ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES
FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

SOFTWARE DEVELOPMENT FOR DETERMINATION OF BEARING CAPACITY FOR SHALLOW FOUNDATIONS

A thesis submitted to the School of Graduate Studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (GEOTECHNIQUES)

By
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Declaration

This thesis is my original work and has not been presented for a degree in any other university, and that all sources of material used for the thesis have been duly acknowledged.

Candidate's name ______________________________

Signature _____________________________________

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Abstract

Goals of this research have been the development of a user-friendly computer program for determination of soil bearing capacity for design of sallow foundation. In the hope of achieving these goals a computer program named BEARING has been developed. The program demands small effort from the user and provides the user with the ability to determine bearing capacity of soil. The analysis capabilities comprises of uniform soil layer and stratified soil layer (with stratification up to five layers).

CHAPTER 1
INTRODUCTION

1.1. GENERAL

The function of a foundation is to transfer the load of the structure to the soil on which it is resting. A properly designed foundation transfers the load to the soil without overstressing it. Overstressing can result in either excessive settlement or shear failure of the soil, both of which cause damage to the structure. Thus, geotechnical and structural engineers, who design foundations, must evaluate the bearing capacity and settlement of the soil.

Much investigation on the subject of bearing capacity has been carried out during the past century. Subsequently, numerous proposals have been advance regarding considerations, criteria, and procedure for the evaluation of the bearing capacity of soils[20]. In fact, the development of a bearing capacity equation is still undergoing some degree of evaluation. Among the very early contribution were Prandtl, Terzaghi, and Tylor; somewhat more recent refinements were made by Meyerhof, Vesic, Hansen, and many, many others. It is perhaps exhaustive and counterproductive to attempt to delineate the work of all of the numerous contributors to the subject. Hence, an attempt to provide a merely sequential overview of development of current bearing capacity equations will be described, and based on those equations software will be developed in this research.

1.2 OBJECTIVE OF THE THESIS

The objective of the thesis is development of software for the determination of bearing capacity of a soil for Shallow foundations supporting various types of structures for uniform and stratified soil type. The program incorporates known bearing capacity formulas as well as the Ethiopian Building Code Standards EBCS-7. The estimation of the ultimate bearing capacity is an essential component in the design of foundations.

1.3. ORGANIZATION OF THE THESIS

The thesis consists of four chapters. The first chapter discusses briefly the objective and the scope of the thesis. The second chapter is a literature review of bearing capacity of soil for design of foundations. Here the theoretical background, failure modes, factors that influence bearing capacity and methodology to determine the bearing capacity of the soil including the Ethiopia Code are discussed. The third chapter is devoted to the development of software to determine the bearing capacities of the soil. The fourth chapter consists of conclusions and recommendations of the thesis.

CHAPTER TWO

2. LITERATURE REVIEW

Bearing capacity is the ability of soil or rock to-safely carries the load placed on the soil from any engineered structure without undergoing a shear failure. Applying a bearing pressure, which is safe with respect to failure, does not ensure that settlement of the foundation will be within acceptable limits. Therefore, settlement analysis should also be performing.

The general accepted method of bearing capacity analysis is to assume that the soil below the foundation along a critical plane of failure (slip path) is on the verge of failure and to calculate the bearing pressure applied by the foundation required to cause this failure condition. This is the ultimate bearing capacity q_u

2.1. TYPES OF BEARING CAPACITY FAILURES

Experimental investigations have indicated that when a footing fails due to insufficient bearing capacity, distinct failure patterns are developed, depending upon type of failure mechanism. Failure is accompanied by appearance of failure surfaces and by building of sheared mass of soil. Vesic (1963) observed three types of bearing capacity failures as shown in Figure 2-1. These include general shear failure, local shear failure and Punching shear failure

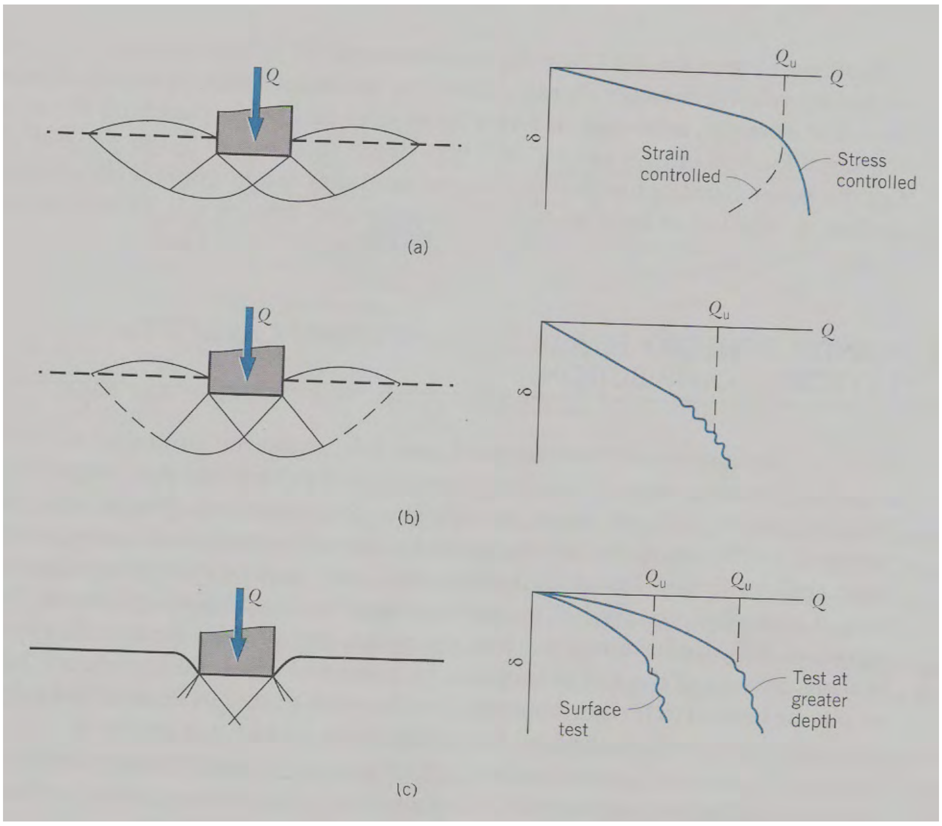

Fig. 2.1 Mode of Bearing Capacity failures (a) General shear. (b) Local shear (c) Punching shear (After Vesic [60] as cited in [20])

2.1.1. GENERAL SHEAR FAILURE

In the case of general shear failure, continuous failure surfaces develop between the edges of the footing and the ground surface, as shown in Fig. 2.2 (a). When the pressure approaches the value of q_u, the state of plastic equilibrium is reached initially in the soil around the edges of the footing, and it then gradually spreads downwards and outwards. Ultimately, the state of plastic equilibrium fully develops throughout the soil above the failure surfaces. The failure is accompanied by appearance of failure surfaces and by considerable bulging of sheared mass of soil. However, the final slip movement would occur only on one side, accompanied by tilting of the footing. Such a failure occurs in soils of low compressibility, i.e. dense or stiff soil, and the pressure-settlement curve is of the general form as shown in Fig. 2.1(a)

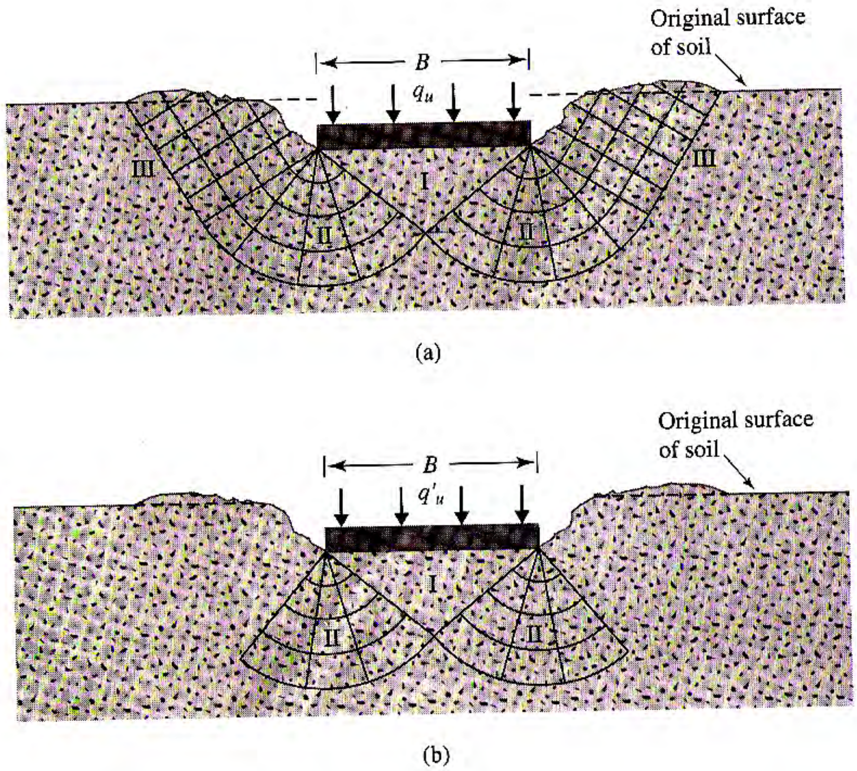

Fig.2.2 Mode of bearing capacity failure in soil: (a) general shear failure of soil; (b) local shear failure of soil [17]

Following are the typical characteristics of general shear failure.

I. It has defined failure surfaces, reaching up to the ground surface.
II. There is considerable bulging of sheared mass of soil adjacent to the footing.
III. Failure accompanied by tilting of the footing in vertical loading.
IV. Failure is sudden, with pronounced peak resistance.
V. The ultimate bearing capacity is well defined.

2.1.2. LOCAL SHEAR FAILURE

In local shear failure, there is significant compression of the soil under the footing and only partial development of state of plastic equilibrium. Due to this reason, the failure surfaces do not reach the ground surface and only slight heaving occurs. Curve of Fig. 2.1(b) represents the pressure-settlement curve, where the peak of the base resistance may never be reached. In such a failure, tilting of foundations is not expected in vertical loading. Local shear failure is associated with soils of medium compressibility and in sands having relative density lying between 35 and 70 percent [20]. The failure is not sudden, and it is characterized by occurrence of relatively large settlements when we compare to general shear failure, which would not be acceptable in practice. In addition, the ultimate bearing capacity in such a failure is not well defined.

Following are typical characteristics of local shear failure:

I. The failure pattern is clearly defined only immediately below the footing.
II. The failure surfaces do not reach ground surface.
III. There is only slight bulging of soil around the footing.
IV. Failure is not sudden and there is no tilting of footing for vertical loading.
V. Failure defined by large settlements.
VI. Ultimate bearing capacity is not well defined.

2.1.3. PUNCHING SHEAR FAILURE

Punching shear failure occurs where there is relatively high compression of soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing. Punching shear may occur in relatively loose sand with relative density less than 35%. Relatively large settlements occur in this mode as shown in Curve of Fig. 2.1 (c). The ultimate bearing capacity is not well defined [9].

Following are the characteristics of punching shear failure:

I. No failure pattern observed.
II. The failure surface, which is vertical or slightly inclined, follows the perimeter of the base.
III. There is no bulging of soil around the footing
IV. There is not tilting of footing
V. Failure is characterized in terms of very large settlements
VI. The ultimate bearing capacity is not well defined.

2.2. CONDITIONS FOR TYPICAL MODE OF FAILURE

The conditions, under which typical modes of failure develop, are given in Fig. 2.3 and Table 2.1. As the relative depth/width ratio increases, the limiting relative densities at which failure type change increase.

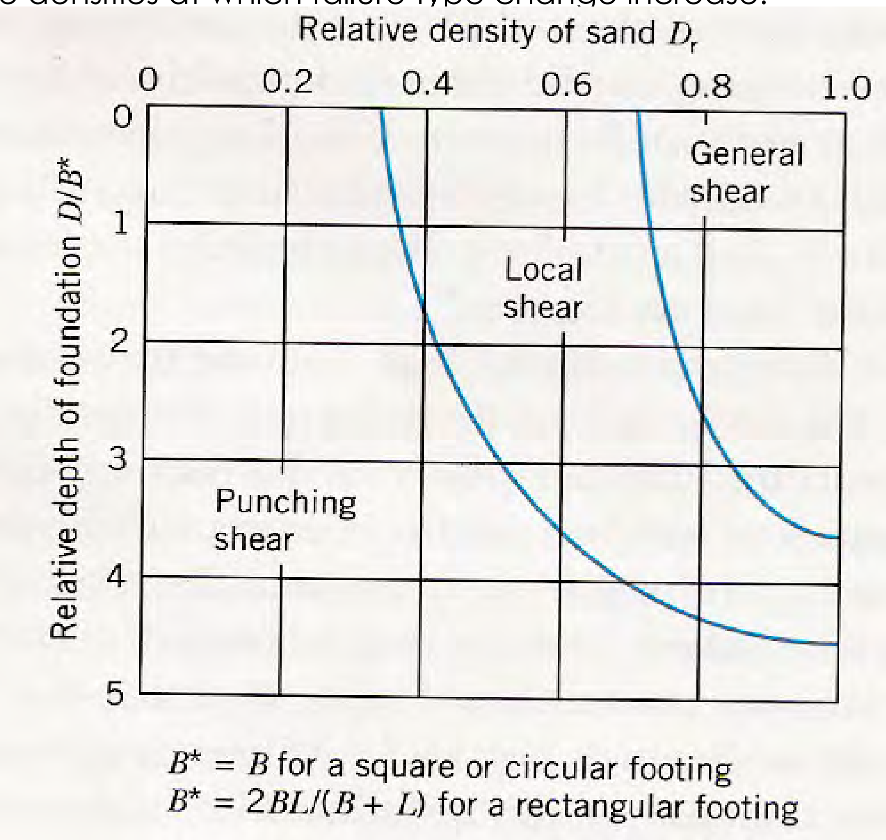

Fig. 2.3. Expected Modes of failure of footing in sand (After Vesic [60] (as cited in [20])

Table 2.1: Conditions of typical mode of failure [17]

Condition	Mode of failure
1. Footing on ground surface or at shallow depth, in very dense sand	General shear failure
2. Footing on saturated, normally consolidated clay, under un-drained loading	General shear failure
3. Very deep footing in dense sand	Punching shear failure
4. Footing on the surface or shallow depth in loose sand	punching shear failure
5. Footing in very dense sand, loaded by transient dynamic loads	punching shear failure
6. Footing on very dense sand underlain by loose sand or soft clay	punching shear failure
7. Footing on saturated, normally consolidated clay under drained loading	punching/local shear failure
8. Footing on ground surface or at shallow depth, in soils of high compressibility	local shear failure
9.(a) Footing at ground surface or at shallow depth, in sands of relative density between 0.3 to 0.7 (b)Footing at great depth, in sands of relative density between 0.7 to 0.9	local shear failure local shear failure

2.3. BEARING CAPACITY EQUATIONS

Shallow foundations such as footings or mats may undergo either a general or local shear failure and sometimes as described above (Table 2.1) punching shear failure may also observed. Local shear occurs in loose sands, which undergo large strains without complete failure. Local shear may also occur for foundations in sensitive soils with high ratios of peak to residual strength. The failure pattern for general shear is modeled in Figure 2-2. Solutions of the general equation are provided using the Terzaghi, Meyerhof, Hansen and Vesic models. Each of these models has different capabilities for considering foundation geometry and soil conditions. Two or more models should be used for each design case, when practical, to increase confidence in the bearing capacity analyses. [17]

2.3.1. ANALYSIS OF TERZAGHI

The following assumptions were made while developing the equation (Analysis).

1. The soil is homogeneous and isotropic and its shear strength represented by Coulomb's equation.
2. The strip footing has a rough base, and the problem is essentially two-dimensional.
3. The elastic zone has straight boundaries inclined at an internal angle of shearing resistance of the soil ($\psi = \phi$) to the horizontal, and the plastic zones are fully developed.
4. The ultimate bearing capacity consists of three components, which can be calculated separately and added, although the critical surfaces for these components are not identical.
5. Failure zones do not extend above the horizontal plane through the base of the footing, i.e. the shear resistance of soil above the base is neglected and the effect of soil around the footing considered by including the surcharge. It is the intensity of vertical pressure at the base level of the foundation and determined by multiplying the unit weight of the soil above the foundation with the depth of the foundation ($\sigma = \gamma_1 D_f$).

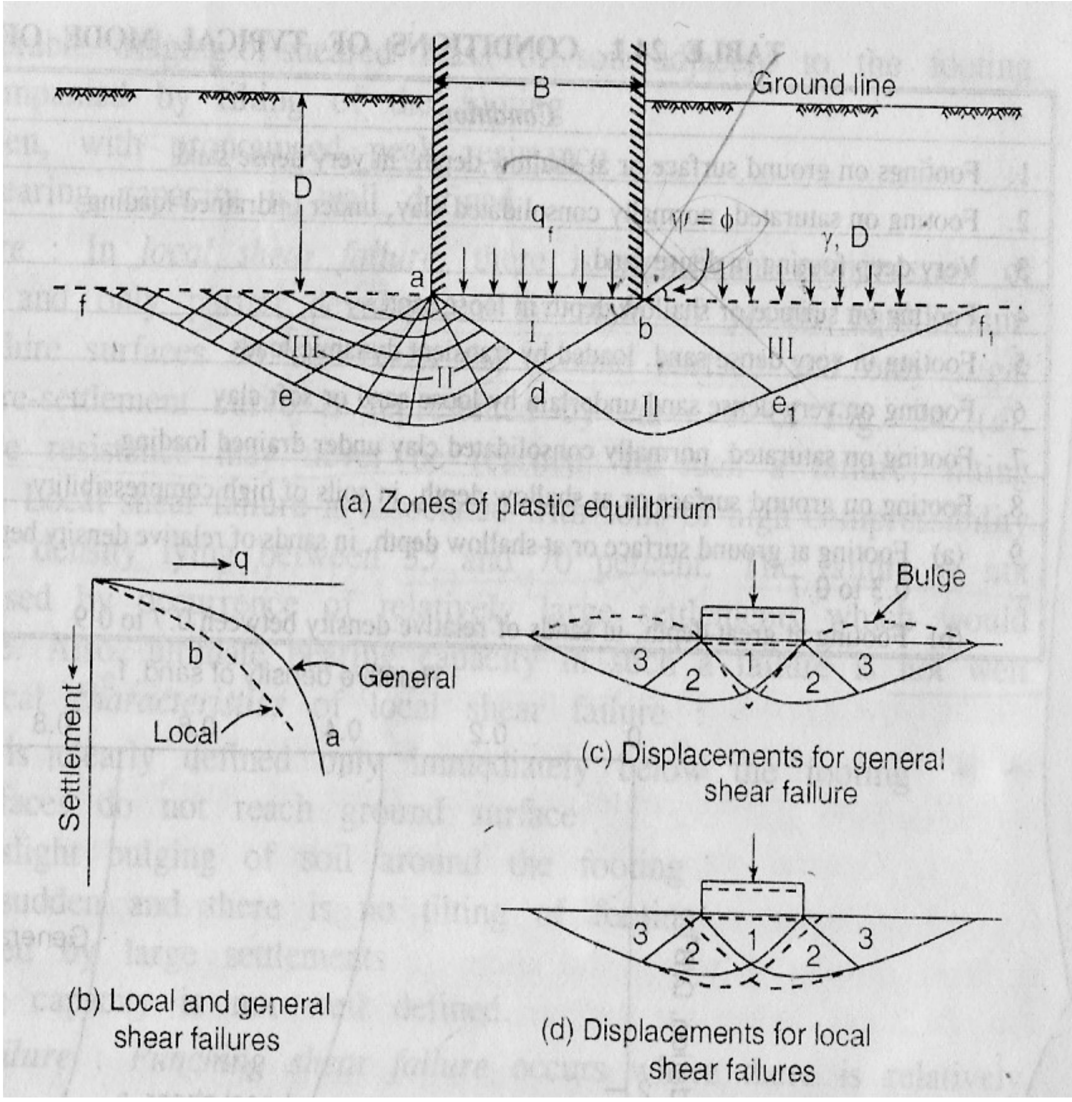

Fig. 2.4 TERZAGHI'S ANALYSIS (1956(as cited in[17]))

With the above assumptions, the analysis done for complete bearing capacity failure, usually termed general shear failure, assuming that the soil behaves like an ideally plastic material. The concept was first developed by Prandtl(1920), and later extended by Terzaghi, Meyerhof and others. Terzaghi derived a general bearing capacity equation from a modification of equations proposed by Prandtl. Fig. 2.4 shows a footing of width B subjected to a loading of intensity q_u causing failure. The foundation is shallow, i.e., the depth D_f of the footing is equal to or less than width B of the footing. In addition, it is a strip footing ($L \succ 5B$).The loaded soil fails along the composite surface as shown in Fig. 2.4(a) $(fede_1f_1)$. This region can be divided into five zones. Zone I, two pairs of zone II, and two pairs of zone III. When the base of the footing (ab) sinks into the ground, zone I (soil wedge (abd) immediately beneath the footing is prevented from undergoing any lateral yield by the friction and adhesion between the soil and the base of footing. Thus, zone I remains in a state of elastic equilibrium, and it acts as if it were part of the footing. Its boundaries (db) and (da) are assumed as plane surface, rising at an angle $\psi = \phi$ with the horizontal. Zone II is called the zone of radial shear, as the lines that constitute one set in the shear pattern radiating from the outer edge of the base of the footing. These radial lines are straight while the lines of the other set are the logarithmic spirals with their centers located at the outer edges of the base of the footing. Zone III is called the zone of linear shear, and is identical with that for passive Rankine state. The boundaries of zone III rise at $45^0 - \frac{\phi}{2}$ with the horizontal. The failure zones are assumed not to extend to the horizontal plane through the base (ab) of the footing. This implies that the shear resistance of the soil above the horizontal plane through the base (ab) of the footing is neglected, and the soil above this plane is replaced with a surcharge ($q = \overline{\sigma} = \gamma D_f$)

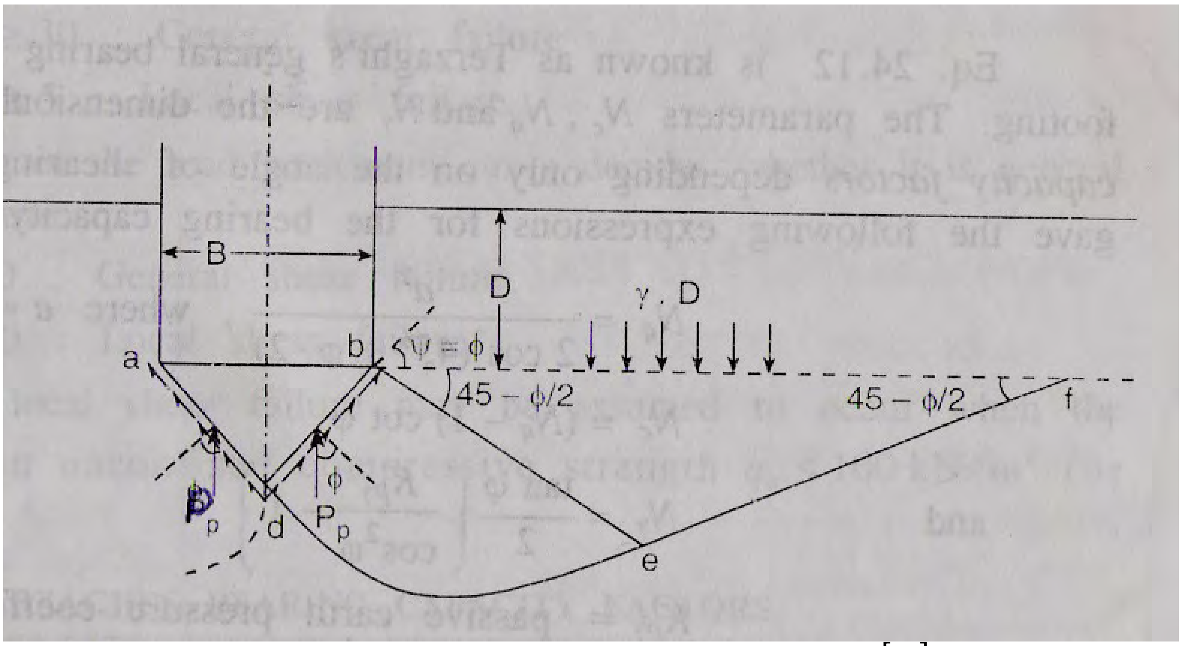

Fig. 2.5 Force Acting to Restrain Failure [17]

The application of the load intensity q_u on the footing tends to push the wedge of this soil (adb) into the ground with lateral displacement of zones II and III, but this lateral displacement is resisted by forces on the plane (db) and (da). These forces are: (i) the resultant of the passive pressure P_p and (ii) the cohesion c acting along the surface (db) and (da). The passive pressure resultant makes an angle ϕ with the normal to the surface (db) and (da). If it is assumed that surfaces (db) and (da) intersect the horizontal line at an angle ϕ, the passive pressure acts vertically. At the instant of failure, the downward and upward forces on the wedge (adb) of unit length must balance.

The downward forces are

I. $q_u B$

II. The weight $\frac{1}{4}\gamma B^2 \tan\phi$ of the wedge.

The upward forces are

I. The resultant Passive pressure (P_p) on each of the surface (db) and (da), and

II. The vertical component of cohesion acting along the lengths (db) and (da), The length $db = da = \frac{B/2}{\cos\phi}$ and hence vertical component of cohesion on each of the surfaces $= c\frac{B/2}{\cos\phi}\sin\phi = \frac{B}{2}c\tan\phi.$

$$q_u B + \frac{1}{4}\gamma B^2 \tan\phi = 2P_p + 2\times\frac{B}{2}c\tan\phi$$

Hence: $$q_u B = 2P_p + B\,c\tan\phi - \frac{1}{4}\gamma B^2\tan\phi \qquad (2.1)$$

The result of passive earth pressure (P_p) can be divided into three components:

I. $p_{p\gamma}$ produced by weight of the shear zone $(dbfe)$,

II. p_{pc} produced by soil cohesion, and

III. p_{pq} Produced by surcharge

These components of passive pressure are computed separately and then added to obtain the value of p_p substituting these components in Eq. 2.1

$$q_u B = 2(P_{p\gamma} + P_{pc} + P_{pq}) + B\,c\tan\phi - \frac{1}{4}\gamma B^2\tan\phi \qquad (2.2)$$

$$q_u B = \left(2P_{p\gamma} - \frac{1}{4}\gamma B^2\tan\phi\right) + (2P_{pc} + B\,c\tan\phi) + 2P_{pq} \qquad (2.3)$$

Let:
$$2P_{p\gamma} - \frac{1}{4}\gamma B^2\tan\phi = B\frac{1}{2}\gamma B N_\gamma$$
$$2P_{pc} + B\,c\tan\phi = B\,c\,N_c$$
$$2P_{pq} = B\times\bar{\sigma}N_q$$

The above Eq. (2.3) will become:

$$q_u = cN_c + \overline{\sigma}.N_q + 0.5\gamma BN_\gamma \qquad (2.4)$$

Where $\overline{\sigma}$ = Effective surcharge at the base level of foundation (It is the intensity of vertical pressure at the base level of foundation, computed assuming total unit weight for the portion of the soil above the water table and submerged unit weight for the portion below the water table).

q_{nu} = The net ultimate bearing capacity (which is the minimum net pressure intensity causing shear failure of the soil) = $q_u - \overline{\sigma}$

$$q_{nu} = cN_c + \overline{\sigma}(N_q - 1) + 0.5\gamma BN_\gamma \qquad (2.5)$$

q_s = Safe bearing capacity (It is the maximum pressure which the soil can carry safely without risk of shear failure) = $\frac{q_{nu}}{F.S} + \overline{\sigma}$, and

$F.S$ = Factor of Safety

$$q_s = \frac{1}{F.S}\left[c\,N_c + \overline{\sigma}(N_q - 1) + 0.5\gamma BN_\gamma\right] + \overline{\sigma} \qquad (2.6)$$

Eq.2.4 is known as Terzaghi's general bearing capacity equation for a continuous footing. The parameters N_c, N_q and N_γ are dimensionless numbers, known as bearing capacity factors depending only on the angle of shearing resistance of the soil. Terzaghi gave the following expressions for the bearing capacity factors are given:

$$N_q = \frac{a^2}{2\cos^2(45^o + \phi/2,)} \qquad (2.4a)$$

$$where:\ a = e^{(0.75\Pi - \phi/2)\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi \qquad (2.4b)$$

$$N_\gamma = \frac{\tan\phi}{2}\left[\frac{K_{p\gamma}}{\cos^2\phi} - 1\right] \qquad (2.4c)$$

$K_{p\gamma}$ = coefficient, dependent on ϕ

Terzaghi never explained very well how he obtained $K_{p\gamma}$ used to compute N_γ. He did, however, give a curve of ϕ versus N_γ and three specific values of N_γ at ϕ= 0, 34° and 48°. The value of the bearing capacity factor N_c, N_q and N_γ can be obtained from Table 2.2. The values of N_γ for ϕ of 34° and 48° are the original Terzaghi values.

The type of failure analyzed above is called the general shear failure and the bearing capacity factors N_c, N_q and N_γ correspond to the general shear failure. In such a failure, the soil properties are assumed to be such that a slight downward movement of footing develops fully plastic zones and the soil bulges out (Fig. 2.2(a)). In the case of fairly soft or loose and compressible soil, large deformations may occur below the footing before the failure zones are fully developed. Such a failure is called a local shear failure (Fig. 2.2(b)) which is associated with considerable vertical soil movement before soil bulging takes place. In the absence of analytical solution, Terzaghi has proposed the following local shear-failure soil parameters as input

$$c_m = \frac{2}{3} c$$

$$\tan\phi_m = \frac{2}{3} \tan\phi$$

The bearing capacity factors are determined with respect to these reduced parameters c_m and ϕ_m. The bearing capacity factors in Table 2.2 corresponding to the local shear failure are indicated with dashes $(N_c', N_q' and\, N_\gamma')$. The bearing capacity equation would then be written:

$$q_u = \frac{2}{3} c N_c' + \overline{\sigma} N_q' + 0.5 \gamma B N_\gamma' \qquad (2.8)$$

It is difficult to define the limiting conditions for which general or local shear failure should be assumed at a given site. However, the following points may be used as a guide [17]:

I. Stress strain test $(c-\phi\, soil)$: General shear failure at low strain, say < 5% while for local shear failure, stress strain curve continues to rise at strains of 10% to 20%.

II. Angle of shear resistance: For $\phi > 36^o$, General shear failure

$\phi < 28^o$, Local shear failure

III. Penetration test: N ≥ 30: General Shear failure

N ≤ 5: Local shear failure

IV. Plate load test: Shape of the load settlement curve decides whether it is general shear failure or local shear failure.

V. Density Index: $I_D > 70$: General shear failure

$I_D < 20$: Local shear failure

For purely cohesive soil, local shear failure may be assumed to occur when the soil is soft to medium, with an unconfined compressive strength $Q_u \le 100\, kN/m^2 \left(or\, c_u \le 50\, kN/m^2\right)$.

Table 2.2 Terzaghi's bearing capacity factors [17]

ϕ	General Share failure			Local shear failure		
	N_c	N_q	N_γ	N'_c	N'_q	N'_γ
0	5.7*	1.0	0.0	5.7	1.0	0.0
5	7.3	1.6	0.5	6.7	1.4	0.2
10	9.6	2.7	1.2	8.0	1.6	0.5
15	12.9	4.4	2.5	9.7	2.7	0.9
20	17.7	7.4	5.0	11.8	3.9	1.7
25	25.1	12.7	9.7	14.8	5.6	3.2
30	37.2	22.5	19.7	19.0	8.3	5.7
34	52.6	36.5	35.0	23.7	11.7	9.0
35	57.8	41.4	42.4	25.2	12.6	10.1
40	95.7	81.3	100.4	34.9	20.5	18.8
45	172.3	173.3	297.5	51.2	35.1	37.7
48	258.3	287.9	780.0	66.8	50.5	60.4
50	347.5	415.1	1153.2	81.3	65.6	87.1

* $(1.5\pi + 1)$

In order to take into account the shape of the footing (i.e. strip, round, square etc.) Terzaghi used only shape factors for the cohesion (s_c) and base (s_γ) terms, taking into account these factors, Terzaghi's original equation (Eq. 2.4) can be expressed as:

$$q_u = cN_c s_c + \overline{\sigma} N_q + 0.5\gamma B N_\gamma s_\gamma. \qquad (2.4\,e)$$

The values of s_c *and* s_γ are given below [20]

Shape	Strip	Round	Square
s_c	1.0	1.3	1.3
s_γ	1.0	0.6	0.8

Based on these values, Terzaghi gave the following semi–empirical equations for square, circular and rectangular footings.

(a) Frictional cohesive soil (c-ϕ soil)

For circular footing:

$$q_u = 1.3cN_c + \overline{\sigma} N_q + 0.3\gamma BN_\gamma \quad (2.9)$$

Where B= diameter of the footing

For square footing:

$$q_u = 1.3cN_c + \overline{\sigma} N_q + 0.4\gamma BN_\gamma \quad (2.10)$$

Where B= width

(b) Cohesive soil $[\phi = 0; c > 0]$

For circular footing:

$$q_u = 1.3cN_c + \overline{\sigma} = 7.4c + \overline{\sigma} \quad (2.11)$$

For square footing:

$$q_u = 1.3c\mathrm{N}_c + \overline{\sigma} = 7.4c + \overline{\sigma} \quad (2.12)$$

(c) Non-cohesive soil $(\phi > 0; c = 0)$

For circular footing:

$$q_u = \overline{\sigma} N_q + 0.3\gamma BN_\gamma \quad (2.13)$$

For square footing:

$$q_u = \overline{\sigma} N_q + 0.4\gamma BN_\gamma \quad (2.14)$$

Terzaghi assumed the value of angle $\psi = \phi$ (Fig 2.4), which is not true. In actual practice, ψ was found to vary from $45^0 - \frac{\phi}{2}$ for perfectly smooth base to $45^0 + \frac{\phi}{2}$ for perfectly rough base [17]. Since footings are normally rough, ψ has been found close to $45^0 + \frac{\phi}{2}$ than to ϕ. In addition, Terazaghi's bearing capacity equations do not have provisions for depth effects, inclination factors, etc. Moreover, the following limitations are observed:

1) As the soil compresses, ϕ changes, slight downward movement of footing may not develop fully the plastic zones.
2) Error due to assumption 5 (Failure zones do not extend above the horizontal plane through the base of the footing…) increases with depth of foundation, and hence the theory is suitable for shallow foundation only.

2.3.2. MIXED STATE OF LOCAL AND GENERAL SHEAR FAILURE

For intermediate value of $\phi\ (28^0 \prec \phi \prec 36^0)$ there is a mixed state of local and general shear failure, for which no separate equations are available. As ϕ increases above 28^0, there is gradual transition from local to general shear failure. Peck, Hansen and Thornburn give curves for N_q and N_γ for the transition state from $\phi = 28^0$ to $\phi = 36^0$, in which they assumed general shear failure when $\phi \succ 28^0$. These values can be obtained from curves for N_γ *and* N_q sown in Fig. 2.6. It will be seen that when $\phi \succ 38^0$, the values are the same as given in Table 2.1.

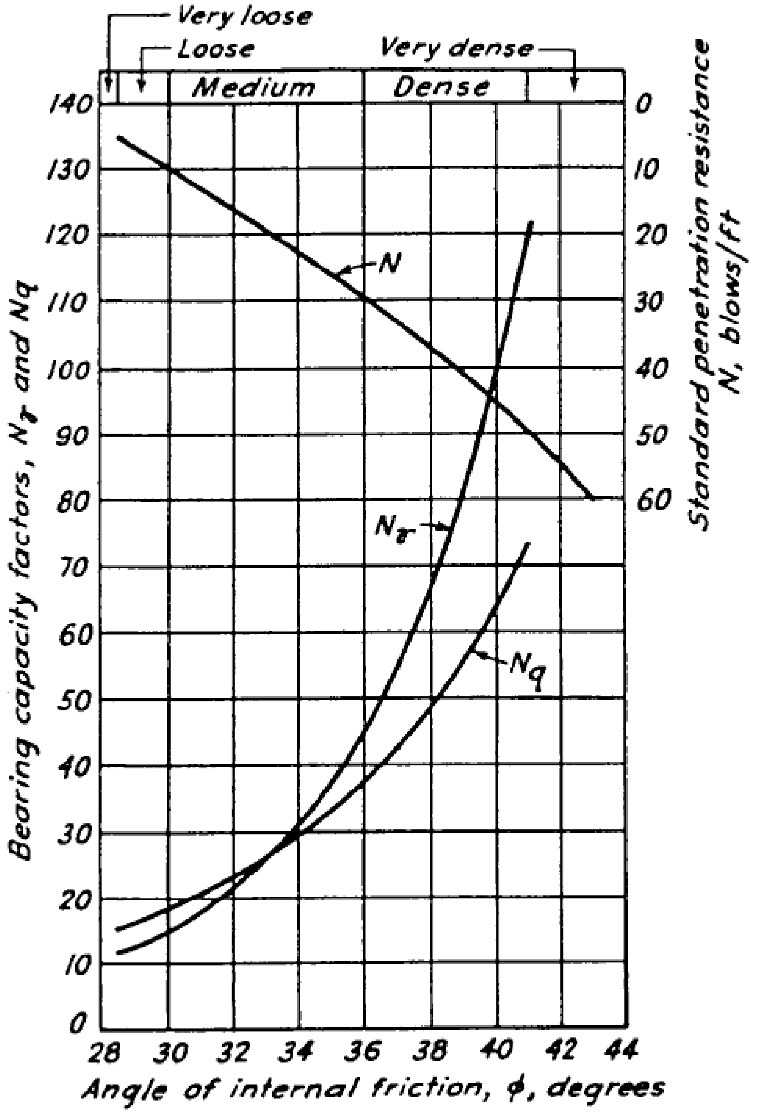

FIG 2.6. Bearing capacity factors N_q and N_γ, which automatically incorporate allowance for punching and local shear failure. (Reproduced from R. B. Peck, W. E. Hanson, and T. H. Thornburn, ''Foundation Engineering,'' John Wiley & Sons, Inc., New York, reproduced with permission of John Wiley & Sons, Inc.) [21]

2.3.3. ANALYSIS ACCORDING TO MEYERHOF

Meyerhof extended the analysis of plastic equilibrium of a surface footing to shallow and deep foundations. Fig. 2.7 (a) and (b) show the failure mechanisms for shallow and deep foundations according to both Terzaghi and Meyerhof analysis. In the Meyerhof's analysis, (abd) is the elastic zone, (bde) is the radial shear zone and $(befg)$ is the zone of mixed shear in which shear varies between radial and plane shear, depending largely upon the depth and roughness of the foundation. The plastic equilibrium in these zones can be established from the boundary conditions starting from the foundation shaft. To simplify the analysis, Meyerhof introduced a parameter β, the angle to define the line (bf), joining point (b) to (f) where the assumed boundary failure slip line intersects the soil surface. The resultant effect of the wedge of soil (bfg) is representing by the normal stresses (P_o) and tangential stresses (s_o) on (bf). The plane (bf) is termed as the equivalent free surface, and (P_o) and (s_o) are termed as the equivalent free surface stresses. The angle β increases with depth, and become 90^0 for deep foundation. Besides this, Meyerhof solution considers correction factors for eccentricity, load inclination and foundation depth as described below.

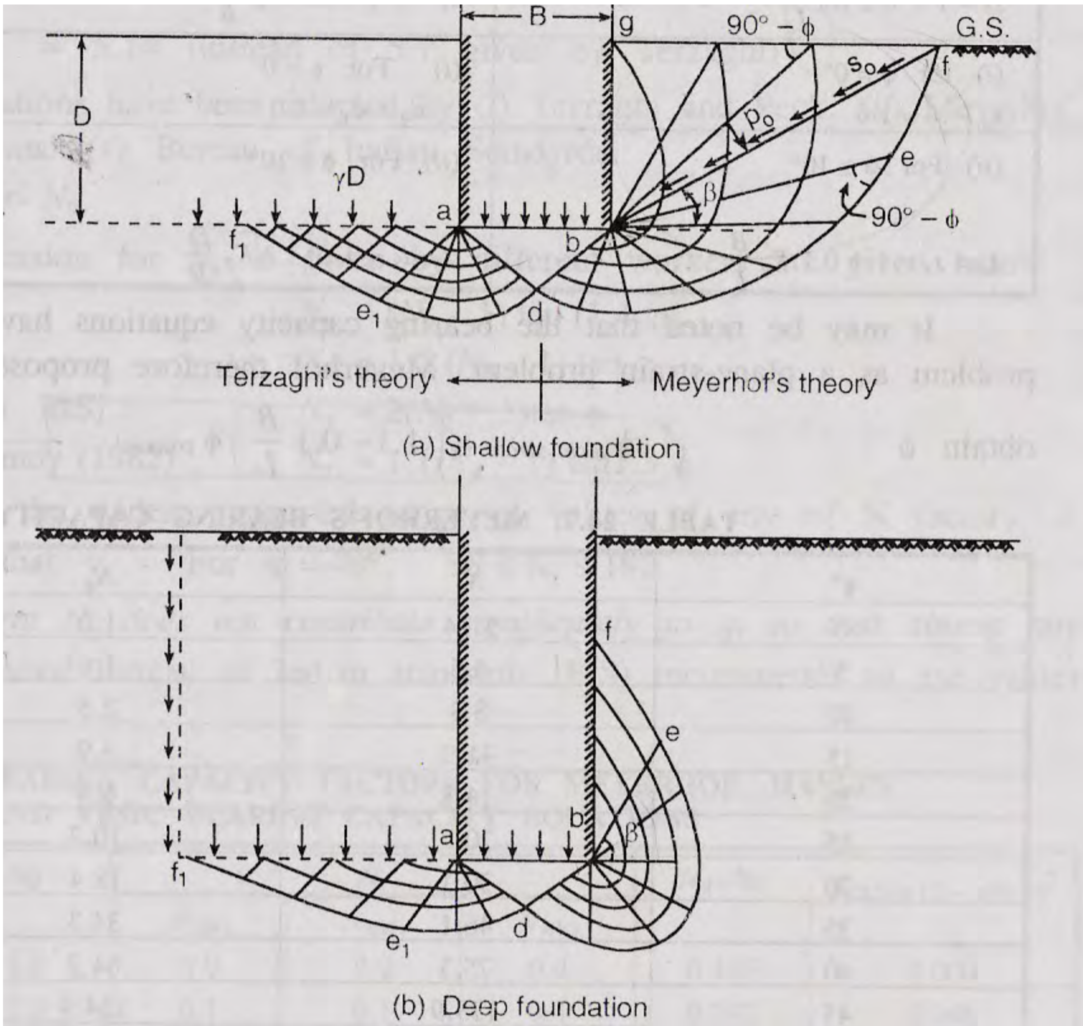

FIG.2.7. MEYEROF'S ANALYSIS [17]

Depth Factors: The bearing capacity factors do not consider the shearing resistance of the failure plane passing through the soil zone above the level of the foundation base. If this upper soil zone possesses significant shearing strength, the ultimate value of bearing capacity would increase (Meyerhof, 1951). For this case, depth factors are applied. The use of the depth factors is conditional upon the soil above the foundation level being not significantly inferior in shear strength characteristics to this level.

Factor For Eccentric Loads: A foundation engineer frequently comes across footings subjected to eccentric loads, e.g. in the case of the foundations of retaining walls, abutments, columns, stanchions, portal framed buildings, etc. The eccentricity may be one way or two ways as shown in the figs. 2.8. The effect of eccentricity can be considered as follows.

One-way eccentricity (Fig.2.8a): If the load has an eccentricity e, with respect to the centroid of the foundation in only one direction, then a length equal to 2e shall reduce the effective dimension of the footing in the direction of eccentricity. The modified dimension shall be used in the bearing capacity equation and in determining the effective area of the footing in resisting the load.

Two-way eccentricity (Fig.2.8b): If the load has double eccentricity $(e_L \, and \, e_B)$ with respect to the centroid of the footing, then the effective dimensions of the footing to be used in determining the bearing capacity as well as in computing the effective area of the footing resisting the load shall be determined as given below:

$$L' = L - 2e_L \qquad (2.15a)$$

$$B' = B - 2e_B \qquad (2.15b)$$

$$A' = L' \times B' \qquad (2.15c)$$

For a design, eccentricity should be less than one-sixth of the foundation dimension to prevent the condition of separation occurring beneath the part of the foundation.[10]

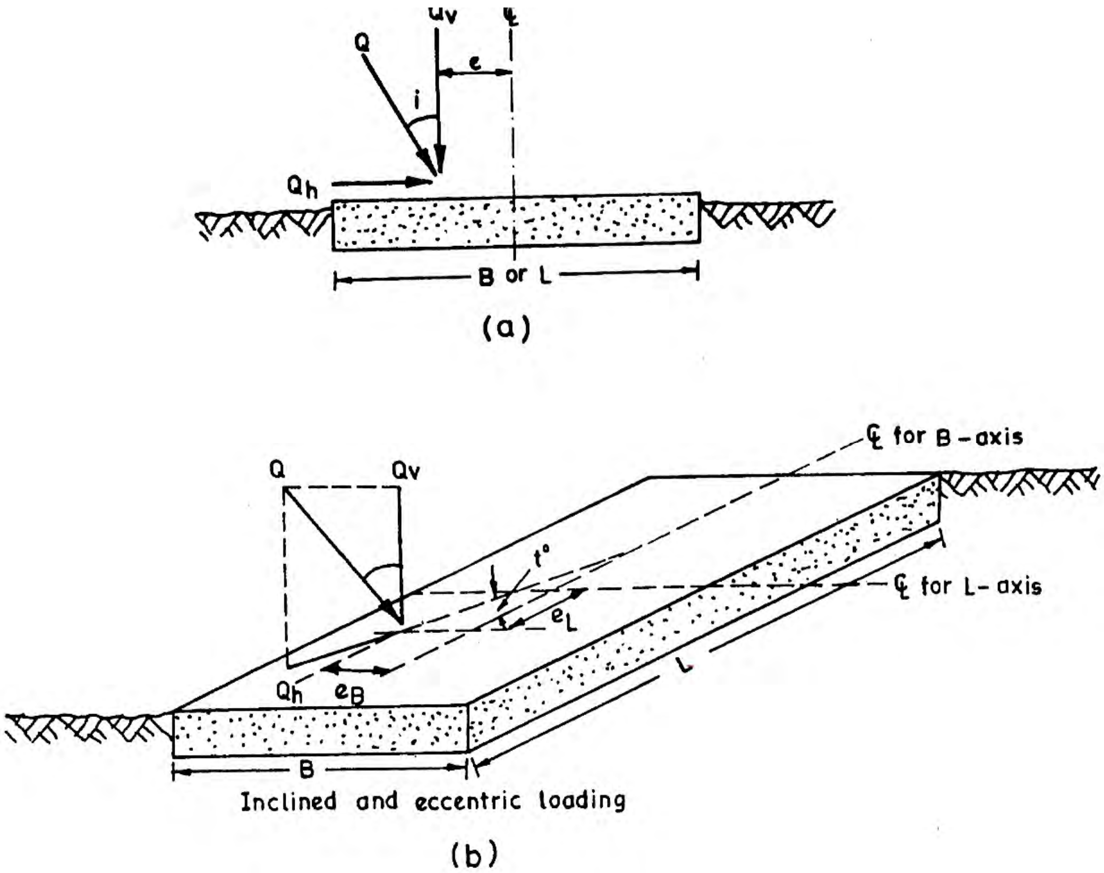

Fig 2.8 Eccentric and Inclined loaded footing: (a) one-way Eccentricity (b) Two-way Eccentricity

Bearing Capacity with Inclined Loads: The conventional method of checking the stability of footing subjected to inclined loads is to resolve the inclined load, Q in to a vertical component (V) and a horizontal load (in the direction of the width (H_b) or in the direction of the length (H_l) or in both direction). The bearing capacity of the footings is found as though the footing is subjected to only the vertical component V and its stability against general shear or local shear failure is checked by finding whether the resistance to horizontal movement, $(R_H = V \tan\phi + cA')$ is greater or less than the horizontal load (H). The ratio of $\frac{R_H}{H}$ =1.5 to 2.0 is generally taken to be sufficient. Meayerhof (1953) developed a reduction factor for determining the bearing capacity of continuous footings with inclined loads from the bearing capacity of the same footings subjected to normal loads. For this condition, base inclination factors are suggested as given below:

$$q_u = cN_c s_c d_c i_c + \overline{\sigma} N_q s_q d_q i_q + 0.5B' N_\gamma s_\gamma d_\gamma i_\gamma \qquad (2.16)$$

Where: $$N_q = e^{\pi \tan\phi} \tan^2\left(45^0 + \frac{\phi}{2}\right) \qquad (2.17)$$

$$N_c = (N_q - 1)\cot\phi \qquad (2.18)$$

$$N_\gamma = (N_q - 1)\tan(1.4\phi) \qquad (2.19)$$

The bearing capacity factors N_c, N_q and N_γ are given in Table 2.3. It will be seen that the factors N_c and N_q are the same as given by Hansen (Table 2.5) while the factor N_γ is different, and

Hear: $s =$ shape factors; $d =$ depth factors; $i =$ inclination factors and the values of these factors are given in Table 2.3

TABLE 2.3 SHAPE, DEPTH AND INCLINATION FACTORS FOR THE MEYERHOF EQUATION[8]

Shape factors	Depth factors	Inclination factors
For any ϕ $s_c = 1 + 0.2K_p \frac{B'}{L'}$	For any ϕ $d_c = 1 + 0.2\sqrt{K_p}\frac{D}{B}$	For any ϕ $i_c = i_q = (1 - \frac{\alpha}{90^0})^2$
(i) for $\phi = 0^o$ $s_q = s_y = 1.0$	(i) for $\phi = 0^0$ $d_q = d_y = 1.0$	$i_\gamma = 1$ for $\phi = 0$ $i_\gamma = \left(1 - \frac{\alpha}{\phi}\right)^2$ for $\alpha < \phi$ and $\phi \succ 0$ $i_\gamma = 0$ for $\alpha > \phi$ and $\phi = 0$
(ii) For $\phi \geq 10^0$ $s_q = s_y = 1 + 0.1K_p \frac{B'}{L'}$ Linear Interpolation B/n $\phi = 0$ and $\phi = 10$	(ii) for $\phi \geq 10^0$ $d_q = d_y = 1 + 0.1\sqrt{K_p}\frac{D}{B}$ Linear Interpolation B/n $\phi = 0$ and $\phi = 10$	α = angle of resultant measured from vertical axis $i_c = i_q = i_\gamma = 1$ for $\alpha = 0$ $K_p = \tan^2\left(45^0 + \frac{\phi}{2}\right)$

It may be noted that the bearing capacity equations have been derived, treating the problem as a plane-strain problem. Meyerhof therefore proposed the following equation to obtain $\phi_{plane\ strain}$:

$$\phi_{Plane\ strain} = \phi_{triaxial}\left(1.1 - 0.1\frac{B}{L}\right) \quad (2.20)$$

TABLE 2.4 MEYERHOF'S BEARING CAPACITY FACTORS [1]

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0
5	6.5	1.6	0.1
10	8.3	2.5	0.4
15	11.0	3.9	1.1
20	14.8	6.4	2.9
25	20.7	10.7	6.8
30	30.1	18.4	15.7
35	46.1	33.3	37.1
40	75.3	64.2	93.7
45	133.9	134.9	262.7
50	266.9	319.0	873.7

2.3.4. ANALYSIS ACCORDING TO BRINCH HANSEN

Starting from 1961, many new equations have been proposed by various workers, giving the ultimate bearing capacity of foundations (These equations are based on the assumption of various values of angle ψ and on the shape of failure surfaces). Notable amongst the workers are Hansen (1970) Hu (1964), Chen and Davidson (1973) and Balla (1962) (as cited in [17]). Of these, the one proposed by Hansen is based on the extension of Meyerhof's work to include tilting base and slopes of the ground supporting the footing, bearing capacity analysis of both shallow and deep foundations in addition to foundation shape and eccentricity, load inclination, and foundation depth.

According to Hansen, the ultimate bearing capacity is given by

$$q_u = cN_c s_c d_c i_c g_c b_c + \sigma_0 N_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma BN_\gamma s_y d_y i_y g_y b_y \qquad (2.21a)$$

$$q_{uB} = cN_c s_{cB} d_c i_{cB} g_c b_c + \sigma_0 N_q s_{qB} d_q i_q B g_q b_q + \frac{1}{2}\gamma BN_\gamma s_{yB} d_y i_{yB} g_y b_y \qquad (2.21b)$$

$$q_{uL} = cN_c s_{cL} d_c i_{cL} g_c b_c + \sigma_0 N_q s_{qL} d_q i_{qL} g_q b_q + \frac{1}{2}\gamma BN_\gamma s_{yL} d_y i_{yL} g_y b_y \qquad (2.21c)$$

This method required a trial process to determine the ultimate bearing capacity of a given soil in the case of an inclined load with the horizontal component in the direction of the width (H_B) that results (q_{uB}), or in the direction length (H_L) that results (q_{uL}), or both direction $\left(H_i = \sqrt{\left(H_B^2 + H_L^2\right)}\right)$ that results (q_u), and take the smaller value. For the case of vertical loading condition, the inclination factors will be one and Eq. $(2.21a)$ is used.

Where: $\sigma_0 = \overline{\sigma}$ = effective overburden pressure at foundation level.

s = shape factor, to account for the effect of the shape of the foundation in developing a failure surface.

d = depth factor to account for the embedment depth and the additional shearing resistance in the top soil.

i = inclination factor to account for both horizontal and vertical components of foundation loads.

g = ground factor, and b = base factor.

γ = density of soil below the foundation level.

For $\phi = 0$ or undrained condition, the above equation is modified to:

$$q_u = 5.14\, c\left[1 + S'_C + d'_c - i'_c - b'_c - g'_c\right] + \sigma_o \qquad (2.22)$$

Where the primed factors are for $\phi = 0$ (Undrained) condition.

In Eq. 2.21 the bearing, capacity factors are computed by the following equations:

$$N_q = \tan^2\left(45^0 + \frac{\phi}{2}\right)\left(e^{\pi \tan\phi}\right) = K_p e^{\pi \tan\phi} \qquad (2.23a)$$

$$N_c = \left(N_q - 1\right)\cot\phi \qquad (2.23b)$$

and $$N_\gamma = 1.5\left(N_q - 1\right)\tan\phi \qquad (2.23c)$$

The bearing capacity factors are given in Table 2.5 and Fig. 2.11 Hansen considered the bearing capacity as a plane- strain problem. If ϕ is found by triaxial tests, its corresponding value for the plane-strain case can be computed from the expression

$$\phi_{plane\ strain} = 1.1\ \phi_{triaxial} \qquad (2.24)$$

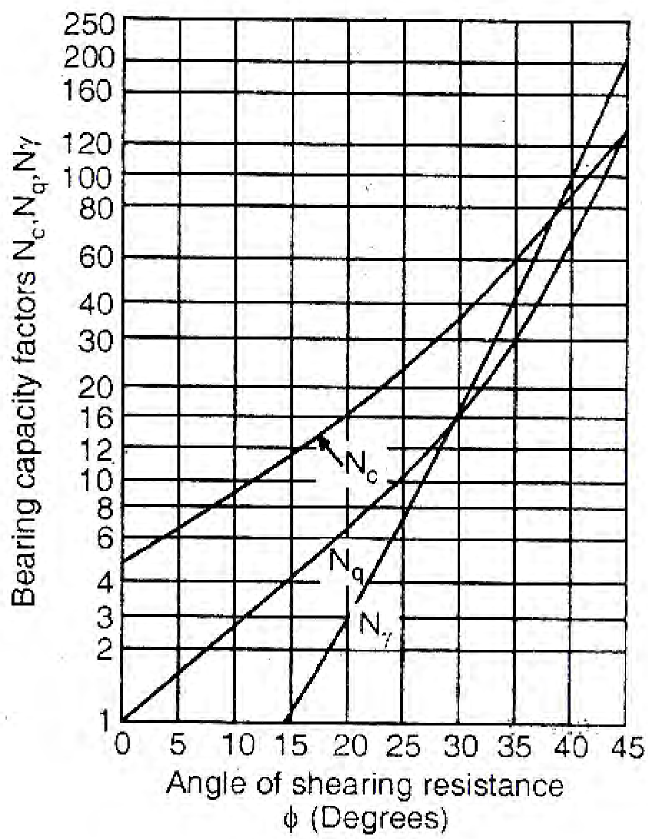

FIG. 2.9 BRINCH-HANSEN'S FACTORS N_c, N_q and N_γ

The values of shape factors, ground factors and base factors are given in

Table 2.6 while the values of depth factors and inclination factors are given in Table 2.7. The following symbols are used in these tables.

$A' =$ Effective footing contact area $= B' \times L'$

$L' =$ Effective footing length $= L - 2e_L$

$B' =$ Effective footing length $= B - 2e_B$

$e_B, e_L =$ Eccentricity of load, with respect to the center of footing area

$D =$ Depth of footing in ground

$H, V =$ Load components parallel and perpendicular to the footing respectively

$\eta =$ Inclination of the base (supporting the footing) with horizontal, reckoned positive for up-slop and negative for down slopes

$\beta =$ Ground slope with horizontal, reckoned positive for down slopes and negative for up slopes

$\delta = \phi =$ For concrete poured on ground

$L =$ Length of footing and $B =$ Width of footing

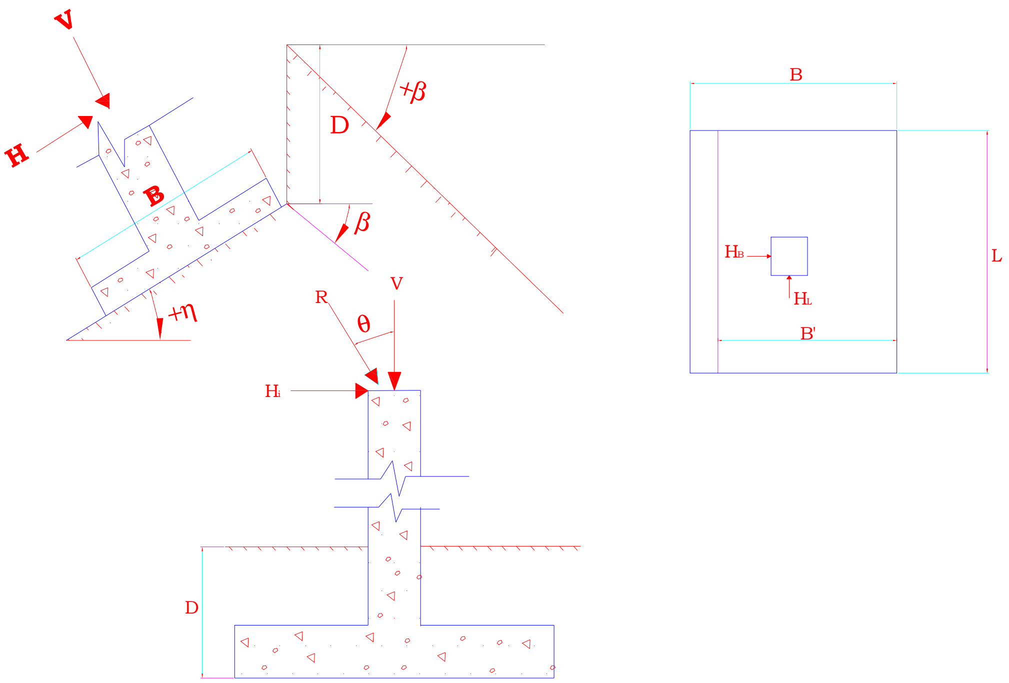

Fig 2.10 HANSEN'S ANALYSIS

Table 2.5 HANSEN'S bearing capacity factors [1]

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0
5	6.49	1.6	0.1
10	8.34	2.5	0.4
15	10.98	3.9	1.2
20	14.83	6.4	2.9
25	20.72	10.7	6.8
26	22.25	11.9	7.9
28	25.80	14.7	10.9
30	30.14	18.4	15.1
32	35.49	23.2	20.8
34	42.16	29.4	28.8
36	50.59	37.8	40.1
38	61.35	48.9	56.2
40	75.31	64.2	79.5
45	133.87	134.9	200.8
50	266.38	319.1	563.6

Table 2.6 and 2.7 have the following limitations:

$H \leq V \tan\delta + c.A'$, $i_q, i_f > 0.0$ and $\beta \leq \phi$

TABLE 2.6. HANSEN'S SHAPE FACTORS, GROUND FACTORS AND BASE FACTORS[1]

Shape factors (s)	Ground inclination (slope) Factors (g)	Base inclination factors (b)
$s_c' = 0.2\,\frac{B'}{L'}$ if $H_B = H_L = 0$ $s_{cB}' = 0.2\, i_{cB}\,\frac{B'}{L'}$ if $H_B \neq 0$ $s_{cL}' = 0.2\, i_{cL}\,\frac{L'}{B'}$ if $H_L \neq 0$	$g_c' = \beta^0/147^0$	$b_c' = \eta^0/147^0$
$s_c = 1 + \frac{N_q}{N_c}\frac{B'}{L'}$ if $H_B = H_L = 0$ $s_{cB} = 1 + i_{cB}\frac{N_q}{N_c}\frac{B'}{L'}$ if $H_B \neq 0$ $s_{cL} = 1 + i_{cL}\frac{N_q}{N_c}\frac{L'}{B'}$ if $H_L \neq 0$	$g_c = 1 - (\beta^0/147^0)$	$b_c = 1 - (\eta^0/147^0)$
$s_q = 1 + \frac{B'}{L'}\sin\phi$ if $H_B = H_L = 0$ $s_{qB} = 1 + i_{qB}\frac{B'}{L'}\sin\phi$ if $H_B \neq 0$ $s_{qL} = 1 + i_{qL}\frac{L'}{B'}\sin\phi$ if $H_L \neq 0$	$g_q = g_\gamma = (1 - 0.5\tan\beta^0)^5$	$b_q = e^{-2\eta\tan\phi}$ $b_y = e^{-2.7\eta\tan\phi}$
$s_\gamma = 1 - 0.4\frac{B'}{L'}$ if $H_B = H_L = 0$ $s_{\gamma B} = 1 - 0.4\frac{i_{\gamma B}}{i_{\gamma L}}\frac{B'}{L'} \geq 0.6$ if $H_B \neq 0$ $s_{\gamma L} = 1 - 0.4\frac{i_{\gamma L}}{i_{\gamma B}}\frac{L'}{B'} \geq 0.6$ if $H_L \neq 0$	For horizontal ground $g_c = g_q = g_y = 1.0$	For horizontal base $b_c' = 0.0$ $b_c = b_q = b_y = 1.0$

Note:-primed factors are for undrained conditions and $\phi = 0, \eta\ is in radians for b_q and b_y$

TABLE 2.7. HANSEN'S DEPTH FACTORS AND INCLINATION FACTORS [1]

Depth factors	**Load Inclination factors**
$d_c' = 0.4\dfrac{D}{B}\ for\ \ D \le B$ $d_c' = 0.4\ \tan^{-1}\dfrac{D}{B}\ for\ D > B$	$i_c' = 0.5 - 0.5\sqrt{1 - \dfrac{H_i}{c\,A'}}$ $i_c = i_q - \dfrac{1 - i_q}{N_q - 1}$
$d_c = 1 + 0.4\dfrac{D}{B}\ for\ \ D \le B$ $d_c = 1 + 0.4\ \tan^{-1}\dfrac{D}{B}\ for\ D > B$	$i_q = \left[1 - \dfrac{0.5H_i}{v + cA'\cot\phi}\right]^5$
$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\dfrac{D}{B}\quad for\, D \le B$ $d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\tan^{-1}\dfrac{D}{B}\qquad for\, D > B$	$i_\gamma = \left[1 - \dfrac{0.7H_i}{v + cA'\cot\phi}\right]^5$ for horizontal ground $i_\gamma = \left[1 - \dfrac{\left(0.7 - \dfrac{\eta^0}{450^0}\right)H_i}{v + cA'\cot\phi}\right]^5$ for sloping ground
$d_\gamma = 1.0\, for\, all\, \phi$ Note: primed factors are for undrained conditions and $\phi = 0$	$H_i = H_B$ if H is parallel to B $H_i = H_L$ if H is parallel to L $H_i = \sqrt{(H_B{}^2 + H_L{}^2)}$ if H is parallel to B& L $i_{cB}, i_{qB}, i_{\gamma B}$ are inclination factors when H is parallel to B $i_{cL}, i_{qL}, i_{\gamma L}$ are inclination factors when H is parallel to L

2.3.5. ANALYSIS ACCORDING TO VESIC

Vesic's bearing capacity equation is similar to Hansen's equation (Eq. 2.21a). The essential differences in Vesic's and Hansens' procedures are: (I) The use of slightly different value of N_γ and (II) a variation on some of Hansen's inclination, base and ground slope factors. The values of these factors are given in Table 2.9 and Table 2.10.

Vesic's Bearing capacity factors are given by the following equations:

$$q_u = cN_c s_c d_c i_c g_c b_c + \sigma_0 N_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma B N_\gamma s_y d_y i_y g_y b_y \qquad (2.21a)$$

$$N_q = \tan^2\left(45^o + \frac{\phi}{2}\right)e^{\pi \tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = 2(N_q + 1)\tan\phi \qquad (2.25)$$

Thus, the values of N_q and N_c are the same as in Hansen's analysis while the values of N_γ are different.

Table 2.8 VESIC'S bearing capacity factors [1]

ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0
5	6.49	1.6	0.4
10	8.34	2.5	1.2
15	10.98	3.9	2.6
20	14.83	6.4	5.4
25	20.72	10.7	10.9
26	22.25	11.9	12.5
28	25.80	14.7	16.7
30	30.14	18.4	22.4
32	35.49	23.2	30.3
34	42.16	29.4	41.0
36	50.59	37.8	56.2
38	61.35	48.9	77.9
40	75.31	64.2	109.3
45	133.87	134.9	271.3

50	266.38	319.1	761.3

TABLE 2.9. VESIC'S DEPTH FACTORS AND INCLINATION FACTORS[8]

Depth factors	Inclination factors
$d'_c = 0.4\frac{D}{B}$ for $D \le B$ $d'_c = 0.4\tan^{-1}\frac{D}{B}$ for $D > B$	$i'_c = 0.5 - 0.5\sqrt{1 - \frac{mH}{c\,A'N_c}}$ $i_c = i_q - \frac{1 - i_q}{N_q - 1}$
$d_c = 1 + 0.4\frac{D}{B}$ for $D \le B$ $d_c = 1 + 0.4\tan^{-1}\frac{D}{B}$ for $D > B$	$i_q = \left[1 - \frac{H}{v + cA'\cot\phi}\right]^m \succ 0$
$d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\frac{D}{B}$ for $D \le B$ $d_q = 1 + 2\tan\phi(1 - \sin\phi)^2\tan^{-1}\frac{D}{B}$ for $D > B$ $d'_q = 1.0$	$i_\gamma = \left[1 - \frac{H}{v + cA'\cot\phi}\right]^{m+1} \succ 0$
$d_\gamma = 1.0$ for all ϕ (All the prime factors are factors for ϕ=0)	$m = \left[\frac{2+R}{1+R}\right]$ Where: $R = \frac{B}{L}$ for H parallel to B $R = \frac{L}{B}$ for H parallel to L $m = \sqrt{\left(\left(\frac{B}{L}\right)^2 + \left(\frac{L}{B}\right)^2\right)}$ for H parallel to B &L

TABLE 2.10. VESIC'S SHAPE FACTORS, GROUND FACTORS AND BASE FACTORS[8]

Shape factors (s)	Ground Factors (g)	Base factors (b)
$s_c^{'} = 0.2\frac{B'}{L'}$ $s_c = 1 + \frac{N_q}{N_c}\frac{B'}{L'}$ Sc for strip =1	$g_c^{'} = 1 - \beta^0 \ /147^0$ $g_c = g_q - ((1 - g_q)/147^0)$	$b_c^{'} = 1 - \eta^0 /147^0$
$s'_q = 1$ $s_q = 1 + \frac{B'}{L'}\tan\phi$ Sq for strip =1	$g_q = g_\gamma = (1 - \tan\beta)^2$	$b_c = b_q - ((1 - b_q)/147^0)$
$s'_\gamma = 1$ $s_\gamma = 1 - 0.4\frac{B'}{L'}$ Sγ for strip =1		$b_q = b_\gamma = (1 - 0.017\eta\tan\phi)^2$

2.3.6. ANALYSIS ACCORDING TO THE ETHIOPIAN BUILDING CODE STANDARD (EBCS-7)

The basis of the Ethiopian code is DIN-4017, which is the German code, with slight differences.

A) **Undrained Conditions:** the design resistance is calculated from:

$$Q_{ult} = (2 + \pi)c_u s_c i_c + q \qquad (2.26)$$

With the design values of dimensionless factors for:

I. The shape of the foundation:

- For a rectangular shape

$$s_c = 1 + 0.2\left(\frac{B'}{L'}\right) \qquad (2.27\,a)$$

- For a square or circular shape

$$s_c = 1.2 \qquad (2.27b)$$

II. The inclination of the load, due to by a horizontal load H:

$$i_c = 0.5\left(1+\sqrt{\left(1-\frac{H}{A'C_u}\right)}\right) \qquad (2.27c)$$

B) **Drained Conditions**: The design resistance is calculated from:

$$Q_{ult} = c' N_c s_c i_c + q' N_q s_q i_q + 0.5B\gamma' N_\gamma i_\gamma s_\gamma \qquad (2.28)$$

With the design values of dimensionless factors for the bearing resistance:

$$N_q = \tan^2\left(45^0 + \frac{\phi'}{2}\right)\left(e^{\pi \tan\phi}\right) \qquad (2.29a)$$

$$N_c = \left(N_q - 1\right)\cot\phi' \qquad (2.29b)$$

$$N_\gamma = 2\left(N_q - 1\right)\tan\phi' \qquad (2.29c)$$

I. The shape of foundation:

- For a rectangular shape;

$$s_q = 1 + \left(\frac{B'}{L'}\right) Sin\ \phi' \qquad (2.29d)$$

$$s_\gamma = 1 - 0.3\left(\frac{B'}{L'}\right) \qquad (2.29e)$$

- For square or circular shape

$$s_q = 1 + \sin\ \phi' \qquad (2.29f)$$

$$s_\gamma = 0.7 \qquad (2.29g)$$

- For rectangular, square or circular shape

$$s_c = \frac{\left(s_q N_q - 1\right)}{\left(N_q - 1\right)} \qquad (2.29h)$$

II. The inclination of the load, due to a horizontal load H parallel to L' (the longer side):

$$i_q = i_\gamma = 1 - \frac{H}{\left(V + A'c\cot\phi'\right)}$$

$$i_c = \frac{(i_q N_q - 1)}{(N_q - 1)}$$

III. The inclination of the load, due to a horizontal load H parallel to B' (the shorter side):

$$i_q = \{\ 1 - 0.7H / (V + A'c'\cot\phi')\ \}^3$$

$$i_\gamma = \{\ 1 - H / (V + A'c'\cot\phi')\ \}^3$$

$$i_c = (i_q N_q - 1) / (N_q - 1)$$

The additional influences of embedment depth, inclination of the base of the foundation and of the ground surface should also be considered.

2.3.7. EFFECT OF WATER TABLE ON BEARING CAPACITY [20]

The soil's unit weight used in the second and third term (the γ in N_q and N_γ terms) of the bearing capacity equations presented in the preceding sections are the effective unit weights. Of course, if dry subsoil becomes saturated with a rising of the water table, the unit weight of the submerged soil is reduced to perhaps half the weight for that soil of the water table; obviously, we have to account for the buoyant effect of the water. A reduction in the unit weight results in a decrease in the ultimate bearing capacity of the soil. When the water level is at a distance of B or below the below the bottom of the footing, no adjustment in the γ value is needed; the γ in the second and third terms of the bearing capacity equations is merely the unit weight of the soil. However, adjustment is recommended when the water level ranges between the ground surface and a distance B below the base of the footing.

For any position of the water table, Eq. 2.4, may be modified as under: [17]

$$q_u = cN_c + \gamma_1 D N_q R_{w1} + \frac{1}{2}\gamma_2 B N_\gamma . R_{w2} \qquad (2.30)$$

Where R_{w1} and R_{w2} are the reduction factors for water table, computed as follows:

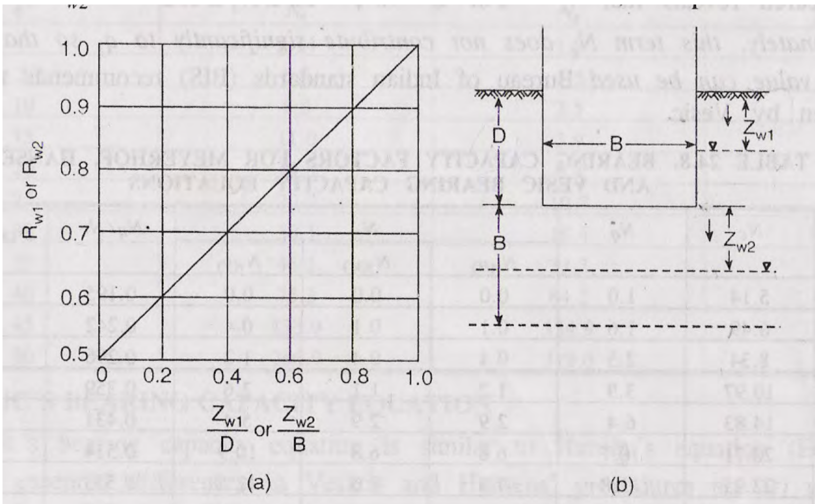

Fig. 2.11 Water reduction factors [17]

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) \qquad (2.31a)$$

$at \quad Z_{w1} = 0,\ R_{w1} = 1/2;\ At Z_{w1} = D,\ R_{w1} = 1$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) \qquad (2.31b)$$

$at \quad Z_{w2} = 0,\ R_{w2} = 0.5;\ At\ Z_{w2} = B,\ R_{w2} = 1$

In Eq. 2.30: γ_1 = average unit weight of the surcharge soil situated above the water table.

γ_2 = average unit weight of the soil in the wedge zone, situated with in a depth B below the base of the footing.

When the water table is just at the base of the footing, $\gamma_2 = \gamma_{sat}$

When the water table is at the ground surface, both γ_1 and γ_2 is the saturated weights. For intermediate locations of water table, γ_1 and γ_2 can be computed, using the above methods.

2.3.8 COMPARISON OF BEARING CAPACITY FACTORS

The values of bearing capacity factors N_c, N_q and N_γ, originally given by Terzaghi have now been superseded by the analysis given by other workers such as Hansen, Meyerhof, Vesic etc. The following are commonly used expressions:

(I) **Expressions for** N_c *and* N_q: $N_c = (N_q - 1)\cot\phi$ (2.25 a)

$$N_q = \tan^2\left(45^0 + \frac{\phi}{2}\right)e^{\pi \tan\phi} \quad (2.25b)$$

When: $\phi = 0,\ N_c = 5.14$ (*instead of* 5.7 *given by Terzaghi*)

The above two expressions have been the same for (a) Terzaghi, (b) Meyerhof, (c) Hansen,(d) Vesic and (e)EBCS-7

(II) **Expressions for** N_γ

The expression for N_γ as given by different authors are given below

(a) Meyerhof: $N_\gamma = (N_q - 1)\tan(1.4\phi)$

(b) Hansen: $N_\gamma = 1.5(N_q - 1)\tan\phi$

(c) Vesic: $N_\gamma = 2(N_q + 1)\tan\phi$

(d) EBCS-7: $N_\gamma = 2(N_q - 1)\tan\phi'$

The N_γ factor has the widest suggested range of values of any of the factors. A closer look reveals that for example for $\phi = 40^0$ values of N_γ will found in a range of 79.7 up to 109.3

The following Table (Table 2.11) gives the values of bearing capacity factors for Meyerhof ($N_{\gamma(M)}$), Hansen ($N_{\gamma(H)}$),Vesic ($N_{\gamma(V)}$)and EBCS-7($N_{\gamma(EBCS)}$). It is to be noted that values of N_c and N_q are the same for all the four methods

Table 2.11 Bearing Capacity Factors According To Meyerhof, Hansen Vesic and EBCS-7

ϕ^0	N_c	N_q	N_γ			
			$N_{\gamma(H)}$	$N_{\gamma(M)}$	$N_{\gamma(V)}$	$N_{\gamma(EBCS)}$
0	5.14	1.0	0.0	0.0	0.0	0.0
5	6.49	1.6	0.1	0.1	0.4	0.1
10	8.34	2.5	0.4	0.4	1.2	0.5
15	10.97	3.9	1.2	1.1	2.6	1.6
20	14.83	6.4	2.9	2.9	5.4	3.9
25	20.71	10.7	6.8	6.8	10.9	9.0
26	22.25	11.8	7.9	8.0	12.5	10.5
28	25.79	14.7	10.9	11.2	16.7	14.6
30	30.13	18.4	15.1	15.7	22.4	20.1
32	35.47	23.2	20.8	22.0	30.2	27.7
34	42.14	29.4	28.7	31.1	41.0	38.3
36	50.55	37.7	40.0	44.4	56.2	53.3
38	61.31	48.9	56.1	64.0	77.9	74.8
40	75.25	64.1	79.7	93.6	109.3	105.9
45	133.73	134.7	200.5	262.3	271.3	267.4
50	266.50	318.5	567.4	871.7	761.3	756.8

(III) WHICH EQUATION TO USE

The Terzaghi equations, being the first proposed, have been very widely used, because of their ease of use they are still used, probably more than they should be. They are only suitable for a centrically loaded footing on a horizontal ground. They are not applicable for footings with horizontal shear and /or a moment or tilted bases.

Both the Meyerhof and Hansen methods are widely used. The Vesic's method has not been much used. As previously noted there is very little difference between the Hansen and Vesic's methods, as illustrated by the computed q_u values.

From these observations, one may suggest the following:

Table 2.12 THE COMPUTED q_u VALUES

Use	Applicability
Terzaghi	-For cohesive soils, where $\frac{D}{B} \leq 1$, or for a quick estimation of q_u.
Meyerhof, EBCS-7	-For all soil types, depending on use preference or familiarity with a particular method
Hansen, Vesic	- For all soil types and when base is tilted, footing on a slope or when $\frac{D}{B} \geq 1$.

It is good practice to use at least two methods other than Terzaghi and compare the computed value of q_u. If the two values do not compare well, use a third method.

2.3.9. BEARING CAPACITY OF LAYERED SOILS

All the above methods are applicable for uniform soils. Footings may be placed on stratified deposits. The usual methods of determining bearing capacity are not applicable. The bearing capacity becomes dependent on the extent of the rupture surface in weaker or stronger material. One may, however determine the bearing capacity of the ground approximately. The following methods are used for the analysis of multiple layers as indicated in Fig 2.12.

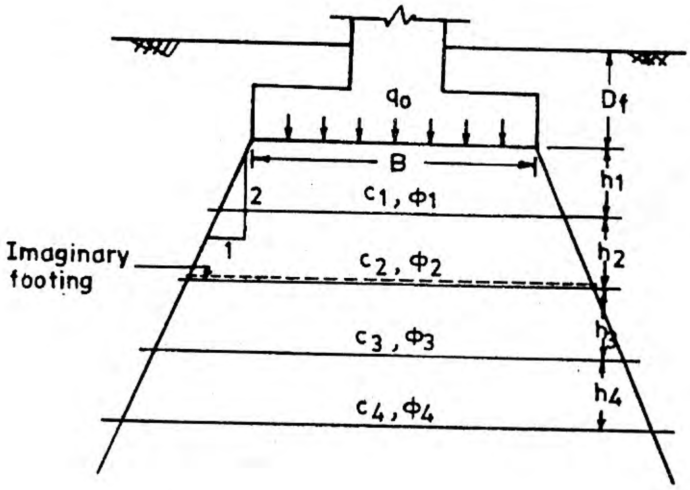

Fig.2.12 Footing on Layered Soils [10]

(I) First Method

Consider the different layers of soil within effective shear depth, which is approximately equal to $\left(0.5B\tan\left(45^0 + \frac{\phi}{2}\right)\right)$. If the thickness of the first layer below the base of the footing is more than the significant shear depth, analysis of single layer holds good. [10]

(II) Second Method

Weighted average values of c and ϕ are obtained as

$$c_{av} = \frac{c_1 h_1 + c_2 h_2 + ... + c_n h_n}{\sum \left[h_1 + h_2 + h_3 + ... + h_n\right]} \qquad (2.33)$$

$$\phi_{av} = \tan^{-1} \frac{h_1 \tan \phi_1 + h_2 \tan \phi_2 + ... + h_n \tan \phi_n}{\sum \left[h_1 + h_2 + h_3 + ... + h_n\right]} \qquad (2.34)$$

The bearing capacity of the footing is then determined with average shear strength parameter c_{av} and ϕ_{av} using any of the methods for uniform soil.

(III).Third Method

This is a method proposed by DIN 4017 for stratified soils. Here one assumes an internal angel of friction ϕ_i, or take the value of the angel of internal friction, ϕ_1 of the first layer under the foundation. One then establishes the rupture line as indicated in Fig 2.13. From the rapture line one calculates the weighted average of the angel of internal friction, ϕ_m and that of the cohesion c_m. If the difference between the internal ϕ_i and that of ϕ_m is bigger than 3%, one alters the magnitude of the angel of internal friction, which would consequently give a different rupture line and repeats the calculation until the difference between altered and calculated is less than 3%. It should be noted that this method is permissible only if the difference between the mean value of the angel of internal friction of the whole layer ϕ_{av} and the respective value of the angel of internal friction of each layer is not greater than 5%.

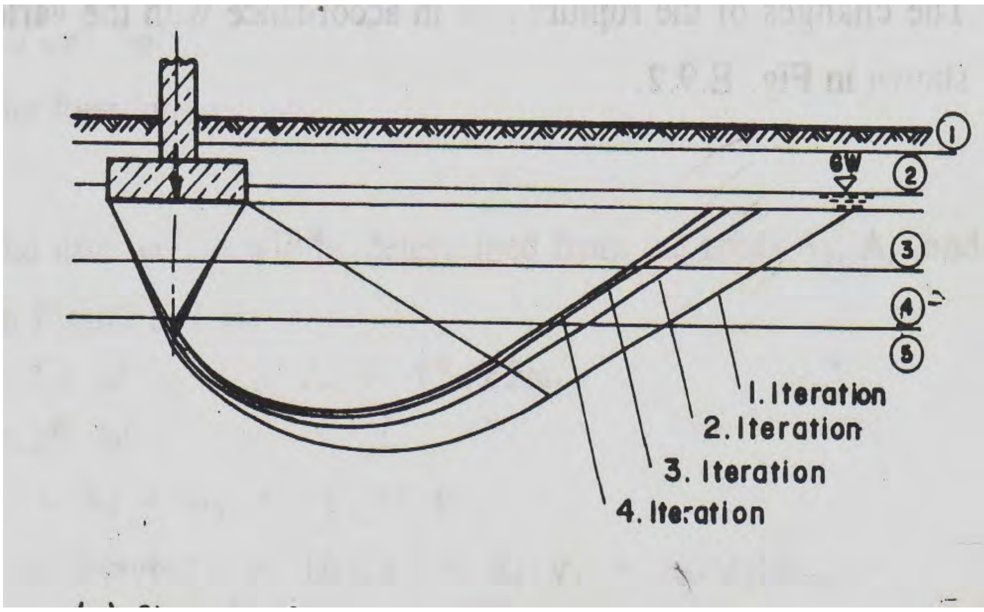

Fig.2.13. Changes of the rupture lines corresponding to the Iteration [3]

CHAPTER THREE

3.1. SOFTWARE DEVELOPMENT

3.1.1. GENERAL

The Software developed in this work is used to determine the bearing capacity of the soil using different methods that have been proposed by different authors.

A user friendly Software is developed using Visual Basic 6.0 development tool.

3.1.2. PROGRAM ALGORITHM

In writing the code, the Select Case …. End Select Control Structure is used.

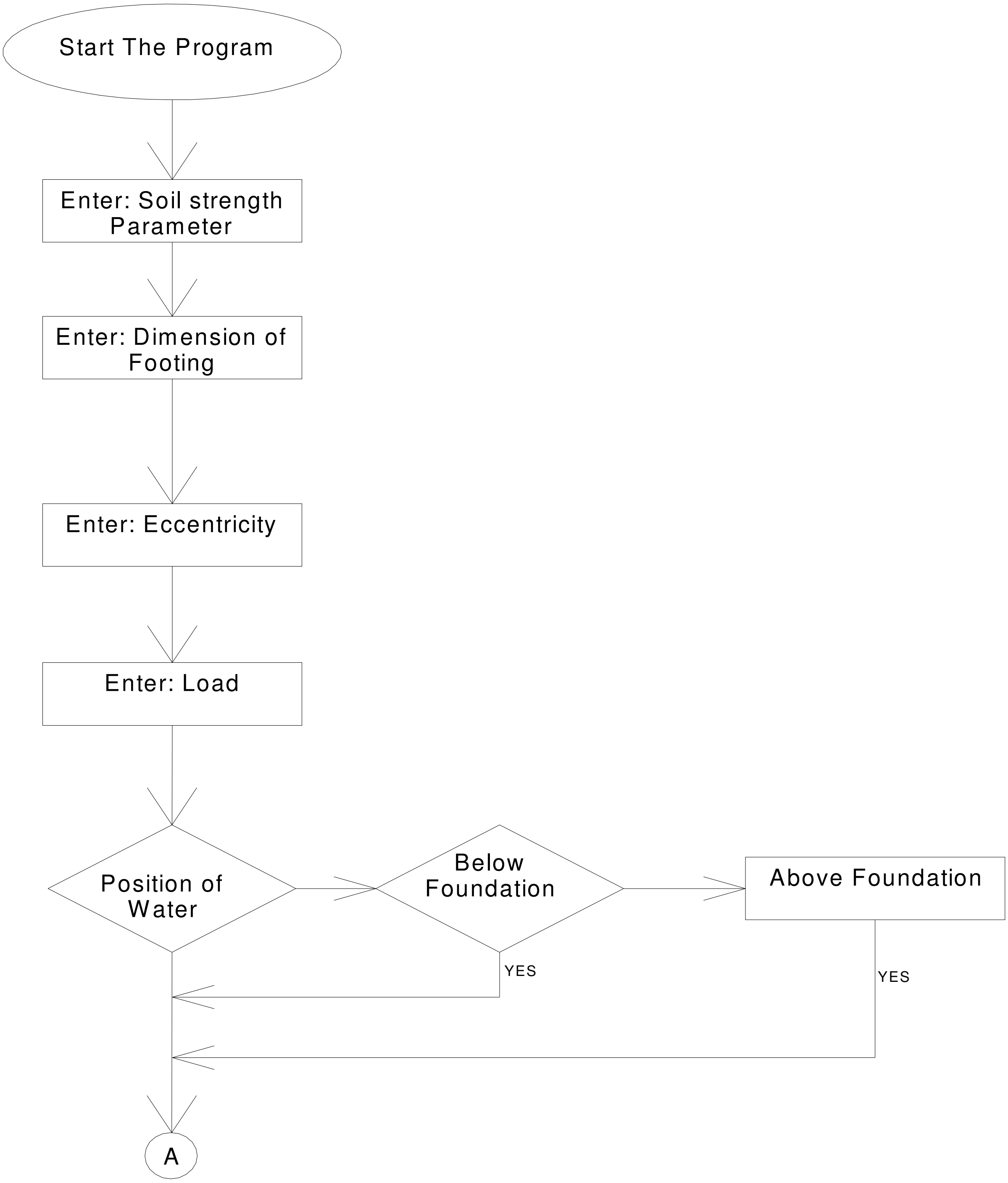

Fig.3.1 Program Flow chart

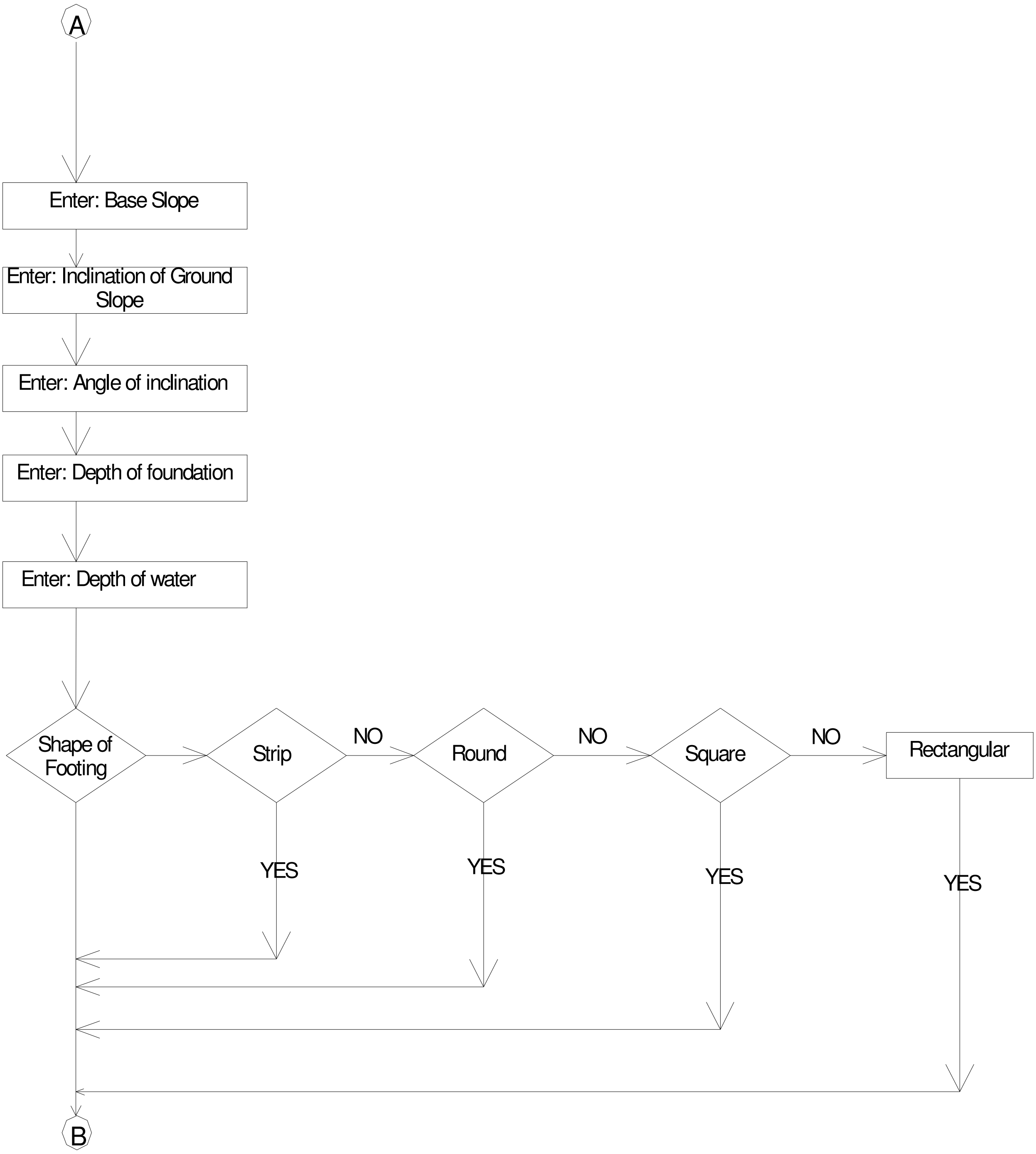

Fig.3.1 Program Flow chart Continuation

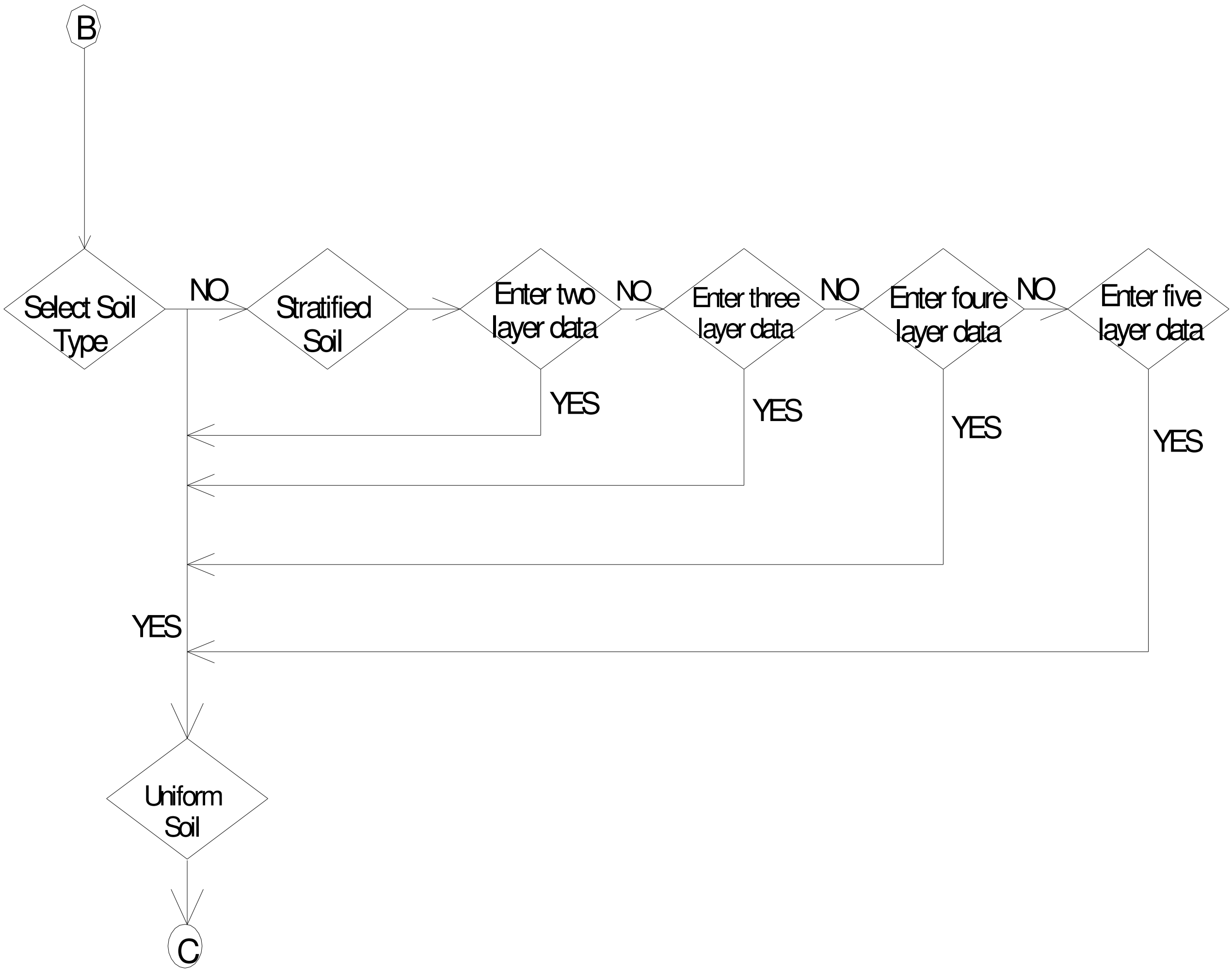

Fig.3.1 Program Flow chart Continuation

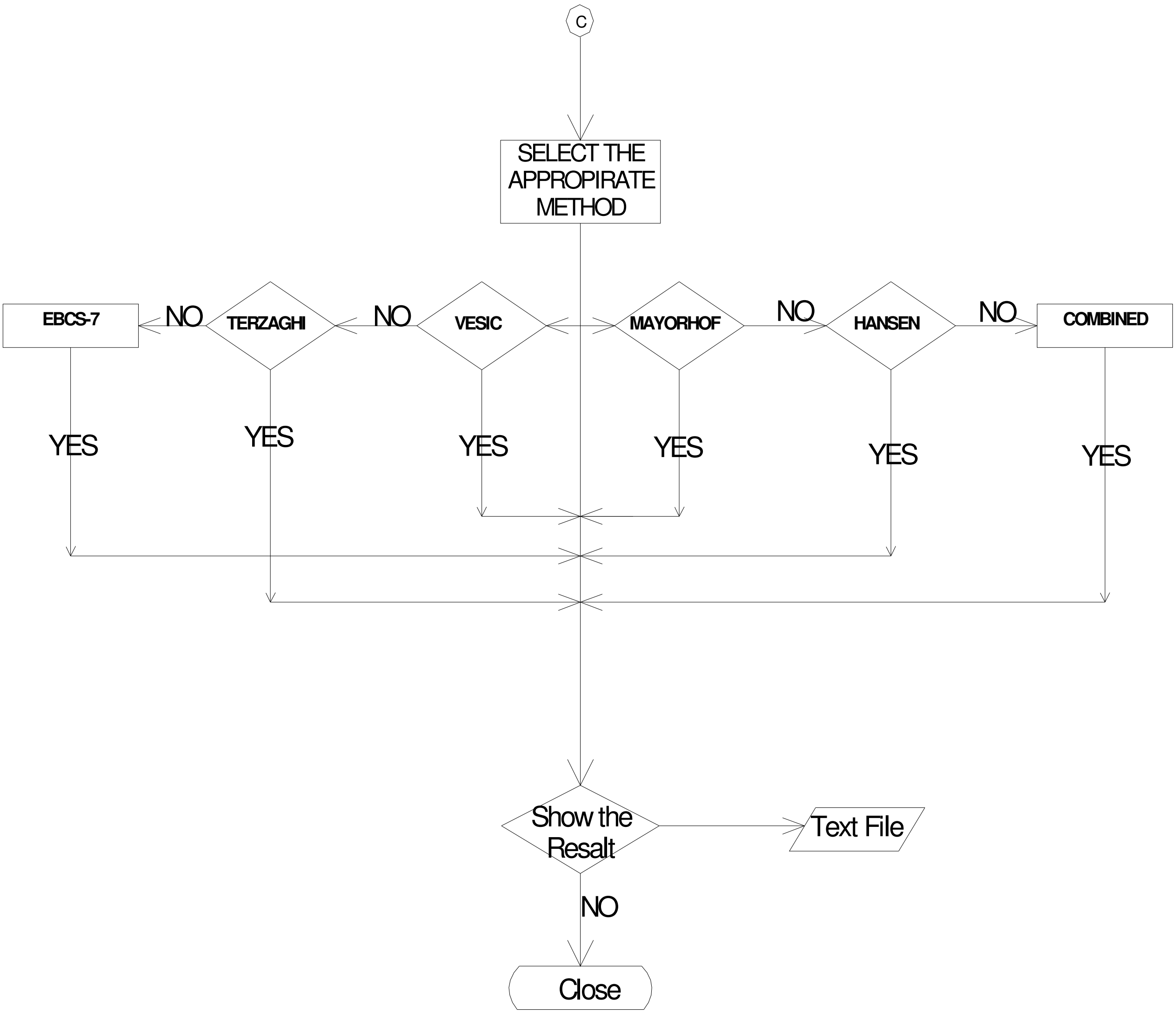

Fig.3.1 Program Flow chart Continuation

3.1.3. ENTERING THE CODES AND DEVELOPING THE APPLICATION

In the development of the Software, programming codes are written for five inbuilt Command Button Controls (i.e. corresponding to the five methods of analysis) as shown in Appendix 1. Then a Text Box Control is used to fill input values for soil strength parameter, eccentricity, load, ground slope, inclination of footing base slope, the loading condition, the depth of water table, the shape and the depth of foundation, which are declared as a Single variable.

After the users enters all the Input data in the **Text Box**, selects the appropriate **option button**, and clicks the **calculate button**, the result will be visible when the user clicks **show the output button**. In addition to the bearing capacity, the appropriate factors (Bearing capacity factors, shape factors, load inclination factors, ground and base inclination factors, and depth of the foundation factors) are the program output.

This program is developed using the Microsoft Package and Development wizard, which can be readily installed in any personal computer. The name of the developed application Software is 'BEARING'. The user enter face of the software is given in Fig. 3.2 up to Fig. 3.7 shown below.

Here two option are considered namely Uniform and stratified soil layer.

For the case of stratified soil layer (up to five layers), a secondary interface (Fig. 3.7) will be appear when stratified soil type option pressed. To generate the weighted average soil strength parameters c and ϕ internally after filling the required input data, two methods (options) are available as well. These options are DIN-4017 and Weighted average method.

In the case of DIN-4017, an assumed rupture surface, as indicated in the figure below, is considered.   An iteration process using different rupture surface is undertaken until the final soil parameters are obtained. Using these soil parameters, the bearing capacity is calculated using the bearing capacity formulas for uniform soils.

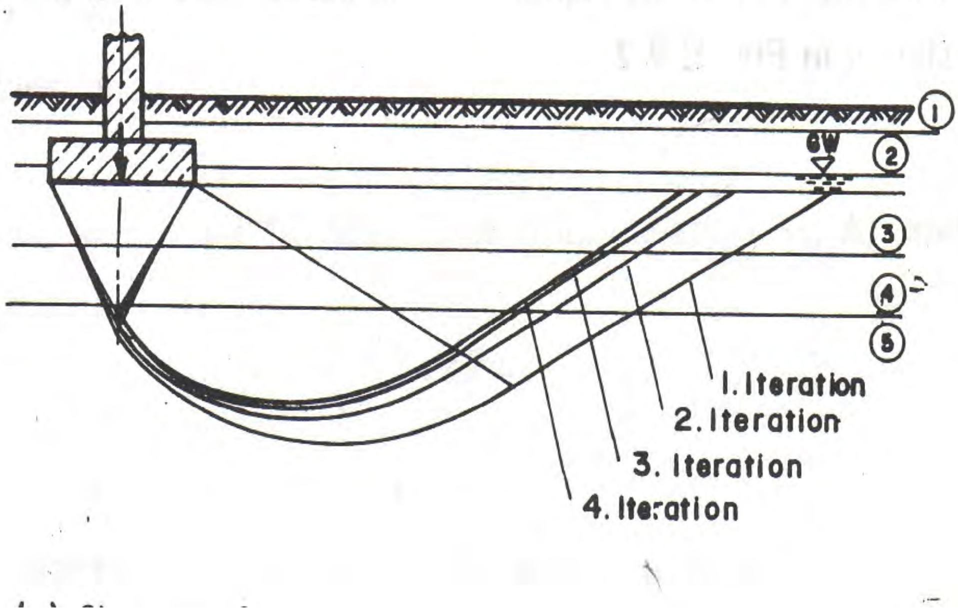

Fig.3.2 Changes of the rupture lines corresponding to the Iteration

In the case of weighted average, the average soil parameters are calculated by using the soil parameters of the given layers and their respective depths, as formulated below:

$$c_{av} = \frac{c_1 h_1 + c_2 h_2 + \ldots + c_n h_n}{\sum \left[h_1 + h_2 + h_3 + \ldots + h_n \right]}$$

$$\phi_{av} = \tan^{-1} \frac{h_1 \tan \phi_1 + h_2 \tan \phi_2 + \ldots + h_n \tan \phi_n}{\sum \left[h_1 + h_2 + h_3 + \ldots + h_n \right]}$$

The average values thus obtained are generated (internally) to calculate the bearing capacity, when the user click Ok button. Then after, the bearing capacity is calculated according the bearing capacity formulas for uniform soils.

3.1.4. SAMPLE DETERMINATION OF BEARING CAPACITY OF SOIL

1) WHEN THE USER selects EBCS-7 Analysis option

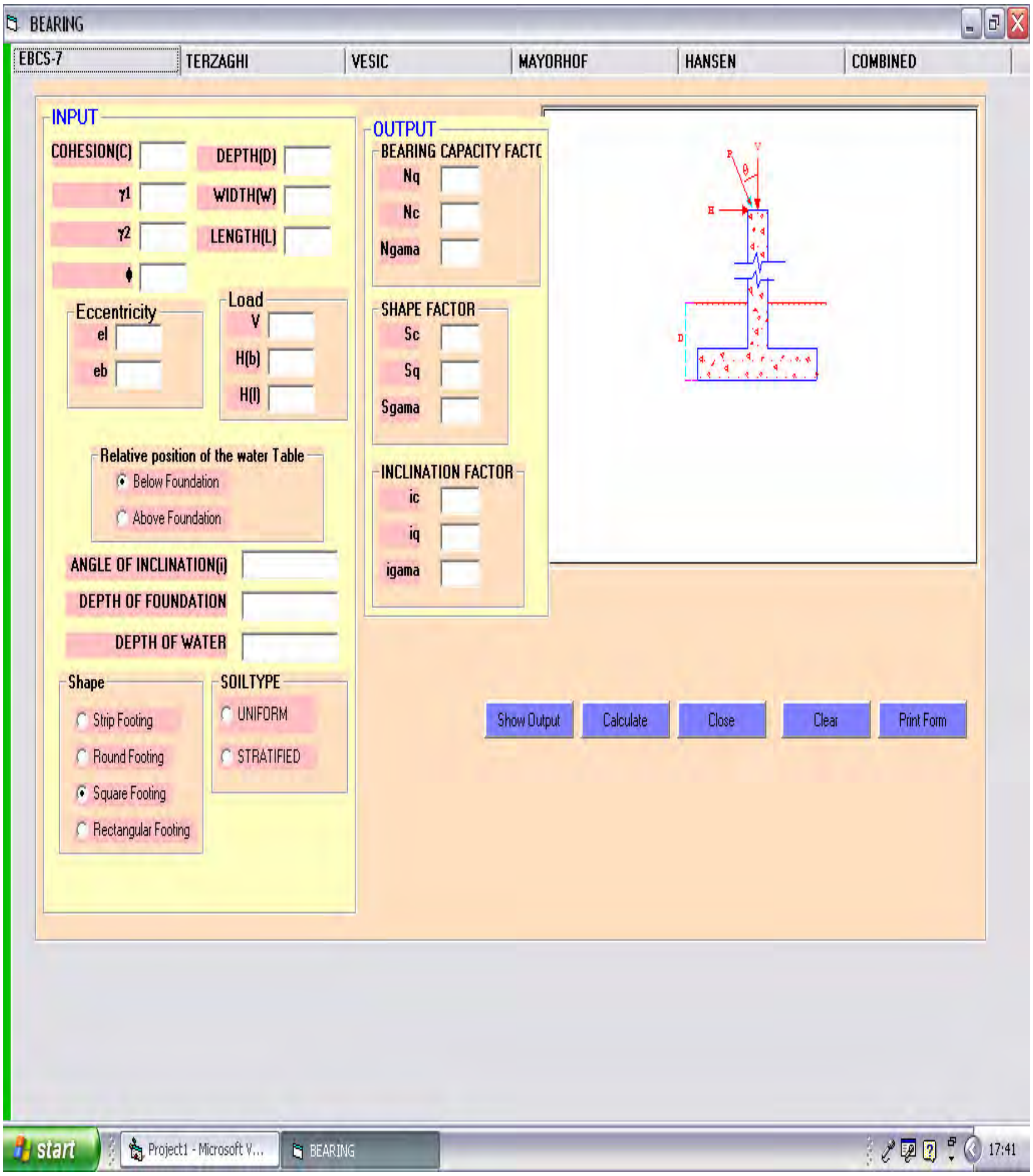

Fig.3.3 EBCS-7 Analysis option interface

2) WHEN THE USER selects TERZAGHI Analysis option

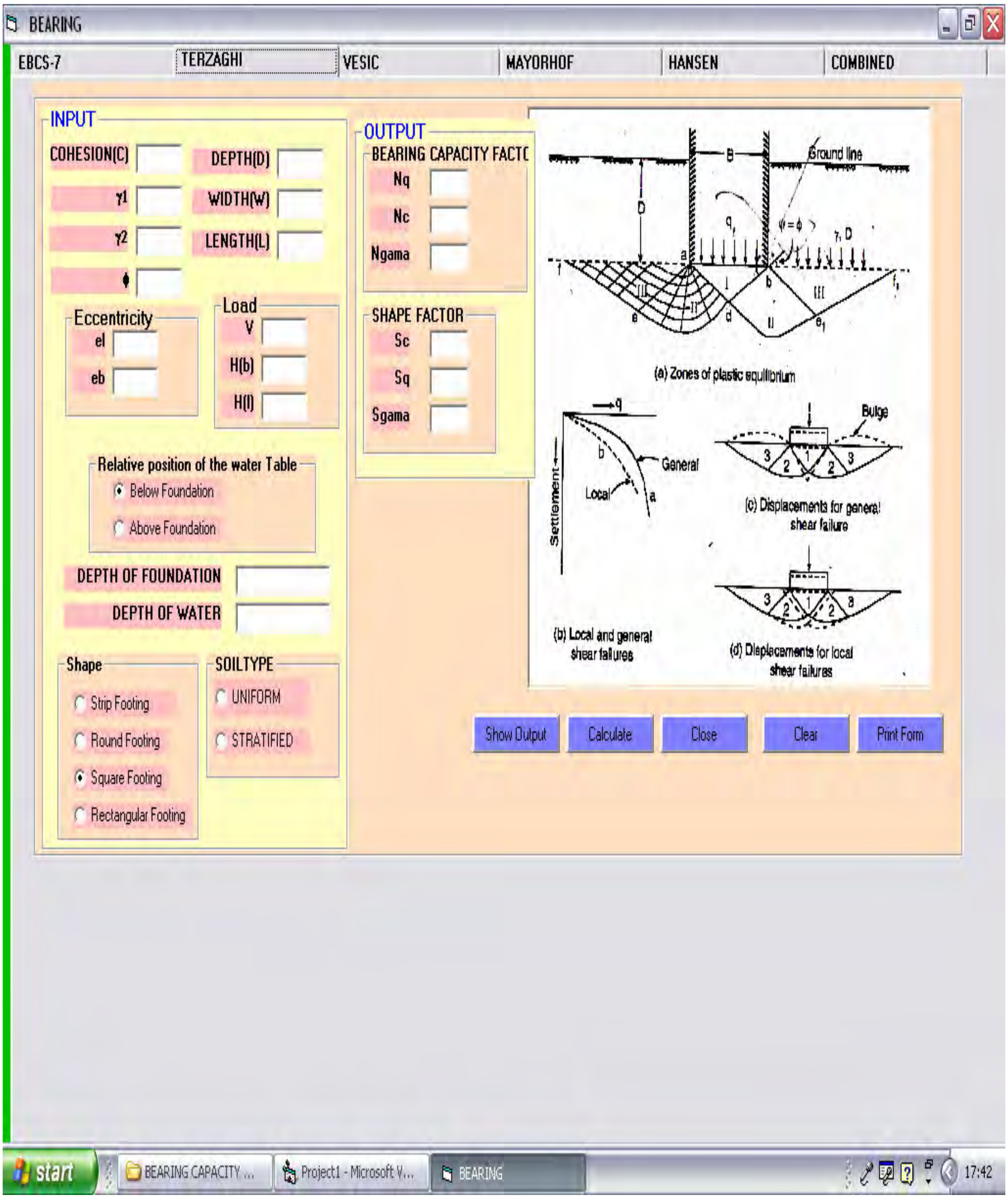

Fig.3.4 TERZAGHI Analysis option interface

3) WHEN THE USER selects VESIC Analysis option

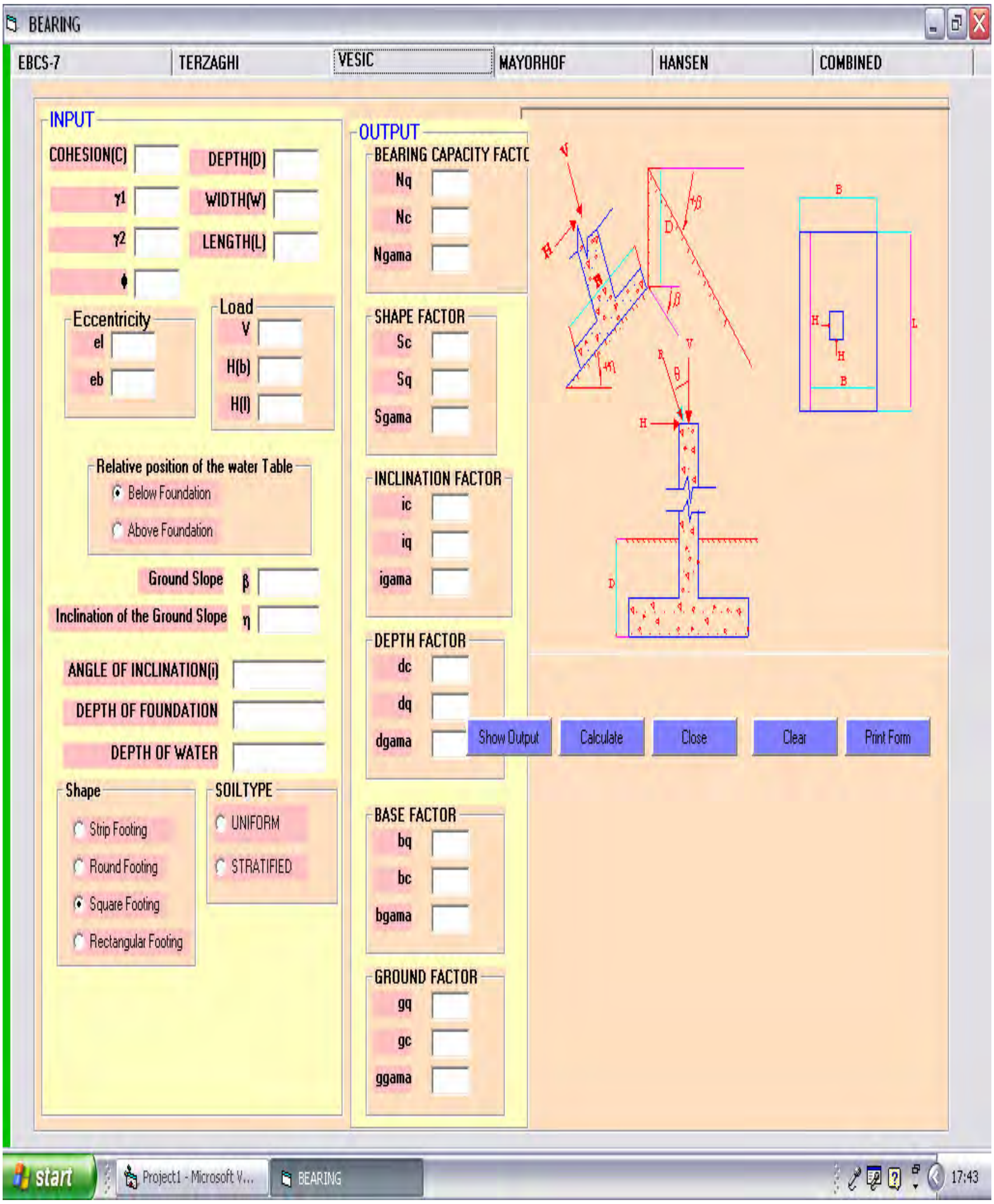

Fig.3.5 VESIC Analysis option interface

4) WHEN THE USER selects MEYERHOF Analysis option

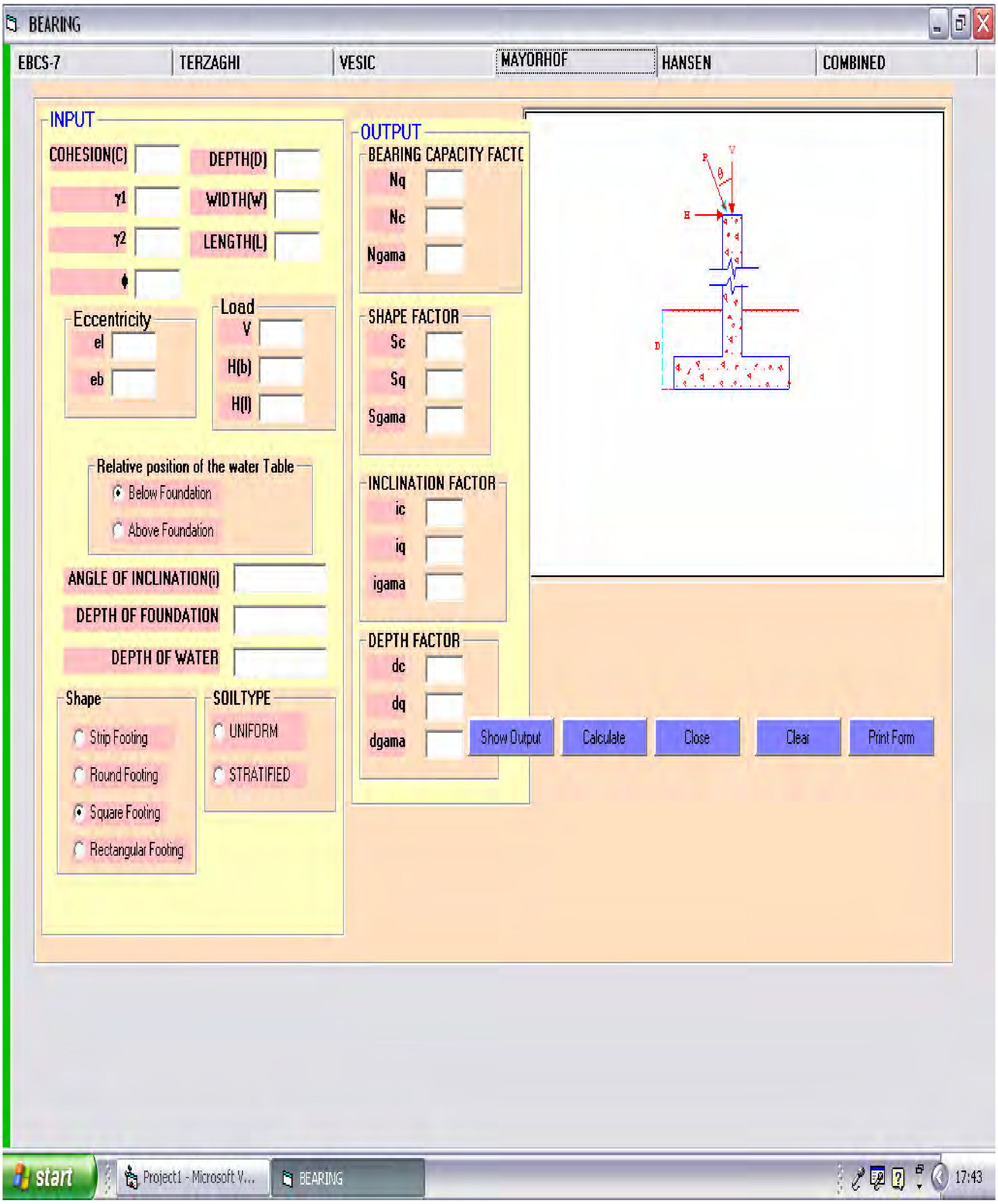

Fig.3.6 MEYERHOF Analysis option interface

5) WHEN THE USER selects HANSEN Analysis option

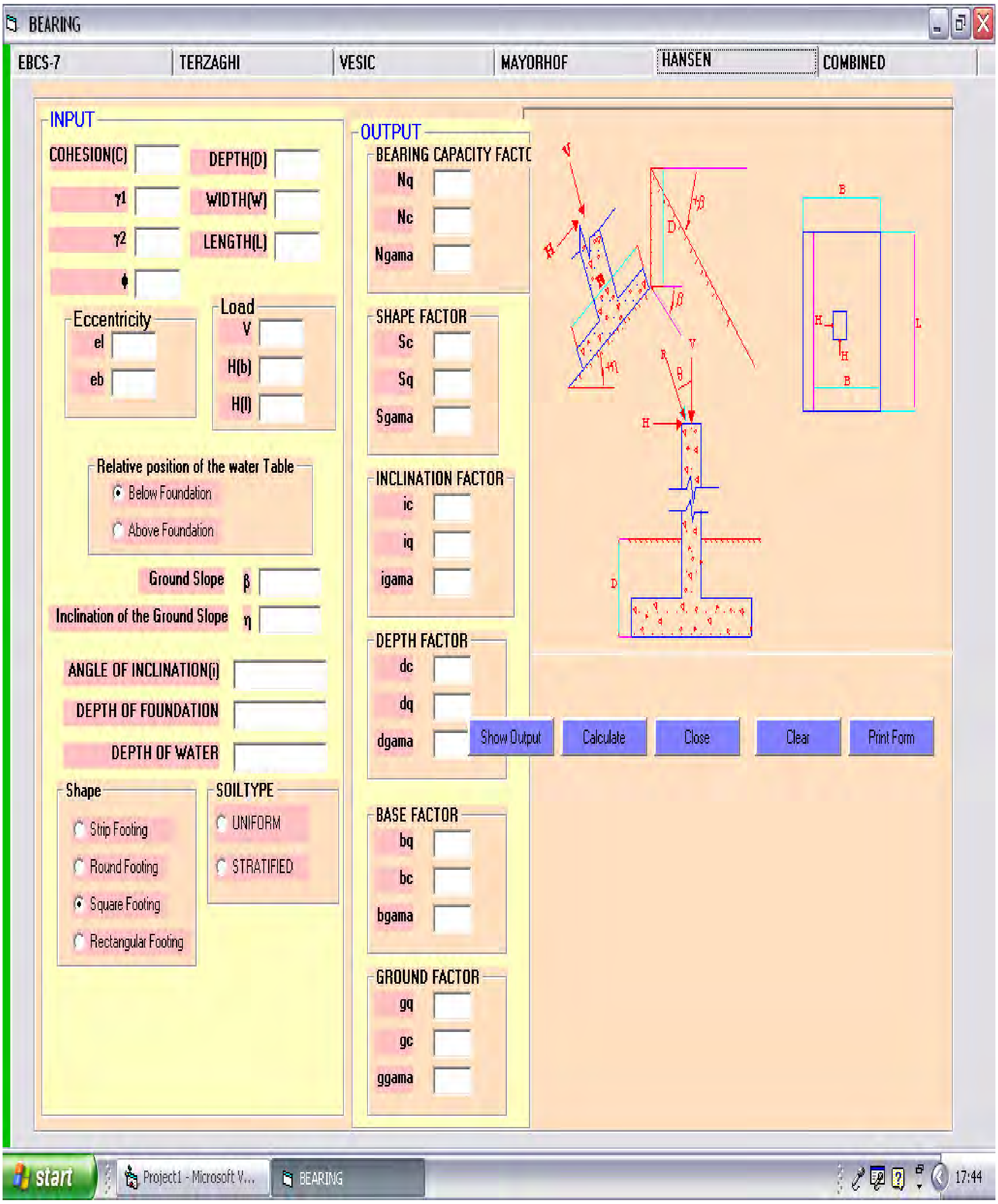

Fig.3.7 HANSEN Analysis option interface

6) WHEN THE USER selects COMBINED Analysis option

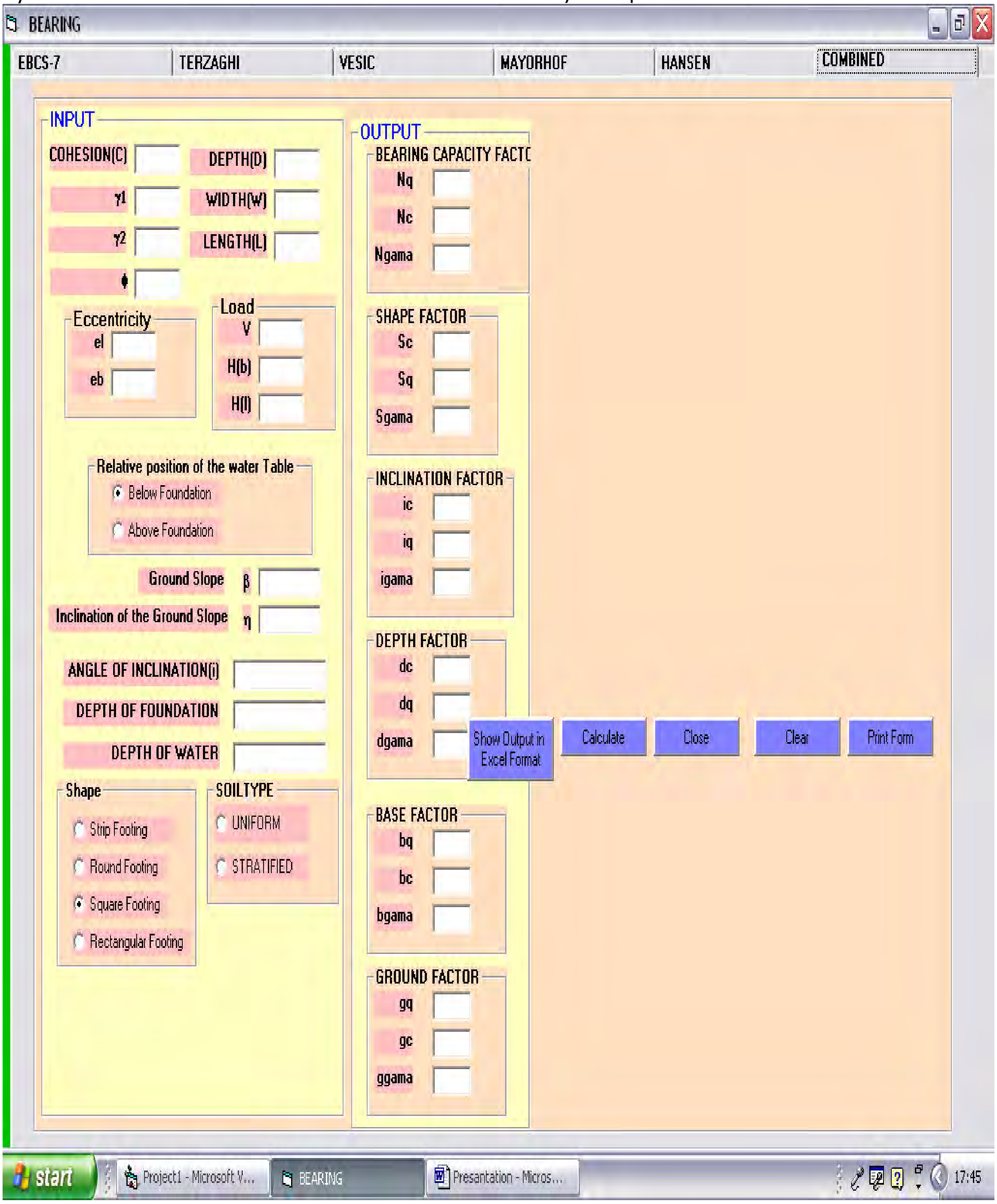

Fig.3.8 COMBINED Analysis option interface

7) WHEN THE USER selects LAYERED SOIL STRATA Analysis option

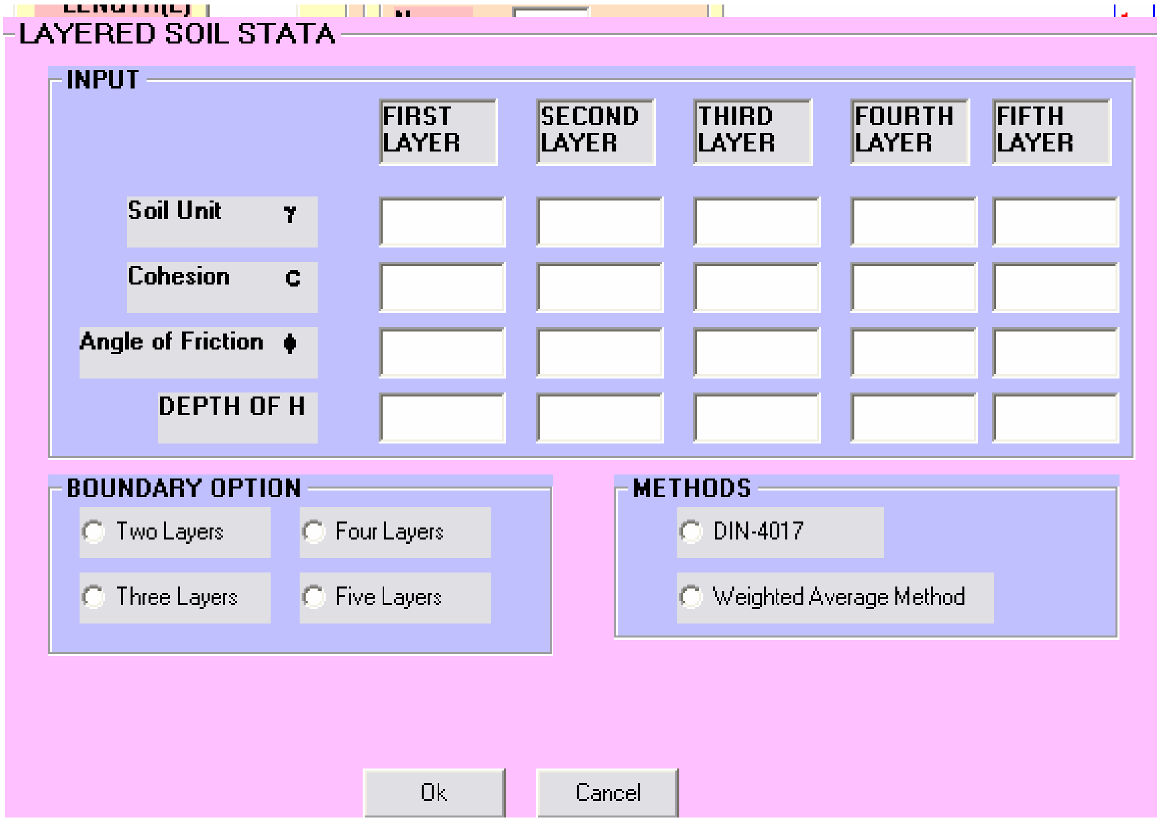

Fig.3.9 Stratified soil type analysis option interface

CHAPTER FOUR

4. CONCLUSION AND RECOMMENDATION

Within the framework of this research, the following conclusions and recommendations are arrived at:

4.1. CONCLUSION

1. To facilitate the calculations of bearing capacity, software is developed for uniform and stratified soils (up to five stratification). This is a useful tool for the practice.
2. The comparison between long hand calculation and the developed program output are presented.

4.2. RECOMMENDATION

1. For the case of stratified soil, centrically loaded foundations are considered. Eccentric loads are omitted due to time limitation. It is therefore recommended to developed a program for eccentric loading as well.
2. The software that has been developed calculates bearing capacity. The effect of settlement has not been considered. Since settlement plays an important role in foundation designs. It would be appropriate to incorporate the effect of settlement in the program.
3. The Bearing capacities are calculated only for static loading. In-order to make the method comprehensive the dynamic effects on bearing capacity should be examined.

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APPENDIX

APPENDIX-1

Examples 1 (Uniform Soil)

A strip footing 2m wide carries a load intensity of 400 $\frac{KN}{m^2}$ at a depth of 1.2m in sand. The saturated unit weight of sand is $19.5\frac{KN}{m^3}$ and unit weight above water table is 16.8 $\frac{KN}{m^3}$. The shear strength parameters are c=0 and $\phi = 35^0$. Determine the ultimate Bearing capacity of the soil with respect to the following case of location of water table is at 4m, 1.2m, 2.5m, 0.5m below the ground level and at the ground level itself

Solution:-

1.1. Long Hand Calculations

1.1.1. Method Of Hansen

$$q_{ult} = cN_c S_c d_c i_c g_c b_c + \overline{\sigma}_o N_q S_q d_q i_q g_q b_q R_{w1} + \frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma R_{w2}$$

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right)\times e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{35^0}{2}\right)\times e^{\pi \tan 35^0} = 33.3$
- $N_c = (Nq - 1)\times \cot\phi = (33.3 - 1)\times \cot 35^0 = 46.13$
- $N_\gamma = 1.5\ (Nq - 1)\times \tan\phi = 1.5\times(33.3 - 1)\times \tan 35^0 = 33.93$

b) Shape Factor

- $S_c = 1 + \left(\frac{Nq}{Nc}\right)\times\left(\frac{B'}{L'}\right)$
- $S_q = 1 + \left(\frac{B'}{L'}\right) Sin\phi$
- $S_\gamma = 1 - 0.4\times\left(\frac{B'}{L'}\right)$

But in our case $S_c = S_q = S_\gamma$ = 1.0 since it is strip footing.

c) Depth Factor

- $d_c = 1 + 0.4 \times \left(\frac{D}{B}\right) = 1 + 0.4 \times \left(\frac{1.2}{2}\right) = 1.24$

- $d_q = 1 + 2 \times \tan\phi (1 - Sin\phi)^2 \times \left(\frac{D}{B}\right)$

  $= 1 + 2 \times \tan 35^0 \times (1 - Sin35^0)^2 \times \left(\frac{1.2}{2}\right) = 1.15$

- $d_\gamma = 1$

d) Load, Ground and Base inclination Factor

- All $i_i = g_i = b_i = 1$ (not zero)

$$q_{ult} = cN_c S_c d_c i_c g_c b_c + \overline{\sigma}_o N_q S_q d_q i_q g_q b_q R_{w1} + \frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma R_{w2}$$

$$\therefore q_{ult} = (\gamma_1 \times 1.2 \times 33.3 \times 1 \times 1.15 \times R_{w1}) + (0.5 \times \gamma_2 \times 2 \times 33.93 \times 1 \times R_{w2}) = 45.95\gamma_1 R_{w1} + 33.93\gamma_2 R_{w2}$$

$$\therefore q_{ult} = 45.95\gamma_1 R_{w1} + 33.93\gamma_2 R_{w2}$$

Case 1: When water table is 4m below G.L.

Since Z_{w2} = 4-1.2 =2.8m > B= 2m then $R_{w2} = 1.0$, $R_{w1} = 1.0$ and there will be no effect of water table. $\gamma_1 = \gamma_2 = 16.8\ KN/m^3$

$$\therefore q_{ult} = 45.95 \times 16.8 \times 1 + 33.93 \times 16.8 \times 1 = 1{,}341.98 KPa$$

Case 2: when water table is just at the base of footing

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the surcharge term, use $\gamma_1 = 16.8\ KN/m^3$ because the surcharge soil is situated above water table, for the wedge term use $\gamma_2 = \gamma_{sat} = 19.5\ KN/m^3$ since the wedge soil is situated below water table.

$$\therefore q_{ult} = 45.95 \times 16.8 \times 1 + 33.93 \times 19.5 \times 0.5 = 1{,}102.78 KPa$$

Case 3: when water table is 2.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{1.3}{2}\right) = 0.825$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$, for the wedge term use γ_2 will be taken as average unit weight of soil situated below the footing level, since the soil up to depth B below the footing level is partially above water table and partially below the water table.

$$\gamma_2 = \gamma_{av} = \frac{(16.8\times1.3)+(19.5\times0.7)}{(1.3+0.7)} = 17.75\frac{KN}{m^3}$$

$$\therefore q_{ult} = 45.95\times16.8\times1 + 33.93\times17.75\times0.825 = 1{,}268.82KPa$$

Case 4: when water table is 0.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0.5}{1.2}\right) = 0.708$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the wedge term, use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ and for the surcharge term use γ_1 will be taken as average unit weight of soil situated above the base of footing, since this soil is located partially above the water table and partially below the water.

$$\gamma_1 = \gamma_{av} = \frac{(16.8\times0.5)+(19.5\times0.7)}{(0.5+0.7)} = 18.38\frac{KN}{m^3}$$

$$\therefore q_{ult} = 45.95\times18.38\times0.708 + 33.93\times19.5\times0.5 = 928.77KPa$$

Case 5: when water table is at G.L. itself

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0}{1.2}\right) = 0.5$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the wedge term and the surcharge term use $\gamma_1 = \gamma_2 = 19.5\frac{KN}{m^3}$

$$\therefore q_{ult} = 45.95\times19.5\times0.5 + 33.93\times19.5\times0.5 = 778.83KPa$$

1.1.2. Method Of Vesic

$$q_{ult} = CN_cS_cd_ci_cg_cb_c + \bar{\sigma}_oN_qS_qd_qi_qg_qb_qR_{w1} + \frac{1}{2}\gamma BN_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma R_{w2}$$

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right)e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{35^0}{2}\right)\times e^{\pi \tan 35^0} = 33.3$
- $N_c = (Nq - 1)\cot\phi = (33.3 - 1)\times \cot 35^0 = 46.13$
- $N_\gamma = 2\ (Nq + 1)\tan\phi = 2\times(33.3 + 1)\times \tan 35^0 = 48.03$

b) Shape Factor

- $S_c = S_q = S_\gamma$ = 1.0 since it is strip footing.

c) Depth Factor

- $d_c = 1 + 0.4\left(\frac{D}{B}\right) = 1 + 0.4\times\left(\frac{1.2}{2}\right) = 1.24$
- $d_q = 1 + 2\tan\phi(1 - Sin\phi)^2\ \left(\frac{D}{B}\right)$

  $= 1 + 2\times\tan 35^0 \times\left(1 - Sin35^0\right)^2 \times\left(\frac{1.2}{2}\right) = 1.15$
- $d_\gamma = 1$

d) Ground and Base Factor

- All $i_i = g_i = b_i = 1$ (not zero)

$$q_{ult} = CN_cS_cd_ci_cg_cb_c + \overline{\sigma}_oN_qS_qd_qi_qg_qb_q + \frac{1}{2}\gamma BN_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

$$\therefore q_{ult} = (\gamma_1\times 1.2\times 33.3\times 1\times 1.15\times 1\times 1\times 1\times R_{w1}) + (0.5\times\gamma_2\times 2\times 48.03\times 1\times 1\times 1\times 1\times 1\times R_{w2})$$

$$= 45.95\gamma_1R_{w1} + 48.03\gamma_2R_{w2}$$

Case 1: when water table is 4m below G.L.

Since Zw2 = 4-1.2 =2.8m > B= 2m then $R_{w2} = 1.0$, $R_{w1} = 1.0$ and there will be no effect of water table. $\gamma_1 = \gamma_2 = 16.8\frac{KN}{m^3}$

$\therefore q_{ult} = 45.95\times 16.8\times 1 + 48.05\times 16.8\times 1 = 1{,}579.2KPa$

Case 2: when water table is just at the base of footing

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$ because the surcharge soil is situated above water table, for the wedge term use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ since the wedge soil is situated below water table.

$$\therefore q_{ult} = 45.95 \times 16.8 \times 1 + 48.05 \times 19.5 \times 0.5 = 1{,}240.45 KPa$$

Case 3: when water table is 2.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{1.3}{2}\right) = 0.825$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$, for the wedge term use γ_2 will be taken as average unit weight of soil situated below the footing level, since the soil up to depth B below the footing level is partially above water table and partially below the water table.

$$\gamma_2 = \gamma_{av} = \frac{(16.8 \times 1.3) + (19.5 \times 0.7)}{(1.3 + 0.7)} = 17.75\frac{KN}{m^3}$$

$$\therefore q_{ult} = 45.95 \times 16.8 \times 1 + 48.05 \times 17.75 \times 0.825 = 1{,}475.59 KPa$$

Case 4: when water table is 0.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0.5}{1.2}\right) = 0.708$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the wedge term, use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ and for the surcharge term use γ_1 will be taken as average unit weight of soil situated above the base of footing, since this soil is located partially above the water table and partially below the water.

$$\gamma_1 = \gamma_{av} = \frac{(16.8 \times 0.5) + (19.5 \times 0.7)}{(0.5 + 0.7)} = 18.38 \frac{KN}{m^3}$$

$$\therefore q_{ult} = 45.95 \times 18.38 \times 0.708 + 48.05 \times 19.5 \times 0.5 = 1{,}066.44 KPa$$

Case 5: when water table is at G.L. itself

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0}{1.2}\right) = 0.5$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the wedge term and the surcharge term use $\gamma_1 = \gamma_2 = 19.5 \frac{KN}{m^3}$

$$\therefore q_{ult} = 45.95 \times 19.5 \times 0.5 + 48.05 \times 19.5 \times 0.5 = 916.5 KPa$$

1.1.3. Method Of Meyerhof

$$q_{ult} = CN_c S_c d_c + \bar{\sigma}_o N_q S_q d_q R_{w1} + \frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma R_{w2}$$ (vertical Load)

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{35^0}{2}\right) \times e^{\pi \tan 35^0} = 33.3$
- $N_c = (Nq - 1) \times \cot\phi = (33.3 - 1) \times \cot 35^0 = 46.13$
- $N_\gamma = (Nq - 1)\tan(1.4\phi) = 33.3 \times \tan(1.4 \times 35^0) = 37.16$

b) Shape Factor

- $K_p = \tan^2(45 + \frac{\phi}{2}) = \tan^2(45 + \frac{35^0}{2}) = 3.69$
- $\sqrt{K_p} = 1.92$
- $S_c = S_q = S_\gamma = 1.0$ since it is strip footing.

c) Depth Factor

- $d_c = 1 + 0.2\sqrt{K_p}\left(\frac{D}{B}\right) = 1 + 0.2 \times 1.92 \times \left(\frac{1.2}{2}\right) = 1.23$

- $d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\left(\frac{D}{B}\right) = 1 + 0.1 \times 1.92 \times \left(\frac{1.2}{2}\right) = 1.12$

$$q_{ult} = cN_cS_cd_c + \overline{\sigma}_oN_qS_qd_qR_{w1} + \frac{1}{2}\gamma BN_\gamma S_\gamma d_\gamma R_{w2}$$

$$\therefore q_{ult} = (\gamma_1 \times 1.2 \times 33.3 \times 1 \times 1.12 \times 1 \times 1 \times 1 \times R_{w1}) + (0.5 \times \gamma_2 \times 2 \times 37.16 \times 1 \times 1.12 \times 1 \times R_{w2}) = 44.76\gamma_1R_{w1} + 41.62\gamma_2R_{w2}$$

Case 1: when water table is 4m below G.L.

Since Z_{w2} = 4-1.2 =2.8m > B= 2m then $R_{w2} = 1.0$, $R_{w1} = 1.0$ and also there will be no effect of water table. $\gamma_1 = \gamma_2 = 16.8\frac{KN}{m^3}$

$$\therefore q_{ult} = 44.76 \times 16.8 \times 1 + 41.62 \times 16.8 \times 1 = 1{,}451.18KPa$$

Case 2: when water table is just at the base of footing

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$ because the surcharge soil is situated above water table, for the wedge term use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ since the wedge soil is situated below water table.

$$\therefore q_{ult} = 44.76 \times 16.8 \times 1 + 41.62 \times 19.5 \times 0.5 = 1{,}157.76KPa$$

Case 3: when water table is 2.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{1.3}{2}\right) = 0.825$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$, for the wedge term use γ_2 will be taken as average unit weight of soil situated below the footing level, since the soil up to depth B below the footing level is partially above water table and partially below the water table.

$$\gamma_2 = \gamma_{av} = \frac{(16.8\times1.3)+(19.5\times0.7)}{(1.3+0.7)} = 17.75\frac{KN}{m^3}$$

$$\therefore q_{ult} = 44.76\times16.8\times1 + 41.62\times17.75\times0.825 = 1{,}361.44KPa$$

<u>Case 4:</u> when water table is 0.5m below the G.L.

$$R_{w1} = 0.5\left(1+\frac{Z_{w1}}{D}\right) = 0.5\left(1+\frac{0.5}{1.2}\right) = 0.708$$

$$R_{w2} = 0.5\left(1+\frac{Z_{w2}}{B}\right) = 0.5\left(1+\frac{0}{2}\right) = 0.5$$

For the wedge term, use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ and for the surcharge term use γ_1 will be taken as average unit weight of soil situated above the base of footing, since this soil is located partially above the water table and partially below the water.

$$\gamma_1 = \gamma_{av} = \frac{(16.8\times0.5)+(19.5\times0.7)}{(0.5+0.7)} = 18.38\frac{KN}{m^3}$$

$$\therefore q_{ult} = 44.76\times18.38\times0.708 + 41.62\times19.5\times0.5 = 988.26KPa$$

<u>Case 5:</u> when water table is at G.L. itself

$$R_{w1} = 0.5\left(1+\frac{Z_{w1}}{D}\right) = 0.5\left(1+\frac{0}{1.2}\right) = 0.5$$

$$R_{w2} = 0.5\left(1+\frac{Z_{w2}}{B}\right) = 0.5\left(1+\frac{0}{2}\right) = 0.5$$

For the wedge term and the surcharge term use $\gamma_1 = \gamma_2 = 19.5\frac{KN}{m^3}$

$$\therefore q_{ult} = 44.76\times19.5\times0.5 + 41.62\times19.5\times0.5 = 842.21KPa$$

1.1.4. Method Of EBCS-7

$$q_{ult} = cN_cS_ci_c + \bar{\sigma}_oN_qS_qi_qR_{w1} + \frac{1}{2}\gamma BN_\gamma S_\gamma R_{w1}$$

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right)e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{35^0}{2}\right)\times e^{\pi \tan 35^0} = 33.3$
- $N_c = (Nq - 1)\cot\phi = (33.3 - 1)\times \cot 35^0 = 46.13$
- $N_\gamma = 2\,(Nq - 1)\tan\phi = 2\times(33.3 - 1)\times \tan 35^0 = 45.23$

b) Shape Factor

- $S_c = S_q = S_\gamma = 1.0$ since it is strip footing.

c) Load inclination Factor

- All $i_i = 1$ (not zero)

$$q_{ult} = cN_cS_ci_c + \bar{\sigma}_oN_qS_qi_qR_{w1} + \frac{1}{2}\gamma BN_\gamma S_\gamma R_{w1}$$

$$\therefore q_{ult} = (\gamma_1\times1.2\times33.3\times1\times1\times R_{w1}) + (0.5\times\gamma_2\times2\times45.23\times1\times R_{w2})$$

$$= 39.96\gamma_1R_{w1} + 45.23\gamma_2R_{w2}$$

<u>Case 1:</u> when water table is 4m below G.L.

Since Z_{w2} = 4-1.2 =2.8m > B= 2m then $R_{w2} = 1.0$, $R_{w1} = 1.0$ and also there will be no effect of water table. $\gamma_1 = \gamma_2 = 16.8\frac{KN}{m^3}$

$$\therefore q_{ult} = 39.96\times16.8\times1 + 45.23\times16.8\times1 = 1{,}431.19KPa$$

<u>Case 2:</u> when water table is just at the base of footing

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$ because the surcharge soil is situated above water table, for the wedge term use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ since the wedge soil is situated below water table.

$$\therefore q_{ult} = 39.96\times16.8\times1 + 45.23\times19.5\times0.5 = 1{,}112.32 KPa$$

<u>Case 3:</u> when water table is 2.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{1.2}{1.2}\right) = 1.0$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{1.3}{2}\right) = 0.825$$

For the surcharge term, use $\gamma_1 = 16.8\frac{KN}{m^3}$, for the wedge term use γ_2 will be taken as average unit weight of soil situated below the footing level, since the soil up to depth B below the footing level is partially above water table and partially below the water table.

$$\gamma_2 = \gamma_{av} = \frac{(16.8\times1.3)+(19.5\times0.7)}{(1.3+0.7)} = 17.75\frac{KN}{m^3}$$

$$\therefore q_{ult} = 39.96\times16.8\times1 + 45.25\times17.75\times0.825 = 1{,}333.96 KPa$$

<u>Case 4:</u> when water table is 0.5m below the G.L.

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0.5}{1.2}\right) = 0.708$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the wedge term, use $\gamma_2 = \gamma_{sat} = 19.5\frac{KN}{m^3}$ and for the surcharge term use γ_1 will be taken as average unit weight of soil situated above the base of footing, since this soil is located partially above the water table and partially below the water.

$$\gamma_1 = \gamma_{av} = \frac{(16.8\times0.5)+(19.5\times0.7)}{(0.5+0.7)} = 18.38\frac{KN}{m^3}$$

$$\therefore q_{ult} = 39.96\times18.38\times0.708 + 45.23\times19.5\times{*}0.5 = 960.99 KPa$$

<u>Case 5:</u> when water table is at G.L. itself

$$R_{w1} = 0.5\left(1 + \frac{Z_{w1}}{D}\right) = 0.5\left(1 + \frac{0}{1.2}\right) = 0.5$$

$$R_{w2} = 0.5\left(1 + \frac{Z_{w2}}{B}\right) = 0.5\left(1 + \frac{0}{2}\right) = 0.5$$

For the wedge term and the surcharge term use $\gamma_1 = \gamma_2 = 19.5\frac{KN}{m^3}$

$$\therefore q_{ult} = 39.96 \times 19.5 \times 0.5 + 45.23 \times 19.5 \times 0.5 = 830.60 KPa$$

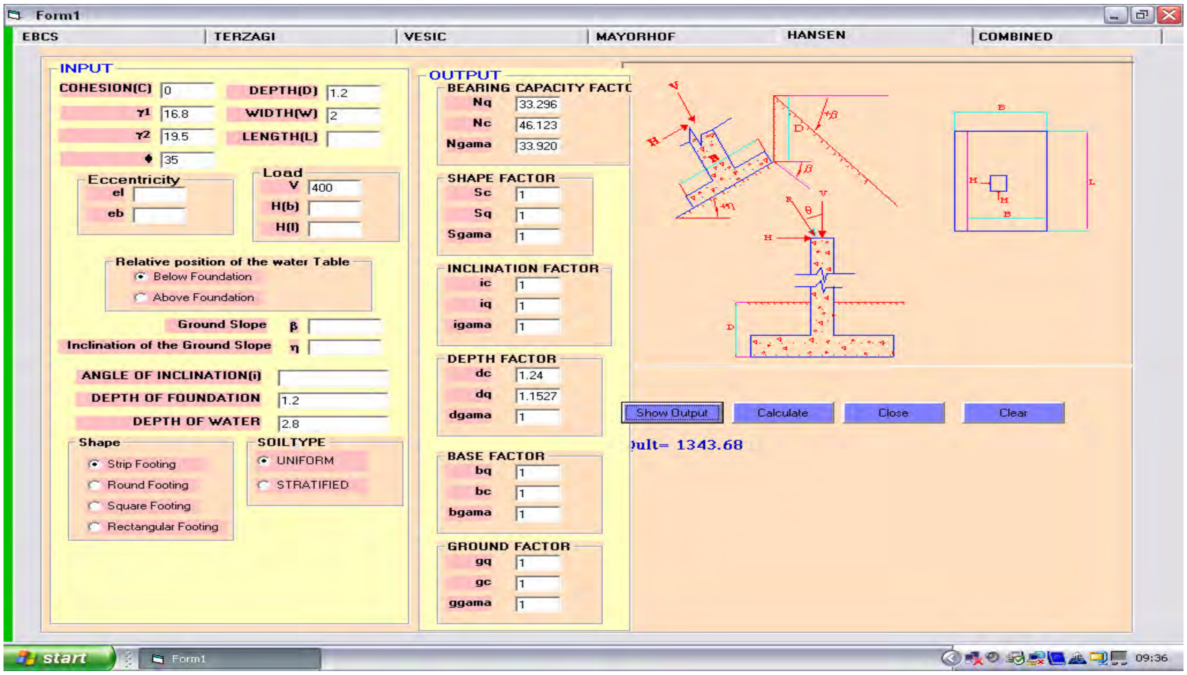
Example 1 (case 1) Composition of long hand calculation and the program output								
	Long Hand Calculations	The program output	Long Hand Calculations	The program output	Long Hand Calculations	The program output	Long Hand Calculations	The program output
	EBCS-7	EBCS-7	Meyerhof	Meyerhof	Hansen	Hansen	Vesic	Vesic
q_u	1,431.19	1,431.08	1,451.18	1,444.71	1,341.98	1,343.68	1,579.20	1,580.68
N_c	46.13	46.123	46.13	46.123	46.13	46.123	46.13	46.123
N_q	33.3	33.296	33.3	33.296	33.3	33.296	33.3	33.296
N_γ	45.23	45.227	37.16	37.152	33.93	33.92	48.03	48.028
s_c	1	1	1	1	1	1	1	1
s_q	1	1	1	1	1	1	1	1
s_γ	1	1	1	1	1	1	1	1
i_c	1	1	1	1	1	1	1	1
i_q	1	1	1	1	1	1	1	1
i_γ	1	1	1	1	1	1	1	1
d_c	-	-	1.23	1.2305	1.24	1.24	1.24	1.24
d_q	-	-	1.12	1.1152	1.15	1.1527	1.15	1.1527
d_γ	-	-	1.12	1.1152	1	1	1	1
b_c	-	-	-	-	1	1	1	1
b_q	-	-	-	-	1	1	1	1
d_γ	-	-	-	-	1	1	1	1
g_c	-	-	-	-	1	1	1	1
g_q	-	-	-	-	1	1	1	1
g_γ	-	-	-	-	1	1	1	1

Example 1 (case 5) Comparison of long hand calculation and the program output								
	Long Hand Calculations	The program output	Long Hand Calculations	The program output	Long Hand Calculations	The program output	Long Hand Calculations	The program output
	EBCS-7	EBCS-7	Meyerhof	Meyerhof	Hansen	Hansen	Vesic	Vesic
q_u	830.60	830.54	842.21	838.45	778.83	779.81	916.50	917.36
N_c	46.13	46.123	46.13	46.123	46.13	46.123	46.13	46.123
N_q	33.3	33.296	33.3	33.296	33.3	33.296	33.3	33.295
N_γ	45.23	45.227	37.16	37.152	33.93	33.92	48.03	48.028
s_c	1	1	1	1	1	1	1	1
s_q	1	1	1	1	1	1	1	1
s_γ	1	1	1	1	1	1	1	1
i_c	1	1	1	1	1	1	1	1
i_q	1	1	1	1	1	1	1	1
i_γ	1	1	1	1	1	1	1	1
d_c	-	-	1.23	1.2305	1.24	1.24	1.24	1.24
d_q	-	-	1.12	1.1152	1.15	1.1527	1.15	1.1527
d_γ	-	-	1.12	1.1152	1	1	1	1
b_c	-	-	-	-	1	1	1	1
b_q	-	-	-	-	1	1	1	1
d_γ	-	-	-	-	1	1	1	1
g_c	-	-	-	-	1	1	1	1
g_q	-	-	-	-	1	1	1	1
g_γ	-	-	-	-	1	1	1	1

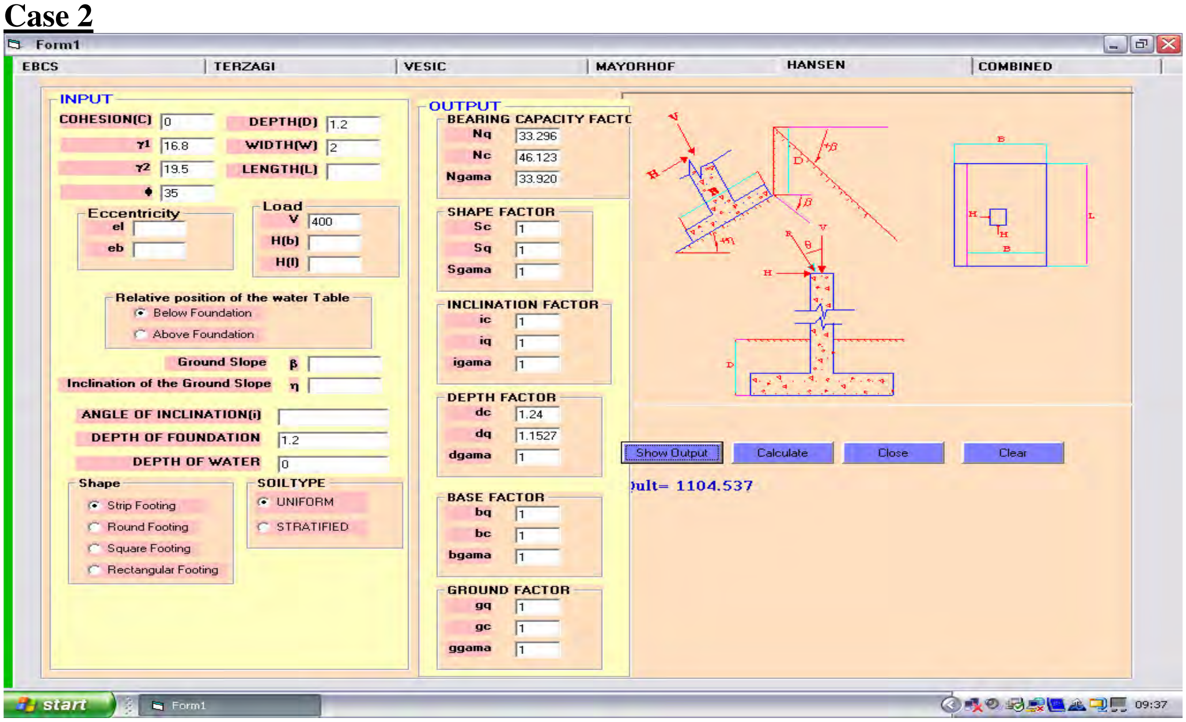
1.2. According To the Developed Software

1.2.1 Method of Hansen

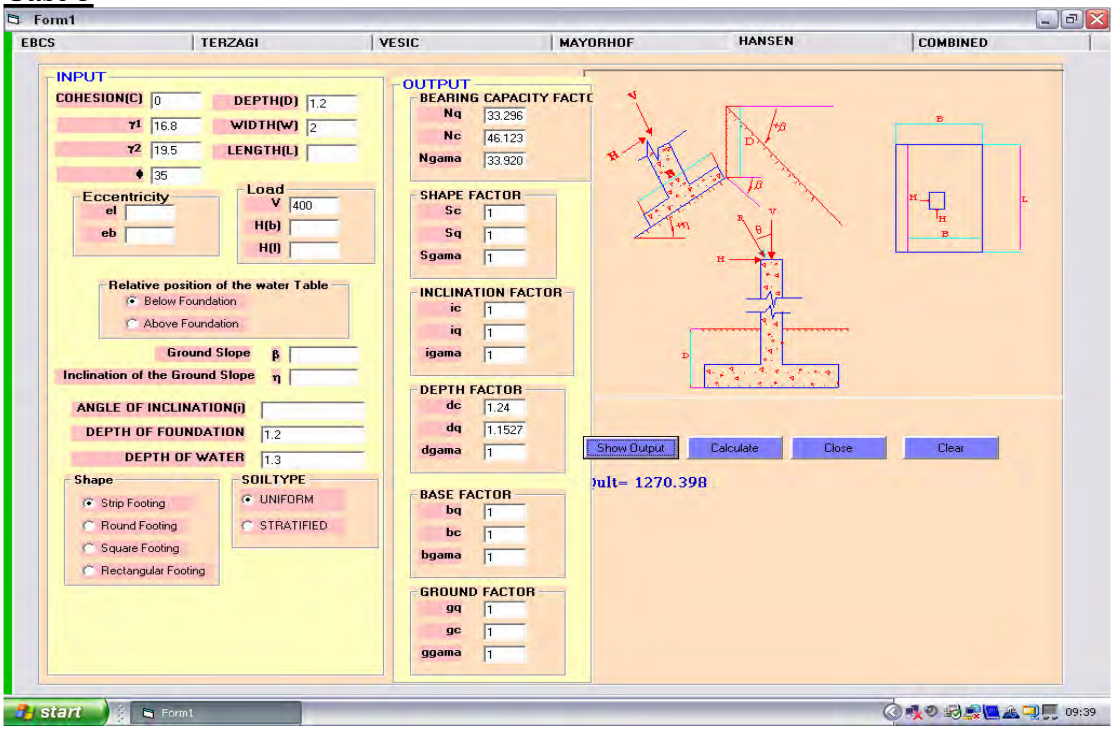
Case 1



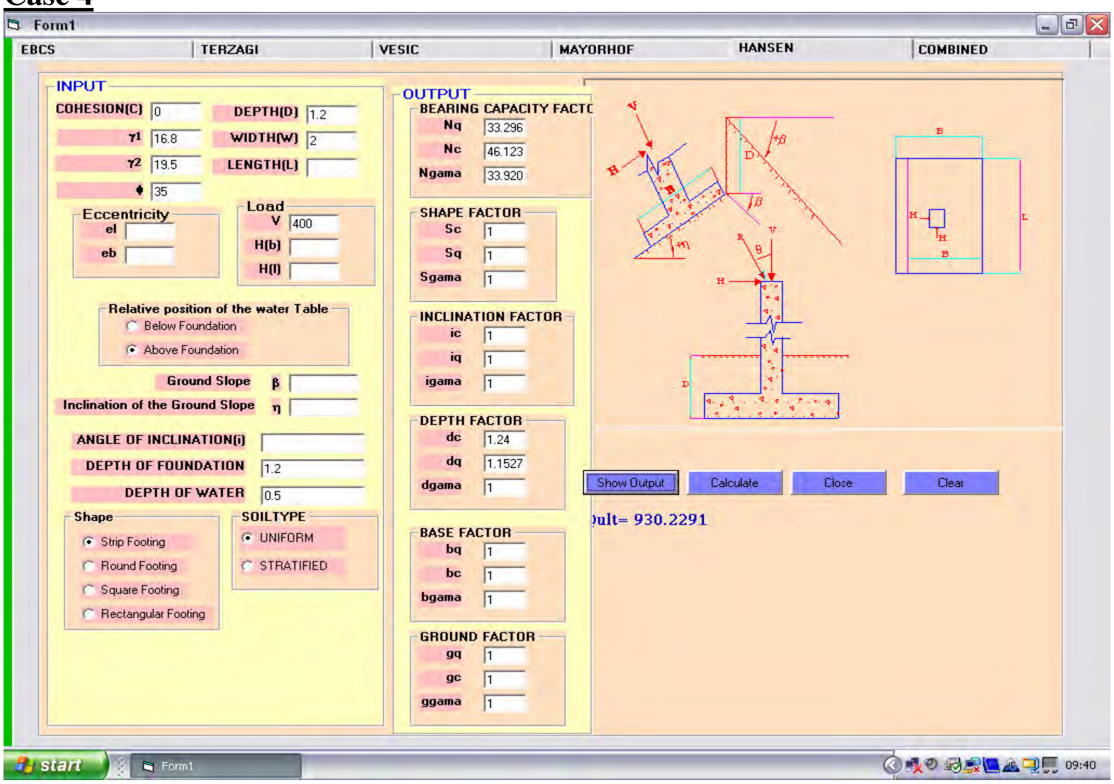
Case 2



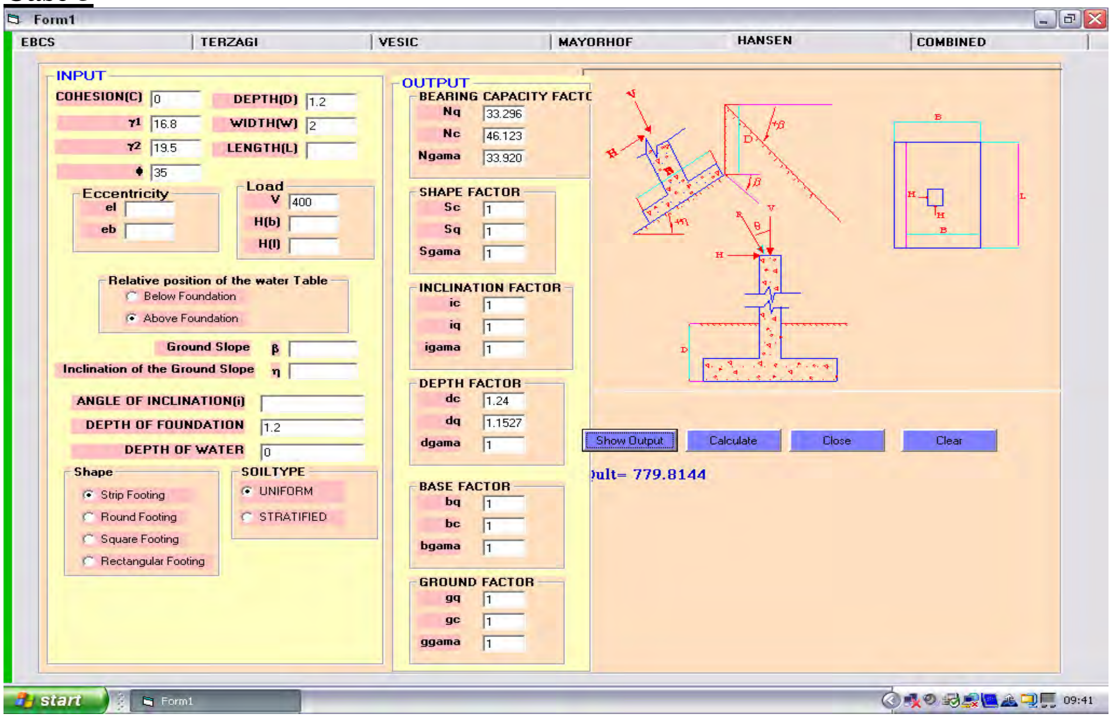
Case 3


Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
ϕ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
Ground Slope β
Inclination of the Ground Slope η
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 1.3
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 33.920
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.24
dq 1.1527
dgama 1
BASE FACTOR
bq 1
bc 1
bgama 1
GROUND FACTOR
gq 1
gc 1
ggama 1
Show Output
Calculate
Close
Clear
)ult= 1270.398
start
Form1
09:39

Case 4

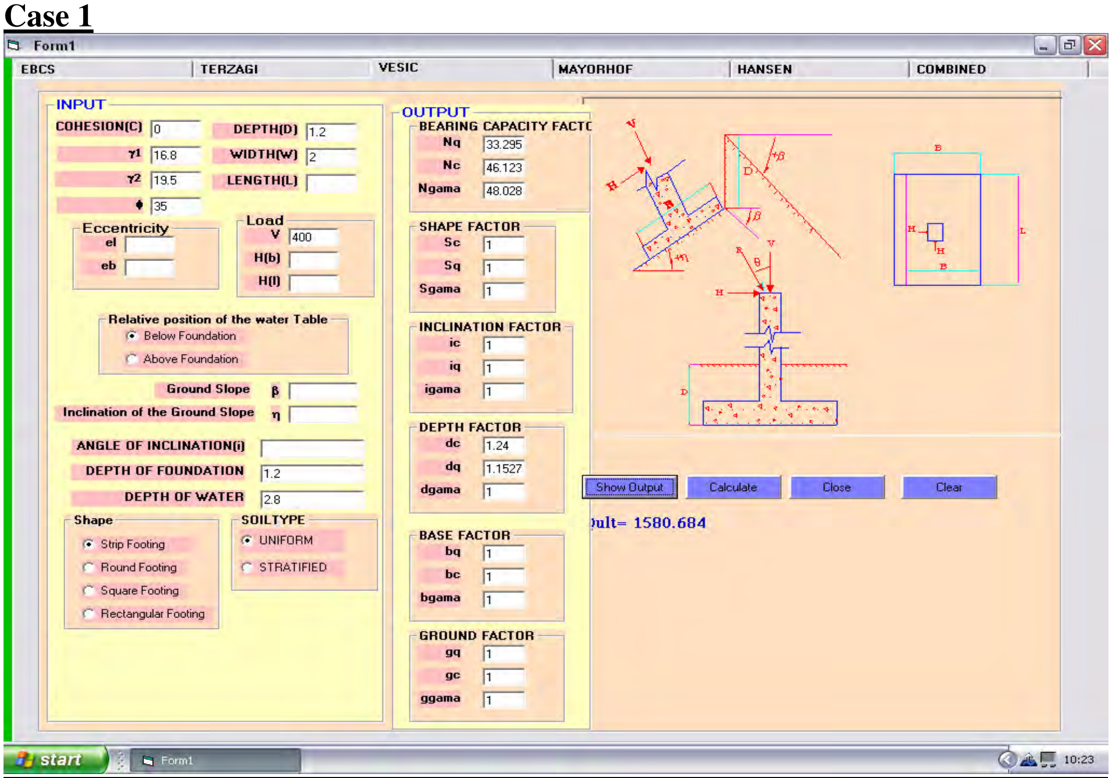

Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
ϕ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
Ground Slope β
Inclination of the Ground Slope η
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0.5
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 33.920
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.24
dq 1.1527
dgama 1
BASE FACTOR
bq 1
bc 1
bgama 1
GROUND FACTOR
gq 1
gc 1
ggama 1
Show Output
Calculate
Close
Clear
)ult= 930.2291
start
Form1
09:40

Case 5

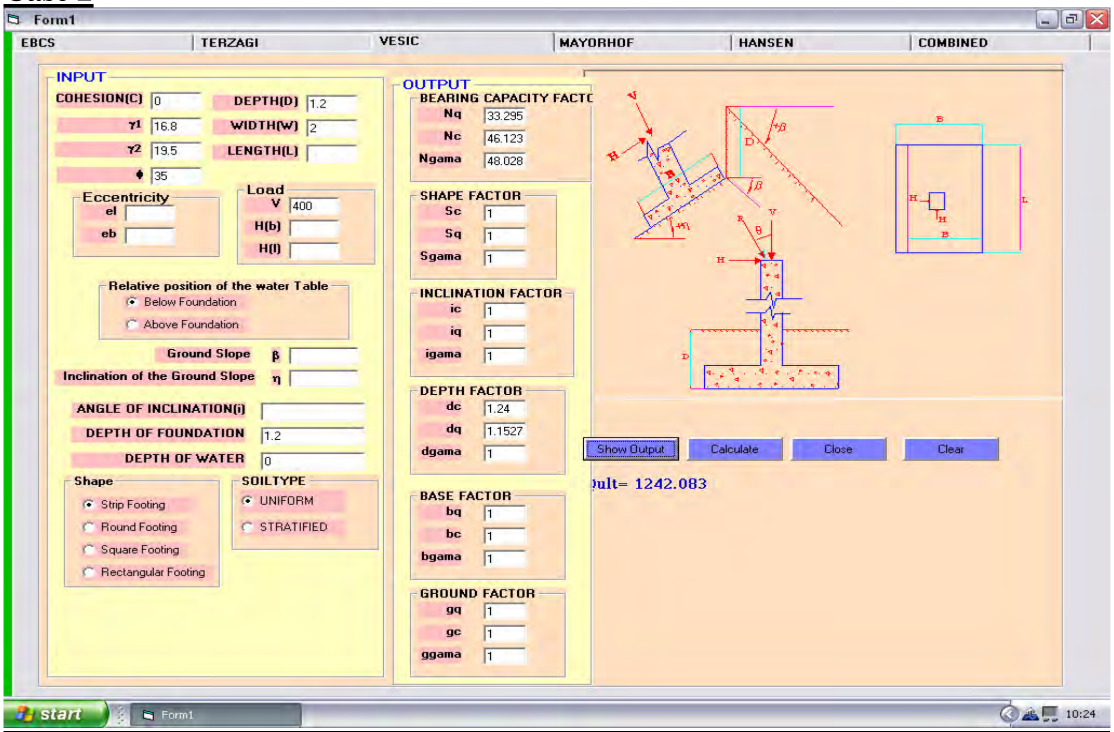


1.2.2. Method Of Vesic

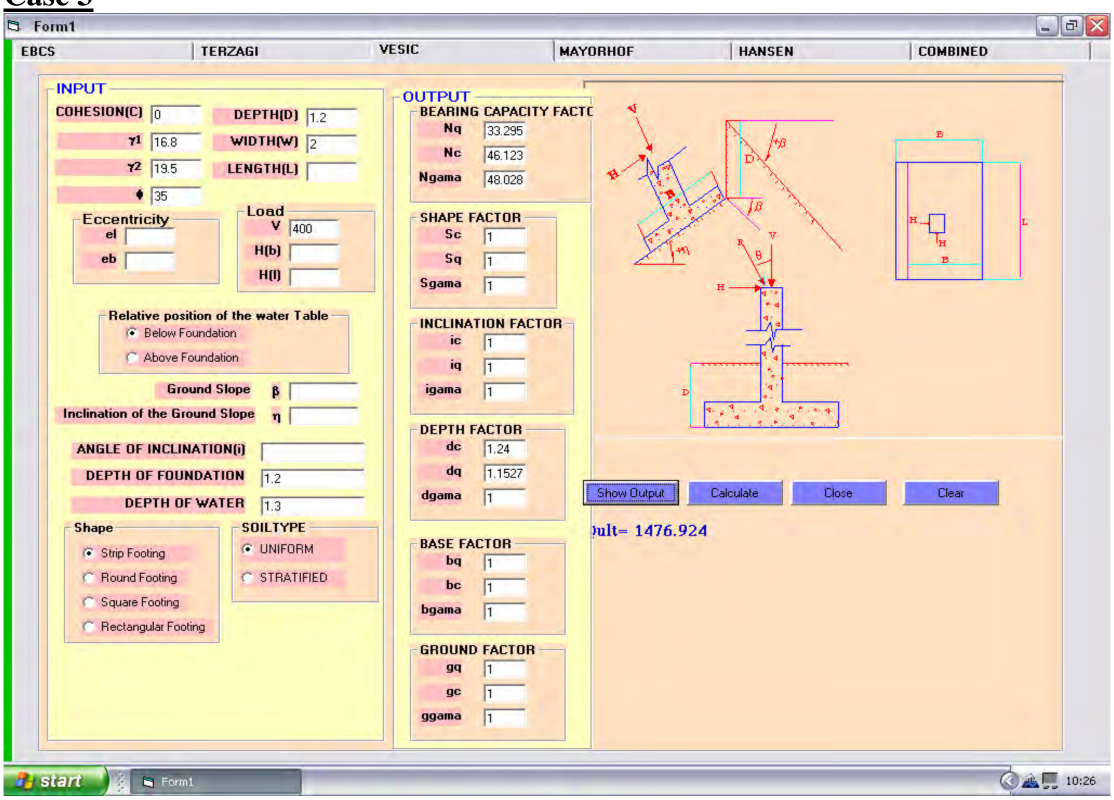
Case 1



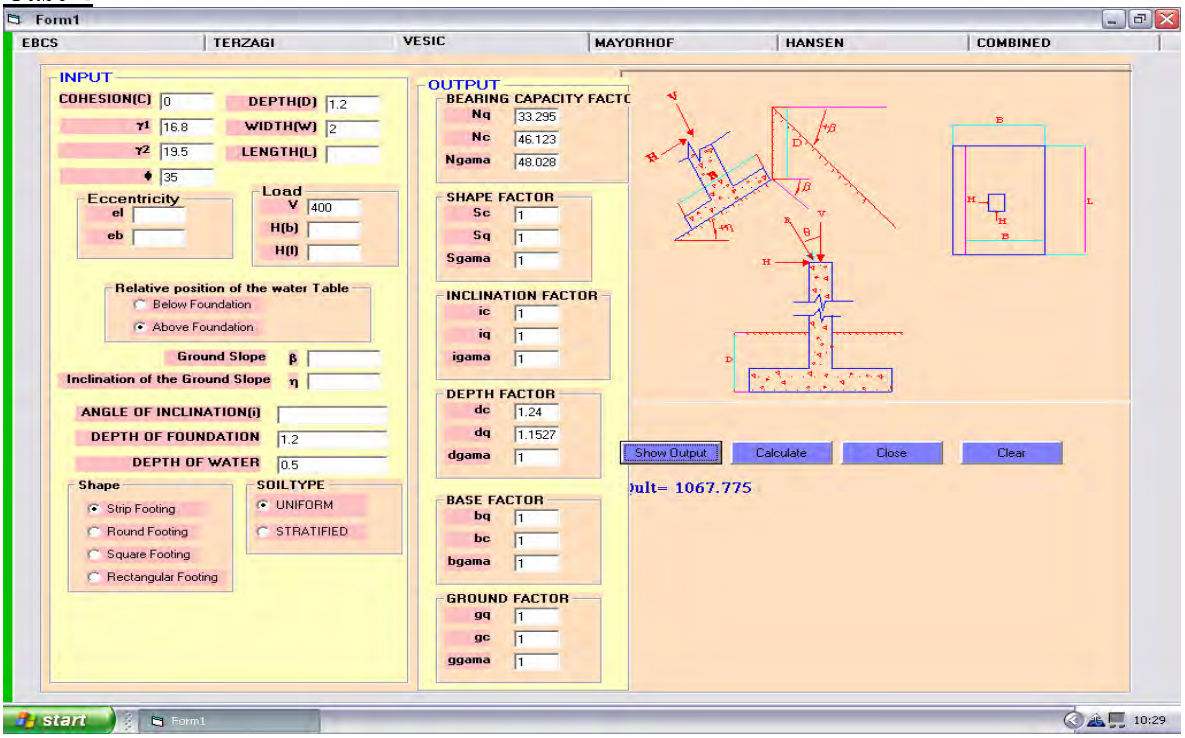
Case 2


Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
ϕ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
Ground Slope β
Inclination of the Ground Slope η
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.295
Nc 46.123
Ngama 48.028
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.24
dq 1.1527
dgama 1
BASE FACTOR
bq 1
bc 1
bgama 1
GROUND FACTOR
gq 1
gc 1
ggama 1
Show Output
Calculate
Close
Clear
)ult= 1242.083
start
Form1
10:24

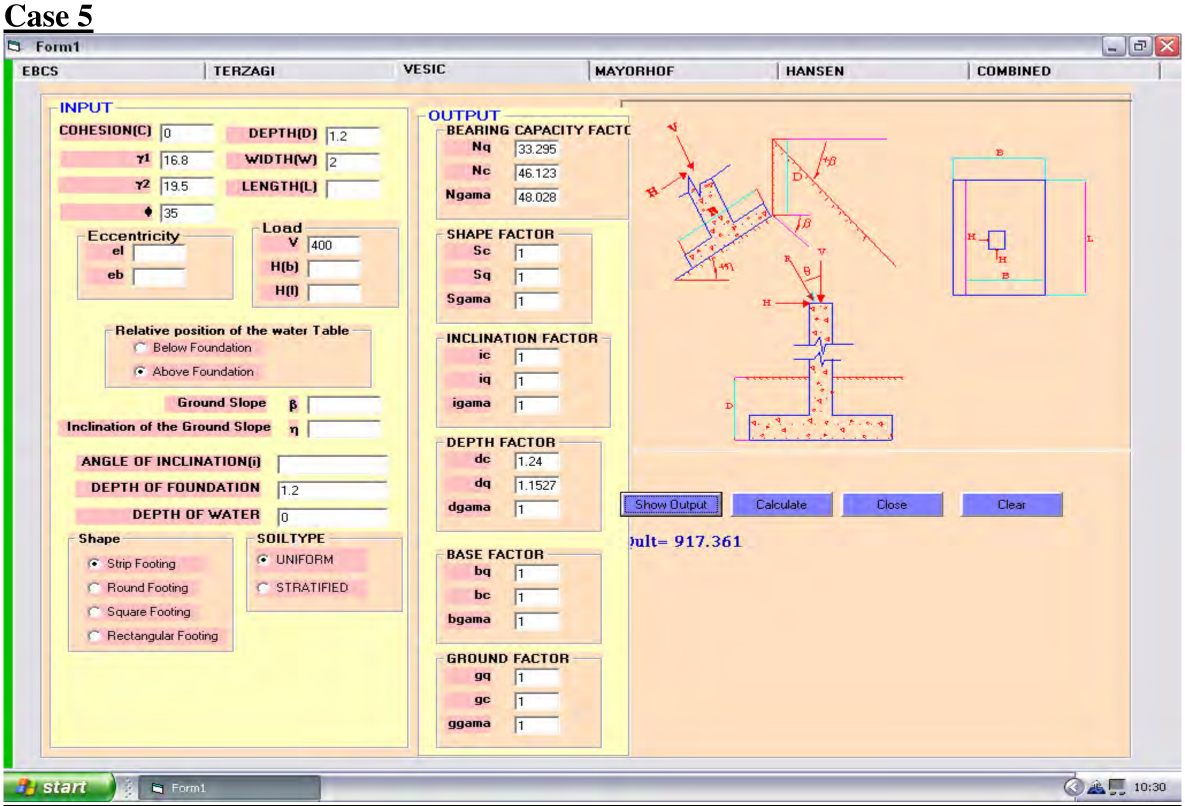
Case 3


Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
ϕ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
Ground Slope β
Inclination of the Ground Slope η
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 1.3
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.295
Nc 46.123
Ngama 48.028
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.24
dq 1.1527
dgama 1
BASE FACTOR
bq 1
bc 1
bgama 1
GROUND FACTOR
gq 1
gc 1
ggama 1
Show Output
Calculate
Close
Clear
)ult= 1476.924
start
Form1
10:26

Case 4

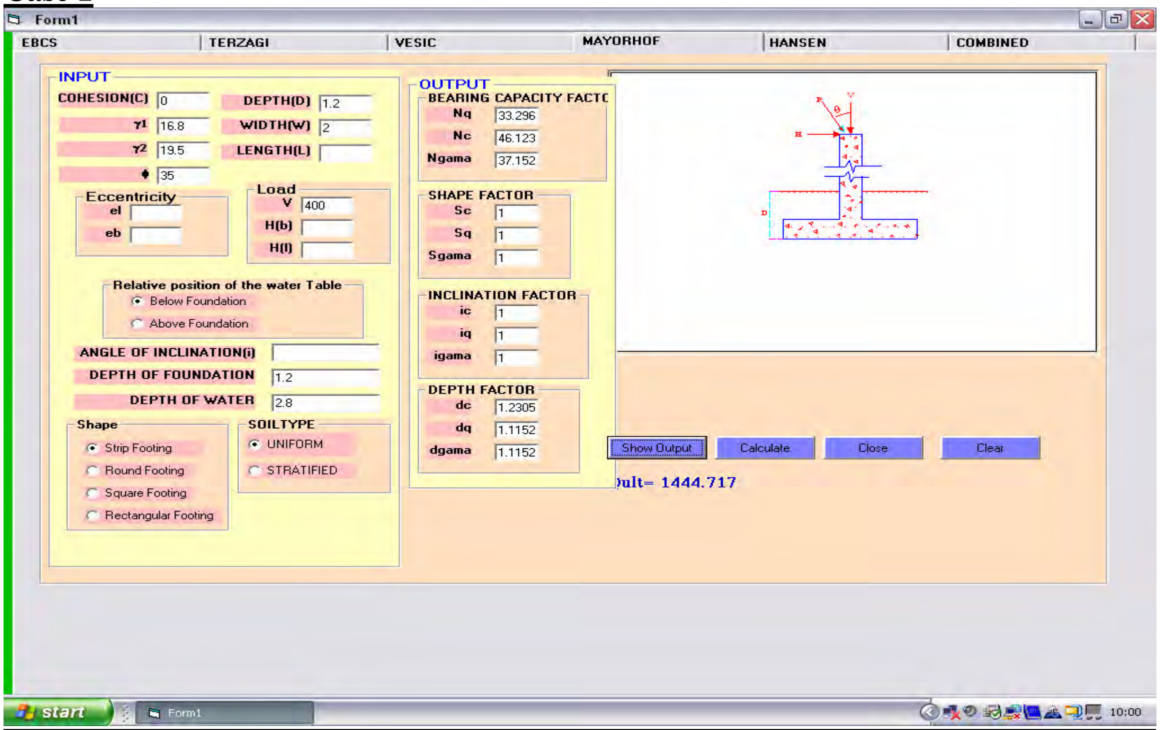

Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
DEPTH(D) 1.2
γ1 16.8
WIDTH(W) 2
γ2 19.5
LENGTH(L)
ϕ 35
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
Ground Slope β
Inclination of the Ground Slope η
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0.5
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.295
Nc 46.123
Ngama 48.028
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.24
dq 1.1527
dgama 1
BASE FACTOR
bq 1
bc 1
bgama 1
GROUND FACTOR
gq 1
gc 1
ggama 1
Show Output
Calculate
Close
Clear
)ult= 1067.775
start
Form1
10:29

Case 5

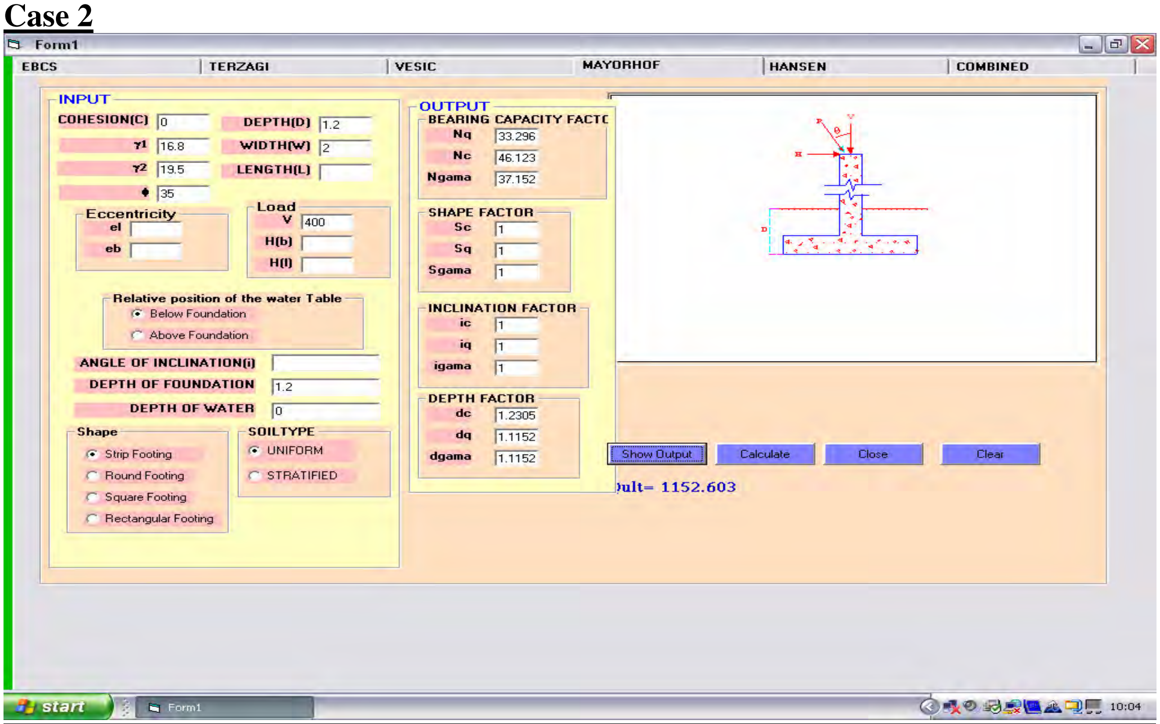

Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
DEPTH(D) 1.2
γ1 16.8
WIDTH(W) 2
γ2 19.5
LENGTH(L)
ϕ 35
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
Ground Slope β
Inclination of the Ground Slope η
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.295
Nc 46.123
Ngama 48.028
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.24
dq 1.1527
dgama 1
BASE FACTOR
bq 1
bc 1
bgama 1
GROUND FACTOR
gq 1
gc 1
ggama 1
Show Output
Calculate
Close
Clear
)ult= 917.361
start
Form1
10:30

1.2.3. Method Of Meyerhof

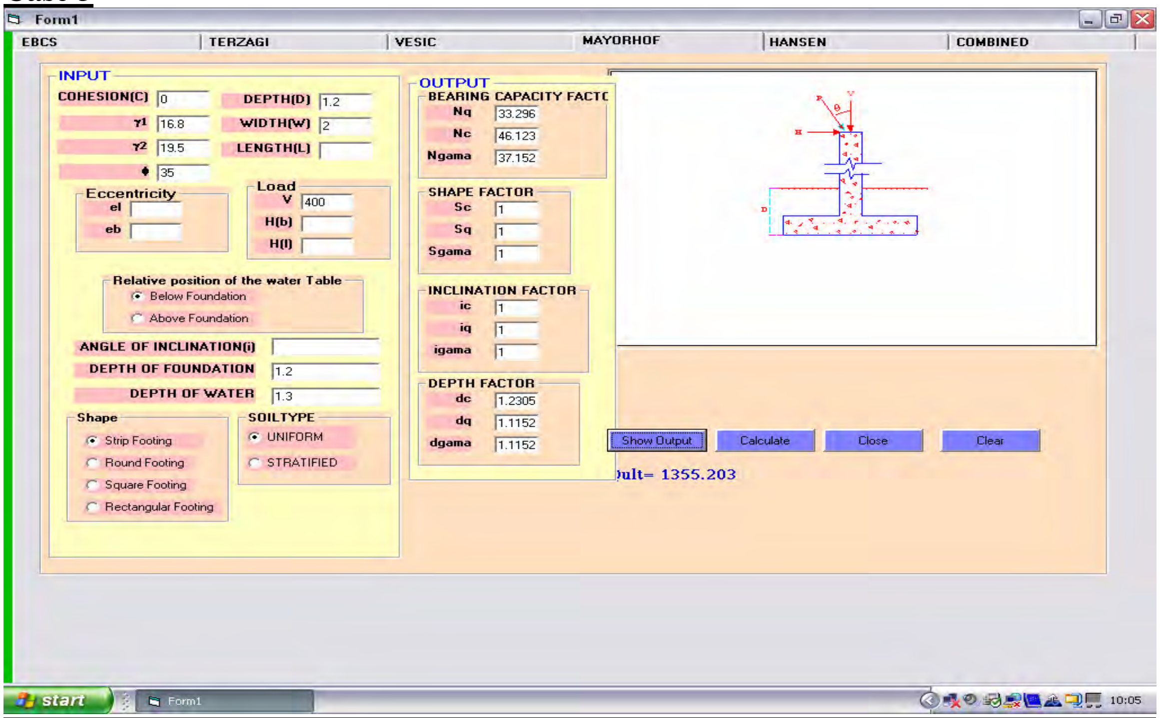
Case 1



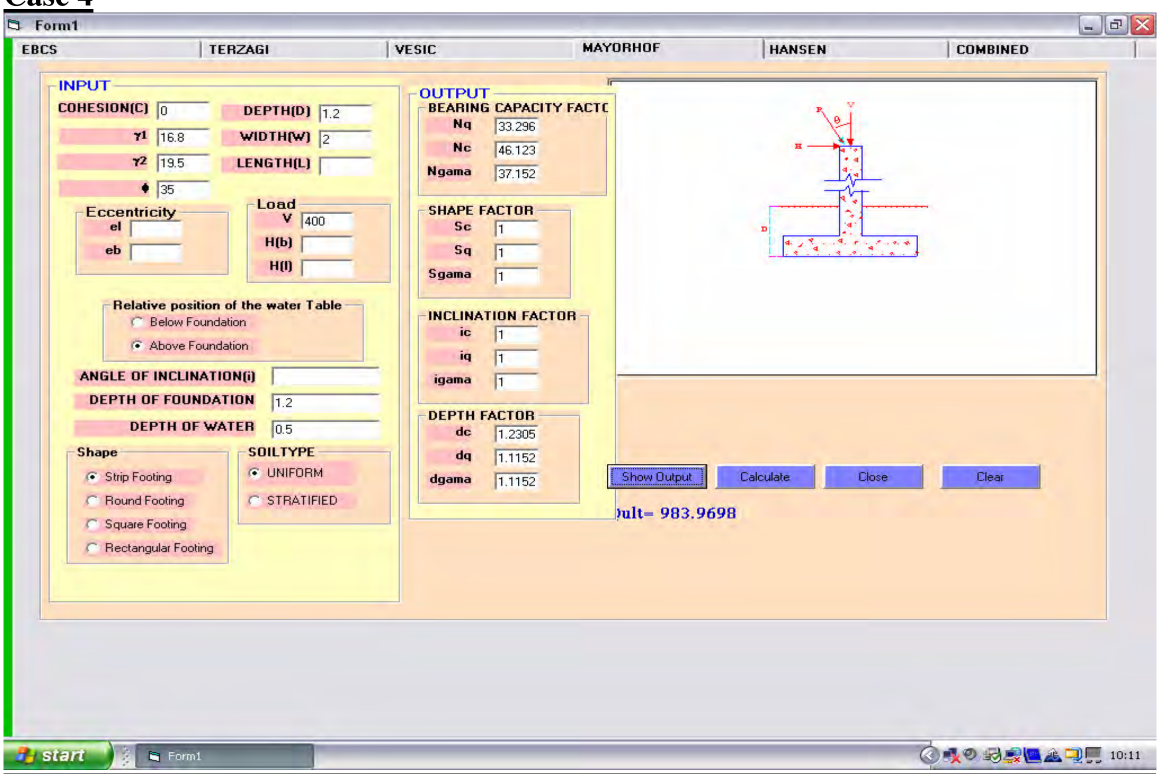
Case 2



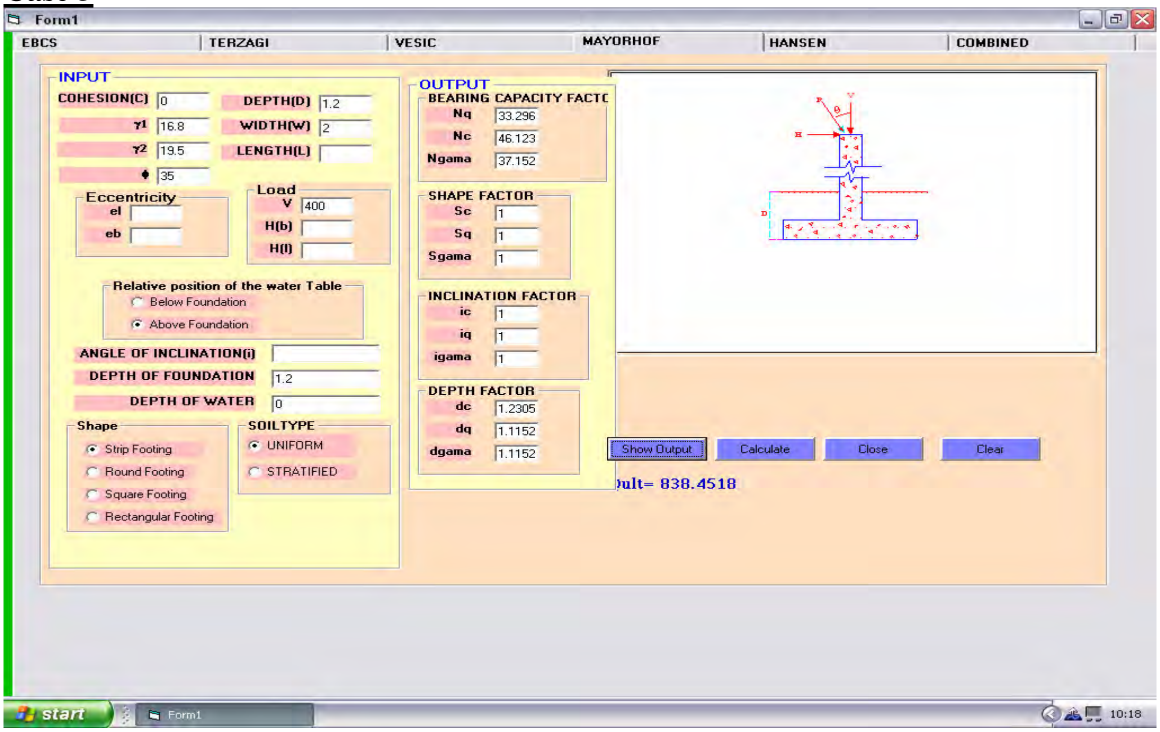
Case 3


Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
ϕ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 1.3
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 37.152
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.2305
dq 1.1152
dgama 1.1152
Show Output
Calculate
Close
Clear
)ult= 1355.203
start
Form1
10:05

Case 4

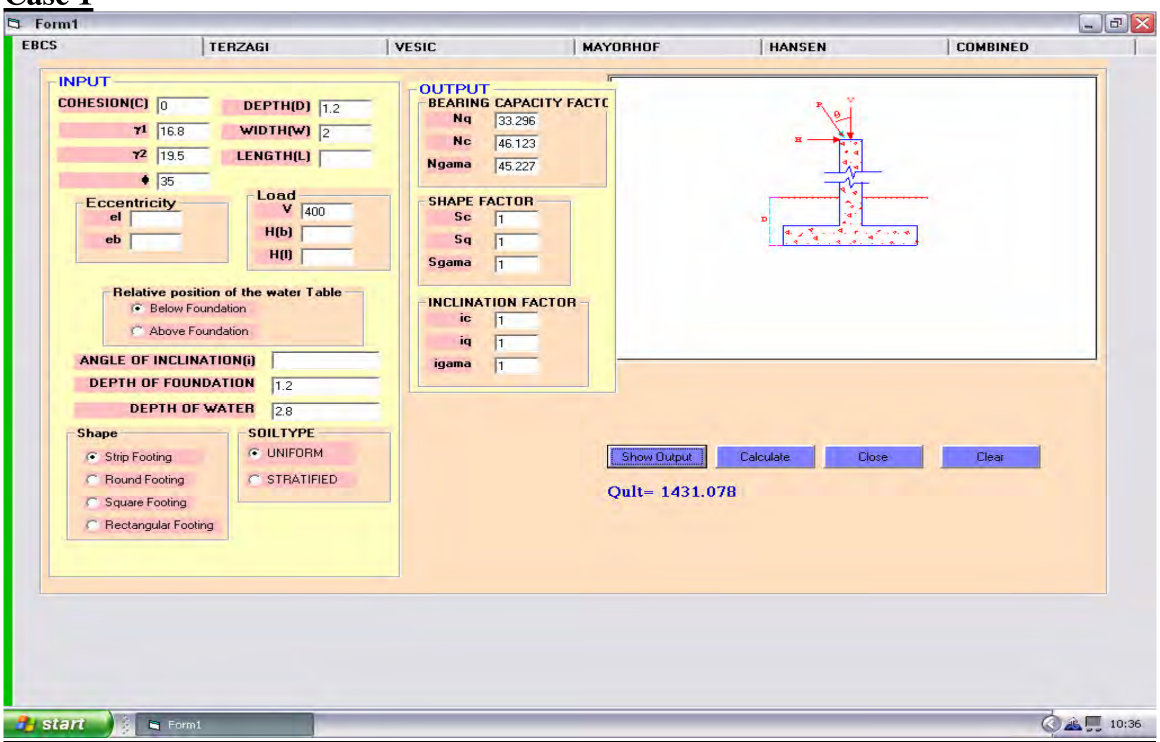

Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
ϕ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0.5
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 37.152
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
DEPTH FACTOR
dc 1.2305
dq 1.1152
dgama 1.1152
Show Output
Calculate
Close
Clear
)ult= 983.9698
start
Form1
10:11

Case 5

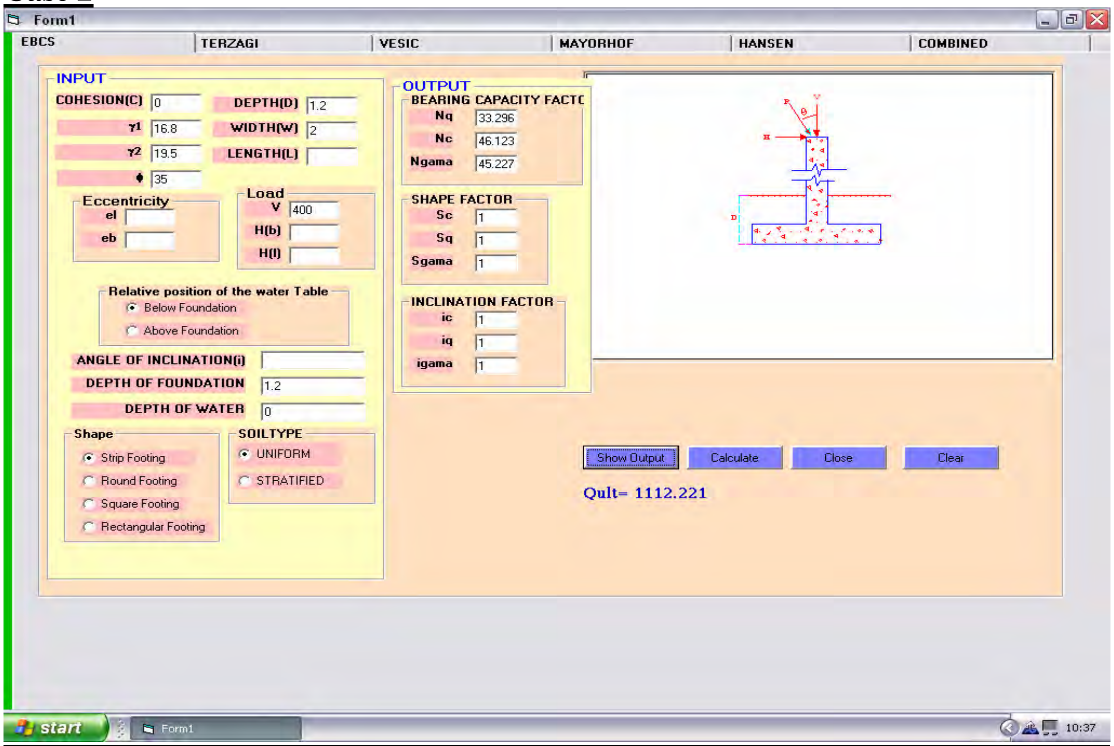


1.2.4. Method Of EBCS-7

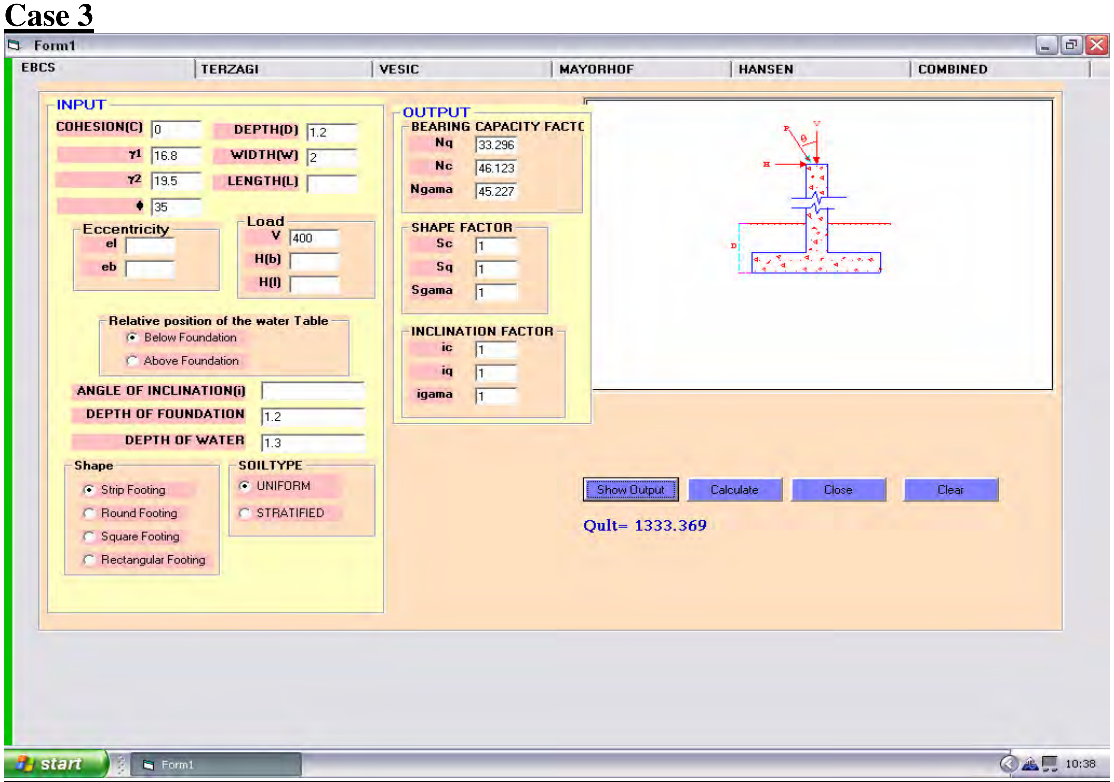
Case 1



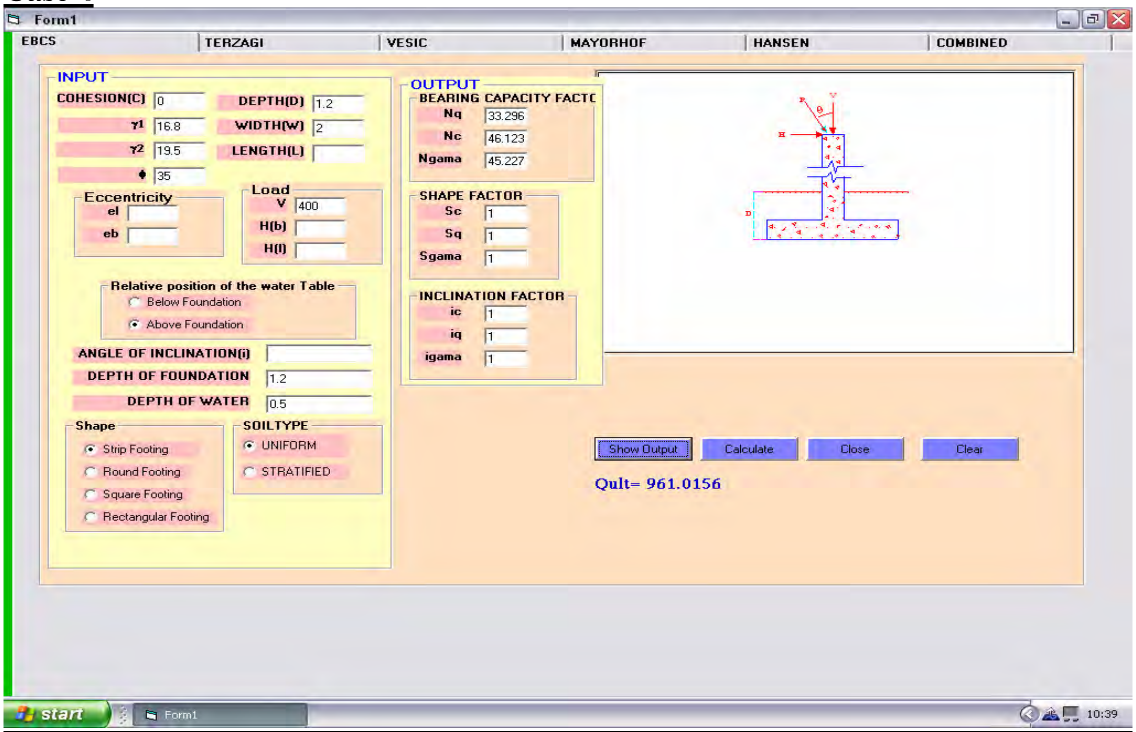
Case 2


Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
φ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 45.227
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
Show Output
Calculate
Close
Clear
Qult= 1112.221
start
Form1
10:37

Case 3

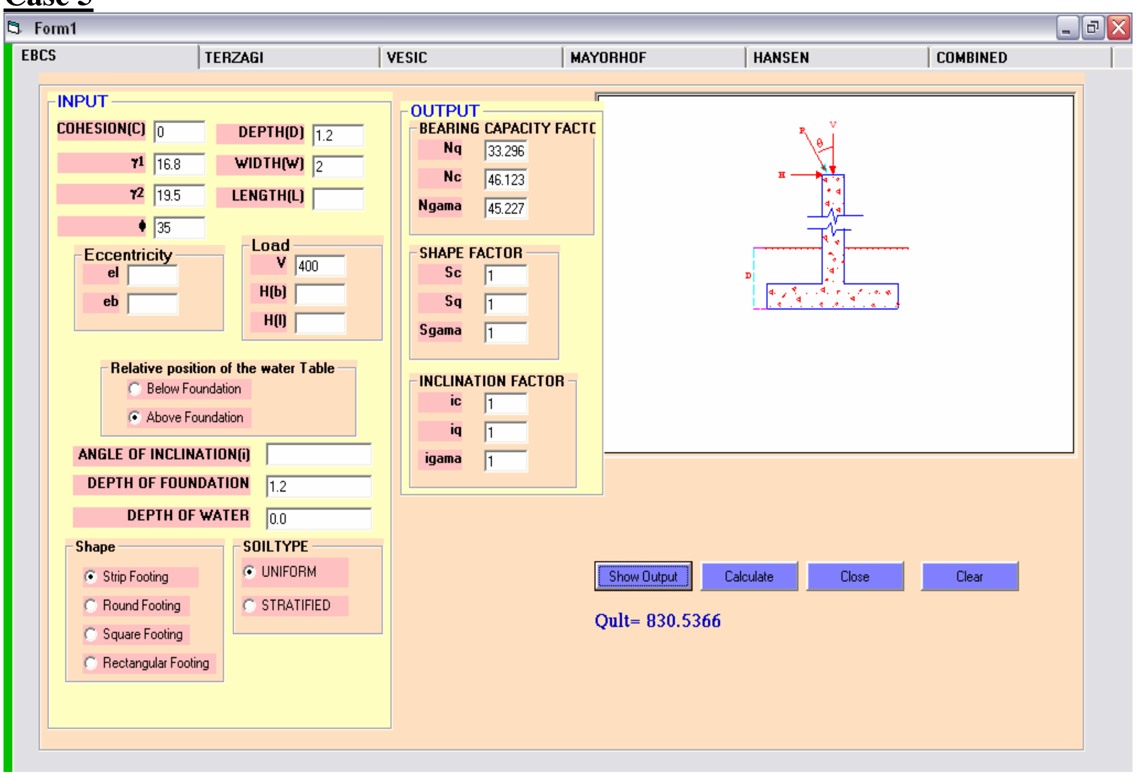

Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
γ1 16.8
γ2 19.5
φ 35
DEPTH(D) 1.2
WIDTH(W) 2
LENGTH(L)
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 1.3
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 45.227
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
Show Output
Calculate
Close
Clear
Qult= 1333.369
start
Form1
10:38

Case 4



Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
DEPTH(D) 1.2
γ1 16.8
WIDTH(W) 2
γ2 19.5
LENGTH(L)
ϕ 35
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0.5
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 45.227
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
Show Output
Calculate
Close
Clear
Qult= 961.0156
start
Form1
10:39

Case 5



Form1
EBCS
TERZAGI
VESIC
MAYORHOF
HANSEN
COMBINED
INPUT
COHESION(C) 0
DEPTH(D) 1.2
γ1 16.8
WIDTH(W) 2
γ2 19.5
LENGTH(L)
ϕ 35
Eccentricity
el
eb
Load
V 400
H(b)
H(l)
Relative position of the water Table
Below Foundation
Above Foundation
ANGLE OF INCLINATION(i)
DEPTH OF FOUNDATION 1.2
DEPTH OF WATER 0.0
Shape
Strip Footing
Round Footing
Square Footing
Rectangular Footing
SOILTYPE
UNIFORM
STRATIFIED
OUTPUT
BEARING CAPACITY FACTC
Nq 33.296
Nc 46.123
Ngama 45.227
SHAPE FACTOR
Sc 1
Sq 1
Sgama 1
INCLINATION FACTOR
ic 1
iq 1
igama 1
Show Output
Calculate
Close
Clear
Qult= 830.5366

Examples 2 (Uniform Soil with eccentricity)

A square footing is $1.8m \times 1.8m$ with a $0.4m \times 0.4m$ square column. It is loaded with an axial load of triaxial test (soil not saturated) gives $\phi = 36^0$ and $c = 20KPa.$ The footing depth $D_f = 1.8m$, the soil unit weight $\gamma = 18\frac{KN}{m^3}$, the water table is at a depth of 6.1m from the ground surface and with a vertical load of $1800KN$, $M_y = 360KNm$ and $M_x = 450KNm$. What is the allowable bearing capacity of the soil

- $e_x = \frac{M_y}{V} = \frac{360.00}{1800.00} = 0.20\ m$
- $e_y = \frac{M_x}{V} = \frac{450.00}{1800.00} = 0.25\ m$
- $L' = L - 2e_y = 1.80 - 2 \times 0.25 = 1.40m$
- $B' = 2e_x = 1.80 - 2 \times 0.20 = 1.30\ m$

Both value of e are $\prec \frac{B}{6} = 0.3m$ also

$$B_{\min} = 4. \times 0.25 + 0.4 = 1.4 \prec 1.8$$
$$L_{\min} = 4. \times 0.20 + 0.4 = 1.2 \prec 1.8$$

Therefore $B' = B - 2e_y = 1.8 - 2 \times 0.25 = 1.30m (B' \prec L')$

$L' = B - 2e_x = 1.8 - 2 \times 0.20 = 1.40m (L' \succ B')$

1.3. Long Hand Calculations

1.3.1. Method Of Hansen

$$q_{ult} = cN_c S_c d_c i_c g_c b_c + \overline{\sigma}_o N_q S_q d_q i_q g_q b_q + \frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{36^0}{2}\right) e^{\pi \tan\phi} = 37.75 \cong 38$
- $N_c = (Nq - 1)\cot\phi = (38 - 1) \times \cot 36^0 = 51$
- $N_\gamma = 1.5\ (Nq - 1)\tan\phi = 1.5 \times (38 - 1) \times \tan 36^0 = 40$

b) Shape Factor

- $S_c = 1 + \left(\frac{Nq}{Nc}\right)\left(\frac{B'}{L'}\right) = 1 + 0.745 \times \left(\frac{1.3}{1.4}\right) = 1.69$
- $S_q = 1 + \left(\frac{B'}{L'}\right) Sin\phi = 1 + \left(\frac{1.3}{1.4}\right) \sin 36^0 = 1.55$
- $S_\gamma = 1 - 0.4 \left(\frac{B'}{L'}\right) = 1 - 0.4 \times \left(\frac{1.3}{1.4}\right) = 0.63 > 0.6$ (Ok!)

c) Depth Factor

- $d_c = 1 + 0.4\left(\frac{D}{B}\right) = 1 + 0.4 \times \left(\frac{1.8}{1.8}\right) = 1.40$
- $d_q = 1 + 2\tan\phi(1 - Sin\phi)^2 \left(\frac{D}{B}\right)$

  $= 1 + 2 \times \tan 36^0 \times (1 - Sin 36^0)^2 \times \left(\frac{1.8}{1.8}\right) = 1.25$
- $d_\gamma = 1$

d) Ground and Base Factor

- All $i_i = g_i = b_i = 1$ (not zero)

$$\therefore q_{ult} = (20 \times 51 \times 1.69 \times 1.4) + (18 \times 1.8 \times 38 \times 1.55 \times 1.25) + (0.5 \times 18 \times 1.3 \times 40 \times 0.63 \times 1) = 5{,}093.61 KPa.$$

1.3.2. Method Of Vesic

$$q_{ult} = cN_c S_c d_c i_c g_c b_c + \bar{\sigma}_o N_q S_q d_q i_q g_q b_q + \frac{1}{2}\gamma B N_\gamma S_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$$

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{36^0}{2}\right) \times e^{\pi \tan\phi} = 37.75 \cong 38$
- $N_c = (Nq - 1)\cot\phi = (38 - 1) \times \cot 36^0 = 51$
- $N_\gamma = 2\ (Nq + 1)\tan\phi = 2 \times (38 + 1) \times \tan 36^0 = 57$

b) Shape Factor

- $S_c = 1 + \left(\frac{Nq}{Nc}\right)\left(\frac{B'}{L'}\right) = 1 + 0.745 \times \left(\frac{1.3}{1.4}\right) = 1.69$

- $S_q = 1 + \left(\frac{B'}{L'}\right)\tan\phi = 1 + \left(\frac{1.3}{1.4}\right)\tan 36^0 = 1.67$
- $S_\gamma = 1 - 0.4\left(\frac{B'}{L'}\right) = 1 - 0.4\times\left(\frac{1.3}{1.4}\right) = 0.63$

c) Depth Factor

- $d_c = 1 + 0.4\left(\frac{D}{B}\right) = 1 + 0.4\times\left(\frac{1.8}{1.8}\right) = 1.40$
- $d_q = 1 + 2\tan\phi(1 - Sin\phi)^2\left(\frac{D}{B}\right)$

  $= 1 + 2\times\tan 36^0 \times\left(1 - Sin36^0\right)^2 \times\left(\frac{1.8}{1.8}\right) = 1.25$
- $d_\gamma = 1$

d) Ground and Base Factor

- All $i_i = g_i = b_i = 1$ (not zero)

$$\therefore q_{ult} = (20\times51\times1.69\times1.4) + (18\times1.8\times38\times1.67\times1.25) + (0.5\times18\times1.3\times57\times0.63\times1) = 5{,}403.60\,KPa.$$

1.1.3. Method Of Meyerhof

$$q_{ult} = cN_cS_cd_c + \overline{\sigma}_oN_qS_qd_q + \frac{1}{2}\gamma BN_\gamma S_\gamma d_\gamma$$ (Vertical Load)

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi\tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{36^0}{2}\right)\times e^{\pi\tan\phi} = 37.75 \cong 38$
- $N_c = (Nq - 1)\cot\phi = (38 - 1)\times\cot 36^0 = 51$
- $N_\gamma = (Nq - 1)\tan(1.4\phi) = (38 - 1)\times\tan\left(1.4\times36^0\right) = 45$

b) Shape Factor

- $K_p = \tan 2\ (45 + \frac{\phi}{2}) = \tan 2\ (45 + \frac{36^0}{2}) = 3.85$
- $\sqrt{K_p} = 1.96$

- $S_c = 1 + 0.2Kp\ \frac{B'}{L'} = 1 + 0.2 \times 3.85 \times \frac{1.3}{1.4} = 1.715$

- $S_q = S_\gamma = 1 + 0.1K_p \frac{B'}{L'} = 1 + 0.1 \times 3.85 \times \frac{1.3}{1.4} = 1.358$

c) Depth Factor

- $d_c = 1 + 0.2\sqrt{K_p}\left(\frac{D}{B}\right) = 1 + 0.2 \times 1.96 \times \left(\frac{1.8}{1.8}\right) = 1.392$

- $d_q = d_\gamma = 1 + 0.1\sqrt{K_p}\left(\frac{D}{B}\right) = 1 + 0.1 \times 1.96 \times \left(\frac{1.8}{1.8}\right) = 1.196$

$$\therefore q_{ult} = (20 \times 51 \times 1.715 \times 1.392) + (18 \times 1.8 \times 38 \times 1.358 \times 1.196)$$
$$+ (0.5 \times 18 \times 1.3 \times 44.72 \times 1.358 \times 1.196) = 5{,}284.5\ KPa.$$

1.3.4. Method Of EBCS-7

$$q_{ult} = cN_cS_ci_c + \overline{\sigma}_oN_qS_qi_q + \frac{1}{2}\gamma BN_\gamma S_\gamma$$

a) Bearing Capacity Factor

- $Nq = \tan^2\left(45 + \frac{\phi}{2}\right)e^{\pi \tan\phi}$
- $Nq = \tan^2\left(45^0 + \frac{36^0}{2}\right) \times e^{\pi \tan 36^0} = 37.75 \cong 38$
- $N_c = (Nq - 1)\cot\phi = (38 - 1) \times \cot 36^0 = 51$
- $N_\gamma = 2\ (Nq - 1)\tan\phi = 2 \times (38 - 1) \times \tan 36^0 = 53.76$

b) Shape Factor

- $S_q = 1 + Sin\phi = 1 + Sin36^0 = 1.588$

- $S_c = \left(\frac{S_qN_q - 1}{N_q - 1}\right) = \left(\frac{1.588 \times 38 - 1}{38 - 1}\right) = 1.604$
- $S_\gamma = 0.7$

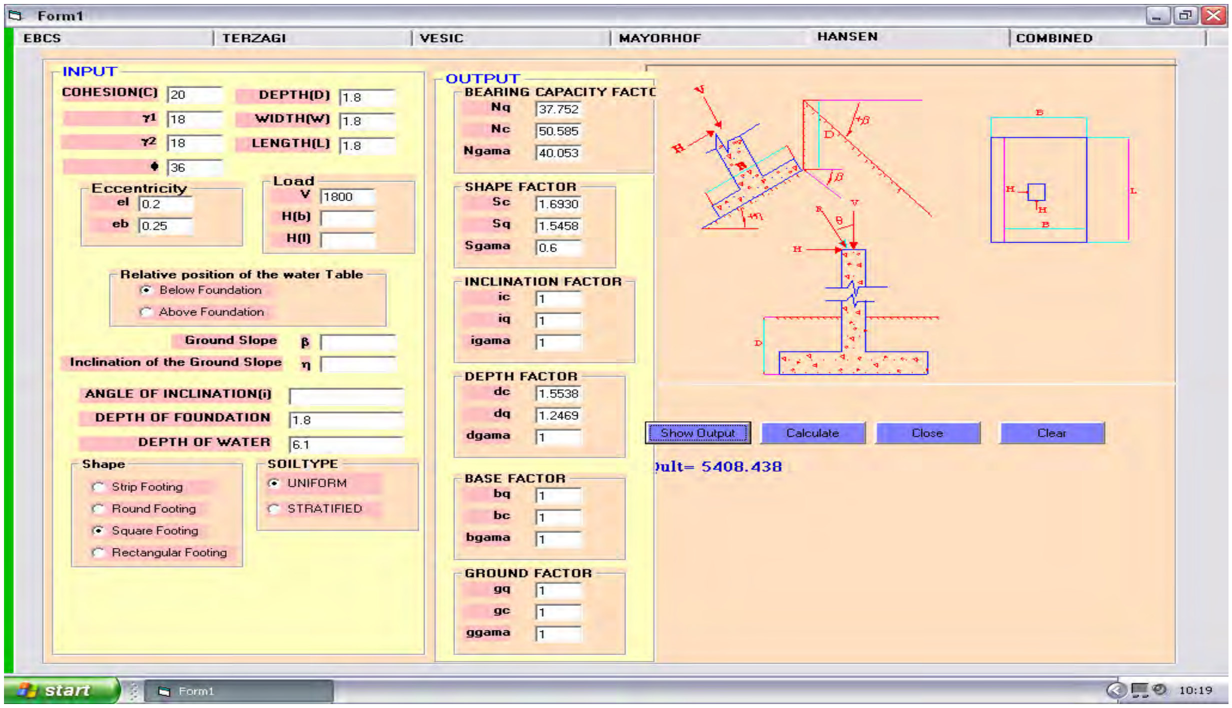
c) Load inclination Factor

- All $i_i = 1$ (not zero)

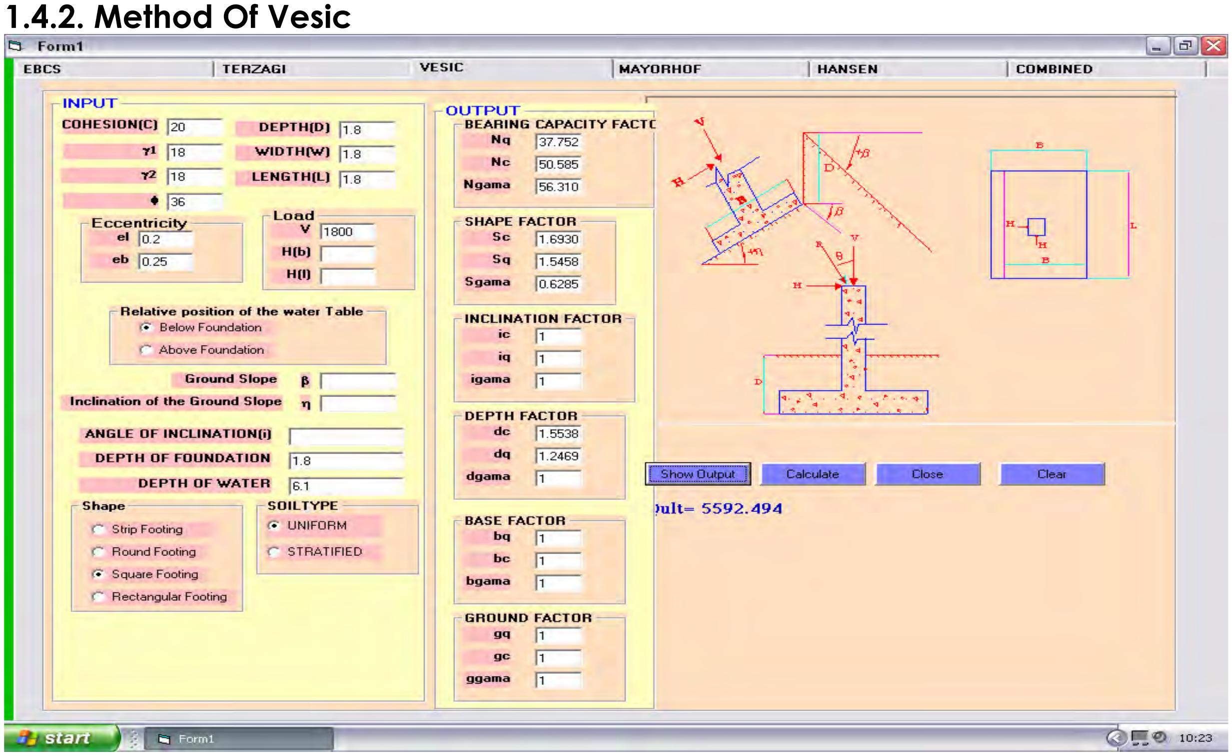
$$q_{ult} = (20 \times 51 \times 1.604) + (18 \times 1.8 \times 38 \times 1.588)$$
$$+ (0.5 \times 1.3 \times 18 \times 53.76 \times 0.7) = 4{,}031.52 KPa.$$

1.4. According To the Developed Software

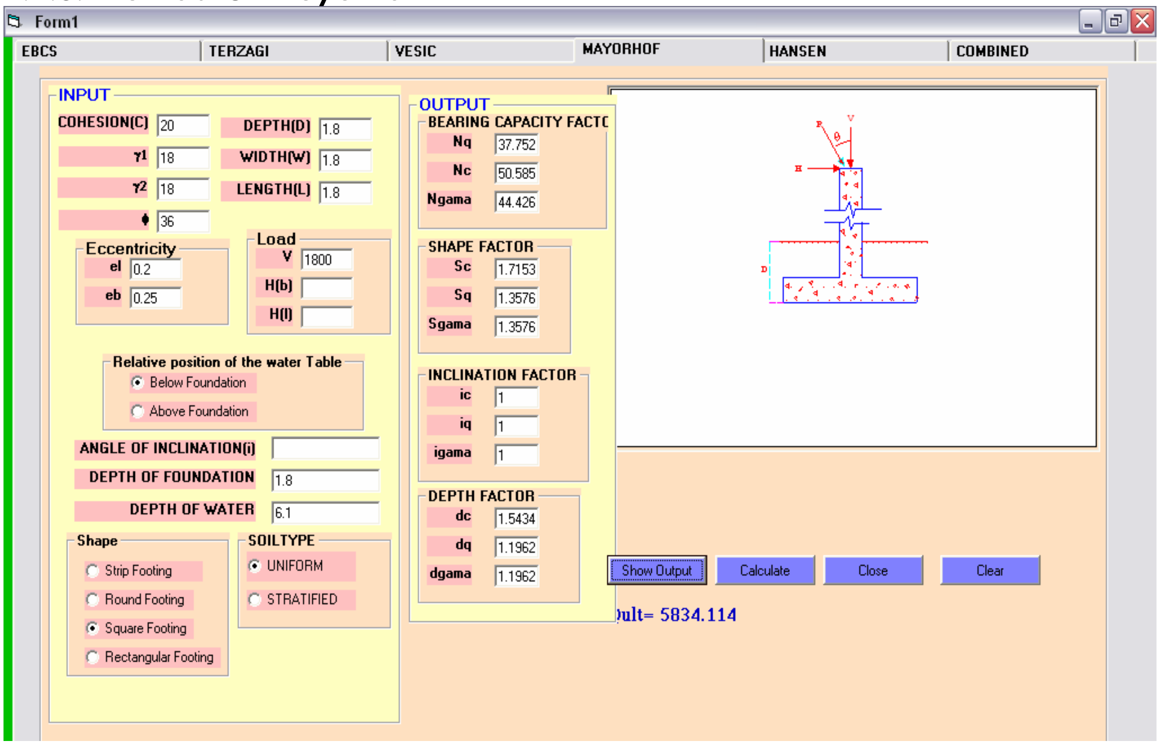
1.4.1. Method Of Hansen



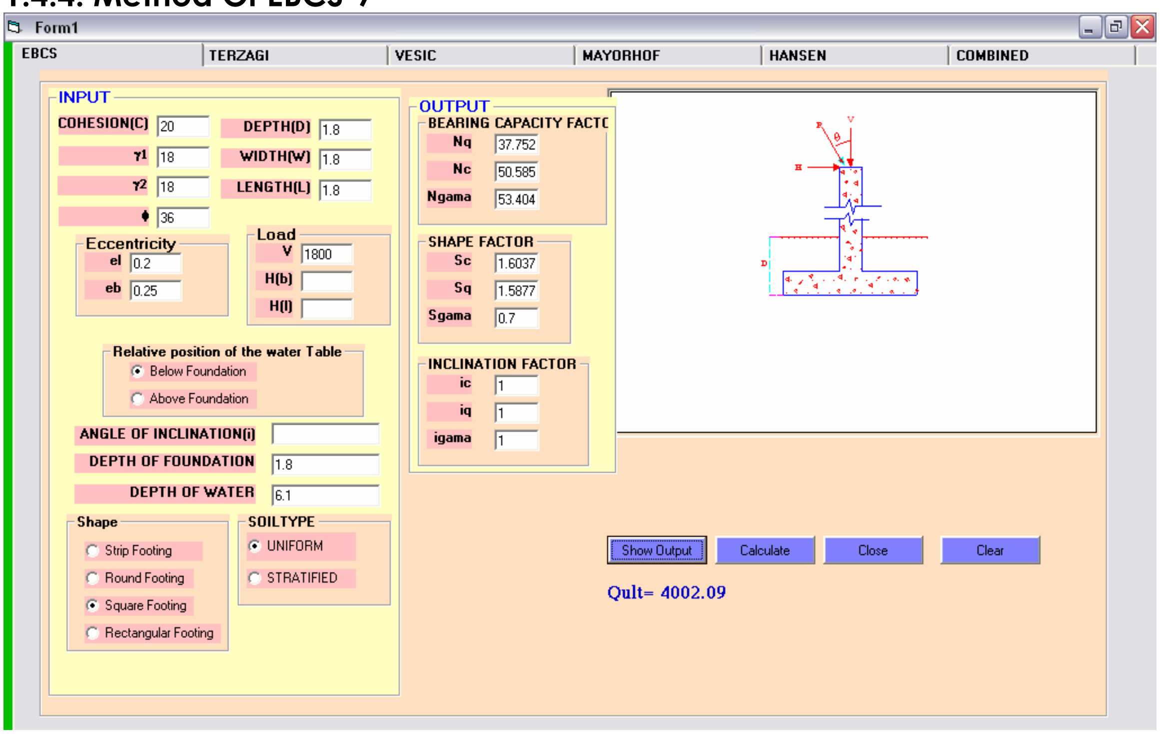
1.4.2. Method Of Vesic



1.4.3. Method Of Meyerhof



1.4.4. Method Of EBCS-7



Example 2: Comparison of long hand calculation and the program output								
	Long Hand Calculations	The program output	Long Hand Calculations	The program output	Long Hand Calculations	The program output	Long Hand Calculations	The program output
	EBCS-7	EBCS-7	Meyerhof	Meyerhof	Hansen	Hansen	Vesic	Vesic
q_u	4,031.52	4,002.09	5,284.5	5,247.42	5,093.61	5,050.17	5,403.6	5,366.22
N_c	51.00	50.585	51.00	50.585	51.00	50.585	51.00	50.585
N_q	38.00	37.752	38.00	37.752	38.00	37.752	38.00	37.752
N_γ	53.76	53.404	45.00	44.426	40.00	40.053	57.00	56.310
s_c	1.604	1.6037	1.715	1.7153	1.69	1.693	1.69	1.693
s_q	1.588	1.5877	1.358	1.3576	1.55	1.5458	1.67	1.6746
s_γ	0.7	0.7	1.358	1.3576	0.63	0.6285	0.63	0.6285
i_c	1	1	1	1	1	1	1	1
i_q	1	1	1	1	1	1	1	1
i_γ	1	1	1	1	1	1	1	1
d_c	-	-	1.392	1.3925	1.40	1.4000	1.40	1.40
d_q	-	-	1.196	1.1962	1.25	1.2469	1.25	1.2469
d_γ	-	-	1.196	1.1962	1	1	1	1
b_c	-	-	1	1	1	1	1	1
b_q	-	-	1	1	1	1	1	1
d_γ	-	-	1	1	1	1	1	1
g_c	-	-	1	1	1	1	1	1
g_q	-	-	1	1	1	1	1	1
g_γ	-	-	1	1	1	1	1	1

Examples 3 (Stratified Soil)

The loading and soil profile as the Fig.E.1 given below. It is required to determine the bearing capacity of the foundation.

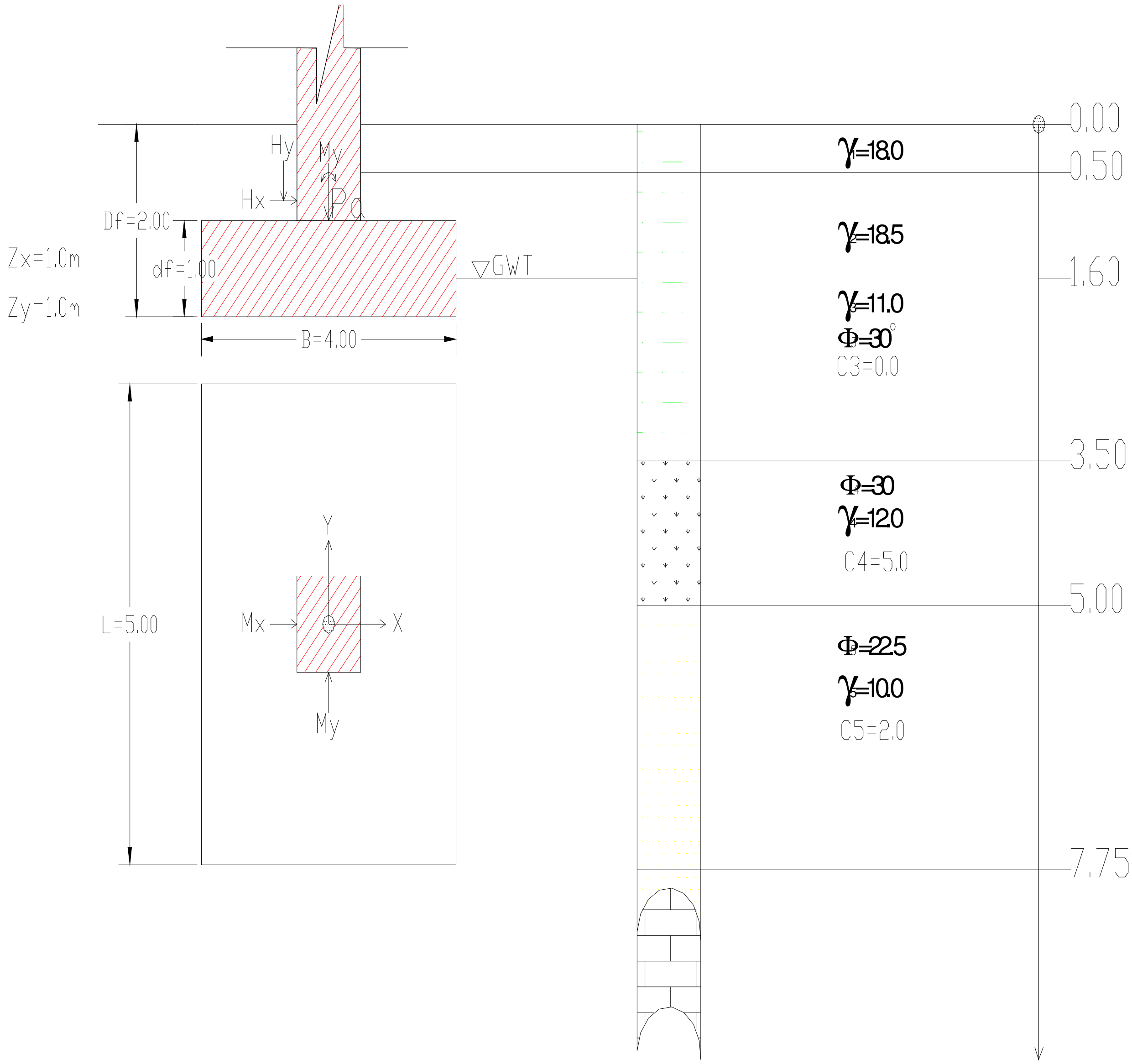


Fig.E.1, loading and Soil profile.

Solution:-

1.5 Long Hand Semi-graphical Iteration

I. Existing Loads

- Given Vertical load, $P_a = 6575.00KN$
- The weight of the foundation $= 4.00 \times 5.00 \times 1.00 \times 24.00$
  $= 480.00KN$
- Buoyant force $= -(4.00 \times 5.00 \times (2.00 - 1.60) \times 10.00)$
  $= -80.00KN$
- Total load, $P = (6575.00 + 480.00 - 80.00)$
  $= 6975.00KN$

II. Iterative calculation of the soil parameters.

- Mean value of $\phi_a = \dfrac{30 + 25 + 22.5}{3} = 25.8^0$
- The difference between the mean value (ϕ_a) and the respective values for each layer is less than 5^0. Hence, calculation using this method is permissible.

b) First iteration

- Start with $\phi_a = 30^0$ and determine the rupture line.

$$\beta = 45^0 - \frac{\phi_0}{2} = 45^0 - \frac{30^0}{2} = 30^0$$

$$\alpha = 45^0 + \frac{\phi_0}{2} = 45^0 + \frac{30^0}{2} = 60^0$$

Hence $\omega = 180^0 - (\beta + \alpha)$

$$= 180^0 - (30^0 + 90^0) = 90^0$$

$$r_0 = \frac{B}{Sin\,(90^0 - 30^0)} \sin\alpha = \frac{4.00}{\sin 60^0} \times \sin 60^0 = 4.00m$$

$$r_1 = r_o e^{(arc\,\omega \tan\phi)} = 4.00 \times e^{\left(90^0 \times \frac{\pi}{180^0} \times \tan 30^0\right)} = 9.91m$$

The distance of the EP6 (Fig. Ex.1-1st Iteration)

$= 2r_1 \cos\beta$

$= 2 \times 9.91 \times \cos 30^0$

$= 17.16m$

The depth of the rupture line:

$$Max.d_s = r_o e^{(arc\,\theta \tan\phi)} \cos\phi$$

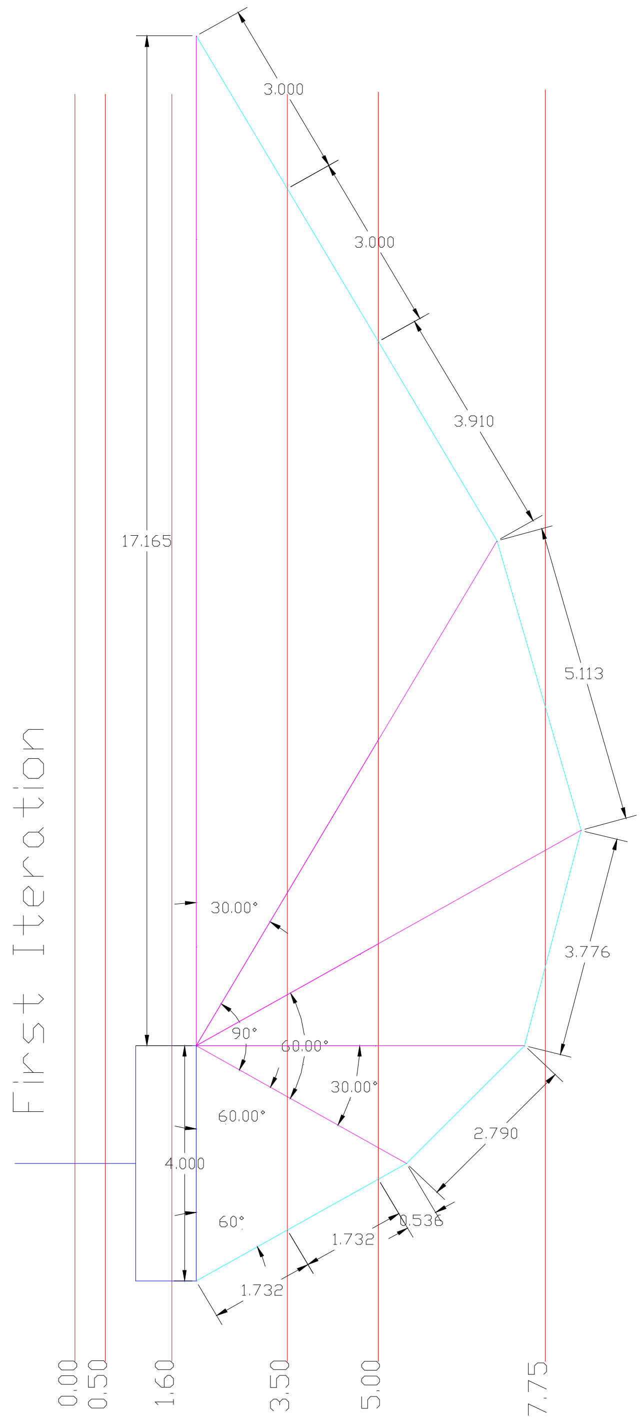
$$= 4.00 \times e^{\left(60^0 \times \frac{\pi}{180^0} \times \tan 30^0\right)} \cos 30^0$$
= 6.34m

Hence, the location of the deepest point of the rupture line is max d_s + d = 6.34 + 2.00 = 8.34m below the ground surface.

$$r_2 = 4.00 \times e^{\left(60^0 \times \frac{\pi}{180^0} \times \tan 30^0\right)} = 7.32m$$
$$r_3 = 4.00 \times e^{\left(30^0 \times \frac{\pi}{180^0} \times \tan 30^0\right)} = 5.41m$$

- Taking P1 as reference point for a coordinate system in X and Z. The coordinate of the point at which the rupture line cutes the horizontal soil layers are as follows:

$S3_l$ (0.87, 1.50)
$S3_r$ (18.56, 1.50)
$S4_l$ (1.73, 3.00)
$S4_r$ (15.96, 3.00)


First Iteration
3.000
3.000
3.910
5.113
3.776
2.790
0.538
1.732
1.732
17.165
4.000
30.00°
90°
60.00°
30.00°
60.00°
60°
0.00
0.50
1.60
3.50
5.00
7.75

- From these data, one may determine the following lengths.

$$l_3 = l3_l + l3_r = 1.732m + 3.00m = 4.732m$$
$$l_4 = l4_l + l4_r = 1.732m + 3.00m = 4.732m$$
$$l_5 = 0.536m + 2.790m + 3.776m + 5.113m + 3.910m = 16.12m$$
$$l_{Total} = 4.732m + 4.732m + 16.12m = 25.58m$$

- With these lengths, the first iteration for the weighted average angel of internal friction will be obtained.

$$\tan\phi_{ml} = \frac{l_3 \tan\phi_3 + l_4 \tan\phi_4 + l_5 \tan\phi_5}{l_3 + l_4 + l_5}$$
$$= \frac{4.732\times\tan 30^0 + 4.732\times\tan 25^0 + 16.12\times\tan 22.5^0}{25.58}$$
$$\phi_{ml} = 24.42^0$$

- The deviation from the initial value of $\phi_a = 30^0$

$$\Delta_1 = \left(\frac{30^0 - 24.42^0}{30^0}\right)\times 100\% = 18.6\%$$

- Since this deviation is greater than 5% it would be necessary to carry another iteration with the following angle of internal friction.

$$\phi_1 = \frac{24.42^0 + 30^0}{2} = 27.21^0$$

c) Second iteration

- Start with $\phi_a = 27.21^0$ and determine the rupture line.

$$\beta = 45^0 - \frac{\phi_0}{2} = 45^0 - \frac{27.21^0}{2} = 31.4^0$$
$$\alpha = 45^0 + \frac{\phi_0}{2} = 45^0 + \frac{27.21^0}{2} = 58.6^0$$

Hence $\omega = 180^0 - (\beta + \alpha)$

$$= 180^0 - (31.4^0 + 58.6^0) = 90^0$$

$$r_0 = \frac{B}{Sin\ (90^0 - 27.21^0)}\sin\alpha$$
$$= \frac{4.00}{Sin\ (62.79^0)}\sin 58.6^0 = 3.84m$$

$$r_1 = r_0\, e^{(arc\,\omega \tan\phi)} = 3.84 \times e^{\left(90^0 \times \frac{\pi}{180^0} \times \tan 27.21^0\right)} = 8.70m$$

$$Max.d_s = r_o e^{(arc\,\theta \tan\phi)} \cos\phi$$

$$= 3.84 \times e^{\left(60^0 \times \frac{\pi}{180^0} \times \tan 27.21^0\right)} \times \cos 27.21^0 = 5.85m$$

$$r_2 = 3.84 \times e^{\left(60^0 \times \frac{\pi}{180^0} \times \tan 27.21^0\right)} = 6.58m$$

$$r_3 = 3.84 \times e^{\left(30^0 \times \frac{\pi}{180^0} \times \tan 27.21^0\right)} = 5.05m$$

- From these data, one may determine the following lengths.

$$l_3 = l3_l + l3_r = 1.757m + 2.879m = 4.636m$$

$$l_4 = l4_l + l4_r = 1.757m + 2.879m = 4.636m$$

$$l_5 = 0.324m + 2.581m + 3.354m + 4.453m + 2.942m = 13.654m$$

$$l_{Total} = 4.636m + 4.636m + 13.654m = 22.926m$$

- With these lengths, the first iteration for the weighted average angel of internal friction will be obtained.

$$\tan\phi_{ml} = \frac{l_3 \tan\phi_3 + l_4 \tan\phi_4 + l_5 \tan\phi_5}{l_3 + l_4 + l_5}$$

$$= \frac{4.636 \times \tan 30^0 + 4.636 \times \tan 25^0 + 13.654 \times \tan 22.5^0}{22.926}$$

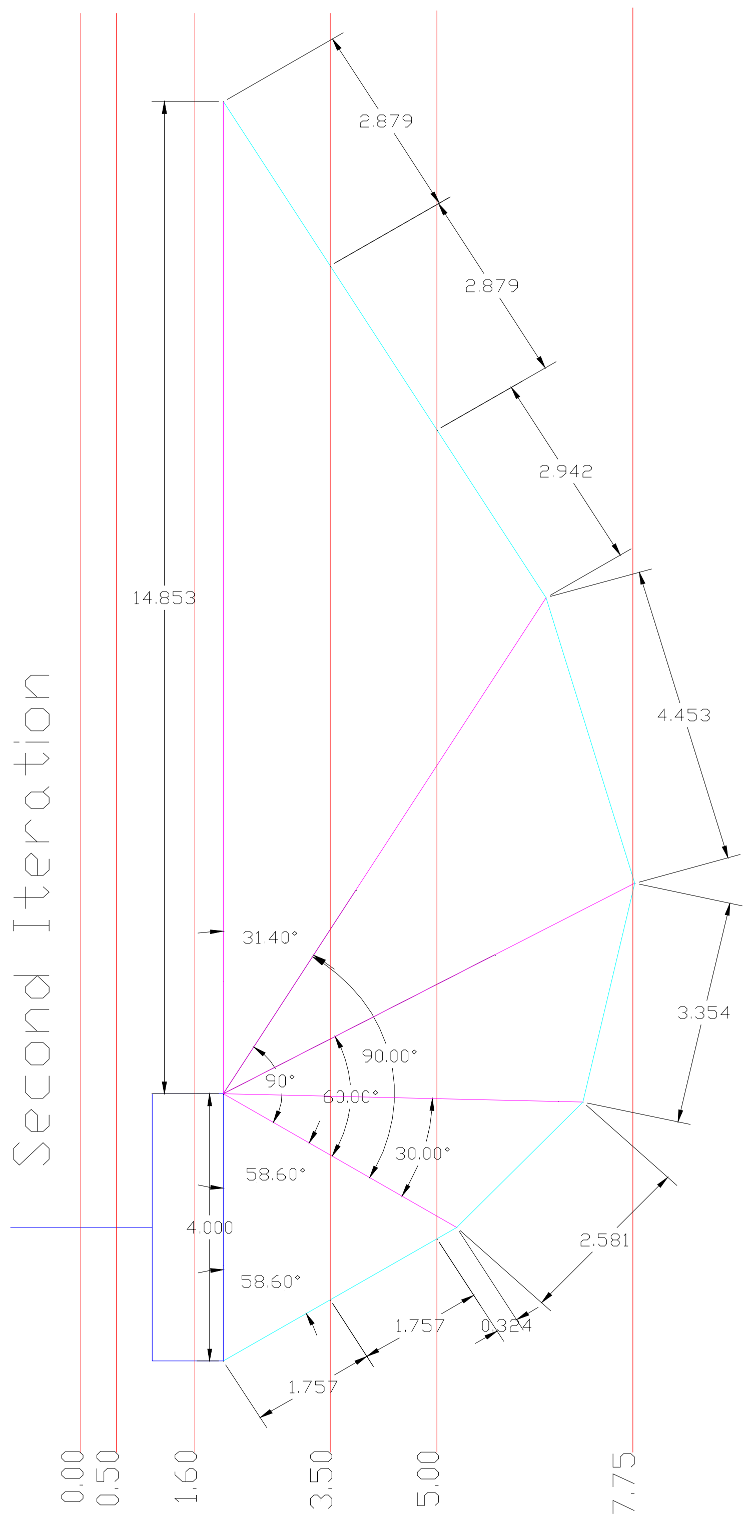
$$\phi_{m2} = 24.60^0$$

- The devation from the initial value of $\phi_2 = 27.21^0$

$$\Delta = \left(\frac{27.21^0 - 24.60^0}{27.21^0}\right) \times 100\% = 9.78\%$$

- Since this deviation is greater than 5% it would be necessary to carry another iteration with the following angle of internal friction.

$$\phi_3 = \frac{27.21^0 + 24.60^0}{2} = 25.91^0$$



Second Iteration
14.853
2.879
2.879
2.942
4.453
3.354
2.581
0.324
1.757
1.757
4.000
31.40°
90.00°
90°
60.00°
30.00°
58.60°
58.60°
0.00
0.50
1.60
3.50
5.00
7.75

d) Third iteration

- Start with $\phi_3 = 25.91^0$ and determine the rupture line.

$$\beta = 45^0 - \frac{\phi_0}{2} = 45^0 - \frac{25.91^0}{2} = 32.05^0$$

$$\alpha = 45^0 + \frac{\phi_0}{2} = 45^0 + \frac{25.91^0}{2} = 57.95^0$$

Hence $\omega = 180^0 - (\beta + \alpha)$

$$= 180^0 - (32.05^0 + 57.95^0) = 90^0$$

$$r_0 = \frac{B}{Sin\ (90^0 - 25.91^0)} \sin\alpha$$

$$= \frac{4.00}{Sin\ (64.09^0)} \sin 57.95^0 = 3.77m$$

$$r_1 = r_o e^{(arc\ \omega \tan\phi)}$$

$$= 3.77 \times e^{\left(90^0 \times \frac{\pi}{180^0} \times \tan 25.91^0\right)} = 8.09m$$

$$r_2 = 3.77 \times e^{\left(60^0 \times \frac{\pi}{180^0} \times \tan 25.91^0\right)} = 6.27m$$

$$r_3 = 3.77 \times e^{\left(30^0 \times \frac{\pi}{180^0} \times \tan 25.91^0\right)} = 4.86m$$

- From these data, one may determine the following lengths.

$$l_3 = l3_l + l3_r = 1.770m + 2.827m = 4.597m$$

$$l_4 = l4_l + l4_r = 1.770m + 2.827m = 4.597m$$

$$l_5 = 0.229m + 2.469m + 3.218m + 4.111m + 2.437m = 12.464m$$

$$l_{Total} = 4.597m + 4.597m + 12.464m = 21.658m$$

- With these lengths, the first iteration for the weighted average angel of internal friction will be obtained.

$$\tan\phi_{ml} = \frac{l_3 \tan\phi_3 + l_4 \tan\phi_4 + l_5 \tan\phi_5}{l_3 + l_4 + l_5}$$

$$= \frac{4.597 \times \tan 30^0 + 4.597 \times \tan 25^0 + 12.467 \times \tan 22.5^0}{21.658}$$

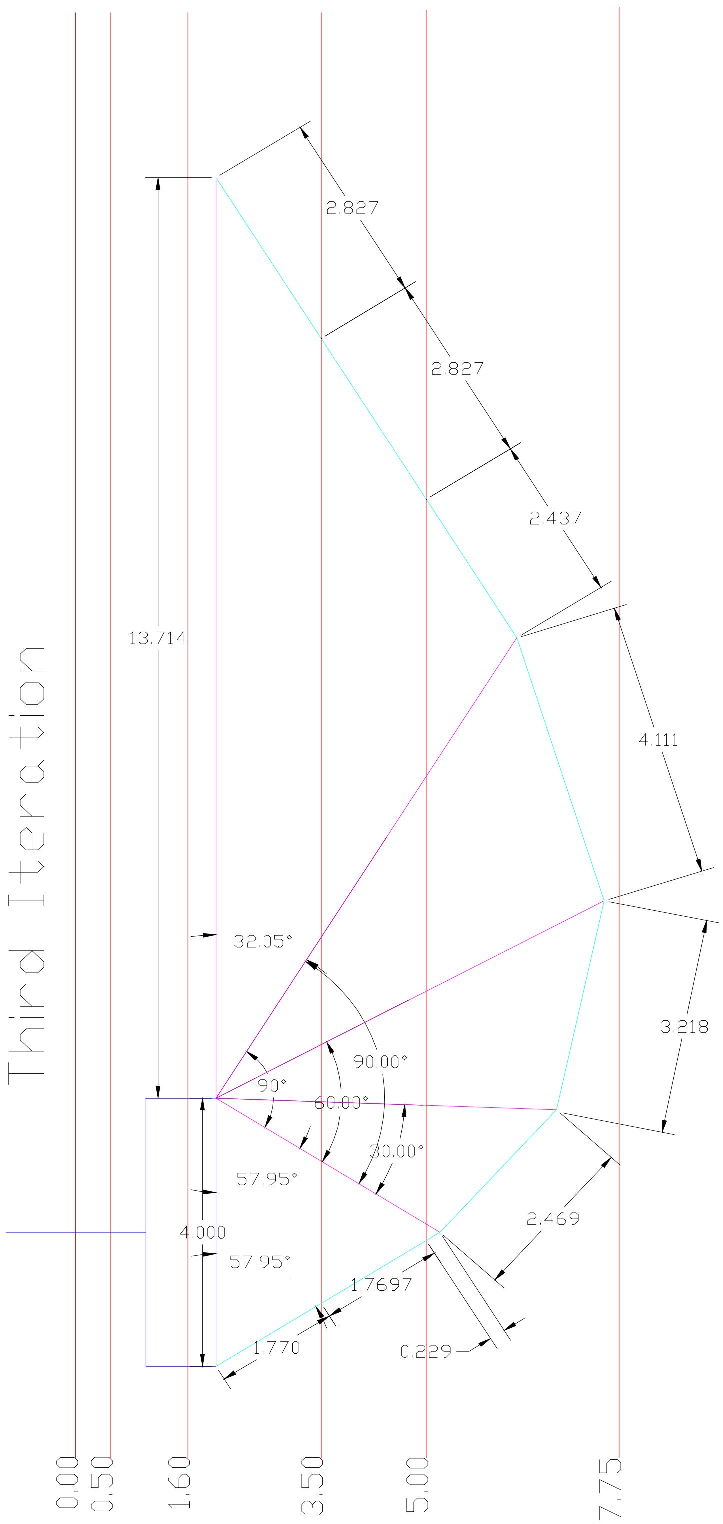
$$\phi_{ml} = 24.70^0$$

- The devation from the initial value of $\phi_3 = 25.91^0$

$$\Delta = \left(\frac{25.91^0 - 24.70^0}{25.91^0} \right) 100\% = 4.67\%$$

- Since this deviation is almost 5% it would be necessary to carry another iteration to have better accuracy with the following angle of internal friction.

$$\phi_4 = \frac{24.70^0 + 25.91^0}{2} = 25.31^0$$


Third Iteration
13.714
2.827
2.827
2.437
4.111
3.218
2.469
32.05°
90°
60.00°
90.00°
30.00°
57.95°
4.000
57.95°
1.7697
1.770
0.229
0.00
0.50
1.60
3.50
5.00
7.75

e) Fourth iteration

- Start with $\phi_a = 25.31^0$ and determine the rupture line.

$$\beta = 45^0 - \frac{\phi_0}{2} = 45^0 - \frac{25.31^0}{2} = 32.35^0$$

$$\alpha = 45^0 + \frac{\phi_0}{2} = 45^0 + \frac{25.31^0}{2} = 57.65^0$$

Hence $\omega = 180^0 - (\beta + \alpha)$

$$= 180^0 - \left(32.35^0 + 57.65^0\right) = 90^0$$

$$r_0 = \frac{B}{Sin\ (90^0 - 25.31^0)} \sin\alpha$$

$$= \frac{4.00}{Sin\ (64.69^0)} \sin 57.62^0 = 3.74m$$

$$r_1 = r_o e^{(arc\ \omega \tan\phi)}$$

$$= 3.74 \times e^{\left(90^0 \times \frac{\pi}{180^0} \times \tan 25.31^0\right)} = 7.86m$$

$$Max.d_s = r_0 e^{(arc\,\theta * \tan\phi)} \cos\phi$$

$$= 3.74 \times e^{\left(60^0 * \pi / 180^0 * \tan 25.31^0\right)} \times \cos 25.31 = 5.55m$$

$$r_2 = 3.74 \times e^{\left(60^0 \times \frac{\pi}{180^0} \times \tan 25.31^0\right)} = 6.14m$$

$$r_3 = 3.74 \times e^{\left(30^0 \times \frac{\pi}{180^0} \times \tan 25.31^0\right)} = 4.79m$$

- From these data, one may determine the following lengths.

$$l_3 = l3_l + l3_r = 1.776m + 2.803m = 4.579m$$

$$l_4 = l4_l + l4_r = 1.776m + 2.803m = 4.579m$$

$$l_5 = 0.188m + 2.43m + 3.109m + 3.981m + 2.258m = 11.971m$$

$$l_{Total} = 4.553m + 4.553m + 11.966m = 21.129m$$

- With these lengths, the first iteration for the weighted average angel of internal friction will be obtained.

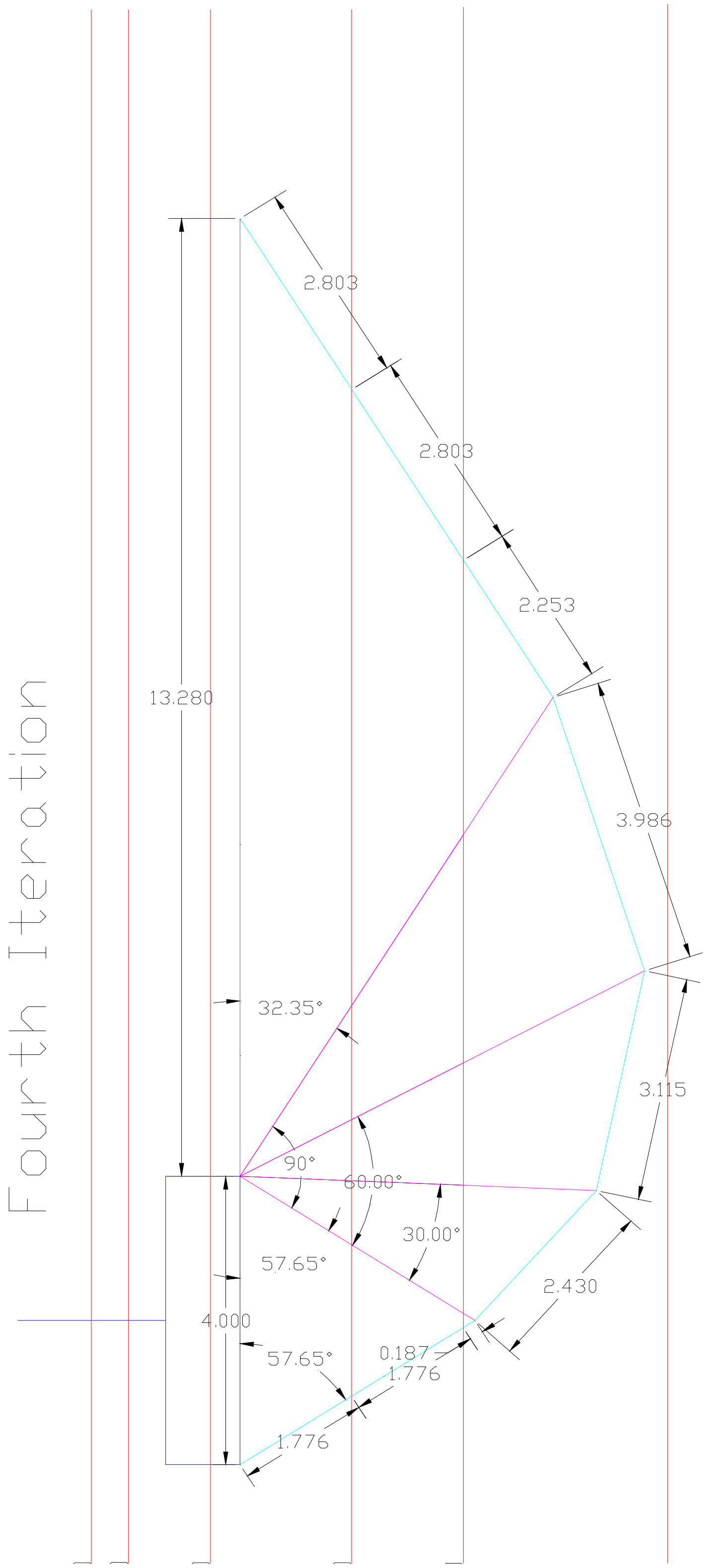
$$\tan\phi_{m4} = \frac{l_3 \times \tan\phi_3 + l_4 \times \tan\phi_4 + l_5 \times \tan\phi_5}{l_3 + l_4 + l_5}$$

$$= \frac{4.579 \times \tan 30^0 + 4.579 \times \tan 25^0 + 11.971 \times \tan 22.5^0}{21.129}$$

$$\phi_{m4} = 24.74^0$$

- The devation from the initial value of $\phi_4 = 25.31^0$

$$\Delta = \left(\frac{25.31^0 - 24.74^0}{25.31^0} \right) \times 100\% = 2.25\%$$


Fourth Iteration
13.280
2.803
2.803
2.253
3.986
3.115
32.35°
90°
60.00°
30.00°
57.65°
57.65°
4.000
2.430
0.187
1.776
1.776

- Since this deviation is less than 5% it would be necessary to stop the iteration and to continue the next calculation

$$\phi = \frac{25.31^0 + 24.74^0}{2} = 25.03^0 \approx 25.00^0$$

III. Bearing Capacity Calculation.

a) Weighted mean Cohesion C would be determined as follow:Use the following data: ϕ = 25.0^0

l_3 = 4.579m

l_4 = 4.579m

l_5 = 11.971m

l_{total} = 21.129m

$$c = \frac{l_3 \times c_3 + l_4 \times c_4 + l_5 \times c_5}{l_{total}}$$

$$c = \frac{4.579 \times 0 + 4.579 \times 5 + 11.971 \times 2}{21.129} = 2.22 \, KN/m^2$$

f) Weighted unit weights under the foundation levels will be as follow:

- $A_3 = \frac{1}{2}(0.950 + 2.386)1.50 + 13.962 \times 1.50 = 23.43 m^2$
- $A_4 = \frac{1}{2}(0.95 + 2.368)1.50 + 10.644 * 1.50 = 18.45 m^2$
- $A_5 = \frac{1}{2}(1.786 + 1.206)1.895 + (4.959 \times 1.786) + \frac{1}{2}(1.890 \times 1.786)$
  $+ \frac{1}{2}(1.904 \times 1.206) + \frac{1}{2}(4.954 \times 0.653) = 16.14 m^2$
- $A_{total} = 58.02 m^2$
- $$\gamma_2 = \frac{A_3 \times \gamma_3 + A_4 \times \gamma_4 + A_5 \times \gamma_5}{A_{total}}$$
  $$= \frac{23.43 \times 11.00 + 18.45 \times 12 + 16.14 \times 10}{58.02} = 11.04 \, KN/m^3$$

g) Weighted unit weights above the foundation levels will be as follow:

- $$\gamma_1 = \frac{0.5 \times 18.00 + 1.1 \times 18.5 + 0.4 \times 11.00}{(0.5 + 1.1 + 0.40)} = 16.875 \, KN/m^3$$

h) Finally the bearing capacity of the soil will be determined using the following data: ϕ = 25.0^0

$c = 2.22KN/m^2$

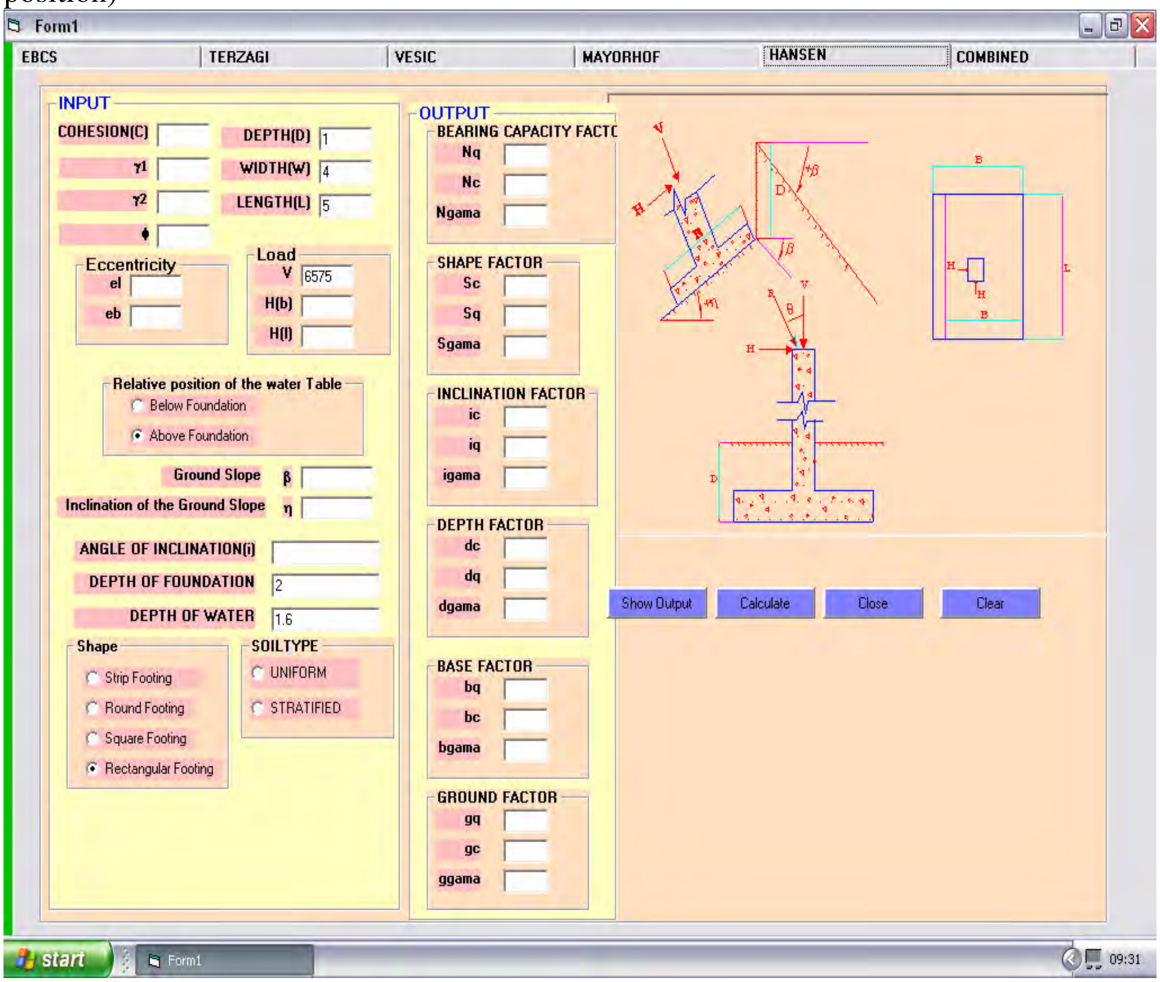
$\gamma_1 = 16.88\ KN/m^3$

$\gamma_2 = 11.04KN/m^3$

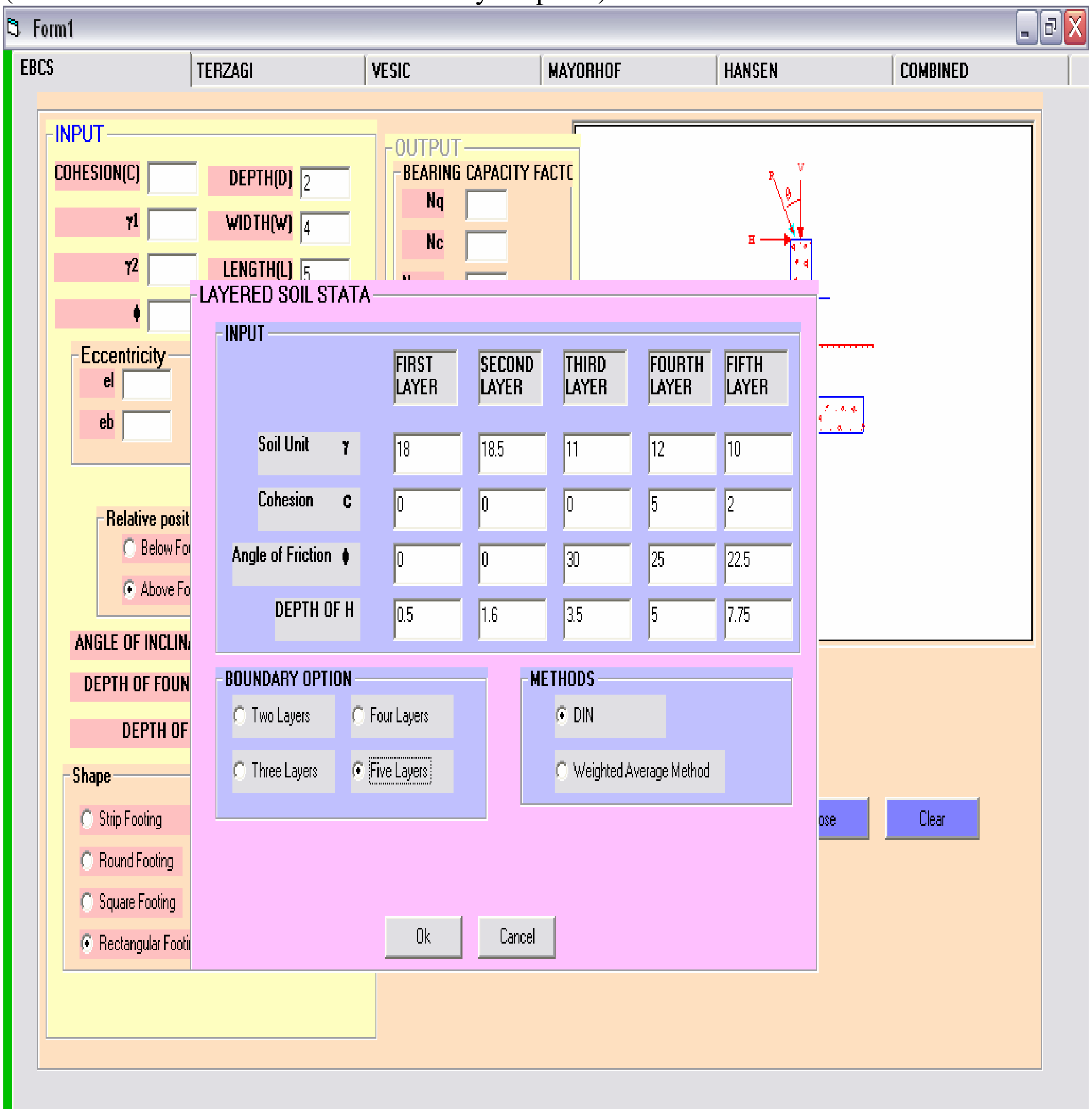
and using the following methods

1.6. According To the Developed Software(for stratified soil layer)

(When the user fills the geometry of the foundation, the loading condition and the water position)



(When the user selects Stratified soil layer option)



(When the user selects Combined option and finaly selects calculate and show the output)

Bearing capacity calculation result using different methods

Vertical Load (KN)=			6575				
Horizontal Load (KN)=		(Along B)=	0				
		(Along L)=	0				
TITA =	0						
ALFA =	0						
GAMA1 =	16.88						
GAMA2 =	11.04						
C =	2.22						
Depth =	1	Length(L)=	5	WIDTH(B)=	4	Diameter(B)=	4

		EBCS-7	Terzaghi	Meyerhof	Hansen	Vesic	Average
Bearing capacity fa.	Nq	10.72	12.8	10.72	10.72	10.72	11.14
	Nc	20.8	25.24	20.8	20.8	20.8	21.69
	Ngama	9.09	9.79	6.83	6.82	10.96	8.7
shape factors	Sq	1.34	1.34	1.2	1.34	1.34	1.31
	Sc	1.37	1.24	1.4	1.41	1.41	1.37
	Sgama	0.76	0.8	1.2	0.68	0.68	0.82
depth factors	dq	0	0	1.08	1.16	1.16	0.68
	dc	0	0	1.16	1.2	1.2	0.71
	dgama	0	0	1.08	1	1	0.62
inclination fact.	iq	1	1	1	1	1	1
	ic	1	1	1	1	1	1
	igama	1	1	1	1	1	1
Ground factors	gq	0	0	0	1	1	0.4
	gc	0	0	0	1	1	0.4
	ggama	0	0	0	1	1	0.4
base factors	bq	0	0	0	0	0	0
	bc	0	0	0	0	0	0
	bgama	0	0	0	0	0	0
	Q-ult	700.36	674.1	736.55	740.27	802.47	730.75

APPENDIX-2

2.1. Manual for BEARING program

When the computer package BEARING is selected, the BEARING user interface will appear. The BEARING user interface contains labels, text boxes, combo boxes, option buttons and command buttons. The functions of each of these will be described below:

1. Frame

A Frame control provides an identifiable grouping for controls. You can also use a Frame to subdivide a form functionally for example, to separate groups of Option Button controls.

2. Labels

The function of labels is to describe the name of input or output parameters

3. Text boxes

Text boxes are blank spaces where the user writes the input or where outputs are displayed

5. Option buttons

When an option button is checked, a selection is made among the different options provided

6. Command buttons

Command buttons are used to enter input data to the combo boxes and also to cause some actions such as calculation of the soil Bearing capacity. The following procedures are followed to calculate the soil Bering capacity using EBCS-7, TERZAGI, VECIC, MAYERHOF and HANSEN methods:

1. Fill all the input data on the input text boxes and select the appropriate option button (main interface). When filling the input data use the following units:
    - KN/m^3 for unit weight
    - KN/m^2 for cohesion
    - KN for Vertical and Horizontal Load
    - Meter for all lengths
    - Degree for all angles
2. Select the soil type options (Uniform, or stratified)
3. Again, fill all the input data on the input text boxes of the second interface and click ‘Ok’ (it is only for stratified soil layer option).
4. Click ‘’Calculate’’ and then ‘’Show output’’ to get the result of the bearing capacity of the soils.
5. To see all the output on excel spreadsheet format the user should select combined option and finally click “Calculation” and then “Show output” command.

APPENDIX-3

Programming Code