CHAPTER 4

THEORETICAL DEVELOPMENTS

4.1 SCOPE:

The scope of this chapter is to draw attention of some of the redundant points of classical Terzaghi's theory of bearing capacity for shallow footings. Further, the description on modifications suggested by several research workers is illustrated with their limitations. The present work proposes a theoretical model for computing bearing capacity of plane footings accounting inherent additional points obscuring (deleting) some of the inherent deficient points and proposing analytical relation exhibiting the frictional and dilatant behaviour of granular mass. Further, the present work has proposed a mechanistic model of confined sand mass within the skirts of skirted strip footings and developed a relation which can describe the improved state of stress. The model is also extended for eccentric, oblique and horizontal loads. Finite element method (FEM) analysis employing ANSYS software is used to confirm the improvement due to confinement of sand within the skirts and load settlement behaviour under axial loads.
4.2 OBSERVATIONS ON TERZAGHI'S CLASSICAL THEORY OF BEARING CAPACITY:

The observations on Terzaghi's classical theory (1943) of bearing capacity are as follows:

(1) The soil considered by Terzaghi is homogeneous and isotropic whose shear strength can be described by Coulomb's equation. The base of footing is rough.

(2) The shear strength of soil located above the base of the footing is neglected. Effect of this soil is equivalent surcharge load.

(3) Theory considers that the formation of rupture plane is not progressive and the value of $\phi$ along the rupture line is constant. Muhs (1965) considered that the shear failure in the soil under the footing is a phenomenon of progressive rupture at variable stress levels. De Beer (1965) supported the same and advocated the use of average mean normal stress.

(4) Classical theory accounts for the compressibility by reducing the strength parameter for the assessment of the bearing capacity factors. Pack, Hansen and Thornburn (1953) suggested mixed mode failure for sands having intermediate denseness (medium dense sand having $\phi$ ranging from 28° to 36°).

(5) Vesic (1965) observed that value of bearing capacity of sand for very small plates decreases with the width of the plate up to 200 mm, and then it increases non-linearly. After a certain width, the bearing capacity should be constant for very large widths. De Beer (1965) and Kerisel (1967) also observed the similar scale effects on shallow foundations.

(6) Vesic (1963) considered cavity expansion concept of an elasto-plastic solids for obtaining equations of bearing capacity factors for illustrating relation between compressibility factors and angle of internal friction, $\phi$. 

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4.3 MODIFIED THEORY OF BEARING CAPACITY OF PLANE FOOTINGS ON SAND:

4.3.1 General:

According to Terzaghi the tendency of the soil located within the zone I (elastic wedge) to spread is counteracted by the friction and adhesion between the soil and the base of the footing. Due to this resistance, the zone I is formed and it remains permanently in the elastic condition.

As the width of plate increases, the depth of this zone also increases. The compressibility of the sand comes into picture and reduces the effective internal angle of friction. This reduction in \( \phi \) depends on the applied pressure on the plate. According to De Beer (1965), the shearing strength parameters decrease by increasing \( \sigma_m/E_n \) values (where \( \sigma_m \) is the mean stress and \( E_n \) is the modulus of elasticity of sand) and therefore the \( N_r \) for larger footing will be less than that for a smaller footing. In this modified theory, the following points are taken into account.

(a) Scale effect

This includes the correction for the local effect of very small widths of plate and that due to increase in width. Increase or decrease of width may induce variation in equation of \( \phi \) accordingly. It is considered that larger the stress in soil, larger is the change in \( \phi \).

*From classical theory*, the bearing capacity due to soil masses \( (q_r) \)

\[
q_r = (0.5 B) \gamma N_r \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (-4-1)
\]

The equation indicates that the increase in width \( (B) \) increases the stress in soil, which means that the reduction in \( \phi \) is some function of width. It is assumed that the reduction in \( \phi \) due to width effect is a function of square root of the width. The frictional parameter under the footing in zone I is modified to \( \phi_m \) taking into account the scale effect.
The theoretical relation for modification is proposed as

\[ \tan \phi_m = \{\exp (C_s \tan \phi)\} \tan \phi \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (4-2) \]

Where \( \exp (C_s \tan \phi) \) represents the coefficient due to scale effect. The factor \( C_s \) is proposed by an analytical relation based on the above assumption and supported by experimental observations as

\[ C_s = 0.15 - 0.4 \sqrt{B_1} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (4-3) \]

Exponential function is chosen similar to the exponential relation used for log spiral in classical theory. The positive constant 0.15 is used to increase the value of \( \phi \) for smaller plates. For an imaginary plate of zero width (i.e. if \( B=0 \) in equation 4-3), the value of \( \tan \phi \) will be increased by 16 %. The factor \( 0.4 \sqrt{B_1} \) takes into account the possible reduction in \( \phi \) due to width. \( B_1 \) is in metres and constant 0.4 shall be used when \( B_1 \) is in metres.

Equating the correction factor \( \{\exp (C_s \tan \phi)\} \) to unity so as to obtain the plate width having no size effect

\[ \exp (0.15 - 0.4 \sqrt{B_1}) = 1.0 = \exp (0) \]

\[ B_1 = 0.140625 \text{ m, say 0.14 m.} \]

This implies that for plate (strip) width of 0.14 m, \( \phi_m = \phi \). This modified value \( \phi_m \) will be used for zone II; hence the side \( bd \) of triangular zone will be at an angle \( \phi_m \) instead of \( \phi \) with horizontal. Thus

For \( B < 0.14 \) m, \( \phi_m > \phi \), and

\[ B > 0.14 \) m, \( \phi_m < \phi. \]

The variation of \( \phi_m \) with respect to the width of plane footing by using equation (4-2) for some values of \( \phi \) is tabulated in table 4-1 and represented graphically in fig. 4-1.
<table>
<thead>
<tr>
<th>Width in Metres</th>
<th>Angle of internal friction φ</th>
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<td>32.05</td>
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<td>40.44</td>
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</table>
Scale effect on $\phi$ for plane footings on sand

Fig. 4-1
PLANE FOOTINGS SUBJECTED TO AXIAL LOADS

FIG. 4.2

$\phi = \phi_0 \tan \phi_1$

AS PER THEORY ASSUMED FOR DERIVATION.
(b) Progressive rupture phenomenon

Consider the inert zone II, of fig. 4-2.

The stress in soil is maximum at level ab. It continuously reduces as we go vertically downward up to point d. This means the value of \( \phi \) at the interface is less than that at d. The equation (4-2) uses the mean value of \( \phi \) in zone \( abd \) which is termed as modified (an equivalent) \( \phi \), denoted by \( \phi_m \).

In zone III, as we go from \( d \) to \( e \), the stresses in soil are reduced. This means while moving from \( d \) to \( e \) on failure surface, the value of \( \phi \) varies from \( \phi_m \) at \( d \) to \( \phi \) at \( e \) because of marginal stress effect at point \( e \). Also in zone III, \( \phi \) remains identical to original \( \phi \). This mechanism of progressive rupture failure is applicable to all types of failure conditions viz. general shear, local shear, and mixed mode failure.

The equation of the radius \( r \) of the log spiral, the failure line, is now:

\[
r = r_0 \exp (\theta \tan \phi_1)
\]

Replacing \( \phi \) by \( \phi_1 \), where \( \phi_1 \) varies from \( \phi_m \) at \( d \) to \( \phi \) at \( e \).

The effect of progressive rupture phenomenon is to reduce \( \phi_1 \) to some extent for smaller plates and to increase to some extent for larger plates changing the bearing capacity factors accordingly.

This log spiral is shown by dotted line in fig. 4-2. The smooth variation of \( \phi_m \) from \( \phi_m \) to \( \phi \) is considered, and its average value is accounted for simplicity. Therefore

\[
r = r_0 \exp \{\theta \tan ((\phi_m+\phi)/2)\} \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (4-4)
\]

With the introduction of this average value, the log spiral in fig. 4-2 will be dark line instead of dotted line, which is in close proximity with the dotted line.
For local shear failure, Terzaghi considers a lower value of $\phi$ given by

$$\tan \phi' = \frac{2}{3} \tan \phi.$$  \hspace{1cm} (4-5)

It is established \cite{Taylor1965} for sands that

- for $\phi < 28^\circ$, the failure is by local shear
- for $\phi > 36^\circ$, the failure is by general shear

For $28^\circ \leq \phi \leq 36^\circ$, the failure is neither by local shear nor by general shear. Such a failure is referred to as the mixed-mode failure. \textit{Pack, Hansen and Thomburn} \cite{Pack1953} has assumed a smooth transition from local to general shear failure, which is adopted, in the present work.

Accordingly, for the condition $28^\circ \leq \phi \leq 36^\circ$, for mixed mode failure, $N_{\text{mix}}$ is

$$N_{\text{mix}} = N_{\text{local}} + (N_{\text{gen}} - N_{\text{local}}) \times \frac{(\phi - 28)}{8}.$$  \hspace{1cm} (4-6)

Table 4-2 and fig. 4-3 represent Terzaghi's bearing capacity factors for cohesionless soil ($N_c$ and $N_r$) with correction for mixed mode failure.

4.3.2 Summary of theoretical work for plane footings:

The general considerations of this theoretical development for plane footing are:

1. Progressive rupture failure concept considers the average value of $\phi$ in zone II / III.
2. In width effect modification, the constants of modification are based on experimental facts.
3. Proposed theoretical development considers the Terzaghi's ideology and reasoning for local shear failure.
4. For mixed mode failure corrections, logical reasoning of Pack, Hansen and Thomburn are adopted.
<table>
<thead>
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<th>PHI</th>
<th>$N_q$</th>
<th>$N'_q$</th>
<th>$N_{q2}$</th>
<th>$N_Y$</th>
<th>$N'_Y$</th>
<th>$N_{y2}$</th>
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<td>147.73</td>
<td>261.60</td>
<td>30.71</td>
<td>261.60</td>
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</table>
Terzaghi's bearing capacity factors for plane footings on sand including mixed mode failure $N_{q2}$ and $N_{g2}$ (before applying scale effect and progressive rupture concept)

Fig. 4-3
4.3.3 Derivation of modified bearing capacity factors:

Considering all the modifications suggested above, Terzaghi's theoretical equations for cohesionless soils are obtained as mentioned in section 4.4.6. The modified equations for bearing capacity factors (refer to section 4.4.5) are:

\[ N_q(p) = \frac{a^2}{\{2 \cos^2 (\pi / 4 + \phi_m / 2)\}} \ldots \ldots \ldots (4-7) \]

and

\[ N_\gamma(p) = 0.5 \tan \phi_m \{(k_p \gamma(p)) / \cos^2 \phi_m\} - 1 \ldots \ldots \ldots (4-8) \]

Where \( \phi_m \) is as obtained by equation (4-2) and \( k_p \gamma(p) \) is obtained as given in section 4.4.5. The bearing capacity factors of this theoretical development can be determined from the general theory of bearing capacity of skirted strip footings as described in section 4.4.6. The values of \( N_q(p) \) and \( N_\gamma(p) \) for some of the widths of plane footings are tabulated and illustrated in table 4-3 and figures 4-4(a) and (b). Table 4-4 and figs. 4-5 (a) and 4-5 (b) represent the bearing capacity factors for \( \phi = 34.5^\circ \) (\( \phi \) value of the sand bed of for present investigation).

Figure 4-5 indicates that for the plate width of 140 mm, the modified bearing capacity factors and Terzaghi's bearing capacity factors are the same for general shear failure. As the size of plate increases from 140 mm, the reduction is observed in modified factors theoretically. Similarly the reduction in size increases the modified factors. For local shear failure, it is observed that as the width increases, the modified factors decrease.
Table 4-3

Bearing capacity parameters for plane footings on sand (Present Investigation)

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<th>$\phi$ in deg.</th>
<th>Width of footing in metres</th>
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<td>45</td>
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<tr>
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Fig. 44 (a)

Bearing capacity factors for plane strip footings on sand (Present investigation)

Fig. 44 (a)
Bearing capacity factors for plane strip footings on sand (Present investigation)

Fig. 4.4 (b)
Table 4-4

Comparison of B.C. factors of Terzaghi for plane strip footings with present investigation with reference to width effect for $\phi = 34.5^\circ$

<table>
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<th>Width of plate in metre</th>
<th>$N_q,\text{Ter}$</th>
<th>$N_q,\text{present}$</th>
<th>$N_r,\text{Ter}$</th>
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<td>12.20</td>
<td>11.96</td>
<td>7.83</td>
<td>11.00</td>
</tr>
<tr>
<td>0.25</td>
<td>12.20</td>
<td>11.80</td>
<td>7.83</td>
<td>11.00</td>
</tr>
<tr>
<td>0.30</td>
<td>12.20</td>
<td>11.65</td>
<td>7.83</td>
<td>10.00</td>
</tr>
<tr>
<td>0.35</td>
<td>12.20</td>
<td>11.52</td>
<td>7.83</td>
<td>10.00</td>
</tr>
<tr>
<td>0.40</td>
<td>12.20</td>
<td>11.40</td>
<td>7.83</td>
<td>10.00</td>
</tr>
<tr>
<td>0.45</td>
<td>12.20</td>
<td>11.29</td>
<td>7.83</td>
<td>9.00</td>
</tr>
<tr>
<td>0.50</td>
<td>12.20</td>
<td>11.19</td>
<td>7.83</td>
<td>9.00</td>
</tr>
<tr>
<td>0.55</td>
<td>12.20</td>
<td>11.09</td>
<td>7.83</td>
<td>9.00</td>
</tr>
<tr>
<td>0.60</td>
<td>12.20</td>
<td>11.00</td>
<td>7.83</td>
<td>9.00</td>
</tr>
</tbody>
</table>
Fig. 4-5 (a)

Comparison of Bearing Capacity factors of Terzaghi for plane strip footings (general shear failure) with present investigation after considering the width effect and progressive rupture failure for $\phi = 34.5^\circ$
Comparison of bearing capacity factors of Terzaghi for plane strip footings with present investigation with reference to width effect for $\phi = 34.5^\circ$

(local shear failure)

Fig. 4-5 (b)
4.4 THEORY OF BEARING CAPACITY FOR SKIRTED STRIP FOOTINGS:

4.4.1 General

For the development of \( N_q \) and \( N_y \) for the case of skirted footings, it is essential to understand the mechanism of confinement of sand within the skirts and the effect of interfacial friction between confined sand mass and the foundation sand. Also the change in state of stress at inter-granular points in the sand packing due to confinement is desired to be accounted in the derivation. Reduction in overall rigidity of the skirted footing system is attributed to consideration of confined sand mass and rigid skirted footing as a single unit.

4.4.2 Mechanism under skirted footings

The foundation with the skirts and confined sand mass within the skirts are considered to act as a single unit up to the aspect ratio \( k \) equal to unity. When a load is applied to this unit, a wedge shaped zone similar to that of Terzaghi's is formed. Because of restricted movement of the sand by the skirts, foundation sand experiences densification in zone I and in turn to zone II (fig. 4-6). Average increase of density in zone I and a gradual increase in zone II from \( D \) to \( C \) is assumed. These assumptions are confirmed by the experimental evidences by actual density measurements within the skirts and in zone \( AA'C \) as shown in fig. 4-6. Also finite element analysis by using ANSYS software has verified the same which is discussed later.

Bottom surface of the top part of the skirted footing should be compatible to the foundation sand mass, to make the confined sand mass within the skirts be effective, thereby the adjacent sand mass will be in the line with the top surface.

A factor \( \exp \left( e-e_{\text{min}} \right) \) is applied to modified \( \phi \) to account for the possible movement of the sand particles in zone II and zone I during densification under the self load (seating load) of the footing.
4.4.3 Effect of roughness of the footing

As described earlier that confined sand mass and skirted footing act as a unit; the roughness or smoothness of the inside surface of the footing has no more vital role on the behaviour of the system. But the friction to friction effect at the interface of the unit of skirted footing and confined sand mass with foundation sand plays an important role in forming zone I having boundaries at an angle $\phi_m$ with horizontal interface.

4.4.4 Increase in density

The increase in density as described earlier depends on available volume of sand within the skirts and possibility to compact the confined sand in zone I. Considering two skirted footings, one with $\alpha = 0$ and the other having $\alpha = \alpha$ and $B_1 = B_t + 2kB_t\cos\alpha$ representing the former as a plane footing (fig.4-7), where $k$ is the aspect ratio (width of the skirt to the top width of skirted footing) and $B_t$ the top width of the footing. In footing with $\alpha=0$ (plane footing), the movement of sand in zone II and zone I is too remote unlike the skirted footings. This is attributed to the available volume of sand mass in zone I, in turn to the area of this zone, being footings of the same lengths. The movement being two directional, this effect can be assumed to be proportional to (to vary as) the length of equivalent square of the area $AA'C$ for $\alpha = 0$ (plane footing) and $ACA'JG$ for $\alpha = \alpha$ (skirted footing). The improvement in friction angle $\phi$ is approximated as the function of the square root of the above areas and is equal to

$$\sqrt{A_2 - A_1}/\sqrt{A_1} = \sqrt{[\frac{A_2}{A_1} - 1]}.$$

Where

$$A_1 = \text{area of inert wedge under footing with } \alpha = 0$$

$$A_2 = \text{area of inert wedge below and under skirted footing with } \alpha = \alpha.$$
The areas $A_1$ and $A_2$ are given by
\[
A_1 = 0.25 (1+2k)^2 B_1^2 \tan \phi, \quad \text{k being the aspect ratio (width of the skirts to the top width ratio)}
\]
and
\[
A_2 = 0.25(1+2k\cos\alpha)^2 B_1^2 \tan \phi + k \sin \alpha (1+k\cos\alpha) B_2^2.
\]

The *inert wedge ratio* is given by $A_2 / A_1$, where $A_1$ and $A_2$ are defined as above. The variation of inert wedge ratio with $\alpha$ for a few values of $k$ is shown in fig. 4-7(C) and is tabulated below in table 4-5.

<table>
<thead>
<tr>
<th>Angle of skirt ($\alpha$, deg.)</th>
<th>Values of inert wedge ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k = 0$</td>
</tr>
<tr>
<td>0</td>
<td>1.0</td>
</tr>
<tr>
<td>15</td>
<td>1.0</td>
</tr>
<tr>
<td>30</td>
<td>1.0</td>
</tr>
<tr>
<td>45</td>
<td>1.0</td>
</tr>
<tr>
<td>60</td>
<td>1.0</td>
</tr>
<tr>
<td>75</td>
<td>1.0</td>
</tr>
<tr>
<td>90</td>
<td>1.0</td>
</tr>
</tbody>
</table>
\[ \Lambda_1 = \frac{0.25(1 + 2K)^2}{B_t^2 \tan \phi} \]

(a)

\[ \Lambda_2 = \frac{0.25(1 + 2K \cos \alpha)^2}{B_t^2 \tan \phi} + K \sin \alpha (1 + K \cos \alpha) B_t^2 \]

(b)

**Calculation of Areas of Inert Wedge**

**Fig: 4.7**
VARIATION OF INERT WEDGE RATIO v/s THE ANGLE OF SKIRT

VALUES OF INERT WEDGE RATIO

ANGLE OF SKIRT $\alpha$ (deg.)

(c)

VARIATION OF INERT WEDGE RATIO v/s THE ANGLE OF SKIRT

FIG 4.7
The confinement factor denoted by \( C_j \) is expressed by the following expression

\[
C_j = \left\{ (A_2 - A_1) - 1 \right\}^{0.5} \cdot (e - e_{\text{min}})^{0.10} \tag{4-9}
\]

The ratio \( A_2 / A_1 \) used in above equation should consider the followings.

1. Width effect as per equation (4-3) used for modifying Terzaghi's bearing capacity factors.
2. Progressive rupture failure as per equation (4-4)
3. Confinement factor as per equation (4-9)

Thus the modified \( \phi_m \) is given by

\[
\tan \phi_m = \{ \exp(C_S \tan \phi + C_i) \} \tan \phi \tag{4-10}
\]

Where

\[
C_S = \exp\{0.15 - 0.4 \times \sqrt{B_1}\}
\]

And

\[
C_i = \left\{ (A_2 / A_1) - 1 \right\}^{0.5} \cdot (e - e_{\text{min}})^{0.10}
\]

The equation of log spiral due to progressive rupture failure is given by

\[
r = r_0 \exp [ \theta \tan \{(\phi_m + \phi)/2\} ] \tag{4-11}
\]

4.4.5 Derivation of bearing capacity factors for axial loads:

Theory considers similar to Terzaghi the continuity of the skirted footing i.e. length of the skirted footing is greater than or equal to five times the projected width of the footings. Further the footing is considered as shallow wherein depth of foundation \( D_f \) is considered to be less than or equal to the projected width of the footing as shown in fig. 4-8 wherein \( D_f \) is assumed from level \( JG \) and the failure zone is expected to extend up to that level. Also the resistance of the soil above level \( AA' \) is neglected along with consideration of equivalent surcharge of \( \gamma D_f \).
(a) PLAN

(b) SECTION

\[ B_t = B_t (1 + 2k \cos \alpha) \]

- \( B_t \): Top width of base of footing
- \( B_s \): Width of skirt
- \( k \): Aspect ratio = \( B_s / B_t \)
- \( \alpha \): Angle of skirt with horizontal
- \( D_l \): Surcharge

TYPICAL SKIRTED STRIP FOOTING

NOMENCLATURE

FIG: 4·8
With the above mechanism, the plastic equilibrium at or just prior to the failure is considered. The equations of equilibrium viz. $\Sigma X = 0$, $\Sigma Y = 0$ and $\Sigma M = 0$ should be satisfied considering each of the loads acting on some selected free body diagram. The curved boundary $CD$ or $CD'$ is assumed to be a log spiral as given by the equation (4-11), viz.

$$r = r_0 \exp (\theta \tan \phi_1), \text{ where } \phi_1 = (\phi_m + \phi) / 2$$

whose centre $O_1$ is located on line $DA$ or on extended line $DA$, (fig. 4-9a), where

$$r = \text{length of any vector } O_1 n \text{ making an angle of } \theta \text{ (radians) with vector } O_1 C$$

and

$$r_0 = O_1 C, \text{ the length of vector when } \theta = 0.$$

Every vector through centre $O_1$ of the logarithmic spiral of equation (4-11) intersects the corresponding tangent to the spiral at an angle $90^\circ - \phi_1$, as shown in fig. 4-9. Since the centre $O_1$ of the spiral is located on line $AD$, the spiral corresponding to equation (4-11) passes without any break into the straight line $DE$.

 Furthermore, at any point of the curved section of the surface of sliding, the reaction $dF$ acts at an angle $\phi_1$ to the normal or at an angle $90^\circ - \phi_1$ to the tangent drawn to the spiral. This direction is identical with that of the vector $O_1 n$. Hence the resultant reaction $F$ along the curved line $CD$ (or $CD'$) also passes through the centre $O_1$.

Terzaghi has assumed the centre of the log spiral at $A$ (or $A'$) for deriving the parameter $N_a$. For the calculation of $N_a$, however, he has assumed the centre to lie on line $DA$ or its extension. In this theory, it is assumed that the centres of log spiral lie on line $DA$ or on its extension for both the calculations of $N_a$ and $N_t$.

The method of finding these values consists of considering equilibrium of mass $ACDH$. Consider the projected width of footing $B_1 = 2B$ for the simplicity of calculations. The footing is subjected to constant pressure of $q_0$. The critical load $Q_D$
Consider a unit length of footing. The weight of the confined sand mass within the skirts is assumed as additive to the self-weight of the skirted footing.

Refer to fig. 4-9 (a) and fig. 4-9 (b)

\[ AC = A'C = B \cos \phi_m \]

Area \( \Delta AA'C = 0.5 \times 2B \times B \tan \phi_m = B^2 \tan \phi_m. \)

Therefore the weight of triangular wedge \( W_1 \) is

\[ W_1 = \gamma \cdot B^2 \tan \phi_m \]

Applying equilibrium conditions:

\[ \Sigma V = 0 \]

\[ Q_0 + W_1 = 2P_p \]

\[ Q_0 = 2P_p - W_1 \]

The passive pressure \( P_p = P_{pq} + P_{py} \) (where \( P_{pq} \) is passive pressure due to surcharge and \( P_{py} \) is the passive pressure due to the weight of the soil) and is vertical due to geometry. It is inclined to the normal of \( AC \) or \( A'C \) at \( \phi_m \), hence vertical as shown in figure. The corresponding load \( Q_0 \) is \( Q_0 = Q_q + Q_y. \)

Substituting for \( Q_0 \) and \( W_1 \) in (2)

\[ Q_0 = Q_q + Q_y \]

\[ = Q_q + 2P_{py} - \gamma B^2 \tan \phi_m \]

(3)
DETERMINATION OF BEARING CAPACITY PARAMETERS
FOR SKIRTED STRIP FOOTINGS (AXIAL LOADS)

FIG: 4.9 (g)
\[ q_D = (\text{APPLIED LOAD + SELF WEIGHT}) \]

of footing

Sand weight \( A G J A' \) \( A' \) is included in self weight of footing

\[ \alpha_1 = \text{ANGLE OF WALL} \]

\[ AC = 180 - \phi_m \]

**DETERMINATION OF BEARING CAPACITY PARAMETERS**

**FOR SKIRTED STRIP FOOTING (AXIAL LOAD)**

**FIG 4.9 (b)**
If $D_f = 0$ (i.e. $q = 0$) i.e. if the base of the footing rests on the horizontal surface of a mass of a cohesionless sand, the passive pressure $P_p$ acting on wall $AC$ is given by

$$P_p = \frac{P_{pn}}{\cos \delta} = \frac{1}{2} \gamma H^2 \frac{kp}{\sin \phi_i \cos \delta}$$ (standard formula).

Where $P_{pn} =$ normal component of passive pressure

$H =$ the vertical projection of the wall

$\phi_i =$ the angle which the wall makes with horizontal (slope angle of the contact face)

and $\delta =$ the angle of wall friction.

Here $P_p = P_{py}$

$H = B \tan \phi_m$

$\delta = \phi_m$

$k_p = k_{py}$

and $\phi' = 180^\circ - \phi_m$

$$P_{py} = \frac{1}{2} \gamma B^2 \tan^2 \phi_m \frac{k_{py}}{\sin \phi_m \cos \phi_m}$$

$$= \frac{1}{2} \gamma B^2 \frac{\tan \phi_m}{\cos^2 \phi_m} k_{py} \ldots \ldots \ldots \ldots$$ (4)

Substituting in (3)

$$Q_D = Q_y = 2 \times \frac{1}{2} \gamma B^2 \frac{\tan \phi_m}{\cos^2 \phi} k_{py} - \gamma B^2 \tan \phi_m$$

$$= \gamma B^2 \tan \phi_m \left( \frac{k_{py}}{\cos^2 \phi_m} - 1 \right) \ldots \ldots \ldots$$ (5)
Similarly passive pressure $P_{pq}$ due to surcharge is given by

$$P_{pq} = \frac{p_m}{\cos \delta} = \frac{H}{\sin \phi_m \cos \delta} \times q \times k_{pq}$$

$$= \frac{B \tan \phi_m}{\sin \phi_m + \cos \phi_m} \times q \times k_{pq}$$

$$= \frac{B}{\cos^2 \phi_m} \times q \times k_{pq} \quad (6)$$

Therefore, for foundation having surcharge also, using (3), (5) and (6)

$$Q_D = Q_q + Q_y$$

$$= \frac{2Bq \cdot k_{pq}}{\cos^2 \phi_m} + \gamma B^2 \tan \phi_m \left( \frac{k_{py}}{\cos^2 \phi_m} - 1 \right)$$

This is written as:

$$Q_D = 2Bq \left( \frac{k_{pq}}{\cos^2 \phi_m} - 1 \right) + 2B^2 \gamma \left\{ \frac{1}{2} \tan \phi_m \left( \frac{k_{py}}{\cos^2 \phi_m} - 1 \right) \right\}$$

$$Q_D = 2B \left( q \cdot N_q + B \gamma \cdot N_y \right) \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (8)$$

Where

$$N_q = \frac{k_{pq}}{\cos^2 \phi_m}$$

$$N_y = \frac{1}{2} \tan \phi_m \left( \frac{k_{py}}{\cos^2 \phi_m} - 1 \right)$$

Substitute $2B = B_1 \left( 1 + 2k \cos \alpha \right)$ in equation (8).

$$Q_D = B_1 \left( q \cdot N_q + 0.5B_1 \gamma \cdot N_y \right) \ldots \ldots \ldots \ldots \ldots \ldots (9)$$
Where

\[ N_{qs} = \frac{(1+2k \cos \alpha) k_{pq}}{\cos^2 \phi_m} \]

\[ N_{ys} = (1+2k \cos \alpha)^2 \times \frac{1}{2} \tan \phi_m \left[ \frac{k_{pq}}{\cos^2 \phi_m} - 1 \right] \] \hspace{1cm} (4-12b)

\[ \frac{A}{qs} \text{ and } \frac{A}{ys} \text{ are the bearing capacity factors for skirted footings.} \]

Thus, if \( q_D \) is the bearing capacity of soil, the failure load is given by

\[ Q_d = q_D \times B \times L \] \hspace{1cm} (4-4)

and

\[ q_D = q \cdot N_{qs} + \frac{1}{2} B_i \gamma N_{ys} \] \hspace{1cm} (4-5)

Where

\[ L \text{ = Length of the footing} \]

\[ N_{qs} \text{ = Bearing capacity factor due to surcharge, and} \]

\[ N_{ys} \text{ = Bearing capacity factor due to weight of soil} \]

The method of finding these values consists of considering equilibrium of mass ACDH. The required passive pressure parameter \( k_{pq} \) and \( k_{pp} \) are obtained by considering the forces acting on free body GACDHL (fig. 4-9) and applying the equilibrium conditions. The log spiral, which gives the least value of the said parameter, is adopted for the respective calculations.

Referring to fig. 4-10 (a) and fig. 4-10 (c)

Let

\[ x = \left( \frac{1}{\cos \phi_m} \right) \]

\[ a_0 = \exp \left[ \theta_1 \tan \left\{ \left( \phi_m + \phi \right)/2 \right\} \right] \]

\[ \beta = 45 + \phi/2 \]

\[ N_0 = \tan^2 \beta \]

\[ AC = \left( B / \cos \phi_m \right) = Bx. \]
From \( \Delta \) O\( _r \)AC

\[
\frac{AC}{\sin \theta_1} = \frac{O_r A}{\sin(45 - \phi / 2 + \phi_m + \theta_1)} = \frac{O_r C}{\sin(45 - \phi / 2 + \phi_m)} = \frac{BX}{\sin \theta_1}
\]

\( O_r A = zB = \{\sin(45 - \phi / 2 + \phi_m + \theta_1) / \sin \theta_1\} \cdot Bx \)

\( O_r C = \{\sin(45 - \phi)\} \)

\( x = 1 / \cos \phi_m \)

\( y = \{\sin(45 - \phi / 2 + \phi_m) / \sin \theta_1\} \cdot a_0 \)

\( z = \{\sin(45 - \phi / 2 + \phi_m + \theta_1) / \sin \theta_1\} \cdot x \)

With these relations, we have

\( O_r A = zB \)

\( O_r D = yB \)

\( AC = xB \)

\( AD = (y-z) B \)

\( HD = (y-z) B \cos \beta \)

\( AH = (y-z) B \sin \beta \)

Also \( LH = kd_1 \sin \alpha \).

Let

\( P_{dy} = \) passive earth pressure due to soil mass

\( P_{dq} = \) passive earth pressure due to surcharge on HE

\( = q \times N_x \times HD \)

\( q = \) surcharge on \( AH = \gamma \cdot D_t \)

\( P_{pq} = \) passive pressure on AC due to surcharge
\( P_{dy} = \) passive pressure on \( AC \) due to weight of the earth
\( F = \) resultant thrust on curve passing through \( O_i \).

The passive pressures are acting at \( \phi_m \) to the normal. The resultant thrust \( F \) passes through the centre of the log-spiral, \( O_i \), due to the geometric properties of the log-spiral. Thus if the moments of the forces acting on the free body selected are taken about the centre of the log-spiral, the force \( F \) does not induce any moment which reflects the excellency of classical theory by selecting the failure curve to be a log-spiral rather than circular or triangular.

The parameters \( k_{pq} \) and \( k_{py} \) are obtained by equating moments of all the forces acting on a free body diagram \( ACDH \) about point \( O \), the centre of the log-spiral, and equated to zero. The value of \( F \) can be obtained by applying the other equilibrium conditions \( \Sigma V = 0 \) and \( \Sigma H = 0 \). The moment calculations are tabulated in table 4-6 and table 4-7 and solved thereafter to prepare equations for calculating bearing capacity factors.

### Table 4-6
**MOMENT CALCULATIONS FOR \( N_{qs} \)**

<table>
<thead>
<tr>
<th>Force Name</th>
<th>Value</th>
<th>Distance from ( O_i ), the centre of log-spiral</th>
<th>Moment @ ( O_i )</th>
<th>Sign</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_{pq} )</td>
<td>( qN_b ) ( LD )</td>
<td>( LD - \frac{1}{2} (LH + zB \cos \beta) )</td>
<td>AM(_1) ((+))</td>
<td></td>
</tr>
<tr>
<td>( qN_b (HD+LH) )</td>
<td></td>
<td>( \frac{1}{2} (HD-LH) + zB \cos \beta )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P_{p1} )</td>
<td>( \frac{1}{2} qN_b ) ( D_1 )</td>
<td>( \frac{1}{2} D_1 + LH - zB \cos \beta )</td>
<td>AM(_2) ((+))</td>
<td></td>
</tr>
<tr>
<td>( Q )</td>
<td>( q ) ( AH )</td>
<td>( \frac{1}{2} AH + zB \sin \beta )</td>
<td>AM(_3) ((+))</td>
<td></td>
</tr>
<tr>
<td>( P_{py} )</td>
<td>( \frac{Bq}{\cos^2 \phi_m} ) ( k_{py} ) ( \frac{1}{2} )</td>
<td>( B - zB \sin \beta )</td>
<td>((-))</td>
<td></td>
</tr>
</tbody>
</table>

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Applying conditions of equilibrium.

\[ \Sigma M = 0 \]

\[ \frac{1}{2} B P_{pq} (1 - 2 z \sin \beta) = q N_\phi (HD + LH) \left[ \frac{1}{2} (HD - LH) + z B \cos \beta \right] \]

\[ - \frac{1}{2} q N_\phi D_t \left[ \frac{1}{3} D_t + LH - z B \cos \beta \right] + q AH \left[ \frac{1}{2} (AH + 2 z B \sin \beta) \right] \]

\[ \frac{1}{2} q N_\phi \left[ (y-z) B \cos \beta + \frac{2kB \sin \alpha}{AX} \right] \left[ (y-z) B \cos \beta - \frac{2kB \sin \alpha}{AX} + 2 z B \cos \beta \right] \]

\[ = \frac{1}{2} q N_\phi B \left[ (y-z) \cos \beta + \frac{2kB \sin \alpha}{AX} \right] \left[ (y-z) \cos \beta - \frac{2kB \sin \alpha}{AX} + 2 z \cos \beta \right] \]

\[ \frac{1}{2} q N_\phi \left[ \frac{1}{3} D_t + \frac{2kB \sin \alpha}{AX} \right] - zB \cos \beta \]

\[ = \frac{1}{2} Q N_\phi D_t \left[ \frac{1}{3} \frac{D_t}{B} + \frac{2kB \sin \alpha}{AX} \right] - z \cos \beta \]

\[ \frac{1}{2} q (y-z) B \sin \beta \left[ (y-z) B \sin \beta + 2 z B \sin \beta \right] \]

Equating

\[ P_{pq} (1 - 2 z \sin \beta) = B q N_\phi \left[ (y-z) \cos \beta + \frac{2kB \sin \alpha}{AX} \right] \left[ (y-z) \cos \beta - \frac{2kB \sin \alpha}{AX} + 2 z \cos \beta \right] \]

\[ - B q N_\phi D_t \left[ \frac{1}{2} \frac{D_t}{B^2} + \frac{2kB \sin \alpha}{AX} \right] - z \cos \beta \right] + q B (y-z) \sin \beta \left[ (y-z) \sin \beta + 2 z \sin \beta \right] \]

Divide both the sides by \( qB \)

\[ \frac{P_{pq}}{qB} \left[ (1 - 2 z \sin \beta) = N_\phi \left[ (y-z) \cos \beta + \frac{2kB \sin \alpha}{AX} \right] \left[ (y-z) \cos \beta - \frac{2kB \sin \alpha}{AX} + 2 z \cos \beta \right] \right. \]

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\[
- N_4 D_1 \left[ \frac{Df}{3B^2} + \frac{2k \sin \alpha}{BAX} \frac{Z}{B} \cos \beta \right] + (y-z) \sin \beta \left[ (y-z) \sin \beta + 2z \sin \beta \right]
\]

\[
N_6 = \frac{AM_1 - AM_2 + AM_3}{(1-2z \sin \beta)}
\]

Where

\[
AM_1 = N_4 \left[ (y-z) \cos \beta + \frac{2k \sin \alpha}{AX} \frac{(y-z) \cos \beta}{AX} - \frac{2k \sin \alpha}{AX} + 2z \cos \beta \right]
\]

\[
AM_2 = N_4 D_1 \left[ \frac{Df}{3B^2} + \frac{2k \sin \alpha}{BAX} \frac{z}{B} \cos \beta \right]
\]

\[
AM_3 = (y-z) \sin \beta \left[ (y-z) \sin \beta + 2z \sin \beta \right]
\]
TABLE 4 - 7
MOMENT CALCULATIONS FOR $N_y$

<table>
<thead>
<tr>
<th>Force Name</th>
<th>Value</th>
<th>Distance from $O_1$, the centre of log-spiral</th>
<th>Moment @ 'O'</th>
<th>Sign</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_{\phi_1}$</td>
<td>$\frac{1}{2} N_\phi \gamma (LD)^2$</td>
<td>$\frac{2}{3} LD - kb_1 \sin \alpha + zB \cos \beta$</td>
<td>$AM_1$</td>
<td>(+)</td>
</tr>
<tr>
<td>$W_1$</td>
<td>$\frac{1}{2} \times AH \times HD \times \gamma$</td>
<td>$\frac{2}{3} \gamma H + zB \sin \beta$</td>
<td>$AM_2$</td>
<td>(+)</td>
</tr>
<tr>
<td>$P_{\phi_2}$</td>
<td>$\frac{1}{2} \beta \left( \tan \phi_m - \beta \right)$</td>
<td>$\frac{2}{3} \beta \gamma H + zB \sin \beta$</td>
<td>$AM_3$</td>
<td>(-)</td>
</tr>
<tr>
<td>$W_2$</td>
<td>$\frac{1}{2} \int_{1/2} r^2 d\theta \gamma$</td>
<td>$\frac{2}{3} r \sin (n)$</td>
<td>$AM_4$</td>
<td>(+)</td>
</tr>
<tr>
<td>$W_3$</td>
<td>$\frac{1}{2} \beta C \times AN \times \gamma$</td>
<td>$\frac{2}{3} \beta \cos \left( \delta - \left( 45 - \frac{\gamma}{2} \right) \right)$ - $OA \sin \beta$</td>
<td>$AM_5$</td>
<td>(+)</td>
</tr>
<tr>
<td>$W_4$</td>
<td>$\gamma B (y - z) \sin \beta \cos \alpha$</td>
<td>$\frac{1}{2} B (y - z) \sin \beta + zB \sin \beta$</td>
<td>$AM_6$</td>
<td>(+)</td>
</tr>
</tbody>
</table>

$\Sigma M = 0$

\[
k_{py} AM_3 = AM_1 + AM_2 + AM_4 + AM_5 + AM_6
\]

\[
k_{py} = \frac{AM_1 + AM_2 + AM_4 + AM_5 + AM_6}{AM_3}
\]

\[
N_y = \frac{1}{2} \tan \phi_m \left( \frac{k_{pr}}{\cos^2 \phi_m} - 1 \right)
\]

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\[ N_{\gamma S} = N_\gamma \times (1+2k\cos\alpha)^2 \]

\[ P_{pq} = N_q \cdot qB \]

\[ P_{pr} = N_\gamma \cdot B^2 \]

\[ M_1 = \frac{1}{2} \frac{N_\gamma (LD)^2}{2} \left[ \frac{2}{3} LD - kb \sin \alpha + zB \cos \beta \right] \]

\[ = \frac{1}{2} \tan^2 \beta \cdot \gamma \left[ (y-z) B \cos \beta + kb \sin \alpha \right]^2 \]

\[ = \left[ \frac{2}{3} (y-z) B \cos \beta + \frac{2}{3} kb \sin \alpha - kb \sin \alpha + zB \cos \beta \right] \]

\[ = \frac{1}{2} \tan^2 \beta \cdot \gamma \left[ B \left( y-z \right) \cos \beta + \frac{2kB}{AX} \sin \alpha \right]^2 \]

\[ \times \left[ \frac{2}{3} (y-z) B \cos \beta - \frac{1}{3} \frac{2kB}{AX} \sin \alpha + zB \cos \beta \right] \]

\[ \frac{M_1}{\gamma B^2} = \frac{1}{2} \tan^2 \beta \left[ (y-z) \cos \beta + \frac{2k \sin \alpha}{AX} \right]^2 \]

\[ \times \left[ \frac{2}{3} (y-z) \cos \beta - \frac{2k \sin \alpha}{3(AX)} + z \cos \beta \right] \]

\[ = \frac{1}{2} \tan^2 \beta \gamma \left[ B \left( y-z \right)^2 \cos^2 \beta + \frac{4k(y-z) \cos \beta \sin \alpha}{AX} + \frac{4k^2 \sin^2 \alpha}{(AX)^2} \right] \]

\[ \times \left[ \frac{2}{3} (y-z) \cos \beta - \frac{2k \sin \alpha}{3(AX)} \times z \cos \beta \right] \]

\[ AM_t = \frac{1}{2} \tan^2 \beta \left[ (y-z)^2 \cos^2 \beta + \frac{4k(y-z) \cos \beta \sin \alpha}{AX} + \frac{4k^2 \sin^2 \alpha}{(AX)^2} \right] \]

\[ \times \left[ \frac{2}{3} (y-z) \cos \beta - \frac{2k \sin \alpha}{3(AX)} + Z \cos \beta \right] \]
\[ M_2 = \frac{1}{2} |AH \times HD| \times \gamma \left[ \frac{2}{3} AH + zB \sin \beta \right] \]
\[ = \frac{1}{2} (y-z) B \sin \beta (y-z) B \cos \beta. \gamma \left[ \frac{2}{3} (y-z) B \sin \beta + zB \sin \beta \right] \]
\[ = \frac{1}{2} (y-z) B \sin \beta (y-z) B \cos \beta. \gamma \left[ \frac{2}{3} yB \sin \beta + \frac{1}{3} zB \sin \beta \right] \]

\[ \frac{M_2}{\gamma B^3} = \frac{1}{6} \sin^2 \beta \cos \beta (y-z)^2 (2y+z) \]

\[ AM_2 = \frac{1}{6} \sin^2 \beta \cos \beta (y-z)^2 (2y+z) \]

\[ M_3 = \frac{1}{2} \gamma B^2 \tan \frac{\phi_m}{\cos^2 \phi_m} - k_p \left( \frac{2}{3} B - z B \sin \beta \right) \]
\[ = \gamma B^3 \left[ \frac{1}{2} \tan \frac{\phi_m}{\cos^2 \phi_m} - k_p \left( \frac{2}{3} - z \sin \beta \right) \right] \]

\[ \frac{M_3}{\gamma B^3} = AM_3 \]
\[ = \frac{1}{2} \times \tan \frac{\phi_m}{\cos^2 \phi_m} \left( \frac{2}{3} - z \sin \beta \right) \]

\[ M_6 = \gamma B (y-z) \sin \beta kB \sin \alpha \times \left( \frac{1}{2} B (y-z) \sin \beta + zB \sin \beta \right) \]

where \[ B_l = \frac{2B}{AX} \]

\[ \frac{M_6}{\gamma B^3} = (y-z) \sin \beta k \frac{2}{AX} \sin \alpha \times \frac{1}{2} \left\{ (y-z) \sin \beta + 2 i \sin \beta \right\} \]
\[ = (y-z) \sin \beta \frac{k \sin \alpha}{AX} \left\{ (y+z) \sin \beta \right\} \]

\[ AM_6 = k \sin \alpha \sin^2 \beta (y-z) (y+z)/AX \]

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\[ M_4 = \int_0^{\theta_1} \frac{1}{2} r^2 \sin^2 \theta \, r \times \frac{2}{3} r \sin (\eta) \]

\[ r = r_0 e^{\theta \tan \phi_1} \quad \text{and} \quad \phi_1 = \frac{\phi_m + \phi}{2} \]

Refer to fig. 4-11

\[ \xi = \theta_1 - \beta \]

\[ \eta = \theta - \xi = \theta - (\theta_1 - \beta) \]

\[ M_4 = \int_0^{\theta_1} \frac{1}{2} r^2 \sin^2 \theta \cos \theta \, d\theta \]

\[ = \int_0^{\theta_1} r^2 \sin \{\theta - (\theta_1 - \beta)\} \, d\theta \]

\[ = \frac{r_0^3}{3} \int_0^{\theta_1} e^{\theta \tan \phi_1} \sin \{\theta - (\theta_1 - \beta)\} \, d\theta \]

Using the standard relation

\[ e^{ax} \sin (bx + c) \, dx = \frac{a x \sin (bx + c) - b \cos (bx + c)}{a^2 + b^2} \]

\[ a = 3 \tan \phi, \quad b = 1, \quad c = -(\theta_1 - \beta) \]

\[ M_4 = \frac{r_0^3}{3} \left[ \frac{3 \tan \phi_1}{9 \tan^2 \phi_1 + 1} \left\{ 3 \tan \phi_1 \sin (\theta_1 - \beta) - \cos (\theta_1 - \beta) \right\} \right]_0^{\theta_1} \]

\[ = \gamma B^3 \times \frac{\sin^3 (\phi_m + \frac{\pi}{4} - \frac{\phi_1}{2})}{\sin^3 \theta_1 \cos^3 \phi_m} \frac{e^{\theta \tan \phi_1}}{9 \tan^2 \phi_1 + 1} \left\{ 3 \tan \phi_1 \sin (\theta_1 - \beta) - \cos (\theta_1 - \beta) \right\} \]

\[ + \frac{1}{9 \tan^2 \phi_1 + 1} \left\{ 3 \tan \phi_1 \sin (\theta_1 - \beta) + \cos (\theta_1 - \beta) \right\} \]
\[ M_4 \gamma B^3 = AM_4 = \frac{1}{27 \tan^2 \phi_1 + 3} \left[ \sin(\phi_m \pm \frac{\pi}{2} - \frac{\phi}{2}) \right]^3 \times [a^2_b(3 \tan \phi_1 \sin \beta - \cos \beta)] \]
\[
+ 3 \tan \phi_1 \sin (\theta_1 \beta) + \cos (\phi_1 \beta) \]

\[ M_5 = \frac{1}{2} \times OC \times AN \times \gamma \left( \frac{2}{3} AM \cos[\delta - (45 - \phi/2)] - OA \sin \beta \right) \]

\[ \beta_2 = (45 - \phi/2 + \phi_m) \]

From fig. 4-10 (b)

\[ \frac{B_x}{\sin \theta_1} = \frac{OC}{\sin \beta_2} \]

\[ \therefore OC = \frac{\sin \beta_2}{\sin \theta_1} \cdot B_x \]

From \( \Delta OAM \)

\[ \frac{OM}{\sin \delta} = \frac{MA}{\sin \theta_1} = \frac{OA}{\sin(\theta_1 + \delta)} \]

\[ AM = \frac{\sin \theta_1}{\sin \delta} \cdot \frac{OM}{2 \sin \delta} = \frac{1}{2} \times \frac{\sin \theta_1}{\sin \delta} \times \frac{\sin \beta_2}{\sin \theta_1} \times B_x = \frac{1}{2} \times \frac{\sin \beta_2}{\sin \delta} \cdot B_x \]

\[ M_6 = \frac{1}{2} \times OC \times AN \times \gamma \left( \frac{2}{3} AM \cos[\delta - (45 - \phi/2)] - OA \sin \beta \right) \]

\[ = \frac{1}{2} \times \frac{\sin \beta_2}{\sin \theta_1} \cdot B_x(z \sin \theta_1) \gamma \]

\[ \times \left[ \left( \frac{2}{3} \times \frac{1}{2} \times \frac{\sin \beta_2}{\sin \delta} \right) B_x \cos[\delta - (45 - \phi/2)] - zB \sin \beta \right] \]

\[ = \frac{1}{2} \times z \gamma. B^2 \sin \beta_2 \left( \frac{1}{3} \sin \beta_2 \times \cos[\delta - (45 - \phi/2)] - z \sin \beta \right) \]
\[ \frac{M_s}{\gamma B^3} = \frac{1}{2} x z \sin \beta_2 \left( \frac{1}{3} \times \frac{\sin \beta_2}{\sin \delta} \times x \cos \{\delta - (45 - \frac{\phi}{2})\} - z \sin \beta \right) = AM_5 \] (let)

Using the trigonometric relation

\[ \cos \{\delta - (45 - \frac{\phi}{2})\} = \cos [90 - (\delta + \phi)] = \cos (90 - (\delta + \beta)) = \sin (\delta + \beta) \]

\[ AM_5 = \frac{1}{2} x z \sin \beta_2 \left( \frac{1}{3} \times \frac{\sin \beta_2}{\sin \delta} \times x \sin (\delta + \beta) - z \sin \beta \right) \]

\[ = \frac{1}{2} x \frac{\sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1} \times \sin \beta_2 \left( \frac{1}{3} \times \frac{\sin \beta_2}{\sin \delta} \times x \sin (\delta + \beta) \right) \]

\[ = \frac{x^3}{6} \frac{\sin^2 \beta_2 \sin (\delta + \beta) \sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1 \sin \delta} \times \frac{x^3}{2} \frac{\sin \beta_2 \sin \beta}{\sin \theta_1} \]

\[ = \frac{x^3}{6} \frac{\sin^2 \beta_2 \sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1} \times \frac{x^3}{2} \frac{\sin \beta_2 \sin \beta}{\sin \theta_1} \]

\[ = \frac{x^3}{6} \frac{\sin^2 \beta_2 \sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1} \times \frac{x^3}{2} \frac{\sin \beta \sin \beta_2}{\sin \theta_1} \]

\[ = \frac{x^3}{6} \frac{\sin^2 \beta_2 \sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1} \times \frac{x^3}{2} \frac{\sin \beta \sin \beta_2}{\sin \theta_1} \]

\[ = \frac{x^3}{6} \frac{\sin^2 \beta_2 \sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1} \times \frac{x^3}{2} \frac{\sin \beta \sin \beta_2}{\sin \theta_1} \]

\[ = \frac{x^3}{6} \frac{\sin^2 \beta_2 \sin(\theta_1 + \phi_m + 45 - \frac{\phi}{2})}{\sin \theta_1} \times \frac{x^3}{2} \frac{\sin \beta \sin \beta_2}{\sin \theta_1} \]

\[ \text{152} \]
Using the standard relation

\[ \cot \delta = 2\cot \beta_2 + \cot \theta_1 \]

\[
M_s = \frac{x^3 \sin^2 \beta_2 \sin(\theta + \beta_2)}{6} \frac{\cos \beta + 2\cot \beta_2 \sin \beta + \cot \theta_1 \sin \beta}{\sin \theta_1}
\]

\[
AM_s = \frac{1}{6} \frac{x^3 \sin^2 \beta_2 \sin(\theta + \beta_2)}{\sin \theta_1} \frac{\sin^2(\theta_1 + \beta_2)}{\sin^2 \theta_1} \frac{\cos \beta + 2\cot \beta_2 \sin \beta + \cot \theta_1 \sin \beta}{\sin \theta_1}
\]

A computer programme is devised to solve the above equations and is included in chapter 3 of vol. II. The derivation of bearing capacity factors for skirted footings includes the sand property parameter \((e-e_{\text{min}})\). Thus the usual diagrams showing the relations between angle of internal friction \(\phi\) and the bearing capacity parameters can not be drawn, or if they are drawn, they are applicable to those foundation sand whose \((e-e_{\text{min}})\) are the same. Once the property \((e-e_{\text{min}})\) is provided as the data in computer programme, the Bearing capacity factors can be easily obtained. A few theoretical diagrams are however drawn using \(\phi\) as the variable assuming \((e-e_{\text{min}})\) as constant to check the nature of the diagrams with available literature on plane footings for the axially loaded case.

Following diagrams are drawn for the axially loaded case. These diagrams / charts are portrayed after making the possible smoothening of theoretical values using Microsoft excel and hence, while comparing the experimental failure load by using computer programme and the charts some discrepancies may be observed.
Typical set of the calculations for $B_t = 50\;\text{mm}$, $\phi = 34.5^\circ$ (obtained in the present investigation by adjusting particular height of raining) with various values of $k = 0, 0.1, 0.25, 0.4, 0.5,$ and $0.6$ and $e-e_{\text{emin}} = 0.222$ are given in table 4-8 and illustrated in figure 4-12 a, b, c, d. Similarly for other values of $B_t = 20\;\text{mm}$ and $75\;\text{mm}$ for different $k$, set of tables and figures can be displayed.

For the same sand used in this investigation i.e. for the same $e_{\text{emin}}$ with different $e$ obtained by changing the height of Rainer i.e. with various $\phi$ values the tables and charts can be illustrated similar to (1) using the same computer programme given in volume II.

For any sand having particular value of $e$ and $e_{\text{emin}}$ similar tables and charts can be displayed by using the computer programme given in volume II which reflects its versatility.

For different sands, the typical $e_{\text{emin}}$ values (Jumikis, '51) are:

- Uniform spheres $= 0.35$
- Clean uniform sand $= 0.40$
- Uniform organic silt $= 0.40$
- Silty sand $= 0.30$
- Fine to coarse sand $= 0.20$
- Micaceous sand $= 0.40$
- Silty sand and gravels $= 0.14$

Theoretical graphs indicate that the bearing capacity factors increase with angle of skirt $\alpha$ up to about $30^\circ$ and then drop. Also the bearing capacity factors increase with the aspect ratio.
4.4.6. TERZAGHI MODIFIED AND CLASSICAL EQUATIONS FOR BEARING CAPACITY FACTORS (PLANE STRIP FOOTINGS) FROM THE DERIVATION OF EQUATIONS OF PRESENT INVESTIGATIONS (SKIRTED STRIP FOOTINGS)

Present Investigation (Terzaghi’s Modified equation)

In the above derivation, substitute \( B_t = B \) for plane strip footing and \( k = 0 \) to transform the skirted footing as the plane footing. The areas of inert wedge are

\[ A_1 = A_2 = 0.25 B^2 \tan \phi \]

Therefore

\[ \text{Inert wedge ratio } \left( \frac{A_1}{A_2} \right) = 1.0 \]

and

\[ C_i = \left[ \sqrt{\frac{A_1}{A_2}} - 1.0 \right] \left[ \sqrt{e - e_{\text{min}}} \right] = 0. \]

Thus the derivation reduces to that for plane strip footing after considering the scale effect and progressive rapture concept of present investigations.

Terzaghi's classical theory

Substituting \( B_t = B \) and \( k = 0 \) the skirted footing can be plane footing and also putting \( C_s = 0 \) the consideration of the size effect can be deleted. Then the improvement factor

\[ C_s \tan \phi + C_i = 0, \text{ i.e. } \phi_m = \phi. \]

Also accounting \( 0.5 (\phi_m + \phi) = 0.5 (\phi + \phi) = \phi \), the effect of progressive rupture failure can be obscured and the derivation can tend to classical Terzaghi.
Table 4-8

Bearing capacity parameter for axially loaded skirted footings on sand for
Bt = 50 mm and $\phi = 34.5^\circ$, $\epsilon_{\text{emin}} = 0.222$

<table>
<thead>
<tr>
<th>Angle of skirt</th>
<th>Aspect Ratio $k$</th>
<th>Bearing capacity factor $N_{qs}$</th>
<th>Bearing capacity factor $N'_{qs}$</th>
<th>Bearing capacity factor $N_{ys}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.00</td>
<td>0.10</td>
<td>0.25</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>41.27</td>
<td>49.10</td>
<td>60.68</td>
<td>72.08</td>
</tr>
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<td>50.66</td>
<td>63.85</td>
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</tr>
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<td>41.27</td>
<td>50.97</td>
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</tr>
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<td>49.36</td>
<td>51.31</td>
</tr>
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Fig. 4-12 (a)

Bearing capacity parameter $N_{qs}$ for skirted footings on sand for $B_t = 50$ mm

Values of $N_{qs}$ vs. Angle of the skirt $\alpha$ in deg.
Bearing capacity parameter $N'_{qs}$ for skirted footings on sand for $B_t = 50$ mm

Fig. 4 - 12 (b)
Values of \( N \), Bearing capacity parameter for skirted footings on sand for \( B_t = 50 \text{ mm} \)

Fig. 4-12 (C)

Bearing capacity parameter \( N_{ts} \) for skirted footings on sand for \( B_t = 50 \text{ mm} \)

Fig. 4 - 12 (c)
Bearing capacity parameter $N'_{s}$ for skirted footings on sand for $B_t = 50 \text{ mm}$

Fig. 4 - 12 (d)
4.5 ECCENTRICALLY, OBLIQUELY LOADED SKIRTED FOOTINGS ON SAND

Consider a skirted footing with eccentric (e) and oblique (i) load in x direction as shown in fig. 4-13 (a). The following assumptions have been made in the development of analytical solution.

(1) One sided failure surface occurs along with the surface as shown in the figure [as observed by Jumikis (1961), fig. 4-13(b)]. The failure region can be divided into three zones (I), (II) and (III). Zone I represents an elastic zone defined by wedge angles \( \alpha_1 \) and \( \alpha_2 \). Zones II and III are similar to those adopted for axially loaded skirted footings.

(2) A similar rupture surface is considered when the footing loses its contact with the soil due to excessive eccentricity \( e > B_i/6 \). The footing loses its contact in a characteristic manner as the load eccentricity increases.

(3) Soil on the left of the failure plane \( A''C \) contributes passive resistance to failure characterised by a mobilisation factor \( m_2 \) where \( \tan \phi_m = m_2 \tan \phi_m \).

(4) Superposition of limit stresses for both the cases i.e. \( q = 0 \) and \( \gamma = 0 \) holds well where \( \gamma \) is the density of soil and \( q \) is the intensity of surcharge.

Analytical solutions are developed for the general case when the footing loses its contact with the soil. The contact width of the footing \( A''A \) is assumed to be \( 2B \times x_i \) as shown in the figure. For complete contact of footing with soil, \( x_i = 1.0 \). The geometry of the failure surface is similar to that of the axially loaded footings.
ECCENTRICALLY OBLIQUELY LOADED SKIRTED FOOTING ON SAND

FIG: 4.13(a)
Two-sided rupture surface from a vertical centrally applied load

One-sided rupture surface from vertical, eccentric load

One-sided rupture surface under obliquely load

Rupture surfaces observed by Jumikis (1961)

Fig. 4-13 (b)
The modified angle of internal friction $\phi_m$ is considered to be similar to the axially loaded case.

$$\tan \phi_m = \{ \exp (C_s \tan \phi + C_i) \} \tan \phi$$

$$C_s = 0.15 - 0.4 \sqrt{B_1}$$

$$C_i = \{(A_1/A_2)^{0.5} - 1\} \{e - e_{min}\}^{0.10}$$

The areas $A_1$ and $A_2$ are as defined previously in fig. 4-7.

4.5.1 Derivation of bearing capacity factors for eccentrically, obliquely loaded skirted strip footing

Consider a unit length of a skirted footing with usual nomenclatures subjected to an eccentric and oblique load as shown in fig. 4-13 (b). This forms an eccentricity $e_1$ and obliquity $i$ at the soil foundation interface. Eccentricity $e_1$ and obliquity $i$ have their positive directions as indicated in the figure. This $e_1$ is considered as eccentricity for the derivation. The weight of the confined sand mass is assumed to be included in the self-weight of the footing whereas the weight of triangular inert wedge under the footing is neglected. Consider the equilibrium of an elastic wedge.

$$\Sigma X = 0$$

$$Q \sin i = P_p \sin (\alpha_1 \cdot \phi_m) - P_{p2} \sin (\alpha_2 - \phi_{m2}) \ldots \ldots \ldots \ldots \ldots (1)$$

$$\Sigma Y = 0$$

$$Q \cos i = P_p \cos (\alpha_1 \cdot \phi_m) + P_{p2} \cos (\alpha_2 - \phi_{m2})$$

The passive pressure $P_p = P_{pq} + P_{pr}$ and $P_{p2} = P_{pq2} + P_{pr2}$. 
Substituting

\[ Q \cos i = [P_{pq} \cos (\alpha_1 \cdot \phi_m) + P_{pq2} \cos (\alpha_2 - \phi_{m2})] \]
+ \[P_{py} \cos (\alpha_1 \cdot \phi_m) + P_{py2} \cos (\alpha_2 - \phi_{m2})] \]
\[ \therefore Q = 2B \left[ q \frac{P_{pq} \cos(\alpha_1 \cdot \phi_m)}{2Bq \cos i} + q \frac{P_{pq2} \cos(\alpha_2 - \phi_{m2})}{2Bq \cos i} \right] \]
+ \[2B \left[ \frac{B\gamma P_{py} \cos(\alpha_1 \cdot \phi_m) + B\gamma P_{py2} \cos(\alpha_2 - \phi_{m2})}{2\gamma B^2 \cos i} \right] \]
\[ \therefore Q = 2B \left[ q, N_q + B, \gamma, N_t \right] \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (1) \]

where
\[ N_q = \frac{P_{pq} \cos(\alpha_1 \cdot \phi_m)}{2Bq \cos i} + \frac{P_{pq2} \cos(\alpha_2 - \phi_{m2})}{2Bq \cos i} \]
and
\[ N_t = \frac{P_{py} \cos(\alpha_1 \cdot \phi_m)}{2\gamma B^2 \cos i} + \frac{P_{py2} \cos(\alpha_2 - \phi_{m2})}{2\gamma B^2 \cos i} \]

Substitute \( 2B = B_t (1 + 2k \cos \alpha) \) in equation (1)

\[ Q = B_t (1 + 2k \cos \alpha) \left[ q, N_q + \frac{1}{2} B_t (1 + 2k \cos \alpha), \gamma, N_t \right] \]

\[ = B_t [q, N_q (1 + 2k \cos \alpha) + \frac{1}{2} B_t (1 + 2k \cos \alpha)^2 \gamma, N_t] \]
\[ \therefore Q = B_t [q, N_{qs}, N_{ys}] \]

where \( N_{qs} = N_q (1 + 2k \cos \alpha) \quad \ldots \quad \ldots \quad \ldots \quad (4-1a) \)

and \( N_{ys} = N_t (1 + 2k \cos \alpha)^2 \quad \ldots \quad \ldots \quad \ldots \quad (4-1b) \)
$N_{qs}$ and $N_{rs}$ are bearing capacity factors for skirted footings on sand.

Thus if $q_0$ is the bearing capacity of soil, the failure load is given by

$$Q_d = q_d \times B_t \times \ell$$

where $\ell = \text{length of the footing}$

and $q_0 = q \cdot N_{qs} + \frac{1}{2} B_t \gamma \cdot N_{rs}.$

The method of finding these values consists of considering equilibrium of mass $GACDHL$. The required passive pressures are obtained by considering the forces acting on free body and applying the equilibrium equations for a selected set of $\alpha_1$ and $\alpha_2$ where $\alpha_2 < \alpha_1 < \phi_m$. The log spiral which gives the least value of the passive pressure is selected for calculations. The wedge equilibrium equation (1) i.e. $\sum X = 0$ is checked with the values of passive pressures obtained. The procedure is repeated for another set of $\alpha_1$ and $\alpha_2$ till the equation is satisfied. Bearing capacity factors $N_{qs}$ and $N_{rs}$ are determined at the end of the iteration process. The calculations are carried out by using Fortran programme in computer.

When $i = 0$, $e/b = 0$, then the case is that of axially loaded footing. However, the values of $N_{qs}$ and $N_{rs}$ obtained from axially loaded case and from eccentrically loaded case are slightly different because of computational limitations. The increment of wedge angles $\alpha_1$ and $\alpha_2$ adopted in computer programme is unity (one) which can not take the case of $\alpha_1 = 36.27^\circ$ for example but $\alpha_1 = 36^\circ$. This limitation is reflected in the results.

4.5.2. WEDGE ANGLES RELATIONSHIP:

The wedge angle relationships can be derived by considering the equilibrium of the elastic wedge. Let the foundation lose its contact with soil such that there exists a contact of $AA'' = 2Bx_1$ as shown in fig. 4-14.
(b) CONTACT WITH FACTOR FOR FOOTING \(x_4\) SUBJECTED TO ECCENTRIC LOAD (assumed)

\[ e_1 = e + k B_t \sin \alpha \cdot \tan \beta \]

(c) RELATION BETWEEN \(e\) AND \(e_1\)
Contact width factor $x_1$ is defined as follows:

$k = 0 \text{ i.e. for plane footings}$

$$x_1 = 1.0 - (1+z_1) z_1 \text{ for } z_1 \leq 0.2$$

$$= 1.25 - 2.5 z_1 - 0.25 \tan (i) \text{ for } z_1 > 0.2$$

$k > 0 \text{ i.e. for skirted footings}$

$$x_1 = 1.0 - (0.75+z_1) z_1 \text{ for } z_1 \leq 0.25$$

$$= 1.5 - 3 z_1 - 0.25 \tan (i) \text{ for } z_1 > 0.25$$

In case of plane footings subjected to the eccentric loads, Meyerhof's concept of effective width is based on frictional resistance between the footing material and soil continuum. Because of the reduction in surface frictional angle due to increase of eccentricity, which is inducing the grip between the footing and soil against eccentricity satisfies the above rate even though the effective area concept seems to be conservative from the work of past research workers (Saran, Agrawal, 1973). Therefore this work assumes the law of contact width as mentioned above instead of $1-2 z_1$ as given by Meyerhof.

In case of skirted footing, the frictional resistance between the soil plug within the skirts and foundation sand mass at the interface induced by eccentricity activate the dilation which may be very less during the initial stages but may increase at the later stages and due to this, the average degree of reduction i.e. linear variation is assumed while adopting the contact width factor.

Relation between $e$ (applied eccentricity) and $e_t$, the eccentricity at interface is given as follows, referring to fig. 4-14 (c)

$$e_t = e + [kB_t \sin \alpha + t_i] \tan i$$
As the designer shall assume the thickness of footing, the bearing
capacity factors are based on $e$ at the inside bottom of the footing. This $e$
should be considered as the input of the program. The wedge angle relationships for both the
cases, viz. (i) $\gamma = 0$ and (ii) $q = 0$ are derived as follows.

From $\triangle AA'C$ of fig. 4-15

$$\frac{AC}{\sin \alpha_2} = \frac{A'C}{\sin \alpha_1} = \frac{2Bx_1}{\sin(\alpha_1 + \alpha_2)}$$

Therefore $AC = \frac{\sin \alpha_2}{\sin(\alpha_1 + \alpha_2)} (2Bx_1)$,

and $AC' = \frac{\sin \alpha_1}{\sin(\alpha_1 + \alpha_2)} (2Bx_1)$

From three force equilibrium of fig. 4-15 (a)

$$\frac{Q_q}{\sin(\alpha_1 + \alpha_2 - \phi_m - \phi_{m2})} = \frac{P_{pp}}{\sin(\alpha_2 - \phi_{m2} + i)} = \frac{P_{pq2}}{\sin(\alpha_1 - \phi_m - i)}$$

= $R$ (assume) ... ... ... (1)

Similarly from fig. 4-15 (b)

$$\frac{Q_q}{\sin(\alpha_1 + \alpha_2 - \phi_m - \phi_{m2})} = \frac{P_{pp}}{\sin(\alpha_2 - \phi_{m2} + i)} = \frac{P_{pq2}}{\sin(\alpha_1 - \phi_m - i)}$$

= $R$ (assume) ... ... ... (2)

Use these equations for the derivation of wedge angle relationships.

To complete the equilibrium conditions, minimise (i.e. tends to zero) the
following.
FIG. 4.15

FORCES ACTING ON WEDGE

THREE FORCE EQUILIBRIUM

CASE I - $\gamma = 0$

CASE II - $\gamma = 0$

$\frac{1}{2} A C \sin \alpha$

$\frac{1}{2} A C \sin \alpha$

$B_1 = 2B$

B

$2B \times 1$

$2B \times 1$

$\phi_{m2}$

$\phi_{m2}$

$P_{P2}$

$P_{P2}$

$P_{Pr}$

$P_{Pr}$

$\alpha_{1-\phi_{m}}$

$\alpha_{1-\phi_{m}}$
\[ R_1 = \frac{P_{yy}}{\sin(\alpha_2 - \phi_{m2} - i)} \]

and \[ R_2 = \frac{P_{y'y}}{\sin(\alpha_1 - \phi_{m} + i)} \]

Minimise the absolute value of \( R_1 - R_2 \).

**Case I \( (\gamma = 0) \):**

Taking moments of all forces @ \( A'' \) (fig. 4 - 16 a)

\[
Q_q \cos i (2 B_i x - B + e_i) + P_{pq} \sin (\alpha_1 - \phi_{m}) \frac{1}{2} AC \sin \alpha_1
\]

\[
= P_{pq} \cos (\alpha_1 - \phi_{m}) (2 B_1 x_1 - \frac{1}{2} AC \cos \alpha_1)
\]

\[
+ P_{pq2} \sin (\alpha_2 - \phi_{m2}) \frac{1}{2} A'C \sin \alpha_2
\]

\[
+ P_{pq2} \cos (\alpha_2 - \phi_{m2}) \frac{1}{2} A'C \cos \alpha_2.
\]

Substitute for \( AC \) and \( A'C \) from (1) and for \( Q_q, P_{pq} \) and \( P_{pq2} \) from (2) and divide both the sides by \( R (2B_1 x_1) \)

\[
sin (\alpha_1 + \alpha_2 - \phi_{m} - \phi_{m2}) \cos i (1 - \frac{1}{2x_1} + \frac{e_1}{2B_2})
\]

\[
+ \sin (\alpha_2 - \phi_{m2} + i) \sin (\alpha_1 - \phi_{m}) \frac{1}{2} \times \frac{\sin \alpha_1 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2)}
\]

\[
= \sin (\alpha_2 - \phi_{m2} + i) \cos (\alpha_1 - \phi_{m}) \left(1 - \frac{1}{2} \times \frac{\sin \alpha_2 \cos \alpha_1}{\sin(\alpha_1 + \alpha_2)}\right)
\]
\[ \sin (\alpha_1 - \phi_m - i) \sin (\alpha_2 - \phi_{m2}) + \sin (\alpha_1 - \phi_m - i) \cos (\alpha_2 - \phi_{m2}) \left\{ \frac{\sin \alpha_1 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2)} \right\} \]

Substitute \( H_1 = \left( 1 - \frac{1}{2x_1} + \frac{e_1}{2Bx_1} \right) \) and rearrange as follows.

\[ 2 \sin (\alpha_1 + \alpha_2) \sin (\alpha_1 + \alpha_2 - \phi_m - \phi_{m2}) H_1 \cos i \]

\[ -2 \sin (\alpha_1 + \alpha_2) \sin (\alpha_2 - \phi_{m2} + i) \cos (\alpha_1 - \phi_m) \]

\[ = \sin (\alpha_1 - \phi_m - i) \{ \sin (\alpha_2 - \phi_{m2}) \sin \alpha_2 + \cos (\alpha_2 - \phi_{m2}) \cos \alpha_2 \} \sin \alpha_1 \]

\[ -\sin (\alpha_2 - \phi_{m2} + i) \{ \cos (\alpha_1 - \phi_m) \cos \alpha_1 + \sin (\alpha_1 - \phi_m) \sin \alpha_1 \} \sin \alpha_2 \]

This gives

\[ 2(\sin \alpha_1 \cos \alpha_2 + \cos \alpha_1 \sin \alpha_2) \{ \sin \alpha_2 \cos (\phi_{m1} - \phi_{m2}) \}
+ \cos \alpha_2 \sin (\phi_{m1} - \phi_{m2}) \} H_1 \cos i - 2(\sin \alpha_1 \cos \alpha_2 + \cos \alpha_1 \sin \alpha_2) \]

\[ x \{ \sin \alpha_2 \cos (\phi_{m2} - i) - \cos \alpha_2 \sin (\phi_{m2} - i) \} \cos (\alpha_1 - \phi_m) \]

\[ = \sin (\alpha_1 - \phi_m - i) \cos \phi_{m2} \sin \alpha_1 - \{ \sin \alpha_2 \cos (\phi_{m2} - i) - \cos \alpha_2 \sin (\phi_{m2} - i) \} \cos \phi_m \sin \alpha_2. \]

Expand and divide by \( \cos^2 \alpha_2 \) (i.e. multiply by \( 1 + \tan^2 \alpha_2 \) where required, on RHS)

\[ 2 \sin \alpha_1 \tan \alpha_2 \cos (\alpha_1 - \phi_m - \phi_{m2}) H_1 \cos i + 2 \sin \alpha_1 \sin (\alpha_1 - \phi_m - \phi_{m2}) H_1 \cos i \]

\[ +2 \cos \alpha_1 \tan^2 \alpha_2 \cos (\alpha_1 - \phi_m - \phi_{m2}) H_1 \cos i + 2 \cos \alpha_1 \tan \alpha_2 \sin (\alpha_1 - \phi_m - \phi_{m2}) H_1 \cos i \]

\[ -2 \sin \alpha_1 \tan \alpha_2 \cos (\phi_{m2} - i) \cos (\alpha_1 - \phi_m) + 2 \sin \alpha_1 \sin (\phi_{m2} - i) \cos (\alpha_1 - \phi_m) \]

\[ -2 \cos \alpha_1 \tan^2 \alpha_2 \cos (\phi_{m2} - i) \cos (\alpha_1 - \phi_m) + 2 \cos \alpha_1 \tan \alpha_2 \sin (\phi_{m2} - i) \cos (\alpha_1 - \phi_m) \]

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\[ = \sin(\alpha_1 \phi_{m-i}) \cos \phi_{m2} \sin \alpha_1 + \sin(\alpha_1 \phi_{m-i}) \cos \phi_{m2} \sin \alpha_1 \tan^2 \alpha_2 \]

\[- \tan^2 \alpha_2 \cos (\phi_{m2-i}) \cos \phi_m + \tan \alpha_2 \sin(\phi_{m2-i}) \cos \phi_m \]

Re-arrange as \( A_q \tan^2 \alpha_2 + B_q \tan \alpha_2 + C_q = 0. \)

\[
[2 \cos \alpha_1 \cos (\alpha_1 \phi_{m-\phi_{m2}}) H_1 \cos \theta - 2 \cos \alpha_1 \cos (\phi_{m2-i}) \cos (\alpha_1 \phi_m)
- \sin (\alpha_1 \phi_{m-i}) \cos \phi_{m2} \sin \alpha_1 + \cos (\phi_{m2-i}) \cos \phi_m] \tan^2 \alpha_2
+ [2 \sin \alpha_1 \cos (\alpha_1 \phi_{m-\phi_{m2}}) H_1 \cos \theta + 2 \cos \alpha_1 \sin (\alpha_1 \phi_{m-\phi_{m2}}) H_1 \cos \theta
- 2 \sin \alpha_1 \cos (\phi_{m2-i}) \cos (\alpha_1 \phi_m) + 2 \cos \alpha_1 \sin (\phi_{m2-i}) \cos (\alpha_1 \phi_m)
- \sin (\phi_{m2-i}) \cos \phi_m] \tan \alpha_2
+ [2 \sin \alpha_1 \sin (\alpha_1 \phi_{m-\phi_{m2}}) H_1 \cos \theta + 2 \sin \alpha_1 \sin (\phi_{m2-i}) \cos (\alpha_1 \phi_m)
- \sin (\alpha_1 \phi_{m-i}) \cos \phi_{m2} \sin \alpha_1] = 0. \]

**Case II \((q = 0)\)**

Taking moments of all forces @ \( A" \) (fig. 4-16 b)

\[
Q_y \cos i (2 B_1 x - B + e_1) + P_{py} \sin (\alpha_1 \phi_m) \frac{2}{3} AC \sin \alpha_1
= P_{py} \cos (\alpha_1 \phi_m) (2 B x_1 - \frac{2}{3} AC \cos \alpha_1) + P_{py2} \sin (\alpha_2 \phi_{m2}) \frac{2}{3} A^* C \sin \alpha_2
+ P_{py2} \cos (\alpha_2 \phi_{m2}) \frac{2}{3} A^* C \cos \alpha_2.
\]

Substitute for \( AC \) and \( A^* C \) from (1) and for \( Q_y, P_{py} \) and \( P_{py2} \) from (2) and divide both the sides by \( R (2Bx_1) \)
\[
\sin (\alpha_1 + \alpha_2 - \phi_m - \phi_{m2}) \cos i \left(1 - \frac{1}{2x_1} + \frac{e_1}{2Bx_1}\right)
\]

\[
+ \sin (\alpha_2 - \phi_{m2} + i) \sin (\alpha_1 - \phi_m) \frac{2}{3} \times \frac{\sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 + \alpha_2)}
\]

\[
= \sin (\alpha_2 - \phi_{m2} + i) \cos (\alpha_1 - \phi_m) \left(1 - \frac{2}{3} \times \frac{\sin \alpha_2 \cos \alpha_1}{\sin (\alpha_1 + \alpha_2)}\right)
\]

\[
+ \sin (\alpha_1 - \phi_m - i) \sin (\alpha_2 - \phi_{m2}) \frac{2}{3} \times \frac{\sin \alpha_1 \sin \alpha_2}{\sin (\alpha_1 + \alpha_2)}
\]

\[
+ \sin (\alpha_1 - \phi_m - i) \cos (\alpha_2 - \phi_{m2}) \frac{2}{3} \times \frac{\sin \alpha_1 \cos \alpha_1}{\sin (\alpha_1 + \alpha_2)}
\]

Substitute \( H_1 = \left(1 - \frac{1}{2x_1} + \frac{e_1}{2Bx_1}\right) \) and rearrange as follows.

\[
\frac{3}{2} \sin (\alpha_1 + \alpha_2) \sin (\alpha_1 + \alpha_2 - \phi_m - \phi_{m2}) H_1 \cos i
\]

\[
- \frac{3}{2} \sin (\alpha_1 + \alpha_2) \sin (\alpha_2 - \phi_{m2} + i) \cos (\alpha_1 - \phi_m)
\]

\[
= \sin (\alpha_1 - \phi_m - i) \{\sin (\alpha_2 - \phi_{m2}) \sin \alpha_2 + \cos (\alpha_2 - \phi_{m2}) \cos \alpha_2 \sin \alpha_1 - \sin (\alpha_2 - \phi_{m2} + i) \{\cos (\alpha_1 - \phi_m) \cos \alpha_1 + \sin (\alpha_1 - \phi_m) \sin \alpha_1\} \sin \alpha_2
\]

This gives

\[
\frac{3}{2} \{\sin \alpha_1 \cos \alpha_2 + \cos \alpha_1 \sin \alpha_2\} \{\sin \alpha_2 \cos (\alpha_1 - \phi_m - \phi_{m2}) \}
\]

\[
+ \cos \alpha_2 \sin (\alpha_1 - \phi_m - \phi_{m2})\} H_1 \cos i - \frac{3}{2} \{\sin \alpha_1 \cos \alpha_2 + \cos \alpha_1 \sin \alpha_2\}
\]

\[
x \{\sin \alpha_2 \cos (\phi_{m2} - i) - \cos \alpha_2 \sin (\phi_{m2} - i)\} \cos (\alpha_1 - \phi_m)
\]

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\[
\frac{3}{2} \sin\alpha_1 \tan\alpha_2 \cos (\alpha_1 - \phi_m - \phi_m) H_1 \cos i + \frac{3}{2} \sin\alpha_1 \sin (\alpha_1 - \phi_m - \phi_m) H_1 \cos i
\]

\[
+ \frac{3}{2} \cos\alpha_1 \tan^2\alpha_2 \cos (\alpha_1 - \phi_m - \phi_m) H_1 \cos i + \frac{3}{2} \cos\alpha_1 \tan\alpha_2 \sin (\alpha_1 - \phi_m - \phi_m) H_1 \cos i
\]

\[
- \frac{3}{2} \sin\alpha_1 \tan\alpha_2 \cos (\phi_m - i) \cos (\alpha_1 - \phi_m) + \frac{3}{2} \sin\alpha_1 \sin (\phi_m - i) \cos (\alpha_1 - \phi_m)
\]

\[
- \frac{3}{2} \cos\alpha_1 \tan^2\alpha_2 \cos (\phi_m - i) \cos (\alpha_1 - \phi_m) + \frac{3}{2} \cos\alpha_1 \tan\alpha_2 \sin (\phi_m - i) \cos (\alpha_1 - \phi_m)
\]

\[
= \sin (\alpha_1 - \phi_m - i) \cos \phi_m \sin \alpha_1 + \sin (\alpha_1 - \phi_m - i) \cos \phi_m \sin \alpha_1 \tan^2\alpha_2
\]

\[- \tan^2\alpha_2 \cos (\phi_m - i) \cos \phi_m + \tan\alpha_2 \sin (\phi_m - i) \cos \phi_m.
\]

Re-arrange as \( A_\tan^2\alpha_2 + B_\tan\alpha_2 + C_\tan = 0. \)
Refer to fig. 4-16.

From $\triangle A''AC$

\[
\frac{A''C}{\sin \alpha_1} = \frac{AC}{\sin \alpha_2} = \frac{2Bx_1}{\sin(\alpha_1 + \alpha_2)}
\]

\[
AC = \frac{2Bx_1 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2)}
\]

\[
A''C = \frac{2Bx_1 \sin \alpha_1}{\sin(\alpha_1 + \alpha_2)}
\]

Let $x = \frac{2 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2)}$. $\therefore AC = Bxx_1$

Let $\beta = 45 + \phi/2$.

From $\triangle OAC$

\[
\frac{Bxx_1}{\sin \theta_1} = \frac{OC}{\sin(45 - \frac{\phi}{2} + \alpha_1)} = \frac{zB}{\sin(\theta_1 + 45 - \frac{\phi}{2} + \alpha_1)}
\]

\[
OC = r_0 \frac{Bxx_1 \sin(\theta_1 + 45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1}
\]

\[
OA = zB = r_0 \frac{Bxx_1 \sin(\theta_1 + 45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1}
\]

Let $a_o = e^{\theta_1 \tan \phi}$

\[
OD = r_o a_o = \frac{Bxx_1 \sin(45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1} = yB \text{ (Let)}
\]
Finally we have

\[ x = \frac{2 \sin \alpha_2}{\sin(\alpha_1 + \alpha_2)} \]

\[ y = \frac{x x_1 \sin(45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1} a_0 \]

\[ z = r_0 = \frac{B x x_1 \sin(\theta_1 + 45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1} \]

\[ a_0 = e^{\theta_1 \tan \phi} \]

\[ OA = zB \]

\[ OC = r_0 = \frac{B x x_1 \sin(\theta_1 + 45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1} \]

\[ 2B = B_t + 2k B_t \cos \alpha \]

\[ = B_t (1 + 2k \cos \alpha) \]

\[ = B_t (AX) \]

\[ AX = (1 + 2k \cos \alpha) \]

\[ OC = r_0 = \frac{B x x_1 \sin(45 - \frac{\phi}{2} + \alpha_1)}{\sin \theta_1} \]

\[ OD = yB \]

\[ AD = (y-z)B \]

\[ HD = (y-z)B \cos \beta \]
\[ AH = (y-z) B \sin \beta \]

\[ LH = k B_t \sin \alpha = \frac{2kB \sin \alpha}{AX} \]

\[ AX = 1 + 2k \cos \alpha. \]

\[ B_t = \frac{2B}{AX}. \]

For the selected value of \( \alpha_1 \), the corresponding value of \( \alpha_2 \) can be determined by using the wedge-angle relationships. As per the directions of forces used in fig. 4-16, \( \alpha_1 > \alpha_2 \). Also, \( \alpha_1 \) can neither be less than \( \phi_m \) nor greater than \( 45^\circ + \phi_m/2 \). Keeping this in view, the computer programme is devised to solve the equations. For every increase of \( 1^\circ \) angle in \( \alpha_1 \), the value of \( \alpha_2 \) is determined. For each set of \( \alpha_1 \) and \( \alpha_2 \), the values of passive pressures are determined. For assumed value of \( m \), the mobilisation factor for shear failure, the values of \( N_{qs} \) and \( N_{qs} \) are determined. The values of bearing capacity factors corresponding to maximum mobilization factor shall be adopted. The moment calculations are tabulated in tables 4-9 and 4-10.

Apply conditions of equilibrium.

\[ \Sigma M = 0 \]

\[ P_{pq} \left[ \cos (\alpha_1-\phi_m) \left( \frac{Bxx}{2} \cos \alpha_1 - z B \sin \beta \right) + \sin (\alpha_1-\phi_m) \left( \frac{Bxx}{2} \sin \alpha_1 + z B \cos \beta \right) \right] \]

\[ = \frac{1}{2} q N_\phi \left( (HD+LH) [(HD - LH) + 2 z B \cos \beta ] + \frac{1}{2} q AH (AH+2 z B \sin \beta) \right) \]

\[ = \frac{1}{2} q N_\phi \left[ (HD^2 - LH^2) + 2 z B (HD - LH) \cos \beta \right] + \frac{1}{2} q AH (AH+2 z B \sin \beta) \]
\[ = \frac{1}{2} q N_{q_b} [(y-z)^2 B^2 \cos^2 \beta] \]

\[ \frac{4k^2 B^2 \sin^2 \alpha}{(AX)^2} + 2zB \cos \beta \{xy-zB \cos \beta + \frac{2kB \sin \alpha}{AX}\} \]

\[ + \frac{1}{2} q (y-z) B \sin \beta \{y-z) B \sin \beta + 2zB \sin \beta\} \]

**TABLE 4-9**

Moment calculations for \( N_{qs} \)

<table>
<thead>
<tr>
<th>Force Name</th>
<th>Value</th>
<th>Distance from ( O_1 ), the centre of log-spiral</th>
<th>Moment @ '( O_1 )'</th>
<th>Sign</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_{aq} )</td>
<td>( QN_b , LD )</td>
<td>( \frac{LD}{2} - LH + zB \cos \beta )</td>
<td>( AM_1 )</td>
<td>(+)</td>
</tr>
<tr>
<td></td>
<td>( QN_b , (HD+LH) )</td>
<td>( \frac{1}{2} (HD-LH) + zB \cos \beta )</td>
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<td></td>
</tr>
<tr>
<td>( Q )</td>
<td>( q , AH )</td>
<td>( \frac{1}{2} AH + zB \sin \beta )</td>
<td>( AM_2 )</td>
<td>(+)</td>
</tr>
<tr>
<td>( P_{pq} )</td>
<td>( P_{pq} \cos (\alpha_1-\phi_m) )</td>
<td>( \frac{1}{2} B_{xx_1} \cos \alpha_1 - zB \sin \beta )</td>
<td>( AM_3 )</td>
<td>(-)</td>
</tr>
<tr>
<td></td>
<td>( P_{pq} \sin (\alpha_1-\phi_m) )</td>
<td>( \frac{1}{2} B_{xx_1} \sin \alpha_1 - zB \cos \beta )</td>
<td>( AM_4 )</td>
<td>(-)</td>
</tr>
</tbody>
</table>
Divide both the sides by 2 q B^2

\[ \frac{P_{pq}}{2qB} \left[ \cos(\alpha_1 - \phi_m) \left( \frac{XX_1}{2} \cos \alpha_1 - z \sin \beta \right) + \sin(\alpha_1 - \phi_m) \left( \frac{XX_1}{2} \sin \alpha_1 + z \cos \beta \right) \right] \]

\[ = \frac{N_\phi}{4} \left[ (y - z)^2 \cos^2 \beta - \frac{4k^2 \sin^2 \alpha}{(AX)^2} + 2z \cos \beta \left( (y - z) \cos \beta + \frac{2k \sin \alpha}{AX} \right) \right] \]

\[ + \frac{1}{4} (y - z) \sin \beta \times \{(y - z) \sin \beta + 2z \sin \beta\} \]

Let

\[ \frac{P_{pq}}{2qB} = \frac{S}{T} \]

Where

\[ S = \frac{N_\phi}{4} \left[ (y - z)^2 \cos^2 \beta - \left( \frac{2k \sin \alpha}{AX} \right)^2 \right] + \frac{N_\phi}{4} \left[ 2z \cos \beta \left( (y - z) \cos \beta + \frac{2k \sin \alpha}{AX} \right) \right] \]

\[ + \frac{1}{4} (y - z) (y + z) \sin^2 \beta \]

And

\[ T = \cos (\alpha_1 - \phi_m) \left( \frac{XX_1}{2} \cos \alpha_1 - z \sin \beta \right) + \sin (\alpha_1 - \phi_m) \left( \frac{XX_1}{2} \sin \alpha_1 + z \cos \beta \right) \]

Now

\[ N_{q_1} = \left( \frac{P_{pq}}{2qB} \right) \frac{\cos(\alpha_1 - \phi_m)}{\cos i} \]
Similarly

\[ N_{q2} = \left( \frac{P_{p2}}{2qB} \right) \frac{\cos(\alpha_2 - \phi_{m2})}{\cos i} \]

\( \phi_m = \phi_{m2} \) in above equations.

\[ N_q = N_{q1} + N_{q2} \]

Now

\[ R_1 = \frac{P_{p1}}{\sin(\alpha_1 - \phi_{m2} + i)} = \frac{S1/T1}{\sin(\alpha_1 - \phi_{m2} + i)} \]

and

\[ R_2 = \frac{P_{p2}}{\sin(\alpha_1 - \phi_{m1} - i)} = \frac{S2/T2}{\sin(\alpha_1 - \phi_{m1} - i)} \]

Minimise Absolute value of \((R_1 - R_2)\).

For assumed value of \(m\), the value of \(N_{qs}\) is obtained from these calculations. The minimum value of \(N_{qs}\) for particular value of \(m\) shall be determined. The value of \(m\) shall be increased from 0.1 to 1.0 in the increment of 0.1 and values of \(N_{qs}\) obtained from each case are compared. The maximum value of \(N_{qs}\) shall be adopted for the design to consider the maximum capacity of foundation sand to be mobilised.

Bearing capacity factors \(N_{ys}\) shall be determined in the same way by replacing \(q\) by \(\gamma\) in the above equations i.e. \(N_{ys} = N_{ys1} + N_{ys2}\), and by using the same equations as per the axially loaded case. The moment calculations for eccentrically loaded case are tabulated in table 4-10. A computer programme is devised to solve the above equations and is included in chapter 1, Vol. II.
TABLE 4-10

Moment calculations for \( N_{ys} \)

<table>
<thead>
<tr>
<th>Force Name</th>
<th>Value</th>
<th>Distance from ( O_i ), the centre of log-spiral</th>
<th>Moment @ 'O'</th>
<th>Sign</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P_{dy} )</td>
<td>( \frac{1}{2} N_y \gamma (L D)^2 )</td>
<td>( \frac{2}{3} LD - k B_t \sin \alpha + z B \cos \beta )</td>
<td>( AM_1 )</td>
<td>(+)</td>
</tr>
<tr>
<td>( W_1 )</td>
<td>( \frac{1}{2} \times A H \times H D \times \gamma )</td>
<td>( \frac{2}{3} A H \sin \beta )</td>
<td>( AM_2 )</td>
<td>(+)</td>
</tr>
<tr>
<td>( P_{pr} )</td>
<td>( P_{pr} \cos (\alpha_1 \phi_m) )</td>
<td>( \frac{2}{3} B_{xx_1} \cos \alpha_1 - z B \sin \beta )</td>
<td>( AM_3 )</td>
<td>(-)</td>
</tr>
<tr>
<td></td>
<td>( P_{pr} \sin (\alpha_1 \phi_m) )</td>
<td>( \frac{2}{3} B_{xx_1} \sin \alpha_1 - z B \cos \beta )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \frac{w_2}{(OCD)} )</td>
<td>( \int \frac{1}{2} \tau^2 d \theta \gamma )</td>
<td>( \frac{2}{3} r \sin (\eta) )</td>
<td>( AM_4 )</td>
<td>(+)</td>
</tr>
<tr>
<td>( \frac{w_3}{(OCA)} )</td>
<td>( \frac{1}{2} O C \times A N \times \gamma )</td>
<td>( \frac{2}{3} A M \cos (\delta - (45 - \phi/2)) + O A \sin \beta )</td>
<td>( AM_5 )</td>
<td>(+)</td>
</tr>
<tr>
<td>( \frac{w_3}{(GAC)} )</td>
<td>( \gamma B (y - z) \sin \beta B_t \sin \alpha )</td>
<td>( \frac{1}{2} B (y - z) \sin \beta + z B \cos \beta )</td>
<td>( AM_6 )</td>
<td>(+)</td>
</tr>
</tbody>
</table>

The following equations are used to determine the values of \( N_{ys} \):

\[
\frac{P_{pr}}{\gamma B^3} = \frac{AM_1 + AM_2 + AM_4 + AM_5 + AM_6}{AM_3},
\]

\[
N_{ys1} = \frac{1}{2} \frac{P_{pr}}{\gamma B^3} X \frac{\cos (\alpha_1 - \phi_m)}{\cos (\iota)} \quad N_{ys2} = \frac{1}{2} \frac{P_{pr_2}}{\gamma B^3} X \frac{\cos (\alpha_2 - \phi_m)}{\cos (\iota)}
\]

and \( N_{ys} = N_{ys1} + N_{ys2} \).

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A number of curves consisting of following combinations are drawn to see the variation of the parameters with $\alpha$.

1. $B_t = 220$ mm, $\phi = 34.5^\circ$, $k = 0$ (i.e. plane footings), $i = 0^\circ$, $5^\circ$, $10^\circ$ and $15^\circ$ and $z_i = e/B_t = 0$, $0.1$, $0.2$, $0.3$ and $0.4$. Refer to tables 4-11 to 4-14 and figures 4-17 to 4-20.

2. $B_t = 75$ mm; $\phi = 34.5^\circ$, $e-e_{\text{min}} = 0.222$, $k = 0.25$ (used for present investigation) with various values of $i = 0^\circ$, $5^\circ$, $10^\circ$ and $15^\circ$; $\alpha = 0^\circ$ to $90^\circ$ and $z_i = e/B_t = 0$, $0.1$, $0.2$, $0.3$ and $0.4$. Refer to tables 4-15 to 4-18 and figures 4-21 to 4-24.

3. $B_t = 300$ mm; $\phi = 34.5^\circ$, $e-e_{\text{min}} = 0.222$, $k = 0.25$ (used for present investigation) with various values of $i = 0^\circ$, $5^\circ$, $10^\circ$ and $15^\circ$; $\alpha = 0^\circ$ to $90^\circ$ and $z_i = e/B_t = 0$, $0.1$, $0.2$, $0.3$ and $0.4$. Refer to tables 4-19 to 4-22 and figures 4-25 to 4-28.

Note: In case where $z_i = e/B_t = 0$ and $i = 0$, the graph shows the Terzaghi's theory after considering scale effect and progressive rupture failure as derived in present investigation.

Following inferences are drawn from the charts.

1. Theoretical bearing capacity parameters ($N_{qs}$ and $N_{ys}$) increase with increase of the angle of internal friction, $\phi$, under axial, eccentric and oblique loads for plane footings for all width of plates.

2. For given angle of internal friction, theoretical bearing capacity factors reduce with increase of eccentricity and inclination, both, keeping the positive direction of inclination of loads.

3. For skirted footings, under eccentric and oblique loads, theoretical bearing capacity factors increase with the angle of skirt in the range of $\alpha = 30^\circ$ to $45^\circ$ and thereafter drop. With increase of eccentricity and inclination, the peak value of bearing capacity factors shifts towards $\alpha = 45^\circ$. This is because of variation in theoretical wedge angles $\alpha_1$ and $\alpha_2$. The bearing capacity factors increase with aspect ratio for all inclinations and eccentricities.
### TABLE NO 4-11

**BEARING CAPACITY PARAMETERS Nqs FOR PLANE FOOTINGS**

- **SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS**

$z_1 = \frac{e}{w}$ mm

<table>
<thead>
<tr>
<th>Angle of int. friction</th>
<th>Eccentricity factor $z_1$ = eccentricity (e) / width of footing</th>
<th>$z_1=0.0$</th>
<th>$z_1=0.1$</th>
<th>$z_1=0.2$</th>
<th>$z_1=0.3$</th>
<th>$z_1=0.4$</th>
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</thead>
<tbody>
<tr>
<td>Load inclination = 0°</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5.00</td>
<td>1.57</td>
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<td>Load inclination = 10°</td>
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<tr>
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</table>
BEARING CAPACITY PARAMETER, Nqs FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

Fig. 4.17(a)
BEARING CAPACITY PARAMETER, $N_{qs}$ FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

\[
Z = \frac{c}{B_L}
\]

$B_t = 220$ mm

$\phi = 5^\circ$

Fig. 4-17(b)
BEARING CAPACITY PARAMETER, $N_{qs}$ FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

Fig. 4-17(c)

$N_{qs}$ vs Bearing Capacity Factor

$\phi$ = Angle of Internal Friction

$Z_1 = \text{variable}$

$B_t = 220 \text{ mm}$

$i = 10^0$
BEARING CAPACITY PARAMETER, $N_{qs}$ FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

$Z_1 = \frac{e}{B_t}$

$B_t = 220$ mm

$i = 15^\circ$

Fig. 4-17(d)
## TABLE NO 4-12
BEARING CAPACITY PARAMETERS NyS FOR PLANE FOOTINGS
SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

<table>
<thead>
<tr>
<th>Angle of int. friction</th>
<th>Eccentricity factor $z1 = \text{ eccentricity (e)} / \text{ width of footing} $</th>
<th>$z1=0.0$</th>
<th>$z1=0.1$</th>
<th>$z1=0.2$</th>
<th>$z1=0.3$</th>
<th>$z1=0.4$</th>
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</table>

### Load inclination = 0°

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<th>Load (kN)</th>
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<th>0.00</th>
<th>0.00</th>
<th>0.00</th>
<th>0.00</th>
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BEARING CAPACITY PARAMETER, $N_{ys}$ FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

Fig. 4-18(a)

$B_t = 220$ mm

$i = 0^\circ$
BEARING CAPACITY PARAMETER, $N_{ys}$
FOR PLANE FOOTING SUBJECTED TO
ECCENTRIC & OBLIQUE LOADS

$\phi$ versus $N_{ys}$

$\phi = 0$, $\phi = 0.2$, $\phi = 0.3$, $\phi = 0.4$

$Bt = 220$ mm
$I = 5^\circ$

Fig. 4-18(b)
BEARING CAPACITY PARAMETER, $N_{ys}$
FOR PLANE FOOTING SUBJECT TO
ECCENTRIC & OBLIQUE LOADS

$Z_{1.20}$
$Z_{1.20.1}$
$Z_{1.20.2}$
$Z_{1.20.3}$
$Z_{1.20.4}$

$Z_{1.2\theta / \beta t}$

$Bt = 220 \text{ mm}$
$i = 10^0$

Fig. 4-18 (c)

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BEARING CAPACITY PARAMETER, \( N_{ys} \)
FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS.

\[ \theta_1 \geq 0.4 \]
\[ \theta_1 \geq 0.3 \]
\[ \theta_1 \geq 0.2 \]
\[ \theta_1 \geq 0.1 \]
\[ \theta_1 = \frac{e}{B_t} \]

\( B_t = 220 \text{ mm} \)
\( i = 15^\circ \)

Fig. 4-18(d)
### TABLE NO 4-13
BEARING CAPACITY PARAMETERS $N_{qs1}$ FOR PLANE FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

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BEARING CAPACITY PARAMETER, $N_{qs1}$
FOR PLANE FOOTING SUBJECTED TO
ECCENTRIC & OBLIQUE LOADS

Fig. 4-19(a)

$N_{qs1}$

$B_t = 220$ mm
$i = 0^\circ$

Fig. 4-19(a)
BEARING CAPACITY PARAMETER, Nqs1
FOR PLANE FOOTING SUBJECTED TO
ECCENTRIC & OBLIQUE LOADS

\[ z_i = \frac{e}{B_t} \]

\[ B_t = 220 \text{ mm} \]
\[ i = 5^0 \]

Fig. 4-18(b)
BEARING CAPACITY PARAMETER, $N_{qs1}$ FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

Fig. 4-19(c)

- $B_t = 220$ mm
- $i = 10^0$

$z_1 > e/B_t$

ANGLE OF INTERNAL FRICTION, $\phi$

BEARING CAPACITY FACTOR $N_{qs1}$
BEARING CAPACITY PARAMETER, \( N_{qs1} \)
FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

Fig. 4-19(d)

\[ B_l = 220 \text{ mm} \]
\[ \theta = 15^\circ \]
## TABLE NO 4-14

**BEARING CAPACITY PARAMETERS $N_{ys}$ FOR PLANE FOOTINGS**

**SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS**

$B_t = z_{zu} \text{ mm}$

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BEARING CAPACITY PARAMETER, $N_{ys1}$
FOR PLANE FOOTING SUBJECTED TO
ECCENTRIC & OBLIQUE LOADS

$B_t = 220 \text{ mm}$
$i = 0^\circ$

Fig. 4-20(e)
BEARING CAPACITY PARAMETER, \( N_{ys1} \)
FOR PLANE FOOTING SUBJECTED TO
ECCENTRIC & OBLIQUE LOADS

\[ z_i \leq 0.4 \quad z_i \geq 0.3 \quad z_i \geq 0.2 \quad z_i \geq 0.1 \]

\[ z_i = e^{1/2t} \]

\( B_t = 220 \text{ mm} \)
\( i = 5^\circ \)

Fig. 4-20(b)
BEARING CAPACITY PARAMETER, N_{ys1}
FOR PLANE FOOTING SUBJECTED TO
ECCENTRIC & OBLIQUE LOADS

Fig. 4-20(c)

\[ Z_1 = \phi \frac{B_t}{i} \]

B_t = 220 mm
i = 10^0

Fig. 4-20(c)
BEARING CAPACITY PARAMETER, $N_{ys1}$ FOR PLANE FOOTING SUBJECTED TO ECCENTRIC & OBLIQUE LOADS

$Z_1 \geq 0.4$
$Z_1 \geq 0.3$
$Z_1 \geq 0.2$
$Z_1 \geq 0.1$

$B_t = 220 \text{ mm}$
$i = 15^\circ$

Fig. 4-38(d)
<table>
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<tr>
<th>Angle of Skirt</th>
<th>Eccentricity factor $z_1 = \text{eccentricity (e)} / \text{width of footing}$</th>
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<td>90.00</td>
<td>34.50</td>
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Load inclination = 5°

| 0.00          | 48.08     | 46.42     | 38.77     | 24.65     | 12.28     |
| 15.00         | 53.41     | 51.57     | 43.29     | 27.49     | 16.66     |
| 30.00         | 55.70     | 53.51     | 44.22     | 27.28     | 17.44     |
| 45.00         | 54.34     | 52.08     | 42.47     | 25.61     | 14.79     |
| 60.00         | 49.15     | 47.22     | 37.40     | 21.35     | 11.93     |
| 75.00         | 42.15     | 40.26     | 29.79     | 15.07     | 6.19      |
| 90.00         | 34.05     | 32.53     | 19.58     | 7.26      | 2.13      |

Load inclination = 10°

| 0.00          | 49.44     | 44.21     | 33.59     | 8.06      | 2.74      |
| 15.10         | 55.34     | 48.32     | 37.62     | 14.37     | 6.93      |
| 30.00         | 57.03     | 49.83     | 39.23     | 14.20     | 9.02      |
| 45.00         | 55.10     | 47.53     | 38.00     | 13.72     | 9.67      |
| 60.00         | 49.05     | 42.93     | 33.91     | 12.63     | 9.26      |
| 75.00         | 39.79     | 37.22     | 27.52     | 9.54      | 6.61      |
| 90.00         | 24.16     | 22.00     | 20.57     | 5.89      | 1.76      |

Load inclination = 15°

| 0.00          | 39.61     | 29.34     | 18.58     | 12.19     | 7.50      |
| 15.00         | 46.87     | 36.44     | 22.25     | 14.86     | 9.23      |
| 30.00         | 48.11     | 40.03     | 23.98     | 16.12     | 9.85      |
| 45.00         | 44.29     | 39.45     | 23.40     | 15.64     | 9.29      |
| 60.00         | 40.44     | 36.15     | 21.56     | 13.64     | 7.92      |
| 75.00         | 33.89     | 30.72     | 18.40     | 10.54     | 5.79      |
| 90.00         | 25.63     | 21.35     | 12.70     | 6.48      | 2.19      |
FIG. 4-21 (a) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$  $k = 0.25$

$Z_1 = \frac{e}{B_t}$

$Z_1 \geq 0$

$Z_1 \geq 0.1$

$Z_1 \geq 0.2$

$Z_1 \geq 0.3$

$Z_1 \geq 0.4$

$\theta$ ANGLE OF SKIRT $\alpha$ IN DEGREES
FIG 4-21 (b) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm
$k = 0.25$

$\theta = 5^0$

$N_{qs}$ vs. Angle of Skirt $\alpha$ in Degrees

$B_t = 75$ mm, $k = 0.25$

FIG 4-21 (b) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
FIG 4-21 (c) BEARING CAPACITY PARAMETER Nqs FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
FIG 4-21 (d) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$Z_1 = \frac{\varepsilon}{B_t}$

$B_t = 75 \text{ mm}$  $k = 0.25$

$Z_1 = 0.0$

$Z_1 = 0.1$

$Z_1 = 0.2$

$Z_1 = 0.3$

$Z_1 = 0.4$

FIG 4-21 (d) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
<table>
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<th>Angle of Skirt $\alpha$</th>
<th>Eccentricity factor $z_1 = \text{eccentricity (e)} / \text{width of footing}$</th>
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</table>
**Figure 4-22 (a)** Bearing capacity parameter $N_{ys}$ for skirted strip footings subjected to eccentric and oblique loads.

- $B_t = 75$ mm
- $k = 0.25$

The graph illustrates the variation of $N_{ys}$ with the angle of skirt $\alpha$ in degrees for different values of $Z_1 = \frac{e}{B_t}$, where $e$ is the eccentricity and $B_t$ is the skirt width.
FIG 4-22 (b) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm} \quad k = 0.25$

$\theta = 5^0$

$B_t = 75 \text{ mm}$

$k = 0.25$
FIG 4-22 (c) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$  $k = 0.25$

\[ i = 10^6 \]

$B_t = 75 \text{ mm}$  $k = 0.25$

$z_1 = 0$

$z_1 = 0.1$

$z_1 = 0.2$

$z_1 = 0.3$

$z_1 = 0.4$

$\theta = \theta_0$

$\theta = \theta_t$

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FIG 4-22 (d) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  
$k = 0.25$

$B_t = 75$ mm  
$k = 0.25$

FIG 4-22 (d) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
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Load inclination = 5°

| 0.00           | 15.74     | 15.21     | 12.23     | 7.69      | 3.10      |
| 15.00          | 17.61     | 16.89     | 13.35     | 8.70      | 4.55      |
| 30.00          | 18.52     | 17.73     | 13.82     | 8.84      | 5.07      |
| 45.00          | 18.29     | 17.47     | 13.57     | 8.36      | 4.28      |
| 60.00          | 16.93     | 16.12     | 12.28     | 7.20      | 3.83      |
| 75.00          | 14.72     | 13.96     | 9.97      | 5.17      | 2.71      |
| 90.00          | 12.15     | 11.42     | 6.70      | 2.87      | 0.98      |

Load inclination = 10°

| 0.00           | 16.22     | 11.39     | 6.15      | 0.00      | 0.00      |
| 15.00          | 19.06     | 15.61     | 9.87      | 0.00      | 0.00      |
| 30.00          | 19.98     | 17.27     | 12.78     | 0.00      | 0.00      |
| 45.00          | 19.12     | 16.94     | 13.49     | 0.00      | 0.00      |
| 60.00          | 17.25     | 15.30     | 11.45     | 0.00      | 0.00      |
| 75.00          | 14.61     | 12.60     | 7.98      | 0.00      | 0.00      |
| 90.00          | 11.08     | 8.58      | 4.08      | 0.00      | 0.00      |

Load inclination = 15°

| 0.00           | 13.68     | 11.60     | 6.30      | 0.00      | 0.00      |
| 15.00          | 15.97     | 12.80     | 7.53      | 0.00      | 0.00      |
| 30.00          | 14.89     | 14.00     | 8.38      | 0.00      | 0.00      |
| 45.00          | 14.67     | 14.15     | 8.35      | 0.00      | 0.00      |
| 60.00          | 13.97     | 12.88     | 7.69      | 0.00      | 0.00      |
| 75.00          | 12.59     | 11.78     | 6.80      | 0.00      | 0.00      |
| 90.00          | 10.28     | 9.68      | 4.92      | 0.00      | 0.00      |
FIG 4-23 (a) BEARING CAPACITY PARAMETER $N_{qs1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$N_{qs1}$ vs $\alpha$ for different $Z_1$ values:
- $Z_1 = 0$
- $Z_1 = 0.1$
- $Z_1 = 0.2$
- $Z_1 = 0.3$
- $Z_1 = 0.4$

Parameters:
- $B_t = 75 \text{ mm}$
- $k = 0.25$
- $i = 0^\circ$

$B_t$ = 75 mm $\quad k = 0.25$

$Z_1 = \frac{e}{B_t}$

ANGLE OF SKIRT $\alpha$ IN DEGREES

0.00 10.00 20.00 30.00 40.00 50.00 60.00 70.00 80.00 90.00 100.00

0 2 4 6 8 10 12 14 16 18 20
FIG 4-23 (b) BEARING CAPACITY PARAMETER \( N_{qs1} \) FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

\[
\begin{align*}
&i = 5^\circ \\
&B_1 = 75 \text{ mm} \\
&k = 0.25
\end{align*}
\]

FIG 4-23 (b) BEARING CAPACITY PARAMETER \( N_{qs1} \) FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

\[ B_1 = 75 \text{ mm} \quad k = 0.25 \]
FIG 4-23 (c) BEARING CAPACITY PARAMETER $N_{qs1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$N_{qs1}$ vs. ANGLE OF SKIRT $\alpha$ IN DEGREES

$B_t = 75$ mm $\quad k = 0.25$

$Z_i = e^{|\theta_L|}$

$i = 10^0$

$B_t = 75$ mm $\quad k = 0.25$
FIG 4-23 (d) BEARING CAPACITY PARAMETER $N_{qs1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_i = 75$ mm  \hspace{1cm} k = 0.25
Table NO 4-18
BEARING CAPACITY PARAMETERS Ngs1 FOR SKIRTED FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
Bt = 75 mm \( k = 0.25 \)

<table>
<thead>
<tr>
<th>Angle of Skirt ( \alpha )</th>
<th>Eccentricity factor ( z_1 = \text{eccentricity (e)} / \text{width of footing} )</th>
<th>( z_1=0.0 )</th>
<th>( z_1=0.1 )</th>
<th>( z_1=0.2 )</th>
<th>( z_1=0.3 )</th>
<th>( z_1=0.4 )</th>
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<td>0.00</td>
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<td>1.42</td>
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</table>

| Load inclination = 5° | 0.00 | 23.24 | 13.80 | 4.24 | 1.83 | 0.85 |
| | 15.00 | 29.35 | 18.66 | 5.57 | 2.63 | 1.55 |
| | 30.00 | 33.58 | 22.08 | 6.53 | 3.20 | 2.08 |
| | 45.00 | 34.44 | 22.90 | 6.76 | 3.34 | 1.91 |
| | 60.00 | 30.94 | 20.91 | 6.44 | 3.10 | 1.74 |
| | 75.00 | 25.10 | 15.51 | 5.08 | 2.40 | 1.18 |
| | 90.00 | 18.26 | 9.74 | 2.63 | 1.14 | 0.47 |

| Load inclination = 10° | 0.00 | 22.26 | 17.69 | 7.96 | 1.87 | 0.77 |
| | 15.00 | 28.36 | 24.09 | 11.71 | 4.24 | 1.57 |
| | 30.00 | 32.42 | 28.81 | 13.87 | 5.46 | 2.18 |
| | 45.00 | 33.00 | 30.30 | 13.55 | 5.85 | 2.37 |
| | 60.00 | 29.39 | 27.76 | 12.00 | 4.97 | 2.10 |
| | 75.00 | 23.19 | 22.38 | 8.51 | 3.55 | 1.37 |
| | 90.00 | 16.93 | 16.34 | 4.96 | 1.12 | 0.26 |

| Load inclination = 15° | 0.00 | 22.11 | 17.87 | 11.81 | 4.77 | 2.55 |
| | 15.00 | 28.20 | 24.73 | 18.97 | 6.64 | 3.13 |
| | 30.00 | 32.08 | 30.00 | 21.20 | 9.23 | 4.68 |
| | 45.00 | 32.77 | 31.98 | 27.18 | 11.93 | 5.60 |
| | 60.00 | 30.61 | 29.64 | 24.80 | 8.36 | 4.16 |
| | 75.00 | 25.78 | 24.07 | 16.48 | 6.32 | 2.63 |
| | 90.00 | 19.73 | 17.84 | 9.63 | 3.14 | 1.35 |
FIG 4-24 (a) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS
SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  \hspace{1cm}  k = 0.25
FIG 4-24 (b) BEARING CAPACITY PARAMETER Nys1 FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

Bt = 75 mm \quad k = 0.25

\begin{align*}
\theta_1 &\geq \frac{\varepsilon}{\theta_t} \\
\theta_1 &\geq 0 \\
\theta_1 &\geq 0.1 \\
\theta_1 &\geq 0.2 \\
\theta_1 &\geq 0.3 \\
\theta_1 &\geq 0.4
\end{align*}
FIG 4-24 (c) BEARING CAPACITY PARAMETER $N_{y51}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm} \quad k = 0.25$
FIG 4-24 (d) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$  \quad  k = 0.25

$Z_1 = \text{ eccentricity factor}$

$\theta$ = angle of skirt in degrees

$N_{ys1}$ vs. angle of skirt $\theta$ in degrees

$B_t = 75 \text{ mm}$  \quad  k = 0.25

FIG 4-24 (d) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
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<th>Angle of Skirt (°)</th>
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</table>
FIG 4-25 (a) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \, \text{mm}$ \hspace{1cm} $k = 0.25$

$Z_1 = 0, 0.1, 0.2, 0.3, 0.4$

$\theta = 0^{\circ}$

$B_t = 300 \, \text{mm}$

$Z_1 = \frac{\theta}{B_t}$
FIG 4-25 (b) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$  \hspace{1cm} $k = 0.25$

$\theta = 5^\circ$

$B_t = 300 \text{ mm}$

$k = 0.25$

$z_1 \leq \frac{c}{b\theta}$

$z_1 \leq 0$

$z_1 \leq 0.1$

$z_1 \leq 0.2$

$z_1 \leq 0.3$

$z_1 \leq 0.4$

$\alpha$ IN DEGREES

$N_{qs}$

$0 \leq 10 \leq 20 \leq 30 \leq 40 \leq 50 \leq 60 \leq 70 \leq 80 \leq 90 \leq 100$
FIG 4-25 (c) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  \hspace{1cm}  k = 0.25

$Z_1 = e_1 B_t$

$Z_1 = 0$

$Z_1 = 0.1$

$Z_1 = 0.2$

$Z_1 = 0.3$

$Z_1 = 0.4$

$N_{qs}$

ANGLE OF SKIRT $\alpha$ IN DEGREES
FIG 4-25 (d) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  $k = 0.25$

$z_1 > 0$

$z_1 = 0.1$

$z_1 = 0.2$

$z_1 = 0.3$

$z_1 = 0.4$

ANGLE OF SKIRT $\alpha$ IN DEGREES

$B_t = 75$ mm  $k = 0.25$

FIG 4-25 (d) BEARING CAPACITY PARAMETER $N_{qs}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
### TABLE NO 4-20

**BEARING CAPACITY PARAMETERS \( N_{gs} \) FOR SKIRTED FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS**

\( B_t \geq 300 \text{ mm} \quad k = 0.25 \)

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<th>Angle of Skirt</th>
<th>Eccentricity factor ( z_1 = \frac{e}{\text{width of footing}} )</th>
<th>( z_1=0.0 )</th>
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FIG 4-26 (a) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$

$k = 0.25$

$Z_1 = 0$

$Z_1 = 0.1$

$Z_1 = 0.2$

$Z_1 = 0.3$

$Z_1 = 0.4$

$Z_1 = e^i B_t$

ANGLE OF SKIRT $\alpha$ IN DEGREES
FIG 4-26 (b) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75\text{ mm}$  
$k = 0.25$

$z_1 = \frac{e}{B_t}$

$B_t = 300\text{ mm}$

$z_1 \geq 0$

$z_1 = 0.1$

$z_1 = 0.2$

$z_1 = 0.3$

$z_1 = 0.4$

ANGLE OF SKIRT $\alpha$ IN DEGREES
FIG 4-26 (c) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$ \quad $k = 0.25$

FIG 4-26 (c) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
FIG 4-26 (d) BEARING CAPACITY PARAMETER $N_{ys}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm $k = 0.25$

$Z_1 = 0.3$
$Z_1 = 0.2$
$Z_1 = 0.1$
$Z_1 = 0$

$\theta = 15^\circ$
$B_t=300$ mm
$k = 0.25$

$Z_1 \geq e / B_t$
### TABLE NO 4-21

**BEARING CAPACITY PARAMETERS Nqs1 FOR SKIRTED FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS**

<table>
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<th>Angle of Skirt</th>
<th>Eccentricity factor ( z_1 ) = eccentricity (( e )) / width of footing</th>
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FIG 4-27 (a) BEARING CAPACITY PARAMETER Nqs1 FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$  $k = 0.25$

$z_1 \leq 0$
$z_1 \leq 0.1$
$z_1 \leq 0.2$
$z_1 \leq 0.3$
$z_1 \leq 0.4$
FIG 4-27 (b) BEARING CAPACITY PARAMETER N qs1 FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

B_t = 75 mm  
k = 0.25
FIG 4-27 (c) BEARING CAPACITY PARAMETER $N_{qs1}$ FOR SKIRTED STRIP FOOTINGS
SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \text{ mm}$  $k = 0.25$

$Z_1 \geq \frac{i}{1B_t}$
FIG 4-27 (d) BEARING CAPACITY PARAMETER Nqs1 FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  \hspace{1cm}  k = 0.25

$z_1 \geq 0.1$  \hspace{1cm}  $z_1 \geq 0.2$

$B_t = 300$ mm  \hspace{1cm}  $k = 0.25$

$\alpha$ IN DEGREES

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### TABLE NO 4-22
BEARING CAPACITY PARAMETERS $N_{gs1}$ FOR SKIRTED FOOTINGS
SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 300 \text{ mm}$  $k = 0.25$

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FIG 4-28 (a) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  
$k = 0.25$

$Z_1 \geq 0$
$Z_1 \geq 0.1$
$Z_1 \geq 0.2$
$Z_1 \geq 0.3$
$Z_1 \geq 0.4$

$B_t = 300$ mm  
$k = 0.25$

$Z_1 \geq e_l B_t$
FIG 4-28 (b) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75$ mm  \hspace{1cm} k = 0.25

$z_1 = e / B_t$

$z_1 \geq 0$

$z_1 \geq 0.1$

$z_1 \geq 0.2$

$z_1 \geq 0.4$

$B_t = 75$ mm  \hspace{1cm} k = 0.25

FIG 4-28 (b) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
FIG 4-28 (c) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_s = 75$ mm  $k = 0.25$

$\theta_e = 300$ mm  $k = 0.25$

$z_1 \leq 1B_s$

$N_{ys1}$ vs ANGLE OF SKIRT $\alpha$ IN DEGREES

$B_s = 75$ mm  $k = 0.25$

FIG 4-28 (c) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS
FIG 4-28 (d) BEARING CAPACITY PARAMETER $N_{ys1}$ FOR SKIRTED STRIP FOOTINGS SUBJECTED TO ECCENTRIC AND OBLIQUE LOADS

$B_t = 75 \, \text{mm}$  
$k = 0.25$

$\theta_1 \geq \theta_1 B_t$

$\theta_1 \geq 0.2$

$\theta_1 \geq 0.1$

$\theta_1 \geq 0.04$

$\theta_1 \geq 0.01$

$\theta_1 \geq 0.005$

$\theta_1 \geq 0.001$

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4.6. HORIZONTAL LOAD:

The mathematical treatment as described earlier in section 4.3 for the width effect and the state of stress in the sand within the skirts is also applicable to plane and skirted strip footings subjected to horizontal load provided that sufficiently large normal load and horizontal load simultaneously act upon the footing.

The relation proposed for modified friction parameter in presence of the normal load in the same section, viz.

\[ \tan \phi_m = \exp [c_\phi \tan \phi] \tan \psi \] for plane footings

and \[ \tan \phi_m = \exp [c_\phi \tan \phi + c_i] \tan \psi \] for skirted footings is applicable.

In the absence of appropriate normal load, the relation \( \tan \phi_m = \tan \phi \) is applied to the plane footing with only self-weight acting vertically. For skirted footing, provided that the sand within the skirts is compacted by the seating load, the equation for improved state of stress within the skirts, viz.

\[ \tan \phi_m = C_i \tan \phi, \] where

\[ C_i = \left\{(A_2/A_1)^{0.5} - 1\right\} (e_{\text{emin}})^{0.1} \] will be applicable with self-weight and weight of confined sand acting vertically. This is because the effect of width of plate will be negligible in this case.

The ultimate sliding load against frictional resistance for plane surface footing is given by \( Q_m = \tan \psi \times R \), where \( R \) is the normal reaction (fig. 4-29). In case of skirted strip footings, on the surface of the sand mass, the passive resistance acting on skirts is also added (fig. 4-30).

* Note: This equation is applicable provided that the sand mass within the skirts is confined under an action of (sufficiently large normal load) at least 10% failure (load due to normal loading which is removed after the application (this facilitates the required densification of sand within the skirts).
Referring to fig. 4-30

\[ Q_{th} = Q_{fr} + P_{p0} + P_{p0} \]

\[ = R \tan \phi_m + \frac{1}{2} k_{pm} \gamma. (kB_t \sin \alpha)^2 + \frac{1}{2} k_p \gamma. (kB_t \sin \alpha)^2 \]

Thus \[ Q_m = \tan \phi_m \times R + \frac{1}{2} (k_{pm} + k_p) \gamma. (kB_t \sin \alpha)^2 \]

where \[ k_{pm} = \tan^2 \left(45+\phi_m/2\right) \]

\[ k_p = \tan^2 \left(45+\phi/2\right) \]

Inserting the value of \( \phi_m \), the equation for ultimate sliding load for surface footings are as follows.

\[ Q_{th} = \exp \left[ C_s \tan \phi + C_1 \right] \tan \phi \times R + \frac{1}{2} (k_{pm} + k_p) \gamma (kB_t \sin \alpha)^2 \]...

(4-15a)

if normal load is simultaneously acting.

The energy correction (\( \tan \delta / \tan \phi \)) as mentioned in chapter 2 [E. Schultz and Horn (1967)] may vary between 0.24 and 1.08.

And \[ Q_m = \left( \tan \delta / \tan \phi \right) \left\{ \exp \left[ C_1 \right] \tan \phi \times R + \frac{1}{2} (k_{pm} + k_p) \gamma (kB_t \sin \alpha)^2 \right\} \]...

(4-15b)

If normal load is absent and sand within the skirts is properly densified, the energy correction (\( \tan \delta / \tan \phi \)) will tend to unity.

The variation in \( \phi_m \) for different angles of skirts with and without normal loads for one of the cases of present work viz. \( \phi = 34.5^\circ, \gamma = 1490 \text{ kg/m}^3, e = 0.792, e_{\text{min}} = 0.57 \) [i.e. \( (e-e_{\text{min}} = 0.222) \)], \( B_t = 50 \text{ mm} \) and \( k = 0.25 \) is shown in figure 4-31. The correction factor in present investigation is assumed as equal to 0.9 because the sand has \( \phi = 34.5^\circ \), and has a mixed mode failure state.

From the above theoretical analysis it infers that the modified angle of internal friction increases with angle of skirt up to 45° to 50°, thereafter reduces.
RESISTANCE TO HORIZONTAL LOAD: plane footing

\[ Q_{th} \rightarrow R \tan \phi \rightarrow R \]

**FIG: 4-29**

RESISTANCE TO HORIZONTAL LOAD: skirted footing

\[ Q_{fm} \rightarrow P_{p(i)} \rightarrow P_{p(0)} \rightarrow R \]

**FIG: 4-30**
The curve will modify as per energy correction factor depending on material of foundation, foundation depth and water level.

Variation of $\phi_m$ in presence and absence of the normal load ($p$) for skirted footing subjected to horizontal load.

FIG: 4-31
4.7. VERIFICATION OF VARIATION IN MODULUS OF ELASTICITY BY FEM USING ANSYS SOFTWARE:

The proposed mechanistic model explained in section 4.4 for confined sand mass exhibits increase in its modulus of elasticity $E$ which is verified by FEM analysis employing the ANSYS (1987), a general purpose FEM software package. This package has an extensive element library besides capability of modelling plasticity, large strain, and swelling and creeps behaviour.

Finite element investigations were undertaken for plane and skirted strip footings. Load settlement curves were simulated by FEM investigations for a sand underlain by plane and skirted strip footings and also the sand mass within the skirts. Four nodded rectangular elements were used in the analysis. These elements were used for both the plane and skirted strip footings. As the applied pressure, the geometry of footing, and the tank were symmetric, only the symmetric half was simulated. The axial loads were applied. Two-dimensional modelling principles were used to model the symmetric half of the tank. The typical grid used in the analysis (for 1000-mm sand mass underlain by plane and skirted strip footings) is shown in fig. 4-32. The properties of sand used in the analysis are given in chapter 5. Triaxial tests were performed to determine the modulus of elasticity of sand. Results of These tests are presented in chapter 5. From these tests, the parameters required to evaluate the modulus are obtained by using the method suggested by Duncan and Chang (1970).

Accordingly

$$E_i = k P_4 (\sigma_3/P_3)^n$$  \hspace{1cm} (4-16a)

$$E_i = E_i [1 - \frac{R_{t}(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_{ult}}]^2$$  \hspace{1cm} (4-16b)

$$\sigma_1 - \sigma_3 = \frac{2c\cos\phi + 2\sigma_3\sin\phi}{1 - \sin\phi}$$  \hspace{1cm} (4-16c)
MESH USED FOR ANSYS SOLUTION PLANE FOOTING

Fig: 4.32a
VARIATION OF EQUIVALENT MODULUS OF ELASTICITY OF SAND V/S ANGLE OF SKIRT $\alpha$

FIG: 4.33
Where $E_i$, $E_t$ = Initial and tangent moduli

$\sigma_1 - \sigma_3$ = deviator stress

$R_f$ = failure ratio (always less than one.)

$p_a$ = atmospheric pressure

$c$ = unit cohesion which is zero for present investigation.

The value of $R_f$ required for calculations is 0.65. Thus if $E_i$ is known, $E_t$ can be determined at any level of loading from equation (4-16b).

The sand mass can be divided into number of layers and the modulus determined for each of these layers. However, for present study, the sand under plane and skirted strip footings and the sand within the skirts were assigned equivalent modulus till it gave a fairly good fit to the experimental curves. These made it feasible to compare the equivalent modulus obtained for sand below plane and skirted strip footings to study the improvement in modulus within the skirts.

For example, one metre sand layer underlain by plane footing of 50 mm $\times$ 600 mm worked out an equivalent modulus of 20 kg/cm² gave a fairly good fit with experimental results. Such an approach is also adopted by Mhaisker S.Y. (1993).

For sand underlain with skirted footings, the initial modulus $E_i = 20$ kg/cm² was used for confined sand and the foundation sand mass. $E_i$ of confined mass was changed (usually increased) to obtain good fit with experimental curves. Simultaneously $E$ of foundation soil and confined mass are increased according to Duncan Chang's equation (4-16b). Due to the non-linear nature of the problem for all cases (plasticity), the load was applied in steps with adequate iterations in each step. For skirted footings, load steps were 6 with 10 iterations in each case for a convergence criterion of 5 percent.

The FEM analysis employing ANSYS software suggests that the equivalent modulus of sand within the skirts increases up to an angle of skirt of 30° which drops thereafter, reaches to more or less same value of plane footing at an angle of skirt of 90° as shown in fig. 4-33.