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Bearing Capacity Analysis of Spread Footing on Massif in the "Corniche Ouest" of the Dakar Peninsula (Senegal, West Africa)

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Abstract

This study presents various approaches to calculating the bearing capacity of spread footings applied to the rock mass of the western corniche at the tip of the Dakar peninsula. The bearing capacity was estimated using empirical, analytical and numerical approaches based on the parameters of the rock mass and the foundation. Laboratory tests were carried out on basanite, as well as on the other facies detected. The results of these studies give a range of allowable bearing capacity values varying between 1.92 and 11.39 MPa for the empirical methods and from 7.13 to 25.50 MPa for the analytical methods. A wide dispersion of results was observed according to the different approaches. This dispersion of results is explained by the use of different rock parameters depending on the method used. The allowable bearing capacity results obtained with varying approaches of calculation remain admissible to support the loads. On the other hand, the foundation calculations show acceptable settlement of the order of a millimeter for all the layers, especially in the thin clay layers resting on the bedrock at shallow depths, where the rigidity of the rock reduces settlement.

Keywords

Peninsula of Dakar, Bearing Capacity, Basanites, Rock Mass, Spread Footings

1. Introduction

Over the last few decades, the considerable growth in infrastructure development has often posed the problem of finding suitable sites to construct foundations for structures such as buildings, bridges, power stations and tunnels to withstand heavy loads. Design of foundations on soils, whether shallow or deep, has been the subject of study for a very long time and is now governed by calculation rules validated by several building codes, including those used in Senegal, which are DTU, Fascicule 62, Eurocode 7. One possible reason for this vast collection of literature on soil foundations is that the design and construction of soil have generally been considered more difficult than rock foundations. In the rock field, until recent years, the idea prevailed that there are few problems of bearing capacity or settlement because "any structure founded on rock" was considered to be a guarantee of stability for the works. Rock is usually an excellent foundation material. However, the behaviour of rock becomes very complex when it is heavily jointed, very soft or weathered. In order to provide an optimum foundation design, it is crucial to estimate the ultimate bearing and settlement of rock masses. In the literature, there are numerous approaches suggested to predict the bearing capacity of the rock mass, the most commonly used of which are [1] building codes, analytical methods, semi-empirical and experimental methods. **Figure 1** shows a series of spread footings founded on basanite blocks. Sarr *et al.* [2] studied the behaviour of rock facies foundations on the western corniche using classification systems and finite element numerical modelling. This work complements the work of [2] through the use of other calculation approaches widely used in Senegal on rock masses, namely the pressuremeter test and Eurocode7 (EC7) on soils and altered rocks. However, in order to use the rock mass as a foundation support to its full capacity, it is important to assess the characteristics of the rock masses. To achieve our objective, engineering geological studies, including field and laboratory tests, were carried out on the west corniche of Dakar, specifically in Almadies-Ngor-Yoff-Aeroport (zone 1); Mamelles-Ouakam-Sacre-cœur (zone 2) and Fann-Point E (zone 3) (**Figure 2**). For each zone, core sampling, pressurementer test and laboratory tests were carried out. The in-situ and laboratory parameters obtained were used in empirical, analytical and numerical equations to estimate the ultimate bearing capacity and displacement of spread footings. This will provide a clear understanding of the impact of the parameters, depending on the methods used, on the bearing capacity. In fact, this would be the source of the discrepancies noted in the bearing capacity values.

(a) (b)

Figure 1. Spread foundations on rock mass ((a) Site Ouakam, (b) Site Almadies, Dakar, Senegal).

2. Geographic and Geological Framework of the Study Area

The study area is located in the Dakar region, at the head of the Cap-vert peninsula. The Cap-vert peninsula constitutes the western extremity of the Senegalese-Mauritanian coastal basin situated on the western of the West African craton. It has a Sudano-Sahelian climate, with a rainy season (July-October) alternating with a relatively dry season (November-June). Its temperature varies between 17 and 25˚C from December to April and 27˚C to 30˚C from May to November [3]. The geological of the study area is integrated into the general geology of the Cap-vert peninsula, the seat of Tertiary and Quaternary volcanism [2]. Tertiary volcanic formations are scattered throughout the Cap-vert peninsula and the Thiès region, while Quaternary is located at the head of the Cap-vert peninsula [4] [5]. Quaternary volcanism was stratigraphically distinguished from Tertiary volcanism very early on. Indeed, at the Pointe of Fann, the finite Tertiary lateritic cuirass, itself developed on a basanitic body. In addition, flows from the Mamelles volcano overlie "infrabasaltic" aquifer sands, reputed to be Quaternary. In the interior of the Cap-Vert peninsula (Dakar region), the relatively flat relief is formed by clayey sands and recent sand dunes on which part of the city of Dakar is built, making it difficult to access the volcanic formations because of the settlements along the coast or on the volcanic plateau.

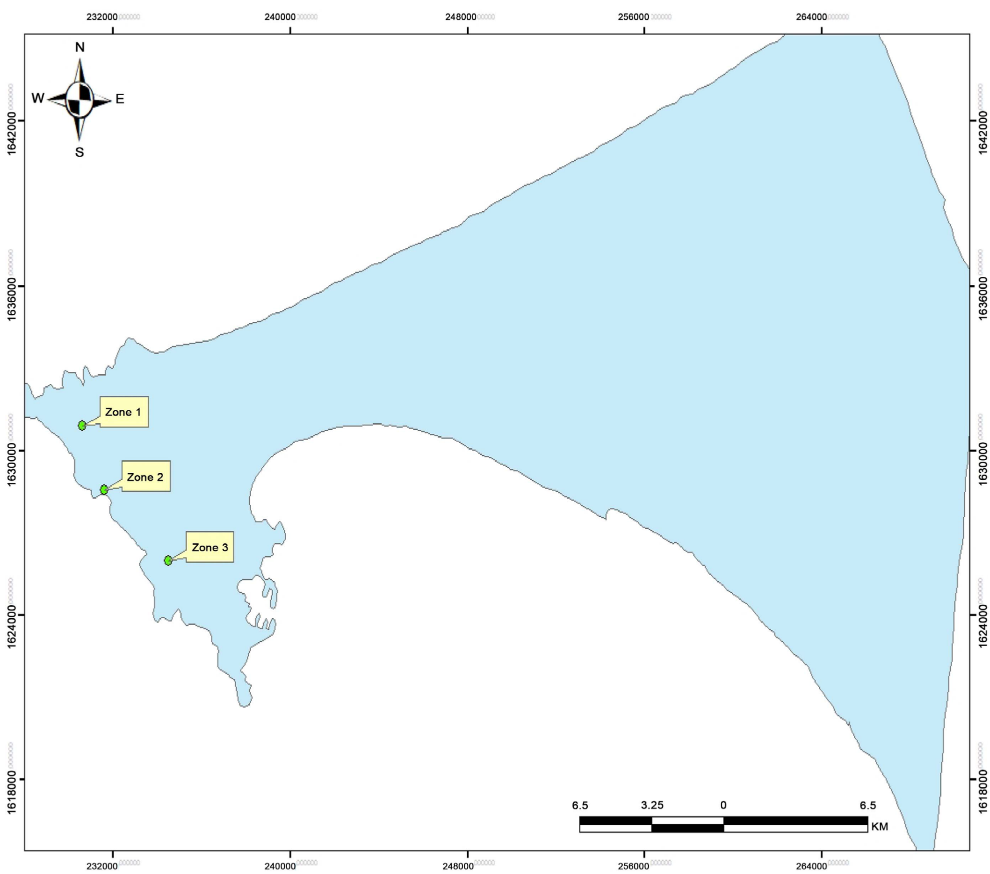

Figure 2. Location of study areas on the map of the Cap-Vert peninsula.

3. Research Methodologies

3.1. Geotechnical Investigations

The geotechnical investigations were carried out using core drilling coupled pressuremeter testing for each zone. The coring was realized by rotation using a 90 mm diameter double core barrel on a drilling machine. According to the boreholes drilled, the water table varied between 3.0 and 10.0 m during the investigation periods. There is a sandy-clay layer at an average depth of between 0.0 m and 5.0 m, underlain by blocks of basanites with olivines. These basanites have a micro-grained to grainy texture, sometimes healthy and altered by the presence of vesicles (**Figure 3**). The rock quality designation (RQD) values of the rock vary between 45 and 80 %, which corresponds to poor and good quality, according to D. Deere. The pressuremeter test carried out only in the clay layers and in the soft, weathered rock has been made in accordance with Standard NF 94 110, 2000 using a Menard pressuremeter (**Figure 4**). The average limit pressure varies from 0.270 to 1.33 MPa and from 2.20 to 19.20 MPa for the Menard modulus (EM) for the sand clay layers. This means that the materials have a firm consistency [6]. As for the basanites, the Menard modulus values vary between 23.50 and 297 MPa and between 1.28 and 7.01 MPa for the limit pressure. The lithology of the three zones is characterised by the presence of recent sandy-clay formations overlying basaltic volcanic formations. At zones 1 and 2, the rock layer is sub-surface. In zone 3, however, it is interbedded between two layers of soil. The lithological sections and results of the pressuremeter parameters for the different zones are shown in **Figures 5(a)-(c)**.

(a) (b)

Figure 3. Intact rock cores in core box collected between 0.0 and 20.0 m.

(a) (b)

Figure 4. Core drilling and pressuremeter testing.

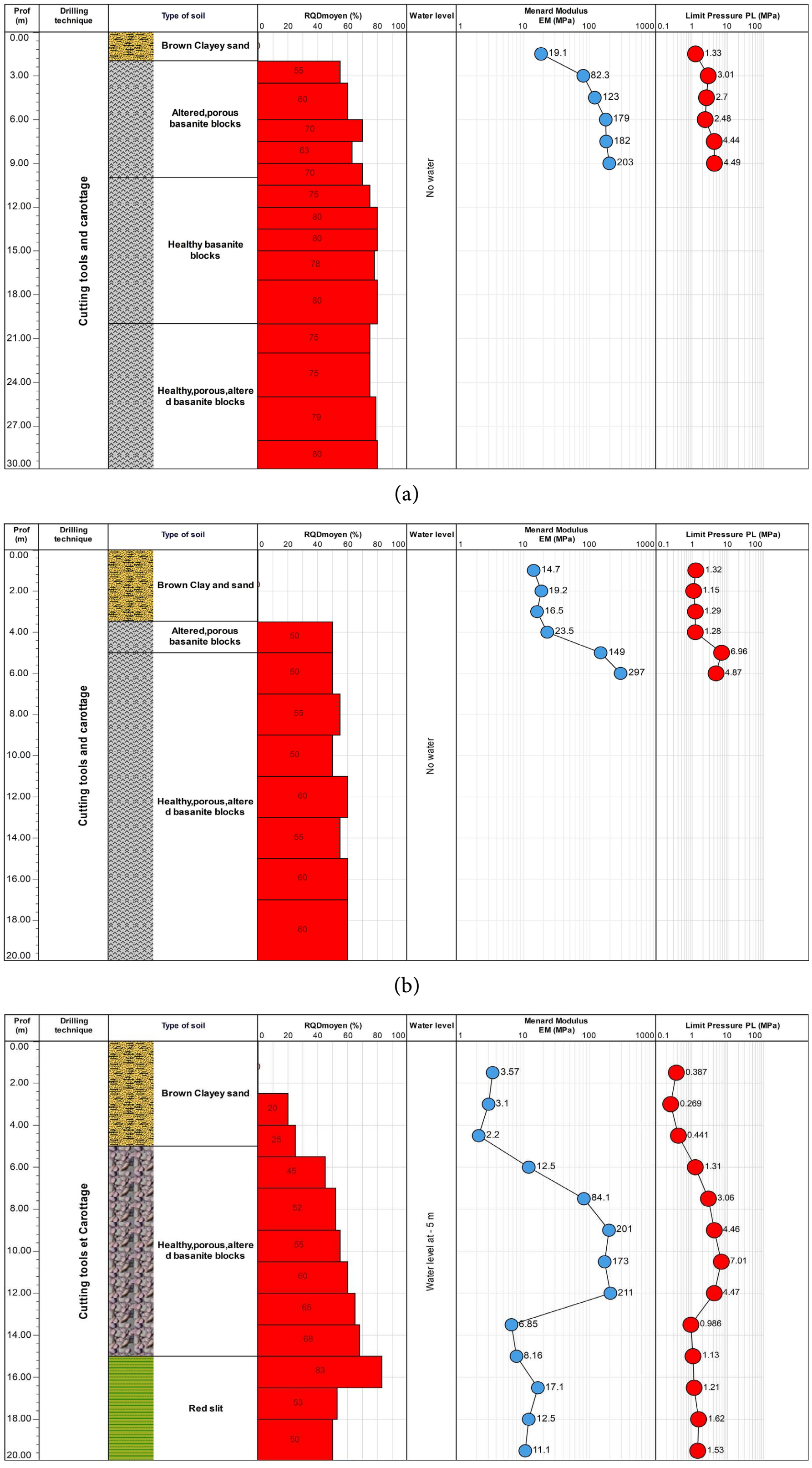

Figure 5. Borehole cross-sections and results of pressuremeter tests ((a) zone 1; (b) zone 2 and (c) zone 3).

3.2. Rock Mass Classification

The most widely known rock mass systems are D. Deere's RQD [7], Bieniawski's RMR [8], Q-system of Barton [9] and Geological Strength Index (GSI) [10]. In this study, only the RMR and GSI will be used to characterize the rock and determine the Hoek and Brown parameters. The RMR was first introduced by Bieniawski (1973) and later modified over the years by the author until its latest version in 1989. Six parameters characterizing the intact rock and joints are taken into account to express the RMR: (01) strength of intact rock, (02) the value of the RQD, (03) mean spacing of discontinuities, (04) condition of discontinuities, (04) groundwater, (05) conditions and (06) the orientation of the discontinuities. The value of the RMR varies from 0 to 100. The sum of the six parameters is used to estimate the RMR value.

$$RMR = R_1 + R_2 + R_3 + R_4 + R_5 + R_6 \quad (1)$$

The Geological Strength Index (GSI) is a recent classification system for hard and soft rock masses. It was introduced by Hoek (1994), Hoek *et al.* (1995) and improved by Hoek and Brown (1997). It was introduced specifically for estimating rock properties. Relations have been proposed by Hoek and Brown (1997) to estimate the GSI [11] from the *RMR*. These relationships are as follows:

$$GSI = RMR' - 5 \quad \text{if } RMR' > 23 \quad (2)$$

RMR': Modified Bieniawski rock mass rating, calculated using a value of 15 for the coefficient relating to water ($R_5 = 15$) and a zero value for the correction coefficient relating to the orientation of discontinuities ($R_6 = 0$). Outcrops, rock faces and core holes are the most widely used sources of information for estimating RMR and GSI values for rock massifs [12] (**Figure 6**).

(a) (b)

Figure 6. Geomechanical study for foundation design ((a) Basanite blocks in a clay matrix, (b) Healthy basanite blocks).

3.3. Evaluation of Ultimate Bearing Capacity and Settlement

For the numerical calculation of bearing capacity, the finite element method with the Plaxis 2D code is chosen. The behaviour of the soil is governed by an elastic-perfectly plastic law and the modified Hoek-Brown failure criterion for rock mass. A rigid spread footing was modeled with a width of 1.0 m and subjected to a vertical load on the surface of the homogeneous rock mass. During the generation of the mesh, the 15-node triangular elements with a fine mesh were used. As for boundary conditions, horizontal and vertical displacements are fixed on the extremities. The load-displacement curve was used to estimate the bearing capacity. The progressive displacement of the footing is accompanied by an increase in stress in the massif. This stress under the footing stabilizes at a value that indicates a yield point, which corresponds to the load-bearing capacity. Other approaches to determining the bearing capacity of rock foundations are empirical and analytical methods. Empirical methods are based on observations, tests and the parameters of the Hoek-Brown failure criterion. Analytical approaches, on the other hand, are based on limit equilibrium theorems, the method of characteristics or slip lines, and the limit analysis method with its two approaches, kinematic and static. As with ultimate bearing capacity, calculating foundation settlement is an important design parameter. Settlement is given by the expression of Schleicher (1926), pressuremeter tests and numerical modelling. Table 1 and Table 2 give some expressions for calculating bearing capacity and settlement respectively.

Table 1. Empirical and analytical methods for calculating bearing capacity.

Authors	Equations		References
Empirical methods			
Peck *et al.*	$q_a = 1 + \dfrac{\frac{RQD}{16}}{1 - \frac{RQD}{130}}$	(3)	[13]
Canadian Code	$q_a = K_s \times q_c$	(4)	[14]
Wyllie	$q_{ult} = C_{f1} s^{1/2} \sigma_{u(r)} \left[1 + \left(m s^{-1/2} + 1\right)^{1/2}\right]$	(5)	[15]
Americain Code	$q_a = 0.2 \times q_c$	(6)	[16]
Pressuremeter	$q_{ult} = K_p \times Pl_e + q_0$	(7)	[17]
Analytical methods			
Serrano *et al.*	$q_{ult} = P_n = \beta_n \left(N_\beta - \xi_n\right)$	(8)	[18]
Goodman	$q_{ult} = q_u \left(N_\varphi + 1\right)$	(9)	[19]
Terzaghi	$q_{ult} = cN_c + 0.5\gamma BN_\gamma + \gamma DN_q$	(10)	[20]
Merifield	$q_{ult} = q_c \times N_\sigma$	(11)	[21]

q_{ult}: ultimate bearing capacity; q_a: allowable bearing capacity; q_u: unconfined compressive strength; *RQD*: rock quality designation; q_c: uniaxial compression strength of intact rock; m and s: material constants in the Hoek and Brown failure criterion; K_p: bearing capacity factor; K_s: empirical coefficient depending on the spacing of discontinuities; C: cohesion; φ: internal friction angle; ζ_n and β_n: constants of rock mass; N_σ and N_β: bearing capacity factor; Pl_e: limit pressure; q_0: overburden pressure after construction.

Table 2. Settlements calculated using Schleicher and Pressuremeter methods.

Authors	Equations	References
Schleicher	$$\delta = \frac{C_d qB\left(1-\upsilon^2\right)}{E} \quad (12)$$	[15]
Pressuremeter	$$s = \frac{\alpha}{9E_c}\left(q-\sigma_{v0}\right)\lambda_c B + \frac{2}{9E_d}\left(q-\sigma_{v0}\right)B_0\left(\lambda_d \frac{B}{B_0}\right)^{\alpha} \quad (13)$$	[17]

δ, S: settlement in mm; E: Young's modulus; ν: Poisson's ratio; C_d: Shape factor of foundation; B: footing width; λ_c, λ_d: Shape coefficients depending on the footing dimensions; E_d, E_c: Pressuremeter equivalent modulus; α: rheological coefficient of the soil or rock; q: vertical stress applied by the foundation; σ_{v0}: Total vertical stress.

4. Results and Discussions

4.1. Laboratory Tests and Rock Mass Classification

Tables 3-5 summarize the physical and mechanical properties and geomechanical classification of the sandy clay and basanite strata. Laboratory tests on soil samples taken from clayey sand strata showed a unit weight ranging from 13.83 to 19.06 kN/m^3, with a plasticity index varying between 15.78 and 20.40, confirming the clay's low plasticity. The results of the shear test illustrate soil with friction angle and cohesion values confirming the clayey tendency. The basanite facies in the study areas were geomechanically classified using the RMR and GSI systems. The rock matrix has an average compressive strength of between 24.27 and 34 MPa, corresponding to a grade ranging from 2 to 4. The RQD varies between 58.20% and 72.96%, corresponding to a grade of 13. The average spacing of the discontinuities ranges from 150 to 390 mm, giving a score of 8 to 10. The discontinuities have slightly rough shoulders with slightly too heavily weathered surfaces. A score of 10 is awarded for wet rock (presence of water table) and 15 for dry rock. Finally, the RMR of our rock mass ranges from 60 to 62, with a GSI of 55 ± 5, which corresponds to medium to good quality basanites overall.

4.2. Bearing Capacity and Settlement

The results of the bearing capacity according to the different approaches are presented in Table 6 and Table 7. A comparison was made by means of a histogram, which gave the allowable bearing capacity (Figure 7). There is a scattering of results between the different methods. The range of bearing capacity values is between 1.92 and 11.39 MPa for the empirical approaches and from 7.13 to 25.50 MPa for the analytical approaches. For the empirical approaches, the results obtained with the Canadian code, Eurocode 7 [22] and the pressuremeter test give very similar bearing capacity values. The advantage of the Canadian and Eurocode 7 approaches is that they take into account both the joint spacing and the compressive strength of the rock. With the pressuremeter test in our case study, the value of the capacity is underestimated, due to the fact that the limiting pressure of the rock is much lower than its breaking pressure. The pressuremeter

Table 3. Laboratory test results of clay sand.

Zones	Depth	Porosity	Natural unit weight	Plasticity index	Cohesion	Internal friction angle	Soil type
	(m)	n (%)	g (kN/m^3)	Ip (%)	c (kPa)	φ (˚)	
Zone 1	0.0 - 3.50	44.67	13.83	17.57	5.5	30.66	Brown clayey sand
Zone 2	0.0 - 1.50	60	16.4	20.4	7.9	34	Brown clayey sand
Zone 3	0.0 - 5.00	34.67	19.06	15.78	14.8	28.8	Brown clayey sand

Table 4. Geomechanics' parameters of the basanite.

Zones	Uniaxial compression strength UCS (MPa)	RQD %	RMR	GSI	Rock mass classes based on RMR
Zone 1	28	58.2	62	60	Good rock
Zone 2	24.27	72.96	60	55	Good rock
Zone 3	34	65.38	60	55	Good rock

Table 5. Results of Hoek-Brown mechanical parameters on basanites.

Zones	Classification			Hoek and Brown			Mohr-coulomb		Rock mass parameters			
	GSI			Parameters			Parameters		MPa			
	GSI	mi	D	mb	s	a	C (MPa)	φ (˚)	σ3max	scm	σt	Em
Zone 1	60	25	0	5.38	0.008	0.504	2.01	40.55	7	8.76	−0.04	1368.796
Zone 2	55	25	0	5.01	0.006	0.504	1.7	39.95	7.298	7.29	−0.033	902.772
Zone 3	55	25	0	5.01	0.006	0.504	2.38	39.95	8.5	10.22	−0.046	1390.45

method, derived from soil mechanics, assumes a continuous, homogeneous soil and does not take discontinuities into account when estimating bearing capacity, so it is less suitable for fractured or only slightly fractured rock masses. With Peck's approach, the bearing capacity increases with the RQD and not with the compressive strength. This means that two rock formations with different strengths [23], for example, healthy basanite and weathered vesicular basanite having the same RQD, will have the same bearing capacity, even if they have different compressive strengths. The limitations of determining bearing capacity using RQD quickly show. The American code gives very pessimistic and conservative capacity estimates and does not take into account the joints of the rock mass. As far as analytical approaches are concerned, Merifield's method gives the highest bearing capacity values. This overestimation of the bearing capacity with the Merrified approach is explained by its bearing factor, which is significantly higher than those proposed by Serrano-Ollala, Wyllie Duncan and Terzaghi. The Finite Element Method (FEM) using load-displacement curves was used to estimate the ultimate bearing capacity (**Figures 8(a)-(c)**). The material parameters are mi = 25, GSI = 60 and 55 and Rc = 28; 24.27 and 34 MPa. The results obtained using the Finite Element Method give values between 13.98 and 19.77 MPa.

Table 6. Allowable bearing capacity of basanites using empirical methods.

Zones	Solutions				
	Peck	EC7	Canadian	Americain code	Pressuremeter
Zone 1	7.58	2.5	2.8	5.6	2.07
Zone 2	11.39	2.5	2.42	4.85	1.92
Zone 3	9.22	3.75	3.4	6.8	2.57

Table 7. Allowable bearing capacity of basanites using analytical methods.

Zones	Solutions				
	Wyllie	Terzaghi	Goodman	Serrano-Olalla	Merifield
Zone 1	9.2	16.9	16.69	10.98	21
Zone 2	7.13	13.98	13.58	8.74	18.2
Zone 3	9.99	19.27	19.03	12	25.5

Foundation displacements were obtained from pressuremeter tests, the Schleicher method and numerical modelling. The results show very low displacements (**Table 8**). In the case of a thin clay sand layer resting on the bedrock at a shallow depth (**Figure 9(a)**, **Figure 9(b)**), the rigidity of the bedrock reduces the settlement of the foundation, compared with the case where the bedrock is at a great depth (**Figure 9(c)**). The difference in rigidity between the clay layer and the rock layer leads to differential settlement when footings are laid at different levels, especially for spread footings. In addition, settlement calculations based on the results of the finite element method in a compressible layer on a rigid base are more suitable than pressuremeter tests and the Schleicher method. These results confirm those of [2].

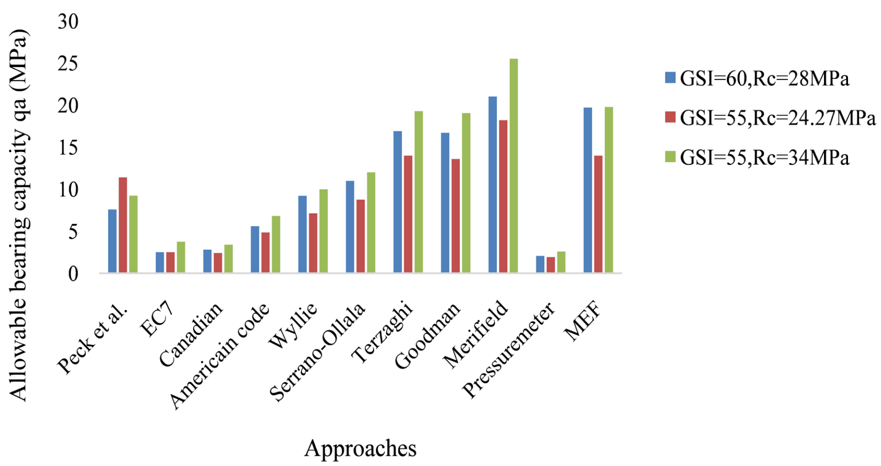

Figure 7. Histogram of Allowable bearing capacity values using different approaches.

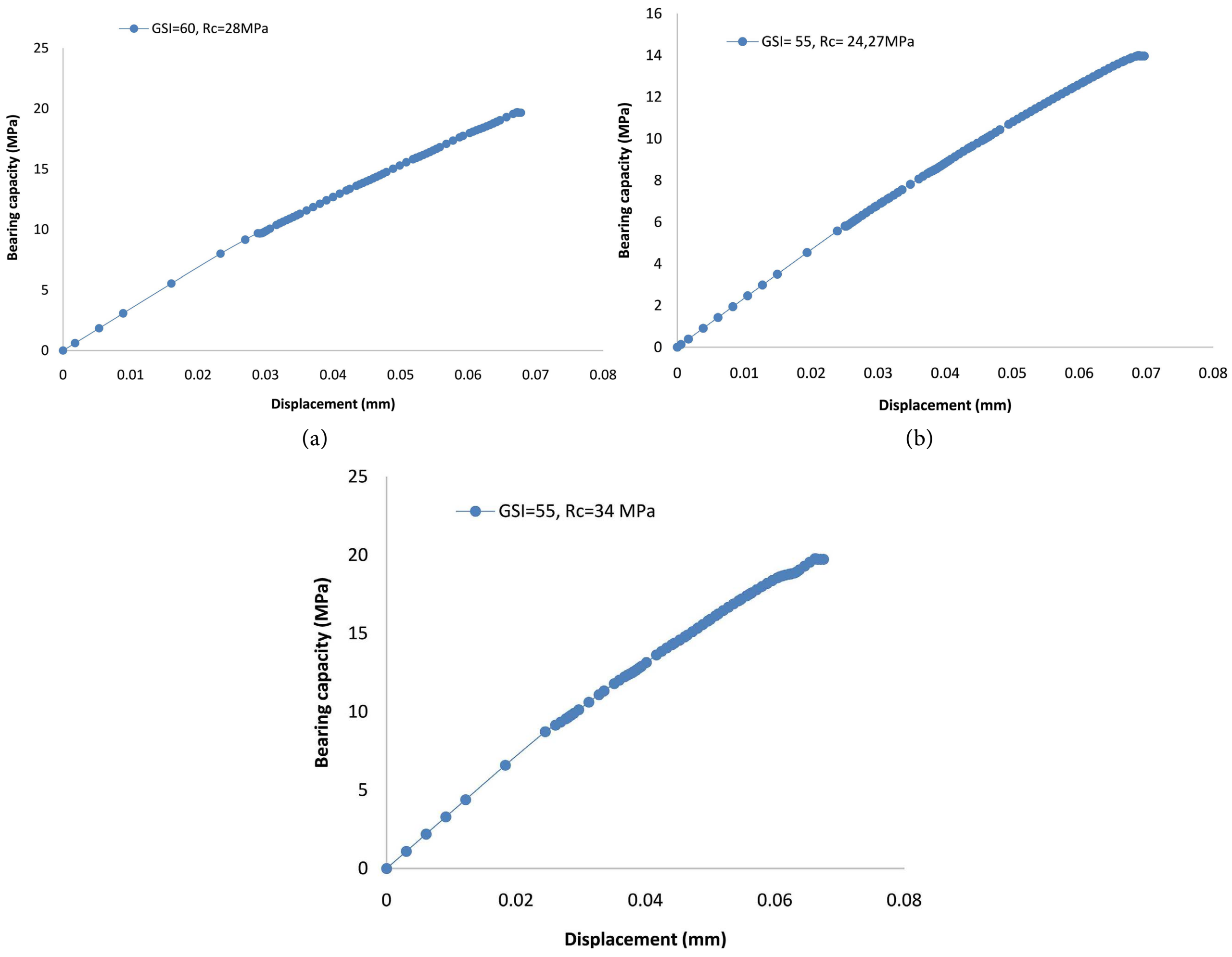

Figure 8. Displacement and Bearing capacity curve ((a) zone 1, (b) zone 2 and (c) zone 3).

Table 8. Comparison of settlement using empirical and finite element methods for compressible layer on rigid base.

Zones	Settlements solutions		
	FEM	Schleicher	Pressuremeter
Zone 1	3.61E−03	2.88E−03	2.80E−03
Zone 2	7.27E−02	9.56E−03	7.00E−04
Zone 3	9.05E−02	1.17E−02	1.87E−02

5. Conclusion

In this article, we present the results of the bearing capacity of spread foot on a rock mass on the western corniche of Dakar, which was studied using different approaches. The laboratory and geomechanical results obtained on basanite rock facies are of medium to good quality. The admissible bearing capacity and settlement results obtained with varying approaches of calculation remain

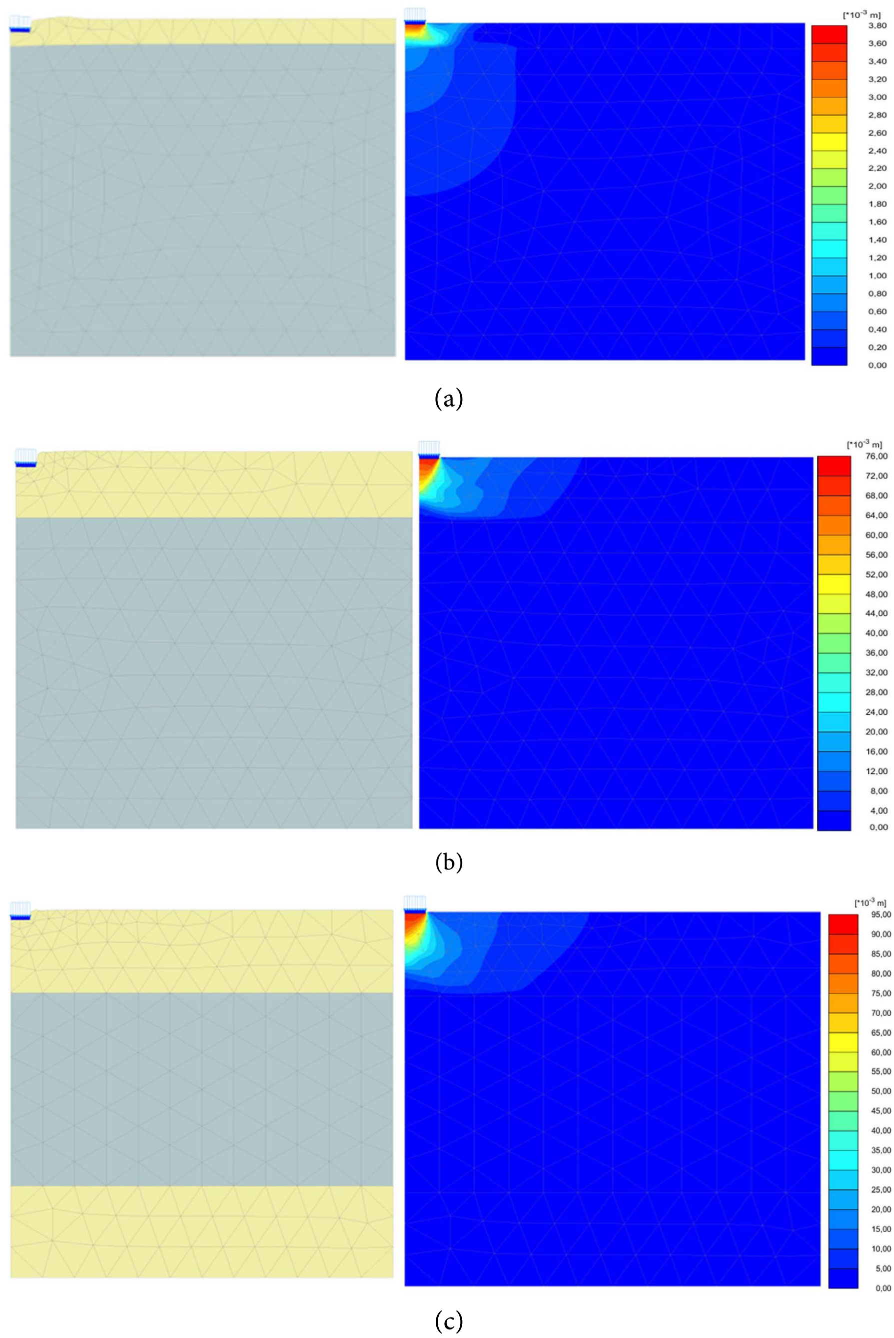

Figure 9. Mesh deformed and total displacements for each zone ((a): zone 1; (b): zone 2; (c): zone 3).

admissible. Our study shows that the bearing capacity values are lower than the simple compressive strength of rock. Although the bearing capacity of the rock remains lower than the simple compressive strength of the rock and that of the concrete generally used in Senegal for the design of foundations on rock masses, the difference in stiffness between the rock layer and the sandy-clay layer can lead to differential settlement in the case of an oversized foundation. In addition, the properties of sound rock and concrete are insufficient to assess the bearing capacity of a foundation. It is essential to take into account as many geomechanical parameters as possible when designing foundations in general.

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Conflicts of Interest

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

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