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Assessment of the Liquefaction Risk of a Sandy Soil in a Coastal Environment Using the Direct PMT Method and the PMT-SPT Correlation: Case of Cotonou (Bénin)

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Abstract

West African coastal cities experiencing rapid urban growth are (prone) facing to flooding and seismic tremors, which expose infrastructure to structural damage during seismic events. This study aims to assess the liquefaction risk of sandy soil in the coastal environment of Cotonou. Geotechnical characterization reveals loose sand with an average permeability of 0.00002429 m/s, high friction angles ranging from 37.41˚ to 38.93˚, and low cohesion values ranging from 0.39 kPa to 6.17 kPa. The calculation of liquefaction potential by layer, derived from the cyclic stress ratio, is based on the direct PMT method and the PMT-SPT correlation. For a seismic coefficient of 0.108 g in loose sandy soils and earthquake magnitudes below 7 on the Richter scale, all layers exhibit a liquefaction potential factor greater than the value 1. This threshold is exceeded at depths between 22 and 25 m for a magnitude of 9.1, triggering localized liquefaction that does not pose a significant risk. A moderate risk is indicated starting at from a magnitude of 10.8. Mitigating this risk would require localized improvement measures, such as cement grout injection to consolidate the soil and reduce flow, or vibroflottation and stone columns to densify the sand and dissipate excess pore water pressure. This study enhances the understanding of natural phenomena, facilitating the design and construction of durable structures that ensure the safety of people and property while minimizing economic losses.

Keywords

Earthquake, Floods, Magnitude, Liquefaction, Coastal, PMT-SPT

1. Introduction

Soil liquefaction manifests as a sudden loss of shear strength in saturated, non-cohesive soils subjected to cyclic seismic, vibratory, or hydrodynamic loading. The phenomenon primarily results from an increase in pore water pressure during cyclic loading, which can lead to a significant reduction in effective stress and, in worst-case scenarios, an almost total loss of shear strength. Consequences may include settlement, lateral displacement, foundation failure, and damage to infrastructure.

Coastal environments characterized by recent sandy deposits, a shallow water table, and low relative density are naturally susceptible to this phenomenon. This susceptibility is heightened in areas undergoing rapid urbanization, where buildings and infrastructure are constructed on sedimentary formations that are often heterogeneous and possess mechanical properties that vary significantly with depth. Cotonou city is situated on the coastal barrier of the Gulf of Guinea at anelevation near sea level. The study site, intended for the construction of an eight-story building (ground floor plus seven upper floors), is located in the port area, in the immediate vicinity of the (Autonomous) Sea Port of Cotonou, potential source of mechanical vibrations and a heavily trafficked interstate highway. This setting combines geotechnical, hydrogeological, and dynamic conditions that could trigger liquefaction. Despite significant urban and real estate development along the Beninese coast, quantitative studies analyzing liquefaction potential are virtually non-existent. The availability of geotechnical data also presents a major constraint. In the absence of site-specific SPT or CPT data, this study relies on Ménard pressuremeter data from a geotechnical investigation, combined with two complementary assessment methods within the framework of the NF EN 1998-5 standard.

The overall objective of the study is to assess the liquefaction risk of sandy soil in the coastal environment of Cotonou. To achieve this objective, three specific aspects were investigated:

- Determination of the geotechnical characteristics of the soil at the study site;
- Quantification of the soil liquefaction potential at the site based on available pressuremeter parameters, applying the direct PMT method and the PMT-SPT correlation;
- Formulation of appropriate technical recommendations to reduce the risk of liquefaction for structures founded on the soils of the Cotonou coastal ridge.

2. The Study Context

The site in study is located in Cotonou, the so-called economic capital city of Benin on the coastal strip of the Gulf of Guinea between the Atlantic Ocean and Lake Nokoué (438,066.02 m E, 703,033.54 m N, UTM Zone 31N). Intended for the construction of an eight-store building (ground floor plus seven upper floors), the site is situated near the port area, close to the (Autonomous) Sea Port of Cotonou as shown in **Figure 1**, and a busy interstate highway, two potential sources of mechanical

Figure 1. Overview of the study area.

vibration. The natural ground level, ranging from +3.74 m to +4.39 m NGF, is close to sea level, indicating a vulnerability to soil saturation.

The geographical coordinates and elevations of the main geotechnical investigation points are presented in Table 1. These data provide the precise spatial reference of the boreholes and other investigation points used for the geotechnical characterization of the study site.

Table 1. Coordinates and depths of the geotechnical investigation points.

Borehole	X (m)	Y (m)	Z (m NGF)	Depth (m)
SP3	436,791.243	702,372.036	+4.299	25
SP4	436,752.000	702,338.470	+3.740	25
SC3-PZ03	436,760.844	702,349.342	+3.588	25
SC4-PZ04	436,797.195	702,349.329	+4.394	25
LFr2	436,784.528	702,354.950	+4.003	2 - 4

2.1. Hydrological and Hydrogeological Context

Hydrologically, Cotonou (experiences) is subjected to a sub-equatorial climate with four seasons and an average annual rainfall of approximately 1300 mm [1]. Situated between the Atlantic Ocean and Lake Nokoué, the city is subject to hydrological exchanges that facilitate the recharge of shallow sandy formations through precipitation and seasonal flooding.

Hydrogeologically, the site is underlain by the unconfined Quaternary aquifer of the coastal sandbar. Piezometric monitoring conducted in March 2025 revealed a stable water table at an average depth of 2.04 m. A Lefranc test performed at a depth of 2 to 4 m measured a permeability of $K = 2.429 \times 10^{-5}$ m/s, characteristic of permeable fine-to-medium sand [2]. The aquifer, recharged by precipitation and connected to both Lake Nokoué and the Atlantic Ocean, exhibits seasonal fluctuations ranging from 0.5 to 2.5 m. Its shallow depth keeps the surface sandy layers saturated, thereby increasing their susceptibility to liquefaction. However, the presence of potentially liquefiable soil is not sufficient to define a significant risk, as this also depends on the intensity of seismic loading and soil's resistance [3].

2.2. Geology Context

The site is located within the Beninese coastal sedimentary basin [4], which extends along the Gulf of Guinea and is the result of continuous sedimentation from the Late Cretaceous to the present day. According to the 1:200,000 scale geological map of Benin [5], the surface geology of the Cotonou area consists primarily of clayey sands and sandy clays of deltaic and lagoonal origin, overlain by recent Quaternary deposits forming the coastal barrier; these deposits compressed between marine and aeolian sands reworked by coastal dynamics. Core samples taken at the site confirm this structure: the observed lithology is dominated by clean sand at the surface, followed by silty sands, sandy silts, and finally clayey to highly clayey sands with increasing depth, down to 25 m. This sequence, typical of West African coastal barriers, reflects a gradual transition from an open marine depositional environment to appease, lagoonal conditions, and explains the alternation between loose sandy layers and finer, denser layers observed.

2.3. Geotechnical Context

Liquefaction is a geotechnical phenomenon in which saturated, loose, non-cohesive soil temporarily loses its shear strength under cyclic loading due to an increase in pore water pressure. Its assessment relies on the simplified method developed by Seed and Idriss [6] and updated by the NCEER/NSF [7], which compares cyclic stress (CSR) with the soil's cyclic resistance (CRR). Initially based on the SPT, this approach was extended to the CPT and subsequently to the Ménard pressuremeter test, thanks to the work of [8] and [9] introduced the Liquefaction Potential Index (LPI), which incorporates the entire geotechnical profile. Although these methods have been successfully applied in Beyrouth [10] and Tunisia, they remain poorly documented in West Africa, where pressuremeter tests often constitute the primary source of geotechnical data.

West Africa is a region of low to moderate seismicity, lacking any major active faults [11]. The most significant documented earthquake remains the one that struck Accra, Ghana, on December 22, 1939, with a magnitude of 6.4 [12]. In Benin, historical earthquakes are rare and generally have magnitudes below 5, justifying its classification as a seismic zone 2 area (a_gR = 0.06 g). Consequently, the low regional seismic hazard is the primary reason for the "no-liquefaction" result obtained.

3. Materials and Methods

3.1. Materials

The geotechnical investigation was carried out within the SImAU building footprint at the Cotonou Financial City site. Figure 2 presents the location of the main investigation points, including the pressuremeter and cored boreholes, providing an overview of their spatial distribution within the study area.

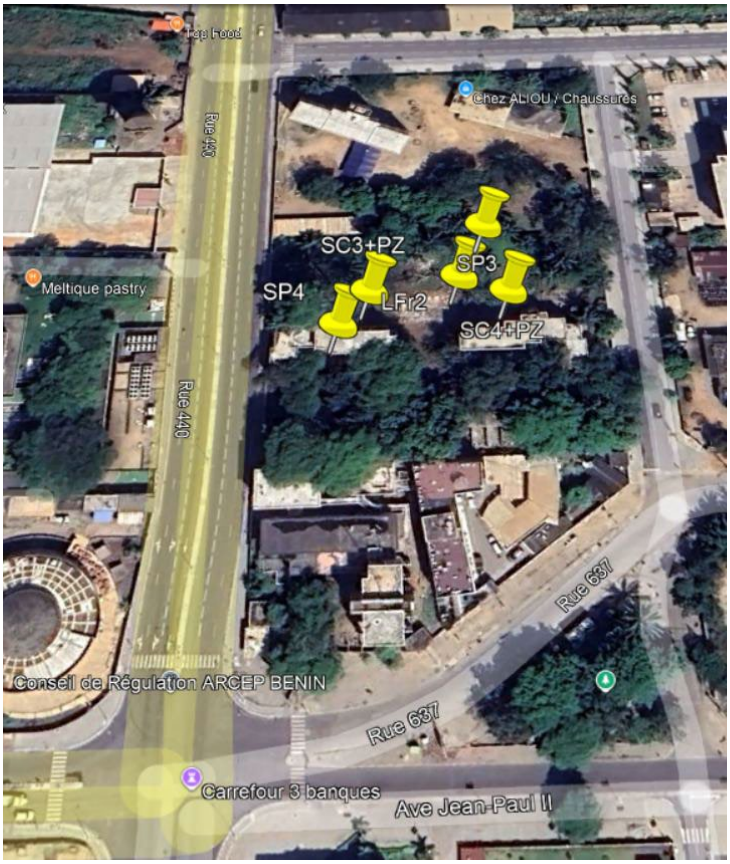

Figure 2. Aerial view of the SImAU site showing the location of the geotechnical investigation points.

The geotechnical campaign utilized GPS for positioning boreholes, a piezometer for monitoring the water table, a Ménard pressuremeter for the mechanical characterization of soils, and a core drilling rig for lithological assessment and sample collection. The Ménard pressuremeter enables the determination of the limit pressure and the pressuremeter modulus parameters essential for evaluating liquefaction potential. The investigation comprised two Ménard pressuremeter boreholes (SP3 and SP4) and two cored boreholes (SC3 and SC4), all drilled to 25 m depth. Pressuremeter tests were performed at 1.00 m intervals throughout each borehole, yielding 25 test levels per sounding. The boreholes are located within the SImAU building footprint at the Cotonou Financial City site. The type of ressurometer used is presented in Figure 3.

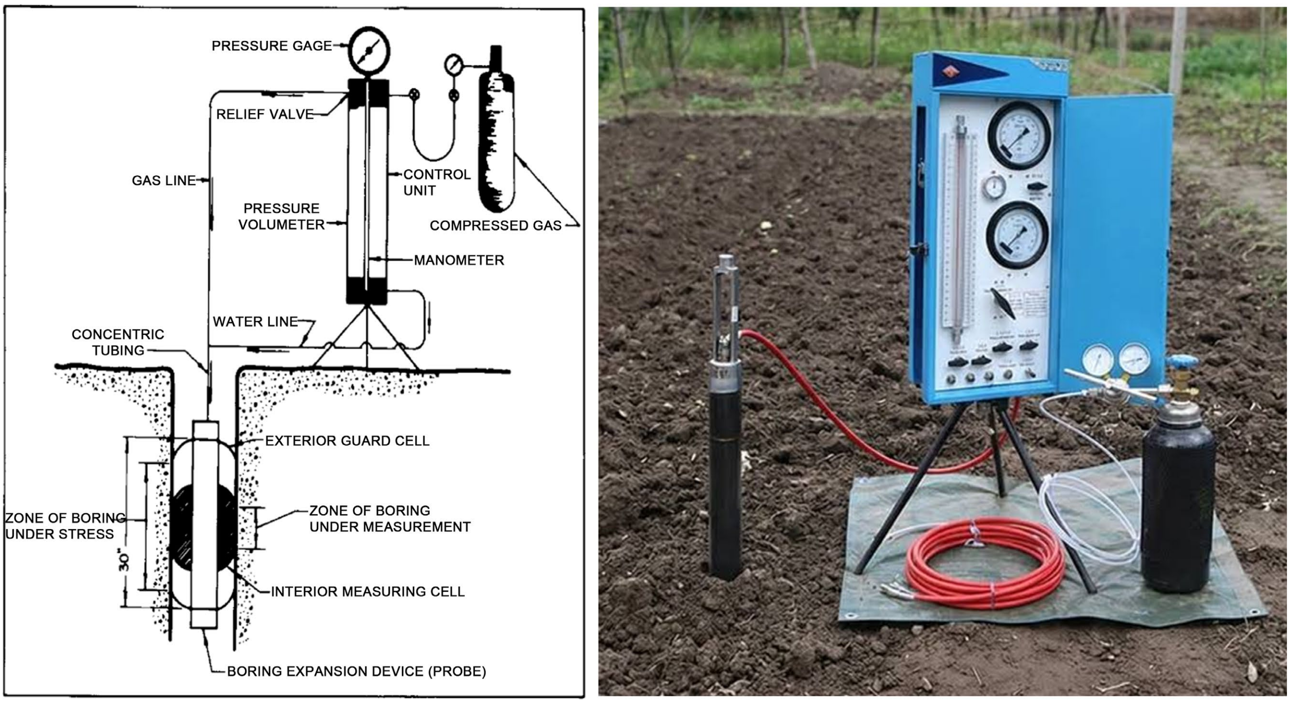

Figure 3. Schematic diagram of the Ménard pressuremeter test and equipment.

The core drilling system that is shown in Figure 4 provides information on the lithological succession of the formations penetrated, as well as representative samples for soil identification and laboratory testing. The combined use of these two pieces of equipment ensures a comprehensive geotechnical characterization of the site by linking the mechanical properties of the ground to its geological nature.

3.2. Methods

3.2.1. Determination of the Seismic Coefficient

NF EN 1998-5 [13] governs the assessment of liquefaction. The conditions for exemption are not met due to the presence of clean sands with a low fines content (<2%), loose layers, and a groundwater table located at a depth of 2.04 m. An assessment of liquefaction potential is therefore required for the site under study. The seismic coefficient is determined using the method specified in NF EN 1998-5, in accordance with the following equation:

$$kh = \gamma_I \cdot a_g R \cdot \frac{S}{r} \tag{2.1}$$

With $r = 1$ (saturated, cohesionless soils)

The design magnitude adopted (Mw = 5.1) corresponds to the maximum

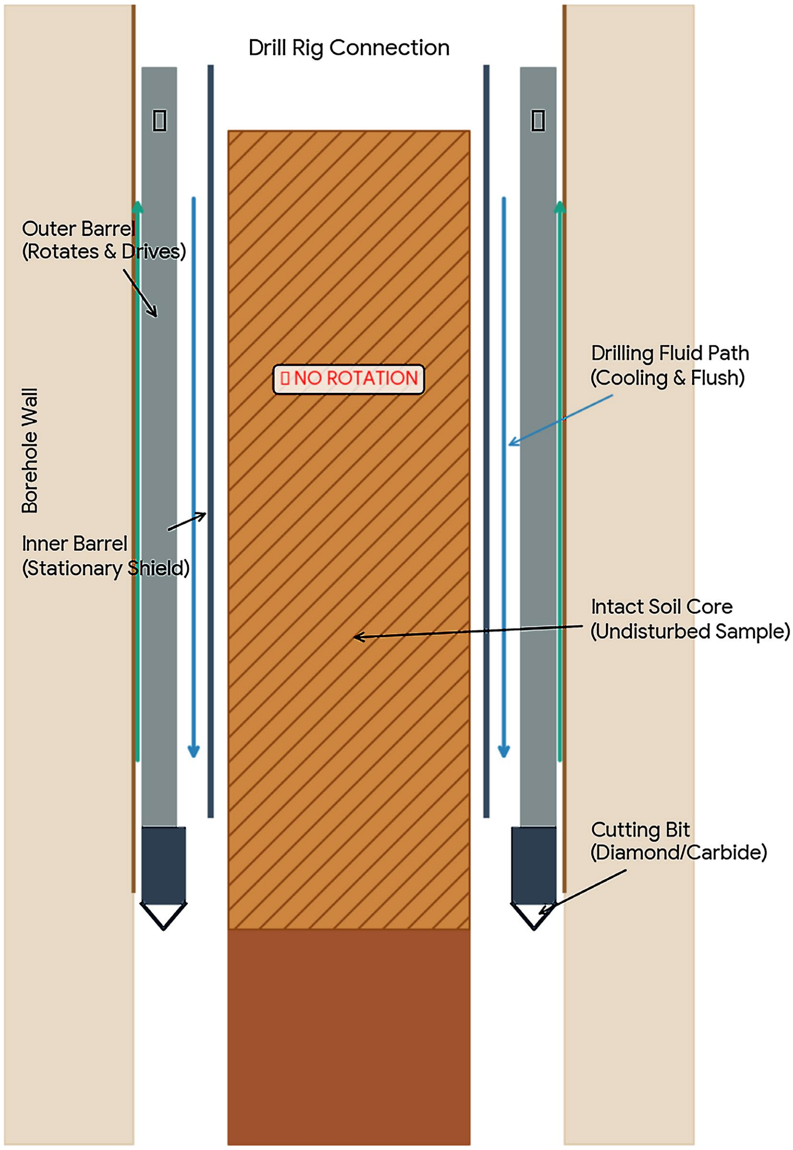

Figure 4. Diagram of the core sampling principle.

historical earthquake recorded in Benin at Nikki on August 3, 2014. This choice provides an initial assessment consistent with available data but does not replace a probabilistic study of local seismic hazard. The following input values were used: total and effective vertical stresses (σ_v, σ'_v) computed from measured soil unit weights (γ_h = 17.14 - 19.00 kN/m^3) and groundwater table depth hw = 2.04 m; depth-reduction factor rd after Idriss and Boulanger (2004); reference ground acceleration $a_gR = 0.06\ g$ (Poggi *et al.* [14], seismic Zone 2, Benin); soil amplification factor $S = 1.8$ (site class D, Eurocode 8); importance factor $\gamma_I = 1.0$ (ordinary structure, class II); yielding ag·S = $\gamma_I \times a_gR \times S = 0.108$ g.

$$Pl_1N = \left(\frac{Pl}{P_a}\right)\cdot\left(\frac{P_a}{\sigma'_{V0}}\right)^{0.5} \quad (2.2)$$

where P_a = 100 kPa.

Le CRR (7.5) is given by:

$$\mathrm{CRR}(7,5)=\frac{1}{(34-2\cdot Pl_1N)}+\frac{2\cdot Pl_1N}{135}+\frac{10}{(20\cdot Pl_1N+25)^2}-\frac{1}{200} \tag{2.3}$$

The magnitude correction is introduced by the factor:

$$\mathrm{MSF}=6.9\cdot\exp\left(-\frac{\mathrm{Mw}}{4}\right)-0.058\leq 1.8 \tag{2.4}$$

We have: $\mathrm{MSF}=1.8$ pour $\mathrm{Mw}=5.1$

The factor of safety is then calculated using the relationship:

$$\mathrm{FS}_1=\frac{\mathrm{CRR}(7,5)\cdot\mathrm{MSF}}{\mathrm{CSR}} \tag{2.5}$$

This method allows for the direct use of pressuremeter results without requiring a standard penetration test performed on-site.

3.2.2. PMT-SPT Correlation

The PMT-SPT correlation method, proposed by [15], is based on the indirect estimation of Standard Penetration Test (SPT) resistance from pressuremeter parameters. In the absence of SPT test results for the site under study, this approach allows the pressuremeter modulus to be converted into an equivalent standard penetration index (N_{60}) using the following empirical relationship:

$$N_{60}=\mathrm{Em}\times\frac{10}{1.5} \tag{2.6}$$

The overload correction from [16]

$$\mathrm{CN}=\left(\frac{P_a}{\sigma'_{V0}}\right)^{0.5}\leq 2.0 \tag{2.7}$$

$$N_1(60)=\mathrm{CN}\times N_{60} \tag{2.8}$$

For $N_1(60)<30$ the CRR (7.5) is calculated using the following formula:

$$\mathrm{CRR}(7,5)=\frac{1}{(34-N_1(60))}+\frac{N_1(60)}{135}+\frac{50}{(10\cdot N_1(60)+45)^2}-\frac{1}{200} \tag{2.9}$$

When $N_1(60)\geq 30$, the soil is directly classified as non-liquefiable within the framework of the method in question. The factor of safety is then obtained by:

$$\mathrm{FS}_2=\frac{\mathrm{CRR}(7,5)\cdot\mathrm{MSF}}{\mathrm{CSR}} \tag{2.10}$$

This approach provides an indirect solution for utilizing pressuremeter data in the absence of SPT tests. However, it must be interpreted with caution, as the relationship between pressuremeter parameters and the SPT index can depend on the soil type, its density, and the geological context.

3.2.3. Determination of the Liquefaction Potential Index

The liquefaction potential of the profile is summarized using the Liquefaction Potential Index (LPI). This index integrates the contribution of each layer weighted by its proximity to the surface over a reference depth of 20 m:

$$\text{LPI} = \int_0^{20} F(z) \times W(z)\,\mathrm{d}z \tag{2.11}$$

where

$$F(z) = \max(1 - \text{FS}; 0) \tag{2.12}$$

$$W(z) = 10 - 0.5z \tag{2.13}$$

The function $F(z)$ represents the degree of susceptibility to liquefaction, while $W(z)$ assigns greater weight to horizons close to the surface.

4. Results and Discussion

4.1. Results

4.1.1. Factors of Safety

Table 1 shows that both methods yield consistent conclusions. All layers exhibit a factor of safety greater than 1. Layer C5 is the most critical (FS = 2.80), whereas layers C2 and C4 fall outside the liquefiable range.

The value of the factor of safety per layer for each method is shown in Table 2.

Table 2. Factor of safety values per layer according to each method.

Layer	CRR (7.5)	FS_1	N_1 (60)	FS_2	Pl_hN
	Direct PMT method		PMT-SPT Method		
1	0.304	7.90	68.7	>>1	12.80
2	→ ∞	>>1	139.0	>>1	31.12
3	0.477	8.99	134.5	>>1	15.06
4	→ ∞	>>1	246.3	>>1	22.64
5	0.131	2.80	60.4	>>1	6.09
6	0.218	5.30	68.2	>>1	10.14
7	0.179	5.06	42.6	>>1	8.43

It should be noted that both methods yield consistent results. All layers exhibit a factor of safety greater than 1, indicating no risk of liquefaction. Layer 5 is the most unfavorable.

All the layers exhibit FS > 1 for both methods. Consequently, the severity function $F(z) = \max(1 - \text{FS}, 0)$ equals zero for every layer throughout the investigated profile. The depth-weighted, profile-wide Liquefaction Potential Index (LPI), integrated from the surface to $z = 20$ m, is therefore LPI = 0, regardless of the integration bound selected (10 m or 20 m), since no layer contributes to the summation. An LPI of zero places the site in the lowest risk class ("very low to nil") of the Iwasaki *et al.* [17] classification.

4.1.2. Determination of the Critical Magnitude

Table 3 presents an isolated parametric sensitivity analysis for the most vulnerable

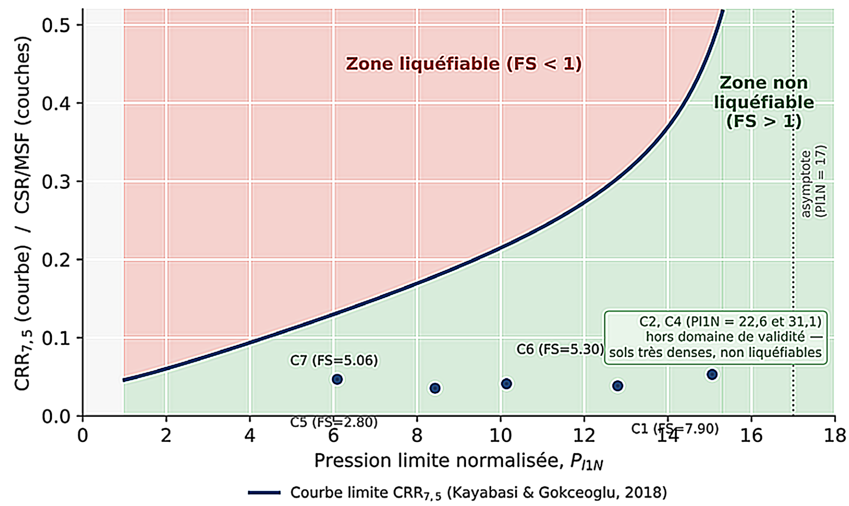

Figure 5. Position of the layers relative to the liquefaction limit curve.

layer (C5), obtained by varying Mw while keeping ag·S = 0.108 g constant. This is a methodological exercise only: the seismic coefficient is not updated consistently with each magnitude value, as no regional ground-motion prediction equation is available for Benin. The sole purpose is to identify the parametric threshold at which FS = 1 under the fixed design acceleration. These Mw values do not represent physically consistent earthquake scenarios and must not be interpreted as design-risk references.

The liquefaction zones of the soil tested are shown in **Figure 5**.

Table 3. Parametric sensitivity of FS (layer C5, most vulnerable) to varying Mw at fixed ag·S = 0.108 g isolated methodological exercise, not a physically consistent hazard scenario.

Mw	MSF	FS (C5)	LPI	Risk level
5.1	1.800	2.785	0	Very low
6.0	1.482	2.292	0	Very low
7.5	1.000	1.547	0	Very low
9.1	0.671	1.000	0	Critical threshold
9.5	0.584	0.903	1.28	Weak
>10.8	—	—	>5	moderate

Under the fixed ag·S = 0.108 g condition, layer C5 remains above the liquefaction threshold (FS > 1) for all magnitudes up to Mw ≈ 9.1. Beyond this parametric threshold, magnitudes above Mw = 9.1 have never been recorded on Earth and are entirely outside the seismotectonic context of the West African passive margin (historical maximum: Mw = 6.4 in Ghana, 1939). These results are presented solely to characterize the mathematical behavior of the method and carry no design significance.

4.2. Discussion

The results obtained differ from those reported for the Beirut coastline, where safety factors close to or below 1 were calculated for several sand layers due to higher regional seismic hazard. Conversely, the study by [18], conducted in a Tunisian coastal sedimentary basin subject to moderate seismicity comparable to that of Benin, yields consistent conclusions, with safety factors exceeding 2 and a low to very low liquefaction risk. This comparison demonstrates that the level of regional seismic hazard rather than soil type accounts for the differences observed between the studied contexts. From a practical standpoint, this study represents one of the first documented applications of pressuremeter-based liquefaction assessment methods in Benin, where SPT and CPT tests are rarely available. The convergence of the two methods used, along with their consistency with results from comparable coastal basins, confirms the suitability of this approach for the West African context and provides a reproducible methodological framework for the seismic design of foundations along the Beninese coast.

However, three main limitations should be noted: the absence of SPT and CPT tests necessitates the use of correlations not calibrated for the coastal sands of the Gulf of Guinea; the selected magnitude is based on the maximum historical value rather than a probabilistic study of local seismic hazard; and finally, seasonal groundwater fluctuations and potential sand disturbance remain factors requiring monitoring. A supplementary campaign of CPTu and SPT tests would help confirm the results, particularly for Layer 5, which was identified as the most vulnerable. Although the risk of liquefaction is ruled out, foundations should be anchored in the most resistant layers (Layer 2 or 4), while local seismic microzoning and piezometric monitoring would usefully complement this assessment.

5. Conclusion

This study presents the first quantitative assessment of liquefaction risk in a site located at Cotonou coastal barrier, conducted using pressuremeter data in accordance with NF EN 1998-5. Geotechnical characterization revealed a seven-layer profile extending to a depth of 25.5 m, comprising a superficial layer of loose sand and several denser to very dense horizons, situated beneath a groundwater table stabilized at 2.04 m. The two pressuremeter-based methods yield unequivocal results: none of the seven layers is susceptible to liquefaction at the design magnitude of 5.1 showing a minimum factor of safety of 2.80 and an LPI of zero and sensitivity analysis confirms the robustness of these findings up to a magnitude of 9.1; the moderate regional seismic hazard, rather than uniform soil quality, is the primary reason for this conclusion of no liquefaction risk. Layer 5, the most vulnerable horizon, will require special attention should the reference seismic hazard be revised; furthermore, while the results confirm the suitability of pressuremeter methods in the absence of penetration tests, they would benefit from validation through CPTu and SPT testing, as well as seismic microzonation studies across Benin. Finally, regarding design, the absence of significant liquefaction risk does not obviate the need for a thorough assessment of bearing capacity and settlement. Foundations must be anchored below the superficial layer of low bearing capacity, and monitoring the groundwater table during the operational phase serves as an additional safety measure.

Author Contributions

Conceptualization, Kassa Issifou MOUNOU SAMBIENI, Euphorie GAMAVO and de Paule CODO; Methodology, Kassa Issifou MOUNOU SAMBIENI X.; software, Kassa Issifou MOUNOU SAMBIENI; validation, Kassa Issifou MOUNOU SAMBIENI, Euphorie and de Paule CODO.; formal analysis, Kassa Issifou MOUNOU SAMBIENI.; investigation, Kassa Issifou MOUNOU SAMBIENI.; resources, Kassa Issifou MOUNOU SAMBIENI.; data curation, Kassa Issifou MOUNOU SAMBIENI and Euphorie GAMAVO.; writing original draft preparation, Kassa Issifou MOUNOU SAMBIENI.; writing review and editing, Kassa Issifou MOUNOU SAMBIENI.; visualization, Kassa Issifou MOUNOU SAMBIENI.; supervision, Kassa Issifou MOUNOU SAMBIENI and de Paule CODO.; project administration, Kassa Issifou MOUNOU SAMBIENI.; funding acquisition, Kassa Issifou MOUNOU SAMBIENI. All authors have read and agreed to the published version of the manuscript.

Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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