

Application of the Characterization of Partially Saturated Soils in the Modeling of the Resistant Behavior of Isolated Footings Subjected to Axial Load

Aplicación de la caracterización de suelos parcialmente saturados en la modelación del comportamiento resistente de cimientos aislados sometido a carga axial

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Abstract

This research addresses the aspects related to obtaining the bearing capacity and the settlement of an isolated footing subjected to axial load supported on the soil from the Capdevila formation by using computer modeling. For this purpose, the Abaqus/CAE software was used, which details the modeling process of the partially saturated soil from the Capdevila formation. This research also addresses aspects related to the geometric design of the model, loads, as well as the definition of the boundary and interface conditions between the bodies that make up the system. It addresses the entire mathematical calibration process of the model, associated with the selection of the type of finite element and the appropriate mesh density, according to an adequate computational cost.

Keywords: Settlement; bearing capacity; shallow foundation; MEF; partially saturated soils

Resumen

En el desarrollo de esta investigación se tratan los aspectos concernientes a la obtención de la capacidad resistente y del asentamiento de un cimiento superficial aislado sometido a carga axial apoyado en el suelo Capdevila, utilizando la modelación computacional empleándose el programa Abaqus/CAE, detallándose el proceso de modelación del suelo Capdevila bajo la condición de saturación parcial, tratándose aspectos relacionados con la concepción geométrica del modelo, las cargas, así como la definición de las condiciones de borde e interfase entre los cuerpos que componen el sistema. Se aborda todo el proceso de calibración matemática del modelo, vinculado a la selección del tipo de elemento finito y de la densidad de malla adecuada, acorde con un adecuado costo computacional.

Palabras clave: Asentamiento; capacidad de carga; cimiento superficial; MEF; suelos parcialmente saturados

1. Introduction

The study of the geotechnical behavior of the soils was carried out considering that the soils were saturated or dry. However, there are significant geotechnical problems where the study of partial saturation is essential since a large part of the geological formations of the planet is made up of partially saturated soils (Tristán et al., 2020).

In general, the ultimate bearing capacity of foundations is estimated based on the assumption that the soil is isotropic with respect to shear strength (Al-shamrani et al., 2015), assuming that the soil is in a saturated state. Shallow foundations are placed above the water table and the variation of stresses in relation to depth, associated with loads of the upper structures, is distributed across the substructure (shallow foundations), in partially saturated soils. The results of the study show that the ultimate bearing capacity of these partially saturated soils is significantly influenced by suction, stress, overloading, and dilation. The framework to estimate ultimate bearing capacity is simple, promising and can be extended in the geotechnical engineering practice to design foundations using the mechanics of partially saturated soils. Estimating the ultimate bearing capacity of shallow foundations through conventional soil mechanics for partially saturated soils may underestimate these values and lead to conservative and costly designs (Mohamed et al., 2011).

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In Cuba, soils in nature do not permanently experience saturation values below 60%, so obtaining the bearing capacity and forecasting settlement for foundations on partially saturated soils is valid for suctions between 0 - 200 kPa, where these represent a degree of saturation that varies from 100 - 70% in the soils used in the research.

2. Obtaining the Bearing Capacity by Using FEM (Finite Element Method).

A shallow foundation with no influence of the water table will be chosen as a case study. The analyzed shallow foundation is square, with B and L sides of 1.5 m, and a foundation depth of 1.5 m. The foundation will be supported on the soil from the Capdevila formation where the resistant capacity will be determined when variations in the degree of saturation occur (Figure 1).

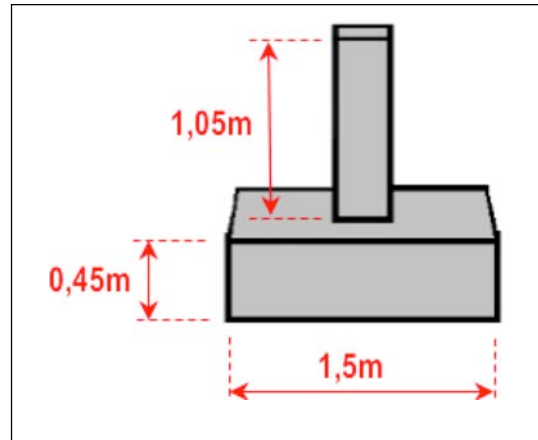


Figure 1. Geometric characteristics of the foundation of the case study.

2.1 Physical-mathematical model to obtain the bearing capacity and settlements in the case of an isolated footing supported on a partially saturated soil

(Figure 2) shows the physical model used to determine the bearing capacity and settlement experienced by an isolated footing subjected to the axial load applied in the center of the footing pedestal. This reaction will be analyzed when there is a variation in the degree of saturation of the bearing soil.

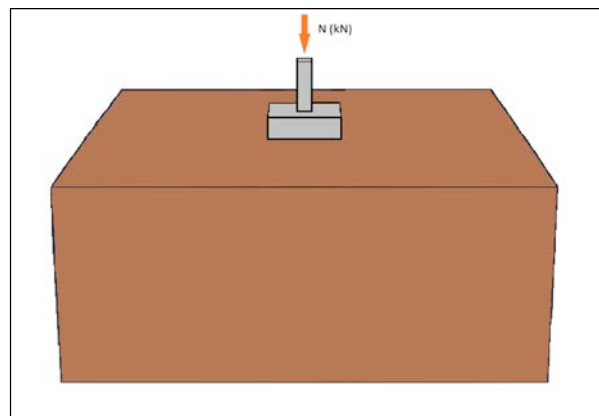


Figure 2. Diagram of the physical model.



2.1.1 Geometric model

The 3D model is made up of a shallow foundation supported on the soil represented as a homogeneous half-space.

Each of the parts that make up the structure has been built separately in the module and then assembled in the software assembly module. The following is a brief description of the design of each part.

- a) *Shallow foundation: It consists of two parts, the base and the pedestal. (Figure 3)*
- b) *Bearing soil: hexahedral structure representing an infinite half-space whose volumetric dimensions are $9 \times 9 \times 3.75$ m. For the dimensioning of the soil subdomain, the Active Power criterion (H_a) was used, taking into account the proposals of the Cuban Standard (Figure 3).*

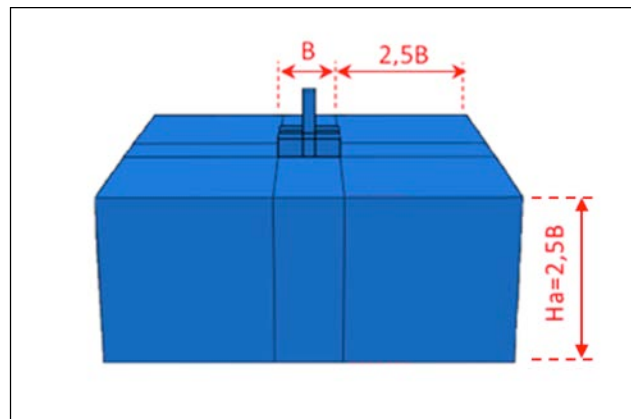


Figure 3. Subdomain dimensions.

2.1.2 Load modeling

The axial load to which the model will be subjected was placed on the foundation pedestal of the case study. Its position, distribution, and magnitude correspond to a value that would allow the foundation to reach the failure; and to be able to obtain the load vs. settlement curve.

In geotechnical problems involving large volumes of natural land, especially in the analysis of the full-scale load-bearing capacity of foundations, it is necessary to add to the mathematical model an initial condition that reproduces the stress state of self-weight in the soil mass.

The pressure on the floor was considered by applying pressure in the supporting half-space of the foundation as pressure on its surface.

2.1.3 Modeling of boundary conditions

Boundary conditions arise from criteria developed by several authors and reflected in the works of (Tristá, 2015), where they consider the restriction of movements in the horizontal directions U_1 and U_2 in the periphery of the subdomain. At the lateral boundary, the horizontal movements are restricted and the vertical movements are not restricted to allow the rearrangement of the soil particles by imposing a tensile state of self-weight, gravitational load, and floor pressure, allowing the movements in that direction to spread to the end of the continuum without generating distortions in the numerical model. At the bottom of the continuum, restrictions were placed in the three directions, considering this boundary to be outside the influence zone of the foundation (Figure 4).

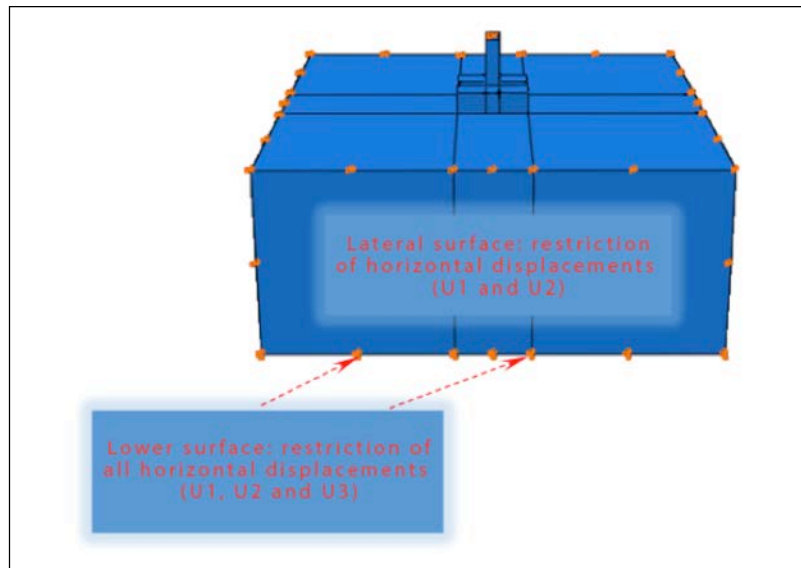


Figure 4. Boundary conditions of the model.

2.1.4 Interaction model

In this case, the contact between the materials of each element of the model is defined, i.e. the concrete of the foundation and the surrounding soil:

Soil-structure: for this interaction, a normal "hard-contact" type of contact was used, with normal frictional and tangential behavior. The foundation was established as the master surface and the bearing soil as the slave surface, so that the interface responds to vertical stresses and at the same time simulates the non-bonding condition of the foundation to the soil, allowing it to separate in some areas. This represents a phenomenon that occurs in the real physical model (Figure 5).

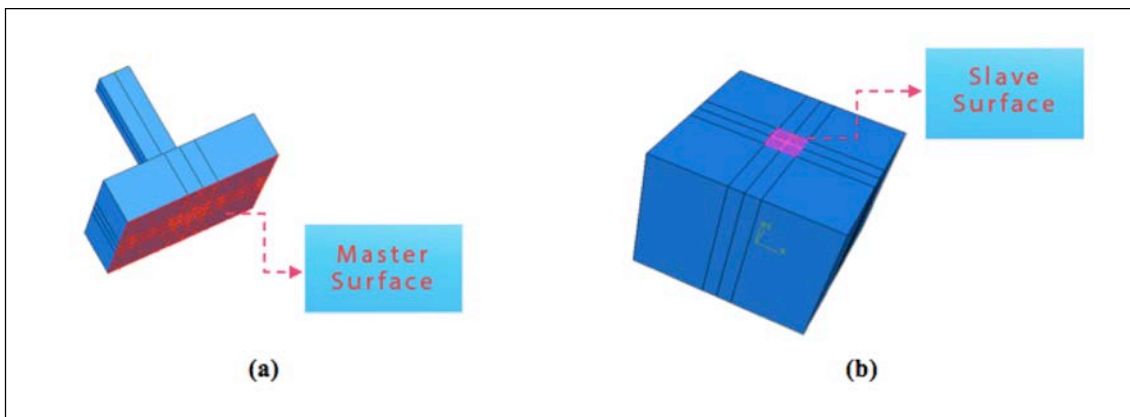


Figure 5. (a) Master surface in the foundation-soil interaction (b) slave surface in the foundation-soil interaction

2.1.5 Constitutive model for material

The study of constitutive models has evolved greatly in academic circles but its practical application in design has been limited, mainly because of its poor dissemination and the need to use a greater number of parameters than those required in traditional methodologies (Camacho and Reyes, 2005).



The constitutive modeling for a material is one of the most important problem-solving elements in engineering. If a proper constitutive model is not assumed, the results obtained would not be valid, taking into account that a wrong behavior of the material is considered when faced with the effect of the loads.

a) Bearing soil modeling

The most used model in geotechnics is the elastic model with the Mohr Coulomb failure criterion. This constitutive model is very popular because it has only 4 parameters and all of them have a physical explanation. In addition, the parameters can be obtained from a triaxial test (Mendoza et al., 2018).

The constitutive model for soil is assumed with an elastic-plastic behavior according to the Mohr-Coulomb criterion in order to take advantage, in the determination of the ultimate bearing capacity, of the behavior of the material subjected to stresses that exceed the limit of elastic linearity and the formation of plasticizing zones.

The creep or failure criterion allows the evaluation of the tensile state of the material at each stress increase to determine its behavior state (elastic or plastic). In the Mohr-Coulomb criterion, the failure at one point in the soil is given by the linear relationship between shear stress (τ) and normal stress (σ), through the shear strength parameters: cohesion (c) and angle of internal friction (ϕ) (Equation 1).

$$\tau = c + \sigma \tan \phi \quad (1)$$

The behavior model attributed to the soil has the parameters of the elastic behavior (elastic modulus E and Poisson's ratio ν) and the parameters that characterize its mechanical shear strength: cohesion (c), angle of internal friction (ϕ) and angle of dilation (ψ_d).

The angle of dilation is controversial and influences the geotechnical behavior of shallow foundations (Gonzalez-Cueto et al., 2013); (Mohamed et al., 2011). Therefore, in the absence of a single criterion, a value of 10% of the angle of internal friction ($\psi_d = 10\% \phi$) was considered in the case study as proposed by (Vanapalli and Mohamed, 2007) for partially saturated soils. The relationship between the elastic modulus (E) and the angle of friction of the modeled soils was established according to (Vanapalli and Mohamed, 2007) and the soil cohesion parameter was modified considering the approach of (Fredlund and Rahardjo, 1993) (Equation 2) and (Equation 3) and (Vanapalli and Mohamed, 2007) (Equation 6) and (Equation 8). (Table 1).

- **(Fredlund and Rahardjo, 1993) formula** cited by (Tristá et al., 2017)

$$c_{unsat} = c_{sat}' + (u_a - u_w) \tan \phi^b \quad (2)$$

$$\phi_{unsat} = \phi_{sat} \quad (3)$$

Where c_{sat}' is the effective cohesion, $(u_a - u_w)$ is the matric suction, ϕ_{sat}' is the effective angle of friction and ϕ^b is the angle that indicates the increase rate of shear strength relative to matric suction, provided that $\phi^b < \phi_{sat}$.

On the other hand, Fredlund states that there is a relationship between ϕ^b and ϕ_{sat} where it is given by:

$$\chi = \frac{\tan \phi^b}{\tan \phi_{sat}} \quad (4)$$

If the criterion $\chi = S_r$ is considered to be valid, then:

$$\phi^b = \arctan(S_r \tan \phi_{sat}) \quad (5)$$

- **(Vanapalli and Mohamed, 2007) formula** cited by (Tristá et al., 2017)

$$c = c' + (u_a - u_w)_b (\tan \phi_{sat} - S_r^\psi \tan \phi_{sat}) + (u_a - u_w)_{AVR} S_r^\psi \tan \phi_{sat} \quad (6)$$

$$\psi = 1.0 + 0.34 (IP) - 0.0031 (IP^2) \quad (7)$$

$$\phi_{unsat} = \phi_{sat} + \psi_d \quad (8)$$



Where c' is the effective cohesion, γ is the moist unit weight, d is the depth of the foundation, B is the width of the foundation, $(u_a - u_w)_b$ is the air inlet value of the soil water retention curve, $(u_a - u_w)_{AVR}$ is the suction of the interval to be analyzed, ϕ_{sat} is the effective angle of friction, S_r is the degree of saturation, Ψ is the model adjustment parameter. The parameter Ψ depends on the soil plasticity index (IP), (Equation 7).

Table 1. Properties of the soil from the Capdevila formation used in the model

Physical-mechanical properties						
S (kPa)	ϕ' (°)	Specific weight (kN/m³)	E (kPa)	ν	C_{Vanapalli} (kPa)	C_{Fredlund} (kPa)
0	31.88	18,1	20000	0,3	2,34	2,34
50		18,1			23,79	25,08
100		18,0			46,60	46,91
150		17,7			59,60	64,42
200		17,4			61,62	78,74

b) Concrete modeling

The foundation sub-domain is composed entirely of concrete, with an elastic linear physical-mechanical behavior, characterized by the properties shown in (Table 2).

Table 2. Concrete properties used in the foundation

f'c (MPa)	E (MPa)	γ(kN/m³)
25	23500	24

2.1.6 Model correction

To apply the FEM it is necessary to select the type of finite element to be used and to determine the minimum mesh density necessary to calculate the bearing capacity of the shallow foundation of the case study.

In the selected software, three basic finite element geometries are available for the three-dimensional design of the model: the tetrahedra, triangular base prisms ("wedges"), and hexahedra which can be of linear interpolation if they are considered as nodes only at their vertices (C3D4, C3D6 and C3D8, respectively) or quadratic interpolation if they are considered as intermediate nodes at the edges (C3D10, C3D15 and C3D20). Although quadratic interpolation ensures greater accuracy of the results in the numerical modeling, the computational cost is also higher, so it can be assessed as an alternative to achieve the required accuracy by densifying the meshes of elements and not in the order of individual interpolation, which was the decision made for this research.

(Haramboure, 2014) states that tetrahedral elements (C3D4 and C3D10) are not recommended to be used in regions with high stress and strain gradients and "wedge" type elements (C3D6 and C3D15) are recommended to be used to complete meshes in abrupt geometries. For this reason, both are excluded from this research. In view of the above, it was decided to apply hexahedral elements of linear interpolation.

Within the hexahedral elements of linear interpolation included in Abaqus/CAE, it is possible to consider the order of maximum (C3D8) or reduced (C3D8R) integration. Reduced integration means a decrease in the running time of the numerical models as they have fewer points of Gaussian integration especially in the three-dimensional analysis, also depending on the nature of the physical problem to be addressed.

In the case of the selection of the finite element type, in the studies carried out by (Haramboure, 2014) it was shown that the first order hexahedral elements of interpolation and reduced integration, C3D8R, have a lower variation in the load curves vs. strain with respect to the physical test, thus allowing a better adjustment of the



numerical modeling results with respect to the physical tests, with a mean absolute error of ($e_{ma} = 0.04$) compared to the C3D8 elements.

Another aspect to consider in the numerical solution method used is the recommended mesh density. For its selection, mesh densities ranging from (0.25 to 0.55 m) were studied, taking into account the number of nodes and the running time of the model. For this model, a mesh was selected where the values in the plots of settlement vs. number of nodes and settlement vs. running time began to have an asymptotic behavior. The results of the calculation of the absolute error (Equation 9) for the different mesh densities were also taken into consideration.

$$e_a = \left| \frac{S_n - S_t}{S_t} \right| \quad (9)$$

Where S_n is the settlement value of the node under study determined with the “n” mesh and S_t is the settlement value of the node under study determined in the denser mesh.

The numerical correction process shows the movement results (U_3) obtained for different mesh densities, the time generated in each run, the number of elements, and the number of nodes (Figure 7). Node 46 (Figure 6) was chosen for this analysis, observing the behavior of the stresses in the reference nodes within the soil and the running time of the model.

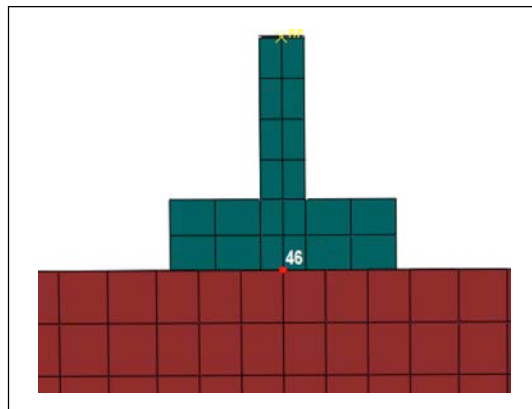


Figure 6. Reference node (46), for settlement control calculated with different mesh densities.

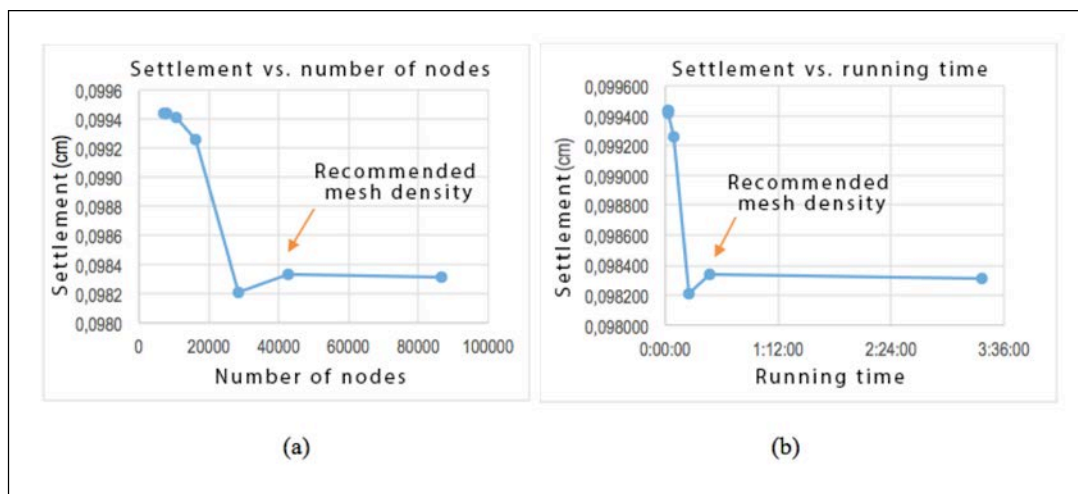


Figure 7. Mesh correction. (a) settlement vs. number of nodes, (b) settlement vs. running time.



Finally, the recommended mesh to perform the modeling was 0.3 m where both the "settlement vs. running time" and "settlement vs. the number of nodes" curves remain relatively constant with an absolute error equal to zero ($e_a = 0,00$) (Figure 8).

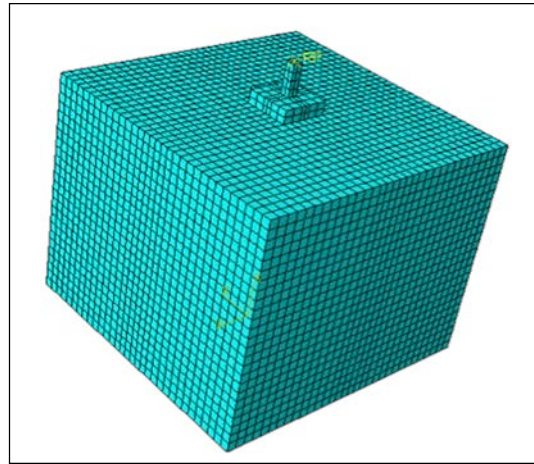


Figure 8. Mesh density to be used in the model, 0.3 m.

The soil-structure interaction between the foundation and the soil for all the cases studied was defined by the Coulomb's friction law (Román and Chio, 2018); (Tristá, 2015) (Equation 10) whose application in Abaqus/CAE (as a "penalty method") describes it through the proportionality between the tangential stresses (τ_T) and the normal stresses (σ_N) that are generated in the contact surface through a friction coefficient μ .

$$\tau_T = \mu \sigma_N \quad (10)$$

The value of the soil-structure coefficient of friction to be used in different cases depends on the type and material of the structure, construction method, and physical-mechanical properties of the soil, among other aspects. It can even take different values for static friction ("peak" value) and dynamic friction ("residual" value).

To determine the soil-structure coefficient of friction in the contact of the shallow foundations with the base soil, numerical load tests on the flat shallow foundation were modeled in this study for different values of the coefficient of friction, and the ultimate bearing capacity values obtained from these tests were compared with those established analytically, modifying the soil strength parameters in the equation of bearing capacity proposed by Brinch-Hansen by taking into account the transformation of the soil strength parameters proposed by Vanapalli and Fredlund (Figure 9).

From this analysis, it was determined that the soil-structure coefficient of friction that provided the best adjustment to the analytical results of ultimate bearing capacity was a value of $\mu = 0.08$. Therefore, it is evident that there will always be an influence of friction between the soil and the structure, although in this case it is small. For practical reasons, it was decided to adopt $\mu=0$ as the soil-structure coefficient of friction and not to consider the influence of friction in both cases.



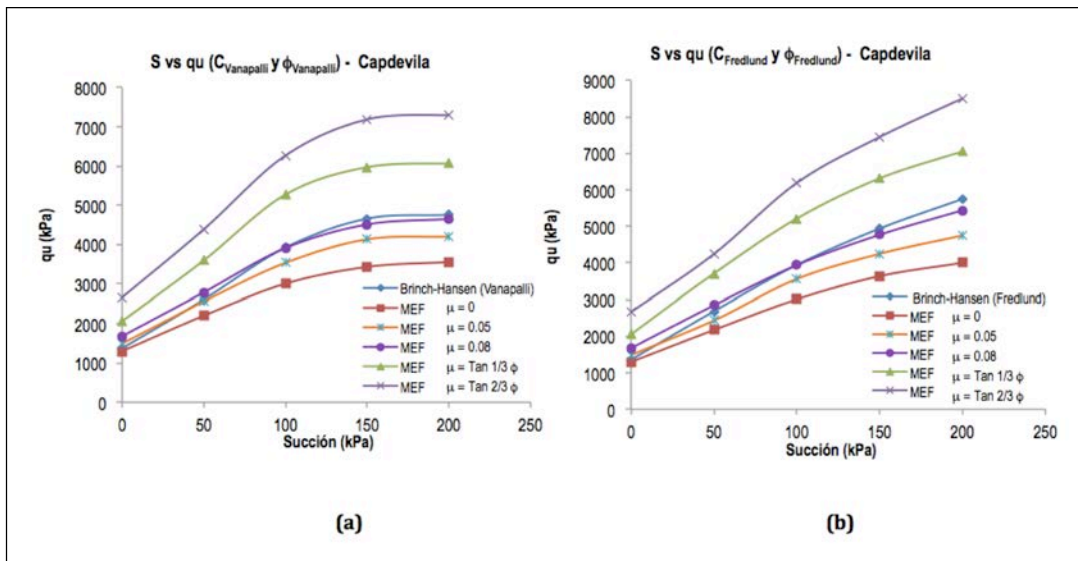


Figure 9. Obtaining the soil-structure coefficient of friction (μ) for the application of FEM calculated by analytical and numerical methods. (a) Ultimate bearing capacity for the Brinch-Hansen method with $(C_{Vanapalli}$ and $\phi_{Vanapalli}$), (b) ultimate bearing capacity for the Brinch Hansen method with $(C_{Fredlund}$ and $\phi_{Fredlund}$).

3. Results

The bearing capacity is then obtained, using the finite element method (FEM), and the settlements experimented under a shallow foundation supported on the soil from the Capdevila formation when a variation in suction occurs due to changes in the degree of saturation of the soil.

Several factors are involved in the determination of the bearing capacity of a square shallow foundation. These factors depend on the shape, depth, slope, and shear strength of the soil, which is defined by the cohesion and angle of internal friction of the soil, which, in turn, depends on the existing variation of the suction of the soil.

3.1 Resistant bearing capacity

For the analysis of the resistant bearing capacity of the aforementioned foundation, the characteristics and properties listed in (Table 3) were taken as initial data.

Table 3. Data from the foundation and the soil from the Capdevila formation used in the modeling

Foundation			Capdevila formation				
B (m)	L (m)	D_f (m)	ϕ_{sat} (°)	ψ_d	C_{sat} (kPa)	IP (%)	$(u_a - u_w)_b$ (kPa)
1.5	1.5	1.5	33.4	3.34	3.4	30.6	120

- Finite Element Method (FEM)

The results of the numerical modeling in the software for the voids content ($e = 0,842$) are detailed as follows.

For the plots obtained from the models analyzed for the same suction range, where both the strength parameters and their physical-mechanical properties vary, the characteristics of the soil from the Capdevila formation were taken into account. The plots obtained from the modeling are shown below (Figure 10).



(Figure 10) shows that as the suction increases, the bearing capacity increases until it reaches a point where the suction is 150 kPa for the bearing capacity modified with ($C_{Vanapalli}$ and $\phi_{Vanapalli}$). There, it is observed that it begins to decrease as a rearrangement of the soil particles is produced and consequently, there is a drop in the stress and resistant capacity. This differs from the bearing capacity modified with ($C_{Fredlund}$ and $\phi_{Fredlund}$), which increases as the suction on the soil increases.

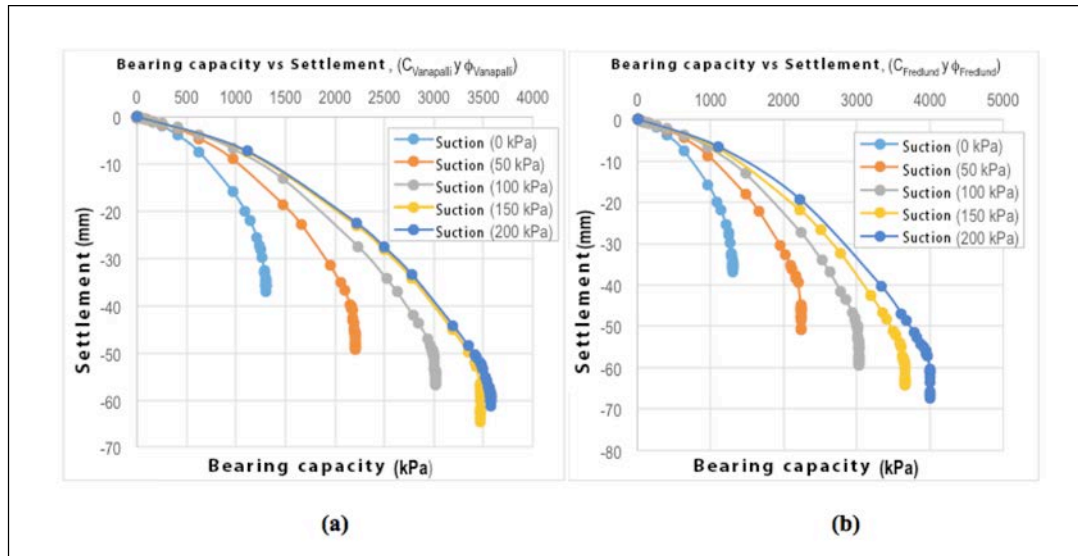


Figure 10. Bearing capacity results obtained in MEF modeling for a voids content of 0.842. (a) Modified with ($C_{Vanapalli}$ and $\phi_{Vanapalli}$) and (b) modified with ($C_{Fredlund}$ and $\phi_{Fredlund}$).

4. Conclusions

The ultimate bearing capacity values experience a high increase when the coefficient of friction μ has values other than zero, moving away from the behavior obtained when analytical formulations are used, taking as a criterion that the valid coefficient of friction for this analysis is $\mu = 0$.

The ultimate bearing capacity obtained by FEM at the base of the shallow foundation under study using the Abaqus/CAE software does not exceed the capacities calculated analytically through the Brinch-Hansen formula when the strength parameters of the partially saturated soil are modified using Vanapalli and Fredlund. In this case study, it was concluded that the use of FEM provides more conservative ultimate bearing capacity values than those obtained analytically.

The method that best describes the behavior of the partially saturated soil from the Capdevila formation is the approach of Vanapalli for the modification of the soil strength parameters, which includes in its formula the soil characteristics through the use of suction, the degree of saturation and the soil plasticity index, and unlike the formula proposed by Fredlund in which the bearing capacity value increases as the matric suction of the soil increases indefinitely. In the case of the Vanapalli's formula, this bearing capacity value begins to decrease for a given suction value, which is a function of the type of soil, revealing a readjustment of the particles when there is a decrease in the degree of saturation.



5. References

- Al-shamrani, M. A.; Ali, A.; Moghal, B. (2015).** Closed-form solutions for bearing capacity of footing on anisotropic cohesive soils. *Revista Ingeniería de Construcción*, V. 30(2), 109–125. <http://ricuc.cl/index.php/ric/article/view/574/pdf>.
- Camacho, J.; Reyes, O. (2005).** Aplicación del modelo cam-clay modificado en arcillas reconstituidas del la sabana de Bogotá. *Revista Ingeniería de Construcción*, V. 20(1), 47–58. <http://ricuc.cl/index.php/ric/article/view/CAMACHO/pdf>.
- Fredlund, D. G.; Rahardjo, H. (1993).** *Soil Mechanics for Unsaturated Soils* (I. JOHN WILEY & SONS, ed.). University of Saskatchewan: A Wiley-Interscience.
- Gonzalez-Cueto, A. V.; González, E.; Quevedo Sotolongo, G. (2013).** Asentamientos, Aplicación de Métodos Analíticos y Numéricos en el cálculo de The, de cimentaciones superficiales en suelos friccionales. 10mo Simposio Internacional de Estructuras, Geotecnia y Materiales de Construcción. Cayo Santamaría, Villa Clara, Cuba. ISBN: 978-959-250-929-0.
- Haramboure, Y. (2014).** Influencia de la directriz de un cimientto lanimar de revolución en el estado tenso-deformacional del suelo. Caso de estudio: Depósitos elvados tipo Guira. Instituto Superior Politécnico "José Antonio Echeverría". Tesis de Doctorado. <http://tesis.cujae.edu.cu:8080/xmlui/handle/123456789/6315>.
- Mendoza, C.; Ruge, J. C.; & Caicedo, B. (2018).** The geological history analysis of the friction angle in transported soils and their importance in the bearing capacity of shallow foundations. *Revista Internacional de Métodos Numéricos Para Cálculo y Diseño En Ingeniería*, 34(1). <https://doi.org/10.23967/j.rimni.2017.8.003>
- Mohamed, F. M. O.; Vanapalli, S. K.; Saatcioglu, M. (2011).** Bearing capacity and settlement behaviour of footings in an unsaturated sand. In 14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering: Vol. October 2 (pp. 1–8).
- Román, D.; Chio, G. (2018).** Análisis de modelos de deslizamiento en bloque para predecir el comportamiento dinámico del fenómeno de remoción en masa: Modelo Uniparamétrico y Modelo Biparamétrico. *Boletín de Geología*, 40(2), 113–124. <https://doi.org/10.18273/revbol.v40n2-2018007>.
- Tristá, J.; Cobelo, W.; Quevedo, G. (2020).** Análisis del comportamiento tenso-deformacional de un suelo parcialmente saturado para un cimientto circular bajo carga axial. *Ingeniería y desarrollo: revista de la División de Ingeniería de la Universidad del Norte*, 38(1), 181–196. <https://doi.org/http://dx.doi.org/10.14482/inde.35.2.10169>.
- Tristá, J. G. (2015).** Estudio del comportamiento tenso-deformacional de suelos parcialmente saturados en Cuba. Universidad Central "Marta Abreu" de las Villas. Facultad de Construcciones. Tesis de Doctorado. <http://tesis.cujae.edu.cu:8080/xmlui/handle/123456789/5336>.
- Tristá, J. G.; Cobelo, W.; Quevedo, G. (2017).** Bearing capacity of footings in unsaturated soils employing analytic methods. *Ingeniería y Desarrollo. Universidad Del Norte*, 35(2), 417–430. <https://doi.org/http://dx.doi.org/10.14482/inde.35.2.10169>.
- Vanapalli, S. K.; Mohamed, F. M. O. (2007).** Bearing capacity of model footings in unsaturated soils. In Pringer-Verlag (Ed.), *In Experimental Unsaturated Soil Mechanics* (pp. 483–493). Berlin Heidelberg: Springer Proceedings in Physics.

