BEARING CAPACITY OF MODEL FOOTINGS ON SAND

A PROJECT SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

By

K.R.Srivatsan

&

Satyabrata behera

Bachelor of Technology

In

CIVIL Engineering

Department of Civil Engineering

National Institute of Technology

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Under the guidance of

Prof. N.R.Mohanty

Department of Civil Engineering

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CERTIFICATE

This is to certify that the project entitled **"Bearing capacity of model footings on sand''** submitted by **K.R.Srivatsan, Roll No: 10301032** and **Satyabrata Behera, Roll No: 10301009** in the partial fulfillment of the requirement for the award of **Bachelor of Technology** in **Civil Engineering**, National Institute of Technology, Rourkela, is being carried out under my supervision.

To the best of my knowledge the matter embodied in the project has not been submitted to any other university/institute for the award of any degree or diploma.

Prof. N.R.Mohanty

Department of Civil Engineering

National Institute of Technology

Rourkela.

Date:

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ABSTRACT

Soil mechanics engineering is one of most important aspects of civil engineering involving the study of soil , its behaviour and application as an engineering material.good soil engineering embodies the use of the best practices in exploration,testing ,design and construction control,in addition to the basic idealized theories. with increasing load on soil due to construction of multi storeyed buildings there is a need to construct footing by conducting a test of their model in laboratory on the soil over which the foundation is to be laid.

Sand is one of the soils over which foundations are laid ,so it is necessary to conduct experiments by placing different model footings over sand and find out their ultimate bearing capacity and based on these values ,it can be incorporated on to the field and foundations can be laid. Square footings of different sizes are taken and model testing of these footings are conducted and the ultimate bearing capacity of different footings are found and on the basis of these values foundations are laid on sandy soils .these values can also be compared with theoretical analysis of Terzaghi and Meyerhof ‘s to check out the difference in values of ultimate bearing capacity between a theoretical and practical analysis.

LIST OF FIGURES:

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CHAPTER 1

DEFINITIONS

1. Definitions:

Bearing capacity:-

The supporting power of a soil or rock id referred to as its bearing capacity. The term bearing capacity is defined after attaching certain qualifying prefixes, as defined below.

Gross pressure intensity (q):

The gross pressure intensity q is the total pressure at the base of the footing due to the weight of the superstructure, self-weight of the footing and the weight of the earth fill, if any.

Net pressure intensity (q_n):

It is defined as the excess pressure , or the difference in intensities of the structure and the original overburden pressure. The construction of the structure and the effective overburden pressure. if, D is the depth of the footing

$$q_n = q - \gamma d$$

$\gamma =$ Average unit weight of soil above the foundation base.

Ultimate bearing capacity (q_f):

The ultimate bearing capacity is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear.

Net ultimate bearing capacity (q_{nf}):

It is the minimum net pressure intensity causing shear failure of the soil. the ultimate bearing capacity q_f and the net ultimate bearing capacity are connected by the following relation :

$$q_f = q_{nf} + \sigma -$$

Net safe bearing capacity (q_{ns}):

The net safe bearing capacity is the net ultimate bearing capacity divided by the factor of safety F

$$q_{ns} = \frac{q_{nf}}{f}$$

Safe bearing capacity (q_s):

the maximum pressure which the soil can carry without risk of shear failure is called the safe bearing capacity .it is equal to the net safe bearing capacity plus original overburden pressure:

$$q_s = \frac{q_{nf}}{f} + \gamma d$$

Sometimes the safe bearing capacity is also referred as the ultimate bearing *capacity* divided by a factor of safety f.

CHAPTER 2

METHODS OF FINDING OUT BEARING CAPACITY

2. Methods of finding out bearing capacity:

There are various methods to find out bearing capacity ,some of the methods are

1. Determination of building capacity by building code method

2. By plate load test

3. Theoritical analysis

Theoretical analysis is done by two methods, they are

1. Terzaghi's analysis.

2. Meyerhof's analysis

CHAPTER 3

TEST ON A MODEL FOOTING

3. Test on a model footing:

The ultimate bearing capacity of a soil and the probable settlement under a given loading is found out by testing the soil on various sizes of model footings.the test essentially consists in loading a rigid plate at the foundation level and determining the settlements corresponding at each load increment. the ultimate bearing capacity is then taken as the load at which the plate starts sinking at a rapid rate .the method assumes that down to the depth of influence of stresses ,the soil strata is reasonably uniform.

The bearing plate is square of minimum recommended size 30 cm square and maximum size recommended is 75 cm square. the plate is machined on sides and edges and should have a thickness sufficient to withstand effectively the bending stresses that would be caused by maximum anticipated load. the thickness of steel plate should not be less than 25 mm.

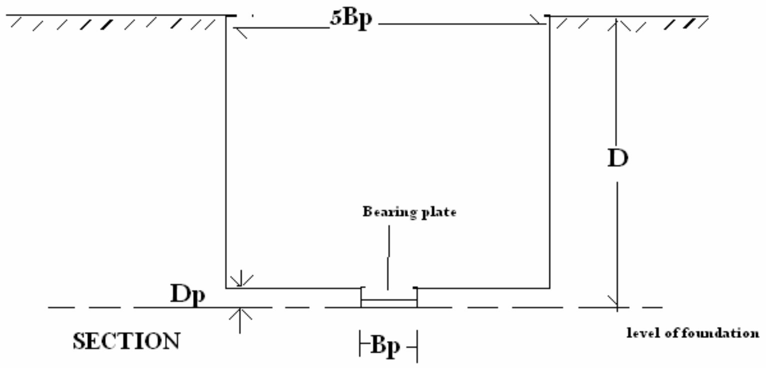
the test pit width is made five times the width of the plate b_p . at the centre of the pit, a small square hole is dug whose size is equal to the size of the plate and the bottom level of which correspond to the level of actual depth formation . the depth d_p of the hole should be such that

$$\frac{d_p}{b_{[p}} = \frac{foundationdepth}{foundationwidth} = \frac{D}{B}$$

3.1Plate load test:

the loading to the test plate may be applied with the help of a hydraulic jack. the reaction of the hydraulic jack may be borne either of the following two methods

1. Gravity loading platform method
2. Reaction truss method



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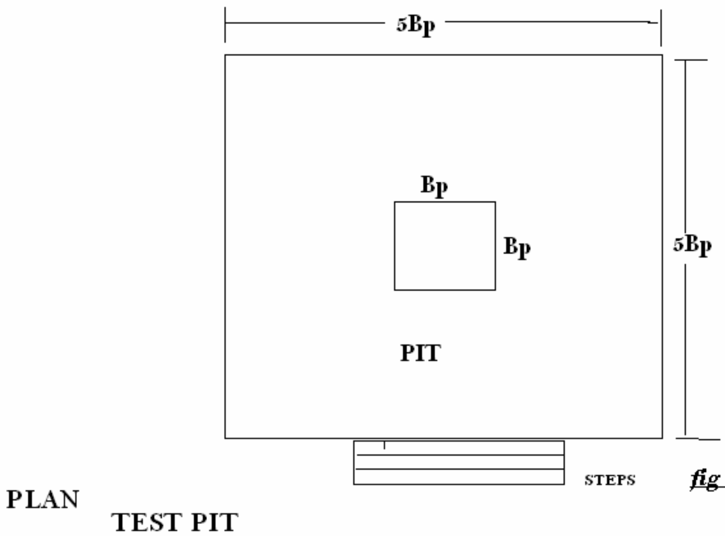

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fig no 3.2

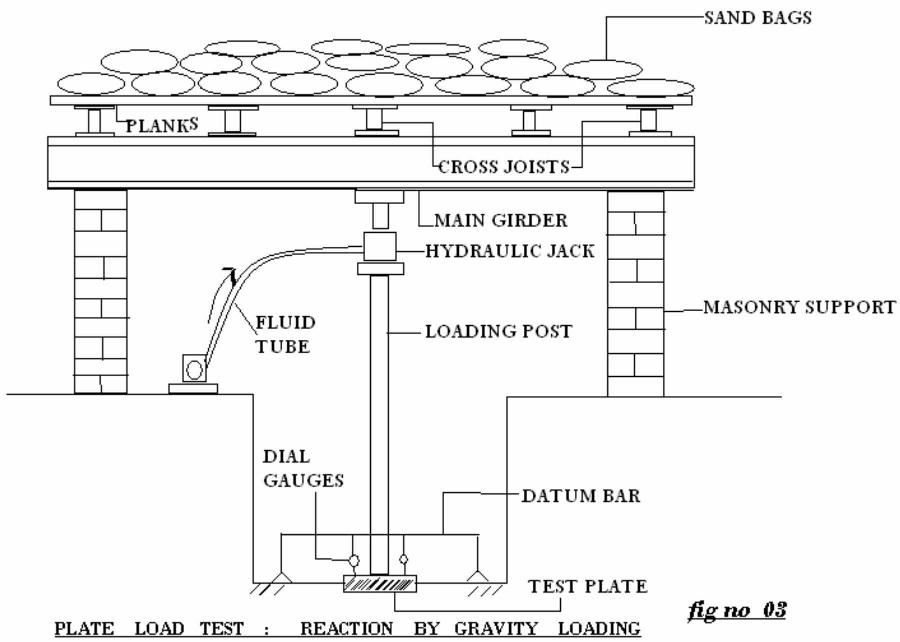


fig no 3.3

In case of gravity loading method, a platform is constructed over a vertical column resting on the test plate and the loading is done with the help of sand bags ,stones or concrete blocks.the general arrangement of the test set-up for this method is shown

When load is applied to the plate,it sinks or settles. the settlement of the plate is measured with the help of sensitive dial gauges .for square plates , two dial gauges are used .the dial gauges are mounted in independently supported datum bar. as the plate settles ,the ram of the dial gauge moves down and the settlement is recorded . the load is indicated on the load-gauge of the hydraulic jack.

The below figure shows the arrangement when the reaction of the jack is borne by a reaction truss. The truss held to the ground through soil anchors. these anchors are firmly driven in the soil with the help of hammers .the reaction truss is usually made of mild steel sections .guy ropes are used for the lateral stability of the truss.

Indian standard code (IS: 1888-1962) recommends that the loading of the plate should invariably be borne either by gravity or loading platform or by the reaction truss.the use of the reaction truss is more popular now-a-days since this is simple ,quick and less clumsy.

3.2Test procedure:

The plate is firmly seated in the hole and if the ground is slightly uneven, a thin layer of sand is spread underneath the plate. Indian standard (IS:1888-1962) recommends a seating load of 70 g\cm^2 which is released before the actual test is started.the load is applied with the help of a hydraulic jack in convenient increments, say of about one-fifth of the expected safe bearing capacity or one-tenth of the ultimate bearing capacity. settlement of the plate is observed by 2 dial gauges fixed at diametrically opposite ends,with sensitivity of .02 mm .Settlement of the plate is observed for each increment of load after an interval of 1,4,10,20,40 and 60 minutes and thereafter at hourly intervals until the rate of settlement becomes less than about 0.02 mm per hour. After this,the next load increment is applied. the maximum load that is to be applied corresponds to $1\frac{1}{2}$ times the estimated ultimate load or to a 3 times the proposed allowable bearing pressure.

the water table has a marked influence on the bearing capacity of sandy or gravelly soil. if the water table is already above the level of the footing, it should be lowered by pumping and the bearing plate seated after the water table has been lowered just below the footing level.even if the water table is located above 1 m below the base level of the footing ,the load test should be made at the level of the water table itself.

The load intensity and settlement observation of the plate load test are plotted. curve 1 corresponds to general shear failure , curve 2 corresponds to local shear failure ,curve 3 is a typical of dense cohesionless soils which do not show any marked sign of shear failure under the loading intensities of the test. Is: 1888-1962 recommends a log-log plot giving two straight lines the intersection of which may be considered the yield value of the soil.when a load settlement curve does not indicate any marked breaking point failure may alternatively be assumed corresponding equal to one-fifth of the width of the test plate .In order to determine the safe bearing capacity it would be normally sufficient to use a factor of safety 2 or 2.5 on ultimate bearing capacity.

3.3 Limitations of plate load test:

1. the test reflects only the character of the soil located within a depth less than twice the width of the bearing plate .since the foundations are generally larger the settlement and resistance against shear failure will depend on the properties of a much thicker stratum.
2. it is essentially a short duration test , and hence the test does not give the ultimate settlement ,particularly in case of cohesive soil.
3.another limitation is the effect of size of foundation .for clayey soils the ultimate pressure for a large foundation is the same as that of the test plate .but in dense sandy soils the bearing capacity increases with the size in foundation and the test on smaller size bearing plates tend to give conservative values.

CHAPTER 4

BUILDING CODE METHOD

4. Building code method:

Before the 19^{th} century the framework for most of the buildings consisted of strong but somewhat flexible main walls interconnected by massive but equally flexible partition walls intersecting each other at right angles. since such buildings could stand large settlements without damage ,their builders gave little considerations to foundations other to increase the wall thickness of the base .the development of highly competitive during the 19^{th} century led to demand for large but inexpensive buildings. the types that developed was more sensitive to differential settlement than their predecessors. hence designers need found themselves in a need of more reliable procedures, applicable under soil conditions, for proportioning the footings of a given building in such a manner that they would all experience nearly the same settlement. to satisfy this need the concept of ”allowable soil pressure” was developed during the 1870's in several different countries. the concept was based on the obvious fact that under fairly similar soil conditions ,footings transmitted pressures of high intensity to the subsoil generally settled more than those transmitting pressures at low intensity .the pressure beneath the footings of all those footings that showed signs of damage due to settlement were considered too great for the given soil conditions.the values obtained for each type of soil for a given locality is given in the table below of allowable soil pressures that was subsequently incorporated into the building code governing construction in that locality.

The building codes do not offer any hint regarding the origin of the values,or explaining the meaning of the term “allowable soil pressure” .these omissions have fostered the belief that settlement will be uniform and of no consequence if the pressure on the soil beneath each footing is equal to allowable soil pressure. the size of loaded area and the type of building are considered immaterial. but because of various confusions the engineers assumed that wrong allowable pressures have been selected because the terms used to describe the soil in the field and the building codes did not have the same building . in order to avoid this difficulty ,it gradually became customary to select the soil pressure on the basis of the results of load tests.

Character of Foundation bed(tn\ft ^{2})	Akron 1920	Atlanta 1911	Boston 1926
Quick sand or alluvial soil	0.5	-	-
Soft or wet clay, atleast 15 cm thick	1	1	
Clay in thick beds			1
Hard clay		3-4	
Clay in thick beds always dry	4		
Rock	10	15	100
Gravel and coarse sand in thick beds	5		
Hard shale unexposed	6		

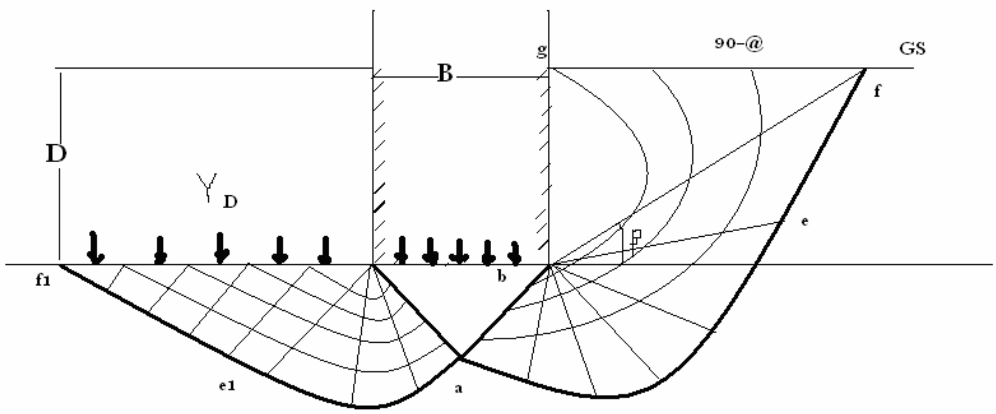
Table no: 4.1

CHAPTER 5

TERZAGHI'S ANALYSIS

5. Terzaghi's analysis:

an analysis of the condition of complete bearing capacity failure,usually termed general shear failure,can be made by assuming the soil behaves like an ideally plastic material. this concept was first developed by Prandtl and later extended by terzaghi. he considered a footing of width B and subjected to a loading intensity q_f to cause failure.the footing is shallow is equal to or less than width B of the footing. the loading soils fails along the composite surface $fede_1f_1$ .this region is divided into three zones zone1 ,two pairs of zone 2 and two pairs of zone 3. when the base of the footing sinks into the ground ,zone 1 is prevented from undergoing any lateral yield by the fraction and adhesion between the soil and the base of the footing. thus zone 1 remains in the state of elastic equilibrium and it acts as if it were a part of the footing .its boundaries da and db are assumed as plane surfaces ,rising at an angle $\varphi = \phi$ with the horizontal .zone 2 is called the zone of radial shear .these lines are straight while the lines of the other set are the logarithmic spirals with their located at the outer edges of the base of the footing .zone 3 is called the zone of linear shear,and is identical with that for rankines passive state .the boundary of zone 3 rise at $(45^o - \frac{\phi}{2})$ with the horizontal the failure zones are assumed not to extend above the horizontal plane through ab of the footing .this implies the shear resistance of the soil above the horizontal plane through the base of the footing is neglected ,and the soil above this plane is replaced with a surcharge $q = \gamma d$



TERZAGHI'S THEORY--------MEYERHOF'S THEORY

SHALLOW FOUNDATION *fig no 04*

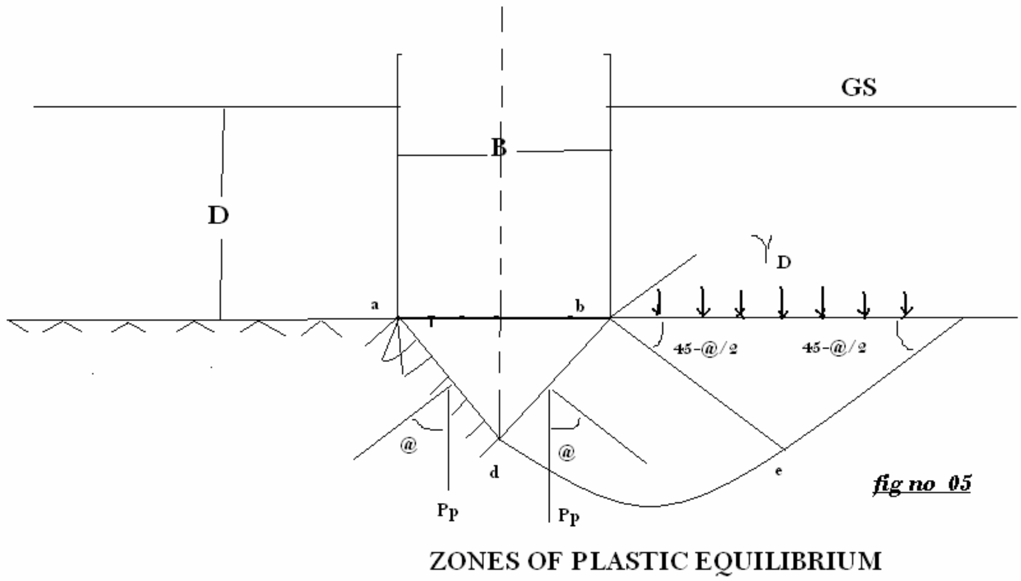


Fig no 5.1

the application of the load intensity q_f on the footing tends to push the wedge of the soil abd into the ground with lateral displacements of zone 2 and zone 3 but this lateral displacement is resisted by forces on the plane db and da . these forces are : 1. the resultant of the passive pressure p_p and 2. the cohesion c acting along the surface da and db . the passive pressure resultant makes an angle ϕ with the normal to the surfaces da and db .if it is assumed that surfaces da and db intersect the horizontal line at an angle ϕ,the passive pressure acts vertically. at the instant of failure ,the downward and upward forces are (i) q_f B and (ii)the weight $\frac{1}{4}\gamma B^2 \tan\phi$ of the wedge .the upward forces are (i) the resultant pressure p_a on each of the surfaces da and db (ii) the vertical component of cohesion acting along the lengths ad and bd .the length db = da= $\frac{b/2}{\cos\phi}$

so by equating forces

$$q_f b + \frac{1}{4}\gamma b^2 \tan\phi = 2p_p + 2\frac{b}{2}c\tan\phi \cdots\cdots 1$$

$$q_f b = 2(p_{p\gamma} + p_{pc} + p_{pq}) + B.c\tan\phi - \frac{1}{4}\gamma b^2 \tan\phi \cdots\cdots 2$$

let $2p_{p\gamma} - \frac{1}{4}\gamma b^2 \tan\phi = b\frac{1}{2}\gamma b n_y \cdots\cdots 3$

$2p_{pc} + bc\tan\phi = bcn_c \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 4$

$2\,p_{pq} = b\bar{\sigma}n_q \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 5$

therefore

$q_f = cn_c + \bar{\sigma}n_q + .5\gamma bn_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 6$

if the water table is below the base of the footing , $\bar{\sigma} = \gamma d$ and hence

$q_f = cn_c + \gamma dn_q + .5\gamma bn_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 7$

if the water table is below the base of the footing the above eqn becomes

$q_f = cn_c + \gamma d(n_q - 1) + .5\gamma bn_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 8$

for purely cohesive soils the eqn becomes

$q_f = 5.7c + \bar{\sigma} \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 9$

for square footing the equation for ultimate bearing capacity becomes

$q_f = 1.3cn_c + \bar{\sigma}n_q + .4\gamma bn_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 10$

for circular footing the formula becomes

$q_f = 1.3cn_c + \bar{\sigma}n_q + .3\gamma bn_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 11$

5.1Assumptions in Terzaghi's analysis

1. The soil is homogenous , isotropic and its shear strength is represented by coulomb's equation
2. The strip footing has a rough base and the problem is essentially two dimensional.
3. The elastic zones has straight boundaries is inclined at $\varphi = \phi$ to the horizontal and the plastic zones fully develop.
4. p_p consists of three components which can be calculated separately and added although the critical surface for these components are not identical.
5. Failure zones do not extend below the horizontal below the base of the footing (i.e) the shear resistance of the soil above base is neglected and the effect of soil around the footing is considered equivalent to a surcharge $\sigma = \gamma d$.

5.2 Limitations:

1.As the soil compresses, ϕ changes slight downward movement of footing may not develop fully the plastic zones

2. The assumption that term p_p consists of three components which can be calculated separately and added although the critical surface for these components are not identical, is small and on the safe side.

3. The assumption that Failure zones do not extend below the horizontal below the base of the footing (i.e) the shear resistance of the soil above base is neglected and the effect of soil around the footing is considered equivalent to a surcharge $\sigma = \gamma d$ increases with depth of foundation and the theory is suitable for shallow foundations only.

Terzaghi's bearing capacity factors(**table 5.1**).

Φ	General shear failure			Local shear failure		
	n_c	n_q	n_γ	n_c	n_q	n_γ
0	5.7	1	0	5.7	1	0
5	7.3	1.6	.5	6.7	1.4	0.2
10	9.6	2.7	1.2	8	1.9	0.5
15	12.9	4.4	2.5	9.7	2.7	0.9
20	17.7	7.4	5	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	8.3	5.7
34	52.6	36.5	35	23.7	11.7	9
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	780.1	66.8	50.5	60.4
50	347.5	415.1	1153.2	81.3	65.6	87.1

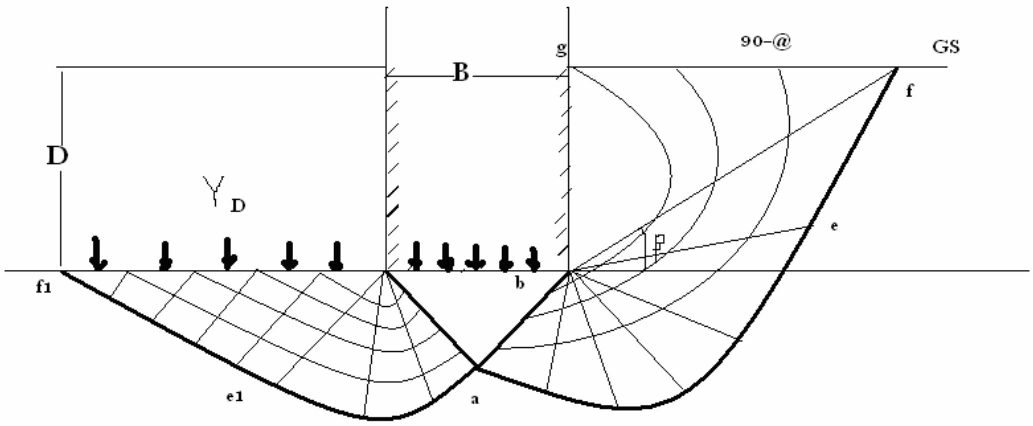
CHAPTER 6

MEYERHOF’S ANALYSIS

6. Meyerhof's analysis:

Meyerhof extended the analysis of plasic equilibrium of a surface footing to shallow and deep foundations .the below figures show the failure mechanisms for shallow and deep foundations. in this 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 upon the depth and roughness of the foundation. the plastic equilibrium in these zones can be established frm the boundary conditions starting from the foundation shift.to simplify this meyerhof established a factor β the angle to define the line bf,joining point b to f where the assumed boundary failure slip intersects the soil surface .the resultant effect of wedge of soil

bfg is represented by the normal and tangential stresses p_0 and s_o on bf .the plane bf is termed as equivalent free surface and p_0 and s_o are termed as equivalent free surface stresses.the angle β increases with depth and becomes 90 for deep foundations.



TERZAGHI'S THEORY--------MEYERHOF'S THEORY

SHALLOW FOUNDATION

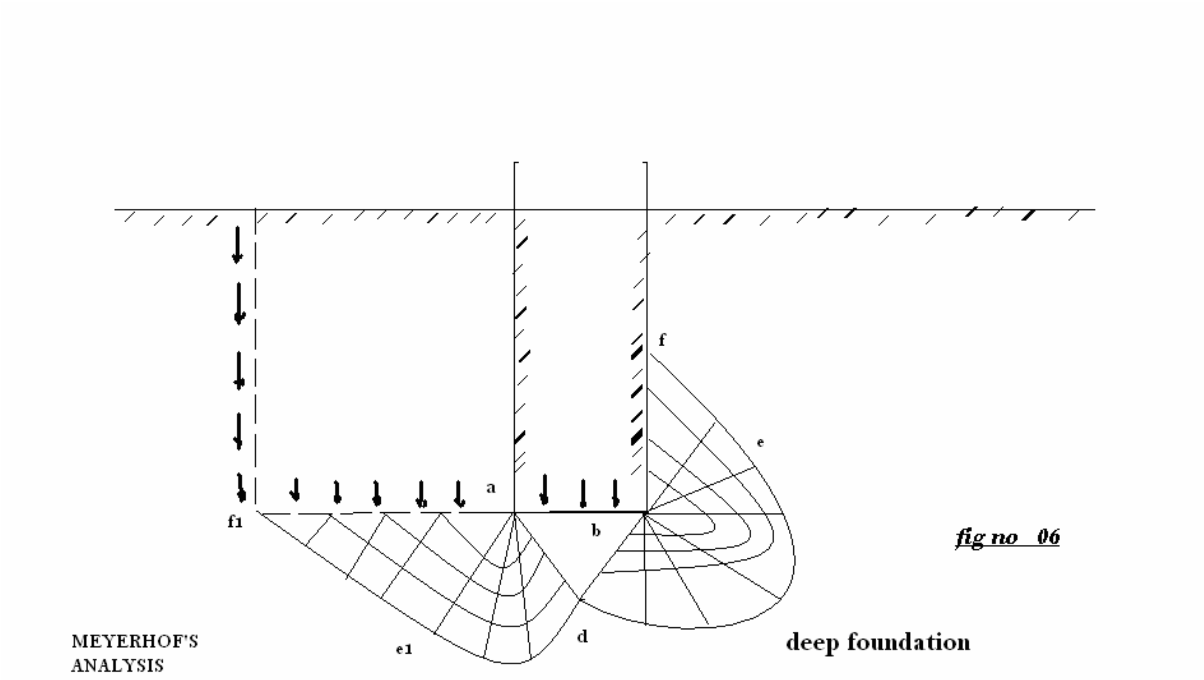


Fig no 6.2

meyerhof gave the following equations for ultimate bearing capacity,taking into account shape, depth and inclination factors:

vertical load:

$$q_f = cn_c s_c d_c + \bar{\sigma} n_q s_q d_q + .5\gamma b n_\gamma s_\gamma d_\gamma \cdots\cdots 1$$

for inclined load:

$$q_f = cn_c i_c d_c + \bar{\sigma} n_q i_q d_q + .5\gamma b n_\gamma s_\gamma d_\gamma \cdots\cdots 2$$

where

$$n_q = e^{\pi \tan\phi} \tan^2(45 + \frac{\phi}{2}) \cdots\cdots 3$$

$$n_c = (n_q - 1)\cot\phi \cdots\cdots 4$$

$$n_\gamma = (n_q - 1)\tan(1.4\phi) \cdots\cdots 5$$

where

s = shape factors

d = depth factors

I = inclination factors

table 6.1

Shape factors	Depth factors	Inclination factors
$s_c = 1 + 0.2k_p \dfrac{b}{l}$	$d_c = 1 + .2\sqrt{k_p}\dfrac{d}{b}$	$i_c = i_q = 1 - \dfrac{\alpha}{90}$
(i)for $\phi = 0$ $s_q = s_\gamma = 1$	(i)for $\phi = 0$ $d_q = d_\gamma = 1$	$i_\gamma = (1 - \dfrac{\alpha}{\phi})^2$
For ϕ>=10 $s_q = s_\gamma = 1 + .1k_p \dfrac{b}{l}$	For ϕ>=10 $d_q = d_\gamma = 1 + .1\sqrt{k_p}\dfrac{d}{b}$	$k_p = \tan^2(45 + \dfrac{\phi}{2})$

shape, depth and inclination factors for the meyerhof's equation

Table no 6.2

Φ	n_c	n_q	n_γ
0	5.14	1	0
5	6.5	1.6	0.1
10	8.3	2.5	0.4
15	11	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	873.7

meyerhof's bearing capacity factors

CHAPTER 7

COHESIVE SOILS

7 cohesive soils:

Cohesive soil is one in which the major component of settlement is due to consolidation, which is time dependant. All clays below the water table will undergo consolidation under load irrespective of the actual facility for drainage or the number of drainage faces ,the latter affecting only the time-rate of settlement and not the total settlement due to consolidation .

Cohesionless soil:

Sandy soils are considered to be cohesionless because their main source of settlement is due to elastic deformation of the soil within the zone of influence under the footing defined arbitrarily as the “bulb of pressure”. the sandy soil with which we conducted our experiment was a cohesive soil.

CHAPTER 8

BEARING CAPACITY OF MODEL FOOTINGS ON SAND

8. Bearing capacity of model footings on sand:

We had done our project thesis on bearing capacity of model footing on sand. In this wehad three different square footings of size (3.75 cm *3.75 cm),(5 cm * 5 cm)and(6.3 cm*6.3 cm) and found out the bearing capacity of these footings on sandy soil. A tank was constructed of size 60 cm * 60 cm * 42 cm. sandy soil was filled in it to a depth of 35 cm. the sandy soil was filled in five layers of depth 7cm.each layer was compacted by giving certain number of blows. the experimental set up of our apparatus is shown below

Procedure:

A tank was constructed of size 60 cm * 60 cm * 42 cm. sandy soil was filled in it to a depth of 35 cm. the sandy soil was filled in five layers of depth 7cm.each layer was compacted by giving certain number of blows. then the square footing whose bearing capacity has to determined was placed in the centre of the tank just resting above the sand. our footing was a surface footing,a dead load of 10 kg was given and two dial gauges of sensitivity 0.01mm was kept at diametrically opposite ends and the settlement was noted in the dial gauge.the settlement was noted at time intervals of(1,2.25,4,6.25,9,12.25,16,25,36,49,64minutes) and after 24 hours. After 24 hour is completed for the first load then load is incremented by 5 kgs and once again the settlement is noted in the above mentioned manner. this process is repeaed until failure of the footing takes place .failure is marked when footing does not withstand the load and goes into the sandy soil. a graph of load vs pressure intensity is plotted and the bearing capacity of the footing on sand is found out. the curve obtained for the three footings is shown below .

Footing 1:

Size of the footing: 3.75cm * 3.75 cm.

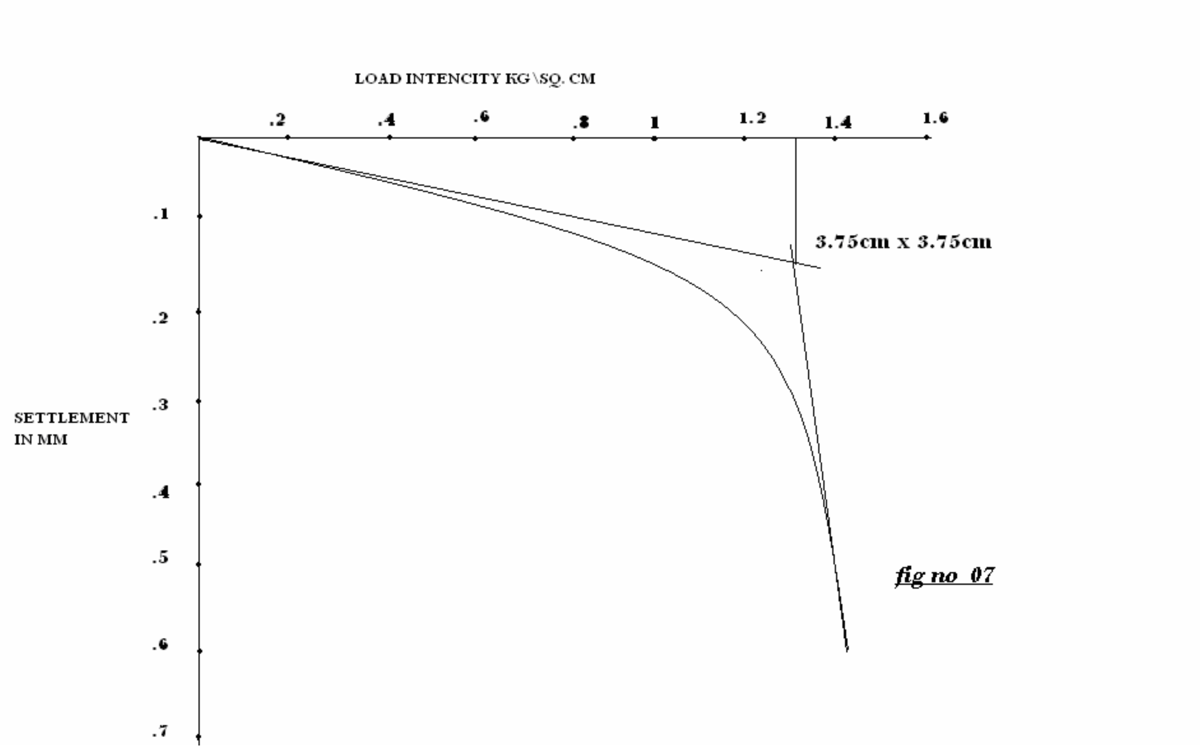


Fig no 8.1

Table 8.1

STRESS (kg/sq. cm)	SETTLEMENT In mm
0.71	.13
1.06	.185
1.42	.58

This footing failed when a load of 25 kg was applied.

Footing 2:

Size of the footing: 5 cm * 5 cm

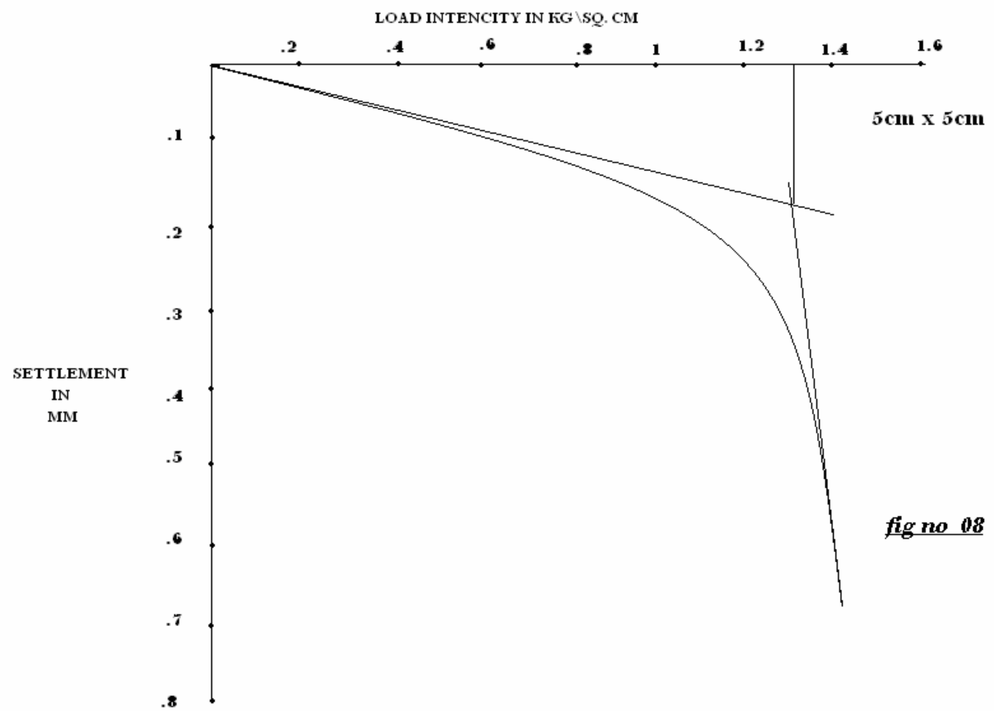


Fig no 8.2

Table 8.2

STRESS (kg/sq. cm)	SETTLEMENT In mm
0.4	.087
0.6	.12
0.8	.15
1	.18
1.2	.27
1.4	.62

This footing failed when a load of 40 kg was applied.

Footing 3:

Size of the footing: 6.2 cm * 6.2 cm

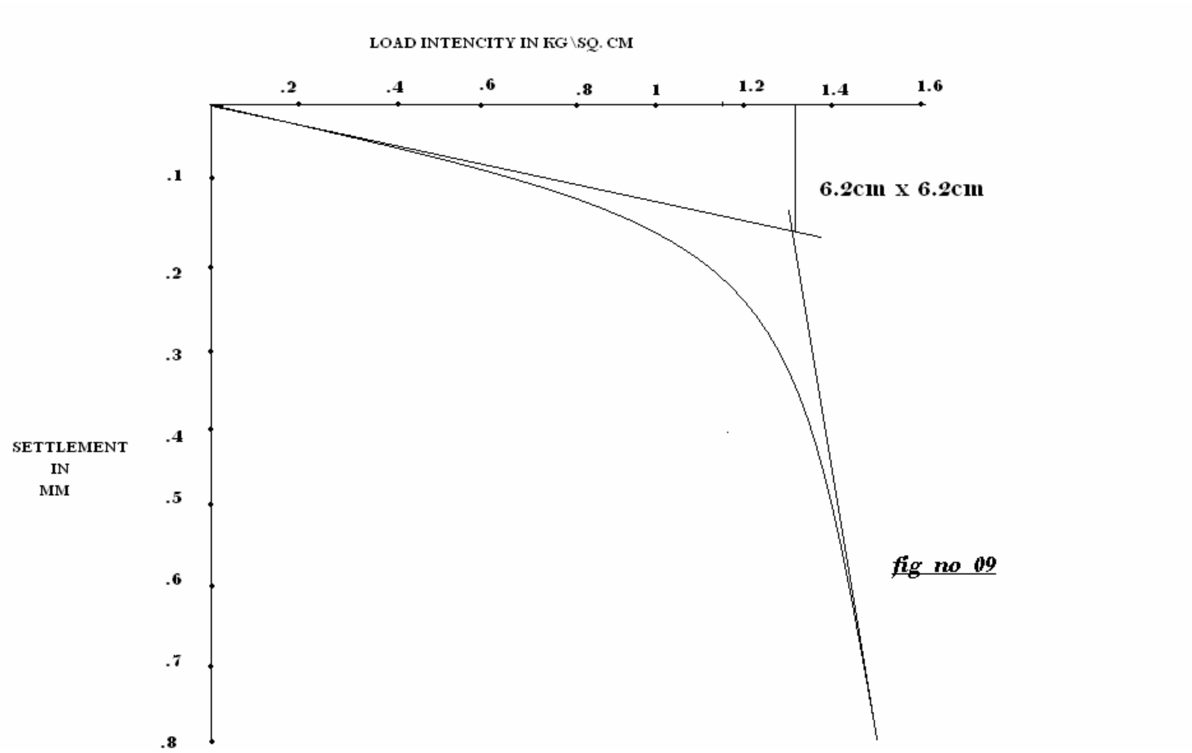


Fig no 8.3

Table 8.3

STRESS (kg/sq. cm)	SETTLEMENT In mm
.26	.06
.39	.075
.52	.1
.65	.13
.78	.15
.91	.18
1.04	.19
1.17	.24
1.3	.36
1.43	.68

This footing failed when a load of 60 kg was applied.

CHAPTER 9

DIRECT SHEAR TEST

9. **Direct shear test**:

To determine the bearing capacity of the square footing using terzaghi's equation Φ- angle of internal friction of sand is to be found out for determining the values of n_γ, n_q required in the equation. for this reason direct shear test was performed on sanb.

the shear strength of a soil is given by mohr-coulomb's equation:

$s = c + \sigma \tan \phi$

s = shear strength (kg\cm^2)

σ = normal stress on failure plane(kg\cm^2)

c= unit cohesion((kg\cm^2)

Φ = angle of internal friction(degrees).

In a strength test of soil, there are two basic stages .first a normal load is applied to the specimen and then failure is induced by appliying shear stress .If no water is allowed to escape enter from or enter into specimen either during consolidation or during shearing then it is called undrained test or unconsolidated undrained test (quick test).if specimen is allowed to consolidate under normal load but no drainage of water is allowed during shear it is called consolidated undrained test.if the specimen is consolidated under normal load and sheared under fully drained conditions it is called consolidated drained 0r slow test.undrained tests can be performed in a shear box only on highly impermeable clay.

Preparation of sample:

Since sand is a non-cohesive soil it is tamped in the shear box itself with the base plate and grid plate or a porous stone as required in place at the bottom of the box.

Procedure:

1. The sandy soil is prepared as described above.the soil does not contain particles more than 4.75 mm size.

2.it is noted that the semations of both top and bottom grid plates are at right angles to the direction of shear loading pad is placed on the top of the grid plate.

3. the box is transferred into the water jacket placed on the platform of apparatus provided with adjustable loading frame.

4.the leverage ratio is determined and the desired normal load intensity at the range of 0.5 to 2 kg\cm^2 is applied .the proving ring is adjusted so that it is attached spindle touches the water jackets outer surface.

5.A dial gauge is attached to the fitting fixed to the vertical end plate.this gauge measures the shear displacement.

6. Shear displacement at a rate of about 0.6mm\min is induced.

7. Readings of proving ring dial gauge are taken at every 0.6 mm of shear displacement till failure(or till a displacement of 12 mm(20 % strain)) which occurs earlier.

8. The test is repeated on two or more identical specimens under increased normal loads.

Table 9.1

s.no	Proving ring reading	Shear load(kg)	Shear stresss = (p *0.3852)/36(kg\cm^2)	Normal stress= (shearload*5)/36(kg\cm^2)
1	22	1.944	0.2	0.27
2	39	3.816	0.38	0.52
3	61	6.12	0.64	0.85

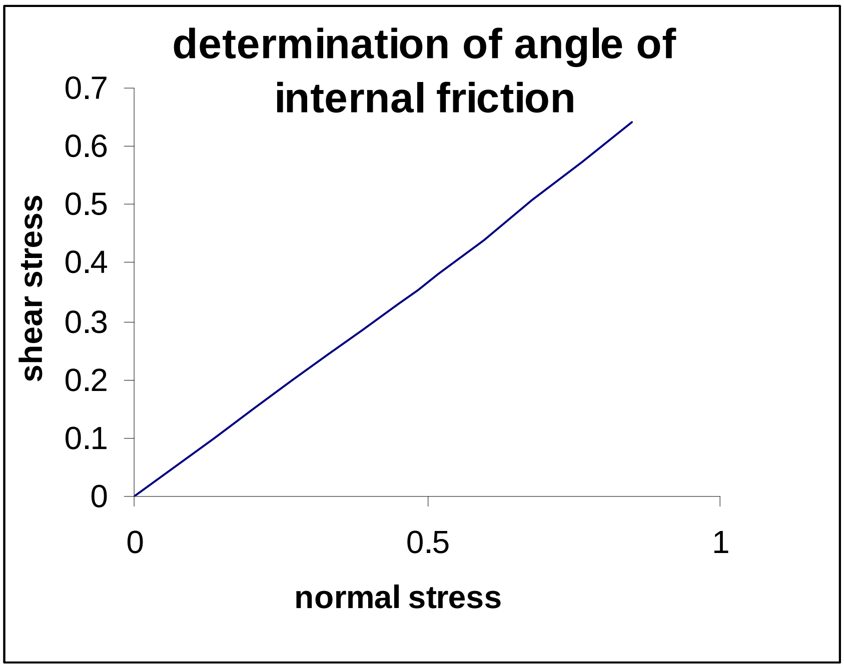


Fig no 10

Fig no 9.1

from the above graph the value of angle of internal friction Φ of sand was 45 degrees.

Theoretical analysis:

We compared the bearing capacities of the above square footings on sand we got experimentally with the values we calculated theoretically using terzaghi's bearing capacity equations.

the equation for ultimate bearing capacity of a square footing using terzaghi's equation is

$$q_f = 1.3cn_c + \bar{\sigma}n_q + .4\gamma bn_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 1$$

case 1:

when we considered the case of a surface footing ,

d=0;

c=0;

equation 1 reduces to

$q_f = 0.4\gamma b n_\gamma$

where

q_f - ultimate bearing capacity of a square footing

γ- unit weight of soil in(kn\m^3)

B-width of footing

n_γ -

For, Footing 1:

Size of the footing: 3.75cm * 3.75 cm

γ = 16.52 kn\m^3

b= .0375m

n_γ = 297.5(from value of φ =45 deg obtained from direct shear test)

therefore

$q_f = 0.4 * 0.0375 * 16.52 * 297.5$

$q_f = 73.23 \quad kn/m^2$

For, Footing 2:

Size of the footing: 5cm * 5 cm

γ = 16.52 kn\m^3

b= 0.05 m

n_γ = 297.5(from value of φ =45 deg obtained from direct shear test)

therefore

$q_f = 0.4 * 0.05 * 16.52 * 297.5$

$q_f =$ 98.175 kn/m^2

For, Footing 3:

Size of the footing: 6.2 cm * 6.2 cm

γ = 16.52 kn\m^3

b= 0.062 m

n_γ = 297.5(from value of φ =45 deg obtained from direct shear test)

therefore: $q_f = 0.4 * 0.062 * 16.52 * 297.5$

q_f = 121.88 kn/m^2

Table 9.2

From terzaghi's equation the bearing capacity fot the above three square footings are:

Size of footing(cm^2)	Ultimate bearing capacity(kn/m^2)
14.0625	73.123
25.00	98.175
38.44	121.88

case 2:

when the settlement of footing during failure is also taken into account:

terzaghi's equation becomes

$q_f = \bar{\sigma} n_q + .4\gamma b n_\gamma \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots 1$

For, Footing 1:

Size of the footing: 3.75cm * 3.75 cm

γ = 16.52 kn\m^3

b= .0375m

n_γ = 297.5(from value of φ =45 deg obtained from direct shear test)

n_q = 173.3

d = 0.014 m

$q_f = 16.52 * 0.014 * 173.3 + 0.4 * 16.52 * 0.0375 * 297.5$

$q_f = 113.80\ kn/m^2$

For, Footing 2:

Size of the footing: 5 cm * 5 cm

γ = 16.52 kn\m^3

b= .05m

n_γ = 297.5(from value of φ =45 deg obtained from direct shear test)

n_q = 173.3

d = 0.023 m

$q_f = 16.52 * 0.023 * 173.3 + 0.4 * 16.52 * 0.5 * 297.5$

$q_f = 163.53 \ kn/m^2$

For, Footing 3:

Size of the footing: 6.2 cm * 6.2 cm

γ = 16.52 kn\m^3

b= .062m

n_γ = 297.5(from value of φ =45 deg obtained from direct shear test)

n_q = 173.3

d = 0.037 m

$q_f = 16.52 * 0.037 * 173.3 + 0.4 * 16.52 * 0.062 * 297.5$

$q_f = 227.80 \ kn/m^2$

Table 9.3

Ultimate Bearing capacity of footings from graph (by model testing on footings)	Ultimate Bearing capacities by terzaghi's analysis Considering footings as surface footings
$126 \, kn/m^2$	$73.23 \ kn/m^2$
$129 \, kn/m^2$	$98.175 \, kn/m^2$
$135 \, kn/m^2$	$121.88 \, kn/m^2$

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Table 9.4

Ultimate Bearing capacity of footings from graph (by model testing on footings)	Ultimate Bearing capacities by terzaghi's analysis Considering settlement of footing during failure
$126\,kn/m^2$	$113.80\,kn/m^2$
$129\,kn/m^2$	$163.53\,kn/m^2$
$135\,kn/m^2$	$227.80\,kn/m^2$

CHAPTER 10

CONCLUSION

10.Conclusion:

Bearing capacity increases with the increase in size of model footing(square footing)on sand. The value of ultimate bearing capacity obtained from performing load test on model footings and that obtained from terzaghi's analysis were found to vary slightly. the value obtained by load test on footing was more than that obtained by terzaghi's analysis. It is possible to perform plate load test on model footings on a particular type of soil and these can be incorporated to the field by considering suitable criterias and foundation for the particular system can be laid.

11. References:

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