*Indian Geotechnical Journal, 39(2), April 2009, 216 - 226*

TECHNICAL NOTE

# Allowable Bearing Pressure on Clays-Reliability Based Approach

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## Introduction

The design of shallow foundation requires following two criteria to be satisfied for the acceptable performance: (i) it should have sufficient factor of safety against shear failure (ii) the settlements should be within tolerable limits. Since, In these cases almost all the settlements take place at the end of the construction and hence do not pose any problems to the structures founded on such soils subsequently. In case of clays, both the shear failure and settlements are the governing criteria (Bowles 1996). The settlement analysis requires the following two aspects to be investigated: (i) An estimate of the net "final" settlement at the end of the consolidation and (ii) time for settlement. The settlement calculations in clays are based on the classical Terzaghi's consolidation theory (Terzaghi and Peck 1948). For the safe design, it is required that the applied load on the footing should be such that it should provide enough factor of safety from complete shear failure as well as the estimated settlement with the applied load should be within tolerable limits. With experience and from economic considerations, it is established that a factor of safety of not more than 3.0 is appropriate for the design of shallow foundation resting on clayey soils in case where the shear failure is the governing criterion (Terzaghi and Peck 1948, Bowles 1996). In this context, the guidelines based on measurements of final settlements assume considerable importance. Skempton (1951) quantified the selection of appropriate value of factor of safety for the footings resting on clay based upon theoretical and field observations and concluded that a factor of safety of at least 3.0 is desirable in estimating the allowable bearing capacity. In cases where the foundation design is controlled by settlement considerations, a factor of safety greater than 3.0 is desirable to restrict the settlement to a magnitude compatible with the structural requirements. Skempton (1951) has also addressed the variation of soil properties in an indirect manner and suggested the factors of safety. Although the factor of safety approach is widelyused in the conventional design of shallow foundations, it is well recognized that it does not consider different sources of uncertainties i.e. natural heterogeneity, measurement errors, and model transformation uncertainties in an explicit manner. In the recent years, it has been established that reliability analysis is a useful tool to understand the role of variability and a few studies have been conducted on the reliability analysis of

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shallow foundations (Sivakumar Babu *et. al*, 2006 and Sivakumar Babu and Amit Srivastava, 2007). In the light of above, it is desirable that the results presented by Skempton (1951) should be re-examined using reliability analysis principles and the results presented in this paper fulfil this objective.

## Objective of the Present Study

In the present study, a review on the estimation of bearing capacity of clays and the corresponding factor of safety evaluation from shear failure and settlement considerations using the data of Skempton (1951) are presented. It needs to be reiterated that his study is perhaps the only study in Geotechnical literature that considers the theories as well as measurements of settlements in a comprehensive manner and provides guidelines. Hence, the concept of probabilistic approach has been used to reinvestigate the selection of appropriate value for factor of safety by considering the effect of variation in the input parameters.

## Ultimate Bearing Capacity of Clayey Soil

The net ultimate bearing capacity ($q_{nu}$) of a shallow foundation on a clayey soil can be estimated with the help of the following equation (Bowles 1996).

$$q_{nu} = cN_C \tag{1}$$

where $c$ is the undrained cohesion (for a saturated clayey soil for which undrained shear strength parameter $\phi_u$ = 0) and $N_c$ is the bearing capacity factor. Applying a factor of safety ($FS$), the allowable bearing capacity ($q_a$) of the foundation soil can be calculated as below:

$$q_a = \frac{q_{nu}}{FS} = \frac{cN_c}{FS} \tag{2}$$

It can be seen that the ultimate bearing capacity of shallow foundation on clayey soil depends on two factors $c$ and $N_c$ and a global factor of safety ($FS$) is applied to $cN_c$ to obtain the allowable bearing capacity of the foundation soil. According to Skempton (1951), if the shear strength of the soil does not vary by more than ±50% of the average value for a depth of $2/3B$ below the footing, the average value of "$c$" can be used for the calculation of ultimate bearing capacity of the clayey soil.

For the estimation of bearing capacity factor ($N_c$), theoretical solutions are available in the literature. The most popular of them is the Prandtl (1920) solution in which the analysis of the bearing capacity of the strip footing on the surface ($D$ = 0) showed value of bearing capacity factor ($N_c$) equal to $2+\pi$ = 5.14. In the analysis, the failure mechanism assumes that the footing pushes a wedge of clay in front of it, which, in turn, pushes the adjacent material sideways and upwards.

For the footings placed at a considerable depth, Meyerhof (1950) modified the Prandtl (1920) solution in which the slip surface is assumed to curve back on the sides of the foundation. The corresponding value of $N_c$ for strip footing is 8.3.

The solution provides an upper limit to the bearing capacity factor ($N_c$) value because of the greater length of assumed shear surface.

Ishlinsky (1944) obtained a rigorous solution for $N_c$ for circular footings, with a smooth base placed on the surface of the clay. The solution provided a numerical value of $N_c$ = 5.68. For a rough footing, Meyerhof (1950) used an approximate analysis and for the same case, a value of $N_c$ equal to 6.2 was reported.

For circular footings located at a considerable depth beneath the surface, three theoretical solutions available in the literature. (i) Meyerhof (1950) calculated a value of $N_c$ equal to 9.3, which is certainly an upper limit. (ii) Gibson (1950) combined the approach originated by Bishop *et al.*, (1945) for metals with large strain theory by Swainger (1947) and found the solution for bearing capacity factor for clays. The failure mechanism assumed that the penetration of footing at ultimate failure is equivalent to expanding a spherical cavity in the clay of diameter equal to the diameter of the footing ($B$). A plastic zone develops with a radius of $\frac{B}{2}\sqrt[3]{\frac{E}{c}}$ (where $E$ is the secant modulus of clays at a stress equal to ½ the yield value) beyond which the clay remains in elastic state. The following expression for $N_c$ was proposed by Gibson (1950);

$$N_c = \frac{4}{3}\left(\log_e \frac{E}{c} + 1\right) + 1 \tag{3}$$

The range of $E/c$ for majority of undisturbed clays lies in the range of 50 to 200 that provides a corresponding $N_c$ values between 7.6 to 9.4 calculated using Eq. 3 with an average value of $N_c$ = 8.5. (iii) Wilson (1950) provided a value of $N_c$ equal to 8.0 for the bearing capacity of clay loaded at depth by a rigid circular plate by finding the foundation pressure necessary to bring about the merging of the two plastic zones originating from the edges of the footing. It can be seen that the available theoretical solutions for $N_c$ although involve different approaches, yet they lead to values of $N_c$ in the range of ±10% covered by Gibson(1950) analysis for clays.

Skempton (1951) proposed the following expression for the estimation of bearing capacity factor ($N_c$), on the basis of theory, laboratory tests and field observations:

For strip footings:

$$N_c = 5\left(1 + 0.2\frac{D_f}{B}\right);\ \text{limiting to maximum 7.5.} \tag{4a}$$

For square and circular footings:

$$N_c = 6\left(1 + 0.2\frac{D_f}{B}\right);\ \text{limiting to maximum 9.0.} \tag{4b}$$

For rectangular footings:

$$N_c = 5.0\left(1+0.2\frac{D_f}{B}\right)\left(1+0.2\frac{B}{L}\right);\ \text{for } \frac{D_f}{B} \le 2.5 \tag{4c}$$

$$N_c = 7.5\left(1+0.2\frac{B}{L}\right);\text{for } \frac{D_f}{B} > 2.5 \tag{4d}$$

Table 1 summarizes the available field evidence on the ultimate bearing capacity of clays that provides a satisfactory confirmation to the suggested values of $N_c$ by Skempton (1951).

**Table 1 Comparison of Field data with the Suggested $N_c$ Value for the Ultimate Bearing Capacity of Clays (Skempton 1951)**

| References | Dimensions (m) | Avg. (S) settlement at failure (cm) | S/B % | Net foundation pressure at failure ($q_{nf}$) kPa | Avg. shear strength of clay beneath the foundation (kPa) | Value of $N_c$ | |
|---|---|---|---|---|---|---|---|
| | | | | | | Actual ($q_{nf}/c$) | Proposed (Skempton1951) |
| Odenstad (1948) Cadling and Odenstad (1950) | B = 0.40<br>L = 2.00<br>D = 0.0 (L)<br>0.3 (U) | 12.70 | 3 | 41.20 | 7 kPa (Vane)<br>6.4 kPa (compr.) | 5.88<br>6.44 | 5.4<br>6.5 |
| Skempton (1942) | B = 2.44<br>L = 2.74<br>D = 1.68 | 25.40 | 10 | 91.0*<br>110.0** | 15.32 | 5.58<br>7.18 | 7.2 |
| Morgan (1944) Skempton (1950) | B = 2.44<br>L = 2.44<br>D = 15.24 | 27.94 | 12 | 182.0 | 21.00 | 8.67 | 9.0 |
| Wilson (1950) | B = 2.44<br>L = 2.44<br>D = 1.83 (in clay)<br>6.00 (total) | 35.56 | 15 | 277.0 | 34.47 | 8.00 | 7.4<br>8.6 |
| Nixon (1949) | B = 7.62<br>L = 7.62<br>D = 0.0 | - | - | 80.0 | 12.93 | 6.18 | 6.2 |

* with side friction and ** without side friction, L = lower, U = Upper
B = Width, L = Length, D = Depth

## Settlement Analysis

For the shallow foundation resting on a clayey soil, the final settlement and rate of settlement can be calculated from Terzaghi's theory of one dimensional consolidation (Terzaghi and Peck 1948). Considering the estimation of net final settlement of the clayey soil under an applied net pressure ($q_n$), the following Eq. 5 can be used for the foundation resting directly on a thick layer of clayey soil:

$$S_n = m_v q_n B I_f \tag{5}$$

where $m_v$ is the coefficient of volumetric compressibility at a depth beneath the foundation, as measured in oedometer tests on undisturbed samples and determined over the range of overburden pressure from ($p_o$) to ($p_o+\sigma_z$). $p_o$ is the initial effective overburden pressure at depth $z$ and $\sigma_z$ is the increment of vertical pressure due to application of net foundation pressure ($q_n$). $I_f$ = influence factor at a given depth $z$ and the value can be obtained from published literature (Terzaghi 1943).

Skempton (1951) indicated that the above conventional approach is adequate for estimating the final settlement of foundations on deep beds of clays and it can be written in the following form to obtain a relationship between final settlement and factor of safety against ultimate failure.

$$S_n = \frac{q_n}{q_{nu}} \frac{q_{nu}}{c} B m_v c I_f \tag{6}$$

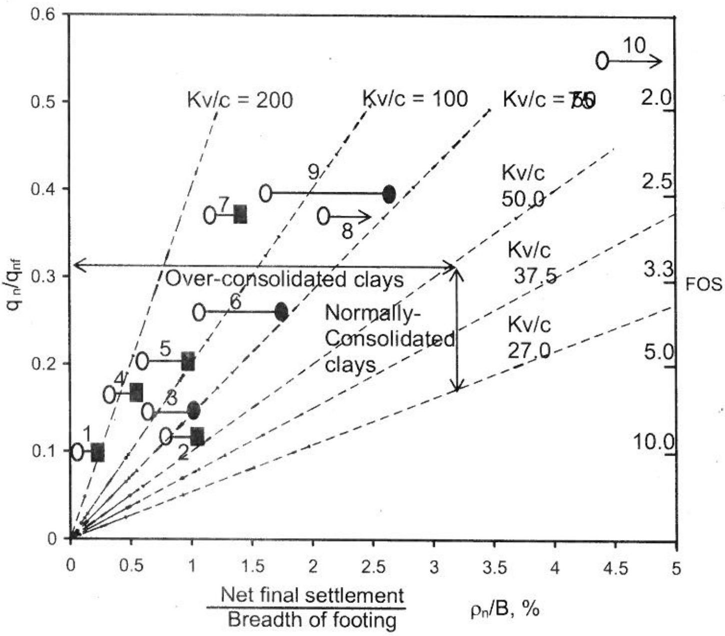
Defining the modulus of compressibility $K_v = 1/m_v$ and using the values of $I_f$, the net final settlement is given by the following expression (Skempton, 1951).

$$S_n = \frac{5}{K_v / c} \frac{q_n}{q_{nu}} B = \frac{5}{K_v / c} \frac{1}{FS} B \tag{7}$$

For over-consolidated clays $K_v/c$ lies in the range from 70 to 200 while for normally- consolidated clays the range is approximately from 25 to 80 (Skempton 1951). Eq. 7 has been plotted in Figure 1 for several typical values of $K_v/c$ and it is compared with the field observation results (Skempton 1951). It can be seen that for any given clay the settlement is approximately proportional to the width of the footing for a given factor of safety. Conversely, the allowable net foundation pressure on given clay will decrease in direct proportion to the foundation width, if it is required to restrict the settlement to some specified magnitude. A close observation of Figure 1 leads to the following inferences:

- For a limiting value of final settlement and breadth of the footing, the normally- consolidated soil requires higher factor of safety than over-consolidated soil.
- For a limiting allowable settlement, a larger width of the footing means a higher factor of safety from settlement considerations and hence the design is governed by settlement considerations.
- The stability criterion is relevant only for small footings on over-consolidated clays. For other cases, the design is governed by settlement considerations.

From the above brief discussion on the bearing capacity evaluation of clayey soil based on shear failure and settlement criteria, it can be concluded that a factor of safety of atleast 3.0 is desirable. In circumstances, where the design of foundation is controlled by the settlement considerations, a factor of safety greater than 3.0 could be used in order to restrict the settlements within the tolerable limits.




● Estimated settlement after 100 years

■ Estimated final settlement

O Recorded settlement ( Skempton 1951)

| Sr.No. | References | | |
|---|---|---|---|
| 1 | Skempton (1951) | 6 | Cooling (1948) |
| 2 | Gibson (1950) | 7 | Buckton and Fereday (1938) |
| 3 | Wilson ( 1950) | 8 | Tschebotarioff and Schuyler (1948) |
| 4 | Hanna ( 1950) | 9 | Jeppesen (1948) |
| 5 | Tschebotarioff (1940) | 10 | Glick (1936) |

**Fig. 1 Final Settlements of the Foundations in Saturated Clays (Comparison of Theoretical and Recorded Settlement Values as Reported in Skempton 1951)**

The analysis conducted in the early 1950s provided a basis for the selection of appropriate value of factor of safety in the conventional design of shallow foundations resting on a clayey soil and since then it has been used routinely in the conventional designs. Probabilistic approach which has been used in the present study to reinvestigate the selection of factor of safety for the foundation design on clayey soils takes into account the variability in a mathematical framework and provides a rational basis.

## Probabilistic Approach

In the probabilistic analysis, the input soil parameters required for the estimation of ultimate bearing capacity of the foundation soil are treated as random variables. The random variables are characterized by their probability density function (*pdf*) and parameters of distribution i.e. mean and variance. The variance defines the extent of variation in the estimated mean of the random variables. The variability is also defined by coefficient of variation (CoV) which is the ratio of the standard deviation ($\sigma$ = square root of variance) to the expected mean ($\mu$) of the random variable. Hence, CoV is = $\sigma/\mu$ which is normally reported in percent (%). Considering the variability in the input parameters, the performance of the system is assessed either in terms of probability of failure or in terms of a safety index defined as reliability index ($\beta$). The available explicit or implicit functional relationship between input soil parameters and output response are used for the reliability analysis. The explicit relationships for reliability analysis can be obtained from conventional bearing capacity and settlement equations. These relationships are implicit in numerical analysis. For the calculation of reliability index, methods such as First Order Reliability method (FORM), Second Order Reliability Method (SORM), Point Estimate method (PEM), and Monte Carlo simulation (MCS) are available in the literature (Baecher and Christian 2003).

Normally, in the analysis, the performance function *g()* is defined as *C-D*; where *C* is the capacity and *D* is the demand. *g()* > 0 will be in the safe state while *g()* < 0 will be in the unsafe state.

From shear failure consideration, *C* is taken as ultimate bearing capacity of the foundation soil and *D* is the applied pressure on the footing. On the other hand, from the settlement considerations *C* is the allowable settlement and *D* is the calculated settlement. For uncorrelated normally distributed *C* and *D*, reliability index ($\beta$) can be calculated using the following equation:

$$\beta = \frac{\mu_C - \mu_D}{\sqrt{\sigma_C^2 + \sigma_D^2}} \tag{8}$$

On the other hand, for uncorrelated log-normally distributed *C* and *D*, reliability index ($\beta$) can be calculated as below:

$$\beta = \frac{\ln\left[\frac{\mu_C}{\mu_D}\sqrt{\frac{1+CoV_D^2}{1+CoV_C^2}}\right]}{\sqrt{\ln\left[\left(1+CoV_C^2\right)\left(1+CoV_D^2\right)\right]}} \tag{9}$$

where $\mu_C$, $\mu_D$ are the mean values of *C* and *D*; $\sigma_C$ and $\sigma_D$ are the standard deviations in the estimation of *C* and *D*. *CoV* is the coefficient of variation which is obtained by dividing the standard deviation ($\sigma$) by the mean ($\mu$). For the good performance of the system, the calculated value of reliability index ($\beta$) should not be less than 3 (USACE 1997).

## Results and Discussion

It is assumed that a strip footing of width *B* is placed on a thick layer of clayey soil and parametric study is performed by taking different values of coefficient of variation in the input parameters, and the reliability analysis is performed using different values of factor of safety (FS) i.e. 2.0, 2.5 and 3.0. A factor of safety for a given case is considered appropriate when it provides a corresponding reliability index ($\beta$) value of more than 3.0. From the shear failure consideration, for the calculation of reliability index ($\beta$) values, the mean value of capacity ($\mu_C$) is taken as the ultimate bearing capacity of the foundation soil ($q_{nu}$) and mean value of demand ($\mu_D$) is the applied pressure on the foundation soil which can be obtained by applying the factor of safety to the ultimate bearing capacity. Hence, $\mu_C = q_{nu}$ and $\mu_D = q_a = q_{nu}/FS$. It should be noted that the variation in the applied pressure on the foundation soil is not considered and the coefficient of variation ($CoV$) in the capacity ($C$) is taken from 10% to 30% which is the normal expected range as reported in the literature (Duncan 2000). Putting the mean value of $\mu_C = q_{nu} = cN_c$ and $\mu_D = q_a = q_{nu}/FS$ in Eqs. 8 and 9; expressions for reliability index values for normally and log-normally distributed input variables can be obtained as below:

$$\beta = \frac{\left(1 - \frac{1}{FS}\right)}{(CoV)_C} \tag{10}$$

For normally distributed input variables,

$$\beta = \frac{\ln\left[FS\sqrt{\frac{1}{1+CoV_C^2}}\right]}{\sqrt{\ln\left[\left(1+CoV_C^2\right)\right]}} \tag{11}$$

For lognormally distributed input variables,

Using Eqs 10 and 11, the range of factor of safety normally used in the conventional design of shallow foundation on clayey soils can be reinvestigated by considering the variability in the input parameters.

For different values of CoVs and for a given factor of safety value, the corresponding reliability index ($\beta$) value can be evaluated that will help in deciding whether the considered factor of safety provides the acceptable level of safety and the associated risk can be evaluated. Tables 2 and 3 provide the reliability index values from shear failure criterion for normally and log-normally distributed input variables, respectively.

**Table 2 Reliability Index Values for Normally Distributed Input Variables**

| $(CoV)_C$(%) | 10 | 15 | 20 | 25 | 30 |
|---|---|---|---|---|---|
| Case (1) [FS = 2.0] | 5.00 | 3.33 | 2.50 | 2.00 | 1.67 |
| Case (2) [FS = 2.5] | 6.00 | 4.00 | 3.00 | 2.40 | 2.00 |
| Case (3) [FS = 3.0] | 6.67 | 4.44 | 3.33 | 2.67 | 2.22 |

The results indicate that the selection of appropriate value of factor of safety depends on the extent of uncertainty involved in the determination of input soil parameters and acceptable performance level of the structure defined in terms of reliability index ($\beta$). For normally distributed input variables, a value of $FS$ = 3.0 is adequate for low values of $CoVs$ (up to 20%) to achieve the corresponding reliability index ($\beta$) values more than 3.0. For higher values of $CoVs$ (more than

20%), a factor of safety of 3.0 is not adequate and designers need to adopt higher factor of safety. On the other hand, for log-normally distributed input variables, the calculated values of reliability index are higher for the same cases considered for normally distributed input variables. But in these cases also a factor of safety of 3.0 is adequate for *CoVs* up to 35% only.

**Table 3 Reliability Index Values for Log- Normally Distributed Input Variables**

| (CoV)C(%) | 15 | 20 | 25 | 30 | 35 |
|---|---|---|---|---|---|
| Case (1) [FS = 2.0] | 4.57 | 3.40 | 2.69 | 2.21 | 1.87 |
| Case (2) [FS = 2.5] | 6.07 | 4.53 | 3.60 | 2.97 | 2.53 |
| Case (3) [FS = 3.0] | 7.29 | 5.45 | 4.34 | 3.60 | 3.06 |

From settlement consideration, a parametric study is performed by assuming different values of width of footing (*B*) and the mean values of $K_v/c$ which are taken as 25, 50, 100, 150 and 200 as reported in the literature (Skempton 1951). The allowable settlement (*C*) is taken as 76 mm which is normally the tolerable limit for most of the structures founded on clayey soil (BIS: 1904-1986; Eurocode 7.0 BS EN 1997-1:2004). Table 4 provides the reliability index ($\beta$) values for different widths of the footing (*B*) and for different values of coefficient of variation (CoV) in $K_v/c$.

**Table 4 Reliability Index Values from Settlement Consideration (FS = 3.0)**

| Width of the footing (B) | Mean Kv/c | CoV = 10% | CoV = 15% | CoV = 20% | CoV = 25% |
|---|---|---|---|---|---|
| B = 1.5 m | 25 | * | * | * | * |
| | 50 | 5.00 | 3.33 | 2.50 | 2.00 |
| | 100 | 20.0 | 13.3 | 10.0 | 8.00 |
| | 150 | 34.5 | 23.0 | 17.5 | 14.0 |
| | 200 | 50.0 | 33.3 | 25.0 | 20.0 |
| B = 3.0 m | 100 | 5.00 | 3.33 | 2.50 | 2.00 |
| | 150 | 12.5 | 8.33 | 6.25 | 5.00 |
| | 200 | 20.0 | 13.3 | 10.0 | 8.00 |
| B = 4.5 m | 150 | 5.00 | 3.33 | 2.50 | 2.00 |
| | 200 | 10.00 | 6.67 | 5.00 | 4.00 |
| B = 6.0 m | 150 | 1.25 | 0.83 | 0.63 | 0.50 |
| | 200 | 5.00 | 3.33 | 2.50 | 2.00 |

From the results of the analysis presented in Table 4, the following observations can be made:

1. For smaller widths of footing placed on a normally-consolidated soil (represented by low value of $K_v/c$), the assumed value of factor of safety of 3.0 is adequate for lower coefficient of variation (*CoV*) in $K_v/c$ (10-15%). For

higher *CoVs* in $K_v/c$, for the considered factor of safety of 3.0, the corresponding reliability index values are less than 3.0. Hence, a larger factor of safety is required in these cases.

2. For the footing placed on over-consolidated clay (high values of $K_v/c$), the reliability index values are more than 3.0 and therefore a factor of safety value of 3.0 is adequate even for higher *CoVs* in $K_v/c$. For the cases, where a lower width of footing is placed on over-consolidated clay, it can be seen that the shear failure criteria is the governing criteria in the design since it provides very high values of reliability index ($\beta$) from settlement considerations.
3. In contrast to the above, for the cases where a lower width of footing is placed on normally-consolidated clay, settlement is the governing criteria and a factor of safety of more than 3.0 is required to achieve the acceptable performance level measured in terms of reliability index values.
4. For larger widths of footing placed on normally-consolidated and over-consolidated clays, with a given factor of safety of 3.0, the calculated values of reliability indices are less than 3.0. Hence, in such cases the value of factor of safety = 3.0 will not be adequate (except for cases where the coefficient of variation in $K_v/c$ is of very low value). The design needs to be governed by the settlement consideration requiring a factor of safety much more than 3.0.

With these observations, it can be inferred that the selection of appropriate value of safety factor in the conventional design depends on the extent of variation in the expected mean value of the input parameters, width of the footing and type of soil on which footing is placed. The probabilistic approach provides a rational basis for the selection of factor of safety with sound mathematical treatment to the uncertainties involved.

## Concluding Remarks

The factor of safety which is routinely used in the conventional geotechnical design is reinvestigated in a probabilistic framework using the data of Skempton (1951). It is found that a factor of safety of 3.0 is adequate for smaller width footings placed on over-consolidated clays for a range of coefficient of variation in the input parameter not more than 30 – 35%. For larger width footings, settlement is the governing criterion in the design and it will require the selection of factor of safety much more than 3.0 to reduce the settlement below the tolerable limit. In addition, the effect of width of footing and type of foundation soil on the selection of factor of safety and interdependency of shear and settlement consideration in the foundation design are clearly demonstrated.

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