MATHEMATICAL MODELING OF THE COEFFICIENTS USED IN THE CORRELATION BETWEEN THE STANDARD PENETRATION TEST AND THE SOILS LOAD CAPACITY

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Abstract: In this article, the different calculations referring to the soil bearing capacity and its friction angle are analyzed. Specifically, from the sand when the Standard Penetration Test or also called S.P.T. is taken as a starting point.

The modification of formula factors is proposed to obtain the friction angle of two different authors starting from a triaxial test. Several disturbed samples from the soil under study were taken both to carry out the triaxial test as well as the respective S.P.T tests from different points in the sector under study soil.

Based on the data obtained, the respective correlation is carried out using R software, from which representative differences were found between the classical formulas and the research proposal.

Effectively correlating the factor values used in the formulas commonly used in our calculations of both the admissible capacity of soils and the angle of friction are far from its actual capacity.

Giving a new acquisition formula with a factor for the friction angle, representative in the treated soil.

Keywords: Standard penetration test, triaxial study, correlational, friction angle.

INTRODUCTION

Various types of buildings which over the years are being built with greater and greater detail in terms of comfort and obviously in construction. All these buildings or structures are built on the earth's surface which, depending on their topology or classification, the respective tests and calculations are carried out.

Among the main problems in soil study and more clearly in the calculation of foundations and retaining walls in sandy soils, these have been made from foreign formulas and within them coefficients and constants that do not correspond in any way to any type of soil that exists in our country and especially our city. With this background, the calculation accuracy is affected and consequently there are economic losses, either due to under-dimensioning or over-dimensioning.
Factor data and exact soil coefficients are required from our place of study, for a greater calculation effectiveness and risk reduction in structures founded on this type of soil in particular.
For this, the correlations factors between the S.P.T. tests and the soil carrying capacity analyzed by the soil surface cut theory in the city of Riobamba. For which the topographic and planimeter survey of the neighborhoods under study is carried out, perforations are carried out by SPT test and collection of samples, as well as triaxial tests to define the soil carrying capacity by the cutting theory.

SOIL CAPACITY THEORY DEFINITION MODEL

Terzaghi (1943) was the first to present a complete theory for evaluating the ultimate bearing capacity of shallow foundations. According to this, a foundation is shallow if the depth, Df of the foundation is less than or equal to its width. However, later researchers suggest that foundations with Df equal to 3 or 4 times the width of the foundation can be defined as shallow foundations.

Terzaghi suggested that for a strip foundation (that is, when the ratio of width to length of the foundation tends to zero). The soil effect above the bottom of the foundation can also be assumed to be replaced by an effective equivalent surcharge q = γ*Df (where γ = specific weight of soil). The failure zone under the foundation can be separated into three parts:

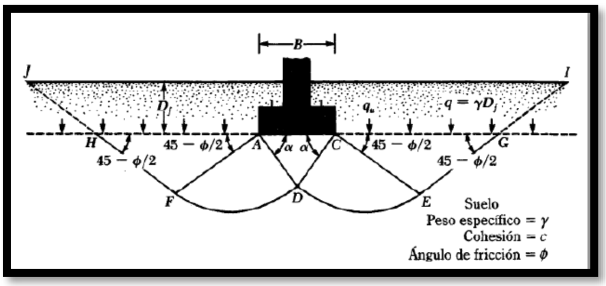

Figure 1. Failure due to soil bearing capacity under a continuous rigid foundation.
Source: Baja M. Das, "Foundation Engineering Principles"

The triangular zone ACD immediately below the foundation
The radial slice zones ADF and CDE, with the curves DE and DF as arcs of a logarithmic spiral
Two passive triangular Rankine zones AFH and CEG

It is assumed that the angles CAD and ACD are equal to the friction Angle in the ground, ø.
Note that, replacing the soil above the bottom of the foundation by an equivalent surcharge q, Soil shear strength along the GI and HJ failure surfaces was neglected.
Using equilibrium analysis, Terzaghi expressed the ultimate carrying capacity in the form.

$$q_u = c.N_C + q.N_q + \frac{1}{2}.\boldsymbol{\gamma}.B.N_\gamma \qquad \text{(Ec. 1.1)}$$

Where:
c = soil cohesion
γ = specific soil weight
q = γ * Df
Nc, Nq, Nγ, = dimensionless bearing capacity factors that are solely a function of the soil friction angle ø.
Carrying capacity factors, Nc Nq y Nγ are defined by the expressions:

$N_c = cot\text{ø} \left[\frac{e^{2\left(\frac{3\pi}{4}-\frac{\text{ø}}{2}\right)tan\text{ø}}}{2cos^2\left(\frac{\pi}{4}+\frac{\text{ø}}{2}\right)} - 1\right] = \cot\text{ø}$ (Ec. 1.2)

$N_q = \frac{e^{2\left(\frac{3\pi}{4}-\frac{\text{ø}}{2}\right)tan\text{ø}}}{2cos^2\left(45+\frac{\text{ø}}{2}\right)}$ (Ec. 1.3)

$N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{cos^2\text{ø}} - 1\right)\tan\text{ø}$ (Ec. 1.4)

Where:
K p y = passive thrust coefficient

The carrying capacity factors defined by these equations are given in Table 1.1 below.

To estimate the ultimate carrying capacity of square and circular foundations, the equation can be modified to:

$q_u = 1{,}3.c.N_C + q.N_q + 0{,}4.\boldsymbol{\gamma}.B.N_{\boldsymbol{\gamma}}$ (Square foundation) (Ec. 1.5)
y
$q_u = 1{,}3.c.N_C + q.N_q + 0{,}3.\boldsymbol{\gamma}.B.N_{\boldsymbol{\gamma}}$ (Circular foundation) (Eq. 1.6)

In the equation for square foundations, B is equal to the dimension of each side of the foundation; in the equation for circular foundations, B is equal to the diameter of the foundation. For foundations that show local shear failure in soils, Terzaghi suggested modifying the equations of qu.

$q_u = \frac{2}{3}.c.N_C + q.N_q + \frac{1}{2}.\boldsymbol{\gamma}.B.N_{\boldsymbol{\gamma}}$ (Strip foundation) (Eq. 1.7)

$q_u = 0{,}867.c.N_C + q.N_q + 0{,}4.\boldsymbol{\gamma}.B.N_{\boldsymbol{\gamma}}$ (Square foundation) (Eq. 1.8)

$q_u = 0{,}867.c.N_C + q.N_q + 0{,}3.\boldsymbol{\gamma}.B.N_{\boldsymbol{\gamma}}$ (Circular foundation) (Eq. 1.9)

N´c, N´q y N´γ are the modified carrying capacity factors. These are calculated using the equations for the carrying capacity factor (for Nc, Nq, and Nγ) replacing ø with:

$\text{ø}' = tan^{-1}\left(\frac{2}{3}tan\text{ø}\right)$ (Ec. 1.10)

The variation of N´c, N´q, N´γ with the angle ø of soil friction.

Terzaghi's carrying capacity equations were modified to account for the foundation shape effects (B/L), depth of embedment (Df), and load inclination. These are given in the General Equation of Carrying capacity. However, many engineers still use the Terzaghi carrying capacity equation which gives quite good results considering the uncertainty of soil conditions.

LOAD EXPERIMENTAL ANALYSIS

Since the population is huge, the neighborhood "EL DESPERTAR" has been taken as a representative sample in the study universe, which covers an area of 21.43 hectares.

Considering that the investigation will yield general data, we estimate to carry out the sampling by means of perforations in a 250-meter grid. According to the needs in the field, it will be carried out at a shorter distance; Sampling will be carried out between 0.50 and 2 meters high taking necessary samples for comparing results.

After resting the sampler at the bottom of the drilled well, it is driven in by blows from the hammer launched in free fall from a height of 75 ± 5 cm above the blow head.

To lift the hammer, a manila rope or similar must be used and, in case of being mechanically actuated, the rope must not be winded more than two times on the winch.

Care must be taken that the rigid steel bar that guides the drop of the hammer is entirely vertical, thus ensuring that the effect of the friction of the hammer on the said bar does not diminish the energy of the blows.

The sampler must first penetrate 15 cm, which is considered a necessary preparation and adjustment process for the test. Therefore, the number of necessary blows for this first driving section has only a relative orientation value.

The test then begins by counting the number of blows (N) necessary to make the sampler penetrate 30 cm (note 3), which determines the resistance to penetration of said soil.

Marking the guide bar at 70, 75, and 80 cm above the top of the fall's head is one approach for keeping an even drop. In this way, the operator will be able to lift the hammer until the 70 or 75 cm mark appear on its lower part, but never the 80 cm mark.

It is convenient to keep track of the number of blows required for each 15 cm penetration whilst driving.

If 50 blows are exceeded without having penetrated the entire measure indicated in numeral d, the number of blows made and the fraction or length of penetration of the sampler must be recorded.

After driving and before extracting the sampler to the surface, it must be rotated at least two revolutions, in order to cut the sample at the bottom. The sampler is then pulled to the surface, opened, and the sample length retrieved. The soil sample is removed, discarding the upper part that is considered unrepresentative.

The sample is described as established in the INEN 693 Standard, after which it must be conditioned and hermetically closed in an appropriate container for shipment to the laboratory.

When the sampler mentioned in numeral 2 is used, once the sample is obtained, the sampler is disassembled and the inner tube is removed, leveled, sealed and sent to the laboratory.

For the sample identification, the containers must be properly labeled.

a) Job designation.
b) Perforation number.
c) Sample number.
d) Extraction depth or elevation.
e) Number of blows (N) and all the additional data that is useful for its easy recognition and identification.

CALCULATIONS

Determination of corrected N

With PECK's expression, (1974), so that:

$$Nc = N \times \left(0{,}77 \times Log\left(\frac{20}{q}\right)\right) \qquad \text{(Ec. 1.11)}$$

Where:

Nc = Number of corrected blows
N = Number of blows on the field
Log = Base 10 logarithm
q = Overload on Kg/cm2
q > 0.25 Kg/cm^2.

Determining the internal friction angle.

Logarithmic regression of the correlations proposed by Osaki, Meyerhof and Schmertmann.

$\emptyset = (3{,}612 \times ln(Nc)) + 20{,}58$ (Ec 1.12)

Where:

Ø = Internal friction angle
Nc = Number of corrected blows
Ln = Natural logarithm

Determining the soil bearing capacity

With the MEYERHOF expression (1965), Modified by BOWLES (1977) we will determine the bearing capacity.

MEYERHOF (1965)

$Q_{net(adm)} \frac{kN}{m^2} = 19.16\ N_{cor}\ F_d \left(\frac{Se}{25.4}\right)$ (For B 1.22 m) (Ec 1.13)

BOWLES (1977)

$Q_{net(adm)} = 11.98\ N_{cor} \left(\frac{3.28B+1}{3.28B}\right) F_d \left(\frac{Se}{25.4}\right)$ (Para B > 1.22 m) (Ec 1.14)

Where:

$Q_{net(adm)}$= Allowable bearing capacity of the soil
N_{cor}= Number of corrected blows
F_d= 1 + 0.33 ($\boldsymbol{D_f}$/B) ≤ **1.33**
B = Footing width in meters
St = Tolerable settlement in mm

Triaxial Compression Test according to ASTM -D2850-82

(Unconsolidated undrained compression)

This test allows us to obtain soil parameters and also to know the stress-strain relationship through the determination of the shear stress.

It is a very complex test, but the information obtained is the most representative of the shear stress suffered by the soil mass when subjected to weight.

It is so complex that a special system is needed to contain a cylindrical soil sample placed inside a waterproof rubber or rubber membrane. This chamber is filled with water and subjected to pressure loads inside. Once said sample is in equilibrium and maintaining its lateral pressures, it is subjected to axial loads until it fails.

It is necessary to obtain the results of 3 tests and these must be carried out with 3 different types of lateral pressures to be able to graph them in Mohr's circles. These represent the failure stresses of each sample and by drawing a tangent line to these circles the cohesion (c) and the angle of internal friction of the soil (Ø) are obtained.
Such as the sand conditions in this case

Calculations
We obtain the initial height of the sample (Lo) by applying an arithmetic mean to the measurements made.

We calculate the diameter (D) of the specimen:
$D = (d_i + 2 * d_m + d_3) / 4$ (cm.) (Ec 1.15)
Where:
d_i = bottom diameter (cm.)
d_m= mid diameter (cm.)
d_3 = top diameter (cm.)

We calculate the area (A) and the volume (V) of the specimen.
$A = \pi * (D/2)$ (cm^2) and (Ec 1.16)
$V = A * L_o$ (cm^3) (Ec 1.17)

Calculate the unit strain (ε) for each load application using the expression:
$\varepsilon = \Delta L / L_o$ (Ec 1.18)
Where:
ΔL = height variation of the specimen.

Calculate the corrected area (Ac) for each load application, using the expression:

$A_c = A / (1 - \varepsilon)$ (cm^2) (Ec 1.19)

Calculate the deviator stress (σc) for each area unit using the expression:
$\sigma_c = P / A_c$ (kg / cm^2) (Ec 1.20)
Where:
P = applied load on (kg)

Plot the strain (ε * 10-2) against the deviator stress for each confining pressure.
Draw Mohr's circles for all the tests on the same graph and draw a tangent or envelope to them. Obtain the parameters Ø and c from the ground, measuring the tangent slope that corresponds to the internal friction angle (Ø) and the intercept with the ordinate, which will correspond to cohesion.

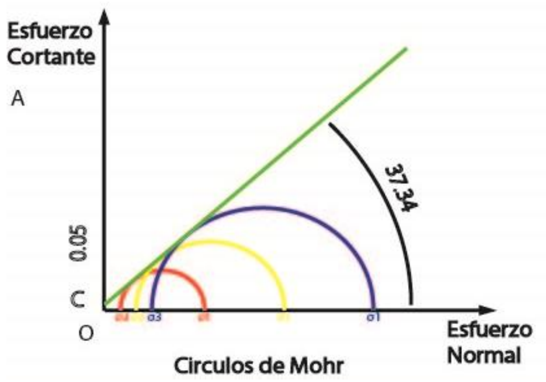

Figure 2. Mohr's Circles Graph

RESULTS OBTAINED FROM THE TESTS

After the preliminary tests, the results are shown in the following tables regarding the Triaxial test as well as the S.P.T. Test.
These tables are labeled with their respective data and field information as well as the order in which they were taken and tested.
The SPT test follows the INEN 689 standard.

This method is widely used in soil exploration (also very economical), which allows determining the characteristics, thickness and stratification of materials found in the soil. It also allows knowing the resistance to penetration depending on the number of blows (N) of the different strata that make up the subsoil at different depths.

This type of test is generally used for sand. On the other hand, for clayey soils it presents many interpretation difficulties. Care must be taken in soils containing gravels with the influence that generates the size of soil particles.

Table 20 shows the following data.
To obtain the corrected N, PECK's expression (1974) is used.

Table 1*. SPT test results*

TABLA DE RESULTADOS DEL BARRIO "NUEVO AMANECER".

Provincia:	Chimborazo			LABORATORIO DE MECANICA DE SUELOS DE LA UNACI			
Ciudad:	Riobamba			FECHA:		ene-15	
Barrio:	Nuevo Amanecer			ELABORADO	Juan Romero		
N- de perforacion:	u						

Tabulación de Datos Ensayo S.P.T.

						Regresión logarítmica Ec (2.1) ø= (A)*ln(Nc)+20,58	Hatanaka y Uchida ø=((B*Nc)^0,5)+20
						A=3,612	B=20
Toma #	**Cota(cm)**	**No Golpes**	**N**	**Ncorr**	**Qneta adm**	**ø CLASICO**	**ø CLASICO**
1	15	4					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
2	15	4					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
3	15	3					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
4	15	3					
	30	3					
	45	3	6	8.79227574	186.0506349	28.43203137	33.2606755
	cota(cm)	No Golpes					
5	15	3					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
6	15	4					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
7	15	4					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
8	15	3					
	30	3					
	45	4	7	10.257655	217.0590741	28.98882362	34.32316657
	cota(cm)	No Golpes					
9	15	3					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914
	cota(cm)	No Golpes					
10	15	4					
	30	4					
	45	4	8	11.7230343	248.0675132	29.47113901	35.31210914

Additionally, to determine the angle value and using the Logarithmic Regression equation of the correlations proposed by Osaki, Meyerhof, and Schmertmann.

Soil Average Internal Friction Angle:

ø=29.31

Maximum internal friction angle of de ground:
ø=29.47

Minimum internal friction angle of the ground:
ø=28.98

Table 2. *Triaxial Test Results Summary*

Tabulación de Datos Ensayo Traxial

TOMA	Presión(psi)	Mpa σ3	Respuesta(KN)	σ1	Mpa σ1	ø
		0.006894757			0.001	
	25	0.172368932	1.8	935.3458547	0.935345855	
						37.26541781
1	50	0.344737865	3.15	1636.855246	1.636855246	
						40.06451116
	75	0.517106797	4.68	2431.899222	2.431899222	
	25	0.172368932	1.79	930.1494889	0.930149489	
						37.26541781
2	50	0.344737865	3.14	1631.65888	1.63165888	
						37.43348961
	75	0.517106797	4.5	2338.364637	2.338364637	
	25	0.172368932	1.82	945.7385864	0.945738586	
						36.22180762
3	50	0.344737865	3.11	1616.069782	1.616069782	
						40.09314631
	75	0.517106797	4.642	2412.153032	2.412153032	
	25	0.172368932	1.8	935.3458547	0.935345855	
						36.40007018
4	50	0.344737865	3.1	1610.873416	1.610873416	
						37.59995769
	75	0.517106797	4.47	2322.775539	2.322775539	
	25	0.172368932	1.81	940.5422206	0.940542221	
						36.57656344
5	50	0.344737865	3.12	1621.266148	1.621266148	
						36.92435825
	75	0.517106797	4.45	2312.382807	2.312382807	
	25	0.172368932	1.79	930.1494889	0.930149489	
						36.57656344
6	50	0.344737865	3.1	1610.873416	1.610873416	
						38.40911357
	75	0.517106797	4.52	2348.757369	2.348757369	
	40	0.275790292	3.1	1610.873416	1.610873416	
						33.26078518
7	60	0.413685438	4.01	2083.74271	2.08374271	
						39.06808694
	80	0.551580583	5.18	2691.717515	2.691717515	
	40	0.275790292	3.1	1610.873416	1.610873416	
						36.40007018
8	60	0.413685438	4.14	2151.295466	2.151295466	
						35.72221588
	80	0.551580583	5.15	2676.128418	2.676128418	
	40	0.275790292	3.1	1610.873416	1.610873416	
						37.88749408
9	60	0.413685438	4.21	2187.670027	2.187670027	
						34.53264382
	80	0.551580583	5.17	2686.521149	2.686521149	
	40	0.275790292	3.05	1584.891587	1.584891587	
						38.87690792
10	60	0.413685438	4.21	2187.670027	2.187670027	
						34.53264382
	80	0.551580583	5.17	2686.521149	2.686521149	

(Unconsolidated undrained compression)
This test allows us to obtain soil parameters and also to know the stress-strain relationship through the determination of the shear stress.
It is a very complex test, but the information obtained is the most representative of the shear stress suffered by soil mass when subjected to weight.

In addition, to determine the angle and using the graphs of Mohr's circles mentioned in chapter II, it indicates that drawing a tangent on the graphs and this line angle will give us the internal friction angle.
Soil Average Internal Friction Angle:
ø=37.34

Soil Minimum Internal Friction Angle:
ø=34.53

Soil Maximum Internal Friction Angle:
ø=39.068

The Allowable Conveyed Effort is obtained empirically by observing that the maximum pressure does not cause structural damage in different soil conditions. This does not mean that settlements will occur. This allowable pressure is valid for foundation sizes and types of structures for which practical rules have been established. The values are conservative, and it is difficult to find out what data they have been based on. The registered flaws are attributed to bad soil classification, instead of a bad empirical rule. In many cases, it is verified with load tests, which may not be significant.

If the shear theory is considered to determine allowable soil stresses, the results obtained between the SPT test and the Triaxial test will be different because the angle of internal friction has a considerable variation in the two tests.

INTERNAL FRICTION ANGLE OF THE SOIL

The internal friction angle mentioned in this study is obtained by two methods. The first was proposed for the relationship N blows of the test (SPT) / friction angle through the PECK equation (1974). In this way, it is on the safe side and adapts better to sandy soils. In the same way, by obtaining the flaws envelopes provided by the triaxial test, a tangent line is drawn from the three soil responses and as a result of it, we get the internal friction angle.
Knowing the average internal friction angle and the average corrected N from the SPT test, these formulas are related to find the corrected coefficients.

ANALYSIS AND RESULTS

The modifications made to the correlation factors serve to have more accurate values in the calculation of the bearing capacity of the soil. Thus benefiting students, calculation engineers, teachers and other users. Also avoiding over-dimensioning in the case of foundation structures and reducing or lowering costs in this type of structure for the city. This type of analysis, studies and modifications are of great importance since in different sectors of the city there are no soils with identical characteristics to those of our study. Prior to a calculation with established formulas, their respective studies must be carried out in order to determine the true physical capacity of the soil, especially in sands, which is the case of our study.

Soils change much from one place to another. The chemical composition and physical structure of the soil at a given location are determined by the type of geotechnical material from which it originates. It may be due to the vegetation cover, the length of time weathering has taken place, the topography and artificial changes resulting from human activities.

Soil variations in nature are gradual, except those derived from natural disasters.

Basic knowledge of soil texture is important to civil engineers who construct buildings, roads, and other structures above and below the earth's surface.

The soil-structure interaction at the soil-foundation interface is fundamental in estimating settlements. This aspect is seldom taken into account; however it is well known that the stress distribution does not have a uniform distribution under a foundation.

SOIL CHARACTERISTICS

The soil deforms under load through a continuous or isolated foundation. Designing an adequate foundation consists of limiting the possible deformations to values that do not produce detrimental effects on the dwelling, avoiding total settlement and differential settlement.
Rock disintegration produces fragments that are carried by river torrents and give rise to boulders, gravel, and others. This is how other granular materials form the subsoil. They become compacted or joined together over time, sometimes by simple compression or being agglomerated or bound by natural cements of various kinds, forming a product of different consistency.

The proposal consists of making modifications to the formulas established for calculating the bearing capacity and internal friction angle of the soil in such a way that it adjusts to the specific properties of any soil by means of a correlation between the SPT tests and the triaxial test.

Taking as the main reference is the internal friction angle of the soil obtained from the triaxial test.
Corrections are made so that the formula used in the SPT test adjusts to the properties of the soil type. This means that through mathematical calculations we restate the factors of the internal friction angle formula for the soil properties under study.

Explained another way it would be.

Taking the exposed formula:

$$\emptyset = (3{,}612 \times ln(Nc)) + 20{,}58$$

The respective corrections are made to this formula, and we obtain the following equation.

$$\emptyset = (6{,}92 \times ln(Nc)) + 20{,}58$$

Thus, the following table presents factors already modified to apply them to this type of soil.
With the same principle used in the modifications of the previous expressions, modifications are made to the formulas exposed in the following lines which correspond to more recent studies.

Performing a series of drained triaxial tests on samples
Undisturbed sand, from which its friction angle (∅) is determined.
The friction angle obtained was compared with the empirical equations proposed by many researchers using the SPT test using the following expression.

$$\emptyset = \sqrt{20 \times (Nc)} + 20$$

In the same way that we made the corrections in our previous formula and as an additional study for our thesis, we obtain the following equation.

$$\emptyset = \sqrt{26{,}69 \times (Nc)} + 20$$

Finally, and as the most important result of this research work, the linearization of the data was carried out to obtain the expression proposed in this study.

Which resulted in the following:

$$\emptyset = 1{,}1587\text{Nc} + 24{,}175$$

The verification of the proposed expressions is presented in the following table.

***Table 3**. Internal friction angle results, using the Classical and Proposed correlations.*

TABLA DE RESULTADOS DEL BARRIO "NUEVO AMANECER".										
Provincia:	Chimborazo				LABORATORIO DE MECANICA DE SUELOS DE LA UNACH					
Ciudad:	Riobamba				FECHA:	ene-15				
Barrio:	Nuevo Amanecer				ELABORADO	Juan Romero				
N- de perforacion:	u									
Tabulación de Datos Ensayo S.P.T.										
						Regresión logarítmica Ec (2.1) ø= (A)*ln(Nc)+20,58		Hatanaka y Uchida ø=((B*Nc)^0,5)+20		Propuesta ø = 1,1587Nc + 24,175
						A=3,612	A=6,92	B=20	B=26,685	
Toma #	**Cota(cm)**	**No Golpes**	**N**	**Ncorr**	**Qneta adm**	**ø CLASICO**	**ø NUEVO**	**ø CLASICO**	**ø NUEVO**	**ø NUEVO propuesto**
	15	4								
1	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	4								
2	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	3								
3	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	3								
4	30	3								
	45	3	6	8.79227574	186.0506349	28.43203137	35.6232052	33.2606755	35.3173718	34.3626099
	cota(cm)	No Golpes								
	15	3								
5	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	4								
6	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	4								
7	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	3								
8	30	3								
	45	4	7	10.257655	217.0590741	28.98882362	36.6899279	34.32316657	36.5446524	36.06054488
	cota(cm)	No Golpes								
	15	3								
9	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
	cota(cm)	No Golpes								
	15	4								
10	30	4								
	45	4	8	11.7230343	248.0675132	29.47113901	37.6139651	35.31210914	37.6869774	37.75847987
PROMEDIOS				11.2834205	238.7649815	29.31899671	37.3224854	35.00807152	37.3357844	37.24909937

CONCLUSIONS

Indeed, the factor values used in the formulas commonly used for our calculations, both the soil admissible capacity and the friction angle are far from their actual capacity.
The cohesion is close to zero in all the tests carried out, assuming that the treated soil is sand.
Logically, the soil characteristics vary from one sector to another, and even more so if the difference is between continents and regions.
In this case, formulas adjusted to the soil of other countries were used, under sizing the bearing capacity of the soil and therefore oversizing the foundations or being too conservative in the values.

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