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BEARING CAPACITY BASED ON SPT-COMPUTER INTERPOLATION

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ABSTRACT:- Any structural design must be accompanied with sound analysis referring to the foundation design. The columns carrying the total load of the building may be in very high stress. The actual stress that exists in columns may reach in actual cases to half f'c or more. On the other hand, the maximum carrying stress of soil is very much small compared with that for reinforced concrete, the situation that necessitate the enlargement of column end to have "a footing". While the compressive strength of concrete is easy to measure, the bearing stress of soil is not. Methods of evaluating the soil bearing capacity are numerous and consist of field and laboratory. The SPT is one of these field methods. Scientists tried to relate the SPT-N value with the soil strength properties resulting in large number of tables, charts, and graphs. This research considers the most famous methods to evaluate the bearing capacity from the SPT. A BASIC computer program is written to aid in using these formulas. In going to this step all tables, curves, and graphs must be converted to numerical equations. This is done by using the usual FD technique of interpolation. The authors feel that this program must be used with caution since it is not a replacement of sound hand calculations associated with engineering judgment and experience. This is because the very SPT is used only as a guide and never as a replacement of laboratory testing program except for sands since it is very difficult to get undisturbed samples.

Keywords: Bearing capacity, Interpolation, structural.

LIST OF SYMBOLS

γ_b	submerged unit weight of soil.
γ_{moist}	moist unit weight of soil.
γ_s	saturated unit weight of soil.
φ	effective angle of internal friction for soil particles.
ρ	soil settlement.
σ_{vo}	initial effective overburden pressure of soil.
B	width of footing.
c_u	undrained cohesion of clay.
D_f	depth of footing.
d_w	depth of water table = z_w.
$K_{p\gamma}$	Terzaghi bearing capacity coefficient used in the formula of K_γ.
N, N'	corrected and uncorrected value of SPT.
N_c, N_q, K_γ	bearing capacity factors based on φ.

PI plasticity index of soil = LL-PL.
q_{all}, q_{ult} allowable and ultimate bearing stress of soil.
R_w, $R_{w'}$ SPT corrections based of A.R.E.A. for the presence of water table.

INTRODUCTION

The bearing capacity is a criterion for structural stability. Any structure, unless it floats, must eventually be founded on soil. The failure criterion for foundation soil is known as the ultimate bearing capacity or simply the bearing capacity of soil and is considered as one of the corner stones of soil mechanics. For such a purpose, scientists from about many decades ago tried to establish sound bearing capacity equations, which take into account the most variables encountered. Nowadays, the available bearing capacity equations are "how we say" numerous. Some of them have succeeded to float on surface while others have not.

By bearing capacity equations, here, authors mean, as well, all techniques used in field and laboratory to "estimate" the ultimate bearing stress of soil. Most of the field data available are presented as tables with boundary limits or nomograghs in terms of well standard tests such as the SPT (standard penetration test) and the CPT (cone penetration test). The bearing capacity equations based of theoretical approaches and laboratory tests are, of most, consist of equations or nomograghs. The huge data, as a background, available for the SPT have kept the light focused on such field test and instead of canceling it from soil investigation record; it is still floating of surface till now.

The research has taken into account the most famous and well-proven tables and graphs for field and theory bearing capacity estimations. Then a BASIC program has been established to facilitate the use of these bearing capacity equations. For programming purposes, tables and nomographs have been transformed into equations using the FD with most methods found suitable for each table and graph. In carrying out this step, personal experiences have been used based on authors' background. The aspects of the program are presented with brief explanation for the procedures used and the FD interpolation equations reached. Some of the source tables or data are presented in Engineering System. All data are transformed into N-kg-metric system.

ONE STEP FURTHER

To schedule programming the following scheme is adopted, see figure (1). The first program page consists of many ways of estimating the bearing capacity from the SPT. These methods are numbered and the user has to choose the method he wishes. The program will require the parameters needed in the particular method if order to compute the bearing capacity. Here, and in order to illustrate the program, each method is listed individually with brief discussion about it. Some methods consist of direct application of a series of equations leading directly to bearing capacity of soil. On the other hand, other methods consist of tables and graphs. Moreover, as mentioned before, using FD interpolation the latter are transformed into equations for use in programming. It must be mentioned here that in FD theory the closer the value to the pivotal point the less error we get in interpolation, also the higher the degree of FD polynomial the less error we get as well. We say in the outset that the pivotal points, degree of FD polynomial, the FD method, are chosen in accordance to authors experience, to the nature of point, soil type, numerical distance between points and so on. Any change in one of those parameters will, in sense, change the FD polynomial. Nevertheless, authors believe that most of these changes may be of minor effect on the value of bearing capacity.

FIRST PAGE CHOICES

The following choices will appear on running the program. A brief discussion will follow with the number of case is referred:

1- Finite differences interpolation of submerged and moist unit weight, and angle of friction- after Terzaghi and Peck- for sandy soils.
2- Finite differences interpolation of unconfined compressive strength of clayey soils- after Terzaghi and Peck.
3- Allowable bearing stress based on the ultimate bearing capacity of sandy soils- after Teng.
4- Allowable bearing stress based on one inch (25 mm) of settlement of sands- after Terzaghi and Peck.
5- Allowable bearing stress for clayey soils- after Terzaghi and Peck.
6- Bearing capacity factors N_q and N_γ obtained directly from the SPT-N value- with the allowance of local shear failure in foundation soil- after Peck, Hansen, and Thornburn- for sandy soils
7- Meryerhof equations for one inch (25 mm) or any settlement of sands.
8- Finite differences interpolation of angle of friction, and moist unit weight of sands- after Bowles.
9- Theoretical Hansen bearing capacity equations- "for comparison", with Kenny equation for very plastic soils.
10- It is usually costumed to refer to bearing capacity method by its scent's name or names. The number between brackets show the reference from which bearing capacity method is taken.

1-FD INTERPOLATION OF γ_b, γ_{moist}, AND φFOR STANDY SOILS

This method is based originally on the empirical tables presented by Terzaghi and Peck (1948), received many modifications later on, relating the SPT-N value with the relative density, the angle of friction, and the unit weight of sands (submerged and moist). The N values are corrected in accordance to the effective overburden pressure by a graph after Gibbs and Holtz (1957). No mention to N correction for the presence of water table in bore-hole. On the other hand, Terzaghi and Peck suggested increasing the angle of friction by 5 degrees for soils containing 5% of fine sands of silts. The foregoing suggestion is not incorporated in program for factor of safety.

The table for Terzaghi and Peck is transformed into numerical equations using the FD interpolation. The relationships between N and γ_b, N and γ_{moist} are obtained by using Newton divided FD, pivot N is selected as 10 degrees, while the equation relating N and φ is obtained by direct linear fitting. The equations obtained are:

$$\gamma_b = \frac{N^2}{848} + 9.58 \qquad (kN/m^3) \qquad -----(1)$$

$$\gamma_{moist} = \frac{N^2}{390} - \frac{N}{11.5} + 19.15 \qquad (kN/m^3) \qquad -----(2)$$

$$\phi = 0.28\,N + 27^\circ \qquad -----(3)$$

The correction for depth is $N = \dfrac{50\,N'}{\dfrac{\gamma\ z_w}{6.895} + 10}$,

where:

z_w : depth of water table from the natural ground level.
N' : uncorrected N-value.
γ : moist unit weight or submerged unit, depending on the level of water table at the time of the SPT-test.

Two conditions restrict the depth factor equation, namely, 1) $\dfrac{\gamma\ z_w}{6.895}$ must not exceed 40, and 2) if N>2N' then the corrected N must be divided by a safety factor of 2. In practice, the authors believe that this depth correction should be treated with caution since high values can be obtained.

The FD fitting of γ_b and γ_{moist} do not match for N=10 and smaller with the original table for Terzaghi and Peck by an error of 25%. This is not a serious problem since the angle of friction for pure sand does exceed 26.5. This is called the "particle-to-particle friction angle or φ_μ.

And in order to find the bearing capacity of the sandy soil the unit weights and angle of friction are used in Hansen equations to obtain the plain strain case of loading. It is worth to mention that no shape or other factors are used in the Hansen equations since the Terzaghi-Peck tables are considered crude.

It should be mentioned here that in case of the presence of water table in the D_F range, the soil water must be drained, by pumping for instance, for the purpose of concrete casting of foundations. This ground water should never be used as a substitute for mixing water according to ACI 318–08 (3.4.1 and 3.4.3). On other hand, the values of q_{all} or q_{ult} should be compared with actual soil pressure under footing with ample safety factor as in ACI 318–08 (15.10.3).

2-FD INTERPOLATION OF UNCONFINED COMPRESSIVE STRENGTH OF CLAYS c_u

Terzaghi and Peck presented a table similar to case(1) before but relating the SPT-number with the confined compressive strength and with the saturated unit weight for clays. Linear fitting is used between the undrained strength and the SPT-N while the correlation between N and γ_{moist} is ignored by program because:

1- It has minor effect on bearing capacity and,
2- The relationships between the undrained strength and SPT-N is very unreliable as stated by Terzaghi and Peck.

Thus the bearing capacity is based on the term (c_u N_c) with c_u=5.14. This step is towards the safety factor and is positive. No depth or shape factors are used. In sense:

c_u=5.985*N in kN/m^2, linear fitting

And

$$q_{ult}=c_u\,N_c = 5.14 * 5.985 * N \quad \text{in kN/m}^2 \qquad --(4)$$

No correction for N is used and q_{ult} is the ultimate bearing capacity of soils.

3- q_{all} BASED ON q_{ult} FOR SANDS

Here q_{all} is the allowable bearing stress of soils. Teng (1962) presented two empirical equations relating the SPT-N number with the bearing pressures of granular soils. Gibbs and Holtz correction for effective overburden stress, and A.R.E.A. correction for the water table level are used in these equations. The equations provides q_{ult} of soils and Teng suggests a factor of safety ***not less than 3***. The following simple steps illustrate the procedure of calculation q_{ult} with the aid of figure (2).

ALGORITHM

Enter depth of W_T from NGL

If $d_w \le D_f$ -----then-----R_w'=0.5, $R_w = 1 - \dfrac{D_f - d_w}{2D_f}$

If $d_w > D_f$ and $d_w<(D_f+B)$------then------ $R_w' = \dfrac{1}{2} + \dfrac{d_w - D_f}{2B}$, R_w=1.0

If $d_w \ge (D_f+B)$------then-------$R_w' = R_w = 1.0$

Correct SPT-N for effective overburden stress by using Gibbs and Holtz (1957) graph

Then for plain strain (PS) loading;

$$q_{ult} = 0.15709\ [2N^2\ B\ R_w+6\ (100+N^2)\ D_f\ R_w'] \quad \text{for square footing,} \quad --(5)$$

$$q_{ult} = 0.150[3N^2\ B\ R_w+5\ (100+N^2)\ D_f\ R_w'] \quad \text{for PS loading} \quad --(6)$$

Again, a safety factor of more than 3 is recommended to get q_{all}.

4- q_{all} BASED ON INCH SETTLEMENT IN SANDS

Terzaghi and Peck presented two equations for allowance bearing pressure based on settlement of 25 mm in sands. Same corrections used in (3) before are used here as well. The equations are,

$$q_{all} = 34.47\ (N-3)[\frac{(B+0.3)}{2B}]^2\ R_w'(1+D_f/B) \quad \text{in case of } (1+D_f/B) < 2, \text{ and} \quad --(7)$$

$$q_{all} = 6.89(N-3)\ [\frac{(B+0.3)}{2B}]^2 \quad \text{in case of } (1+D_f/B) \ge 2 \quad \_\ \_(8)$$

5- q_{all} FOR CLYEY SOILS

Terzaghi and Peck presented a table between the SPT-N value versus the allowable bearing pressures of square and plain strain loading for footing resting on clays. A safety factor of three in incorporated in the table with *large* settlement expected- as stated by Terzaghi. FD-Newton divided differences of interpolations are used, setting the pivotal N=11. At low N values, polynomial errors of about 13% are encountered between table data and polynomial. The FD equations are,

q_{all}=16.9N-0.048N^2-7.76 in(kN/m^2) for square footing, and −−(9)

q_{all}=13.55N-12.88-0.053N^2 in(kN/m^2) for PS loading. −−(10)

6-N_q AND N_γ OBTAINED DIRECTLY FROM THE SPT IN SANDY SOILS

Thornburn, Hansen, and Peck presented a nomograph relating the SPT-N value with φ, N_q, and N_γ, and allowing for local shear failure. Now using the FD-Everett method and setting N_o=20 for sands. The nomograph is transformed into two polynomials and then the Hansen bearing capacity equations are used to find q_{ult}. The equations are,

$$N_q=N^3-\frac{N^2}{200}+0.8834 \qquad ------(11)$$

$$N_\gamma=N^3-0.14N^2+3.2837N-15 \qquad ------(12)$$

The usual Gibbs and Holtz depth-overburden correction is used on the SPT-N value.

7- MEYERHOF EQUATION FOR SETTLEMENT OF ONE INCH ON SANDY SOILS

These simple equations presented by Meyerhof give the allowable bearing stress on sands based on 25mm of settlement or any other settlement.

$$q_{all}=0.47N\rho \quad \text{in case of } B\leq 1.2\text{m} \qquad ------(13)$$

$$q_{all}=0.4N\rho(B+\frac{0.3}{B})^2 \qquad ------(14)$$

Where:

q_{all} in kN/m^2

ρ is the settlement in mm.

The N-value is corrected for overburden stresses. Based on authors experience the settlement is restricted to 50mm as a maximum limit since beyond this limit the building may suffer large distresses.

8-FD INTERPOLATION OF φ AND γ_{moist} FOR SANDY SOILS

Bowles (1982) presented a table similar to that for Terzaghi and Peck relating the N-value with φ and γ_{moist} for sands. FD-interpolation for the table with N_o=12 and using the FD-Newton divided differences, results in the following two equations,

$$\phi=26.2+0.626N-\frac{N^2}{143} \qquad -----(15)$$

$$\gamma_{moist}=16.074+0.147N-\frac{N^2}{500} \quad in\ \ kN/m^2 \qquad -----(16)$$

Bowles incorporated two types of SPT corrections:

1- According to depth from Bazaraa, and as follows

In case of $\sigma_{vo}\leq$ 75kN/m^2 ------- N=4N' / (1+0.04 σ_{vo})

In case of $\sigma_{vo} \geq 75 kN/m^2$ ------- $N=4N' / (3.25+0.01\ \sigma_{vo})$

Where,

σ_{vo} is the effective overburden stress at the level of the SPT test.

N, N' are the corrected and uncorrected SPT-value, respectively.

2- According to the presence of water table in the bore-hole in addition the existence of fine sands or silts (producing a negative pore water pressure when the SPT arm is pulled up-resulting in a false increase in the SPT-N value). In such a case;

$$N = 15 + \frac{N' - 15}{2} \qquad \text{for} \quad N' \geq 15 \qquad --(17)$$

Bowles did not include in his tables a relation between SPT-N and the submerged unit weight. Thus, unlike the Terzaghi and Peck, the program will require the input of the effective vertical stress at the level of the SPT-test, and γ_{moist} is listed in program as a comparison with the in-situ one.

BRIEFS COMMENTS ON TERZAGHI AND HANSEN BEARING CAPACITY EQUATIONS

The bearing capacity equations, in general, have similar form, that is, cNc+qNq+0.5γBNγ for plain strain loading. The worldwide equations used by soil engineers are that which belong to Terzaghi, Meyerhof, and Hansen. These equations have proven to be the best among others and have great history and practice ever. The general plain strain equation cNc+qNq+0.5γBNγ is modified by each scientist by adding factors that have some effect of the bearing capacity of soil, such as the CD or UU for c_u and φ, shape and depth of footing, presence of eccentricity, ground inclination, and so on.

In sense, the Karl Terzaghi equations are the most famous and have long history of successful use, but for program applications the $K_{p\gamma}$ factor of N_γ was presented by Table(1). Using the FD interpolation to simulate $K_{p\gamma}$ by one and only one polynomial is not an accurate task, it requires either:

1-The use of several polynomials, that is, to subdivide the large range of φ (from zero to 45) into subintervals and each one is treated with, say, Everett formula. The total combination of these equations will scope to full range the "mathematical equation of $K_{p\gamma}$".

2-Or the use of one high degree polynomial, say nine, if all nodal point are to be considered.

In Table (1), two polynomials are shown, their equations are,

$$K_{p\gamma} = \frac{\phi^4}{1500} - \frac{\phi^3}{15.31} + 2.523\phi^2 - 42.667\phi + 285 \quad (\text{a 4th degree polynomial}) \text{-------------}(18)$$

Moreover,

$$K_{p\gamma} = \frac{43\phi^5}{375000} - \frac{\phi^4}{604.83} + 0.9523\phi^3 - 27.1467\phi^2 + 393.32\phi - 2123 \ (\text{a 5th degree polynomial}) \text{------}(19)$$

because the Newton Forward FD is used, with initial φ_o is 20, the points before φ_o do not match (ever) with the Terzaghi $K_{p\gamma}$ coefficient. A more practical choice to the Terzaghi equations for programming is the Meyerhof or the Hansen equations which, as well, have a very successful record in foundation engineering with the advantages of the presence of many factors that take into account many situations with affect the bearing capacity. The Hansen bearing capacity equations are programmed as the last step in the first page, so that one in concern can calculate the bearing capacity based on theory as for comparison with any SPT past result. The Hansen equations are used without any correction factors (for plain strain condition).

For very plastic soils, the determination of effective φ for drained condition analysis is very uncertain due to the long period of triaxial tests needed. Kenney (1959), [5] provided an empirical chart relating the plasticity index PI of clay with the sine of drained angle of friction. It has been found that the linear logarithm equation will fit well. The equation is,

$$\phi = \sin^{-1}[0.806 - 0.229\log(PI)] \qquad in\ raidians, and, \qquad -----(20)$$

$$\phi = \sin^{-1}[0.806 - 0.229\log(PI)](\pi/180) \qquad in\deg rees \qquad -----(21)$$

Which can be used directly to find the bearing capacity for plastic soils in drained conditions. In determining the bearing capacity using Hansen equations the following approach is followed;

1- If the water table level is within the depth of footing, $z_w \leq D_f$ then the soil is assumed to be fully saturated.
2- If the water table level is $D_f \leq z_w \leq (D_f+B)$ the water table level is assumed to be at the footing level, $z_w=D_f$.
3- If the water table level is below (D_f+B) or $z_w \geq (D_f+B)$ then the presence of pore-water-pressure in soil is ignored.

These assumptions are considered in the program, and regarded on the safe side of design.

CONCLUSIONS

1- The bearing capacity of soils is a very difficult and complicated problem because of the so many variables involved in the soil strength parameters and in the loading conditions. As a sequence, the concept and derivation of the ultimate bearing stress that a soil can withstand, differ from one scientist to another leading to so many equations, nomograghs, tables, and so on. Because of that having one single program to calculate the bearing capacity is an impossible task at least in the recent times. The program in this research is considered rather simple but is collective for many theories and self-experience.

2- The program is based fundamentally on the FD approximation to interpolate polynomials. The FD method, the pivot points and the degree of polynomial are considered as self-experience.
3- The program is mainly useful for office routine works, since in many situations *risk analysis* is considered especially for the level of the water table.
4- As stated in earlier paragraphs, this program is not a substitute for actual design and laboratory works, since the SPT is used only as a guide. Thus, the SPT is not a substitute for the actual site investigation work as well.
5- It is rather difficult to differentiate between the methods as which one has more accuracy since each method has it own assumptions.

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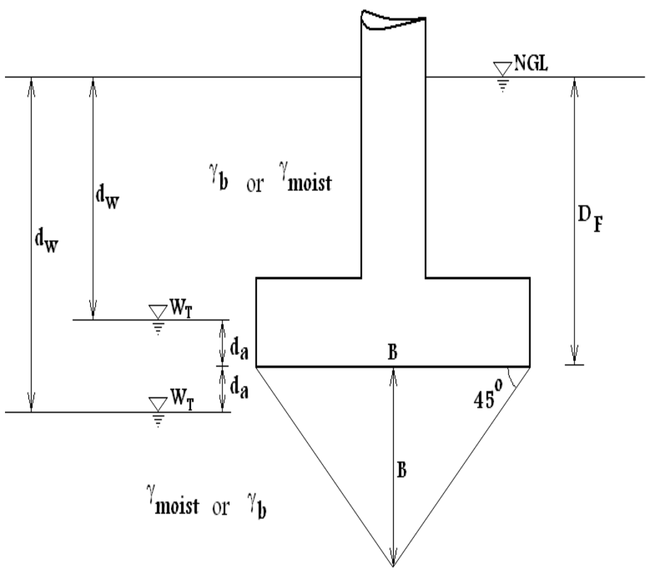

Fig.(2): A.R.E.A. (American Railway Engineering Association, Chicago, Illinois) .

Table (1): Comparison between actual $K_{p\gamma}$ values and the FD-approximation.

φ	**Values of $K_{p\gamma}$**		
	Terzaghi	**Forth degree polynomial**	**Fifth degree polynomial**
0	10.8	285	-2123
5	12.2	126.992	-776.005
10	14.7	51.98	-206.039
15	18.6	25.976	-17.132
$20=\varphi_o$	25	24.992	24.682
25	35	35.042	34.365
30	52	52.137	50.877
35	82	82.289	80.173
40	141	141.52	138.204
45	298	255.818	293.919

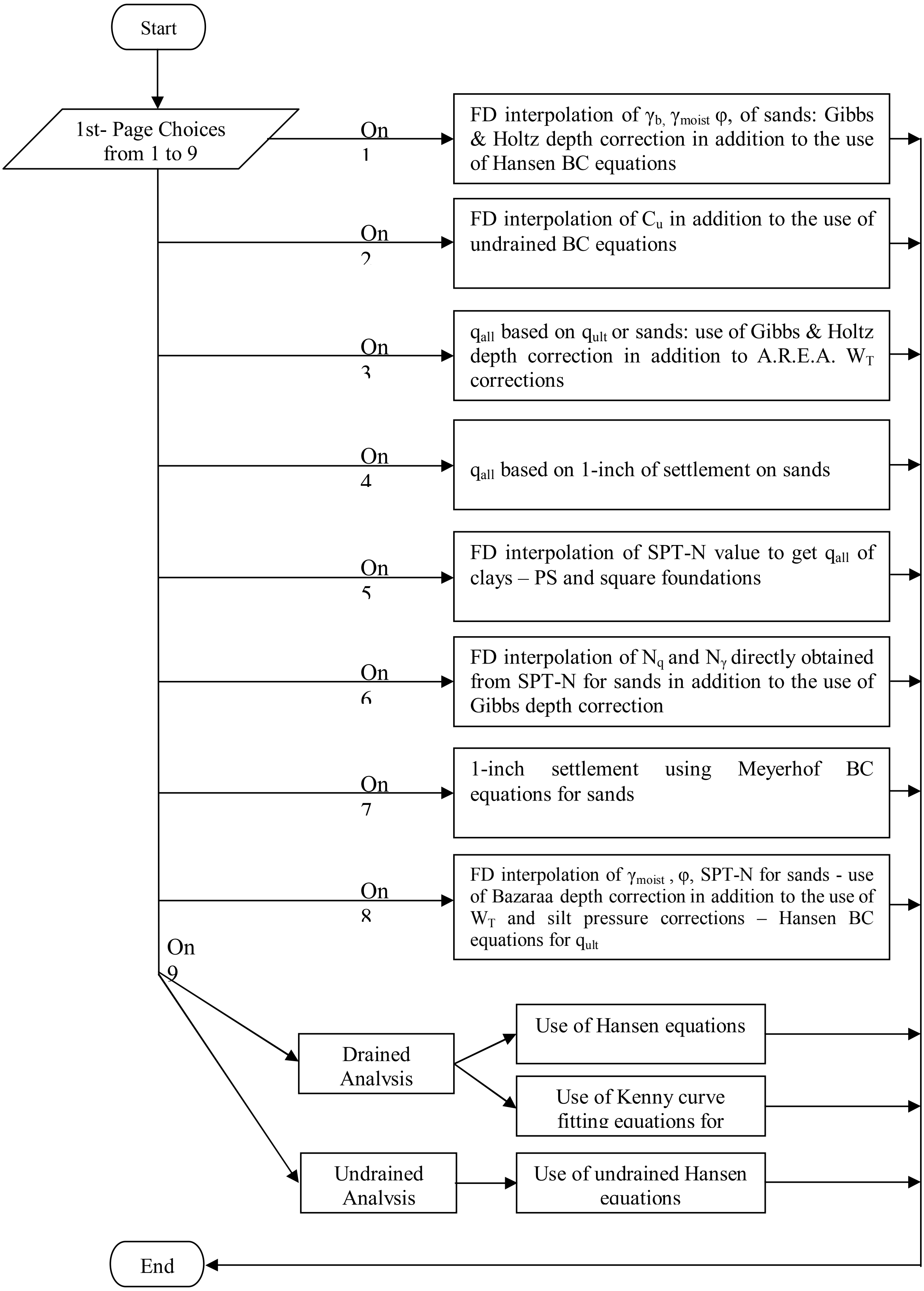

Fig.(1): BASIC program flowchart.

حساب قابلية تحمل التربة بالاعتماد على برمجة طريقة فحص الاختراق القياسي (SPT)

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الخلاصة

من المعلوم أن أي منشأ يجب أن يتم تصميمه بشكل سليم وهذا التصميم يشمل الجزء العلوي من المنشأ superstructure والجزء السفلي منه substructure الذي هو الأسس ومن ضمنها التربة التي يستند عليها المنشأ. أن ثقل المنشأ يتم حمله بواسطة الأعمدة (أو الجدران الساندة) مما يجعلها تحت ضغط شديد. في بعض الأحيان هذا الضغط يصل إلى نصف قوة تحمل الكونكريت أو يزيد. ومن ناحية الأخرى أن قابلية تحمل التربة أقل بكثير من تحمل الكونكريت. وبهذا يجب تعريض نهايات الأعمدة لكي نحصل على أساس. أن قابلية تحمل الكونكريت للانضغاط يمكن قياسه بسهولة أما قابلية تحمل التربة الأقصى فلا تتمتع بهذه الخاصية. أن طرق حساب تحمل التربة الأقصى كثيرة ومتنوعة والنتائج المستحصلة تختلف فما بينها (اختلافا ليس بسيطا) وتتضمن طرق مختبريه_بأخذ نماذج من التربة وفحصها_ أو طرق حقلية_الاختراق الديناميكي أو الستاتيكي. أن أحد الطرق الحقلية المشاع استخدامها في أرجاء البسيطة والعالم هو فحص الاختراق القياسي(SPT) The Standard Penetration Test. لقد حاول العلماء ومنذ أوائل القرن المنصرم أن يحددوا العلاقة بين فحص الاختراق القياسي وخصائص تحمل التربة القصوى مثل الانضغاط، الكثافة، زاوية احتكاك حبيبات التربة، جهد الالتصاق وغيرها وتم وضع تلك العلاقات على شكل جداول ومنحنيات. أن هذا البحث يستخدم أشهر(وليس الأحدث) الطرق المستخدمة لحساب خصائص التربة وقابلية تحملها الأقصى من فحص الاختراق القياسي. تم إعداد برنامج مبسط بلغة QBASIC لهذا الغرض. ومن المعلوم بأن البرامج هذه تستخدم المعادلات بشكل مباشر. ولأجل الحصول على تلك المعادلات تم استخدام طريقة الفروقات المحددة FD لغرض تحويل تلك الجداول والمنحنيات إلى معادلات متعددة الحدود Interpolating Polynomials. أن هذا التحويل يحتاج إلى اختيار نقاط مسمارية Pivotal Points يتمركز عليها التقريب. ولأجل ذلك تم اعتماد الخبرات الشخصية للباحثين.

ويجب التنبيه بالنهاية الى ان هذا البرنامج مع البحث يمكن استخدامه كدليل لحساب قابلية تحمل التربة الأقصى وليس بديلا عن الفحوص والتصاميم المختبرية.