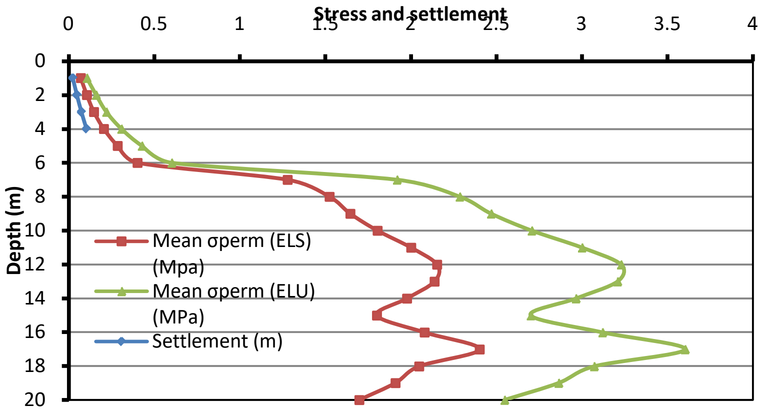



**Figure n°5 :** Stress and settlement as a function of depth

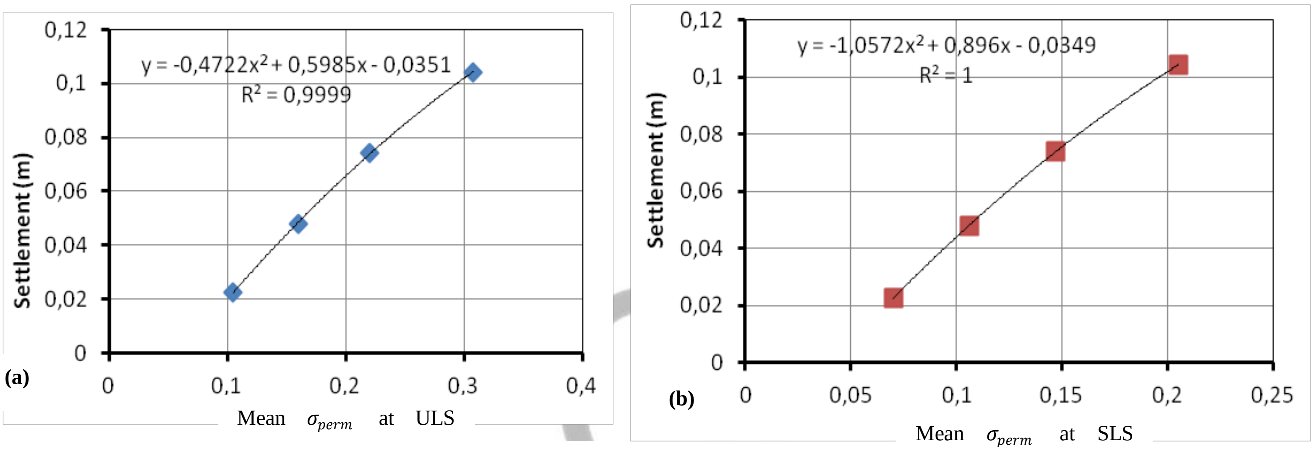



**Figure n°6:** correlation between settlement under footing and the permissible stress at (a) ULS and (b) SLS

Settlement under a shallow foundation (between 1 and 4 m deep) increases relatively with the permissible stress. Figures n°5 and n°6 show that the correlation which exists between the permissible stress and the settlement is a polynomial of order 2. The correlation is practically perfect. It is expressed with a correlation coefficient of 1.000 for soils at the serviceability limit state and 0.999 for soils at the ultimate limit state under a shallow foundation.

The variation of the stresses or settlements of soils under a foundation is quite common in many research works such as those of Corneille (2007) on the mechanical behavior of the ballasted columns loaded on rigid footings, as well Ali et al, (2010) on the pile limit load and settlement estimation from the pile load test, of Bouassida et al (2003) on the settlement of a rigid foundation soil carrying a reinforced concrete column.

## Conclusion

This study, as initially defined, was aimed primarily at determining the soil bearing pressure and compressibility of the different soil layers of Kotto, located in the north-east of the city of Douala (Cameroon). In order to achieve this objective, 4 soil tests were carried using the SPT to a depth of about 20 m. This study shows that the site has a variable lithology with 7 layers of distinct resistances. The results of the identification tests show a fairly homogeneous soil consisting essentially of clayey sand, whose class is A-7-6 (4) according to the HRB classification.

It emerges that the first layer of soil has a low soil bearing pressure ranging from 0.6 bar to 1.54 bar (minimum value). These values show that this layer cannot bear isolated spreadfootings.

The foundation calculation was thus made up to 6 meters in the soil and the results obtained were satisfactory between 5 and 6 meters. Associated with the results of the oedometric test, the settlement calculation gave a value of 4.80 cm at 2 meters under the base of the footing.

The calculation of the foundations has given very large dimensions in the first layer which is not acceptable from the economic point of view. In order to implement the dimensions obtained (1.30 mx 1.95 m), it is recommended, from a technical point of view, to avoid the use of deep foundations, to make use of pozzolanic gravelmechanical stabilization present at the vicinity to improve upon the bearing capacity of the soil.

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