Methodology for calculate bearing capacity of soft soils under cyclic loading

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Abstract. The bearing capacity in soft soils under cyclic loading has not been widely studied as the static bearing capacity. In soft saturated soils under cyclic load, the phenomenon of cyclic softening occurs, this consists of an increase in the generation of pore-water pressure due to the cyclic load and a decrease in effective soil stresses. The purpose of this work was to propose a methodology for calculating the bearing capacity in soft soils under cyclic loading. The research is based on the results of monotonic and cyclic simple shear tests carried out with samples from the PRAT port of Barcelona. From an analysis carried out on all the experimental data, an equation for the generation of pore-water pressure is proposed. This formulation was implemented in the FLAC2D finite difference software and is part of the methodology. The methodology of the proposed calculation process consists of 6 stages and verification under two criteria. This process is iterative in case of not meeting any of the two verification criteria. Under this methodology of the calculation process, charts are proposed to calculate the bearing capacity of soft soils under cyclic loading. This application takes into account the properties of the soil, the static bearing capacity (Phe), and the effective load outside the foundation (q). Also, an analysis of the parameters that most influence the bearing capacity of soft soils under cyclic loading was carried out.

1. Introduction

Bearing capacity under dynamic loads (Phd) over time has been resolved indirectly, through an equivalent pseudostatic approach or through a reduction coefficient [1]. The first who worked in a pseudostatic approach were Meyerhof and Shinohara [2,3] who applied horizontal and vertical accelerations to the center of gravity of the structure and the problem is reduced to a static case with inclined eccentric loads. However, this type of bearing capacity in cohesive soils had not been much studied until the earthquakes, Northridge in 1994, Kocaeli in 1999 and Chi-Chi in 1999 [4,5]. Marcuson [6] defined the phenomenon of liquefaction as the transformation of a granular material from solid state to a liquefied state as a consequence of the increase in the pore water pressure and reduction of the effective stress. A similar phenomenon occurs in cohesive soils (clays), but unlike granular soils, the effective stress does not necessary reach to zero value and the soil fails, this phenomenon is called cyclic softening.

Cyclic softening leads in engineering practice to problems in the design of foundations for structures built along the coast, offshore installations, structures such as ports, breakwaters
and storm surge barriers or other types of foundations that must resist cyclic loading and are founded on saturated cohesive soils [7, 8]. Tsai et al. [9] argues that cyclic softening and loss of strength in saturated clays are fundamental problems in engineering, such as slope stability, dam safety or the support capacity of foundations.

As consequence to this problem, the interest arises to study the calculation of the bearing capacity of soft soils under cyclic loading. The present work is based on cyclic simple shear tests carried out by Patiño [10] to samples from the port of Barcelona at PRAT pier. A formulation of pore water pressure generation was obtained from the tests and was later implemented in the FLAC2D finite difference software [11]. A methodology of the calculation process is proposed in the software to be able to determine the behavior and bearing capacity of soft soils under cyclic loading before cyclic softening occurs.

2. Background

2.1. Cyclic softening

One of the first to investigate the behavior of soft clays under cyclic loading were Idriss et al. [12]. However, one of the first conceptualizations of cyclic softening was made by Ishihara [13], describing it as the reduction of the stiffness and strength of the soil due to a repeated cyclic load in a clay. The first reported case of cyclic softening corresponds to the 1964 Alaska earthquake [14, 15]. As well as, Bray and Sancio [4] from observations and results of cyclic tests showed cyclic softening in fine-grained soils during the Northridge (1994), Kocaeli (1999) and Chi-Chi (1999) earthquakes.

From these events, various types of analysis for cyclic softening arise, one of them proposed by Bray and Sancio [4] suggests the analysis based on the plasticity index. Boulanger et al. [16, 14, 17] presented a criterion based on the mechanical stress-strain behavior. Tsai et al. [9] proposed a simplified procedure based on the amplitude of the cyclic shear stress and the equivalent number of cycles. This way, the main factors that affect cyclic softening began to be investigated. Bray and Sancio [4] takes into account factors such as confining pressure, static initial shear stress, stress-path, soil mineralogy, void ratio and overconsolidation ratio. Patiño et al. [18] showed that the initial shear stiffness of a soft clay reduces after several loading cycles, in the same way the cyclic shear stress ratio necessary to reach failure decreases as the number increases of cycles. On the other hand, Leboeuf et al. [19] indicates that factors such as the accumulation of pore-water pressure, the effect of fatigue and strain speed are very important. Martinez et al. [20] proposed a methodology to study the risk of sudden failure, allowing an adequate analysis of the degradation of the modulus of shear stiffness, the influence of the static shear stress, generation of pore-water pressure and the evolution with the number of cycles. Kumar et al. [21] indicated that the reduction of the shear modulus and the damping ratio are not affected by changes in initial dry density and water content.

On the other hand, Chavez et al. [22] reproduced the collapse of a coastal structure due to cyclic softening in small-scale laboratory conditions and under controlled conditions. Eslami et al. [23] indicate from their studies that the plasticity index is an insufficient indicator to evaluate the potential cyclic softening.

2.2. Pore-water pressure generation

The generation of pore-water pressure was developmented along the time by various empirical formulations, from the first given by Seed et al. [24] as well as Martin et al. [25], Byrne [26] or the case of Matasović and Vucetic[27], who developed pore-water pressure generation
models of generalized degradation. Hyde and Ward [28] proposed a power model for the pore-water pressure of silty clay. Later, after the events mentioned above, models such as the one proposed by Green et al. [29] arise that relate the generation of residual pore-water pressure with the energy dissipated per unit volume of the soil. Galindo Aires et al. [30, 31, 32] proposed through the evaluation of the degradative damage produced, solve the evolution of the cycles and generation of pore-water pressure under cyclic loads. Ni et al. [33] presented a constitutive model that predicts the behavior of soft clays under undrained cyclic triaxial load. Ren et al. [34] proposed a hyperbolic model to predict the development of undrained pore-water pressure.

2.3. Bearing capacity under cyclic load

It is very important to know the bearing capacity under cyclic loading for design of foundations in offshore structures, near the coast or on land [35, 34]. A structure such as a foundation under cyclic loading produces various stress conditions below the soil. The Figure 1, shows in a simplified way the stress conditions along a potential ground fault surface. Stress conditions that can be simulated in the laboratory with a simple cyclic shear test (case 1 and 3) or triaxial compression test (case 2) or triaxial extension (case 4). In this research it will focus on cases 1 and 3 because there are cyclic simple shear tests available.

Figure 1. Hypothetical fault scheme to represent stress conditions that can be simulated with cyclic simple shear tests and triaxial tests [18].

Chu et al. [36] performs a bearing capacity analysis using strength reduction to account for cyclic softening by comparing safety factors during the earthquake versus bearing capacity failure. Cascone and Casablanca [37] uses the characteristics method to evaluate the static and seismic bearing capacity of both rough and smooth strip footing under a pseudostatic approach. Moreover, Pane et al. [38] work based on the pseudostatic approach in the finite difference method (FDM) to evaluate the seismic effects on the bearing capacity of strip footing. Conti [39] worked on the seismic bearing capacity of strip footings based on a pseudostatic approach and limit analysis in cohesive-frictional and purely cohesive soils.

Most of the methods do not take into account the effects of pore-water pressure, nor the reduction of shear strength of the soil due to seismic effects. Cinicioglu and Erkli [40] considers a pseudostatic coefficient to define the seismic load but does not take into account the pore-water
pressures. As well, argues that for the calculation of the seismic bearing capacity in cohesive
soils the influence of all the elements is not considered parameters, and therefore in practice has
its limited use.

3. Experimental data
The experimental data of the present work were carried out by Patiño Nieto [10] to unaltered
samples from subsoil borings of pier the Prat in Barcelona. The soil deposit is of alluvial origin
that belongs to the Holocene era and was classified as silty clays and clayey silts of low to medium
plasticity. The behavior of this cohesive soil was described as typically plastic, with positive
pore-water pressure that shows a contractive behavior, making it a normally consolidated soil
or with a low degree of consolidation.

3.1. Identification and classification
The geotechnical characterization was based on the classification and index properties as shown
in Table 1. These samples for all the tests were determined from a depth between 30 and 52
meters with respect to sea level.

| Parameter          | Unit       | N° of data | Average | Range   |
|--------------------|------------|------------|---------|---------|
| Natural moisture   | %          | 78         | 29.24   | 24-37   |
| Natural unit weight| g/cm³      | 78         | 1.97    | 1.85-2.08|
| Content of fines   | %          | 78         | 98.66   | 87.26-99.99|
| D<2µ               | %          | 14         | 30.21   | 14-41   |
| Liquid limit       | %          | 16         | 40.25   | 30.50-45.30|
| Plastic limit      | %          | 16         | 22.58   | 18.20-25.50|
| Plasticity index   | %          | 16         | 17.67   | 12.30-19.90|
| Specific gravity   | -          | 16         | 2.78    | 2.74-2.80|

Patiño Nieto [41] based on identification and classification tests indicated:
- The soil deposit studied is heterogeneous, laminated and made up of small-thickness
  intercalations.
- Natural moisture is between the liquid limit and plastic limit, which is typical for deposits
  with a small degree of pre-consolidation.
- According to the consistency limits and based on the plasticity chart, the deposit is
  considered to be made up of silty clays of low plasticity.

| Mineral           | [%] |
|-------------------|-----|
| Calcite           | 42  |
| Quartz            | 26 - 36|
| Chlorite          | 11 - 16|
| Albite (feldspar) | 7 - 11|
| Muscovite (mica)  | 4 - 6|
The mineralogical content was obtained by 13 determinations by the x-ray diffraction method and the results are shown in Table 2.

3.2. Monotonic and cyclic simple shear tests
The experimental data consists of 16 monotonic simple shear tests and 78 cyclic simple shear tests at different combinations of static and cyclic shear stresses as shown in Figure 2. On the vertical axis, it show the cyclic shear stress (Δτ_c) normalized with respect to the vertical effective stress of consolidation (σ'_ov) and on the horizontal axis, the static shear stress (τ_0) also normalized by the vertical effective stress of consolidation. The tests were first consolidated to the field stress and subsequently the different combinations of shear stresses were applied.

![Figure 2. Combination of stresses from static and cyclic simple shear tests.](image)

The description of equipment used, the treatment of the samples so that they are unaltered (unconventional method), the conditions that were set during the tests to eradicate the factors that could influence the behavior of the soil and the complete results can be consulted at Patiño et al. [18].

The behavior of the monotonic simple shear tests showed that the resistance to undrained shear reaches maximum values for very large shear strains that vary from 12 to 22%. Moreover, the maximum pore-water pressure generation that it developed was 50% of the effective vertical stress in situ, which in terms of shear strain represents values greater than 15%. It is worth mentioning that, for this cohesive soil, the maximum stress values reached are for large strains, something that may be unacceptable for certain structures that want to be cemented on this soil.

In the case of the cyclic simple shear tests, the evaluation and interpretation of the results was carried out in a more detailed way and is part of the objective of this work. In this way, the analysis and interpretation of the results will be shown below to determine a model of pore-water pressure generation under cyclic loading and the development of a numerical methodology.

4. Pore-water pressure generation
4.1. Pore-water pressure generation under static load
The generation of pore-water pressure and shear stresses under monotonic load was analyzed for the 16 tests and can be seen in Figure 3. From these tests it can be clearly seen that the
maximum values of pore-water pressure and shear stress are for large shear strains between 15 and 20%.

![Figure 3. Shear stresses and pore-water pressure under monotonic load.](image)

An analysis was performed to determine the model that best fits for the evaluation of shear stresses and pore-water pressure generation under monotonic load with the following expressions:

\[
\tau_{xy} = e^{\left(\beta_0 + \frac{\beta_1}{\gamma_m}\right)} \cdot \sigma'_{ov} \quad (1)
\]

\[
u = e^{\left(\beta_2 + \frac{\beta_3}{\gamma_m}\right)} \cdot \sigma'_{ov} \quad (2)
\]

Where:

- \(\tau_{xy}\) Shear stress.
- \(\sigma'_{ov}\) effective vertical stress “in situ”.
- \(\nu\) pore-water pressure generation.
- \(\gamma_m\) monotonic shear strain.
- \(\beta_0, \beta_1, \beta_2, \beta_3\) empirical constants.

The best fit for the models of shear stress has a correlation of 82%, while for the generation of pore-water pressure it is 86.3%.

4.2. Pore-water pressure generation under cyclic load

The generation of pore-water pressure under cyclic loading was analyzed from various parameters that can influence it. The most influential parameters that were recorded in the tests were the void ratio, the static and cyclic shear stress. The void ratio was analyzed as shown in Figure 4, it can see that there is a linear fit of the data almost horizontal, which denotes that there is a mean value for each case of cyclic shear stress.
Moreover, an analysis of the initial shear stress and cyclic shear stress with the generation of pore-water pressure was performed, as can be seen in Figure 5. Figure 5 a) shows how the generation of pore-water pressure decreases with the increase of normalized initial shear stress. The maximum pore-water pressure generation values occur for zero initial shear stress, and varying from 0.60 to 0.90. The minimum pore-water pressure generation values occur for maximum initial shear stress values; these values are around a value of 0.40. In Figure 5 b), it is shown how the pore-water pressure generation increases as the cyclic shear stress increases. It is shown that the cyclic shear stress values are in a range between 0.05 and 0.25; this is because a null value indicates that it is a static case. The minimum pore-water pressure values are in the order of 0.40 for a cyclic shear stress of 0.05, while the values for a cyclic shear stress of 0.25 are around the value of 0.70.

**Figure 4.** Variation of void ratio with normalized cyclic shear stress.

**Figure 5.** Variation of pore-water pressure with normalized initial shear stress and normalized cyclic shear stress.
As a first conclusion of the analysis, it can indicate that the generation of pore-water pressure never reached a value of 1, which would represent canceling the effective stress and that it is common to observe in a granular soil (sands) with the phenomenon of liquefaction. Consequently, it can be verified that a cohesive soil reaches the fault without precisely having to decrease the effective stresses to zero, a phenomenon that is called cyclic softening.

Based on three analyzes carried out and under a careful study, the most suitable fit model for the experimental data was determined. This adjustment carries with it a structure of two main parts ($\Delta u = A + B$), where the first part $A$, depends on the initial shear stress, cyclic shear stress and voids ratio. In this first part the variable of the initial and average void ratio is introduced, which significantly increases the correlation. Finally, part $B$, depends only on the cyclic shear stress. The adjustment for the formulation had a correlation of 85%, which is considered very adequate due to the variables entered. Therefore, the formulation is detailed as follows,

$$\frac{\Delta u}{\sigma_{ov}} = a_1 \cdot \left( \frac{\tau_0}{\sigma_{ov}} - \frac{\tau_c}{\sigma_{ov}} \right) \cdot \left( e_0 - \bar{e} \right) + e \left( \beta_0 + \beta_1 \frac{\tau_c}{\sigma_{ov}} \right)$$

(3)

Where:

- $\Delta u$ pore-water pressure generation.
- $\sigma_{ov}$ effective vertical stress “in situ”.
- $\tau_0$ initial shear stress.
- $\tau_c$ cyclic shear stress.
- $e_0$ voids ratio.
- $\bar{e}$ average voids ratio.
- $a_1, \beta_0, \beta_1$ empirical constants.

![Proposed equation](image)

**Figure 6.** The fit of pore-water pressure generation under cyclic loading for estimated and actual data.
The fit of pore-water pressure generation under cyclic loading for estimated and actual data is shown in Figure 6. It can be observed that the most significant variable in the generation of pore-water pressure is the cyclic shear stress, above the voids ratio or the initial shear stress. However, it can be seen that the number of charge cycles is not taken into account. This is because it focuses only on the critical point when the cyclic softening of the soil will occur. For this reason, the intermediate behaviors that the soil may have are not considered. In other words, the proposed formulation is only for the maximum generation of pore-water pressure, or represent the generation of pore-water pressure in the critical state and the failure of the soil would occur.

5. Numerical methodology
The implementation of the equation proposed pore-water pressure generation was carried out in a finite difference computational code FLAC2D [11]. This software allows, through an internal programming option (FISH), to obtain and calculate variables as well as to control the analysis process. In this way, the proposed formulation was inserted under the general considerations of plane deformation, small strains, the Mohr-Coulomb failure criterion and a flow rule not associated with an angle of dilatation equal to zero.

![Scheme of numerical model used in FLAC2D.](image)

5.1. Numerical model
The numerical model was determined from an analysis of variables such as the generation of pore-water pressure, the slide surface, the mesh size and the velocity of load application. To determine the size of the model, the zone of influence of the pore-water pressure and the slide surface were considered. From this analysis, the most suitable size of (20 × B) wide by (12 × B) high was determined, where B is the width of the footing as shown in Figure 7. The influence
of pore-water pressure limited the height of the model, while the width was the slide surface. As shown in Figure 7, there is an intermediate zone at $6 \times B$, this zone to optimize and reduce calculation time, the mesh is more refined because it is the zone of influence of the pore-water pressure and the slide surface when soil failure. Moreover, the application of vertical load on the footing according to the software conditions is given by a controlled descending velocity on the nodes of the footing surface.

For the mesh size and the velocity of load application, a convergence analysis was made as shown in Figure 8 to determine these two variables. This analysis was carried out in order to optimize the calculation time.

![Figure 8](image)

**Figure 8.** Convergence analysis to determine mesh size and velocity of application of the load on the footing.

In Figure 8 a) it can see the behavior of the load as the mesh size decreases, being for a size from 0.5m that there is not much variation in the static bearing capacity ($P_{he}$). In the same way, it shows at Figure 8 b) that for values of load application velocity of 2.5e-7, the values of static bearing capacity ($P_{he}$) tend to stabilize. Therefore, the mesh size in the most refined zone is 0.5m and the velocity of application of load on the footing is 2.5e-7 (step/m).

### 5.2. Properties of soil

The properties of the soil used for modeling in FLAC2D were determined from want to reproduce the soil tested in the experimental part. This soil corresponds to a silty clay of low plasticity, for which a value of modulus of elasticity ($E$) of 5 MPa and Poisson’s coefficient ($\nu$) of 0.25 was chosen following the study of the soil of the Port of Barcelona [41]. It is worth mentioning that it worked with the assumption without the self-weight of the soil, so the width of the footing B does not influence the calculations made. For the determination of the cohesion and friction angle parameters, the envelopes of the monotonic tests of Figure 9 are taken into account.

According to the previous analysis, this type of soil based on the tests, fails for large shear strains. The choice of parameters was for a value of 5 and 15% of the shear strain measured in the tests. In this way, the cohesion will be 10 kPa and the internal friction angle $20^\circ$ for a
monotonic shear strain of 5%. For the shear strain of 15%, the cohesion will be 25 kPa and the internal friction angle of 25º as shown in Figure 9. While the dilatation for the present work was taken as a null value.

5.3. Calculation process in FLAC2D

Once the models were established together with the soil parameters, an analysis was carried out under three possible methods of calculating the bearing capacity in FLAC2D. The first corresponds to applying velocity to the nodes of the footing, the second is by calculating the safety factor with the SRM (Strength Reduction Method) technique applied to cohesion and angle of friction, and the third corresponds to calculating the safety factor with reduction only of the cohesion parameter.

From an analysis carried out, it was determined that the most suitable method for our purpose was the bearing capacity. For the present work, not all the methods will be developed and it will only focus on showing the operation of the bearing capacity method for subsequent calculations. The bearing capacity method in FLAC2D models consists of applying a vertical downward velocity along the width of the footing. This velocity is applied to the model at the nodes of the footing, it is controlled and quantified as the vertical displacement for each calculation step it performs and it must be small enough to minimize any inertial effect. Subsequently, the application of this load with the displacement produced is plotted and the bearing capacity of the soil is obtained when this load tends to be asymptotic at a constant value.

5.4. Proposed calculation methodology

The proposed calculation methodology for the bearing capacity of a shallow foundation under cyclic loading in FLAC2D is shown in Figure 10. This methodology consists of a 6 stages calculation process, the first one consists of the creation of the mesh, designation of soil and water properties, self-weight, and contour conditions of the model. At this stage, due to the fact that it worked with the assumption without self-weight, it is not a calculation stage or one that has an influence on the proposed formulation. The second stage corresponds to the determination of the permanent load ($PL$) on the footing and the effective load outside the
foundation \((q)\). In this calculation stage, it proceeds to obtain parameters such as the effective vertical stress and the initial shear stress produced by the permanent load and the effective load outside the foundation. These variables that will later be used to calculate the pore-water pressure generation formulation. The third stage is the assignment of the cyclic loading \((CL)\) on the footing and the calculation due to this load to later extract the variable that corresponds to the increase in shear stress produced by the cyclic loading. The fourth stage corresponds to the calculation of the pore-water pressure generation through the formulation proposed in equation (3) based on the permanent load \((PL)\), the effective load outside the foundation \((q)\) and the cyclic loading \((CL)\). The fifth stage is the one that corresponds to the calculation of the equilibrium state of stresses. In this stage the stresses of the soil are balanced with the pore-water pressure to generate the new state of stresses due to the reduction by the pore-water pressure. The sixth stage is the calculation of the bearing capacity of the soil.

![Diagram](image)

**Figure 10.** Proposed calculation methodology for the bearing capacity of a shallow foundation under cyclic loading.

Once the value of the bearing capacity was obtained, an analysis is made under two criteria as shown in Figure 11. The first criterion in Figure 11 a), verifies that the value of the bearing capacity is zero or close to zero. In the second criterion Figure 11 b), it must be verified that the Mohr-Coulomb failure criterion is met. If any of the two criteria is not met, it is necessary to return to stage three and change the value of the cyclic loading to a higher or less value.
as appropriate and continue until both conditions are met. This process is iterative until the maximum value of cyclic loading on the footing that can resist is found.

![Image]

**Figure 11.** Verification criteria for the proposed methodology.

For the analysis of bearing capacity under cyclic loading, it was determined to carry out various combinations for the different types of soils ($c$, $\varphi$), effective load outside the foundation ($q$), voids ratio ($e_0$) and permanent load ($PL$) as shown in the Table 3.

| $c$ [kPa] | $\varphi$ [º] | $q$ [% Phe] | $e_0$ | $PL$ [%Phe] |
|-----------|----------------|-------------|-------|-------------|
| 10,25     | 20,25          | 0; 10; 20; 40 | 0.74; 0.82; 0.90 | 1; 10; 25; 50; 75; 90 |

6. Analysis and discussion of results

The analysis of the results obtained starts from the generation of the proposed design charts that are shown in Figure 12. These charts correspond to cohesion ($c$) of 10 and 25 kPa, a friction angle ($\varphi$) of 20 and $25^\circ$, an effective load outside the foundation ($q$) of 0, 0.10, 0.20 and 0.40 of the static bearing capacity ($P_{he}$) and voids ratio ($e_0$). For a better understanding of they will denominate the types of soils numerically, the soil 1 corresponds to $c=10$ kPa and $\varphi=20^\circ$, soil 2 to $c=10$ kPa and $\varphi=25^\circ$, the soil 3 to $c=25$ kPa and $\varphi=20^\circ$, and finally, soil 4 to $c=25$ kPa and $\varphi=25^\circ$.

6.1. Full charts

In Figure 12 the charts are shown for 4 different types of soils, part a) corresponds to a soil with cohesion of 10kPa and friction angle of $20^\circ$. Part b), the soil has a cohesion of 10 kPa but with a friction angle of $25^\circ$. Part c) is a soil with a cohesion of 25 kPa and a friction angle of $20^\circ$. Finally, part d) is a soil with a cohesion of 25 kPa and a friction angle of $25^\circ$. 
Figure 12. Design charts proposed for different types of soils ($c, \phi$), effective load outside the foundation ($q$) and permanent load ($PL$).

6.2. Influence of the void ratio
The influence of the voids ratio on the proposed charts was analyzed for the first chart (Figure 12 a.), and for the effective load values outside the foundation of $q=0, 0.10, 0.20$ and $0.40*Phe$; and for three different voids ratio $e_1 = 0.74; e_2 = 0.82$ and $e_3 = 0.90$ as shown in Figure 13. The voids ratio $e_1$ and $e_3$ arise from the sum of the average value of the voids ratio measured in the experimental par $\pm$ their variation.

It shows in Figure 13 that the influence of voids ratio is low and therefore only the best fit for each group is shown. The maximum variation due to the void ratio was 3% in global terms for values from $e_0 = 0.74$ to $e_0 = 0.90$. From this analysis it is evidenced that the most critical void ratio is the highest and the least critical is the lowest; something that is consistent with the theory, since for a higher void ratio it is expected to be more susceptible to cyclic softening. According to this analysis, the charts shown in Figure 12 correspond only for the void ratio of $e_0 = 0.90$, this being the most critical and susceptible to cyclic softening.
6.3. Influence of an effective load outside the foundation

The influence of the effective load outside the foundation is shown in Figure 12, where the variation of the curves can be observed as a function of the load \((q)\) applied to the soil for the same void ratio. It is observed that for values greater than 50% of \(PL/Phe\) there is the same trend of the curves with very similar values, in this way, it can indicate that there is little influence of the effective load outside the foundation for values greater than 0.50 of \(PL/Phe\). However, for values less than 50% of \(PL/Phe\), there is a considerable variation between each value of \((q)\), this variation being up to 13.7% in global terms.

In all cases, the maximum variation for a value \(q=0^\circ Phe\) and a value \(q=0.40^\circ Phe\), corresponds to the value of 0.01 \(Phd/Phe\). Soil 1 has a maximum variation in global terms of 13.6%, soil 2 is 8.2%, for soil 3 the variation is 13.7% and for soil 4, the maximum variation is 11%. This would show that the maximum variation occurs in soil 3. However, if it analyzes the variation between the value \(q=0^\circ Phe\) and the next one, which will be \(q=0.10^\circ Phe\), the maximum variation occurs in soil 1 with 9.5% in global terms, for soil 2 it is 5%, for soil 3 the variation is 9.3% and for soil 4 it is 7.6% in global terms.

6.4. Influence of cohesion

The influence of cohesion can be seen in Figure 14, where it is shown on the one hand, Figure 14 a) for soils 1 and 3 that have a friction angle of 20° and Figure 14 b) for soils 2 and 4 that have a friction angle of 25°. These charts are for a void ratio of 0.90 and effective load outside the foundation of \(q=0, 0.10, 0.20\) and 0.40 of the static bearing capacity for each type of soil. The analysis in Figure 14 a) was carried out from the point of view of how cohesion from 10 to 25 kPa influences with the same friction angle of 20° for the different loads \((q)\). It can see that the influence for the same load \((q = 0.40 * Phe)\) is very small of the order of 1.2% in global terms for a value of 0.10 of \(PL/Phe\). In the case of Figure 14 b) it is observed that for a friction angle value \(\varphi = 25^\circ\), there is a greater influence of cohesion with a value of 2.6% in global terms for a value of 0.01 of PL/Phe.

In the general, it can be seen in Figure 14 a) that for values greater than 0.75 of \(PL/Phe\), the cohesion parameter tends to have no influence, and for Figure 14 b) this tendency occurs for values from 0.90 \(PL/Phe\).
6.5. Influence of internal friction angle

The influence of the friction angle was analyzed by separating the soil types by the cohesion value 10kPa and 25 kPa as shown in Figure 15. The Figure 15 a) corresponds to type 1 and 2 soils, it can be seen that the maximum variation is 14.3% in global terms for a value of 0.01 of $PL/Phe$. The variation decreases to a value of 9.73% for $q = 0.10 \cdot Phe$, to a value of 8.77% for $q = 0.20 \cdot Phe$ and to a value of 8.88% for $q = 0.40 \cdot Phe$. This indicates that the higher the friction angle, the lower the resistance to cyclic softening.

Moreover, Figure 15b corresponds to type 3 and 4 soils, where the maximum variation has a value of 11.5% in global terms for a value of 0.01 of $PL/Phe$. The variation decreases to 9.83% for $q = 0.10 \cdot Phe$, to a value of 9.08% for $q = 0.20 \cdot Phe$ and to a value of 8.89% for $q = 0.40 \cdot Phe$. This indicates that although the resistance capacity decreases when the value of the friction angle increases, the increase in cohesion influences that it is less than the previous case.

Figure 15. Influence of friction angle for soils with $c = 10kPa$ and $c = 25kPa$. 

Figure 14. Influence of cohesion for soils with $\varphi = 20$ and $\varphi = 25$. 

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Based on the analysis presented, it can mention the following conclusions. The internal friction angle ($\phi$) has an influence of almost 15% in global terms for a low permanent load ($PL$) in relation to the static bearing capacity ($Phe$), decreasing as the permanent load increases Figure 12 or Figure 15. The influence of the voids ratio ($e_0$) is very small, which is why the determination was made to work only with the most critical void ratio ($e_0 = 0.90$). The effective load outside the foundation ($q$) represents an influential parameter for footing design when the permanent load has a very low value in relation to the static bearing capacity as can be seen in Figure 12. Cohesion ($c$) has a greater influence when the value of the friction angle increases, this is reproduced in a maximum value of 2.6% in global terms (Figure 14b).

7. Conclusions
It was possible to determine a methodology to calculate the bearing capacity of softs soil under cyclic loading. The work starts with the analysis of experimental data to fit and propose an equation for the generation of pore-water pressure. This formulation was implemented in finite difference software (FLAC2D). In turn, a calculation process methodology is proposed that allowed us to generate design charts. The main aspects and conclusions are the following:

- It was determined that pore-water pressure generation under cyclic loading depends on variables such as initial shear stress, cyclic shear stress, effective vertical consolidation stress and voids ratio. It proposed a formulation for generation of pore-water pressure, and the best model fit was 85% for the experimental data.
- The formulation was implemented in the FLAC2D program by means of the FISH programming code to apply it to the particular case of a footing that can vary the properties of the soil ($c$, $\phi$), effective load outside the foundation ($q$), voids ratio ($e_0$) and permanent load ($PL$), to determine the value of the maximum cyclic loading that the analyzed soil can resist.
- A methodology of the calculation process is proposed in FLAC2D, which consists of 6 stages that allow considering the different stress states that the soil acquires in each phase of the analysis. In this way, the generation of pore-water pressure produced by cyclic loading in soils susceptible to cyclic softening is considered.
- The design charts within the data range in which the analysis was performed are valid, although a similar trend is expected for soils with geomechanical characteristics that are outside the studied range.
- The parameters that most influence the dynamic bearing capacity ($Phd$) are the internal friction angle and the effective load outside

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