



(RESEARCH ARTICLE)



## COMPARATIVE EVALUATION OF DIFFERENTIAL SETTLEMENT AND LOAD-TRANSFER MECHANISMS BETWEEN RAFT AND DEEP PILE FOUNDATIONS OF CONSTRUCTION PROJECTS IN SOFT SOILS: APPLICATION TO THE BIIC (R+5) BUILDING IN COTONOU

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### ABSTRACT

Urban development along the Beninese coastline involves medium-rise projects in areas with sandy formations of low surface compaction and a shallow water table, necessitating a complex foundation system. This study presents a comparative evaluation of geotechnical performance, execution constraints, and economic impacts between a general raft foundation and pile foundations for the BIIC building (R+5) of the Financial City of Cotonou. The methodology is based on a G3-type geotechnical characterization derived from pressuremeter and core drilling data up to 25 m, followed by dimensioning according to the NF P 94-261 and NF P 94-262 standards, respectively for shallow and deep foundations. The design profile reveals a shallow layer of loose sand (equivalent net limit pressure  $Pl^* = 0.39$  MPa) beneath a stabilized water table at a depth of 1.99 m, overlying a very dense horizon ( $Pl^* = 3.47$  MPa) at a depth of 3.50 m. The results indicate that the general foundation slab transmits a stress of 232.80 kPa and has an allowable ultimate limit state (ULS) bearing capacity of 1006.48 kPa, but induces a maximum settlement of 4.81 cm. Conversely, piles with a diameter of 0.80 m, anchored at 8.50 m, limit the settlement to 0.80 cm and provide a bearing capacity of 2546.23 kN. Although both options meet regulatory bearing capacity requirements, the decision analysis shows that the optimal choice depends on site constraints. The general foundation slab requires significant management of groundwater inflows (groundwater drawdown). Before soil improvement, the pile solution, although more expensive in terms of equipment, offers superior structural safety and eliminates the hydraulic risks associated with excavation.

**Keywords:** Foundation; Raft Foundation; Bored Piles; Load-transfer Settlement.

## 1 INTRODUCTION

In Benin, rapid urban expansion on the coastal strip forces the construction of medium-height structures on sandy sedimentary formations with low compactness, characterized by a subsurface water table [1] and [2]. In such a geotechnical context, the choice of foundation system is a determining factor for structural stability. It is a matter of arbitrating between shallow foundations of the general raft type, which optimize the distribution of surface stresses, and deep foundations on piles, which ensure the transfer of loads to mechanically more competent geological horizons. This study is part of the project to construct the R+5 building of the International Bank for Industry and Commerce (BIIC), located in the Financial City of Cotonou. Based on in situ geotechnical investigations carried out on the site, including investigation boreholes associated with Menard pressuremeter tests (PMT) up to 25 meters deep, this work proposes a rigorous comparative analysis between a general raft option and a variant on deep foundations. The central issue is: to what extent do these two types of foundations meet the load-bearing and deformability requirements of the BIIC project (R+5), given the site's specific lithological and hydrogeological constraints? The overall objective of this study is to evaluate the mechanical behavior and technical performance of a general raft foundation in relation to a pile foundation system, to identify the optimal solution validating the safety and durability criteria of the structure, in accordance with the requirements of Eurocode 7 and its national application standards [3] and [4].

## 2 SITUATION OF THE STUDY AREA

### 2.1 Location and geotechnical framework of the study area

The study site is located within the perimeter of the Cotonou Financial City project, a large-scale real estate complex for tertiary activities currently under development. The infrastructure consists of three main building complexes: the BIIC building (R+5), the CAA-CDCB complex (R+7), and the SIMAU building (R+7). This comparative study is limited to the BIIC building. The input data is based on the geotechnical engineering execution study (type G3, according to [5], entrusted to the engineering firm AGTS Benin and finalized during the year 2025. From a geographical and environmental perspective, the project site is located in the port area of Cotonou, bordered to the west by the Autonomous Port of Cotonou. The site is immediately adjacent to a national interstate highway, characterized by heavy traffic with high aggressiveness, inducing continuous dynamic and vibratory stresses on the surrounding environment, as shown in Figure 1.



Figure 1: Overview of the study area

## 3 MATERIAL

### 3.1 Investigation program

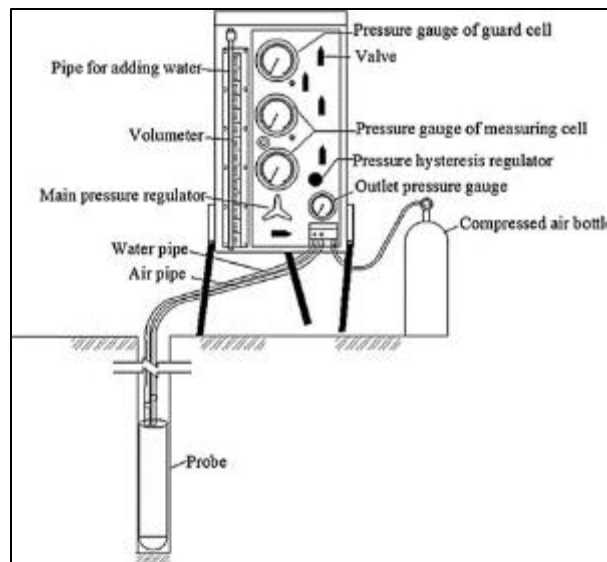
The data used for this study comes from the type G3 geotechnical execution study report [6]. They were acquired using pressuremeter tests and core drilling investigation techniques carried out to a depth of 25.00 m

### 3.1.1 Menard pressuremeter test

Performed according to the specifications of standard [7], this test allows the in-situ characterization of the pseudo-elastic and rupture properties of the horizons traversed. The protocol consists of applying uniform radial pressure increments to the wall of a previously drilled borehole, using a three-cell expandable probe (central measuring cell and guard cells) as shown in Figure 2. Recording the volumetric deformation of the probe as a function of the effective stress applied allows the rheological curve of the soil to be established.

The analysis of the pressuregram allows us to extract two fundamental parameters:

- The Pressuremeter Modulus (PM) helps to calculate the pseudo-elastic behavior phase (linear phase) to characterize the rigidity of the soil in the face of deviatoric stresses.
- The net limit pressure ( $p_l^*$ ) is determined by asymptotic extrapolation to volume infinity (doubling the initial volume of the cavity). It quantifies the ultimate bearing capacity and the shear strength of the soil matrix.



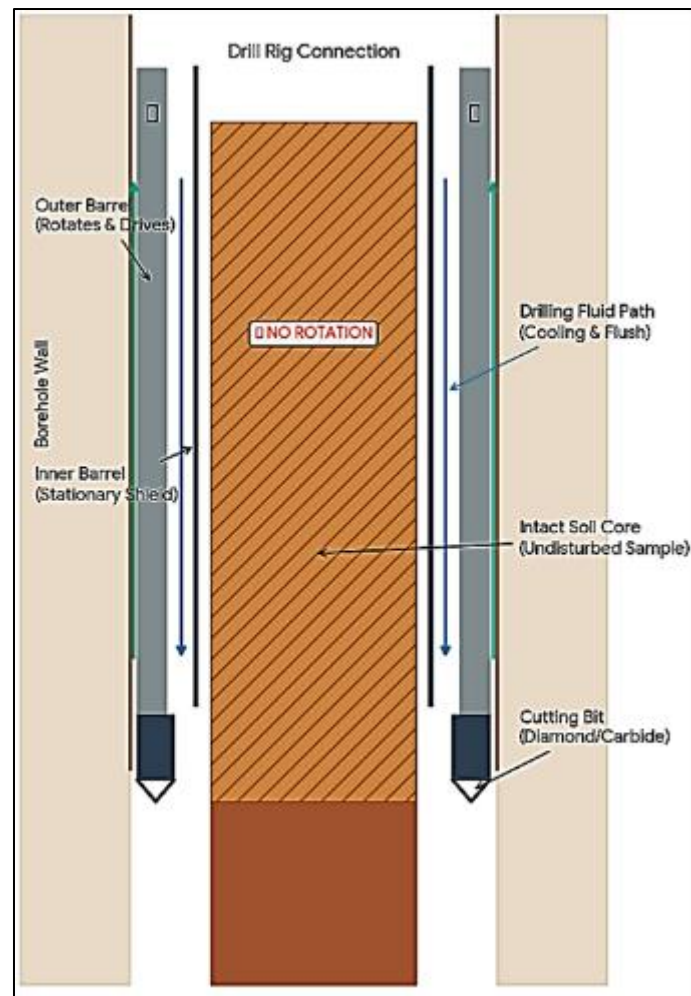
**Figure 2: Schematic diagram of the Menard pressuremeter test and apparatus [8]**

### 3.1.2 Sampling by rotary core drilling

Core drilling, carried out according to the recommendations of standard [9], aims to extract intact and continuous soil samples. This rotary drilling technique employs a core barrel fitted with a diamond bit that cuts through rock or cohesive soil to extract a cylinder (the core). This method allows for the establishment of a precise lithological cross-section of the different layers traversed and for the collection of intact samples for laboratory identification tests (particle size distribution, Atterberg limits, direct shear). Continuous core drilling aims at the extraction of samples of undisturbed soils

The process relies on peripheral destructive rotary drilling, where the cutting of the soil is ensured by a diamond core bit fixed to the end of a core barrel as shown in Figure 3. This mechanical action allows for the continuous recovery of coaxial cylinders of ground material, known as cores.

The analysis of these cores makes it possible to determine with accuracy the fine stratigraphy and the lithological profile of the horizons crossed and to have undisturbed samples essential for the characterization of intrinsic properties in the laboratory. These tests consist of identification of state parameters, particle size analysis by sedimentation or sieving, Atterberg limits, natural water content, and density. The samples are also used to determine mechanical behavior parameters such as direct shear strength, compressibility characteristics, etc.



**Figure 3: Diagram of the core sampling principle**

### 3.1.3 In-situ Lefranc permeability test

The evaluation of the equivalent permeability coefficient of the surface and deep horizons is carried out in situ by means of Lefranc tests, in accordance with the protocol requirements of standard [10]. This hydrogeological test is based on the injection of water under constant or variable hydraulic head into an injection cavity (or lantern) made at the bottom of the borehole, as shown in Figure 4. Quantifying the absorption rate or the dissipation kinetics of the load allows the calculation of the permeability coefficient ( $k$ ), which is essential for sizing groundwater lowering devices and for modeling transient flows during the pumping phases for foundation work.

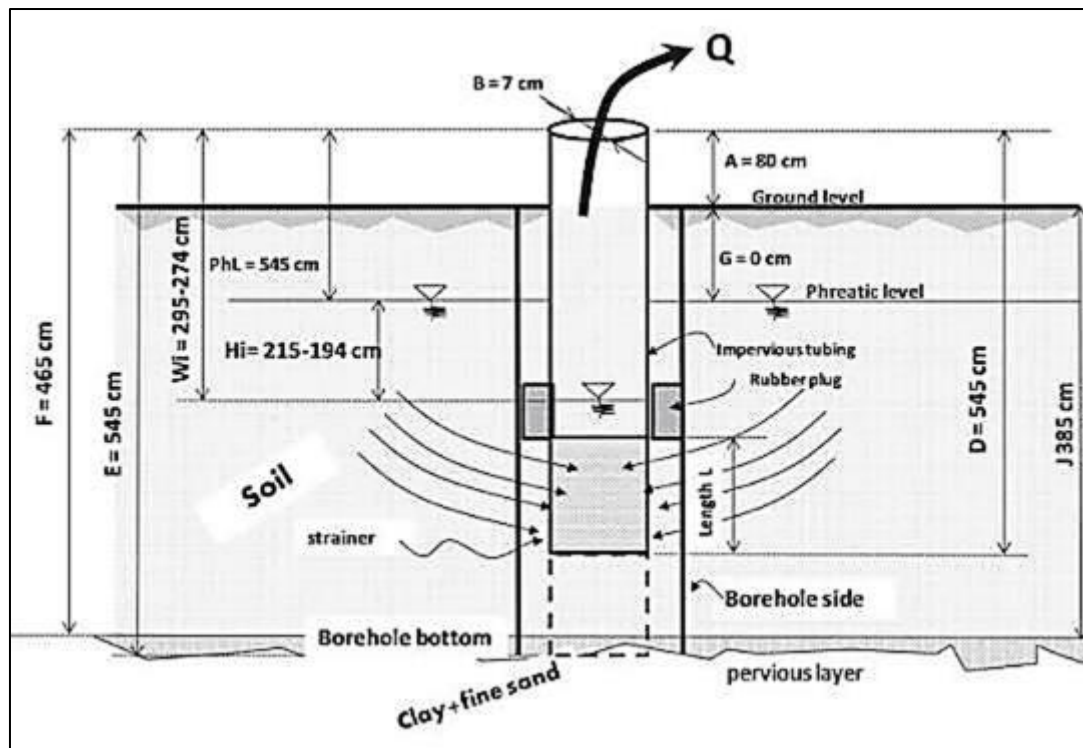


Figure 4: Diagram of the principle of the Lefranc permeability test. [11]

## 4 METHODS

### 4.1 Dimensioning method

The comparative analysis of foundation solutions is based exclusively on the pressuremeter method, guaranteeing the homogeneity of the calculations with the study of the raft foundation carried out by the design office. This approach directly exploits the rupture parameters (net limit pressure  $p_l^*$  and deformation (pressuremeter modulus from in situ tests. It thus avoids the bias of empirical correlations and adapts to the sandy horizons of the site according to the regulatory charts of the standards

#### 4.1.1 Design of the general raft foundation

The raft (42.00 m × 42.00 m) is modeled using soil-structure interaction analysis via the Tasplaq software. The thickness of the slab is dimensioned with respect to shear and the relative stiffness criterion. The latter ensures rigid structural behavior and a linear (planar) distribution of contact stresses. This criterion is defined by  $B \leq 2L_0$  where:

$$L_0 = \left( \frac{EI}{k} \right)^{1/4} \quad (1)$$

Where  $B$  is the span between columns,  $L_0$  the transfer length,  $EI$  the flexural stiffness of the foundation slab, and  $k$  the soil reaction coefficient.

The geotechnical performance was verified by comparing the contact stress  $q'_{ref}$  to the allowable stresses (ULS and SLS), and by ensuring that the angular distortion  $\Omega_{3D}$  remains below the threshold of 1/500.

The preliminary sizing of the thickness first verifies the absolute minimum thickness condition imposed by Fascicle 62 ( $h \geq 0.50$  m), then the shear condition of BAEL 91 :

$$\tau_u = V_u(B * 0,9h) \leq \bar{\tau} = 0,05 * f_{c28} \quad (2)$$

where  $V_u$  is the design shear force and  $b$  is the design unit width (1.00 m). The reinforcement of the foundation slab is then determined according to the ultimate limit state method, assimilating each calculation strip to a continuous beam of unit width subjected to the ultimate bending moment  $M_u$  from the Tasplaq modeling. The reduced moment is given by the formula:

$$\mu_u = M_u(b * d^2 * f_{bu}) \quad (3)$$

Compared to the limiting reduced moment  $\mu_l = 0.392$  (FeE500 steel), it allows verification that no compression reinforcement is required. The position of the neutral axis leads to the lever arm is:

$$z = d(1 - 0,4\alpha) \quad (4)$$

Hence the required steel section is:

$$A_s = M_u / (z * f_{ed}) \quad (5)$$

This value is compared to the minimum section ( $A_{min}$ ) imposed by the non-brittleness condition (BAEL art. A.4.2.1).

$$A_{min} = 0,23 b * d * f_{t28} / f_e \quad (6)$$

The selected reinforcement corresponds to the larger of the two values from the preceding formulas

#### 4.1.2 Dimensioning of pile foundations

The site context (water table at 1.99 m, urban environment) led to the choice of bored cased piles with recovered casing . Their load-bearing capacity is evaluated by summing the peak and lateral friction resistances according to the pressuremeter model.

$$R_c = R_b + R_s \quad (7)$$

The design resistance  $R_{c,d}$  is obtained by weighting  $R_c$  by the model coefficients  $\gamma_{R;d1}, \gamma_{R;d2}$ . A negative friction  $F_{n;d}$  generated by the settlement of the embankment and the loose layer was calculated and added to the loads of the structure. Verification at the Ultimate Limit State (ULS) therefore involves ensuring that the final load per pile satisfies the following condition:

$$\frac{N_{ELU} + n \times F_{n;d}}{n} \leq R_{c,d} \quad (8)$$

Where  $n$  is the number of piles and  $N_{ELU}$  is the ultimate axial load. The SLS verification showed that the load remained below the critical creep load ,  $Q_c$ , guaranteeing operation in the elastic range.

## 5 RESULTS

### 5.1 Interpretation of the geotechnical profile

As shown in Figure 5, the pressuremeter tests (SP5/SP6) and core drilling (SC5/SC6) revealed a strong mechanical dichotomy in the first 25 meters. The surface layer (0 to 2.50 m) is a loose, weak sand ( $Pl^* = 0.39$  MPa,  $EM = 1.25$  MPa) whose low recovery rates (50-55%) reflect insufficient compactness. This layer is saturated by a stagnant water table at a depth of 1.99 m. Below 3.50 m, the profile shows a clear increase in resistance with a very dense sand ( $Pl^* = 3.47$  MPa,  $EM = 31.70$  MPa), constituting the main bearing horizon. Further tests as Lefranc permeability ( $K = 2.152 \times 10^{-6}$  m/s), confirm a permeable sandy site and chemically low aggressiveness (class XA1).

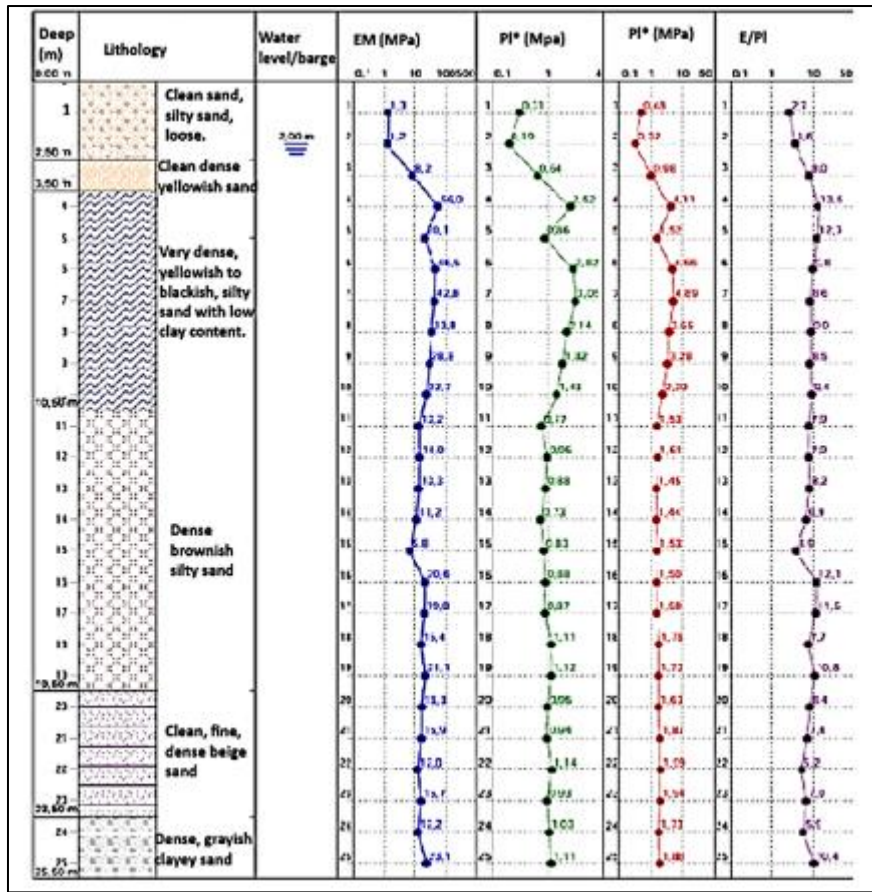


Figure 5: Results of the variation of the limit pressure PI and the E/PI ratio with depth.

## 5.2 General foundation slab dimensioning

The Tasplaq modeling analysis shows that the thickness of 0.90 m in the column zone is the dimensioning condition, allowing the rigidity criterion ( $B = 7.70 \text{ m} \leq 2L0 = 10.59 \text{ m}$ ). From the point of view of bearing capacity, the foundation slab is largely oversized. The contact stress (232.80 kPa) only mobilizes 23% of the allowable capacity of the soil at the ULS (1006.48 kPa). However, the analysis of the deformations reveals a vulnerability. The absolute settlement reaches 4.81 cm, remaining below the permissible threshold of 5.00 cm retained for rigid shallow foundations, and the maximum angular distortion is  $\Omega 3D = 1/450$ . This value exceeds the normative threshold of  $1/500$  recommended for rigid structures, highlighting the sensitivity of the foundation slab to differential settlements generated by the compressibility of the surface layer.

The preliminary dimensioning confirms a thickness of 0.50 m in the normal zone and 0.90 m in the loaded zone, with a shear stress  $\tau_u = 0.601 \text{ MPa}$  much lower than the regulatory limit  $\bar{\tau} = 1.50 \text{ MPa}$ , leaving a margin of 0.899 MPa. The reinforcement associated with the loaded zone ( $h = 0.90 \text{ m}$ ), installed independently along the two foundation slab directions (X-X and Y-Y) and at the two characteristic bending positions (span and support), gives results in the following steel sections as shown in Table 1

Table 1: Foundation slab reinforcement sections in the loaded zone ( $h = 0.90 \text{ m}$ )

| Case         | $M_u$ (kN·m/m) | $\mu_u$ | $z$ (m) | $A_s$ calculated/(cm <sup>2</sup> /m) | Bars, spacing       |
|--------------|----------------|---------|---------|---------------------------------------|---------------------|
| X-X, span    | 1570.30        | 0.1278  | 0.7917  | 45.61 / 45.61                         | 10 HA25, St = 10 cm |
| X-X, support | 586.50         | 0.0477  | 0.8293  | 16.27 / 16.27                         | 7 HA20, St = 14 cm  |
| Y-Y, span    | 875.40         | 0.1527  | 0.7792  | 55.37 / 55.37                         | 7 HA32, St = 14 cm  |
| Y-Y, support | 229.90         | 0.0187  | 0.8420  | 6.28 / 9.38 ( $A_s$ , min)            | 7 HA14, St = 14 cm  |

The most unfavorable moment, in span Y-Y (875.40 kN·m/ml), leads to the largest steel section. At the Y-Y support, the lower stress causes the dimensioning to shift to the minimum non-brittle section rather than to the calculation in bending. All the selected spacings comply with the condition  $St \leq \min(3h; 33 \text{ cm}) = 33 \text{ cm}$  imposed by BAEL 91

### 5.3 Evaluation of the performance of pile foundations

The pile solution with 0.80 m diameter and 8.50 m length is anchored at 5.00 m in the dense layer, which allows a design bearing capacity of  $R_{c,d} = 2546.23$  kN. The negative friction induced by the settlement of the embankment and the loose layer was estimated at 117.53 kN per pile; This phenomenon remains marginal and does not compromise load-bearing capacity. To the right of the most stressed post (26R,  $N_{ELU} = 4658.38$  kN), a group of 3 piles is required. The design load per pile (1,670.32 kN) satisfies the Ultimate Limit State (ULS) with a comfortable safety margin. At the SLS, the creep ratio  $N/Q_c = 0.562$  (less than 0.70) confirms strictly elastic behavior, which results in a very low estimated settlement of 0.80 cm.

### 5.4 Comparative performance summary

Although the foundation slab formally meets the absolute settlement threshold (5.00 cm), it still uses 96% of its permissible margin, compared to only 32% for the pile solution, which reflects a significantly smaller safety reserve against deformations, consistent with the joint exceedance observed on the angular distortion. The results show that while both solutions respect the ultimate limit states, their behavior differs fundamentally. The foundation slab offers a large reserve of bearing capacity but remains sensitive to deformations of the surface layer, while the piles are completely independent of this layer to guarantee the dimensional stability of the structure. To guide decision-making based on the three pillars (Technical, Execution, Economic), the comparative matrix is shown in Table 2.

**Table 2: Mat Foundation vs. Piled Foundation (FTR)**

| Performance Criterion         | Mat Foundation (Raft)                   | Piled Foundation                                       | Optimal solution and justification                                    |
|-------------------------------|-----------------------------------------|--------------------------------------------------------|-----------------------------------------------------------------------|
| Bearing Capacity Mobilization | 23%<br>(232.80/1,006.48 kPa)            | 65%<br>(1,670 / 2,546 kN)                              | Piles: Better optimization and utilization of bearing capacity.       |
| Allowable Settlement          | 96% of the limit<br>(4.81 / 5.00 cm)    | 32% of the limit<br>(0.80 / 2.50 cm)                   | Piles: Maximum safety with a very clear margin of comfort.            |
| Absolute Settlement           | 4.81 cm                                 | 0.80 cm                                                | Piles: Drastic reduction in deformation (divided by 6).               |
| Angular Distortion            | 1/450 (Exceeds the 1/500 limit)         | Non-critical                                           | Piles: Eliminates the risk of structural damage due to differentials. |
| Sensitivity to Groundwater    | High (Requires dewatering/pumping)      | Low (FTR technique adapted to saturated soils)         | Piles: Secured construction site against hydraulic risks.             |
| Execution Complexity          | Low (Standard earthwork and concreting) | Medium (Drilling, casing, and concreting under shroud) | Mat: Simpler, faster, and technically less demanding process.         |

#### Limitations

The raft foundation presents a major structural risk. Its angular distortion exceeds the safety limit, and its settlement reaches 96% of the permissible value, even though it is utilizing only 23% of its bearing capacity. These high values necessitate more in-depth testing, such as triaxial tests and construction measures for soil densification, which were not examined in this study. The study reveals that the high groundwater level requires intensive dewatering. A lack of specific geotechnical data across the full depth could lead to unquantified structural, environmental, and efficiency-related risks for both raft foundations and pile foundations.

## 6 CONCLUSION

This comparative geotechnical study has evaluated the raft and the pile foundation for the BIIC building in the Financial City of Cotonou. Pressuremeter tests reveal a marked bilayer stratigraphy beneath an outcropping water table (-1.99 m), suggesting a saturated soil. The compressible surface layer of  $PI^* = 0.39$  MPa overlies a dense and efficient horizon of  $PI^* = 3.47$  MPa. Modeling at the ultimate and serviceability limit states (ULS/SLS), in accordance with NF P 94-261 and NF P 94-262 Standards. The design confirms the technical viability of both systems. The pile option eliminates the risk of surface settlement, while the more economical raft foundation remains viable only with rigorous quality control during execution, particularly regarding the compaction of backfill in untreated granular materials. This study developed an approach that directly protects human lives and prevents catastrophic structural failures in a high-density

urban area such as Cotonou. It also offers a clear choice between a lower-cost option (raft) and a very low-settlement option (piles), allowing decision-makers to optimize public or private investments. Finally, this study enriches the regional engineering database with high-quality local pressuremeter data, providing a reliable reference for future tower projects in the area.

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## STATEMENTS ON COMPLIANCE WITH ETHICAL STANDARDS

### *Disclosure of conflict of interest*

No conflict of interest to be disclosed

### *Statement of informed consent*

Informed consent was obtained from all individual participants included in the study

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## REFERENCES

- [1] OBRGM & DGM. Geological map of Benin at 1:200,000. Cotonou: OBRGM & DGM, 2007.
- [2] Laïbi, A. Physico-chemical and geotechnical characterization of two clay sites in Benin with a view to their valorization in eco-construction. *International Journal of Biological and Chemical Sciences*, 2017, 11, 500–514.
- [3] AFNOR. NF P 94-261. Justification of geotechnical works – National application standards of Eurocode 7. Shallow foundations. Saint-Denis : AFNOR, 2013.
- [4] AFNOR. NF P 94-262 – Justification of geotechnical works – National application standards of Eurocode 7 – Deep foundations. Saint-Denis: AFNOR, 2012.
- [5] AFNOR. (2013). NF P94-500: Geotechnical engineering missions — Classification and specifications. La Plaine Saint-Denis : AFNOR.
- [6] AGTS Bénin. Geotechnical execution study report type G3 – BIIC Building, Cotonou Financial City. Ref. No. P.187122-02-2025.1. Cotonou: AGTS Bénin, May 2025.
- [7] NF EN ISO 22476-4 (2021). Geotechnical investigations and testing Field testing. Part 4: Prebored pressure meter test by Menard procedure. <https://www.boutique.afnor.org>
- [8] Li, J., Kong, L. & Jin, L. (2024). In-Situ Creep Behavior of Expansive Soils Based on Pressuremeter Test. *Geotech GeolEng* 42, 1209–1222 (2024). <https://doi.org/10.1007/s10706-023-02614-2>
- [9] AFNOR. NF EN ISO 22475-1 – Geotechnical investigation and testing – Water and soil sampling – Part 1: General principles. Saint-Denis: AFNOR, 2018.
- [10] AFNOR. NF EN ISO 22282-2 – Geotechnical investigation and testing – Determination of soil permeability – Part 2: Borehole testing. Saint-Denis: AFNOR, 2012.
- [11] Keïta, A. (2015). Subsurface Drainage of Valley Below Irrigated Rice Schemes in Tropical Savannah: Case Studies of Tiefora and Moussodougou in Burkina Faso. PhD thesis, Wageningen University, Delft.